STRUCTURAL ANALYSIS OF
PROPOSED RESIDENTIAL HOME

10JAN2024

Table of content

1 DESIGN CRITERIA:

1.1 DESIGN CODES & REFERENCES

- 2023 Florida building code
- 2021 Edition of the International Building Code
- 2021 Edition of the International Residential Code
- ASCE 7-16
- Steel design: AISC 360-16: LRFD Specification for Structural Steel Buildings
- Seismic AISC 341-16 Seismic Provisions for Structural Steel Buildings
- Concrete: Reinforced Concrete Design Handbook (ACI)
- Ultimate Strength Design Handbook (ACI)

Calculation performed by SCIA Engineer 20.0

1.2 CODE CRITERIA

2021 IBC

Seismic Design Category:	**A**
Wind Speed:	**165** MPH
Wind Exposure:	**D**
Snow Load (Roof):	**0** psf

1.3 MATERIALS

Hot Rolled Steel Grade

- WIDE FLANGE BEAMS - ASTM A992
- HOLLOW STRUCTURAL SECTIONS - ASTM A500 Gr.B

Cold Formed Steel Grade

- ASTM A653 GRADE 33, TYPE H 33-43 mil GALV. STEEL
- ASTM A 913 GRADE 50, TYPE H 54-97 mil GALV. STEEL

1.4 STRUCTURAL LOADS

Floor Live Load	**40.00 psf**
Floor Dead Loads	
Finish	2 psf
Sheathing	2.2 psf
Floor Joists	2.8 psf
Insulation	1.0 psf
Add. flooring	3 psf
Misc.	4 psf
Total Floor Dead Load	**15.00 psf**
Roof Live Load	**30.00 psf**
Roof Dead Loads	
Finish	2 psf
Sheathing	2.2 psf
Framing	3.8 psf
Insulation	2 psf
Solar panels	5 psf
Misc.	5 psf
Total roof Dead Load	**20.00 psf**
Exterior Wall Dead Loads	
3” Gunite concrete	33.75 psf
Framing	2 psf
Insulation	2.05 psf
Drywall	2.2 psf
Misc.	5 psf
Total wall Dead Load	**45 psf**

Interior Wall Dead Loads

Drywall	2.2 psf
Framing	2 psf
Insulation	2.05 psf
Drywall	2.2 psf
Misc.	5 psf
Total wall Dead Load	**13.45 psf**

LOAD SUMMARY

	Dead load (DL), psf	Live Load (LL), psf
Roof	20	30
Floor	15	40
Wall	45 / 13.45	

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

S =snow load

R =rain load

W =wind load

E =earthquake load

SNOW LOADS

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Snow

Results:

Ground Snow Load, p_g :	0 lb/ft^2
Mapped Elevation:	3.3 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Mon Dec 18 2023

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

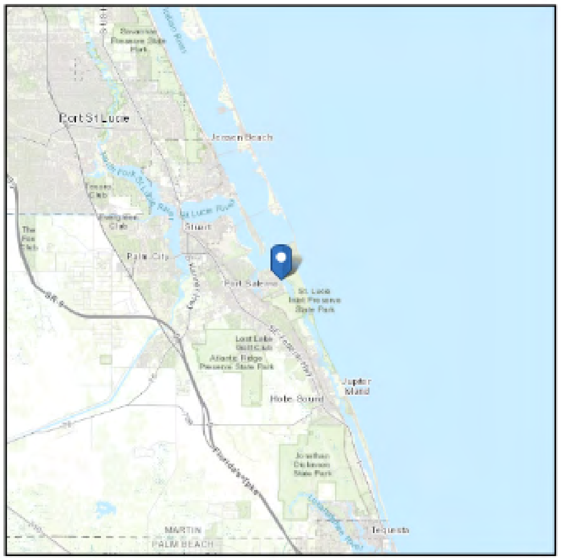
Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported ‘elevation’ and ‘mapped elevation’ differ significantly from each other.

WIND LOADS

ASCE 7 Hazards Report

ASCE AMERICAN SOCIETY OF CIVIL ENGINEERS

Address:		**Standard:**	ASCE/SEI 7-16	**Latitude:**	27.154874
No Address at This Location		**Risk Category:**	II	**Longitude:**	-80.172908
		Soil Class:	D - Stiff Soil	**Elevation:**	3.3336851526096796 ft (NAVD 88)



Wind

Results:

Wind Speed	165 Vmph
10-year MRI	88 Vmph
25-year MRI	109 Vmph
50-year MRI	123 Vmph
100-year MRI	134 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2

Date Accessed: Mon Dec 18 2023

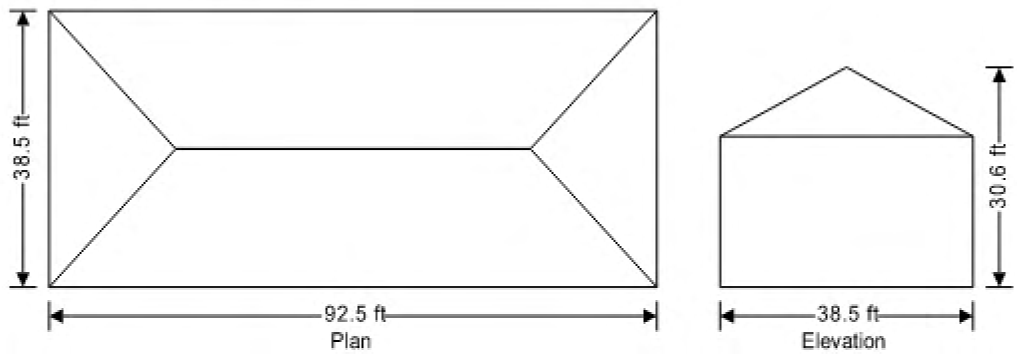
Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2. Glazed openings shall be protected against wind-borne debris as specified in Section 26.12.3.

WIND LOADING

In accordance with ASCE7-16

Using the directional design method



Building data

Type of roof	Hipped
Length of building	b = **92.50** ft
Width of building	d = **38.50** ft
Height to eaves	H = **21.00** ft
Pitch of main slope	α_0 = **26.6** deg
Pitch of gable slope	α_{90} = **26.6** deg
Mean height	h = **25.81** ft

General wind load requirements

Basic wind speed	V = **165.0** 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	$K_e = \exp(-0.0000362 * z_{gl}/1ft)$ = **1.00**
Exposure category (cl 26.7.3)	D
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)
15.00	1.03	61.02
15.00	1.03	61.02
21.00	1.09	64.45
25.81	1.13	66.74

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **66.74** psf

Pressures and forces

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

Net force $F_w = p * A_{ref}$

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)	25.81	-0.33	66.74	-30.51	1576.57	-48.10
B (-ve)	25.81	-0.60	66.74	-46.05	1576.57	-72.60
C (-ve)	25.81	-1.01	66.74	-69.36	186.26	-12.92
D (-ve)	25.81	-0.83	66.74	-59.20	462.43	-27.37
E (-ve)	25.81	-0.57	66.74	-44.24	179.95	-7.96

Total vertical net force $F_{w,v}$ = **-151.11** kips

Total horizontal net force $F_{w,h}$ = **10.96** 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	15.00	0.80	61.02	29.48	1387.50	40.90
A_2	15.00	0.80	61.02	29.48	0.00	0.00
A_3	21.00	0.80	64.45	31.82	555.00	17.66
B	25.81	-0.50	66.74	-40.38	1942.50	-78.43
C	25.81	-0.70	66.74	-51.72	808.50	-41.82
D	25.81	-0.70	66.74	-51.72	808.50	-41.82

Overall loading

Projected vertical plan area of wall	$A_{vert\_w\_0} = b * H =$ **1942.50** ft^2
Projected vertical area of roof	$A_{vert\_r\_0} = b * d/2 * \tan(\alpha_0) - (d/2 * \tan(\alpha_0))^2 / \tan(\alpha_{90}) =$ **705.18** ft^2
Minimum overall horizontal loading	$F_{w,total\_min} = p_{min\_w} * A_{vert\_w\_0} + p_{min\_r} * A_{vert\_r\_0} =$ **36.72** kips
Leeward net force	$F_l = F_{w,wB} =$ **-78.4** kips
Windward net force	$F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **58.6** kips
Overall horizontal loading	$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **148.0** 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)	25.81	0.15	66.74	20.70	1576.57	32.64
B (+ve)	25.81	-0.60	66.74	-22.02	1576.57	-34.72
C (+ve)	25.81	-0.18	66.74	1.80	186.26	0.34
D (+ve)	25.81	-0.18	66.74	1.80	462.43	0.83
E (+ve)	25.81	-0.18	66.74	1.80	179.95	0.32

Total vertical net force	$F_{w,v} =$ **-0.52** kips
Total horizontal net force	$F_{w,h} =$ **30.13** 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	15.00	0.80	61.02	53.51	1387.50	74.24
A_2	15.00	0.80	61.02	53.51	0.00	0.00
A_3	21.00	0.80	64.45	55.84	555.00	30.99
B	25.81	-0.50	66.74	-16.35	1942.50	-31.76
C	25.81	-0.70	66.74	-27.70	808.50	-22.39
D	25.81	-0.70	66.74	-27.70	808.50	-22.39

Overall loading

Projected vertical plan area of wall	$A_{vert\_w\_0} = b * H =$ **1942.50** ft^2
Projected vertical area of roof	$A_{vert\_r\_0} = b * d/2 * \tan(\alpha_0) - (d/2 * \tan(\alpha_0))^2 / \tan(\alpha_{90}) =$ **705.18** ft^2

Minimum overall horizontal loading	$F_{w,total\_min} = p_{min\_w} * A_{vert\_w\_0} + p_{min\_r} * A_{vert\_r\_0}$ = **36.72** kips
Leeward net force	$F_l = F_{w,wB}$ = **-31.8** kips
Windward net force	$F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **105.2** kips
Overall horizontal loading	$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **167.1** 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)	25.81	-0.21	66.74	-23.81	414.32	-9.86
B (-ve)	25.81	-0.60	66.74	-46.05	414.32	-19.08
C (-ve)	25.81	-0.90	66.74	-63.07	186.26	-11.75
D (-ve)	25.81	-0.90	66.74	-63.07	510.60	-32.20
E (-ve)	25.81	-0.50	66.74	-40.38	1111.17	-44.86
F (-ve)	25.81	-0.30	66.74	-29.03	1345.10	-39.05

Total vertical net force	$F_{w,v}$ = **-140.24** kips
Total horizontal net force	$F_{w,h}$ = **4.12** 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	61.02	29.48	577.50	17.02
A_2	15.00	0.80	61.02	29.48	0.00	0.00
A_3	21.00	0.80	64.45	31.82	231.00	7.35
B	25.81	-0.28	66.74	-27.89	808.50	-22.55
C	25.81	-0.70	66.74	-51.72	1942.50	-100.47
D	25.81	-0.70	66.74	-51.72	1942.50	-100.47

Overall loading

Projected vertical plan area of wall	$A_{vert\_w\_90} = d * H$ = **808.50** ft^2
Projected vertical area of roof	$A_{vert\_r\_90} = d^2/4 * \tan(\alpha_0)$ = **185.32** ft^2
Minimum overall horizontal loading	$F_{w,total\_min} = p_{min\_w} * A_{vert\_w\_90} + p_{min\_r} * A_{vert\_r\_90}$ = **14.42** kips
Leeward net force	$F_l = F_{w,wB}$ = **-22.5** kips
Windward net force	$F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **24.4** kips
Overall horizontal loading	$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **51.0** 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)	25.81	0.29	66.74	28.37	414.32	11.75
B (+ve)	25.81	-0.60	66.74	-22.02	414.32	-9.12
C (+ve)	25.81	-0.18	66.74	1.80	186.26	0.34
D (+ve)	25.81	-0.18	66.74	1.80	510.60	0.92
E (+ve)	25.81	-0.18	66.74	1.80	1111.17	2.00
F (+ve)	25.81	-0.18	66.74	1.80	1345.10	2.42

Total vertical net force $F_{w,v}$ = **7.43** kips

Total horizontal net force $F_{w,h}$ = **9.33** 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	61.02	53.51	577.50	30.90
A_2	15.00	0.80	61.02	53.51	0.00	0.00
A_3	21.00	0.80	64.45	55.84	231.00	12.90
B	25.81	-0.28	66.74	-3.86	808.50	-3.12
C	25.81	-0.70	66.74	-27.70	1942.50	-53.80
D	25.81	-0.70	66.74	-27.70	1942.50	-53.80

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d * H =$ **808.50** ft^2

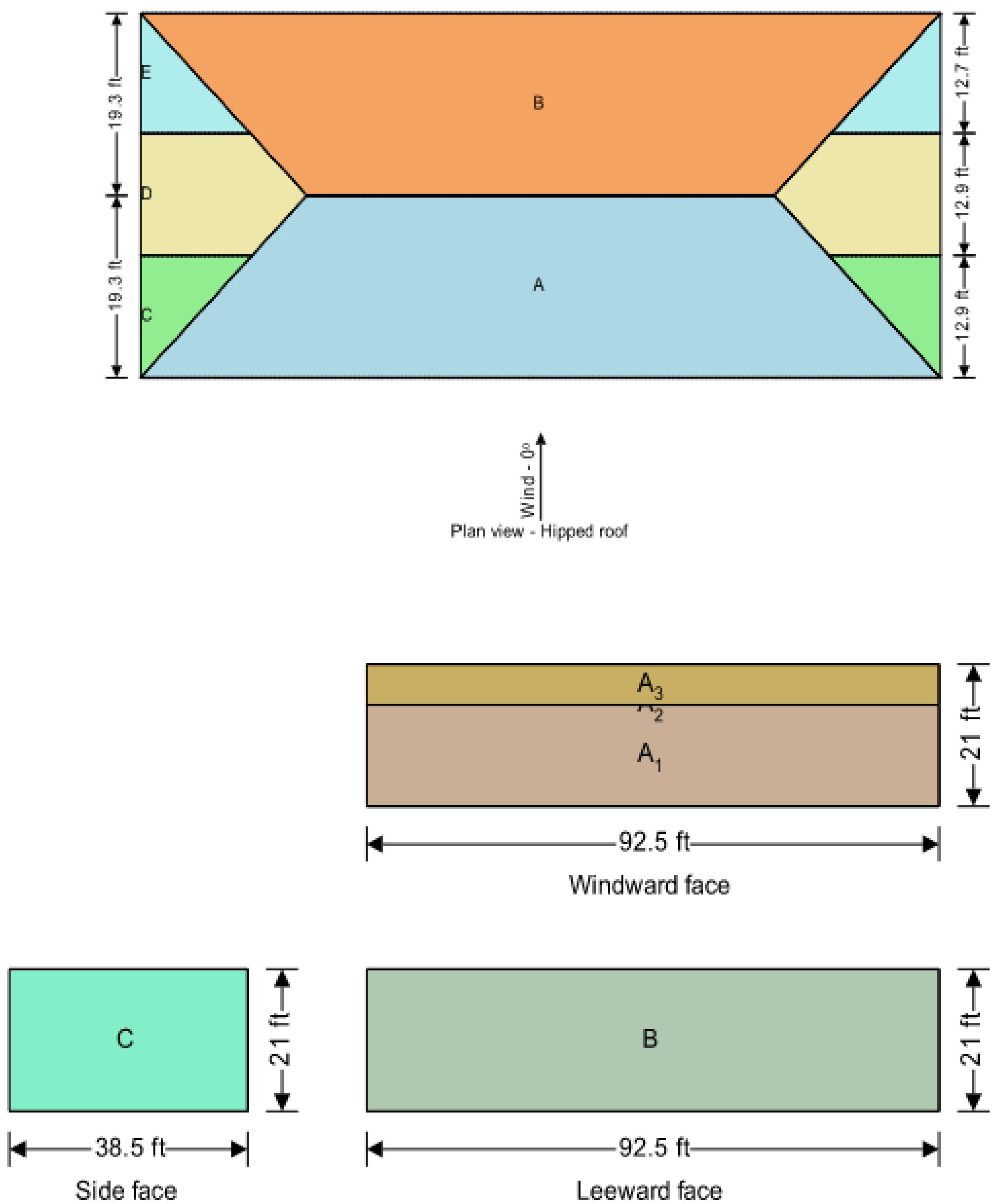
Projected vertical area of roof $A_{vert\_r\_90} = d^2/4 * \tan(\alpha_0) =$ **185.32** ft^2

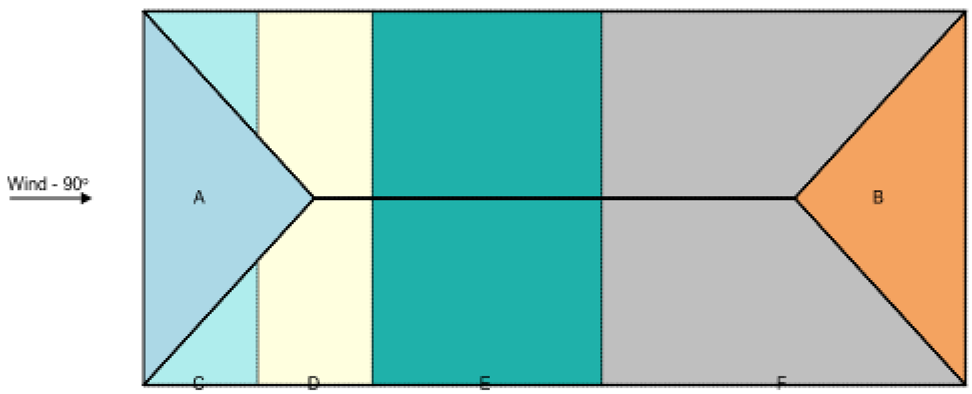
Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} * A_{vert\_w\_90} + p_{min\_r} * A_{vert\_r\_90} =$ **14.42** kips

Leeward net force $F_l = F_{w,wB} =$ **-3.1** kips

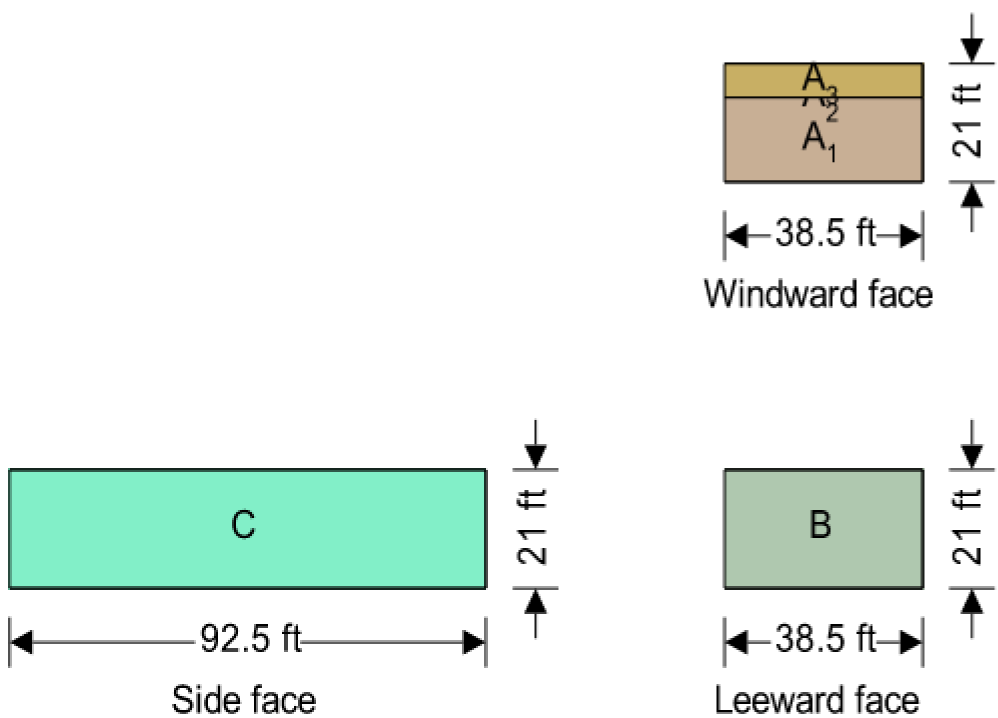
Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **43.8** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **56.3** kips


19.3 ft
19.3 ft
E
D
C
B
A
12.7 ft
12.9 ft
12.9 ft
Wind - 0º
Plan view - Hipped roof
A3
A2
A1
21 ft
92.5 ft
Windward face
C
21 ft
38.5 ft
Side face
B
21 ft
92.5 ft
Leeward face



Plan view - Hipped roof



SEISMIC LOADS

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

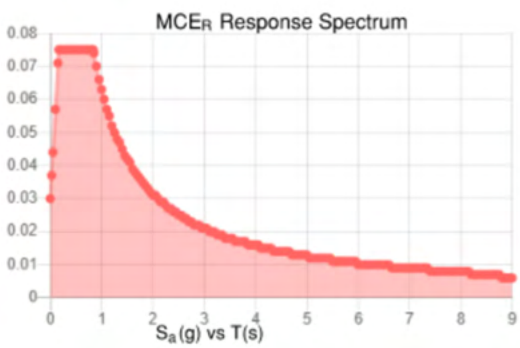
Seismic

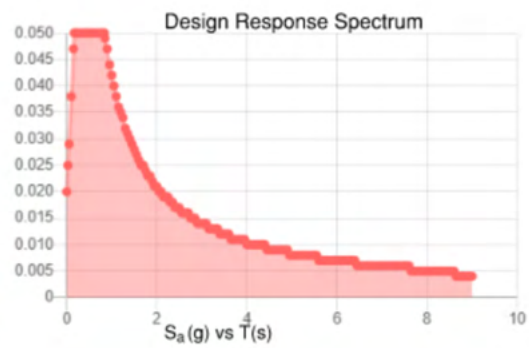
Seismic Design Category: A

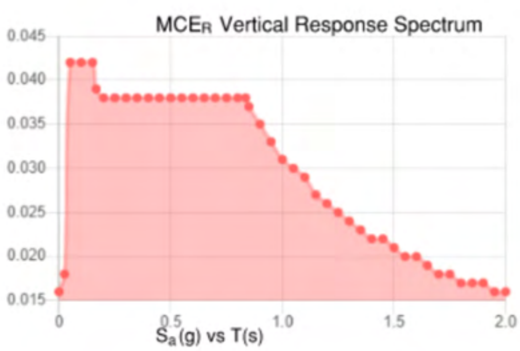
Site Soil Class: D - Stiff Soil

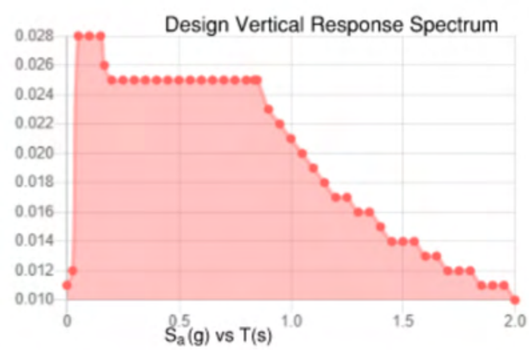
Results:

S_S :	0.047	S_{D1} :	0.042
S_1 :	0.026	T_L :	8
F_a :	1.6	PGA :	0.022
F_v :	2.4	PGA_M :	0.035
S_{MS} :	0.075	F_{PGA} :	1.6
S_{M1} :	0.063	I_e :	1
S_{DS} :	0.05	C_v :	0.7









Data Accessed: **Mon Dec 18 2023**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

2. STRUCTURAL ANALYSIS

2.1 BUILDING DESCRIPTION

The two-storey building is complexly shaped in plan, the building is 92'-10" long and 39'-7" wide. The longitudinal walls of the first floor are sized to extend beyond the longitudinal walls of the ground floor and are supported by the floor structures. Walls, floor and roof trusses are designed of LGS structural framing.
The stability of the building is provided by a vertical x-bracing strap and sprayed concrete shear wall.
Foundation is a slab-on-grade with footings where required for steel columns.

The design of the structure is based on the requirements of the 2021 Edition of the International Building Code and reference codes.

The analytical model is presented as a spatial model.

Structural rigidity in the longitudinal direction is achieved due to the sprayed concrete shear wall, transverse directions are achieved due sprayed concrete shear wall and to x-brace strap.

Structural analysis is done in SCIA Engineer 20 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 B2: Steel special concentrically braced frames.

To determine the design forces from dynamic loads (earthquake), two types of calculations are used: Modal Mode, Equivalent Lateral Forces or 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.

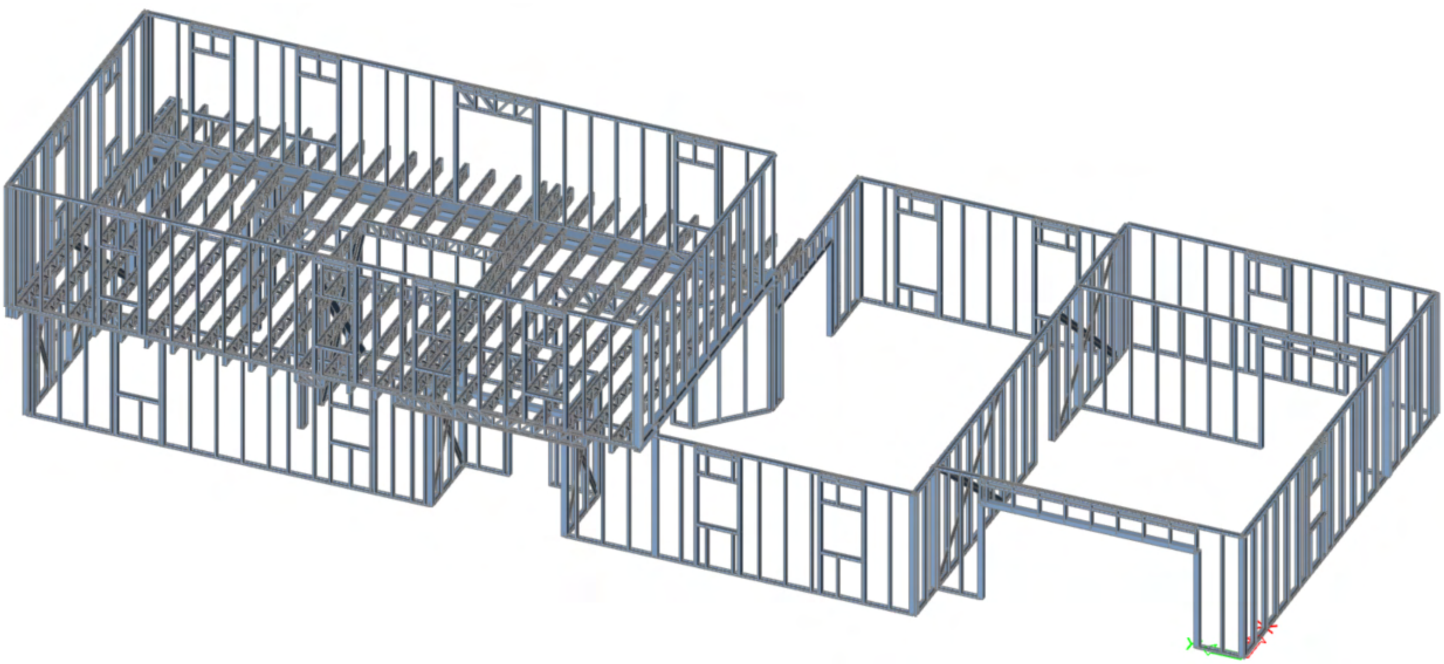
Response Spectrum Method:
Seismic analysis is based on the response spectrum method. All data is defined the same way as in modal analysis. Additionally, parameters required by a specific national code to establish the response spectrum shape must be specified. Calculations and results are the same as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

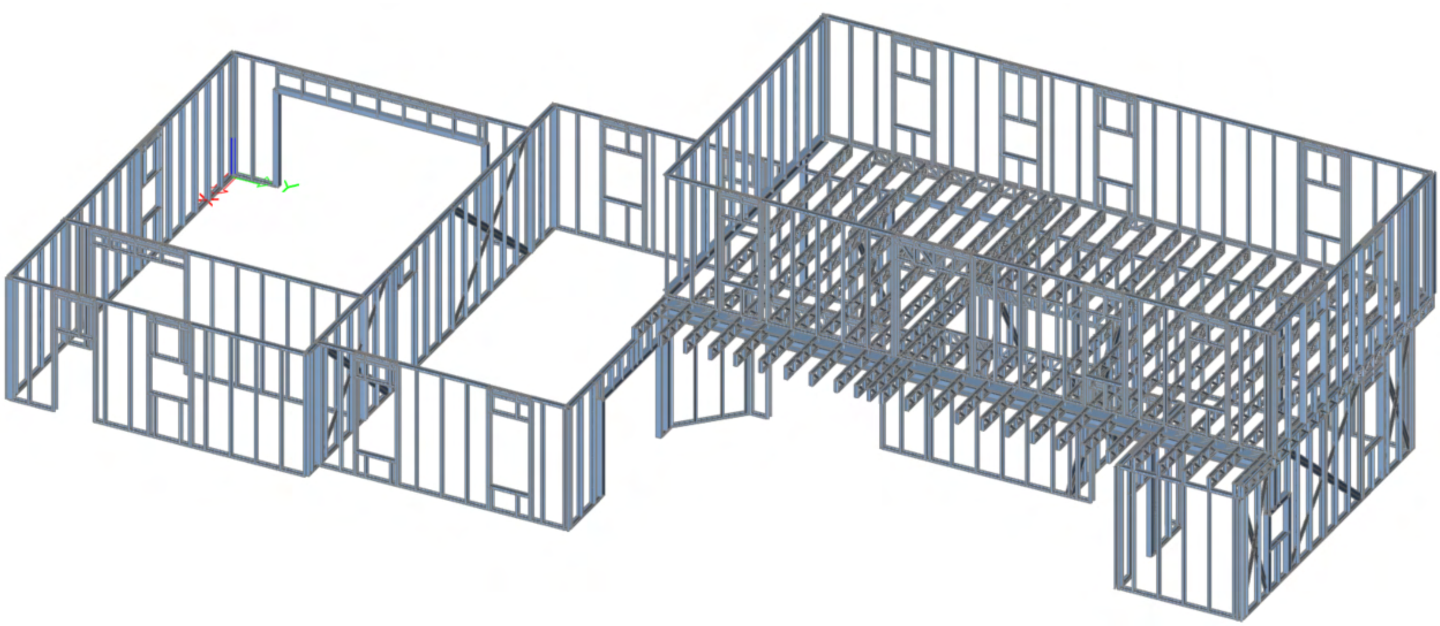
- Seismic excitation multiplier (value of the accelerating excitation spectrum).
- Seismic participation factors calculated as those for the modal analysis. However, vector D describing excitation direction is user defined. Coefficients are specified for each dynamic degree of freedom according to the 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.
- Pseudostatic 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.

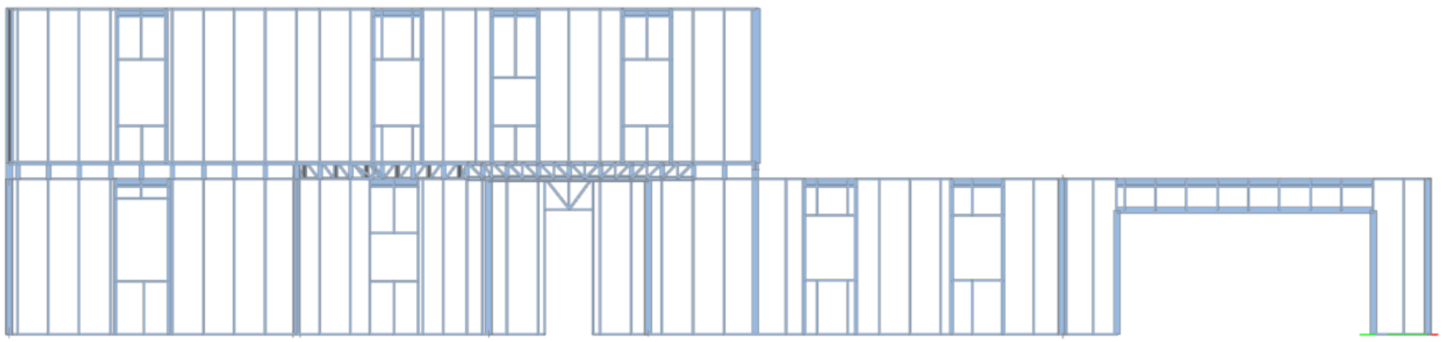
2.2 GENERAL SCHEME

General views

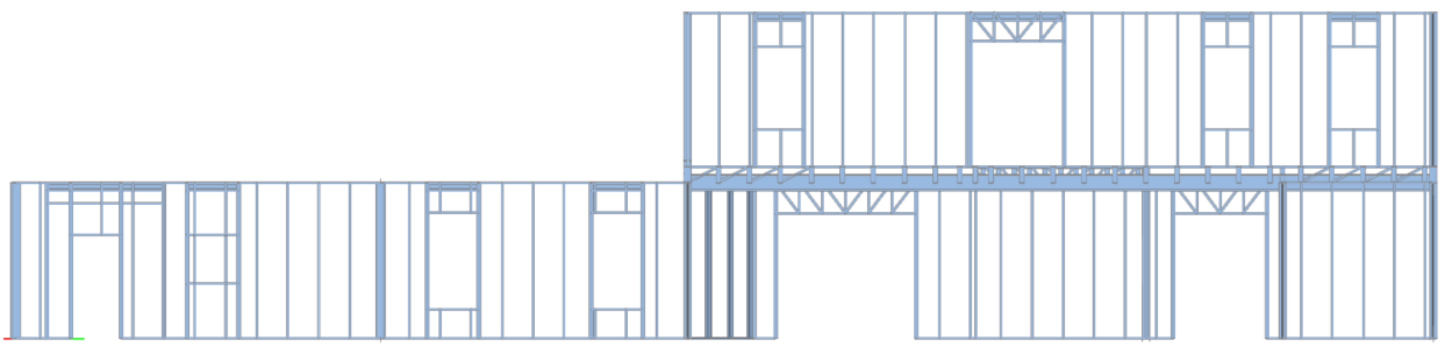




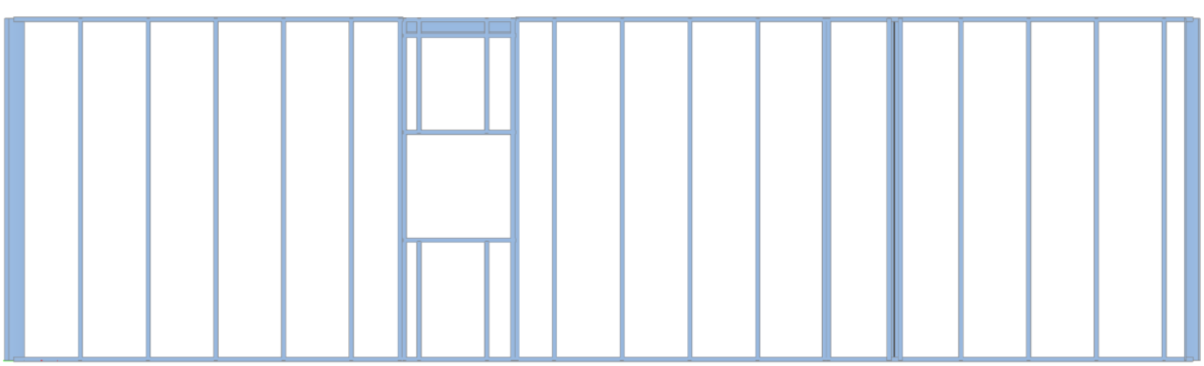
FRONT ELEVATION VIEW



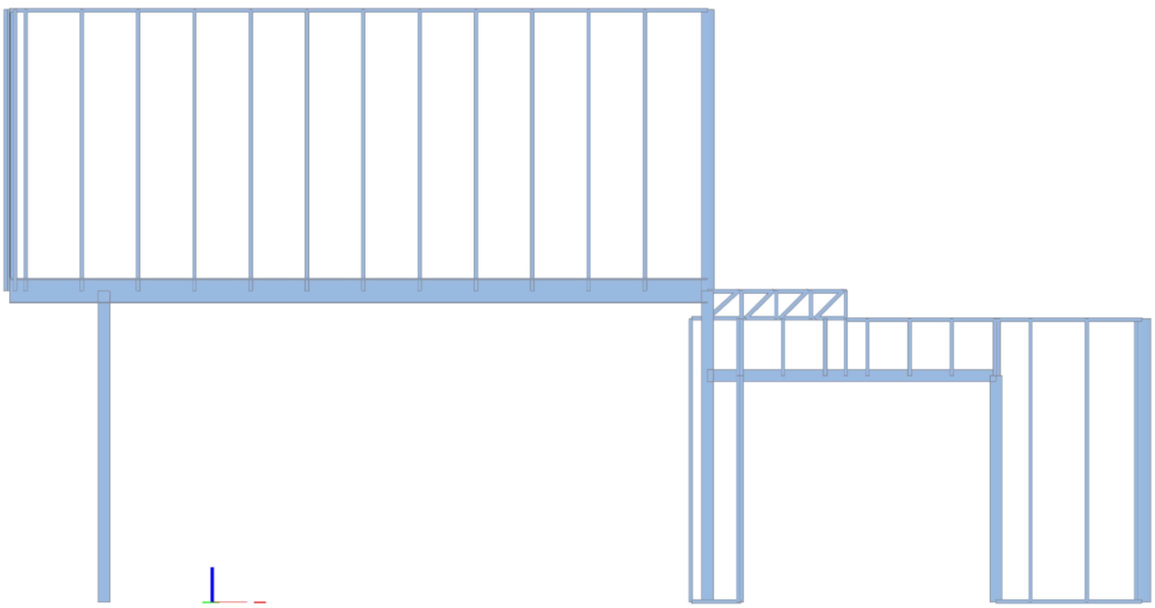
FRONT ELEVATION VIEW



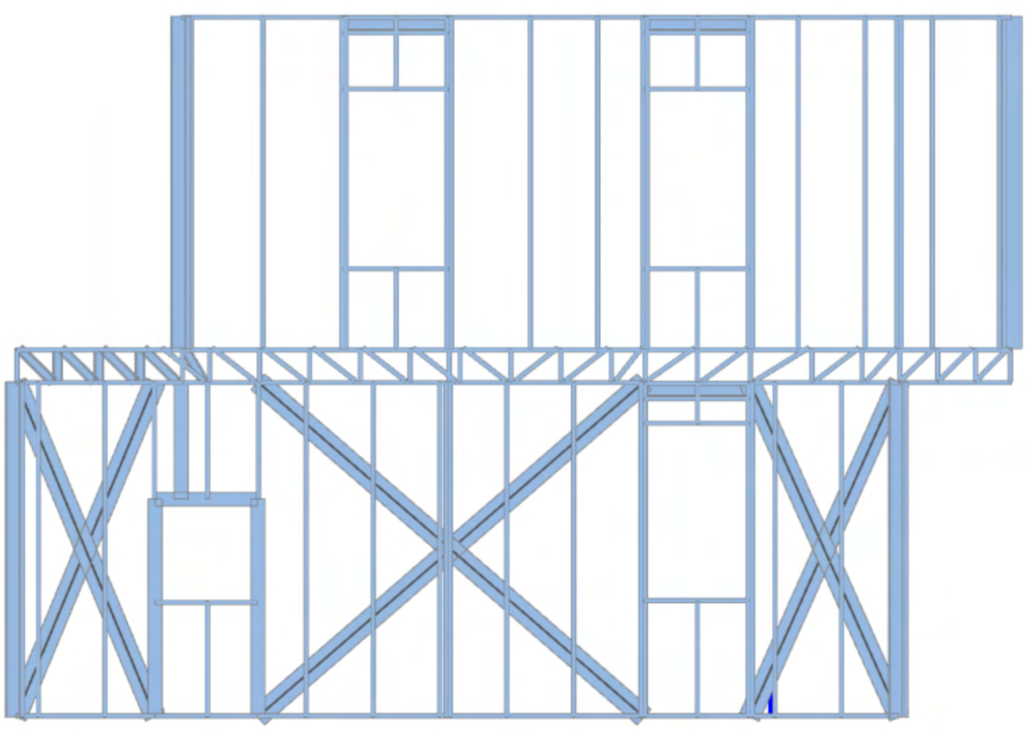
RIGHT SIDE ELEVATION VIEW



SECOND STORY RIGHT SIDE ELEVATION VIEW



LEFT SIDE ELEVATION VIEW



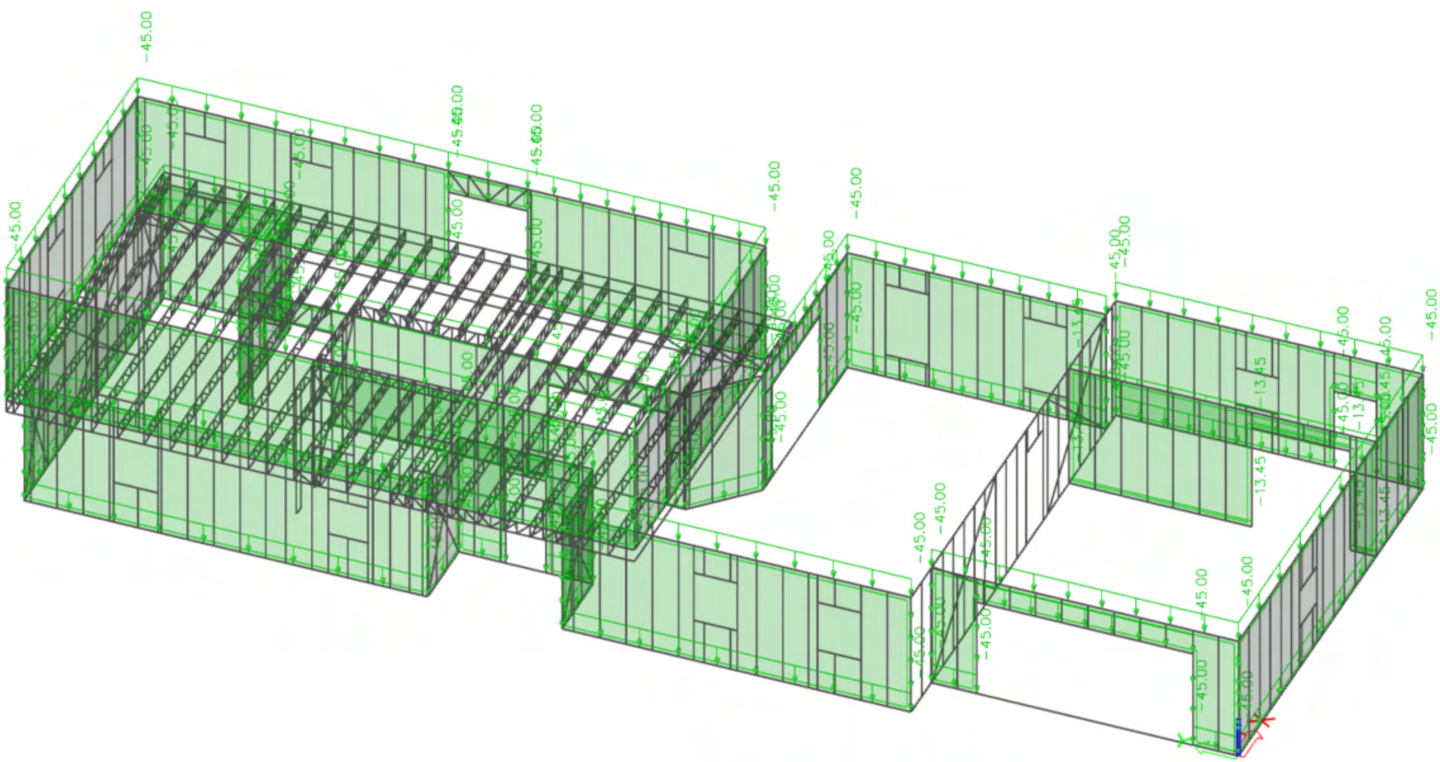
2.3 APPLIED LOADS

1. DL2 Dead weight of steel structures (automatically)

2. DL2 Wall dead load

Dead loads distribution

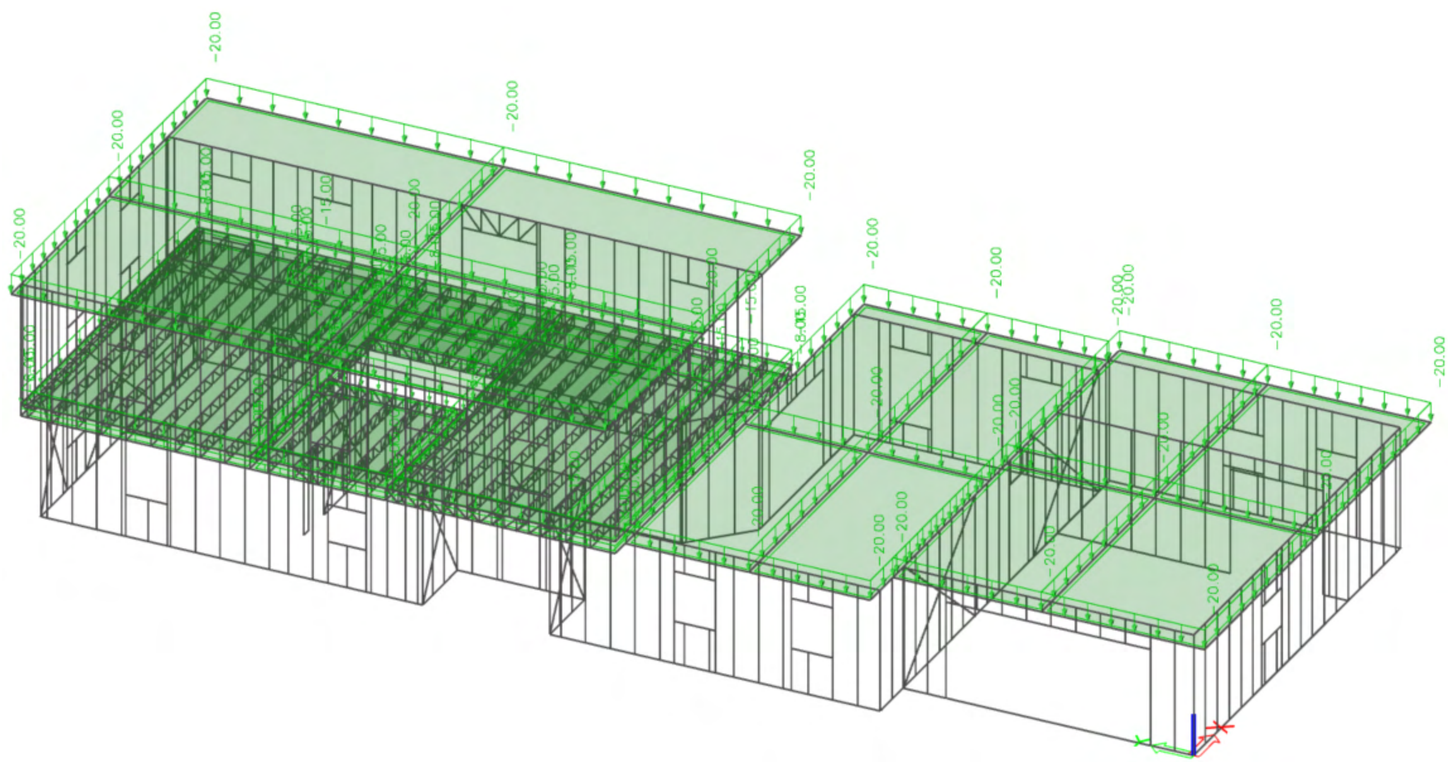
DL = 45 psf / DL = 13.45 psf



3. DL3 Roof, Floor, Ceiling dead load

Dead loads distribution

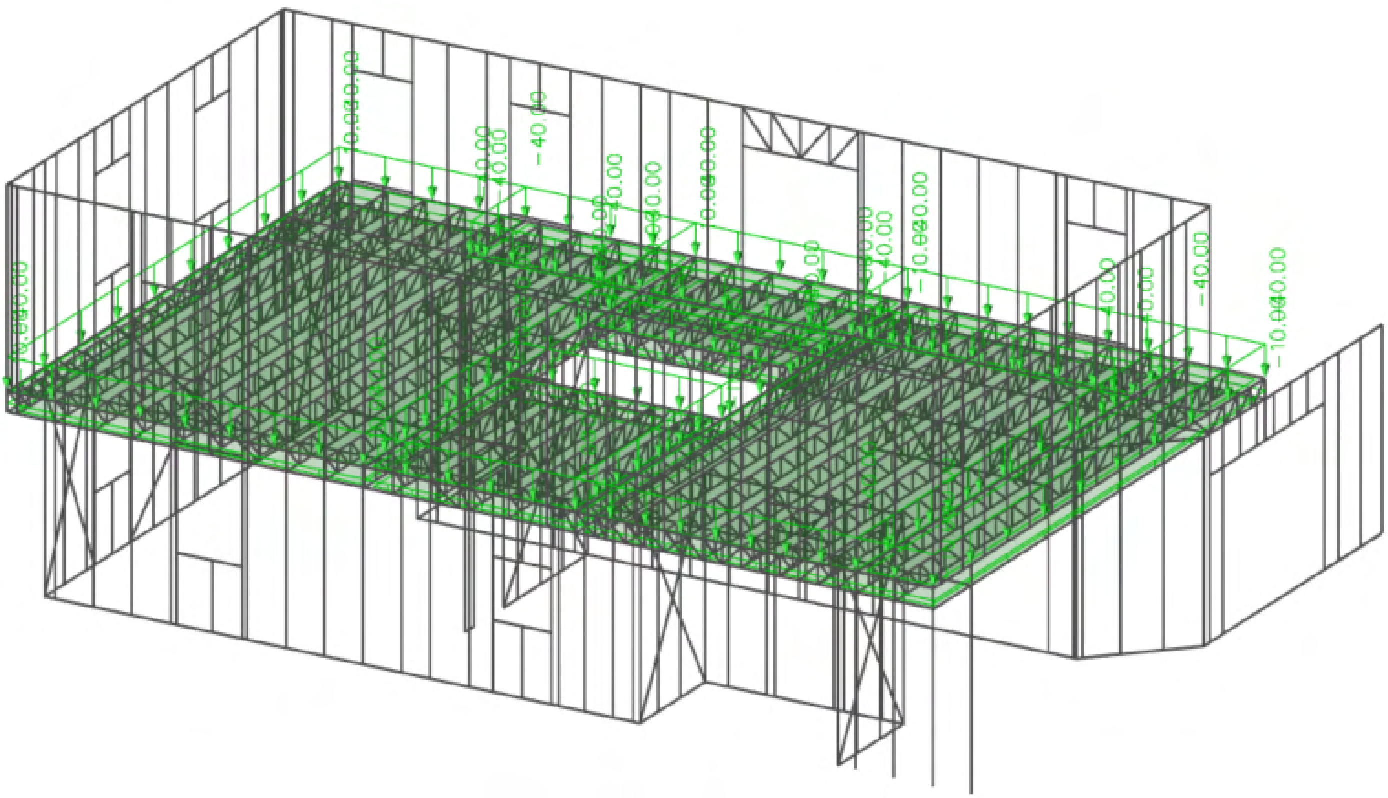
Roof dead load - 20 psf. Floor dead load - 15 psf. Ceiling dead load - 8 psf.



4. L Live load

Live loads distribution

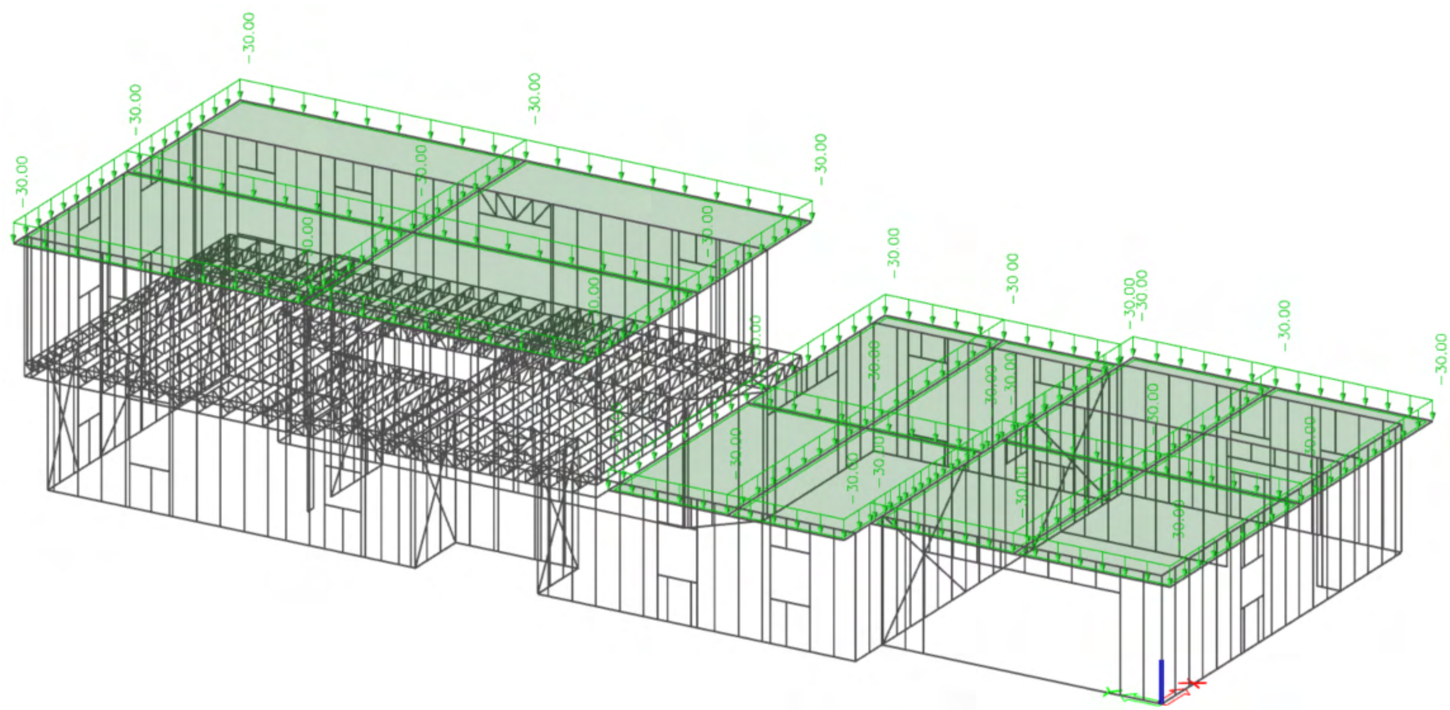
L=40 psf floor live load / L=10 psf ceiling live load



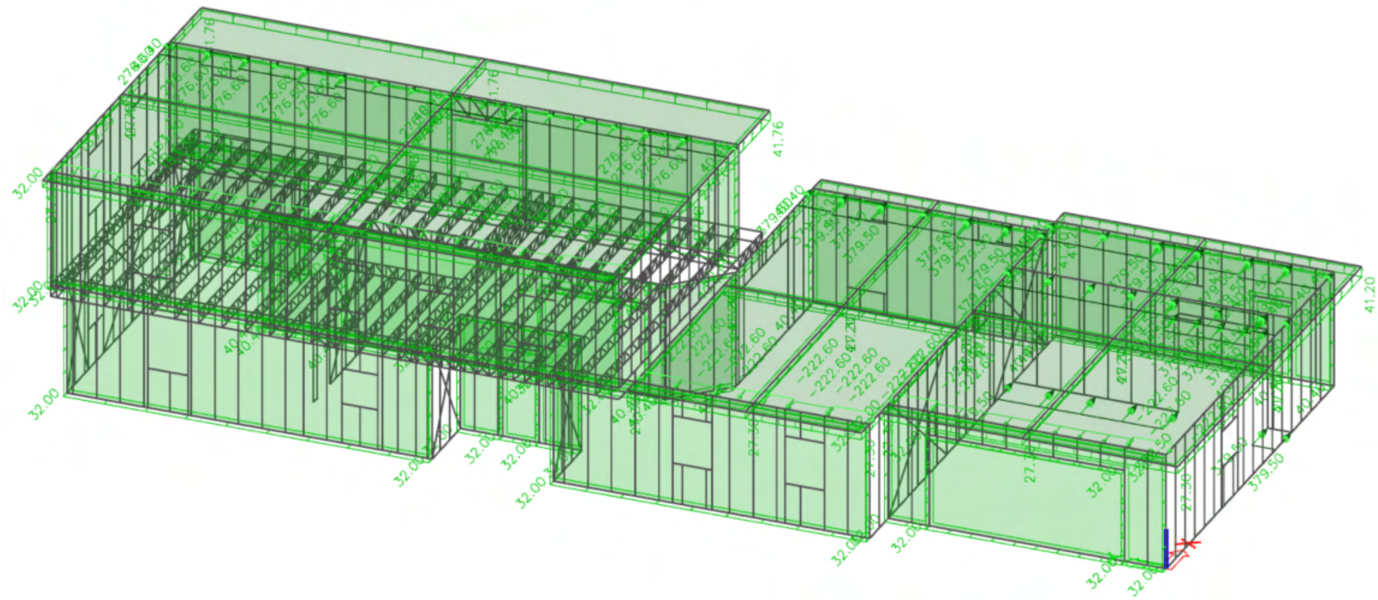
5. LR Roof live load

Live loads distribution

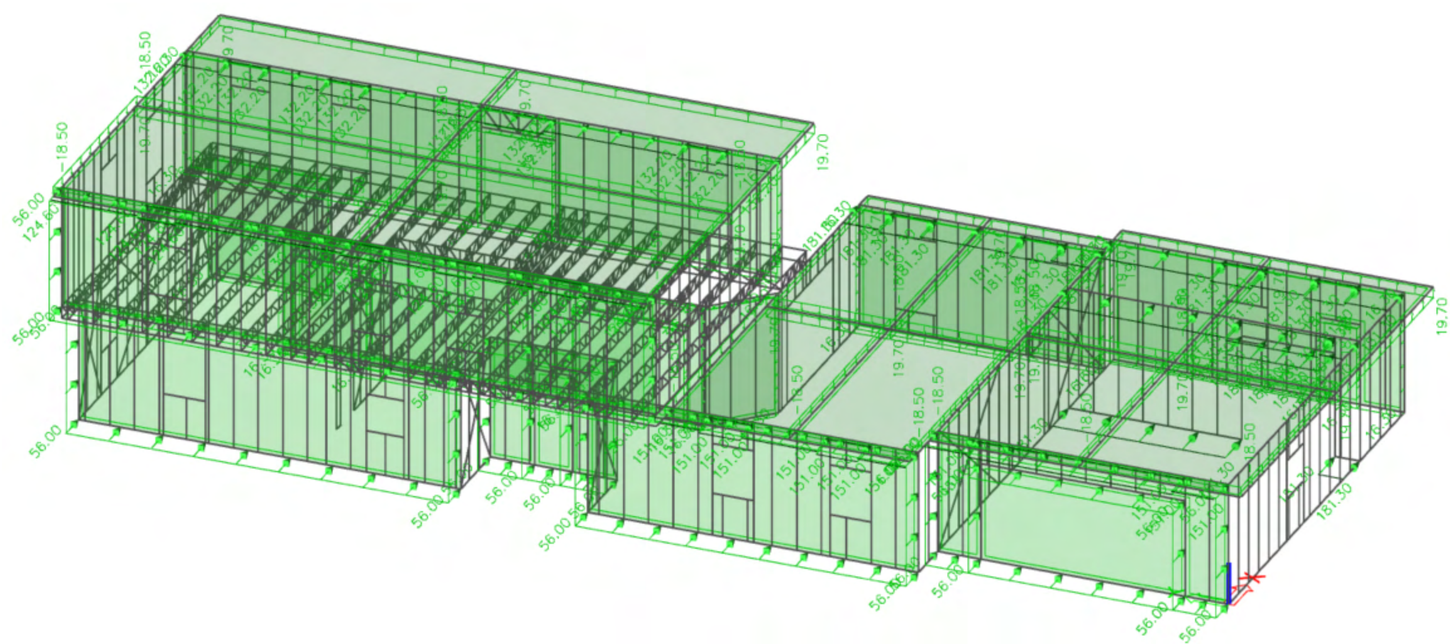
Lr=30psf



6. **Wx+ (0.18) Wind load**
Windward wind pressure - 31.82 psf, Leeward wind pressure - 40.32 psf,
Winward Roof wind pressure - 30.51 psf, Leeward Roof wind pressure - 46.05 psf

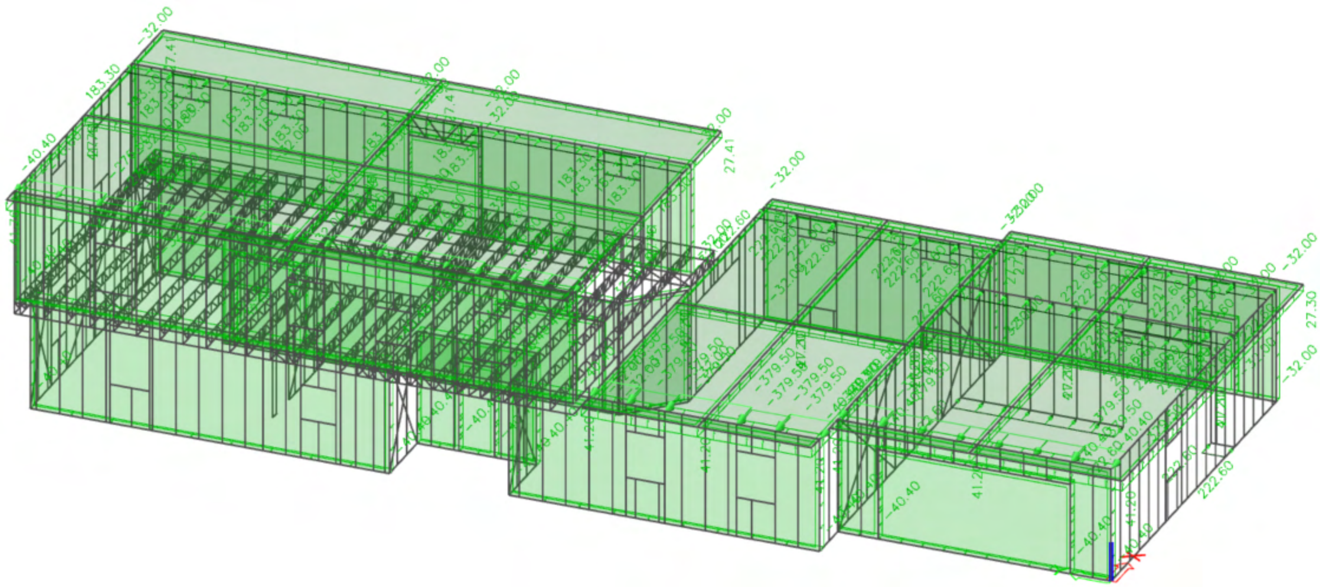


7. **Wx+ (-0.18) Wind load**
Windward wind pressure - 55.93 psf, Leeward wind pressure - 16.35 psf,
Winward Roof wind pressure - 20.7 psf, Leeward Roof wind pressure - 22.02 psf (Up)



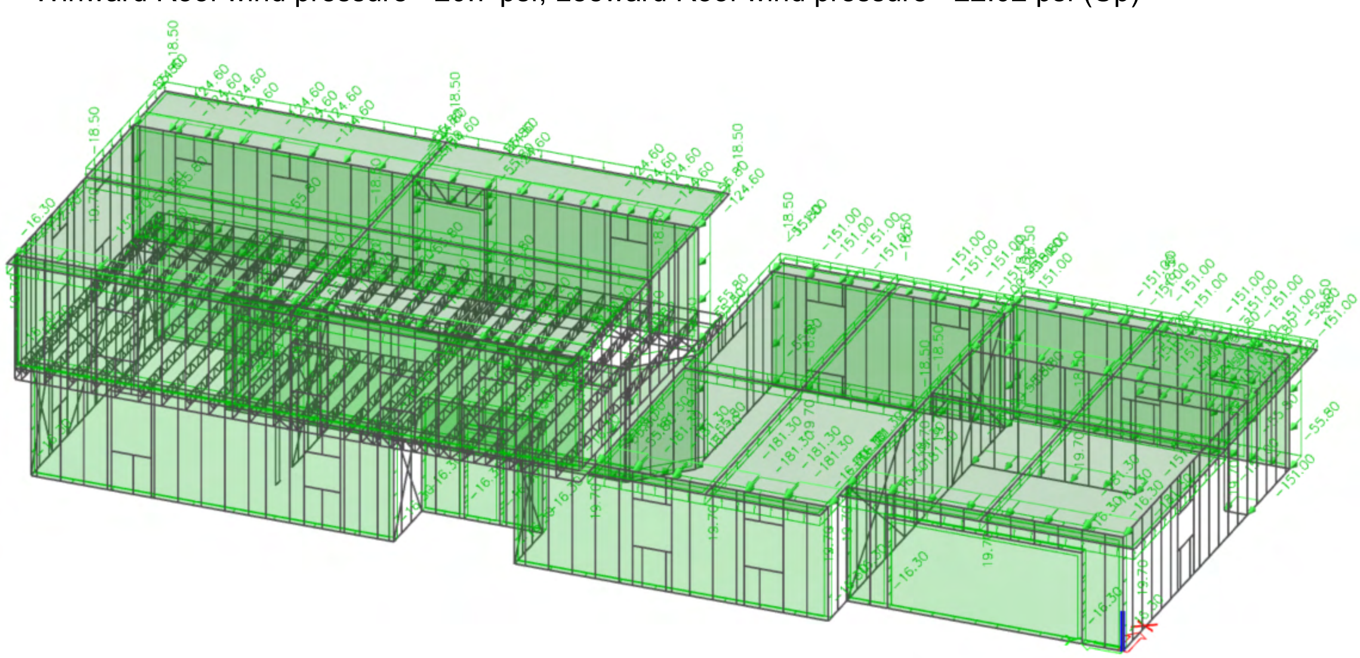
8. Wx- (0.18) Wind load

Windward wind pressure - 31.82 psf, Leeward wind pressure - 40.32 psf,
Winward Roof wind pressure - 30.51 psf (Up), Leeward Roof wind pressure - 46.05 psf (Up)



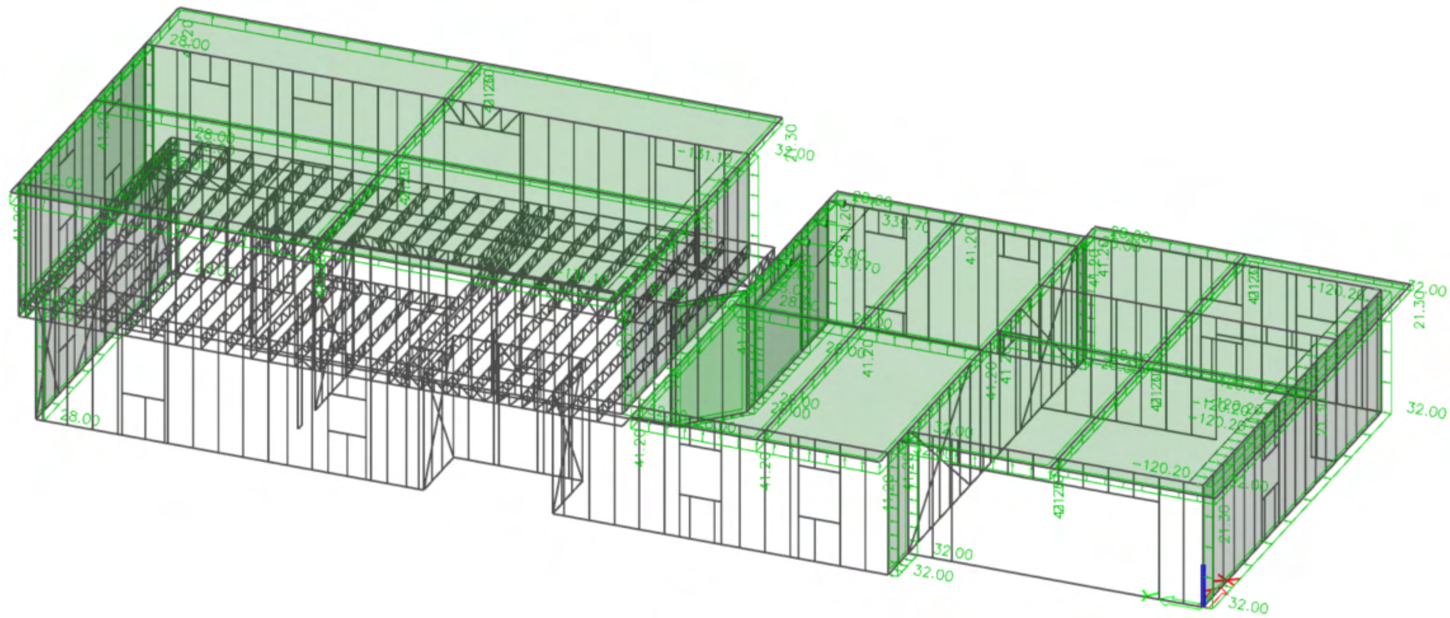
8. Wx- (-0.18) Wind load

Windward wind pressure - 55.93 psf, Leeward wind pressure - 16.35 psf,
Winward Roof wind pressure - 20.7 psf, Leeward Roof wind pressure - 22.02 psf (Up)



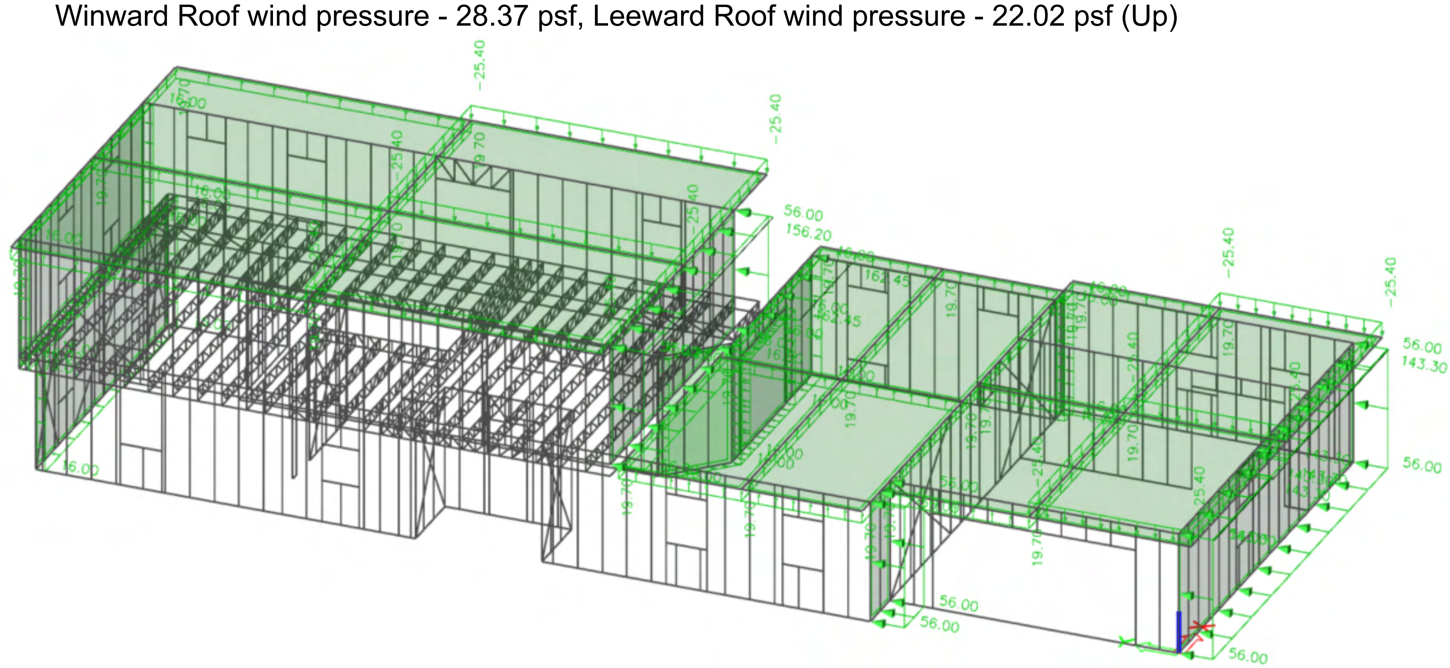
10. Wy+ (0.18) Wind load

Windward wind pressure - 31.82 psf, Leeward wind pressure - 27.89 psf,
Winward Roof wind pressure - 23.81 psf (Up), Leeward Roof wind pressure - 46.05 psf (Up)



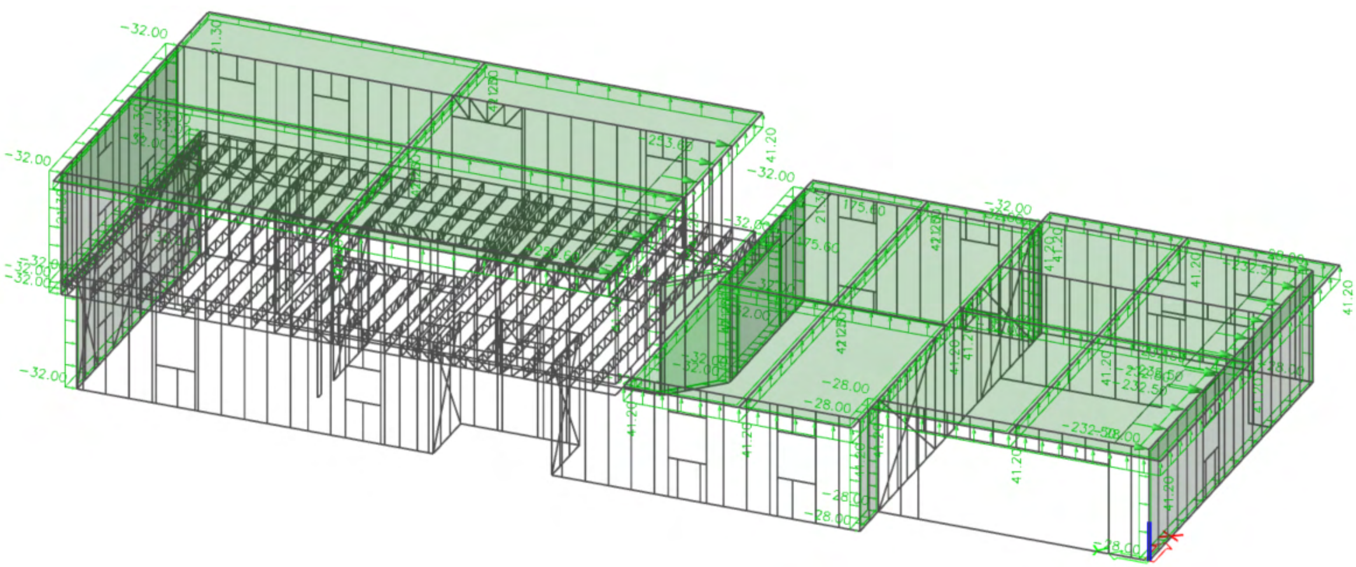
11. Wy+ (-0.18) Wind load

Windward wind pressure - 55.86 psf, Leeward wind pressure - 16.00 psf,
Winward Roof wind pressure - 28.37 psf, Leeward Roof wind pressure - 22.02 psf (Up)



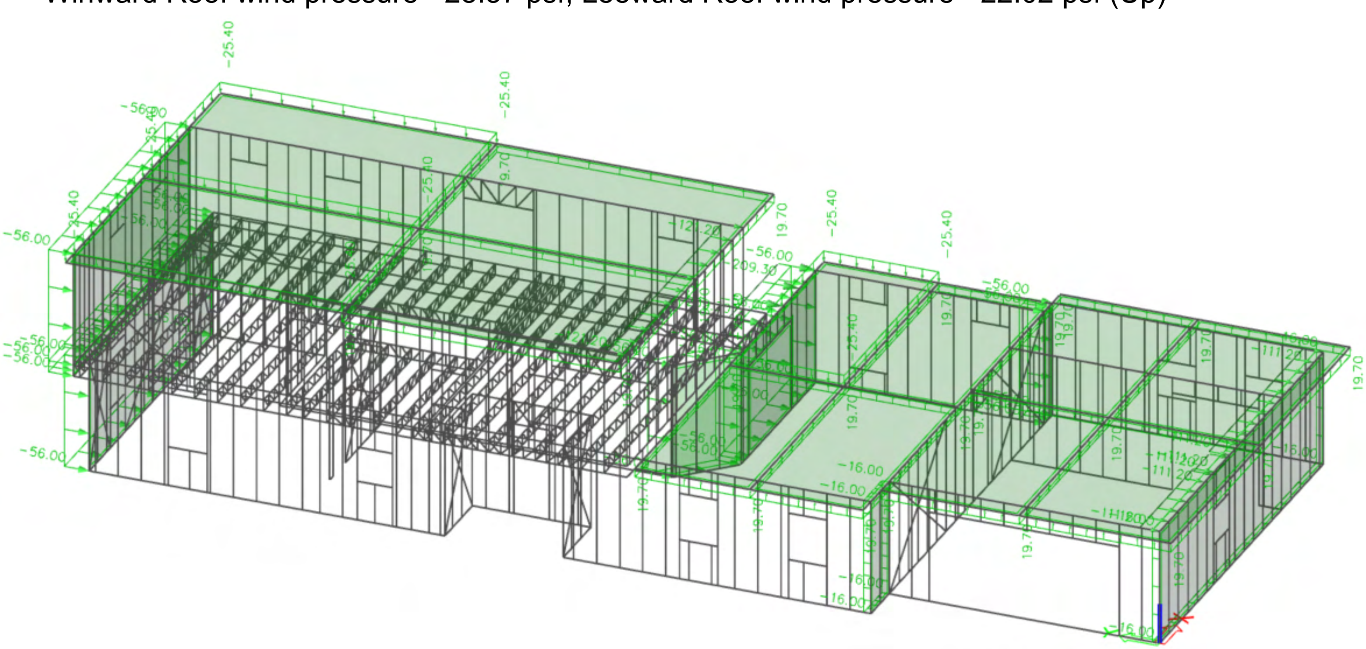
12. Wy- (0.18) Wind load

Windward wind pressure - 31.82 psf, Leeward wind pressure - 27.89 psf,
Winward Roof wind pressure - 23.81 psf (Up), Leeward Roof wind pressure - 46.05 psf (Up)



13. Wy- (-0.18) Wind load

Windward wind pressure - 55.86 psf, Leeward wind pressure - 16.00 psf,
Winward Roof wind pressure - 28.37 psf, Leeward Roof wind pressure - 22.02 psf (Up)



Load cases

Case # **Case name**

Name	Description Spec	Action type / Load type	Load group	Direction	Duration
DL1	Self weight	Permanent / Self weight	DL	-Z	
DL2	Wall dead load	Permanent / Standard	DL		
DL3	Roof, Floor, Ceiling dead load	Permanent / Standard	DL		
L	Live Load Floor	Variable / Static	L		Short
Lr	Roof Live Load Roof	Variable / Static	Lr		Short
Wx+(+0.18)	Wind Load	Variable / Static	W		Short
Wx+(-0.18)	Wind Load	Variable / Static	W		Short
Wx-(+0.18)	Wind Load	Variable / Static	W		Short
Wx-(-0.18)	Wind Load	Variable / Static	W		Short
Wy+(+0.18)	Wind Load	Variable / Static	W		Short
Wy+(-0.18)	Wind Load	Variable / Static	W		Short
Wy-(+0.18)	Wind Load	Variable / Static	W		Short
Wy-(-0.18)	Wind Load	Variable / Static	W		Short
Ex	Seismic Load	Variable / Static equivalent	E		
Ey	Seismic Load	Variable / Static equivalent	E		

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

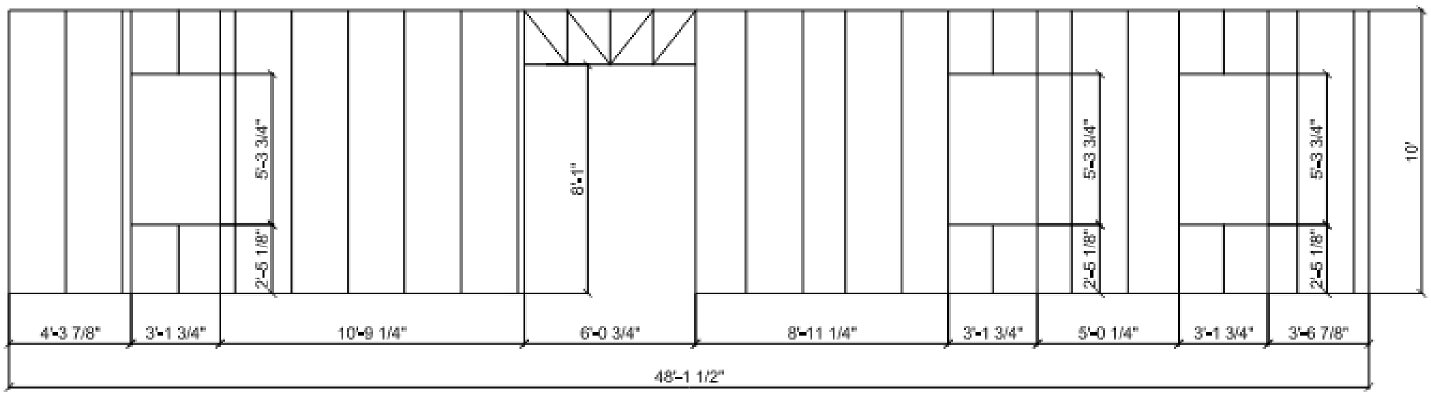
#	Comb. Name	Load cases
22	LRFD-Ult (auto) 22	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy+(-0.18)
23	LRFD-Ult (auto) 23	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy-(+0.18)
24	LRFD-Ult (auto) 24	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy-(-0.18)
25	LRFD-Ult (auto) 25	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Wx+(+0.18)
26	LRFD-Ult (auto) 26	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Wx+(-0.18)
27	LRFD-Ult (auto) 27	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Wx-(+0.18)
28	LRFD-Ult (auto) 28	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr
29	LRFD-Ult (auto) 29	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Wx-(-0.18)
30	LRFD-Ult (auto) 30	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Wy+(+0.18)
31	LRFD-Ult (auto) 31	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Wy+(-0.18)
32	LRFD-Ult (auto) 32	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Wy-(+0.18)
33	LRFD-Ult (auto) 33	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Wy-(-0.18)
34	LRFD-Ult (auto) 34	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Wx+(+0.18)
35	LRFD-Ult (auto) 35	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + 1.0*Wx+(+0.18)
36	LRFD-Ult (auto) 36	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Wx+(-0.18)
37	LRFD-Ult (auto) 37	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + 1.0*Wx+(-0.18)
38	LRFD-Ult (auto) 38	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Wx-(+0.18)
39	LRFD-Ult (auto) 39	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + 1.0*Wx-(+0.18)
40	LRFD-Ult (auto) 40	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Wx-(-0.18)
41	LRFD-Ult (auto) 41	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + 1.0*Wx-(-0.18)
42	LRFD-Ult (auto) 42	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Wy+(+0.18)

#	Comb. Name	Load cases
43	LRFD-Ult (auto) 43	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + 1.0*Wy+(+0.18)
44	LRFD-Ult (auto) 44	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Wy+(-0.18)
45	LRFD-Ult (auto) 45	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + 1.0*Wy+(-0.18)
46	LRFD-Ult (auto) 46	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Wy-(+0.18)
47	LRFD-Ult (auto) 47	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + 1.0*Wy-(+0.18)
48	LRFD-Ult (auto) 48	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Wy-(-0.18)
49	LRFD-Ult (auto) 49	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + 1.0*Wy-(-0.18)
50	LRFD-Ult (auto) 50	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.0*Wx+(+0.18)
51	LRFD-Ult (auto) 51	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.0*Wx+(-0.18)
52	LRFD-Ult (auto) 52	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.0*Wx-(+0.18)
53	LRFD-Ult (auto) 53	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.0*Wx-(-0.18)
54	LRFD-Ult (auto) 54	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.0*Wy+(+0.18)
55	LRFD-Ult (auto) 55	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.0*Wy+(-0.18)
56	LRFD-Ult (auto) 56	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.0*Wy-(+0.18)
57	LRFD-Ult (auto) 57	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.0*Wy-(-0.18)
58	LRFD-Ult (auto) 58	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Ex
59	LRFD-Ult (auto) 59	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + (-1.0)*Ex
60	LRFD-Ult (auto) 60	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.0*Ey
61	LRFD-Ult (auto) 61	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + (-1.0)*Ey
62	LRFD-Ult (auto) 62	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Ex
63	LRFD-Ult (auto) 63	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + (-1.0)*Ex

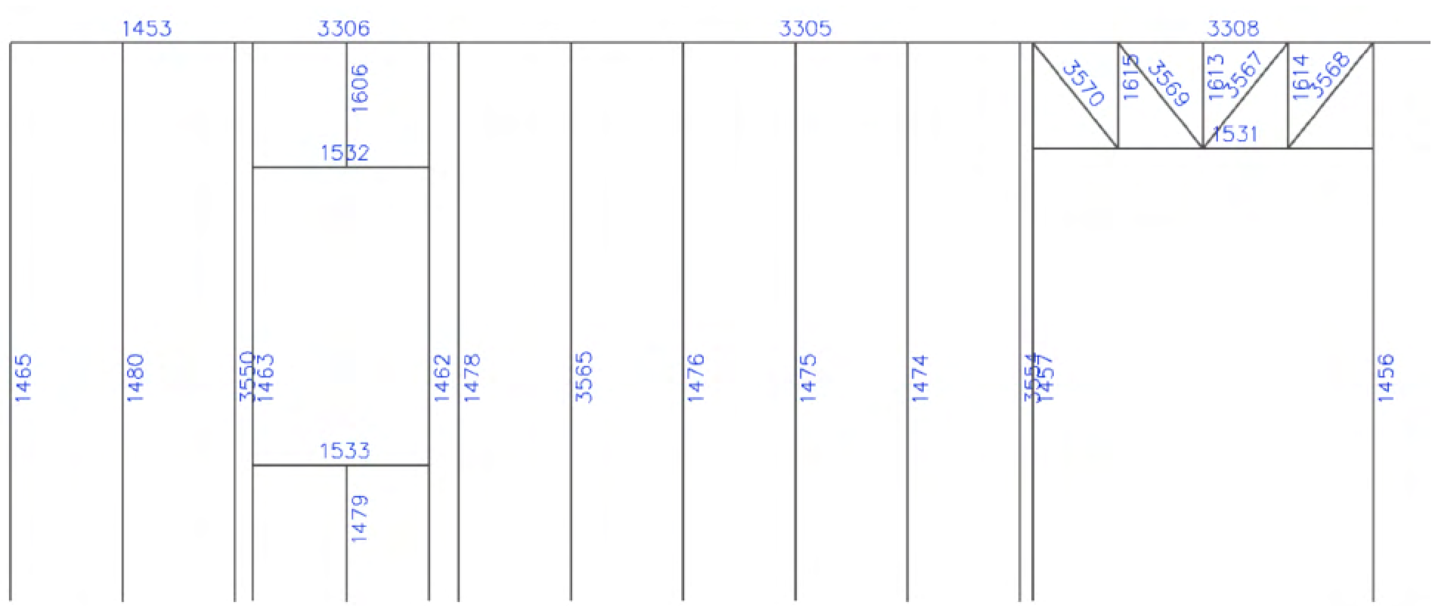
#	Comb. Name	Load cases
64	LRFD-Ult (auto) 64	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.0*Ey
65	LRFD-Ult (auto) 65	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + (-1.0)*Ey
66	LRFD-Ult (auto) 66	0.9*DL1 + 0.9*DL2 + 0.9*DL3
67	LRFD-Ult (auto) 67	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Wx+(+0.18)
68	LRFD-Ult (auto) 68	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Wx+(-0.18)
69	LRFD-Ult (auto) 69	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Wx-(+0.18)
70	LRFD-Ult (auto) 70	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Wx-(-0.18)
71	LRFD-Ult (auto) 71	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Wy+(+0.18)
72	LRFD-Ult (auto) 72	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Wy+(-0.18)
73	LRFD-Ult (auto) 73	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Wy-(+0.18)
74	LRFD-Ult (auto) 74	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Wy-(-0.18)
75	LRFD-Ult (auto) 75	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Ex
76	LRFD-Ult (auto) 76	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + (-1.0)*Ex
77	LRFD-Ult (auto) 77	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 1.0*Ey
78	LRFD-Ult (auto) 78	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + (-1.0)*Ey

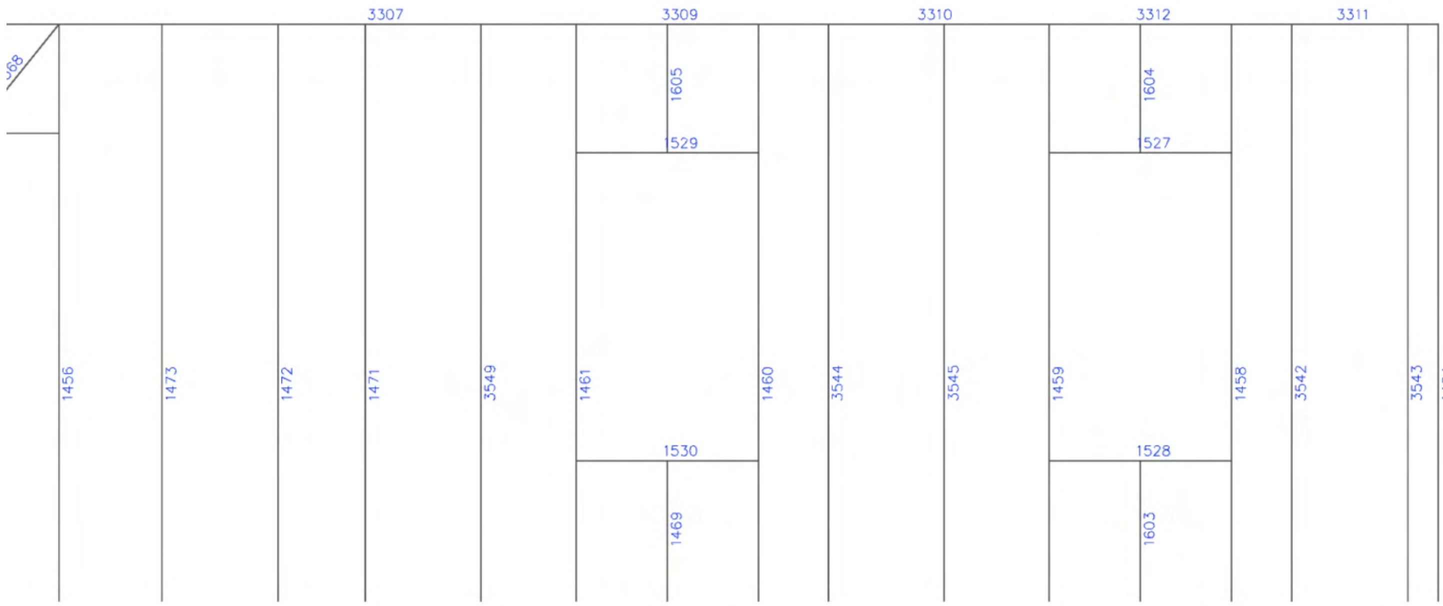
2.4 STEEL STRUCTURE CHECK
2.4.1 SECOND-FLOOR WALL DESIGN
2.4.1.1 REAR WALL DESIGN

General scheme

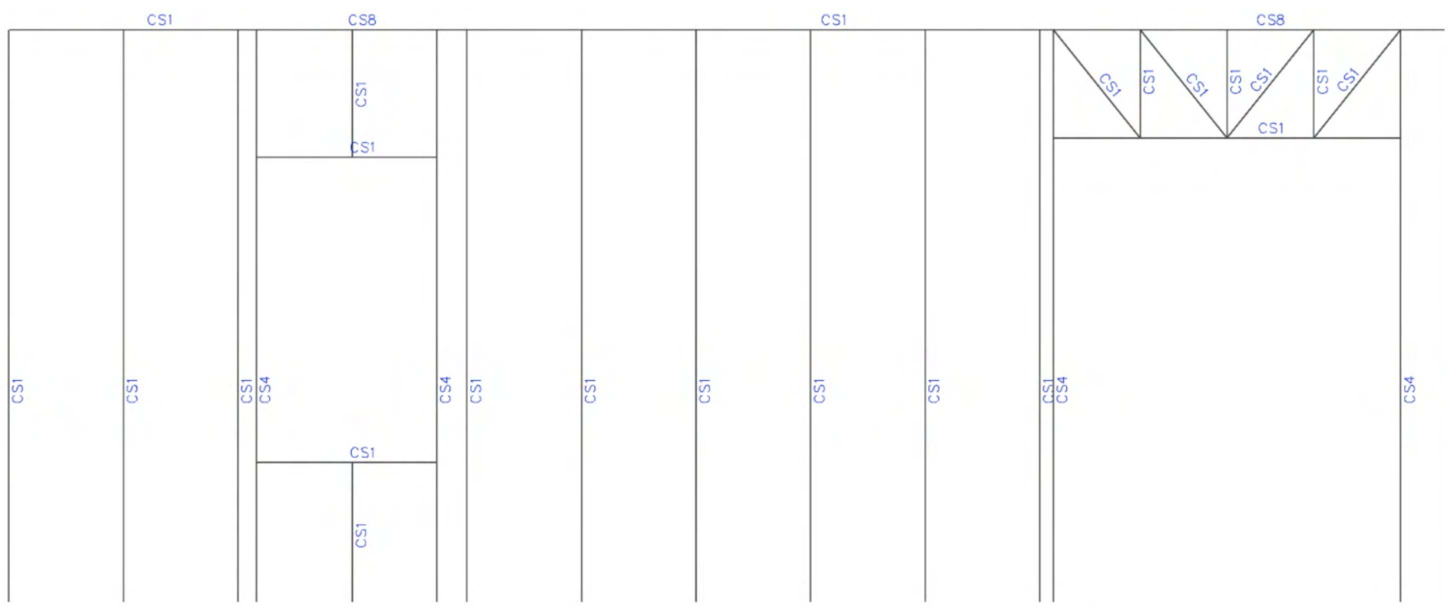


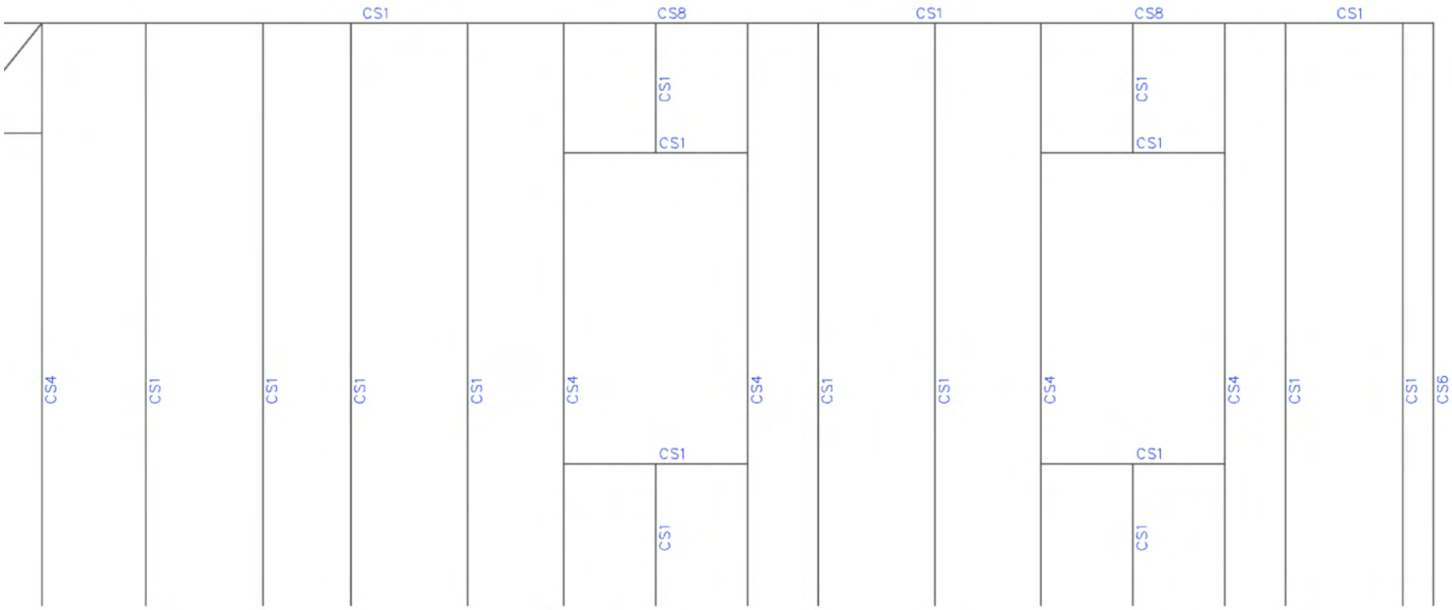
Member numbers



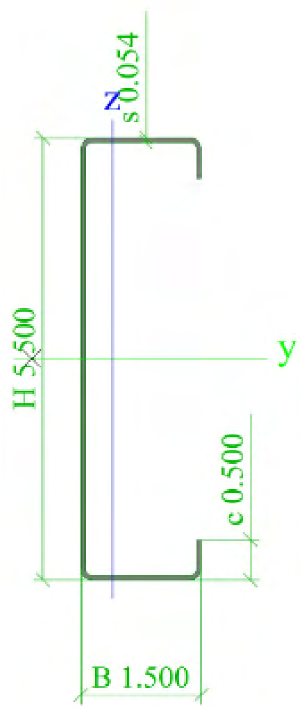


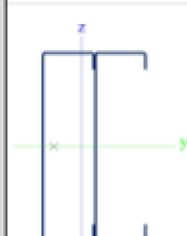
Cross-sections of elements.

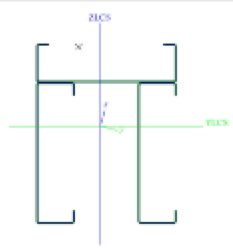


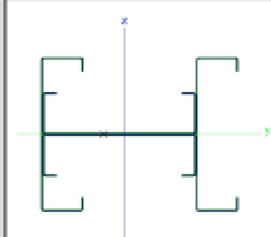

CS1
CS8
CS1
CS8
CS1
CS1
CS1
CS1
CS1
CS4
CS1
CS1
CS1
CS1
CS4
CS4
CS1
CS1
CS4
CS4
CS1
CS1
CS6
CS1
CS1
CS1
CS1

Cross-sections properties

CS1		
Type	Cold formed C section	
Detailed	5.500; 1.500; 0.054; 0.086; 0.500	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.491	
A_y [inch^2], A_z [inch^2]	0.164	0.303
A_L [inch^2/inch], A_D [inch^2/inch]	1.83e+01	1.83e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.392	2.750
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.122	0.139
i_y [inch], i_z [inch]	2.080	0.532
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.772	0.126
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.924	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.05e+00	9.05e+00
d_y [inch], d_z [inch]	-0.999	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.880
β_y [inch], β_z [inch]	0.000	5.904
Picture		

CS4		
Type	2x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.987	
A_y [inch^2], A_z [inch^2]	0.303	0.590
A_L [inch^2/inch], A_D [inch^2/inch]	2.16e+01	3.52e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	3.102	1.925
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.289	0.838
i_y [inch], i_z [inch]	2.084	0.921
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.560	0.451
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.864	0.741
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.32e+01	9.32e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.70e+01	3.70e+01
d_y [inch], d_z [inch]	-0.812	0.000
I_t [inch^4], I_w [inch^6]	1.030	2.525
β_y [inch], β_z [inch]	0.000	2.932
Picture		

CS6		
Type	3x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.481	
A_y [inch^2], A_z [inch^2]	0.596	0.834
A_L [inch^2/inch], A_D [inch^2/inch]	5.01e+01	5.01e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	5.971	2.973
$I_{Y.LCS}$ [inch^4], $I_{Z.LCS}$ [inch^4]	7.684	6.418
$I_{YZ.LCS}$ [inch^4]	0.368	
α [deg]	-15.10	
I_y [inch^4], I_z [inch^4]	7.783	6.319
i_y [inch], i_z [inch]	2.292	2.066
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.812	1.642
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.878	2.808
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.44e+02	1.44e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.40e+02	1.40e+02
d_y [inch], d_z [inch]	-1.628	2.757
I_t [inch^4], I_w [inch^6]	0.002	41.712
$β_y$ [inch], $β_z$ [inch]	-6.538	3.765
Picture		

CS8		
Type	4x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.975	
A_y [inch^2], A_z [inch^2]	0.768	0.979
A_L [inch^2/inch], A_D [inch^2/inch]	5.16e+01	5.27e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	6.321	0.604
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.725	12.252
i_y [inch], i_z [inch]	1.547	2.491
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.718	3.013
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.253	4.606
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.13e+02	1.13e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.30e+02	2.30e+02
d_y [inch], d_z [inch]	-0.749	0.000
I_t [inch^4], I_w [inch^6]	0.005	37.816
$β_y$ [inch], $β_z$ [inch]	0.000	1.807
Picture		

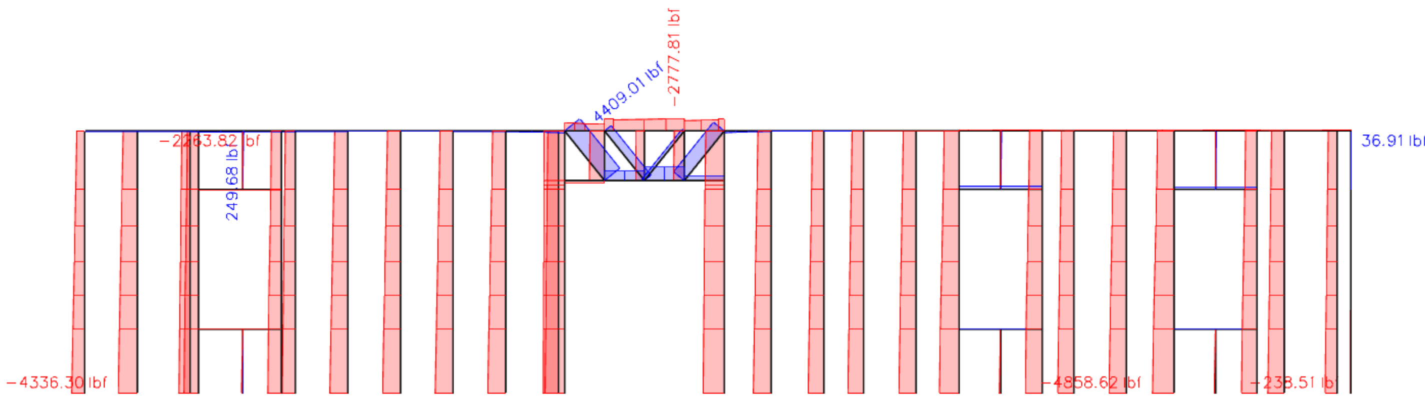
Explanations of symbols	
A	Area
A_y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A_z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input 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
i_z	Radius of gyration about the principal z-axis

Explanations of symbols	
$W_{el.y}$	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 - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Maximum force diagram

Axial force diagram N,

LRFD-Ult (auto)8 (1.2xDL1+1.2xDL2+1.2xDL3+1.6xL+0.5xLr), lbf.



Shear force diagram Vz ,

LRFD-Ult (auto)70 (0.9xDL1+0.9xDL2+0.9xDL3+Wx-(-0.18)), lbf.

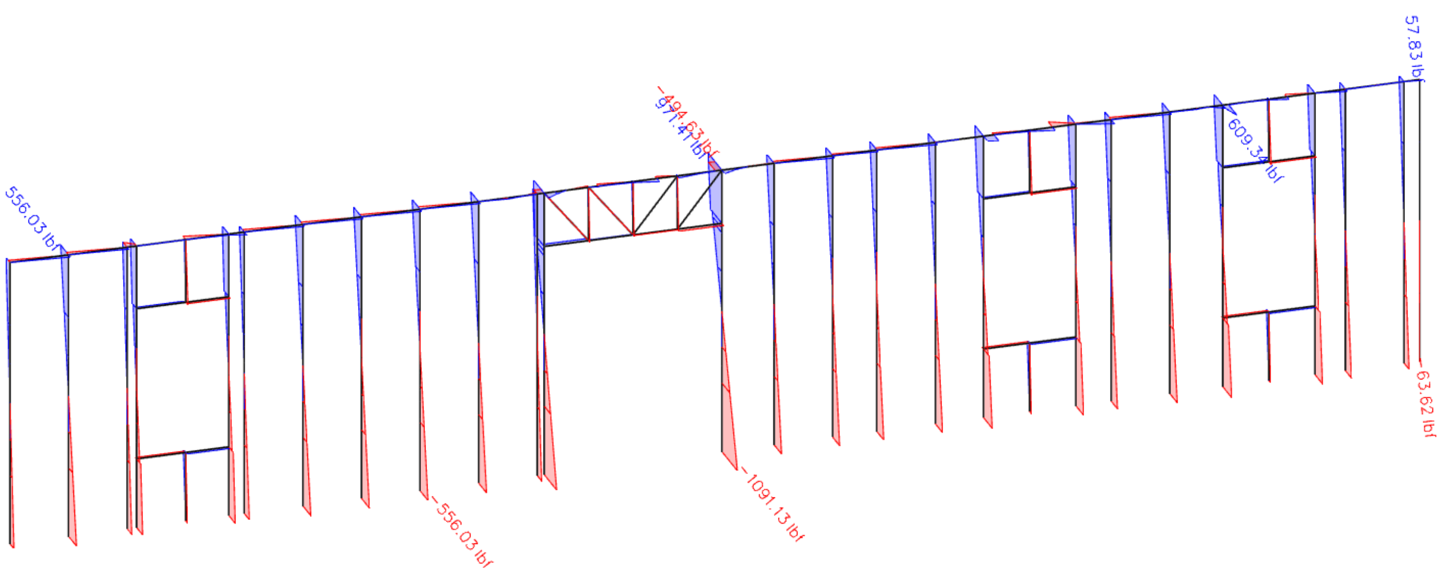
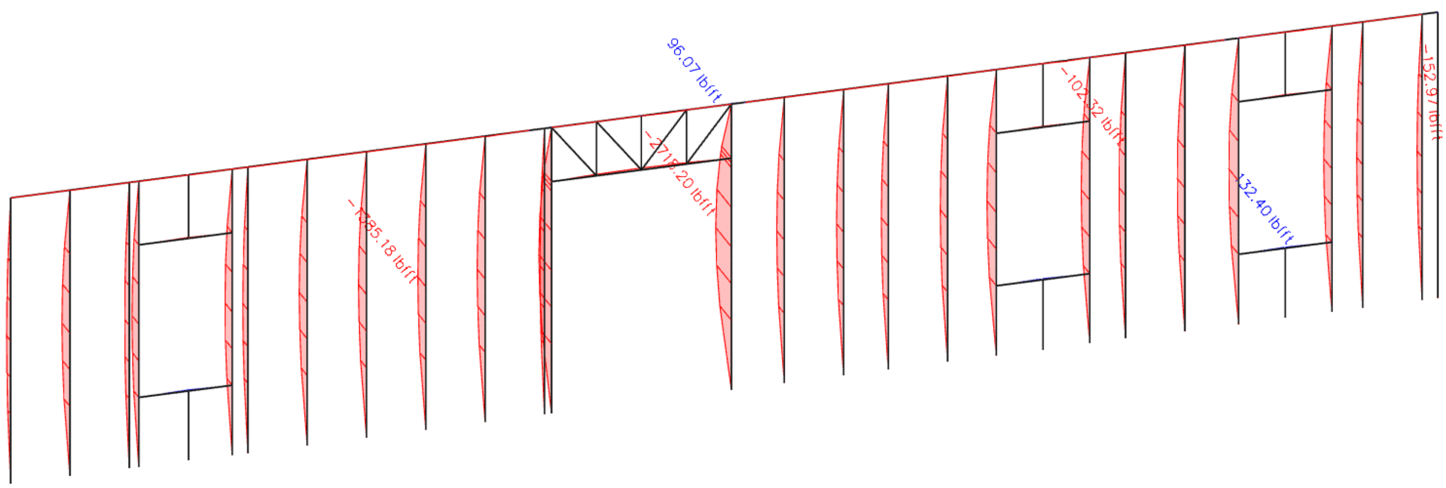


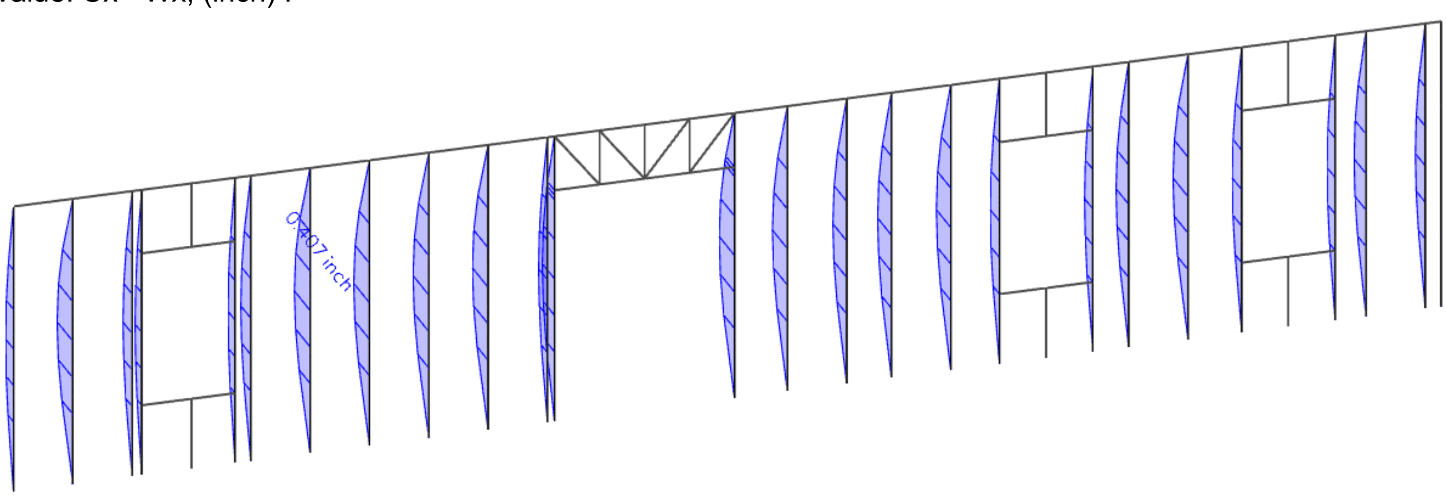
Diagram of moment My,

LRFD-Ult (auto)70 (0.9xDL1+0.9xDL2+0.9xDL3+Wx-(-0.18)), lbf*ft.



Displacement of elements

Value: Ux - Wx, (inch) .



The maximum deflection is 0.407" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 10'= 10'*12"=120"/240 =0.5"

0.407" < 0.50" **<u>Deflection is OK!</u>**

STEEL MEMBER 1456 CHECK

AISI S100-16 LRFD Check

Member 1456	2x550S150-54	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 8.08 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	-7291.51	lbf
Vux	1575.47	lbf
Vuy	0.01	lbf
Mut	-0.60	lbfft
Mux	0.05	lbfft
Muy	641.75	lbfft

Note: Mux and/or Muy include additional moments as defined in art. H1.2

...::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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 10.574	0.21	6.558	239.763	0.457	1.000	- 1.446	0.519 0.927	-	- -	-
3	4.500	10.574 10.574	1.00	4.000	15.101	0.837	0.881	3.964 -	- -	-	- -	-
4	1.446	50.000 10.574	0.21	6.558	239.763	0.457	1.000	- 1.446	0.519 0.927	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	9.838 9.838	1.00	4.000	5467.304	0.042	1.000	0.473 -	- -	-	- -	-
7	1.446	9.102 -30.324	3.33	175.211	6406.202	0.038	1.000	- 1.446	0.228 0.723	-	- -	-
8	5.446	-30.324 -30.324	-	-	-	-	-	- -	- -	-	- -	-
9	1.446	9.102 -30.324	3.33	175.211	6406.202	0.038	1.000	- 1.446	0.228 0.723	-	- -	-
10	0.473	9.838 9.838	1.00	4.000	5467.304	0.042	1.000	0.473 -	- -	-	- -	-

Table of values		
Sye	0.450	$inch^3$
Mnyo	1875.06	lbfft
Resistance factor	0.90	
Unity check	0.38	-

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

Table of values		
Sigma,ex	86.980	ksi
Kt	1.00	
Lt	4.000 in	ft
Sigma,t	9807.275	ksi
Cb	1.00	
Sfy	0.451	$inch^3$
Fcre	4887.768	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	0.078	2342.52
3	0.000	0.00
4	0.078	2342.52
5	0.000	0.00
6	0.000	0.00
7	0.078	2342.52
8	0.000	0.00
9	0.078	2342.52
10	0.000	0.00

Table of values		
Vn,x	9370.08	lbf
Resistance factor	0.95	
Unity check	0.18	-

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

Table of values		
Mnyo	1875.06	lbfft
Vnx	9370.08	lbf
Resistance factor shear	0.95	
Resistance factor bending y	0.90	

Unity check (My, Vx) = sqrt(0.14+0.03) = 0.42

...::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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	4.500	50.000 50.000	1.00	4.000	15.101	1.820	0.483	2.174 -	- -	-	- -	-
4	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 50.000	1.00	4.000	5467.304	0.096	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	5.446	50.000 50.000	1.00	4.000	10.311	2.202	0.409	2.226 -	- -	-	- -	-
9	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
10	0.473	50.000 50.000	1.00	4.000	5467.304	0.096	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.704	inch ^{2}
Pno	35191.38	lbf
Resistance factor	0.85	
Unity check	0.24	-

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	10	3/8	ft
Effective Length factor K	1.00	0.96	
Effective Length	10	3/8	ft
Slenderness	57.37	4.15	
Flexural Buckling stress Fcre	86.980	16618.966	ksi

Torsional (-Flexural) Buckling Strength

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

Table of values		
Sigma,ex	86.980	ksi
Sigma,ey	16618.966	ksi
Kt	1.00	
Lt	3/8	ft
Sigma,t	9807.275	ksi
Sigma,TF	86.893	ksi
Torsional (-Flexural) buckling stress Fcre	86.893	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	0.473	39.298 39.298	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
2	1.446	39.298 39.298	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
3	4.500	39.298 39.298	1.00	4.000	15.101	1.613	0.535	2.409 -	- -	-	- -	-
4	1.446	39.298 39.298	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
5	0.473	39.298 39.298	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
6	0.473	39.298 39.298	1.00	4.000	5467.304	0.085	1.000	0.473 -	- -	-	- -	-
7	1.446	39.298 39.298	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
8	5.446	39.298 39.298	1.00	4.000	10.311	1.952	0.454	2.475 -	- -	-	- -	-
9	1.446	39.298 39.298	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
10	0.473	39.298 39.298	1.00	4.000	5467.304	0.085	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	86.893	ksi
lambda, c	0.76	
Fn	39.298	ksi
Ae	0.730	inch^2
Pn	28686.82	lbf
Resistance factor	0.85	
Unity check	0.30	-

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	0.473	7.385 7.385	1.00	0.430	146.934	0.224	1.000	0.473 -	- -	-	- -	-
2	1.446	7.385 7.385	1.00	4.000	146.251	0.225	1.000	1.446 -	- -	-	- -	-
3	4.500	7.385 7.385	1.00	4.000	15.101	0.699	0.980	4.411 -	- -	-	- -	-

4	1.446	7.385 7.385	1.00	4.000	146.251	0.225	1.000	1.446 -	- -	-	- -	-
5	0.473	7.385 7.385	1.00	0.430	146.934	0.224	1.000	0.473 -	- -	-	- -	-
6	0.473	7.385 7.385	1.00	4.000	5467.304	0.037	1.000	0.473 -	- -	-	- -	-
7	1.446	7.385 7.385	1.00	4.000	146.251	0.225	1.000	1.446 -	- -	-	- -	-
8	5.446	7.385 7.385	1.00	4.000	10.311	0.846	0.874	4.762 -	- -	-	- -	-
9	1.446	7.385 7.385	1.00	4.000	146.251	0.225	1.000	1.446 -	- -	-	- -	-
10	0.473	7.385 7.385	1.00	4.000	5467.304	0.037	1.000	0.473 -	- -	-	- -	-

Table of values		
Centerline shift ex	0.041	inch
Centerline shift ey	0.000	inch
Additional moment Mx	0.00	lbfft
Additional moment My	-25.05	lbfft
Mny	1875.06	lbfft
PEy	16409148.58	lbf
Alfa y	1.00	
Cmy	0.68	
Pn	28686.82	lbf
Pno	35191.38	lbf
Resistance factor compression	0.85	

Table of values		
Resistance factor bending y	0.90	

Unity check = 0.30+0.00+0.26 = 0.56 - (H1.2-1)
Unity check = 0.24+0.00+0.38 = 0.62 -

The member satisfies the check !

STEEL MEMBER 1464 CHECK

AISI S100-16 LRFD Check

Member 1464	3x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.19

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

The critical check is on position 7.75 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	-102.73	lbf
Vux	-627.82	lbf
Vuy	0.00	lbf
Mut	0.00	lbfft
Mux	-0.00	lbfft
Muy	1411.63	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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 26.703	0.53	5.134	187.721	0.516	1.000	- 1.446	0.586 0.860	-	- -	-
3	5.446	26.703 26.703	1.00	4.000	10.311	1.609	0.536	2.921 -	- -	-	- -	-
4	1.446	50.000 26.703	0.53	5.134	750.883	0.258	1.000	- 1.446	0.586 0.860	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 42.379	0.85	0.487	166.309	0.548	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	2.554	26.703 -14.446	0.54	14.401	168.778	0.398	1.000	- 2.554	0.721 1.277	-	- -	-
9	1.446	-37.743 -37.743	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-30.122 -37.743	-	-	-	-	-	- -	- -	-	- -	-
11	0.473	-14.446 -14.446	-	-	-	-	-	- -	- -	-	- -	-
12	1.446	-14.446 -37.743	-	-	-	-	-	- -	- -	-	- -	-

13	5.446	-37.743 -37.743	-	-	-	-	-	- -	- -	-	- -	-
14	1.446	-14.446 -37.743	-	-	-	-	-	- -	- -	-	- -	-
15	0.473	-14.446 -14.446	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	1.981	$inch^3$
Mnyo	8253.00	lbfft
Resistance factor	0.90	
Unity check	0.19	-

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

Table of values		
Sigma,ex	105.223	ksi
Kt	1.00	
Lt	2 ft 2.636 in	ft
Sigma,t	575.637	ksi
Cb	1.00	
Sfy	2.148	$inch^3$
Fcre	754.359	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	0.078	2342.52
3	0.000	0.00
4	0.156	4685.04
5	0.000	0.00
6	0.026	766.26
7	0.000	0.00
8	0.138	4137.48
9	0.000	0.00
10	0.026	766.26
11	0.000	0.00
12	0.078	2342.52
13	0.000	0.00
14	0.156	4685.04
15	0.000	0.00

Table of values		
Vn,x	19725.12	lbf
Resistance factor	0.95	
Unity check	0.03	-

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

Table of values		
Mnyo	8253.00	lbfft
Vnx	19725.12	lbf
Resistance factor shear	0.95	
Resistance factor bending y	0.90	

Unity check (My, Vx) = sqrt(0.04+0.00) = 0.19

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	10	2 1/4	ft
Effective Length factor K	1.00	0.83	
Effective Length	10	1 7/8	ft
Slenderness	52.16	10.74	
Flexural Buckling stress Fcre	105.223	2480.732	ksi

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

Table of values		
Sigma,ex	105.223	ksi
Sigma,ey	2480.732	ksi
Kt	1.00	
Lt	2 1/4	ft
Sigma,t	575.637	ksi
Sigma,TF	102.252	ksi
Torsional (-Flexural) buckling stress Fcre	102.252	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	0.473	40.746 40.746	1.00	0.430	146.934	0.527	1.000	0.473 -	- -	-	- -	-
2	1.446	40.746 40.746	1.00	4.000	146.251	0.528	1.000	1.446 -	- -	-	- -	-
3	5.446	40.746 40.746	1.00	4.000	10.311	1.988	0.447	2.436 -	- -	-	- -	-
4	1.446	40.746 40.746	1.00	4.000	585.004	0.264	1.000	1.446 -	- -	-	- -	-
5	0.473	40.746 40.746	1.00	0.430	146.934	0.527	1.000	0.473 -	- -	-	- -	-
6	0.473	40.746 40.746	1.00	0.430	146.934	0.527	1.000	0.473 -	- -	-	- -	-

7	1.446	40.746 40.746	1.00	4.000	146.251	0.528	1.000	1.446 -	- -	-	- -	-
8	2.554	40.746 40.746	1.00	4.000	46.881	0.932	0.820	2.093 -	- -	-	- -	-
9	1.446	40.746 40.746	1.00	4.000	146.251	0.528	1.000	1.446 -	- -	-	- -	-
10	0.473	40.746 40.746	1.00	0.430	146.934	0.527	1.000	0.473 -	- -	-	- -	-
11	0.473	40.746 40.746	1.00	0.430	146.934	0.527	1.000	0.473 -	- -	-	- -	-
12	1.446	40.746 40.746	1.00	4.000	146.251	0.528	1.000	1.446 -	- -	-	- -	-
13	5.446	40.746 40.746	1.00	4.000	10.311	1.988	0.447	2.436 -	- -	-	- -	-
14	1.446	40.746 40.746	1.00	4.000	585.004	0.264	1.000	1.446 -	- -	-	- -	-
15	0.473	40.746 40.746	1.00	0.430	146.934	0.527	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	102.252	ksi
lambda, c	0.70	
Fn	40.746	ksi
Ae	1.155	$inch^2$
Pn	47058.51	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	0.473	0.069 0.069	1.00	0.430	146.934	0.022	1.000	0.473 -	- -	-	- -	-
2	1.446	0.069 0.069	1.00	4.000	146.251	0.022	1.000	1.446 -	- -	-	- -	-
3	5.446	0.069 0.069	1.00	4.000	10.311	0.082	1.000	5.446 -	- -	-	- -	-
4	1.446	0.069 0.069	1.00	4.000	585.004	0.011	1.000	1.446 -	- -	-	- -	-
5	0.473	0.069 0.069	1.00	0.430	146.934	0.022	1.000	0.473 -	- -	-	- -	-
6	0.473	0.069 0.069	1.00	0.430	146.934	0.022	1.000	0.473 -	- -	-	- -	-
7	1.446	0.069 0.069	1.00	4.000	146.251	0.022	1.000	1.446 -	- -	-	- -	-
8	2.554	0.069 0.069	1.00	4.000	46.881	0.038	1.000	2.554 -	- -	-	- -	-
9	1.446	0.069 0.069	1.00	4.000	146.251	0.022	1.000	1.446 -	- -	-	- -	-
10	0.473	0.069 0.069	1.00	0.430	146.934	0.022	1.000	0.473 -	- -	-	- -	-

11	0.473	0.069 0.069	1.00	0.430	146.934	0.022	1.000	0.473 -	- -	-	- -	-
12	1.446	0.069 0.069	1.00	4.000	146.251	0.022	1.000	1.446 -	- -	-	- -	-
13	5.446	0.069 0.069	1.00	4.000	10.311	0.082	1.000	5.446 -	- -	-	- -	-
14	1.446	0.069 0.069	1.00	4.000	585.004	0.011	1.000	1.446 -	- -	-	- -	-
15	0.473	0.069 0.069	1.00	0.430	146.934	0.022	1.000	0.473 -	- -	-	- -	-

Table of values		
Mny	8253.00	lbfft
Pn	47058.51	lbf
Resistance factor compression	0.85	
Resistance factor bending y	0.90	

Unity check = 0.00+0.00+0.19 = 0.19 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER 1480 CHECK

AISI S100-16 LRFD Check

Member 1480	Cold formed C section (5.500; 1.500; 0.054; 0.086; 0.500)	A913 grade 50	LRFD-Ult (auto)	0.78

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

The critical check is on position 4.98 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	-2607.20	lbf
Vux	3.82	lbf
Vuy	0.00	lbf
Mut	-0.00	lbfft
Mux	-1385.04	lbfft
Muy	14.83	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	0.361	47.939 41.334	0.86	0.481	282.818	0.412	1.000	0.336 -	- -	-	- -	-
3	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
5	5.221	47.939 -47.717	1.00	23.880	66.973	0.846	0.875	- 4.566	1.143 2.283	-	- -	-
7	1.221	-49.778 -49.778	-	-	-	-	-	- -	- -	-	- -	-
9	0.360	-41.112 -47.717	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	0.768	inch^3
Mnxo	3199.68	lbfft
Resistance factor	0.90	
Unity check	0.48	-

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	0.361	47.568 41.010	0.86	0.481	282.841	0.410	1.000	0.343 -	- -	-	- -	-
3	1.221	49.615 49.615	1.00	3.149	161.485	0.554	1.000	1.221 -	0.581 0.640	30.95	0.000 0.000	0.343
5	5.221	47.568 -47.413	1.00	23.915	67.072	0.842	0.877	- 4.580	1.146 2.290	-	- -	-
7	1.221	-49.460 -49.460	-	-	-	-	-	- -	- -	-	- -	-
9	0.360	-40.855 -47.413	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Lltb	2 ft 9.153 in	ft
Sigma,ey	73.820	ksi
Kt	1.00	
Lt	2 ft 9.153 in	ft
Sigma,t	85.329	ksi
Cb	1.09	
Sfx	0.772	inch ^{3}
Fcre	129.881	ksi

Table of values		
Fc	49.615	ksi
Scx	0.769	inch ^{3}
Mnx	3180.09	lbfft
Resistance factor	0.90	
Unity check	0.48	-

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

Table of values		
Sfy	0.772	inch ^{3}
My	3215.29	lbfft
L	1 ft 0.708 in	ft
Beta	1.09	
k,phi,fe	291.72	lbf
k,phi,we	273.76	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	inch ^{2}
k,phi,wg	0.002	inch ^{2}
Fd	73.527	ksi
Sf	0.772	inch ^{3}
Mcrd	4728.21	lbfft
Lambda,d	0.82	
Mn	2858.84	lbfft
Resistance factor	0.90	
Unity check	0.54	-

Data		
Lm	2 ft 9.153 in	ft
Lcr	1 ft 0.708 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	-0.056	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	0.361	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.361 -	- -	-	- -	-
3	1.221	44.796 -11.683	0.26	10.530	539.981	0.288	1.000	- 1.221	0.374 0.610	-	- -	-
5	5.221	-16.887 -16.887	-	-	-	-	-	- -	- -	-	- -	-
7	1.221	44.796 -11.683	0.26	10.530	539.981	0.288	1.000	- 1.221	0.374 0.610	-	- -	-
9	0.360	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.360 -	- -	-	- -	-

Table of values		
Sye	0.126	$inch^3$
Mnyo	523.01	lbfft
Resistance factor	0.90	
Unity check	0.03	-

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

Table of values		
Sigma,ex	86.587	ksi
Kt	1.00	
Lt	2 ft 9.153 in	ft
Sigma,t	85.329	ksi
Cs	-1.00	
CTF	0.84	
Sfy	0.126	$inch^3$
j	3.076	inch
Fcre	321.664	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	0.126	$inch^3$
My	522.96	lbfft
L	1 ft 0.708 in	ft
Beta	1.09	
k,phi,fe	291.72	lbf
k,phi,we	273.76	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	$inch^2$
k,phi,wg	0.009	$inch^2$
Fd	38.077	ksi
Sf	0.126	$inch^3$
Mcrd	398.26	lbfft
Lambda,d	1.15	
Mn	368.75	lbfft
Resistance factor	0.90	
Unity check	0.04	-

Data		
Lm	2 ft 9.153 in	ft
Lcr	1 ft 0.708 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	-0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	0.056	inch
Ksi,web	0.00	

Number of compressed flanges: 2

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

...::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	0.361	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-
3	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
5	5.221	50.000 50.000	1.00	4.000	11.218	2.111	0.424	2.215 -	- -	-	- -	-
7	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
9	0.360	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.326	$inch^2$
Pno	16296.53	lbf
Resistance factor	0.85	
Unity check	0.19	-

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	10	2 7/8	ft
Effective Length factor K	1.00	0.67	
Effective Length	10	1 7/8	ft
Slenderness	57.50	42.00	
Flexural Buckling stress Fcre	86.587	162.305	ksi

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

Table of values		
Sigma,ex	86.587	ksi
Sigma,ey	162.305	ksi
Kt	1.00	
Lt	2 7/8	ft
Sigma,t	85.329	ksi
Sigma,TF	60.445	ksi
Torsional (-Flexural) buckling stress Fcre	60.445	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	0.361	35.368 35.368	1.00	0.430	252.949	0.374	1.000	0.361 -	- -	-	- -	-
3	1.221	35.368 35.368	1.00	3.202	164.223	0.464	1.000	1.221 -	0.610 0.610	36.66	0.000 0.000	0.361
5	5.221	35.368 35.368	1.00	4.000	11.218	1.776	0.493	2.576 -	- -	-	- -	-
7	1.221	35.368 35.368	1.00	3.202	164.223	0.464	1.000	1.221 -	0.610 0.610	36.66	0.000 0.000	0.360
9	0.360	35.368 35.368	1.00	0.430	252.949	0.374	1.000	0.360 -	- -	-	- -	-

Table of values		
Fe	60.445	ksi
lambda, c	0.91	
Fn	35.368	ksi
Ae	0.348	$inch^2$
Pn	12309.16	lbf
Resistance factor	0.85	
Unity check	0.25	-

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

Table of values		
Py	24535.65	lbf
L	1 ft 2.047 in	ft
k,phi,fe	205.02	lbf
k,phi,we	152.10	lbf
k,phi	0.00	lbf
k,phi,fg	0.006	$inch^2$
k,phi,wg	0.007	$inch^2$
Fd	27.271	ksi
Pcrd	13382.33	lbf
Lambda,d	1.35	
Pn	14090.87	lbf
Resistance factor	0.85	
Unity check	0.22	-

Data		
Lm	2 ft 9.153 in	ft
Lcr	1 ft 2.047 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	-0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	0.056	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	0.361	5.313 5.313	1.00	0.430	252.949	0.145	1.000	0.361 -	- -	-	- -	-
3	1.221	5.313 5.313	1.00	4.000	205.118	0.161	1.000	1.221 -	0.610 0.610	94.58	- 0.000	0.361
5	5.221	5.313 5.313	1.00	4.000	11.218	0.688	0.989	5.161 -	- -	-	- -	-
7	1.221	5.313 5.313	1.00	4.000	205.118	0.161	1.000	1.221 -	0.610 0.610	94.58	- 0.000	0.360
9	0.360	5.313 5.313	1.00	0.430	252.949	0.145	1.000	0.360 -	- -	-	- -	-

Table of values		
Mnx	2858.84	lbfft
Mny	368.75	lbfft
PEx	42489.38	lbf
PEy	79645.12	lbf
Alfa x	0.94	
Alfa y	0.97	
Cmx	0.85	
Cmy	0.84	
Pn	12309.16	lbf
Pno	16296.53	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.25+0.49+0.04 = 0.78 - (H1.2-1)
Unity check = 0.19+0.54+0.04 = 0.77 -

The member satisfies the check !

STEEL MEMBER 3306 CHECK

AISI S100-16 LRFD Check

Member 3306	4x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.34

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

The critical check is on position 1.47 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	249.68	lbf
Vux	-0.92	lbf
Vuy	1452.78	lbf
Mut	-0.01	lbfft
Mux	-2153.60	lbfft
Muy	-0.56	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	0.473	31.421 25.563	0.81	0.501	171.216	0.428	1.000	0.473 -	- -	-	- -	-
2	1.446	31.756 31.756	1.00	4.000	585.004	0.233	1.000	- 1.446	0.723 0.723	-	- -	-
3	5.446	31.421 -36.031	1.15	28.079	289.506	0.329	1.000	- 5.446	1.313 2.723	-	- -	-
4	1.446	-36.365 -36.365	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-30.173 -36.031	-	-	-	-	-	- -	- -	-	- -	-
6	0.473	-18.790 -18.790	-	-	-	-	-	- -	- -	-	- -	-
7	1.446	-18.790 -36.700	-	-	-	-	-	- -	- -	-	- -	-
8	1.250	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-
9	1.446	-18.790 -36.700	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-18.790 -18.790	-	-	-	-	-	- -	- -	-	- -	-

11	0.473	-30.173 -36.031	-	-	-	-	-	- -	- -	-	- -	-
12	1.446	-36.365 -36.365	-	-	-	-	-	- -	- -	-	- -	-
13	1.446	31.756 31.756	1.00	4.000	585.004	0.233	1.000	- 1.446	0.723 0.723	-	- -	-
14	0.473	31.421 25.563	0.81	0.501	171.216	0.428	1.000	0.473 -	- -	-	- -	-
15	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
16	1.446	50.000 32.090	0.64	4.808	175.805	0.533	1.000	- 1.446	0.613 0.833	-	- -	-
17	1.250	32.090 32.090	1.00	4.000	195.711	0.405	1.000	- 1.250	0.625 0.625	-	- -	-
18	1.446	50.000 32.090	0.64	4.808	175.805	0.533	1.000	- 1.446	0.613 0.833	-	- -	-
19	0.473	50.000 50.000	1.00	0.431	147.393	0.582	1.000	0.473 -	- -	-	- -	-
20	1.250	32.090 32.090	1.00	4.000	195.711	0.405	1.000	- 1.250	0.625 0.625	-	- -	-
29	1.250	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-
34	0.054	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-
39	0.054	32.090 32.090	1.00	4.000	104869.215	0.017	1.000	0.054 -	- -	-	- -	-

Table of values		
Sxe	4.451	$inch^3$
Mnxo	18547.37	lbfft
Resistance factor	0.90	
Unity check	0.13	-

Lateral-Torsional Buckling Strength

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

Table of values		
Lltb	1 ft 8.150 in	ft
Sigma,ey	1687.180	ksi
Kt	1.00	
Lt	1 ft 8.150 in	ft
Sigma,t	1477.685	ksi
Cb	1.83	
Sfx	1.718	$inch^3$
Fcre	10065.707	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
2	0.156	4685.04
3	0.000	0.00
4	0.156	4685.04
5	0.000	0.00
6	0.026	766.26
7	0.000	0.00
8	0.068	2025.00
9	0.000	0.00
10	0.026	766.26
11	0.000	0.00
12	0.156	4685.04
13	0.156	4685.04
14	0.000	0.00
15	0.026	766.26
16	0.000	0.00
17	0.067	2025.00
18	0.000	0.00
19	0.026	766.26
20	0.068	2025.00
29	0.067	2025.00
34	0.003	87.48
39	0.003	87.48

Table of values		
Vn,y	30080.16	lbf
Resistance factor	0.95	
Unity check	0.05	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	18547.37	lbfft
Vny	30080.16	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.02+0.00) = 0.14

Combined Tensile Axial Load and Bending

According to article H1.1 and formulas (H1.1-1), (H1.1-2)

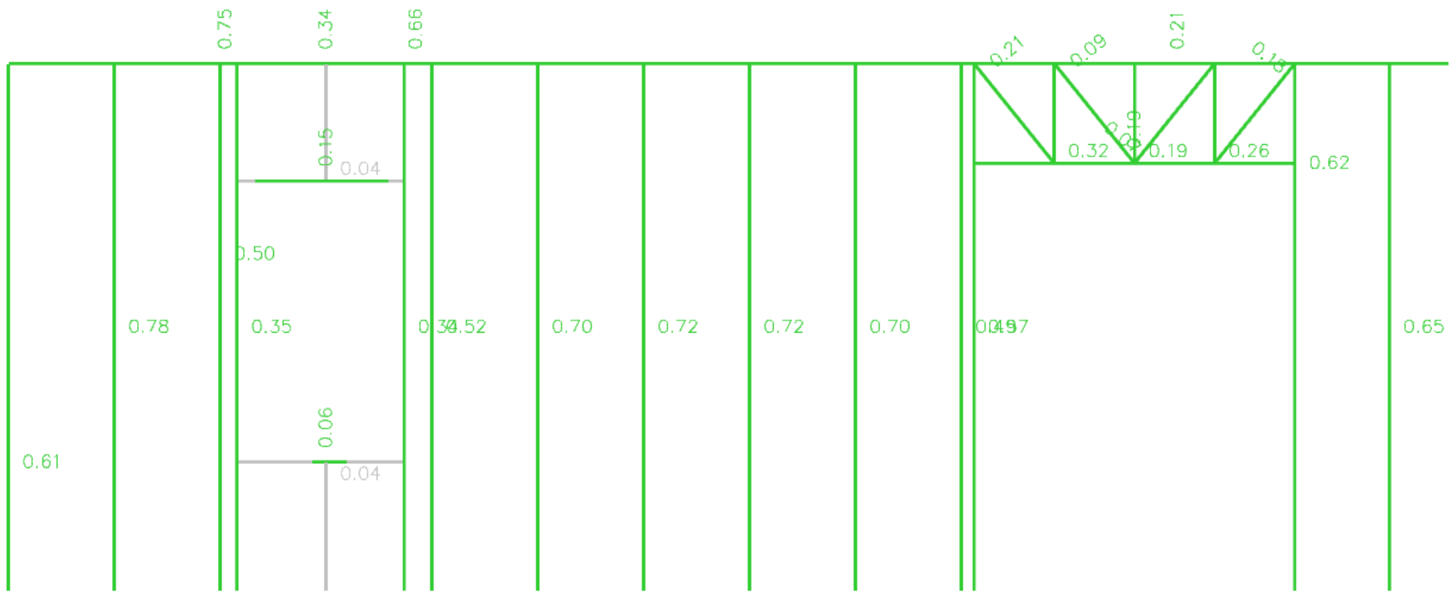
Table of values		
Sftx	1.718	inch ^{3}
Mnxt	7159.00	lbfft
Mnx	18547.37	lbfft
Tn	98737.48	lbf
Resistance factor tension	0.95	
Resistance factor bending x	0.90	

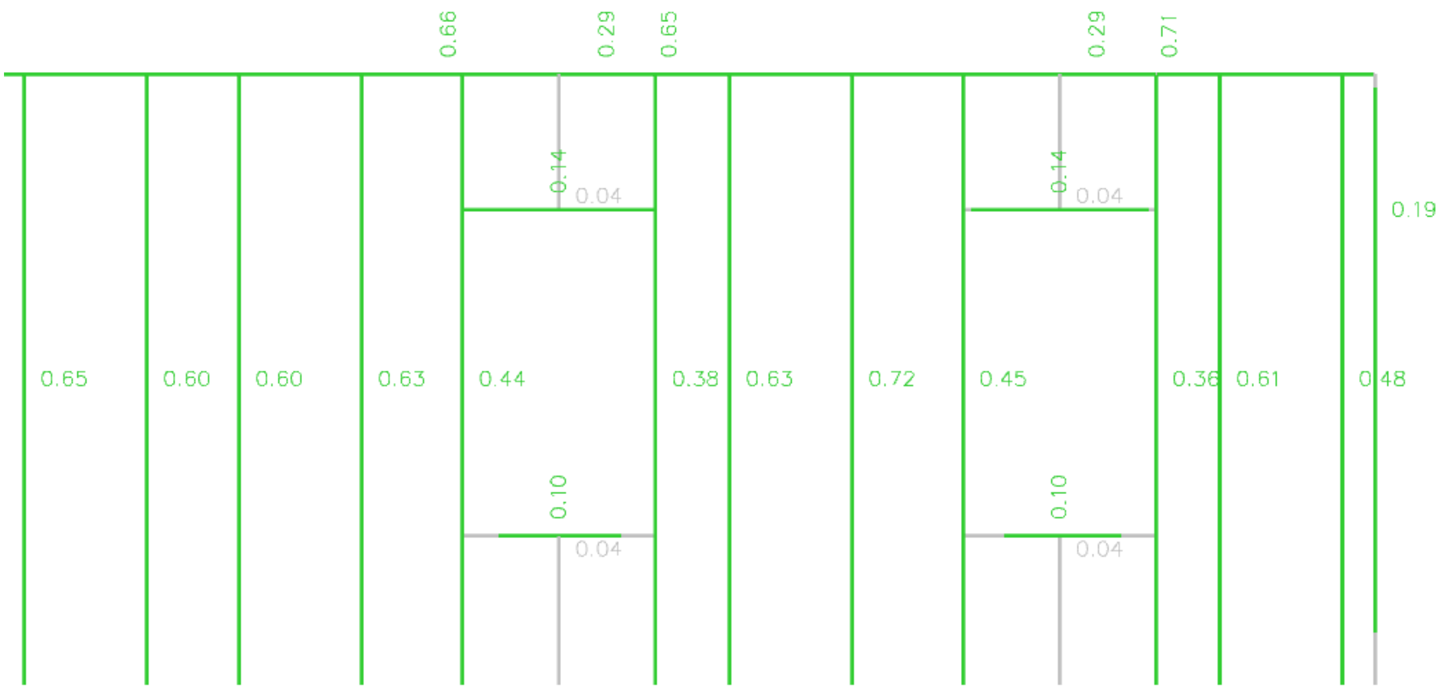
Unity check = 0.33+0.00+0.00 = 0.34 - (H1.1-1)

Unity check = 0.13+0.00-0.00 = 0.13 - (H1.1-2)

The member satisfies the check !

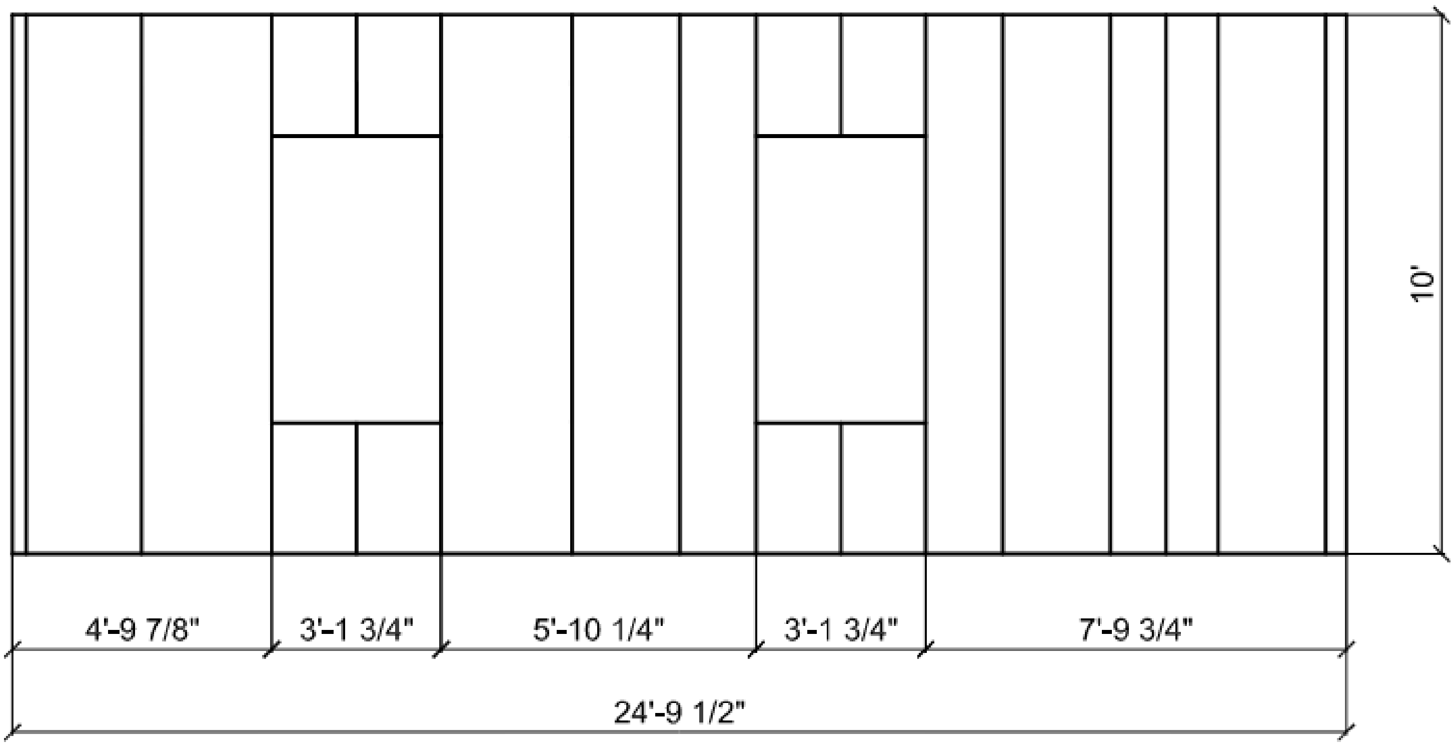
Unity check


0.75
0.34
0.66
0.21
0.09
0.21
0.18
0.15
0.04
0.32
0.19
0.19
0.26
0.62
0.50
0.78
0.35
0.70
0.72
0.72
0.70
0.65
0.61
0.06
0.04

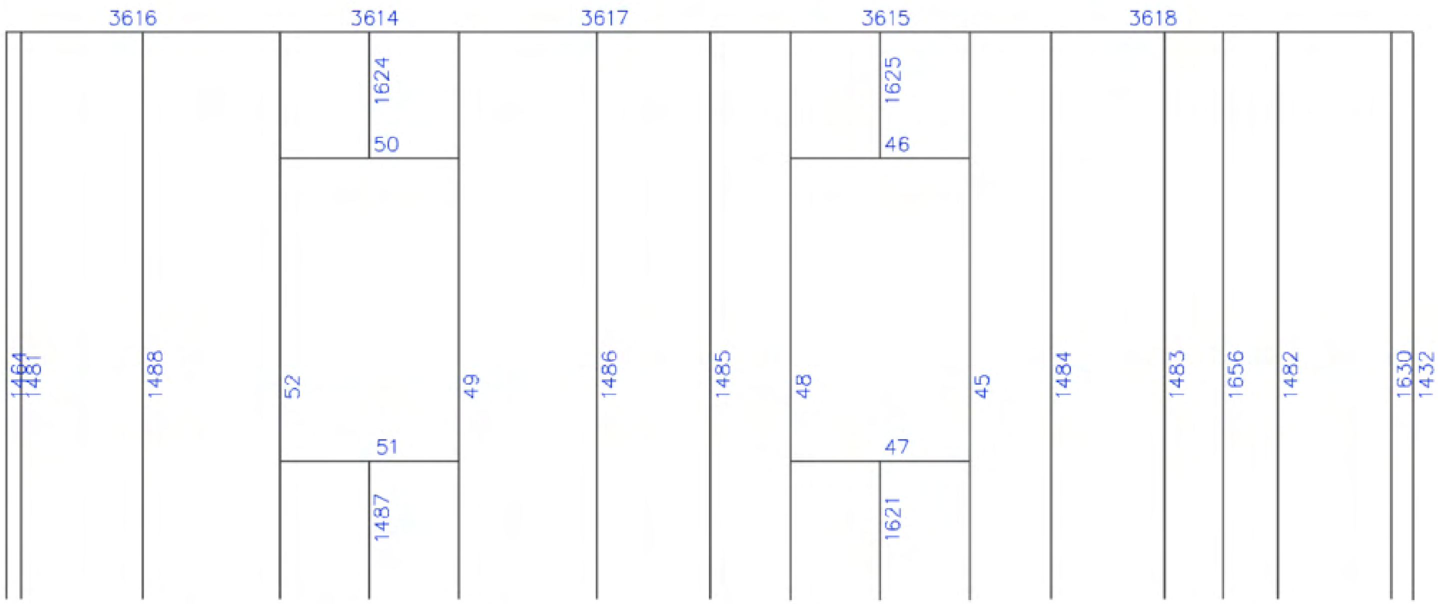

0.66
0.29
0.65
0.29
0.71
0.14
0.04
0.14
0.04
0.19
0.65
0.60
0.60
0.63
0.44
0.38
0.63
0.72
0.45
0.36
0.61
0.48
0.10
0.04
0.10
0.04

2.4.1.2 LEFT SIDE WALL DESIGN

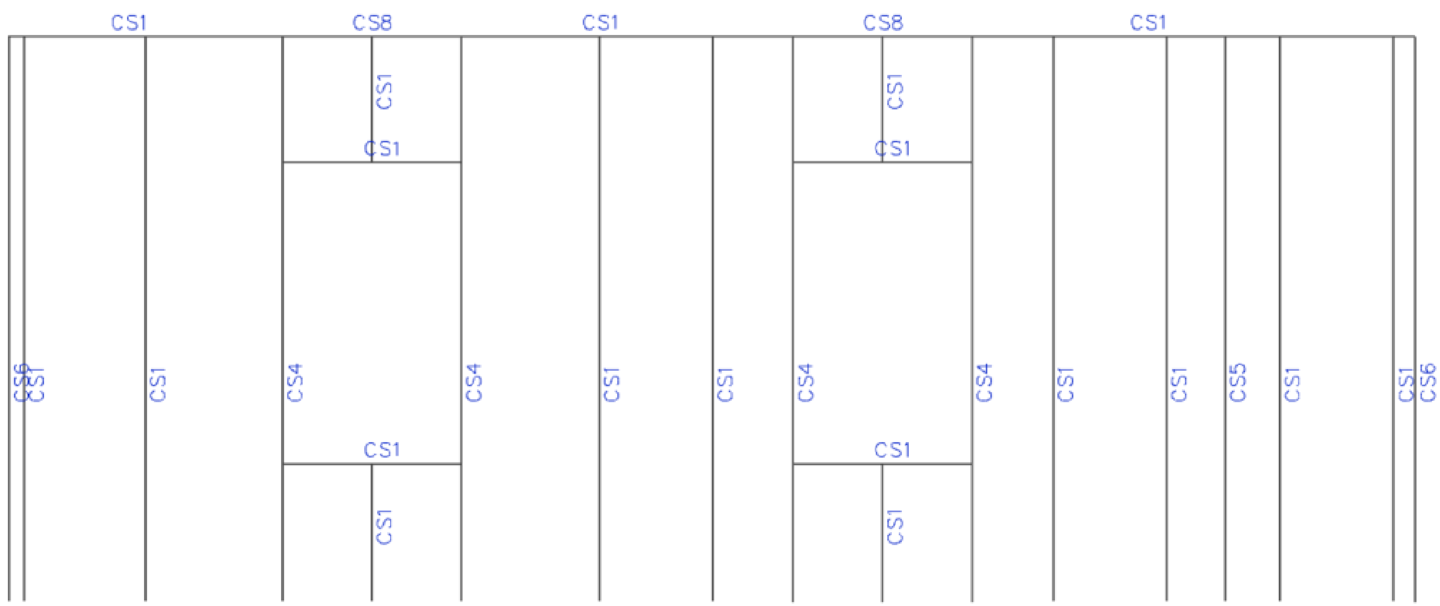
General scheme



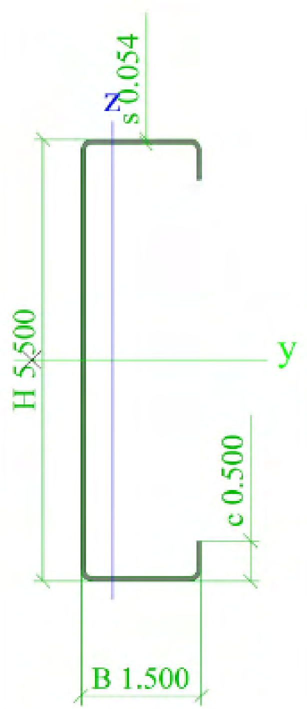
Member numbers

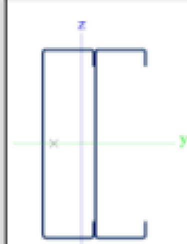


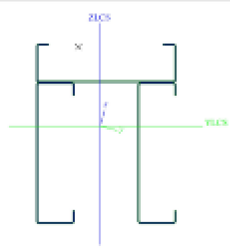
Cross-sections of elements.

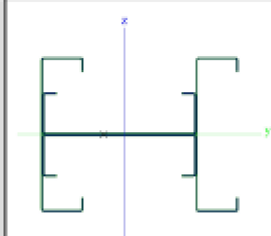


Cross-sections properties

CS1		
Type	Cold formed C section	
Detailed	5.500; 1.500; 0.054; 0.086; 0.500	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch^2]	0.491	
A_y [inch^2], A_z [inch^2]	0.164	0.303
A_L [inch^2/inch], A_D [inch^2/inch]	1.83e+01	1.83e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.392	2.750
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.122	0.139
i_y [inch], i_z [inch]	2.080	0.532
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.772	0.126
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.924	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.05e+00	9.05e+00
d_y [inch], d_z [inch]	-0.999	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.880
$β_y$ [inch], $β_z$ [inch]	0.000	5.904
Picture		

CS4		
Type	2x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch^2]	0.987	
A_y [inch^2], A_z [inch^2]	0.303	0.590
A_L [inch^2/inch], A_D [inch^2/inch]	2.16e+01	3.52e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	3.102	1.925
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.289	0.838
i_y [inch], i_z [inch]	2.084	0.921
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.560	0.451
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.864	0.741
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.32e+01	9.32e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.70e+01	3.70e+01
d_y [inch], d_z [inch]	-0.812	0.000
I_t [inch^4], I_w [inch^6]	1.030	2.525
$β_y$ [inch], $β_z$ [inch]	0.000	2.932
Picture		

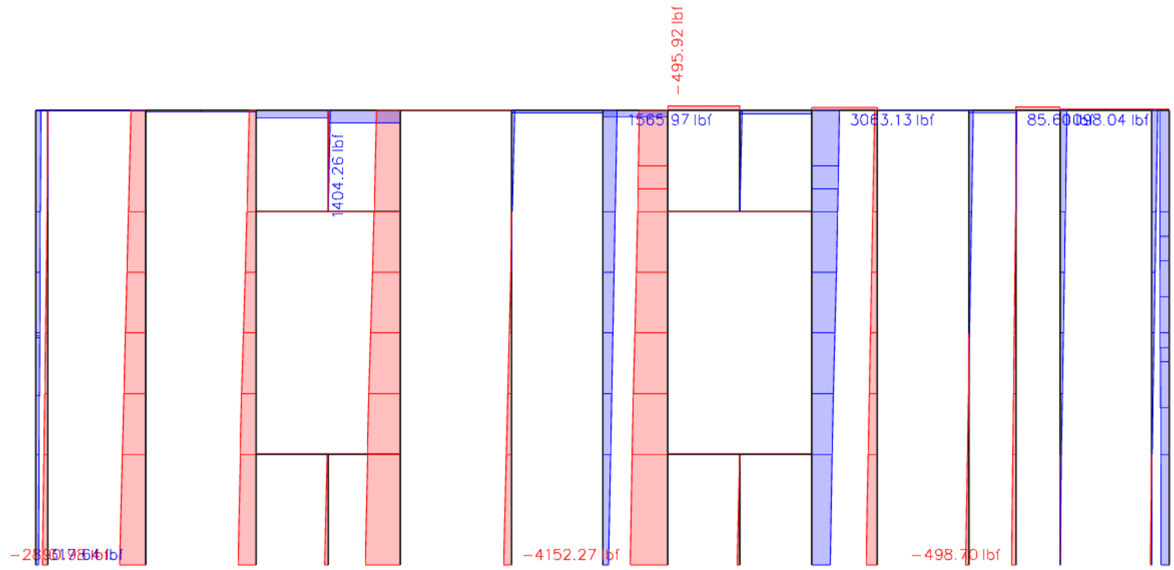
CS6		
Type	3x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.481	
A_y [inch^2], A_z [inch^2]	0.596	0.834
A_L [inch^2/inch], A_D [inch^2/inch]	5.01e+01	5.01e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	5.971	2.973
$I_{Y.LCS}$ [inch^4], $I_{Z.LCS}$ [inch^4]	7.684	6.418
$I_{YZ.LCS}$ [inch^4]	0.368	
α [deg]	-15.10	
I_y [inch^4], I_z [inch^4]	7.783	6.319
i_y [inch], i_z [inch]	2.292	2.066
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.812	1.642
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.878	2.808
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.44e+02	1.44e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.40e+02	1.40e+02
d_y [inch], d_z [inch]	-1.628	2.757
I_t [inch^4], I_w [inch^6]	0.002	41.712
β_y [inch], β_z [inch]	-6.538	3.765
Picture		

CS8		
Type	4x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.975	
A_y [inch^2], A_z [inch^2]	0.768	0.979
A_L [inch^2/inch], A_D [inch^2/inch]	5.16e+01	5.27e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	6.321	0.604
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.725	12.252
i_y [inch], i_z [inch]	1.547	2.491
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.718	3.013
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.253	4.606
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.13e+02	1.13e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.30e+02	2.30e+02
d_y [inch], d_z [inch]	-0.749	0.000
I_t [inch^4], I_w [inch^6]	0.005	37.816
β_y [inch], β_z [inch]	0.000	1.807
Picture		

Explanations of symbols	
A	Area
A_y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A_z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
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
i_z	Radius of gyration about the principal z-axis

Explanations of symbols	
$W_{el.y}$	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 - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Maximum force diagram
Axial force diagram N,
LRFD-Ult (auto)51 (1.2xDL1+1.2xDL2+1.2xDL3+0.5xL+0.5xLr+1xWx+(-0.18)), lbf.



Shear force diagram Vz ,
LRFD-Ult (auto)57 (1.2xDL1+1.2xDL2+1.2xDL3+0.5xL+0.5xLr+Wy-(-0.18)), lbf.

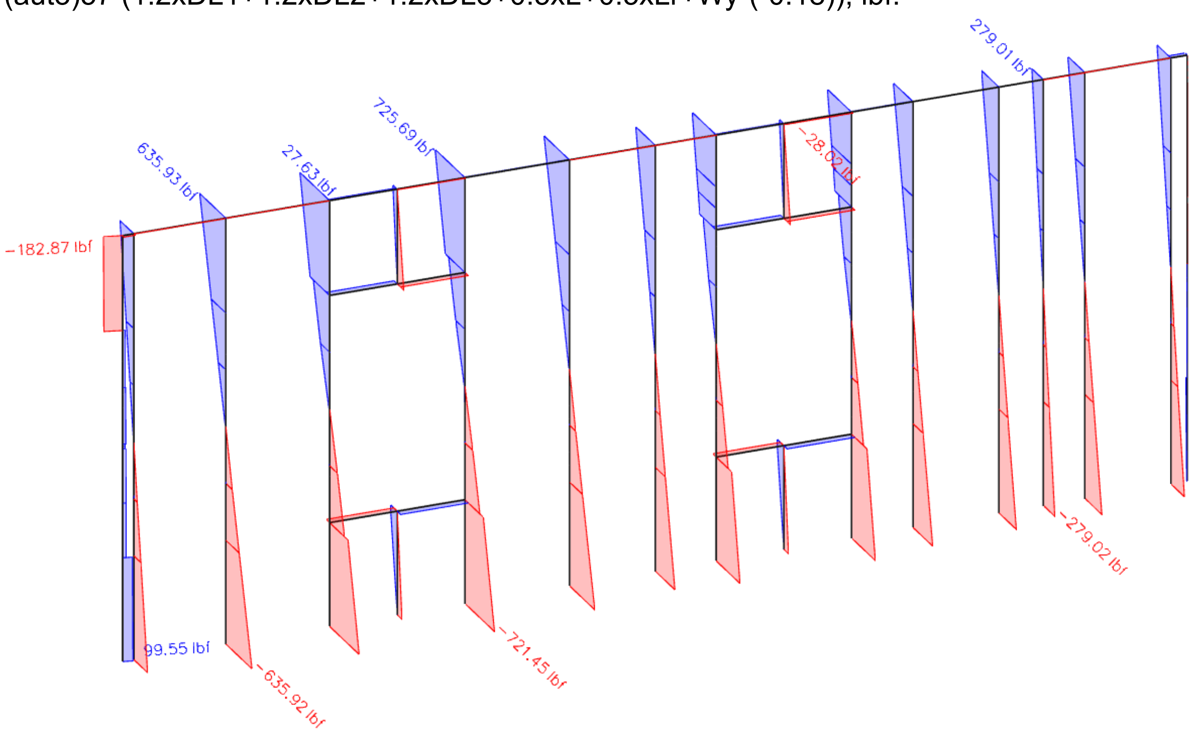
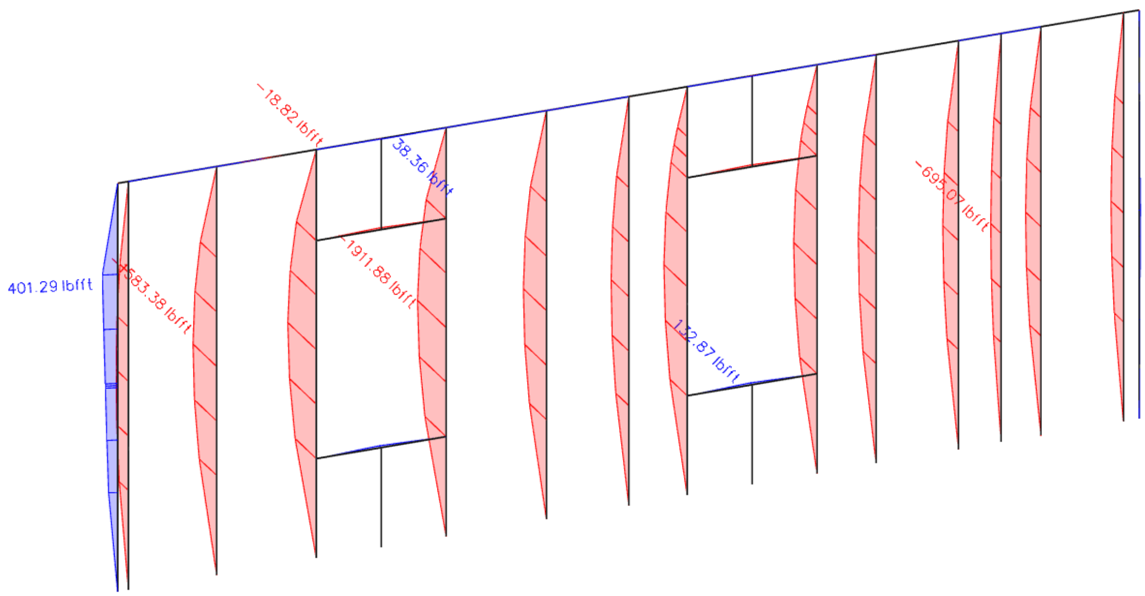
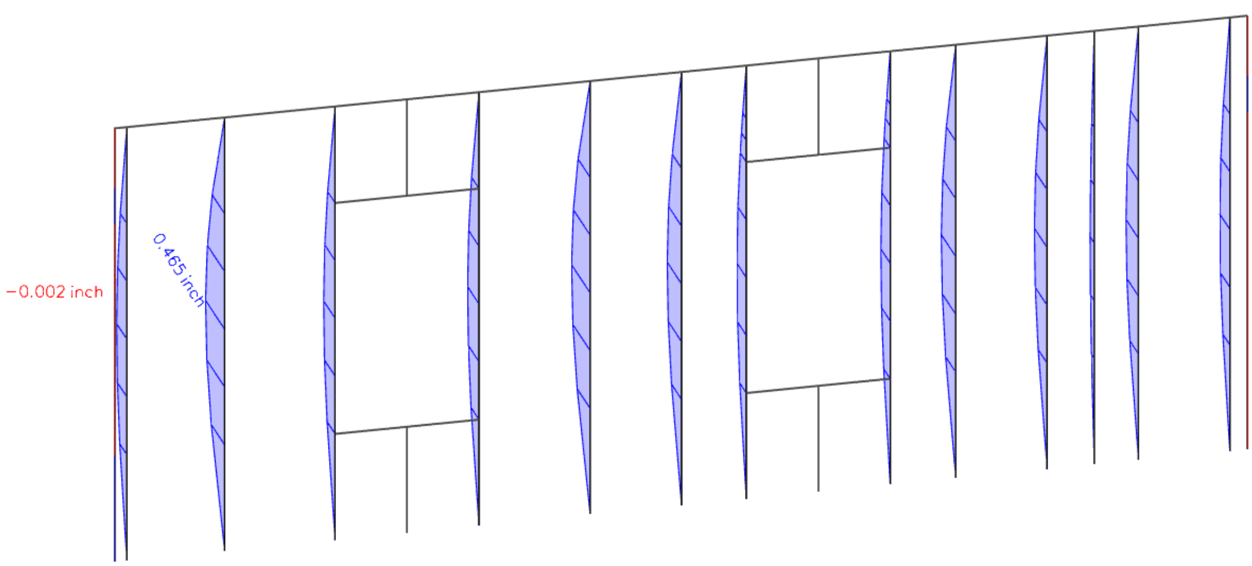


Diagram of moment My,
LRFD-Ult (auto)57 (1.2xDL1+1.2xDL2+1.2xDL3+0.5xL+0.5xLr+Wy-(-0.18)), lbf*ft.



Displacement of elements
Value: Ux - Wx, (inch) .



The maximum deflection is 0.465" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 10'= 10'*12"=120"/240 =0.5"
0.465" < 0.50" **<u>Deflection is OK!</u>**

STEEL MEMBER 49 CHECK

AISI S100-16 LRFD Check

Member 49	2x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.42

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

The critical check is on position 5.09 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	-2301.56	lbf
Vux	-0.09	lbf
Vuy	16.33	lbf
Mut	-0.05	lbfft
Mux	-1911.88	lbfft
Muy	-0.94	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	0.473	50.000 41.315	0.83	0.496	169.345	0.543	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	4.500	41.315 -41.315	1.00	24.000	90.607	0.675	0.998	- 4.493	1.123 2.246	-	- -	-
4	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-
6	0.473	50.000 41.315	0.83	4.358	5956.483	0.092	1.000	- 0.473	0.218 0.255	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	5.446	50.000 -50.000	1.00	24.000	61.863	0.899	0.840	- 4.575	1.144 2.288	-	- -	-
9	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	1.560	$inch^3$
Mnxo	6498.93	lbfft
Resistance factor	0.90	
Unity check	0.33	-

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

Table of values		
Lltb	3.190 in	ft
Sigma,ey	23868.713	ksi
Kt	0.10	
Lt	3.190 in	ft
Sigma,t	14282.597	ksi
Cb	1.07	
Sfx	1.560	$inch^3$
Fcre	30237.616	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.
...::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	86.980	ksi
Kt	0.10	
Lt	2 ft 7.900 in	ft
Sigma,t	14282.597	ksi
Cb	1.00	
Sfy	0.732	$inch^3$
Fcre	3636.363	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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	4.500	50.000 50.000	1.00	4.000	15.101	1.820	0.483	2.174 -	- -	-	- -	-
4	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 50.000	1.00	4.000	5467.304	0.096	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-

8	5.446	50.000 50.000	1.00	4.000	10.311	2.202	0.409	2.226 -	- -	-	- -	-
9	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
10	0.473	50.000 50.000	1.00	4.000	5467.304	0.096	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.704	$inch^2$
Pno	35191.38	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	10	2 3/4	ft
Effective Length factor K	1.00	0.10	
Effective Length	10	3/8	ft
Slenderness	57.37	3.46	
Flexural Buckling stress Fcre	86.980	23868.713	ksi

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

Table of values		
Sigma,ex	86.980	ksi
Sigma,ey	23868.713	ksi
Kt	0.10	
Lt	2 3/4	ft
Sigma,t	14282.597	ksi
Sigma,TF	86.920	ksi
Torsional (-Flexural) buckling stress Fcre	86.920	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	0.473	39.301 39.301	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
2	1.446	39.301 39.301	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
3	4.500	39.301 39.301	1.00	4.000	15.101	1.613	0.535	2.409 -	- -	-	- -	-
4	1.446	39.301	1.00	4.000	146.251	0.518	1.000	1.446	-	-	-	-
5	0.473	39.301 39.301	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
6	0.473	39.301 39.301	1.00	4.000	5467.304	0.085	1.000	0.473 -	- -	-	- -	-
7	1.446	39.301 39.301	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
8	5.446	39.301 39.301	1.00	4.000	10.311	1.952	0.454	2.475 -	- -	-	- -	-

9	1.446	39.301 39.301	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
10	0.473	39.301 39.301	1.00	4.000	5467.304	0.085	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	86.920	ksi
lambda, c	0.76	
Fn	39.301	ksi
Ae	0.730	$inch^2$
Pn	28688.68	lbf
Resistance factor	0.85	
Unity check	0.09	-

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	0.473	2.331 2.331	1.00	0.430	146.934	0.126	1.000	0.473 -	- -	-	- -	-
2	1.446	2.331 2.331	1.00	4.000	146.251	0.126	1.000	1.446 -	- -	-	- -	-
3	4.500	2.331 2.331	1.00	4.000	15.101	0.393	1.000	4.500 -	- -	-	- -	-
4	1.446	2.331 2.331	1.00	4.000	146.251	0.126	1.000	1.446 -	- -	-	- -	-
5	0.473	2.331 2.331	1.00	0.430	146.934	0.126	1.000	0.473 -	- -	-	- -	-
6	0.473	2.331 2.331	1.00	4.000	5467.304	0.021	1.000	0.473 -	- -	-	- -	-
7	1.446	2.331 2.331	1.00	4.000	146.251	0.126	1.000	1.446 -	- -	-	- -	-
8	5.446	2.331 2.331	1.00	4.000	10.311	0.475	1.000	5.446 -	- -	-	- -	-
9	1.446	2.331 2.331	1.00	4.000	146.251	0.126	1.000	1.446 -	- -	-	- -	-
10	0.473	2.331 2.331	1.00	4.000	5467.304	0.021	1.000	0.473 -	- -	-	- -	-

Table of values		
Mnx	6498.93	lbfft
Mny	1557.22	lbfft
Pn	28688.68	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.09+0.33+0.00 = 0.42 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER 1656 CHECK

AISI S100-16 LRFD Check

Member 1656	2x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.15

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

The critical check is on position 5.09 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	-898.33	lbf
Vux	-0.04	lbf
Vuy	5.85	lbf
Mut	0.00	lbfft
Mux	-695.07	lbfft
Muy	-0.49	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 [inch⁴]	ds [inch]
1	0.473	50.000 41.315	0.83	0.496	169.345	0.543	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	5.446	50.000 -50.000	1.00	24.000	247.452	0.450	1.000	- 5.446	1.362 2.723	-	- -	-
4	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-
6	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
9	0.473	50.000 41.315	0.83	0.496	169.345	0.543	1.000	0.473 -	- -	-	- -	-

Table of values		
Sxe	1.560	inch³
Mnxo	6498.93	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		
Lltb	3.190 in	ft
Sigma,ey	12413.394	ksi
Kt	0.10	
Lt	3.190 in	ft
Sigma,t	20890.763	ksi
Cb	1.07	
Sfx	1.560	$inch^3$
Fcre	23852.479	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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	5.446	50.000 50.000	1.00	4.000	41.242	1.101	0.727	3.958 -	- -	-	- -	-
4	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
9	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.842	$inch^2$
Pno	42118.80	lbf
Resistance factor	0.85	
Unity check	0.03	-

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	10	2 3/4	ft
Effective Length factor K	1.00	0.10	
Effective Length	10	3/8	ft
Slenderness	57.37	4.80	
Flexural Buckling stress Fcre	86.980	12413.394	ksi

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

Table of values		
Sigma,ex	86.980	ksi
Sigma,ey	12413.394	ksi
Kt	0.10	
Lt	2 3/4	ft
Sigma,t	20890.763	ksi
Sigma,TF	86.980	ksi
Torsional (-Flexural) buckling stress Fcre	86.980	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	0.473	39.308 39.308	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
2	1.446	39.308 39.308	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
3	5.446	39.308 39.308	1.00	4.000	41.242	0.976	0.793	4.321 -	- -	-	- -	-
4	1.446	39.308 39.308	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
5	0.473	39.308 39.308	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
6	0.473	39.308 39.308	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
7	1.446	39.308 39.308	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
8	1.446	39.308 39.308	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
9	0.473	39.308 39.308	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	86.980	ksi
lambda, c	0.76	
Fn	39.308	ksi
Ae	0.882	inch^2
Pn	34654.93	lbf
Resistance factor	0.85	
Unity check	0.03	-

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	0.473	0.910 0.910	1.00	0.430	146.934	0.079	1.000	0.473 -	- -	-	- -	-
2	1.446	0.910 0.910	1.00	4.000	146.251	0.079	1.000	1.446 -	- -	-	- -	-
3	5.446	0.910 0.910	1.00	4.000	41.242	0.149	1.000	5.446 -	- -	-	- -	-
4	1.446	0.910 0.910	1.00	4.000	146.251	0.079	1.000	1.446 -	- -	-	- -	-
5	0.473	0.910 0.910	1.00	0.430	146.934	0.079	1.000	0.473 -	- -	-	- -	-
6	0.473	0.910 0.910	1.00	0.430	146.934	0.079	1.000	0.473 -	- -	-	- -	-
7	1.446	0.910 0.910	1.00	4.000	146.251	0.079	1.000	1.446 -	- -	-	- -	-
8	1.446	0.910 0.910	1.00	4.000	146.251	0.079	1.000	1.446 -	- -	-	- -	-
9	0.473	0.910 0.910	1.00	0.430	146.934	0.079	1.000	0.473 -	- -	-	- -	-

Table of values		
Mnx	6498.93	lbfft
Pn	34654.93	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.03+0.12+0.00 = 0.15 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER 3615 CHECK

AISI S100-16 LRFD Check

Member 49	2x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.42

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

The critical check is on position 5.09 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	-2301.56	lbf
Vux	-0.09	lbf
Vuy	16.33	lbf
Mut	-0.05	lbfft
Mux	-1911.88	lbfft
Muy	-0.94	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	0.473	50.000 41.315	0.83	0.496	169.345	0.543	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	4.500	41.315 -41.315	1.00	24.000	90.607	0.675	0.998	- 4.493	1.123 2.246	-	- -	-
4	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-
6	0.473	50.000 41.315	0.83	4.358	5956.483	0.092	1.000	- 0.473	0.218 0.255	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	5.446	50.000 -50.000	1.00	24.000	61.863	0.899	0.840	- 4.575	1.144 2.288	-	- -	-
9	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	1.560	$inch^3$
Mnxo	6498.93	lbfft
Resistance factor	0.90	
Unity check	0.33	-

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

Table of values		
Lltb	3.190 in	ft
Sigma,ey	23868.713	ksi
Kt	0.10	
Lt	3.190 in	ft
Sigma,t	14282.597	ksi
Cb	1.07	
Sfx	1.560	$inch^3$
Fcre	30237.616	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.
...::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	86.980	ksi
Kt	0.10	
Lt	2 ft 7.900 in	ft
Sigma,t	14282.597	ksi
Cb	1.00	
Sfy	0.732	$inch^3$
Fcre	3636.363	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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	4.500	50.000 50.000	1.00	4.000	15.101	1.820	0.483	2.174 -	- -	-	- -	-
4	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 50.000	1.00	4.000	5467.304	0.096	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-

8	5.446	50.000 50.000	1.00	4.000	10.311	2.202	0.409	2.226 -	- -	-	- -	-
9	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
10	0.473	50.000 50.000	1.00	4.000	5467.304	0.096	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.704	$inch^2$
Pno	35191.38	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	10	2 3/4	ft
Effective Length factor K	1.00	0.10	
Effective Length	10	3/8	ft
Slenderness	57.37	3.46	
Flexural Buckling stress Fcre	86.980	23868.713	ksi

Torsional (-Flexural) Buckling Strength

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

Table of values		
Sigma,ex	86.980	ksi
Sigma,ey	23868.713	ksi
Kt	0.10	
Lt	2 3/4	ft
Sigma,t	14282.597	ksi
Sigma,TF	86.920	ksi
Torsional (-Flexural) buckling stress Fcre	86.920	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	0.473	39.301 39.301	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
2	1.446	39.301 39.301	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
3	4.500	39.301 39.301	1.00	4.000	15.101	1.613	0.535	2.409 -	- -	-	- -	-
4	1.446	39.301 39.301	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
5	0.473	39.301 39.301	1.00	0.430	146.934	0.517	1.000	0.473 -	- -	-	- -	-
6	0.473	39.301 39.301	1.00	4.000	5467.304	0.085	1.000	0.473 -	- -	-	- -	-
7	1.446	39.301 39.301	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
8	5.446	39.301 39.301	1.00	4.000	10.311	1.952	0.454	2.475 -	- -	-	- -	-

9	1.446	39.301 39.301	1.00	4.000	146.251	0.518	1.000	1.446 -	- -	-	- -	-
10	0.473	39.301 39.301	1.00	4.000	5467.304	0.085	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	86.920	ksi
lambda, c	0.76	
Fn	39.301	ksi
Ae	0.730	$inch^2$
Pn	28688.68	lbf
Resistance factor	0.85	
Unity check	0.09	-

Combined Compressive Axial Load and Bending

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

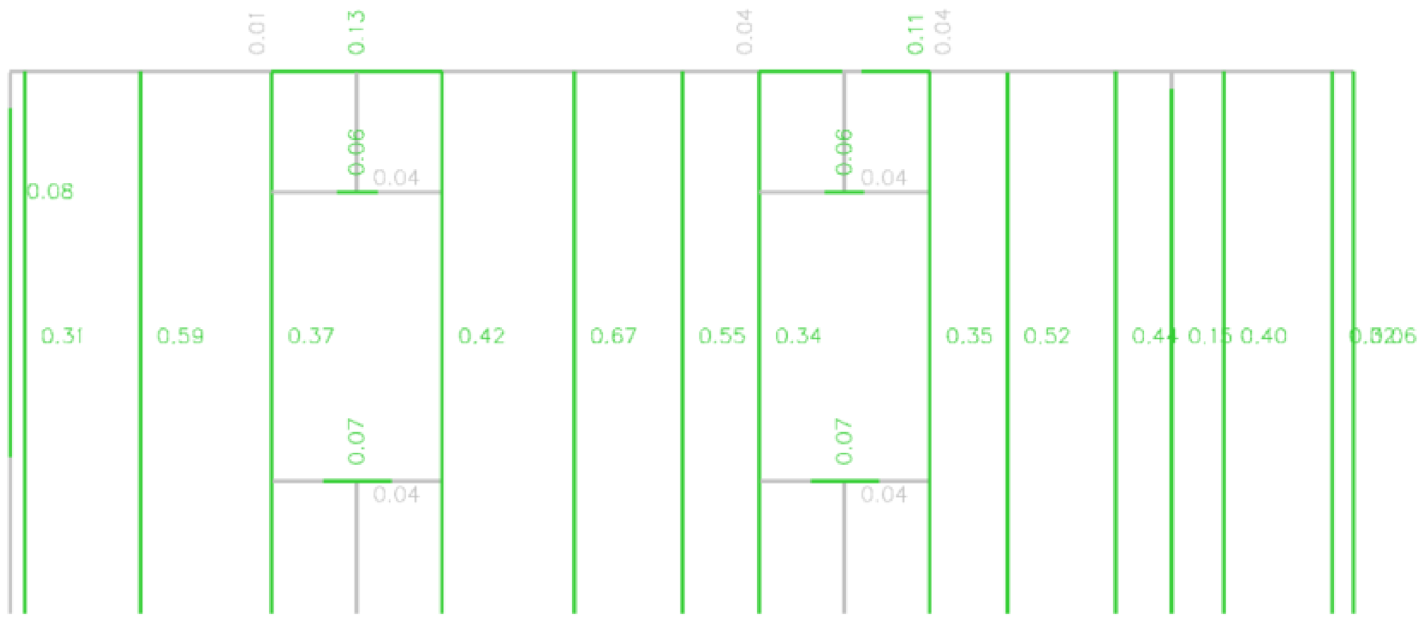
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.473	2.331 2.331	1.00	0.430	146.934	0.126	1.000	0.473 -	- -	-	- -	-
2	1.446	2.331 2.331	1.00	4.000	146.251	0.126	1.000	1.446 -	- -	-	- -	-
3	4.500	2.331 2.331	1.00	4.000	15.101	0.393	1.000	4.500 -	- -	-	- -	-
4	1.446	2.331 2.331	1.00	4.000	146.251	0.126	1.000	1.446 -	- -	-	- -	-
5	0.473	2.331 2.331	1.00	0.430	146.934	0.126	1.000	0.473 -	- -	-	- -	-
6	0.473	2.331 2.331	1.00	4.000	5467.304	0.021	1.000	0.473 -	- -	-	- -	-
7	1.446	2.331 2.331	1.00	4.000	146.251	0.126	1.000	1.446 -	- -	-	- -	-
8	5.446	2.331 2.331	1.00	4.000	10.311	0.475	1.000	5.446 -	- -	-	- -	-
9	1.446	2.331 2.331	1.00	4.000	146.251	0.126	1.000	1.446 -	- -	-	- -	-
10	0.473	2.331 2.331	1.00	4.000	5467.304	0.021	1.000	0.473 -	- -	-	- -	-

Table of values		
Mnx	6498.93	lbfft
Mny	1557.22	lbfft
Pn	28688.68	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.09+0.33+0.00 = 0.42 - (C5.2.1-3)

The member satisfies the check !

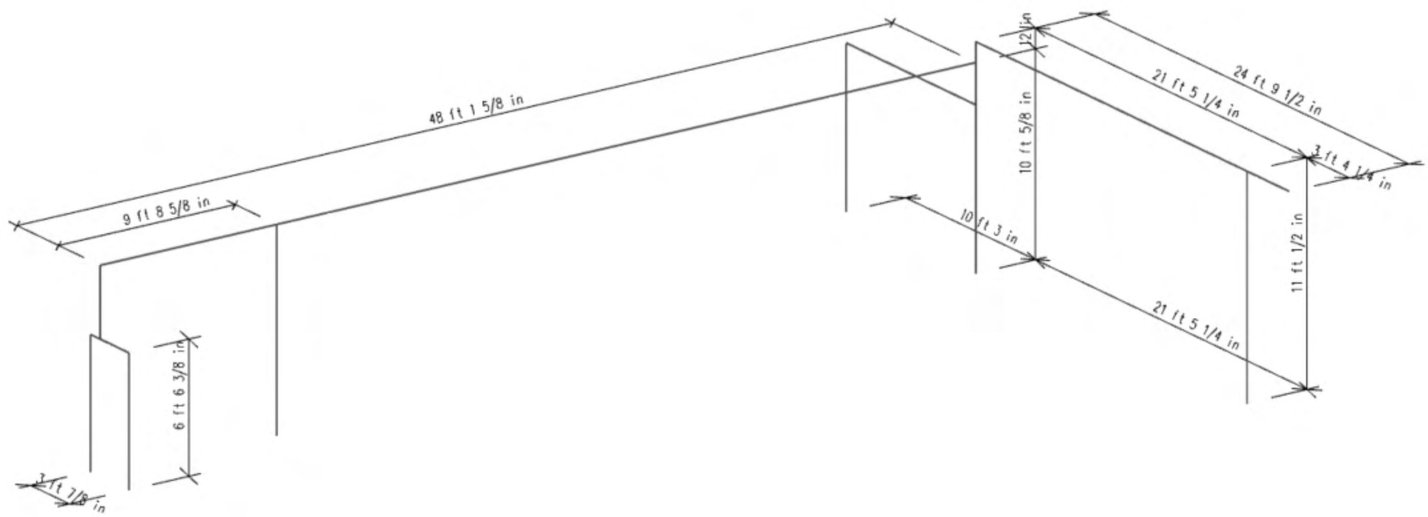
Unity check


0.01
0.13
0.04
0.11
0.04
0.08
0.06
0.04
0.06
0.04
0.31
0.59
0.37
0.42
0.67
0.55
0.34
0.35
0.52
0.44
0.15
0.40
0.07
0.04
0.07
0.04

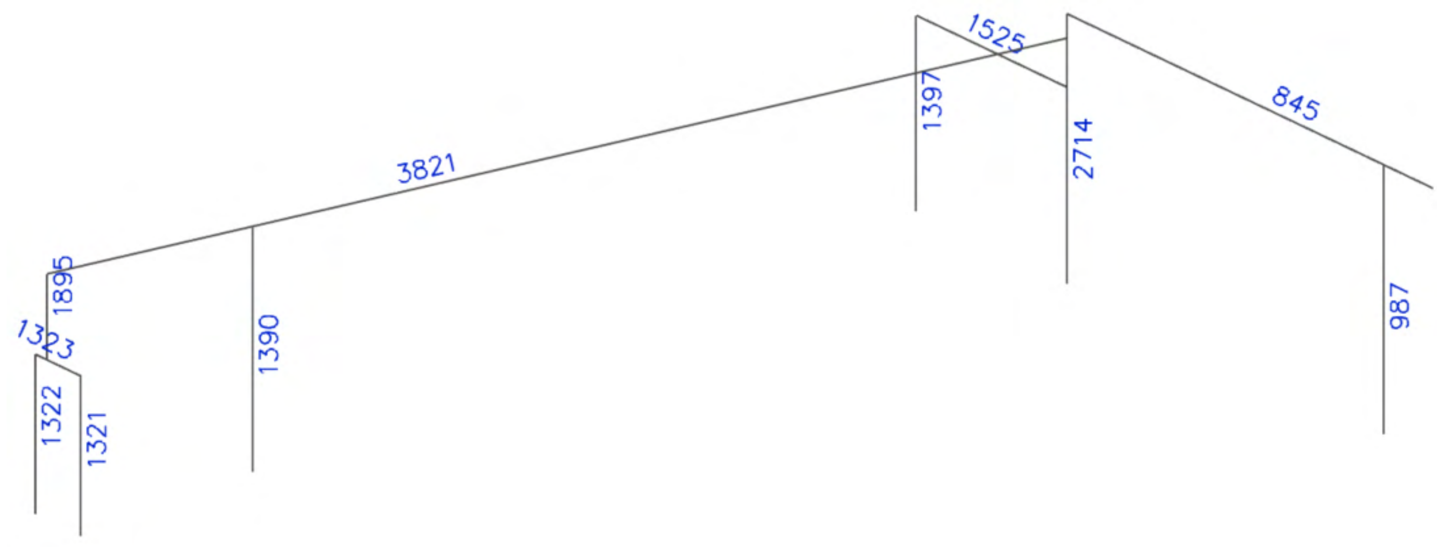
2.4.2 FIRST FLOOR STRUCTURAL ANALYSIS

2.4.2.1 2-ND FLOOR WALL FRAME STRUCTURAL DESIGN

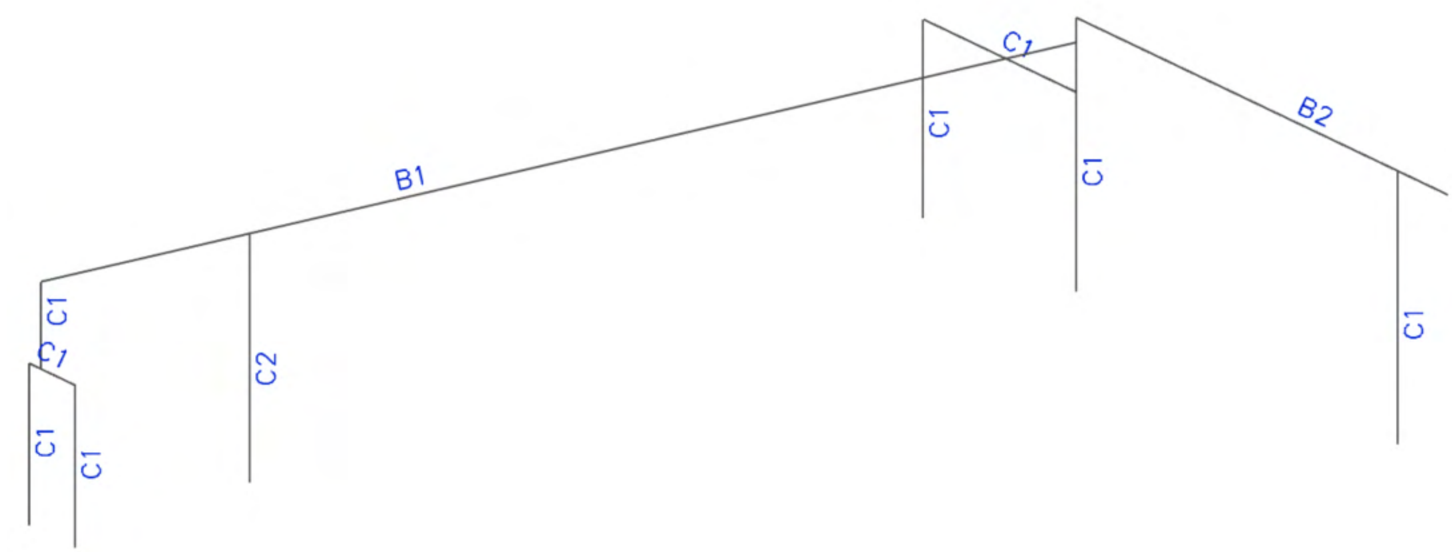
General scheme



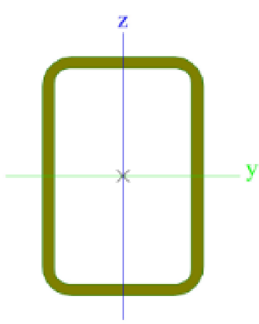
Member numbers



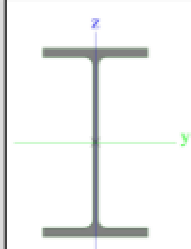
Cross-sections of elements.



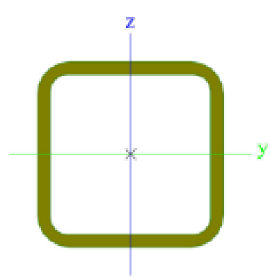
Cross-sections properties

B1		
Type	HSS12X8X5/8	
Formcode	2 - Rectangular hollow section	
Shape type	Thin-walled	
Item material	A500 grade B	
Fabrication	rolled	
Colour		
A [inch^2]	21.000	
A_y [inch^2], A_z [inch^2]	8.202	12.303
A_L [inch^2/inch], A_D [inch^2/inch]	3.75e+01	7.16e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	4.000	6.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	397.000	210.000
i_y [inch], i_z [inch]	4.348	3.162
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	66.100	52.500
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	82.100	61.900
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3.32e+03	3.32e+03
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.52e+03	2.52e+03
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	454.000	4462.080
β_y [inch], β_z [inch]	0.000	0.000
Picture		

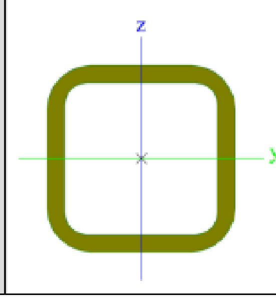
Cross-sections properties

B2		
Type	W10X30	
Formcode	1 - I section	
Shape type	Thin-walled	
Item material	A992	
Fabrication	rolled	
Colour		
A [inch^2]	8.840	
A_y [inch^2], A_z [inch^2]	5.745	3.185
A_L [inch^2/inch], A_D [inch^2/inch]	4.29e+01	4.29e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	2.905	5.250
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	170.000	16.700
i_y [inch], i_z [inch]	4.385	1.374
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	32.400	5.750
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	36.600	8.840
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.86e+03	1.86e+03
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	4.43e+02	4.43e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.622	414.001
β_y [inch], β_z [inch]	0.000	0.000
Picture		

Cross-sections properties

C1		
Type	HSS5X5X3/8	
Formcode	2 - Rectangular hollow section	
Shape type	Thin-walled	
Item material	A500 grade B	
Fabrication	rolled	
Colour		
A [inch^2]	6.180	
A_y [inch^2], A_z [inch^2]	2.996	2.996
A_L [inch^2/inch], A_D [inch^2/inch]	1.85e+01	3.49e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	2.500	2.500
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	21.700	21.700
i_y [inch], i_z [inch]	1.874	1.874
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	8.680	8.680
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	10.600	10.600
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.26e+02	4.26e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	4.26e+02	4.26e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	36.100	90.885
β_y [inch], β_z [inch]	0.000	0.000
Picture		

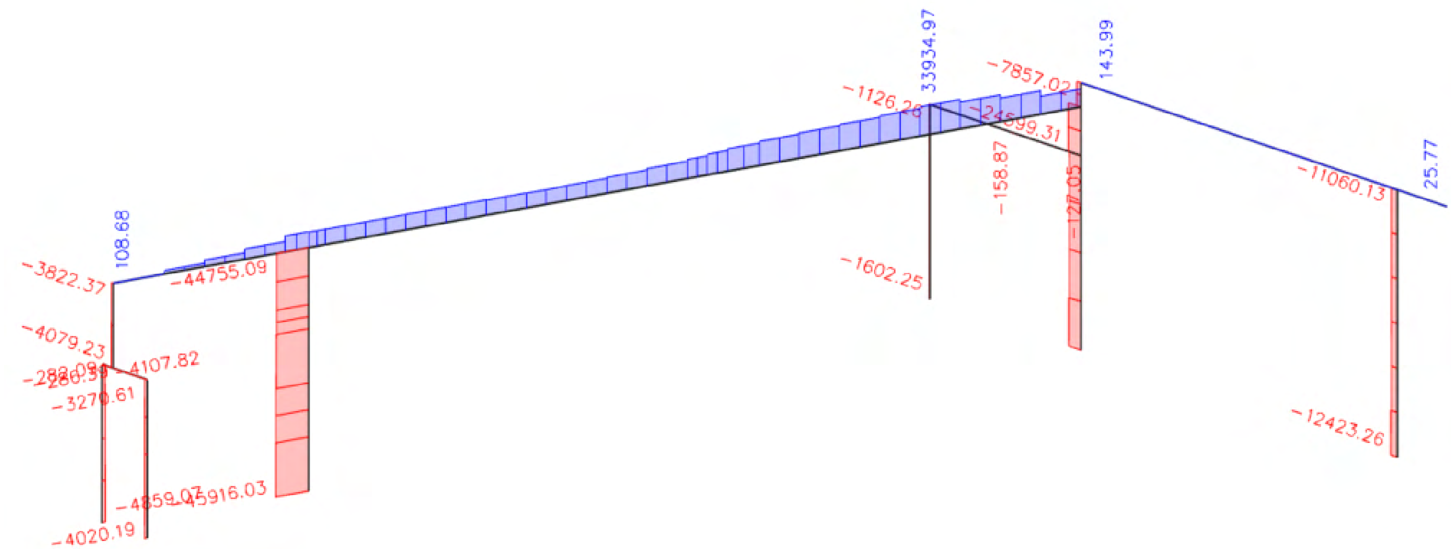
Cross-sections properties

C2		
Type	HSS5X5X1/2	
Formcode	2 - Rectangular hollow section	
Shape type	Thin-walled	
Item material	A500 grade B	
Fabrication	rolled	
Colour		
A [inch^2]	7.880	
A_y [inch^2], A_z [inch^2]	3.784	3.784
A_L [inch^2/inch], A_D [inch^2/inch]	1.80e+01	3.33e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	2.500	2.500
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	26.000	26.000
i_y [inch], i_z [inch]	1.816	1.816
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	10.400	10.400
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	13.100	13.100
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.19e+02	5.19e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	5.19e+02	5.19e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	44.600	121.094
β_y [inch], β_z [inch]	0.000	0.000
Picture		

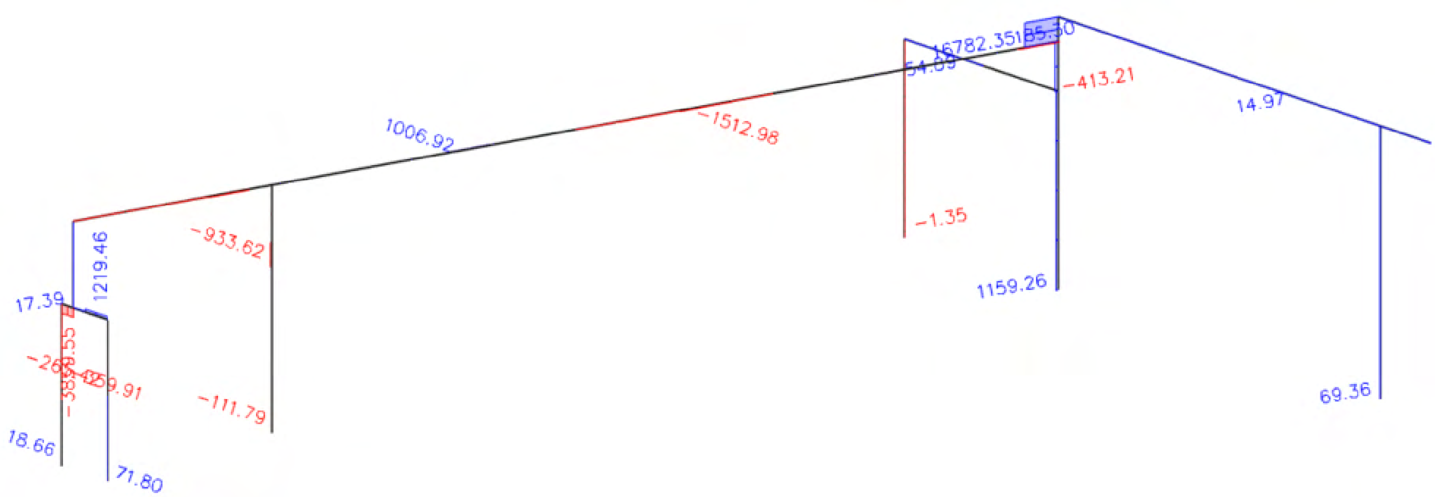
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction 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	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

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



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



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

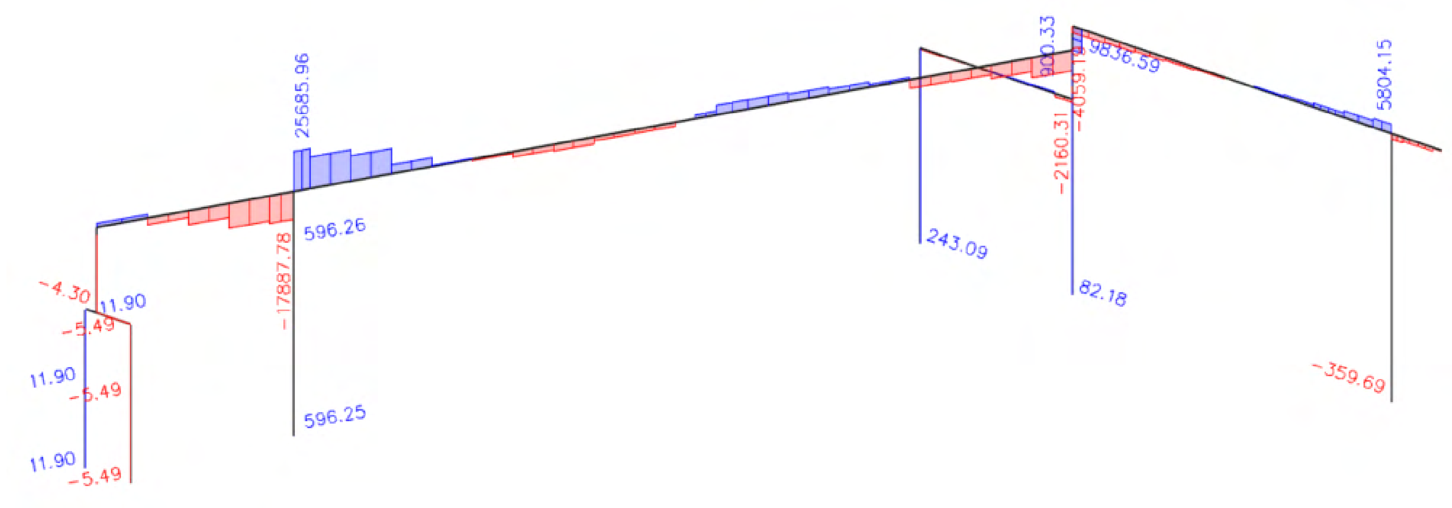


Diagram of moment My,
LRFD-Ult (auto)8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.6*Lr), lbf*ft.

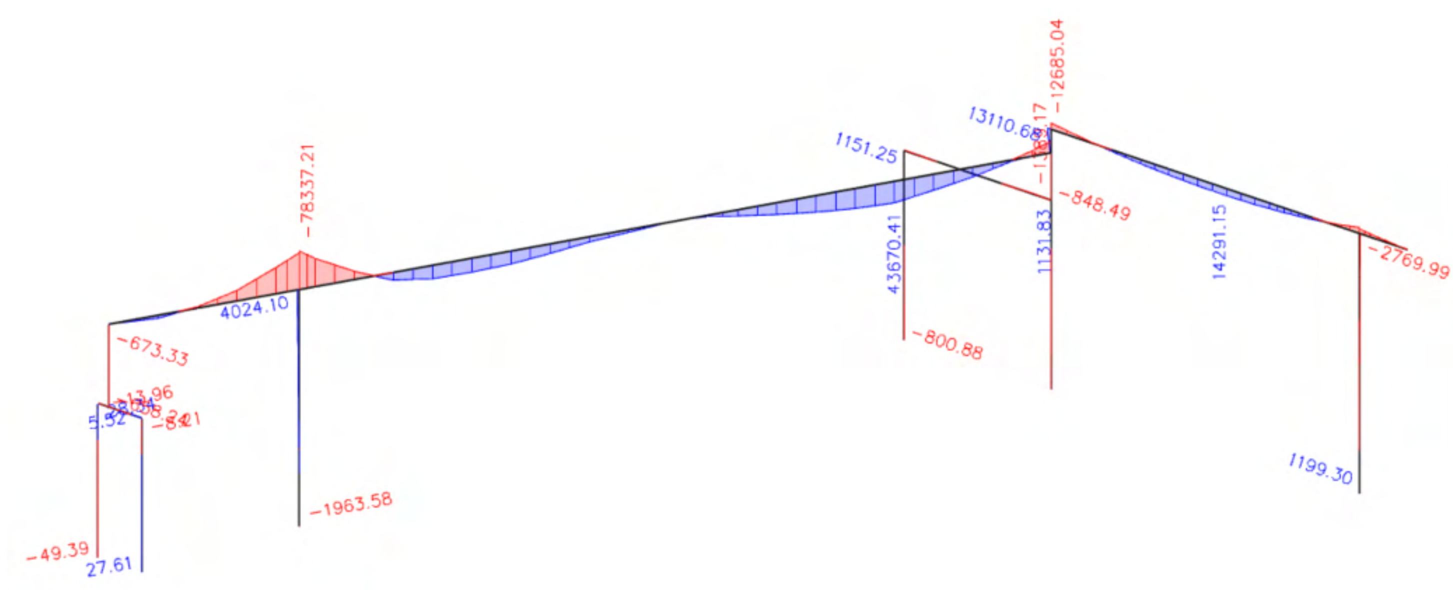
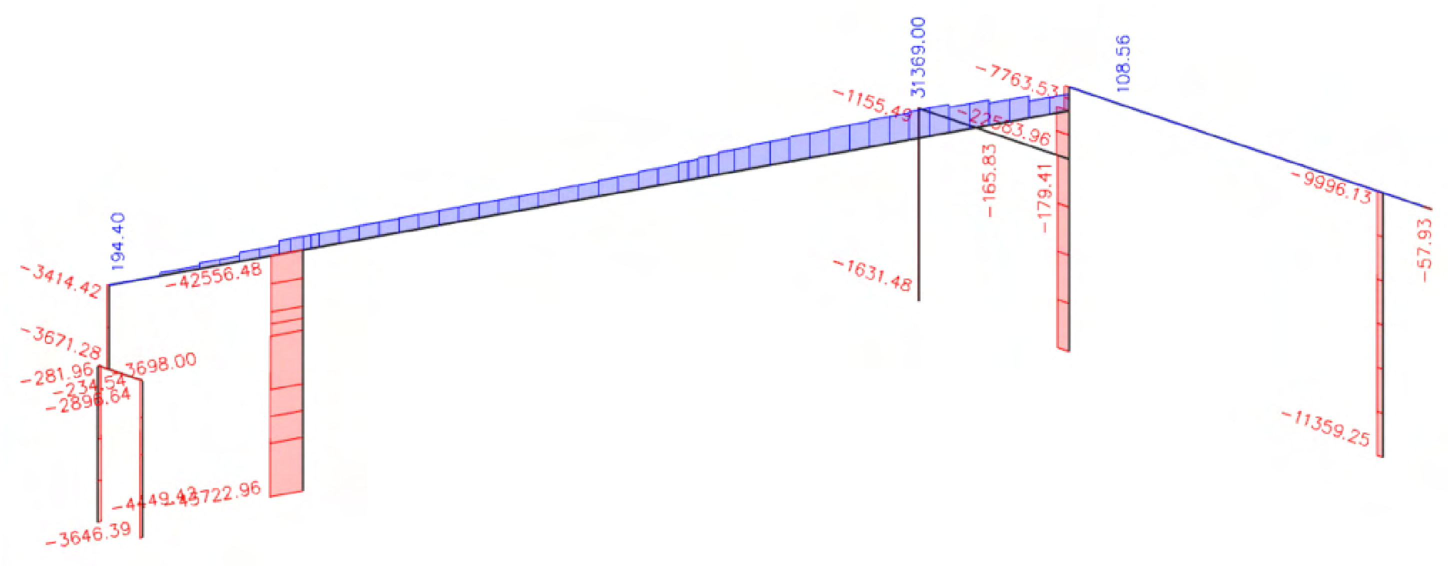
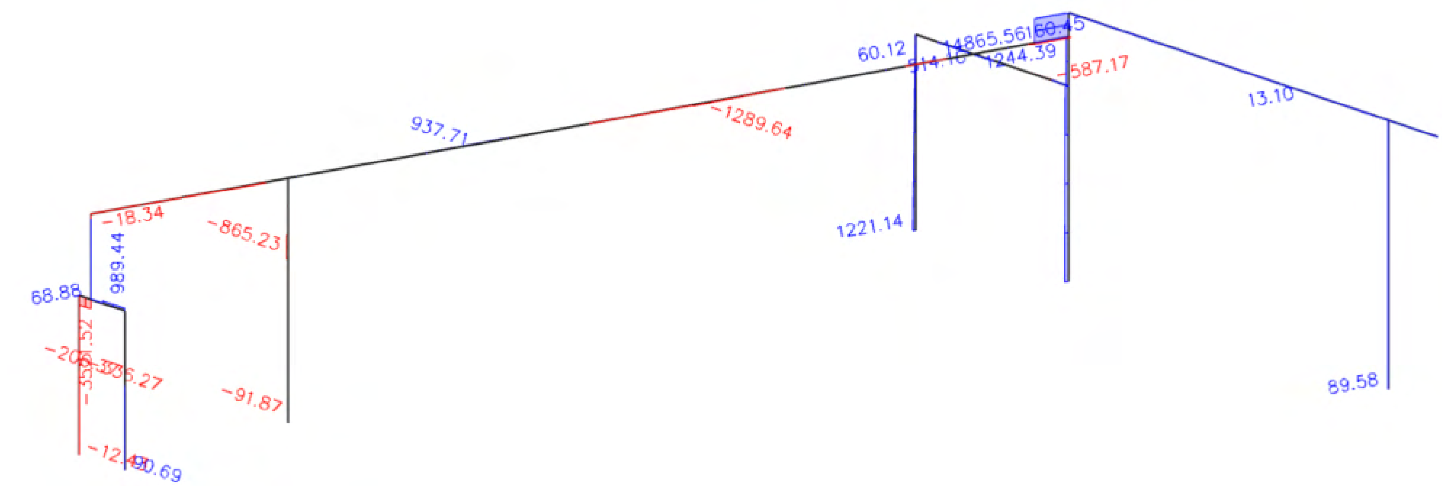


Diagram of moment Mz,
LRFD-Ult (auto)8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 1.6*Lr), lbf*ft.

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



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



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

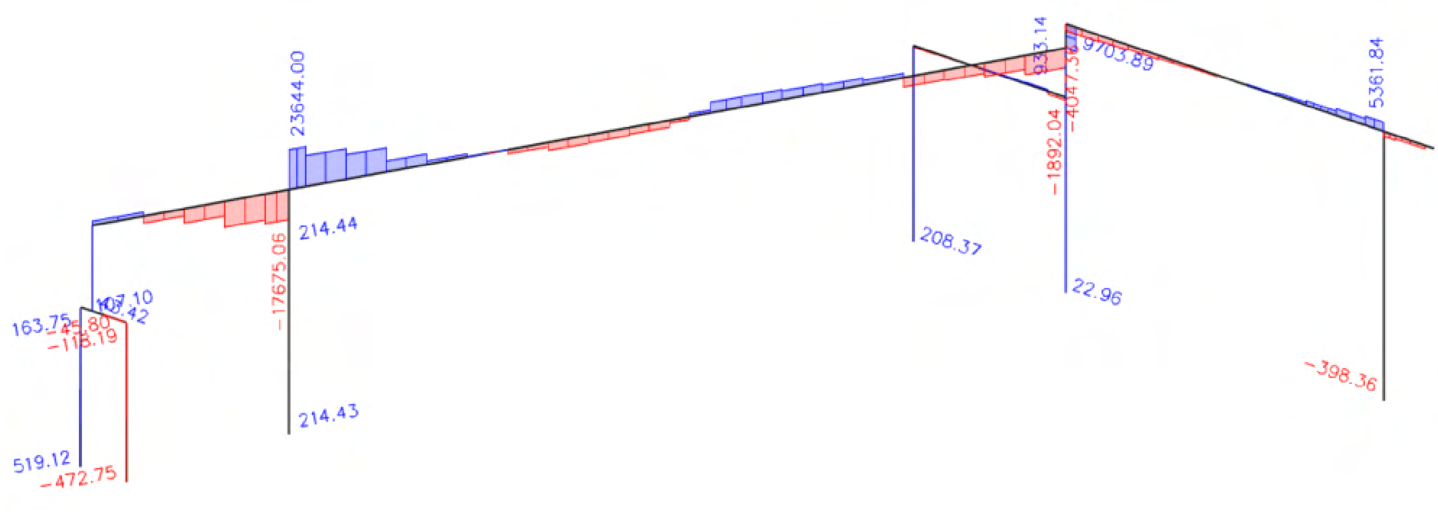


Diagram of moment My,
LRFD-Ult (auto)24 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy-(-0.18)), lbf*ft.

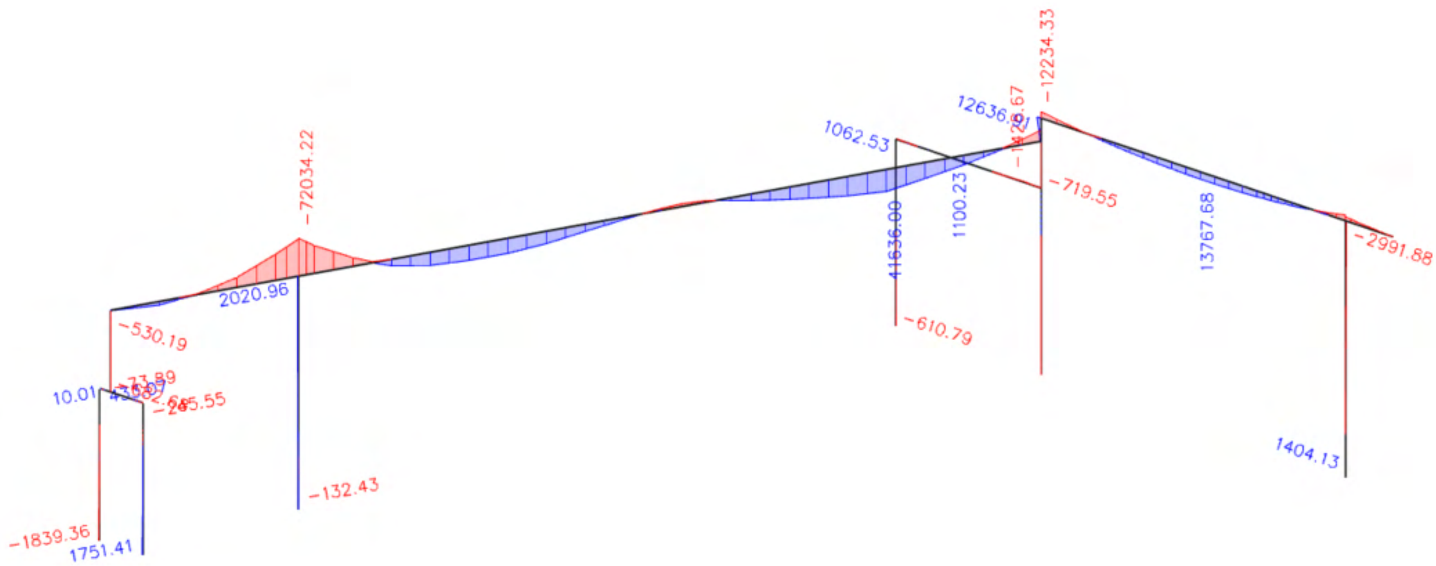
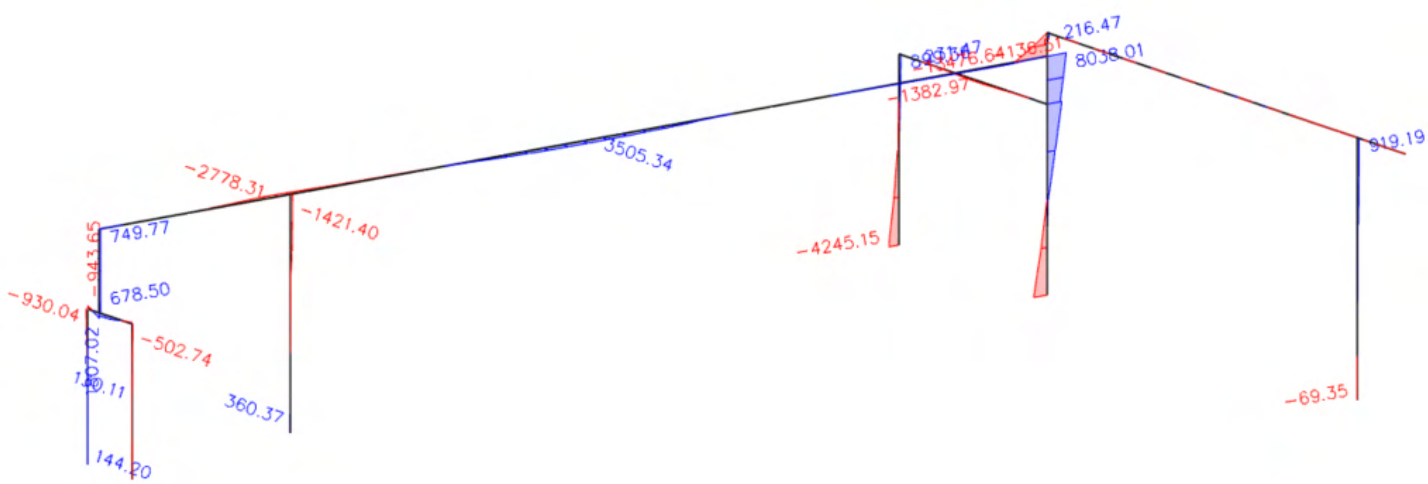
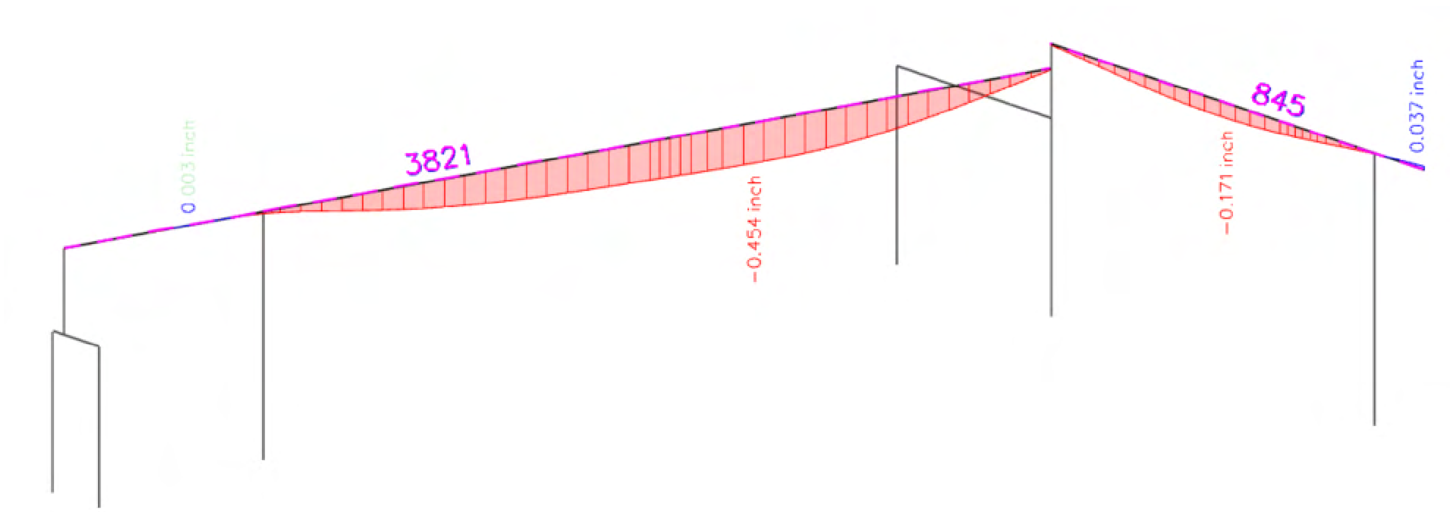


Diagram of moment Mz,
LRFD-Ult (auto)24 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy-(-0.18)), lbf*ft.



Displacement of elements

Value: Uz - DL+L+Lr, (inch) .



The maximum deflection for beam 845 is 0.171" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 21'-5"= 21'*12"+5"=257"/240 =1.07"

0.171" < 1.07" <u>Deflection is OK!</u>

The maximum deflection for beam 3821 is 0.454" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 38'-5"= 38'*12"+5"=461"/240 =1.92"

0.454" < 1.92" <u>Deflection is OK!</u>

STEEL MEMBER 1390 CHECK

AISC 360-16 LRFD Check

Member 1390	HSS5X5X1/2	A500 grade B	LRFD-Ult (auto)	0.28

Material data		
Yield stress Fy	42.00	ksi
Tensile stress Fu	58.00	ksi
fabrication	rolled	

The critical check is on position 7.70 ft

Classification for Axial Compression
according to article B4 and Table B4.1a

Compression N	ratio	Lambda,r (Non-slender)
Webs	6.25	36.79
Internal flanges	6.25	36.79

Section is classified as non-slender section.
Classification for Flexure
according to article B4 and Table B4.1b

Bending Mx	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	6.25	63.60	149.80
Internal flanges	6.25	29.43	36.79

Section is classified as compact section.

Bending My	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	6.25	29.43	36.79
Internal flanges	6.25	29.43	36.79

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	-44980.14	lbf
Vux	-151.05	lbf
Vuy	596.26	lbf
Mut	80.92	lbfft
Mux	2625.78	lbfft
Muy	-1245.20	lbfft

Buckling parameters	xx	yy	
type	sway	non-sway	
Slenderness	67.66	14.23	
Reduced slenderness	0.82	0.17	
Length	10.04	2.35	ft
Effective length factor, K	1.02	0.92	
Effective length, Lc	10.24	2.15	ft

Buckling check
according to article E3 and formula (E3-1)

Table of values		
Pn	249855.45	lbf
Pu	44980.14	lbf
Fcr	31.71	ksi
Resistance factor	0.90	
unity check	0.20	

Torsional buckling check
according to article E4 and formula (E4-1)

Table of values		
Pn	330401.63	lbf
Pu	44980.14	lbf
Fe	10410.88	ksi
Fez	10410.88	ksi
Resistance factor	0.90	
unity check	0.15	

LTB data		
Lb	2.35	ft
Cb	1.16	

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

Table of values		
Mn	45850.10	lbfft
Mu	2625.78	lbfft
Resistance factor	0.90	
unity check	0.06	

Weak axis bending check
according to article F7 and formula (F7-1)

Table of values		
Mn	45850.10	lbfft
Mu	1245.20	lbfft
Resistance factor	0.90	
unity check	0.03	

Shear stress check
according to article G5 and formula (G2-1)
in buckling field 1

Table of values		
a	10.04	ft
h	0.15	ft
tw	0.47	inch
kv	5.34	
Vn	40872.39	lbf
Vu	596.26	lbf
Resistance factor	0.90	
unity check	0.02	

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

Table of values		
Pr	44980.14	lbf
Mrx	2625.78	lbfft
Mry	1245.20	lbfft
Pc	224869.90	lbf
Mcx	41265.09	lbfft
Mcy	41265.09	lbfft
Res. factor compression	0.90	
Res. factor flexure	0.90	

unity check = 0.20+8/9(0.06+0.03) =0.28 (H1-1a)
Torsion is neglected since Tr < 0,20 Tc.

The member satisfies the check !

STEEL MEMBER 845 CHECK

AISC 360-16 LRFD Check

Member 845	W10X30	A992	LRFD-Ult (auto)	0.22

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	rolled	

The critical check is on position 3.35 ft

Classification for Flexure
according to article B4 and Table B4.1b

Bending Mx	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	28.55	90.56	137.29
Outstanding flanges	5.70	9.15	24.09

Section is classified as compact section.

Bending My	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Outstanding flanges	5.70	9.15	24.09

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	160.61	lbf
Vux	50.13	lbf
Vuy	7143.98	lbf
Mut	-11.49	lbfft
Mux	-26408.92	lbfft
Muy	-34.75	lbfft

Normal force check
according to article D2 and formula (D2-1)

Table of values		
Pn	441999.72	lbf
Pu	160.61	lbf
Resistance factor	0.90	
unity check	0.00	

LTB data		
Lb	1.24	ft
Cb	1.15	

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

Table of values		
Lr	16.15	ft
Lp	4.86	ft
Mn	152500.14	lbfft
Mu	-26408.92	lbfft
Resistance factor	0.90	
unity check	0.19	

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

Table of values		
Mn	36833.43	lbfft
Mu	34.75	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	24.79	ft
h	0.71	ft
tw	0.30	inch
kv	5.34	
Vn	94500.01	lbf
Vu	7143.98	lbf
Resistance factor	1.00	
unity check	0.08	

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

Table of values		
Pr	160.61	lbf
Mrx	-26408.92	lbfft
Mry	34.75	lbfft
Pc	397799.75	lbf
Mcx	137250.12	lbfft
Mcy	33150.09	lbfft
Res. factor tension	0.90	
Res. factor flexure	0.90	

unity check = 0.00+0.19+0.00=0.19 (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	-26408.92	lbfft
Muy	34.75	lbfft
fun	9.85	ksi
Fny	50.00	ksi
Res. factor torsion	0.90	

unity check = 9.85/45.00=0.22 (H3-7)
according to article H3 and formula (H3-8)
Critical fibre position 8

Table of values		
Vux	50.13	lbf
Vuy	7143.98	lbf
Mut	11.49	lbfft
fsx	0.000	lbf/ft^2
fsy	51.514	lbf/ft^2
fT	776.904	lbf/ft^2
Stress	2.62	ksi
Fn	30.00	ksi
Res. factor torsion	0.90	

unity check = 2.62/27.00=0.10 (H3-8)
according to article H3 and formula (H3-9)

Table of values		
Pux	160.61	lbf
fua	0.02	ksi
Fnb	50.00	ksi
Res. factor torsion	0.90	

unity check = 0.02/45.00=0.00 (H3-9)

The member satisfies the check !

STEEL MEMBER 3821 CHECK

AISC 360-16 LRFD Check

Member 3821	HSS12X8X5/8	A500 grade B	LRFD-Ult (auto)	0.33

Material data		
Yield stress Fy	42.00	ksi
Tensile stress Fu	58.00	ksi
fabrication	rolled	

The critical check is on position 9.71 ft

Classification for Flexure
according to article B4 and Table B4.1b

Bending Mx	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	16.15	63.60	149.80
Internal flanges	9.27	29.43	36.79

Section is classified as compact section.

Bending My	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	16.15	29.43	36.79
Internal flanges	9.27	29.43	36.79

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	17789.73	lbf
Vux	587.22	lbf
Vuy	25685.96	lbf
Mut	2992.15	lbfft
Mux	-78337.21	lbfft
Muy	-3065.05	lbfft

Normal force check
according to article D2 and formula (D2-1)

Table of values		
Pn	882002.72	lbf
Pu	17789.73	lbf
Resistance factor	0.90	
unity check	0.02	

LTB data		
Lb	0.82	ft
Cb	1.12	

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

Table of values		
Mn	287350.47	lbfft
Mu	-78337.21	lbfft
Resistance factor	0.90	
unity check	0.30	

Weak axis bending check
according to article F7 and formula (F7-1)

Table of values		
Mn	216650.19	lbfft
Mu	3065.05	lbfft
Resistance factor	0.90	
unity check	0.02	

Shear stress check
according to article G5 and formula (G2-1)
in buckling field 1

Table of values		
a	48.12	ft
h	0.66	ft
tw	0.58	inch
kv	5.34	
Vn	232268.03	lbf

Table of values		
Vu	25685.96	lbf
Resistance factor	0.90	
unity check	0.12	

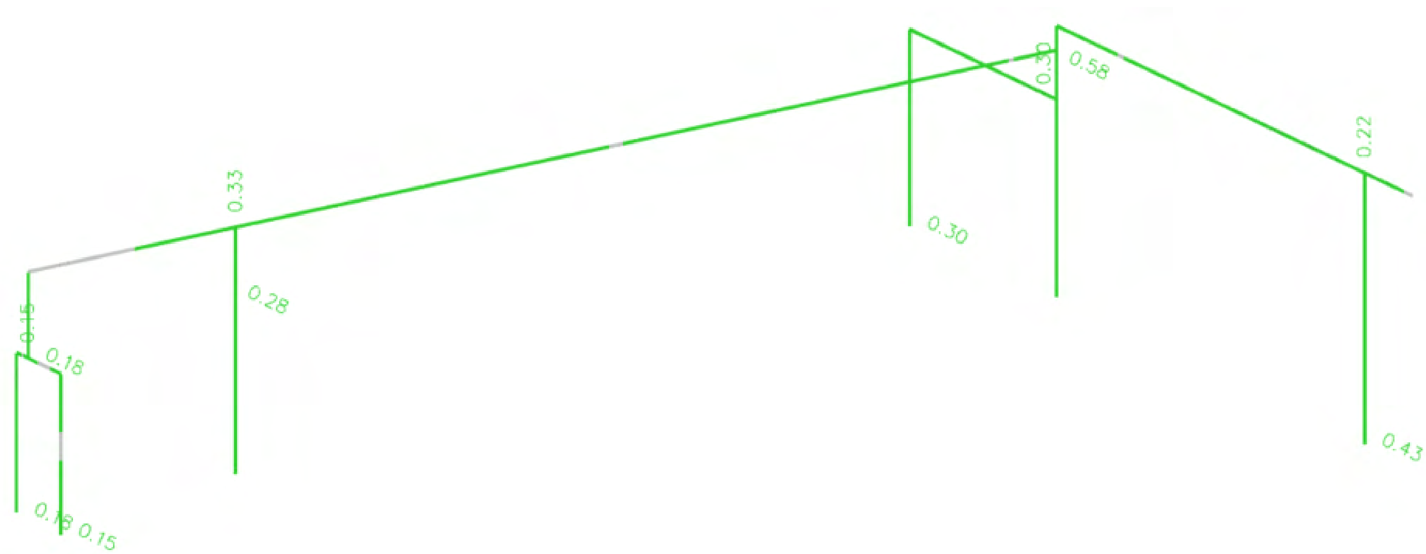
Combined stresses check
according to article H1.2 and formula (H1-1b)

Table of values		
Pr	17789.73	lbf
Mrx	-78337.21	lbfft
Mry	3065.05	lbfft
Pc	793802.45	lbf
Mcx	258615.43	lbfft
Mcy	194985.17	lbfft
Res. factor tension	0.90	
Res. factor flexure	0.90	

unity check = 0.01+0.30+0.02=0.33 (H1-1b)
Torsion is neglected since Tr < 0,20 Tc.

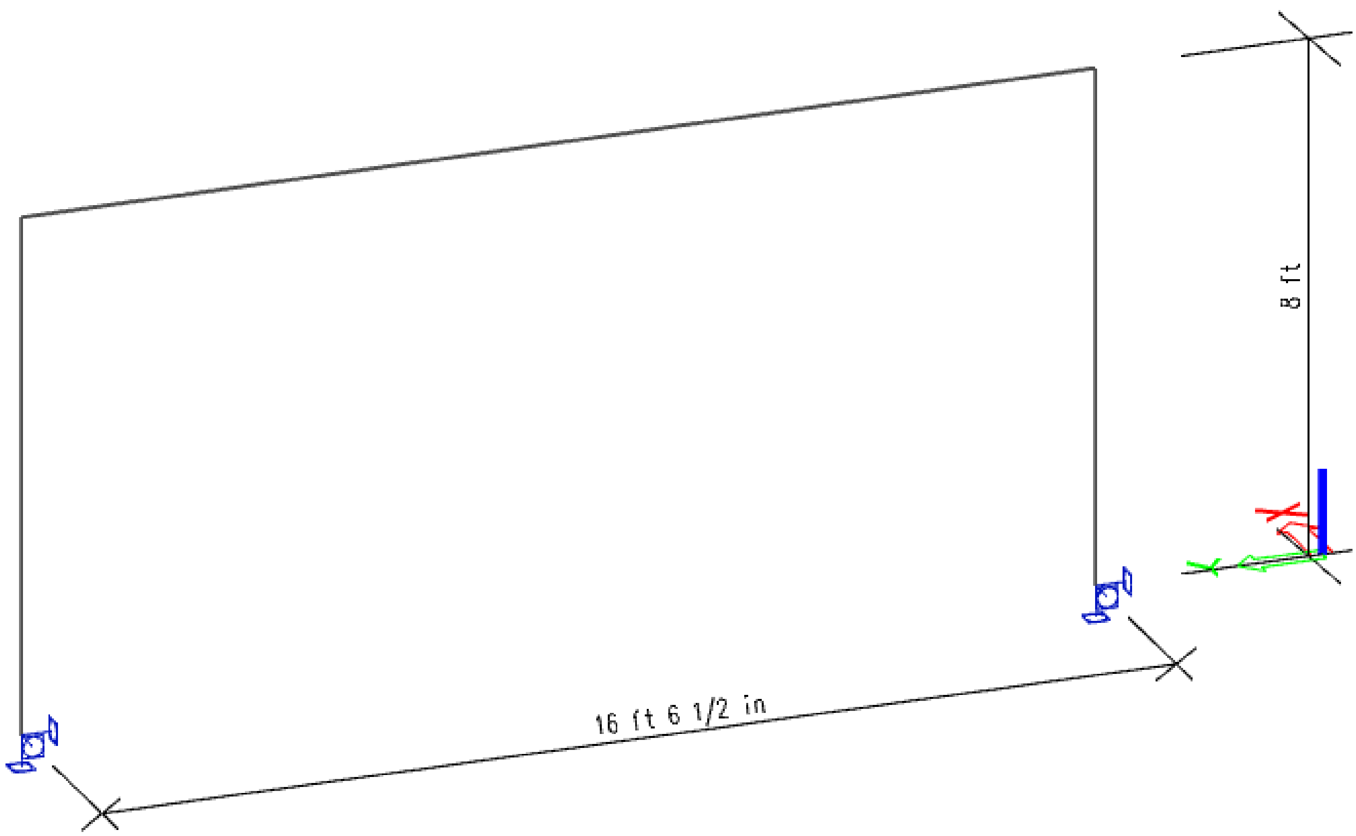
The member satisfies the check !

Unity check

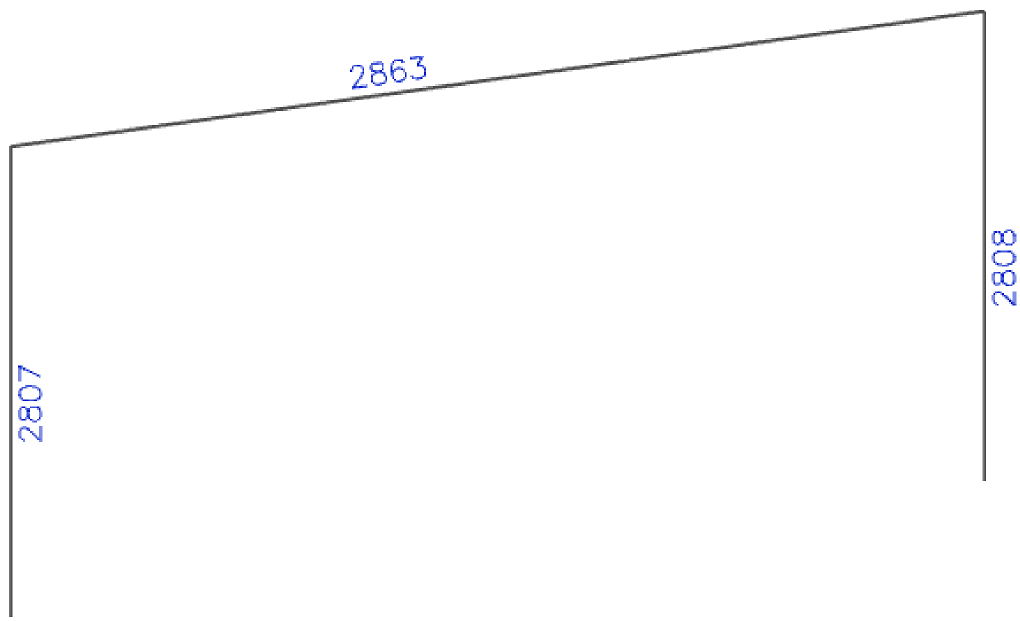


2.4.2.2 GARAGE DOOR FRAME DESIGN

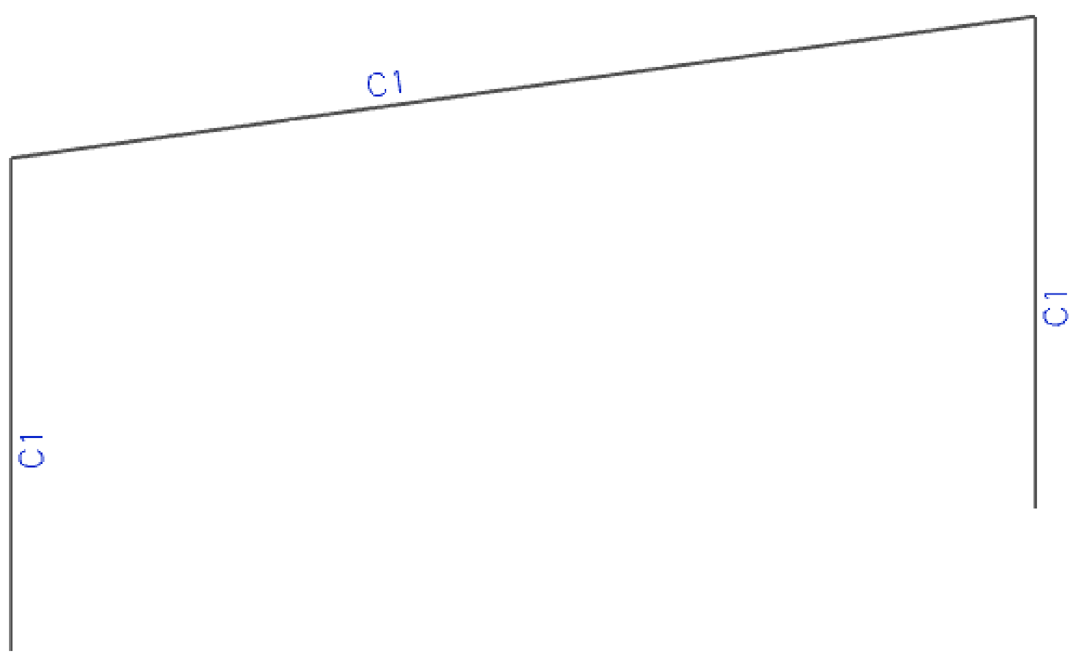
General scheme



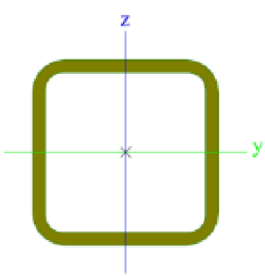
Member numbers



Cross-sections of elements.



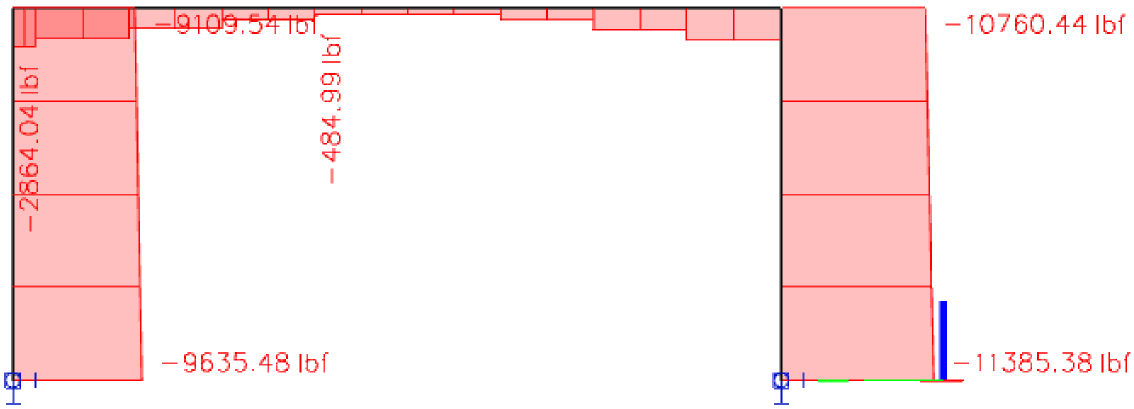
Cross-sections properties

C1		
Type	HSS5X5X3/8	
Formcode	2 - Rectangular hollow section	
Shape type	Thin-walled	
Item material	A500 grade B	
Fabrication	rolled	
Colour		
A [$inch^2$]	6.180	
A_y [$inch^2$], A_z [$inch^2$]	2.996	2.996
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	1.85e+01	3.49e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	2.500	2.500
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	21.700	21.700
i_y [inch], i_z [inch]	1.874	1.874
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	8.680	8.680
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	10.600	10.600
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.26e+02	4.26e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	4.26e+02	4.26e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [$inch^4$], I_w [$inch^6$]	36.100	90.885
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	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

Maximum force diagram
Axial force diagram N,
LRFD-Ult (auto)22 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy+(-0.18)), lbf.



Shear force diagram Vy ,
LRFD-Ult (auto)22 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy+(-0.18)), lbf.

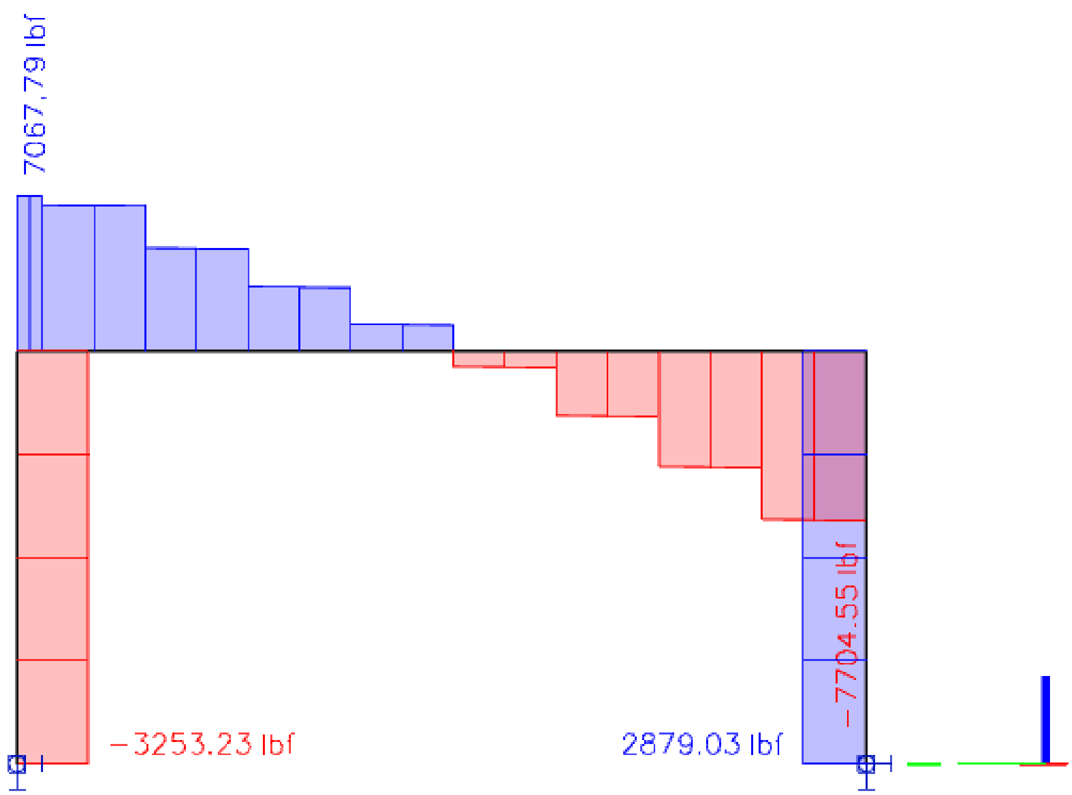
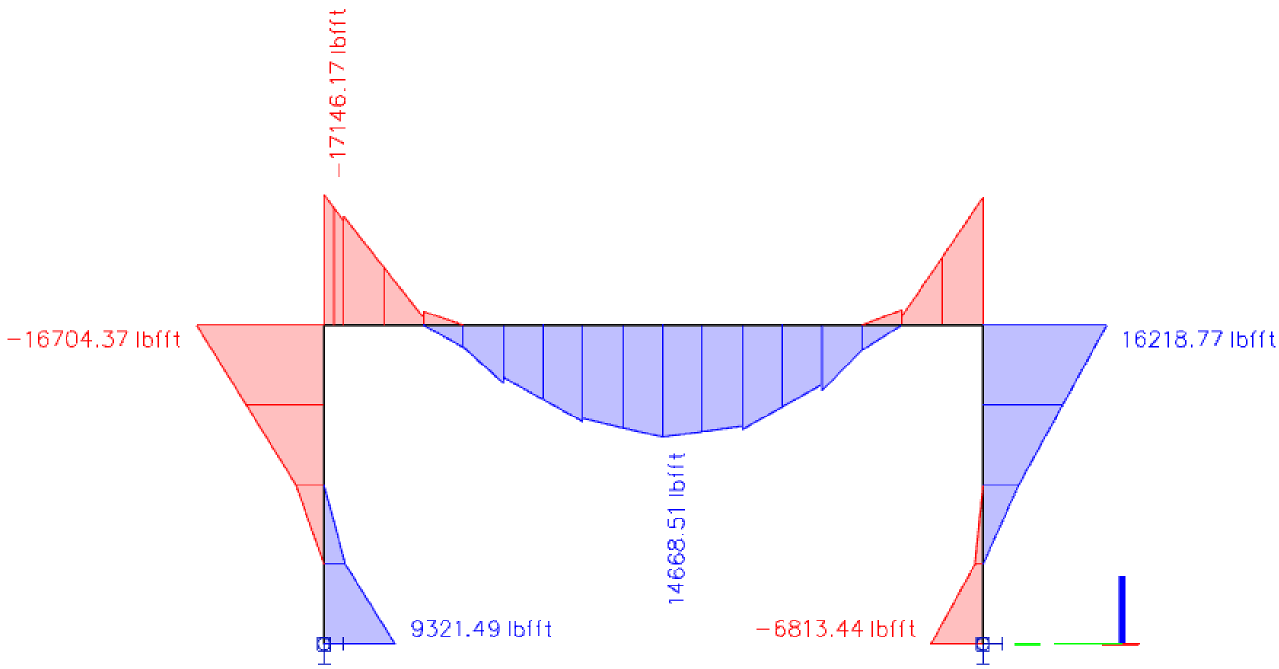
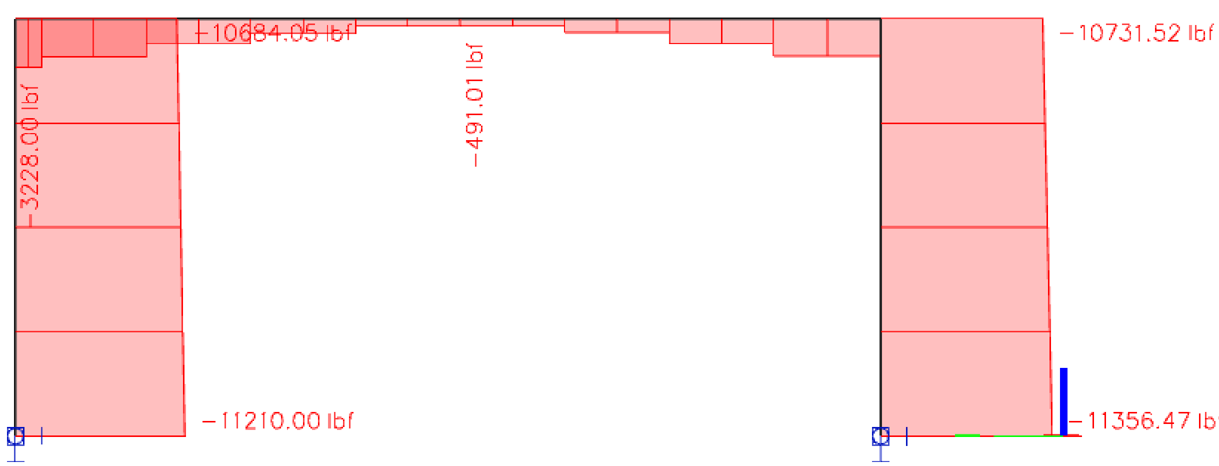


Diagram of moment Mz,
LRFD-Ult (auto)22 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy+(-0.18)), lbf*ft.



Axial force diagram N,
LRFD-Ult (auto)18 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wx+(-0.18)), lbf.



Shear force diagram Vy ,
LRFD-Ult (auto)18 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wx+(-0.18)), lbf.

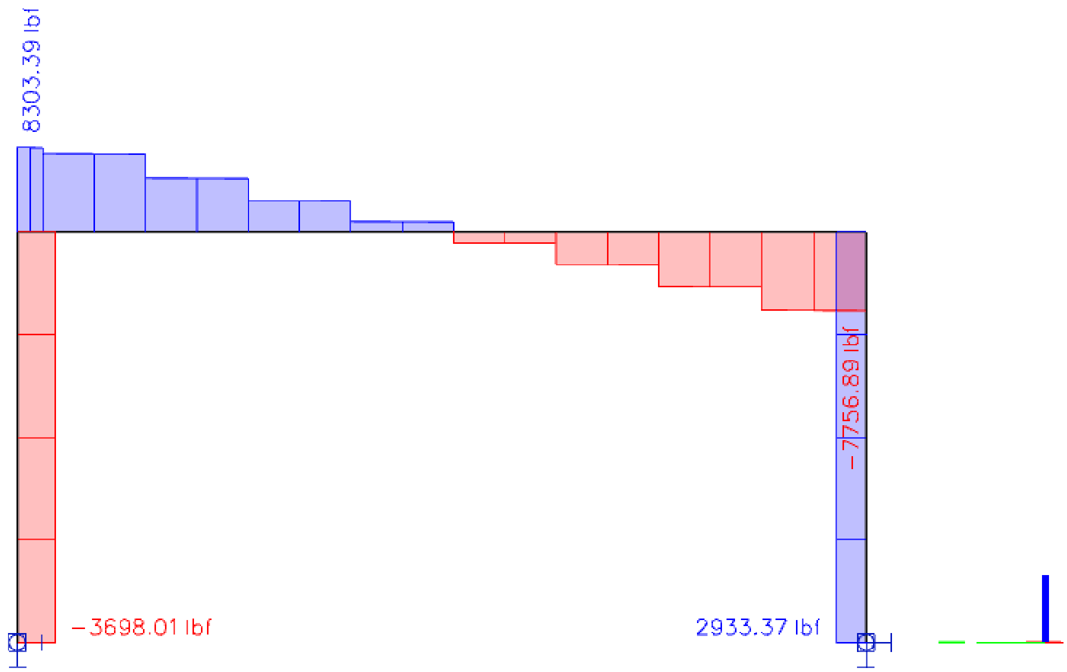
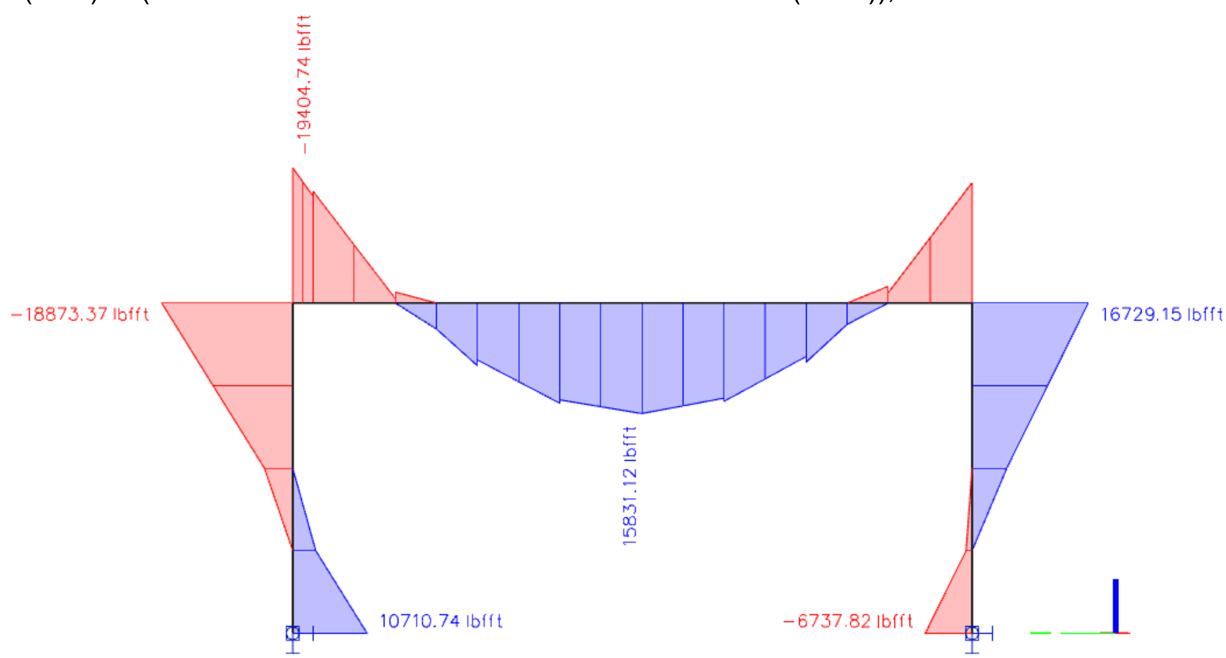
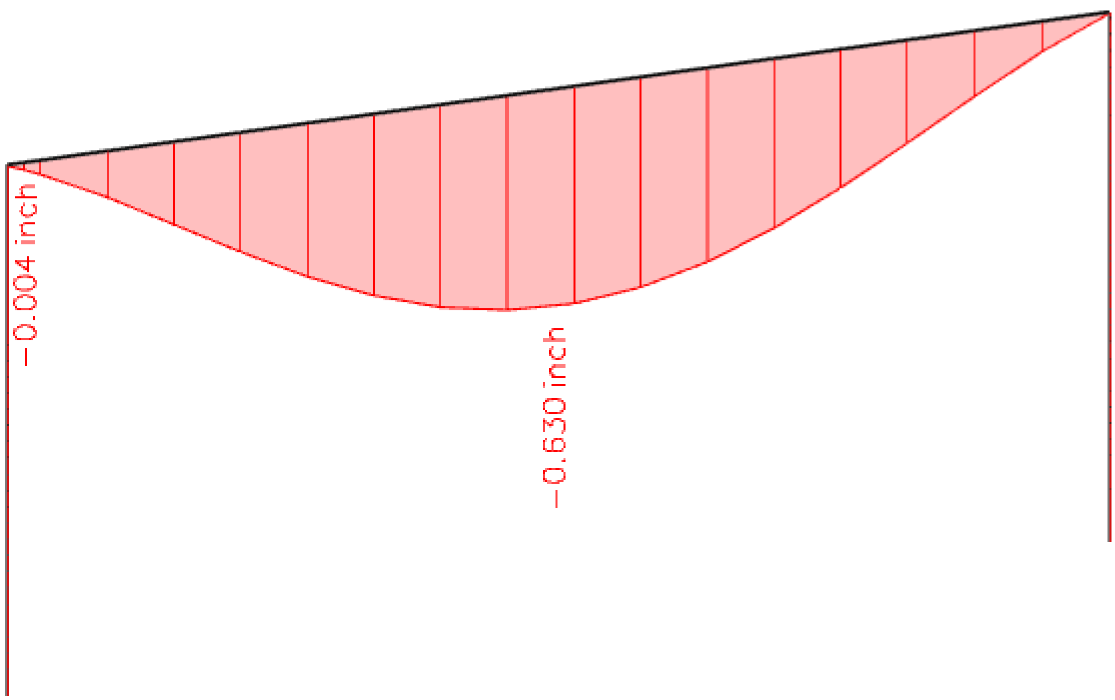


Diagram of moment Mz,
LRFD-Ult (auto)18 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wx+(-0.18)), lbf*ft.



Displacement of elements

Value: Uz - DL+Lr, (inch) .



The maximum deflection for terrace joist is 0.630" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 16’-6”= 16’*12”+6”=198”/240 =0.825”

0.630” < 0.825” **<u>Deflection is OK!</u>**

STEEL MEMBER 2807 CHECK

AISC 360-16 LRFD Check

Member 2807	HSS5X5X3/8	A500 grade B	LRFD-Ult (auto)	0.96

Material data		
Yield stress Fy	42.00	ksi
Tensile stress Fu	58.00	ksi
fabrication	rolled	

The critical check is on position 0.00 ft

Classification for Axial Compression
according to article B4 and Table B4.1a

Compression N	ratio	Lambda,r (Non-slender)
Webs	9.83	36.79
Internal flanges	9.83	36.79

Section is classified as non-slender section.
Classification for Flexure
according to article B4 and Table B4.1b

Bending Mx	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	9.83	63.60	149.80
Internal flanges	9.83	29.43	36.79

Section is classified as compact section.

Bending My	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	9.83	29.43	36.79
Internal flanges	9.83	29.43	36.79

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	-8744.59	lbf
Vux	4561.52	lbf
Vuy	3258.66	lbf
Mut	-535.18	lbfft
Mux	-10536.00	lbfft
Muy	-20347.04	lbfft

Buckling parameters	xx	yy	
type	sway	sway	
Slenderness	66.73	102.46	
Reduced slenderness	0.81	1.24	
Length	8.00	8.00	ft
Effective length factor, K	1.30	2.00	
Effective length, Lc	10.42	16.00	ft

Buckling check
according to article E3 and formula (E3-1)

Table of values		
Pn	136230.40	lbf
Pu	8744.59	lbf
Fcr	22.04	ksi
Resistance factor	0.90	
unity check	0.07	

Torsional buckling check
according to article E4 and formula (E4-1)

Table of values		
Pn	259072.32	lbf
Pu	8744.59	lbf
Fe	9345.23	ksi
Fez	9345.23	ksi
Resistance factor	0.90	
unity check	0.04	

LTB data		
Lb	8.00	ft
Cb	2.21	

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

Table of values		
Mn	37100.03	lbfft
Mu	-10536.00	lbfft
Resistance factor	0.90	
unity check	0.32	

Weak axis bending check
according to article F7 and formula (F7-1)

Table of values		
Mn	37100.03	lbfft
Mu	20347.04	lbfft
Resistance factor	0.90	
unity check	0.61	

Shear stress check
according to article G5 and formula (G2-1)
in buckling field 1

Table of values		
a	8.00	ft
h	0.21	ft
tw	0.35	inch
kv	5.34	
Vn	44959.02	lbf
Vu	3258.66	lbf
Resistance factor	0.90	
unity check	0.08	

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

Table of values		
Pr	8744.59	lbf
Mrx	-10536.00	lbfft
Mry	20347.04	lbfft
Pc	122607.36	lbf
Mcx	33390.03	lbfft
Mcy	33390.03	lbfft
Res. factor compression	0.90	
Res. factor flexure	0.90	

unity check = 0.04+0.32+0.61=0.96 (H1-1b)
Torsion is neglected since Tr < 0,20 Tc.

The member satisfies the check !

STEEL MEMBER 2863 CHECK

AISC 360-16 LRFD Check

Member 2863	HSS5X5X3/8	A500 grade B	LRFD-Ult (auto)	0.61

Material data		
Yield stress Fy	42.00	ksi
Tensile stress Fu	58.00	ksi
fabrication	rolled	

The critical check is on position 0.00 ft

Classification for Axial Compression
according to article B4 and Table B4.1a

Compression N	ratio	Lambda,r (Non-slender)
Webs	9.83	36.79
Internal flanges	9.83	36.79

Section is classified as non-slender section.

Classification for Flexure
according to article B4 and Table B4.1b

Bending Mx	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	9.83	63.60	149.80
Internal flanges	9.83	29.43	36.79

Section is classified as compact section.

Bending My	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	9.83	29.43	36.79
Internal flanges	9.83	29.43	36.79

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	-3228.00	lbf
Vux	8303.39	lbf
Vuy	230.75	lbf
Mut	15.91	lbfft
Mux	-267.61	lbfft
Muy	-19404.74	lbfft

Buckling parameters	xx	yy	
type	sway	sway	
Slenderness	32.02	141.07	
Reduced slenderness	0.39	1.71	
Length	0.50	16.54	ft
Effective length factor, K	10.00	1.33	
Effective length, Lc	5.00	22.03	ft

Buckling check
according to article E3 and formula (E3-1)

Table of values		
Pn	77965.53	lbf
Pu	3228.00	lbf
Fcr	12.62	ksi
Resistance factor	0.90	
unity check	0.05	

Torsional buckling check
according to article E4 and formula (E4-1)

Table of values		
Pn	259069.71	lbf
Pu	3228.00	lbf
Fe	9295.40	ksi
Fez	9295.40	ksi
Resistance factor	0.90	
unity check	0.01	

LTB data		
Lb	16.54	ft
Cb	1.21	

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

Table of values		
Mn	37100.03	lbfft
Mu	-267.61	lbfft
Resistance factor	0.90	
unity check	0.01	

Weak axis bending check
according to article F7 and formula (F7-1)

Table of values		
Mn	37100.03	lbfft
Mu	19404.74	lbfft
Resistance factor	0.90	
unity check	0.58	

Shear stress check
according to article G5 and formula (G2-1)
in buckling field 1

Table of values		
a	16.54	ft
h	0.21	ft
tw	0.35	inch
kv	5.34	
Vn	44959.02	lbf
Vu	230.75	lbf
Resistance factor	0.90	
unity check	0.01	

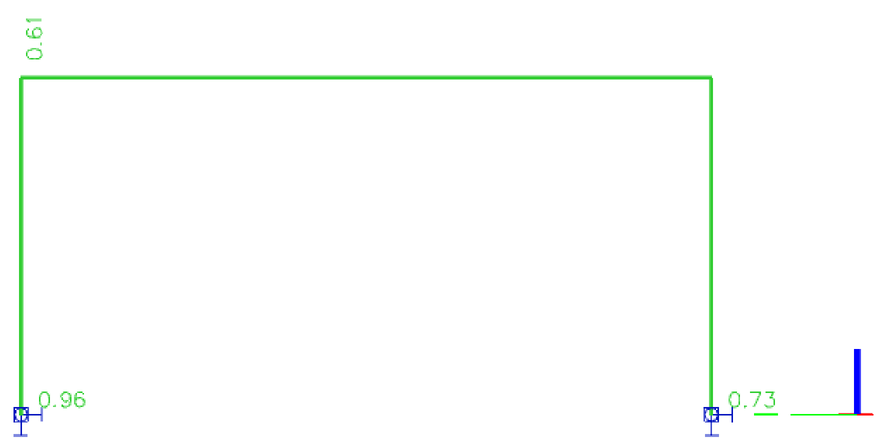
Combined stresses check
according to article H1.1 and formula (H1-1b)

Table of values		
Pr	3228.00	lbf
Mrx	-267.61	lbfft
Mry	19404.74	lbfft
Pc	70168.97	lbf
Mcx	33390.03	lbfft
Mcy	33390.03	lbfft
Res. factor compression	0.90	
Res. factor flexure	0.90	

unity check = 0.02+0.01+0.58=0.61 (H1-1b)
Torsion is neglected since Tr < 0,20 Tc.

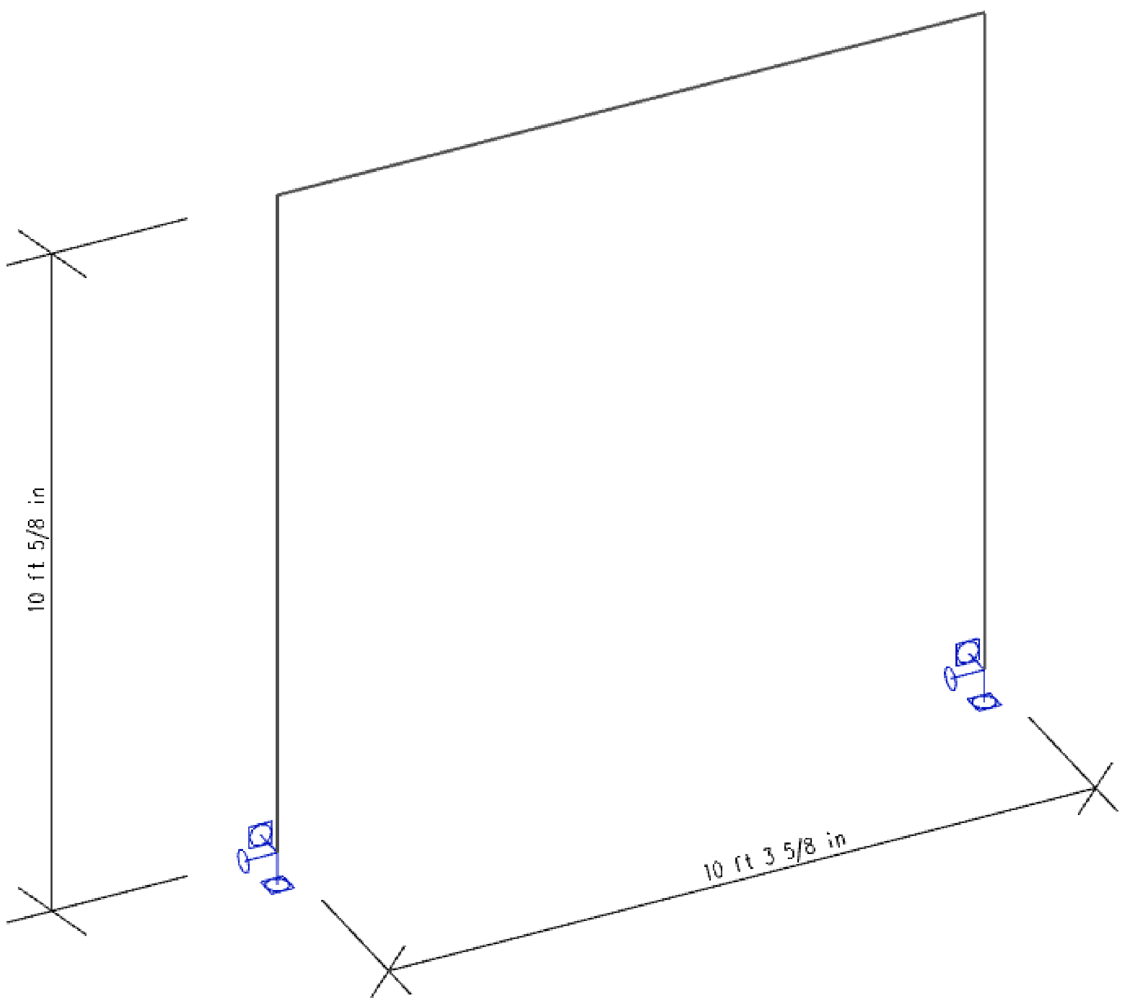
The member satisfies the check !

Unity check

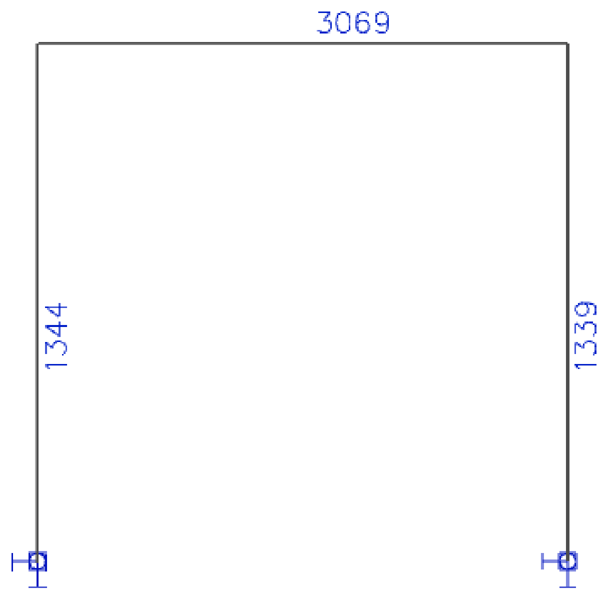


2.4.2.3 DOOR FRAME DESIGN

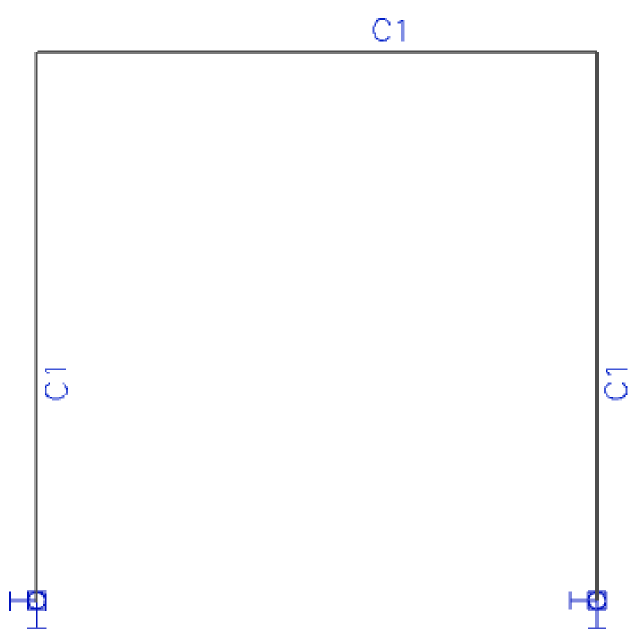
General scheme



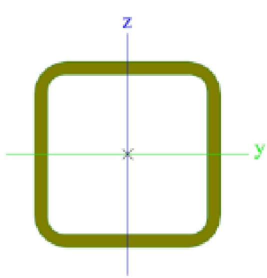
Member numbers



Cross-sections of elements.



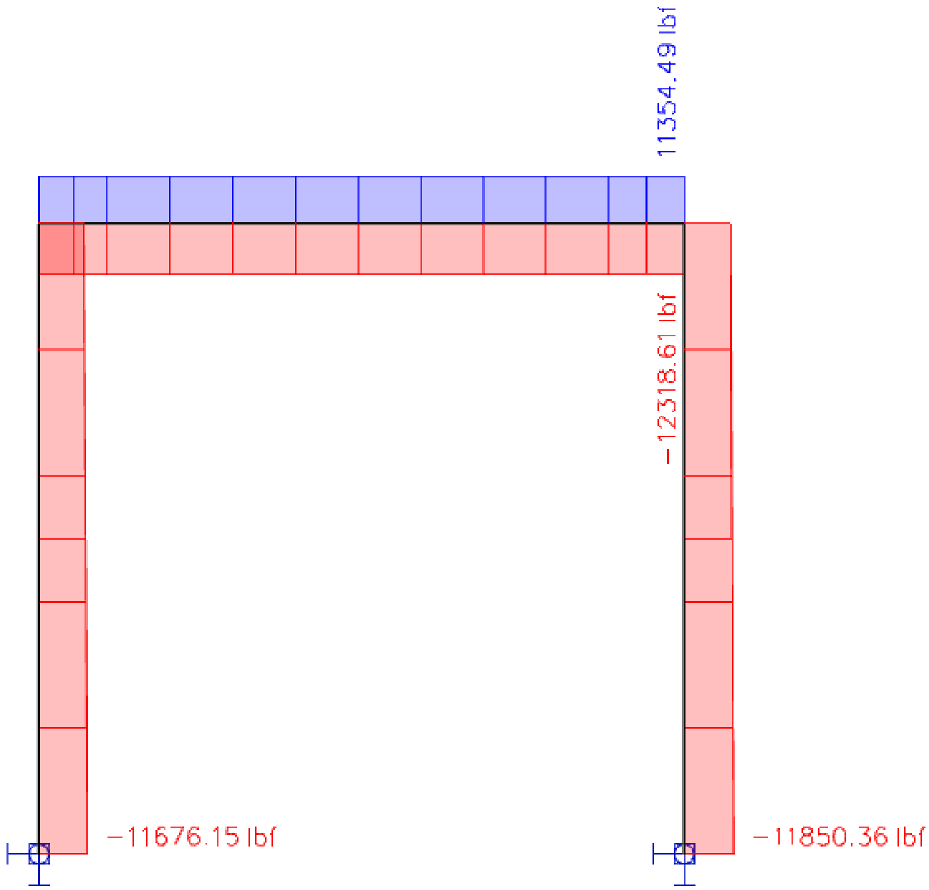
Cross-sections properties

C1		
Type	HSS5X5X3/8	
Formcode	2 - Rectangular hollow section	
Shape type	Thin-walled	
Item material	A500 grade B	
Fabrication	rolled	
Colour		
A [inch^2]	6.180	
A_y [inch^2], A_z [inch^2]	2.996	2.996
A_L [inch^2/inch], A_D [inch^2/inch]	1.85e+01	3.49e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	2.500	2.500
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	21.700	21.700
i_y [inch], i_z [inch]	1.874	1.874
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	8.680	8.680
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	10.600	10.600
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.26e+02	4.26e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	4.26e+02	4.26e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	36.100	90.885
$β_y$ [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	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

Maximum force diagram
Axial force diagram N,
LRFD-Ult (auto)22 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy+(-0.18)), lbf.



Shear force diagram Vy ,
LRFD-Ult (auto)22 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy+(-0.18)), lbf.

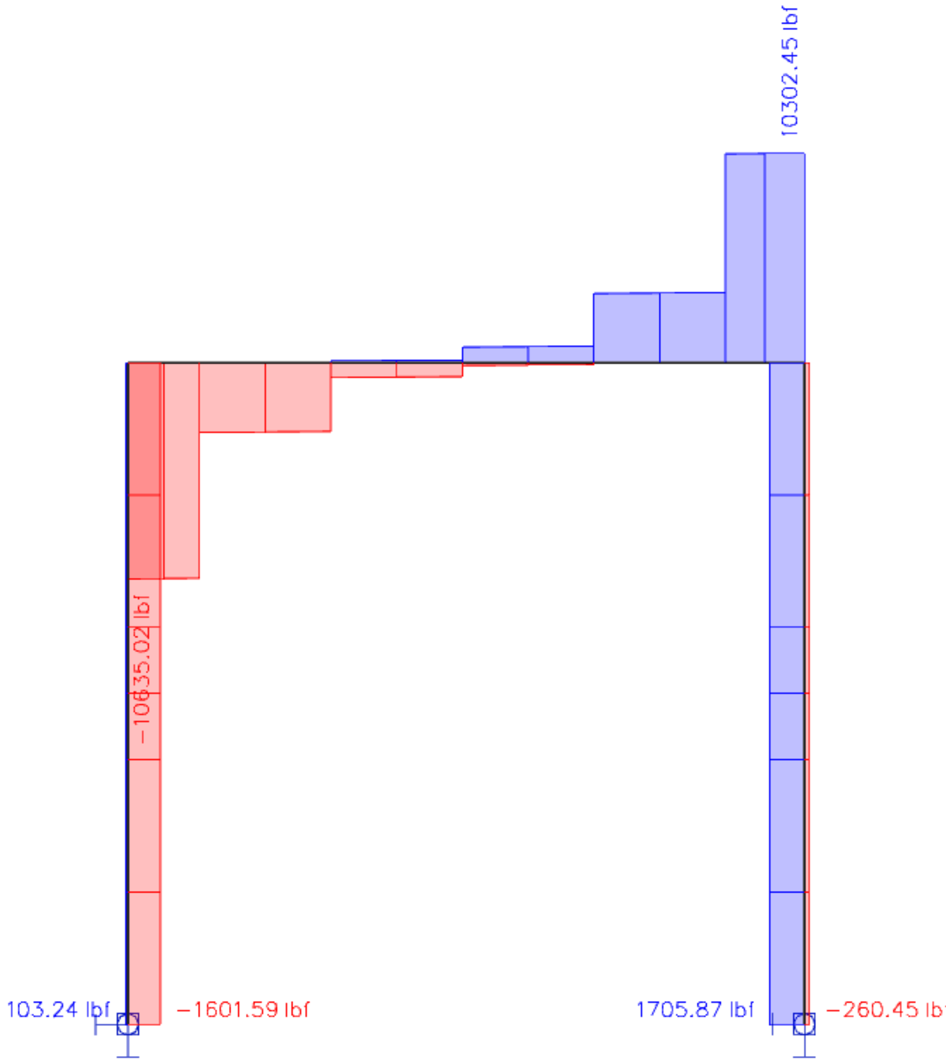
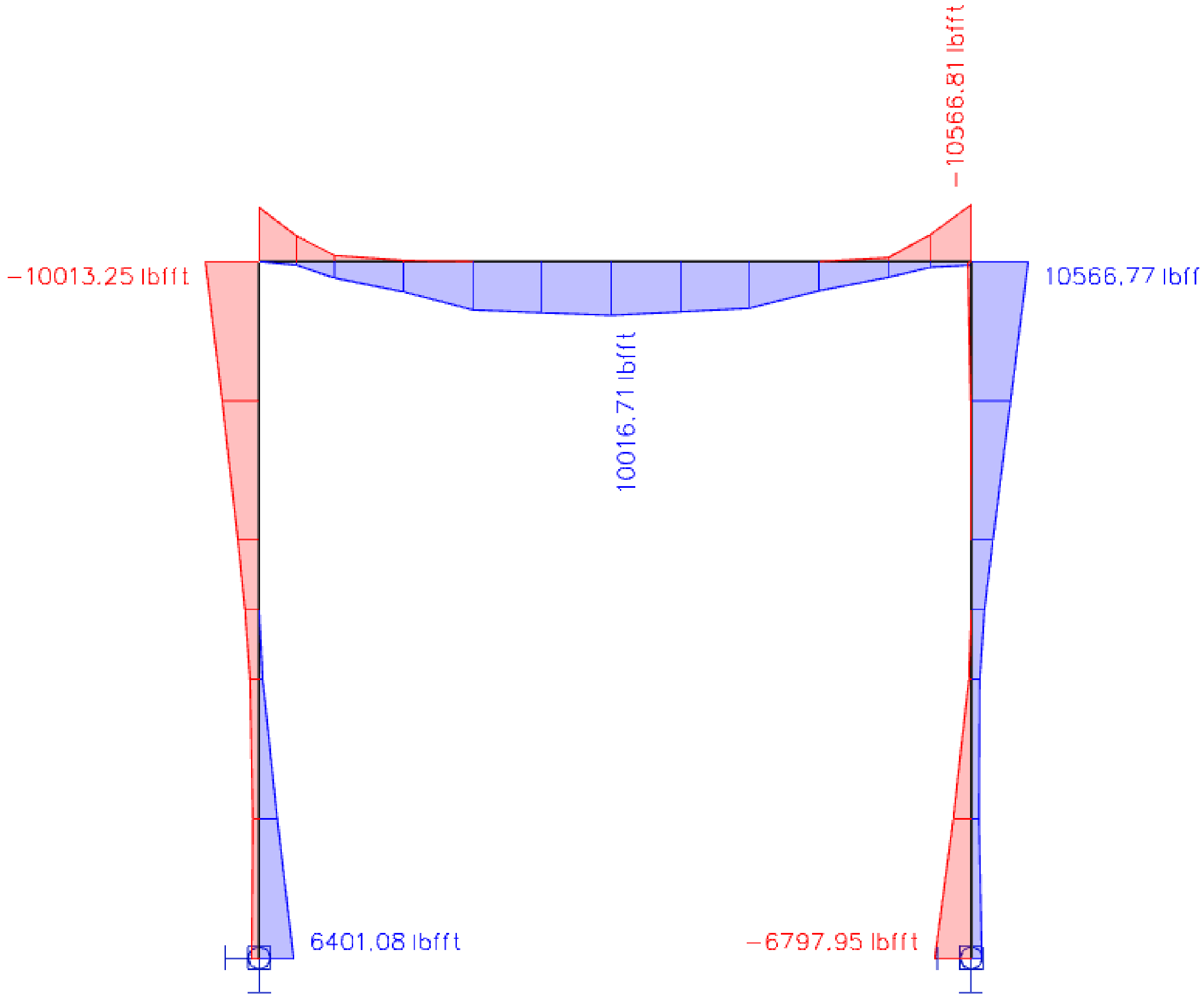
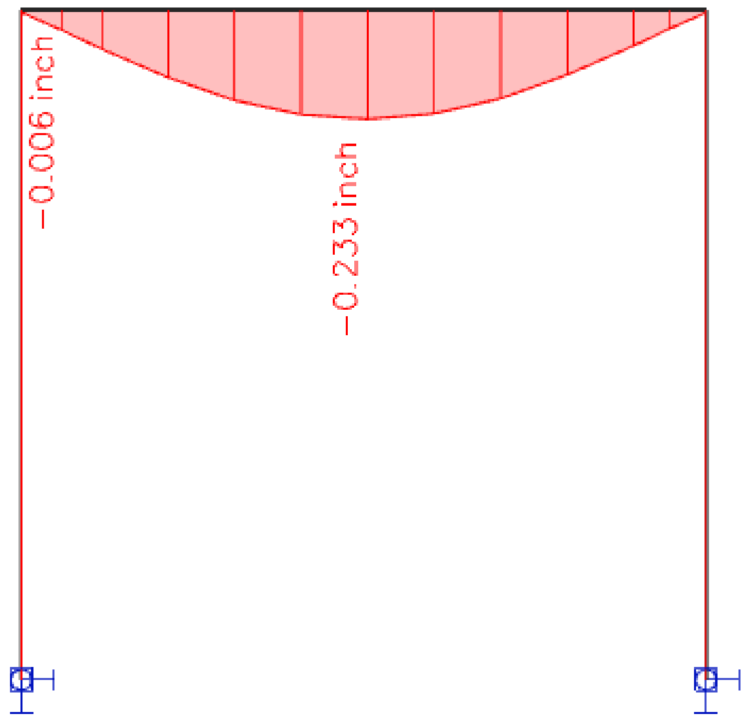


Diagram of moment Mz,
LRFD-Ult (auto)22 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wy+(-0.18)), lbf*ft.



Displacement of elements

Value: Uz - DL+Lr, (inch) .



The maximum deflection for terrace joist is 0.233" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 10’-3.5”= ’*12”+6”=198”/240 =0.825”

0.630” < 0.825” **<u>Deflection is OK!</u>**

STEEL MEMBER 1339 CHECK

AISC 360-16 LRFD Check

Member 1339	HSS5X5X3/8	A500 grade B	LRFD-Ult (auto)	0.39

Material data		
Yield stress Fy	42.00	ksi
Tensile stress Fu	58.00	ksi
fabrication	rolled	

The critical check is on position 10.04 ft

Classification for Axial Compression
according to article B4 and Table B4.1a

Compression N	ratio	Lambda,r (Non-slender)
Webs	9.83	36.79
Internal flanges	9.83	36.79

Section is classified as non-slender section.
Classification for Flexure
according to article B4 and Table B4.1b

Bending Mx	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	9.83	63.60	149.80
Internal flanges	9.83	29.43	36.79

Section is classified as compact section.

Bending My	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	9.83	29.43	36.79
Internal flanges	9.83	29.43	36.79

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	-10755.43	lbf
Vux	-29.50	lbf
Vuy	1644.31	lbf
Mut	-550.55	lbfft
Mux	10545.97	lbfft
Muy	-243.72	lbfft

Buckling parameters	xx	yy	
type	sway	sway	
Slenderness	74.47	128.62	
Reduced slenderness	0.90	1.56	
Length	10.04	10.04	ft
Effective length factor, K	1.16	2.00	
Effective length, Lc	11.63	20.08	ft

Buckling check
according to article E3 and formula (E3-1)

Table of values		
Pn	93796.93	lbf
Pu	10755.43	lbf
Fcr	15.18	ksi
Resistance factor	0.90	
unity check	0.13	

Torsional buckling check
according to article E4 and formula (E4-1)

Table of values		
Pn	259071.08	lbf
Pu	10755.43	lbf
Fe	9321.47	ksi
Fez	9321.47	ksi
Resistance factor	0.90	
unity check	0.05	

LTB data		
Lb	10.04	ft
Cb	2.19	

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

Table of values		
Mn	37100.03	lbfft
Mu	10545.97	lbfft
Resistance factor	0.90	
unity check	0.32	

Weak axis bending check
according to article F7 and formula (F7-1)

Table of values		
Mn	37100.03	lbfft
Mu	243.72	lbfft
Resistance factor	0.90	
unity check	0.01	

Shear stress check
according to article G5 and formula (G2-1)
in buckling field 1

Table of values		
a	10.04	ft
h	0.21	ft
tw	0.35	inch
kv	5.34	
Vn	44959.02	lbf
Vu	1644.31	lbf
Resistance factor	0.90	
unity check	0.04	

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

Table of values		
Pr	10755.43	lbf
Mrx	10545.97	lbfft
Mry	243.72	lbfft
Pc	84417.23	lbf
Mcx	33390.03	lbfft
Mcy	33390.03	lbfft
Res. factor compression	0.90	
Res. factor flexure	0.90	

unity check = 0.06+0.32+0.01=0.39 (H1-1b)
Torsion is neglected since Tr < 0,20 Tc.

The member satisfies the check !

STEEL MEMBER CHECK

AISC 360-16 LRFD Check

Member 3069	HSS5X5X3/8	A500 grade B	LRFD-Ult (auto)	0.40

Material data		
Yield stress Fy	42.00	ksi
Tensile stress Fu	58.00	ksi
fabrication	rolled	

The critical check is on position 0.00 ft

Classification for Axial Compression
according to article B4 and Table B4.1a

Compression N	ratio	Lambda,r (Non-slender)
Webs	9.83	36.79
Internal flanges	9.83	36.79

Section is classified as non-slender section.

Classification for Flexure
according to article B4 and Table B4.1b

Bending Mx	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	9.83	63.60	149.80
Internal flanges	9.83	29.43	36.79

Section is classified as compact section.

Bending My	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	9.83	63.60	149.80
Internal flanges	9.83	29.43	36.79

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	-12318.61	lbf
Vux	-245.57	lbf
Vuy	9019.25	lbf
Mut	965.90	lbfft
Mux	-10332.80	lbfft
Muy	1804.74	lbfft

Buckling parameters	xx	yy	
type	sway	non-sway	
Slenderness	17.43	59.67	
Reduced slenderness	0.21	0.72	
Length	1.20	10.29	ft
Effective length factor, K	2.26	0.91	
Effective length, Lc	2.72	9.32	ft

Buckling check
according to article E3 and formula (E3-1)

Table of values		
Pn	208589.21	lbf
Pu	12318.61	lbf
Fcr	33.75	ksi
Resistance factor	0.90	
unity check	0.07	

Torsional buckling check
according to article E4 and formula (E4-1)

Table of values		
Pn	259070.97	lbf
Pu	12318.61	lbf
Fe	9319.49	ksi
Fez	9319.49	ksi
Resistance factor	0.90	
unity check	0.05	

LTB data		
Lb	10.29	ft
Cb	1.37	

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

Table of values		
Mn	37100.03	lbfft
Mu	-10332.80	lbfft
Resistance factor	0.90	
unity check	0.31	

Weak axis bending check
according to article F7 and formula (F7-1)

Table of values		
Mn	37100.03	lbfft
Mu	-1804.74	lbfft
Resistance factor	0.90	
unity check	0.05	

Shear stress check
according to article G5 and formula (G2-1)
in buckling field 1

Table of values		
a	10.29	ft
h	0.21	ft
tw	0.35	inch
kv	5.34	
Vn	44959.02	lbf
Vu	9019.25	lbf
Resistance factor	0.90	
unity check	0.22	

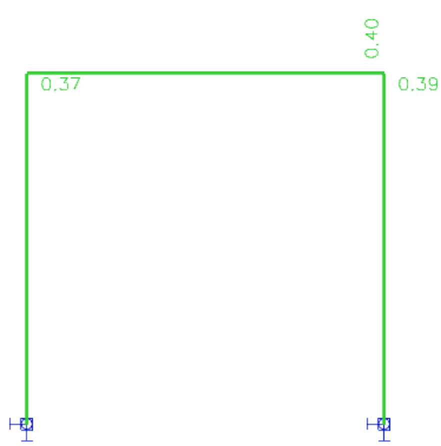
Combined stresses check
according to article H1.1 and formula (H1-1b)

Table of values		
Pr	12318.61	lbf
Mrx	-10332.80	lbfft
Mry	-1804.74	lbfft
Pc	187730.29	lbf
Mcx	33390.03	lbfft
Mcy	33390.03	lbfft
Res. factor compression	0.90	
Res. factor flexure	0.90	

unity check = 0.03+0.31+0.05=0.40 (H1-1b)
Torsion is neglected since Tr < 0,20 Tc.

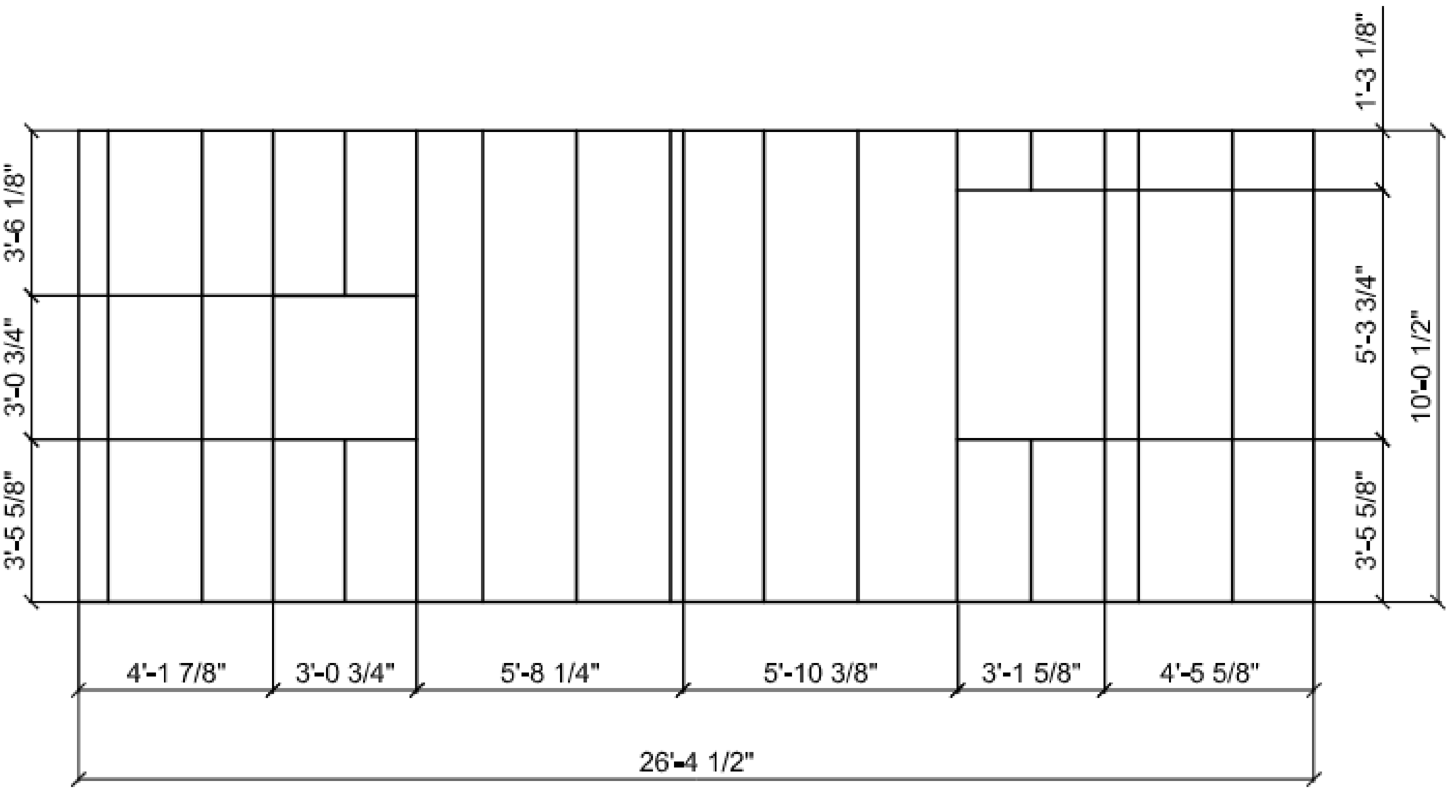
The member satisfies the check !

Unity check

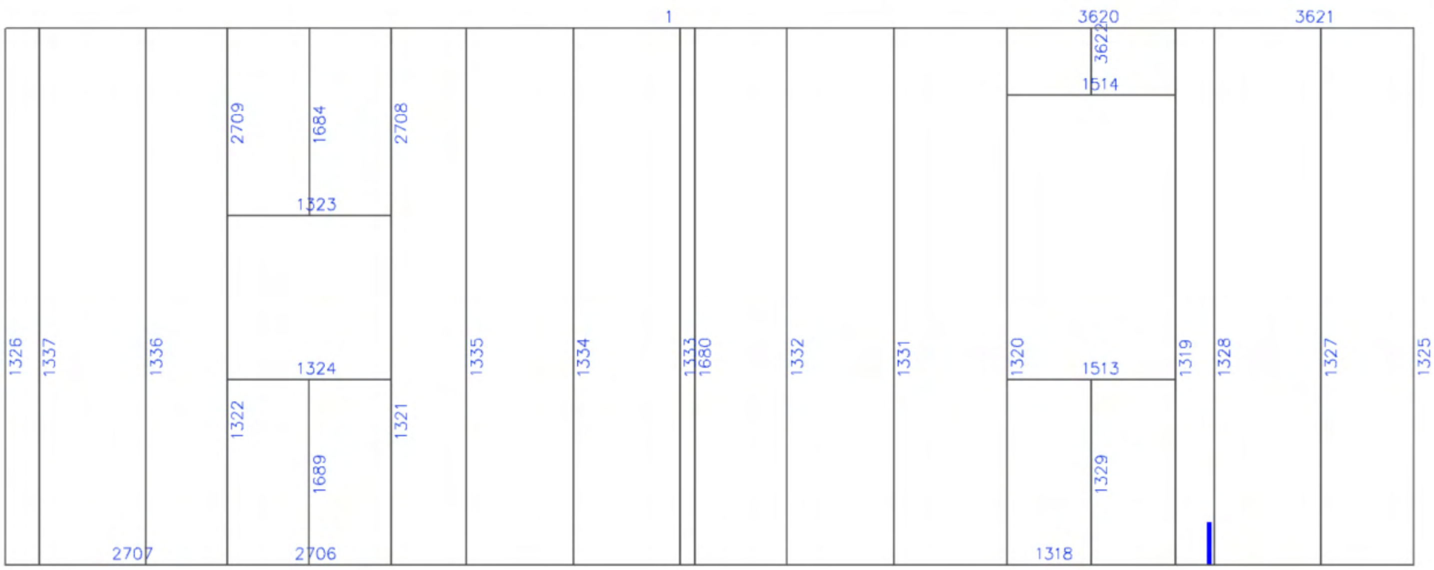


2.4.2.4 LEFT SIDE WALL DESIGN

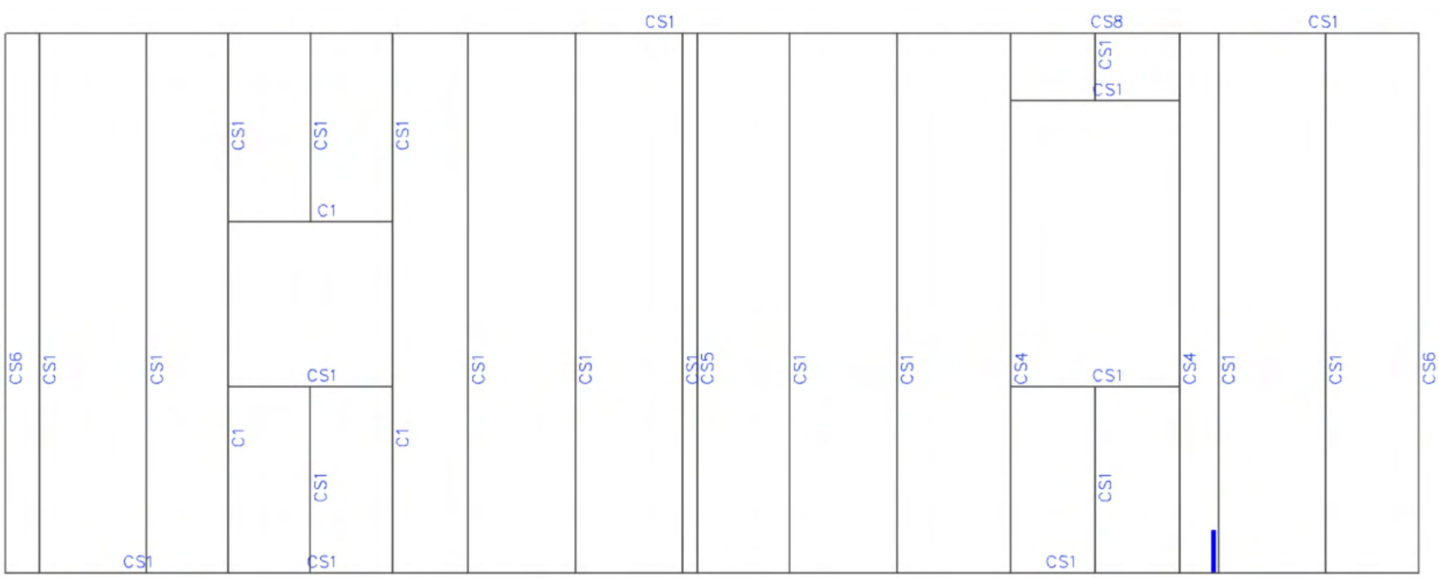
General scheme



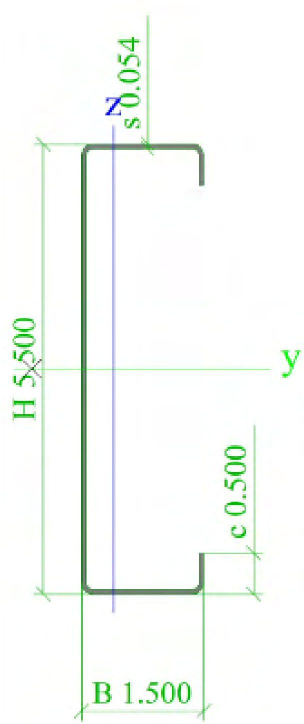
Member numbers

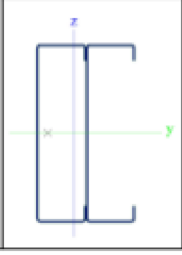


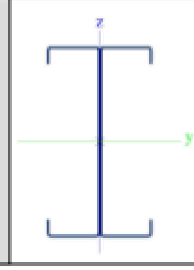
Cross-sections of elements.

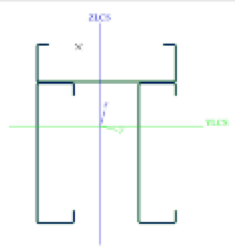


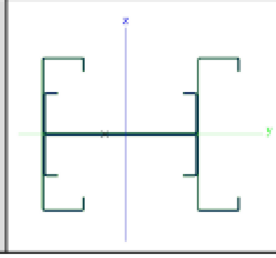
Cross-sections properties

CS1		
Type	Cold formed C section	
Detailed	5.500; 1.500; 0.054; 0.086; 0.500	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.491	
A_y [inch^2], A_z [inch^2]	0.164	0.303
A_L [inch^2/inch], A_D [inch^2/inch]	1.83e+01	1.83e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.392	2.750
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.122	0.139
i_y [inch], i_z [inch]	2.080	0.532
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.772	0.126
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.924	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.05e+00	9.05e+00
d_y [inch], d_z [inch]	-0.999	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.880
β_y [inch], β_z [inch]	0.000	5.904
Picture		

CS4		
Type	2x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.987	
A_y [inch^2], A_z [inch^2]	0.303	0.590
A_L [inch^2/inch], A_D [inch^2/inch]	2.16e+01	3.52e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	3.102	1.925
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.289	0.838
i_y [inch], i_z [inch]	2.084	0.921
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.560	0.451
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.864	0.741
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.32e+01	9.32e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.70e+01	3.70e+01
d_y [inch], d_z [inch]	-0.812	0.000
I_t [inch^4], I_w [inch^6]	1.030	2.525
$β_y$ [inch], $β_z$ [inch]	0.000	2.932
Picture		

CS5		
Type	2x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.987	
A_y [inch^2], A_z [inch^2]	0.322	0.589
A_L [inch^2/inch], A_D [inch^2/inch]	2.62e+01	2.62e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	3.458	1.925
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.289	0.436
i_y [inch], i_z [inch]	2.084	0.664
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.560	0.290
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.864	0.389
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.32e+01	9.32e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.95e+01	1.95e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.003	3.507
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

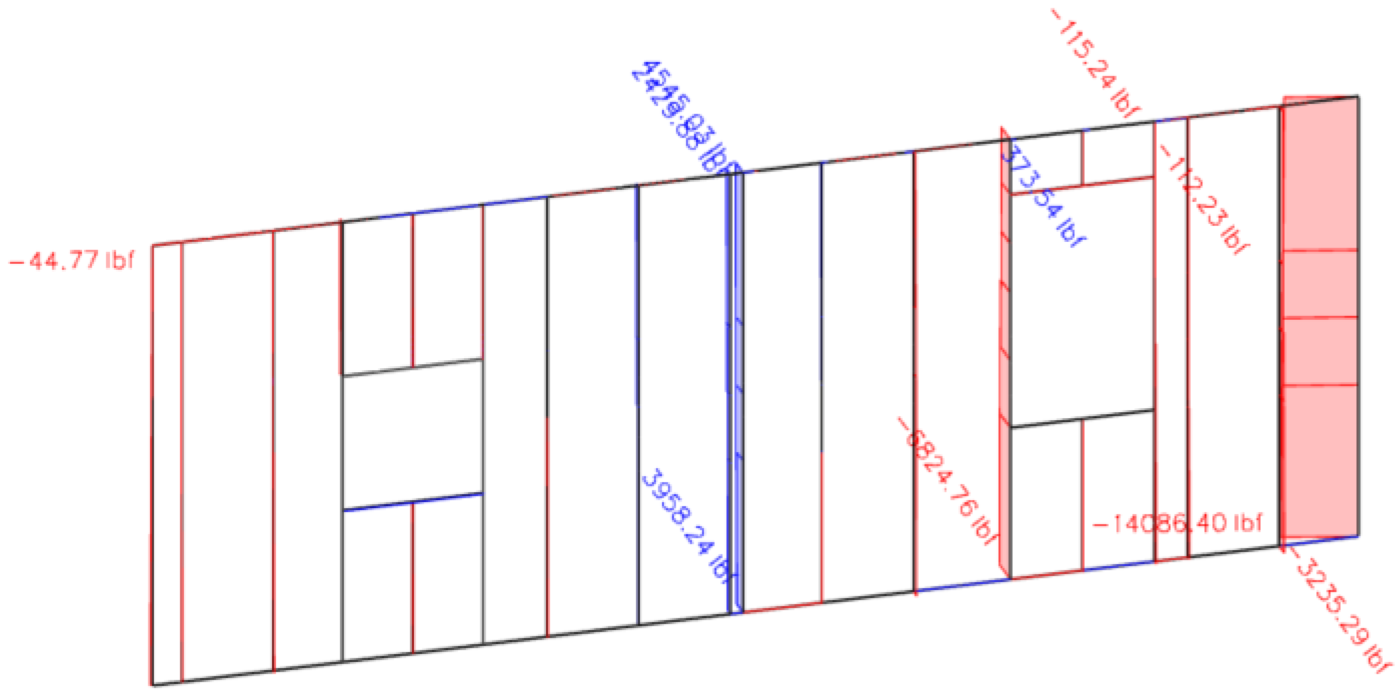
CS6		
Type	3x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch^2]	1.481	
A_y [inch^2], A_z [inch^2]	0.596	0.834
A_L [inch^2/inch], A_D [inch^2/inch]	5.01e+01	5.01e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	5.971	2.973
$I_{Y.LCS}$ [inch^4], $I_{Z.LCS}$ [inch^4]	7.684	6.418
$I_{YZ.LCS}$ [inch^4]	0.368	
α [deg]	-15.10	
I_y [inch^4], I_z [inch^4]	7.783	6.319
i_y [inch], i_z [inch]	2.292	2.066
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.812	1.642
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.878	2.808
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.44e+02	1.44e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.40e+02	1.40e+02
d_y [inch], d_z [inch]	-1.628	2.757
I_t [inch^4], I_w [inch^6]	0.002	41.712
β_y [inch], β_z [inch]	-6.538	3.765
Picture		

CS8		
Type	4x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch^2]	1.975	
A_y [inch^2], A_z [inch^2]	0.768	0.979
A_L [inch^2/inch], A_D [inch^2/inch]	5.16e+01	5.27e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	6.321	0.604
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.725	12.252
i_y [inch], i_z [inch]	1.547	2.491
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.718	3.013
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.253	4.606
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.13e+02	1.13e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.30e+02	2.30e+02
d_y [inch], d_z [inch]	-0.749	0.000
I_t [inch^4], I_w [inch^6]	0.005	37.816
β_y [inch], β_z [inch]	0.000	1.807
Picture		

Explanations of symbols	
A	Area
A_y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A_z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
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
i_z	Radius of gyration about the principal z-axis

Explanations of symbols	
$W_{el.y}$	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 - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Maximum force diagram
Axial force diagram N,
LRFD-Ult (auto)41 (1.2xDL1+1.2xDL2+1.2xDL3+0.5xLr+1xWx-(-0.18)), lbf.



Shear force diagram Vz ,
LRFD-Ult (auto)74 (0.9xDL1+0.9xDL2+0.9xDL3+1xWy-(-0.18)), lbf.

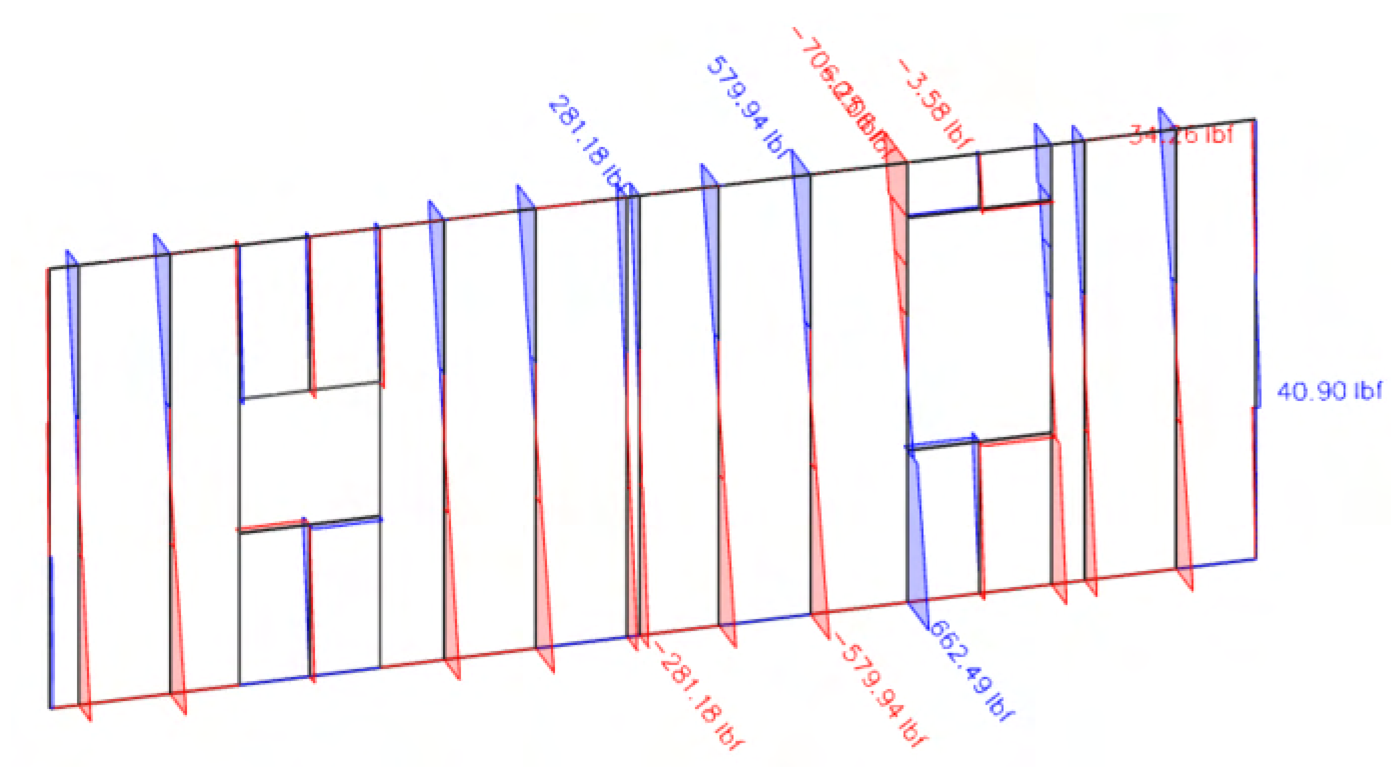
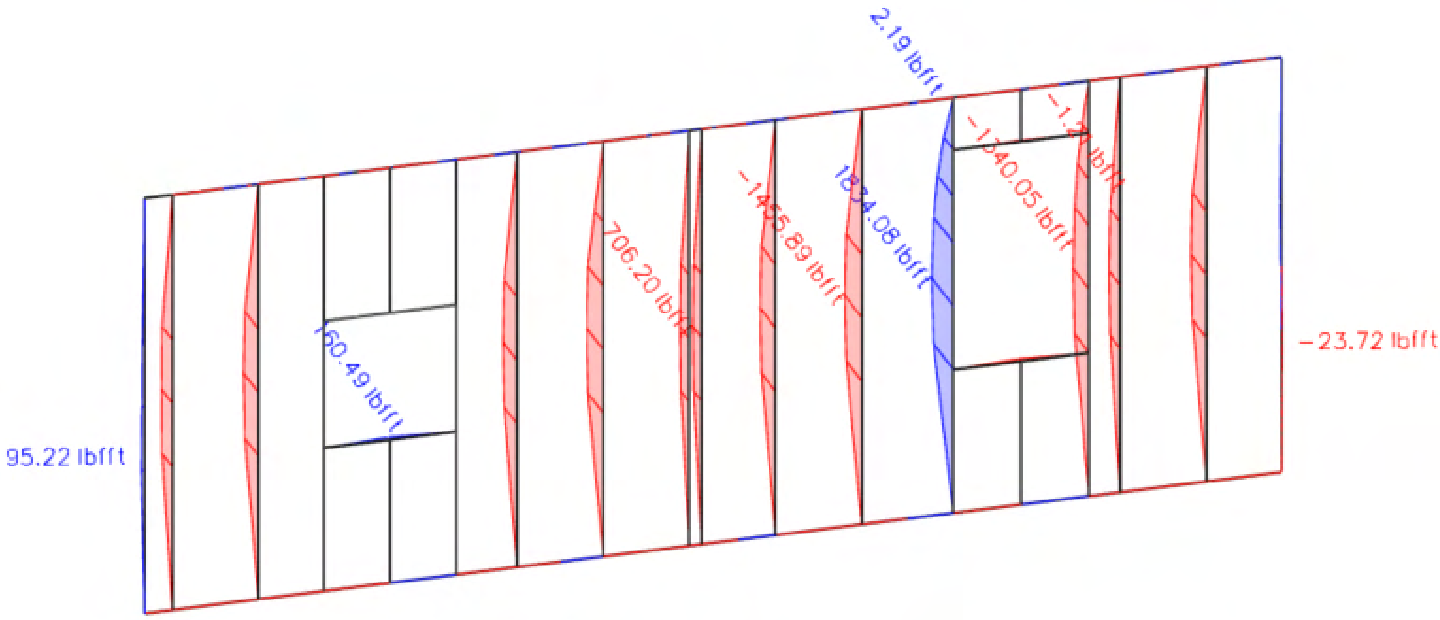
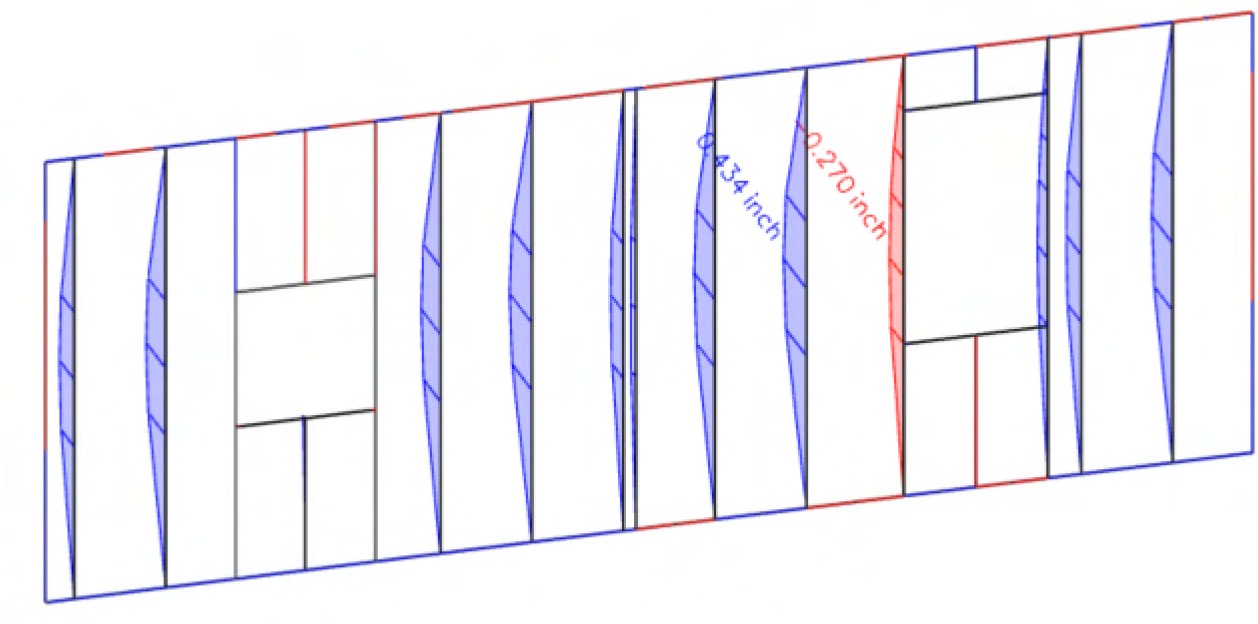


Diagram of moment My,
LRFD-Ult (auto)74 (0.9xDL1+0.9xDL2+0.9xDL3+1xWy-(-0.18)), lbf*ft.



Displacement of elements

Value: Uy - Wy, (inch) .



The maximum deflection is 0.434" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 10'= 10'*12"=120"/240 =0.5"

0.434" < 0.50" **<u>Deflection is OK!</u>**

STEEL MEMBER 1331 CHECK

AISI S100-16 LRFD Check

Member 1331	Cold formed C section (5.500; 1.500; 0.054; 0.086; 0.500)	A913 grade 50	LRFD-Ult (auto)	0.72

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

The critical check is on position 5.00 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	-1359.50	lbf
Vux	-0.02	lbf
Vuy	-2.78	lbf
Mut	-0.00	lbfft
Mux	-1455.89	lbfft
Muy	0.20	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 [inch⁴]	ds [inch]
1	0.361	47.939 41.334	0.86	0.481	282.818	0.412	1.000	0.336 -	- -	-	- -	-
3	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
5	5.221	47.939 -47.717	1.00	23.880	66.973	0.846	0.875	- 4.566	1.143 2.283	-	- -	-
7	1.221	-49.778 -49.778	-	-	-	-	-	- -	- -	-	- -	-
9	0.360	-41.112 -47.717	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	0.768	$inch^3$
Mnxo	3199.68	lbfft
Resistance factor	0.90	
Unity check	0.51	-

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	0.361	45.824 39.496	0.86	0.481	282.894	0.402	1.000	0.361 -	- -	-	- -	-
3	1.221	47.799 47.799	1.00	3.202	164.223	0.540	1.000	1.221 -	0.610 0.610	31.53	0.000 0.000	0.361
5	5.221	45.824 -45.824	1.00	24.000	67.310	0.825	0.889	- 4.641	1.160 2.320	-	- -	-
7	1.221	-47.799 -47.799	-	-	-	-	-	- -	- -	-	- -	-
9	0.360	-39.496 -45.824	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Lltb	3 ft 0.799 in	ft
Sigma,ey	59.915	ksi
Kt	1.00	
Lt	3 ft 0.799 in	ft
Sigma,t	69.629	ksi
Cb	1.02	
Sfx	0.772	inch^3
Fcre	99.477	ksi
Fc	47.799	ksi
Scx	0.772	inch^3
Mnx	3073.75	lbfft
Resistance factor	0.90	
Unity check	0.53	-

Distortional Buckling Strength

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

Table of values		
Sfy	0.772	inch^3
My	3215.29	lbfft
L	1 ft 0.708 in	ft
Beta	1.01	
k,phi,fe	291.72	lbf
k,phi,we	273.76	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	inch^2
k,phi,wg	0.002	inch^2
Fd	67.860	ksi
Sf	0.772	inch^3
Mcrd	4363.79	lbfft
Lambda,d	0.86	
Mn	2785.74	lbfft
Resistance factor	0.90	
Unity check	0.58	-

Data		
Lm	3 ft 0.799 in	ft
Lcr	1 ft 0.708 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	-0.056	inch
Ksi,web	2.00	

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

...::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	0.361	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-
3	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
5	5.221	50.000 50.000	1.00	4.000	11.218	2.111	0.424	2.215 -	- -	-	- -	-
7	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
9	0.360	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.326	$inch^2$
Pno	16296.53	lbf
Resistance factor	0.85	
Unity check	0.10	-

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	10 1/8	3 1/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 1/8	3 1/8	ft
Slenderness	57.95	69.13	
Flexural Buckling stress Fcre	85.256	59.915	ksi

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

Table of values		
Sigma,ex	85.256	ksi
Sigma,ey	59.915	ksi
Kt	1.00	
Lt	3 1/8	ft
Sigma,t	69.629	ksi
Sigma,TF	53.538	ksi
Torsional (-Flexural) buckling stress Fcre	53.538	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	0.361	33.823 33.823	1.00	0.430	252.949	0.366	1.000	0.361 -	- -	-	- -	-
3	1.221	33.823 33.823	1.00	3.202	164.223	0.454	1.000	1.221 -	0.610 0.610	37.49	0.000 0.000	0.361
5	5.221	33.823 33.823	1.00	4.000	11.218	1.736	0.503	2.626 -	- -	-	- -	-
7	1.221	33.823 33.823	1.00	3.202	164.223	0.454	1.000	1.221 -	0.610 0.610	37.49	0.000 0.000	0.360
9	0.360	33.823 33.823	1.00	0.430	252.949	0.366	1.000	0.360 -	- -	-	- -	-

Table of values		
Fe	53.538	ksi
lambda, c	0.97	
Fn	33.823	ksi
Ae	0.351	inch ^{2}
Pn	11862.38	lbf
Resistance factor	0.85	
Unity check	0.13	-

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

Table of values		
Py	24535.65	lbf
L	1 ft 2.047 in	ft
k,phi,fe	205.02	lbf
k,phi,we	152.10	lbf
k,phi	0.00	lbf
k,phi,fg	0.006	inch ^{2}
k,phi,wg	0.007	inch ^{2}
Fd	27.271	ksi
Pcrd	13382.33	lbf
Lambda,d	1.35	
Pn	14090.87	lbf
Resistance factor	0.85	
Unity check	0.11	-

Data		
Lm	3 ft 0.799 in	ft
Lcr	1 ft 2.047 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	-0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	0.056	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	0.361	2.770 2.770	1.00	0.430	252.949	0.105	1.000	0.361 -	- -	-	- -	-
3	1.221	2.770 2.770	1.00	4.000	205.118	0.116	1.000	1.221 -	0.610 0.610	130.98	- 0.000	0.361
5	5.221	2.770 2.770	1.00	4.000	11.218	0.497	1.000	5.221 -	- -	-	- -	-
7	1.221	2.770 2.770	1.00	4.000	205.118	0.116	1.000	1.221 -	0.610 0.610	130.98	- 0.000	0.360
9	0.360	2.770 2.770	1.00	0.430	252.949	0.105	1.000	0.360 -	- -	-	- -	-

Table of values		
Mnx	2785.74	lbfft
Pn	11862.38	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.13+0.58+0.00 = 0.72 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER 1618 CHECK

Check of steel

Linear calculation, Extreme : Cross-section
Selection : 1680
Combinations : LRFD-Ult (auto)
Layer : Ex wall

AISI S100-16 LRFD Check

Member 1680	2x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.34

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

The critical check is on position 6.53 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	-8862.41	lbf
Vux	-9.13	lbf
Vuy	-0.00	lbf
Mut	-0.01	lbfft
Mux	0.00	lbfft
Muy	32.07	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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 0.916	0.02	7.855	287.214	0.417	1.000	- 1.446	0.485 0.961	-	- -	-
3	5.446	- -	-	-	-	-	-	- -	- -	-	- -	-
4	1.446	50.000 0.916	0.02	7.855	287.214	0.417	1.000	- 1.446	0.485 0.961	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	1.446	-0.916 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	1.446	-0.916 -50.000	-	-	-	-	-	- -	- -	-	- -	-
9	0.473	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.290	$inch^3$
Mnyo	1210.13	lbfft
Resistance factor	0.90	
Unity check	0.03	-

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

Table of values		
Sigma,ex	85.643	ksi
Kt	1.00	
Lt	3 ft 6.142 in	ft
Sigma,t	125.701	ksi
Cb	1.00	
Sfy	0.290	$inch^3$
Fcre	771.637	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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	5.446	50.000 50.000	1.00	4.000	41.242	1.101	0.727	3.958 -	- -	-	- -	-
4	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
9	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.842	$inch^2$
Pno	42118.80	lbf
Resistance factor	0.85	
Unity check	0.25	-

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	10 1/8	3 5/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 1/8	3 5/8	ft
Slenderness	57.82	63.44	
Flexural Buckling stress Fcre	85.643	71.126	ksi

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

Table of values		
Sigma,ex	85.643	ksi
Sigma,ey	71.126	ksi
Kt	1.00	
Lt	3 5/8	ft
Sigma,t	125.701	ksi
Sigma,TF	71.126	ksi
Torsional (-Flexural) buckling stress Fcre	71.126	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	0.473	37.255 37.255	1.00	0.430	146.934	0.504	1.000	0.473 -	- -	-	- -	-
2	1.446	37.255 37.255	1.00	4.000	146.251	0.505	1.000	1.446 -	- -	-	- -	-
3	5.446	37.255 37.255	1.00	4.000	41.242	0.950	0.809	4.404 -	- -	-	- -	-
4	1.446	37.255 37.255	1.00	4.000	146.251	0.505	1.000	1.446 -	- -	-	- -	-
5	0.473	37.255 37.255	1.00	0.430	146.934	0.504	1.000	0.473 -	- -	-	- -	-
6	0.473	37.255 37.255	1.00	0.430	146.934	0.504	1.000	0.473 -	- -	-	- -	-
7	1.446	37.255 37.255	1.00	4.000	146.251	0.505	1.000	1.446 -	- -	-	- -	-
8	1.446	37.255 37.255	1.00	4.000	146.251	0.505	1.000	1.446 -	- -	-	- -	-
9	0.473	37.255 37.255	1.00	0.430	146.934	0.504	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	71.126	ksi
lambda, c	0.84	
Fn	37.255	ksi
Ae	0.891	inch^2
Pn	33176.69	lbf
Resistance factor	0.85	
Unity check	0.31	-

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	0.473	8.976 8.976	1.00	0.430	146.934	0.247	1.000	0.473 -	- -	-	- -	-
2	1.446	8.976 8.976	1.00	4.000	146.251	0.248	1.000	1.446 -	- -	-	- -	-
3	5.446	8.976 8.976	1.00	4.000	41.242	0.467	1.000	5.446 -	- -	-	- -	-
4	1.446	8.976 8.976	1.00	4.000	146.251	0.248	1.000	1.446 -	- -	-	- -	-
5	0.473	8.976 8.976	1.00	0.430	146.934	0.247	1.000	0.473 -	- -	-	- -	-
6	0.473	8.976 8.976	1.00	0.430	146.934	0.247	1.000	0.473 -	- -	-	- -	-
7	1.446	8.976 8.976	1.00	4.000	146.251	0.248	1.000	1.446 -	- -	-	- -	-
8	1.446	8.976 8.976	1.00	4.000	146.251	0.248	1.000	1.446 -	- -	-	- -	-
9	0.473	8.976 8.976	1.00	0.430	146.934	0.247	1.000	0.473 -	- -	-	- -	-

Table of values		
Mny	1210.13	lbfft
PEy	70228.19	lbf
Alfa y	0.87	
Cmy	0.85	
Pn	33176.69	lbf
Pno	42118.80	lbf
Resistance factor compression	0.85	
Resistance factor bending y	0.90	

Unity check = 0.31+0.00+0.03 = 0.34 - (H1.2-1)
Unity check = 0.25+0.00+0.03 = 0.28 -

The member satisfies the check !

STEEL MEMBER 3620 CHECK

AISI S100-16 LRFD Check

Member 3620	4x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.29

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

The critical check is on position 3.15 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	299.44	lbf
Vux	-102.48	lbf
Vuy	-214.63	lbf
Mut	0.23	lbfft
Mux	-1786.70	lbfft
Muy	-61.36	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	0.473	31.421 25.563	0.81	0.501	171.216	0.428	1.000	0.473 -	- -	-	- -	-
2	1.446	31.756 31.756	1.00	4.000	585.004	0.233	1.000	- 1.446	0.723 0.723	-	- -	-
3	5.446	31.421 -36.031	1.15	28.079	289.506	0.329	1.000	- 5.446	1.313 2.723	-	- -	-
4	1.446	-36.365 -36.365	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-30.173 -36.031	-	-	-	-	-	- -	- -	-	- -	-
6	0.473	-18.790 -18.790	-	-	-	-	-	- -	- -	-	- -	-
7	1.446	-18.790 -36.700	-	-	-	-	-	- -	- -	-	- -	-
8	1.250	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-
9	1.446	-18.790 -36.700	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-18.790 -18.790	-	-	-	-	-	- -	- -	-	- -	-
11	0.473	-30.173 -36.031	-	-	-	-	-	- -	- -	-	- -	-

12	1.446	-36.365 -36.365	-	-	-	-	-	- -	- -	-	- -	-
13	1.446	31.756 31.756	1.00	4.000	585.004	0.233	1.000	- 1.446	0.723 0.723	-	- -	-
14	0.473	31.421 25.563	0.81	0.501	171.216	0.428	1.000	0.473 -	- -	-	- -	-
15	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
16	1.446	50.000 32.090	0.64	4.808	175.805	0.533	1.000	- 1.446	0.613 0.833	-	- -	-
17	1.250	32.090 32.090	1.00	4.000	195.711	0.405	1.000	- 1.250	0.625 0.625	-	- -	-
18	1.446	50.000 32.090	0.64	4.808	175.805	0.533	1.000	- 1.446	0.613 0.833	-	- -	-
19	0.473	50.000 50.000	1.00	0.431	147.393	0.582	1.000	0.473 -	- -	-	- -	-
20	1.250	32.090 32.090	1.00	4.000	195.711	0.405	1.000	- 1.250	0.625 0.625	-	- -	-
29	1.250	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-
34	0.054	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-
39	0.054	32.090 32.090	1.00	4.000	104869.215	0.017	1.000	0.054 -	- -	-	- -	-

Table of values		
Sxe	4.451	$inch^3$
Mnxo	18547.37	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	1.167 in	ft
Sigma,ey	503338.591	ksi
Kt	1.00	
Lt	1.167 in	ft
Sigma,t	439877.035	ksi
Cb	1.00	
Sfx	1.718	$inch^3$
Fcre	1644346.959	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	0.473	-27.047 -27.047	-	-	-	-	-	- -	- -	-	- -	-
2	1.446	-0.496 -27.047	-	-	-	-	-	- -	- -	-	- -	-

3	5.446	- -	-	8.000	82.484	-	1.000	- 5.446	1.815 3.631	-	- -	-
4	1.446	-0.496 -27.047	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-27.047 -27.047	-	-	-	-	-	- -	- -	-	- -	-
6	0.473	50.000 41.315	0.83	0.496	169.345	0.543	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	- 1.446	0.723 0.723	-	- -	-
8	1.250	50.000 27.047	0.54	5.112	250.098	0.447	1.000	- 1.250	0.508 0.742	-	- -	-
9	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-
11	0.473	27.047 27.047	1.00	0.430	146.934	0.429	1.000	0.473 -	- -	-	- -	-
12	1.446	27.047 0.496	0.02	7.855	1148.855	0.153	1.000	- 1.446	0.485 0.961	-	- -	-
13	1.446	27.047 0.496	0.02	7.855	1148.855	0.153	1.000	- 1.446	0.485 0.961	-	- -	-
14	0.473	27.047 27.047	1.00	0.431	147.393	0.428	1.000	0.473 -	- -	-	- -	-
15	0.473	50.000 41.315	0.83	0.496	169.345	0.543	1.000	0.473 -	- -	-	- -	-
16	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	- 1.446	0.723 0.723	-	- -	-
17	1.250	50.000 27.047	0.54	5.112	250.098	0.447	1.000	- 1.250	0.508 0.742	-	- -	-
18	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
19	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-
20	1.250	-27.047 -50.000	-	-	-	-	-	- -	- -	-	- -	-
29	1.250	-27.047 -50.000	-	-	-	-	-	- -	- -	-	- -	-
34	0.054	0.496 -0.496	1.00	24.000	629215.289	0.001	1.000	- 0.054	0.013 0.027	-	- -	-
39	0.054	0.496 -0.496	1.00	24.000	629215.289	0.001	1.000	- 0.054	0.014 0.027	-	- -	-

Table of values		
Sye	1.162	inch^3
Mnyo	4841.40	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	1305231.420	ksi
Kt	1.00	
Lt	1.167 in	ft
Sigma,t	439877.035	ksi
Cb	1.00	
Sfy	4.101	$inch^3$
Fcre	1104130.557	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
2	0.156	4685.04
3	0.000	0.00
4	0.156	4685.04
5	0.000	0.00
6	0.026	766.26
7	0.000	0.00
8	0.068	2025.00
9	0.000	0.00
10	0.026	766.26
11	0.000	0.00
12	0.156	4685.04
13	0.156	4685.04
14	0.000	0.00
15	0.026	766.26
16	0.000	0.00
17	0.067	2025.00
18	0.000	0.00
19	0.026	766.26
20	0.068	2025.00
29	0.067	2025.00
34	0.003	87.48
39	0.003	87.48

Table of values		
Vn,y	30080.16	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	18547.37	lbfft
Vny	30080.16	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.01+0.00) = 0.11

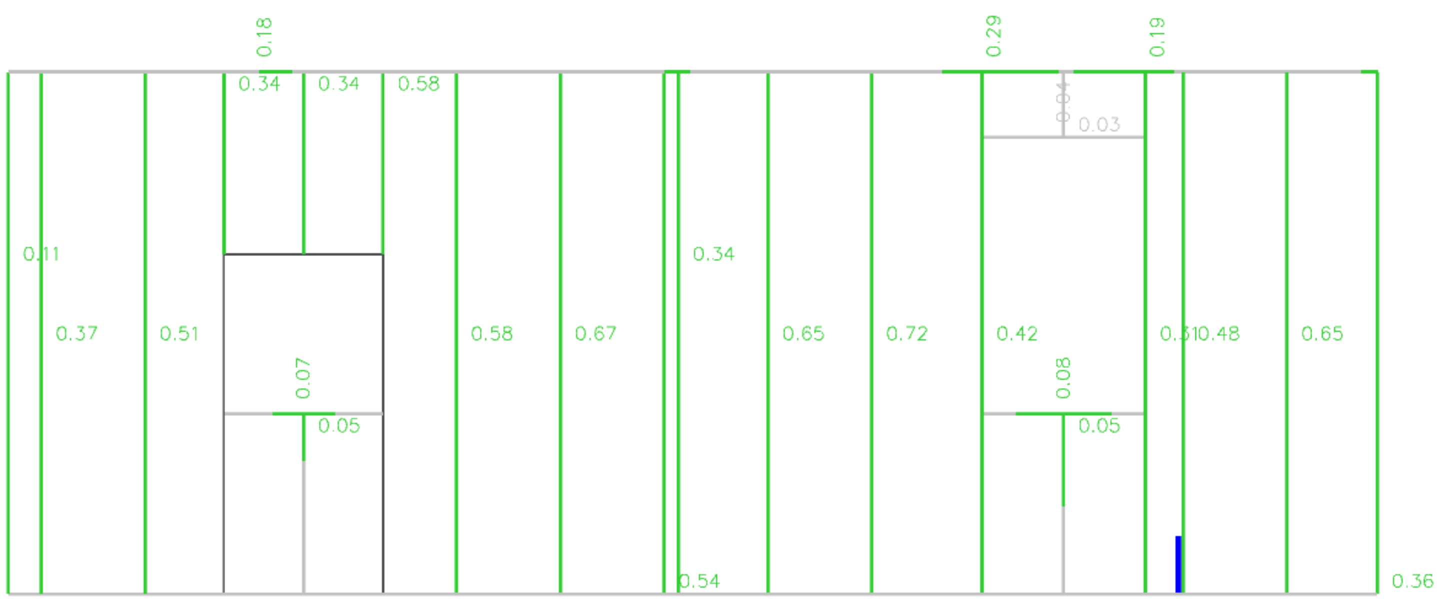
Combined Tensile Axial Load and Bending
According to article H1.1 and formulas (H1.1-1), (H1.1-2)

Table of values		
Sftx	1.718	$inch^3$
Sfty	3.013	$inch^3$
Mnxt	7159.00	lbfft
Mnyt	12554.48	lbfft
Mnx	18547.37	lbfft
Mny	4841.40	lbfft
Tn	98737.48	lbf
Resistance factor tension	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.28+0.01+0.00 = 0.29 - (H1.1-1)
Unity check = 0.11+0.01-0.00 = 0.12 - (H1.1-2)

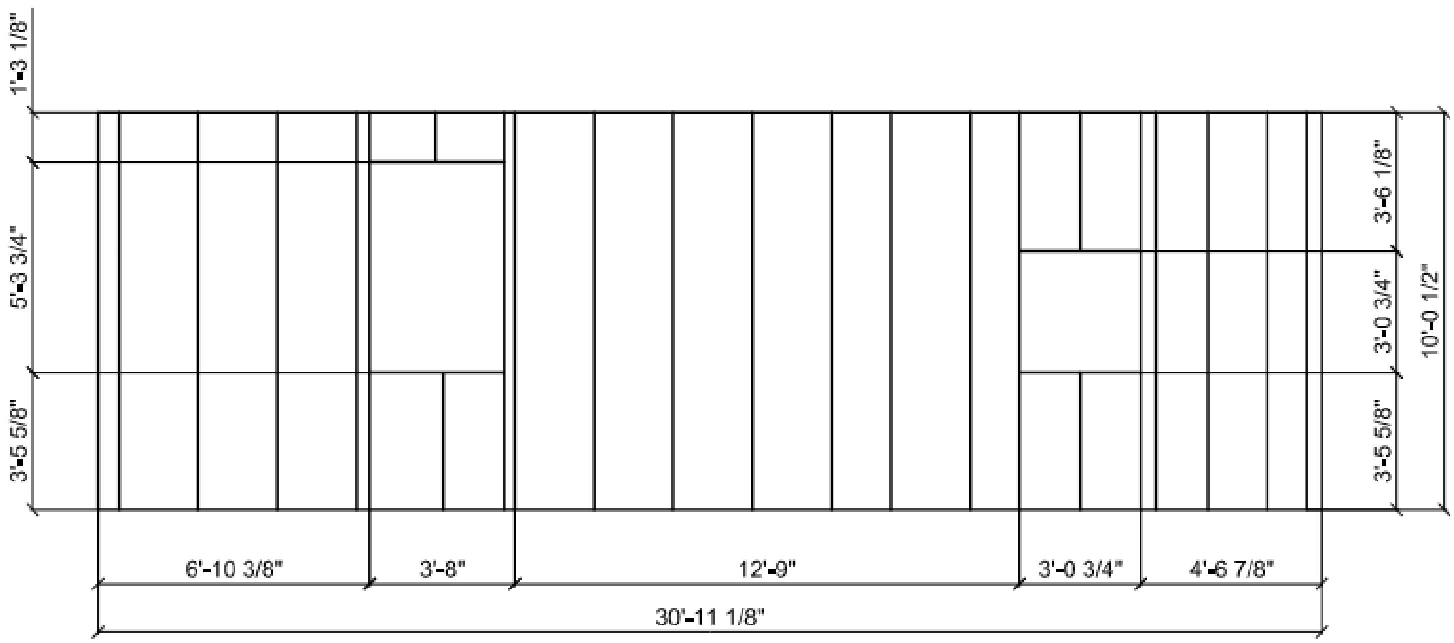
The member satisfies the check !

Unity check

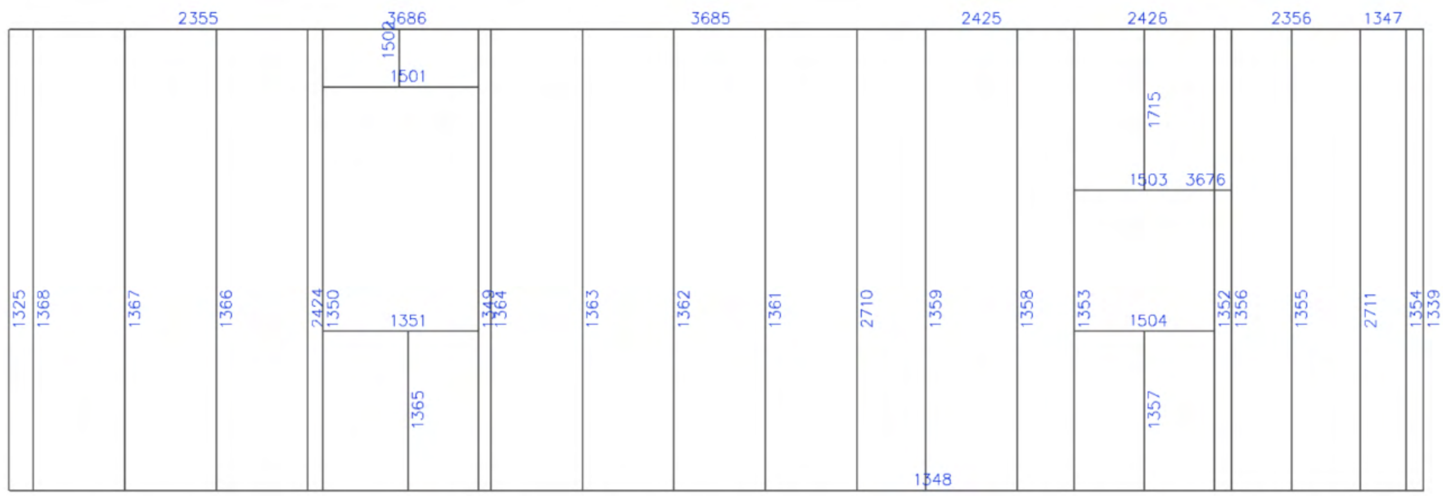


2.4.2.5 FRONT WALL DESIGN

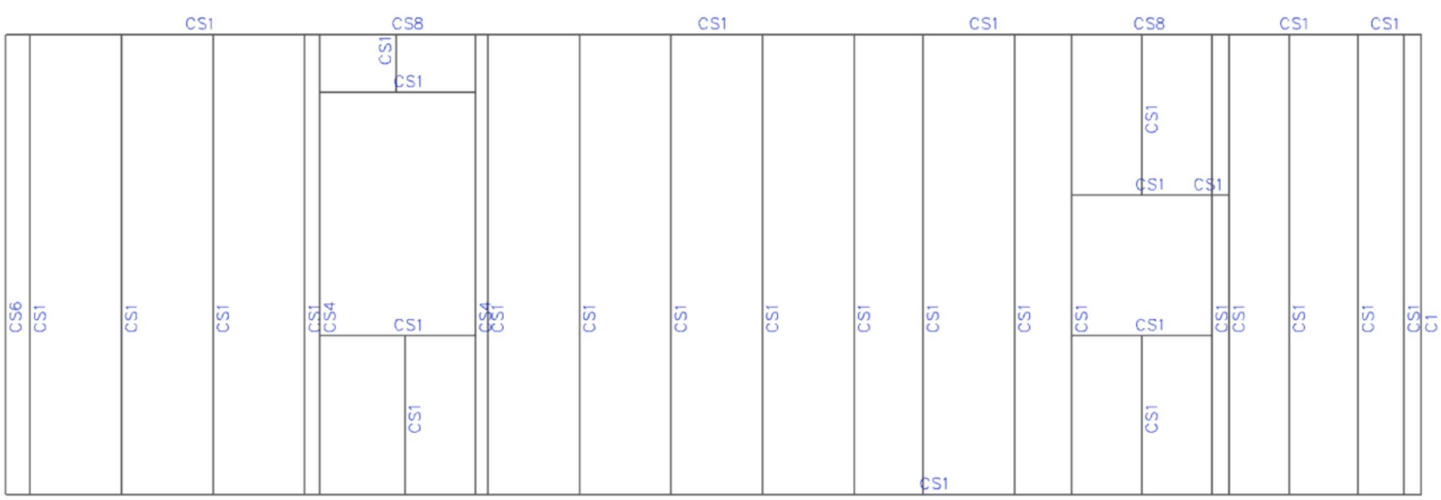
General scheme



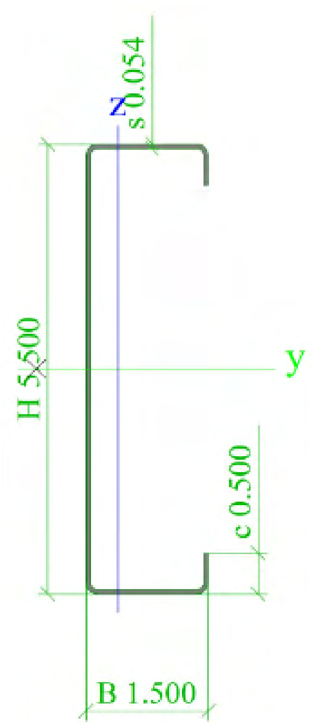
Member numbers

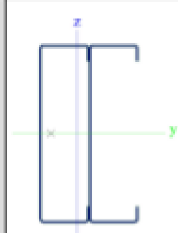


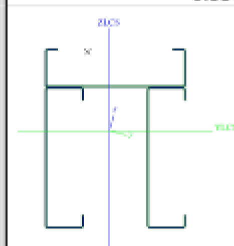
Cross-sections of elements.

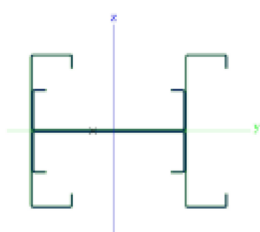


Cross-sections properties

CS1		
Type	Cold formed C section	
Detailed	5.500; 1.500; 0.054; 0.086; 0.500	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.491	
A_y [inch^2], A_z [inch^2]	0.164	0.303
A_L [inch^2/inch], A_D [inch^2/inch]	1.83e+01	1.83e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.392	2.750
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.122	0.139
i_y [inch], i_z [inch]	2.080	0.532
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.772	0.126
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.924	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.05e+00	9.05e+00
d_y [inch], d_z [inch]	-0.999	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.880
$β_y$ [inch], $β_z$ [inch]	0.000	5.904
Picture		

CS4		
Type	2x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch^2]	0.987	
A_y [inch^2], A_z [inch^2]	0.303	0.590
A_L [inch^2/inch], A_D [inch^2/inch]	2.16e+01	3.52e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	3.102	1.925
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.289	0.838
i_y [inch], i_z [inch]	2.084	0.921
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.560	0.451
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.864	0.741
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.32e+01	9.32e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.70e+01	3.70e+01
d_y [inch], d_z [inch]	-0.812	0.000
I_t [inch^4], I_w [inch^6]	1.030	2.525
β_y [inch], β_z [inch]	0.000	2.932
Picture		

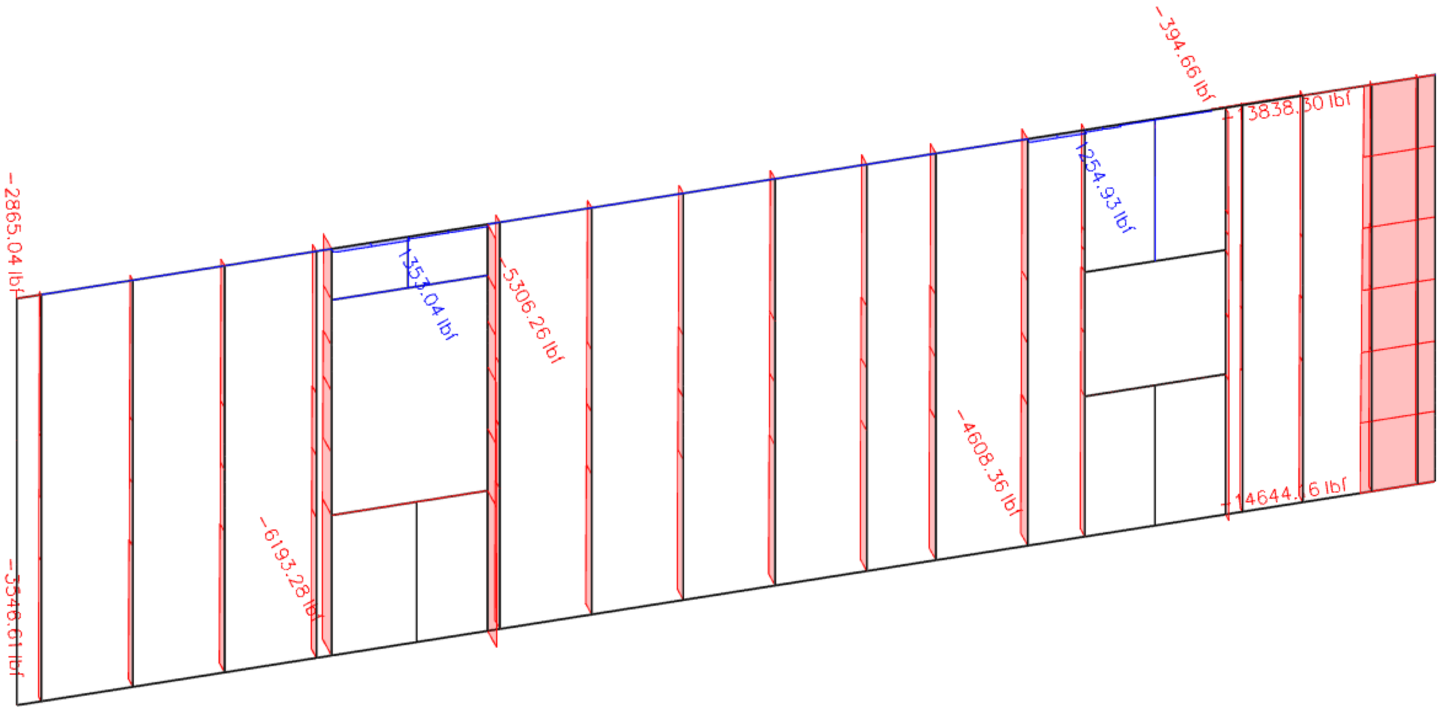
CS6		
Type	3x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch^2]	1.481	
A_y [inch^2], A_z [inch^2]	0.596	0.834
A_L [inch^2/inch], A_D [inch^2/inch]	5.01e+01	5.01e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	5.971	2.973
$I_{Y.LCS}$ [inch^4], $I_{Z.LCS}$ [inch^4]	7.684	6.418
$I_{YZ.LCS}$ [inch^4]	0.368	
α [deg]	-15.10	
I_y [inch^4], I_z [inch^4]	7.783	6.319
i_y [inch], i_z [inch]	2.292	2.066
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.812	1.642
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.878	2.808
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.44e+02	1.44e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.40e+02	1.40e+02
d_y [inch], d_z [inch]	-1.628	2.757
I_t [inch^4], I_w [inch^6]	0.002	41.712
β_y [inch], β_z [inch]	-6.538	3.765
Picture		

CS8		
Type	4x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch^2]	1.975	
A_y [inch^2], A_z [inch^2]	0.768	0.979
A_L [inch^2/inch], A_D [inch^2/inch]	5.16e+01	5.27e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	6.321	0.604
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	4.725	12.252
i_y [inch], i_z [inch]	1.547	2.491
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.718	3.013
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.253	4.606
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.13e+02	1.13e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.30e+02	2.30e+02
d_y [inch], d_z [inch]	-0.749	0.000
I_t [inch^4], I_w [inch^6]	0.005	37.816
β_y [inch], β_z [inch]	0.000	1.807
Picture		

Explanations of symbols	
A	Area
A_y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A_z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
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
i_z	Radius of gyration about the principal z-axis

Explanations of symbols	
$W_{el.y}$	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 - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Maximum force diagram
Axial force diagram N,
LRFD-Ult (auto)8 (1.2xDL1+1.2xDL2+1.2xDL3+0.5xL+1.6xLr, lbf.



Shear force diagram Vz ,
LRFD-Ult (auto)26 (1.2xDL1+1.2xDL2+1.2xDL3+1xWx+(-0.18)), lbf.

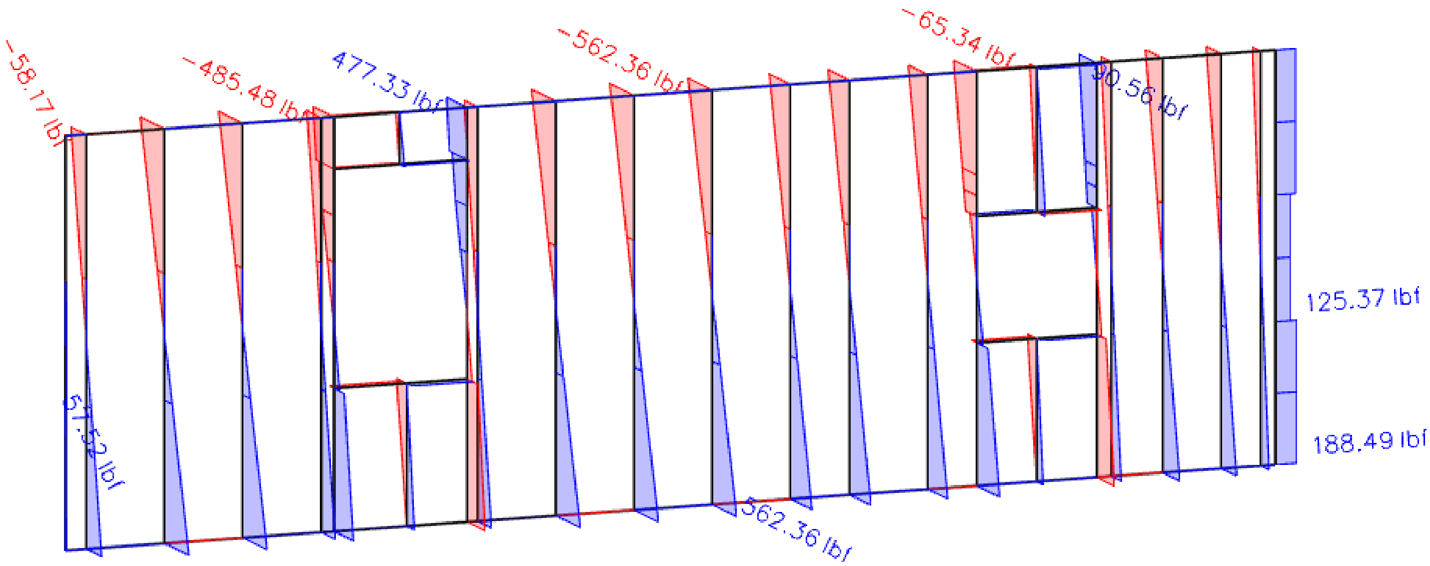
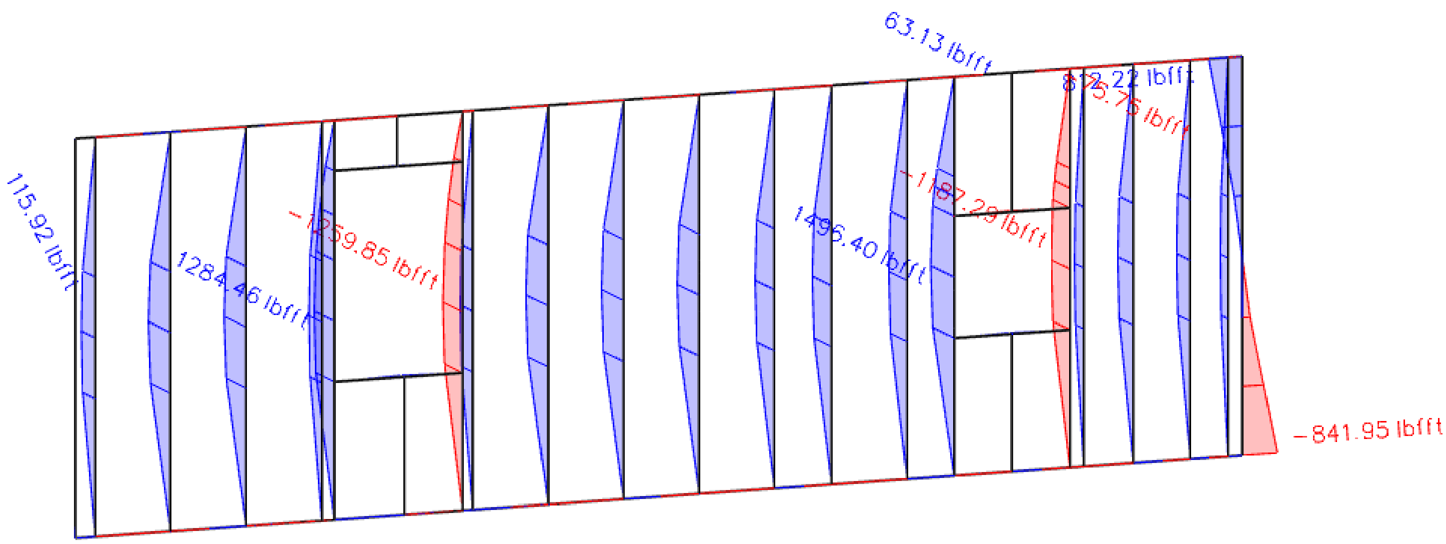
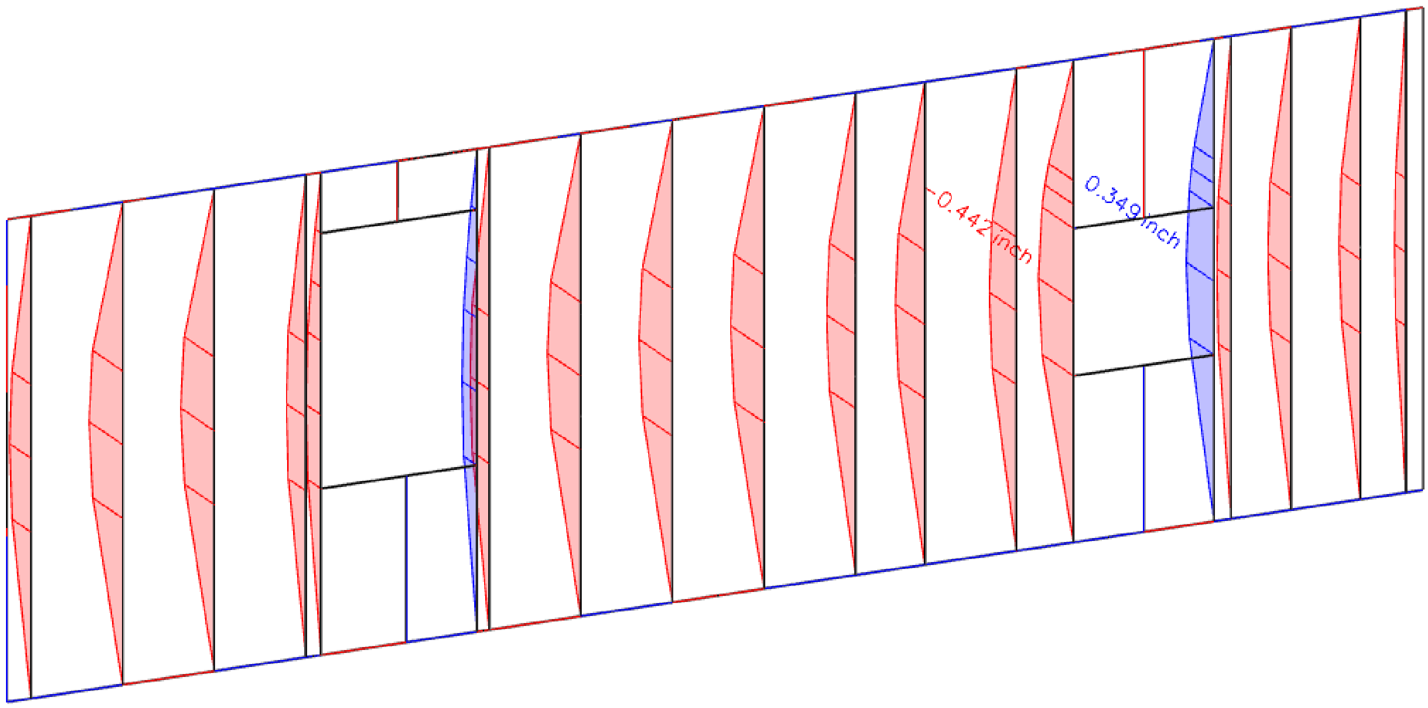


Diagram of moment My,
LRFD-Ult (auto)26 (1.2xDL1+1.2xDL2+1.2xDL3+1xWx+(-0.18)), lbf*ft.



Displacement of elements

Value: Ux - Wx, (inch) .



The maximum deflection is 0.442" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 10'= 10'*12"=120"/240 =0.5"

0.442" < 0.50" <u>Deflection is OK!</u>

STEEL MEMBER 1361 CHECK

AISI S100-16 LRFD Check

Member 1361	Cold formed C section (5.500; 1.500; 0.054; 0.086; 0.500)	A913 grade 50	LRFD-Ult (auto)	0.89

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

The critical check is on position 5.00 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	-3530.67	lbf
Vux	-1.55	lbf
Vuy	2.70	lbf
Mut	0.00	lbfft
Mux	1411.80	lbfft
Muy	-6.24	lbfft

Note: Mux and/or Muy include additional moments as defined in art. H1.2

...::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	0.361	-41.112 -47.717	-	-	-	-	-	- -	- -	-	- -	-
3	1.221	-49.778 -49.778	-	-	-	-	-	- -	- -	-	- -	-
5	5.221	47.939 -47.717	1.00	23.880	66.973	0.846	0.875	- 4.566	1.143 2.283	-	- -	-
7	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
9	0.360	47.939 41.334	0.86	0.481	282.818	0.412	1.000	0.336 -	- -	-	- -	-

Table of values		
Sxe	0.768	inch^3
Mnxo	3199.68	lbfft
Resistance factor	0.90	
Unity check	0.49	-

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 [inch⁴]	ds [inch]
1	0.361	-39.496 -45.824	-	-	-	-	-	- -	- -	-	- -	-
3	1.221	-47.799 -47.799	-	-	-	-	-	- -	- -	-	- -	-
5	5.221	45.824 -45.824	1.00	24.000	67.310	0.825	0.889	- 4.641	1.160 2.320	-	- -	-
7	1.221	47.799 47.799	1.00	3.202	164.223	0.540	1.000	1.221 -	0.610 0.610	31.53	0.000 0.000	0.360
9	0.360	45.824 39.496	0.86	0.481	282.894	0.402	1.000	0.360 -	- -	-	- -	-

Table of values		
Lltb	3 ft 0.799 in	ft
Sigma,ey	59.915	ksi
Kt	1.00	
Lt	3 ft 0.799 in	ft
Sigma,t	69.629	ksi
Cb	1.02	

Sfx	0.772	$inch^3$
Fcre	99.477	ksi
Fc	47.799	ksi
Scx	0.772	$inch^3$
Mnx	3073.75	lbfft
Resistance factor	0.90	
Unity check	0.51	-

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

Table of values		
Sfy	0.772	$inch^3$
My	3215.29	lbfft
L	1 ft 0.708 in	ft
Beta	1.01	
k,phi,fe	291.72	lbf
k,phi,we	273.76	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	$inch^2$
k,phi,wg	0.002	$inch^2$
Fd	67.860	ksi
Sf	0.772	$inch^3$
Mcrd	4363.79	lbfft
Lambda,d	0.86	
Mn	2785.74	lbfft
Resistance factor	0.90	
Unity check	0.56	-

Data		
Lm	3 ft 0.799 in	ft
Lcr	1 ft 0.708 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	-0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	0.056	inch
Ksi,web	2.00	

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

...::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	0.361	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	1.221	19.216 -44.161	2.30	82.349	4222.840	0.067	1.000	- 1.221	0.230 0.610	-	- -	-
5	5.221	25.055 25.055	1.00	4.000	11.218	1.494	0.571	2.979 -	- -	-	- -	-
7	1.221	19.216 -44.161	2.30	82.349	4222.840	0.067	1.000	- 1.221	0.230 0.610	-	- -	-
9	0.360	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.119	$inch^3$
Mnyo	496.24	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	85.256	ksi
Kt	1.00	
Lt	3 ft 0.799 in	ft
Sigma,t	69.629	ksi
Cs	1.00	
CTF	0.23	
Sfy	0.355	$inch^3$
j	3.076	inch
Fcre	3553.232	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	0.361	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-
3	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
5	5.221	50.000 50.000	1.00	4.000	11.218	2.111	0.424	2.215 -	- -	-	- -	-
7	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
9	0.360	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.326	inch^2
Pno	16296.53	lbf
Resistance factor	0.85	
Unity check	0.25	-

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	10 1/8	3 1/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 1/8	3 1/8	ft
Slenderness	57.95	69.13	
Flexural Buckling stress Fcre	85.256	59.915	ksi

Torsional (-Flexural) Buckling Strength

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

Table of values		
Sigma,ex	85.256	ksi
Sigma,ey	59.915	ksi
Kt	1.00	
Lt	3 1/8	ft
Sigma,t	69.629	ksi
Sigma,TF	53.538	ksi
Torsional (-Flexural) buckling stress Fcre	53.538	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	0.361	33.823 33.823	1.00	0.430	252.949	0.366	1.000	0.361 -	- -	-	- -	-
3	1.221	33.823 33.823	1.00	3.202	164.223	0.454	1.000	1.221 -	0.610 0.610	37.49	0.000 0.000	0.361
5	5.221	33.823 33.823	1.00	4.000	11.218	1.736	0.503	2.626 -	- -	-	- -	-
7	1.221	33.823 33.823	1.00	3.202	164.223	0.454	1.000	1.221 -	0.610 0.610	37.49	0.000 0.000	0.360
9	0.360	33.823 33.823	1.00	0.430	252.949	0.366	1.000	0.360 -	- -	-	- -	-

Table of values		
Fe	53.538	ksi
lambda, c	0.97	
Fn	33.823	ksi
Ae	0.351	inch ^{2}
Pn	11862.38	lbf
Resistance factor	0.85	
Unity check	0.35	-

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

Table of values		
Py	24535.65	lbf
L	1 ft 2.047 in	ft
k,phi,fe	205.02	lbf
k,phi,we	152.10	lbf
k,phi	0.00	lbf
k,phi,fg	0.006	inch ^{2}
k,phi,wg	0.007	inch ^{2}
Fd	27.271	ksi
Pcrd	13382.33	lbf
Lambda,d	1.35	
Pn	14090.87	lbf
Resistance factor	0.85	
Unity check	0.29	-

Data		
Lm	3 ft 0.799 in	ft
Lcr	1 ft 2.047 in	ft
h0	5.500	inch
Ixf	0.002	inch ^{4}
Iyf	0.018	inch ^{4}
Ixyf	-0.003	inch ^{4}
Cwf	0.000	inch ^{6}
Jf	0.000	inch ^{4}
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	inch ^{2}
y0f	0.056	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	0.361	7.195 7.195	1.00	0.430	252.949	0.169	1.000	0.361 -	- -	-	- -	-
3	1.221	7.195 7.195	1.00	4.000	205.118	0.187	1.000	1.221 -	0.610 0.610	81.27	- 0.000	0.361
5	5.221	7.195 7.195	1.00	4.000	11.218	0.801	0.906	4.728 -	- -	-	- -	-
7	1.221	7.195 7.195	1.00	4.000	205.118	0.187	1.000	1.221 -	0.610 0.610	81.27	- 0.000	0.360
9	0.360	7.195 7.195	1.00	0.430	252.949	0.169	1.000	0.360 -	- -	-	- -	-

Table of values		
Centerline shift ex	0.021	inch
Centerline shift ey	0.000	inch
Additional moment Mx	0.00	lbfft
Additional moment My	-6.15	lbfft
Mnx	2785.74	lbfft
Mny	496.24	lbfft
PEx	41836.32	lbf
PEy	29401.21	lbf
Alfa x	0.92	
Alfa y	0.88	
Cmx	0.85	
Cmy	0.85	
Pn	11862.38	lbf
Pno	16296.53	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.35+0.52+0.01 = 0.89 - (H1.2-1)
Unity check = 0.25+0.56+0.01 = 0.83 -

The member satisfies the check !

STEEL MEMBER 1364 CHECK

AISI S100-16 LRFD Check

Member 1364	Cold formed C section (5.500; 1.500; 0.054; 0.086; 0.500)	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 5.00 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	-3183.95	lbf
Vux	-1.43	lbf
Vuy	1.53	lbf
Mut	-0.07	lbfft
Mux	800.73	lbfft
Muy	-3.87	lbfft

Note: Mux and/or Muy include additional moments as defined in art. H1.2

...::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	0.361	-41.112 -47.717	-	-	-	-	-	- -	- -	-	- -	-
3	1.221	-49.778 -49.778	-	-	-	-	-	- -	- -	-	- -	-
5	5.221	47.939 -47.717	1.00	23.880	66.973	0.846	0.875	- 4.566	1.143 2.283	-	- -	-
7	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
9	0.360	47.939 41.334	0.86	0.481	282.818	0.412	1.000	0.336 -	- -	-	- -	-

Table of values		
Sxe	0.768	inch^3
Mnxo	3199.68	lbfft
Resistance factor	0.90	
Unity check	0.28	-

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	0.361	-39.496 -45.824	-	-	-	-	-	- -	- -	-	- -	-
3	1.221	-47.799 -47.799	-	-	-	-	-	- -	- -	-	- -	-
5	5.221	45.824 -45.824	1.00	24.000	67.310	0.825	0.889	- 4.641	1.160 2.320	-	- -	-
7	1.221	47.799 47.799	1.00	3.202	164.223	0.540	1.000	1.221 -	0.610 0.610	31.53	0.000 0.000	0.360
9	0.360	45.824 39.496	0.86	0.481	282.894	0.402	1.000	0.360 -	- -	-	- -	-

Table of values		
Lltb	3 ft 0.799 in	ft
Sigma,ey	59.915	ksi
Kt	1.00	
Lt	3 ft 0.799 in	ft
Sigma,t	69.629	ksi
Cb	1.02	
Sfx	0.772	$inch^3$
Fcre	99.477	ksi
Fc	47.799	ksi
Scx	0.772	$inch^3$
Mnx	3073.75	lbfft
Resistance factor	0.90	
Unity check	0.29	-

Distortional Buckling Strength

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

Table of values		
Sfy	0.772	$inch^3$
My	3215.29	lbfft
L	1 ft 0.708 in	ft
Beta	1.01	
k,phi,fe	291.72	lbf
k,phi,we	273.76	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	$inch^2$
k,phi,wg	0.002	$inch^2$
Fd	67.860	ksi
Sf	0.772	$inch^3$
Mcrd	4363.81	lbfft
Lambda,d	0.86	
Mn	2785.75	lbfft
Resistance factor	0.90	
Unity check	0.32	-

Data		
Lm	3 ft 0.799 in	ft
Lcr	1 ft 0.708 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	-0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	0.056	inch
Ksi,web	2.00	

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

...::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	0.361	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	1.221	19.216 -44.161	2.30	82.349	4222.840	0.067	1.000	- 1.221	0.230 0.610	-	- -	-
5	5.221	25.055 25.055	1.00	4.000	11.218	1.494	0.571	2.979 -	- -	-	- -	-
7	1.221	19.216 -44.161	2.30	82.349	4222.840	0.067	1.000	- 1.221	0.230 0.610	-	- -	-
9	0.360	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.119	$inch^3$
Mnyo	496.24	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	85.256	ksi
Kt	1.00	
Lt	3 ft 0.799 in	ft
Sigma,t	69.629	ksi
Cs	1.00	
CTF	0.23	
Sfy	0.355	$inch^3$
j	3.076	inch
Fcre	3505.917	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	0.361	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-
3	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
5	5.221	50.000 50.000	1.00	4.000	11.218	2.111	0.424	2.215 -	- -	-	- -	-
7	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
9	0.360	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.326	inch ^{2}
Pno	16296.53	lbf
Resistance factor	0.85	
Unity check	0.23	-

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	10 1/8	3 1/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 1/8	3 1/8	ft
Slenderness	57.95	69.13	
Flexural Buckling stress Fcre	85.256	59.915	ksi

Torsional (-Flexural) Buckling Strength

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

Table of values		
Sigma,ex	85.256	ksi
Sigma,ey	59.915	ksi
Kt	1.00	
Lt	3 1/8	ft
Sigma,t	69.629	ksi
Sigma,TF	53.538	ksi
Torsional (-Flexural) buckling stress Fcre	53.538	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	0.361	33.823 33.823	1.00	0.430	252.949	0.366	1.000	0.361 -	- -	-	- -	-
3	1.221	33.823 33.823	1.00	3.202	164.223	0.454	1.000	1.221 -	0.610 0.610	37.49	0.000 0.000	0.361
5	5.221	33.823 33.823	1.00	4.000	11.218	1.736	0.503	2.626 -	- -	-	- -	-
7	1.221	33.823 33.823	1.00	3.202	164.223	0.454	1.000	1.221 -	0.610 0.610	37.49	0.000 0.000	0.360
9	0.360	33.823 33.823	1.00	0.430	252.949	0.366	1.000	0.360 -	- -	-	- -	-

Table of values		
Fe	53.538	ksi
lambda, c	0.97	
Fn	33.823	ksi
Ae	0.351	inch ^{2}
Pn	11862.38	lbf
Resistance factor	0.85	
Unity check	0.32	-

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

Table of values		
Py	24535.65	lbf
L	1 ft 2.047 in	ft
k,phi,fe	205.02	lbf
k,phi,we	152.10	lbf
k,phi	0.00	lbf
k,phi,fg	0.006	inch ^{2}
k,phi,wg	0.007	inch ^{2}
Fd	27.271	ksi
Pcrd	13382.33	lbf
Lambda,d	1.35	
Pn	14090.87	lbf
Resistance factor	0.85	
Unity check	0.27	-

Data		
Lm	3 ft 0.799 in	ft
Lcr	1 ft 2.047 in	ft
h0	5.500	inch
Ixf	0.002	inch ^{4}
Iyf	0.018	inch ^{4}
Ixyf	-0.003	inch ^{4}
Cwf	0.000	inch ^{6}
Jf	0.000	inch ^{4}
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	inch ^{2}
y0f	0.056	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	0.361	6.488 6.488	1.00	0.430	252.949	0.160	1.000	0.361 -	- -	-	- -	-
3	1.221	6.488 6.488	1.00	4.000	205.118	0.178	1.000	1.221 -	0.610 0.610	85.58	- 0.000	0.361
5	5.221	6.488 6.488	1.00	4.000	11.218	0.761	0.935	4.879 -	- -	-	- -	-
7	1.221	6.488 6.488	1.00	4.000	205.118	0.178	1.000	1.221 -	0.610 0.610	85.58	- 0.000	0.360
9	0.360	6.488 6.488	1.00	0.430	252.949	0.160	1.000	0.360 -	- -	-	- -	-

Table of values		
Centerline shift ex	0.014	inch
Centerline shift ey	0.000	inch
Additional moment Mx	0.00	lbfft
Additional moment My	-3.78	lbfft
Mnx	2785.75	lbfft
Mny	496.24	lbfft
PEx	41836.32	lbf
PEy	29401.21	lbf
Alfa x	0.92	
Alfa y	0.89	
Cmx	0.85	
Cmy	0.85	
Pn	11862.38	lbf
Pno	16296.53	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.32+0.29+0.01 = 0.62 - (H1.2-1)
Unity check = 0.23+0.32+0.01 = 0.56 -

The member satisfies the check !

STEEL MEMBER 2426 CHECK

AISI S100-16 LRFD Check

Member 2426	4x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.25

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

The critical check is on position 2.31 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	873.14	lbf
Vux	1.49	lbf
Vuy	2260.13	lbf
Mut	0.00	lbfft
Mux	-1554.06	lbfft
Muy	1.57	lbfft

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

Table of values		
Tn	98737.48	lbf
Resistance factor	0.90	
Unity check	0.01	-

...::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	0.473	31.421 25.563	0.81	0.501	171.216	0.428	1.000	0.473 -	- -	-	- -	-
2	1.446	31.756 31.756	1.00	4.000	585.004	0.233	1.000	- 1.446	0.723 0.723	-	- -	-
3	5.446	31.421 -36.031	1.15	28.079	289.506	0.329	1.000	- 5.446	1.313 2.723	-	- -	-
4	1.446	-36.365 -36.365	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-30.173 -36.031	-	-	-	-	-	- -	- -	-	- -	-
6	0.473	-18.790 -18.790	-	-	-	-	-	- -	- -	-	- -	-
7	1.446	-18.790 -36.700	-	-	-	-	-	- -	- -	-	- -	-
8	1.250	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-

9	1.446	-18.790 -36.700	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-18.790 -18.790	-	-	-	-	-	- -	- -	-	- -	-
11	0.473	-30.173 -36.031	-	-	-	-	-	- -	- -	-	- -	-
12	1.446	-36.365 -36.365	-	-	-	-	-	- -	- -	-	- -	-
13	1.446	31.756 31.756	1.00	4.000	585.004	0.233	1.000	- 1.446	0.723 0.723	-	- -	-
14	0.473	31.421 25.563	0.81	0.501	171.216	0.428	1.000	0.473 -	- -	-	- -	-
15	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
16	1.446	50.000 32.090	0.64	4.808	175.805	0.533	1.000	- 1.446	0.613 0.833	-	- -	-
17	1.250	32.090 32.090	1.00	4.000	195.711	0.405	1.000	- 1.250	0.625 0.625	-	- -	-
18	1.446	50.000 32.090	0.64	4.808	175.805	0.533	1.000	- 1.446	0.613 0.833	-	- -	-
19	0.473	50.000 50.000	1.00	0.431	147.393	0.582	1.000	0.473 -	- -	-	- -	-
20	1.250	32.090 32.090	1.00	4.000	195.711	0.405	1.000	- 1.250	0.625 0.625	-	- -	-
29	1.250	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-
34	0.054	-36.700 -36.700	-	-	-	-	-	- -	- -	-	- -	-
39	0.054	32.090 32.090	1.00	4.000	104869.215	0.017	1.000	0.054 -	- -	-	- -	-

Table of values		
Sxe	4.451	$inch^3$
Mnxo	18547.37	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	9.025 in	ft
Sigma,ey	8410.797	ksi
Kt	1.00	
Lt	9.025 in	ft
Sigma,t	7353.537	ksi
Cb	1.78	
Sfx	1.718	$inch^3$
Fcre	48650.965	ksi

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

...::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	21810.440	ksi
Kt	1.00	
Lt	9.025 in	ft
Sigma,t	7353.537	ksi
Cb	1.00	
Sfy	3.013	$inch^3$
Fcre	25118.169	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
2	0.156	4685.04
3	0.000	0.00
4	0.156	4685.04
5	0.000	0.00
6	0.026	766.26
7	0.000	0.00
8	0.068	2025.00
9	0.000	0.00
10	0.026	766.26
11	0.000	0.00
12	0.156	4685.04
13	0.156	4685.04
14	0.000	0.00
15	0.026	766.26
16	0.000	0.00
17	0.067	2025.00
18	0.000	0.00
19	0.026	766.26
20	0.068	2025.00
29	0.067	2025.00
34	0.003	87.48
39	0.003	87.48

Table of values		
Vn,y	30080.16	lbf
Resistance factor	0.95	
Unity check	0.08	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	18547.37	lbfft
Vny	30080.16	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.01+0.01) = 0.12

Combined Tensile Axial Load and Bending

According to article H1.1 and formulas (H1.1-1), (H1.1-2)

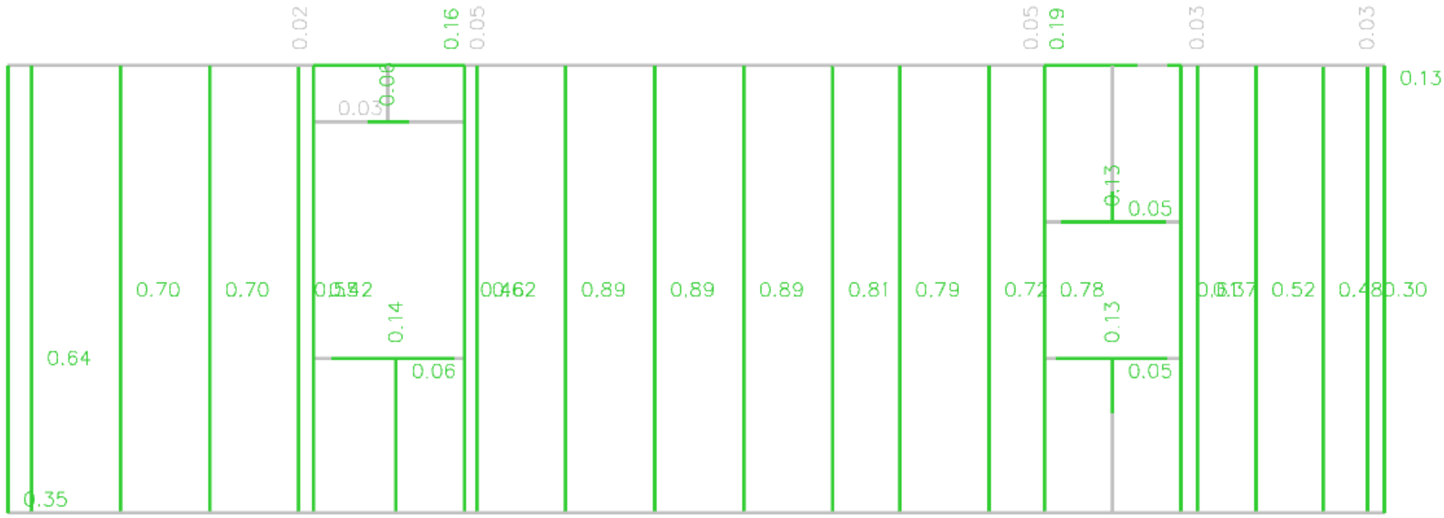
Table of values		
Sftx	1.718	$inch^3$
Sfty	4.101	$inch^3$
Mnxt	7159.00	lbfft
Mnyt	17088.16	lbfft
Mnx	18547.37	lbfft
Mny	4841.40	lbfft
Tn	98737.48	lbf
Resistance factor tension	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.24+0.00+0.01 = 0.25 - (H1.1-1)

Unity check = 0.09+0.00-0.01 = 0.08 - (H1.1-2)

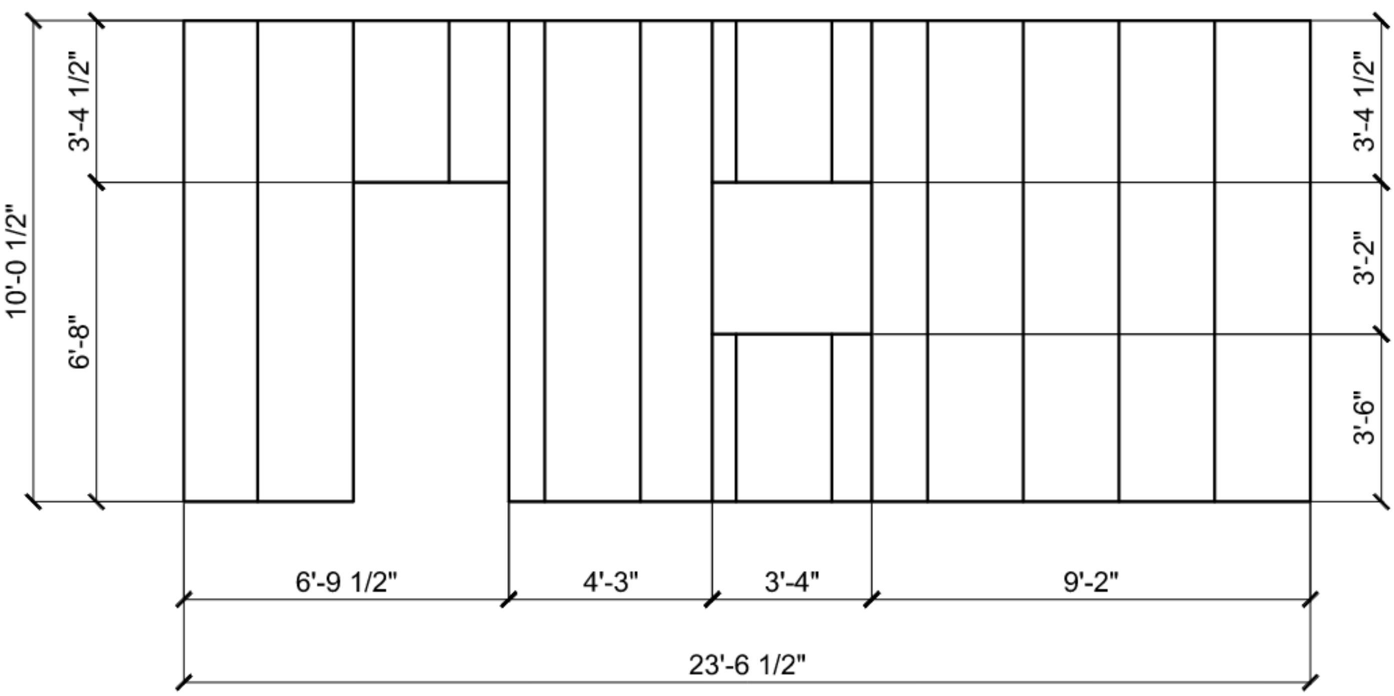
The member satisfies the check !

Unity check

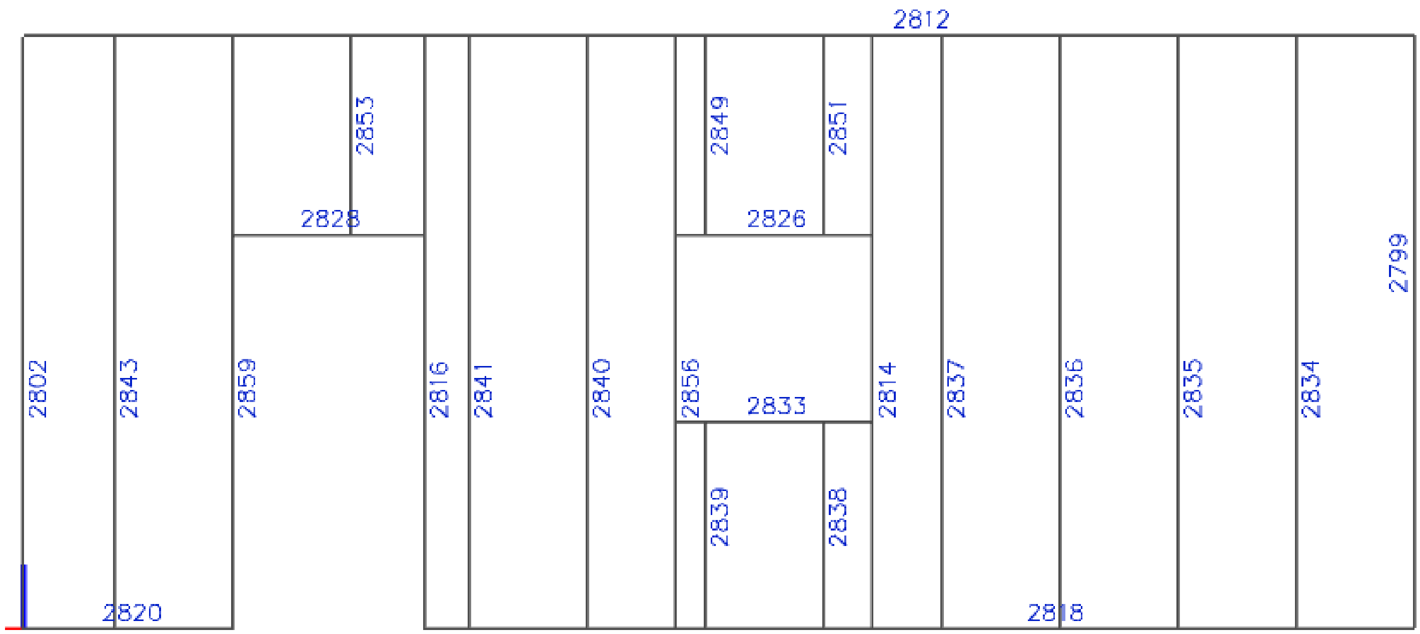


2.4.2.6 REAR GARAGE WALL DESIGN

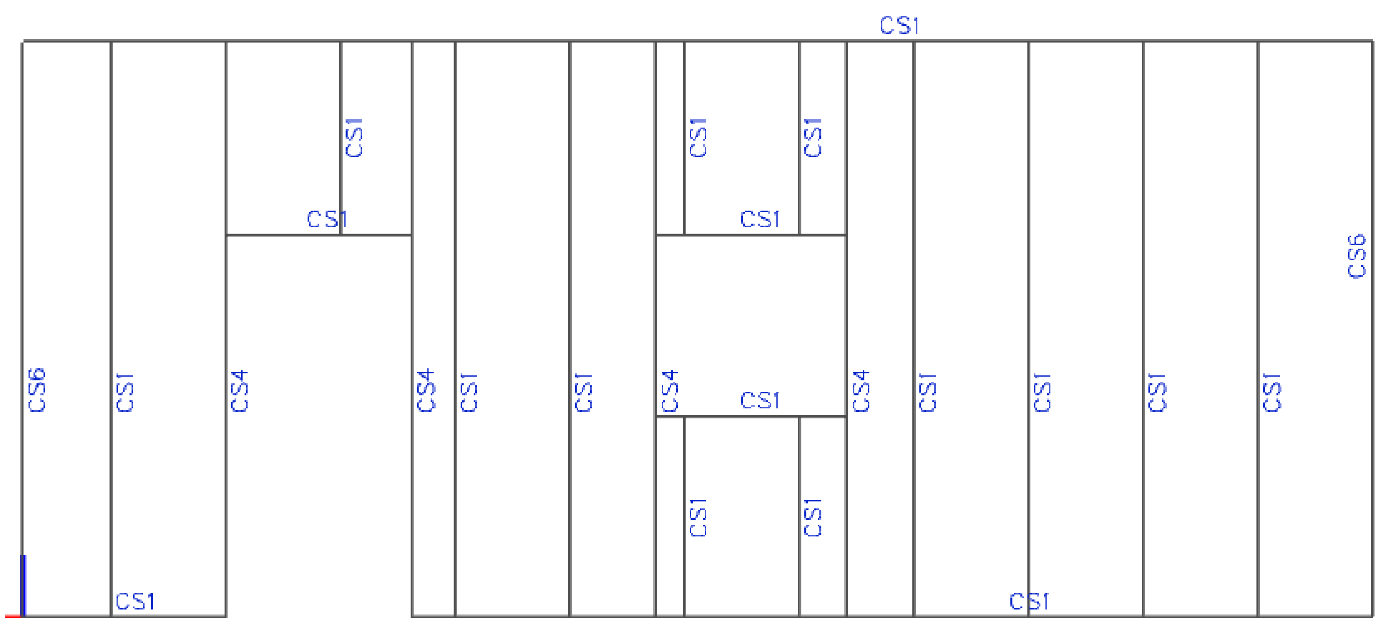
General scheme



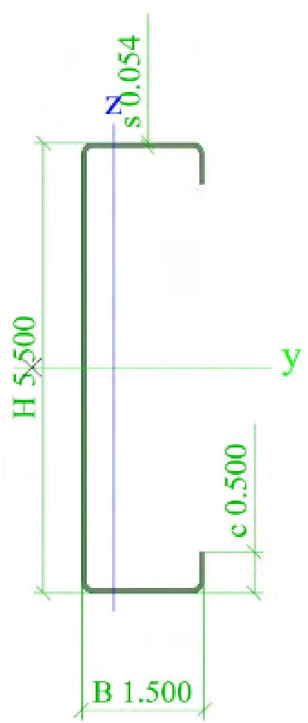
Member numbers

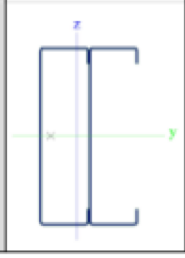


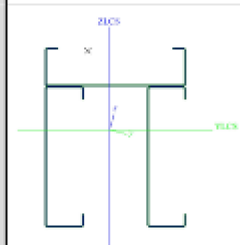
Cross-sections of element



Cross-sections properties

CS1		
Type	Cold formed C section	
Detailed	5.500; 1.500; 0.054; 0.086; 0.500	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.491	
A_y [inch^2], A_z [inch^2]	0.164	0.303
A_L [inch^2/inch], A_D [inch^2/inch]	1.83e+01	1.83e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.392	2.750
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.122	0.139
i_y [inch], i_z [inch]	2.080	0.532
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.772	0.126
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.924	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.05e+00	9.05e+00
d_y [inch], d_z [inch]	-0.999	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.880
$β_y$ [inch], $β_z$ [inch]	0.000	5.904
Picture		

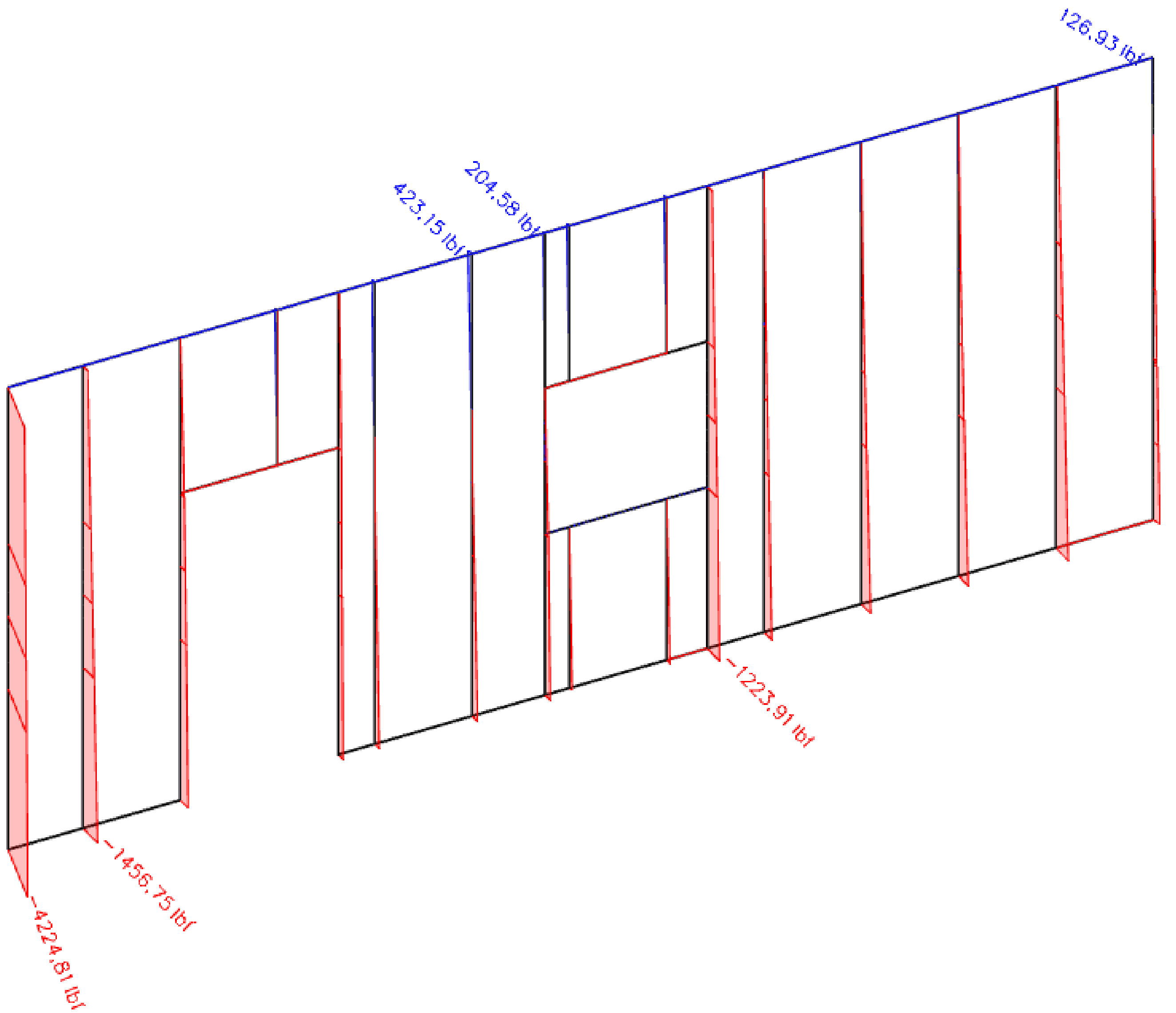
CS4		
Type	2x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [$inch^2$]	0.987	
A_y [$inch^2$], A_z [$inch^2$]	0.303	0.590
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	2.16e+01	3.52e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	3.102	1.925
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	4.289	0.838
i_y [inch], i_z [inch]	2.084	0.921
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	1.560	0.451
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	1.864	0.741
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.32e+01	9.32e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.70e+01	3.70e+01
d_y [inch], d_z [inch]	-0.812	0.000
I_t [$inch^4$], I_w [$inch^6$]	1.030	2.525
β_y [inch], β_z [inch]	0.000	2.932
Picture		

CS6		
Type	3x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [$inch^2$]	1.481	
A_y [$inch^2$], A_z [$inch^2$]	0.596	0.834
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	5.01e+01	5.01e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	5.971	2.973
$I_{Y.LCS}$ [$inch^4$], $I_{Z.LCS}$ [$inch^4$]	7.684	6.418
$I_{YZ.LCS}$ [$inch^4$]	0.368	
α [deg]	-15.10	
I_y [$inch^4$], I_z [$inch^4$]	7.783	6.319
i_y [inch], i_z [inch]	2.292	2.066
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	1.812	1.642
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	2.878	2.808
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.44e+02	1.44e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.40e+02	1.40e+02
d_y [inch], d_z [inch]	-1.628	2.757
I_t [$inch^4$], I_w [$inch^6$]	0.002	41.712
β_y [inch], β_z [inch]	-6.538	3.765
Picture		

Explanations of symbols	
A	Area
A_y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A_z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
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
i_z	Radius of gyration about the principal z-axis

Explanations of symbols	
$W_{el.y}$	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 - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Maximum force diagram
Axial force diagram N,
LRFD-Ult (auto)11 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + 0.5*Lr + 1.6*Wx+(-0.18)), lbf.



Shear force diagram Vz ,
LRFD-Ult (auto)29 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + Wx-(-0.18)), lbf.

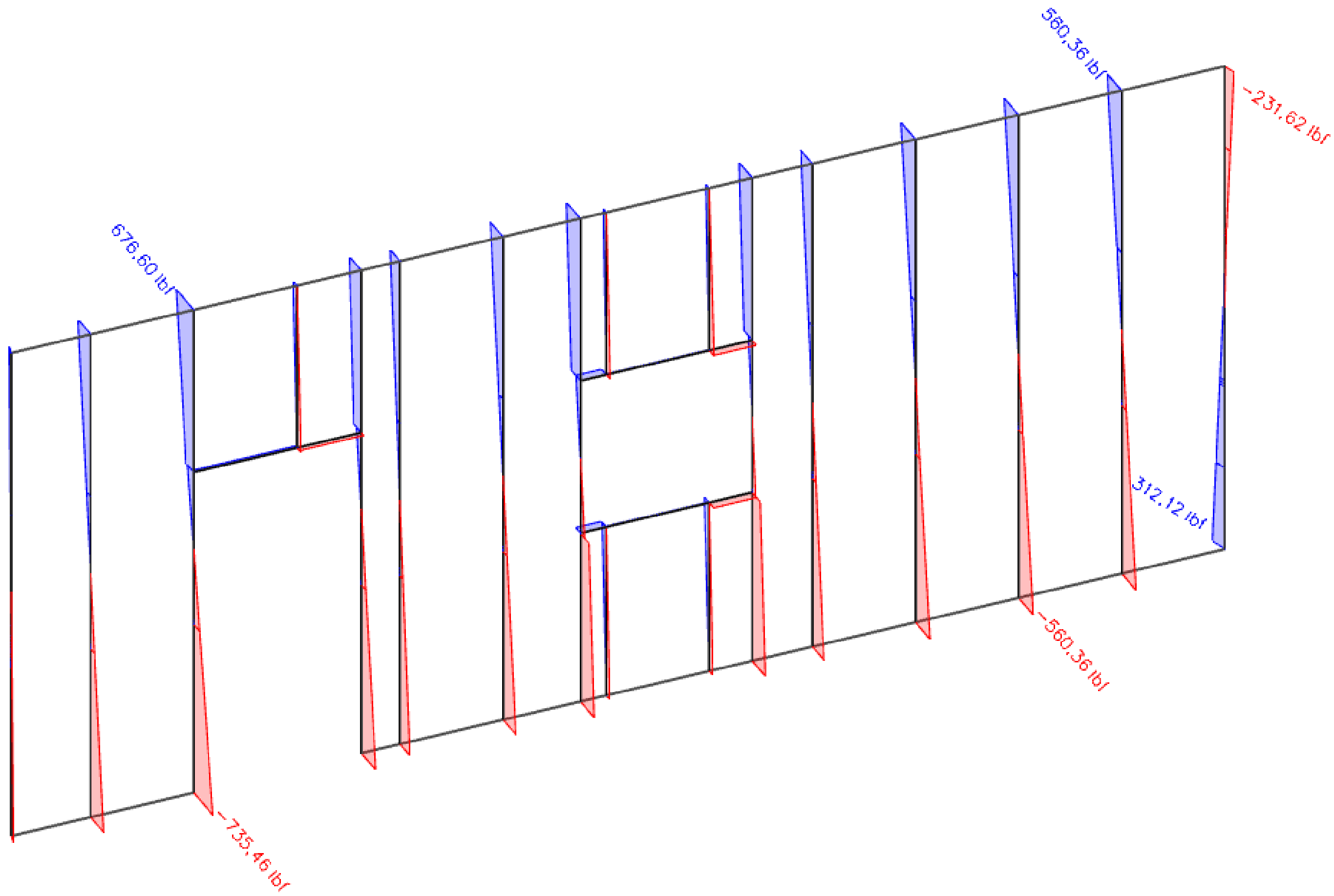
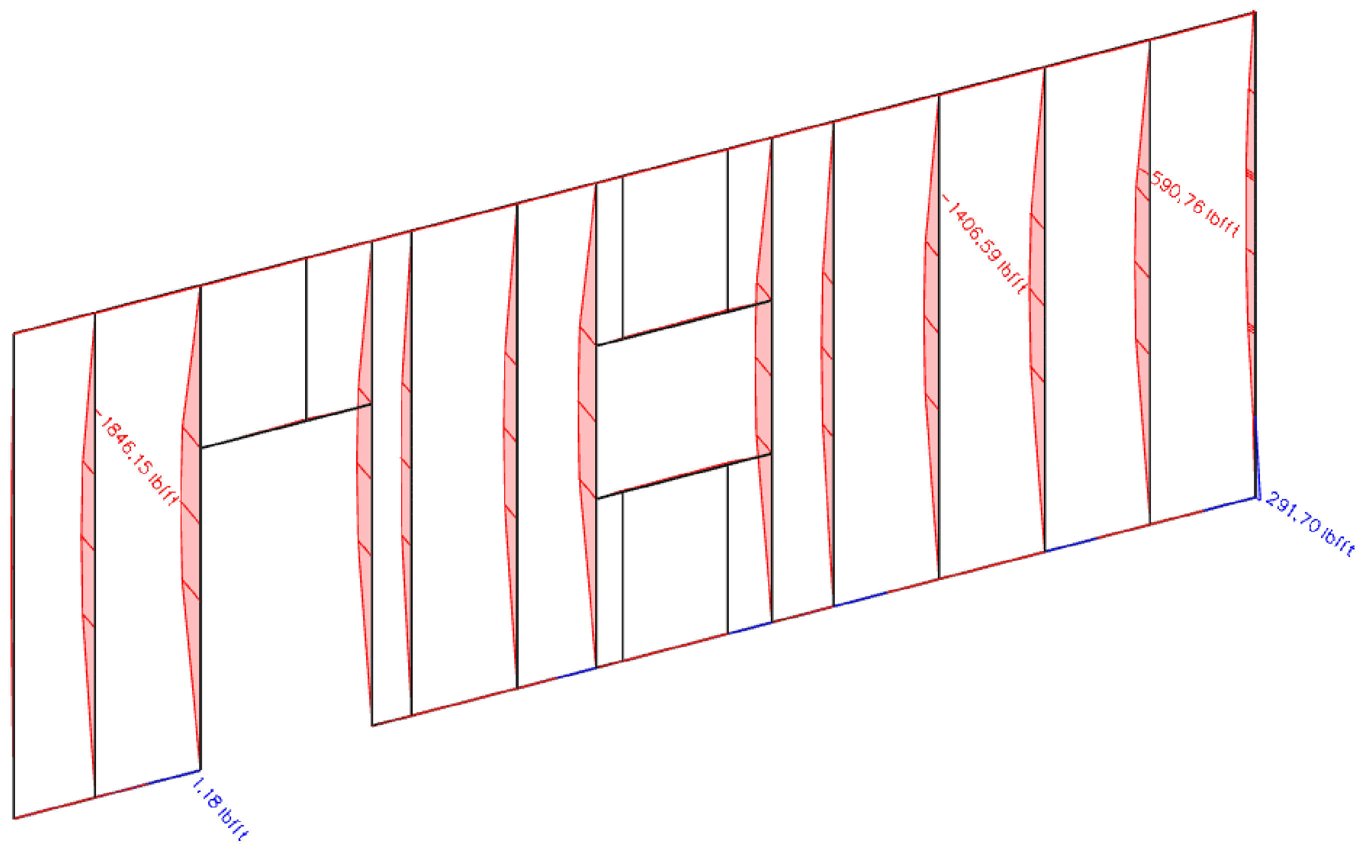
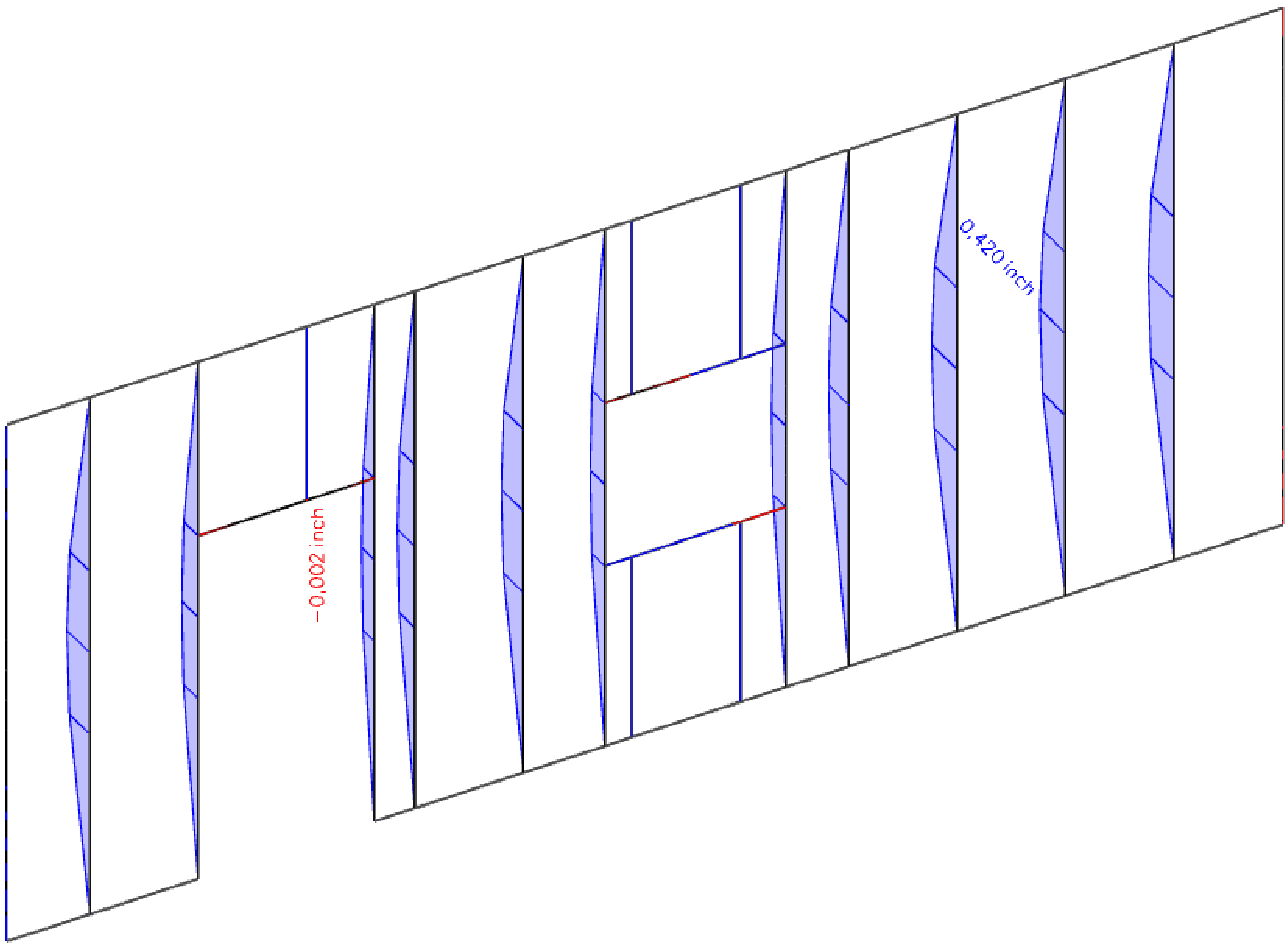


Diagram of moment My,
LRFD-Ult (auto)70 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + Wx-(-0.18)), lbf*ft.



Displacement of elements
Value: Ux - Wx, (inch) .



The maximum deflection is 0.420" according to table 1604.3 the code IBC 2021 - the deflection limits L/240. L = 10'= 10'*12"=120"/240 =0.5"
0.420" < 0.50" **<u>Deflection is OK!</u>**

STEEL MEMBER 2835 CHECK

AISI S100-16 LRFD Check

Member 2835	Cold formed C section (5,500; 1,500; 0,054; 0,086; 0,500)	A913 grade 50	LRFD-Ult (auto)	0.64

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

The critical check is on position 5.08 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	-816.63	lbf
Vux	-0.12	lbf
Vuy	6.94	lbf
Mut	-0.00	lbfft
Mux	-1406.59	lbfft
Muy	-0.34	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	0.361	47.939 41.334	0.86	0.481	282.818	0.412	1.000	0.336 -	- -	-	- -	-
3	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
5	5.221	47.939 -47.717	1.00	23.880	66.973	0.846	0.875	- 4.566	1.143 2.283	-	- -	-
7	1.221	-49.778 -49.778	-	-	-	-	-	- -	- -	-	- -	-
9	0.360	-41.112 -47.717	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	0.768	inch ^{3}
Mnxo	3199.68	lbfft
Resistance factor	0.90	
Unity check	0.49	-

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	0.361	45.350 39.088	0.86	0.481	282.894	0.400	1.000	0.361 -	- -	-	- -	-
3	1.221	47.305 47.305	1.00	3.202	164.223	0.537	1.000	1.221 -	0.610 0.610	31.70	0.000 0.000	0.361
5	5.221	45.350 -45.350	1.00	24.000	67.310	0.821	0.892	- 4.656	1.164 2.328	-	- -	-
7	1.221	-47.305 -47.305	-	-	-	-	-	- -	- -	-	- -	-
9	0.360	-39.088 -45.350	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Lltb	3 ft 2.000 in	ft
Sigma,ey	56.189	ksi
Kt	1.00	
Lt	3 ft 2.000 in	ft
Sigma,t	65.421	ksi
Cb	1.02	
Sfx	0.772	inch ^{3}
Fcre	93.519	ksi
Fc	47.305	ksi

Table of values		
Scx	0.772	inch ^{3}
Mnx	3041.97	lbfft
Resistance factor	0.90	
Unity check	0.51	-

Distortional Buckling Strength

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

Table of values		
Sfy	0.772	inch ^{3}
My	3215.29	lbfft
L	1 ft 0.708 in	ft
Beta	1.01	
k,phi,fe	291.72	lbf
k,phi,we	273.76	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	inch ^{2}
k,phi,wg	0.002	inch ^{2}
Fd	68.215	ksi
Sf	0.772	inch ^{3}
Mcrd	4386.64	lbfft
Lambda,d	0.86	
Mn	2790.51	lbfft
Resistance factor	0.90	
Unity check	0.56	-

Data		
Lm	3 ft 2.000 in	ft
Lcr	1 ft 0.708 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	-0.056	inch
Ksi,web	2.00	

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

...::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	0.361	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-
3	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
5	5.221	50.000 50.000	1.00	4.000	11.218	2.111	0.424	2.215 -	- -	-	- -	-
7	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
9	0.360	50.000 50.000	1.00	0.430	252.949	0.445	1.000	0.336 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.326	$inch^2$
Pno	16296.53	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	sway	
Unbraced Length L	10 1/8	3 1/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 1/8	3 1/4	ft
Slenderness	57.95	71.38	
Flexural Buckling stress Fcre	85.256	56.189	ksi

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

Table of values		
Sigma,ex	85.256	ksi
Sigma,ey	56.189	ksi
Kt	1.00	
Lt	3 1/4	ft
Sigma,t	65.421	ksi
Sigma,TF	51.465	ksi
Torsional (-Flexural) buckling stress Fcre	51.465	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	0.361	33.294 33.294	1.00	0.430	252.949	0.363	1.000	0.361 -	- -	-	- -	-
3	1.221	33.294 33.294	1.00	3.202	164.223	0.450	1.000	1.221 -	0.610 0.610	37.78	0.000 0.000	0.361
5	5.221	33.294 33.294	1.00	4.000	11.218	1.723	0.506	2.644 -	- -	-	- -	-
7	1.221	33.294 33.294	1.00	3.202	164.223	0.450	1.000	1.221 -	0.610 0.610	37.78	0.000 0.000	0.360
9	0.360	33.294 33.294	1.00	0.430	252.949	0.363	1.000	0.360 -	- -	-	- -	-

Table of values		
Fe	51.465	ksi
lambda, c	0.99	
Fn	33.294	ksi
Ae	0.352	$inch^2$
Pn	11708.89	lbf
Resistance factor	0.85	
Unity check	0.08	-

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

Table of values		
Py	24535.65	lbf
L	1 ft 2.047 in	ft
k,phi,fe	205.02	lbf
k,phi,we	152.10	lbf
k,phi	0.00	lbf
k,phi,fg	0.006	$inch^2$
k,phi,wg	0.007	$inch^2$
Fd	27.271	ksi
Pcrd	13382.33	lbf
Lambda,d	1.35	
Pn	14090.87	lbf
Resistance factor	0.85	
Unity check	0.07	-

Data		
Lm	3 ft 2.000 in	ft
Lcr	1 ft 2.047 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	-0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	0.056	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	0.361	1.664 1.664	1.00	0.430	252.949	0.081	1.000	0.361 -	- -	-	- -	-
3	1.221	1.664 1.664	1.00	4.000	205.118	0.090	1.000	1.221 -	0.610 0.610	168.99	- 0.000	0.361
5	5.221	1.664 1.664	1.00	4.000	11.218	0.385	1.000	5.221 -	- -	-	- -	-
7	1.221	1.664 1.664	1.00	4.000	205.118	0.090	1.000	1.221 -	0.610 0.610	168.99	- 0.000	0.360
9	0.360	1.664 1.664	1.00	0.430	252.949	0.081	1.000	0.360 -	- -	-	- -	-

Table of values		
Mnx	2790.51	lbfft
Pn	11708.89	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.08+0.56+0.00 = 0.64 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER 2859 CHECK

AISI S100-16 LRFD Check

Member 2859	2x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.36

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

The critical check is on position 5.08 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	-957.15	lbf
Vux	-1.08	lbf
Vuy	9.11	lbf
Mut	-1.19	lbfft
Mux	-1846.15	lbfft
Muy	-1.56	lbfft

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

Table of values		
Critical fibre	24	
Sigma Mx	14.204	ksi
Sigma My	0.054	ksi
f bending	14.257	ksi
Tau t	0.001	ksi
f torsion	0.001	ksi
Composed Stress	14.257	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	0.473	50.000 41.315	0.83	0.496	169.345	0.543	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	4.500	41.315 -41.315	1.00	24.000	90.607	0.675	0.998	- 4.493	1.123 2.246	-	- -	-
4	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-
6	0.473	50.000 41.315	0.83	4.358	5956.483	0.092	1.000	- 0.473	0.218 0.255	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	5.446	50.000 -50.000	1.00	24.000	61.863	0.899	0.840	- 4.575	1.144 2.288	-	- -	-
9	1.446	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-41.315 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	1.560	inch^3
Mnxo	6498.93	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).

Table of values		
Lltb	3 ft 2.000 in	ft
Sigma,ey	168.203	ksi
Kt	1.00	
Lt	3 ft 2.000 in	ft
Sigma,t	2074.695	ksi
Cb	1.02	
Sfx	1.560	$inch^3$
Fcre	926.790	ksi

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

...::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	85.643	ksi
Kt	1.00	
Lt	3 ft 2.000 in	ft
Sigma,t	2074.695	ksi
Cb	1.00	
Sfy	0.732	$inch^3$
Fcre	1375.236	ksi

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

Note: The Web Crippling Check is not executed since the specification does not give provisions for this type of cross-section.

...::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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	4.500	50.000 50.000	1.00	4.000	15.101	1.820	0.483	2.174 -	- -	-	- -	-
4	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 50.000	1.00	4.000	5467.304	0.096	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	5.446	50.000 50.000	1.00	4.000	10.311	2.202	0.409	2.226 -	- -	-	- -	-
9	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
10	0.473	50.000 50.000	1.00	4.000	5467.304	0.096	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.704	inch ^{2}
Pno	35191.38	lbf
Resistance factor	0.85	
Unity check	0.03	-

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	10 1/8	3 1/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 1/8	3 1/4	ft
Slenderness	57.82	41.26	
Flexural Buckling stress Fcre	85.643	168.203	ksi

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

Table of values		
Sigma,ex	85.643	ksi
Sigma,ey	168.203	ksi
Kt	1.00	
Lt	3 1/4	ft
Sigma,t	2074.695	ksi
Sigma,TF	85.232	ksi
Torsional (-Flexural) buckling stress Fcre	85.232	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	0.473	39.114 39.114	1.00	0.430	146.934	0.516	1.000	0.473 -	- -	-	- -	-
2	1.446	39.114 39.114	1.00	4.000	146.251	0.517	1.000	1.446 -	- -	-	- -	-
3	4.500	39.114 39.114	1.00	4.000	15.101	1.609	0.536	2.414 -	- -	-	- -	-
4	1.446	39.114 39.114	1.00	4.000	146.251	0.517	1.000	1.446 -	- -	-	- -	-
5	0.473	39.114 39.114	1.00	0.430	146.934	0.516	1.000	0.473 -	- -	-	- -	-
6	0.473	39.114 39.114	1.00	4.000	5467.304	0.085	1.000	0.473 -	- -	-	- -	-
7	1.446	39.114 39.114	1.00	4.000	146.251	0.517	1.000	1.446 -	- -	-	- -	-
8	5.446	39.114 39.114	1.00	4.000	10.311	1.948	0.455	2.480 -	- -	-	- -	-
9	1.446	39.114 39.114	1.00	4.000	146.251	0.517	1.000	1.446 -	- -	-	- -	-
10	0.473	39.114 39.114	1.00	4.000	5467.304	0.085	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	85.232	ksi
lambda, c	0.77	
Fn	39.114	ksi
Ae	0.731	inch ^{2}
Pn	28573.28	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 [inch⁴]	ds [inch]
1	0.473	0.969 0.969	1.00	0.430	146.934	0.081	1.000	0.473 -	- -	-	- -	-
2	1.446	0.969 0.969	1.00	4.000	146.251	0.081	1.000	1.446 -	- -	-	- -	-
3	4.500	0.969 0.969	1.00	4.000	15.101	0.253	1.000	4.500 -	- -	-	- -	-
4	1.446	0.969 0.969	1.00	4.000	146.251	0.081	1.000	1.446 -	- -	-	- -	-
5	0.473	0.969 0.969	1.00	0.430	146.934	0.081	1.000	0.473 -	- -	-	- -	-
6	0.473	0.969 0.969	1.00	4.000	5467.304	0.013	1.000	0.473 -	- -	-	- -	-
7	1.446	0.969 0.969	1.00	4.000	146.251	0.081	1.000	1.446 -	- -	-	- -	-
8	5.446	0.969 0.969	1.00	4.000	10.311	0.307	1.000	5.446 -	- -	-	- -	-
9	1.446	0.969 0.969	1.00	4.000	146.251	0.081	1.000	1.446 -	- -	-	- -	-
10	0.473	0.969	1.00	4.000	5467.304	0.013	1.000	0.473	-	-	-	-

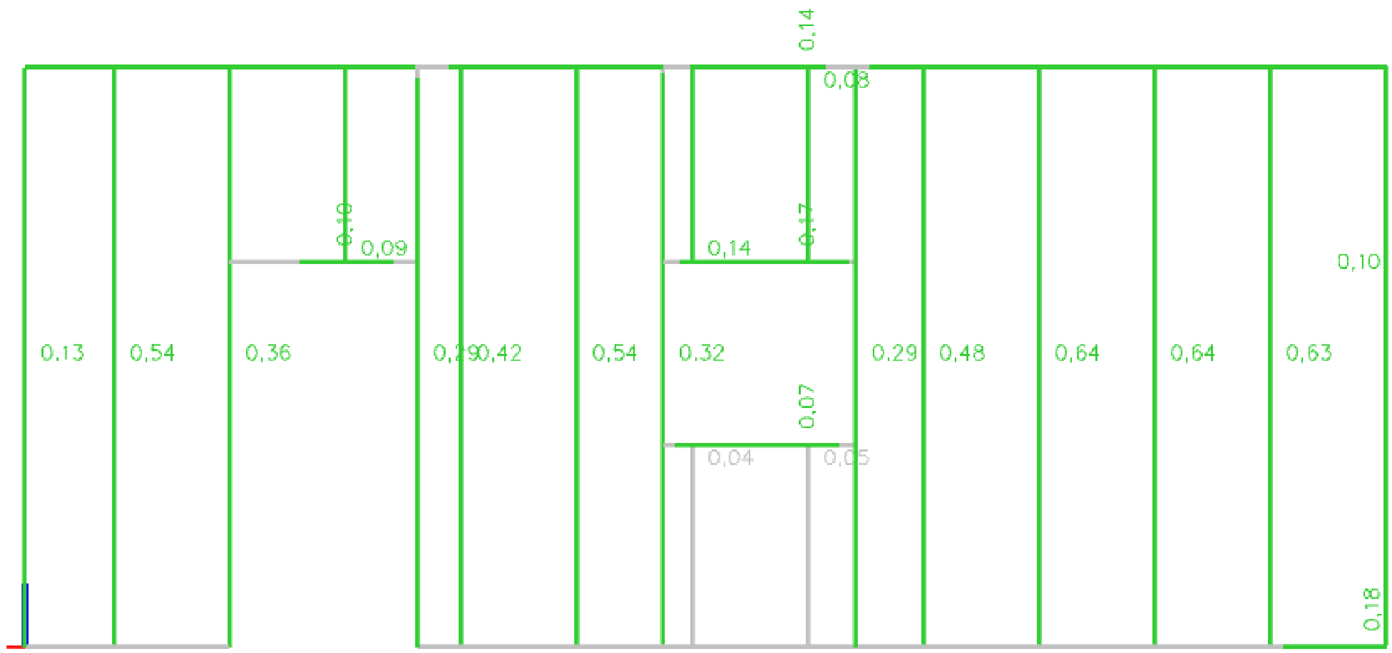
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
		0.969						-	-		-	

Table of values		
Mnx	6498.93	lbfft
Mny	1557.22	lbfft
Pn	28573.28	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.04+0.32+0.00 = 0.36 - (C5.2.1-3)

The member satisfies the check !

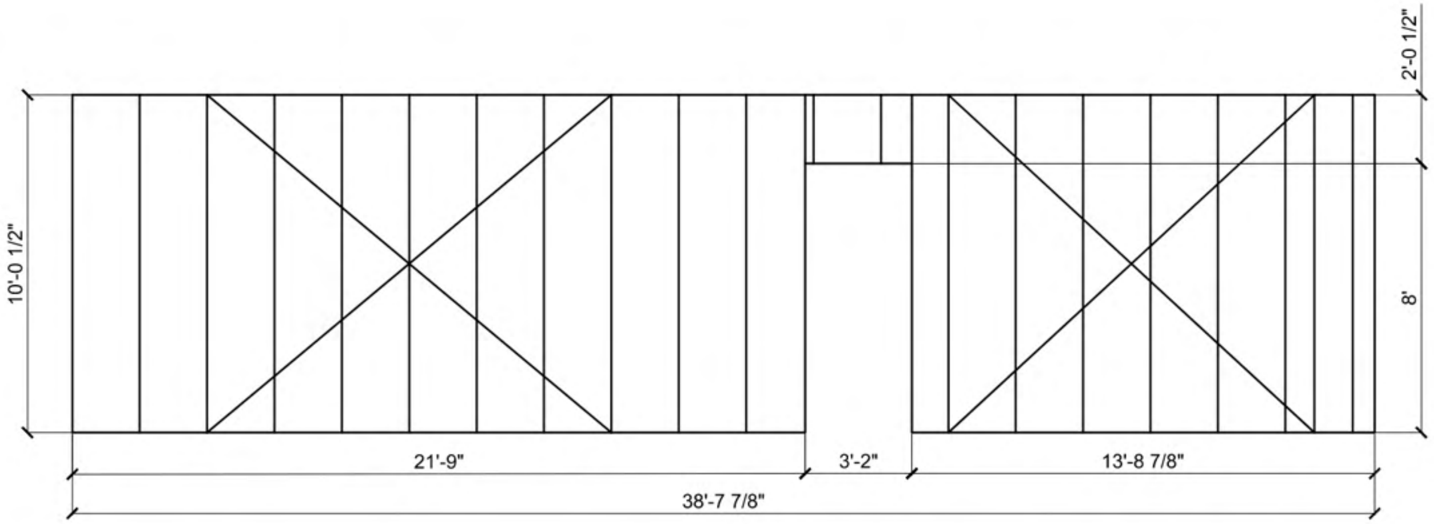
Unity check



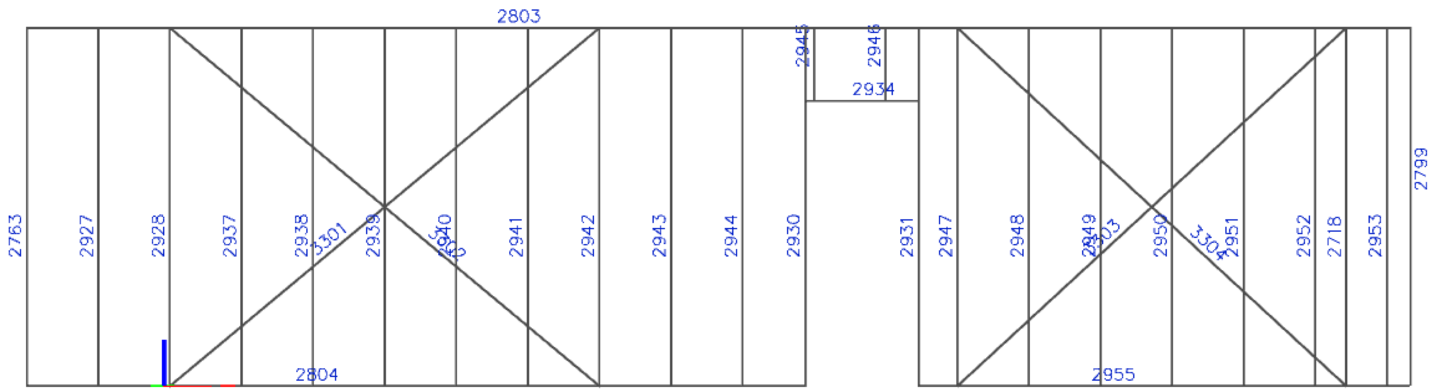
0,14
0,08
0,10
0,09
0,14
0,17
0,10
0,13
0,54
0,36
0,29
0,42
0,54
0,32
0,29
0,48
0,64
0,64
0,63
0,07
0,04
0,05
0,18

2.4.2.7 SIDE GARAGE WALL DESIGN

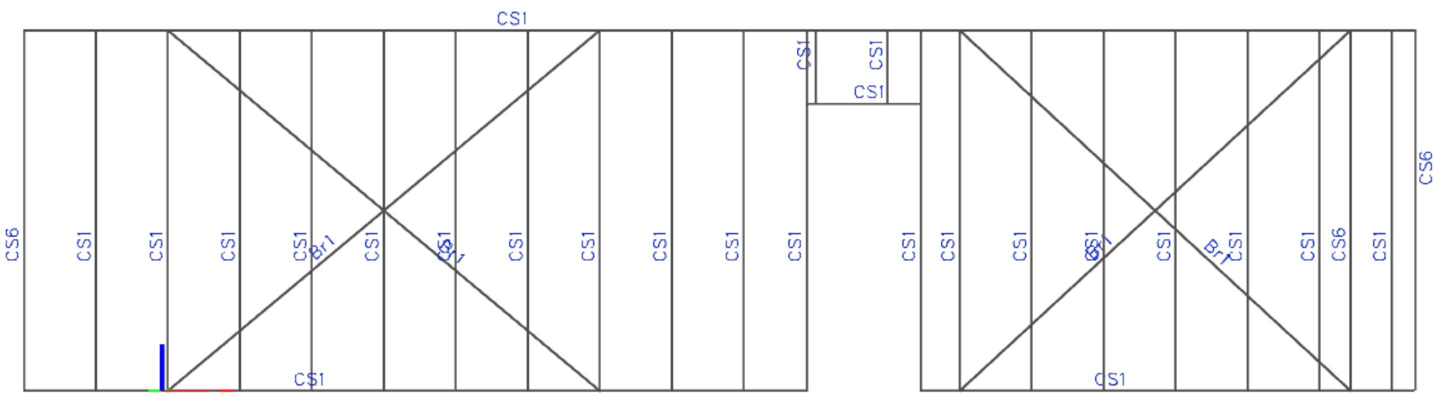
General scheme



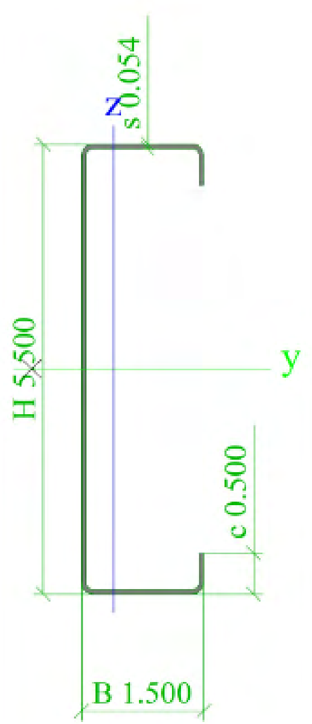
Member numbers

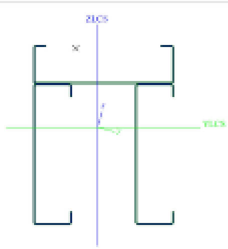


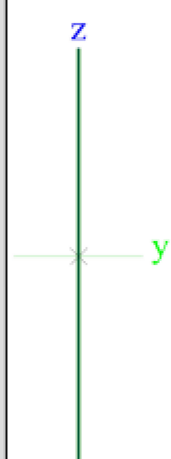
Cross-sections of element



Cross-sections properties

CS1		
Type	Cold formed C section	
Detailed	5.500; 1.500; 0.054; 0.086; 0.500	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.491	
A_y [inch^2], A_z [inch^2]	0.164	0.303
A_L [inch^2/inch], A_D [inch^2/inch]	1.83e+01	1.83e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.392	2.750
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.122	0.139
i_y [inch], i_z [inch]	2.080	0.532
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.772	0.126
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.924	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.05e+00	9.05e+00
d_y [inch], d_z [inch]	-0.999	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.880
$β_y$ [inch], $β_z$ [inch]	0.000	5.904
Picture		

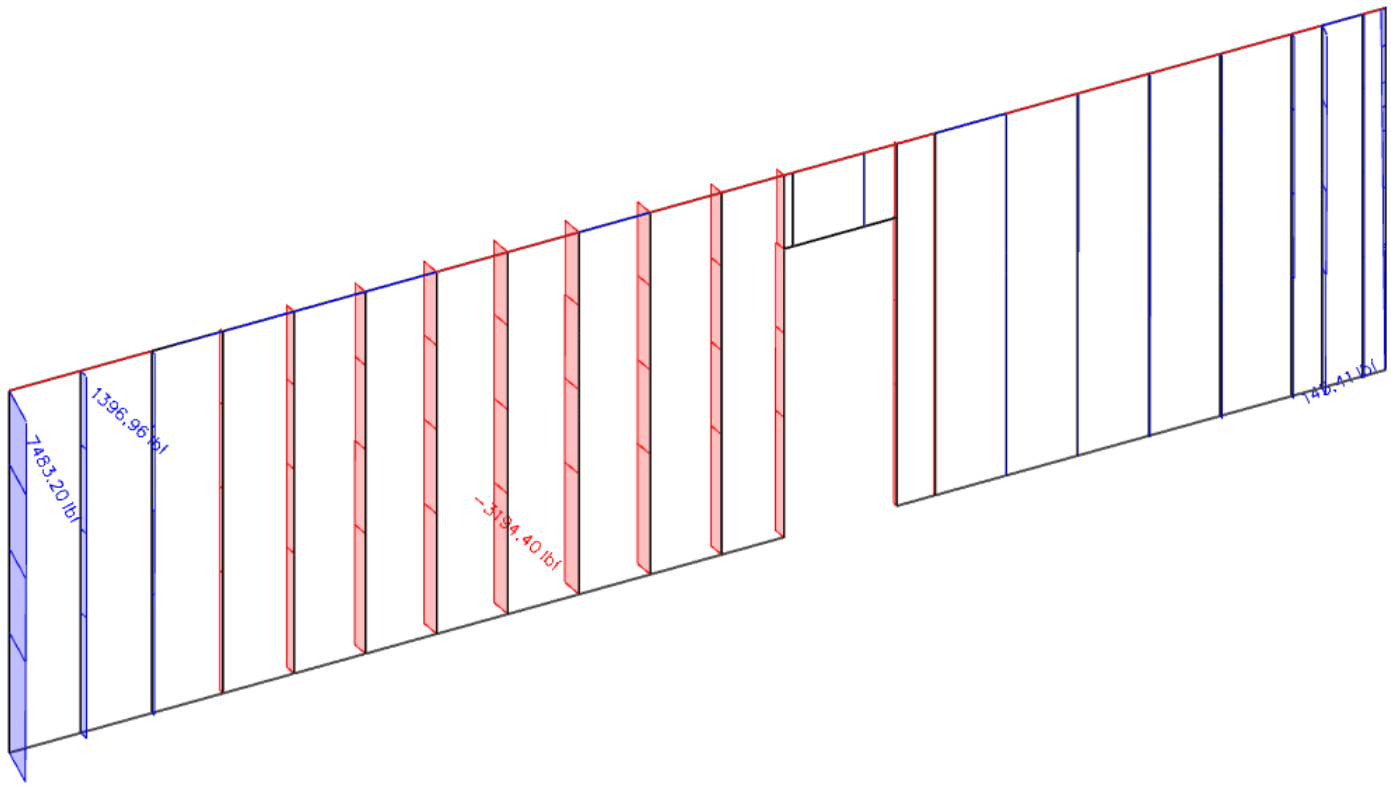
CS6		
Type	3x550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.481	
A_y [inch^2], A_z [inch^2]	0.596	0.834
A_L [inch^2/inch], A_D [inch^2/inch]	5.01e+01	5.01e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	5.971	2.973
$I_{Y.LCS}$ [inch^4], $I_{Z.LCS}$ [inch^4]	7.684	6.418
$I_{YZ.LCS}$ [inch^4]	0.368	
α [deg]	-15.10	
I_y [inch^4], I_z [inch^4]	7.783	6.319
i_y [inch], i_z [inch]	2.292	2.066
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.812	1.642
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.878	2.808
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.44e+02	1.44e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.40e+02	1.40e+02
d_y [inch], d_z [inch]	-1.628	2.757
I_t [inch^4], I_w [inch^6]	0.002	41.712
$β_y$ [inch], $β_z$ [inch]	-6.538	3.765
Picture		

Br1		
Type	Pl 7.5 x 0.054	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.405	
A_y [inch^2], A_z [inch^2]	0.397	0.338
A_L [inch^2/inch], A_D [inch^2/inch]	1.51e+01	1.51e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.000	3.750
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	1.898	0.000
i_y [inch], i_z [inch]	2.165	0.016
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.506	0.004
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.759	0.005
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3.80e+01	3.80e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.73e-01	2.73e-01
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	
A	Area
A_y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A_z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
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
i_z	Radius of gyration about the principal z-axis

Explanations of symbols	
$W_{el.y}$	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 - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Maximum force diagram
Axial force diagram N,
LRFD-Ult (auto)18 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.6*Lr + 0.5*Wx+(-0.18)), lbf.



Shear force diagram Vz ,
LRFD-Ult (auto)31 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + Wy+(-0.18)), lbf.

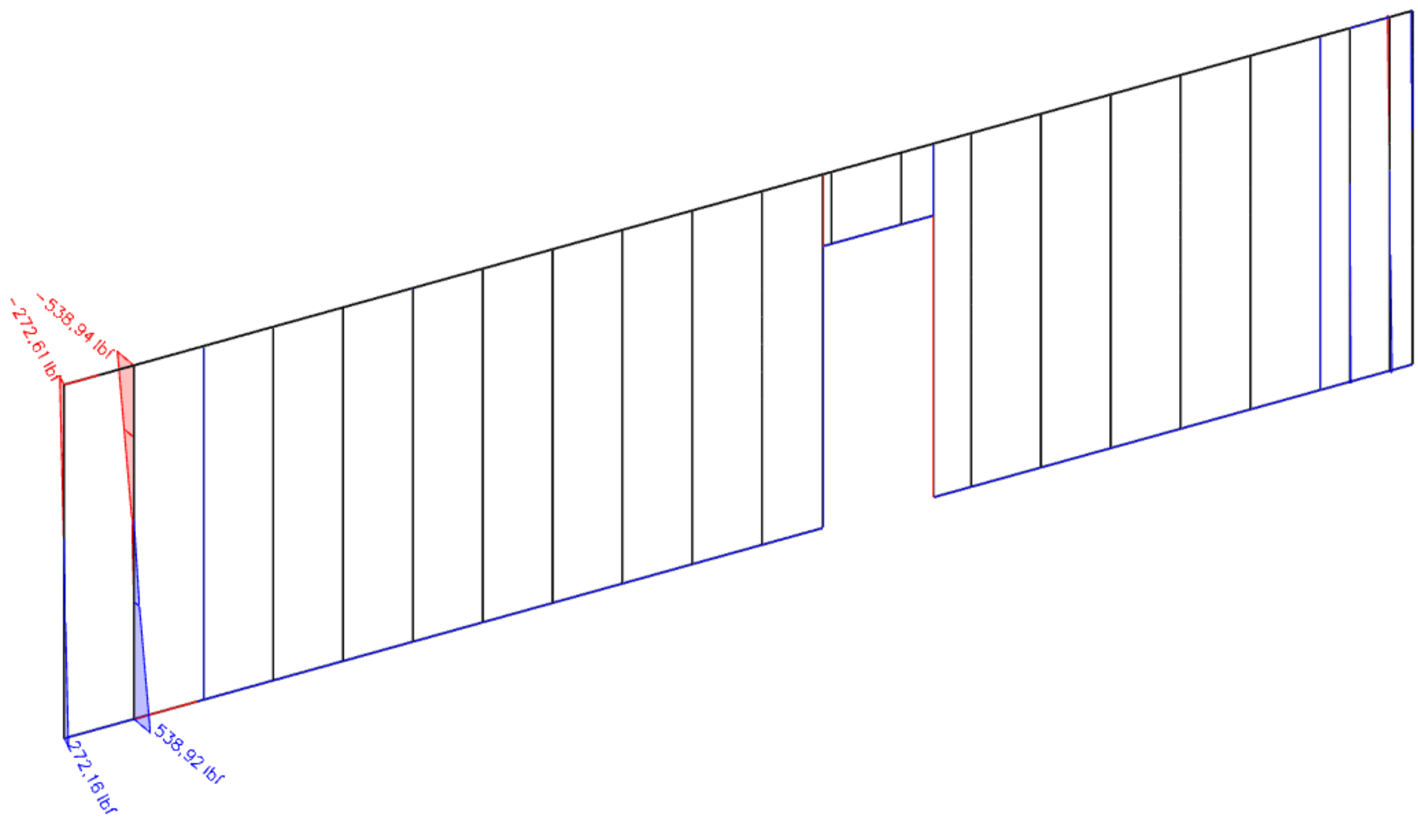
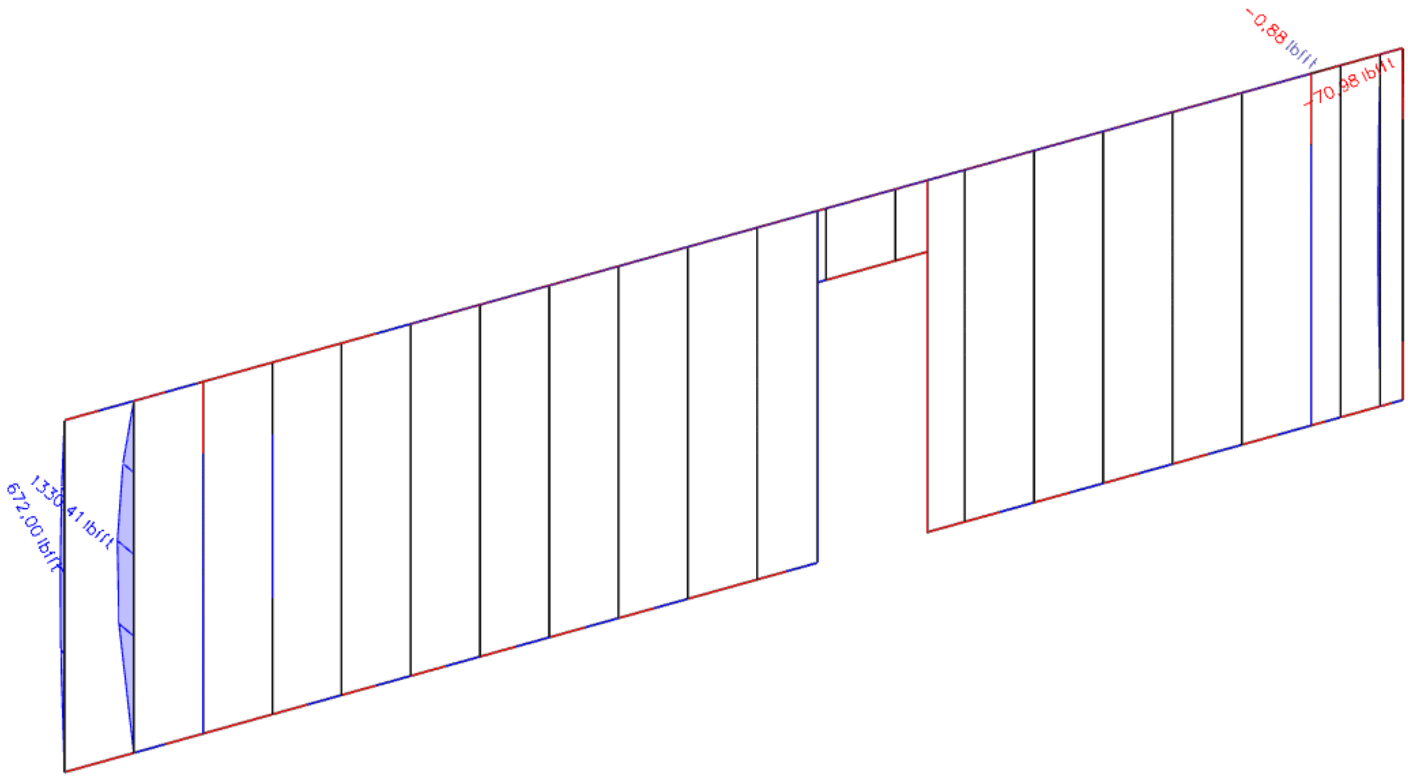
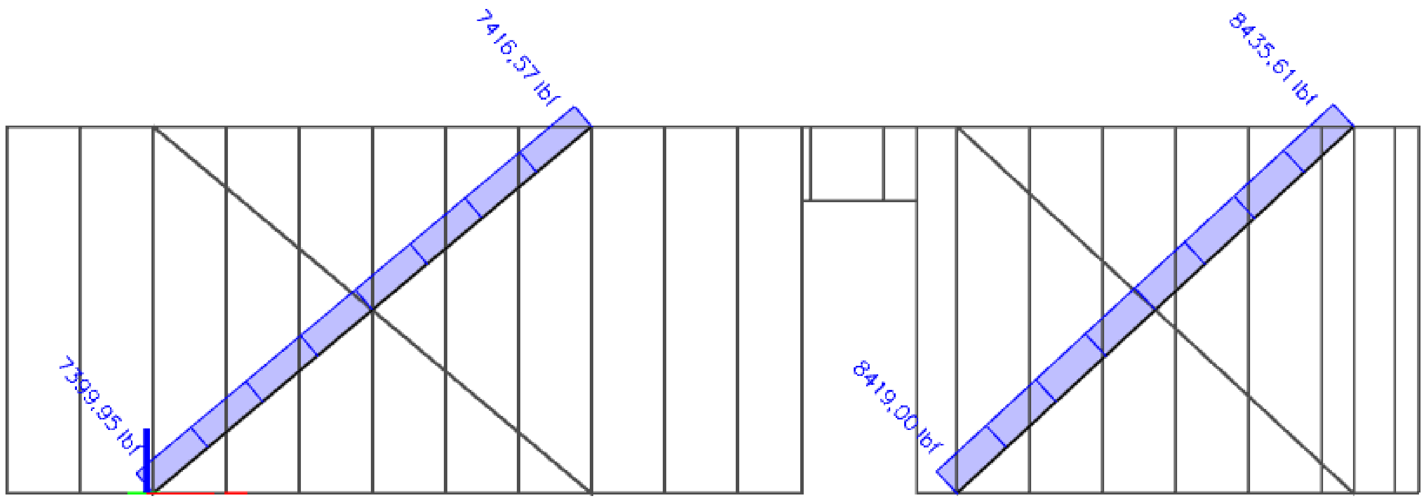


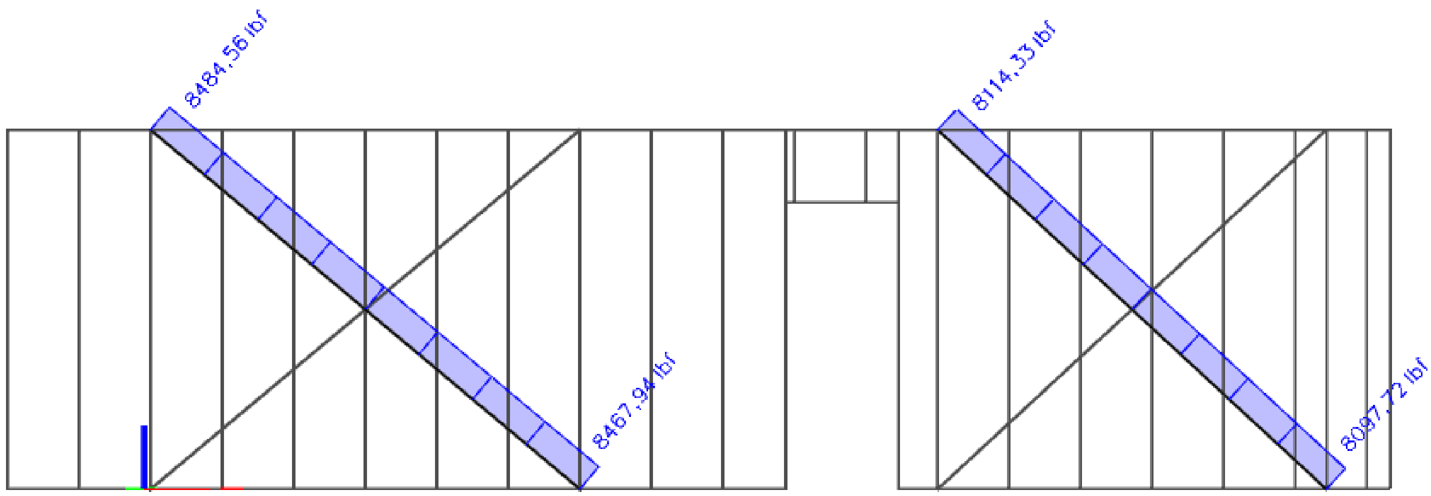
Diagram of moment My,
LRFD-Ult (auto)45 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5Lr + Wy+(-0.18)), lbf*ft.



Axial force diagram for X-bracing N,
LRFD-Ult (auto)37 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*Lr + Wx+(-0.18)), lbf.



Axial force diagram for X-bracing N,
LRFD-Ult (auto)40 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 0.5*L + Wx-(-0.18)), lbf.



STEEL MEMBER 2799 CHECK

AISI S100-16 LRFD Check

Member 2799	3x550S150-54	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	

The critical check is on position 3.38 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	-2908.48	lbf
Vux	-14.14	lbf
Vuy	1.79	lbf
Mut	0.00	lbfft
Mux	-12.52	lbfft
Muy	177.35	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	3 ft 2.000 in	ft
Sigma,ey	845.867	ksi
Kt	1.00	
Lt	3 ft 2.000 in	ft
Sigma,t	283.215	ksi
Cb	1.14	
Sfx	2.023	$inch^3$
Fcre	1820.039	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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 26.703	0.53	5.134	187.721	0.516	1.000	- 1.446	0.586 0.860	-	- -	-
3	5.446	26.703 26.703	1.00	4.000	10.311	1.609	0.536	2.921 -	- -	-	- -	-
4	1.446	50.000 26.703	0.53	5.134	750.883	0.258	1.000	- 1.446	0.586 0.860	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 42.379	0.85	0.487	166.309	0.548	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	2.554	26.703 -14.446	0.54	14.401	168.778	0.398	1.000	- 2.554	0.721 1.277	-	- -	-
9	1.446	-37.743 -37.743	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-30.122 -37.743	-	-	-	-	-	- -	- -	-	- -	-
11	0.473	-14.446 -14.446	-	-	-	-	-	- -	- -	-	- -	-
12	1.446	-14.446 -37.743	-	-	-	-	-	- -	- -	-	- -	-
13	5.446	-37.743 -37.743	-	-	-	-	-	- -	- -	-	- -	-
14	1.446	-14.446 -37.743	-	-	-	-	-	- -	- -	-	- -	-
15	0.473	-14.446 -14.446	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	1.981	inch ^{3}
Mnyo	8253.00	lbfft
Resistance factor	0.90	
Unity check	0.02	-

Lateral-Torsional Buckling Strength

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

Table of values		
Sigma,ex	103.605	ksi
Kt	1.00	
Lt	3 ft 2.000 in	ft
Sigma,t	283.215	ksi
Cb	1.00	
Sfy	2.148	$inch^3$
Fcre	525.047	ksi

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

Note: The Web Crippling Check is not executed since the specification does not give provisions for this type of cross-section.

...::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	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
3	5.446	50.000 50.000	1.00	4.000	10.311	2.202	0.409	2.226 -	- -	-	- -	-
4	1.446	50.000 50.000	1.00	4.000	585.004	0.292	1.000	1.446 -	- -	-	- -	-
5	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
6	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
7	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
8	2.554	50.000 50.000	1.00	4.000	46.881	1.033	0.762	1.946 -	- -	-	- -	-
9	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
10	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
11	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-
12	1.446	50.000 50.000	1.00	4.000	146.251	0.585	1.000	1.446 -	- -	-	- -	-
13	5.446	50.000 50.000	1.00	4.000	10.311	2.202	0.409	2.226 -	- -	-	- -	-
14	1.446	50.000 50.000	1.00	4.000	585.004	0.292	1.000	1.446 -	- -	-	- -	-
15	0.473	50.000 50.000	1.00	0.430	146.934	0.583	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	1.124	$inch^2$
Pno	56213.85	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	sway	
Unbraced Length L	10 1/8	3 1/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 1/8	3 1/4	ft
Slenderness	52.57	18.40	
Flexural Buckling stress Fcre	103.605	845.867	ksi

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

Table of values		
Sigma,ex	103.605	ksi
Sigma,ey	845.867	ksi
Kt	1.00	
Lt	3 1/4	ft
Sigma,t	283.215	ksi
Sigma,TF	96.707	ksi
Torsional (-Flexural) buckling stress Fcre	96.707	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	0.473	40.271 40.271	1.00	0.430	146.934	0.524	1.000	0.473 -	- -	-	- -	-
2	1.446	40.271 40.271	1.00	4.000	146.251	0.525	1.000	1.446 -	- -	-	- -	-
3	5.446	40.271 40.271	1.00	4.000	10.311	1.976	0.450	2.449 -	- -	-	- -	-
4	1.446	40.271 40.271	1.00	4.000	585.004	0.262	1.000	1.446 -	- -	-	- -	-
5	0.473	40.271 40.271	1.00	0.430	146.934	0.524	1.000	0.473 -	- -	-	- -	-
6	0.473	40.271 40.271	1.00	0.430	146.934	0.524	1.000	0.473 -	- -	-	- -	-
7	1.446	40.271 40.271	1.00	4.000	146.251	0.525	1.000	1.446 -	- -	-	- -	-
8	2.554	40.271 40.271	1.00	4.000	46.881	0.927	0.823	2.102 -	- -	-	- -	-
9	1.446	40.271 40.271	1.00	4.000	146.251	0.525	1.000	1.446 -	- -	-	- -	-
10	0.473	40.271 40.271	1.00	0.430	146.934	0.524	1.000	0.473 -	- -	-	- -	-
11	0.473	40.271 40.271	1.00	0.430	146.934	0.524	1.000	0.473 -	- -	-	- -	-
12	1.446	40.271 40.271	1.00	4.000	146.251	0.525	1.000	1.446 -	- -	-	- -	-
13	5.446	40.271 40.271	1.00	4.000	10.311	1.976	0.450	2.449 -	- -	-	- -	-
14	1.446	40.271 40.271	1.00	4.000	585.004	0.262	1.000	1.446 -	- -	-	- -	-
15	0.473	40.271 40.271	1.00	0.430	146.934	0.524	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	96.707	ksi
lambda, c	0.72	
Fn	40.271	ksi
Ae	1.157	inch ^{2}
Pn	46582.52	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	0.473	1.964 1.964	1.00	0.430	146.934	0.116	1.000	0.473 -	- -	-	- -	-
2	1.446	1.964 1.964	1.00	4.000	146.251	0.116	1.000	1.446 -	- -	-	- -	-
3	5.446	1.964 1.964	1.00	4.000	10.311	0.436	1.000	5.446 -	- -	-	- -	-
4	1.446	1.964 1.964	1.00	4.000	585.004	0.058	1.000	1.446 -	- -	-	- -	-
5	0.473	1.964 1.964	1.00	0.430	146.934	0.116	1.000	0.473 -	- -	-	- -	-
6	0.473	1.964 1.964	1.00	0.430	146.934	0.116	1.000	0.473 -	- -	-	- -	-

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	1.446	1.964 1.964	1.00	4.000	146.251	0.116	1.000	1.446 -	- -	-	- -	-
8	2.554	1.964 1.964	1.00	4.000	46.881	0.205	1.000	2.554 -	- -	-	- -	-
9	1.446	1.964 1.964	1.00	4.000	146.251	0.116	1.000	1.446 -	- -	-	- -	-
10	0.473	1.964 1.964	1.00	0.430	146.934	0.116	1.000	0.473 -	- -	-	- -	-
11	0.473	1.964 1.964	1.00	0.430	146.934	0.116	1.000	0.473 -	- -	-	- -	-
12	1.446	1.964 1.964	1.00	4.000	146.251	0.116	1.000	1.446 -	- -	-	- -	-
13	5.446	1.964 1.964	1.00	4.000	10.311	0.436	1.000	5.446 -	- -	-	- -	-
14	1.446	1.964 1.964	1.00	4.000	585.004	0.058	1.000	1.446 -	- -	-	- -	-
15	0.473	1.964 1.964	1.00	0.430	146.934	0.116	1.000	0.473 -	- -	-	- -	-

Table of values		
Mnx	7537.31	lbfft
Mny	8253.00	lbfft
Pn	46582.52	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.07+0.00+0.02 = 0.10 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER 2927 CHECK

AISI S100-16 LRFD Check

Member 2927	Cold formed C section (5,500; 1,500; 0,054; 0,086; 0,500)	A913 grade 50	LRFD-Ult (auto)	0.60

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

The critical check is on position 5.67 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	-36.91	lbf
Vux	0.05	lbf
Vuy	-69.30	lbf
Mut	0.00	lbfft
Mux	1330.41	lbfft
Muy	-0.45	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	0.361	-41.112 -47.717	-	-	-	-	-	- -	- -	-	- -	-
3	1.221	-49.778 -49.778	-	-	-	-	-	- -	- -	-	- -	-
5	5.221	47.939 -47.717	1.00	23.880	66.973	0.846	0.875	- 4.566	1.143 2.283	-	- -	-
7	1.221	50.000 50.000	1.00	3.127	160.326	0.558	1.000	1.221 -	0.569 0.652	30.83	0.000 0.000	0.336
9	0.360	47.939 41.334	0.86	0.481	282.818	0.412	1.000	0.336 -	- -	-	- -	-

Table of values		
Sxe	0.768	inch^3
Mnxo	3199.68	lbfft
Resistance factor	0.90	
Unity check	0.46	-

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	0.361	-31.784 -36.877	-	-	-	-	-	- -	- -	-	- -	-
3	1.221	-38.466 -38.466	-	-	-	-	-	- -	- -	-	- -	-
5	5.221	36.877 -36.877	1.00	24.000	67.310	0.740	0.949	- 4.957	1.239 2.479	-	- -	-
7	1.221	38.466 38.466	1.00	3.202	164.223	0.484	1.000	1.221 -	0.610 0.610	35.15	0.000 0.000	0.360
9	0.360	36.877 31.784	0.86	0.481	282.894	0.361	1.000	0.360 -	- -	-	- -	-

Table of values		
Lltb	4 ft 8.000 in	ft
Sigma,ey	25.873	ksi
Kt	1.00	
Lt	4 ft 8.000 in	ft
Sigma,t	31.190	ksi
Cb	1.06	
Sfx	0.772	inch ^{3}
Fcre	45.151	ksi
Fc	38.466	ksi
Scx	0.772	inch ^{3}
Mnx	2473.60	lbfft
Resistance factor	0.90	
Unity check	0.60	-

Distortional Buckling Strength

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

Table of values		
Sfy	0.772	$inch^3$
My	3215.29	lbfft
L	1 ft 0.708 in	ft
Beta	1.06	
k,phi,fe	291.72	lbf
k,phi,we	273.76	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	$inch^2$
k,phi,wg	0.002	$inch^2$
Fd	71.304	ksi
Sf	0.772	$inch^3$
Mcrd	4585.27	lbfft
Lambda,d	0.84	
Mn	2830.90	lbfft
Resistance factor	0.90	
Unity check	0.52	-

Data		
Lm	4 ft 8.000 in	ft
Lcr	1 ft 0.708 in	ft
h0	5.500	inch
Ixf	0.002	$inch^4$
Iyf	0.018	$inch^4$
Ixyf	-0.003	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.493	inch
hxf	-0.940	inch
Af	0.095	$inch^2$
y0f	0.056	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	0.282	4223.26
7	0.000	0.00

Table of values		
Vn,y	4223.26	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	3199.68	lbfft
Vny	4223.26	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.21+0.00) = 0.46

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	10 1/8	4 3/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 1/8	4 3/4	ft
Slenderness	57.95	105.19	
Flexural Buckling stress Fcre	85.256	25.873	ksi

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

Table of values		
Sigma,ex	85.256	ksi
Sigma,ey	25.873	ksi
Kt	1.00	
Lt	4 3/4	ft
Sigma,t	31.190	ksi
Sigma,TF	25.873	ksi
Torsional (-Flexural) buckling stress Fcre	25.873	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	0.361	22.268 22.268	1.00	0.430	252.949	0.297	1.000	0.361 -	- -	-	- -	-
3	1.221	22.268 22.268	1.00	3.202	164.223	0.368	1.000	1.221 -	0.610 0.610	46.20	0.000 0.000	0.361
5	5.221	22.268 22.268	1.00	4.000	11.218	1.409	0.599	3.127 -	- -	-	- -	-
7	1.221	22.268 22.268	1.00	3.202	164.223	0.368	1.000	1.221 -	0.610 0.610	46.20	0.000 0.000	0.360
9	0.360	22.268 22.268	1.00	0.430	252.949	0.297	1.000	0.360 -	- -	-	- -	-

Table of values		
Fe	25.873	ksi
lambda, c	1.39	
Fn	22.268	ksi
Ae	0.378	inch ^{2}
Pn	8412.59	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 [inch ^{4}]	ds [inch]
1	0.361	0.075 0.075	1.00	0.430	252.949	0.017	1.000	0.361 -	- -	-	- -	-
3	1.221	0.075 0.075	1.00	4.000	205.118	0.019	1.000	1.221 -	0.610 0.610	794.93	- 0.000	0.361
5	5.221	0.075 0.075	1.00	4.000	11.218	0.082	1.000	5.221 -	- -	-	- -	-
7	1.221	0.075 0.075	1.00	4.000	205.118	0.019	1.000	1.221 -	0.610 0.610	794.93	- 0.000	0.360
9	0.360	0.075 0.075	1.00	0.430	252.949	0.017	1.000	0.360 -	- -	-	- -	-

Table of values		
Mnx	2473.60	lbfft
Pn	8412.59	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.01+0.60+0.00 = 0.60 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER 3302 CHECK (X-BRACING)

AISI S100-16 LRFD Check

Member 3302	Pl 7.5 x 0.054	A913 grade 50	NC_LRFD-Ult (au	0.68

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

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

The critical check is on position 15.65 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

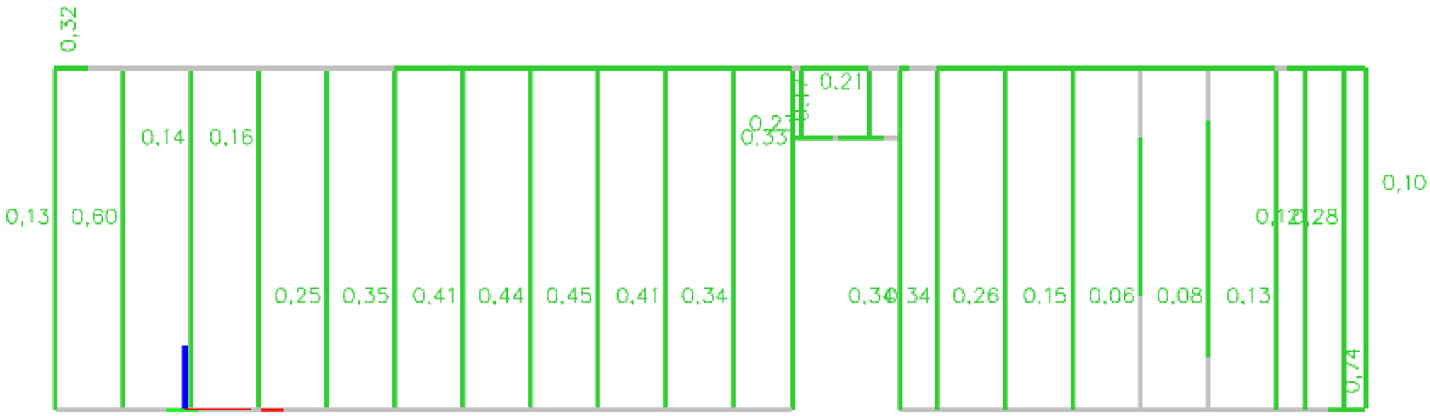
Internal forces		
Pu	12428.45	lbf
Vux	-0.00	lbf
Vuy	-0.00	lbf
Mut	-0.00	lbfft
Mux	0.00	lbfft
Muy	0.00	lbfft

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

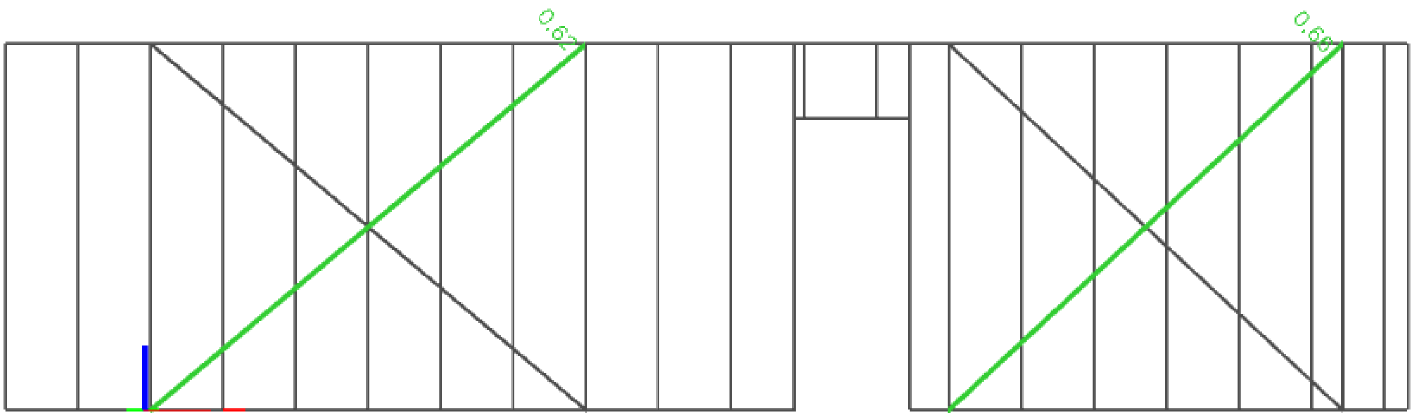
Table of values		
Tn	20250.00	lbf
Resistance factor	0.90	
Unity check	0.68	-

The member satisfies the check !

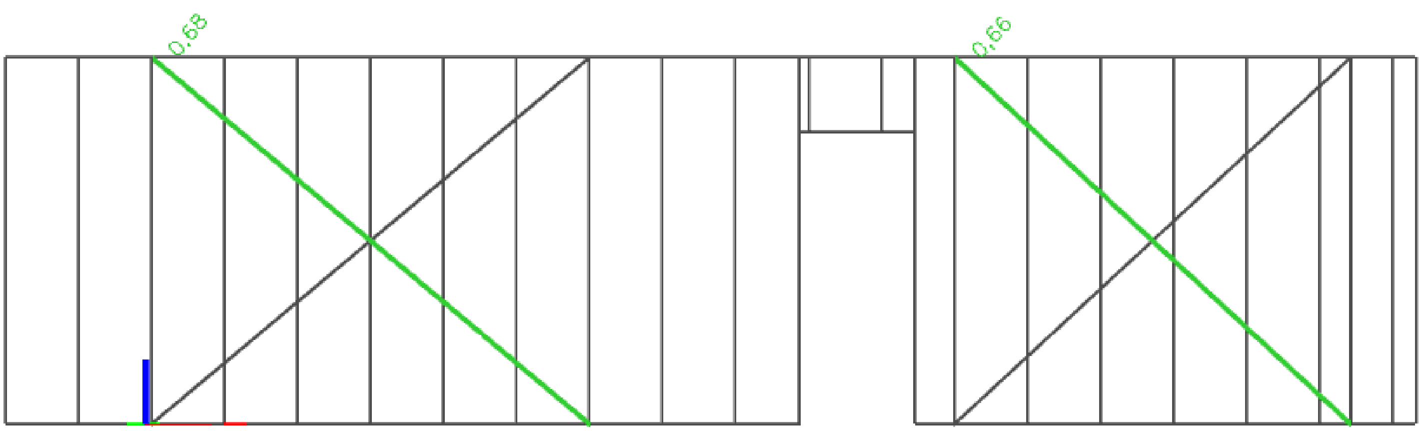
Unity check



X-Bracing Unity check



X-Bracing Unity check



2.5 LATERAL ANALYSIS

2.5.1 BUILDING SEISMIC WEIGHT

Second floor					
Type of constr.	h or w, ft	L, ft	Area, ft2	Weight, psf	Weight, kip
Roof	32.83	51.5	1690.7	20	33.81
Exterior LGS wall	5	48.6	243.0	45	10.94
	5	48.6	243.0	45	10.94
	5	25	125.0	45	5.63
	5	25	125.0	45	5.63
Total seismic weight					**66.93**

First floor					
Type of constr.	h or w, ft	L, ft	Area, ft2	Weight, psf	Weight, kip
Roof	29	38	1102	20	22.04
	21.25	40	850	20	17.00
Floor	30	48.6	1458	15	21.87
Exterior LGS wall	10	26.3	263.3	45	11.85
	10	9.7	96.7	45	4.35
	10	9.7	96.7	45	4.35
	10	33.8	338.3	45	15.23
	10	6.1	60.8	45	2.74
	5	15.5	77.3	45	3.48
	5	19.8	99.2	45	4.46
	5	1.8	8.8	45	0.39
	5	23.7	118.3	45	5.33
	5	34.6	172.9	45	7.78
	5	23.7	118.3	45	5.33
	5	3.8	19.2	45	0.86
	5	19.8	99.2	45	4.46
	10	6.9	69.2	45	3.11
	10	6.8	68.3	45	3.08
	10	10.3	102.5	45	4.61
	10	6.8	68.3	45	3.08
	10	30.9	309.2	45	13.91
Total seismic weight					**159.30**

Total seismic weight: **226.23**

2.5.2 BASE SHEAR SEISMIC FORCES (ASCE 7-16)

Site parameters

Site class — D, Soil properties not known

Mapped acceleration parameters (Section 11.4.2)

at short period — S_S = **0.047**

at 1 sec period — S_1 = **0.025**

Site coefficientat short period (Table 11.4-1) — F_a = **1.600**

at 1 sec period (Table 11.4-2) — F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1) — $S_{MS} = F_a * S_S =$ **0.075**

at 1 sec period (Eq. 11.4-2) — $S_{M1} = F_v * S_1 =$ **0.060**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) — $S_{DS} = 2 / 3 * S_{MS} =$ **0.050**

at 1 sec period (Eq. 11.4-4) — $S_{D1} = 2 / 3 * S_{M1} =$ **0.040**

Seismic design category

Occupancy category (Table 1-1) — II

Seismic design category based on short period response acceleration (Table 11.6-1) — A

Seismic design category based on 1 sec period response acceleration (Table 11.6-2) — A

Seismic base shear (Sect 11.7)

Effective seismic weight of the structure — W = **226.2** kips

Seismic base shear (Eq. 9.5.3-1) — $V = 0.01 * W =$ **2.26** kips

Vertical distribution of seismic forces (Sect 11.7)

Lateral force induce at level i — $F_x = 0.01 * w_x$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips)	Lateral force induced at Level i (kips),
1	12.00	159.3	1.6;
2	22.00	66.9	0.7;

2.5.3 WIND BASE SHEAR AND WIND LOAD ON ROOF DIAPHRAGM

SECOND STOREY ROOF DIAPHRAGM LOAD

Loading	Surface type	Wind pressure, psf	Area, sqft	Load , lb	Total load , lb
Wx (0.18)	Roof	-30.51	645.70	-8820.99	22037.49
	Roof	-46.05	645.70	-13313.89	
	Wall	31.82	243.00	7732.26	
	Wall	-40.38	243.00	-9812.34	
Wx (-0.18)	Roof	20.70	645.70	5984.74	**29893.29**
	Roof	-22.02	645.70	-6366.38	
	Wall	55.84	243.00	13569.12	
	Wall	-16.35	243.00	-3973.05	
Wy (0.18)	Roof	-23.81	304.40	-3245.25	10495.01
	Roof	-46.05	304.40	-6276.52	
	Wall	31.82	125.00	3977.50	
	Wall	-27.89	125.00	-3486.25	
Wy (-0.18)	Roof	28.37	304.40	3866.77	**15850.55**
	Roof	-22.02	304.40	-3001.28	
	Wall	55.86	125.00	6982.50	
	Wall	-16.00	125.00	-2000.00	

SECOND STOREY FLOOR DIAPHRAGM LOAD

Loading	Surface type	Wind pressure, psf	Area, sqft	Load , lb	Total load , lb
Wx (0.18)	Wall	31.82	550.80	17526.46	39767.76
	Wall	-40.38	550.80	-22241.30	
Wx (-0.18)	Wall	55.84	550.80	30756.67	**39762.25**
	Wall	-16.35	550.80	-9005.58	
Wy (0.18)	Wall	31.82	141.75	4510.49	13304.55
	Wall	31.82	152.13	4840.65	
	Wall	-27.89	141.75	-3953.41	
Wy (-0.18)	Wall	55.86	141.75	7918.16	**18683.92**
	Wall	55.86	152.13	8497.76	
	Wall	-16.00	141.75	-2268.00	

FIRST STOREY ROOF DIAPHRAGM LOAD

Loading	Surface type	Wind pressure, psf	Area, sqft	Load, lb	Wind force per storey, lb
Wx (0.18)	Roof	-30.51	712.70	-9736.29	22750.38
	Roof	-46.05	805.20	-16602.67	
	Wall	31.82	220.00	7000.40	
	Wall	-40.38	220.00	-8883.60	
Wx (-0.18)	Roof	20.70	712.70	6605.74	**30426.53**
	Roof	-22.02	805.20	-7938.99	
	Wall	55.84	220.00	12284.80	
	Wall	-16.35	220.00	-3597.00	
Wy (0.18)	Roof	-23.81	393.00	-4189.83	16713.35
	Roof	-46.05	173.00	-3567.14	
	Wall	31.82	197.90	6297.18	
	Wall	-27.89	395.80	-11038.86	
Wy (-0.18)	Roof	28.37	393.00	4992.25	**24085.46**
	Roof	-22.02	173.00	-1705.72	
	Wall	55.86	197.90	11054.69	
	Wall	-16.00	395.80	-6332.80	

Total wind shear:

Wx(0.18) - **84.56 kip**

Wx(-0.18) - **100.08 kip**

Wy(0.18) - **40.51 kip**

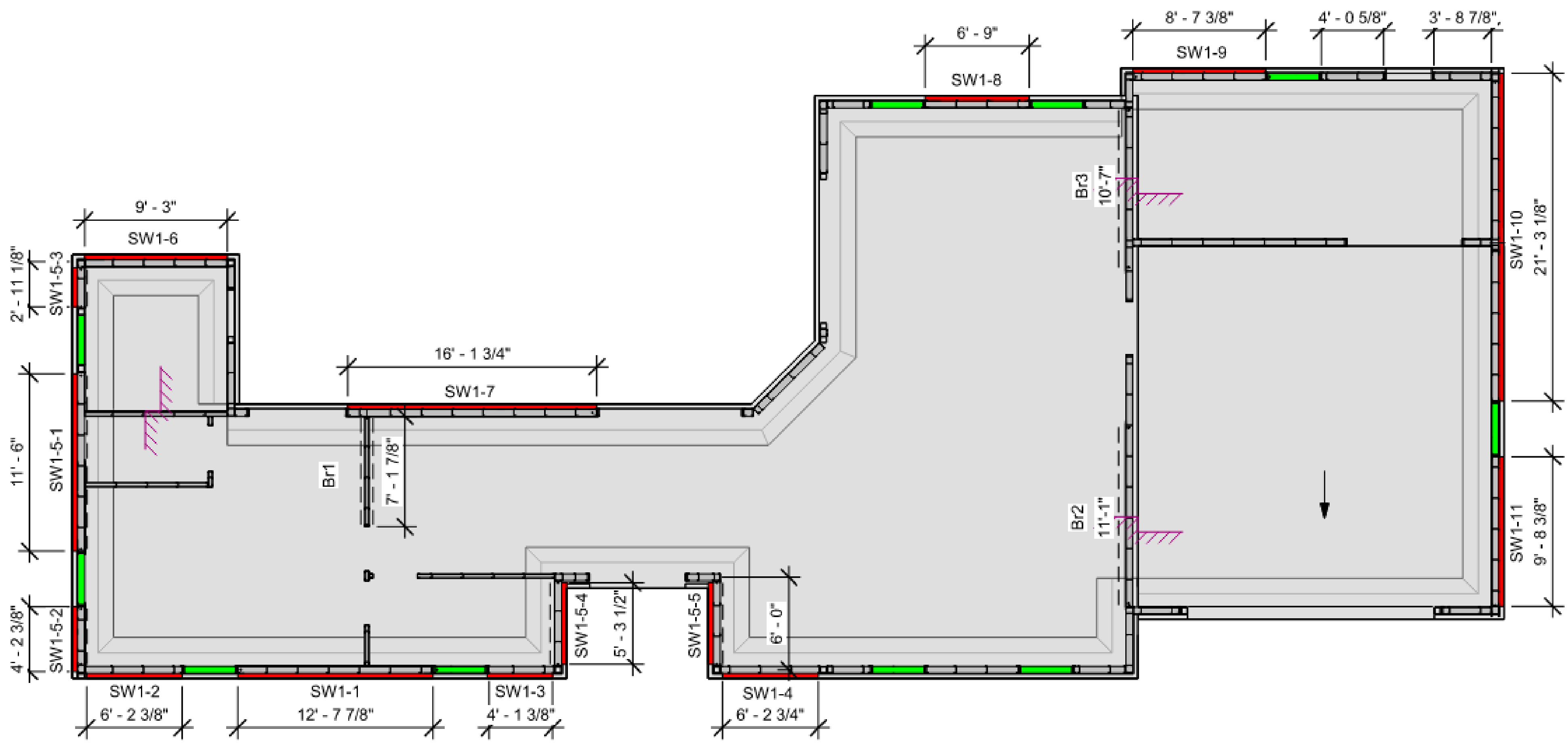
Wy(-0.18) - **58.62 kip**

Transverse direction wind base shear	**100.08 kip**
Longitudinal direction base shear	**58.62 kip**
Seismic base shear	**2.26 kip**

According to the results of the calculation, wind actions affect the calculation in the transverse and longitudinal directions.

2.5.4 SPRAYED CONCRETE SHEAR WALL & X-BRACING - LATERAL DESIGN

SECOND FLOOR SPRAYED CONCRETE SHEAR WALL LOCATION PLAN



FIRST FLOOR SPRAYED CONCRETE SHEAR WALL & X-BRACING LOCATION PLAN

SHEAR WALL & X-BRACING - LATERAL DESIGN TABLE

FLOOR	Total Panel Length (ft)	Force due to wind (lbs)	Force due to siesmic (lbs)	Spray rock capacity (lbs)	Spray rock capacity check	Overloading (lbs)	Type of shear resisting system	Straps	Number of Shear panels	Wall height (ft)	Panel Length (ft)	Governing Force	X-brace axial force (lbs)	X-brace capacity (lbs)	X-brace capacity check	Uplift Force (lbs)	Holdown
1	2	3	4	5	6	7	8	9	10								
2-ND FLOOR																	
Transverse direction																	
SW2-1	7	6540.63	0	7000	OK!	-	Sheathing			10	7					9343.75	S/HDU9
SW2-2	5	4671.88	0	5000	OK!	-	Sheathing			11	5					10278.13	S/HDU9
SW2-3	4	3737.50	0	4000	OK!	-	Sheathing			12	4					11212.50	S/HDS8
SW2-4	20	14950.00	0	20000	OK!	-	Sheathing			13	20					9717.50	S/HDU9
Longitudinal direction																	
SW2-5	12	7925.00	0	12000	OK!	-	Sheathing			16	12					10566.67	S/HDU9
SW2-6	8	3522.22	0	8000	OK!	-	Sheathing			17	8					7484.72	S/HDU9
SW2-7	10	4402.78	0	10000	OK!	-	Sheathing			18	10					7925.00	S/HDU9
1-ST FLOOR																	
Transverse direction																	
SW1-5-1	11	14741.77	0	11000	NOT OK!	3741.77	X Bracing	7.5 x 54mil	1	10	11.00	3741.77	5056.86	20250	OK!	3401.61	S/HDU9
SW1-5-2	4	5360.64	0	4000	NOT OK!	1360.64	X Bracing	7.5 x 54mil	1	10	4.00	1360.64	3663.65	20250	OK!	3401.61	S/HDU9
SW1-5-3	2	2680.32	0	2000	NOT OK!	680.32	X Bracing	7.5 x 54mil	1	10	2.00	680.32	3468.98	20250	OK!	3401.61	S/HDU9
SW1-5-4	5.29	9054.36	0	5291.67	NOT OK!	3762.69	X Bracing	7.5 x 54mil	1	10	5.29	3762.69	8044.78	20250	OK!	7110.60	S/HDU9
SW1-5-5	5.29	6970.42	0	5291.67	NOT OK!	1678.75	X Bracing	7.5 x 54mil	1	10	5.29	1678.75	3589.24	20250	OK!	3172.45	S/HDU9
Br1	7	13150.38	0	0		13150.38	X Bracing	7.5 x 54mil	2	10	7	13150.38	22931.55	40500	OK!	18786.26	S/HDS10
SW1-10	21	4564.00	0	21000	OK!	-	Sheathing			10	21					2173.33	S/HDU9
SW1-11	9	1956.00	0	9000	OK!	-	Sheathing			10	9					2173.33	S/HDU9
Br2	11	9575	0	0		9575.00	X Bracing	7.5 x 54mil	1	10	11	9575.00	12940.24	20250	OK!	8704.55	S/HDU9
Br3	10.5	9575	0	0		9575.00	X Bracing	7.5 x 54mil	1	10	10.5	9575.00	13222.62	20250	OK!	9119.05	S/HDU9
Longitudinal direction																	
SW1-1	12	6171.43	0	12000	OK!	-	Sheathing			10	12					5142.86	S/HDU9
SW1-2	6	3085.71	0	6000	OK!	-	Sheathing			11	6					5657.14	S/HDU9
SW1-3	4	2057.14	0	4000	OK!	-	Sheathing			12	4					6171.43	S/HDU9
SW1-4	6	3085.71	0	6000	OK!	-	Sheathing			13	6					6685.71	S/HDU9
SW1-6	9	1370.00	0	9000	OK!	-	Sheathing			14	9					2131.11	S/HDU9
SW1-7	16	9345.00	0	16000	OK!	-	Sheathing			15	16					8760.94	S/HDU9
SW1-8	6.75	5512.12	0	6750	OK!	-	Sheathing			16	6.75					13065.76	S/HDS8
SW1-9	8	6532.88	0	8000	OK!	-	Sheathing			17	8					13882.37	S/HDS8

2.5.4.1 ANCHOR BOLT DESIGN

ANCHOR BOLT ANCHORING CALCULATIONS FOR HOLDOWN S/HDU9

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Purgavie
Location:
Fastening description: Holdown S/HDU9

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 9.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.00
c_{ac} (inch): 17.18
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Uncracked
Compressive strength, f'_c (psi): 3500
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: Yes
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ 7/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	10280	20096	0.51	Pass
Concrete breakout	**10280**	**10649**	**0.97**	**Pass (Governs)**
Adhesive	10280	15580	0.66	Pass

SET-3G w/ 7/8"Ø F1554 Gr. 36 with hef = 9.000 inch meets the selected design criteria.

ANCHOR BOLT ANCHORING CALCULATIONS FOR HOLDOWN S/HDS8

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Purgavie
Location:
Fastening description: Holdown S/HDS8

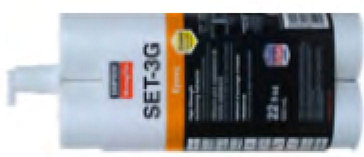
2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 9.500
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.50
c_{ac} (inch): 23.89
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Uncracked
Compressive strength, f'_c (psi): 3500
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: Yes
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ 7/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057



11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	13885	20096	0.69	Pass
Concrete breakout	**13885**	**19118**	**0.73**	**Pass (Governs)**
Adhesive	13885	25856	0.54	Pass

SET-3G w/ 7/8"Ø F1554 Gr. 36 with hef = 9.500 inch meets the selected design criteria.

ANCHOR BOLT ANCHORING CALCULATIONS FOR HOLDOWN S/HDS10

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Purgavie
Location:
Fastening description: Holdown S/HDS10

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 9.500
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.50
c_{ac} (inch): 23.89
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Uncracked
Compressive strength, f'_c (psi): 3500
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: Yes
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ 7/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	18790	20096	0.94	Pass
Concrete breakout	**18790**	**19118**	**0.98**	**Pass (Governs)**
Adhesive	18790	25856	0.73	Pass

SET-3G w/ 7/8"Ø F1554 Gr. 36 with hef = 9.500 inch meets the selected design criteria.

Summary Anchors design result

Model	Max Tension load (lb.)	Anchor
S/HDU9	10278.13	7/8-DIA F1554 GR36 WITH SIMPSON HIGH-STRENGTH EPOXY ADHESIVE "SET-3G", MIN 9" EMBEDMENT
S/HDS8	13882.37	7/8-DIA F1554 GR36 WITH SIMPSON HIGH-STRENGTH EPOXY ADHESIVE "SET-3G", MIN 9.5" EMBEDMENT
S/HDS10	18786.26	7/8-DIA F1554 GR36 WITH SIMPSON HIGH-STRENGTH EPOXY ADHESIVE "SET-3G", MIN 9.5" EMBEDMENT

2.5.5 DIAPHRAGM DESIGN

2.5.5.1 2-ND STORY ROOF DIAPHRAGM DESIGN

Building Dimensions

Building length	$L =$	48.13	ft
Building width	$B =$	24.58	ft
Total wind load (Transverse direction)	$Q_{Tw} =$	29.9	kip
Total wind load (Longitudinal direction)	$Q_{Lw} =$	15.85	kip
Seismic diaphragm force	$S =$	2.26	kip

From Table 4.2B SDPW 2021 Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Nominal Shear Strength for Diaphragms Sheathed - Structural I - 19/32" THK

	$V_{nw} =$	1875	lb/ft
	(LRFD) $\phi_v =$	0.65	
Available Strength	$V_{aw} =$	1218.75	

The maximum diaphragm wind shear in the roof diaphragm is:

Transverse direction

$V_t = Q_{Tw} / 2B =$ 608.14 lb/ft

Longitudinal direction

$V_L = Q_{Lw} / 2L =$ 164.68 lb/ft

Shear capacity for wind loading;

$V_{aw} / \max(V_t; V_L) \leq 1$

$\max(V_t; V_L) =$ 608.14 lb/ft

$V_{aw} =$ 1218.75 lb/ft

$V_{aw} / \max(V_t; V_L) =$ **0.50** **PASS!**

The maximum diaphragm seismic shear in the roof diaphragm is:

Transverse direction

$S_t = S / 2B =$ 45.97 lb/ft

Longitudinal direction

$S_L = S / 2L =$ 23.48 lb/ft

Shear capacity for seismic loading;

$V_{aw} / \max(S_t; S_l) \leq 1$

$\max(S_t; S_L) =$ 45.97 lb/ft

$V_{aw} =$ 1218.75 lb/ft

$V_{aw} / \max(S_t; S_l) =$ **0.04** **PASS!**

2.5.5.2 1-ST STORY ROOF DIAPHRAGM DESIGN

Building Dimensions

Building length	$L =$	43.5	ft
Building width	$B =$	34.7	ft
Total wind load (Transverse direction)	$Q_{Tw} =$	30.43	kip
Total wind load (Longitudinal direction)	$Q_{Lw} =$	24.09	kip
Seismic diaphragm force	$S =$	2.26	kip

From Table 4.2B SDPW 2021 Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Nominal Shear Strength for Diaphragms Sheathed - Structural I - 15/32" THK

	$V_{nw} =$	1875	lb/ft
	(LRFD) $\phi_v =$	0.65	
Available Strength	$V_{aw} =$	1218.75	

The maximum diaphragm wind shear in the roof diaphragm is:

Transverse direction

$V_t = Q_{Tw} / 2B =$ 438.47 lb/ft

Longitudinal direction

$V_L = Q_{Lw} / 2L =$ 276.90 lb/ft

Shear capacity for wind loading;

$V_{aw} / \max(V_t; V_L) \leq 1$

$\max(V_t; V_L) =$ 438.47 lb/ft

$V_{aw} =$ 1218.75 lb/ft

$V_{aw} / \max(V_t; V_L) =$ **0.36** **PASS!**

The maximum diaphragm seismic shear in the roof diaphragm is:

Transverse direction

$S_t = S / 2B =$ 32.56 lb/ft

Longitudinal direction

$S_L = S / 2L =$ 25.98 lb/ft

Shear capacity for seismic loading;

$V_{aw} / \max(S_t; S_l) \leq 1$

$\max(S_t; S_L) =$ 32.56 lb/ft

$V_{aw} =$ 1218.75 lb/ft

$V_{aw} / \max(S_t; S_l) =$ **0.03** **PASS!**

2.5.5.3 FLOOR DIAPHRAGM DESIGN

Building Dimensions

Building length	$L =$	48.13	ft
Building width	$B =$	24.58	ft
Total wind load (Transverse direction)	$Q_{Tw} =$	39.8	kip
Total wind load (Longitudinal direction)	$Q_{Lw} =$	18.7	kip
Seismic diaphragm force	$S =$	2.26	kip

Diaphragms Sheathed - Magnesium Oxide Board - 3/4" THK

Shear Strength: Not less than 391 psi when tested in accordance with ASTM E 455

Nominal Shear Strength for Diaphragms	$V_{nw} =$	2458	lb/ft
	(LRFD) $\phi_v =$	0.67	
Available Strength	$V_{aw} =$	1646.9	lb/ft

The maximum diaphragm wind shear in the floor diaphragm is:

Transverse direction

$V_t = Q_{Tw} / 2B =$ 809.49 lb/ft

Longitudinal direction

$V_L = Q_{Lw} / 2L =$ 194.29 lb/ft

Shear capacity for wind loading;

$V_{aw} / \max(V_t; V_L) \leq 1$

$\max(V_t; V_L) =$ 809.49 lb/ft

$V_{aw} =$ 1646.86 lb/ft

$V_{aw} / \max(V_t; V_L) =$ **0.49** **PASS!**

The maximum diaphragm seismic shear in the floor diaphragm is:

Transverse direction

$S_t = S / 2B =$ 45.97 lb/ft

Longitudinal direction

$S_L = S / 2L =$ 23.48 lb/ft

Shear capacity for seismic loading;

$V_{aw} / \max(S_t; S_l) \leq 1$

$\max(S_t; S_L) =$ 45.97 lb/ft

$V_{aw} =$ 1646.86 lb/ft

$V_{aw} / \max(S_t; S_l) =$ **0.03** **PASS!**

2.6 CONNECTION DESIGN
2.6.1 X-BRACING CONNECTION DESIGN

Maximum tensile force in brace Br3 - 13223 lb,
For connecting x-brace strap to wall framing use X Metal Screw. maximum shear capacity for #10 screw & 54 mil steel - 810 lbs.

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

Minimum number of screws 13223 / 810 = 16.32 pcs. = **18** pcs one side.

2.6.2 WIDE FLANGE W10X30 BEAM TO HSS 5X5X3/8 COLUMN CONNECTION DESIGN

Project data

Project name	Purgavie
Project number	23-126
Author	DZ
Description	W12X96 + W10X30 + HSS5X5X3/8
Date	04.01.2024
Design code	AISC 360-16

Material

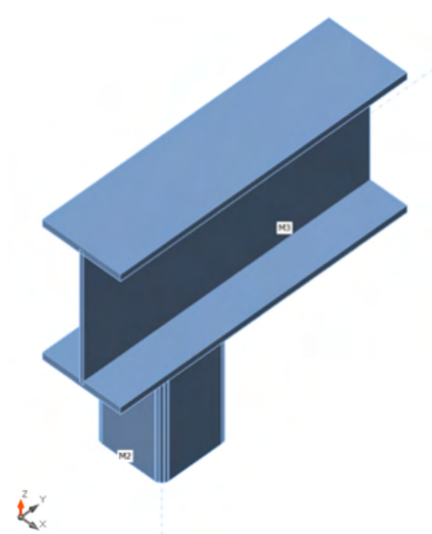
Steel	A36, A500, Gr. B, shaped, A992

Design

Name	CON1
Description	
Analysis	Stress, strain/ simplified loading
Design code	AISC - LRFD 2016

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]	Forces in
M2	1 - HSS(Imp)5X5X3/8	0.0	90.0	0.0	-10.000	0.000	0.000	Node
M3	3 - W(Imp)10X30	90.0	0.0	0.0	-6.000	0.000	17.122	Node



Cross-sections

Name	Material
1 - HSS(Imp)5X5X3/8	A500, Gr. B, shaped
3 - W(Imp)10X30	A992

Load effects (equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [lbf.ft]	My [lbf.ft]	Mz [lbf.ft]
LE1	M3	0.500	0.150	-5.000	0.00	14380.00	162.00
LE2	M3	-1.400	0.100	-1.900	0.00	0.20	0.20

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Welds	80.5 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

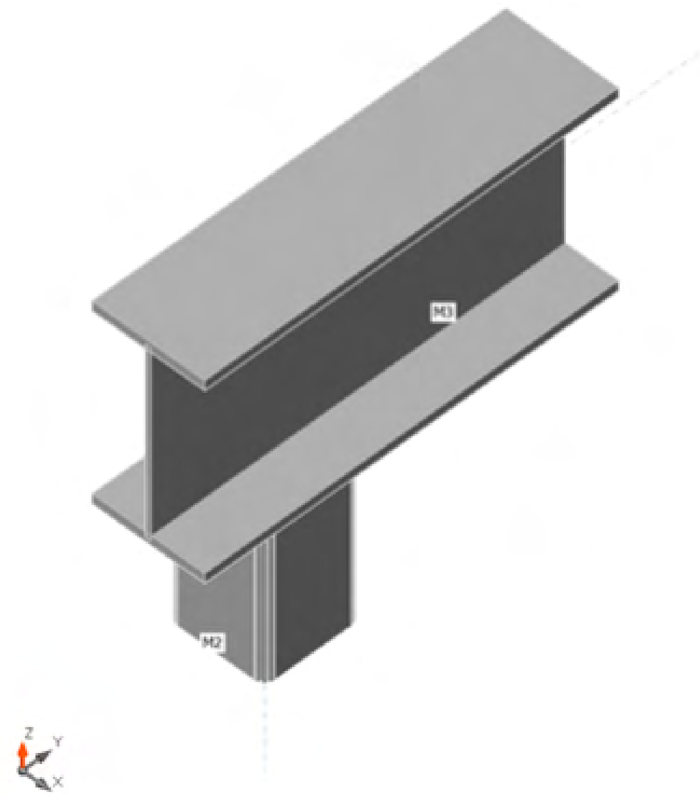
Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
M2	A500, Gr. B, shaped	46.0	3/8	LE1	30.5	0.0	0.0	OK
M3-bfl 1	A992	50.0	1/2	LE1	25.4	0.0	0.0	OK
M3-tfl 1	A992	50.0	1/2	LE1	4.1	0.0	0.0	OK
M3-w 1	A992	50.0	5/16	LE1	32.0	0.0	0.0	OK

Design data

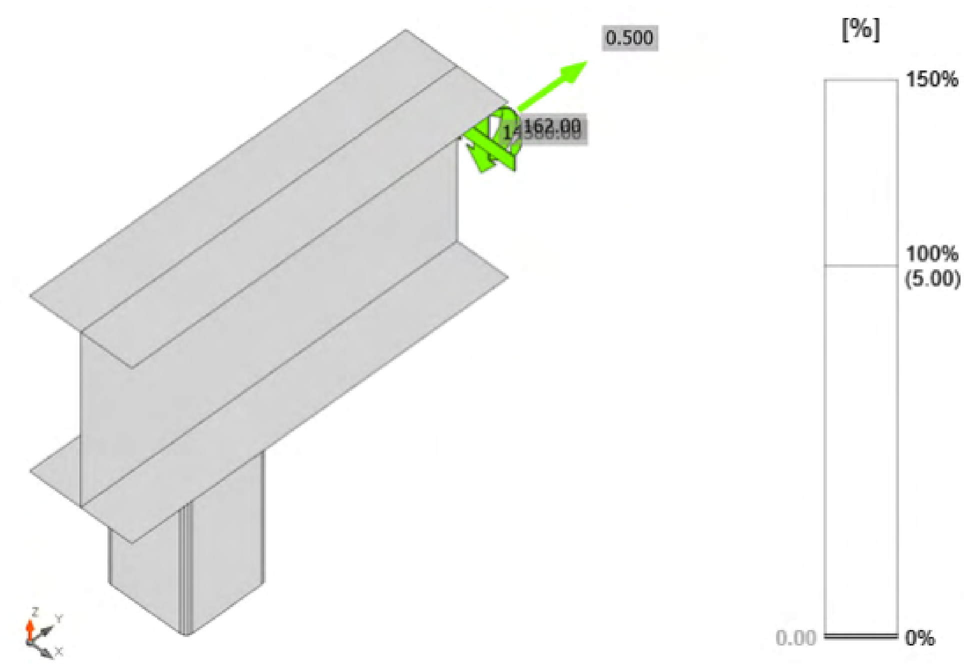
Material	f_y [ksi]	ε_{lim} [%]
A500, Gr. B, shaped	46.0	5.0
A992	50.0	5.0

Symbol explanation

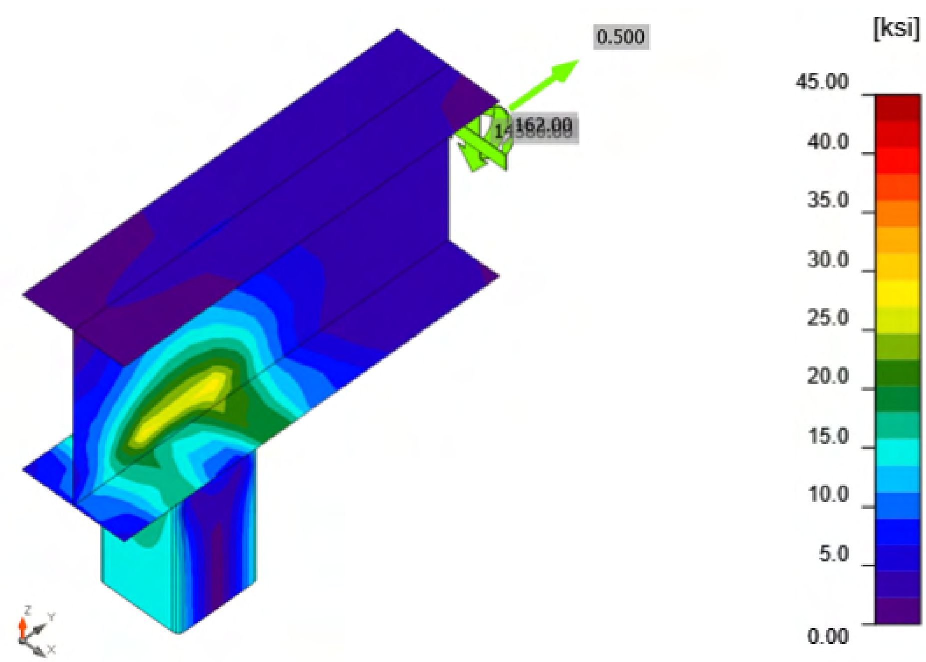
ε_{Pl}	Plastic strain
σc_{Ed}	Contact stress
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain



Overall check, LE1



Strain check, LE1



Equivalent stress, LE1

Weld sections

Item	Edge	Xu	T_h [in]	L_s [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Status
M3-bfl 1	M2	E70xx	◢1/4	◢3/8	17.782	0.391	LE1	3.695	4.588	80.5	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

Detailed result for M3-bfl 1 / M2

Weld resistance check (AISC 360-16: J2-4)

$$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = \quad 4.588 \quad \text{kip} \quad \geq \quad F_n = \quad 3.695 \quad \text{kip}$$

Where:

$F_{nw} = 62.4$ ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 79.2° – angle of loading measured from the weld longitudinal axis

$A_{we} = 0.098$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

2.6.3 HSS12X8X5/8 BEAM TO HSS5X5X3/8 COLUMN CONNECTION DESIGN

Project data

Project name	Purgavie
Project number	23-126
Author	DZ
Description	HSS12X8X5/8 + HSS5X5X3/8
Date	08.01.2024
Design code	AISC 360-16

Material

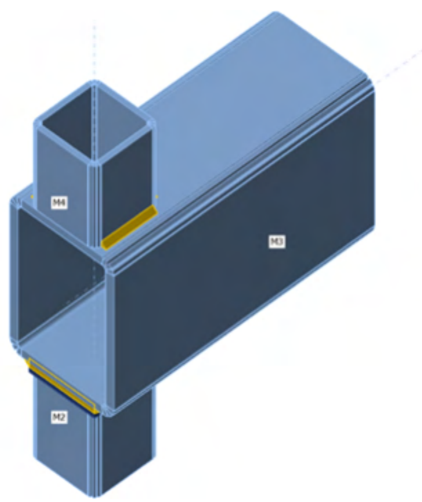
Steel	A36, A500, Gr. B, shaped, A992

Design

Name	CON1
Description	
Analysis	Stress, strain/ simplified loading
Design code	AISC - LRFD 2016

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]	Forces in
M2	1 - HSS(Imp)5X5X3/8	0.0	90.0	0.0	0.000	0.000	0.000	Node
M3	5 - HSS(Imp)12X8X5/8	90.0	0.0	0.0	-2.500	0.000	0.000	Node
M4	1 - HSS(Imp)5X5X3/8	0.0	-90.0	0.0	0.000	0.000	0.000	Node



Cross-sections

Name	Material
1 - HSS(Imp)5X5X3/8	A500, Gr. B, shaped
5 - HSS(Imp)12X8X5/8	A500, Gr. B, shaped

Load effects (equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [lbf.ft]	My [lbf.ft]	Mz [lbf.ft]
LE1	M3	20.800	1.150	-13.200	0.00	23860.00	651.00
	M4	-7.900	-16.780	9.800	0.00	-3440.00	-1527.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	3.4 < 5.0%	OK
Welds	98.0 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

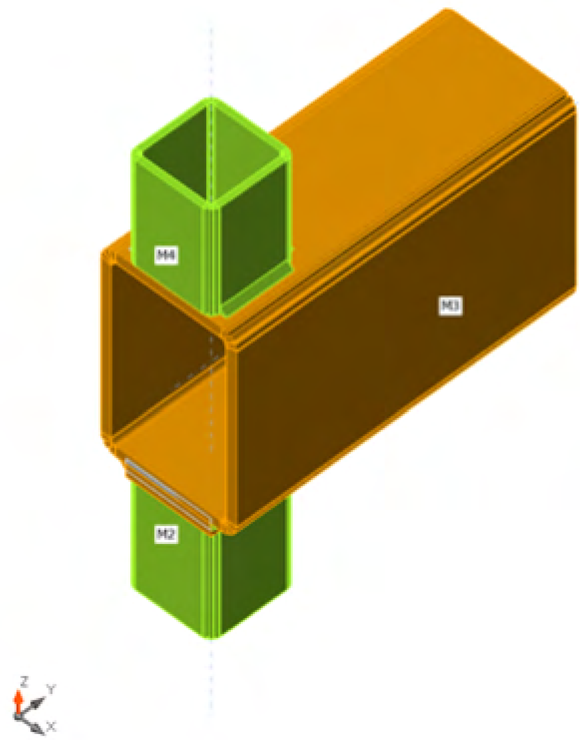
Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
M2	A500, Gr. B, shaped	46.0	3/8	LE1	41.5	0.3	0.0	OK
M3	A500, Gr. B, shaped	46.0	5/8	LE1	42.4	3.4	0.0	OK
M4	A500, Gr. B, shaped	46.0	3/8	LE1	34.0	0.0	0.0	OK
SP1	A36	36.0	3/4	LE1	33.3	3.1	0.0	OK

Design data

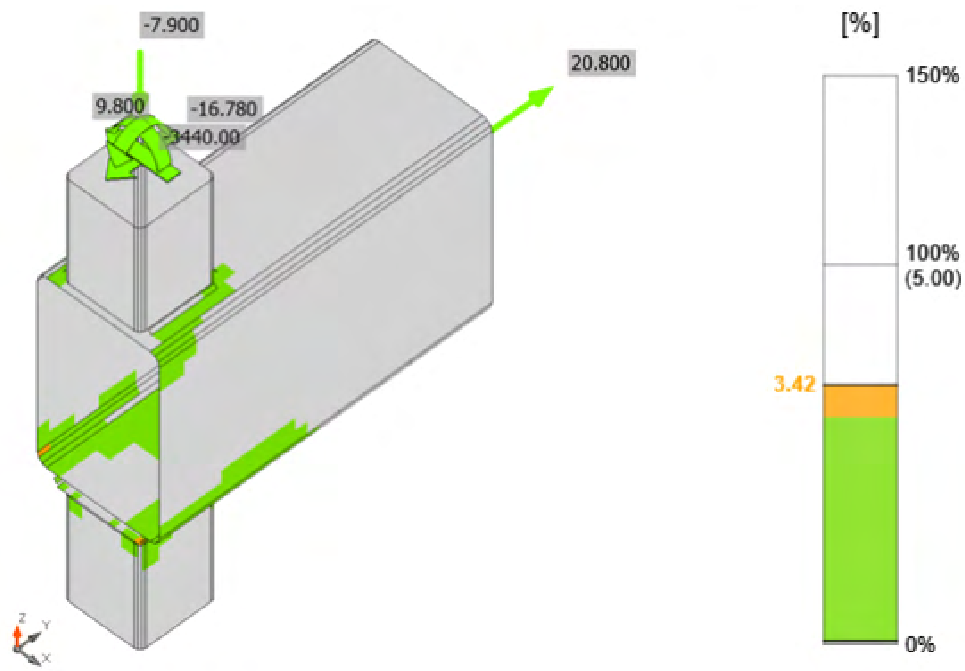
Material	f_y [ksi]	ε_{lim} [%]
A500, Gr. B, shaped	46.0	5.0
A36	36.0	5.0

Symbol explanation

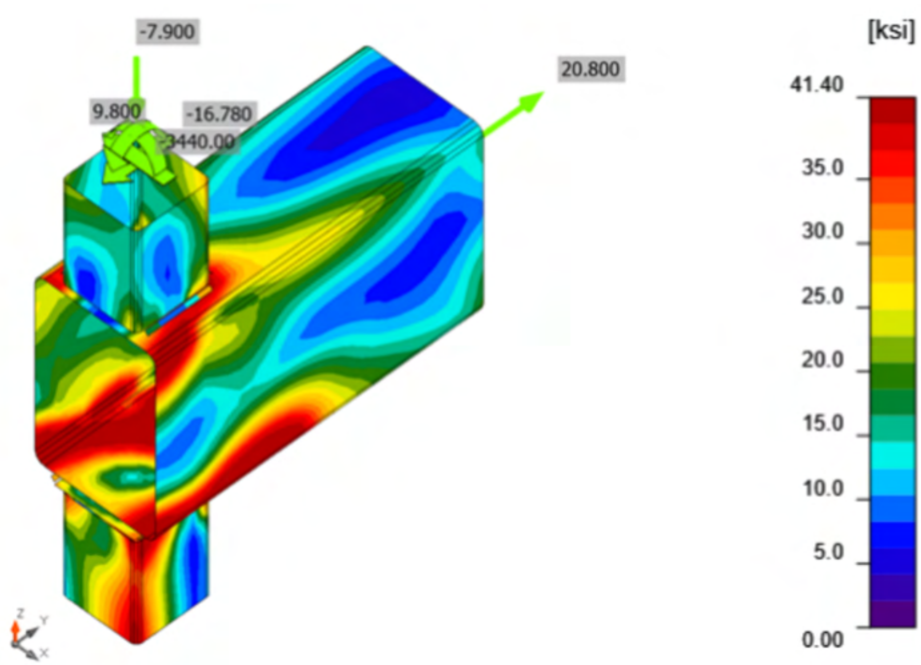
ε_{Pl}	Plastic strain
σc_{Ed}	Contact stress
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain



Overall check, LE1



Strain check, LE1



Equivalent stress, LE1

Weld sections

Item	Edge	Xu	T_h [in]	L_s [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Status
SP1	M2-w 2	E90xx	-	-	3.950	-	-	-	-	-	OK
SP1	M2-w 4	E90xx	-	-	3.950	-	-	-	-	-	OK
M3-w 1	SP1	E90xx	◢1/4	◢3/8	4.949	0.381	LE1	4.911	6.080	80.8	OK
M3-w 1	SP1	E90xx	◢1/4	◢3/8	4.974	0.383	LE1	5.959	6.080	98.0	OK
SP1	M2-w 3	E90xx	-	-	3.950	-	-	-	-	-	OK
SP1	M2-w 1	E90xx	-	-	3.950	-	-	-	-	-	OK
M3-w 1	SP1	E90xx	-	-	5.000	-	-	-	-	-	OK
M3-w 3	M4-w 4	E90xx	-	-	3.950	-	-	-	-	-	OK
M3-w 1	M3	E90xx	◢1/4	◢3/8	4.974	0.383	LE1	5.650	5.954	94.9	OK
M3-w 3	M4	E90xx	◢1/4	◢3/8	3.940	0.788	LE1	9.607	12.654	75.9	OK
M3-w 3	M4	E90xx	◢1/4	◢3/8	3.940	0.788	LE1	8.895	11.741	75.8	OK
M3-w 3	M4	E90xx	◢1/4	◢3/8	3.940	0.788	LE1	6.359	12.502	50.9	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

Detailed result for M3-w 1 / SP1

Weld resistance check (AISC 360-16: J2-4)

$$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = \quad 6.080 \text{ kip} \geq F_n = \quad 5.959 \text{ kip}$$

Where:

$F_{nw} =$ 79.9 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 90.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 76.5° – angle of loading measured from the weld longitudinal axis

$A_{we} =$ 0.101 in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

2.6.4 HSS5X5X3/8 BEAM TO HSS5X5X3/8 COLUMN CONNECTION DESIGN

Project data

Project name	Purgavie
Project number	23-126
Author	DZ
Description	HSS5X5X3/8+HSS5X5X3/8
Date	04.01.2024
Design code	AISC 360-16

Material

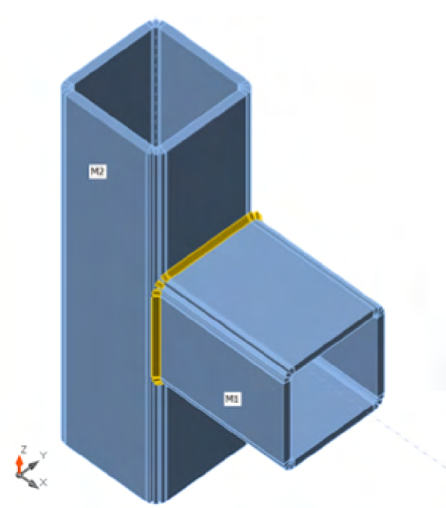
Steel	A500, Gr. B, shaped

Design

Name	CON1
Description	
Analysis	Stress, strain/ simplified loading
Design code	AISC - LRFD 2016

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]	Forces in
M1	1 - HSS(Imp)5X5X3/8	0.0	0.0	0.0	0.000	0.000	0.000	Node
M2	1 - HSS(Imp)5X5X3/8	0.0	90.0	0.0	0.000	0.000	0.000	Node



Cross-sections

Name	Material
1 - HSS(Imp)5X5X3/8	A500, Gr. B, shaped

Load effects (equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [lbf.ft]	My [lbf.ft]	Mz [lbf.ft]
LE1	M1	0.950	0.000	-4.360	0.00	8613.00	-600.00
LE2	M1	1.650	-0.445	6.480	0.00	-9218.00	-500.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Welds	80.6 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

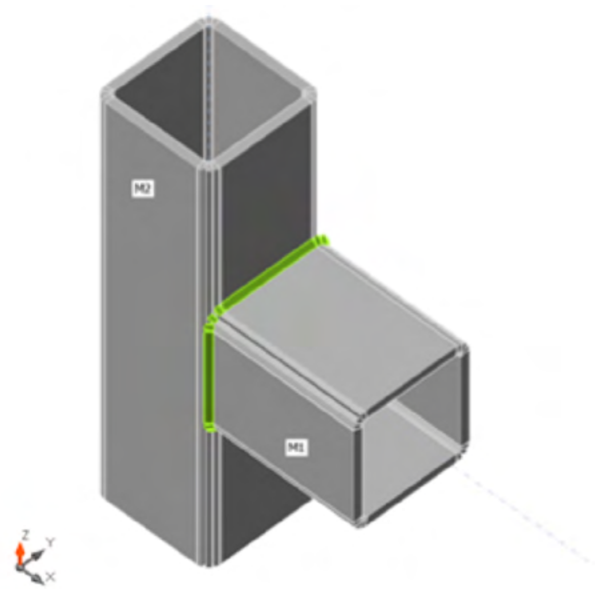
Name	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
M1	46.0	3/8	LE2	21.1	0.0	0.0	OK
M2	46.0	3/8	LE2	29.9	0.0	0.0	OK

Design data

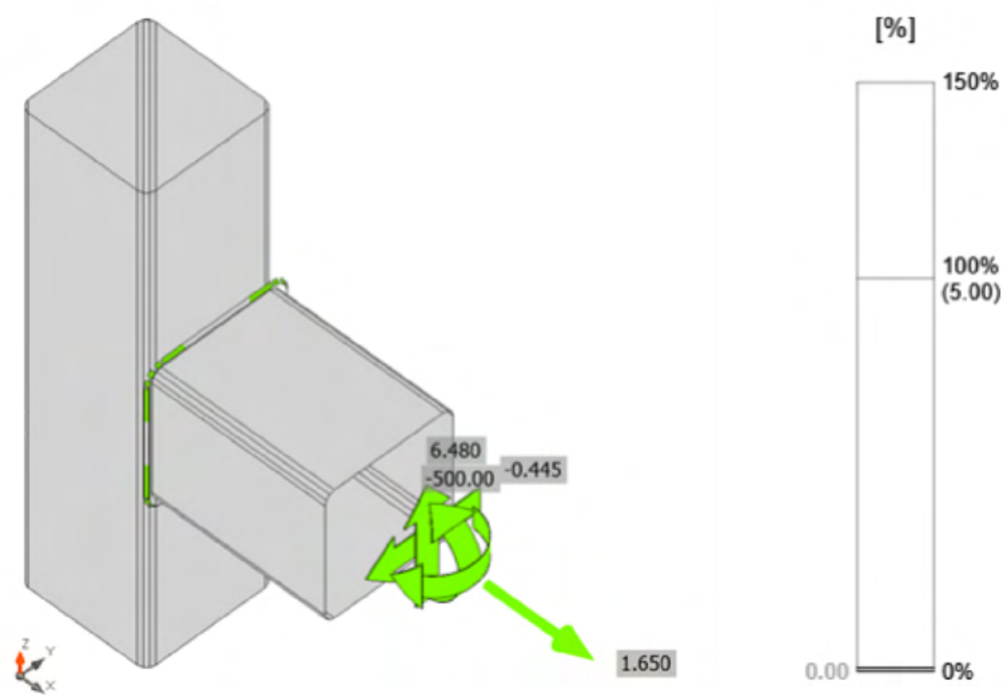
Material	f_y [ksi]	ε_{lim} [%]
A500, Gr. B, shaped	46.0	5.0

Symbol explanation

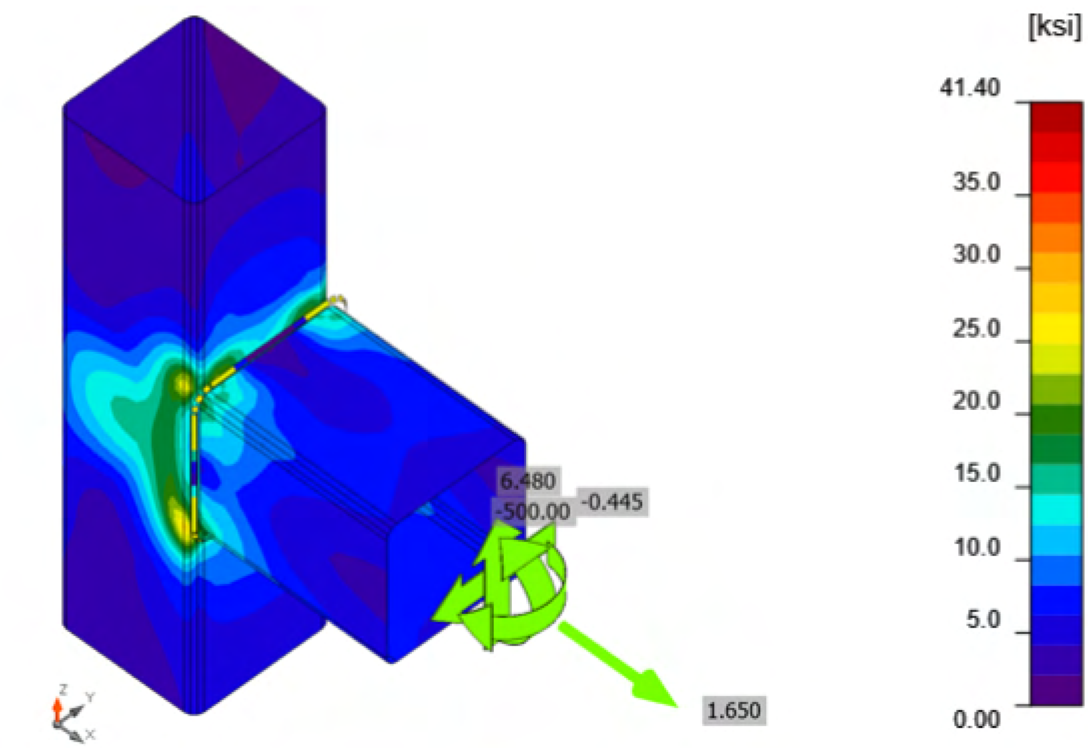
ε_{Pl}	Plastic strain
σc_{Ed}	Contact stress
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain



Overall check, LE2



Strain check, LE2



Equivalent stress, LE2

Weld sections

Item	Edge	Xu	T_h [in]	L_s [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Status
M2-w 3	M1	E70xx	◢1/8	◢3/16	17.821	0.175	LE2	0.928	1.152	80.6	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

Detailed result for M2-w 3 / M1

Weld resistance check (AISC 360-16: J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 1.152 kip ≥ $F_n =$ 0.928 kip

Where:

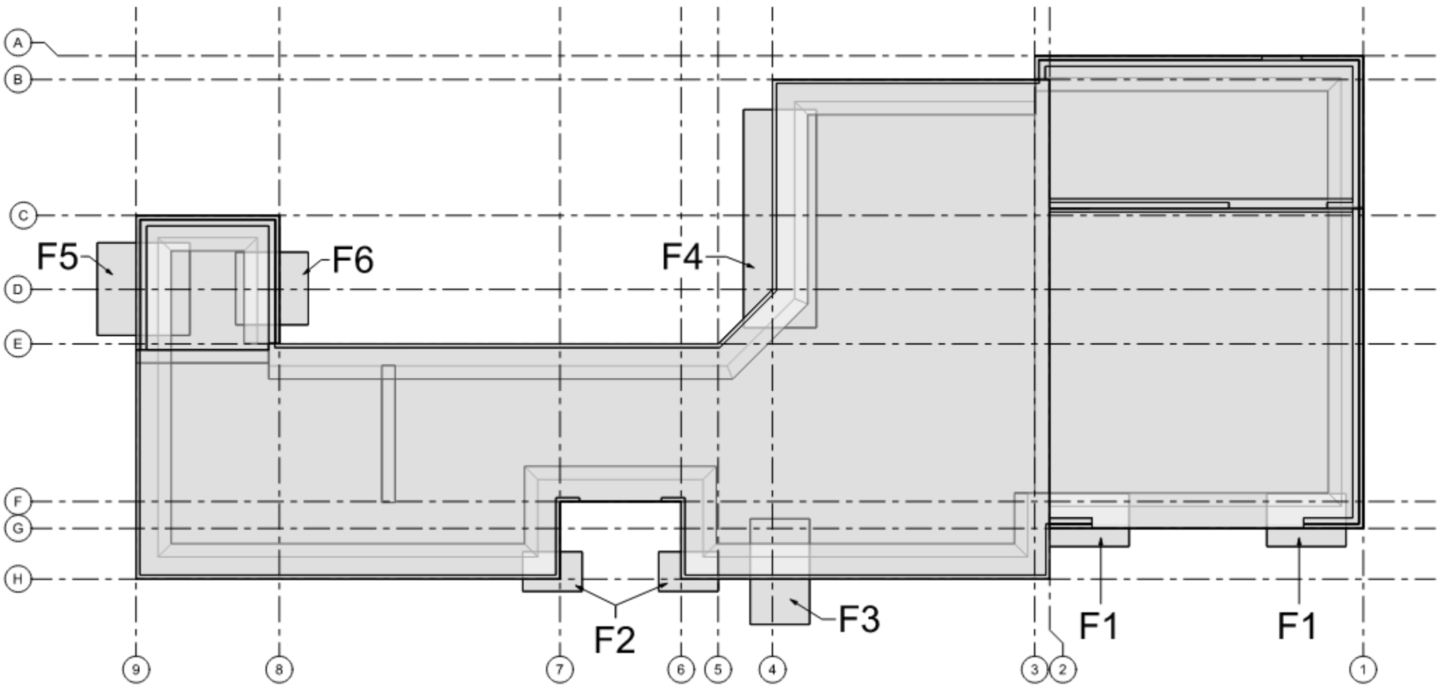
$F_{nw} =$ 63.0 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 88.3° – angle of loading measured from the weld longitudinal axis

$A_{we} =$ 0.024 in^2 – effective area of weld critical element

$\phi =$ 0.75 – resistance factor for welded connections

3.0 FOUNDATION ANALYSIS



FOUNDATION PLAN

3.1 FOOTINGS ANALYSIS & DESIGN
EXTERIOR WALL FOOTING ANALYSIS & DESIGN

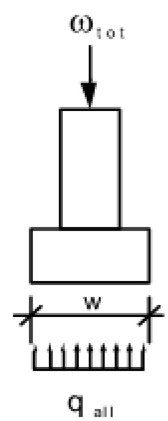
Footing **Exterior Wall Foundation**

Description of Variables:			DL	LL	
	1r = 1 ft of roof trib	Roof	20	20	psf
	2f = 2 ft of floor trib	Floor	15	40	psf
	3g = 3 ft of garage trib	Garage	0	0	psf
	4d = 4 ft of deck trib	Deck	0	0	psf
	5w = 5 ft of wall trib	Wall	45	0	psf

Conventional Footing Design Parameters:

Allowable Bearing Pressure 2000 psf

FT	TRIB (ft)		Wdl (plf)	Wll (plf)
ExWF	20.0	r	400	400
	11.3	f	170	453
	0.0	g	0	0
	0.0	d	0	0
	20.0	w	900	0
		TL	**1470**	**853**



w_{tot} 2323 plf

Try Footing Width, w = 18 inches

q_{bear} 1549 psf

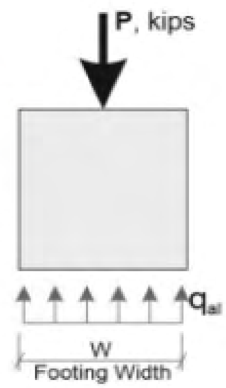
q_{allow} 2000 psf

q_{allow}/q_{bear} 1.29 Footing OK

EXTERIOR WALL FOOTING REINFORCEMENT ANALYSIS

Footing **Exterior wall Foundation**

Reactions on Footing		**D**	**L**	**E**	**W**	
Pt Load from	F1	1.5	1.0	0.0	0.0	kips
	Total	1.5	1.0	0.0	0.0	kips
				U =	**3.3**	kips
				P =	**2.4**	kips



Required Strength:	ASD:	
U = 1.2D + 1.6L	P = D + L	
U = 1.2D + 1.6L + 0.5W	P = D + 0.6W	P = D + 0.75[L+0.6W]
U = 1.2D + 1.6W + 0.5L	P = D + 0.7E	P = D + 0.75[L+0.7E]
U = 1.2D + 1.0E + 1.0L		

Conventional Footing Design Parameters:

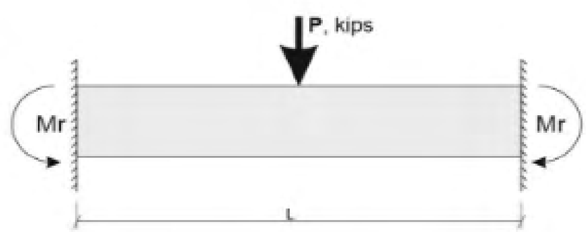
Allowable Bearing Pressure			2000	psf
Seismic or Wind Loading?	N	--->	2000	psf

Footing Width	18	inches
A_{req}	1.2	ft^2
L_{req}	**0.8**	ft

Model Foundation as Fixed/Fixed Member

Eqn. 1: $M_u = PL/8$

$M_u =$ **0.331** k-ft



Steel Reinforcing Design:

Steel Depth	9	in
Bar size (#)	5	bar
# of bars	2	

$øM_n$ =	23.5	kip-ft	*(Design Cap.)*
M_u=	0.331	kip-ft	*(From Eqn. 1)*
øMn/Mu =	**71.0**	OK	

Therefore use 18 inch x 12 inch footing, with (2) #5 bar top and bottom. See Details.

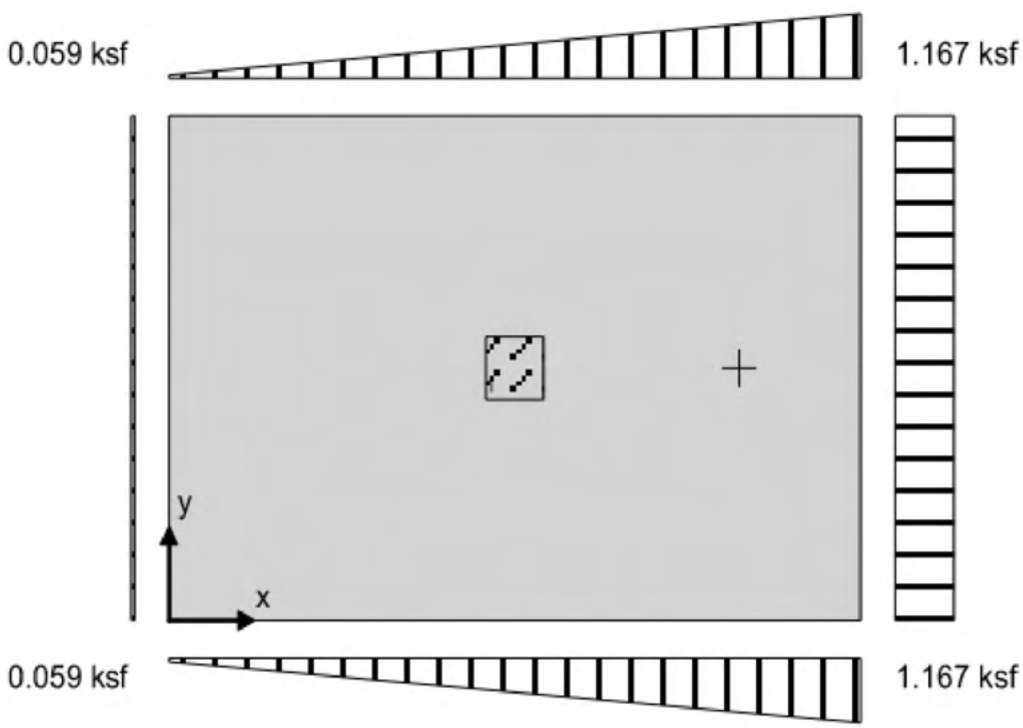
FOOTING F1 ANALYSIS & DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **6** ft
Width of foundation	L_y = **4** ft
Foundation area	$A = L_x \times L_y$ = **24** ft^2
Depth of foundation	h = **18** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **6.00** in
Width of column	l_{y1} = **6.00** in
position in x-axis	x_1 = **36.00** in
position in y-axis	y_1 = **24.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**

Foundation loads

Self weight	$F_{swt} = h * \gamma_{conc}$ = **225** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **2.8** kips
Live roof load in z	F_{Lrz1} = **3.2** kips
Wind load in z	F_{Wz1} = **-0.5** kips
Dead load moment in x	M_{Dx1} = **2.5** kip_ft
Live load moment in x	M_{Lx1} = **3.1** kip_ft
Wind load moment in x	M_{Wx1} = **18.8** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.313)
1.0D + 1.0L (0.378)
1.0D + 1.0Lr (0.380)
1.0D + 1.0S (0.313)
1.0D + 1.0R (0.313)
1.0D + 0.75L + 0.75Lr (0.411)
1.0D + 0.75L + 0.75S (0.361)
1.0D + 0.75L + 0.75R (0.361)
1.0D + 0.6W (0.544)
1.0D + 0.75L + 0.75Lr + 0.45W (0.583)
1.0D + 0.75L + 0.75S + 0.45W (0.534)
1.0D + 0.75L + 0.75R + 0.45W (0.534)

Combination 11 results: 1.0D + 0.75L + 0.75Lr + 0.45W

Forces on foundation

Force in z-axis	$F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1}$ = **14.7** kips

Moments on foundation

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1+M_{Dx1}) + \gamma_L * (M_{Lx1}) + \gamma_{Lr} * (F_{Lrz1} * x_1) + \gamma_W * (F_{Wz1} * x_1+M_{Wx1})$ = **57.4** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1) + \gamma_W * (F_{Wz1} * y_1)$ = **29.4** kip_ft

Uplift verification

Vertical force — F_{dz} = **14.713** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is 0

Overturning moment — $M_{OTx0} = \gamma_W * (F_{Wz1} * x_1)$ = **-0.62** kip_ft

Resisting moment — $M_{Rx0} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1+M_{Dx1}) + \gamma_L * (M_{Lx1}) + \gamma_{Lr} * (F_{Lrz1} * x_1) + \gamma_W * (M_{Wx1})$ = **58.04** kip_ft

Factor of safety — $abs(M_{Rx0} / M_{OTx0})$ = **93.470**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in x direction, moment about x is L_x

Overturning moment — $M_{OTxL} = \gamma_D * (M_{Dx1}) + \gamma_L * (M_{Lx1}) + \gamma_W * (F_{Wz1} * (x_1 - L_x)+M_{Wx1})$ = **13.91** kip_ft

Resisting moment — $M_{RxL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2)) + \gamma_D * (F_{Dz1} * (x_1 - L_x)) + \gamma_{Lr} * (F_{Lrz1} * (x_1 - L_x))$ = **-44.76** kip_ft

Factor of safety — $abs(M_{RxL} / M_{OTxL})$ = **3.219**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in y direction, moment about y is 0

Overturning moment — $M_{OTy0} = \gamma_W * (F_{Wz1} * y_1)$ = **-0.41** kip_ft

Resisting moment — $M_{Ry0} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1)$ = **29.84** kip_ft

Factor of safety — $abs(M_{Ry0} / M_{OTy0})$ = **72.077**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in y direction, moment about y is L_y

Overturning moment — $M_{OTyL} = \gamma_W * (F_{Wz1} * (y_1 - L_y))$ = **0.41** kip_ft

Resisting moment — $M_{RyL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2)) + \gamma_D * (F_{Dz1} * (y_1 - L_y)) + \gamma_{Lr} * (F_{Lrz1} * (y_1 - L_y))$ = **-29.84** kip_ft

Factor of safety — $abs(M_{RyL} / M_{OTyL})$ = **72.077**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **10.835** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) =$ **0.059** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) =$ **0.059** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) =$ **1.167** ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) =$ **1.167** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **0.059** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **1.167** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} =$ **2** ksf

$q_{max} / q_{allow} =$ **0.583**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete $f'_c =$ **3000** psi

Yield strength of reinforcement $f_y =$ **60000** psi

Cover to reinforcement $c_{nom} =$ **3** in

Concrete type Normal weight

Concrete modification factor $\lambda =$ **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.018)

1.2D + 1.6L + 0.5Lr (0.022)

1.2D + 1.6L + 0.5S (0.015)

1.2D + 1.6L + 0.5R (0.015)

1.2D + 1.0L + 1.6Lr (0.038)

1.2D + 1.0L + 1.6S (0.015)

1.2D + 1.0L + 1.6R (0.015)
1.2D + 1.6Lr + 0.5W (0.037)
1.2D + 1.6S + 0.5W (0.014)
1.2D + 1.6R + 0.5W (0.014)
1.2D + 1.0L + 0.5Lr + 1.0W (0.011)
1.2D + 1.0L + 0.5S + 1.0W (0.007)
1.2D + 1.0L + 0.5R + 1.0W (0.007)
0.9D + 1.0W (0.005)

Combination 11 results: 1.2D + 1.0L + 0.5Lr + 1.0W

Forces on foundation

Ultimate force in z-axis — $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1}$ = **16.2** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1 + M_{Dx1}) + \gamma_L * (M_{Lx1}) + \gamma_{Lr} * (F_{Lrz1} * x_1) + \gamma_W * (F_{Wz1} * x_1 + M_{Wx1})$ = **73.4** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1) + \gamma_W * (F_{Wz1} * y_1)$ = **32.3** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **18.486** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Length of bearing in x-axis — $L'_{xu} = \min(L_x, 3 * (L_x / 2 - \mathrm{abs}(e_{ux})))$ = **52.543** in

Pad base pressures

$q_{u1} = 0$ kN/m^2 = **0** ksf

$q_{u2} = 0$ kN/m^2 = **0** ksf

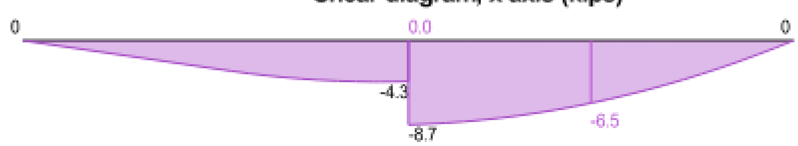
$q_{u3} = 2 * F_{uz} / (3 * L_y * (L_x / 2 - e_{ux}))$ = **1.846** ksf

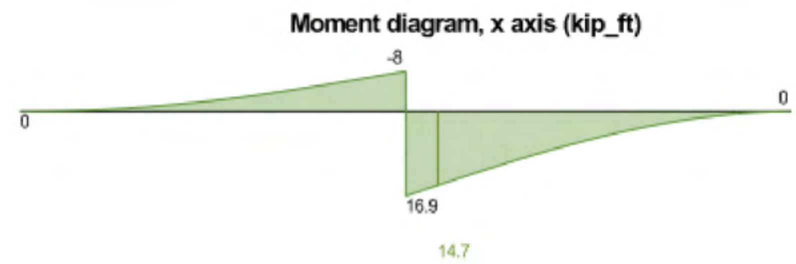
$q_{u4} = 2 * F_{uz} / (3 * L_y * (L_x / 2 - e_{ux}))$ = **1.846** ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **0** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **1.846** ksf

Shear diagram, x axis (kips)





Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **14.722** kip_ft
Tension reinforcement provided	7 No.5 bottom bars (6.8 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 * L_y * h = **1.555** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	d = h - c_{nom} - $\phi_{x.bot}$ / 2 = **14.688** in
Depth of compression block	a = $A_{sx.bot.prov}$ * f_y / (0.85 * f'_c * L_y) = **1.064** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	c = a / β_1 = **1.251** in
Strain in tensile reinforcement (8.3.3.1)	ε_t = 0.003 * d / c - 0.003 = **0.03221**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	M_n = $A_{sx.bot.prov}$ * f_y * (d - a / 2) = **153.589** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f * M_n = **138.23** kip_ft
	$M_{u.x.max}$ / ϕM_n = **0.107**
	PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment	$M_{u.x.min}$ = **-6.95** kip_ft
Tension reinforcement provided	7 No.5 top bars (6.8 in c/c)
Area of tension reinforcement provided	$A_{sx.top.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 * L_y * h = **1.555** in^2
	PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	d = h - c_{nom} - $\phi_{x.top}$ / 2 = **14.688** in
Depth of compression block	a = $A_{sx.top.prov}$ * f_y / (0.85 * f'_c * L_y) = **1.064** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	c = a / β_1 = **1.251** in
Strain in tensile reinforcement (8.3.3.1)	ε_t = 0.003 * d / c - 0.003 = **0.03221**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	M_n = $A_{sx.top.prov}$ * f_y * (d - a / 2) = **153.589** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f * M_n = **138.23** kip_ft
	abs($M_{u.x.min}$) / ϕM_n = **0.050**

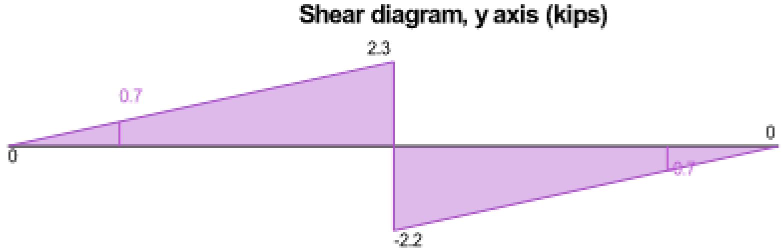
PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

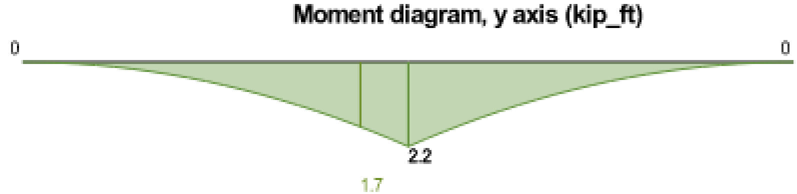
Ultimate shear force	$V_{u.x}$ = **6.484** kips
Depth to reinforcement	d_v = min(h - c_{nom} - $\phi_{x.bot}$ / 2,h - c_{nom} - $\phi_{x.top}$ / 2) = **14.688** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	V_n = 2 * λ * O(f'_c * 1 psi) * L_y * d_v = **77.229** kips
Design shear capacity	ϕV_n = ϕ_v * V_n = **57.922** kips
	$V_{u.x}$ / ϕV_n = **0.112**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **1.724** kip_ft
Tension reinforcement provided	8 No.5 bottom bars (9.3 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **2.48** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 * L_x * h = **2.333** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	d = h - c_{nom} - $\phi_{x.bot}$ - $\phi_{y.bot}$ / 2 = **14.063** in
Depth of compression block	a = $A_{sy.bot.prov}$ * f_y / (0.85 * f'$_c$ * L_x) = **0.810** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	c = a / β_1 = **0.953** in
Strain in tensile reinforcement (8.3.3.1)	ε_t = 0.003 * d / c - 0.003 = **0.04125**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	M_n = $A_{sy.bot.prov}$ * f_y * (d - a / 2) = **169.35** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f * M_n = **152.415** kip_ft
	$M_{u.y.max}$ / ϕM_n = **0.011**
	PASS - Design moment capacity exceeds ultimate moment load
Footing geometry factor (13.3.3.3)	β_f = L_x / L_y = **1.500**
Area of reinf. req. for uniform distribution (CRSI)	A_{sreq} = ($M_{u.y.max}$ / (ϕ_f * f_y * (d - a / 2))) * 2 * β_f / (β_f + 1) = **0.034** in^2
	PASS - Reinforcement can be distributed uniformly

One-way shear design, y direction

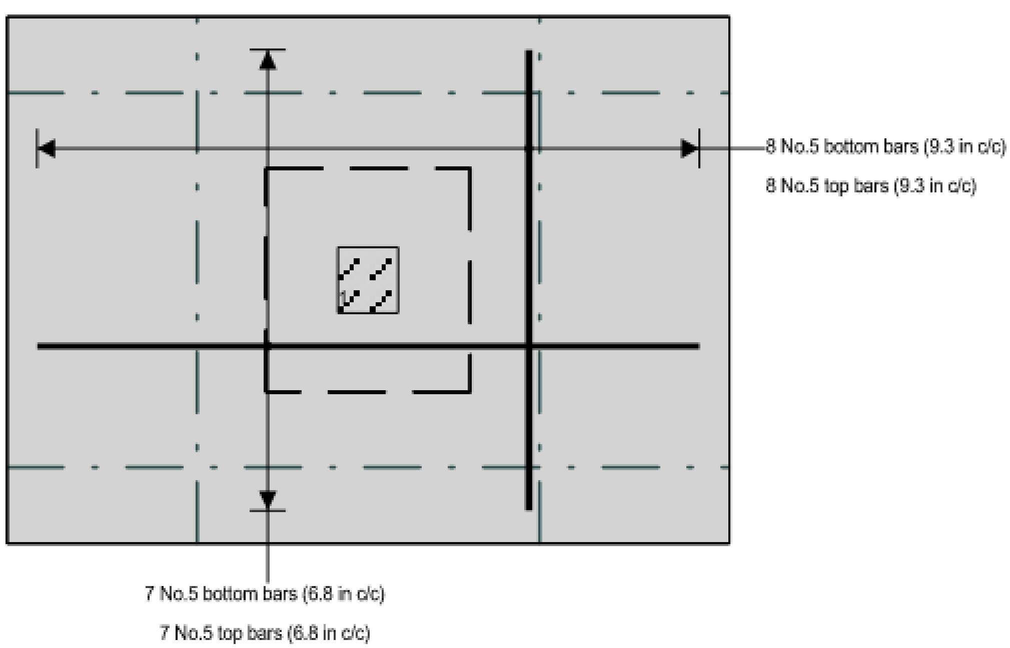
Ultimate shear force	$V_{u.y}$ = **0.651** kips
Depth to reinforcement	d_v = min(h - c_{nom} - $\phi_{x.bot}$ - $\phi_{y.bot}$ / 2,h - c_{nom} - $\phi_{y.top}$ / 2) = **14.063** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	V_n = 2 * λ * O(f'$_c$ * 1 psi) * L_x * d_v = **110.914** kips
Design shear capacity	ϕV_n = ϕ_v * V_n = **83.185** kips
	$V_{u.y}$ / ϕV_n = **0.008**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	d_{v2} = **14.375** in
Shear perimeter length (22.6.4)	l_{xp} = **20.375** in
Shear perimeter width (22.6.4)	l_{yp} = **20.375** in

Shear perimeter (22.6.4)	$b_o = 2 * (l_{x1} + d_{v2}) + 2 * (l_{y1} + d_{v2})$ = **81.500** in
Shear area	$A_p = l_{x,perim} * l_{y,perim}$ = **415.141** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} * l_{y1}$ = **379.141** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **1.265** ksf
Ultimate shear load	$F_{up} = \gamma_D * F_{Dz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} - q_{up.avg} * A_p$ = **2.201** kips
Ultimate shear stress from vertical load	$v_{ug} = max(F_{up} / (b_o * d_{v2}),0\ psi)$ = **1.879** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1}$ = **1.00**
Column location factor (22.6.5.3)	α_s =**40**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) * \lambda * O(f'_c * 1\ psi)$ = **328.634** psi
	$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * O(f'_c * 1\ psi)$ = **495.975** psi
	$v_{cpc} = 4 * \lambda * O(f'_c * 1\ psi)$ = **219.089** psi
	$v_{cp} = min(v_{cpa},v_{cpb},v_{cpc})$ = **219.089** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **219.089** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v * v_n$ = **164.317** psi
	$v_{ug} / \phi v_n$ = **0.011**

PASS - Design shear stress capacity exceeds ultimate shear stress load



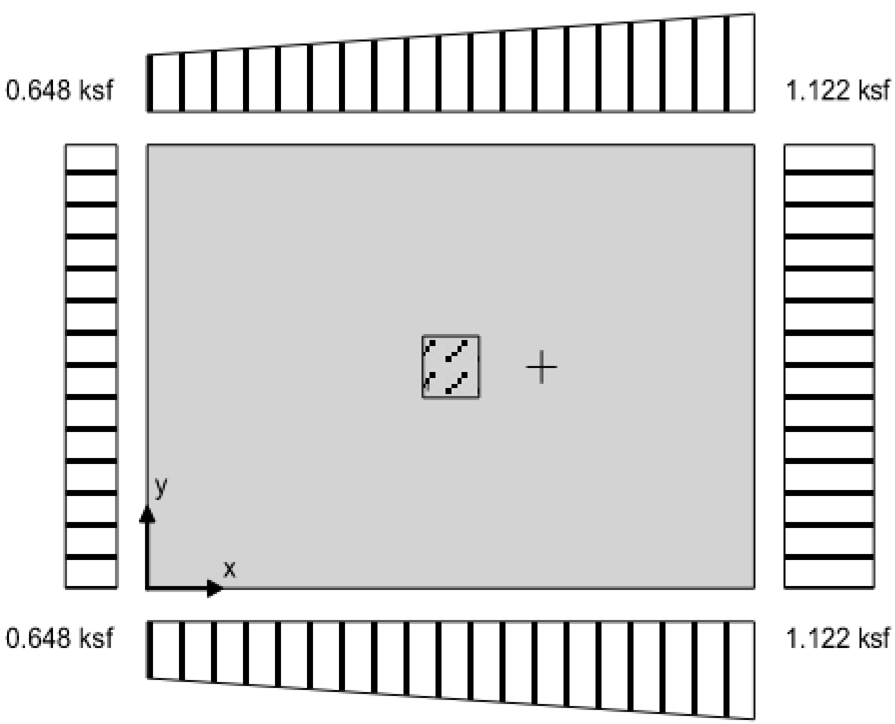
FOOTING F2 ANALYSIS & DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **4.5** ft
Width of foundation	L_y = **3** ft
Foundation area	$A = L_x \times L_y$ = **13.5** ft^2
Depth of foundation	h = **12** in
Depth of soil over foundation	h_{soil} = **12** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **5.00** in
Width of column	l_{y1} = **5.00** in
position in x-axis	x_1 = **27.00** in
position in y-axis	y_1 = **18.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Self weight	$F_{swt} = h * \gamma_{conc}$ = **150** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **6.0** kips
Live load in z	F_{Lz1} = **1.2** kips
Live roof load in z	F_{Lrz1} = **2.3** kips
Wind load in z	F_{Wz1} = **-1.9** kips
Dead load moment in x	M_{Dx1} = **2.4** kip_ft
Live load moment in x	M_{Lx1} = **-1.5** kip_ft
Wind load moment in x	M_{Wx1} = **2.4** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.476)
1.0D + 1.0L (0.444)
1.0D + 1.0Lr (0.561)
1.0D + 1.0S (0.476)
1.0D + 1.0R (0.476)
1.0D + 0.75L + 0.75Lr (0.516)
1.0D + 0.75L + 0.75S (0.452)
1.0D + 0.75L + 0.75R (0.452)
1.0D + 0.6W (0.505)
1.0D + 0.75L + 0.75Lr + 0.45W (0.538)
1.0D + 0.75L + 0.75S + 0.45W (0.474)
1.0D + 0.75L + 0.75R + 0.45W (0.474)
0.6D + 0.6W (0.418)

Combination 3 results: 1.0D + 1.0Lr

Forces on foundation

Force in z-axis	$F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_{Lr} * F_{Lrz1}$ = **11.9** kips

Moments on foundation

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1 + M_{Dx1}) + \gamma_{Lr} * (F_{Lrz1} * x_1) =$ **29.3** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1) =$ **17.9** kip_ft

Uplift verification

Vertical force — $F_{dz} =$ **11.945** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment — $M_{OTxL} = \gamma_D * (M_{Dx1}) =$ **2.4** kip_ft

Resisting moment — $M_{RxL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2)) + \gamma_D * (F_{Dz1} * (x_1 - L_x)) + \gamma_{Lr} * (F_{Lrz1} * (x_1 - L_x)) =$ **-26.88** kip_ft

Factor of safety — $abs(M_{RxL} / M_{OTxL}) =$ **11.198**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **2.411** in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) =$ **0.648** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) =$ **0.648** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) =$ **1.122** ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) =$ **1.122** ksf

Minimum base pressure — $q_{min} = min(q_1,q_2,q_3,q_4) =$ **0.648** ksf

Maximum base pressure — $q_{max} = max(q_1,q_2,q_3,q_4) =$ **1.122** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} =$ **2** ksf

$q_{max} / q_{allow} =$ **0.561**

PASS - Allowable bearing capacity exceeds design base pressure

---

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete	f'_c = **3000** psi
Yield strength of reinforcement	f_y = **60000** psi
Cover to reinforcement	c_{nom} = **3** in
Concrete type	Normal weight
Concrete modification factor	λ = **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.103)
1.2D + 1.6L + 0.5Lr (0.125)
1.2D + 1.6L + 0.5S (0.111)
1.2D + 1.6L + 0.5R (0.111)
1.2D + 1.0L + 1.6Lr (0.148)
1.2D + 1.0L + 1.6S (0.103)
1.2D + 1.0L + 1.6R (0.103)
1.2D + 1.6Lr + 0.5W (0.181)
1.2D + 1.6S + 0.5W (0.077)
1.2D + 1.6R + 0.5W (0.077)
1.2D + 1.0L + 0.5Lr + 1.0W (0.093)
1.2D + 1.0L + 0.5S + 1.0W (0.079)
1.2D + 1.0L + 0.5R + 1.0W (0.079)
0.9D + 1.0W (0.043)

Combination 8 results: 1.2D + 1.6Lr + 0.5W

Forces on foundation

Ultimate force in z-axis	$F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1}$ = **14.3** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0	$M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1+M_{Dx1}) + \gamma_{Lr} * (F_{Lrz1} * x_1) + \gamma_W * (F_{Wz1} * x_1+M_{Wx1})$ = **36.3** kip_ft
Ultimate moment in y-axis, about y is 0	$M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1) + \gamma_W * (F_{Wz1} * y_1)$ = **21.5** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2 =$ **3.423** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0** in

Pad base pressures

$q_{u1} = F_{uz} * (1 - 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y) =$ **0.657** ksf

$q_{u2} = F_{uz} * (1 - 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y) =$ **0.657** ksf

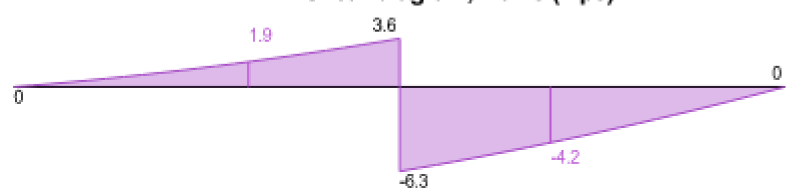
$q_{u3} = F_{uz} * (1 + 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y) =$ **1.463** ksf

$q_{u4} = F_{uz} * (1 + 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y) =$ **1.463** ksf

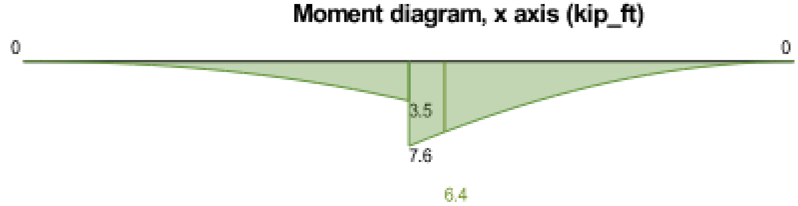
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) =$ **0.657** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) =$ **1.463** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max} =$ **6.357** kip_ft

Tension reinforcement provided — 3 No.5 bottom bars (14.6 in c/c)

Area of tension reinforcement provided — $A_{sx.bot.prov} =$ **0.93** in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 * L_y * h =$ **0.778** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 * h, 18 \text{ in}) =$ **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{x.bot} / 2 =$ **8.688** in

Depth of compression block — $a = A_{sx.bot.prov} * f_y / (0.85 * f'_c * L_y) =$ **0.608** in

Neutral axis factor — $\beta_1 =$ **0.85**

Depth to neutral axis — $c = a / \beta_1 =$ **0.715** in

Strain in tensile reinforcement (8.3.3.1) $\varepsilon_t = 0.003 * d / c - 0.003 =$ **0.03345**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity $M_n = A_{sx.bot.prov} * f_y * (d - a / 2) =$ **38.984** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f * M_n =$ **35.085** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.181**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x} =$ **4.175** kips

Depth to reinforcement $d_v = \min(h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2) =$ **8.062** in

Shear strength reduction factor $\phi_v =$ **0.75**

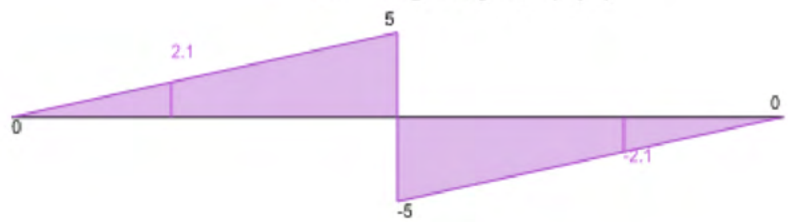
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 * \lambda * \sqrt{}(f'_c * 1 \text{ psi}) * L_y * d_v =$ **31.795** kips

Design shear capacity $\phi V_n = \phi_v * V_n =$ **23.846** kips

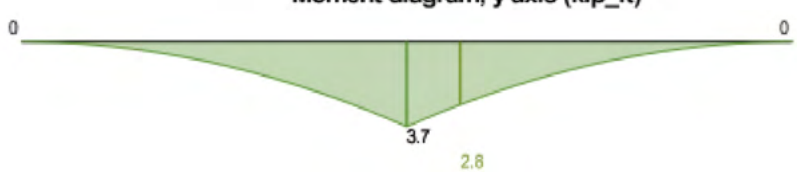
$V_{u.x} / \phi V_n =$ **0.175**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max} =$ **2.761** kip_ft

Tension reinforcement provided 4 No.5 bottom bars (15.7 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov} =$ **1.24** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 * L_x * h =$ **1.166** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	d = h - c_{nom} - $\phi_{x.bot}$ - $\phi_{y.bot}$ / 2 = **8.062** in
Depth of compression block	a = $A_{sy.bot.prov}$ * f_y / (0.85 * f'_c * L_x) = **0.540** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	c = a / β_1 = **0.636** in
Strain in tensile reinforcement (8.3.3.1)	ε_t = 0.003 * d / c - 0.003 = **0.03505**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	M_n = $A_{sy.bot.prov}$ * f_y * (d - a / 2) = **48.313** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f * M_n = **43.481** kip_ft
	$M_{u.y.max}$ / ϕM_n = **0.064**
	PASS - Design moment capacity exceeds ultimate moment load
Footing geometry factor (13.3.3.3)	β_f = L_x / L_y = **1.500**
Area of reinf. req. for uniform distribution (CRSI)	A_{sreq} = ($M_{u.y.max}$ / (ϕ_f * f_y * (d - a / 2))) * 2 * β_f / (β_f + 1) = **0.094** in^2
	PASS - Reinforcement can be distributed uniformly

One-way shear design, y direction

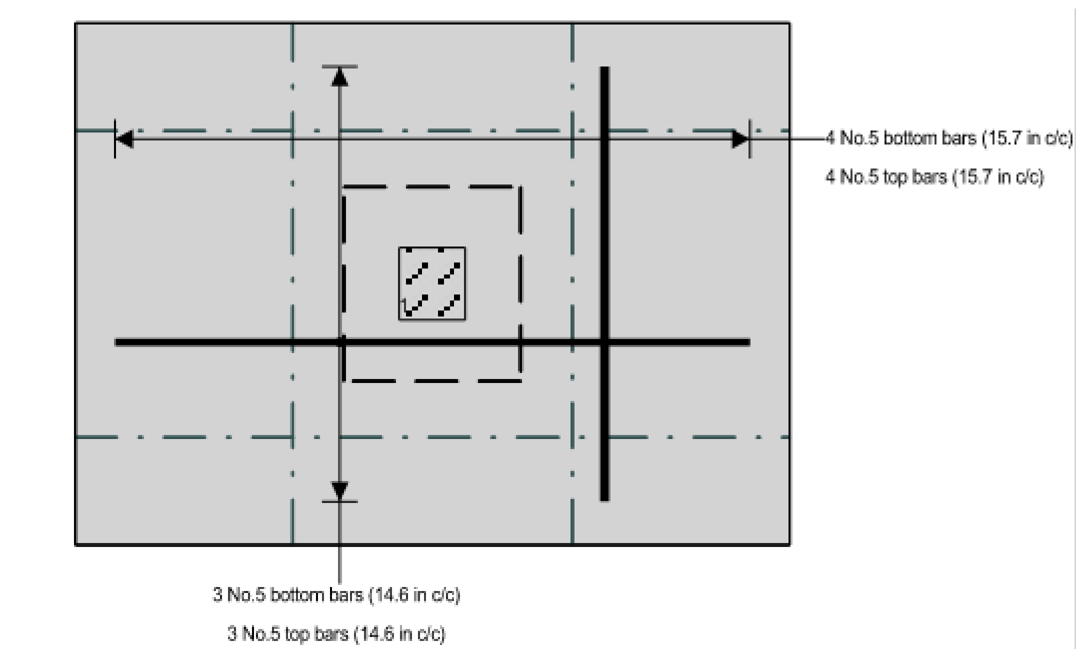
Ultimate shear force	$V_{u.y}$ = **2.052** kips
Depth to reinforcement	d_v = min(h - c_{nom} - $\phi_{y.bot}$ / 2,h - c_{nom} - $\phi_{y.top}$ / 2) = **8.688** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	V_n = 2 * λ * O(f'_c * 1 psi) * L_x * d_v = **51.39** kips
Design shear capacity	ϕV_n = ϕ_v * V_n = **38.543** kips
	$V_{u.y}$ / ϕV_n = **0.053**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	d_{v2} = **8.375** in
Shear perimeter length (22.6.4)	l_{xp} = **13.375** in
Shear perimeter width (22.6.4)	l_{yp} = **13.375** in
Shear perimeter (22.6.4)	b_o = 2 * (l_{x1} + d_{v2}) + 2 * (l_{y1} + d_{v2}) = **53.500** in
Shear area	A_p = $l_{x,perim}$ * $l_{y,perim}$ = **178.891** in^2
Surcharge loaded area	A_{sur} = A_p - l_{x1} * l_{y1} = **153.891** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **1.060** ksf
Ultimate shear load	F_{up} = γ_D * F_{Dz1} + γ_{Lr} * F_{Lrz1} + γ_W * F_{Wz1} + γ_D * A_p * F_{swt} + γ_D * A_{sur} * F_{soil} - $q_{up.avg}$ * A_p = **8.991** kips

Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o * d_{v2}),0\ psi) = \mathbf{20.067}$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} = \mathbf{1.00}$
Column location factor (22.6.5.3)	$\alpha_s = \mathbf{40}$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) * \lambda * O(f'_c * 1\ psi) = \mathbf{328.634}$ psi
	$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * O(f'_c * 1\ psi) = \mathbf{452.511}$ psi
	$v_{cpc} = 4 * \lambda * O(f'_c * 1\ psi) = \mathbf{219.089}$ psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{219.089}$ psi
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = \mathbf{219.089}$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v * v_n = \mathbf{164.317}$ psi
	$v_{ug} / \phi v_n = \mathbf{0.122}$

PASS - Design shear stress capacity exceeds ultimate shear stress load



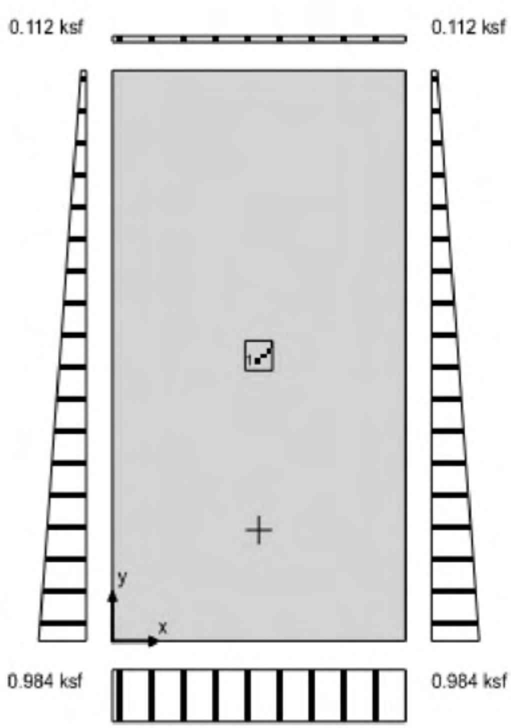
FOOTING F3 ANALYSIS & DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **4.5** ft
Width of foundation	L_y = **8** ft
Foundation area	$A = L_x \times L_y$ = **36** ft^2
Depth of foundation	h = **18** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **5.00** in
Width of column	l_{y1} = **5.00** in
position in x-axis	x_1 = **27.00** in
position in y-axis	y_1 = **48.00** in

Soil properties

Gross allowable bearing pressure — q_{allow_Gross} = **2** ksf

Density of soil — γ_{soil} = **120.0** lb/ft^3

Angle of internal friction — ϕ_b = **30.0** deg

Design base friction angle — δ_{bb} = **30.0** deg

Coefficient of base friction — $\tan(\delta_{bb})$ = **0.577**

Self weight — $F_{swt} = h * \gamma_{conc}$ = **225** psf

Soil weight — $F_{soil} = h_{soil} * \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z — F_{Dz1} = **9.9** kips

Live load in z — F_{Lz1} = **0.8** kips

Live roof load in z — F_{Lrz1} = **0.8** kips

Wind load in z — F_{Wz1} = **-7.9** kips

Dead load moment in y — M_{Dy1} = **-0.4** kip_ft

Live load moment in y — M_{Ly1} = **-0.2** kip_ft

Wind load moment in y — M_{Wy1} = **-34.2** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.344)
1.0D + 1.0L (0.357)
1.0D + 1.0Lr (0.355)
1.0D + 1.0S (0.344)
1.0D + 1.0R (0.344)
1.0D + 0.75L + 0.75Lr (0.362)
1.0D + 0.75L + 0.75S (0.354)
1.0D + 0.75L + 0.75R (0.354)
1.0D + 0.6W (0.492)
1.0D + 0.75L + 0.75Lr + 0.45W (0.473)
1.0D + 0.75L + 0.75S + 0.45W (0.465)
1.0D + 0.75L + 0.75R + 0.45W (0.465)

Combination 9 results: 1.0D + 0.6W

Forces on foundation

Force in z-axis — $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_W * F_{Wz1}$ = **19.7** kips

Moments on foundation

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_W * (F_{Wz1} * x_1)$ = **44.4** kip_ft

Moment in y-axis, about y is 0	$M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1+M_{Dy1}) + \gamma_W * (F_{Wz1} * y_1+M_{Wy1})$ = **58.0** kip_ft

Uplift verification

Vertical force	F_{dz} = **19.74** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is 0

Overturning moment	$M_{OTx0} = \gamma_W * (F_{Wz1} * x_1)$ = **-10.66** kip_ft
Resisting moment	$M_{Rx0} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1)$ = **55.08** kip_ft
Factor of safety	$abs(M_{Rx0} / M_{OTx0})$ = **5.165**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in x direction, moment about x is L_x

Overturning moment	$M_{OTxL} = \gamma_W * (F_{Wz1} * (x_1 - L_x))$ = **10.66** kip_ft
Resisting moment	$M_{RxL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2)) + \gamma_D * (F_{Dz1} * (x_1 - L_x))$ = **-55.08** kip_ft
Factor of safety	$abs(M_{RxL} / M_{OTxL})$ = **5.165**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in y direction, moment about y is 0

Overturning moment	$M_{OTy0} = \gamma_D * (M_{Dy1}) + \gamma_W * (F_{Wz1} * y_1+M_{Wy1})$ = **-39.88** kip_ft
Resisting moment	$M_{Ry0} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1)$ = **97.92** kip_ft
Factor of safety	$abs(M_{Ry0} / M_{OTy0})$ = **2.455**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in y direction, moment about y is L_y

Overturning moment	$M_{OTyL} = \gamma_W * (F_{Wz1} * (y_1 - L_y))$ = **18.96** kip_ft
Resisting moment	$M_{RyL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2)) + \gamma_D * (F_{Dz1} * (y_1 - L_y)+M_{Dy1}) + \gamma_W * (M_{Wy1})$ = **-118.84** kip_ft
Factor of safety	$abs(M_{RyL} / M_{OTyL})$ = **6.268**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis	$e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in
Eccentricity of base reaction in y-axis	$e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **-12.717** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.984** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.112** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) = \mathbf{0.984}$ ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) = \mathbf{0.112}$ ksf

Minimum base pressure $q_{min} = min(q_1,q_2,q_3,q_4) = \mathbf{0.112}$ ksf

Maximum base pressure $q_{max} = max(q_1,q_2,q_3,q_4) = \mathbf{0.984}$ ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} = \mathbf{2}$ ksf

$q_{max} / q_{allow} = \mathbf{0.492}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete $f'_c = \mathbf{3000}$ psi

Yield strength of reinforcement $f_y = \mathbf{60000}$ psi

Cover to reinforcement $c_{nom} = \mathbf{3}$ in

Concrete type Normal weight

Concrete modification factor $\lambda = \mathbf{1.00}$

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.106)

1.2D + 1.6L + 0.5Lr (0.105)

1.2D + 1.6L + 0.5S (0.102)

1.2D + 1.6L + 0.5R (0.102)

1.2D + 1.0L + 1.6Lr (0.108)

1.2D + 1.0L + 1.6S (0.098)

1.2D + 1.0L + 1.6R (0.098)

1.2D + 1.6Lr + 0.5W (0.137)

1.2D + 1.6S + 0.5W (0.127)

1.2D + 1.6R + 0.5W (0.127)

1.2D + 1.0L + 0.5Lr + 1.0W (0.011)

1.2D + 1.0L + 0.5S + 1.0W (0.172)

1.2D + 1.0L + 0.5R + 1.0W (0.172)

0.9D + 1.0W (0.167)

Combination 11 results: 1.2D + 1.0L + 0.5Lr + 1.0W

Forces on foundation

Ultimate force in z-axis — $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1} =$ **22.7** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_{Lr} * (F_{Lrz1} * x_1) + \gamma_W * (F_{Wz1} * x_1) =$ **51.0** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1+M_{Dy1}) + \gamma_L * (F_{Lz1} * y_1+M_{Ly1}) + \gamma_{Lr} * (F_{Lrz1} * y_1) + \gamma_W * (F_{Wz1} * y_1+M_{Wy1}) =$ **55.8** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **-18.458** in

Length of bearing in y-axis — $L'_{yu} = \min(L_y, 3 * (L_y / 2 - abs(e_{uy}))) =$ **88.625** in

Pad base pressures

$q_{u1} = 2 * F_{uz} / (3 * L_x * (L_y / 2 + e_{uy})) =$ **1.365** ksf

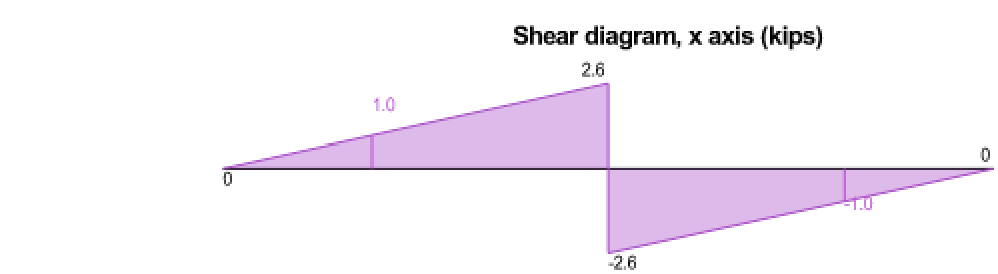
$q_{u2} = 0$ kN/m^2 = **0** ksf

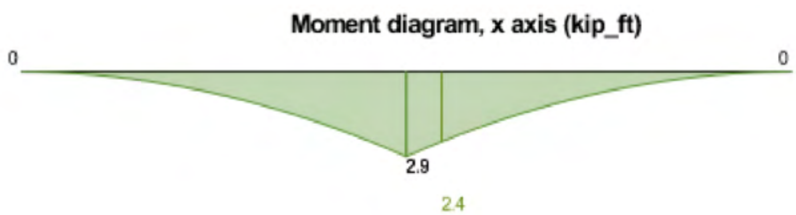
$q_{u3} = 2 * F_{uz} / (3 * L_x * (L_y / 2 + e_{uy})) =$ **1.365** ksf

$q_{u4} = 0$ kN/m^2 = **0** ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **0** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **1.365** ksf





Moment design, x direction, positive moment

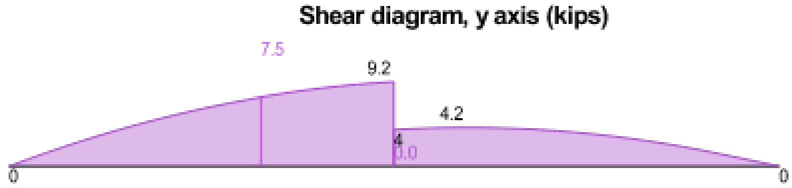
Ultimate bending moment — $M_{u.x.max} =$ **2.399** kip_ft

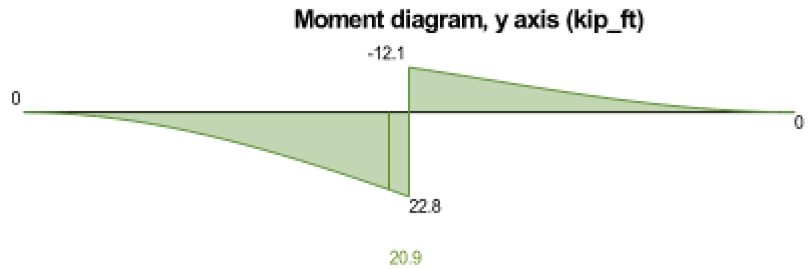
Tension reinforcement provided	11 No.5 bottom bars (8.9 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **3.41** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 * L_y * h$ = **3.11** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2$ = **14.063** in
Depth of compression block	$a = A_{sx.bot.prov} * f_y / (0.85 * f'_c * L_y)$ = **0.836** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.983** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 * d / c - 0.003$ = **0.03991**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sx.bot.prov} * f_y * (d - a / 2)$ = **232.641** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	$\phi M_n = \phi_f * M_n$ = **209.377** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.011**
	PASS - Design moment capacity exceeds ultimate moment load
Footing geometry factor (13.3.3.3)	$\beta_f = L_y / L_x$ = **1.778**
Area of reinf. req. for uniform distribution (CRSI)	$A_{sreq} = (M_{u.x.max} / (\phi_f * f_y * (d - a / 2))) * 2 * \beta_f / (\beta_f + 1)$ = **0.050** in^2
	PASS - Reinforcement can be distributed uniformly

One-way shear design, x direction

Ultimate shear force	$V_{u.x}$ = **1.001** kips
Depth to reinforcement	d_v = min(h - c_{nom} - $\phi_{y.bot}$ - $\phi_{x.bot}$ / 2,h - c_{nom} - $\phi_{x.top}$ / 2) = **14.063** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 * \lambda * \sqrt{f'_c * 1 \text{ psi}} * L_y * d_v$ = **147.885** kips
Design shear capacity	$\phi V_n = \phi_v * V_n$ = **110.914** kips
	$V_{u.x} / \phi V_n$ = **0.009**
	PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)





Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **20.867** kip_ft
Tension reinforcement provided	6 No.5 bottom bars (9.4 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **1.86** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 * L_x * h = **1.75** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	d = h - c_{nom} - $\phi_{y.bot}$ / 2 = **14.688** in
Depth of compression block	a = $A_{sy.bot.prov}$ * f_y / (0.85 * f'_c * L_x) = **0.810** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	c = a / β_1 = **0.953** in
Strain in tensile reinforcement (8.3.3.1)	ε_t = 0.003 * d / c - 0.003 = **0.04321**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	M_n = $A_{sy.bot.prov}$ * f_y * (d - a / 2) = **132.825** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f * M_n = **119.543** kip_ft
	$M_{u.y.max}$ / ϕM_n = **0.175**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, y direction, negative moment

Ultimate bending moment	$M_{u.y.min}$ = **-11.277** kip_ft
Tension reinforcement provided	6 No.5 top bars (9.4 in c/c)
Area of tension reinforcement provided	$A_{sy.top.prov}$ = **1.86** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 * L_x * h = **1.75** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom} - \phi_{y.top} / 2 =$ **14.688** in
Depth of compression block	$a = A_{sy.top.prov} * f_y / (0.85 * f'_c * L_x) =$ **0.810** in
Neutral axis factor	$\beta_1 =$ **0.85**
Depth to neutral axis	$c = a / \beta_1 =$ **0.953** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 * d / c - 0.003 =$ **0.04321**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sy.top.prov} * f_y * (d - a / 2) =$ **132.825** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f * M_n =$ **119.543** kip_ft

$abs(M_{u.y.min}) / \phi M_n =$ **0.094**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y} =$ **7.505** kips

Depth to reinforcement $d_v = \min(h - c_{nom} - \phi_{y.bot} / 2, h - c_{nom} - \phi_{y.top} / 2) =$ **14.688** in

Shear strength reduction factor $\phi_v =$ **0.75**

Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 * \lambda * O(f'_c * 1 \text{ psi}) * L_x * d_v =$ **86.882** kips

Design shear capacity $\phi V_n = \phi_v * V_n =$ **65.162** kips

$V_{u.y} / \phi V_n =$ **0.115**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement $d_{v2} =$ **14.375** in

Shear perimeter length (22.6.4) $l_{xp} =$ **36.688** in

Shear perimeter width (22.6.4) $l_{yp} =$ **57.688** in

Shear perimeter (22.6.4) $b_o = l_{x.perim} + l_{y.perim} =$ **94.375** in

Shear area $A_p = l_{x.perim} * l_{y.perim} =$ **2116.410** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} * l_{y1} =$ **2091.410** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg} =$ **-0.558** ksf

Ultimate shear load $F_{up} = \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} - q_{up.avg} * A_p =$ **20.482** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o * d_{v2}), 0 \text{ psi}) =$ **15.097** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3) $\alpha_s =$**20**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) * \lambda * O(f'_c * 1 \text{ psi}) =$ **328.634** psi

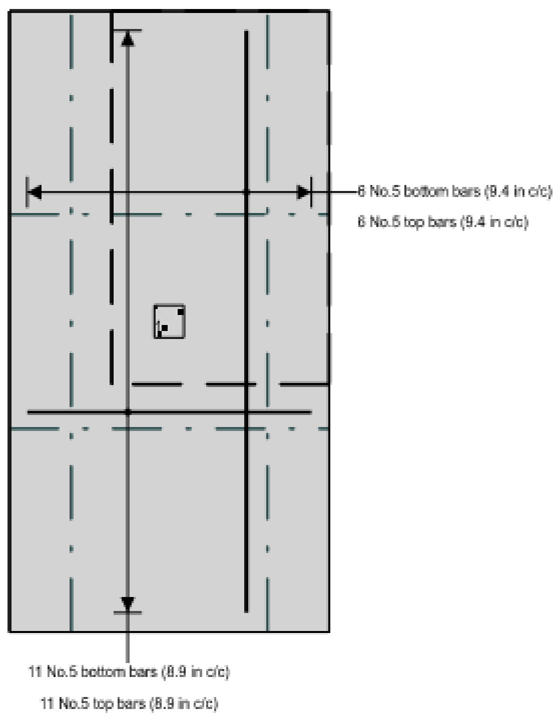
$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * O(f'_c * 1 \text{ psi}) =$ **276.400** psi

$v_{cpc} = 4 * \lambda * O(f'_c * 1 \text{ psi}) =$ **219.089** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **219.089** psi

Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **219.089** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v * v_n$ = **164.317** psi
	$v_{ug} / \phi v_n$ = **0.092**

PASS - Design shear stress capacity exceeds ultimate shear stress load



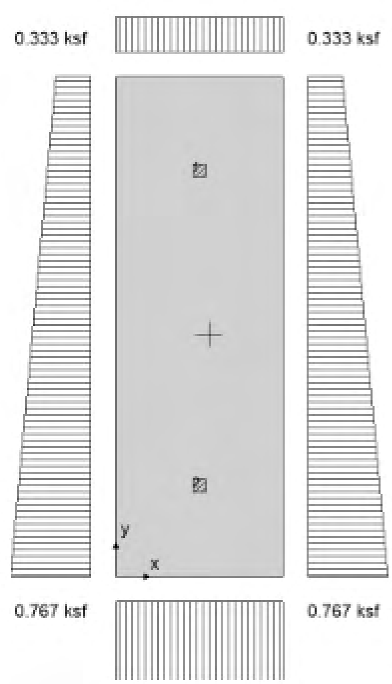
FOOTING F4 ANALYSIS & DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **5.5** ft
Width of foundation	L_y = **16.5** ft
Foundation area	$A = L_x \times L_y$ = **90.75** ft^2
Depth of foundation	h = **18** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **5.00** in
Width of column	l_{y1} = **5.00** in
position in x-axis	x_1 = **33.00** in
position in y-axis	y_1 = **161.00** in

Column no.2 details

Length of column	l_{x2} = **5.00** in
Width of column	l_{y2} = **5.00** in
position in x-axis	x_2 = **33.00** in
position in y-axis	y_2 = **36.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Self weight	$F_{swt} = h * \gamma_{conc}$ = **225** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **180** ps

Column no.1 loads

Dead load in z	F_{Dz1} = **1.1** kips
Live load in z	F_{Lz1} = **0.2** kips
Live roof load in z	F_{Lrz1} = **0.2** kips
Wind load in z	F_{Wz1} = **-1.1** kips
Wind load moment in x	M_{Wx1} = **8.0** kip_ft
Wind load moment in y	M_{Wy1} = **8.2** kip_ft

Column no.2 loads

Dead load in z	F_{Dz2} = **8.4** kips
Live load in z	F_{Lz2} = **2.4** kips
Live roof load in z	F_{Lrz2} = **2.1** kips
Wind load in z	F_{Wz2} = **-2.6** kips
Wind load moment in x	M_{Wx2} = **3.7** kip_ft
Wind load moment in y	M_{Wy2} = **8.9** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.332)
1.0D + 1.0L (0.368)
1.0D + 1.0Lr (0.364)
1.0D + 1.0S (0.332)
1.0D + 1.0R (0.332)
1.0D + 0.75L + 0.75Lr (0.384)
1.0D + 0.75L + 0.75S (0.359)
1.0D + 0.75L + 0.75R (0.359)
1.0D + 0.6W (0.331)

1.0D + 0.75L + 0.75Lr + 0.45W (0.383)
1.0D + 0.75L + 0.75S + 0.45W (0.359)
1.0D + 0.75L + 0.75R + 0.45W (0.359)
0.6D + 0.6W (0.199)

Combination 6 results: 1.0D + 0.75L + 0.75Lr

Forces on foundation

Force in z-axis — $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_D * F_{Dz2} + \gamma_L * F_{Lz2} + \gamma_{Lr} * F_{Lrz2}$ = **49.9** kips

Moments on foundation

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_{Lr} * (F_{Lrz1} * x_1) + \gamma_D * (F_{Dz2} * x_2) + \gamma_L * (F_{Lz2} * x_2) + \gamma_{Lr} * (F_{Lrz2} * x_2)$ = **137.3** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1) + \gamma_D * (F_{Dz2} * y_2) + \gamma_L * (F_{Lz2} * y_2) + \gamma_{Lr} * (F_{Lrz2} * y_2)$ = **357.6** kip_ft

Uplift verification

Vertical force — F_{dz} = **49.921** kips

PASS - Foundation is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **-13.037** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.767** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.333** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.767** ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.333** ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.333** ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **0.767** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross}$ = **2** ksf

q_{max} / q_{allow} = **0.384**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete	f'_c = **3000** psi
Yield strength of reinforcement	f_y = **60000** psi
Cover to reinforcement	c_{nom} = **3** in
Concrete type	Normal weight
Concrete modification factor	λ = **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.061)
1.2D + 1.6L + 0.5Lr (0.077)
1.2D + 1.6L + 0.5S (0.071)
1.2D + 1.6L + 0.5R (0.071)
1.2D + 1.0L + 1.6Lr (0.081)
1.2D + 1.0L + 1.6S (0.064)
1.2D + 1.0L + 1.6R (0.064)
1.2D + 1.6Lr + 0.5W (0.078)
1.2D + 1.6S + 0.5W (0.064)
1.2D + 1.6R + 0.5W (0.064)
1.2D + 1.0L + 0.5Lr + 1.0W (0.099)
1.2D + 1.0L + 0.5S + 1.0W (0.095)
1.2D + 1.0L + 0.5R + 1.0W (0.095)

0.9D + 1.0W (0.074)

Combination 11 results: 1.2D + 1.0L + 0.5Lr + 1.0W

Forces on foundation

Ultimate force in z-axis

$F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1} + \gamma_D * F_{Dz2} + \gamma_L * F_{Lz2} + \gamma_{Lr} * F_{Lrz2} + \gamma_W * F_{Wz2}$ = **55.5** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0

$M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_{Lr} * (F_{Lrz1} * x_1) + \gamma_W * (F_{Wz1} * x_1 + M_{Wx1}) + \gamma_D * (F_{Dz2} * x_2) + \gamma_L * (F_{Lz2} * x_2) + \gamma_{Lr} * (F_{Lrz2} * x_2) + \gamma_W * (F_{Wz2} * x_2 + M_{Wx2})$ = **164.4** kip_ft

Ultimate moment in y-axis, about y is 0

$M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1) + \gamma_W * (F_{Wz1} * y_1 + M_{Wy1}) + \gamma_D * (F_{Dz2} * y_2) + \gamma_L * (F_{Lz2} * y_2) + \gamma_{Lr} * (F_{Lrz2} * y_2) + \gamma_W * (F_{Wz2} * y_2 + M_{Wy2})$ = **420.9** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{2.547}$ in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{-7.976}$ in

Pad base pressures

$q_{u1} = F_{uz} * (1 - 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y) = \mathbf{0.618}$ ksf

$q_{u2} = F_{uz} * (1 - 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y) = \mathbf{0.322}$ ksf

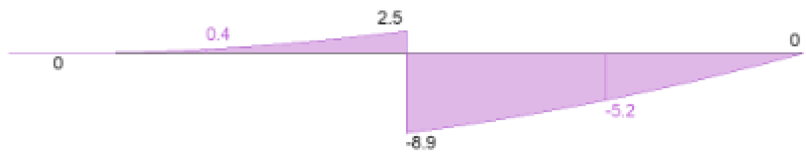
$q_{u3} = F_{uz} * (1 + 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y) = \mathbf{0.901}$ ksf

$q_{u4} = F_{uz} * (1 + 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y) = \mathbf{0.605}$ ksf

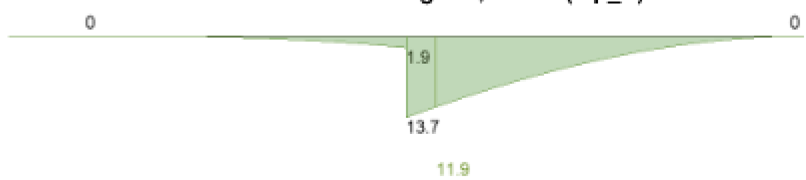
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{0.322}$ ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{0.901}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max} = \mathbf{11.909}$ kip_ft

Tension reinforcement provided — 21 No.5 bottom bars (9.5 in c/c)

Area of tension reinforcement provided — $A_{sx.bot.prov} = \mathbf{6.51}$ in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 * L_y * h = \mathbf{6.415}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 * h, 18 \text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2 = \mathbf{14.063}$ in

Depth of compression block — $a = A_{sx.bot.prov} * f_y / (0.85 * f'_c * L_y) = \mathbf{0.774}$ in

Neutral axis factor — $\beta_1 = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{0.910}$ in

Strain in tensile reinforcement (8.3.3.1) — $\varepsilon_t = 0.003 * d / c - 0.003 = \mathbf{0.04335}$

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity $M_n = A_{sx.bot.prov} * f_y * (d - a / 2) =$ **445.144** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f * M_n =$ **400.629** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.030**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment $M_{u.x.min} =$ **-0.017** kip_ft

Tension reinforcement provided 21 No.5 top bars (9.5 in c/c)

Area of tension reinforcement provided $A_{sx.top.prov} =$ **6.51** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 * L_y * h =$ **6.415** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 * h, 18 \text{ in}) =$ **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{y.top} - \phi_{x.top} / 2 =$ **14.063** in

Depth of compression block $a = A_{sx.top.prov} * f_y / (0.85 * f'_c * L_y) =$ **0.774** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **0.910** in

Strain in tensile reinforcement (8.3.3.1) $\varepsilon_t = 0.003 * d / c - 0.003 =$ **0.04335**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity $M_n = A_{sx.top.prov} * f_y * (d - a / 2) =$ **445.144** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f * M_n =$ **400.629** kip_ft

$\mathrm{abs}(M_{u.x.min}) / \phi M_n =$ **0.000**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x} =$ **5.239** kips

Depth to reinforcement $d_v = \min(h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2) =$ **14.063** in

Shear strength reduction factor $\phi_v =$ **0.75**

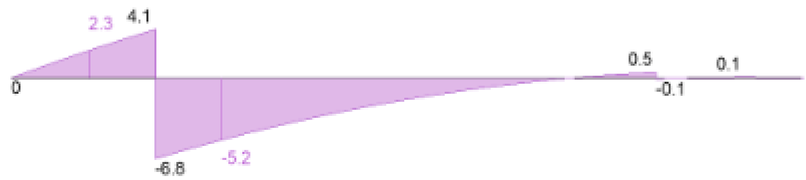
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 * \lambda * \sqrt{f'_c * 1 \text{ psi}} * L_y * d_v =$ **305.013** kips

Design shear capacity $\phi V_n = \phi_v * V_n =$ **228.76** kips

$V_{u.x} / \phi V_n =$ **0.023**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)

Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **13.837** kip_ft
Tension reinforcement provided	7 No.5 bottom bars (9.8 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 * L_x * h$ = **2.138** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{y.bot} / 2$ = **14.688** in
Depth of compression block	$a = A_{sy.bot.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.774** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.910** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 * d / c - 0.003$ = **0.04541**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sy.bot.prov} * f_y * (d - a / 2)$ = **155.162** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f * M_n$ = **139.646** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.099**
	PASS - Design moment capacity exceeds ultimate moment load

Moment design, y direction, negative moment

Ultimate bending moment	$M_{u.y.min}$ = **-8.753** kip_ft
Tension reinforcement provided	7 No.5 top bars (9.8 in c/c)
Area of tension reinforcement provided	$A_{sy.top.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 * L_x * h$ = **2.138** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 * h, 18 \text{ in})$ = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{y.top} / 2$ = **14.688** in
Depth of compression block	$a = A_{sy.top.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.774** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.910** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 * d / c - 0.003$ = **0.04541**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sy.top.prov} * f_y * (d - a / 2)$ = **155.162** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f * M_n$ = **139.646** kip_ft
	$\text{abs}(M_{u.y.min}) / \phi M_n$ = **0.063**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **5.192** kips
Depth to reinforcement	$d_v = \min(h - c_{nom} - \phi_{y.bot} / 2, h - c_{nom} - \phi_{y.top} / 2)$ = **14.688** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 * \lambda * O(f'_c * 1 \text{ psi}) * L_x * d_v$ = **106.19** kips
Design shear capacity	$\phi V_n = \phi_v * V_n$ = **79.642** kips
	$V_{u.y} / \phi V_n$ = **0.065**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	d_{v2} = **14.375** in
Shear perimeter length (22.6.4)	l_{xp} = **19.375** in
Shear perimeter width (22.6.4)	l_{yp} = **46.688** in
Shear perimeter (22.6.4)	$b_o = l_{x,perim} + 2 * l_{y,perim}$ = **112.750** in
Shear area	$A_p = l_{x,perim} * l_{y,perim}$ = **904.570** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} * l_{y1}$ = **879.570** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **0.429** ksf
Ultimate shear load	$F_{up} = \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} - q_{up.avg} * A_p$ = **0.878** kips

Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o * d_{v2}), 0\ psi) =$ **0.542** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (22.6.5.3)	$\alpha_s =$**30**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) * \lambda * O(f'_c * 1\ psi) =$ **328.634** psi
	$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * O(f'_c * 1\ psi) =$ **319.039** psi
	$v_{cpc} = 4 * \lambda * O(f'_c * 1\ psi) =$ **219.089** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **219.089** psi
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} =$ **219.089** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v * v_n =$ **164.317** psi
	$v_{ug} / \phi v_n =$ **0.003**
	PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 2

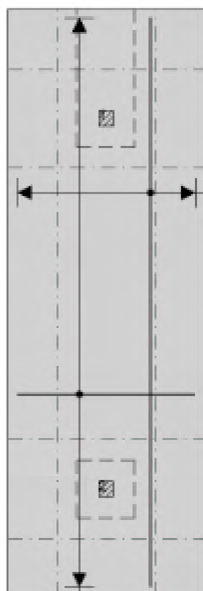
Depth to reinforcement	$d_{v2} =$ **14.375** in
Shear perimeter length (22.6.4)	$l_{xp} =$ **19.375** in
Shear perimeter width (22.6.4)	$l_{yp} =$ **19.375** in
Shear perimeter (22.6.4)	$b_o = 2 * (l_{x2} + d_{v2}) + 2 * (l_{y2} + d_{v2}) =$ **77.500** in
Shear area	$A_p = l_{x,perim} * l_{y,perim} =$ **375.391** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x2} * l_{y2} =$ **350.391** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} =$ **0.677** ksf
Ultimate shear load	$F_{up} = \gamma_D * F_{Dz2} + \gamma_L * F_{Lz2} + \gamma_{Lr} * F_{Lrz2} + \gamma_W * F_{Wz2} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} - q_{up.avg} * A_p =$ **10.296** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o * d_{v2}), 0\ psi) =$ **9.242** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y2} / l_{x2} =$ **1.00**
Column location factor (22.6.5.3)	$\alpha_s =$**40**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) * \lambda * O(f'_c * 1\ psi) =$ **328.634** psi
	$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * O(f'_c * 1\ psi) =$ **515.919** psi
	$v_{cpc} = 4 * \lambda * O(f'_c * 1\ psi) =$ **219.089** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **219.089** psi

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp}$ = **219.089** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v * v_n$ = **164.317** psi

$v_{ug} / \phi v_n$ = **0.056**



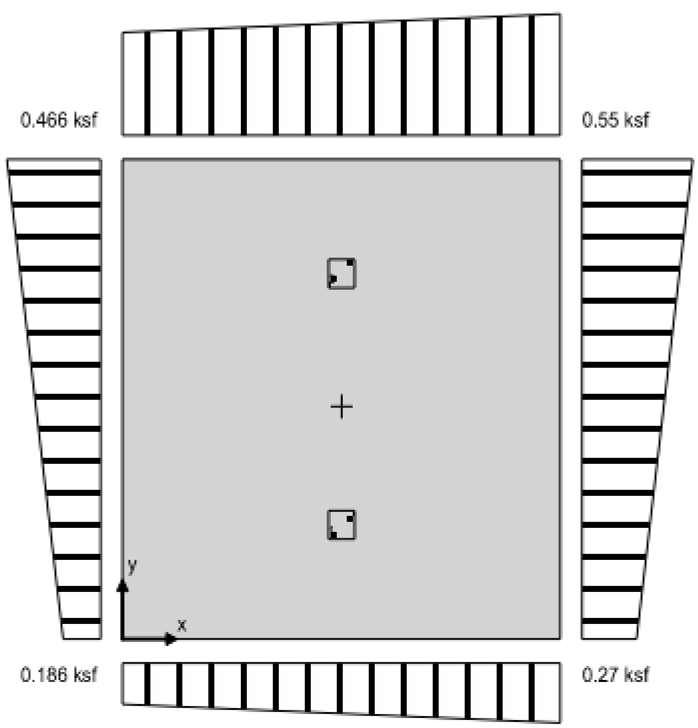
FOOTING F5 ANALYSIS & DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **7** ft
Width of foundation	L_y = **7** ft
Foundation area	$A = L_x \times L_y$ = **49** ft^2
Depth of foundation	h = **12** in
Depth of soil over foundation	h_{soil} = **12** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **5.00** in
Width of column	l_{y1} = **5.00** in
position in x-axis	x_1 = **42.00** in
position in y-axis	y_1 = **20.00** in

Column no.2 details	
Length of column	l_{x2} = **5.00** in
Width of column	l_{y2} = **5.00** in
position in x-axis	x_2 = **42.00** in
position in y-axis	y_2 = **64.00** in

Soil properties	
Gross allowable bearing pressure	q_{allow_Gross} = **2** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Self weight	$F_{swt} = h * \gamma_{conc}$ = **150** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **120** psf

Column no.1 loads	
Dead load in z	F_{Dz1} = **2.6** kips
Live load in z	F_{Lz1} = **0.6** kips
Live roof load in z	F_{Lrz1} = **0.3** kips
Wind load in z	F_{Wz1} = **-12.0** kips
Wind load moment in x	M_{Wx1} = **1.5** kip_ft
Wind load moment in y	M_{Wy1} = **-14.7** kip_ft

Column no.2 loads	
Dead load in z	F_{Dz2} = **1.6** kips
Live load in z	F_{Lz2} = **1.5** kips
Wind load in z	F_{Wz2} = **13.0** kips
Wind load moment in x	M_{Wx2} = **2.5** kip_ft
Wind load moment in y	M_{Wy2} = **-14.8** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.194)
1.0D + 1.0L (0.200)
1.0D + 1.0Lr (0.202)
1.0D + 1.0S (0.194)
1.0D + 1.0R (0.194)
1.0D + 0.75L + 0.75Lr (0.205)
1.0D + 0.75L + 0.75S (0.199)
1.0D + 0.75L + 0.75R (0.199)

1.0D + 0.6W (0.411)
1.0D + 0.75L + 0.75Lr + 0.45W (0.316)
1.0D + 0.75L + 0.75S + 0.45W (0.320)
1.0D + 0.75L + 0.75R + 0.45W (0.320)

Combination 9 results: 1.0D + 0.6W

Forces on foundation

Force in z-axis — $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_W * F_{Wz1} + \gamma_D * F_{Dz2} + \gamma_W * F_{Wz2} =$ **18.0** kips

Moments on foundation

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_W * (F_{Wz1} * x_1+M_{Wx1}) + \gamma_D * (F_{Dz2} * x_2) + \gamma_W * (F_{Wz2} * x_2+M_{Wx2}) =$ **65.5** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_W * (F_{Wz1} * y_1+M_{Wy1}) + \gamma_D * (F_{Dz2} * y_2) + \gamma_W * (F_{Wz2} * y_2+M_{Wy2}) =$ **71.1** kip_ft

Uplift verification

Vertical force — $F_{dz} =$ **18.03** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is 0

Overturning moment — $M_{OTx0} = \gamma_W * (F_{Wz1} * x_1) =$ **-25.2** kip_ft

Resisting moment — $M_{Rx0} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_W * (M_{Wx1}) + \gamma_D * (F_{Dz2} * x_2) + \gamma_W * (F_{Wz2} * x_2+M_{Wx2}) =$ **90.7** kip_ft

Factor of safety — $abs(M_{Rx0} / M_{OTx0}) =$ **3.599**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in x direction, moment about x is L_x

Overturning moment — $M_{OTxL} = \gamma_W * (F_{Wz1} * (x_1 - L_x)+M_{Wx1}) + \gamma_W * (M_{Wx2}) =$ **27.6** kip_ft

Resisting moment — $M_{RxL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2)) + \gamma_D * (F_{Dz1} * (x_1 - L_x)) + \gamma_D * (F_{Dz2} * (x_2 - L_x)) + \gamma_W * (F_{Wz2} * (x_2 - L_x)) =$ **-88.3** kip_ft

Factor of safety — $abs(M_{RxL} / M_{OTxL}) =$ **3.199**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in y direction, moment about y is 0

Overturning moment — $M_{OTy0} = \gamma_W * (F_{Wz1} * y_1+M_{Wy1}) + \gamma_W * (M_{Wy2}) =$ **-29.68** kip_ft

Resisting moment — $M_{Ry0} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_D * (F_{Dz2} * y_2) + \gamma_W * (F_{Wz2} * y_2) =$ **100.77** kip_ft

Factor of safety — $abs(M_{Ry0} / M_{OTy0}) =$ **3.395**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in y direction, moment about y is L_y

Overturning moment $M_{OTyL} = \gamma_W * (F_{Wz1} * (y_1 - L_y))$ = **38.4** kip_ft

Resisting moment $M_{RyL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2)) + \gamma_D * (F_{Dz1} * (y_1 - L_y)) + \gamma_W * (M_{Wy1}) + \gamma_D * (F_{Dz2} * (y_2 - L_y)) + \gamma_W * (F_{Wz2} * (y_2 - L_y) + M_{Wy2})$ = **-93.52** kip_ft

Factor of safety $abs(M_{RyL} / M_{OTyL})$ = **2.435**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **1.597** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **5.314** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.186** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.466** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.27** ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.55** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.186** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **0.55** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **2** ksf

q_{max} / q_{allow} = **0.275**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete f'_c = **3000** psi

Yield strength of reinforcement f_y = **60000** psi

Cover to reinforcement c_{nom} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.050)

1.2D + 1.6L + 0.5Lr (0.059)

1.2D + 1.6L + 0.5S (0.059)

1.2D + 1.6L + 0.5R (0.059)
1.2D + 1.0L + 1.6Lr (0.057)
1.2D + 1.0L + 1.6S (0.051)
1.2D + 1.0L + 1.6R (0.051)
1.2D + 1.6Lr + 0.5W (0.111)
1.2D + 1.6S + 0.5W (0.111)
1.2D + 1.6R + 0.5W (0.111)
1.2D + 1.0L + 0.5Lr + 1.0W (0.216)
1.2D + 1.0L + 0.5S + 1.0W (0.216)
1.2D + 1.0L + 0.5R + 1.0W (0.216)
0.9D + 1.0W (0.191)
$(0.9 - 0.2 * S_{DS})D + 1.0E$ (0.025)

Combination 11 results: 1.2D + 1.0L + 0.5Lr + 1.0W

Forces on foundation

Ultimate force in z-axis — $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1} + \gamma_D * F_{Dz2} + \gamma_L * F_{Lz2} + \gamma_W * F_{Wz2}$ = **24.1** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_{Lr} * (F_{Lrz1} * x_1) + \gamma_W * (F_{Wz1} * x_1+M_{Wx1}) + \gamma_D * (F_{Dz2} * x_2) + \gamma_L * (F_{Lz2} * x_2) + \gamma_W * (F_{Wz2} * x_2+M_{Wx2})$ = **88.5** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1) + \gamma_W * (F_{Wz1} * y_1+M_{Wy1}) + \gamma_D * (F_{Dz2} * y_2) + \gamma_L * (F_{Lz2} * y_2) + \gamma_W * (F_{Wz2} * y_2+M_{Wy2})$ = **100.1** kip_ft

Eccentricity of base reaction

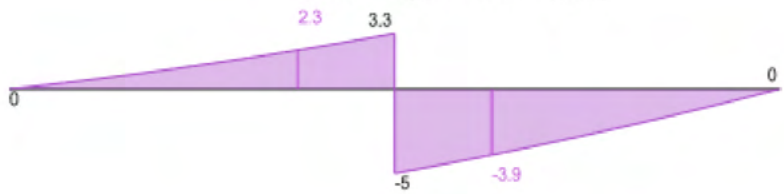
Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **1.988** in
Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **7.74** in

Pad base pressures

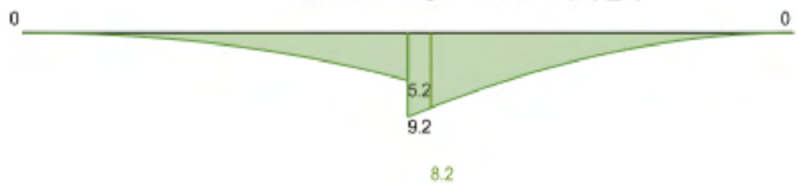
$q_{u1} = F_{uz} * (1 - 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y)$ = **0.15** ksf
$q_{u2} = F_{uz} * (1 - 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y)$ = **0.695** ksf
$q_{u3} = F_{uz} * (1 + 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y)$ = **0.29** ksf
$q_{u4} = F_{uz} * (1 + 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y)$ = **0.835** ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.15** ksf
Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.835** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **8.222** kip_ft
Tension reinforcement provided	7 No.5 bottom bars (12.8 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 * L_y * h$ = **1.814** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2$ = **8.062** in
Depth of compression block	$a = A_{sx.bot.prov} * f_y / (0.85 * f'_c * L_y)$ = **0.608** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.715** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 * d / c - 0.003$ = **0.03082**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sx.bot.prov} * f_y * (d - a / 2)$ = **84.181** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f * M_n$ = **75.763** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.109**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force	$V_{u.x}$ = **3.898** kips
Depth to reinforcement	$d_v = \min(h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2)$ = **8.062** in

Shear strength reduction factor — ϕ_v = **0.75**

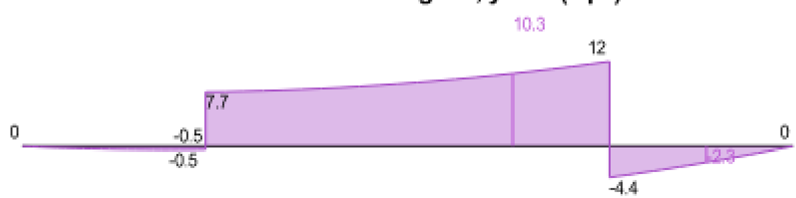
Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 * \lambda * \sqrt{(f'_c * 1\ psi)} * L_y * d_v$ = **74.189** kips

Design shear capacity — $\phi V_n = \phi_v * V_n$ = **55.642** kips

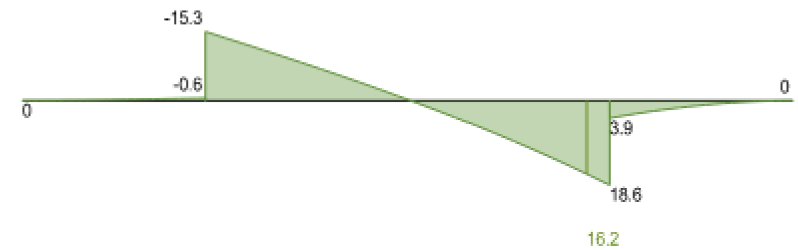
$V_{u.x} / \phi V_n$ = **0.070**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **16.18** kip_ft

Tension reinforcement provided — 7 No.5 bottom bars (12.8 in c/c)

Area of tension reinforcement provided — $A_{sy.bot.prov}$ = **2.17** in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 * L_x * h$ = **1.814** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — s_{max} = min(2 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{y.bot} / 2$ = **8.688** in

Depth of compression block — $a = A_{sy.bot.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.608** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **0.715** in

Strain in tensile reinforcement (8.3.3.1) — $\varepsilon_t = 0.003 * d / c - 0.003$ = **0.03345**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity — $M_n = A_{sy.bot.prov} * f_y * (d - a / 2)$ = **90.962** kip_ft

Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	$\phi M_n = \phi_f * M_n$ = **81.866** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.198**
	PASS - Design moment capacity exceeds ultimate moment load

Moment design, y direction, negative moment

Ultimate bending moment	$M_{u.y.min}$ = **-13.679** kip_ft
Tension reinforcement provided	7 No.5 top bars (12.8 in c/c)
Area of tension reinforcement provided	$A_{sy.top.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 * L_x * h$ = **1.814** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{y.top} / 2$ = **8.688** in
Depth of compression block	$a = A_{sy.top.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.608** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.715** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 * d / c - 0.003$ = **0.03345**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sy.top.prov} * f_y * (d - a / 2)$ = **90.962** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	$\phi M_n = \phi_f * M_n$ = **81.866** kip_ft
	abs($M_{u.y.min}$) / ϕM_n = **0.167**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **10.321** kips
Depth to reinforcement	d_v = min(h - c_{nom} - $\phi_{y.bot}$ / 2,h - c_{nom} - $\phi_{y.top}$ / 2) = **8.688** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 * \lambda *$ O(f'_c * 1 psi) * $L_x * d_v$ = **79.94** kips
Design shear capacity	$\phi V_n = \phi_v * V_n$ = **59.955** kips
	$V_{u.y} / \phi V_n$ = **0.172**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

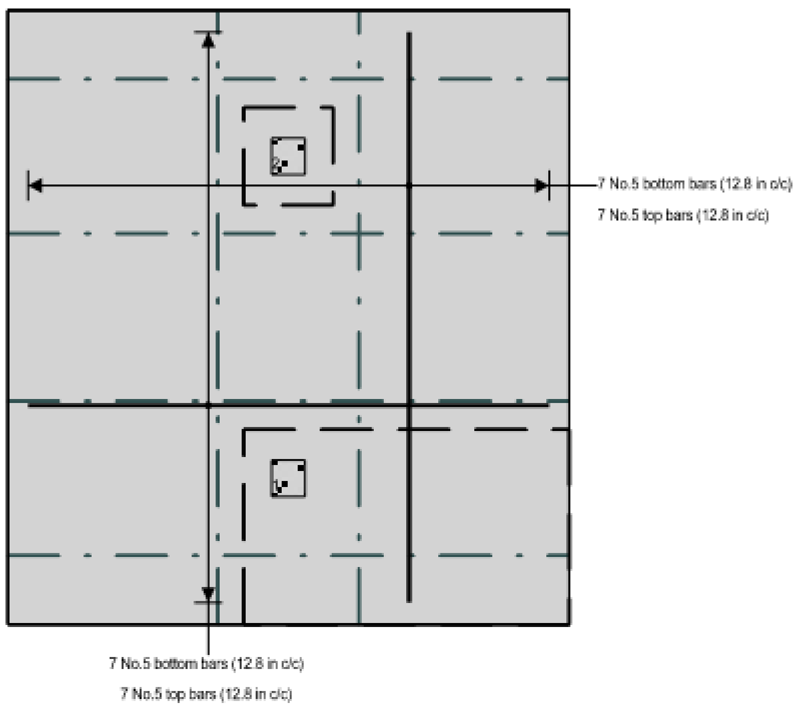
Depth to reinforcement	d_{v2} = **8.375** in
Shear perimeter length (22.6.4)	l_{xp} = **48.688** in
Shear perimeter width (22.6.4)	l_{yp} = **26.687** in
Shear perimeter (22.6.4)	$b_o = l_{x,perim} + l_{y,perim}$ = **75.375** in
Shear area	$A_p = l_{x,perim} * l_{y,perim}$ = **1299.348** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} * l_{y1}$ = **1274.348** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **0.509** ksf
Ultimate shear load	$F_{up} = \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} - q_{up.avg} * A_p$ = **-9.848** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o * d_{v2}), 0 \text{ psi})$ = **0.000** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1}$ = **1.00**
Column location factor (22.6.5.3)	α_s =**20**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) * \lambda * \sqrt{(f'_c * 1 \text{ psi})}$ = **328.634** psi
	$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * \sqrt{(f'_c * 1 \text{ psi})}$ = **231.261** psi
	$v_{cpc} = 4 * \lambda * \sqrt{(f'_c * 1 \text{ psi})}$ = **219.089** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **219.089** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **219.089** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v * v_n$ = **164.317** psi
	$v_{ug} / \phi v_n$ = **0.000**

PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 2

Depth to reinforcement	d_{v2} = **8.375** in
Shear perimeter length (22.6.4)	l_{xp} = **13.375** in
Shear perimeter width (22.6.4)	l_{yp} = **13.375** in
Shear perimeter (22.6.4)	$b_o = 2 * (l_{x2} + d_{v2}) + 2 * (l_{y2} + d_{v2})$ = **53.500** in
Shear area	$A_p = l_{x,perim} * l_{y,perim}$ = **178.891** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x2} * l_{y2}$ = **153.891** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **0.722** ksf
Ultimate shear load	$F_{up} = \gamma_D * F_{Dz2} + \gamma_L * F_{Lz2} + \gamma_W * F_{Wz2} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} - q_{up.avg} * A_p$ = **15.900** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o * d_{v2}), 0 \text{ psi})$ = **35.487** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y2} / l_{x2}$ = **1.00**
Column location factor (22.6.5.3)	α_s =**40**

Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) * \lambda * \sqrt{f'_c * 1\ psi}$ = **328.634** psi
	$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * \sqrt{f'_c * 1\ psi}$ = **452.511** psi
	$v_{cpc} = 4 * \lambda * \sqrt{f'_c * 1\ psi}$ = **219.089** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **219.089** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **219.089** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v * v_n$ = **164.317** psi
	$v_{ug} / \phi v_n$ = **0.216**



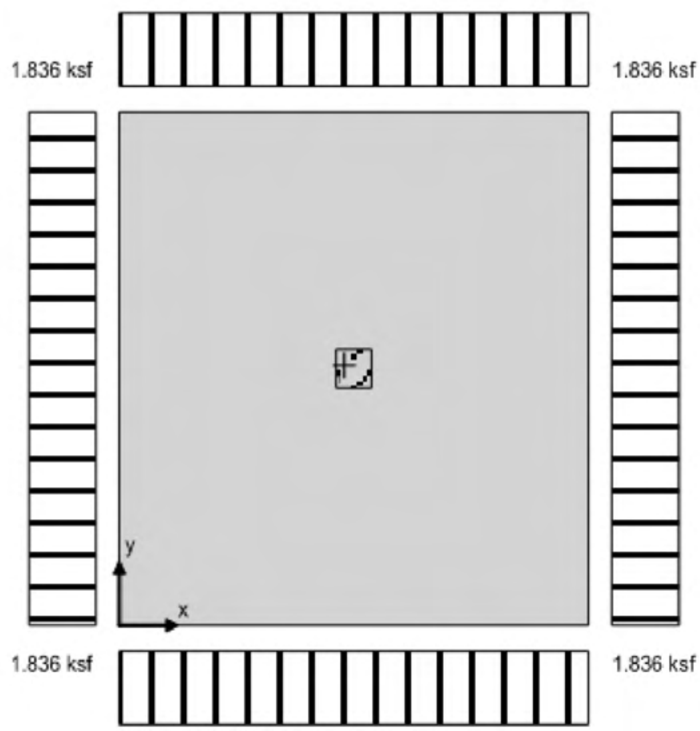
FOOTING F6 ANALYSIS & DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **5.5** ft
Width of foundation	L_y = **5.5** ft
Foundation area	$A = L_x \times L_y$ = **30.25** ft^2
Depth of foundation	h = **18** in
Depth of soil over foundation	h_{soil} = **12** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **5.00** in
Width of column	l_{y1} = **5.00** in
position in x-axis	x_1 = **33.00** in
position in y-axis	y_1 = **33.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Self weight	$F_{swt} = h * \gamma_{conc}$ = **225** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **27.4** kips
Live load in z	F_{Lz1} = **11.5** kips
Live roof load in z	F_{Lrz1} = **12.2** kips
Wind load in z	F_{Wz1} = **-15.1** kips
Wind load moment in x	M_{Wx1} = **-4.7** kip_ft
Wind load moment in y	M_{Wy1} = **1.5** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.625)
1.0D + 1.0L (0.815)
1.0D + 1.0Lr (0.826)
1.0D + 1.0S (0.625)
1.0D + 1.0R (0.625)
1.0D + 0.75L + 0.75Lr (0.918)
1.0D + 0.75L + 0.75S (0.767)
1.0D + 0.75L + 0.75R (0.767)
1.0D + 0.75L + 0.75Lr + 0.45W (0.856)
1.0D + 0.75L + 0.75S + 0.45W (0.705)
1.0D + 0.75L + 0.75R + 0.45W (0.705)

Combination 6 results: 1.0D + 0.75L + 0.75Lr

Forces on foundation

Force in z-axis	$F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1}$ = **55.5** kips

Moments on foundation

Moment in x-axis, about x is 0	$M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_{Lr} * (F_{Lrz1} * x_1)$ = **152.7** kip_ft

Moment in y-axis, about y is 0 | $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1)$ = **152.7** kip_ft

Uplift verification

Vertical force | F_{dz} = **55.536** kips

PASS - Foundation is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis | $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis | $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **1.836** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **1.836** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **1.836** ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **1.836** ksf

Minimum base pressure | $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **1.836** ksf

Maximum base pressure | $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.836** ksf

Allowable bearing capacity

Allowable bearing capacity | $q_{allow} = q_{allow\_Gross}$ = **2** ksf

q_{max} / q_{allow} = **0.918**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete | f'_c = **3000** psi

Yield strength of reinforcement | f_y = **60000** psi

Cover to reinforcement | c_{nom} = **3** in

Concrete type | Normal weight

Concrete modification factor | λ = **1.00**

Column type | Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.191)

1.2D + 1.6L + 0.5Lr (0.286)

1.2D + 1.6L + 0.5S (0.255)

1.2D + 1.6L + 0.5R (0.255)

1.2D + 1.0L + 1.6Lr (0.318)

1.2D + 1.0L + 1.6S (0.221)

1.2D + 1.0L + 1.6R (0.221)

1.2D + 1.6Lr + 0.5W (0.223)

1.2D + 1.6R + 0.5W (0.131)

1.2D + 1.0L + 0.5Lr + 1.0W (0.176)

1.2D + 1.0L + 0.5S + 1.0W (0.151)

1.2D + 1.0L + 0.5R + 1.0W (0.151)

Combination 5 results: 1.2D + 1.0L + 1.6Lr

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1}$ = **76.3** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_{Lr} * (F_{Lrz1} * x_1)$ = **209.8** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1)$ = **209.8** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} * (1 - 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y)$ = **2.522** ksf

$q_{u2} = F_{uz} * (1 - 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y)$ = **2.522** ksf

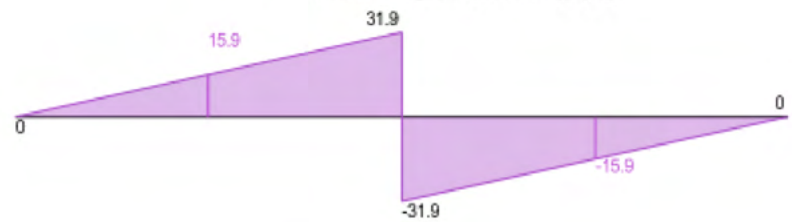
$q_{u3} = F_{uz} * (1 + 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y)$ = **2.522** ksf

$q_{u4} = F_{uz} * (1 + 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y)$ = **2.522** ksf

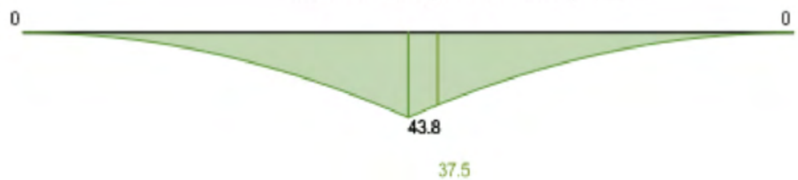
Minimum ultimate base pressure $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **2.522** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **2.522** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **37.451** kip_ft
Tension reinforcement provided	8 No.5 bottom bars (8.4 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **2.48** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 * L_y * h = **2.138** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing

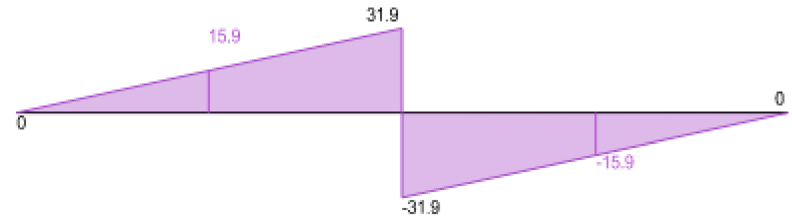
Depth to tension reinforcement	d = h - c_{nom} - $\phi_{x.bot}$ / 2 = **14.688** in
Depth of compression block	a = $A_{sx.bot.prov}$ * f_y / (0.85 * f'_c * L_y) = **0.884** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	c = a / β_1 = **1.040** in
Strain in tensile reinforcement (8.3.3.1)	ε_t = 0.003 * d / c - 0.003 = **0.03936**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	M_n = $A_{sx.bot.prov}$ * f_y * (d - a / 2) = **176.643** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f * M_n = **158.979** kip_ft
	$M_{u.x.max}$ / ϕM_n = **0.236**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

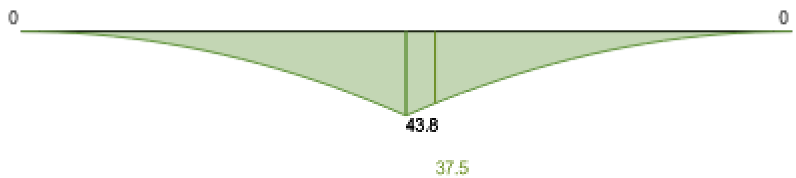
Ultimate shear force	$V_{u.x}$ = **15.882** kips
Depth to reinforcement	$d_v = \min(h - c_{nom} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2)$ = **14.688** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 * \lambda * \surd(f'_c * 1 \text{ psi}) * L_y * d_v$ = **106.19** kips
Design shear capacity	$\phi V_n = \phi_v * V_n$ = **79.642** kips
	$V_{u.x} / \phi V_n$ = **0.199**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **37.451** kip_ft
Tension reinforcement provided	8 No.5 bottom bars (8.4 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **2.48** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 * L_x * h$ = **2.138** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 * h, 18 \text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **14.063** in
Depth of compression block	$a = A_{sy.bot.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.884** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **1.040** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 * d / c - 0.003$ = **0.03756**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	$M_n = A_{sy.bot.prov} * f_y * (d - a / 2) =$ **168.893** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9) =$ **0.900**
Design moment capacity	$\phi M_n = \phi_f * M_n =$ **152.004** kip_ft
	$M_{u.y.max} / \phi M_n =$ **0.246**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y} =$ **15.882** kips
Depth to reinforcement	$d_v = \min(h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom} - \phi_{y.top} / 2) =$ **14.063** in
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 * \lambda * O(f'_c * 1\ psi) * L_x * d_v =$ **101.671** kips
Design shear capacity	$\phi V_n = \phi_v * V_n =$ **76.253** kips
	$V_{u.y} / \phi V_n =$ **0.208**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	$d_{v2} =$ **14.375** in
Shear perimeter length (22.6.4)	$l_{xp} =$ **19.375** in
Shear perimeter width (22.6.4)	$l_{yp} =$ **19.375** in
Shear perimeter (22.6.4)	$b_o = 2 * (l_{x1} + d_{v2}) + 2 * (l_{y1} + d_{v2}) =$ **77.500** in
Shear area	$A_p = l_{x,perim} * l_{y,perim} =$ **375.391** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} * l_{y1} =$ **350.391** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} =$ **2.522** ksf
Ultimate shear load	$F_{up} = \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} - q_{up.avg} * A_p =$ **58.249** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o * d_{v2}), 0\ psi) =$ **52.286** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (22.6.5.3)	$\alpha_s =$ **40**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) * \lambda * O(f'_c * 1\ psi) =$ **328.634** psi
	$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * O(f'_c * 1\ psi) =$ **515.919** psi
	$v_{cpc} = 4 * \lambda * O(f'_c * 1\ psi) =$ **219.089** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **219.089** psi

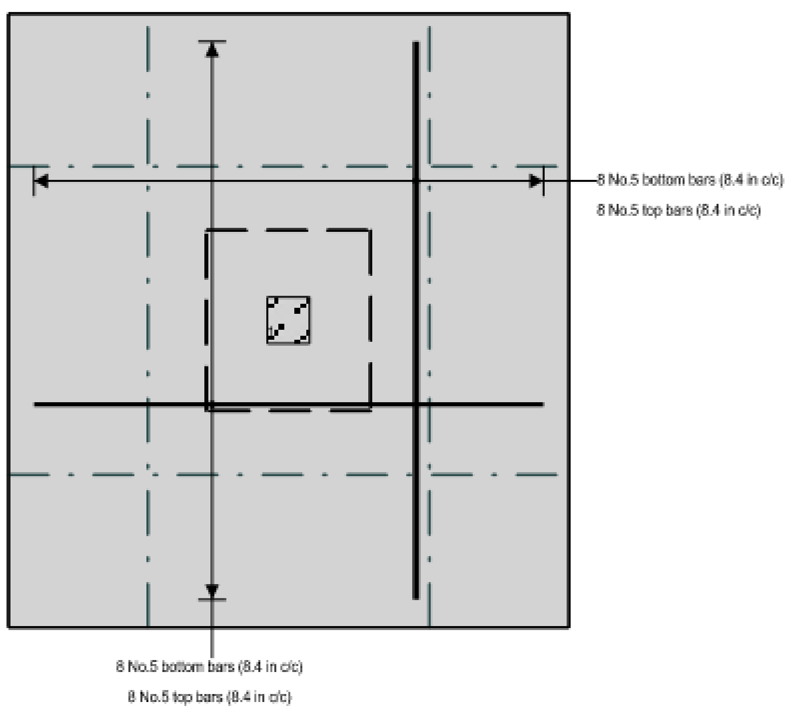
Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp}$ = **219.089** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v * v_n$ = **164.317** psi

$v_{ug} / \phi v_n$ = **0.318**

PASS - Design shear stress capacity exceeds ultimate shear stress load



3.2 EMBED PLATE STUDS DESIGN

EMBED PLATE STUDS DESIGN FOR 12" THICK FOUNDATION (FOOTING F2, F5)

Design moment 6.8 kip*ft

Force per row of studs 6.8 kip*ft x 6 in /12 in = 13.6 kip

ANCHOR BOLT DESIGN

In accordance with ACI318-14

Anchor bolt geometry

Anchor bolt	Cast-in headed stud anchor	Diameter of anchor bolt	d_a = **0.5** in
Number of bolts in x direction	N_{boltx} = **2**	Number of bolts in y direction	N_{bolty} = **2**
Total number of bolts	n_{total} = **4**	Total No. of bolts in tension	n_{tens} = **4**
Spacing of bolts in x direction	s_{boltx} = **6** in	Spacing of bolts in y direction	s_{bolty} = **6** in
Effective area of anchor	A_{se} = **0.196** in^2	Embed. depth of anchor bolt	h_{ef} = **8** in

Foundation geometry

Member thickness	h_a = **12** in		
CL baseplate - left edge	x_{ce1} = **20** in	CL baseplate - right edge	x_{ce2} = **20** in
CL baseplate - bot. edge	y_{ce1} = **20** in	CL baseplate - top edge	y_{ce2} = **20** in

Material details

Minimum yield strength of steel	f_{ya} = **36** ksi		
Nom. tensile stress of steel	f_{uta} = **58** ksi	Compressive strength of conc	f'_c = **3** ksi
Concrete modification factor	λ = **1.00**		
Mod. factor, concrete failure	λ_a = **1.00**		

Strength reduction factors

Tension of steel element	$\phi_{t,s}$ = **0.75**	Shear of steel element	$\phi_{v,s}$ = **0.70**
Concrete tension	$\phi_{t,c}$ = **0.65**	Concrete shear	$\phi_{v,c}$ = **0.70**
Concrete tension for pullout	$\phi_{t,cB}$ = **0.70**	Concrete shear for pryout	$\phi_{v,cB}$ = **0.70**

Anchor forces

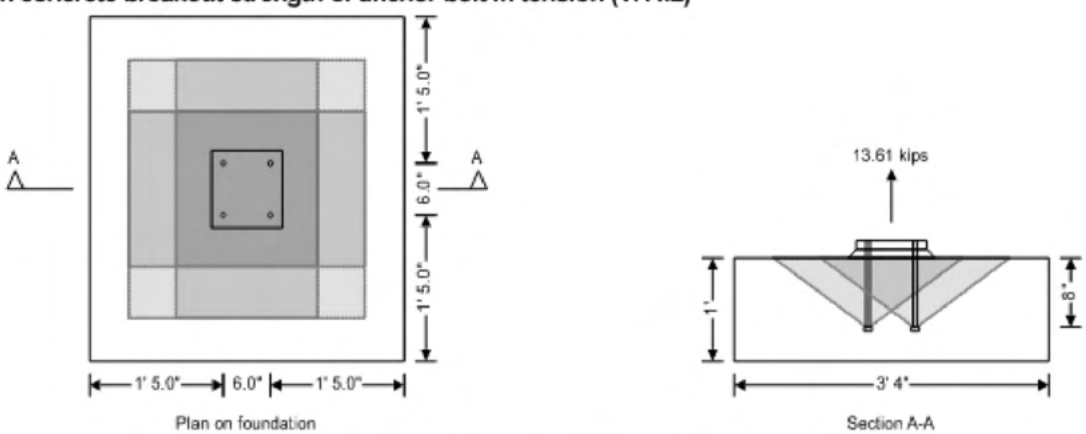
No. of bolt rows in tension	N_{boltN} = **2**		
Axial force in bolts for row 1	N_1 = **13.60** kips		
Axial force in bolts for row 2	N_2 = **0.01** kips		
Total axial force on bolt group	N_R = **13.61** kips	Max axial force to single bolt	$N_{max,s}$ = **6.80** kips
Eccentricity of axial load	e'_N = **3.00** in	Shear force to bolt group	V = **0.00** kips

Steel strength of anchor in tension (17.4.1)

Nom strength of anchor	N_{sa} = **11.39** kips	Steel strength of anchor	ϕN_{sa} = **8.54** kips

PASS - Steel strength of anchor exceeds max tension in single bolt

Check concrete breakout strength of anchor bolt in tension (17.4.2)



Concrete breakout - tension

Coeff for basic breakout	k_c = **24**	Breakout strgth single anchor	N_b = **29.74** kips
Proj area - groups of anchors	A_{Nc} = **900.00** in^2	Proj area - single anchor	A_{Nco} = **576.00** in^2
Min dist to edge of concrete	$c_{a,min}$ = **17.00** in	Ecc. mod factor for groups	$\psi_{ec,N}$ = **0.80**
Mod factor for edge effects	$\psi_{ed,N}$ = **1.00**	Mod factor for no cracking	$\psi_{c,N}$ = **1.000**
Mod factor for cracked conc	$\psi_{cp,N}$ = **1.000**		
Nom conc breakout strength	N_{cbg} = **37.19** kips	Concrete breakout strength	ϕN_{cbg} = **24.17** kips

PASS - Breakout strength exceeds tension in bolts

Pullout strength single anchor (17.4.3)

Net brg area head of anchor	A_{brg} = **1** in^2	Mod factor for no cracking	$\psi_{c,P}$ = **1.000**
Pullout strength	N_p = **24.00** kips	Nom pullout strength	N_{pn} = **24.00** kips
Pullout strength	ϕN_{pn} = **16.80** kips		

PASS - Pullout strength of single anchor exceeds maximum axial force in single bolt

Side face blowout strength (17.4.4)

As h_{ef} <= 2.5 * min(c_{a1}, c_{a2}) the edge distance is considered to be far from an edge and blowout strength need not be considered

EMBED PLATE STUDS DESIGN FOR 18" THICK FOUNDATION (FOOTING F1, F3, F4, F6)

Design moment 14.95 kip*ft

Force per row of studs 14.95 kip*ft x 6 in /12 in = 29.9 kip

ANCHOR BOLT DESIGN

In accordance with ACI318-14

Anchor bolt geometry

Anchor bolt	Cast-in headed stud anchor	Diameter of anchor bolt	d_a = **0.875** in
Number of bolts in x direction	N_{boltx} = **2**	Number of bolts in y direction	N_{bolty} = **2**
Total number of bolts	n_{total} = **4**	Total No. of bolts in tension	n_{tens} = **4**
Spacing of bolts in x direction	s_{boltx} = **6** in	Spacing of bolts in y direction	s_{bolty} = **6** in
Effective area of anchor	A_{se} = **0.601** in^2	Embed. depth of anchor bolt	h_{ef} = **16** in

Foundation geometry

Member thickness	h_a = **18** in		
CL baseplate - left edge	x_{ce1} = **36** in	CL baseplate - right edge	x_{ce2} = **36** in
CL baseplate - bot. edge	y_{ce1} = **36** in	CL baseplate - top edge	y_{ce2} = **36** in

Material details

Minimum yield strength of steel	f_{ya} = **36** ksi		
Nom. tensile stress of steel	f_{uta} = **58** ksi	Compressive strength of conc	f'_c = **3** ksi
Concrete modification factor	λ = **1.00**		
Mod. factor, concrete failure	λ_a = **1.00**		

Strength reduction factors

Tension of steel element	$\phi_{t,s}$ = **0.75**	Shear of steel element	$\phi_{v,s}$ = **0.70**
Concrete tension	$\phi_{t,c}$ = **0.65**	Concrete shear	$\phi_{v,c}$ = **0.70**
Concrete tension for pullout	$\phi_{t,cB}$ = **0.70**	Concrete shear for pryout	$\phi_{v,cB}$ = **0.70**

Anchor forces

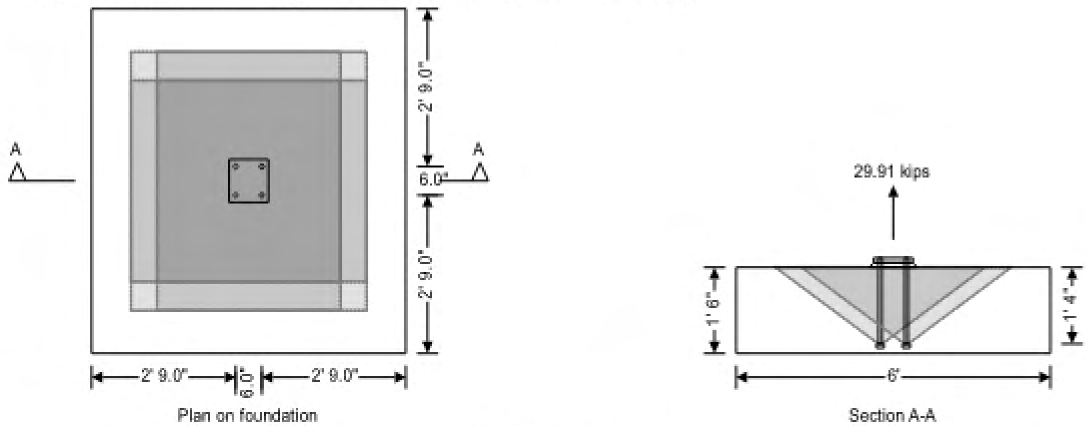
No. of bolt rows in tension	N_{boltN} = **2**		
Axial force in bolts for row 1	N_1 = **29.90** kips		
Axial force in bolts for row 2	N_2 = **0.01** kips		
Total axial force on bolt group	N_R = **29.91** kips	Max axial force to single bolt	$N_{max,s}$ = **14.95** kips
Eccentricity of axial load	e'_N = **3.00** in	Shear force to bolt group	V = **0.00** kips

Steel strength of anchor in tension (17.4.1)

Nom strength of anchor	N_{sa} = **34.88** kips	Steel strength of anchor	ϕN_{sa} = **26.16** kips

PASS - Steel strength of anchor exceeds max tension in single bolt

Check concrete breakout strength of anchor bolt in tension (17.4.2)



Concrete breakout - tension

Coeff for basic breakout	k_c = **24**	Breakout strgth single anchor	N_b = **89.03** kips
Proj area - groups of anchors	A_{Nc} = **2916.00** in^2	Proj area - single anchor	A_{Nco} = **2304.00** in^2
Min dist to edge of concrete	$c_{a,min}$ = **33.00** in	Ecc. mod factor for groups	$\psi_{ec,N}$ = **0.89**
Mod factor for edge effects	$\psi_{ed,N}$ = **1.00**	Mod factor for no cracking	$\psi_{c,N}$ = **1.000**
Mod factor for cracked conc	$\psi_{cp,N}$ = **1.000**		
Nom conc breakout strength	N_{cbg} = **100.17** kips	Concrete breakout strength	ϕN_{cbg} = **65.11** kips

PASS - Breakout strength exceeds tension in bolts

Pullout strength single anchor (17.4.3)

Net brg area head of anchor	A_{brg} = **1** in^2	Mod factor for no cracking	$\psi_{c,P}$ = **1.000**
Pullout strength	N_p = **24.00** kips	Nom pullout strength	N_{pn} = **24.00** kips
Pullout strength	ϕN_{pn} = **16.80** kips		

PASS - Pullout strength of single anchor exceeds maximum axial force in single bolt

Side face blowout strength (17.4.4)

As h_{ef} <= 2.5 * min(c_{a1}, c_{a2}) the edge distance is considered to be far from an edge and blowout strength need not be considered