STRUCTURAL ANALYSIS OF
PROPOSED RESIDENTIAL HOME

11DEC2024

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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)

The calculation performed by SCIA Engineer 20.0

1.2 CODE CRITERIA

2021 IBC

Seismic Design Category:	**A**
Wind Speed:	**150** MPH
Wind Exposure:	**D**
Snow Load (Roof):	**0** psf

1.3 MATERIALS

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	**20.00 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	20
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

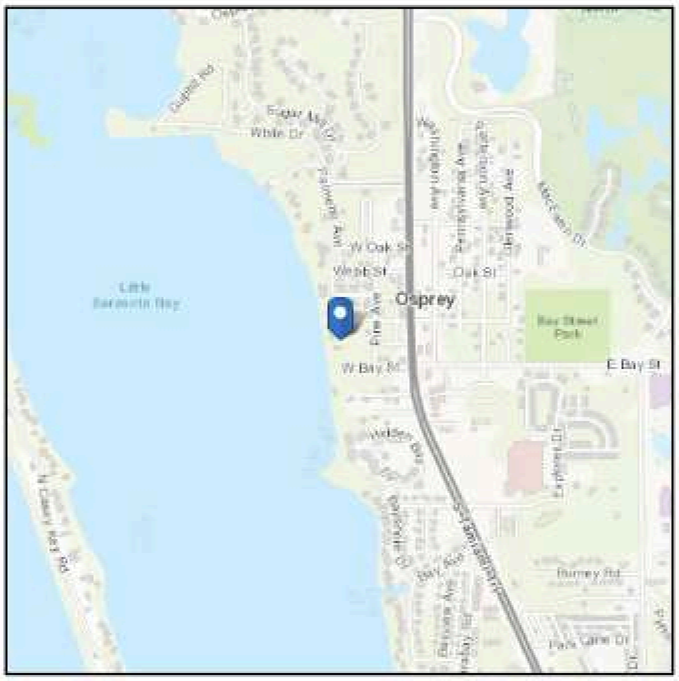
WIND LOADS

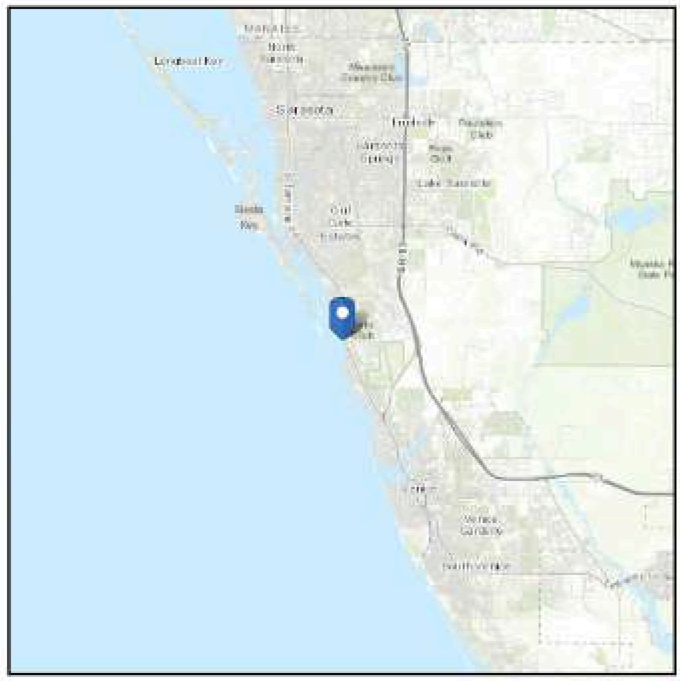
ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-22	**Latitude:**	27.197025
Risk Category:	II	**Longitude:**	-82.492757
Soil Class:	Default	**Elevation:**	9.19234437880068 ft (NAVD 88)





Wind

Results:

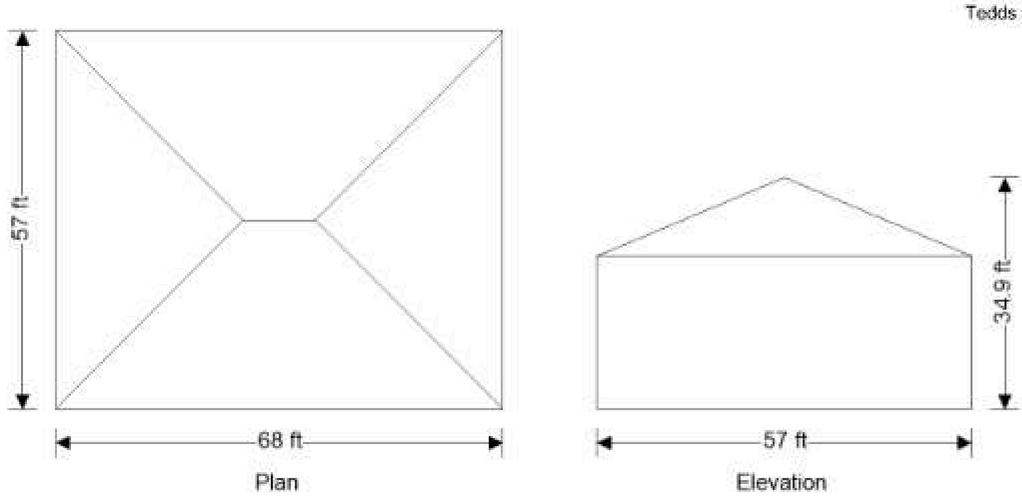
Wind Speed	150 Vmph
10-year MRI	79 Vmph
25-year MRI	99 Vmph
50-year MRI	113 Vmph
100-year MRI	125 Vmph
300-year MRI	140 Vmph
700-year MRI	150 Vmph

1,700-year MRI	160 Vmph
3,000-year MRI	168 Vmph
10,000-year MRI	178 Vmph
100,000-year MRI	207 Vmph
1,000,000-year MRI	227 Vmph

WIND LOADING

Using the directional design method

Tedds calculation version 2.1.03



Building data

Type of roof	Hipped
Length of building	b = **68.00** ft
Width of building	d = **57.00** ft
Height to eaves	H = **23.00** ft
Pitch of main slope	α_0 = **22.6** deg
Pitch of gable slope	α_{90} = **22.6** deg
Mean height	h = **28.94** ft

General wind load requirements

Basic wind speed	V = **150.0** 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	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **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 $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
15.00	1.03	50.43
15.00	1.03	50.43
23.00	1.10	54.05
28.94	1.15	56.38

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **56.38** psf

Pressures and forces

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

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	28.94	-0.35	56.38	-26.98	1219.65	-32.91
B (-ve)	28.94	-0.60	56.38	-38.90	1219.65	-47.45
C (-ve)	28.94	-0.90	56.38	-53.49	226.85	-12.13
D (-ve)	28.94	-0.90	56.38	-53.13	680.14	-36.14
E (-ve)	28.94	-0.50	56.38	-34.26	853.01	-29.22

Total vertical net force $F_{w,v}$ = **-145.69** kips

Total horizontal net force $F_{w,h}$ = **5.59** 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} (ft²)	Net force F_w (kips)
A_1	15.00	0.80	50.43	24.14	1020.00	24.63
A_2	15.00	0.80	50.43	24.14	0.00	0.00
A_3	23.00	0.80	54.05	26.61	544.00	14.47
B	28.94	-0.50	56.38	-34.11	1564.00	-53.35
C	28.94	-0.70	56.38	-43.69	1311.00	-57.28
D	28.94	-0.70	56.38	-43.69	1311.00	-57.28

Overall loading

Projected vertical plan area of wall — $A_{vert\_w\_0} = b \times H =$ **1564.00** ft²

Projected vertical area of roof — $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) - (d/2 \times \tan(\alpha_0))^2 / \tan(\alpha_{90}) =$ **469.30** ft²

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **28.78** kips

Leeward net force — $F_l = F_{w,wB} =$ **-53.3** kips

Windward net force — $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **39.1** kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **98.0** 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} (ft²)	Net force F_w (kips)
A (+ve)	28.94	0.10	56.38	15.05	1219.65	18.35
B (+ve)	28.94	-0.60	56.38	-18.60	1219.65	-22.69
C (+ve)	28.94	-0.18	56.38	1.52	226.85	0.35
D (+ve)	28.94	-0.18	56.38	1.52	680.14	1.04
E (+ve)	28.94	-0.18	56.38	1.52	853.01	1.30

Total vertical net force — $F_{w,v} =$ **-1.53** kips

Total horizontal net force — $F_{w,h} =$ **15.79** 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} (ft²)	Net force F_w (kips)
A_1	15.00	0.80	50.43	44.44	1020.00	45.33
A_2	15.00	0.80	50.43	44.44	0.00	0.00
A_3	23.00	0.80	54.05	46.90	544.00	25.52
B	28.94	-0.50	56.38	-13.81	1564.00	-21.60
C	28.94	-0.70	56.38	-23.40	1311.00	-30.67
D	28.94	-0.70	56.38	-23.40	1311.00	-30.67

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H = \mathbf{1564.00}$ ft²

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) - (d/2 \times \tan(\alpha_0))^2 / \tan(\alpha_{90}) = \mathbf{469.30}$ ft²

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{28.78}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-21.6}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{70.8}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{108.2}$ 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} (ft²)	Net force F_w (kips)
A (-ve)	28.94	-0.32	56.38	-25.37	880.00	-22.33
B (-ve)	28.94	-0.60	56.38	-38.90	880.00	-34.23
C (-ve)	28.94	-0.90	56.38	-53.28	226.85	-12.09
D (-ve)	28.94	-0.90	56.38	-53.28	680.35	-36.25
E (-ve)	28.94	-0.50	56.38	-34.11	1421.17	-48.47
F (-ve)	28.94	-0.30	56.38	-24.52	110.94	-2.72

Total vertical net force $F_{w,v} = \mathbf{-144.07}$ kips

Total horizontal net force $F_{w,h} = \mathbf{4.58}$ 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} (ft²)	Net force F_w (kips)
A_1	15.00	0.80	50.43	24.14	855.00	20.64

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_2	15.00	0.80	50.43	24.14	0.00	0.00
A_3	23.00	0.80	54.05	26.61	456.00	12.13
B	28.94	-0.46	56.38	-32.26	1311.00	-42.29
C	28.94	-0.70	56.38	-43.69	1564.00	-68.34
D	28.94	-0.70	56.38	-43.69	1564.00	-68.34

Overall loading

Projected vertical plan area of wall — $A_{vert\_w\_90} = d \times H =$ **1311.00** ft^2

Projected vertical area of roof — $A_{vert\_r\_90} = d^2/4 \times \tan(\alpha_0) =$ **338.61** ft^2

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **23.68** kips

Leeward net force — $F_l = F_{w,wB} =$ **-42.3** kips

Windward net force — $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **32.8** kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **79.6** 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)	28.94	0.15	56.38	17.29	880.00	15.22
B (+ve)	28.94	-0.60	56.38	-18.60	880.00	-16.37
C (+ve)	28.94	-0.18	56.38	1.52	226.85	0.35
D (+ve)	28.94	-0.18	56.38	1.52	680.35	1.04
E (+ve)	28.94	-0.18	56.38	1.52	1421.17	2.16
F (+ve)	28.94	-0.18	56.38	1.52	110.94	0.17

Total vertical net force — $F_{w,v} =$ **2.36** kips

Total horizontal net force — $F_{w,h} =$ **12.16** 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	50.43	44.44	855.00	38.00
A_2	15.00	0.80	50.43	44.44	0.00	0.00
A_3	23.00	0.80	54.05	46.90	456.00	21.39

B	28.94	-0.46	56.38	-11.96	1311.00	-15.68
C	28.94	-0.70	56.38	-23.40	1564.00	-36.59
D	28.94	-0.70	56.38	-23.40	1564.00	-36.59

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H = \mathbf{1311.00}$ ft^2

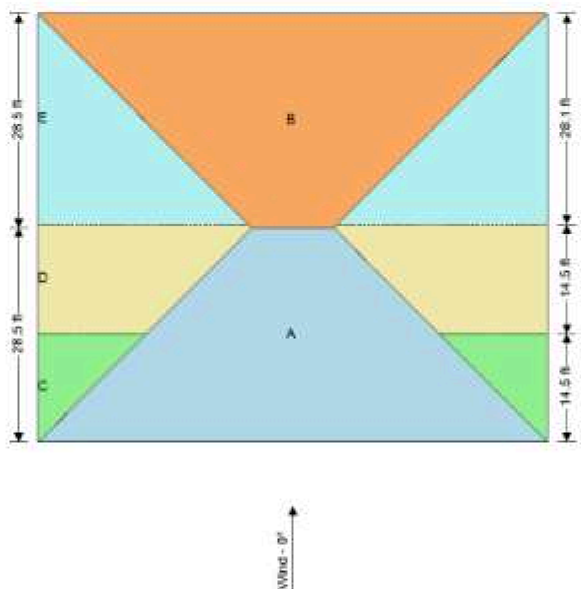
Projected vertical area of roof $A_{vert\_r\_90} = d^2/4 \times \tan(\alpha_0) = \mathbf{338.61}$ ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{23.68}$ kips

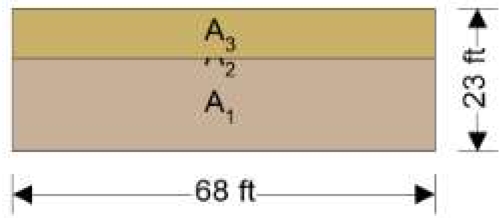
Leeward net force $F_l = F_{w,wB} = \mathbf{-15.7}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{59.4}$ kips

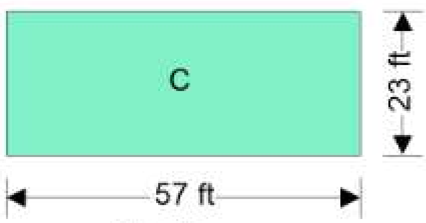
Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{87.2}$ kips



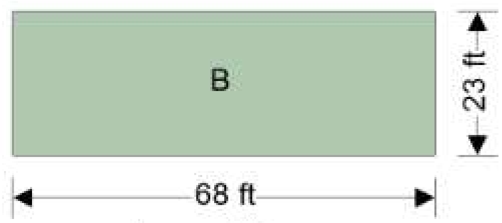
Plan view - Hipped roof



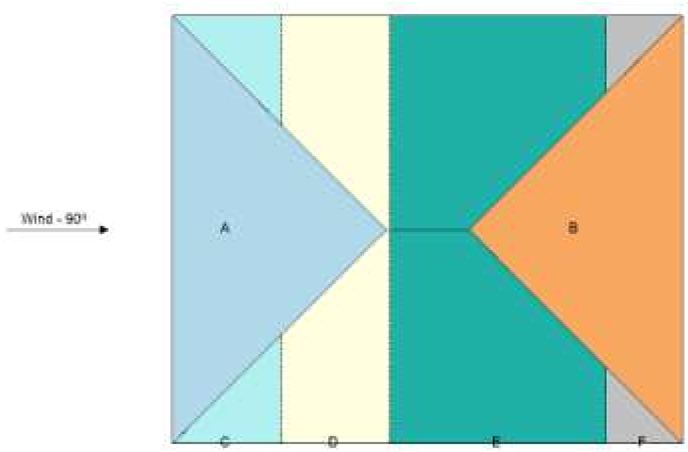
Windward face



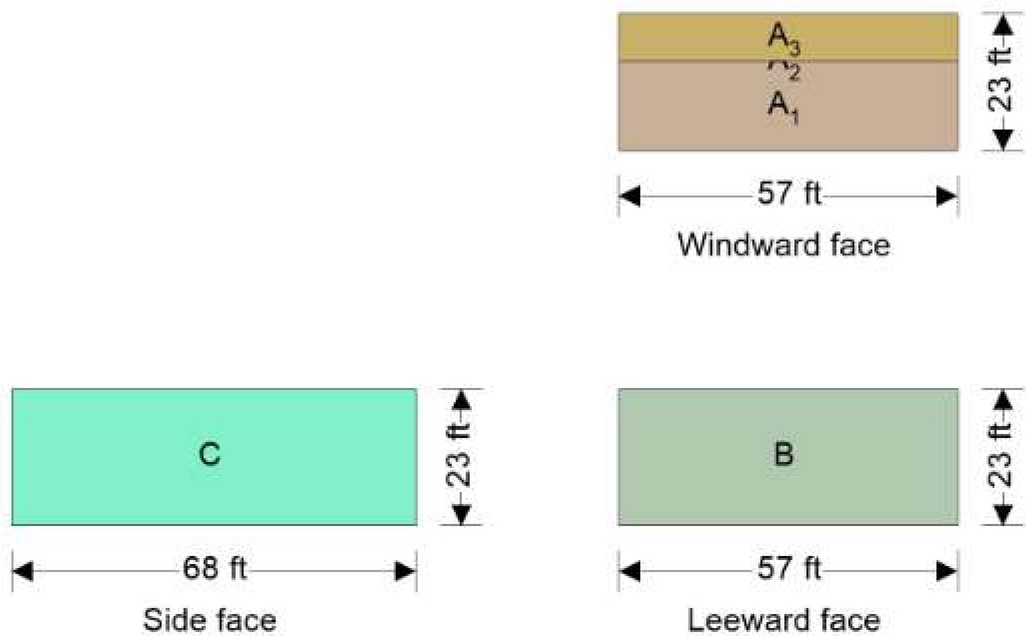
Side face



Leeward face



Plan view - Hipped roof



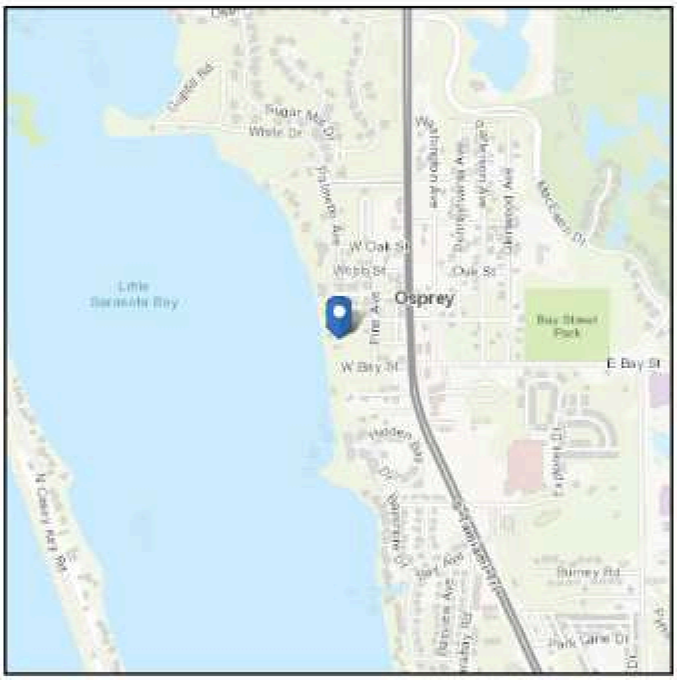
A_3
A_2
A_1
23 ft
57 ft
Windward face
C
23 ft
68 ft
Side face
B
23 ft
57 ft
Leeward face

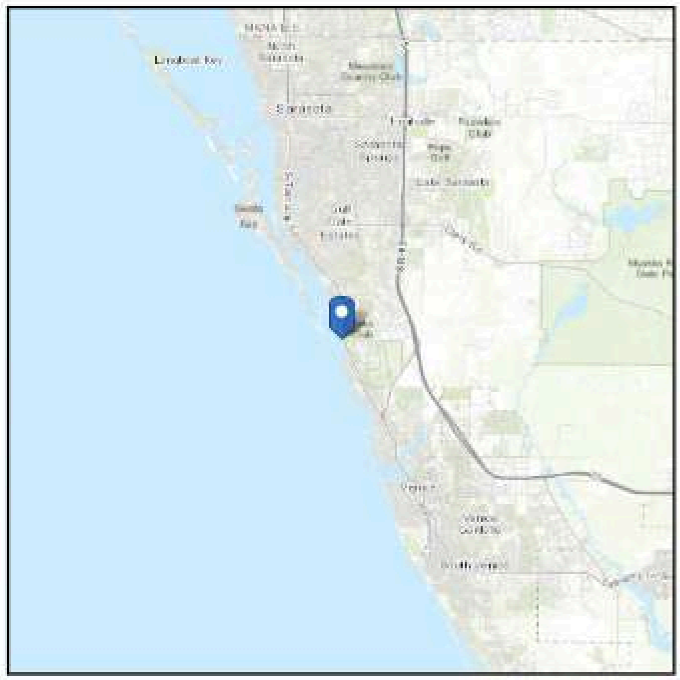
SEISMIC LOADS

ASCE Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-22	**Latitude:**	27.197025
Risk Category:	II	**Longitude:**	-82.492757
Soil Class:	Default	**Elevation:**	9.19234437880068 ft (NAVD 88)



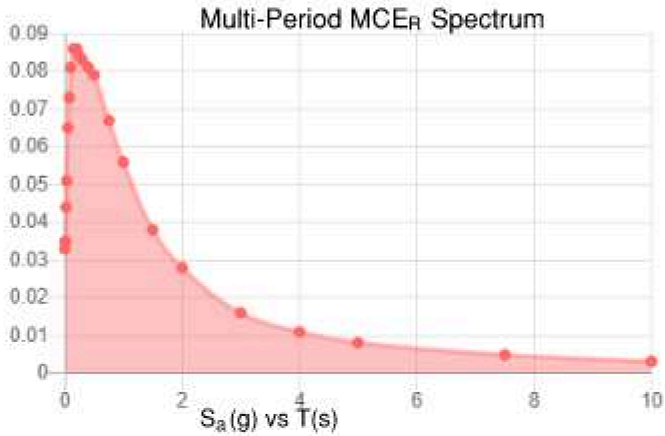


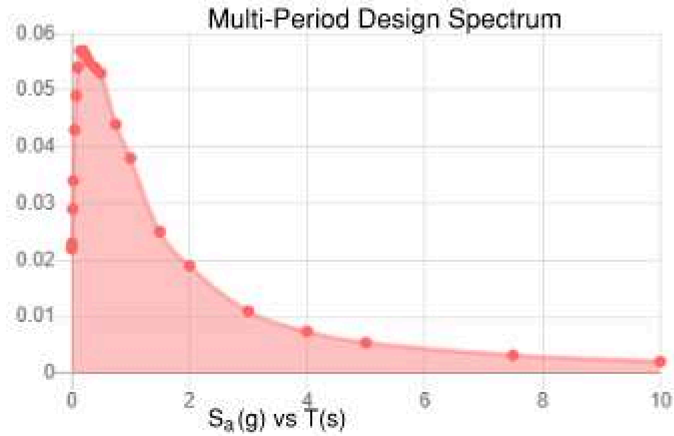
Site Soil Class: Default

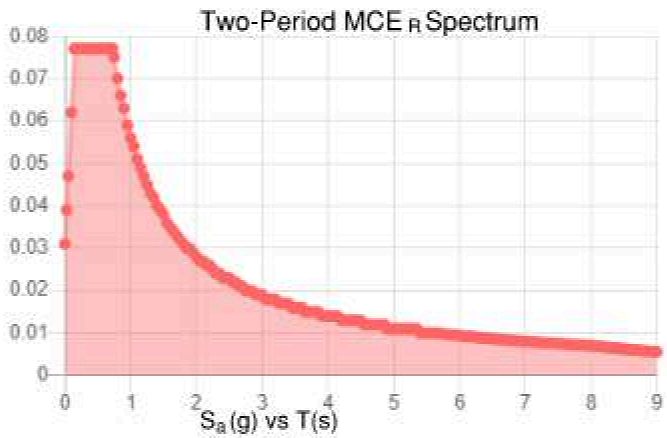
Results:

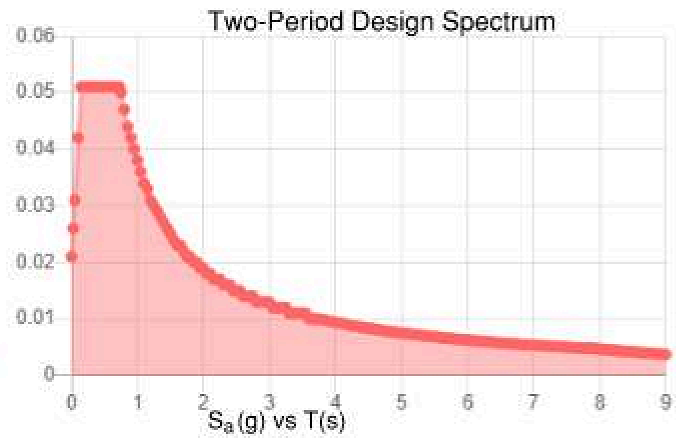
PGA_M :	0.028	T_L :	8
S_{MS} :	0.077	S_S :	0.057
S_{M1} :	0.056	S_1 :	0.026
S_{DS} :	0.051	V_{S30} :	260
S_{D1} :	0.038		

Seismic Design Category: A









MCE$_R$ Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Tue Jul 02 2024**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

2. STRUCTURAL ANALYSIS

2.1 BUILDING DESCRIPTION

The two-story building is complexly shaped in plan. The building is 69' long and 58' wide. Some of the second-floor walls are offset to the interior of the building resting on floor structures. Walls, floor, and roof trusses are designed with LGS structural framing.
The stability of the building is provided by a vertical x-bracing strap and sprayed concrete shear wall.
The foundation is slab-on-grade with footings where required for LGS columns.

The structure's design 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 & transverse direction is achieved due to the sprayed concrete shear wall and to the x-brace strap.

Structural analysis is done in SCIA Engineer 20 software. This software automatically determines 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".
The 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 APPLIED LOADS

Load cases

Name	Description Spec	Action type / Load type	Load Group	Direction	Duration
DL1	Self-weight	Permanent / Self-weight	DL	-Z	
DL2	Floor dead load	Permanent / Standard	DL		
DL3	Wall dead load	Permanent / Standard	DL		
DL4	Roof 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

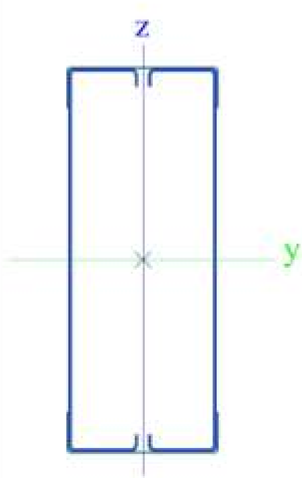
2.3 STEEL STRUCTURE CHECK
2.3.1 ROOF BEAM DESIGN

General scheme

Members number

Members cross-sections

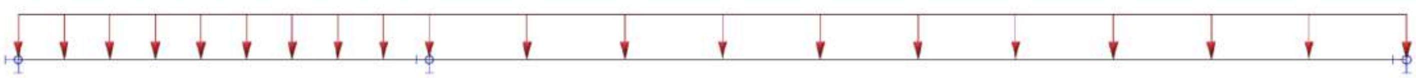
Cross-sections properties

BOX BEAM 14"X5.5"		
Type	14"X2.5"X12GA + 550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	4.683	
A_y [inch^2], A_z [inch^2]	1.460	2.759
A_L [inch^2/inch], A_D [inch^2/inch]	3.92e+01	8.62e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	4.487	9.843
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	136.415	24.819
i_y [inch], i_z [inch]	5.397	2.302
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	19.339	9.025
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	22.949	10.088
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.15e+03	1.15e+03
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	5.04e+02	5.04e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	58.329	165.707
β_y [inch], β_z [inch]	0.000	0.000
Picture		

APPLIED LOADS

LOAD ON ROOF BEAM			
Loading width, (ft)	15.563	Roof slope, α = 22.6	
Load	**Distr. Load, psf**	Sin α = 22.6	**Liner Load, plf**
Roof Dead Load	20.00		311.25
Roof LL	20.00		311.25
Roof Wind (up)	-53.50	0.3848	-320.36
Roof Wind (down)	17.30	0.3848	103.59

Wind loads are applied perpendicular to the roof beam. Liner load with "-" - uplift.



MAXIMUM FORCES

1D internal forces

Linear calculation
Combination: LRFD-Ult (auto)
Coordinate system: Principal
Extreme 1D: Member
Selection: All

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
RB1	17.833	LRFD-Ult (auto)/1	0.0	0.0	**-7962.3**	0.0	0.0	0.0
RB1	0.000	LRFD-Ult (auto)/1	**0.0**	**0.0**	**7962.3**	**0.0**	0.0	**0.0**
RB1	8.941	LRFD-Ult (auto)/2	0.0	0.0	0.6	0.0	**-1019.9**	0.0
RB1	8.941	LRFD-Ult (auto)/1	0.0	0.0	-21.9	0.0	**35499.3**	0.0
RB2	7.500	LRFD-Ult (auto)/1	0.0	0.0	**-3348.7**	0.0	0.0	0.0
RB2	0.000	LRFD-Ult (auto)/1	**0.0**	**0.0**	**3348.9**	**0.0**	0.0	**0.0**
RB2	3.794	LRFD-Ult (auto)/2	0.0	0.0	1.2	0.0	**-180.3**	0.0
RB2	3.794	LRFD-Ult (auto)/1	0.0	0.0	-39.3	0.0	**6278.0**	0.0

Name	Combination key
LRFD-Ult (auto)/1	1.20*DL1 + 1.60*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4
LRFD-Ult (auto)/2	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + W(down)

DISPLACEMENT

Load case DL + Lr, inch:

The maximum deflection for a web header is 0.378”.

According to TABLE 1604.3 the code IBC 2018, maximum member deflection should not exceed - L/240.
L=17’+10”=214”,   214”/240=0.891”.   0.378"< 0.891”. <u>Deflection is OK!</u>

<u>STEEL MEMBER RB1 CHECK</u>

AISI S100-16 LRFD Check

Member RB1	14"X2.5"X12GA + 550S150-54	A913 grade 50	LRFD-Ult (auto)	0.50

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

The critical check is on position 8.94 ft

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

Internal forces		
Pu	0.00	lbf
Vux	0.00	lbf
Vuy	-21.90	lbf
Mut	0.00	lbfft
Mux	35499.33	lbfft
Muy	0.00	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	0.576	-43.8 -47.8	-	-	-	-	-	- -	- -	-	- -	-
2	2.403	-48.0 -48.0	-	-	-	-	-	- -	- -	-	- -	-
3	11.108	39.7 -38.0	0.96	22.942	45.9	0.930	0.821	- 9.117	2.303 2.352	-	- -	-
4	2.403	49.7 49.7	1.00	4.000	414.1	0.346	1.000	- 2.403	1.202 1.202	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
5	0.577	49.5 45.4	0.92	0.459	340.9	0.381	1.000	0.577 -	- -	-	- -	-
6	0.075	50.0 49.5	0.99	4.021	53930.0	0.030	1.000	- 0.075	0.038 0.038	-	- -	-
7	0.076	50.0 50.0	1.00	4.000	53646.5	0.031	1.000	0.076 -	- -	-	- -	-
8	0.075	50.0 49.5	0.99	4.021	53930.0	0.030	1.000	- 0.075	0.038 0.038	-	- -	-
9	0.576	49.5 45.4	0.92	0.459	340.9	0.381	1.000	0.576 -	- -	-	- -	-
10	2.403	49.7 49.7	1.00	4.000	414.1	0.346	1.000	- 2.403	1.202 1.202	-	- -	-
11	11.108	39.7 -38.0	0.96	22.942	45.9	0.930	0.821	- 9.117	2.303 2.352	-	- -	-
12	2.403	-48.0 -48.0	-	-	-	-	-	- -	- -	-	- -	-
13	0.576	-43.8 -47.8	-	-	-	-	-	- -	- -	-	- -	-
14	0.075	-47.8 -48.4	-	-	-	-	-	- -	- -	-	- -	-
15	0.076	-48.4 -48.4	-	-	-	-	-	- -	- -	-	- -	-
16	0.075	-47.8 -48.4	-	-	-	-	-	- -	- -	-	- -	-
17	0.076	-48.4 -48.4	-	-	-	-	-	- -	- -	-	- -	-
22	1.398	49.5 39.7	0.80	4.411	1350.1	0.191	1.000	- 1.398	0.636 0.762	-	- -	-
26	1.398	-38.0 -47.8	-	-	-	-	-	- -	- -	-	- -	-
30	0.489	50.0 50.0	1.00	4.000	1278.8	0.198	1.000	- 0.489	0.244 0.244	-	- -	-
35	1.398	49.5 39.7	0.80	4.411	1350.1	0.191	1.000	- 1.398	0.636 0.762	-	- -	-
39	0.076	50.0 50.0	1.00	4.000	53646.5	0.031	1.000	0.076 -	- -	-	- -	-
44	1.398	-38.0 -47.8	-	-	-	-	-	- -	- -	-	- -	-
48	0.489	-48.4 -48.4	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	19.019	Inch ^{3}
Mnxo	79247.2	lbfft
Resistance factor	0.90	
Unity check	0.50	-

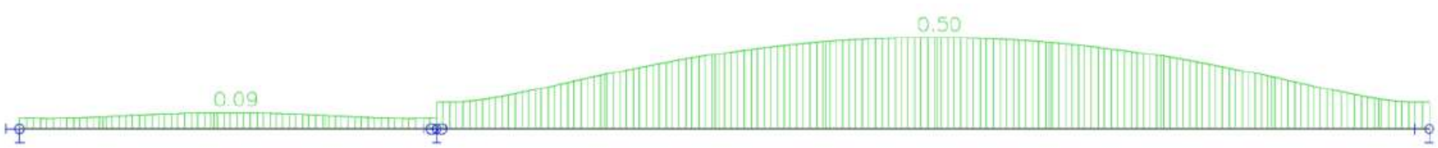
Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	2' 0.000"	ft
Sigma,ey	2634.0	ksi
Kt	1.00	
Lt	2' 0.000"	ft
Sigma,t	4547.0	ksi
Cb	1.00	
Sfx	19.339	$inch^3$
Fcre	4925.3	ksi

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

The member satisfies the check !

Unity check



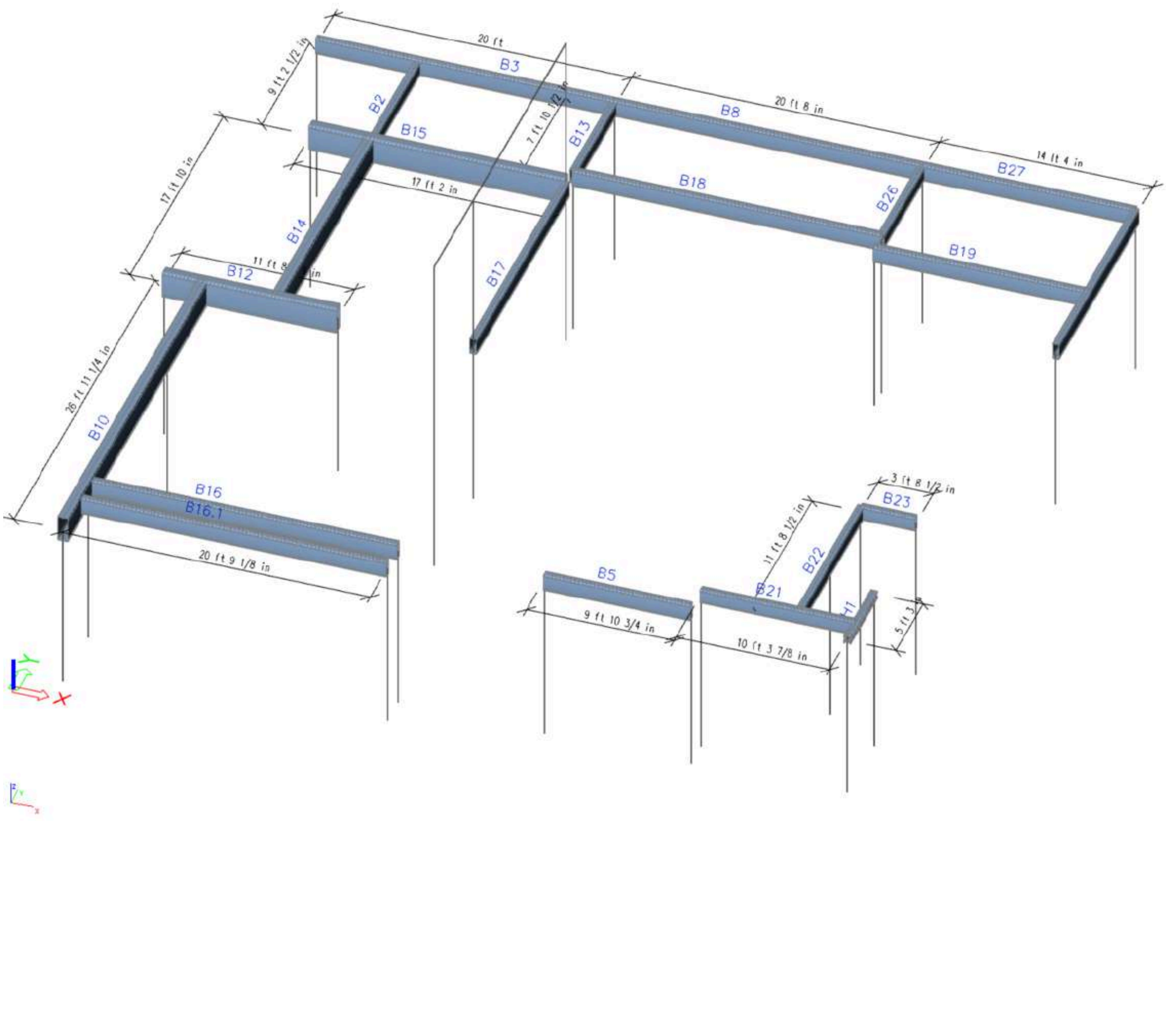
Check of steel

Linear calculation, Extreme : Member
Selection : RB1, RB2
Combinations : LRFD-Ult (auto)

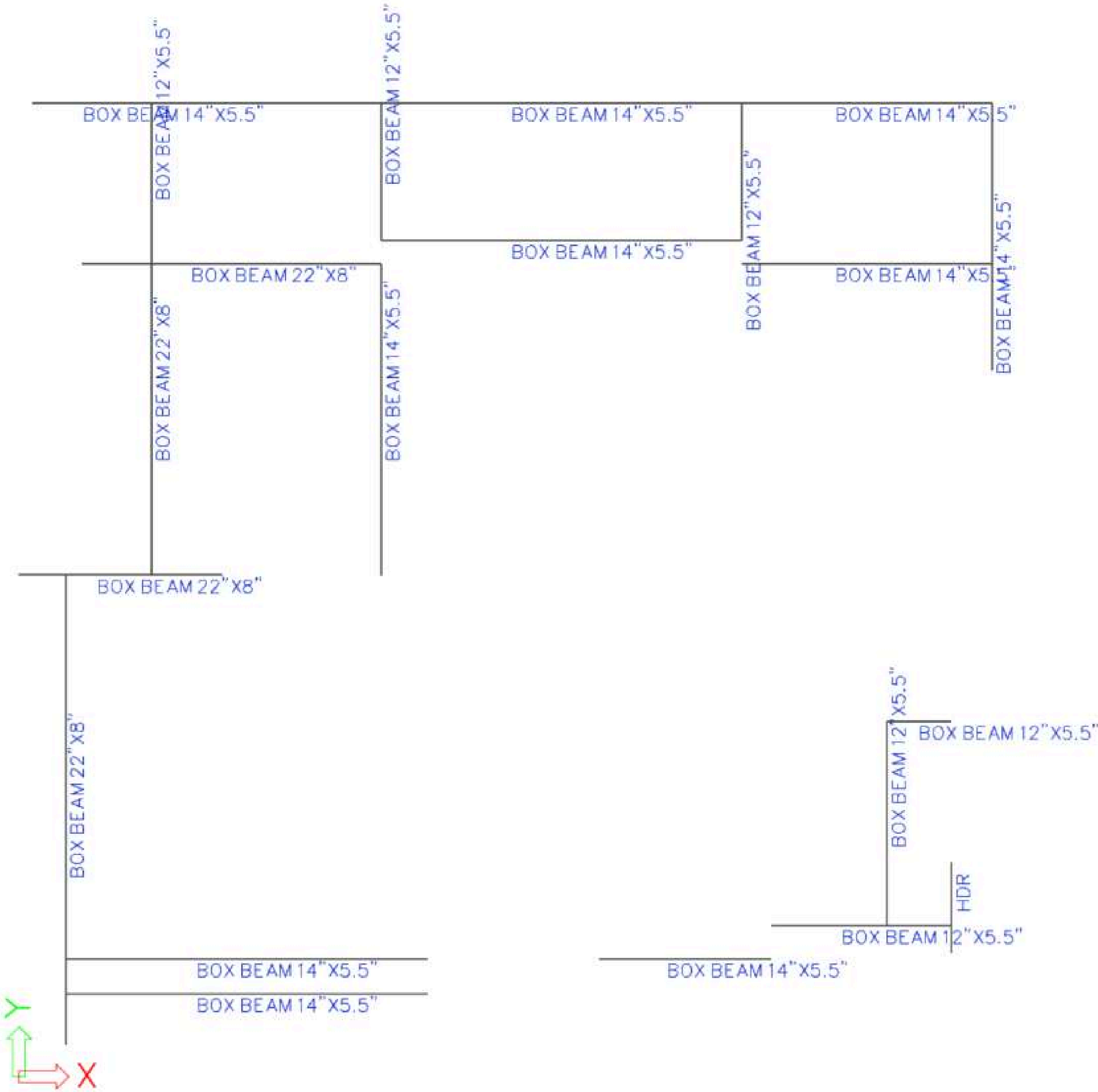
Case	Member	css	mat	dx [ft]	un.check [-]
LRFD-Ult (auto)/1	RB1	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	8' 11.294"	0.50
LRFD-Ult (auto)/1	RB2	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	3' 9.529"	0.09

2.3.2 FLOOR BEAM DESIGN

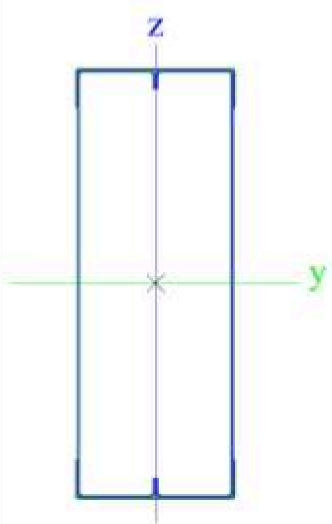
General scheme



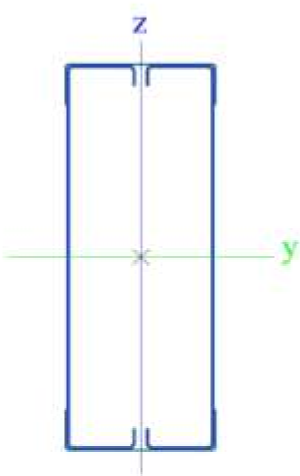
Members cross-sections



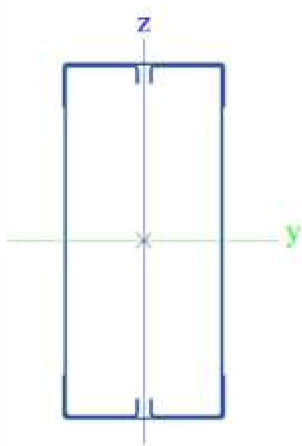
Cross-section properties BOX BEAM 22"X8"

BOX BEAM 22"X8"		
Type	22"X4"X10GA + 8.125"X2"X14GA	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [$inch^2$]	8.965	
A_y [$inch^2$], A_z [$inch^2$]	3.096	5.255
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	6.06e+01	1.33e+02
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	5.222	9.901
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	648.945	103.930
i_y [inch], i_z [inch]	8.508	3.405
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	58.633	25.504
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	69.249	28.232
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3.46e+03	3.46e+03
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.41e+03	1.41e+03
d_y [inch], d_z [inch]	0.000	0.000
I_t [$inch^4$], I_w [$inch^6$]	247.067	2208.056
β_y [inch], β_z [inch]	0.000	0.000
Picture		

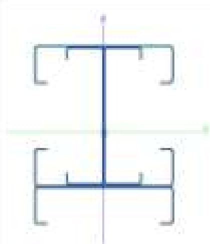
Cross-section properties BOX BEAM 14"X5.5"

BOX BEAM 14"X5.5"		
Type	14"X2.5"X12GA + 550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	4.732	
A_y [inch^2], A_z [inch^2]	1.480	2.765
A_L [inch^2/inch], A_D [inch^2/inch]	3.92e+01	8.72e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	4.487	9.843
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	138.348	24.822
i_y [inch], i_z [inch]	5.407	2.290
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	19.613	9.026
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	23.255	10.099
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.16e+03	1.16e+03
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	5.05e+02	5.05e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	58.329	165.829
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

Cross-section properties BOX BEAM 12"X5.5"

BOX BEAM 12"X5.5"		
Type	12"X2.5"X14GA + 550S150-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	3.309	
A_y [inch^2], A_z [inch^2]	1.197	1.721
A_L [inch^2/inch], A_D [inch^2/inch]	3.52e+01	7.72e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	4.487	10.843
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	76.771	16.838
i_y [inch], i_z [inch]	4.817	2.256
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	12.681	6.123
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	14.672	6.906
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	7.34e+02	7.34e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.45e+02	3.45e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	35.872	103.427
β_y [inch], β_z [inch]	0.000	0.000
Picture		

Cross-section properties HDR

HDR		
Type	5x550S150-54	
Shape type	Thin-walled	
Item material	A572 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	2.456	
A_y [inch^2], A_z [inch^2]	1.314	0.859
A_L [inch^2/inch], A_D [inch^2/inch]	5.94e+01	6.08e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1.322	-0.372
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	14.396	6.806
i_y [inch], i_z [inch]	2.421	1.665
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	3.928	2.475
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	5.311	3.160
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.66e+02	2.66e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.58e+02	1.58e+02
d_y [inch], d_z [inch]	0.000	-0.043
I_t [inch^4], I_w [inch^6]	0.009	61.705
$β_y$ [inch], $β_z$ [inch]	0.634	0.000
Picture		

APPLIED LOADS

Wind loads are applied perpendicular to the roof beam. Liner load with “-” - uplift.

LOAD ON FLOOR BEAM B2, B13					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	1.00		15.00	
Wall Dead Load	45	5.33		240.03	
Roof Dead Load	20.00	8.88		177.50	
Floor Live Load	40.00	1.00		40.00	
Roof Live Load	20.00	8.88		177.50	
Roof Wind (up)	-53.50	8.88	0.3848	-182.70	
Roof Wind (down)	17.30	8.88	0.3848	59.08	

LOAD ON FLOOR BEAM B3, B27					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	4.67		70.00	
Wall Dead Load					
Roof Dead Load					
Floor Live Load	40.00	4.67		186.67	
Roof Live Load					
Roof Wind (up)	-53.50	4.50	0.3848	-92.64	only for B27
Roof Wind (down)	17.30	4.50	0.3848	29.96	only for B27

LOAD ON FLOOR BEAM B5					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	1.00		15.00	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	2.00		40.00	
Floor Live Load	40.00	1.50		60.00	
Roof Live Load	20.00	2.00		40.00	
Roof Wind (up)	-53.50	2.00	0.3848	-41.17	
Roof Wind (down)	17.30	2.00	0.3848	13.31	

LOAD ON FLOOR BEAM B8					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load					
Wall Dead Load	45	1.33		60.00	
Roof Dead Load	20.00	4.00		80.00	
Floor Live Load					
Roof Live Load	20.00	4.00		80.00	
Roof Wind (up)	-53.50	4.00	0.3848	-82.34	
Roof Wind (down)	17.30	4.00	0.3848	26.63	

LOAD ON FLOOR BEAM B10					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	7.75/2		#VALUE!	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	13.00		260.00	
Floor Live Load	40.00	10.40		416.00	
Roof Live Load	20.00	13.00		260.00	
Roof Wind (up)	-53.50	13.00	0.3848	-267.61	
Roof Wind (down)	17.30	13.00	0.3848	86.54	

LOAD ON FLOOR BEAM B12					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	3.88		58.13	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	3.00		60.00	
Floor Live Load	40.00	3.88		155.00	
Roof Live Load	20.00	3.00		60.00	
Roof Wind (up)	-53.50	3.00	0.3848	-61.76	
Roof Wind (down)	17.30	3.00	0.3848	19.97	

LOAD ON FLOOR BEAM B14					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	6.58		98.70	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	12.88		257.50	
Floor Live Load	40.00	6.58		263.20	
Roof Live Load	20.00	12.88		257.50	
Roof Wind (up)	-53.50	12.88	0.3848	-265.04	
Roof Wind (down)	17.30	12.88	0.3848	85.70	

LOAD ON FLOOR BEAM B15					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	5.67		85.00	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	2.00		40.00	
Floor Live Load	40.00	5.60		224.00	
Roof Live Load	20.00	2.00		40.00	
Roof Wind (up)	-53.50	2.00	0.3848	-41.17	
Roof Wind (down)	17.30	2.00	0.3848	13.31	

LOAD ON FLOOR BEAM B16					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	2.00		30.00	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	4.00		80.00	
Floor Live Load	40.00	2.00		80.00	
Roof Live Load	20.00	4.00		80.00	
Roof Wind (up)	-53.50	4.00	0.3848	-82.34	
Roof Wind (down)	17.30	4.00	0.3848	26.63	

LOAD ON FLOOR BEAM B16.1					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	1.00		15.00	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	4.00		80.00	
Floor Live Load	40.00	1.00		40.00	
Roof Live Load	20.00	4.00		80.00	
Roof Wind (up)	-53.50	4.00	0.3848	-82.34	
Roof Wind (down)	17.30	4.00	0.3848	26.63	

LOAD ON FLOOR BEAM B17					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	6.58		98.70	
Wall Dead Load					
Roof Dead Load					
Floor Live Load	40.00	6.58		263.20	
Roof Live Load					
Roof Wind (up)					
Roof Wind (down)					

LOAD ON FLOOR BEAM B18					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load					
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	4.00		80.00	
Floor Live Load					
Roof Live Load	20.00	4.00		80.00	
Roof Wind (up)	-53.50	4.00	0.3848	-82.34	
Roof Wind (down)	17.30	4.00	0.3848	26.63	

LOAD ON FLOOR BEAM B19					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	5.67		85.00	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	2.00		40.00	
Floor Live Load	40.00	5.60		224.00	
Roof Live Load	20.00	2.00		40.00	
Roof Wind (up)	-53.50	2.00	0.3848	-41.17	
Roof Wind (down)	17.30	2.00	0.3848	13.31	

LOAD ON FLOOR BEAM B21					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	1.00		15.00	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	2.50		50.00	
Floor Live Load	40.00	1.00		40.00	
Roof Live Load	20.00	2.50		50.00	
Roof Wind (up)	-53.50	2.50	0.3848	-51.46	
Roof Wind (down)	17.30	2.50	0.3848	16.64	

LOAD ON FLOOR BEAM B22					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	9.29		139.38	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	13.50		270.00	
Floor Live Load	40.00	9.29		371.67	
Roof Live Load	20.00	13.50		270.00	
Roof Wind (up)	-53.50	13.50	0.3848	-277.91	
Roof Wind (down)	17.30	13.50	0.3848	89.87	

LOAD ON FLOOR BEAM B23					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	1.00		15.00	
Wall Dead Load	45	10.00		450.00	
Roof Dead Load	20.00	4.00		80.00	
Floor Live Load	40.00	1.00		40.00	
Roof Live Load	20.00	4.00		80.00	
Roof Wind (up)	-53.50	4.00	0.3848	-82.34	
Roof Wind (down)	17.30	4.00	0.3848	26.63	

LOAD ON FLOOR BEAM B25, B26					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load	15	1.00		15.00	
Wall Dead Load	45	5.33		240.03	
Roof Dead Load	20.00	10.17		203.33	
Floor Live Load	40.00	1.00		40.00	
Roof Live Load	20.00	10.17		203.33	
Roof Wind (up)	-53.50	10.17	0.3848	-209.29	
Roof Wind (down)	17.30	10.17	0.3848	67.68	

MAXIMUM FORCES

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
B15	17' 2.000"	LRFD-Ult (auto)/1	0.0	**-9.8**	**-12845.0**	388.0	0.0	0.0
B15	0.000"	LRFD-Ult (auto)/1	**0.0**	**32.3**	**19181.3**	**388.0**	**0.0**	**0.0**
B15	0.000"	LRFD-Ult (auto)/2	0.0	12.0	6795.6	**143.5**	0.0	0.0
B15	4' 0.000"-	LRFD-Ult (auto)/1	0.0	32.3	18762.1	388.0	**75887.1**	**129.3**
B17	0.000"	LRFD-Ult (auto)/2	**-1.6**	**0.0**	933.3	**0.0**	**0.0**	**0.0**
B17	17' 10.000"	LRFD-Ult (auto)/1	-4.2	0.0	**-5003.8**	0.0	0.0	0.0
B17	9' 0.000"-	LRFD-Ult (auto)/1	**-4.2**	0.0	-46.7	0.0	**22307.4**	0.0
B17	0.000"	LRFD-Ult (auto)/1	-4.2	0.0	**5003.7**	0.0	0.0	**0.0**
B10	0.000"	LRFD-Ult (auto)/2	0.0	**0.0**	760.2	**0.1**	0.0	0.0
B10	19' 11.200"-	LRFD-Ult (auto)/1	0.0	0.0	**-15186.2**	0.3	**-29670.9**	0.1
B10	12' 0.000"-	LRFD-Ult (auto)/1	**0.0**	**0.0**	-375.6	**0.3**	**32059.5**	-0.1
B10	4' 11.199"+	LRFD-Ult (auto)/1	0.0	0.0	**12817.0**	0.3	-11899.7	**-0.3**
B10	4' 11.199"-	LRFD-Ult (auto)/1	0.0	0.0	-5361.3	0.3	-11899.7	**0.1**
B16	20' 9.103"	LRFD-Ult (auto)/1	0.0	0.0	**-4913.5**	0.0	0.0	0.0
B16	0.000"	LRFD-Ult (auto)/1	**0.0**	**0.0**	**5087.3**	**0.0**	**0.0**	**0.0**
B16	10' 0.000"-	LRFD-Ult (auto)/1	0.0	0.0	-15.1	0.0	**18010.1**	0.0
B14	17' 10.000"	LRFD-Ult (auto)/1	0.0	0.0	**-13883.8**	-2908.0	0.0	0.0
B14	0.000"	LRFD-Ult (auto)/1	**0.0**	**0.0**	**13883.8**	-2908.0	**0.0**	**0.0**
B14	9' 0.000"-	LRFD-Ult (auto)/1	0.0	0.0	-129.7	**-2908.0**	**61894.3**	0.0
B14	0.000"	LRFD-Ult (auto)/2	0.0	0.0	4367.8	**-1079.0**	0.0	0.0
B16.1	20' 9.103"	LRFD-Ult (auto)/3	0.0	0.0	**-4462.6**	0.0	0.0	0.0
B16.1	0.000"	LRFD-Ult (auto)/3	**0.0**	**0.0**	**4232.3**	**0.0**	**0.0**	**0.0**
B16.1	11' 0.000"-	LRFD-Ult (auto)/3	0.0	0.0	-393.9	0.0	**33242.2**	0.0
B5	9' 10.075"	LRFD-Ult (auto)/1	0.0	0.0	**-3647.2**	0.0	0.0	0.0
B5	0.000"	LRFD-Ult (auto)/1	**0.0**	**0.0**	**3647.2**	**0.0**	**0.0**	**0.0**
B5	4' 11.037"	LRFD-Ult (auto)/1	0.0	0.0	0.0	0.0	**8971.7**	0.0

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
B2	0.000"	LRFD-Ult (auto)/2	**-15.6**	**0.0**	993.7	64.5	**0.0**	**0.0**
B2	9' 2.440"	LRFD-Ult (auto)/4	-40.0	0.0	**-3889.0**	9.3	0.0	0.0
B2	0.000"	LRFD-Ult (auto)/4	-40.0	0.0	**3889.0**	9.3	0.0	0.0
B2	0.000"	LRFD-Ult (auto)/5	-22.9	0.0	1605.6	**73.5**	0.0	0.0
B2	5' 3.109"	LRFD-Ult (auto)/4	-40.0	0.0	-555.6	9.3	**8765.2**	0.0
B2	0.000"	LRFD-Ult (auto)/1	**-42.2**	0.0	3149.5	**-201.0**	0.0	**0.0**
B3	20' 0.000"	LRFD-Ult (auto)/1	0.0	**-27.8**	**-3924.7**	-333.0	0.0	0.0
B3	0.000"	LRFD-Ult (auto)/1	**0.0**	**14.4**	**4617.5**	-333.0	**0.0**	**0.0**
B3	13' 2.000"+	LRFD-Ult (auto)/1	0.0	-27.8	-3791.0	**-333.0**	26361.2	**189.6**
B3	0.000"	LRFD-Ult (auto)/2	0.0	5.3	937.2	**-123.2**	0.0	0.0
B3	11' 2.000"-	LRFD-Ult (auto)/1	0.0	14.4	155.6	-333.0	**26648.6**	160.8
B8	20' 7.920"	LRFD-Ult (auto)/4	0.0	0.0	**-3399.7**	-400.5	0.0	0.0
B8	0.000"	LRFD-Ult (auto)/4	**0.0**	**0.0**	**3399.7**	-400.5	**0.0**	**0.0**
B8	0.000"	LRFD-Ult (auto)/1	0.0	0.0	2351.2	**-422.1**	0.0	0.0
B8	0.000"	LRFD-Ult (auto)/2	0.0	0.0	606.5	**-156.1**	0.0	0.0
B8	10' 3.960"-	LRFD-Ult (auto)/4	0.0	0.0	0.3	-400.5	**17558.7**	0.0
B26	0.000"	LRFD-Ult (auto)/2	**-2.6**	**0.0**	842.6	**0.0**	**0.0**	**0.0**
B26	7' 10.440"	LRFD-Ult (auto)/4	-6.6	0.0	**-3649.8**	0.0	0.0	0.0
B26	0.000"	LRFD-Ult (auto)/4	-6.6	0.0	**3649.8**	0.0	0.0	0.0
B26	3' 11.220"	LRFD-Ult (auto)/4	-6.6	0.0	0.0	0.0	**7181.0**	0.0
B26	0.000"	LRFD-Ult (auto)/1	**-7.0**	0.0	2880.5	0.0	0.0	**0.0**
B27	0.000"	LRFD-Ult (auto)/1	0.0	-24.8	**2861.2**	**-803.9**	0.0	**9.5**
B27	0.000"	LRFD-Ult (auto)/2	**0.0**	**-9.2**	555.3	**-297.3**	**0.0**	3.5
B27	7' 1.980"	LRFD-Ult (auto)/1	0.0	**-24.8**	0.0	-803.9	**10250.3**	-168.4
B27	14' 3.960"	LRFD-Ult (auto)/1	0.0	-24.8	**-2861.2**	-803.9	0.0	**-346.4**

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
B19	14' 3.960"	LRFD-Ult (auto)/1	0.0	0.0	**-7805.1**	389.9	0.0	0.0
B19	0.000"	LRFD-Ult (auto)/1	**0.0**	**0.0**	**7805.1**	389.9	**0.0**	**0.0**
B19	0.000"	LRFD-Ult (auto)/2	0.0	0.0	3518.0	**145.4**	0.0	0.0
B19	0.000"	LRFD-Ult (auto)/3	0.0	0.0	6347.0	**402.6**	0.0	0.0
B19	7' 1.980"	LRFD-Ult (auto)/1	0.0	0.0	0.0	389.9	**27961.7**	0.0
B22	0.000"	LRFD-Ult (auto)/3	**-1.5**	-15.6	2996.4	-187.5	0.0	**0.0**
B22	5' 10.200"+	LRFD-Ult (auto)/6	**2.9**	10.9	4975.1	130.3	-8723.2	-70.7
B22	5' 10.200"+	LRFD-Ult (auto)/1	1.8	**12.7**	**6969.7**	**152.0**	-11073.6	-82.5
B22	2' 4.080"	LRFD-Ult (auto)/1	-1.4	**-15.9**	-860.6	**-190.6**	**2847.8**	-37.2
B22	5' 10.200"-	LRFD-Ult (auto)/1	-1.4	-15.9	**-7093.4**	-190.6	**-11111.4**	**-92.9**
B23	3' 8.400"	LRFD-Ult (auto)/3	0.0	0.0	**-1508.7**	0.0	0.0	0.0
B23	0.000"	LRFD-Ult (auto)/3	**0.0**	**0.0**	**1508.7**	**0.0**	**0.0**	**0.0**
B23	2' 5.600"	LRFD-Ult (auto)/3	0.0	0.0	-502.9	0.0	**1240.5**	0.0
H1	5' 2.880"	LRFD-Ult (auto)/4	0.0	0.0	**-2139.3**	0.0	0.0	0.0
H1	0.000"	LRFD-Ult (auto)/3	**0.0**	**0.0**	**3730.9**	**0.0**	**0.0**	**0.0**
H1	1' 6.480"-	LRFD-Ult (auto)/3	0.0	0.0	3174.0	0.0	**5316.8**	0.0
B12	11' 8.004"	LRFD-Ult (auto)/1	0.0	0.0	**-13165.3**	0.0	0.0	0.0
B12	7' 7.000"+	LRFD-Ult (auto)/1	**0.0**	**0.0**	-11707.8	**0.0**	**50787.6**	**0.0**
B12	0.000"	LRFD-Ult (auto)/1	**0.0**	0.0	**9464.5**	0.0	**-0.3**	**0.0**
B21	10' 3.840"	LRFD-Ult (auto)/1	0.0	0.0	**-3940.6**	16.8	0.0	0.0
B21	0.000"	LRFD-Ult (auto)/2	-6.9	0.5	2181.6	**5.6**	-82.3	-3.1
B21	0.000"	LRFD-Ult (auto)/1	**-15.9**	1.4	**4485.5**	16.8	**-190.6**	-9.3
B21	6' 7.440"+	LRFD-Ult (auto)/1	**0.0**	**0.0**	-3570.5	16.8	**13905.7**	**0.0**
B21	0.000"	LRFD-Ult (auto)/3	-15.6	**1.5**	4463.2	**17.5**	-187.5	**-9.6**

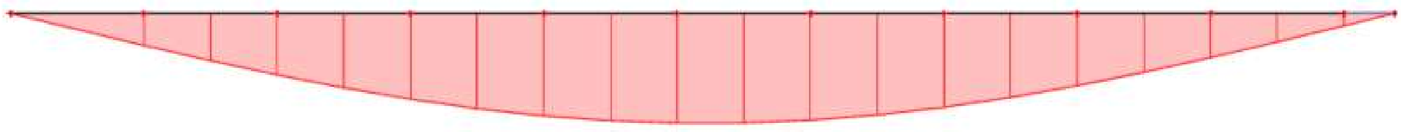
Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
B25	15' 3.960"	LRFD-Ult (auto)/1	-235.1	21.6	**-13326.2**	259.3	0.0	-6.9
B25	0.000"	LRFD-Ult (auto)/3	-234.7	21.6	**10119.1**	258.8	-3321.6	-337.5
B25	9' 2.440"+	LRFD-Ult (auto)/1	**-235.1**	**21.6**	-4616.2	**259.3**	**54960.0**	-139.3
B25	0.000"	LRFD-Ult (auto)/1	-235.1	21.6	9391.7	259.3	**-3327.5**	**-338.1**
B25	15' 3.960"	LRFD-Ult (auto)/2	**-87.0**	**8.0**	-4643.2	**95.9**	0.0	**-2.5**
B18	0.000"	LRFD-Ult (auto)/2	0.0	**0.0**	2959.1	**-30.6**	0.0	0.0
B18	18' 7.920"+	LRFD-Ult (auto)/4	**0.0**	0.1	**6536.3**	-78.5	-11478.9	-0.2
B18	17' 0.920"-	LRFD-Ult (auto)/4	0.0	0.1	**-7873.4**	-78.5	**-18228.1**	1.5
B18	7' 3.960"-	LRFD-Ult (auto)/4	0.0	0.1	-104.2	-78.5	**20649.2**	0.6
B18	18' 7.920"+	LRFD-Ult (auto)/1	0.0	0.1	5655.0	-82.7	-9919.1	**-0.2**
B18	17' 0.920"-	LRFD-Ult (auto)/1	0.0	**0.1**	-6866.0	**-82.7**	-15824.6	**1.6**
B13	0.000"	LRFD-Ult (auto)/1	**7.0**	**0.0**	2693.2	**0.0**	**0.0**	**0.0**
B13	7' 10.440"	LRFD-Ult (auto)/4	6.6	0.0	**-3325.5**	0.0	0.0	0.0
B13	0.000"	LRFD-Ult (auto)/4	6.6	0.0	**3325.5**	0.0	0.0	0.0
B13	3' 11.220"	LRFD-Ult (auto)/4	6.6	0.0	0.0	0.0	**6543.0**	0.0
B13	0.000"	LRFD-Ult (auto)/2	**2.6**	0.0	849.7	0.0	0.0	**0.0**

Name	Combination key
LRFD-Ult (auto)/1	1.20*DL1 + 1.60*L + 0.50*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4
LRFD-Ult (auto)/2	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + W(up)
LRFD-Ult (auto)/3	1.20*DL1 + 0.50*L + 1.60*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4
LRFD-Ult (auto)/4	1.20*DL1 + 1.60*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*W(down)
LRFD-Ult (auto)/5	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + W(up)
LRFD-Ult (auto)/6	1.40*DL1 + 1.40*DL2 + 1.40*DL3 + 1.40*DL4

DISPLACEMENT

Load case DL + Lr, inch:

Maximum displacement per beam B16.1



The maximum deflection for a web header is 0.518”.

According to TABLE 1604.3 the code IBC 2018, maximum member deflection should not exceed - L/240. L=20’+9”=249”, 249”/240=1.037”. 0.518"< 1.037”. <u>Deflection is OK!</u>

STEEL MEMBER B15 CHECK

AISI S100-16 LRFD Check

Member B15	22"X4"X10GA + 8.125"X2"X14GA	A913 grade 50	LRFD-Ult (auto)	0.44

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

The critical check is on position 4.00 ft

Axis definition :

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

Internal forces		
Pu	-0.00	lbf
Vux	32.33	lbf
Vuy	18762.11	lbf
Mut	387.98	lbfft
Mux	75887.11	lbfft
Muy	129.33	lbfft

Combined Bending and Torsional Loading

According to article H4 and formula (H4-1)

Table of values		
Critical fibre	93	
Sigma Mx	15.6	ksi
Sigma My	0.1	ksi
f bending	15.7	ksi
Tau t	-0.1	ksi
f torsion	-0.1	ksi
Composed Stress	15.7	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	0.941	-41.7 -45.8	-	-	-	-	-	- -	- -	-	- -	-
2	3.882	-46.0 -46.0	-	-	-	-	-	- -	- -	-	- -	-
3	18.136	41.4 -37.7	0.91	21.738	24.1	1.310	0.635	- 11.517	2.946 3.086	-	- -	-
4	3.882	49.7 49.7	1.00	4.000	240.7	0.455	1.000	3.882 -	- -	-	- -	-
5	0.941	49.6 45.5	0.92	0.460	189.5	0.512	1.000	0.941 -	- -	-	- -	-
6	0.941	49.6 45.5	0.92	0.460	189.5	0.512	1.000	0.941 -	- -	-	- -	-
7	3.882	49.7 49.7	1.00	4.000	240.7	0.455	1.000	- 3.882	1.941 1.941	-	- -	-
8	18.136	41.4 -37.7	0.91	21.738	24.1	1.310	0.635	- 11.517	2.946 3.086	-	- -	-
9	3.882	-46.0 -46.0	-	-	-	-	-	- -	- -	-	- -	-
10	0.941	-41.7 -45.8	-	-	-	-	-	- -	- -	-	- -	-
11	0.093	50.0 49.6	0.99	4.016	56293.5	0.030	1.000	- 0.093	0.046 0.047	-	- -	-
12	0.093	50.0 50.0	1.00	4.000	56066.0	0.030	1.000	0.093 -	- -	-	- -	-
13	0.093	50.0 49.6	0.99	4.016	56293.5	0.030	1.000	- 0.093	0.046 0.047	-	- -	-
14	0.093	-45.8 -46.2	-	-	-	-	-	- -	- -	-	- -	-
15	0.093	-46.2 -46.2	-	-	-	-	-	- -	- -	-	- -	-
16	0.093	-45.8 -46.2	-	-	-	-	-	- -	- -	-	- -	-
17	0.093	-46.2 -46.2	-	-	-	-	-	- -	- -	-	- -	-
22	1.873	49.6 41.4	0.84	4.338	1121.7	0.210	1.000	- 1.873	0.865 1.008	-	- -	-
26	1.873	-37.7 -45.8	-	-	-	-	-	- -	- -	-	- -	-
30	0.132	50.0 50.0	1.00	4.000	27830.3	0.042	1.000	0.132 -	- -	-	- -	-
35	0.093	50.0 50.0	1.00	4.000	56066.0	0.030	1.000	0.093 -	- -	-	- -	-
40	1.873	49.6 41.4	0.84	4.338	1121.7	0.210	1.000	- 1.873	0.865 1.008	-	- -	-
44	1.873	-37.7 -45.8	-	-	-	-	-	- -	- -	-	- -	-
48	0.132	-46.2 -46.2	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	56.037	$inch^3$
Mnxo	233480.6	lbfft
Resistance factor	0.90	
Unity check	0.36	-

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

Table of values		
Lltb	2' 0.000"	ft
Sigma,ey	5762.1	ksi
Kt	1.00	
Lt	2' 0.000"	ft
Sigma,t	5119.0	ksi
Cb	1.25	
Sfx	58.633	$inch^3$
Fcre	9494.9	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	488.4	ksi
Kt	1.00	
Lt	2' 0.000"	ft
Sigma,t	5119.0	ksi
Cb	1.00	
Sfy	25.504	$inch^3$
Fcre	5093.1	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	0.111	3331.1
2	0.000	0.0
3	2.140	12686.0
4	0.000	0.0
5	0.111	3331.1
6	0.111	3331.1
7	0.000	0.0
8	2.140	12686.0
9	0.000	0.0
10	0.111	3331.1
11	0.006	189.7
12	0.000	0.0
13	0.006	189.7
14	0.006	189.7
15	0.000	0.0

16	0.006	189.7
17	0.000	0.0
22	0.348	10451.3
26	0.348	10451.3
30	0.000	0.0
35	0.000	0.0
40	0.348	10451.3
44	0.348	10451.3
48	0.000	0.0

Table of values		
Vn,y	81260.8	lbf
Resistance factor	0.95	
Unity check	0.24	-

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

Table of values		
Mnxo	233480.6	lbfft
Vny	81260.8	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.13+0.06) = 0.44

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

Table of values		
Mnx	233480.6	lbfft
Mny	64941.2	lbfft
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

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

STEEL MEMBER B22 CHECK

AISI S100-16 LRFD Check

Member B22	12"X2.5"X14GA + 550S150-54	A913 grade 50	LRFD-Ult (auto)	0.31

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

The critical check is on position 5.85 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	-1.40	lbf
Vux	-15.88	lbf
Vuy	-7093.37	lbf
Mut	-190.58	lbfft
Mux	-11111.41	lbfft
Muy	-92.91	lbfft

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

Table of values		
Critical fibre	45	
Sigma Mx	10.5	ksi
Sigma My	0.2	ksi
f bending	10.7	ksi
Tau t	0.1	ksi
f torsion	0.1	ksi
Composed Stress	10.7	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	0.591	49.5 44.6	0.90	0.466	161.7	0.553	1.000	0.591 -	- -	-	- -	-
2	2.432	49.7 49.7	1.00	4.000	263.9	0.434	1.000	- 2.432	1.216 1.216	-	- -	-
3	9.108	37.8 -37.8	1.00	24.000	35.1	1.038	0.759	- 6.915	1.729 3.458	-	- -	-
4	2.432	-49.7 -49.7	-	-	-	-	-	- -	- -	-	- -	-
5	0.591	-44.6 -49.5	-	-	-	-	-	- -	- -	-	- -	-

6	0.061	-49.5 -50.0	-	-	-	-	-	- -	- -	-	- -	-
7	0.061	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-
8	0.061	-49.5 -50.0	-	-	-	-	-	- -	- -	-	- -	-
9	0.591	-44.6 -49.5	-	-	-	-	-	- -	- -	-	- -	-
10	2.432	-49.7 -49.7	-	-	-	-	-	- -	- -	-	- -	-
11	9.108	37.8 -37.8	1.00	24.000	35.1	1.038	0.759	- 6.915	1.729 3.458	-	- -	-
12	2.432	49.7 49.7	1.00	4.000	263.9	0.434	1.000	- 2.432	1.216 1.216	-	- -	-
13	0.591	49.5 44.6	0.90	0.466	161.7	0.553	1.000	0.591 -	- -	-	- -	-
14	0.061	50.0 49.5	0.99	4.020	82597.8	0.025	1.000	- 0.061	0.030 0.031	-	- -	-
15	0.061	50.0 50.0	1.00	4.000	82181.8	0.025	1.000	- 0.061	0.031 0.031	-	- -	-
16	0.061	50.0 49.5	0.99	4.020	82597.8	0.025	1.000	- 0.061	0.030 0.031	-	- -	-
17	0.061	50.0 50.0	1.00	4.000	82181.8	0.025	1.000	0.061 -	- -	-	- -	-
22	1.412	-37.8 -49.5	-	-	-	-	-	- -	- -	-	- -	-
26	1.412	49.5 37.8	0.76	4.500	880.7	0.237	1.000	- 1.412	0.631 0.781	-	- -	-
30	0.460	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-
35	1.412	-37.8 -49.5	-	-	-	-	-	- -	- -	-	- -	-
39	0.061	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-
44	1.412	49.5 37.8	0.76	4.500	880.7	0.237	1.000	- 1.412	0.631 0.781	-	- -	-
48	0.460	50.0 50.0	1.00	4.000	1445.2	0.186	1.000	- 0.460	0.230 0.230	-	- -	-

Table of values		
Sxe	12.681	$inch^3$
Mnxo	52830.6	lbfft
Resistance factor	0.90	
Unity check	0.23	-

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

Table of values		
Lltb	5' 10.200"	ft
Sigma,ey	inf	ksi
Kt	1.00	
Lt	5' 10.200"	ft
Sigma,t	4339.6	ksi
Cb	2.70	
Sfx	12.681	inch^3
Fcre	inf	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	0.591	9.9 9.9	1.00	0.430	149.2	0.257	1.000	0.591 -	- -	-	- -	-
2	2.432	49.0 9.9	0.20	6.617	436.6	0.335	1.000	- 2.432	0.869 1.563	-	- -	-
3	9.108	49.0 49.0	1.00	4.000	5.8	2.896	0.319	2.906 -	- -	-	- -	-
4	2.432	49.0 9.9	0.20	6.617	436.6	0.335	1.000	- 2.432	0.869 1.563	-	- -	-
5	0.591	9.9 9.9	1.00	0.430	149.2	0.257	1.000	0.591 -	- -	-	- -	-
6	0.061	50.0 50.0	1.00	4.000	82181.8	0.025	1.000	0.061 -	- -	-	- -	-
7	0.061	50.0 49.0	0.98	4.039	82989.3	0.025	1.000	- 0.061	0.030 0.031	-	- -	-
8	0.061	-37.7 -37.7	-	-	-	-	-	- -	- -	-	- -	-
9	0.591	2.5 2.5	1.00	0.431	149.7	0.128	1.000	0.591 -	- -	-	- -	-
10	2.432	2.5 -36.7	14.95	8148.664	537609.5	0.002	1.000	- 2.432	0.136 1.216	-	- -	-
11	9.108	-36.7 -36.7	-	-	-	-	-	- -	- -	-	- -	-
12	2.432	2.5 -36.7	14.95	8148.664	537609.5	0.002	1.000	- 2.432	0.136 1.216	-	- -	-
13	0.591	2.5 2.5	1.00	0.430	149.2	0.128	1.000	0.591 -	- -	-	- -	-
14	0.061	-37.7 -37.7	-	-	-	-	-	- -	- -	-	- -	-
15	0.061	-36.7 -37.7	-	-	-	-	-	- -	- -	-	- -	-
16	0.061	50.0 50.0	1.00	4.000	82181.8	0.025	1.000	0.061 -	- -	-	- -	-

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]
17	0.061	50.0 49.0	0.98	4.039	82989.3	0.025	1.000	- 0.061	0.030 0.031	-	- -	-
22	1.412	49.5 49.5	1.00	4.000	782.9	0.251	1.000	- 1.412	0.706 0.706	-	- -	-
26	1.412	49.5 49.5	1.00	4.000	782.9	0.251	1.000	- 1.412	0.706 0.706	-	- -	-
30	0.460	9.9 2.5	0.25	6.349	2294.0	0.066	1.000	- 0.460	0.167 0.293	-	- -	-
35	1.412	-37.1 -37.1	-	-	-	-	-	- -	- -	-	- -	-
39	0.061	-36.7 -37.7	-	-	-	-	-	- -	- -	-	- -	-
44	1.412	-37.1 -37.1	-	-	-	-	-	- -	- -	-	- -	-
48	0.460	9.9 2.5	0.25	6.349	2294.0	0.066	1.000	- 0.460	0.167 0.293	-	- -	-

Table of values		
Sye	4.351	inch ^{3}
Mnyo	18124.8	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	1347.9	ksi
Kt	1.00	
Lt	5' 10.200"	ft
Sigma,t	4339.6	ksi
Cb	1.00	
Sfy	6.123	inch ^{3}
Fcre	6951.9	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
Shear force Vy

Element ID	Aw [inch ^{2}]	Vn [lbf]
1	0.040	1205.6
2	0.000	0.0
3	0.619	4834.2
4	0.000	0.0
5	0.040	1205.6
6	0.003	98.8
7	0.000	0.0
8	0.003	98.8
9	0.040	1205.6
10	0.000	0.0
11	0.619	4834.2

12	0.000	0.0
13	0.040	1205.6
14	0.003	98.8
15	0.000	0.0
16	0.003	98.8
17	0.000	0.0
22	0.172	5167.9
26	0.172	5167.9
30	0.000	0.0
35	0.172	5167.9
39	0.000	0.0
44	0.172	5167.9
48	0.000	0.0

Table of values		
Vn,y	35557.9	lbf
Resistance factor	0.95	
Unity check	0.21	-

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

Table of values		
Mnxo	52830.6	lbfft
Vny	35557.9	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.05+0.04) = 0.31

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	0.591	0.0 0.0	1.00	0.430	149.2	0.002	1.000	0.591 -	- -	-	- -	-
2	2.432	0.0 0.0	1.00	4.000	263.9	0.001	1.000	2.432 -	- -	-	- -	-
3	9.108	0.0 0.0	1.00	4.000	5.8	0.009	1.000	9.108 -	- -	-	- -	-
4	2.432	0.0 0.0	1.00	4.000	263.9	0.001	1.000	2.432 -	- -	-	- -	-
5	0.591	0.0 0.0	1.00	0.430	149.2	0.002	1.000	0.591 -	- -	-	- -	-
6	0.061	0.0 0.0	1.00	4.000	82181.8	0.000	1.000	0.061 -	- -	-	- -	-
7	0.061	0.0 0.0	1.00	4.000	82181.8	0.000	1.000	0.061 -	- -	-	- -	-
8	0.061	0.0 0.0	1.00	4.000	82181.8	0.000	1.000	0.061 -	- -	-	- -	-
9	0.591	0.0 0.0	1.00	0.430	149.2	0.002	1.000	0.591 -	- -	-	- -	-
10	2.432	0.0 0.0	1.00	4.000	263.9	0.001	1.000	2.432 -	- -	-	- -	-

11	9.108	0.0 0.0	1.00	4.000	5.8	0.009	1.000	9.108 -	- -	-	- -	-
12	2.432	0.0 0.0	1.00	4.000	263.9	0.001	1.000	2.432 -	- -	-	- -	-
13	0.591	0.0 0.0	1.00	0.430	149.2	0.002	1.000	0.591 -	- -	-	- -	-
14	0.061	0.0 0.0	1.00	4.000	82181.8	0.000	1.000	0.061 -	- -	-	- -	-
15	0.061	0.0 0.0	1.00	4.000	82181.8	0.000	1.000	0.061 -	- -	-	- -	-
16	0.061	0.0 0.0	1.00	4.000	82181.8	0.000	1.000	0.061 -	- -	-	- -	-
17	0.061	0.0 0.0	1.00	4.000	82181.8	0.000	1.000	0.061 -	- -	-	- -	-
22	1.412	0.0 0.0	1.00	4.000	782.9	0.001	1.000	1.412 -	- -	-	- -	-
26	1.412	0.0 0.0	1.00	4.000	782.9	0.001	1.000	1.412 -	- -	-	- -	-
30	0.460	0.0 0.0	1.00	4.000	1445.2	0.001	1.000	0.460 -	- -	-	- -	-
35	1.412	0.0 0.0	1.00	4.000	782.9	0.001	1.000	1.412 -	- -	-	- -	-
39	0.061	0.0 0.0	1.00	4.000	82181.8	0.000	1.000	0.061 -	- -	-	- -	-
44	1.412	0.0 0.0	1.00	4.000	782.9	0.001	1.000	1.412 -	- -	-	- -	-
48	0.460	0.0 0.0	1.00	4.000	1445.2	0.001	1.000	0.460 -	- -	-	- -	-

Table of values		
Mnx	52830.6	lbfft
Mny	18124.8	lbfft
Pn	117414.9	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.00+0.23+0.01 = 0.24 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B25 CHECK

AISI S100-16 LRFD Check

Member B25	14"X2.5"X12GA + 550S150-54	A913 grade 50	LRFD-Ult (auto)	0.77

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

The critical check is on position 9.20 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	-235.14	lbf
Vux	21.61	lbf
Vuy	-4616.23	lbf
Mut	259.28	lbfft
Mux	54960.02	lbfft
Muy	-139.25	lbfft

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

Table of values		
Critical fibre	47	
Sigma Mx	-33.6	ksi
Sigma My	-0.2	ksi
f bending	-33.9	ksi
Tau t	-0.1	ksi
f torsion	-0.1	ksi
Composed Stress	33.9	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	0.702	-42.9 -47.8	-	-	-	-	-	- -	- -	-	- -	-
2	2.403	-48.0 -48.0	-	-	-	-	-	- -	- -	-	- -	-
3	11.108	39.7 -38.1	0.96	22.954	45.9	0.930	0.821	- 9.119	2.303 2.351	-	- -	-
4	2.403	49.7 49.7	1.00	4.000	414.1	0.346	1.000	- 2.403	1.201 1.202	-	- -	-
5	0.702	49.5 44.6	0.90	0.466	233.5	0.460	1.000	0.702 -	- -	-	- -	-

6	0.076	50.0 49.5	0.99	4.021	53930.1	0.030	1.000	- 0.076	0.038 0.038	-	- -	-
7	0.075	50.0 50.0	1.00	4.000	53646.5	0.031	1.000	0.075 -	- -	-	- -	-
8	0.076	50.0 49.5	0.99	4.021	53930.1	0.030	1.000	- 0.076	0.038 0.038	-	- -	-
9	0.701	49.5 44.6	0.90	0.466	233.5	0.460	1.000	0.701 -	- -	-	- -	-
10	2.403	49.7 49.7	1.00	4.000	414.1	0.346	1.000	- 2.403	1.202 1.202	-	- -	-
11	11.108	39.7 -38.1	0.96	22.954	45.9	0.930	0.821	- 9.119	2.303 2.351	-	- -	-
12	2.403	-48.0 -48.0	-	-	-	-	-	- -	- -	-	- -	-
13	0.701	-42.9 -47.8	-	-	-	-	-	- -	- -	-	- -	-
14	0.075	-47.8 -48.4	-	-	-	-	-	- -	- -	-	- -	-
15	0.075	-48.4 -48.4	-	-	-	-	-	- -	- -	-	- -	-
16	0.075	-47.8 -48.4	-	-	-	-	-	- -	- -	-	- -	-
17	0.075	-48.4 -48.4	-	-	-	-	-	- -	- -	-	- -	-
22	1.397	49.5 39.7	0.80	4.411	1350.1	0.191	1.000	- 1.397	0.636 0.762	-	- -	-
26	1.398	-38.1 -47.8	-	-	-	-	-	- -	- -	-	- -	-
30	0.489	50.0 50.0	1.00	4.000	1278.8	0.198	1.000	- 0.489	0.245 0.245	-	- -	-
35	1.397	49.5 39.7	0.80	4.411	1350.1	0.191	1.000	- 1.397	0.636 0.762	-	- -	-
39	0.075	50.0 50.0	1.00	4.000	53646.5	0.031	1.000	0.075 -	- -	-	- -	-
44	1.398	-38.1 -47.8	-	-	-	-	-	- -	- -	-	- -	-
48	0.489	-48.4 -48.4	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	19.292	$inch^3$
Mnxo	80384.5	lbfft
Resistance factor	0.90	
Unity check	0.76	-

Lateral-Torsional Buckling Strength

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

Table of values		
Lltb	6' 1.520"	ft
Sigma,ey	277.8	ksi
Kt	1.00	
Lt	6' 1.520"	ft
Sigma,t	4042.1	ksi
Cb	1.47	
Sfx	19.613	$inch^3$
Fcre	2210.4	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	0.702	10.8 10.8	1.00	0.430	215.5	0.223	1.000	0.702 -	- -	-	- -	-
2	2.403	48.8 10.8	0.22	6.506	673.5	0.269	1.000	- 2.403	0.865 1.538	-	- -	-
3	11.108	48.8 48.8	1.00	4.000	8.0	2.470	0.369	4.096 -	- -	-	- -	-
4	2.403	48.8 10.8	0.22	6.506	673.5	0.269	1.000	- 2.403	0.865 1.538	-	- -	-
5	0.702	10.8 10.8	1.00	0.430	215.5	0.223	1.000	0.702 -	- -	-	- -	-
6	0.076	50.0 50.0	1.00	4.000	53646.5	0.031	1.000	- 0.076	0.038 0.038	-	- -	-
7	0.075	50.0 48.8	0.98	4.048	54288.1	0.030	1.000	- 0.075	0.037 0.038	-	- -	-
8	0.076	-36.2 -36.2	-	-	-	-	-	- -	- -	-	- -	-
9	0.701	3.0 3.0	1.00	0.431	216.2	0.118	1.000	0.701 -	- -	-	- -	-
10	2.403	3.0 -35.0	11.59	4025.175	416695.8	0.003	1.000	- 2.403	0.165 0.026	-	- -	-
11	11.108	-35.0 -35.0	-	-	-	-	-	- -	- -	-	- -	-
12	2.403	3.0 -35.0	11.59	4025.175	416695.8	0.003	1.000	- 2.403	0.165 0.026	-	- -	-
13	0.701	3.0 3.0	1.00	0.430	215.5	0.118	1.000	0.701 -	- -	-	- -	-
14	0.075	-36.2 -36.2	-	-	-	-	-	- -	- -	-	- -	-
15	0.075	-35.0 -36.2	-	-	-	-	-	- -	- -	-	- -	-
16	0.075	50.0 50.0	1.00	4.000	53646.5	0.031	1.000	- 0.075	0.038 0.038	-	- -	-

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]
17	0.075	50.0 48.8	0.98	4.048	54288.1	0.030	1.000	- 0.075	0.037 0.038	-	- -	-
22	1.397	49.2 49.2	1.00	4.000	1224.3	0.201	1.000	- 1.397	0.699 0.699	-	- -	-
26	1.398	49.2 49.2	1.00	4.000	1224.3	0.201	1.000	- 1.398	0.699 0.699	-	- -	-
30	0.489	10.8 3.0	0.28	6.183	1976.8	0.074	1.000	- 0.489	0.180 0.309	-	- -	-
35	1.397	-35.4 -35.4	-	-	-	-	-	- -	- -	-	- -	-
39	0.075	-35.0 -36.2	-	-	-	-	-	- -	- -	-	- -	-
44	1.398	-35.4 -35.4	-	-	-	-	-	- -	- -	-	- -	-
48	0.489	10.8 3.0	0.28	6.183	1976.8	0.074	1.000	- 0.489	0.180 0.309	-	- -	-

Table of values		
Sye	6.153	inch^3
Mnyo	25637.0	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	247.3	ksi
Kt	1.00	
Lt	6' 1.520"	ft
Sigma,t	4042.1	ksi
Cb	1.00	
Sfy	9.026	inch^3
Fcre	3078.1	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	0.068	2041.4
2	0.000	0.0
3	1.077	11505.4
4	0.000	0.0
5	0.068	2041.4
6	0.004	122.3
7	0.000	0.0
8	0.004	122.3
9	0.068	2041.4
10	0.000	0.0

11	1.077	11505.4
12	0.000	0.0
13	0.068	2041.4
14	0.004	122.3
15	0.000	0.0
16	0.004	122.3
17	0.000	0.0
22	0.211	6330.7
26	0.211	6330.7
30	0.000	0.0
35	0.211	6330.7
39	0.000	0.0
44	0.211	6330.7
48	0.000	0.0

Table of values		
Vn,y	56988.1	lbf
Resistance factor	0.95	
Unity check	0.09	-

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

Table of values		
Mnxo	80384.5	lbfft
Vny	56988.1	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.58+0.01) = 0.76

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	15 3/8	6 1/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	15 3/8	6 1/4	ft
Slenderness	34.02	32.10	
Flexural Buckling stress Fcre	247.3	277.8	ksi

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

Table of values		
Sigma,ex	247.3	ksi
Sigma,ey	277.8	ksi
Kt	1.00	
Lt	6 1/4	ft
Sigma,t	4042.1	ksi
Sigma,TF	247.3	ksi
Torsional (-Flexural) buckling stress Fcre	247.3	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	0.702	45.9 45.9	1.00	0.430	215.5	0.462	1.000	0.702 -	- -	-	- -	-
2	2.403	45.9 45.9	1.00	4.000	414.1	0.333	1.000	2.403 -	- -	-	- -	-
3	11.108	45.9 45.9	1.00	4.000	8.0	2.397	0.379	4.209 -	- -	-	- -	-
4	2.403	45.9 45.9	1.00	4.000	414.1	0.333	1.000	2.403 -	- -	-	- -	-
5	0.702	45.9 45.9	1.00	0.430	215.5	0.462	1.000	0.702 -	- -	-	- -	-
6	0.076	45.9 45.9	1.00	4.000	53646.5	0.029	1.000	0.076 -	- -	-	- -	-
7	0.075	45.9 45.9	1.00	4.000	53646.5	0.029	1.000	0.075 -	- -	-	- -	-
8	0.076	45.9 45.9	1.00	4.000	53646.5	0.029	1.000	0.076 -	- -	-	- -	-
9	0.701	45.9 45.9	1.00	0.430	215.5	0.462	1.000	0.701 -	- -	-	- -	-
10	2.403	45.9 45.9	1.00	4.000	414.1	0.333	1.000	2.403 -	- -	-	- -	-
11	11.108	45.9 45.9	1.00	4.000	8.0	2.397	0.379	4.209 -	- -	-	- -	-
12	2.403	45.9 45.9	1.00	4.000	414.1	0.333	1.000	2.403 -	- -	-	- -	-
13	0.701	45.9 45.9	1.00	0.430	215.5	0.462	1.000	0.701 -	- -	-	- -	-
14	0.075	45.9 45.9	1.00	4.000	53646.5	0.029	1.000	0.075 -	- -	-	- -	-
15	0.075	45.9 45.9	1.00	4.000	53646.5	0.029	1.000	0.075 -	- -	-	- -	-
16	0.075	45.9 45.9	1.00	4.000	53646.5	0.029	1.000	0.075 -	- -	-	- -	-
17	0.075	45.9 45.9	1.00	4.000	53646.5	0.029	1.000	0.075 -	- -	-	- -	-
22	1.397	45.9 45.9	1.00	4.000	1224.3	0.194	1.000	1.397 -	- -	-	- -	-
26	1.398	45.9 45.9	1.00	4.000	1224.3	0.194	1.000	1.398 -	- -	-	- -	-
30	0.489	45.9 45.9	1.00	4.000	1278.8	0.190	1.000	0.489 -	- -	-	- -	-
35	1.397	45.9 45.9	1.00	4.000	1224.3	0.194	1.000	1.397 -	- -	-	- -	-
39	0.075	45.9 45.9	1.00	4.000	53646.5	0.029	1.000	0.075 -	- -	-	- -	-
44	1.398	45.9 45.9	1.00	4.000	1224.3	0.194	1.000	1.398 -	- -	-	- -	-
48	0.489	45.9 45.9	1.00	4.000	1278.8	0.190	1.000	0.489 -	- -	-	- -	-

Table of values		
Fe	247.3	ksi
lambda, c	0.45	
Fn	45.9	ksi
Ae	3.474	$inch^2$
Pn	159591.4	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	0.702	0.0 0.0	1.00	0.430	215.5	0.015	1.000	0.702 -	- -	-	- -	-
2	2.403	0.0 0.0	1.00	4.000	414.1	0.011	1.000	2.403 -	- -	-	- -	-
3	11.108	0.0 0.0	1.00	4.000	8.0	0.079	1.000	11.108 -	- -	-	- -	-
4	2.403	0.0 0.0	1.00	4.000	414.1	0.011	1.000	2.403 -	- -	-	- -	-
5	0.702	0.0 0.0	1.00	0.430	215.5	0.015	1.000	0.702 -	- -	-	- -	-
6	0.076	0.0 0.0	1.00	4.000	53646.5	0.001	1.000	0.076 -	- -	-	- -	-
7	0.075	0.0 0.0	1.00	4.000	53646.5	0.001	1.000	0.075 -	- -	-	- -	-
8	0.076	0.0 0.0	1.00	4.000	53646.5	0.001	1.000	0.076 -	- -	-	- -	-
9	0.701	0.0 0.0	1.00	0.430	215.5	0.015	1.000	0.701 -	- -	-	- -	-
10	2.403	0.0 0.0	1.00	4.000	414.1	0.011	1.000	2.403 -	- -	-	- -	-
11	11.108	0.0 0.0	1.00	4.000	8.0	0.079	1.000	11.108 -	- -	-	- -	-
12	2.403	0.0 0.0	1.00	4.000	414.1	0.011	1.000	2.403 -	- -	-	- -	-
13	0.701	0.0 0.0	1.00	0.430	215.5	0.015	1.000	0.701 -	- -	-	- -	-
14	0.075	0.0 0.0	1.00	4.000	53646.5	0.001	1.000	0.075 -	- -	-	- -	-
15	0.075	0.0 0.0	1.00	4.000	53646.5	0.001	1.000	0.075 -	- -	-	- -	-
16	0.075	0.0 0.0	1.00	4.000	53646.5	0.001	1.000	0.075 -	- -	-	- -	-
17	0.075	0.0 0.0	1.00	4.000	53646.5	0.001	1.000	0.075 -	- -	-	- -	-
22	1.397	0.0 0.0	1.00	4.000	1224.3	0.006	1.000	1.397 -	- -	-	- -	-
26	1.398	0.0 0.0	1.00	4.000	1224.3	0.006	1.000	1.398 -	- -	-	- -	-

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]
30	0.489	0.0 0.0	1.00	4.000	1278.8	0.006	1.000	0.489 -	- -	-	- -	-
35	1.397	0.0 0.0	1.00	4.000	1224.3	0.006	1.000	1.397 -	- -	-	- -	-
39	0.075	0.0 0.0	1.00	4.000	53646.5	0.001	1.000	0.075 -	- -	-	- -	-
44	1.398	0.0 0.0	1.00	4.000	1224.3	0.006	1.000	1.398 -	- -	-	- -	-
48	0.489	0.0 0.0	1.00	4.000	1278.8	0.006	1.000	0.489 -	- -	-	- -	-

Table of values		
Mnx	80384.5	lbfft
Mny	25637.0	lbfft
Pn	159591.4	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.00+0.76+0.01 = 0.77 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER H1 CHECK

AISI S100-16 LRFD Check

Member H1	5x550S150-54	A913 grade 50	LRFD-Ult (auto)	0.37

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

The critical check is on position 1.54 ft

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

Internal forces		
Pu	-0.00	lbf
Vux	-0.00	lbf
Vuy	3174.02	lbf
Mut	-0.00	lbfft
Mux	5316.81	lbfft
Muy	-0.00	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	27.1 27.1	1.00	0.430	146.9	0.430	1.000	0.473 -	- -	-	- -	-
2	1.446	47.0 27.1	0.58	4.997	182.7	0.507	1.000	- 1.446	0.597 0.849	-	- -	-
3	1.250	47.0 47.0	1.00	4.000	195.7	0.490	1.000	1.250 -	- -	-	- -	-
4	1.446	47.0 27.1	0.58	4.997	182.7	0.507	1.000	- 1.446	0.597 0.849	-	- -	-
5	0.473	27.1 27.1	1.00	0.431	147.4	0.429	1.000	0.473 -	- -	-	- -	-
6	0.473	46.3 39.8	0.86	0.482	164.7	0.530	1.000	0.473 -	- -	-	- -	-
7	1.446	46.7 46.7	1.00	4.000	585.0	0.282	1.000	1.446 -	- -	-	- -	-
8	5.446	46.3 -28.6	0.62	15.715	162.0	0.534	1.000	- 5.446	1.505 2.723	-	- -	-
9	1.446	-29.4 -29.4	-	-	-	-	-	- -	- -	-	- -	-
10	0.473	-22.1 -28.6	-	-	-	-	-	- -	- -	-	- -	-

11	0.473	-22.1 -28.6	-	-	-	-	-	- -	- -	-	- -	-
12	1.446	-29.4 -29.4	-	-	-	-	-	- -	- -	-	- -	-
13	1.446	46.7 46.7	1.00	4.000	585.0	0.282	1.000	1.446 -	- -	-	- -	-
14	0.473	46.3 39.8	0.86	0.482	164.7	0.530	1.000	0.473 -	- -	-	- -	-
15	0.473	-9.5 -9.5	-	-	-	-	-	- -	- -	-	- -	-
16	1.446	-9.5 -29.4	-	-	-	-	-	- -	- -	-	- -	-
17	1.250	-29.7 -29.7	-	-	-	-	-	- -	- -	-	- -	-
18	1.446	-9.5 -29.4	-	-	-	-	-	- -	- -	-	- -	-
19	0.473	-9.5 -9.5	-	-	-	-	-	- -	- -	-	- -	-
20	0.473	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-
21	1.446	-30.1 -50.0	-	-	-	-	-	- -	- -	-	- -	-
22	1.250	-29.7 -29.7	-	-	-	-	-	- -	- -	-	- -	-
23	1.446	-30.1 -50.0	-	-	-	-	-	- -	- -	-	- -	-
24	0.473	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-
25	0.054	47.0 47.0	1.00	4.000	104869.2	0.021	1.000	0.054 -	- -	-	- -	-
38	0.054	-29.7 -29.7	-	-	-	-	-	- -	- -	-	- -	-
47	1.250	47.0 47.0	1.00	4.000	195.7	0.490	1.000	1.250 -	- -	-	- -	-

Table of values		
Sxe	3.931	$inch^3$
Mnxo	16380.1	lbfft
Resistance factor	0.90	
Unity check	0.36	-

Lateral-Torsional Buckling Strength

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

Table of values		
Lltb	1' 6.480"	ft
Sigma,ey	2323.4	ksi
Kt	1.00	
Lt	1' 6.480"	ft
Sigma,t	2443.9	ksi
Cb	1.67	
Sfx	4.181	$inch^3$
Fcre	6855.8	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	0.0
2	0.078	2342.5
3	0.000	0.0
4	0.078	2342.5
5	0.000	0.0
6	0.026	766.3
7	0.000	0.0
8	0.588	17645.0
9	0.000	0.0
10	0.026	766.3
11	0.026	766.3
12	0.000	0.0
13	0.000	0.0
14	0.026	766.3
15	0.000	0.0
16	0.078	2342.5
17	0.000	0.0
18	0.078	2342.5
19	0.000	0.0
20	0.000	0.0
21	0.078	2342.5
22	0.000	0.0
23	0.078	2342.5
24	0.000	0.0
25	0.000	0.0
38	0.000	0.0
47	0.000	0.0

Table of values		
Vn,y	34765.2	lbf
Resistance factor	0.95	
Unity check	0.10	-

Combined Bending and Shear

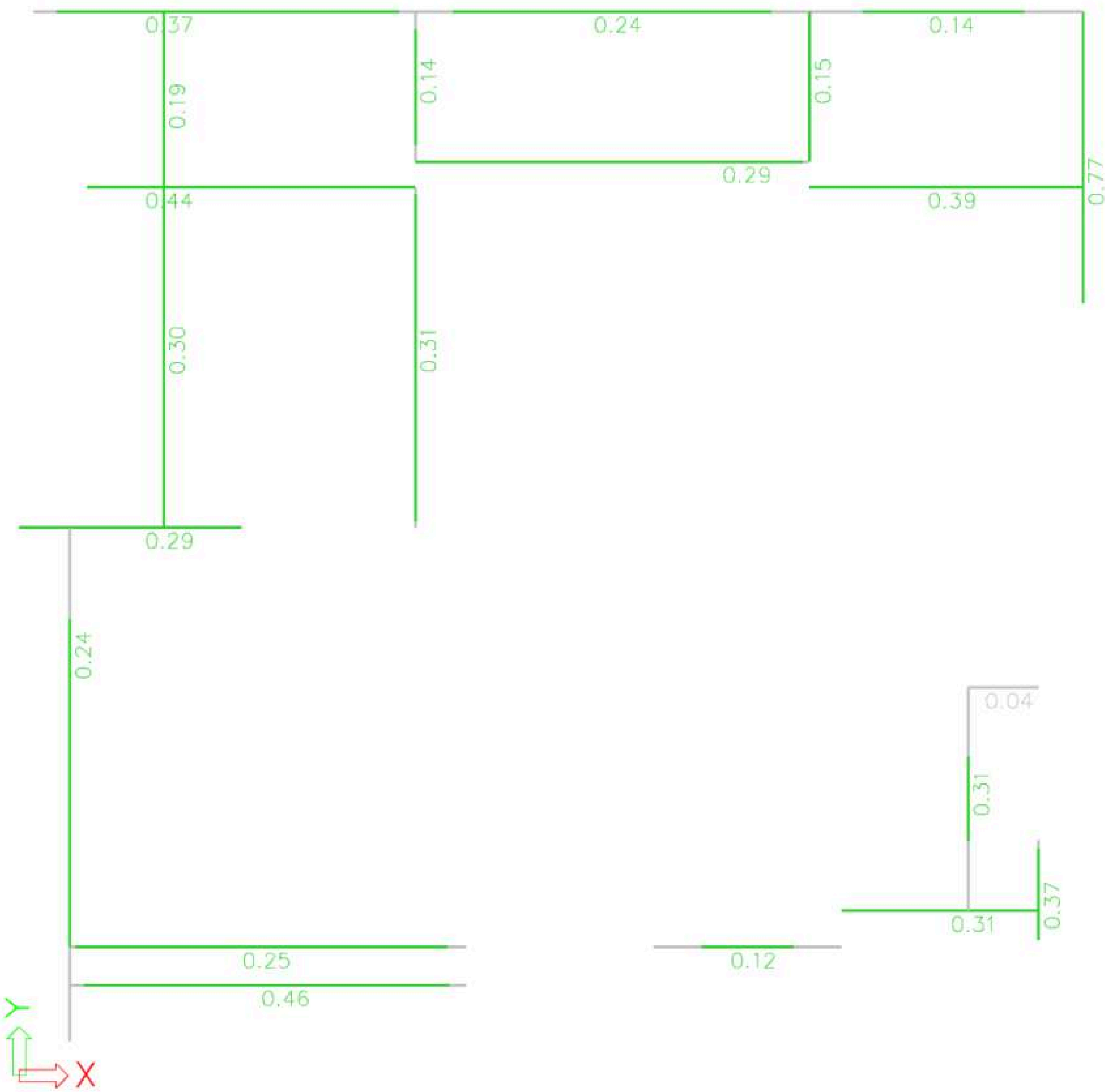
According to article H2 and formula (H2-1)

Table of values		
Mnxo	16380.1	lbfft
Vny	34765.2	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.13+0.01) = 0.37

The member satisfies the check !

Unity check



0.37
0.24
0.14
0.19
0.14
0.15
0.29
0.44
0.39
0.77
0.30
0.31
0.29
0.24
0.04
0.31
0.37
0.31
0.25
0.12
0.46
Y
X

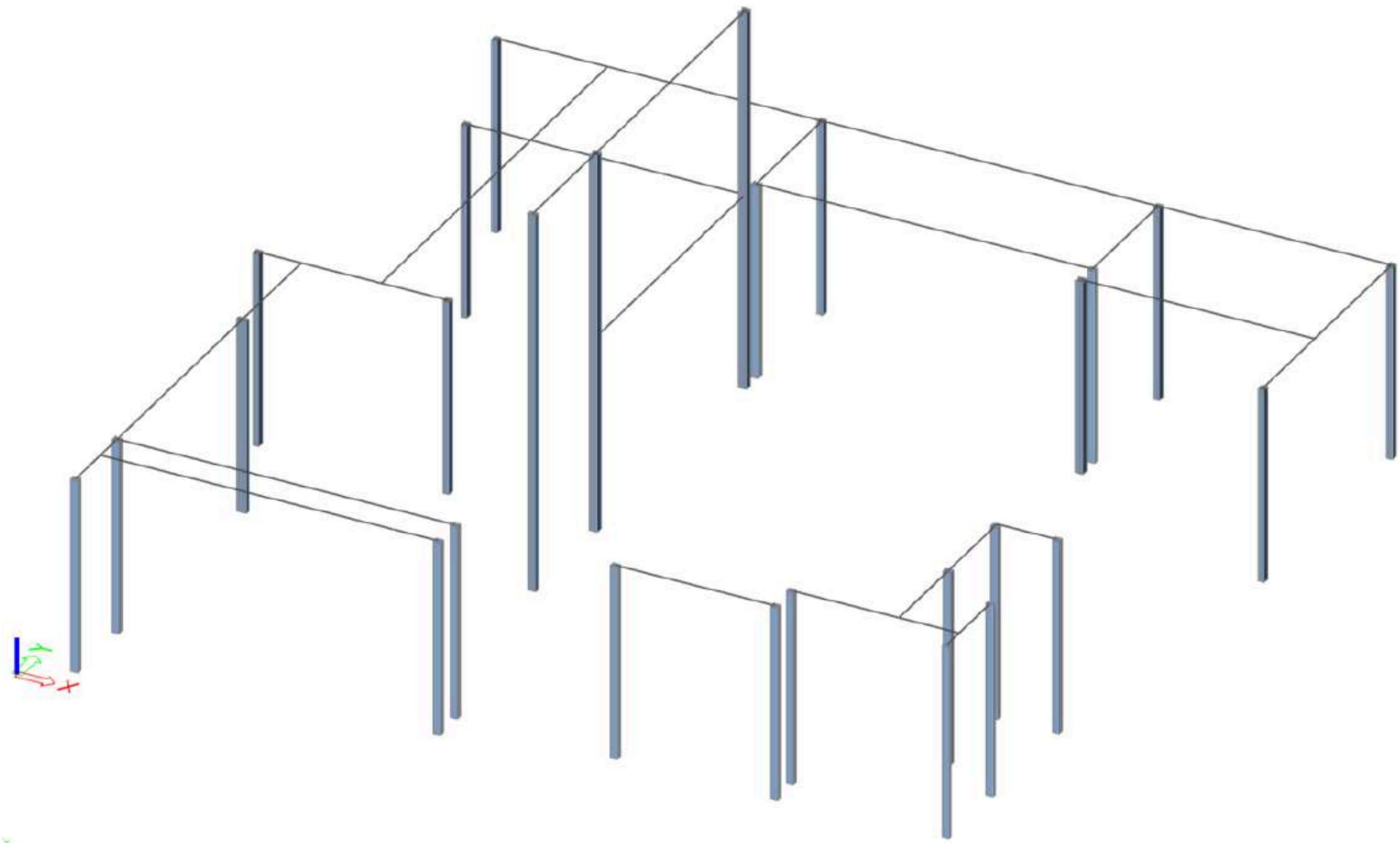
Check of steel

Linear calculation, Extreme : Member
Selection : All
Combinations : LRFD-Ult (auto)

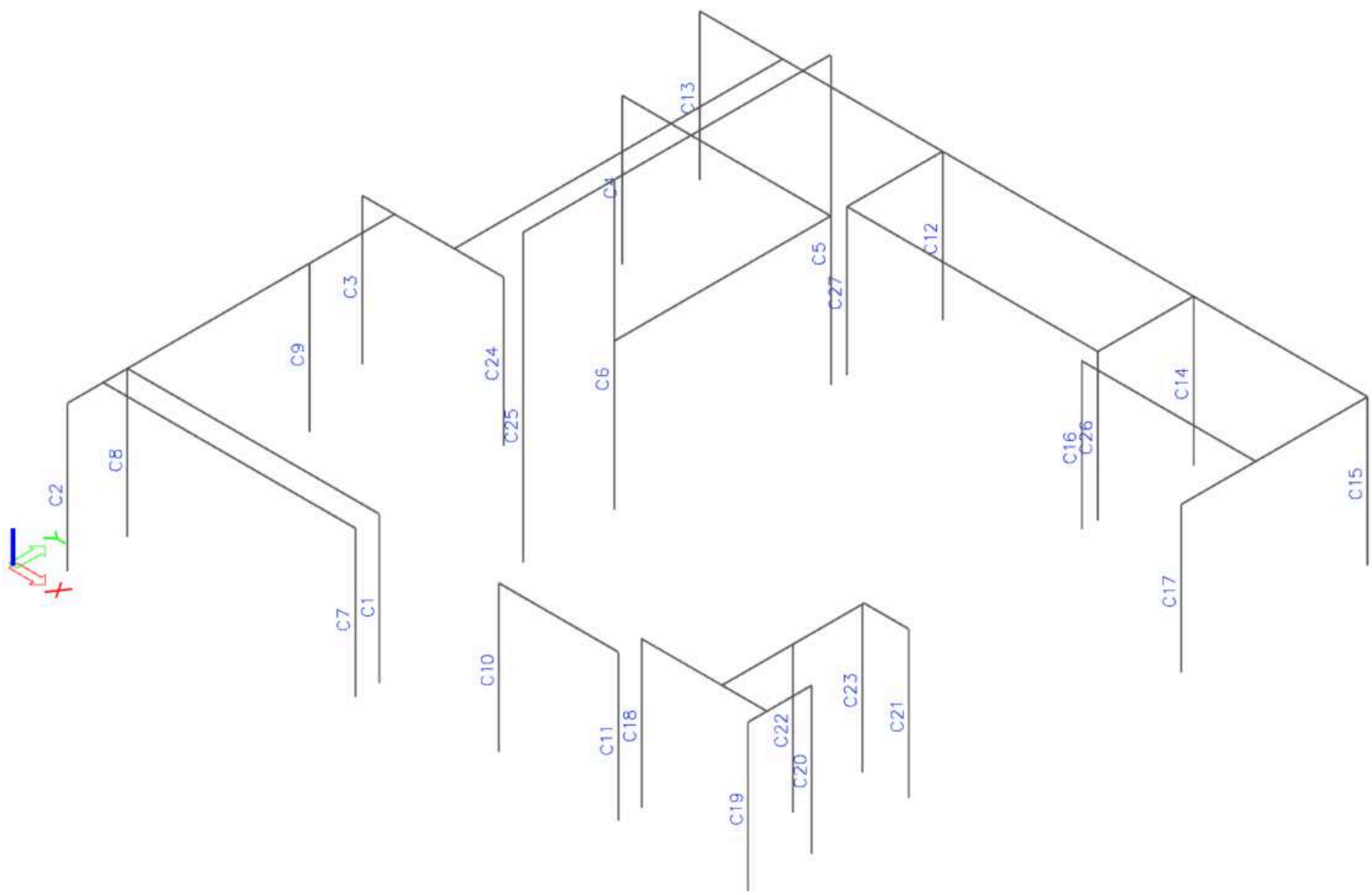
Case	Member	css	mat	dx [ft]	un.check [-]
LRFD-Ult (auto)/1	B15	BOX BEAM 22"X8" - 22"X4"X10GA + 8.125"X2"X14GA	A913 grade 50	4' 0.000"	0.44
LRFD-Ult (auto)/1	B17	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	8' 12.000"	0.31
LRFD-Ult (auto)/1	B10	BOX BEAM 22"X8" - 22"X4"X10GA + 8.125"X2"X14GA	A913 grade 50	19' 11.200"	0.24
LRFD-Ult (auto)/1	B16	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	10' 0.000"	0.25
LRFD-Ult (auto)/1	B14	BOX BEAM 22"X8" - 22"X4"X10GA + 8.125"X2"X14GA	A913 grade 50	8' 12.000"	0.30
LRFD-Ult (auto)/2	B16.1	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	10' 12.000"	0.46
LRFD-Ult (auto)/1	B5	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	4' 11.037"	0.12
LRFD-Ult (auto)/3	B2	BOX BEAM 12"X5.5" - 12"X2.5"X14GA + 550S150-54	A913 grade 50	3' 11.331"	0.19
LRFD-Ult (auto)/1	B3	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	13' 2.000"	0.37
LRFD-Ult (auto)/3	B8	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	10' 3.960"	0.24
LRFD-Ult (auto)/3	B26	BOX BEAM 12"X5.5" - 12"X2.5"X14GA + 550S150-54	A913 grade 50	3' 11.220"	0.15
LRFD-Ult (auto)/1	B27	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	7' 1.980"	0.14
LRFD-Ult (auto)/1	B19	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	7' 1.980"	0.39
LRFD-Ult (auto)/1	B22	BOX BEAM 12"X5.5" - 12"X2.5"X14GA + 550S150-54	A913 grade 50	5' 10.200"	0.31
LRFD-Ult (auto)/2	B23	BOX BEAM 12"X5.5" - 12"X2.5"X14GA + 550S150-54	A913 grade 50	0.000"	0.04
LRFD-Ult (auto)/2	H1	HDR - 5x550S150-54	A913 grade 50	1' 6.480"	0.37
LRFD-Ult (auto)/1	B12	BOX BEAM 22"X8" - 22"X4"X10GA + 8.125"X2"X14GA	A913 grade 50	7' 7.000"	0.29
LRFD-Ult (auto)/1	B21	BOX BEAM 12"X5.5" - 12"X2.5"X14GA + 550S150-54	A913 grade 50	6' 7.440"	0.31
LRFD-Ult (auto)/1	B25	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	9' 2.440"	0.77
LRFD-Ult (auto)/3	B18	BOX BEAM 14"X5.5" - 14"X2.5"X12GA + 550S150-54	A913 grade 50	17' 0.920"	0.29
LRFD-Ult (auto)/3	B13	BOX BEAM 12"X5.5" - 12"X2.5"X14GA + 550S150-54	A913 grade 50	3' 11.220"	0.14

2.3.3 COLUMN DESIGN

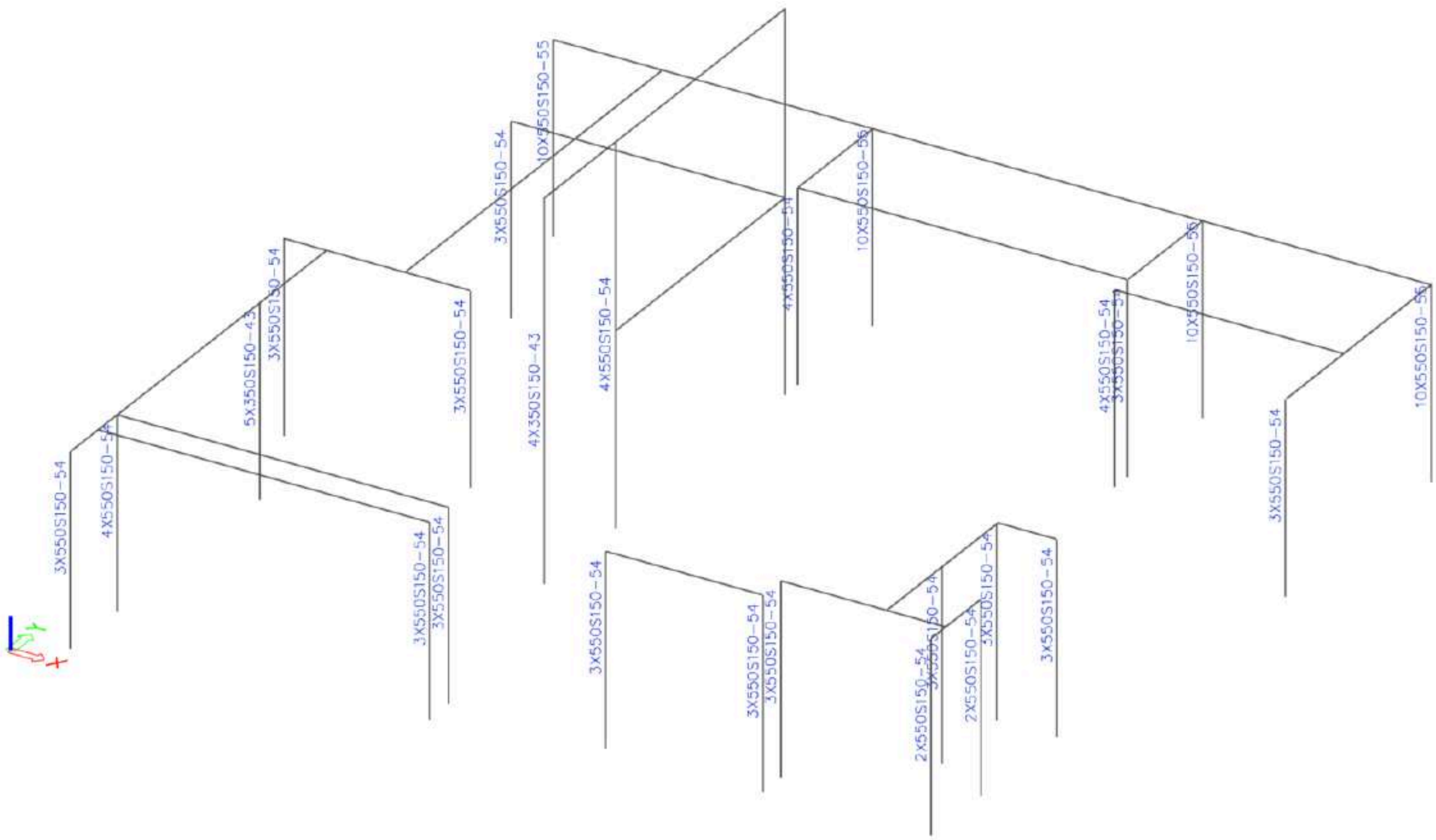
General scheme



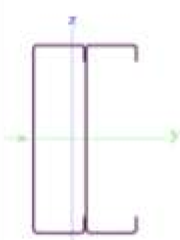
Members number



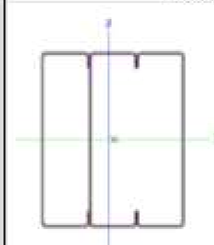
Members cross-sections



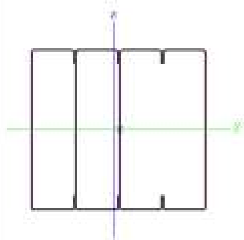
Cross-section properties 2X550S150-54

2X550S150-54		
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch²]	0.982	
A_y [inch²], A_z [inch²]	0.318	0.607
A_L [inch²/inch], A_D [inch²/inch]	2.16e+01	3.51e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0.659	4.800
α [deg]	0.00	
I_y [inch⁴], I_z [inch⁴]	4.251	0.831
i_y [inch], i_z [inch]	2.080	0.920
$W_{el,y}$ [inch³], $W_{el,z}$ [inch³]	1.546	0.447
$W_{pl,y}$ [inch³], $W_{pl,z}$ [inch³]	1.850	0.737
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	9.25e+01	9.25e+01
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	3.68e+01	3.68e+01
d_y [inch], d_z [inch]	-1.457	0.000
I_t [inch⁴], I_w [inch⁶]	0.090	2.759
$β_y$ [inch], $β_z$ [inch]	0.000	4.222
Picture		

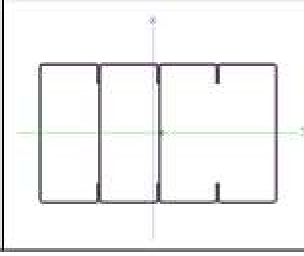
Cross-section properties 3X550S150-54

3X550S150-54		
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour	■	
A [inch²]	1.473	
A_y [inch²], A_z [inch²]	0.533	0.910
A_L [inch²/inch], A_D [inch²/inch]	2.04e+01	5.20e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	1.648	4.800
α [deg]	0.00	
I_y [inch⁴], I_z [inch⁴]	6.376	3.850
i_y [inch], i_z [inch]	2.080	1.616
$W_{el,y}$ [inch³], $W_{el,z}$ [inch³]	2.319	1.625
$W_{pl,y}$ [inch³], $W_{pl,z}$ [inch³]	2.774	2.006
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1.39e+02	1.39e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1.00e+02	1.00e+02
d_y [inch], d_z [inch]	0.155	0.000
I_t [inch⁴], I_w [inch⁶]	0.408	26.734
$β_y$ [inch], $β_z$ [inch]	0.000	0.145
Picture		

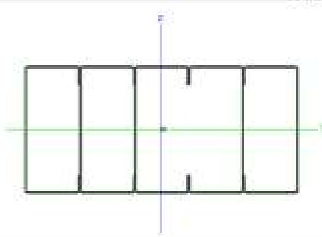
Cross-section properties 4X550S150-54

4X550S150-54		
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.965	
A_y [inch^2], A_z [inch^2]	0.765	1.213
A_L [inch^2/inch], A_D [inch^2/inch]	2.37e+01	6.88e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	2.338	4.800
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	8.501	7.853
i_y [inch], i_z [inch]	2.080	1.999
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	3.091	2.470
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	3.699	3.298
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.85e+02	1.85e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.65e+02	1.65e+02
d_y [inch], d_z [inch]	0.204	0.000
I_t [inch^4], I_w [inch^6]	0.497	50.957
$β_y$ [inch], $β_z$ [inch]	0.000	0.149
Picture		

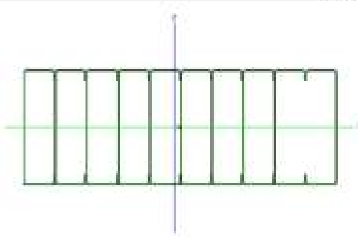
Cross-section properties 4X350S150-43

4X350S150-43		
Type	3x350S150-43	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.241	
A_y [inch^2], A_z [inch^2]	0.595	0.651
A_L [inch^2/inch], A_D [inch^2/inch]	1.94e+01	5.31e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	2.284	4.800
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.400	4.657
i_y [inch], i_z [inch]	1.391	1.937
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.371	1.490
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.592	2.018
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	7.96e+01	7.96e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.01e+02	1.01e+02
d_y [inch], d_z [inch]	0.197	0.000
I_t [inch^4], I_w [inch^6]	0.248	14.699
$β_y$ [inch], $β_z$ [inch]	0.000	-0.041
Picture		

Cross-section properties 5X350S150-43

5X350S150-43		
Type	5x350S150-43	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.551	
A_y [inch^2], A_z [inch^2]	0.745	0.814
A_L [inch^2/inch], A_D [inch^2/inch]	2.26e+01	6.60e+01
$c_{Y.ucs}$ [inch], $c_{Z.ucs}$ [inch]	3.110	4.800
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	3.000	8.988
i_y [inch], i_z [inch]	1.391	2.407
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.714	2.365
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.990	3.252
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.95e+01	9.95e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.63e+02	1.63e+02
d_y [inch], d_z [inch]	0.080	0.000
I_t [inch^4], I_w [inch^6]	0.301	23.876
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

Cross-section properties 10X550S150-54

10X550S150-54		
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	4.911	
A_y [inch^2], A_z [inch^2]	2.299	3.033
A_L [inch^2/inch], A_D [inch^2/inch]	4.35e+01	1.70e+02
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	6.731	4.800
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	21.253	97.531
i_y [inch], i_z [inch]	2.080	4.456
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	7.728	12.526
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	9.248	18.769
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+02	4.62e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.38e+02	9.38e+02
d_y [inch], d_z [inch]	0.238	0.000
I_t [inch^4], I_w [inch^6]	1.072	494.843
β_y [inch], β_z [inch]	0.000	-0.096
Picture		

APPLIED LOADS

Loads on columns are transmitted from floor and roof beams, see sections 2.3.1, 2.3.2.

MAXIMUM FORCES

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
C1	12' 0.000"	LRFD-Ult (auto)/1	**-2358.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C1	0.000"	LRFD-Ult (auto)/2	**-4985.8**	0.0	0.0	0.0	0.0	**0.0**
C2	0.000"	LRFD-Ult (auto)/1	-814.3	0.0	**0.0**	0.0	**0.0**	0.0
C2	12' 0.000"	LRFD-Ult (auto)/2	**233.2**	**0.0**	**0.0**	**0.0**	**0.3**	**0.0**
C2	0.000"	LRFD-Ult (auto)/3	**-1014.0**	0.0	0.0	0.0	0.0	**0.0**
C3	12' 0.000"	LRFD-Ult (auto)/4	**-3263.0**	**0.0**	**0.0**	**0.0**	**0.0**	-0.1
C3	0.000"	LRFD-Ult (auto)/2	**-9536.8**	**0.0**	0.0	0.0	**0.0**	**0.0**
C3	12' 0.000"	LRFD-Ult (auto)/2	-9464.6	0.0	0.0	0.0	0.0	**-0.3**
C4	12' 0.000"	LRFD-Ult (auto)/4	**-6794.4**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C4	0.000"	LRFD-Ult (auto)/2	**-19254.5**	0.0	0.0	0.0	0.0	**0.0**
C5	23' 6.000"	LRFD-Ult (auto)/5	**227.6**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C5	0.000"	LRFD-Ult (auto)/2	**-22940.3**	0.0	0.0	0.0	0.0	**0.0**
C6	23' 6.000"	LRFD-Ult (auto)/5	**323.2**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C6	0.000"	LRFD-Ult (auto)/6	**-13921.4**	0.0	0.0	0.0	0.0	**0.0**
C7	12' 0.000"	LRFD-Ult (auto)/1	**-2316.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C7	0.000"	LRFD-Ult (auto)/6	**-4534.9**	0.0	0.0	0.0	0.0	**0.0**
C8	12' 0.000"	LRFD-Ult (auto)/1	**-8332.5**	**0.0**	**0.0**	**0.1**	**0.0**	**0.0**
C8	0.000"	LRFD-Ult (auto)/2	**-23361.8**	0.0	0.0	**0.4**	0.0	**0.0**
C9	12' 0.000"	LRFD-Ult (auto)/4	**-7809.5**	**0.0**	**0.0**	**0.1**	**0.0**	**0.0**
C9	0.000"	LRFD-Ult (auto)/2	**-26035.4**	0.0	0.0	**0.2**	0.0	**0.0**
C10	12' 0.000"	LRFD-Ult (auto)/1	**-2100.7**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C10	0.000"	LRFD-Ult (auto)/2	**-3719.4**	0.0	0.0	0.0	0.0	**0.0**
C11	12' 0.000"	LRFD-Ult (auto)/1	**-2100.7**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C11	0.000"	LRFD-Ult (auto)/2	**-3719.4**	0.0	0.0	0.0	0.0	**0.0**

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
C12	12' 0.000"	LRFD-Ult (auto)/1	**-2393.6**	0.0	**0.0**	**0.0**	**0.0**	0.0
C12	0.000"	LRFD-Ult (auto)/7	-5040.7	**-1506.8**	0.0	0.0	0.0	**18082.0**
C12	0.000"	LRFD-Ult (auto)/2	**-9903.5**	0.0	0.0	0.0	**0.0**	0.0
C12	0.000"	LRFD-Ult (auto)/8	-5040.7	**1506.8**	0.0	0.0	0.0	**-18082.0**
C13	12' 0.000"	LRFD-Ult (auto)/1	**-543.1**	0.0	**0.0**	**0.0**	**0.0**	0.0
C13	0.000"	LRFD-Ult (auto)/7	-1854.8	**-1514.2**	0.0	0.0	0.0	**18170.4**
C13	0.000"	LRFD-Ult (auto)/2	**-4164.7**	0.0	0.0	0.0	**0.0**	0.0
C13	0.000"	LRFD-Ult (auto)/8	-1854.8	**1514.2**	0.0	0.0	0.0	**-18170.4**
C14	12' 0.000"	LRFD-Ult (auto)/1	**-2004.4**	0.0	**0.0**	**0.0**	**0.0**	0.0
C14	0.000"	LRFD-Ult (auto)/7	-4318.2	**-1506.8**	0.0	0.0	0.0	**18082.1**
C14	0.000"	LRFD-Ult (auto)/6	**-8498.9**	0.0	0.0	0.0	**0.0**	0.0
C14	0.000"	LRFD-Ult (auto)/8	-4318.2	**1506.8**	0.0	0.0	0.0	**-18082.1**
C15	12' 0.000"	LRFD-Ult (auto)/1	**-3858.3**	0.0	**0.0**	**0.0**	**0.0**	0.0
C15	0.000"	LRFD-Ult (auto)/7	-5959.6	**-1512.1**	0.0	0.0	0.0	**18145.5**
C15	0.000"	LRFD-Ult (auto)/2	**-12276.8**	0.0	0.0	0.0	**0.0**	0.0
C15	0.000"	LRFD-Ult (auto)/8	-5959.6	**1512.1**	0.0	0.0	0.0	**-18145.5**
C16	12' 0.000"	LRFD-Ult (auto)/1	**-3518.0**	**0.0**	**-12.2**	**0.0**	-146.0	**0.0**
C16	12' 0.000"	LRFD-Ult (auto)/6	-6347.0	0.0	**-33.5**	0.0	**-402.4**	0.0
C16	0.000"	LRFD-Ult (auto)/2	**-7901.4**	0.0	-32.3	0.0	**0.0**	**0.0**
C17	12' 0.000"	LRFD-Ult (auto)/1	**-4723.5**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C17	0.000"	LRFD-Ult (auto)/2	**-13615.4**	0.0	0.0	0.0	0.0	**0.0**
C18	12' 0.000"	LRFD-Ult (auto)/1	**-2181.6**	**0.5**	**-6.9**	**3.1**	-82.3	5.6
C18	12' 0.000"	LRFD-Ult (auto)/2	-4485.5	1.4	**-15.9**	9.3	**-190.6**	16.8
C18	0.000"	LRFD-Ult (auto)/2	**-4557.7**	1.4	-15.9	9.3	**0.0**	**0.0**
C18	12' 0.000"	LRFD-Ult (auto)/6	-4463.2	**1.5**	-15.6	**9.6**	-187.5	**17.5**

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
		(auto)/6						
C14	0.000"	LRFD-Ult (auto)/8	-4318.2	**1506.8**	0.0	0.0	0.0	**-18082.1**
C15	12' 0.000"	LRFD-Ult (auto)/1	**-3858.3**	0.0	**0.0**	**0.0**	**0.0**	0.0
C15	0.000"	LRFD-Ult (auto)/7	-5959.6	**-1512.1**	0.0	0.0	0.0	**18145.5**
C15	0.000"	LRFD-Ult (auto)/2	**-12276.8**	0.0	0.0	0.0	**0.0**	0.0
C15	0.000"	LRFD-Ult (auto)/8	-5959.6	**1512.1**	0.0	0.0	0.0	**-18145.5**
C16	12' 0.000"	LRFD-Ult (auto)/1	**-3518.0**	**0.0**	**-12.2**	**0.0**	-146.0	**0.0**
C16	12' 0.000"	LRFD-Ult (auto)/6	-6347.0	0.0	**-33.5**	0.0	**-402.4**	0.0
C16	0.000"	LRFD-Ult (auto)/2	**-7901.4**	0.0	-32.3	0.0	**0.0**	**0.0**
C17	12' 0.000"	LRFD-Ult (auto)/1	**-4723.5**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C17	0.000"	LRFD-Ult (auto)/2	**-13615.4**	0.0	0.0	0.0	0.0	**0.0**
C18	12' 0.000"	LRFD-Ult (auto)/1	**-2181.6**	**0.5**	**-6.9**	**3.1**	-82.3	5.6
C18	12' 0.000"	LRFD-Ult (auto)/2	-4485.5	1.4	**-15.9**	9.3	**-190.6**	16.8
C18	0.000"	LRFD-Ult (auto)/2	**-4557.7**	1.4	-15.9	9.3	**0.0**	**0.0**
C18	12' 0.000"	LRFD-Ult (auto)/6	-4463.2	**1.5**	-15.6	**9.6**	-187.5	**17.5**
C19	12' 0.000"	LRFD-Ult (auto)/1	**-1199.4**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C19	0.000"	LRFD-Ult (auto)/6	**-3779.1**	0.0	0.0	0.0	0.0	**0.0**
C20	12' 0.000"	LRFD-Ult (auto)/1	**-615.2**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C20	0.000"	LRFD-Ult (auto)/9	**-2187.5**	0.0	0.0	0.0	0.0	**0.0**
C21	12' 0.000"	LRFD-Ult (auto)/1	**-772.6**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C21	0.000"	LRFD-Ult (auto)/6	**-1581.0**	0.0	0.0	0.0	0.0	**0.0**
C22	12' 0.000"	LRFD-Ult (auto)/1	**-4286.6**	**-2.2**	**12.3**	**-4.5**	148.0	-26.4
C22	0.000"	LRFD-Ult (auto)/2	**-14135.3**	-3.2	28.6	-10.4	**0.0**	**0.0**
C22	12' 0.000"	LRFD-Ult (auto)/2	-14063.1	-3.2	**28.6**	**-10.4**	**342.6**	-37.8
C22	12' 0.000"	LRFD-Ult (auto)/9	-12337.2	**-4.1**	26.8	-9.8	321.3	**-48.7**
C23	12' 0.000"	LRFD-Ult (auto)/1	**-1610.8**	1.7	**-5.5**	**-4.4**	-65.7	20.8
C23	0.000"	LRFD-Ult (auto)/10	-4576.0	**1.7**	-11.9	-9.6	**0.0**	0.0

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
C23	12' 0.000"	LRFD-Ult (auto)/2	-4845.5	1.8	**-12.7**	**-10.2**	**-152.0**	21.0
C23	0.000"	LRFD-Ult (auto)/2	**-4917.8**	1.8	-12.7	-10.2	0.0	**0.0**
C23	12' 0.000"	LRFD-Ult (auto)/11	-3601.3	**2.9**	-10.9	-8.8	-130.3	**34.2**
C24	12' 0.000"	LRFD-Ult (auto)/1	**-4362.4**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C24	0.000"	LRFD-Ult (auto)/2	**-13237.5**	0.0	0.0	0.0	0.0	**0.0**
C25	23' 6.000"	LRFD-Ult (auto)/5	**95.5**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
C25	0.000"	LRFD-Ult (auto)/12	**-3468.8**	0.0	0.0	0.0	0.0	**0.0**
C26	12' 0.000"	LRFD-Ult (auto)/11	**1998.8**	**0.0**	0.0	**0.0**	0.0	**0.0**
C26	12' 0.000"	LRFD-Ult (auto)/8	1952.4	0.0	**0.0**	0.0	**0.0**	0.0
C26	12' 0.000"	LRFD-Ult (auto)/7	1952.4	0.0	**0.0**	0.0	**0.0**	0.0
C26	0.000"	LRFD-Ult (auto)/5	**1154.0**	0.0	0.0	0.0	0.0	**0.0**
C27	12' 0.000"	LRFD-Ult (auto)/1	**-3808.8**	**0.0**	0.0	**0.0**	0.0	**0.0**
C27	12' 0.000"	LRFD-Ult (auto)/7	-5515.2	0.0	**0.0**	0.0	**0.0**	0.0
C27	12' 0.000"	LRFD-Ult (auto)/8	-5515.2	0.0	**0.0**	0.0	**0.0**	0.0
C27	0.000"	LRFD-Ult (auto)/9	**-9160.5**	0.0	0.0	0.0	0.0	**0.0**

Name	Combination key
LRFD-Ult (auto)/1	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + W(up)
LRFD-Ult (auto)/2	1.20*DL1 + 1.60*L + 0.50*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4
LRFD-Ult (auto)/3	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + W(up)
LRFD-Ult (auto)/4	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + Wx+
LRFD-Ult (auto)/5	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + W(down)
LRFD-Ult (auto)/6	1.20*DL1 + 0.50*L + 1.60*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4
LRFD-Ult (auto)/7	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + Wx-
LRFD-Ult (auto)/8	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + Wx+
LRFD-Ult (auto)/9	1.20*DL1 + 1.60*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*W(down)
LRFD-Ult (auto)/10	1.20*DL1 + 1.60*L + 1.20*DL2 + 1.20*DL3 + 1.20*DL4
LRFD-Ult (auto)/11	1.40*DL1 + 1.40*DL2 + 1.40*DL3 + 1.40*DL4
LRFD-Ult (auto)/12	1.20*DL1 + 1.60*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4

STEEL MEMBER C9 CHECK

AISI S100-16 LRFD Check

Member C9	5?350S150-43	A913 grade 50	LRFD-Ult (auto)	0.93

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

The critical check is on position 0.00 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	-26040.07	lbf
Vux	0.00	lbf
Vuy	-0.00	lbf
Mut	0.22	lbfft
Mux	0.00	lbfft
Muy	0.67	lbfft

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.479	50.0 50.0	1.00	4.000	3387.5	0.121	1.000	0.479 -	- -	-	- -	-
2	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-
3	3.457	50.0 50.0	1.00	4.000	16.2	1.755	0.498	1.722 -	- -	-	- -	-
4	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
5	0.478	50.0 50.0	1.00	4.000	3387.5	0.121	1.000	0.478 -	- -	-	- -	-
6	0.479	50.0 50.0	1.00	4.000	3387.5	0.121	1.000	0.479 -	- -	-	- -	-
7	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-
8	2.500	50.0 50.0	1.00	4.000	31.0	1.269	0.651	1.628 -	- -	-	- -	-
9	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-
10	0.478	50.0 50.0	1.00	4.000	3387.5	0.121	1.000	0.478 -	- -	-	- -	-
11	0.478	50.0 50.0	1.00	0.430	364.2	0.371	1.000	0.478 -	- -	-	- -	-
12	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-
13	2.500	50.0 50.0	1.00	4.000	31.0	1.269	0.651	1.628 -	- -	-	- -	-
14	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-
15	0.478	50.0 50.0	1.00	0.430	364.2	0.371	1.000	0.478 -	- -	-	- -	-
16	0.478	50.0 50.0	1.00	4.000	3387.5	0.121	1.000	0.478 -	- -	-	- -	-
17	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-
18	3.457	50.0 50.0	1.00	4.000	16.2	1.755	0.498	1.722 -	- -	-	- -	-
19	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-
20	0.478	50.0 50.0	1.00	4.000	3387.5	0.121	1.000	0.478 -	- -	-	- -	-
21	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-
22	2.500	50.0 50.0	1.00	4.000	31.0	1.269	0.651	1.628 -	- -	-	- -	-
23	1.457	50.0 50.0	1.00	4.000	91.3	0.740	0.950	1.384 -	- -	-	- -	-

Table of values		
Fn	50.0	ksi
Ae	1.284	inch^2
Pno	64210.7	lbf
Resistance factor	0.85	
Unity check	0.48	-

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	12	12	ft
Effective Length factor K	1.00	1.00	
Effective Length	12	12	ft
Slenderness	59.82	103.56	
Flexural Buckling stress Fcre	80.0	26.7	ksi

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

Table of values		
Sigma,ex	80.0	ksi
Sigma,ey	26.7	ksi
Kt	1.00	
Lt	12	ft
Sigma,t	307.1	ksi
Sigma,TF	26.7	ksi
Torsional (-Flexural) buckling stress Fcre	26.7	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	0.479	22.8 22.8	1.00	4.000	3387.5	0.082	1.000	0.479 -	- -	-	- -	-
2	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-
3	3.457	22.8 22.8	1.00	4.000	16.2	1.186	0.687	2.374 -	- -	-	- -	-
4	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-
5	0.478	22.8 22.8	1.00	4.000	3387.5	0.082	1.000	0.478 -	- -	-	- -	-
6	0.479	22.8 22.8	1.00	4.000	3387.5	0.082	1.000	0.479 -	- -	-	- -	-
7	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-
8	2.500	22.8 22.8	1.00	4.000	31.0	0.858	0.867	2.167 -	- -	-	- -	-
9	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-
10	0.478	22.8 22.8	1.00	4.000	3387.5	0.082	1.000	0.478 -	- -	-	- -	-
11	0.478	22.8 22.8	1.00	0.430	364.2	0.250	1.000	0.478 -	- -	-	- -	-
12	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-
13	2.500	22.8 22.8	1.00	4.000	31.0	0.858	0.867	2.167 -	- -	-	- -	-
14	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-

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]
15	0.478	22.8 22.8	1.00	0.430	364.2	0.250	1.000	0.478 -	- -	-	- -	-
16	0.478	22.8 22.8	1.00	4.000	3387.5	0.082	1.000	0.478 -	- -	-	- -	-
17	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-
18	3.457	22.8 22.8	1.00	4.000	16.2	1.186	0.687	2.374 -	- -	-	- -	-
19	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-
20	0.478	22.8 22.8	1.00	4.000	3387.5	0.082	1.000	0.478 -	- -	-	- -	-
21	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-
22	2.500	22.8 22.8	1.00	4.000	31.0	0.858	0.867	2.167 -	- -	-	- -	-
23	1.457	22.8 22.8	1.00	4.000	91.3	0.500	1.000	1.457 -	- -	-	- -	-

Table of values		
Fe	26.7	ksi
lambda, c	1.37	
Fn	22.8	ksi
Ae	1.441	$inch^2$
Pn	32903.4	lbf
Resistance factor	0.85	
Unity check	0.93	-

The member satisfies the check !

STEEL MEMBER C15 CHECK

AISI S100-16 LRFD Check

Member C15	General	A913 grade 50	LRFD-Ult (auto)	0.26

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

The critical check is on position 0.00 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	-7973.42	lbf
Vux	-0.00	lbf
Vuy	-1512.13	lbf
Mut	0.00	lbfft
Mux	18145.53	lbfft
Muy	0.00	lbfft

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

Table of values		
Sxe	25.013	$inch^3$
Mnxo	104222.9	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		
Lltb	12' 0.000"	ft
Sigma,ey	59.7	ksi
Kt	1.00	
Lt	12' 0.000"	ft
Sigma,t	157.8	ksi
Cb	1.67	
Sfx	7.728	$inch^3$
Fcre	506.4	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	0.051	1532.5
2	0.000	0.0
3	0.294	4048.8
4	0.000	0.0
5	0.051	1532.5
6	0.051	1532.5
7	0.000	0.0
8	0.243	4870.5
9	0.000	0.0
10	0.051	1532.5
11	0.051	1532.5
12	0.000	0.0
13	0.243	4870.5
14	0.000	0.0
15	0.051	1532.5
16	0.051	1532.5
17	0.000	0.0
18	0.243	4870.5
19	0.000	0.0
20	0.051	1532.5
21	0.051	1532.5
22	0.000	0.0
23	0.243	4870.5
24	0.000	0.0
25	0.051	1532.5
26	0.051	1532.5
27	0.000	0.0
28	0.243	4870.5
29	0.000	0.0
30	0.051	1532.5
31	0.051	1532.5
32	0.000	0.0
33	0.243	4870.5
34	0.000	0.0
35	0.051	1532.5
36	0.051	1532.5
37	0.000	0.0
38	0.243	4870.5
39	0.000	0.0
40	0.051	1532.5
41	0.051	1532.5
42	0.000	0.0
43	0.243	4870.5
44	0.000	0.0
45	0.051	1532.5
46	0.000	0.0
47	0.294	4048.8
48	0.000	0.0

Table of values		
Vn,y	74646.6	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	104222.9	lbfft
Vny	74646.6	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.04+0.00) = 0.19

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

Table of values		
Fn	50.0	ksi
Ae	3.666	$inch^2$
Pno	183315.7	lbf
Resistance factor	0.85	
Unity check	0.05	-

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	12	12	ft
Effective Length factor K	1.00	1.00	
Effective Length	12	12	ft
Slenderness	32.31	69.22	
Flexural Buckling stress Fcre	274.2	59.7	ksi

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

Table of values		
Sigma,ex	274.2	ksi
Sigma,ey	59.7	ksi
Kt	1.00	
Lt	12	ft
Sigma,t	157.8	ksi
Sigma,TF	59.7	ksi
Torsional (-Flexural) buckling stress Fcre	59.7	ksi

Table of values		
Fe	59.7	ksi
lambda, c	0.92	
Fn	35.2	ksi
Ae	3.857	$inch^2$
Pn	135788.6	lbf
Resistance factor	0.85	
Unity check	0.07	-

Combined Compressive Axial Load and Bending

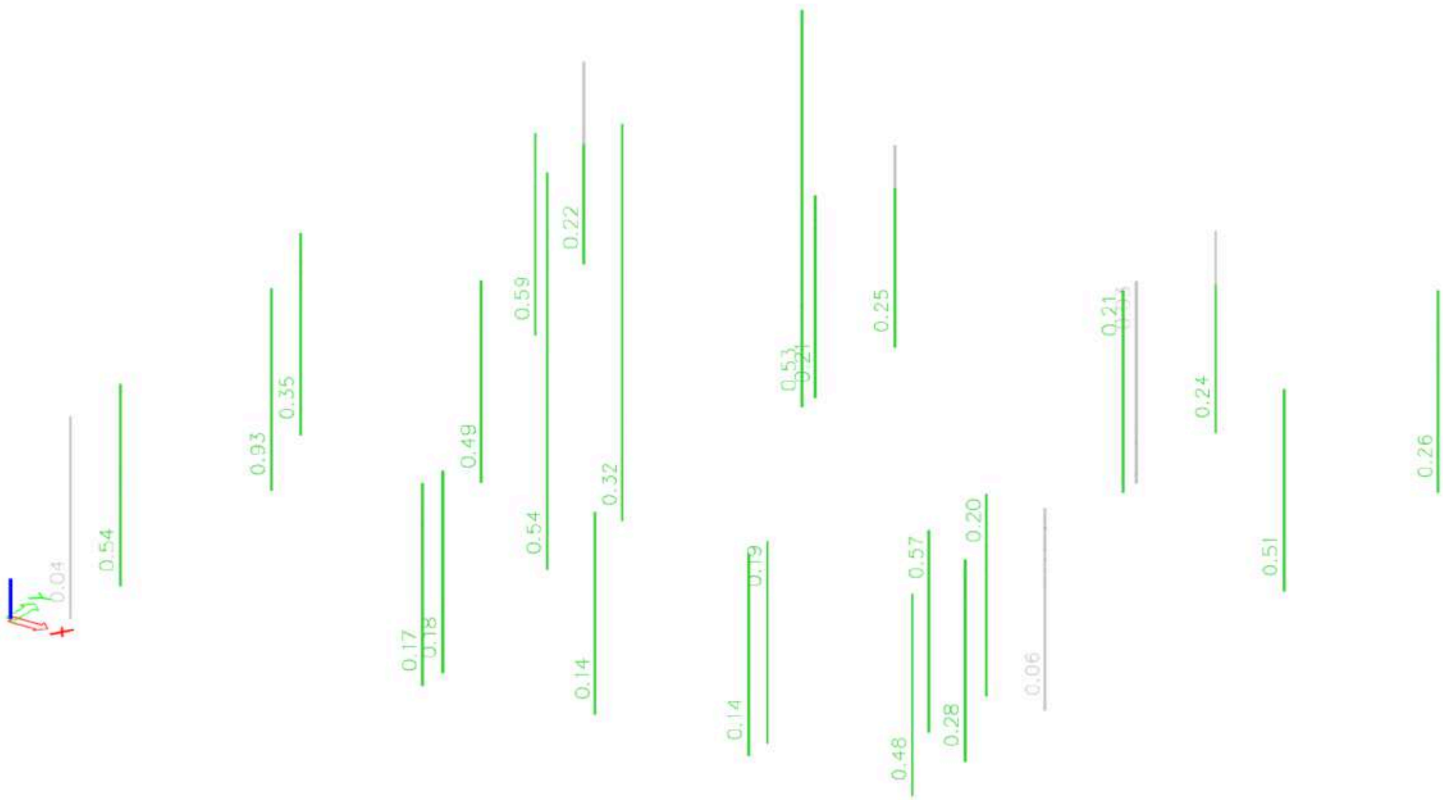
According to article H1.2 and formulas (C5.2.1-3)

Table of values		
Mnx	104222.9	lbfft
Pn	135788.6	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.07+0.19+0.00 = 0.26 - (C5.2.1-3)

The member satisfies the check !

Unity check



0.04
0.54
0.93
0.35
0.17
0.18
0.49
0.59
0.54
0.22
0.14
0.32
0.53
0.21
0.19
0.14
0.25
0.48
0.57
0.28
0.20
0.06
0.21
0.24
0.51
0.26

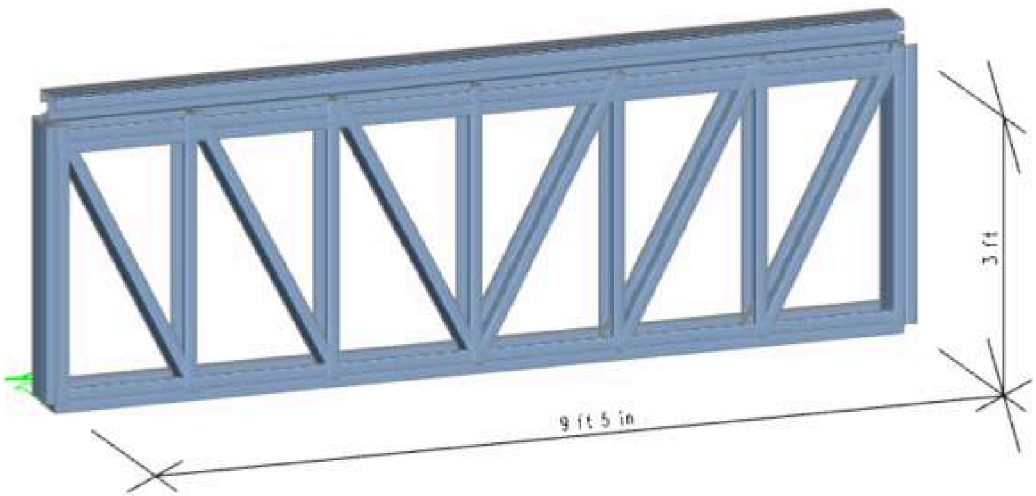
Check of steel

Linear calculation, Extreme : Member
Selection : All
Combinations : LRFD-Ult (auto)

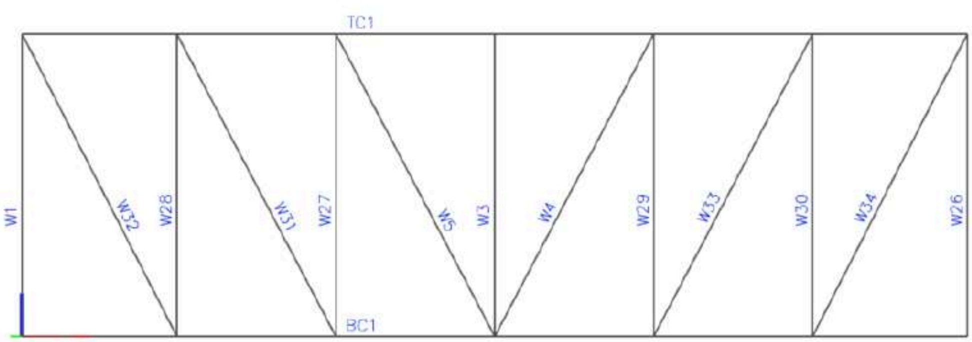
Case	Member	css	mat	dx [ft]	un.check [-]
LRFD-Ult (auto)/2	C1	3X550S150-54	A913 grade 50	0.000"	0.18
LRFD-Ult (auto)/3	C2	3X550S150-54	A913 grade 50	0.000"	0.04
LRFD-Ult (auto)/2	C3	3X550S150-54	A913 grade 50	0.000"	0.35
LRFD-Ult (auto)/2	C4	3X550S150-54	A913 grade 50	0.000"	0.59
LRFD-Ult (auto)/2	C5	4X550S150-54 -	A913 grade 50	0.000"	0.53
LRFD-Ult (auto)/4	C6	4X550S150-54 -	A913 grade 50	0.000"	0.32
LRFD-Ult (auto)/4	C7	3X550S150-54	A913 grade 50	0.000"	0.17
LRFD-Ult (auto)/2	C8	4X550S150-54 -	A913 grade 50	0.000"	0.54
LRFD-Ult (auto)/2	C9	5X350S150-43 - 5x350S150-43	A913 grade 50	0.000"	0.93
LRFD-Ult (auto)/2	C10	3X550S150-54	A913 grade 50	0.000"	0.14
LRFD-Ult (auto)/2	C11	3X550S150-54	A913 grade 50	0.000"	0.14
LRFD-Ult (auto)/1	C12	10X550S150-54	A913 grade 50	0.000"	0.25
LRFD-Ult (auto)/1	C13	10X550S150-54	A913 grade 50	0.000"	0.22
LRFD-Ult (auto)/1	C14	10X550S150-54	A913 grade 50	0.000"	0.24
LRFD-Ult (auto)/1	C15	10X550S150-54	A913 grade 50	0.000"	0.26
LRFD-Ult (auto)/2	C16	4X550S150-54 -	A913 grade 50	12' 0.000"	0.21
LRFD-Ult (auto)/2	C17	3X550S150-54	A913 grade 50	0.000"	0.51
LRFD-Ult (auto)/2	C18	3X550S150-54	A913 grade 50	12' 0.000"	0.19
LRFD-Ult (auto)/4	C19	2X550S150-54 -	A913 grade 50	0.000"	0.48
LRFD-Ult (auto)/5	C20	2X550S150-54 -	A913 grade 50	0.000"	0.28
LRFD-Ult (auto)/4	C21	3X550S150-54	A913 grade 50	0.000"	0.06
LRFD-Ult (auto)/2	C22	3X550S150-54	A913 grade 50	12' 0.000"	0.57
LRFD-Ult (auto)/2	C23	3X550S150-54	A913 grade 50	12' 0.000"	0.20
LRFD-Ult (auto)/2	C24	3X550S150-54	A913 grade 50	0.000"	0.49
LRFD-Ult (auto)/6	C25	4X350S150-43 - 3x350S150-43	A913 grade 50	0.000"	0.54
LRFD-Ult (auto)/7	C26	3X550S150-54	A913 grade 50	12' 0.000"	0.03
LRFD-Ult (auto)/5	C27	4X550S150-54 -	A913 grade 50	0.000"	0.21

2.3.4 GARAGE HEADER GH1 DESIGN

General scheme



Members number



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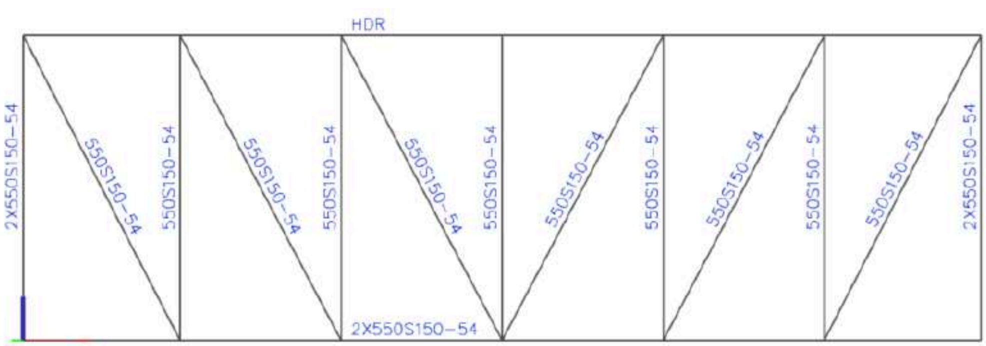
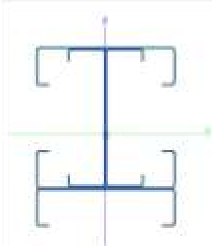
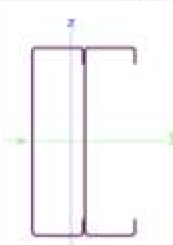
Members cross-sections



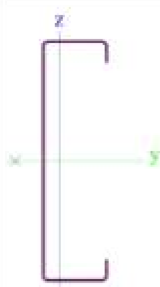
Cross-sections properties HDR

HDR		
Type	5x550S150-54	
Shape type	Thin-walled	
Item material	A572 grade 50	
Fabrication	cold formed	
Colour		
A [$inch^2$]	2.456	
A_y [$inch^2$], A_z [$inch^2$]	1.314	0.859
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	5.94e+01	6.08e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1.322	-0.372
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	14.396	6.806
i_y [inch], i_z [inch]	2.421	1.665
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	3.928	2.475
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	5.311	3.160
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.66e+02	2.66e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.58e+02	1.58e+02
d_y [inch], d_z [inch]	0.000	-0.043
I_t [$inch^4$], I_w [$inch^6$]	0.009	61.705
$β_y$ [inch], $β_z$ [inch]	0.634	0.000
Picture		

Cross-sections properties 2X550S150-54

2X550S150-54		
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [$inch^2$]	0.982	
A_y [$inch^2$], A_z [$inch^2$]	0.318	0.607
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	2.16e+01	3.51e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.659	4.800
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	4.251	0.831
i_y [inch], i_z [inch]	2.080	0.920
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	1.546	0.447
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	1.850	0.737
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.25e+01	9.25e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.68e+01	3.68e+01
d_y [inch], d_z [inch]	-1.457	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.090	2.759
$β_y$ [inch], $β_z$ [inch]	0.000	4.222
Picture		

Cross-sections properties 550S150-54

550S150-54		
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]	-0.091	4.800
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	2.125	0.139
i_y [inch], i_z [inch]	2.080	0.533
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.773	0.126
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	0.925	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.07e+00	9.07e+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	5.901
Picture		

APPLIED LOADS

LOAD ON GARAGE HEADER GH1					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load					
Wall Dead Load	45	3.33		150.00	
Roof Dead Load	20.00	12.17		243.33	
Floor Live Load					
Roof Live Load	20.00	12.17		243.33	
Roof Wind (up)	-53.50	12.17	0.3848	-250.46	
Roof Wind (down)	17.30	12.17	0.3848	80.99	

Wind loads are applied perpendicular to the roof beam. Liner load with “-” - uplift.

MAXIMUM FORCES

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
W1	3' 0.000"	LRFD-Ult (auto)/1	**-595.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W1	0.000"	LRFD-Ult (auto)/2	**-4641.0**	0.0	0.0	0.0	0.0	**0.0**
W3	3' 0.000"	LRFD-Ult (auto)/1	**-144.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W3	0.000"	LRFD-Ult (auto)/2	**-1191.7**	0.0	0.0	0.0	0.0	**0.0**
W4	3' 4.706"	LRFD-Ult (auto)/2	**291.6**	-1.6	**0.0**	**0.0**	**0.0**	0.0
W4	3' 4.706"	LRFD-Ult (auto)/3	185.1	**-1.9**	0.0	0.0	0.0	0.0
W4	0.000"	LRFD-Ult (auto)/3	178.1	**1.9**	0.0	0.0	0.0	0.0
W4	0.000"	LRFD-Ult (auto)/1	**36.7**	1.2	0.0	0.0	0.0	**0.0**
W4	1' 8.353"	LRFD-Ult (auto)/3	181.6	0.0	0.0	0.0	0.0	**1.6**
W5	3' 4.706"	LRFD-Ult (auto)/2	**1008.8**	-1.6	**0.0**	**0.0**	**0.0**	0.0
W5	3' 4.706"	LRFD-Ult (auto)/3	632.6	**-1.9**	0.0	0.0	0.0	0.0
W5	0.000"	LRFD-Ult (auto)/3	625.6	**1.9**	0.0	0.0	0.0	0.0
W5	0.000"	LRFD-Ult (auto)/1	**129.4**	1.2	0.0	0.0	0.0	**0.0**
W5	1' 8.353"	LRFD-Ult (auto)/3	629.1	0.0	0.0	0.0	0.0	**1.6**

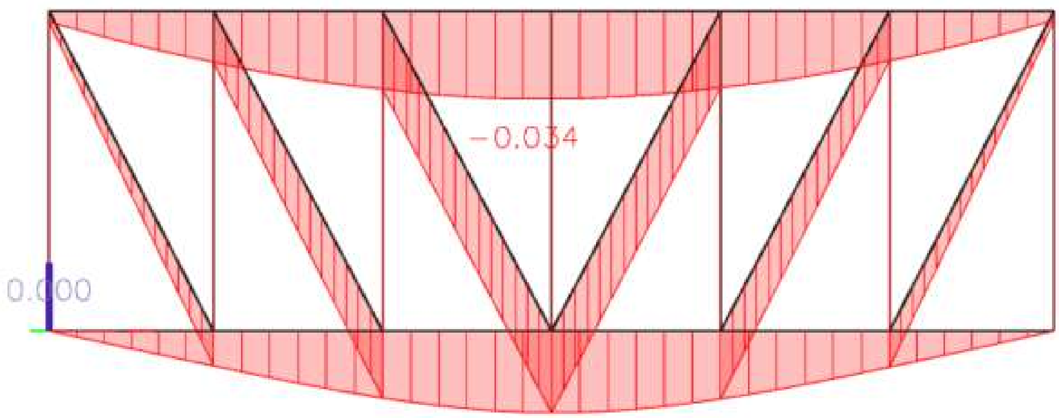
Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
TC1	0.000"	LRFD-Ult (auto)/1	**-116.3**	**0.0**	136.2	**0.0**	**0.0**	**0.0**
TC1	9' 5.000"	LRFD-Ult (auto)/2	-1515.6	0.0	**-1090.1**	0.0	0.0	0.0
TC1	3' 11.000"+	LRFD-Ult (auto)/2	**-2626.0**	0.0	101.7	0.0	**1087.4**	0.0
TC1	0.000"	LRFD-Ult (auto)/2	-901.4	0.0	**1092.6**	0.0	0.0	**0.0**
BC1	6' 3.500"-	LRFD-Ult (auto)/2	**2491.3**	**-32.6**	0.0	0.0	0.0	53.2
BC1	3' 1.500"+	LRFD-Ult (auto)/2	2156.6	**33.9**	**0.0**	**0.0**	**0.0**	51.0
BC1	0.000"	LRFD-Ult (auto)/2	**-914.2**	5.6	0.0	0.0	0.0	**0.0**
BC1	4' 8.500"-	LRFD-Ult (auto)/2	2156.6	27.6	0.0	0.0	0.0	**99.7**
W26	3' 0.000"	LRFD-Ult (auto)/1	**-519.2**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W26	0.000"	LRFD-Ult (auto)/2	**-4054.8**	0.0	0.0	0.0	0.0	**0.0**
W27	3' 0.000"	LRFD-Ult (auto)/1	**-295.6**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W27	0.000"	LRFD-Ult (auto)/2	**-2367.5**	0.0	0.0	0.0	0.0	**0.0**
W28	3' 0.000"	LRFD-Ult (auto)/1	**-440.8**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W28	0.000"	LRFD-Ult (auto)/2	**-3496.0**	0.0	0.0	0.0	0.0	**0.0**
W29	3' 0.000"	LRFD-Ult (auto)/1	**-227.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W29	0.000"	LRFD-Ult (auto)/2	**-1837.1**	0.0	0.0	0.0	0.0	**0.0**
W30	3' 0.000"	LRFD-Ult (auto)/1	**-366.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W30	0.000"	LRFD-Ult (auto)/2	**-2918.3**	0.0	0.0	0.0	0.0	**0.0**
W31	3' 4.706"	LRFD-Ult (auto)/2	**2692.2**	-1.6	**0.0**	**0.0**	**0.0**	0.0
W31	3' 4.706"	LRFD-Ult (auto)/3	1681.8	**-1.9**	0.0	0.0	0.0	0.0
W31	0.000"	LRFD-Ult (auto)/3	1674.8	**1.9**	0.0	0.0	0.0	0.0
W31	0.000"	LRFD-Ult (auto)/1	**345.5**	1.2	0.0	0.0	0.0	**0.0**
W31	1' 8.353"	LRFD-Ult (auto)/3	1678.3	0.0	0.0	0.0	0.0	**1.6**

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
W32	3' 4.475"	LRFD-Ult (auto)/2	**3975.2**	-1.5	**0.0**	**0.0**	**0.0**	0.0
W32	3' 4.475"	LRFD-Ult (auto)/3	2482.0	**-1.8**	0.0	0.0	0.0	0.0
W32	0.000"	LRFD-Ult (auto)/3	2475.0	**1.8**	0.0	0.0	0.0	0.0
W32	0.000"	LRFD-Ult (auto)/1	**510.8**	1.2	0.0	0.0	0.0	**0.0**
W32	1' 8.238"	LRFD-Ult (auto)/3	2478.5	0.0	0.0	0.0	0.0	**1.5**
W33	3' 4.706"	LRFD-Ult (auto)/2	**2093.4**	-1.6	**0.0**	**0.0**	**0.0**	0.0
W33	3' 4.706"	LRFD-Ult (auto)/3	1308.1	**-1.9**	0.0	0.0	0.0	0.0
W33	0.000"	LRFD-Ult (auto)/3	1301.1	**1.9**	0.0	0.0	0.0	0.0
W33	0.000"	LRFD-Ult (auto)/1	**268.1**	1.2	0.0	0.0	0.0	**0.0**
W33	1' 8.353"	LRFD-Ult (auto)/3	1304.6	0.0	0.0	0.0	0.0	**1.6**
W34	3' 4.475"	LRFD-Ult (auto)/2	**3318.9**	-1.5	**0.0**	**0.0**	**0.0**	0.0
W34	3' 4.475"	LRFD-Ult (auto)/3	2072.5	**-1.8**	0.0	0.0	0.0	0.0
W34	0.000"	LRFD-Ult (auto)/3	2065.5	**1.8**	0.0	0.0	0.0	0.0
W34	0.000"	LRFD-Ult (auto)/1	**425.9**	1.2	0.0	0.0	0.0	**0.0**
W34	1' 8.238"	LRFD-Ult (auto)/3	2069.0	0.0	0.0	0.0	0.0	**1.5**

Name	Combination key
LRFD-Ult (auto)/1	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + W(up)
LRFD-Ult (auto)/2	1.20*DL1 + 1.60*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*W(down)
LRFD-Ult (auto)/3	1.40*DL1 + 1.40*DL2 + 1.40*DL3 + 1.40*DL4

DISPLACEMENT

Load case DL + Lr, inch:



The maximum deflection for a web header is 0.034”.

According to TABLE 1604.3 the code IBC 2018, maximum member deflection should not exceed - L/240.

L=9’+5”=113”, 113”/240=0.470”. 0.034"< 0.470”. <u>Deflection is OK!</u>

<u>STEEL MEMBER W28 CHECK</u>

AISI S100-16 LRFD Check

Member W28	A913 grade 50	LRFD-Ult (auto)	0.35

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

The critical check is on position 0.00 ft

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

Internal forces		
Pu	-3495.98	lbf
Vux	-0.00	lbf
Vuy	-0.00	lbf
Mut	-0.00	lbfft
Mux	-0.00	lbfft
Muy	-7.86	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 [inch⁴]	ds [inch]
1	0.473	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-
2	1.446	27.4 -50.0	1.83	54.833	2004.9	0.117	1.000	- 1.446	0.300 0.723	-	- -	-
3	5.446	27.4 27.4	1.00	4.000	10.3	1.629	0.531	2.891 -	- -	-	- -	-
4	1.446	27.4 -50.0	1.83	54.833	2004.9	0.117	1.000	- 1.446	0.300 0.723	-	- -	-
5	0.473	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.125	$inch^3$
Mnyo	520.1	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	955.9	ksi
Kt	1.00	
Lt	3' 0.000"	ft
Sigma,t	72.6	ksi
Cb	1.00	
Sfy	0.355	$inch^3$
Fcre	862.3	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 [inch⁴]	ds [inch]
1	0.473	50.0 50.0	1.00	0.430	146.9	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.0 50.0	1.00	4.000	146.3	0.585	1.000	1.446 -	- -	-	- -	-
3	5.446	50.0 50.0	1.00	4.000	10.3	2.202	0.409	2.226 -	- -	-	- -	-
4	1.446	50.0 50.0	1.00	4.000	146.3	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.0 50.0	1.00	0.430	146.9	0.583	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.0	ksi
Ae	0.327	$inch^2$
Pno	16372.8	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	3	3	ft
Effective Length factor K	1.00	1.00	
Effective Length	3	3	ft
Slenderness	17.31	67.58	
Flexural Buckling stress Fcre	955.9	62.7	ksi

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

Table of values		
Sigma,ex	955.9	ksi
Sigma,ey	62.7	ksi
Kt	1.00	
Lt	3	ft
Sigma,t	72.6	ksi
Sigma,TF	62.7	ksi
Torsional (-Flexural) buckling stress Fcre	62.7	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	35.8 35.8	1.00	0.430	146.9	0.494	1.000	0.473 -	- -	-	- -	-
2	1.446	35.8 35.8	1.00	4.000	146.3	0.495	1.000	1.446 -	- -	-	- -	-
3	5.446	35.8 35.8	1.00	4.000	10.3	1.864	0.473	2.577 -	- -	-	- -	-
4	1.446	35.8 35.8	1.00	4.000	146.3	0.495	1.000	1.446 -	- -	-	- -	-
5	0.473	35.8 35.8	1.00	0.430	146.9	0.494	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	62.7	ksi
lambda, c	0.89	
Fn	35.8	ksi
Ae	0.346	$inch^2$
Pn	12404.9	lbf
Resistance factor	0.85	
Unity check	0.33	-

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

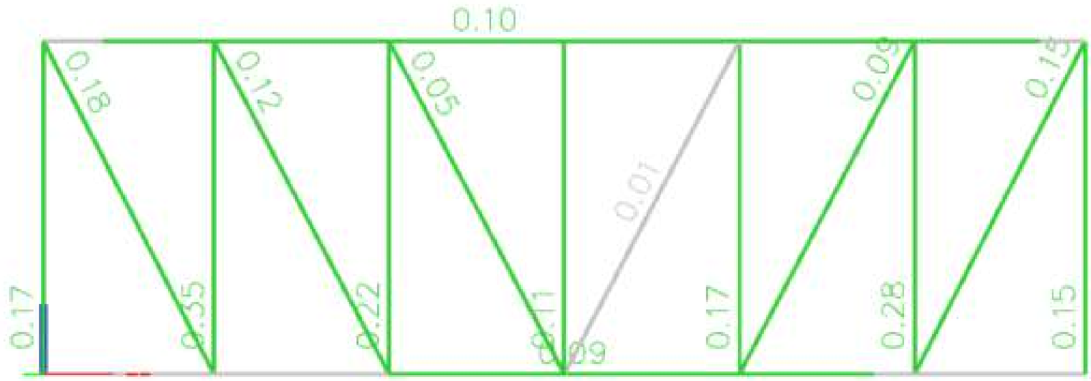
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.473	7.1 7.1	1.00	0.430	146.9	0.220	1.000	0.473 -	- -	-	- -	-
2	1.446	7.1 7.1	1.00	4.000	146.3	0.221	1.000	1.446 -	- -	-	- -	-
3	5.446	7.1 7.1	1.00	4.000	10.3	0.831	0.885	4.819 -	- -	-	- -	-
4	1.446	7.1 7.1	1.00	4.000	146.3	0.221	1.000	1.446 -	- -	-	- -	-
5	0.473	7.1 7.1	1.00	0.430	146.9	0.220	1.000	0.473 -	- -	-	- -	-

Table of values		
Centerline shift ex	0.027	inch
Centerline shift ey	0.000	inch
Additional moment Mx	0.0	lbfft
Additional moment My	-7.9	lbfft
Mny	520.1	lbfft
PEy	30787.0	lbf
Alfa y	0.89	
Cmy	0.85	
Pn	12404.9	lbf
Pno	16372.8	lbf
Resistance factor compression	0.85	
Resistance factor bending y	0.90	

Unity check = 0.33+0.00+0.02 = 0.35 - (H1.2-1)
Unity check = 0.25+0.00+0.02 = 0.27 -

The member satisfies the check !

Unity check



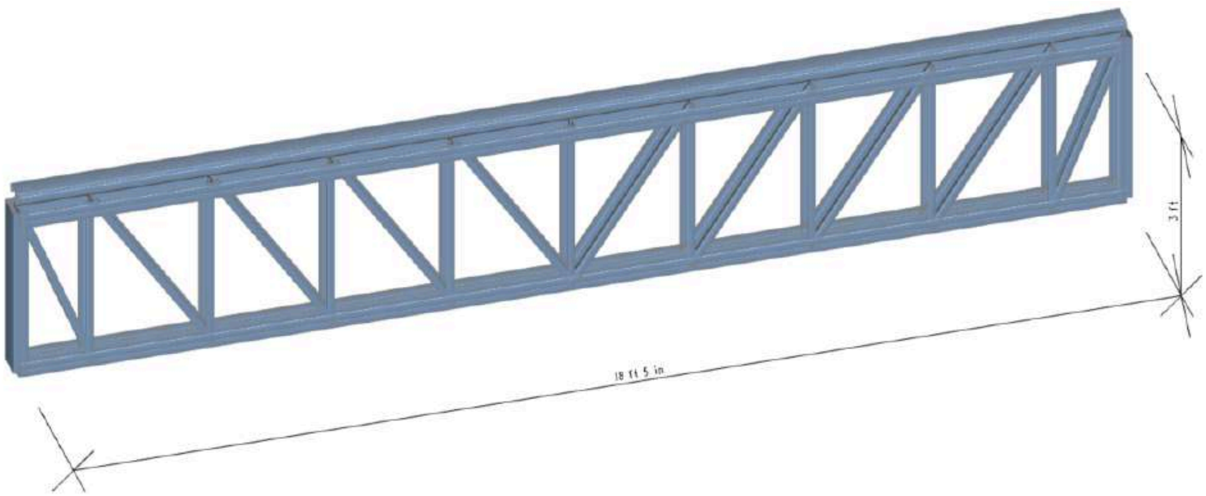
Check of steel

Linear calculation, Extreme : Member
Selection : All
Combinations : LRFD-Ult (auto)

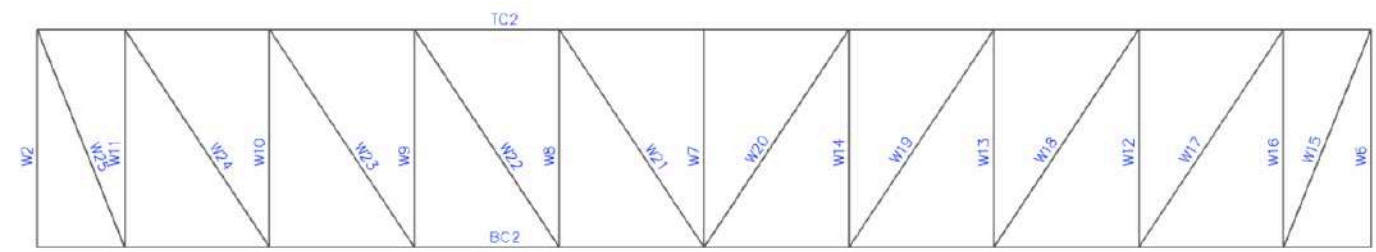
Case	Member	css	mat	dx [ft]	un.check [-]
LRFD-Ult (auto)/1	W1	2X550S150-54 -	A913 grade 50	0.000"	0.17
LRFD-Ult (auto)/1	W3	550S150-54 -	A913 grade 50	0.000"	0.11
LRFD-Ult (auto)/1	W4	550S150-54 -	A913 grade 50	1' 8.353"	0.01
LRFD-Ult (auto)/1	W5	550S150-54 -	A913 grade 50	3' 4.706"	0.05
LRFD-Ult (auto)/1	TC1	HDR - 5x550S150-54	A913 grade 50	3' 11.000"	0.10
LRFD-Ult (auto)/1	BC1	2X550S150-54 -	A913 grade 50	4' 8.500"	0.09
LRFD-Ult (auto)/1	W26	2X550S150-54 -	A913 grade 50	0.000"	0.15
LRFD-Ult (auto)/1	W27	550S150-54 -	A913 grade 50	0.000"	0.22
LRFD-Ult (auto)/1	W28	550S150-54 -	A913 grade 50	0.000"	0.35
LRFD-Ult (auto)/1	W29	550S150-54 -	A913 grade 50	0.000"	0.17
LRFD-Ult (auto)/1	W30	550S150-54 -	A913 grade 50	0.000"	0.28
LRFD-Ult (auto)/1	W31	550S150-54 -	A913 grade 50	3' 4.706"	0.12
LRFD-Ult (auto)/1	W32	550S150-54 -	A913 grade 50	3' 4.475"	0.18
LRFD-Ult (auto)/1	W33	550S150-54 -	A913 grade 50	3' 4.706"	0.09
LRFD-Ult (auto)/1	W34	550S150-54 -	A913 grade 50	3' 4.475"	0.15

2.3.5 GARAGE HEADER GH1 DESIGN

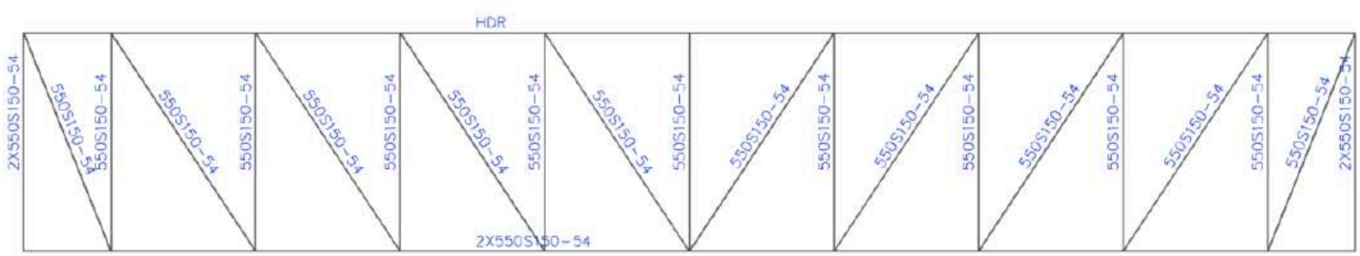
General scheme



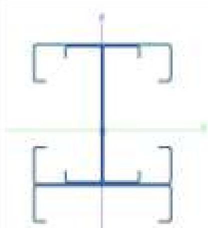
Members number



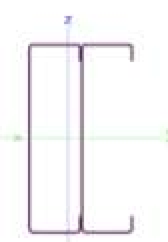
Members cross-sections



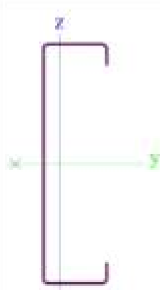
Cross-sections properties HDR

HDR		
Type	5x550S150-54	
Shape type	Thin-walled	
Item material	A572 grade 50	
Fabrication	cold formed	
Colour		
A [$inch^2$]	2.456	
A_y [$inch^2$], A_z [$inch^2$]	1.314	0.859
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	5.94e+01	6.08e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1.322	-0.372
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	14.396	6.806
i_y [inch], i_z [inch]	2.421	1.665
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	3.928	2.475
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	5.311	3.160
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.66e+02	2.66e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.58e+02	1.58e+02
d_y [inch], d_z [inch]	0.000	-0.043
I_t [$inch^4$], I_w [$inch^6$]	0.009	61.705
β_y [inch], β_z [inch]	0.634	0.000
Picture		

Cross-sections properties 2X550S150-54

2X550S150-54		
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [$inch^2$]	0.982	
A_y [$inch^2$], A_z [$inch^2$]	0.318	0.607
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	2.16e+01	3.51e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.659	4.800
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	4.251	0.831
i_y [inch], i_z [inch]	2.080	0.920
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	1.546	0.447
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	1.850	0.737
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.25e+01	9.25e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.68e+01	3.68e+01
d_y [inch], d_z [inch]	-1.457	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.090	2.759
β_y [inch], β_z [inch]	0.000	4.222
Picture		

Cross-sections properties 550S150-54

550S150-54		
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]	-0.091	4.800
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.125	0.139
i_y [inch], i_z [inch]	2.080	0.533
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.773	0.126
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.925	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.07e+00	9.07e+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	5.901
Picture		

APPLIED LOADS

LOAD ON GARAGE HEADER GH2					
Roof slope 5 : 12, α = 22.6					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Note:**
Floor Dead Load					
Wall Dead Load	45	3.33		150.00	
Roof Dead Load	20.00	6.00		120.00	
Floor Live Load					
Roof Live Load	20.00	6.00		120.00	
Roof Wind (up)	-53.50	6.00	0.3848	-123.51	
Roof Wind (down)	17.30	6.00	0.3848	39.94	

Wind loads are applied perpendicular to the roof beam. Liner load with “-” - uplift.

MAXIMUM FORCES

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
W2	3' 0.000"	LRFD-Ult (auto)/1	**-1224.3**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W2	0.000"	LRFD-Ult (auto)/2	**-5117.6**	0.0	0.0	0.0	0.0	**0.0**
W6	3' 0.000"	LRFD-Ult (auto)/1	**-1224.3**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W6	0.000"	LRFD-Ult (auto)/2	**-5117.6**	0.0	0.0	0.0	0.0	**0.0**
TC2	7' 2.500"+	LRFD-Ult (auto)/2	**-7639.2**	0.0	578.3	0.0	560.5	0.0
TC2	0.000"	LRFD-Ult (auto)/1	**-398.2**	**0.0**	233.2	**0.0**	**0.0**	**0.0**
TC2	18' 5.000"	LRFD-Ult (auto)/2	-1659.5	0.0	**-982.2**	0.0	0.0	0.0
TC2	1' 11.500"	LRFD-Ult (auto)/2	-4342.9	0.0	5.2	0.0	**945.7**	0.0
TC2	0.000"	LRFD-Ult (auto)/2	-1659.5	0.0	**982.2**	0.0	0.0	**0.0**
BC2	18' 5.000"	LRFD-Ult (auto)/2	0.0	**-22.0**	0.0	0.0	0.0	0.0
BC2	0.000"	LRFD-Ult (auto)/2	**0.0**	**22.0**	**0.0**	**0.0**	**0.0**	**0.0**
BC2	9' 2.500"-	LRFD-Ult (auto)/2	**7295.8**	5.3	0.0	0.0	0.0	**61.3**
W7	3' 0.000"	LRFD-Ult (auto)/1	**-237.5**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W7	0.000"	LRFD-Ult (auto)/2	**-1033.5**	0.0	0.0	0.0	0.0	**0.0**
W8	3' 0.000"	LRFD-Ult (auto)/1	**-386.2**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W8	0.000"	LRFD-Ult (auto)/2	**-1653.6**	0.0	0.0	0.0	0.0	**0.0**

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
W9	3' 0.000"	LRFD-Ult (auto)/1	**-649.5**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W9	0.000"	LRFD-Ult (auto)/2	**-2751.6**	0.0	0.0	0.0	0.0	**0.0**
W10	3' 0.000"	LRFD-Ult (auto)/1	**-950.4**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W10	0.000"	LRFD-Ult (auto)/2	**-4007.8**	0.0	0.0	0.0	0.0	**0.0**
W11	3' 0.000"	LRFD-Ult (auto)/1	**-980.7**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W11	0.000"	LRFD-Ult (auto)/2	**-4127.1**	0.0	0.0	0.0	0.0	**0.0**
W12	3' 0.000"	LRFD-Ult (auto)/1	**-950.4**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W12	0.000"	LRFD-Ult (auto)/2	**-4007.8**	0.0	0.0	0.0	0.0	**0.0**
W13	3' 0.000"	LRFD-Ult (auto)/1	**-649.5**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W13	0.000"	LRFD-Ult (auto)/2	**-2751.6**	0.0	0.0	0.0	0.0	**0.0**
W14	3' 0.000"	LRFD-Ult (auto)/1	**-386.2**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W14	0.000"	LRFD-Ult (auto)/2	**-1653.6**	0.0	0.0	0.0	0.0	**0.0**
W15	3' 2.810"	LRFD-Ult (auto)/2	**4444.8**	-1.2	**0.0**	**0.0**	**0.0**	0.0
W15	3' 2.810"	LRFD-Ult (auto)/3	3212.6	**-1.4**	0.0	0.0	0.0	0.0
W15	0.000"	LRFD-Ult (auto)/3	3205.6	**1.4**	0.0	0.0	0.0	0.0
W15	0.000"	LRFD-Ult (auto)/1	**1063.6**	0.9	0.0	0.0	0.0	**0.0**
W15	1' 8.898"	LRFD-Ult (auto)/3	3209.4	-0.1	0.0	0.0	0.0	**1.1**
W16	3' 0.000"	LRFD-Ult (auto)/1	**-980.7**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
W16	0.000"	LRFD-Ult (auto)/2	**-4127.1**	0.0	0.0	0.0	0.0	**0.0**
W17	3' 7.267"	LRFD-Ult (auto)/2	**4840.6**	-2.0	**0.0**	**0.0**	**0.0**	0.0
W17	3' 7.267"	LRFD-Ult (auto)/3	3497.8	**-2.3**	0.0	0.0	0.0	0.0
W17	0.000"	LRFD-Ult (auto)/3	3490.8	**2.3**	0.0	0.0	0.0	0.0
W17	0.000"	LRFD-Ult (auto)/1	**1157.7**	1.5	0.0	0.0	0.0	**0.0**
W17	1' 11.076"	LRFD-Ult (auto)/3	3494.6	-0.2	0.0	0.0	0.0	**2.1**
W18	3' 7.267"	LRFD-Ult (auto)/2	**3311.6**	-2.0	**0.0**	**0.0**	**0.0**	0.0
W18	3' 7.267"	LRFD-Ult (auto)/3	2393.5	**-2.3**	0.0	0.0	0.0	0.0

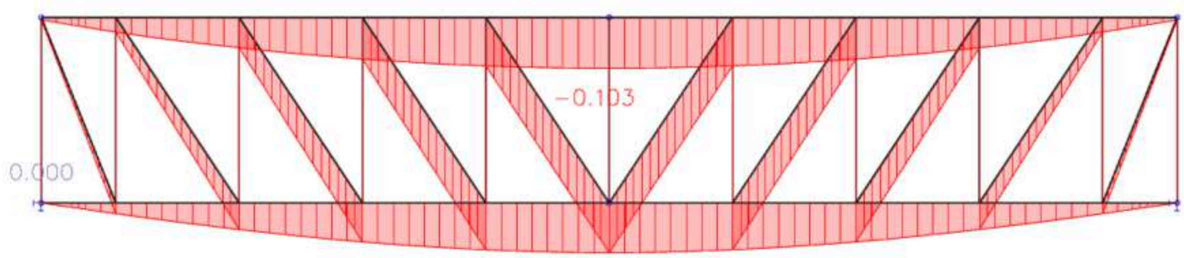
Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
W18	0.000"	LRFD-Ult (auto)/3	2386.5	**2.3**	0.0	0.0	0.0	0.0
W18	0.000"	LRFD-Ult (auto)/1	**791.2**	1.5	0.0	0.0	0.0	**0.0**
W18	1' 11.076"	LRFD-Ult (auto)/3	2390.2	-0.2	0.0	0.0	0.0	**2.1**
W19	3' 7.267"	LRFD-Ult (auto)/2	**2017.7**	-2.0	**0.0**	**0.0**	**0.0**	0.0
W19	3' 7.267"	LRFD-Ult (auto)/3	1458.9	**-2.3**	0.0	0.0	0.0	0.0
W19	0.000"	LRFD-Ult (auto)/3	1451.9	**2.3**	0.0	0.0	0.0	0.0
W19	0.000"	LRFD-Ult (auto)/1	**481.0**	1.5	0.0	0.0	0.0	**0.0**
W19	1' 11.076"	LRFD-Ult (auto)/3	1455.6	-0.2	0.0	0.0	0.0	**2.1**
W20	3' 7.267"	LRFD-Ult (auto)/2	**622.0**	-2.0	**0.0**	**0.0**	**0.0**	0.0
W20	3' 7.267"	LRFD-Ult (auto)/3	451.8	**-2.3**	0.0	0.0	0.0	0.0
W20	0.000"	LRFD-Ult (auto)/3	444.8	**2.3**	0.0	0.0	0.0	0.0
W20	0.000"	LRFD-Ult (auto)/1	**147.5**	1.5	0.0	0.0	0.0	**0.0**
W20	1' 11.076"	LRFD-Ult (auto)/3	448.5	-0.2	0.0	0.0	0.0	**2.1**
W21	3' 7.267"	LRFD-Ult (auto)/2	**622.0**	-2.0	**0.0**	**0.0**	**0.0**	0.0
W21	3' 7.267"	LRFD-Ult (auto)/3	451.8	**-2.3**	0.0	0.0	0.0	0.0
W21	0.000"	LRFD-Ult (auto)/3	444.8	**2.3**	0.0	0.0	0.0	0.0
W21	0.000"	LRFD-Ult (auto)/1	**147.5**	1.5	0.0	0.0	0.0	**0.0**
W21	1' 11.076"	LRFD-Ult (auto)/3	448.5	-0.2	0.0	0.0	0.0	**2.1**
W22	3' 7.267"	LRFD-Ult (auto)/2	**2017.7**	-2.0	**0.0**	**0.0**	**0.0**	0.0
W22	3' 7.267"	LRFD-Ult (auto)/3	1458.9	**-2.3**	0.0	0.0	0.0	0.0
W22	0.000"	LRFD-Ult (auto)/3	1451.9	**2.3**	0.0	0.0	0.0	0.0
W22	0.000"	LRFD-Ult (auto)/1	**481.0**	1.5	0.0	0.0	0.0	**0.0**
W22	1' 11.076"	LRFD-Ult (auto)/3	1455.6	-0.2	0.0	0.0	0.0	**2.1**

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
W23	3' 7.267"	LRFD-Ult (auto)/2	**3311.6**	-2.0	**0.0**	**0.0**	**0.0**	0.0
W23	3' 7.267"	LRFD-Ult (auto)/3	2393.5	**-2.3**	0.0	0.0	0.0	0.0
W23	0.000"	LRFD-Ult (auto)/3	2386.5	**2.3**	0.0	0.0	0.0	0.0
W23	0.000"	LRFD-Ult (auto)/1	**791.2**	1.5	0.0	0.0	0.0	**0.0**
W23	1' 11.076"	LRFD-Ult (auto)/3	2390.2	-0.2	0.0	0.0	0.0	**2.1**
W24	3' 7.267"	LRFD-Ult (auto)/2	**4840.6**	-2.0	**0.0**	**0.0**	**0.0**	0.0
W24	3' 7.267"	LRFD-Ult (auto)/3	3497.8	**-2.3**	0.0	0.0	0.0	0.0
W24	0.000"	LRFD-Ult (auto)/3	3490.8	**2.3**	0.0	0.0	0.0	0.0
W24	0.000"	LRFD-Ult (auto)/1	**1157.7**	1.5	0.0	0.0	0.0	**0.0**
W24	1' 11.076"	LRFD-Ult (auto)/3	3494.6	-0.2	0.0	0.0	0.0	**2.1**
W25	3' 2.810"	LRFD-Ult (auto)/2	**4444.8**	-1.2	**0.0**	**0.0**	**0.0**	0.0
W25	3' 2.810"	LRFD-Ult (auto)/3	3212.6	**-1.4**	0.0	0.0	0.0	0.0
W25	0.000"	LRFD-Ult (auto)/3	3205.6	**1.4**	0.0	0.0	0.0	0.0
W25	0.000"	LRFD-Ult (auto)/1	**1063.6**	0.9	0.0	0.0	0.0	**0.0**
W25	1' 8.898"	LRFD-Ult (auto)/3	3209.4	-0.1	0.0	0.0	0.0	**1.1**

Name	Combination key
LRFD-Ult (auto)/1	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + W(up)
LRFD-Ult (auto)/2	1.20*DL1 + 1.60*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*W(down)
LRFD-Ult (auto)/3	1.40*DL1 + 1.40*DL2 + 1.40*DL3 + 1.40*DL4

DISPLACEMENT

Load case DL + Lr, inch:



The maximum deflection for a web header is 0.103”.

According to TABLE 1604.3 the code IBC 2018, maximum member deflection should not exceed - L/240.
L=18’+5”=221”, 221”/240=0.920”. 0.103"< 0.920”. <u>Deflection is OK!</u>

STEEL MEMBER W16 CHECK

AISI S100-16 LRFD Check

Member W16	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 0.00 ft

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

Internal forces		
Pu	-4127.14	lbf
Vux	0.00	lbf
Vuy	0.00	lbf
Mut	-0.00	lbfft
Mux	-0.00	lbfft
Muy	-13.48	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	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-
2	1.446	27.4 -50.0	1.83	54.833	2004.9	0.117	1.000	- 1.446	0.300 0.723	-	- -	-
3	5.446	27.4 27.4	1.00	4.000	10.3	1.629	0.531	2.891 -	- -	-	- -	-
4	1.446	27.4 -50.0	1.83	54.833	2004.9	0.117	1.000	- 1.446	0.300 0.723	-	- -	-
5	0.473	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.125	inch^3
Mnyo	520.1	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	955.9	ksi
Kt	1.00	
Lt	3' 0.000"	ft
Sigma,t	72.6	ksi
Cb	1.00	
Sfy	0.355	inch^3
Fcre	862.3	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	50.0 50.0	1.00	0.430	146.9	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.0 50.0	1.00	4.000	146.3	0.585	1.000	1.446 -	- -	-	- -	-
3	5.446	50.0 50.0	1.00	4.000	10.3	2.202	0.409	2.226 -	- -	-	- -	-
4	1.446	50.0 50.0	1.00	4.000	146.3	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.0 50.0	1.00	0.430	146.9	0.583	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.0	ksi
Ae	0.327	$inch^2$
Pno	16372.8	lbf
Resistance factor	0.85	
Unity check	0.30	-

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	3	3	ft
Effective Length factor K	1.00	1.00	
Effective Length	3	3	ft
Slenderness	17.31	67.58	
Flexural Buckling stress Fcre	955.9	62.7	ksi

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

Table of values		
Sigma,ex	955.9	ksi
Sigma,ey	62.7	ksi
Kt	1.00	
Lt	3	ft
Sigma,t	72.6	ksi
Sigma,TF	62.7	ksi
Torsional (-Flexural) buckling stress Fcre	62.7	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	35.8 35.8	1.00	0.430	146.9	0.494	1.000	0.473 -	- -	-	- -	-
2	1.446	35.8 35.8	1.00	4.000	146.3	0.495	1.000	1.446 -	- -	-	- -	-
3	5.446	35.8 35.8	1.00	4.000	10.3	1.864	0.473	2.577 -	- -	-	- -	-
4	1.446	35.8 35.8	1.00	4.000	146.3	0.495	1.000	1.446 -	- -	-	- -	-
5	0.473	35.8 35.8	1.00	0.430	146.9	0.494	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	62.7	ksi
lambda, c	0.89	
Fn	35.8	ksi
Ae	0.346	$inch^2$
Pn	12404.9	lbf
Resistance factor	0.85	
Unity check	0.39	-

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

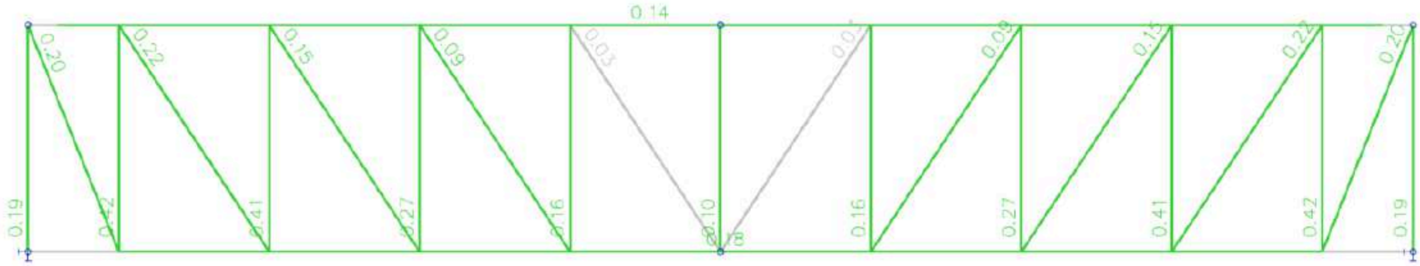
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.473	8.4 8.4	1.00	0.430	146.9	0.239	1.000	0.473 -	- -	-	- -	-
2	1.446	8.4 8.4	1.00	4.000	146.3	0.240	1.000	1.446 -	- -	-	- -	-
3	5.446	8.4 8.4	1.00	4.000	10.3	0.903	0.838	4.562 -	- -	-	- -	-
4	1.446	8.4 8.4	1.00	4.000	146.3	0.240	1.000	1.446 -	- -	-	- -	-
5	0.473	8.4 8.4	1.00	0.430	146.9	0.239	1.000	0.473 -	- -	-	- -	-

Table of values		
Centerline shift ex	0.039	inch
Centerline shift ey	0.000	inch
Additional moment Mx	0.0	lbfft
Additional moment My	-13.5	lbfft
Mny	520.1	lbfft
PEy	30787.0	lbf
Alfa y	0.87	
Cmy	0.85	
Pn	12404.9	lbf
Pno	16372.8	lbf
Resistance factor compression	0.85	
Resistance factor bending y	0.90	

Unity check = 0.39+0.00+0.03 = 0.42 - (H1.2-1)
Unity check = 0.30+0.00+0.03 = 0.33 -

The member satisfies the check !

Unity check



Check of steel

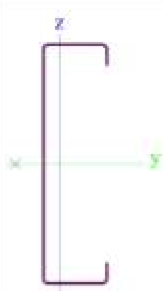
Linear calculation, Extreme : Member
Selection : All
Combinations : LRFD-Ult (auto)

Case	Member	css	mat	dx [ft]	un.check [-]
LRFD-Ult (auto)/1	W2	2X550S150-54 -	A913 grade 50	0.000"	0.19
LRFD-Ult (auto)/1	W6	2X550S150-54 -	A913 grade 50	0.000"	0.19
LRFD-Ult (auto)/1	TC2	HDR - 5x550S150-54	A913 grade 50	8' 2.500"	0.14
LRFD-Ult (auto)/1	BC2	2X550S150-54 -	A913 grade 50	9' 2.500"	0.18
LRFD-Ult (auto)/1	W7	550S150-54 -	A913 grade 50	0.000"	0.10
LRFD-Ult (auto)/1	W8	550S150-54 -	A913 grade 50	0.000"	0.16
LRFD-Ult (auto)/1	W9	550S150-54 -	A913 grade 50	0.000"	0.27
LRFD-Ult (auto)/1	W10	550S150-54 -	A913 grade 50	0.000"	0.41
LRFD-Ult (auto)/1	W11	550S150-54 -	A913 grade 50	0.000"	0.42
LRFD-Ult (auto)/1	W12	550S150-54 -	A913 grade 50	0.000"	0.41
LRFD-Ult (auto)/1	W13	550S150-54 -	A913 grade 50	0.000"	0.27
LRFD-Ult (auto)/1	W14	550S150-54 -	A913 grade 50	0.000"	0.16
LRFD-Ult (auto)/1	W15	550S150-54 -	A913 grade 50	3' 2.810"	0.20
LRFD-Ult (auto)/1	W16	550S150-54 -	A913 grade 50	0.000"	0.42
LRFD-Ult (auto)/1	W17	550S150-54 -	A913 grade 50	3' 7.267"	0.22
LRFD-Ult (auto)/1	W18	550S150-54 -	A913 grade 50	3' 7.267"	0.15
LRFD-Ult (auto)/1	W19	550S150-54 -	A913 grade 50	3' 7.267"	0.09
LRFD-Ult (auto)/1	W20	550S150-54 -	A913 grade 50	3' 7.267"	0.03
LRFD-Ult (auto)/1	W21	550S150-54 -	A913 grade 50	3' 7.267"	0.03
LRFD-Ult (auto)/1	W22	550S150-54 -	A913 grade 50	3' 7.267"	0.09
LRFD-Ult (auto)/1	W23	550S150-54 -	A913 grade 50	3' 7.267"	0.15
LRFD-Ult (auto)/1	W24	550S150-54 -	A913 grade 50	3' 7.267"	0.22
LRFD-Ult (auto)/1	W25	550S150-54 -	A913 grade 50	3' 2.810"	0.20

2.3.6 WALL POST DESIGN TYPICAL

Stud height - 13 ft. Stud spacing - 2 ft.

Cross-sections properties 550S150-54

550S150-54		
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]	-0.091	4.800
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.125	0.139
i_y [inch], i_z [inch]	2.080	0.533
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.773	0.126
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.925	0.181
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.62e+01	4.62e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.07e+00	9.07e+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	5.901
Picture		

APPLIED LOADS

LOAD ON STUD C1					
Roof slope 5 : 12, α = 22.6, Stud spacing - 24"					
Load	**Distr. Load, psf**	**Loading width, (ft)**	**Sin α = 22.6**	**Liner Load, plf**	**Point Load, lb**
Floor Dead Load	15	7.79		116.88	233.75
Roof Dead Load	20.00	10.29			411.67
Floor Live Load	40	7.79			623.33
Roof Live Load	20.00	10.29			411.67
Wall Wind	45	2		90	
Roof Wind (down)	17.30	10.29	0.3848		356.09
Roof Wind (up)	-53.50	10.29	0.3848		-1101.21

Wind loads are applied perpendicular to the roof beam. Liner load with “-” - uplift.

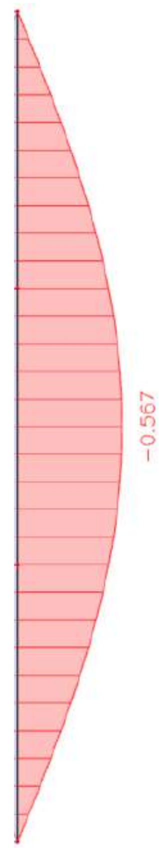
MAXIMUM FORCES

Name	dx [ft]	Case	N [lbf]	V_y [lbf]	V_z [lbf]	M_x [lbfft]	M_y [lbfft]	M_z [lbfft]
C1	13' 0.000"	LRFD-Ult (auto)/1	**520.6**	0.0	**-585.0**	0.0	0.0	0.0
C1	0.000"	LRFD-Ult (auto)/1	501.0	**0.0**	**585.0**	**0.0**	0.0	**0.0**
C1	6' 6.001"-	LRFD-Ult (auto)/2	-1144.2	0.0	0.0	0.0	**-1901.2**	0.0
C1	6' 6.001"-	LRFD-Ult (auto)/1	510.8	0.0	0.0	0.0	**1901.2**	0.0
C1	0.000"	LRFD-Ult (auto)/3	**-2005.7**	0.0	0.0	0.0	0.0	**0.0**

Name	Combination key
LRFD-Ult (auto)/1	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + W(up)
LRFD-Ult (auto)/2	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + W(down)
LRFD-Ult (auto)/3	1.20*DL1 + 1.60*L + 0.50*Lr + 1.20*DL2 + 1.20*DL3 + 1.20*DL4

DISPLACEMENT

Load case W, inch:



The maximum deflection for a web header is 0.567”.

According to TABLE 1604.3 the code IBC 2018, maximum member deflection should not exceed - L/240.
L=13’=221”,   156”/240=0.650”.   0.567"< 0.650”.

Deflection is OK!

STEEL MEMBER C1 CHECK

AISI S100-16 LRFD Check

Member C1	A913 grade 50	LRFD-Ult (auto)	0.97

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

The critical check is on position 6.50 ft

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

Internal forces		
Pu	-1662.24	lbf
Vux	0.00	lbf
Vuy	0.01	lbf
Mut	0.00	lbfft
Mux	-1901.25	lbfft
Muy	0.00	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	50.0 41.3	0.83	0.496	169.3	0.543	1.000	0.473 -	- -	-	- -	-
2	1.446	50.0 50.0	1.00	4.000	146.3	0.585	1.000	1.446 -	- -	-	- -	-
3	5.446	50.0 -50.0	1.00	24.000	61.9	0.899	0.840	- 4.575	1.144 2.288	-	- -	-
4	1.446	-50.0 -50.0	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-41.3 -50.0	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	0.773	$inch^3$
Mnxo	3220.2	lbfft
Resistance factor	0.90	
Unity check	0.66	-

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	0.473	39.7 32.8	0.83	0.496	169.3	0.484	1.000	0.473 -	- -	-	- -	-
2	1.446	39.7 39.7	1.00	4.000	146.3	0.521	1.000	1.446 -	- -	-	- -	-
3	5.446	39.7 -39.7	1.00	24.000	61.9	0.802	0.905	- 4.929	1.232 2.465	-	- -	-
4	1.446	-39.7 -39.7	-	-	-	-	-	- -	- -	-	- -	-
5	0.473	-32.8 -39.7	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Lltb	4' 4.001"	ft
Sigma,ey	30.0	ksi
Kt	1.00	
Lt	4' 4.001"	ft
Sigma,t	34.2	ksi
Cb	1.01	
Sfx	0.773	inch ^{3}
Fcre	48.8	ksi
Fc	39.7	ksi

Scx	0.773	inch ^{3}
Mnx	2559.9	lbfft
Resistance factor	0.90	
Unity check	0.83	-

...::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	50.0 50.0	1.00	0.430	146.9	0.583	1.000	0.473 -	- -	-	- -	-
2	1.446	50.0 50.0	1.00	4.000	146.3	0.585	1.000	1.446 -	- -	-	- -	-
3	5.446	50.0 50.0	1.00	4.000	10.3	2.202	0.409	2.226 -	- -	-	- -	-
4	1.446	50.0 50.0	1.00	4.000	146.3	0.585	1.000	1.446 -	- -	-	- -	-
5	0.473	50.0 50.0	1.00	0.430	146.9	0.583	1.000	0.473 -	- -	-	- -	-

Table of values		
Fn	50.0	ksi
Ae	0.327	inch ^{2}
Pno	16372.8	lbf
Resistance factor	0.85	
Unity check	0.12	-

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	13	4 3/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	13	4 3/8	ft
Slenderness	74.99	97.62	
Flexural Buckling stress Fcre	50.9	30.0	ksi

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

Table of values		
Sigma,ex	50.9	ksi
Sigma,ey	30.0	ksi
Kt	1.00	
Lt	4 3/8	ft
Sigma,t	34.2	ksi
Sigma,TF	28.1	ksi
Torsional (-Flexural) buckling stress Fcre	28.1	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	23.8 23.8	1.00	0.430	146.9	0.402	1.000	0.473 -	- -	-	- -	-
2	1.446	23.8 23.8	1.00	4.000	146.3	0.403	1.000	1.446 -	- -	-	- -	-
3	5.446	23.8 23.8	1.00	4.000	10.3	1.518	0.563	3.068 -	- -	-	- -	-
4	1.446	23.8 23.8	1.00	4.000	146.3	0.403	1.000	1.446 -	- -	-	- -	-
5	0.473	23.8 23.8	1.00	0.430	146.9	0.402	1.000	0.473 -	- -	-	- -	-

Table of values		
Fe	28.1	ksi
lambda, c	1.33	
Fn	23.8	ksi
Ae	0.373	inch ^{2}
Pn	8860.2	lbf
Resistance factor	0.85	
Unity check	0.22	-

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.473	3.4 3.4	1.00	0.430	146.9	0.152	1.000	0.473 -	- -	-	- -	-
2	1.446	3.4 3.4	1.00	4.000	146.3	0.152	1.000	1.446 -	- -	-	- -	-
3	5.446	3.4 3.4	1.00	4.000	10.3	0.573	1.000	5.446 -	- -	-	- -	-
4	1.446	3.4 3.4	1.00	4.000	146.3	0.152	1.000	1.446 -	- -	-	- -	-
5	0.473	3.4 3.4	1.00	0.430	146.9	0.152	1.000	0.473 -	- -	-	- -	-

Table of values		
Mnx	2559.9	lbfft
PEx	25002.7	lbf
Alfa x	0.93	
Cmx	0.85	
Pn	8860.2	lbf
Pno	16372.8	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.22+0.75+0.00 = 0.97 - (H1.2-1)
Unity check = 0.12+0.83+0.00 = 0.94 -

The member satisfies the check !

2.4 LATERAL ANALYSIS

2.4.1 BUILDING SEISMIC WEIGHT

Second floor					
Type of constr.	**h or w, ft**	**L, ft**	**Area, ft2**	**Weight, psf**	**Weight, kip**
Roof			3101.0	20	62.02
Exterior LGS wall	5	181.042	905.2	45	40.73
Second floor total seismic weight					**102.75**

First floor					
Type of constr.	**h or w, ft**	**L, ft**	**Area, ft2**	**Weight, psf**	**Weight, kip**
Roof			1398	20	27.96
Floor			1445	15	21.68
Exterior LGS wall	5	181.042	905.208	45	40.73
	6.5	187.146	1216.448	45	54.74
First floor total seismic weight					**145.11**

Total seismic weight: 247.86

2.4.2 BASE SHEAR SEISMIC FORCES (ASCE 7-16)

SEISMIC FORCES (ASCE 7-16)

Tedds calculation version 3.1.00

Site parameters

Site class D, Soil properties not known

Mapped acceleration parameters (Section 11.4.2)

at short period S_S = **0.057**

at 1 sec period S_1 = **0.026**

Alternate design spectral acceleration parameters (Chap 21)

Design spectral response acceleration at period T (Sect 21.3) S_a = 0.75

at short period (Sect 21.4) S_{DSalt} = **0.051**

at 1 sec period (Sect 21.4) S_{D1alt} = **0.038**

Spectral response acceleration parameters

at short period (Sect 21.4) $S_{MS} = 1.5 \times S_{DSalt} = \mathbf{0.077}$

at 1 sec period (Sect 21.4) $S_{M1} = 1.5 \times S_{D1alt} = \mathbf{0.057}$

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 design category A

Seismic base shear (Sect 11.7)

Effective seismic weight of the structure W = **247.9** kips

Seismic base shear (Eq. 9.5.3-1) V = 0.01 × W = **2.48** kips

Vertical distribution of seismic forces (Sect 11.7)

Lateral force induce at level i $F_x = 0.01 \times w_x$

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	13.00	145.1	1.5;
2	23.00	102.8	1.0;

2.4.3 WIND BASE SHEAR AND WIND LOAD ON ROOF DIAPHRAGM

Total wind shear:

Wx - **66.37 kip**

Wy - **58.55 kip**

Transverse direction wind base shear **66.37 kip**

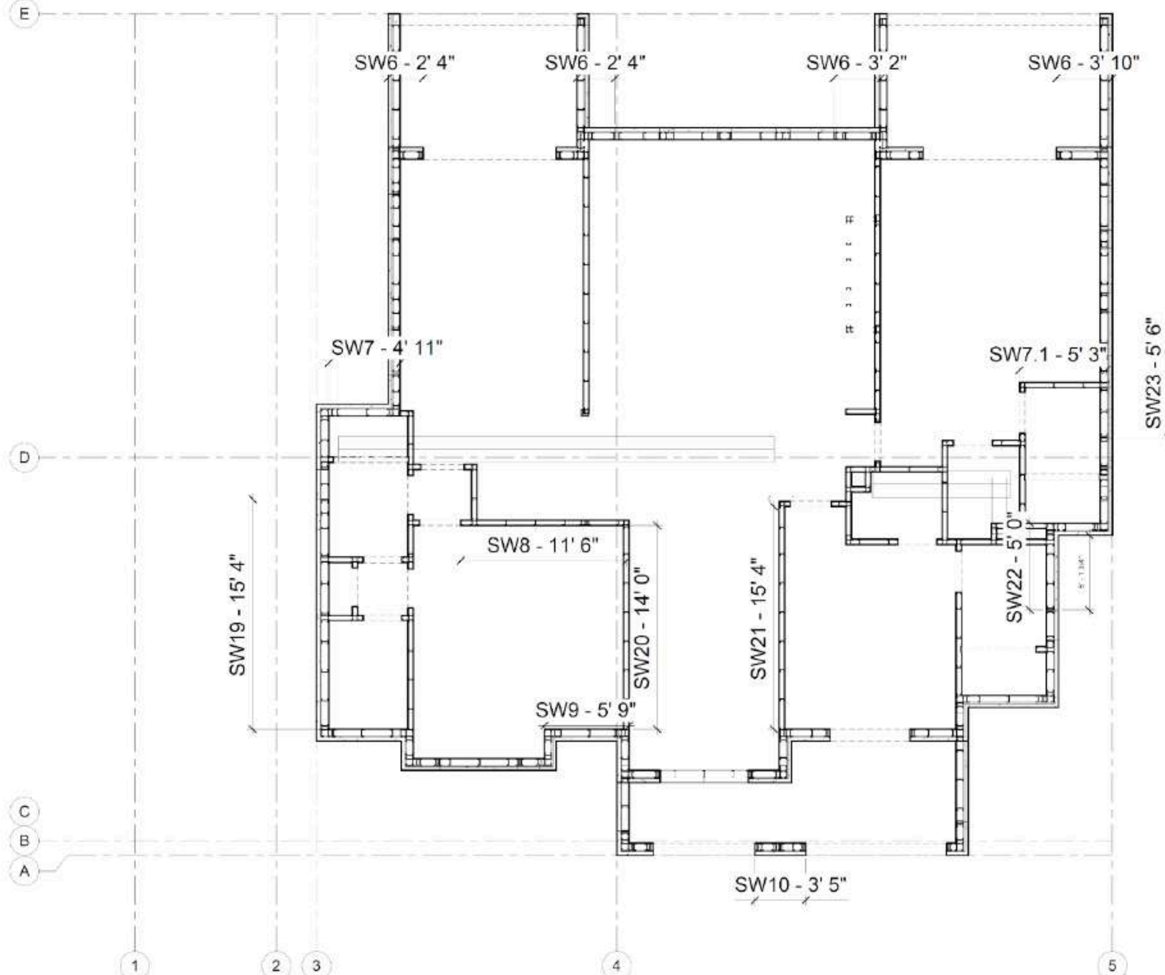
Longitudinal direction base shear **58.55 kip**

Seismic base shear **2.46 kip**

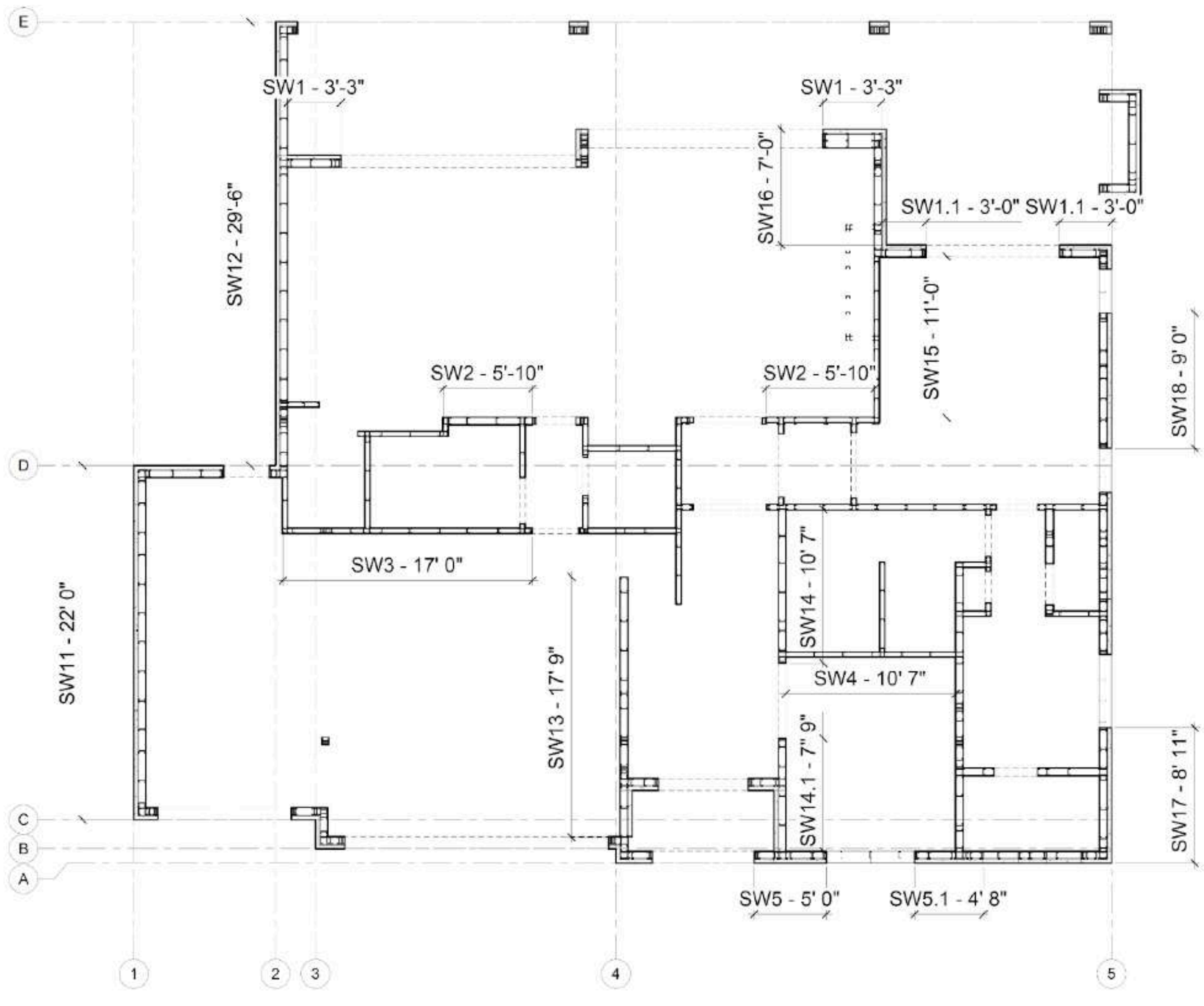
According to the results of the calculation, wind actions affect the calculation in the transverse and longitudinal directions.

2.4.4 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)	Shear wall capacity (lbs/ft)	Shear wall capacity (lbs)	Shear wall 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	11	12	13	14	15	16	17	18
2-ND FLOOR																		
Transverse direction																		
SW19	15.3	4310.00	0	1000	15250	OK!	-	SprayRock SW	-	1	10	15.3	-	-	-	-	2826.23	S/HDU6
SW20	14.0	7730.00	0	0	0	-	-	X Bracing	7,5 x 54mil	1	10	14.0	-	9565.9	20250	OK!	5521.43	S/HDU6
SW21	15.3	7730.00	0	0	0	-	-	X Bracing	7,5 x 54mil	1	10	15.3	-	15095.8	20250	OK!	5042.40	S/HDU6
SW22	5.0	4000.00	0	1000	5000	OK!	-	SprayRock SW	-	1	10	5.0	-	-	-	-	8000.00	S/HDU6
SW23	5.5	560	0	1000	5500	OK!	-	SprayRock SW	-	1	10	5.5	-				1018.18	S/HDU6
Longitudinal direction																		
SW6	7.8	5950.00	0	1000	7750	OK!	-	SprayRock SW	-	4	10	7.8	-	-	-	-	7677.42	S/HDU6
SW7	4.5	3492.00	0	1000	4500	OK!	-	SprayRock SW	-	2	10	4.5	-	-	-	-	7760.00	S/HDU6
SW7.1	5.3	2858.00	0	0	0	-	-	X Bracing	-	1	10	5.3	-	6148.4	20250	OK!	5443.81	S/HDU6
SW8	11.5	5060.00	0	0	0	-	-	X Bracing	-	1	10	11.5	-	-	-	-	4400.00	S/HDU6
SW9	5.8	4400.00	0	1000	5750	OK!	-	SprayRock SW	-	1	10	5.8	-	-	-	-	7652.17	S/HDU6
SW10	3.5	1780.00	0	1000	3500	OK!	-	SprayRock SW	-	1	10	3.5	-	-	-	-	5085.71	S/HDU6
1-ST FLOOR																		
Transverse direction																		
SW11	22.0	3040.00	0	1000	22000	OK!	-	SprayRock SW	-	1	13	22.0	-	-	-	-	1796.36	#4 x 24"1/2 w/ SET-3G, 5"emb.
SW12	29.5	13100.00	0	1000	29500	OK!	-	SprayRock SW	-	1	13	29.5	-	-	-	-	5772.88	#4 x 24"1/2 w/ SET-3G, 5"emb.
SW13	17.8	20000.00	0	0	0	-	-	X Bracing	10 x 54mil	1	13	17.8	-	24750.0	27000	OK!	14579.44	S/HDU11
SW14	7.8	6405.00	0	0	0	-	-	X Bracing	7,5 x 54mil	1	13	7.8	-	12508.2	20250	OK!	10743.87	S/HDU9
SW 14.1	10.5	6405.00	0	0	0	-	-	X Bracing	7,5 x 54mil	1	13	10.5		10193.6	20250	OK!	7930.00	S/HDU6
SW15	11.0	5516.50	0	0	0	-	-	X Bracing	7,5 x 54mil	1	13	11.0	-	8540.2	20250	OK!	6519.50	S/HDU6
SW16	7.0	4513.50	0	1000	7000	OK!		SprayRock SW	-	1	13	7.0		-			8382.21	#4 x 24"1/2 w/ SET-3G, 5"emb.
SW17	8.9	3195.00	0	1000	8920	OK!	-	SprayRock SW	-	1	13	8.9	-	-	-	-	4656.39	#4 x 24"1/2 w/ SET-3G, 5"emb.
SW18	9.0	3195.00	0	1000	9000	OK!		SprayRock SW	-	1	13	9.0		-		-	4615.00	#4 x 24"1/2 w/ SET-3G, 5"emb.
Longitudinal direction																		
SW1	6.5	9940.00	0	1000	6500	NOT OK!	3440.00	SprayRock SW + X Bracing	7,5 x 54mil	2	13	6.5	3440.00	22226.5	40500	OK!	6880.00	S/HDU6
SW1.1	6.0	5410.00	0	1000	6000	OK!	-	SprayRock SW	-	2	13	6.0	-	-	-	-	11721.67	S/HDU11
SW2	11.7	15330.00	0	0	6902.72	-	-	X Bracing	7,5 x 54mil	2	13	11.7	-	22959.5	40500	OK!	8545.88	S/HDU9
SW3	17.0	9930.00	0	0	10064	OK!	-	X Bracing	-	1	13	17.0	-	12500.7	20250	OK!	7593.53	S/HDU6
SW4	10.6	9370.00	0	0	0	-	-	X Bracing	7,5 x 54mil	1	13	10.6	-	14844.2	20250	OK!	11513.23	S/HDU9
SW5	5.0	4285.00	0	1000	5000	OK!	-	SprayRock SW	-	2	13	5.0	-	-	-	-	11141.00	S/HDU9
SW5.1	4.8	4285.00	0	1000	4750	OK!	-	SprayRock SW	-	2	13	4.8	-	-	-	-	11727.37	S/HDU9

2.4.4.1 ANCHOR BOLT DESIGN

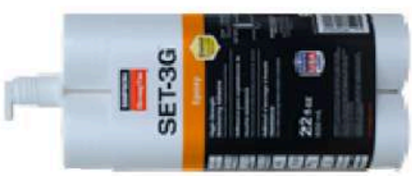
ANCHOR BOLT ANCHORING CALCULATIONS FOR HOLLDOWN S/HDU6

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

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 5.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 6.38
c_{ac} (inch): 8.11
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
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/ 5/8"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	**8000**	**9833**	**0.81**	**Pass (Governs)**
Concrete breakout	8000	11906	0.67	Pass

SET-3G w/ 5/8"Ø F1554 Gr. 36 with hef = 5.000 inch meets the selected design criteria.

ANCHOR BOLT ANCHORING CALCULATIONS FOR HOLLDOWN S/HDU9

1.Project information

Project description:
Location:
Fastening description:

Comment:

2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 55
Diameter (inch): 1.250
Effective Embedment depth, h_{ef} (inch): 9.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.75
c_{ac} (inch): 13.53
C_{min} (inch): 2.75
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 20.00
State: Uncracked
Compressive strength, f'_c (psi): 3500
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: No
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/ 1 1/4"Ø F1554 Gr. 55
Code Report: ICC-ES ESR-4057

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	11727	25988	0.45	Pass
Concrete breakout	**11727**	**13926**	**0.84**	**Pass (Governs)**

SET-3G w/ 7/8"Ø F1554 Gr. 55 with hef = 9.000 inch meets the selected design criteria.

ANCHOR BOLT ANCHORING CALCULATIONS FOR HOLLDOWN S/HDU11

1.Project information

Project description:
Location:
Fastening description:

Comment:

2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 55
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 8.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 10.00
c_{ac} (inch): 12.08
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 20.00
State: Uncracked
Compressive strength, f'_c (psi): 3000
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: No
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. 55
Code Report: ICC-ES ESR-4057

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	14580	25988	0.56	Pass
Concrete breakout	**14580**	**17544**	**0.83**	**Pass (Governs)**

SET-3G w/ 7/8"Ø F1554 Gr. 55 with hef = 8.000 inch meets the selected design criteria.

SUMMARY ANCHORS DESIGN RESULT :

Model	Max Tension load (lb.)	Anchor
S/HDU6	8000,00	5/8"-DIA F1554 GR36 WITH SIMPSON HIGH-STRENGTH EPOXY ADHESIVE "SET-3G", MIN 5" EMBEDMENT
S/HDU9	11727,37	7/8"-DIA F1554 GR55 WITH SIMPSON HIGH-STRENGTH EPOXY ADHESIVE "SET-3G", 9" EMBEDMENT
S/HDU11	14579,44	7/8"-DIA F1554 GR55 WITH SIMPSON HIGH-STRENGTH EPOXY ADHESIVE "SET-3G", 8" EMBEDMENT

"SPRAY-ROCK" CONCRETE WALL ANCHORING

1.Project information

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

Project description:
Location:
Fastening description:

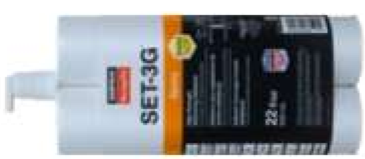
2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Bonded anchor
Material: A615 Grade 60 Rebar
Diameter (inch): 0.500
Effective Embedment depth, h_{ef} (inch): 5.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 6.25
c_{ac} (inch): 8.87
C_{min} (inch): 1.75
S_{min} (inch): 2.50

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 not present
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
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/ #4 A615 Gr. 60 Rebar
Code Report: ICC-ES ESR-4057

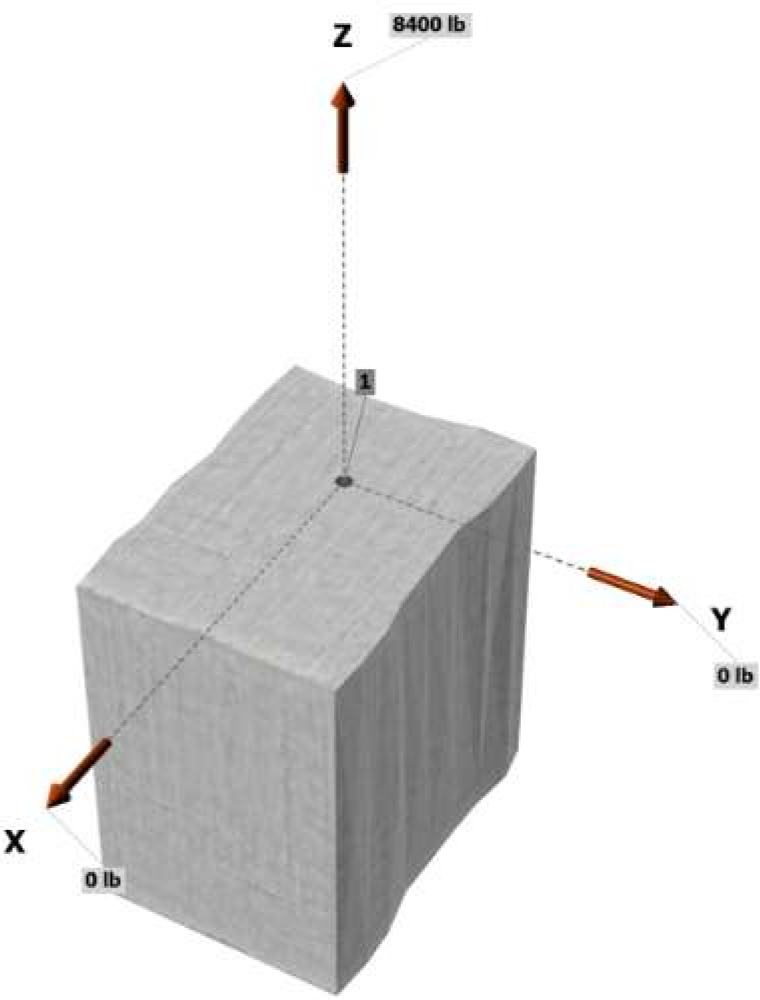


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: Yes
Anchors only resisting wind and/or seismic loads: No

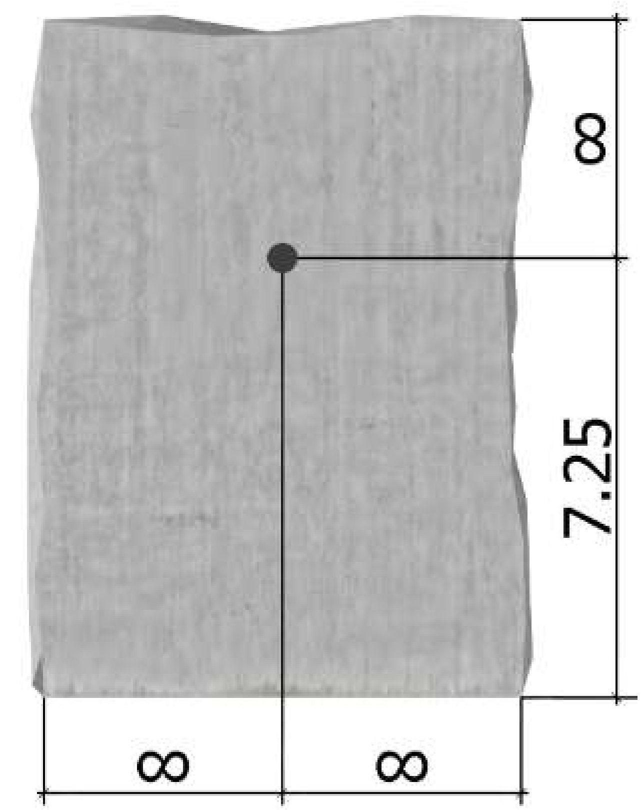
Strength level loads:

N_{ua} [lb]: 8400
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	8400.0	0.0	0.0	0.0
Sum	8400.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 8400
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
18000	0.75	13500

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c}\, h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	3500	5.000	15875

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N}\,N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
221.25	225.00	7.25	0.990	1.00	0.846	15875	0.65	8498

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,uncr} = \tau_{k,uncr} f_{short\text{-}term} K_{sat} (f'_c / 2{,}500)^n$

$\tau_{k,uncr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	f'_c (psi)	n	$\tau_{k,uncr}$ (psi)
2145	1.00	1.00	3500	0.36	2421

$N_{ba} = \lambda_a \tau_{uncr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{uncr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	2421	0.50	5.000	19016

$\phi N_a = \phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\,\Psi_{cp,Na}\,N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
195.00	195.00	6.98	7.25	1.000	0.818	19016	0.65	10108

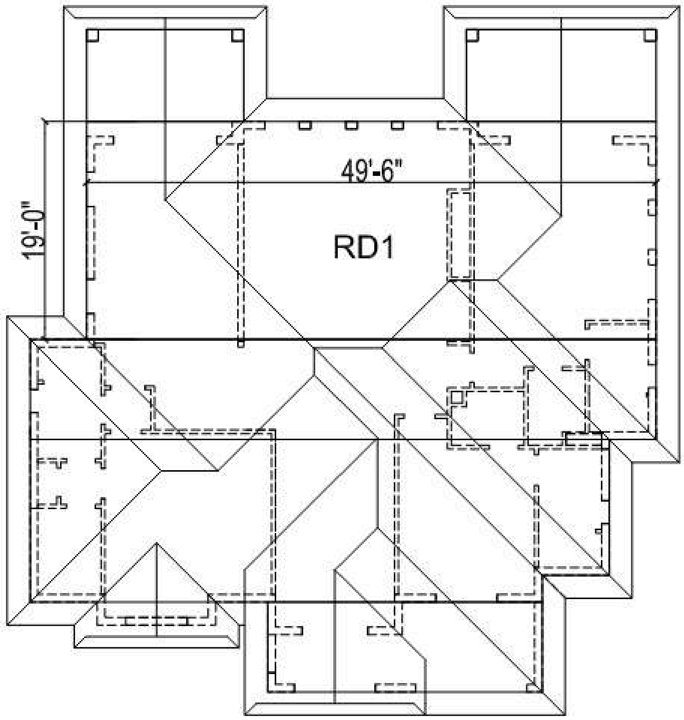
11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	8400	13500	0.62	Pass
Concrete breakout	**8400**	**8498**	**0.99**	**Pass (Governs)**
Adhesive	8400	10108	0.83	Pass

SET-3G w/ #4 A615 Gr. 60 Rebar with hef = 5.000 inch meets the selected design criteria.

2.4.5 DIAPHRAGM DESIGN
2.4.5.1 2-ND STORY ROOF DIAPHRAGM DESIGN



ROOF DIAPHRAGM AREA RD1 CHECK

Diaphragm Dimensions

Diaphragm Length	L =	**49.50**	ft
Diaphragm Width	B =	**19.00**	ft
Distributed linear load in transverse directio	Wt =	**660.00**	lb/ft

15/32" OSB Sheathing (per AISI240 Table B5.4.2-1 Unblocked Wood Structural Panel Diaphragms)

Fastener Spacing Edge/Field - 6/12

Nominal shear capacity for wind design	Vn =	**615**	lb/ft
The design factors for wind Design	LRFD ϕv =	**0.65**	**(AISI240 Section B5.4.3)**
Design shear capacity for wind design	ϕv Vnw =	**399.75**	lb/ft

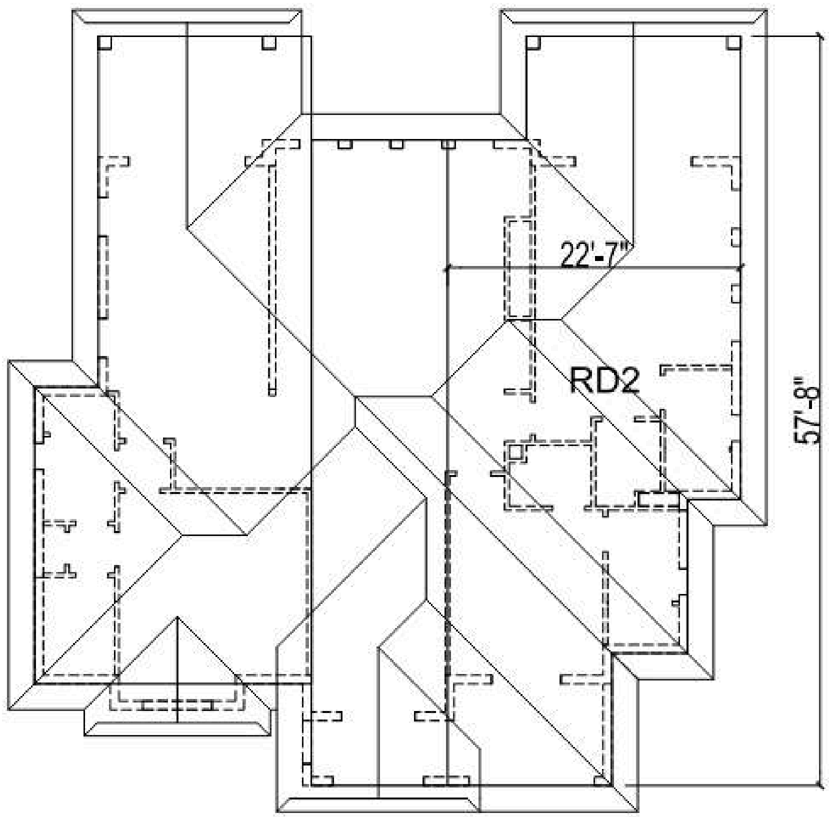
Design criteria **V < ϕvVn**

The maximum diaphragm wind shear in the roof diaphragm is:

Transverse direction $V_t = (W_t * B) / 2L =$ **126.67** lb/ft

Shear capacity for wind loading;

$V_t \leq \phi v\ Vnw$ **126.67** lb/ft < **399.75** lb/ft **PASS!**



ROOF DIAPHRAGM AREA RD2 CHECK

Diaphragm Dimensions

Diaphragm Length	L =	**22.58**	ft
Diaphragm Width	B =	**57.67**	ft
Distributed linear load in longitudinal directi	W_L =	**660.00**	lb/ft

15/32" OSB Sheathing (per AISI240 Table B5.4.2-1 Unblocked Wood Structural Panel Diaphragms)

Fastener Spacing Edge/Field - 6/12

Nominal shear capacity for wind design	Vn =	**615**	lb/ft
The design factors for wind Design	LRFD ϕv =	**0.65**	**(AISI240 Section B5.4.3)**
Design shear capacity for wind design	ϕv Vnw =	**399.75**	lb/ft

Design criteria **V < ϕvVn**

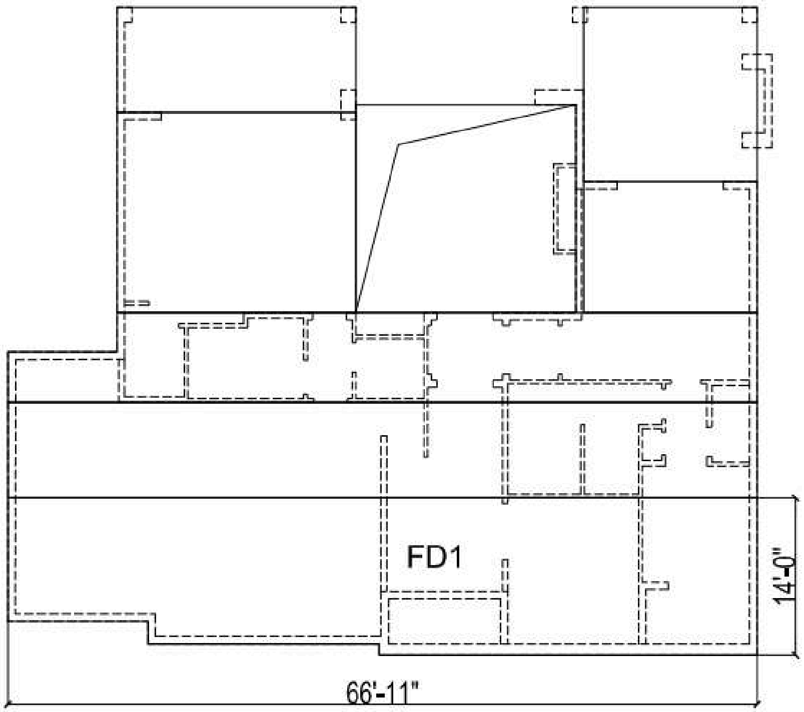
The maximum diaphragm wind shear in the roof diaphragm is:

Longitudinal direction $V_L = (W_L * L) / 2B =$ **129.23** lb/ft

Shear capacity for wind loading:

$V_L \le \phi v\ Vnw$ **129.23** lb/ft < **399.75** lb/ft **PASS!**

2.4.5.1 FLOOR DIAPHRAGM DESIGN



FLOOR DIAPHRAGM AREA FD1 CHECK

Diaphragm Dimensions

Diaphragm Length	L =	**66.92**	ft
Diaphragm Width	B =	**14.00**	ft
Distributed linear load in transverse directio	Wt =	**562.50**	lb/ft
	Load from SW9 =	**4400**	lb

In AISI240 there is no nominal shear value for 3/4" OSB, we take the calculated shear value for 15/32" OSB as the calculated shear value.

Fastener Spacing Edge/Field - 6/12			
Nominal shear capacity for wind design	Vn =	**615**	lb/ft
The design factors for wind Design	LRFD ϕv =	**0.65**	**(AISI240 Section B5.4.3)**
Design shear capacity for wind design	ϕv Vnw =	**399.75**	lb/ft

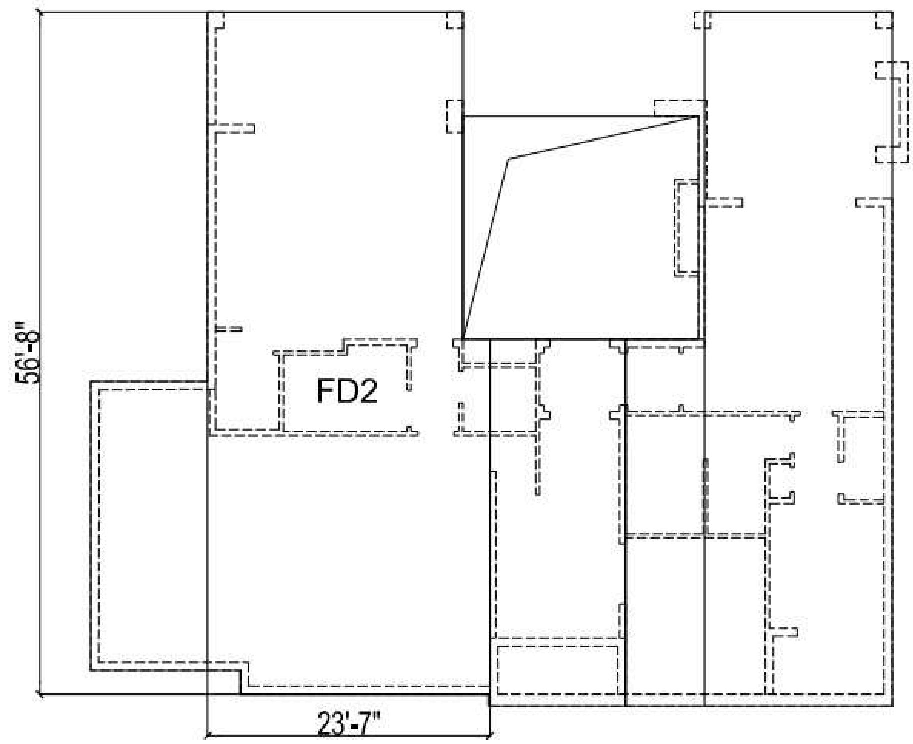
Design criteria **V < ϕvVn**

The maximum diaphragm wind shear in the roof diaphragm is:

Transverse direction $V_t = (W_t * B) / 2L + SW9 / 2L =$ **91.72** lb/ft

Shear capacity for wind loading;

$V_t \leq \phi v$ Vnw **91.72** lb/ft < **399.75** lb/ft **PASS!**



FLOOR DIAPHRAGM AREA FD2 CHECK

Diaphragm Dimensions

Diaphragm Length	L =	**23.58**	ft
Diaphragm Width	B =	**56.67**	ft
Distributed linear load in longitudinal directi	W_L =	**562.50**	lb/ft
	Load from SW19 =	**4310**	lb

In AISI240 there is no nominal shear value for 3/4" OSB, we take the calculated shear value for 15/32" OSB as the calculated shear value.

Fastener Spacing Edge/Field - 6/12

Nominal shear capacity for wind design	Vn =	**615**	lb/ft
The design factors for wind Design	LRFD ϕv =	**0.65**	**(AISI240 Section B5.4.3)**
Design shear capacity for wind design	ϕv Vnw =	**399.75**	lb/ft

Design criteria **V < ϕvVn**

The maximum diaphragm wind shear in the roof diaphragm is:

Longitudinal direction $V_L = (W_L * L) / 2B + SW19 / B$ = **193.11** lb/ft

Shear capacity for wind loading;

$V_L \leq \phi v$ Vnw **193.11** lb/ft < **399.75** lb/ft **PASS!**

2.5 CONNECTION DESIGN

2.5.1 22"X8" BOX BEAM TO 22"X8" BOX BEAM CONNECTION DESIGN

Project data

Project name	Beemsterboer
Project number	24-113
Author	DZ
Description	22"x8" Box beam to 22"x8" box beam connection
Date	7/4/2024
Design code	AISC 360-16

Material

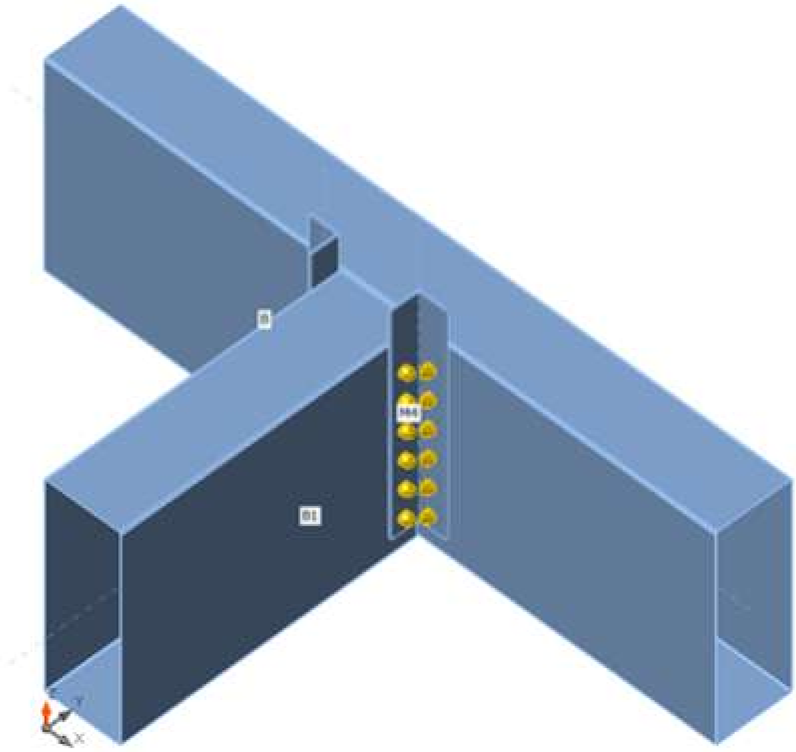
Steel	A913 Gr.50, A36

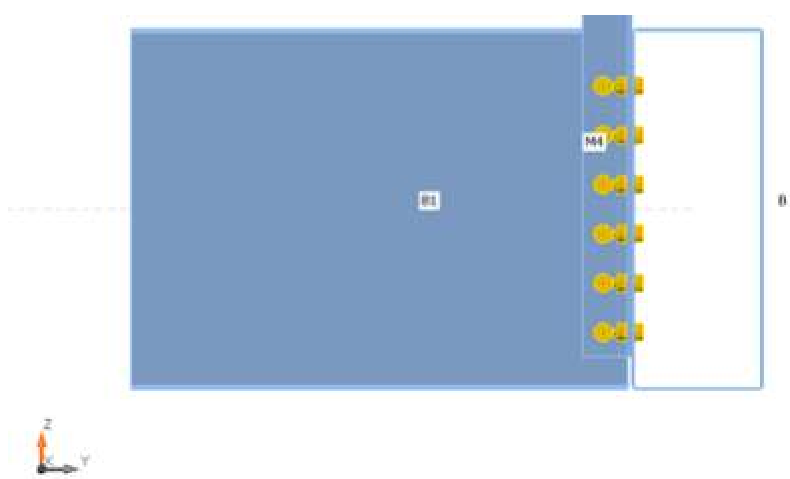
Design

Name	Beemsterboer
Description	22"x8" Box beam to 22"x8" box beam connection
Analysis	Stress, strain/ simplified loading
Design code	AISC - LRFD 2016

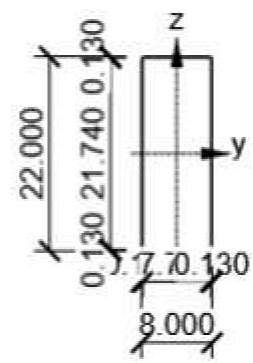
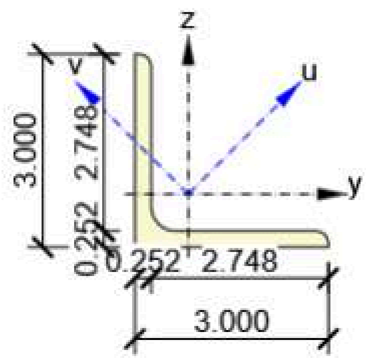
Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]	Forces in	X [in]
B	6 - 22"x8" (RHS558x203)	0.0	0.0	0.0	0.000	0.000	0.000	Node	0.000
B1	6 - 22"x8" (RHS558x203)	-90.0	0.0	0.0	0.000	0.000	0.000	Bolts	5.750
M3	9 - L(Imp)3X3X1/4	90.0	-90.0	0.0	-9.000	4.820	4.820	Bolts	9.000
M4	9 - L(Imp)3X3X1/4	180.0	-90.0	0.0	-9.000	4.820	4.820	Bolts	9.000


B
M4
B1
Y
X


M4
B1
B
Z
Y

Cross-sections

Name	Material	Drawing
6 - 22"x8"(RHS558x203)	A913 Gr.50	
9 - L(Imp)3X3X1/4	A36	

Bolts

Name	Bolt assembly	Diameter [in]	fu [ksi]	Gross area [in^2]
1/2 A325	1/2 A325	0.500	120.0	0.196

Load effects (equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.in]	My [kip.in]	Mz [kip.in]
LE1	B1	0.000	0.000	-13.900	0.00	0.00	0.00
	M3	0.000	0.000	0.000	0.00	0.00	0.00
	M4	0.000	0.000	0.000	0.00	0.00	0.00

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.3 < 5.0%	OK
Bolts	38.2 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

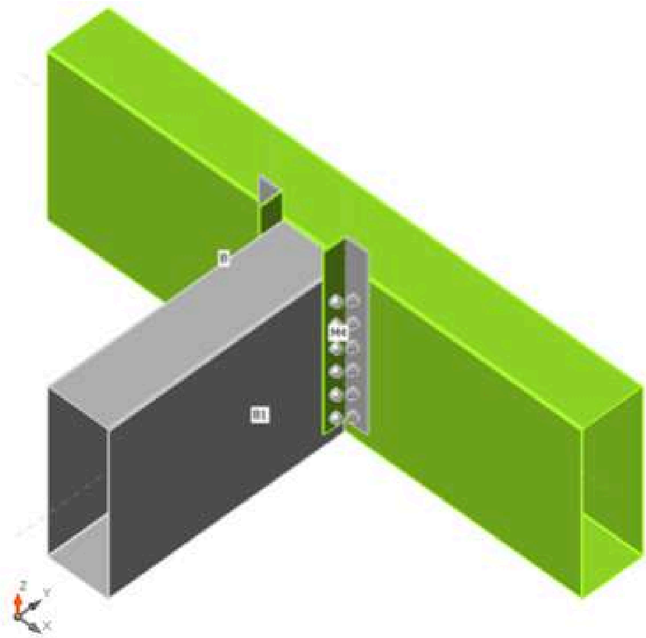
Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
B	A913 Gr.50	50.0	1/8	LE1	45.1	0.3	0.0	OK
B1	A913 Gr.50	50.0	1/8	LE1	19.6	0.0	0.0	OK
M3-bfl 1	A36	36.0	1/4	LE1	23.1	0.0	19.9	OK
M3-w 1	A36	36.0	1/4	LE1	25.1	0.0	19.9	OK
M4-bfl 1	A36	36.0	1/4	LE1	25.1	0.0	19.9	OK
M4-w 1	A36	36.0	1/4	LE1	23.2	0.0	19.9	OK

Design data

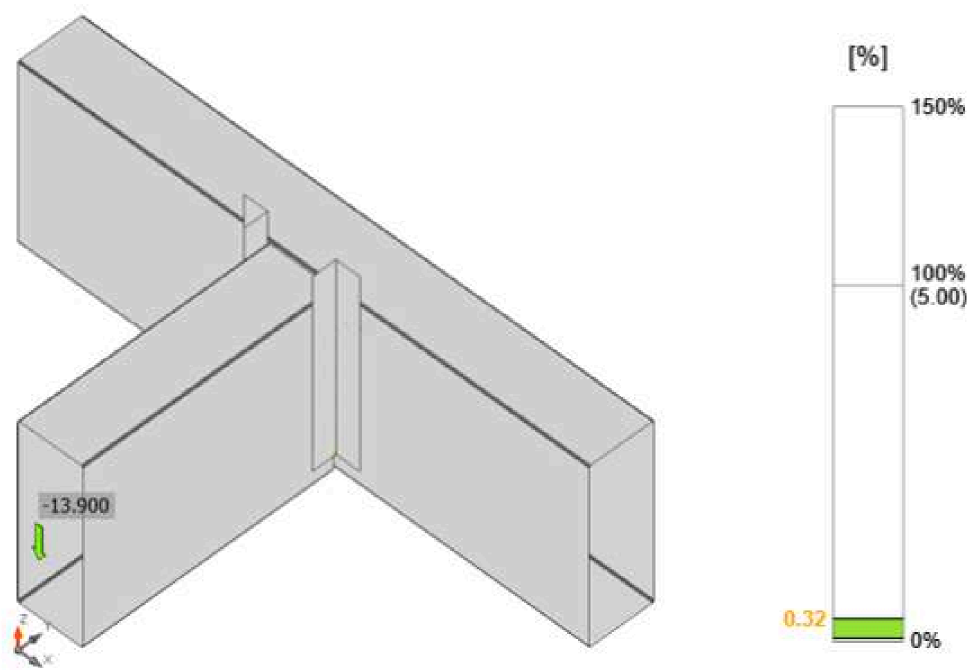
Material	f_y [ksi]	ε_{lim} [%]
A913 Gr.50	50.0	5.0
A36	36.0	5.0

Symbol explanation

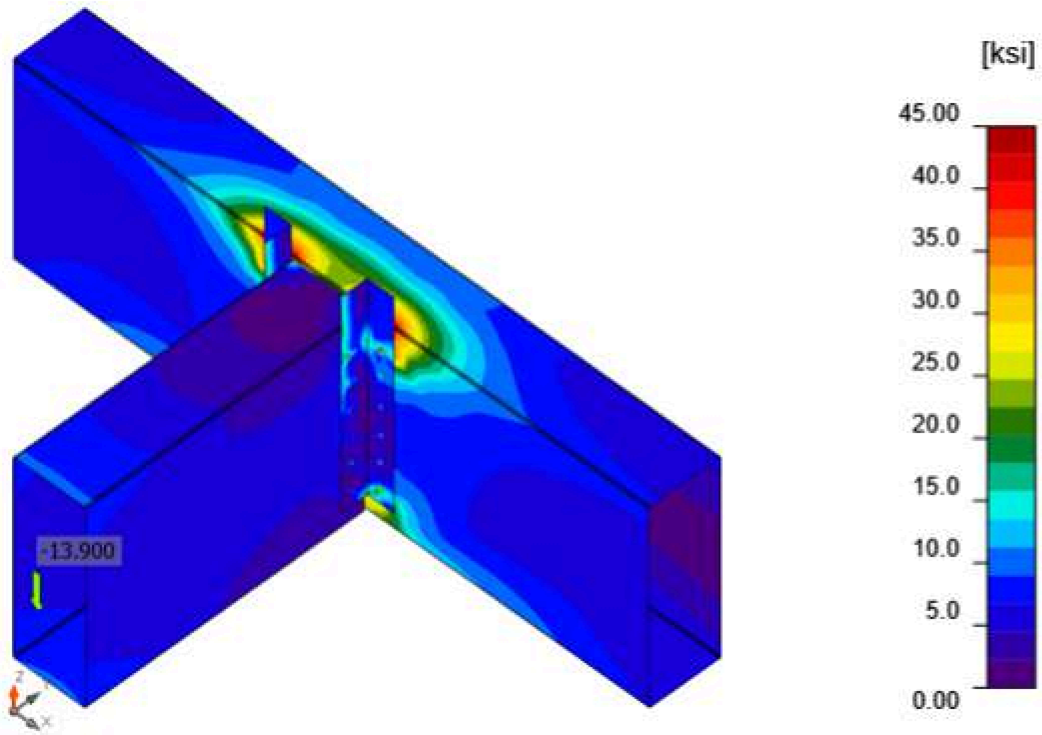
ε_{Pl}	Plastic strain
σc_{Ed}	Contact stress
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain



Overall check, LE1

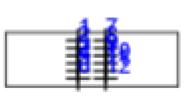


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	1/2 A325 - 1	LE1	5.055	1.407	7.605	38.2	18.5	-	OK
	B2	1/2 A325 - 1	LE1	0.218	1.199	7.605	1.6	15.8	-	OK
	B3	1/2 A325 - 1	LE1	0.280	1.115	7.605	2.1	14.7	-	OK
	B4	1/2 A325 - 1	LE1	0.227	1.077	7.605	1.7	14.2	-	OK
	B5	1/2 A325 - 1	LE1	0.280	1.077	7.605	2.1	14.2	-	OK
	B6	1/2 A325 - 1	LE1	1.628	1.146	7.605	12.3	15.1	-	OK
	B7	1/2 A325 - 1	LE1	5.008	1.407	7.605	37.8	18.5	-	OK
	B8	1/2 A325 - 1	LE1	0.212	1.199	7.605	1.6	15.8	-	OK
	B9	1/2 A325 - 1	LE1	0.275	1.115	7.605	2.1	14.7	-	OK
	B10	1/2 A325 - 1	LE1	0.225	1.077	7.605	1.7	14.2	-	OK
	B11	1/2 A325 - 1	LE1	0.278	1.077	7.605	2.1	14.2	-	OK
	B12	1/2 A325 - 1	LE1	1.595	1.145	7.605	12.0	15.1	-	OK

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B13	1/2 A325 - 1	LE1	0.047	1.217	7.605	0.4	16.0	-	OK
	B14	1/2 A325 - 1	LE1	0.000	1.152	7.605	0.0	15.2	-	OK
	B15	1/2 A325 - 1	LE1	0.004	1.137	7.605	0.0	15.0	-	OK
	B16	1/2 A325 - 1	LE1	0.030	1.150	7.605	0.2	15.1	-	OK
	B17	1/2 A325 - 1	LE1	0.081	1.222	7.605	0.6	16.1	-	OK
	B18	1/2 A325 - 1	LE1	0.367	1.457	7.605	2.8	19.2	-	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
1/2 A325 - 1	13.243	7.946

Symbol explanation

F_t	Tension force
V	Resultant of shear forces Vy, Vz in bolt
$\phi R_{n,bearing}$	Bolt bearing resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance AISC 360-16 J3.6
$\phi R_{n,shear}$	Bolt shear resistance AISC 360-16 – J3.8

Detailed result for B1

Tension resistance check (AISC 360-16: J3-1)

$$\phi R_n = \phi \cdot F_{nt} \cdot A_b = \quad 13.243 \text{ kip} \geq F_t = \quad 5.055 \text{ kip}$$

Where:

$F_{nt} = 89.9$ ksi – nominal tensile stress from AISC 360-16 Table J3.2

$A_b = 0.196$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Shear resistance check (AISC 360-16: J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 7.946 kip ≥ $V =$ 1.407 kip

Where:

$F_{nv} =$ 54.0 ksi – nominal shear stress from AISC 360-16 Table J3.2

$A_b =$ 0.196 in^2 – gross bolt cross-sectional area

$\phi =$ 0.75 – resistance factor

Bearing resistance check (AISC 360-16: J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u$ ≤ $2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 7.605 kip ≥ $V =$ 1.407 kip

Where:

$l_c =$ 2.437 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

$t =$ 0.130 in – thickness of the plate

$d =$ 0.500 in – diameter of a bolt

$F_u =$ 65.0 ksi – tensile strength of the connected material

$\phi =$ 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16: J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

2.5.2 14"X5.5" BOX BEAM TO 22"X8" BOX BEAM CONNECTION DESIGN

Project data

Project name	Beemsterboer
Project number	24-113
Author	DZ
Description	14"X5.5" BOX BEAM TO 22"X8" BOX BEAM CONNECTION DESIGN
Date	7/4/2024
Design code	AISC 360-16

Material

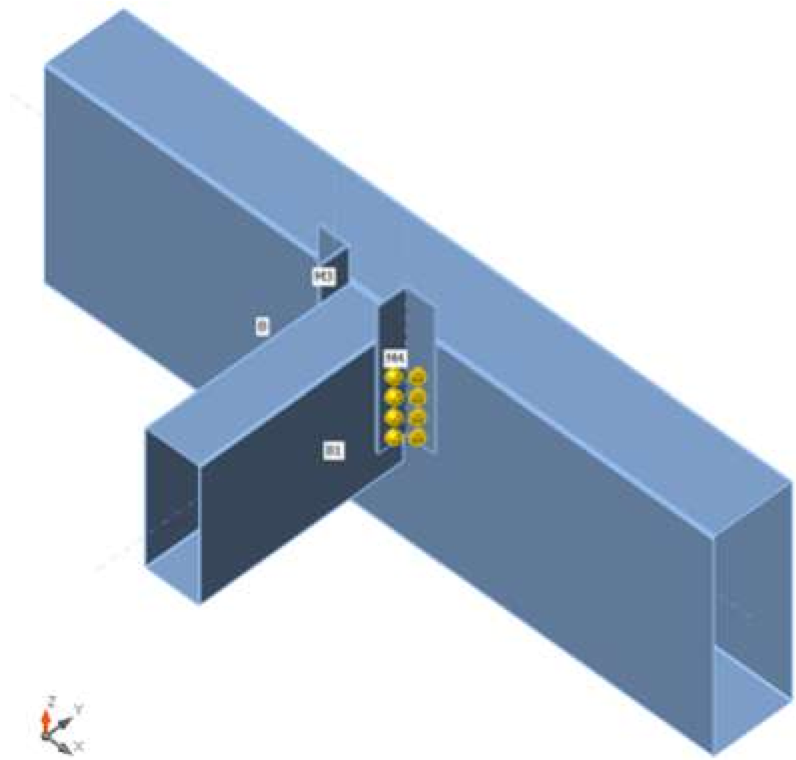
Steel	A913 Gr.50, A36

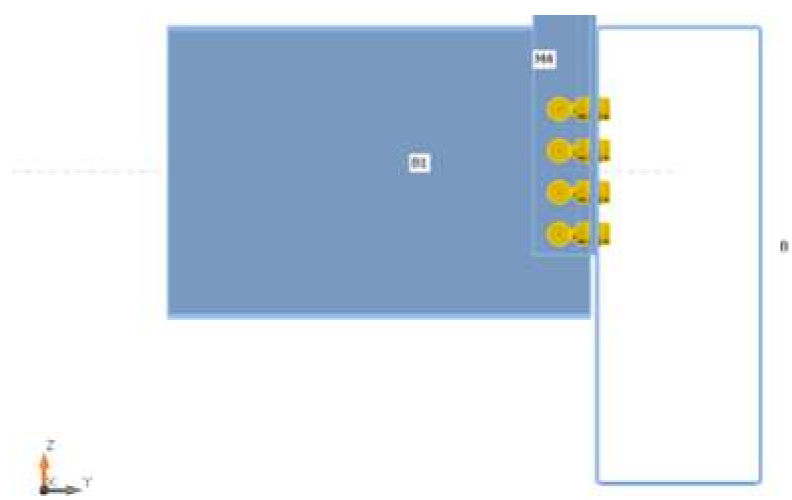
Design

Name	Beemsterboer
Description	14"X5.5" BOX BEAM TO 22"X8" BOX BEAM CONNECTION DESIGN
Analysis	Stress, strain/ simplified loading
Design code	AISC - LRFD 2016

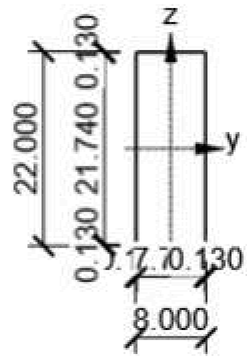
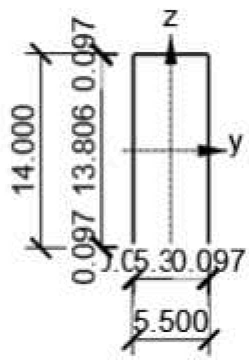
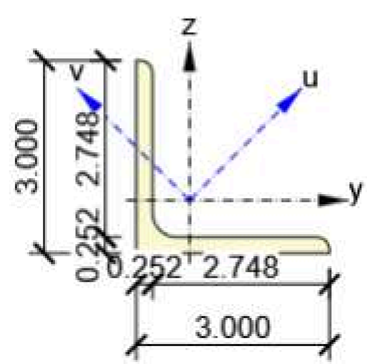
Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]	Forces in	X [in]
B	6 - 22"x8" (RHS558x203)	0.0	0.0	0.0	0.000	0.000	0.000	Node	0.000
B1	7 - 14"x5.5" (RHS355x139)	-90.0	0.0	0.0	0.000	0.000	4.000	Bolts	5.750
M3	9 - L(Imp)3X3X1/4	90.0	-90.0	0.0	0.000	3.575	4.820	Bolts	4.000
M4	9 - L(Imp)3X3X1/4	180.0	-90.0	0.0	0.000	4.820	3.575	Bolts	4.000


Z
Y
X


B
Z
Y

Cross-sections

Name	Material	Drawing
6 - 22"x8"(RHS558x203)	A913 Gr.50	
7 - 14"x5.5"(RHS355x139)	A913 Gr.50	
9 - L(Imp)3X3X1/4	A36	

Bolts

Name	Bolt assembly	Diameter [in]	fu [ksi]	Gross area [in^2]
1/2 A325	1/2 A325	0.500	120.0	0.196

Load effects (equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.in]	My [kip.in]	Mz [kip.in]
LE1	B1	0.000	0.000	-5.100	0.00	0.00	0.00
	M3	0.000	0.000	0.000	0.00	0.00	0.00
	M4	0.000	0.000	0.000	0.00	0.00	0.00

Check

Summary

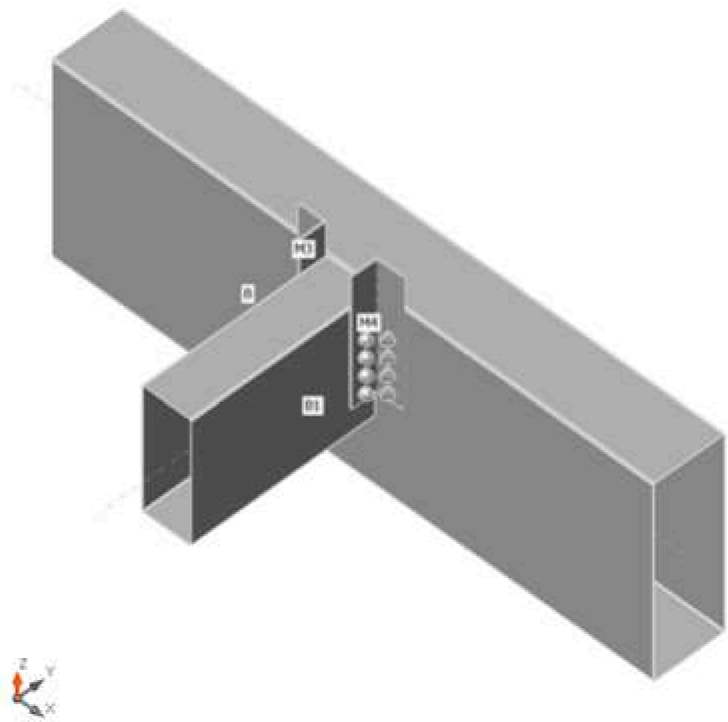
Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Bolts	13.3 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

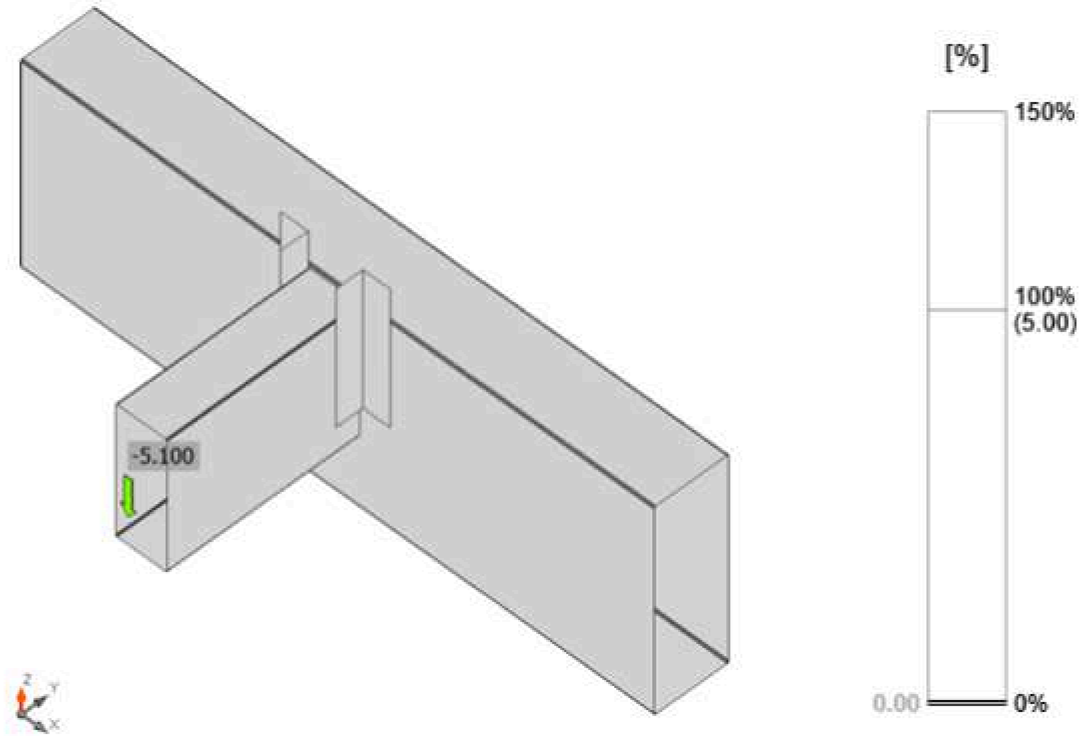
Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
B	A913 Gr.50	50.0	1/8	LE1	15.0	0.0	0.0	OK
B1	A913 Gr.50	50.0	1/8	LE1	15.3	0.0	0.0	OK
M3-bfl 1	A36	36.0	1/4	LE1	10.0	0.0	1.5	OK
M3-w 1	A36	36.0	1/4	LE1	11.3	0.0	0.8	OK
M4-bfl 1	A36	36.0	1/4	LE1	11.3	0.0	0.7	OK
M4-w 1	A36	36.0	1/4	LE1	10.0	0.0	1.5	OK

Design data

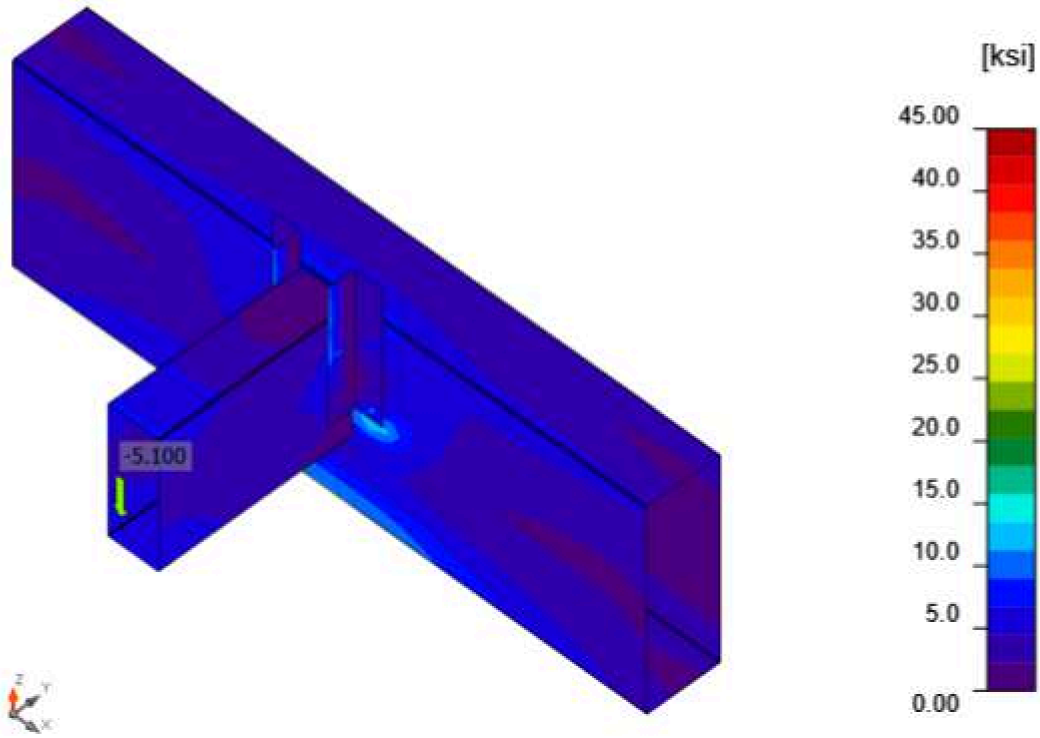
Material	f_y [ksi]	ε_{lim} [%]
A913 Gr.50	50.0	5.0
A36	36.0	5.0



Overall check, LE1

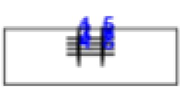


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	1/2 A325 - 1	LE1	0.136	0.669	7.605	1.0	8.8	-	OK
	B2	1/2 A325 - 1	LE1	0.118	0.632	7.605	0.9	8.3	-	OK
	B3	1/2 A325 - 1	LE1	0.171	0.618	7.605	1.3	8.1	-	OK
	B4	1/2 A325 - 1	LE1	0.500	0.634	7.605	3.8	8.3	-	OK
	B5	1/2 A325 - 1	LE1	0.139	0.669	7.605	1.0	8.8	-	OK
	B6	1/2 A325 - 1	LE1	0.118	0.632	7.605	0.9	8.3	-	OK
	B7	1/2 A325 - 1	LE1	0.171	0.618	7.605	1.3	8.1	-	OK
	B8	1/2 A325 - 1	LE1	0.500	0.634	7.605	3.8	8.3	-	OK

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B9	1/2 A325 - 1	LE1	0.007	0.715	5.675	0.1	12.6	-	OK
	B10	1/2 A325 - 1	LE1	0.000	0.642	5.675	0.0	11.3	-	OK
	B11	1/2 A325 - 1	LE1	0.000	0.647	5.675	0.0	11.4	-	OK
	B12	1/2 A325 - 1	LE1	0.000	0.754	5.675	0.0	13.3	-	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
1/2 A325 - 1	13.243	7.946

Detailed result for B12

Tension resistance check (AISC 360-16: J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 13.243 kip ≥ $F_t =$ 0.000 kip

Where:

$F_{nt} = 89.9$ ksi – nominal tensile stress from AISC 360-16 Table J3.2

$A_b = 0.196\ \text{in}^2$ – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Shear resistance check (AISC 360-16: J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 7.946 kip ≥ $V =$ 0.754 kip

Where:

$F_{nv} = 54.0$ ksi – nominal shear stress from AISC 360-16 Table J3.2

$A_b = 0.196\ \text{in}^2$ – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Bearing resistance check (AISC 360-16: J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 5.675 kip ≥ $V =$ 0.754 kip

Where:

$l_c = 2.774$ in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

$t = 0.097$ in – thickness of the plate

$d = 0.500$ in – diameter of a bolt

$F_u = 65.0$ ksi – tensile strength of the connected material

$\phi = 0.75$ – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16: J3-2)

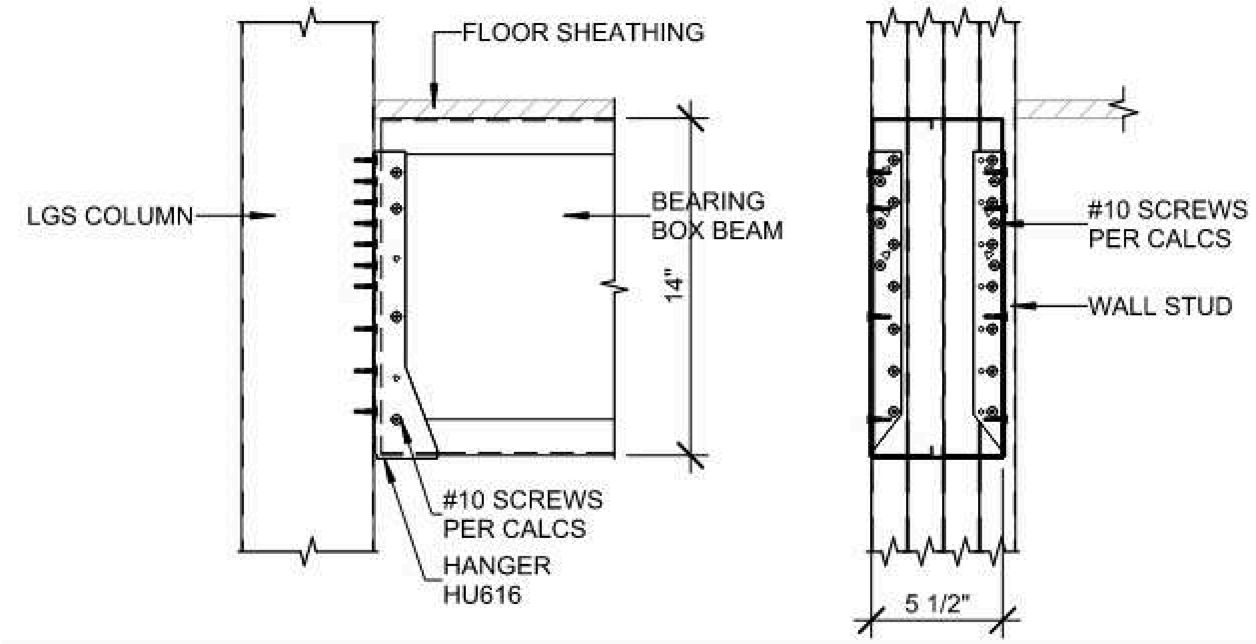
The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Symbol explanation

Symbol	Symbol explanation
ε_{Pl}	Strain
f_y	Yield strength
ε_{lim}	Limit of plastic strain
F_t	Tension force
V	Resultant of shear forces Vy, Vz in bolt
$\phi Rn_{Bearing}$	Plate bearing resistance AISC 360-16 J3.10
Ut	Utilization
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear EN 1993-1-8 table 3.4
$\phi Rn_{Bearing}$	Bolt bearing resistance
ϕRn_{Shear}	Bolt shear resistance AISC 360-16 – J3.8

2.5.3 14"X5.5" TO LGS COLUMN CONNECTION DESIGN

For connection 14” X 5.5” X 12GA use Simpson hanger HU616. Beam support reaction V= 5050 lbs



For connecting Simpson hange to lgs column 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	$\boxed{810}$	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 5050 / 810 = 6.23 pcs. = **8** pcs one side.

2.5.4 LGS COLUMN TO FOUNDATION MOMENT CONNECTION

Project data

Project name	Beemsterboer
Project number	24-113
Author	DZ
Description	LGS COLUMN TO FOUNDATION MOMENT CONNECTION
Date	7/16/2024
Design code	AISC 360-16

Material

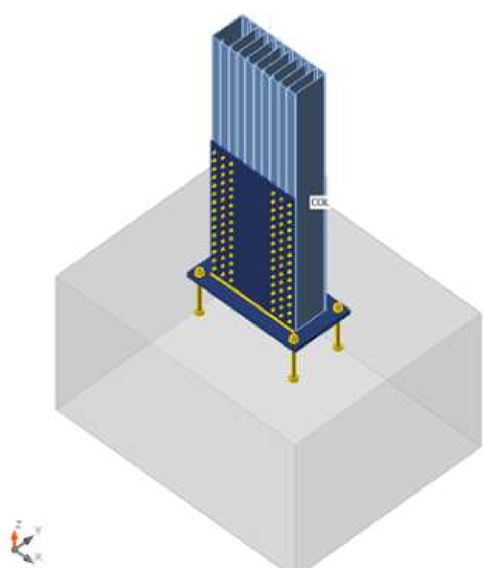
Steel	A500, Gr. C, shaped, A913 Gr.50
Concrete	3000 psi

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
COL	2 - 10X550S150-54(General)	90.0	-90.0	0.0	0.000	0.000	0.000	Node



Cross-sections

Name	Material
2 - 10X550S150-54(General)	A913 Gr.50, A913 Gr.50, A913 Gr.50, A913 Gr.50, A913 Gr.50, A913 Gr.50, A913 Gr.50, A913 Gr.50, A913 Gr.50, A913 Gr.50

Anchors / Bolts

Name	Bolt assembly	Diameter [in]	fu [ksi]	Gross area [in^2]
1/2 A325	1/2 A325	0.500	120.0	0.196
#10	#10	0.190	120.0	0.028

Load effects (equilibrium not required)

Name	Member	N [lbf]	Vy [lbf]	Vz [lbf]	Mx [lbf.ft]	My [lbf.ft]	Mz [lbf.ft]
LE1	COL	-14463.000	-1600.000	0.000	0.00	0.00	-17735.00
LE2	COL	-2554.000	-1514.000	0.000	0.00	0.00	-18070.00
LE3	COL	-5040.000	-1500.000	0.000	0.00	0.00	-18082.00
LE4	COL	-4318.000	-1506.000	0.000	0.00	0.00	-18082.00
LE5	COL	-5960.000	-1512.000	0.000	0.00	0.00	-18145.00

Foundation block

Item	Value	Unit
CB 1		
Dimensions	43.122 x 33.622	in
Depth	24.000	in
Anchor	1/2 A325	
Anchoring length	6.000	in
Shear force transfer	Friction	

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	5.0 < 5.0%	OK
Bolts	48.5 < 100%	OK
Anchors	69.1 < 100%	OK
Welds	94.7 < 100%	OK
Concrete block	9.8 < 100%	OK
Shear	31.6 < 100%	OK
Buckling	Not calculated	

Plates

Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
COL-w 1	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-arc 1	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-arc 2	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-arc 3	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-tfl 1	A913 Gr.50	50.0	1/16	LE1	45.6	2.0	0.3	OK
COL-arc 4	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-arc 5	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-arc 6	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-w 2	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-arc 7	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-arc 8	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-arc 9	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-bfl 1	A913 Gr.50	50.0	1/16	LE1	45.6	2.1	0.2	OK
COL-arc 10	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-arc 11	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-arc 12	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-w 3	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-w 4	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-arc 13	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-arc 14	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-arc 15	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-tfl 2	A913 Gr.50	50.0	1/16	LE1	45.6	2.1	0.3	OK
COL-arc 16	A913 Gr.50	50.0	1/16	LE1	44.9	0.0	0.0	OK
COL-arc 17	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-arc 18	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-w 5	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-arc 19	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-arc 20	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-arc 21	A913 Gr.50	50.0	1/16	LE1	44.3	0.0	0.0	OK
COL-bfl 2	A913 Gr.50	50.0	1/16	LE1	45.7	2.3	0.1	OK
COL-arc 22	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-arc 23	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-arc 24	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-w 6	A913 Gr.50	50.0	1/16	LE1	45.3	1.0	0.0	OK
COL-w 7	A913 Gr.50	50.0	1/16	LE1	45.3	1.0	0.0	OK
COL-arc 25	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK

Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
COL-arc 26	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-arc 27	A913 Gr.50	50.0	1/16	LE1	45.2	0.9	0.0	OK
COL-tfl 3	A913 Gr.50	50.0	1/16	LE1	45.6	2.2	0.3	OK
COL-arc 28	A913 Gr.50	50.0	1/16	LE1	43.2	0.0	0.0	OK
COL-arc 29	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-arc 30	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-w 8	A913 Gr.50	50.0	1/16	LE1	44.1	0.0	0.0	OK
COL-arc 31	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-arc 32	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-arc 33	A913 Gr.50	50.0	1/16	LE1	42.1	0.0	0.0	OK
COL-bfl 3	A913 Gr.50	50.0	1/16	LE1	45.7	2.4	0.2	OK
COL-arc 34	A913 Gr.50	50.0	1/16	LE2	45.3	0.9	0.0	OK
COL-arc 35	A913 Gr.50	50.0	1/16	LE2	45.3	1.0	0.0	OK
COL-arc 36	A913 Gr.50	50.0	1/16	LE2	45.3	1.0	0.0	OK
COL-w 9	A913 Gr.50	50.0	1/16	LE1	45.3	1.1	0.0	OK
COL-w 10	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 37	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 38	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 39	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-tfl 4	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 40	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 41	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 42	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 11	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 43	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 44	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 45	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-bfl 4	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 46	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 47	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 48	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 12	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 13	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 49	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 50	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK

Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
COL-arc 51	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-tfl 5	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 52	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 53	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 54	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 14	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 55	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 56	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 57	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-bfl 5	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 58	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 59	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 60	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 15	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 16	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 61	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 62	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 63	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-tfl 6	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 64	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 65	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 66	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 17	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 67	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 68	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 69	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-bfl 6	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 70	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 71	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 72	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 18	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 19	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-arc 73	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK

Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
COL-arc 74	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-arc 75	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-tfl 7	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-arc 76	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-arc 77	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-arc 78	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-w 20	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-arc 79	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 80	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 81	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-bfl 7	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-arc 82	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 83	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-arc 84	A913 Gr.50	50.0	1/16	LE1	0.0	0.0	0.0	OK
COL-w 21	A913 Gr.50	50.0	1/16	LE2	0.0	0.0	0.0	OK
COL-w 22	A913 Gr.50	50.0	1/16	LE2	45.4	1.3	0.0	OK
COL-arc 85	A913 Gr.50	50.0	1/16	LE2	45.4	1.3	0.0	OK
COL-arc 86	A913 Gr.50	50.0	1/16	LE2	45.4	1.3	0.0	OK
COL-arc 87	A913 Gr.50	50.0	1/16	LE2	45.4	1.2	0.0	OK
COL-tfl 8	A913 Gr.50	50.0	1/16	LE1	45.9	3.2	0.2	OK
COL-arc 88	A913 Gr.50	50.0	1/16	LE1	33.1	0.0	0.0	OK
COL-arc 89	A913 Gr.50	50.0	1/16	LE1	41.5	0.0	0.0	OK
COL-arc 90	A913 Gr.50	50.0	1/16	LE1	45.0	0.0	0.0	OK
COL-w 23	A913 Gr.50	50.0	1/16	LE1	37.9	0.0	0.0	OK
COL-arc 91	A913 Gr.50	50.0	1/16	LE1	44.0	0.0	0.0	OK
COL-arc 92	A913 Gr.50	50.0	1/16	LE1	36.6	0.0	0.0	OK
COL-arc 93	A913 Gr.50	50.0	1/16	LE1	30.6	0.0	0.0	OK
COL-bfl 8	A913 Gr.50	50.0	1/16	LE1	46.1	3.9	0.1	OK
COL-arc 94	A913 Gr.50	50.0	1/16	LE2	45.5	1.6	0.0	OK
COL-arc 95	A913 Gr.50	50.0	1/16	LE2	45.5	1.6	0.0	OK
COL-arc 96	A913 Gr.50	50.0	1/16	LE2	45.5	1.7	0.0	OK
COL-w 24	A913 Gr.50	50.0	1/16	LE2	45.5	1.7	0.0	OK
COL-w 25	A913 Gr.50	50.0	1/16	LE2	45.4	1.2	0.0	OK
COL-arc 97	A913 Gr.50	50.0	1/16	LE2	45.5	1.6	0.0	OK
COL-arc 98	A913 Gr.50	50.0	1/16	LE2	45.5	1.6	0.0	OK

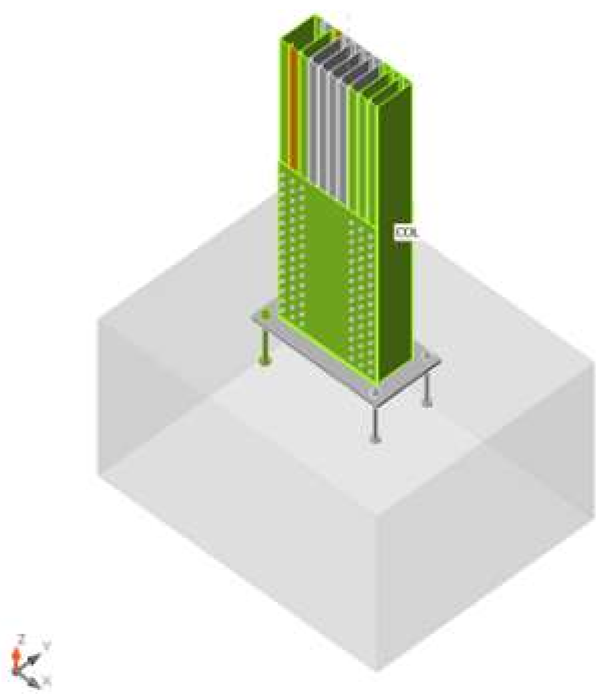
Name	Material	f_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	σc_{Ed} [ksi]	Check status
COL-arc 99	A913 Gr.50	50.0	1/16	LE2	45.5	1.6	0.0	OK
COL-tfl 9	A913 Gr.50	50.0	1/16	LE2	46.4	5.0	0.7	OK
COL-arc 100	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-arc 101	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-arc 102	A913 Gr.50	50.0	1/16	LE1	45.0	0.1	0.0	OK
COL-w 26	A913 Gr.50	50.0	1/16	LE1	44.0	0.0	0.0	OK
COL-arc 103	A913 Gr.50	50.0	1/16	LE1	14.9	0.0	0.0	OK
COL-arc 104	A913 Gr.50	50.0	1/16	LE1	14.5	0.0	0.0	OK
COL-arc 105	A913 Gr.50	50.0	1/16	LE1	13.4	0.0	0.0	OK
COL-bfl 9	A913 Gr.50	50.0	1/16	LE1	15.3	0.0	0.0	OK
COL-arc 106	A913 Gr.50	50.0	1/16	LE1	11.6	0.0	0.0	OK
COL-arc 107	A913 Gr.50	50.0	1/16	LE1	10.9	0.0	0.0	OK
COL-arc 108	A913 Gr.50	50.0	1/16	LE1	9.8	0.0	0.0	OK
COL-w 27	A913 Gr.50	50.0	1/16	LE1	10.0	0.0	0.0	OK
COL-w 28	A913 Gr.50	50.0	1/16	LE1	45.3	1.0	0.0	OK
COL-arc 109	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-arc 110	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.0	OK
COL-arc 111	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.0	OK
COL-tfl 10	A913 Gr.50	50.0	1/16	LE1	45.2	0.8	0.8	OK
COL-arc 112	A913 Gr.50	50.0	1/16	LE2	45.0	0.0	0.0	OK
COL-arc 113	A913 Gr.50	50.0	1/16	LE2	45.0	0.1	0.0	OK
COL-arc 114	A913 Gr.50	50.0	1/16	LE2	45.0	0.1	0.0	OK
COL-w 29	A913 Gr.50	50.0	1/16	LE2	45.0	0.0	0.0	OK
COL-arc 115	A913 Gr.50	50.0	1/16	LE2	45.0	0.0	0.0	OK
COL-arc 116	A913 Gr.50	50.0	1/16	LE2	45.0	0.0	0.0	OK
COL-arc 117	A913 Gr.50	50.0	1/16	LE2	40.2	0.0	0.0	OK
COL-bfl 10	A913 Gr.50	50.0	1/16	LE1	45.3	0.9	0.6	OK
COL-arc 118	A913 Gr.50	50.0	1/16	LE1	45.3	1.0	0.0	OK
COL-arc 119	A913 Gr.50	50.0	1/16	LE1	45.3	1.0	0.0	OK
COL-arc 120	A913 Gr.50	50.0	1/16	LE1	45.3	1.0	0.0	OK
COL-w 30	A913 Gr.50	50.0	1/16	LE1	45.3	1.1	0.0	OK
BP1	A500, Gr. C, shaped	50.0	5/8	LE2	34.8	0.0	0.0	OK
SP1	A500, Gr. C, shaped	50.0	1/8	LE2	37.5	0.1	0.5	OK
SP2	A500, Gr. C, shaped	50.0	1/8	LE2	39.4	0.2	0.7	OK

Design data

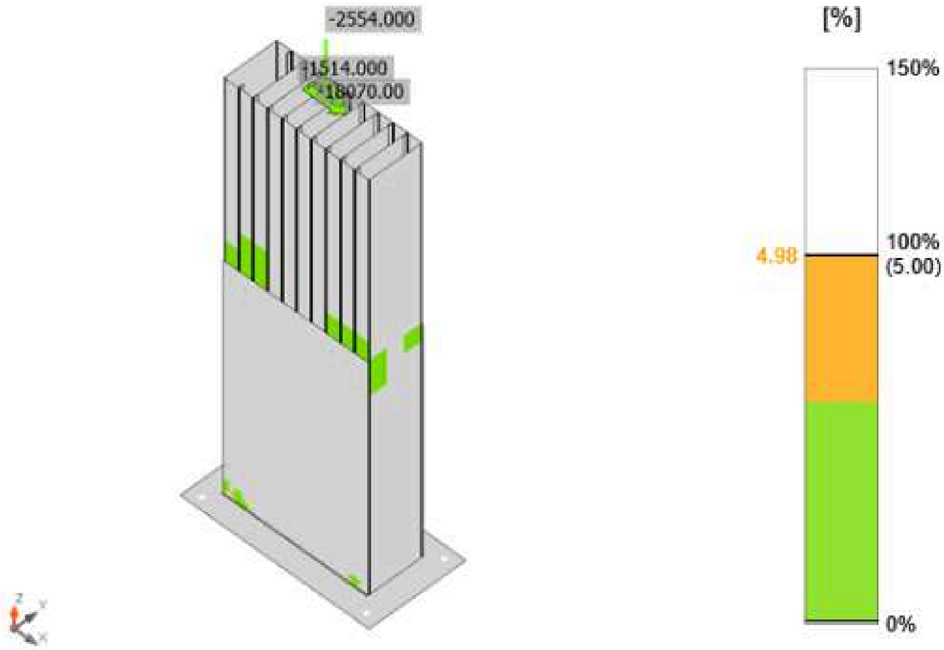
Material	f_y [ksi]	ε_{lim} [%]
A913 Gr.50	50.0	5.0
A500, Gr. C, shaped	50.0	5.0

Symbol explanation

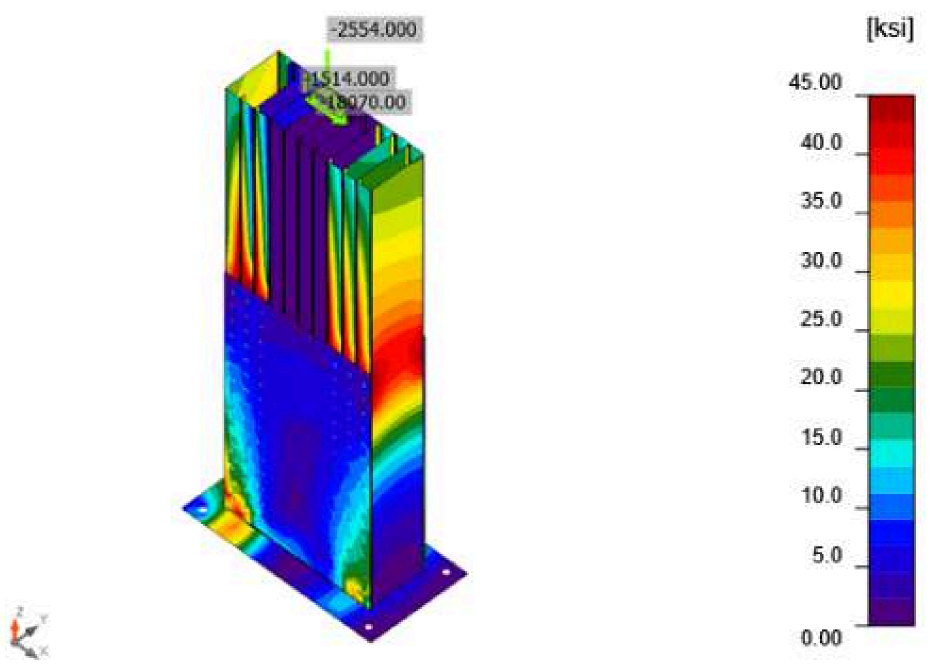
ε_{Pl}	Plastic strain
σc_{Ed}	Contact stress
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain



Overall check, LE2

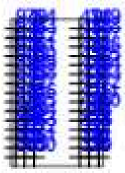


Strain check, LE2

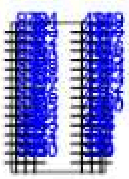


Equivalent stress, LE2

Bolts

Shape	Item	Grade	Loads	F_t [lbf]	V [lbf]	$\phi R_{n,bearing}$ [lbf]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B5	#10 - 1	LE1	14.572	314.795	1200.422	0.8	27.5	-	OK
	B6	#10 - 1	LE1	1.105	296.582	1200.422	0.1	25.9	-	OK
	B7	#10 - 1	LE1	0.000	280.788	1200.422	0.0	24.5	-	OK
	B8	#10 - 1	LE1	0.850	267.960	1200.422	0.0	23.4	-	OK
	B9	#10 - 1	LE1	1.133	257.090	1200.422	0.1	22.4	-	OK
	B10	#10 - 1	LE1	2.079	248.997	1200.422	0.1	21.7	-	OK
	B11	#10 - 1	LE1	3.899	244.110	1200.422	0.2	21.3	-	OK
	B12	#10 - 1	LE1	4.639	242.878	1200.422	0.2	21.2	-	OK
	B13	#10 - 1	LE1	8.206	242.831	1200.422	0.4	21.2	-	OK
	B14	#10 - 1	LE1	13.860	239.402	1200.422	0.7	20.9	-	OK
	B15	#10 - 1	LE1	20.098	227.988	1200.422	1.1	19.9	-	OK
	B16	#10 - 1	LE1	29.022	206.899	1200.422	1.5	18.1	-	OK
	B17	#10 - 1	LE1	34.499	198.709	1200.422	1.8	17.4	-	OK
	B18	#10 - 1	LE1	29.588	264.149	1200.422	1.6	23.1	-	OK
	B19	#10 - 1	LE1	14.347	539.677	1200.422	0.8	47.1	-	OK
	B20	#10 - 1	LE1	12.377	278.513	1200.422	0.6	24.3	-	OK
	B21	#10 - 1	LE1	1.780	259.840	1200.422	0.1	22.7	-	OK
	B22	#10 - 1	LE1	0.000	245.705	1200.422	0.0	21.5	-	OK
	B23	#10 - 1	LE1	0.000	235.213	1200.422	0.0	20.5	-	OK
	B24	#10 - 1	LE1	0.131	226.829	1200.422	0.0	19.8	-	OK
	B25	#10 - 1	LE1	1.096	221.142	1200.422	0.1	19.3	-	OK
	B26	#10 - 1	LE1	2.070	218.550	1200.422	0.1	19.1	-	OK
	B27	#10 - 1	LE1	2.550	219.503	1200.422	0.1	19.2	-	OK
	B28	#10 - 1	LE1	5.163	221.338	1200.422	0.3	19.3	-	OK
	B29	#10 - 1	LE1	11.538	219.497	1200.422	0.6	19.2	-	OK
	B30	#10 - 1	LE1	17.198	209.066	1200.422	0.9	18.3	-	OK
	B31	#10 - 1	LE1	26.652	187.854	1200.422	1.4	16.4	-	OK
	B32	#10 - 1	LE1	31.387	178.988	1200.422	1.6	15.6	-	OK
	B33	#10 - 1	LE1	21.989	246.079	1200.422	1.2	21.5	-	OK
	B34	#10 - 1	LE1	0.976	524.771	1200.422	0.1	45.8	-	OK
	B35	#10 - 1	LE1	9.273	231.874	1200.422	0.5	20.2	-	OK
	B36	#10 - 1	LE1	0.906	220.266	1200.422	0.0	19.2	-	OK
	B37	#10 - 1	LE1	0.000	210.313	1200.422	0.0	18.4	-	OK
	B38	#10 - 1	LE1	0.000	202.840	1200.422	0.0	17.7	-	OK
	B39	#10 - 1	LE1	0.000	197.081	1200.422	0.0	17.2	-	OK
	B40	#10 - 1	LE1	0.000	193.764	1200.422	0.0	16.9	-	OK

Shape	Item	Grade	Loads	F_t [lbf]	V [lbf]	$\phi R_{n,bearing}$ [lbf]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B41	#10 - 1	LE1	0.877	193.473	1200.422	0.0	16.9	-	OK
	B42	#10 - 1	LE1	1.434	196.469	1200.422	0.1	17.2	-	OK
	B43	#10 - 1	LE1	4.320	200.109	1200.422	0.2	17.5	-	OK
	B44	#10 - 1	LE1	9.208	199.592	1200.422	0.5	17.4	-	OK
	B45	#10 - 1	LE1	14.956	189.687	1200.422	0.8	16.6	-	OK
	B46	#10 - 1	LE1	23.778	167.581	1200.422	1.2	14.6	-	OK
	B47	#10 - 1	LE1	24.966	157.774	1200.422	1.3	13.8	-	OK
	B48	#10 - 1	LE1	1.837	228.331	1200.422	0.1	19.9	-	OK
	B49	#10 - 1	LE1	0.000	509.849	1200.422	0.0	44.5	-	OK
	B50	#10 - 1	LE2	5.805	75.557	1200.422	0.3	6.6	-	OK
	B51	#10 - 1	LE2	3.463	72.414	1200.422	0.2	6.3	-	OK
	B52	#10 - 1	LE2	3.263	67.708	1200.422	0.2	5.9	-	OK
	B53	#10 - 1	LE2	0.273	63.662	1200.422	0.0	5.6	-	OK
	B54	#10 - 1	LE2	0.000	61.530	1200.422	0.0	5.4	-	OK
	B55	#10 - 1	LE2	0.000	62.593	1200.422	0.0	5.5	-	OK
	B56	#10 - 1	LE2	0.000	67.260	1200.422	0.0	5.9	-	OK
	B57	#10 - 1	LE1	0.000	81.856	1200.422	0.0	7.1	-	OK
	B58	#10 - 1	LE1	0.000	95.876	1200.422	0.0	8.4	-	OK
	B59	#10 - 1	LE1	0.000	101.730	1200.422	0.0	8.9	-	OK
	B60	#10 - 1	LE1	0.000	91.577	1200.422	0.0	8.0	-	OK
	B61	#10 - 1	LE2	0.000	76.741	1200.422	0.0	6.7	-	OK
	B62	#10 - 1	LE2	0.000	84.185	1200.422	0.0	7.4	-	OK
	B63	#10 - 1	LE2	0.644	165.173	1200.422	0.0	14.4	-	OK
	B64	#10 - 1	LE1	0.000	450.035	1200.422	0.0	39.3	-	OK
	B65	#10 - 1	LE2	16.244	149.023	1200.422	0.9	13.3	-	OK
	B66	#10 - 1	LE2	5.479	134.033	1200.422	0.3	11.9	-	OK
	B67	#10 - 1	LE2	4.365	122.589	1200.422	0.2	10.7	-	OK
	B68	#10 - 1	LE2	1.075	113.590	1200.422	0.1	9.9	-	OK
	B69	#10 - 1	LE2	0.000	106.981	1200.422	0.0	9.3	-	OK
	B70	#10 - 1	LE2	0.000	103.165	1200.422	0.0	9.0	-	OK
	B71	#10 - 1	LE2	0.000	102.521	1200.422	0.0	9.0	-	OK
	B72	#10 - 1	LE2	0.000	105.075	1200.422	0.0	9.2	-	OK
	B73	#10 - 1	LE2	0.000	110.660	1200.422	0.0	9.7	-	OK
	B74	#10 - 1	LE2	0.000	116.262	1200.422	0.0	10.2	-	OK
	B75	#10 - 1	LE2	0.000	119.167	1200.422	0.0	10.4	-	OK

Shape	Item	Grade	Loads	F_t [lbf]	V [lbf]	$\phi R_{n,bearing}$ [lbf]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B76	#10 - 1	LE2	0.000	117.904	1200.422	0.0	10.3	-	OK
	B77	#10 - 1	LE2	0.000	121.872	1200.422	0.0	10.6	-	OK
	B78	#10 - 1	LE2	0.000	176.754	1200.422	0.0	15.4	-	OK
	B79	#10 - 1	LE1	0.000	452.290	1200.422	0.0	39.5	-	OK
	B80	#10 - 1	LE2	2.825	298.232	1200.422	0.1	26.0	-	OK
	B81	#10 - 1	LE2	0.000	279.836	1200.422	0.0	24.4	-	OK
	B82	#10 - 1	LE2	5.398	266.119	1200.422	0.3	23.2	-	OK
	B83	#10 - 1	LE2	4.886	255.905	1200.422	0.3	22.3	-	OK
	B84	#10 - 1	LE2	3.895	248.347	1200.422	0.2	21.7	-	OK
	B85	#10 - 1	LE2	3.031	243.642	1200.422	0.2	21.3	-	OK
	B86	#10 - 1	LE2	1.057	242.055	1200.422	0.1	21.1	-	OK
	B87	#10 - 1	LE2	0.000	243.906	1200.422	0.0	21.3	-	OK
	B88	#10 - 1	LE2	0.000	247.247	1200.422	0.0	21.6	-	OK
	B89	#10 - 1	LE2	0.000	246.643	1200.422	0.0	21.5	-	OK
	B90	#10 - 1	LE2	0.000	237.822	1200.422	0.0	20.8	-	OK
	B91	#10 - 1	LE2	0.000	219.640	1200.422	0.0	19.2	-	OK
	B92	#10 - 1	LE2	0.000	211.670	1200.422	0.0	18.5	-	OK
	B93	#10 - 1	LE2	0.000	283.733	1200.422	0.0	24.8	-	OK
	B94	#10 - 1	LE2	0.000	555.184	1200.422	0.0	48.5	-	OK

Design data

Grade	$\phi R_{n,tension}$ [lbf]	$\phi R_{n,shear}$ [lbf]
#10 - 1	1908.624	1145.174

Symbol explanation

F_t	Tension force
V	Resultant of shear forces Vy, Vz in bolt
$\phi R_{n,bearing}$	Bolt bearing resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance AISC 360-16 J3.6
$\phi R_{n,shear}$	Bolt shear resistance AISC 360-16 – J3.8

Detailed result for B94

Tension resistance check (AISC 360-16: J3-1)

$$\phi R_n = \phi \cdot F_{nt} \cdot A_b = \quad 1908.624 \text{ lbf} \geq F_t = \quad 0.000 \text{ lbf}$$

Where:

$F_{nt} = 89.9$ ksi – nominal tensile stress from AISC 360-16 Table J3.2

$A_b = 0.028$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Shear resistance check (AISC 360-16: J3-1)

$$\phi R_n = \phi \cdot F_{nv} \cdot A_b = \quad 1145.174 \text{ lbf} \geq V = \quad 555.184 \text{ lbf}$$

Where:

$F_{nv} = 54.0$ ksi – nominal shear stress from AISC 360-16 Table J3.2

$A_b = 0.028$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Bearing resistance check (AISC 360-16: J3-6)

$$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$$

$$\phi R_n = \quad 1200.422 \text{ lbf} \geq V = \quad 555.108 \text{ lbf}$$

Where:

$l_c = 0.553$ in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

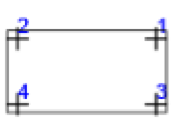
$t = 0.054$ in – thickness of the plate

$d = 0.190$ in – diameter of a bolt

$F_u = 65.0$ ksi – tensile strength of the connected material

$\phi = 0.75$ – resistance factor for bearing at bolt holes

Anchors

Shape	Item	Loads	N_f [lbf]	V [lbf]	ϕN_{cbg} [lbf]	ϕN_p [lbf]	ϕN_{sb} [lbf]	ϕV_{cp} [lbf]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	A1	LE2	7656.726	0.812	21436.193	17317.800	40284.797	76161.080	69.1	0.0	54.1	OK
	A2	LE2	0.000	0.464	-	17317.800	-	76161.080	0.0	0.0	0.0	OK
	A3	LE2	7163.851	0.780	21436.193	17317.800	40284.797	76161.080	69.1	0.0	54.1	OK
	A4	LE2	0.000	0.466	-	17317.800	-	76161.080	0.0	0.0	0.0	OK

Design data

Grade	ϕN_{sa} [lbf]	ϕV_{sa} [lbf]
1/2 A325 - 2	11927.645	6645.402

Symbol explanation

N_f	Tension force
V	Resultant of shear forces Vy, Vz in bolt
ϕN_{cbg}	Concrete breakout strength in tension – ACI 318-14 – 17.4.2
ϕN_p	Pullout strength in tension – ACI 318-14 – 17.4.3
ϕN_{sb}	Concrete side-face blowout strength in tension – ACI 318-14 – 17.4.4
ϕV_{cp}	Concrete pryout strength in shear – ACI 318-14 – 17.5.3
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
ϕN_{sa}	Steel strength of anchor in tension - ACI 318-14 – 17.4.1
ϕV_{sa}	Steel strength of anchor in shear - ACI 318-14 – 17.5.1

Detailed result for A1

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 11927.645 lbf ≥ $N_f =$ 7656.726 lbf

Where:

$\phi = 0.70$ – resistance factor

$A_{se,N} = 0.142$ in^2 – tensile stress area

$f_{uta} = 120.0$ ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125 \text{ ksi}, 1.9 \cdot f_{ya},\ f_u)$, where:
  - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
  - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A1, A3

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b =$ 21436.193 lbf ≥ $N_{fg} =$ 14820.577 lbf

Where:

$N_{fg} =$ 14820.577 lbf – sum of tension forces of anchors with common concrete breakout cone area

$\phi = 0.70$ – resistance factor

$A_{Nc} = 523.430$ in^2 – concrete breakout cone area for group of anchors

$A_{Nc0} = 324.000$ in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N} = 1.00$ – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
    - $c_{a,min} =$ 12.811 in – minimum distance from the anchor to the edge
    - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 6.000 in – depth of embedment, where:
        - $h_{emb} =$ 6.000 in – anchor length
        - $c_{a,max} =$ 13.061 in – maximum distance from the anchor to one of the three closest edges
        - $s =$ 8.000 in – maximum spacing between anchors

$\Psi_{ec,N} = 0.99$ – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
    - $\Psi_{ecx,N} = \frac{1}{1+\frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 1.00 – modification factor that depends on eccentricity in x-direction
    - $e_{x,N} =$ 0.000 in – tension load eccentricity in x-direction
    - $\Psi_{ecy,N} = \frac{1}{1+\frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.99 – modification factor that depends on eccentricity in y-direction
    - $e_{y,N} =$ 0.133 in – tension load eccentricity in y-direction
    - $h_{ef} =$ 6.000 in – depth of embedment

$\Psi_{c,N} = 1.00$ – modification factor for concrete conditions

$N_b = 19235.712$ lbf – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot h_{ef}^{1.5}$, where:
    - $k_c =$ 24.0 – coefficient for cast-in anchors
    - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
    - $f'_c =$ 3.0 ksi – concrete compressive strength
    - $h_{ef} =$ 6.000 in – depth of embedment

Concrete pullout resistance (ACI 318-14 – 17.4.3)

$\phi N_{pn} = \phi \cdot \Psi_{c,P} \cdot N_p =$ 17317.800 lbf ≥ $N_f =$ 7656.726 lbf

Where:

$\phi = 0.70$ – resistance factor

$\Psi_{c,P} = 1.00$ – modification factor for concrete condition

$N_p = 24739.715$ lbf – basic concrete pullout strength for headed anchor:

- $N_{pr} = 8 \cdot A_{brg} \cdot f'_c$, where:
    - $A_{brg} =$ 1.031 in^2 – bearing area of the head of stud or anchor bolt
    - $f'_c =$ 3.0 ksi – concrete compressive strength

Concrete sideface blowout resistance (ACI 318-14 – 17.4.4)

$\phi N_{sb} = r_c \cdot N_{sbg} =$ 40284.797 lbf ≥ $N_f =$ 7656.726 lbf

Where:

$r_c = 0.50$ – reduction factor for anchor close to an edge or multiple anchors with small spacing:

- $r_c = \min(\frac{1+\frac{c_{a2}}{c_{a1}}}{4}, 1+\frac{s}{6 \cdot c_{a1}}),\ 0.5\ \leq r_c \leq 1$, where:
    - $c_{a1} =$ 12.811 in – shorter distance from an anchor to an edge
    - $c_{a2} =$ 13.061 in – longer distance from an anchor to an edge
    - $s =$ 8.000 in – spacing between anchors

$N_{sbg} =$ 79791.055 lbf – concrete side-face blowout strength of headed anchor in tension:

- $N_{sb} = \phi \cdot 160.0 \cdot c_{a1} \cdot \sqrt{A_{bgr}} \cdot \sqrt{f'_c}$, where:
    - $A_{bgr} =$ 1.031 in^2 – bearing area of the head of stud or anchor bolt
    - $f'_c =$ 3.0 ksi – concrete compressive strength
    - $\phi =$ 0.70 – resistance factor

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 6645.402 lbf ≥ $V =$ 0.812 lbf

Where:

$\phi = 0.65$ – resistance factor

$A_{se,V} = 0.142$ in^2 – tensile stress area

$f_{uta} = 120.0$ ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \mathrm{ksi}, 1.9 \cdot f_{ya},\ f_u)$, where:
    - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
    - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 76161.080 lbf ≥ $V_g =$ 0.000 lbf

Where:

$\phi = 0.65$ – resistance factor

$k_{cp} = 2.00$ – concrete pry-out factor

$N_{cp} = 58585.446$ lbf – concrete cone tension break-out resistance in case all anchors are in tension

$V_g = 0.000$ lbf – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

${U_{tt}}^{5/3} + {U_{ts}}^{5/3} =$ 0.54 ≤ 1.0

Where:

$U_{tt} = 0.69$ – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

$U_{ts} = 0.00$ – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Weld sections

Item	Edge	Xu	T_h [in]	L_s [in]	L [in]	L_c [in]	Loads	F_n [lbf]	ϕR_n [lbf]	Ut [%]	Status
BP1	SP1	E70xx	◢1/8	◢3/16	14.976	1.248	LE2	5917.725	6427.857	92.1	OK
BP1	SP2	E70xx	◢1/8	◢3/16	14.976	1.248	LE2	6094.660	6436.141	94.7	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 BP1 / SP2

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

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 6436.141 lbf ≥ $F_n =$ 6094.660 lbf

Where:

$F_{nw} =$ 61.7 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
  - $\theta =$ 73.7° – angle of loading measured from the weld longitudinal axis

$A_{we} =$ 0.139 in^2 – effective area of weld critical element

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

Concrete block

Item	Loads	A_1 [in2]	A_2 [in2]	σ [ksi]	Ut [%]	Status
CB 1	LE1	78.596	974.422	0.3	9.8	OK

Symbol explanation

A_1	Loaded area
A_2	Supporting area
σ	Average stress in concrete
Ut	Utilization

Detailed result for CB 1

Concrete block compressive resistance check (AISC 360-16 Section J8)

$\phi_c f_{p,max} =$ 3.3 ksi ≥ $\sigma =$ 0.3 ksi

Where:

$f_{p,max} = 5.1$ ksi – concrete block design bearing strength:

- $f_{p,max} = 0.85 \cdot f'_c \cdot \sqrt{\frac{A_2}{A_1}} \leq 1.7 \cdot f'_c$, where:
  - $f'_c =$ 3.0 ksi – concrete compressive strength
  - $A_1 =$ 78.596 in^2 – base plate area in contact with concrete surface
  - $A_2 =$ 974.422 in^2 – concrete supporting surface

$\phi_c = 0.65$ – resistance factor for concrete

Shear in contact plane

Item	Loads	V [lbf]	ϕV_r [lbf]	μ [-]	Ut [%]	Status
BP1	LE2	1511.502	4782.129	0.40	31.6	OK

Symbol explanation

V	Shear force
ϕV_r	Shear resistance
μ	Coefficient of friction between base plate and concrete block
Ut	Utilization

Detailed result for BP1

Base plate shear resistance check (ACI 349 – B.6.1.4)

$\phi_c V_r = \phi_c \cdot \mu \cdot C =$ 4782.129 lbf ≥ $V =$ 1511.502 lbf

Where:

$\phi_c = 0.65$ – resistance factor for concrete

$\mu = 0.4$ – coefficient of friction between base plate and concrete

$C = 18392.804$ lbf – compressive force

3.0 FOUNDATION ANALYSIS

FOUNDATION PLAN

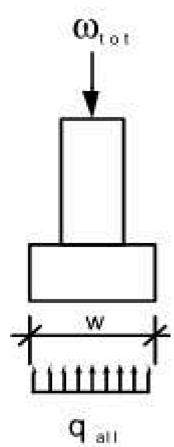
3.1 FOOTING ANALYSIS & DESIGN
WF1 - EXTERIOR WALL FOOTING ANALYSIS & DESIGN

Footing **Exterior Wall Foundation** **WF1**

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

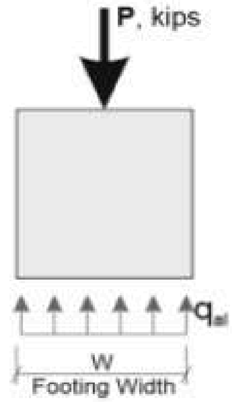
Conventional Footing Design Parameters:
Allowable Bearing Pressure 2000 psf

FT	TRIB	Wdl	Wll
	(ft)	(plf)	(plf)
WF1	10.2 r	204	204
	8.2 f	123	328
	0.0 g	0	0
	0.0 d	0	0
	23.0 w	1035	0
	TL	**1362**	**532**



w_{tot} 1894 plf
Try Footing Width, w = 18 inches
q_{bear} 1263 psf
q_{allow} 2000 psf
q_{allow}/q_{bear} 1.58 Footing OK

Footing **Exterior wall Foundation** **WF1**



Reactions on Footing		**D**	**L**	**E**	**W**	
Pt Load from	F1	1.4	0.5	0.0	0.0	kips
	Total	1.4	0.5	0.0	0.0	kips
				U =	**2.5**	kips
Required Strength:		ASD:		P =	**1.9**	kips

$U = 1.2D + 1.6L$ — $P = D + L$

$U = 1.2D + 1.6L + 0.5W$ — $P = D + 0.6W$ — $P = D + 0.75[L+0.6W]$

$U = 1.2D + 1.6W + 0.5L$ — $P = D + 0.7E$ — $P = D + 0.75[L+0.7E]$

$U = 1.2D + 1.0E + 1.0L$

Conventional Footing Design Parameters:

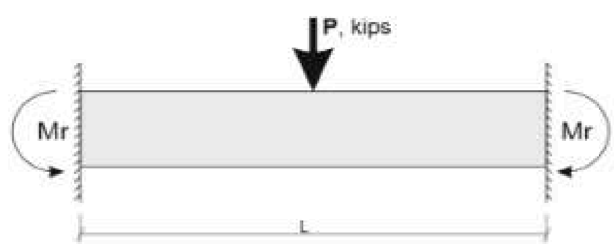
Allowable Bearing Pressure			2000	psf
Seismic or Wind Loading?	N	--->	2000	psf

Footing Width	18	inches
A_{req}	0.9	ft^2
L_{req}	**0.6**	ft

Model Foundation as Fixed/Fixed Member

Eqn. 1: $M_u = PL/8$

$M_u =$ **0.196** 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=	0.196	kip-ft	*(From Eqn. 1)*
ØMn/Mu =	**119.9**	OK	

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

WF2 - INTERIOR WALL FOOTING ANALYSIS & DESIGN

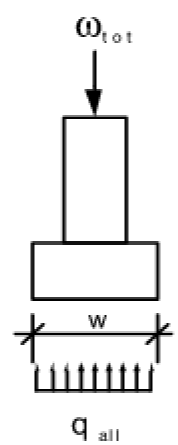
Footing **Exterior Wall Foundation** **WF2**

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	15	0	psf

Conventional Footing Design Parameters:

Allowable Bearing Pressure 2000 psf

FT	TRIB	Wdl	Wll
	(ft)	(plf)	(plf)
WF2	15.8 r	315	315
	10.3 f	155	413
	0.0 g	0	0
	0.0 d	0	0
	23.0 w	345	0
	TL	**815**	**728**



w_{tot} 1543 plf

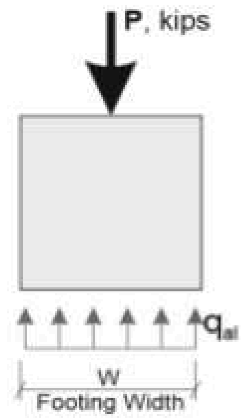
Try Footing Width, w = 16 inches

q_{bear} 1158 psf

q_{allow} 2000 psf

q_{allow}/q_{bear} 1.73 Footing OK

Footing **Exterior wall Foundation** **WF2**



Reactions on Footing		**D**	**L**	**E**	**W**	
Pt Load from	WF2	0.8	0.7	0.0	0.0	kips
	Total	0.8	0.7	0.0	0.0	kips
				U =	**2.1**	kips
				P =	**1.5**	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:

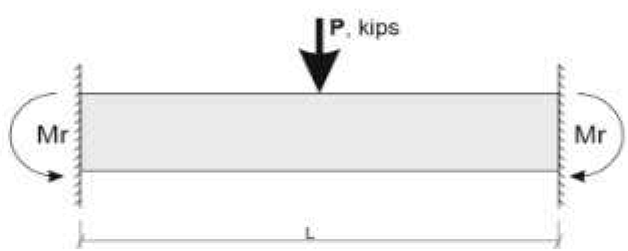
Allowable Bearing Pressure			2000	psf
Seismic or Wind Loading?	N	--->	2000	psf

Footing Width	18	inches
A_{req}	0.8	ft^2
L_{req}	**0.5**	ft

Model Foundation as Fixed/Fixed Member

Eqn. 1: $M_u = PL/8$

$M_u =$ **0.138** 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=	0.138	kip-ft	*(From Eqn. 1)*
øMn/Mu =	**170.3**	OK	

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

WF3 - INTERIOR WALL FOOTING ANALYSIS & DESIGN

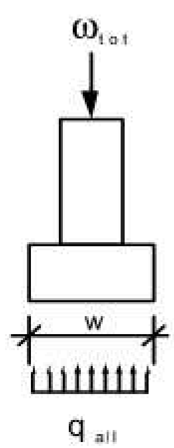
Footing **Exterior Wall Foundation** **WF3**

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	15	0	psf

Conventional Footing Design Parameters:

Allowable Bearing Pressure 2000 psf

FT	TRIB	Wdl	Wll
	(ft)	(plf)	(plf)
WF3	14.6 r	292	292
	9.3 f	139	370
	0.0 g	0	0
	0.0 d	0	0
	23.0 w	345	0
	TL	**775**	**662**



w_{tot} 1437 plf

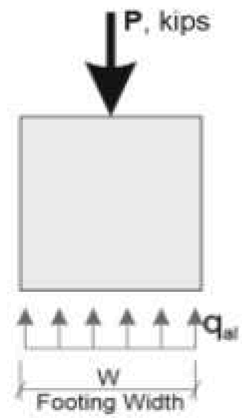
Try Footing Width, w = 16 inches

q_{bear} 1078 psf

q_{allow} 2000 psf

q_{allow}/q_{bear} 1.86 Footing OK

Footing **<u>Exterior wall Foundation</u>** **WF3**



Reactions on Footing		**D**	**L**	**E**	**W**	
Pt Load from	WF3	0.8	0.7	0.0	0.0	kips
	Total	0.8	0.7	0.0	0.0	kips
				U =	**2.0**	kips

Required Strength: ASD: P = **1.4** kips

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		

<u>Conventional Footing Design Parameters:</u>

Allowable Bearing Pressure 2000 psf

Seismic or Wind Loading? N ---> 2000 psf

Footing Width 18 inches

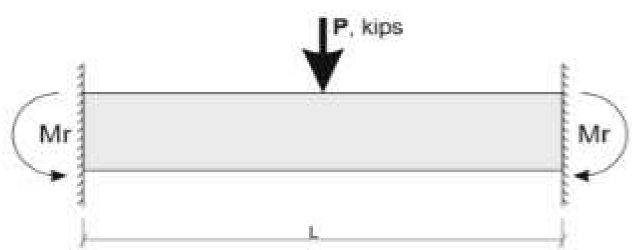
A_{req} 0.7 ft^2

L_{req} **0.5** ft

<u>Model Foundation as Fixed/Fixed Member</u>

Eqn. 1: $M_u = PL/8$

$M_u =$ **0.120** k-ft



<u>Steel Reinforcing Design:</u>

Steel Depth 9 in

Bar size (#) 5 bar

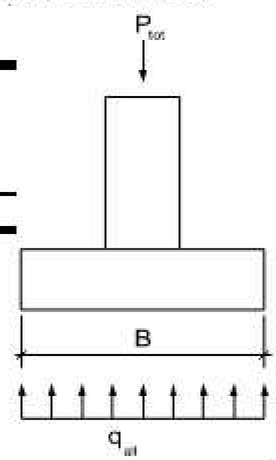
\# of bars 2

$\varnothing M_n$ =	23.5	kip-ft	*(Design Cap.)*
M_u=	0.120	kip-ft	*(From Eqn. 1)*
ØMn/Mu =	**195.3**	OK	

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

P1 - PAD FOOTING ANALYSIS & DESIGN

Pad Footing Calculation



Reactions on Footing		**D**	**L**	**E**	**W**	
Pt Load		12.47	9.12	--	0.00	kips
	SUM:	12.47	9.12	0.00	0.00	kips

Required Strength: U = **29.6** kips

U = 1.2D + 1.6L — ASD: — P = **21.6** kips

U = 1.2D + 1.6L + 0.5W — P = D + L

U = 1.2D + 1.6W + 0.5L — P = D + 0.6W — P = D + 0.75[L+0.6W]

U = 1.2D + 1.0E + 1.0L — P = D + 0.7E — P = D + 0.75[L+0.7E]

Pad Footing Design Parameters:

Allowable Bearing Pressure ⟶ 2000 psf

Seismic or Wind Loading? ⟶ N ⟶ 2000 psf

Footing Width Design:

Try Square Footing Width, B = 48 inches

q_{bear} = 1349.4 psf

q_{allow} = 2000.0 psf

q_{allow}/q_{bear} = 1.48

Reinforcement Design:

Mu = 0.14 kips - in

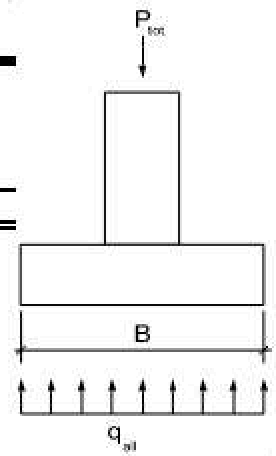
q_U = 0.01 psi

$A_{s\ Req'd}$ 0.000429 in^2

Reinforcement Req'd (4) #5 Bars Spaced Evenly (14" o.c)

P2 - PAD FOOTING ANALYSIS & DESIGN

Pad Footing Calculation



Reactions on Footing		D	L	E	W	
Pt Load		5.62	6.68	--	0.00	kips
	SUM:	5.62	6.68	0.00	0.00	kips

Required Strength: U = **17.4** kips

$U = 1.2D + 1.6L$ ASD: P = **12.3** kips

$U = 1.2D + 1.6L + 0.5W$ $P = D + L$

$U = 1.2D + 1.6W + 0.5L$ $P = D + 0.6W$ $P = D + 0.75[L+0.6W]$

$U = 1.2D + 1.0E + 1.0L$ $P = D + 0.7E$ $P = D + 0.75[L+0.7E]$

Pad Footing Design Parameters:

Allowable Bearing Pressure ⟶ 2000 psf

Seismic or Wind Loading? ⟶ N ⟶ 2000 psf

Footing Width Design:

Try Square Footing Width, B = 36 inches

q_{bear} = 1366.7 psf

q_{allow} = 2000.0 psf

q_{allow}/q_{bear} = 1.46

Reinforcement Design:

Mu = 0.06 kips - in

q_U = 0.01 psi

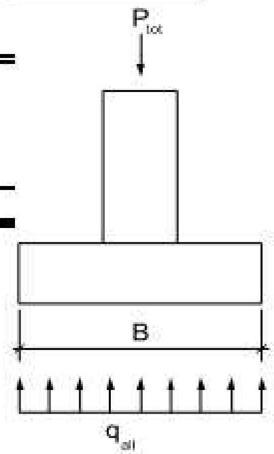
$A_{s\ Req'd}$ 0.000174 in^2

Reinforcement Req'd (3) #5 Bars Spaced Evenly (15" o.c)

P3 - PAD FOOTING ANALYSIS & DESIGN

Pad Footing Calculation

Reactions on Footing		D	L	E	W	
Pt Load		5.07	3.00	--	0.00	kips
	SUM:	5.07	3.00	0.00	0.00	kips



Required Strength:		U =	10.9	kips
U = 1.2D + 1.6L	ASD:	P =	8.1	kips
U = 1.2D + 1.6L + 0.5W	P = D + L			
U = 1.2D + 1.6W + 0.5L	P = D + 0.6W	P = D + 0.75[L+0.6W]		
U = 1.2D + 1.0E + 1.0L	P = D + 0.7E	P = D + 0.75[L+0.7E]		

Pad Footing Design Parameters:

Allowable Bearing Pressure ⟶ 2000 psf

Seismic or Wind Loading? ⟶ N ⟶ 2000 psf

Footing Width Design:

Try Square Footing Width, B = 30 inches

q_{bear} = 1291.2 psf

q_{allow} = 2000.0 psf

q_{allow}/q_{bear} = 1.55

Reinforcement Design:

Mu = 0.03 kips - in

q_U = 0.01 psi

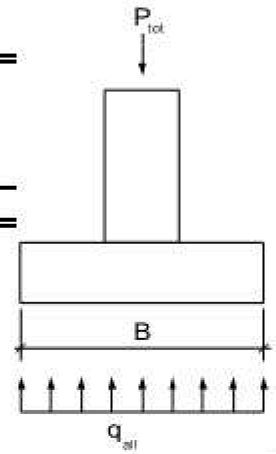
$A_{s\ Req'd}$ 0.000084 in^2

Reinforcement Req'd (3) #5 Bars Spaced Evenly (12" o.c)

P4 - PAD FOOTING ANALYSIS & DESIGN

Pad Footing Calculation

Reactions on Footing		D	L	E	W	
Pt Load		7.23	4.86	--	0.00	kips
	SUM:	7.23	4.86	0.00	0.00	kips



Required Strength:		U =	**16.5**	kips
U = 1.2D + 1.6L	ASD:	P =	**12.1**	kips
U = 1.2D + 1.6L + 0.5W	P = D + L			
U = 1.2D + 1.6W + 0.5L	P = D + 0.6W	P = D + 0.75[L+0.6W]		
U = 1.2D + 1.0E + 1.0L	P = D + 0.7E	P = D + 0.75[L+0.7E]		

Pad Footing Design Parameters:

Allowable Bearing Pressure ⟶ 2000 psf

Seismic or Wind Loading? ⟶ N ⟶ 2000 psf

Footing Width Design:

Try Square Footing Width, B = 30 inches

q_{bear} = 1934.4 psf

q_{allow} = 2000.0 psf

q_{allow}/q_{bear} = 1.03

Reinforcement Design:

Mu = 0.04 kips - in

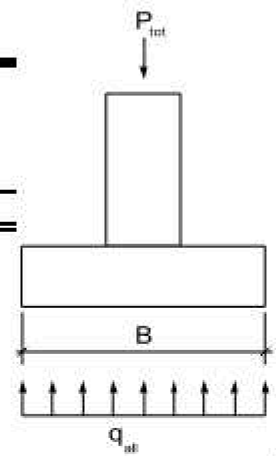
q_U = 0.02 psi

$A_{s\ Req'd}$ 0.000127 in^2

Reinforcement Req'd (3) #5 Bars Spaced Evenly (12" o.c)

P5 - PAD FOOTING ANALYSIS & DESIGN

Pad Footing Calculation



Reactions on Footing		**D**	**L**	**E**	**W**	
Pt Load		7.55	4.30	--	0.00	kips
	SUM:	7.55	4.30	0.00	0.00	kips

Required Strength: U = **15.9** kips

ASD: P = **11.9** kips

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

Pad Footing Design Parameters:

Allowable Bearing Pressure ⟶ 2000 psf

Seismic or Wind Loading? ⟶ N ⟶ 2000 psf

Footing Width Design:

Try Square Footing Width, B = 36 inches

q_{bear} = 1316.7 psf

q_{allow} = 2000.0 psf

q_{allow}/q_{bear} = 1.52

Reinforcement Design:

Mu = 0.05 kips - in

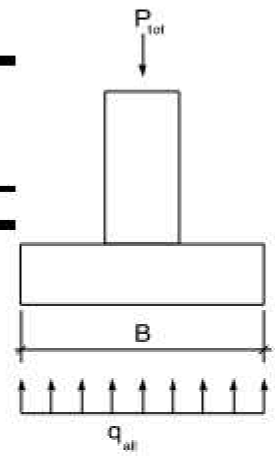
q_U = 0.01 psi

$A_{s\ Req'd}$ 0.000159 in^2

Reinforcement Req'd (3) #5 Bars Spaced Evenly (15" o.c)

P6 - PAD FOOTING ANALYSIS & DESIGN

Reactions on Footing		D	L	E	W	
Pt Load		3.00	0.90	--	0.00	kips
	SUM:	3.00	0.90	0.00	0.00	kips



Required Strength:			U =	**5.0**	kips
U = 1.2D + 1.6L	ASD:		P =	**3.9**	kips
U = 1.2D + 1.6L + 0.5W	P = D + L				
U = 1.2D + 1.6W + 0.5L	P = D + 0.6W	P = D + 0.75[L+0.6W]			
U = 1.2D + 1.0E + 1.0L	P = D + 0.7E	P = D + 0.75[L+0.7E]			

Pad Footing Design Parameters:

Allowable Bearing Pressure ⟶ 2000 psf

Seismic or Wind Loading? ⟶ N ⟶ 2000 psf

Footing Width Design:

Try Square Footing Width, B = 24 inches

q_{bear} = 975.0 psf

q_{allow} = 2000.0 psf

q_{allow}/q_{bear} = 2.05

Reinforcement Design:

Mu = 0.01 kips - in

q_U = 0.01 psi

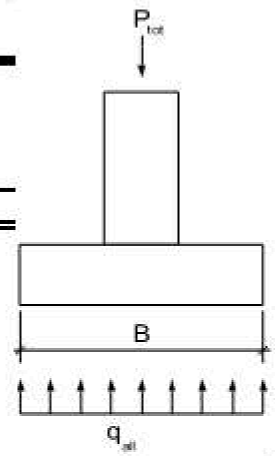
$A_{s\ Req'd}$ 0.000028 in^2

Reinforcement Req'd (2) #5 Bars Spaced Evenly (18" o.c)

P7 - PAD FOOTING ANALYSIS & DESIGN

Pad Footing Calculation

Reactions on Footing		**D**	**L**	**E**	**W**	
Pt Load		19.70	12.40	--	0.00	kips
	SUM:	19.70	12.40	0.00	0.00	kips



Required Strength: U = **43.5** kips

U = 1.2D + 1.6L	ASD:		P =	**32.1** kips
U = 1.2D + 1.6L + 0.5W	P = D + L			
U = 1.2D + 1.6W + 0.5L	P = D + 0.6W	P = D + 0.75[L+0.6W]		
U = 1.2D + 1.0E + 1.0L	P = D + 0.7E	P = D + 0.75[L+0.7E]		

Pad Footing Design Parameters:

Allowable Bearing Pressure ⟶ 2000 psf

Seismic or Wind Loading? ⟶ N ⟶ 2000 psf

Footing Width Design:

Try Square Footing Width, B = 54 inches

q_{bear} = 1585.2 psf

q_{allow} = 2000.0 psf

q_{allow}/q_{bear} = 1.26

Reinforcement Design:

Mu = 0.24 kips - in

q_U = 0.01 psi

$A_{s\ Req'd}$ 0.000731 in^2

Reinforcement Req'd (4) #5 Bars Spaced Evenly (16" o.c)

P8 - PAD FOOTING ANALYSIS & DESIGN

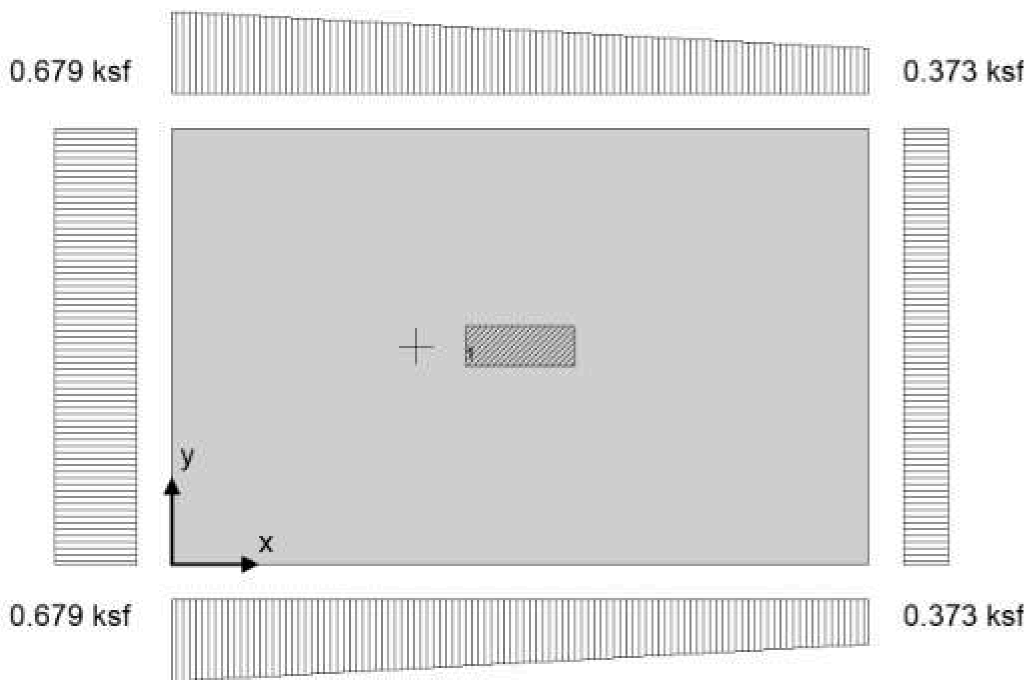
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	L_x = **8** ft
Width of foundation	L_y = **5** ft
Foundation area	$A = L_x \times L_y$ = **40** ft^2
Depth of foundation	h = **12** in
Depth of soil over foundation	h_{soil} = **16** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **15.00** in
Width of column	l_{y1} = **5.50** in
position in x-axis	x_1 = **48.00** in
position in y-axis	y_1 = **30.00** 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	$F_{swt} = h \times \gamma_{conc}$ = **150** psf
Soil weight	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **160** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **6.4** kips
Live load in z	F_{Lz1} = **2.4** kips
Live roof load in z	F_{Lrz1} = **1.7** kips
Wind load in z	F_{Wz1} = **-1.7** kips
Wind load moment in x	M_{Wx1} = **-18.1** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.235)
1.0D + 1.0L (0.264)
1.0D + 1.0Lr (0.256)
1.0D + 1.0S (0.235)
1.0D + 1.0R (0.235)
1.0D + 0.75L + 0.75Lr (0.273)
1.0D + 0.75L + 0.75S (0.257)
1.0D + 0.75L + 0.75R (0.257)
1.0D + 0.6W (0.324)
1.0D + 0.75L + 0.75Lr + 0.45W (0.340)
1.0D + 0.75L + 0.75S + 0.45W (0.324)
1.0D + 0.75L + 0.75R + 0.45W (0.324)
0.6D + 0.6W (0.333)

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.235)
1.0D + 1.0L (0.264)
1.0D + 1.0Lr (0.256)
1.0D + 1.0S (0.235)
1.0D + 1.0R (0.235)
1.0D + 0.75L + 0.75Lr (0.273)
1.0D + 0.75L + 0.75S (0.257)
1.0D + 0.75L + 0.75R (0.257)
1.0D + 0.6W (0.324)
1.0D + 0.75L + 0.75Lr + 0.45W (0.340)
1.0D + 0.75L + 0.75S + 0.45W (0.324)
1.0D + 0.75L + 0.75R + 0.45W (0.324)
0.6D + 0.6W (0.333)

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

Forces on foundation

Force in z-axis

$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1}$ = **21.0** kips

Moments on foundation

Moment in x-axis, about x is 0

$M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + M_{Wx1})$ = **76.0** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **52.6** kip_ft

Uplift verification

Vertical force F_{dz} = **21.048** kips

PASS - Foundation is not subject to uplift

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

Overturning moment $M_{OTx0} = \gamma_W \times (F_{Wz1} \times x_1 + M_{Wx1})$ = **-11.26** kip_ft

Resisting moment $M_{Rx0} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1)$ = **87.29** kip_ft

Factor of safety $abs(M_{Rx0} / M_{OTx0})$ = **7.750**

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 $M_{OTxL} = \gamma_W \times (F_{Wz1} \times (x_1 - L_x))$ = **3.1** kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x)) + \gamma_L \times (F_{Lz1} \times (x_1 - L_x)) + \gamma_{Lr} \times (F_{Lrz1} \times (x_1 - L_x)) + \gamma_W \times (M_{Wx1})$ = **-95.46** kip_ft

Factor of safety $abs(M_{RxL} / M_{OTxL})$ = **30.833**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

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

Overturning moment $M_{OTy0} = \gamma_W \times (F_{Wz1} \times y_1)$ = **-1.93** kip_ft

Resisting moment $M_{Ry0} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1)$ = **54.56** kip_ft

Factor of safety $abs(M_{Ry0} / M_{OTy0})$ = **28.194**

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 $M_{OTyL} = \gamma_W \times (F_{Wz1} \times (y_1 - L_y))$ = **1.93** kip_ft

Resisting moment $M_{RyL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2)) + \gamma_D \times (F_{Dz1} \times (y_1 - L_y)) + \gamma_L \times (F_{Lz1} \times (y_1 - L_y)) + \gamma_{Lr} \times (F_{Lrz1} \times (y_1 - L_y))$ = **-54.56** kip_ft

Factor of safety $abs(M_{RyL} / M_{OTyL})$ = **28.194**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **-4.656** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.679** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.679** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.373}$ ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.373}$ ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{0.373}$ ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{0.679}$ ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} = \mathbf{2}$ ksf

$q_{max} / q_{allow} = \mathbf{0.340}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete $f'_c = \mathbf{4000}$ 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 $\lambda = \mathbf{1.00}$

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.081)
1.2D + 1.6L + 0.5Lr (0.112)
1.2D + 1.6L + 0.5S (0.104)
1.2D + 1.6L + 0.5R (0.104)
1.2D + 1.0L + 1.6Lr (0.116)
1.2D + 1.0L + 1.6S (0.091)
1.2D + 1.0L + 1.6R (0.091)
1.2D + 1.6Lr + 0.5W (0.086)
1.2D + 1.6S + 0.5W (0.062)
1.2D + 1.6R + 0.5W (0.062)
1.2D + 1.0L + 0.5Lr + 1.0W (0.083)
1.2D + 1.0L + 0.5S + 1.0W (0.075)
1.2D + 1.0L + 0.5R + 1.0W (0.075)
0.9D + 1.0W (0.036)

Combination 11 results: 1.2D + 1.0L + 0.5Lr + 1.0W

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} = \mathbf{24.0}$ kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + M_{Wx1})$ = **78.0** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **60.1** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **-9.064** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.941** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.941** ksf

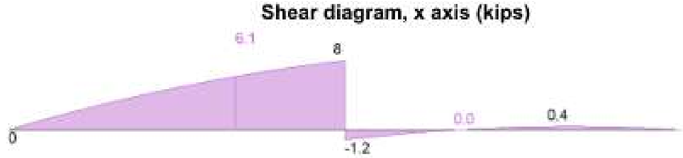
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.26** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.26** ksf

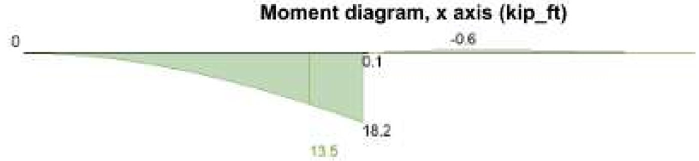
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **0.26** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **0.941** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max}$ = **13.479** kip_ft

Tension reinforcement provided — 5 No.5 bottom bars (13.3 in c/c)

Area of tension reinforcement provided — $A_{sx.bot.prov}$ = **1.55** in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_y \times h$ = **1.296** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18\text{ in})$ = **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 — $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.456** in

Neutral axis factor — β_1 = **0.85**

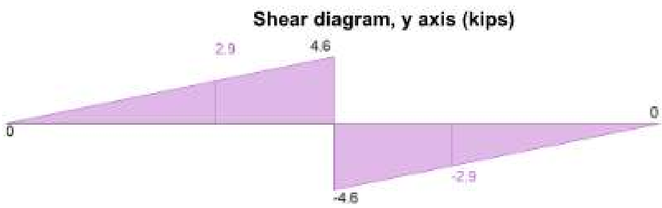
Depth to neutral axis	$c = a / \beta_1 =$ **0.536** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.04559**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **65.562** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n =$ **59.005** kip_ft
	$M_{u.x.max} / \phi M_n =$ **0.228**
	PASS - Design moment capacity exceeds ultimate moment load

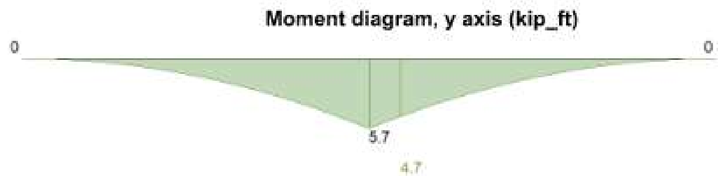
Moment design, x direction, negative moment

Ultimate bending moment	$M_{u.x.min} =$ **-0.639** kip_ft
Tension reinforcement provided	5 No.5 top bars (13.3 in c/c)
Area of tension reinforcement provided	$A_{sx.top.prov} =$ **1.55** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_y \times h =$ **1.296** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 \times h, 18 \text{ in}) =$ **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{x.top} / 2 =$ **8.688** in
Depth of compression block	$a = A_{sx.top.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.456** in
Neutral axis factor	$\beta_1 =$ **0.85**
Depth to neutral axis	$c = a / \beta_1 =$ **0.536** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.04559**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sx.top.prov} \times f_y \times (d - a / 2) =$ **65.562** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n =$ **59.005** kip_ft
	$\text{abs}(M_{u.x.min}) / \phi M_n =$ **0.011**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force	$V_{u.x} =$ **6.137** kips
Depth to reinforcement	$d_v = \min(h - c_{nom} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2) =$ **8.688** in
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v =$ **65.933** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n =$ **49.45** kips
	$V_{u.x} / \phi V_n =$ **0.124**
	PASS - Design shear capacity exceeds ultimate shear load





Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **4.718** kip_ft
Tension reinforcement provided	7 No.5 bottom bars (14.8 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_x \times h$ = **2.074** 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	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.399** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.469** in
Strain in tensile reinforcement (8.3.3.1)	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04854**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **85.314** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **76.783** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.061**

PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor (13.3.3.3)	$\beta_f = L_x / L_y$ = **1.600**
Area of reinf. req. for uniform distribution (CRSI)	$A_{sreq} = (M_{u.y.max} / (\phi_f \times f_y \times (d - a / 2))) \times 2 \times \beta_f / (\beta_f + 1)$ = **0.164** in^2

PASS - Reinforcement can be distributed uniformly

One-way shear design, y direction

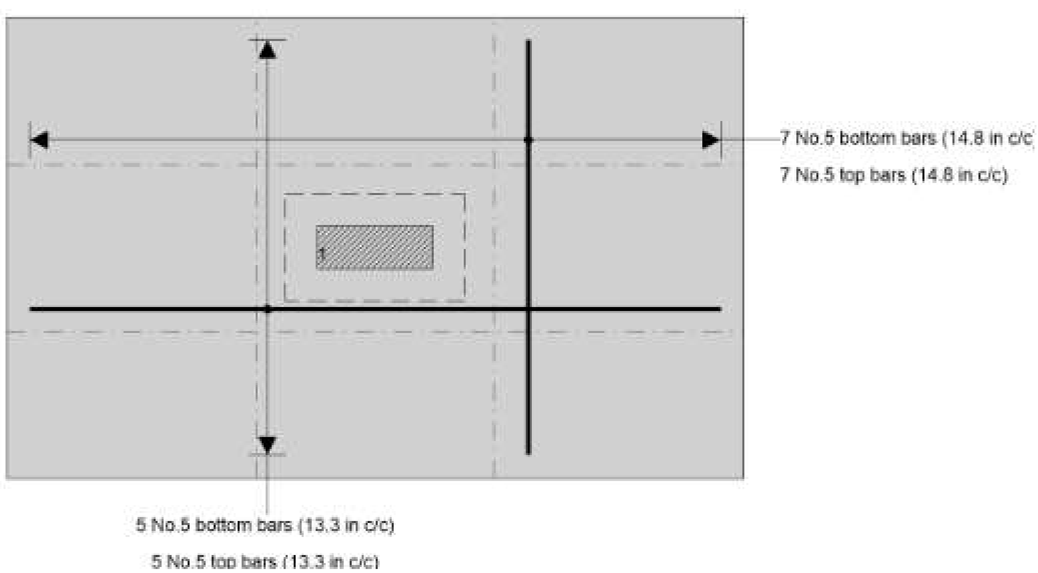
Ultimate shear force	$V_{u.y}$ = **2.926** kips
Depth to reinforcement	$d_v = \min(h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom} - \phi_{y.top} / 2)$ = **8.062** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v$ = **97.904** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **73.428** kips
	$V_{u.y} / \phi V_n$ = **0.040**

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} = **23.375** in
Shear perimeter width (22.6.4)	l_{yp} = **13.875** in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **74.500** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim}$ = **324.328** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1}$ = **241.828** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **0.601** ksf
Ultimate shear load	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **8.525** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi})$ = **13.663** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{x1} / l_{y1}$ = **2.73**
Column location factor (22.6.5.3)	α_s =**40**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **219.251** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **410.884** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **252.982** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **219.251** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **219.251** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n$ = **164.438** psi
	$v_{ug} / \phi v_n$ = **0.083**

PASS - Design shear stress capacity exceeds ultimate shear stress load



3.2 SLAB ON GRADE REINFORCEMENT ANALYSIS

For Grade 60 reinforcing bars:

As = F L w / 2 fs where F (friction factor) = 1.5 (commonly used value), L = 30 ft, w = 40 psf and

fs = 2/3 fy where fy is 65,000 psi.

As = 0.024 sq.in./ft of slab width, required each way.

Use W1.5 (ASTM A185) wire at 6-in, spacings in each direction, designated as: 6 x 6 - W1.5 x W1.5

As = 2 x 0.015 sq.in./ft = 2 x 0.030 sq.in./ft.