STRUCTURAL ANALYSIS OF
PROPOSED RESIDENTIAL HOME

08JUN2023

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1 DESIGN CRITERIA:

1.1 DESIGN CODES & REFERENCES

- 2021 Edition of the International Building Code
- 2022 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)
- TMS 402/602-16 Building Code Requirements and Specification for Masonry Structures

Calculation performed by SCIA Engineer 20.0

1.2 CODE CRITERIA

2021 IBC

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

1.3 MATERIALS

Hot Rolled Steel Grade

- A53 Gr.B

Cold Formed Steel Grade

- ASTM F1008 ST GRADE 33, TYPE H 33-43 mil GALV. STEEL
- ASTM F1008 ST GRADE 50, TYPE H 54-97 mil GALV. STEEL

1.4 STRUCTURAL LOADS

Floor Live Load	**40.00 psf**
Floor Dead Loads	
Finish	5 psf
Sheathing	2.2 psf
Floor Joists	2.7 psf
Insulation	1.05 psf
Add. flooring	3 psf
Misc.	4 psf
Total Floor Dead Load	**17.25 psf**
Roof Live Load	**20.00 psf**
Roof Dead Loads	
Standing Seam Metal	1.5 psf
Sheathing	1.5 psf
Framing	2.7 psf
Insulation	1.05 psf
Solar panels	4 psf
Misc.	2 psf
Total roof Dead Load	**12.75 psf**
AAC Wall Dead Loads	
Plaster 1/4”	0.6 psf
AAC Masonry Walls 8”	33.4 psf
Finish U-Stucco 1/2” thk	1.2 psf
Total wall Dead Load	**35.2 psf**

CFS Wall Dead Loads

Sheathing	2.5 psf
Framing	2.2 psf
Insulation	1.05 psf
Drywall	2.5 psf
Misc.	2 psf
Total wall Dead Load	**10.25 psf**

LOAD SUMMARY

	Dead load (DL), psf	Live Load (LL), psf
Roof	12.75	20
Floor	17.25	40
Wall	10.25 / 35.2	

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

Risk Category

Risk Category II

Project Address

3660 San Ysidro Way, Sacramento, CA 958

Wind and Snow Data

Basic Wind Speed

110 mph

Site Elevation

77.95 ft

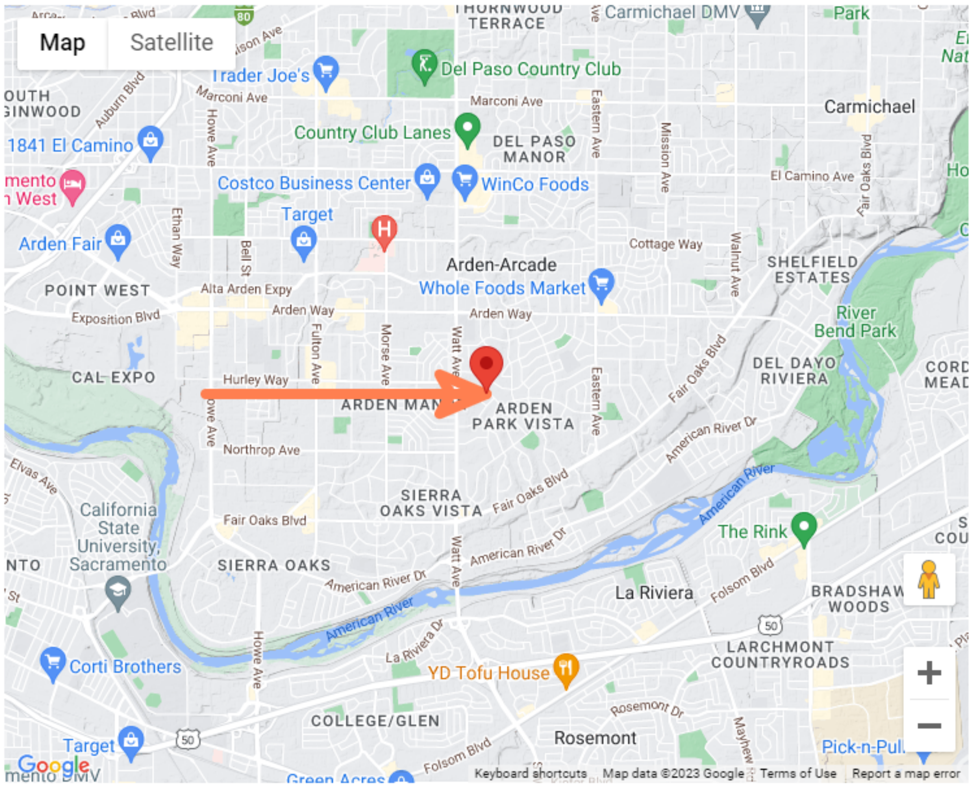
Ground Snow Load

0 psf

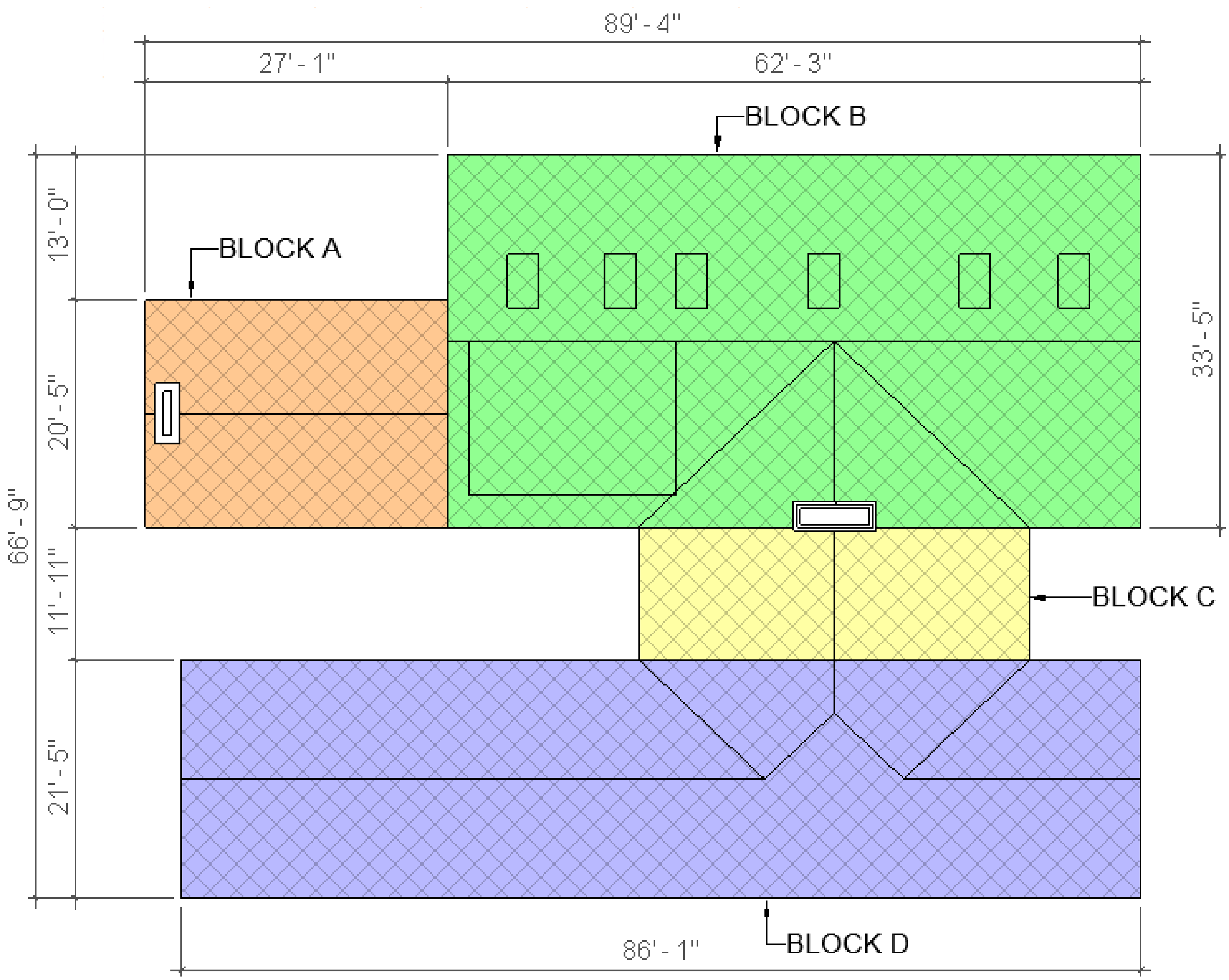
Exposure Category

C

View Map Contours



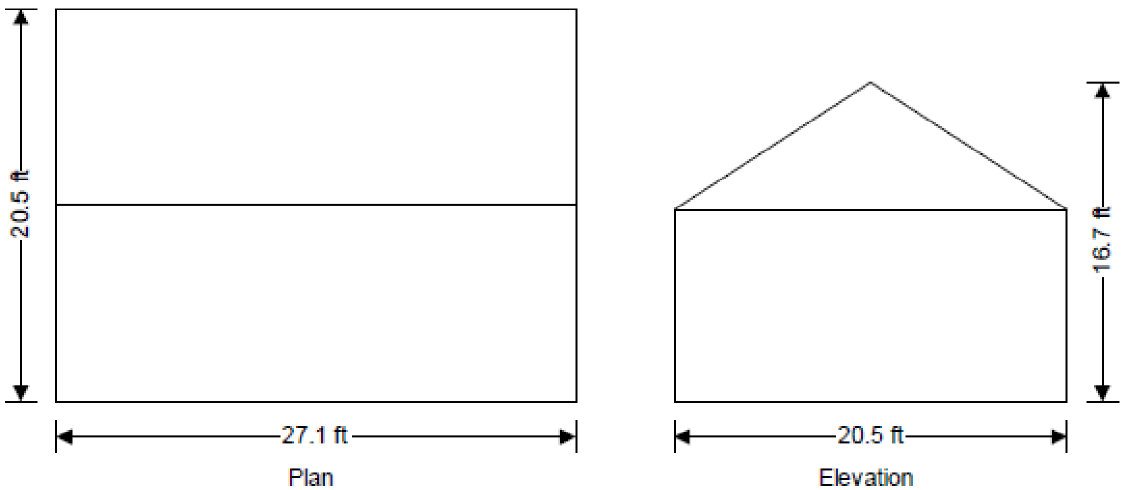
Wind load general scheme



Wind load on block A

In accordance with ASCE7-16

Using the directional design method



Building data

Type of roof	Gable
Length of building	b = **27.10** ft
Width of building	d = **20.50** ft
Height to eaves	H = **10.00** ft
Pitch of roof	α_0 = **33.0** deg
Mean height	h = **13.33** ft

General wind load requirements

Basic wind speed	V = **110.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**

Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.4.7)	p_{min_r} = **8** lb/ft^2

Topography

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

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
10.00	0.57	15.01
13.33	0.57	15.01
15.00	0.57	15.01
16.66	0.59	15.44

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **15.01** 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)	13.33	-0.21	15.01	-5.41	331.21	-1.79
B (-ve)	13.33	-0.60	15.01	-10.36	331.21	-3.43

Total vertical net force $F_{w,v}$ = **-4.38** kips

Total horizontal net force $F_{w,h}$ = **0.89** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	15.01	7.50	271.00	2.03
B	13.33	-0.50	15.01	-9.08	271.00	-2.46
C	13.33	-0.70	15.01	-11.63	273.23	-3.18
D	13.33	-0.70	15.01	-11.63	273.23	-3.18

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H = \mathbf{271.00}$ ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) = \mathbf{180.39}$ ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{5.78}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-2.5}$ kips

Windward net force $F_w = F_{w,wA} = \mathbf{2.0}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{5.8}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	13.33	0.24	15.01	5.79	331.21	1.92
B (+ve)	13.33	-0.60	15.01	-4.95	331.21	-1.64

Total vertical net force $F_{w,v} = \mathbf{0.23}$ kips

Total horizontal net force $F_{w,h} = \mathbf{1.94}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	15.01	12.91	271.00	3.50
B	13.33	-0.50	15.01	-3.68	271.00	-1.00

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)
C	13.33	-0.70	15.01	-6.23	273.23	-1.70
D	13.33	-0.70	15.01	-6.23	273.23	-1.70

Overall loading

Projected vertical plan area of wall — $A_{vert\_w\_0} = b \times H = \mathbf{271.00}$ ft^2

Projected vertical area of roof — $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) = \mathbf{180.39}$ ft^2

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{5.78}$ kips

Leeward net force — $F_l = F_{w,wB} = \mathbf{-1.0}$ kips

Windward net force — $F_w = F_{w,wA} = \mathbf{3.5}$ kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{6.4}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	13.33	-0.90	15.01	-14.18	162.89	-2.31
B (-ve)	13.33	-0.90	15.01	-14.18	162.89	-2.31
C (-ve)	13.33	-0.50	15.01	-9.08	325.79	-2.96
D (-ve)	13.33	-0.30	15.01	-6.53	10.84	-0.07

Total vertical net force — $F_{w,v} = \mathbf{-6.42}$ kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	15.01	7.50	269.00	2.02
A_2	16.66	0.80	15.44	7.80	4.23	0.03
B	13.33	-0.44	15.01	-8.26	273.23	-2.26
C	13.33	-0.70	15.01	-11.63	271.00	-3.15
D	13.33	-0.70	15.01	-11.63	271.00	-3.15

Overall loading

Projected vertical plan area of wall	$A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 =$ **273.23** ft^2
Projected vertical area of roof	$A_{vert\_r\_90} =$ **0.00** ft^2
Minimum overall horizontal loading	$F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **4.37** kips
Leeward net force	$F_l = F_{w,wB} =$ **-2.3** kips
Windward net force	$F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **2.1** kips
Overall horizontal loading	$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **4.4** 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)	13.33	-0.18	15.01	0.41	162.89	0.07
B (+ve)	13.33	-0.18	15.01	0.41	162.89	0.07
C (+ve)	13.33	-0.18	15.01	0.41	325.79	0.13
D (+ve)	13.33	-0.18	15.01	0.41	10.84	0.00

Total vertical net force	$F_{w,v} =$ **0.23** kips
Total horizontal net force	$F_{w,h} =$ **0.00** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	15.01	12.91	269.00	3.47
A_2	16.66	0.80	15.44	13.20	4.23	0.06
B	13.33	-0.44	15.01	-2.86	273.23	-0.78
C	13.33	-0.70	15.01	-6.23	271.00	-1.69
D	13.33	-0.70	15.01	-6.23	271.00	-1.69

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 =$ **273.23** ft^2

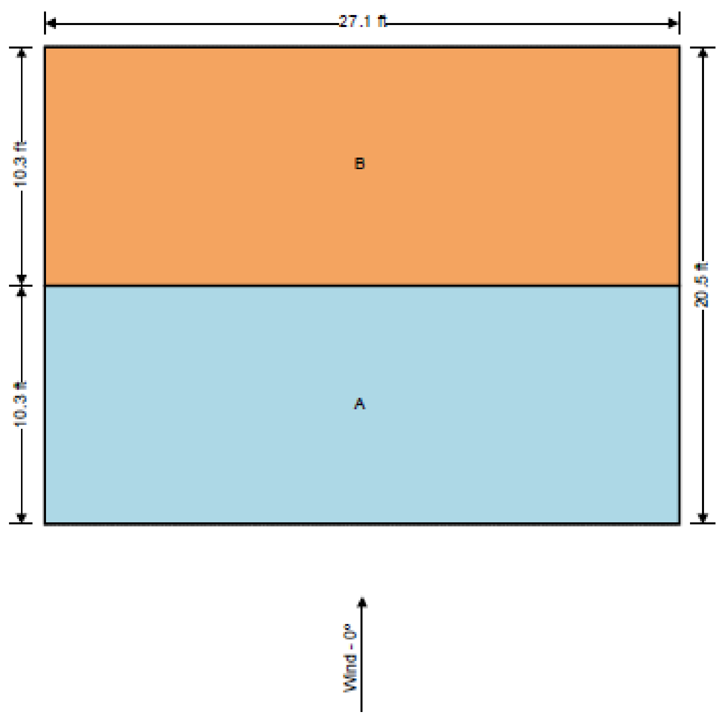
Projected vertical area of roof $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **4.37** kips

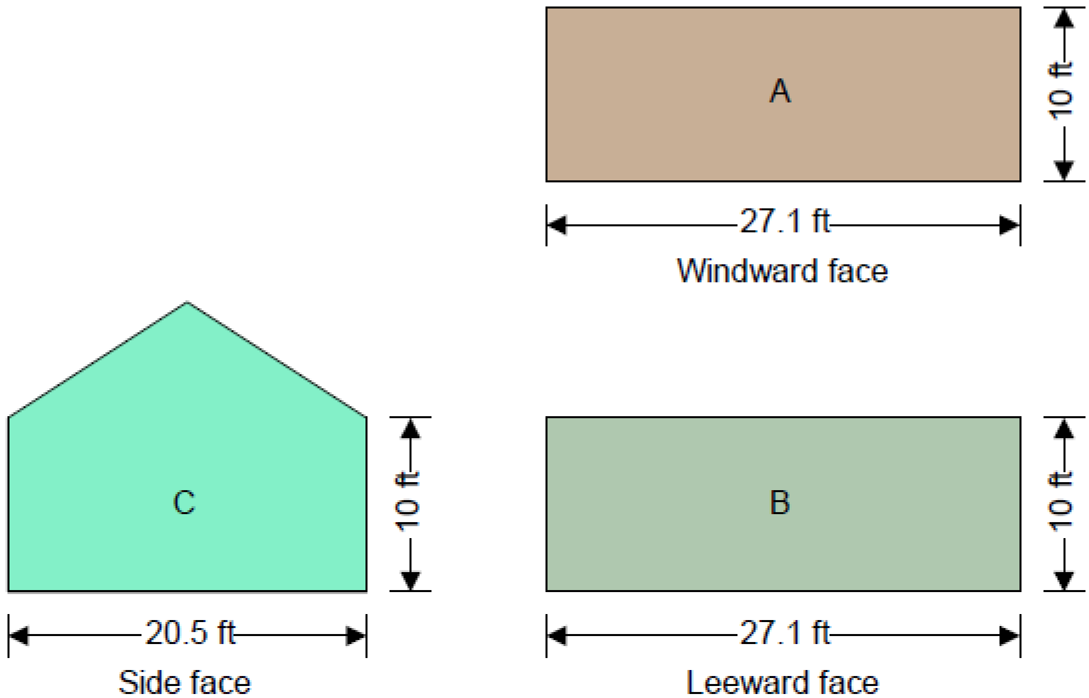
Leeward net force $F_l = F_{w,wB} =$ **-0.8** kips

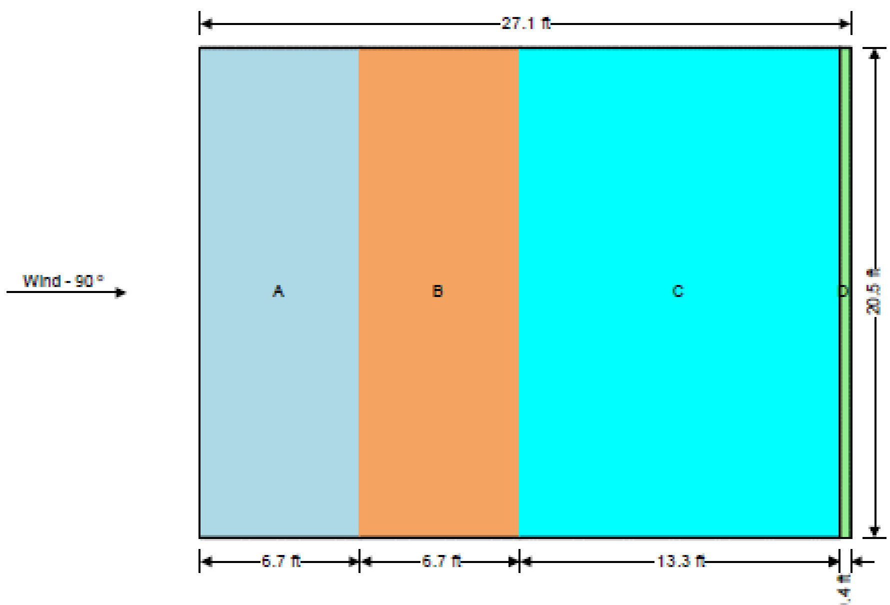
Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **3.5** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **4.4** kips

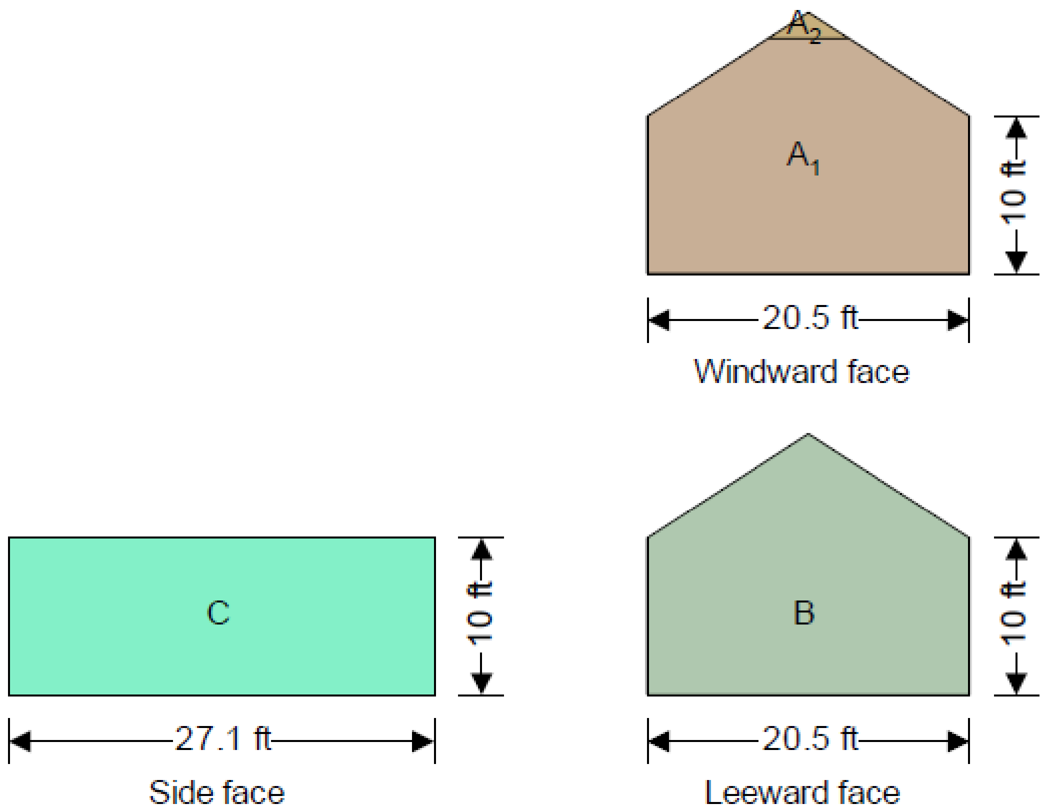


Plan view - Gable roof





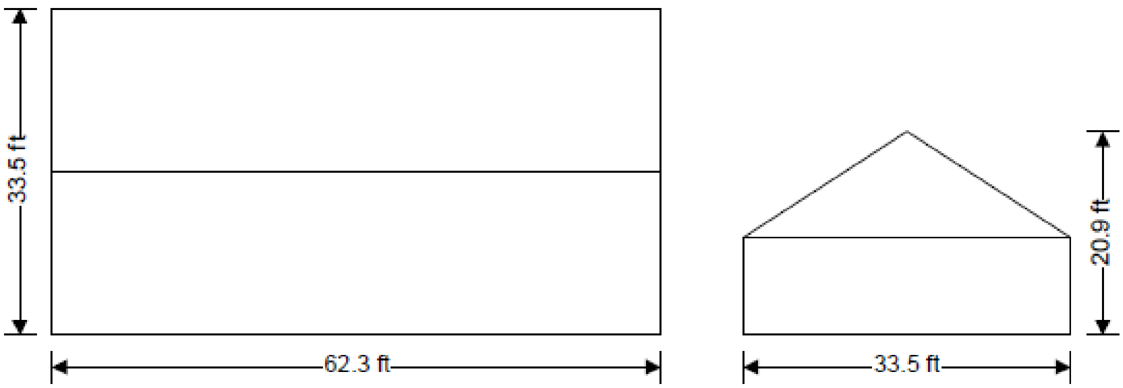
Plan view - Gable roof


A2
A1
10 ft
20.5 ft
Windward face
C
10 ft
27.1 ft
Side face
B
10 ft
20.5 ft
Leeward face

Wind load on block B

In accordance with ASCE7-16

Using the directional design method



Plan   Elevation

Building data

Type of roof	Gable
Length of building	b = **62.25** ft
Width of building	d = **33.50** ft
Height to eaves	H = **10.00** ft
Pitch of roof	α_0 = **33.0** deg
Mean height	h = **15.44** ft

General wind load requirements

Basic wind speed	V = **110.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**

Gust effect factor G_f = **0.85**

Minimum design wind loading (cl.27.4.7) p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant K_{zt} = 1.0

Velocity pressure equation $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
10.00	0.85	22.38
15.00	0.85	22.38
15.00	0.85	22.38
15.44	0.85	22.50
20.88	0.91	23.88

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **22.50** 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)	15.44	-0.18	22.50	-7.51	1243.26	-9.34
B (-ve)	15.44	-0.60	22.50	-15.52	1243.26	-19.30

Total vertical net force $F_{w,v}$ = **-24.02** kips

Total horizontal net force $F_{w,h}$ = **5.42** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	22.38	11.17	622.50	6.95
B	15.44	-0.50	22.50	-13.61	622.50	-8.47
C	15.44	-0.70	22.50	-17.43	517.20	-9.02
D	15.44	-0.70	22.50	-17.43	517.20	-9.02

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **622.50** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) =$ **677.13** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **15.38** kips

Leeward net force $F_l = F_{w,wB} =$ **-8.5** kips

Windward net force $F_w = F_{w,wA} =$ **7.0** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **20.8** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	15.44	0.28	22.50	9.32	1243.26	11.59
B (+ve)	15.44	-0.60	22.50	-7.42	1243.26	-9.23

Total vertical net force $F_{w,v} =$ **1.98** kips

Total horizontal net force $F_{w,h} =$ **11.34** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	22.38	19.27	622.50	11.99
B	15.44	-0.50	22.50	-5.51	622.50	-3.43

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)
C	15.44	-0.70	22.50	-9.34	517.20	-4.83
D	15.44	-0.70	22.50	-9.34	517.20	-4.83

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **622.50** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) =$ **677.13** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **15.38** kips

Leeward net force $F_l = F_{w,wB} =$ **-3.4** kips

Windward net force $F_w = F_{w,wA} =$ **12.0** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **26.8** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	15.44	-0.90	22.50	-21.26	308.34	-6.55
B (-ve)	15.44	-0.90	22.50	-21.26	308.34	-6.55
C (-ve)	15.44	-0.50	22.50	-13.61	616.69	-8.39
D (-ve)	15.44	-0.30	22.50	-9.79	1253.15	-12.26

Total vertical net force $F_{w,v} =$ **-28.32** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	22.38	11.17	464.00	5.18
A_2	15.00	0.80	22.38	11.17	0.00	0.00
A_3	20.88	0.80	23.88	12.19	53.22	0.65
B	15.44	-0.33	22.50	-10.33	517.20	-5.34
C	15.44	-0.70	22.50	-17.43	622.50	-10.85
D	15.44	-0.70	22.50	-17.43	622.50	-10.85

Overall loading

Projected vertical plan area of wall	$A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 =$ **517.20** ft^2
Projected vertical area of roof	$A_{vert\_r\_90} =$ **0.00** ft^2
Minimum overall horizontal loading	$F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **8.28** kips
Leeward net force	$F_l = F_{w,wB} =$ **-5.3** kips
Windward net force	$F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **5.8** kips
Overall horizontal loading	$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **11.2** 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)	15.44	-0.18	22.50	0.61	308.34	0.19
B (+ve)	15.44	-0.18	22.50	0.61	308.34	0.19
C (+ve)	15.44	-0.18	22.50	0.61	616.69	0.37
D (+ve)	15.44	-0.18	22.50	0.61	1253.15	0.76

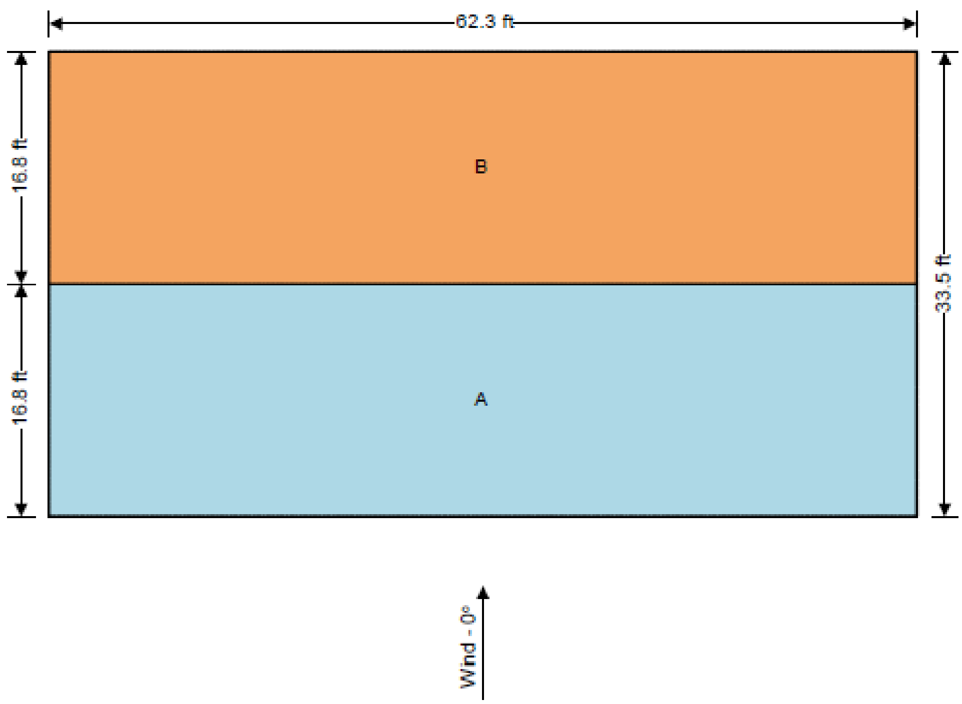
Total vertical net force	$F_{w,v} =$ **1.27** kips
Total horizontal net force	$F_{w,h} =$ **0.00** kips

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

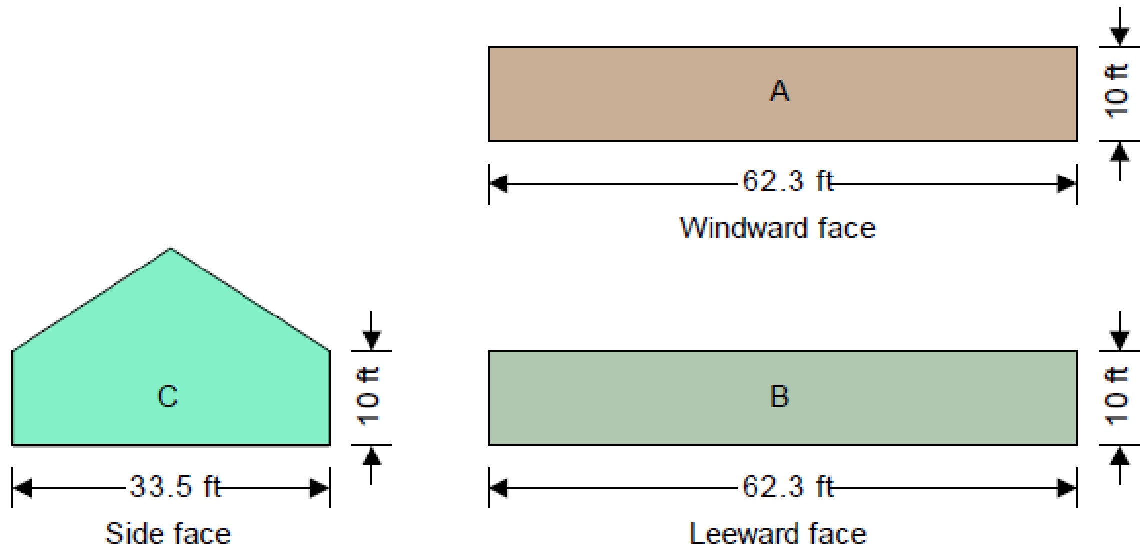
Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	22.38	19.27	464.00	8.94
A_2	15.00	0.80	22.38	19.27	0.00	0.00
A_3	20.88	0.80	23.88	20.29	53.22	1.08
B	15.44	-0.33	22.50	-2.23	517.20	-1.15
C	15.44	-0.70	22.50	-9.34	622.50	-5.81
D	15.44	-0.70	22.50	-9.34	622.50	-5.81

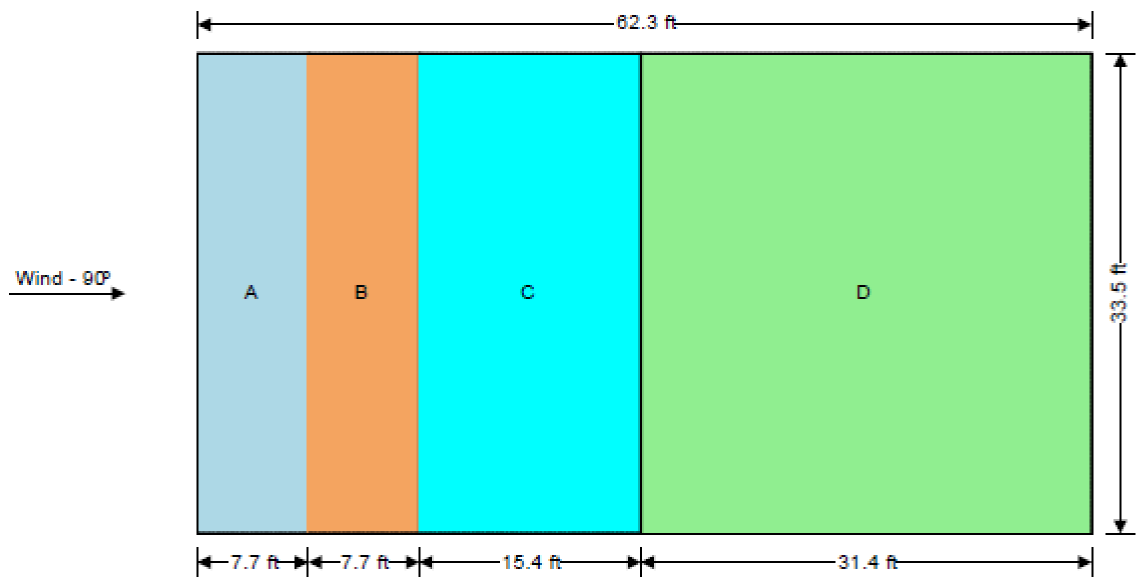
Overall loading

Projected vertical plan area of wall	$A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 =$ **517.20** ft^2
Projected vertical area of roof	$A_{vert\_r\_90} =$ **0.00** ft^2
Minimum overall horizontal loading	$F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **8.28** kips
Leeward net force	$F_l = F_{w,wB} =$ **-1.2** kips
Windward net force	$F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **10.0** kips
Overall horizontal loading	$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **11.2** kips

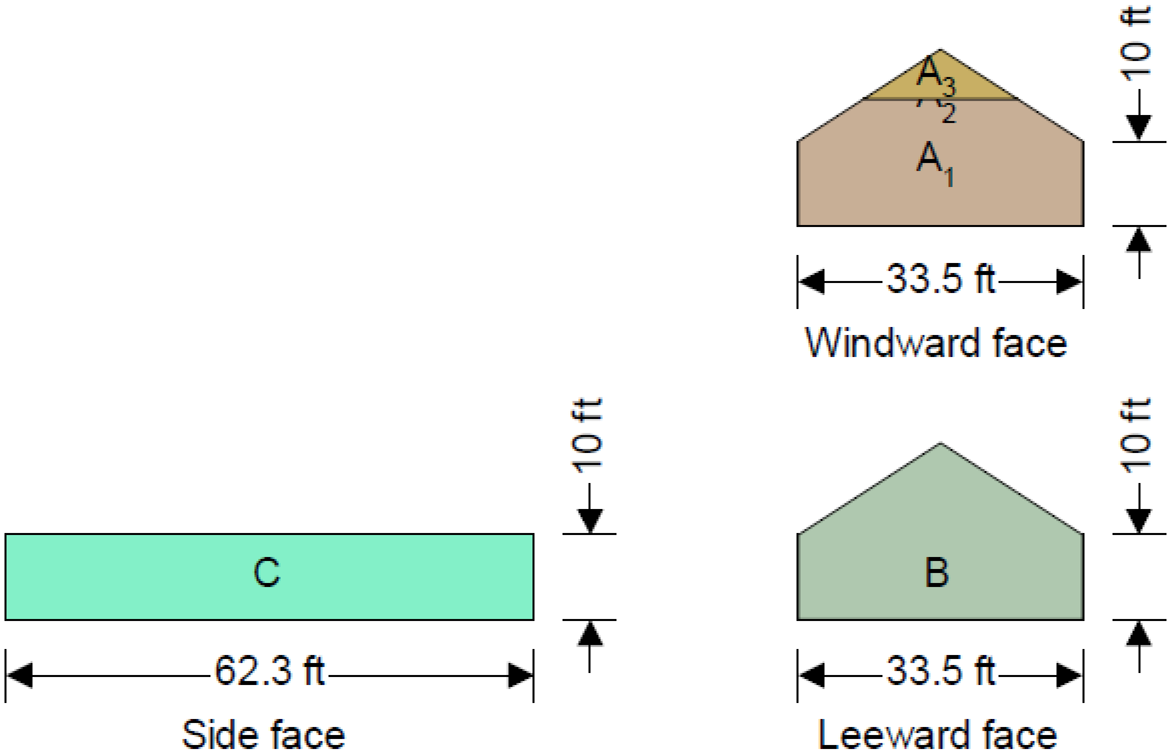


Plan view - Gable roof





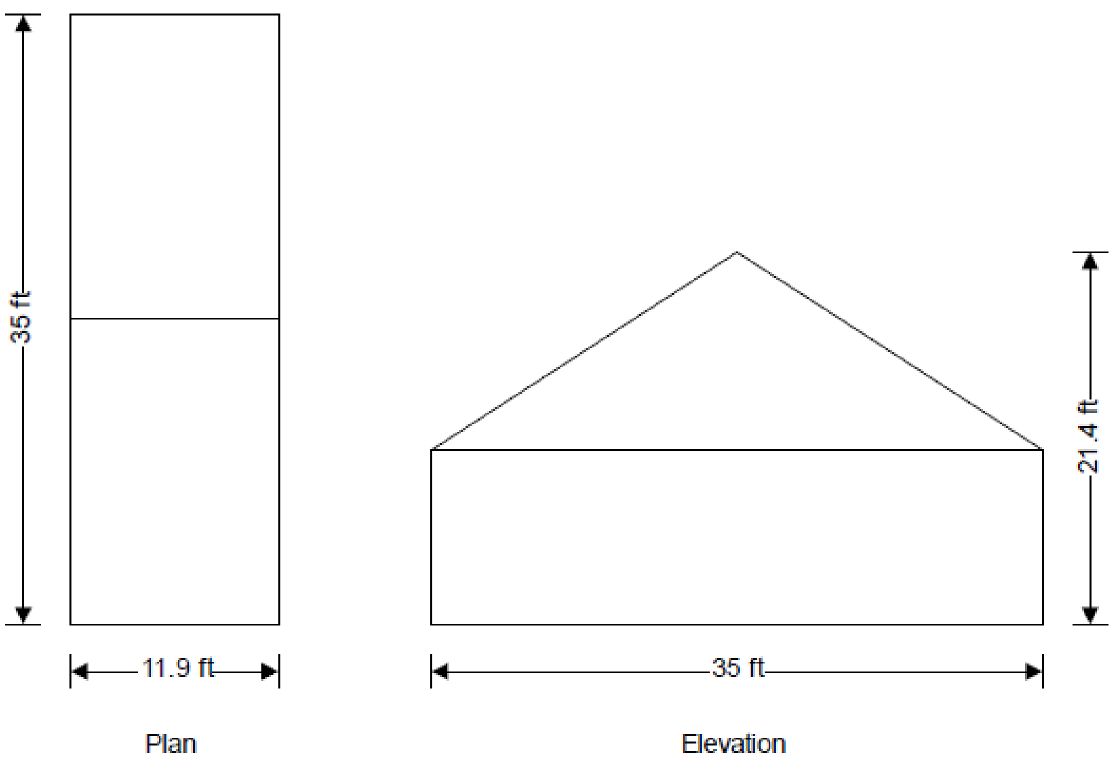
Plan view - Gable roof


A3
A2
A1
10 ft
33.5 ft
Windward face
C
62.3 ft
10 ft
Side face
B
10 ft
33.5 ft
Leeward face

Wind load on block C

In accordance with ASCE7-16

Using the directional design method



Building data

Type of roof	Gable
Length of building	b = **11.90** ft
Width of building	d = **35.00** ft
Height to eaves	H = **10.00** ft
Pitch of roof	α_0 = **33.0** deg
Mean height	h = **15.68** ft

General wind load requirements

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

Topography

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

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
10.00	0.85	22.38
15.00	0.85	22.38
15.00	0.85	22.38
15.68	0.86	22.56
21.36	0.91	23.98

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	q_i = **22.56** 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)	15.68	-0.18	22.56	-7.42	248.31	-1.84
B (-ve)	15.68	-0.60	22.56	-15.57	248.31	-3.87

Total vertical net force $F_{w,v}$ = **-4.79** kips

Total horizontal net force $F_{w,h}$ = **1.11** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	22.38	11.16	119.00	1.33
B	15.68	-0.25	22.56	-8.91	119.00	-1.06
C	15.68	-0.70	22.56	-17.48	548.88	-9.60
D	15.68	-0.70	22.56	-17.48	548.88	-9.60

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **119.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) =$ **135.24** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **2.99** kips

Leeward net force $F_l = F_{w,wB} =$ **-1.1** kips

Windward net force $F_w = F_{w,wA} =$ **1.3** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **3.5** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	15.68	0.28	22.56	9.44	248.31	2.35
B (+ve)	15.68	-0.60	22.56	-7.44	248.31	-1.85

Total vertical net force $F_{w,v}$ = **0.42** kips

Total horizontal net force $F_{w,h}$ = **2.29** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	22.38	19.28	119.00	2.29
B	15.68	-0.25	22.56	-0.79	119.00	-0.09
C	15.68	-0.70	22.56	-9.36	548.88	-5.14
D	15.68	-0.70	22.56	-9.36	548.88	-5.14

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **119.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) =$ **135.24** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **2.99** kips

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

Windward net force $F_w = F_{w,wA} =$ **2.3** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **4.7** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	15.68	-1.17	22.56	-26.42	327.23	-8.64
B (-ve)	15.68	-0.70	22.56	-17.48	169.39	-2.96

Total vertical net force $F_{w,v} =$ **-9.73** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	22.38	11.16	486.50	5.43
A_2	15.00	0.80	22.38	11.16	0.00	0.00
A_3	21.36	0.80	23.98	12.25	62.38	0.76
B	15.68	-0.50	22.56	-13.65	548.88	-7.49
C	15.68	-0.70	22.56	-17.48	119.00	-2.08
D	15.68	-0.70	22.56	-17.48	119.00	-2.08

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = \mathbf{548.88}$ ft^2

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

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{8.78}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-7.5}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{6.2}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{13.7}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	15.68	-0.18	22.56	0.61	327.23	0.20
B (+ve)	15.68	-0.18	22.56	0.61	169.39	0.10

Total vertical net force $F_{w,v} = \mathbf{0.25}$ kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	22.38	19.28	486.50	9.38
A_2	15.00	0.80	22.38	19.28	0.00	0.00
A_3	21.36	0.80	23.98	20.37	62.38	1.27
B	15.68	-0.50	22.56	-5.53	548.88	-3.03
C	15.68	-0.70	22.56	-9.36	119.00	-1.11
D	15.68	-0.70	22.56	-9.36	119.00	-1.11

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = \mathbf{548.88}\ ft^2$

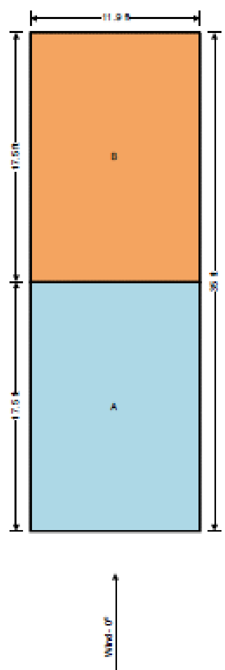
Projected vertical area of roof $A_{vert\_r\_90} = \mathbf{0.00}\ ft^2$

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{8.78}$ kips

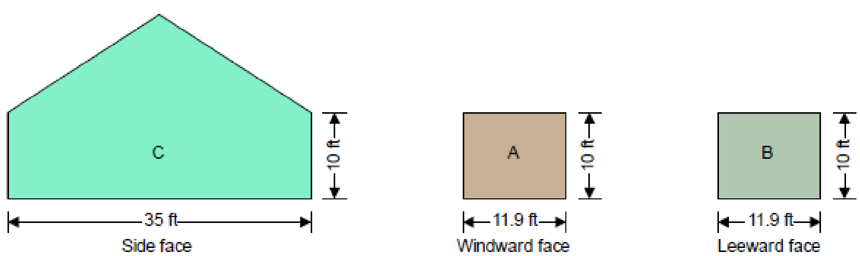
Leeward net force $F_l = F_{w,wB} = \mathbf{-3.0}$ kips

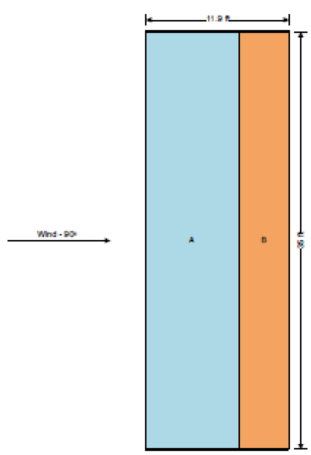
Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{10.7}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{13.7}$ kips

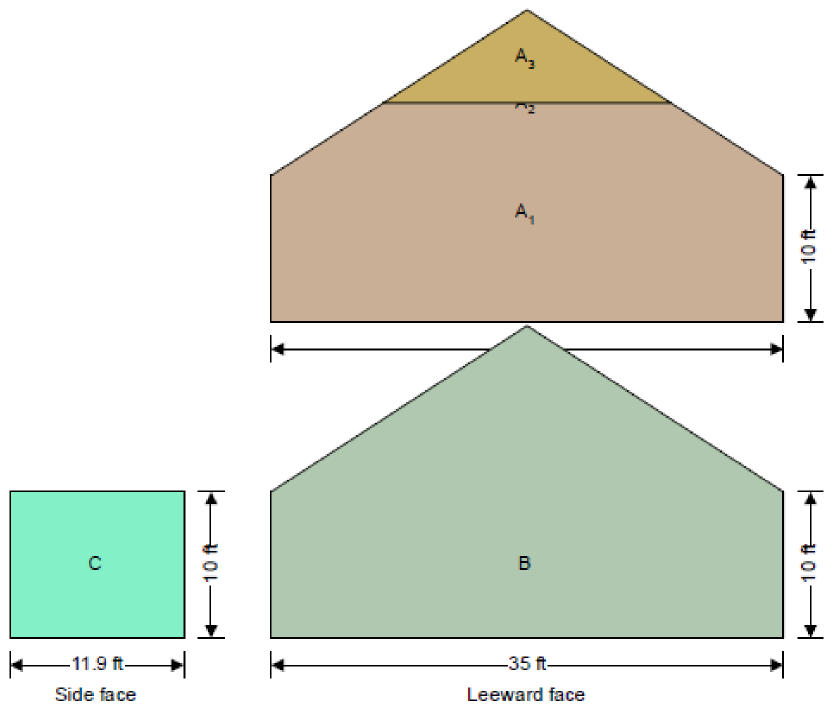


Plan view - Gable roof





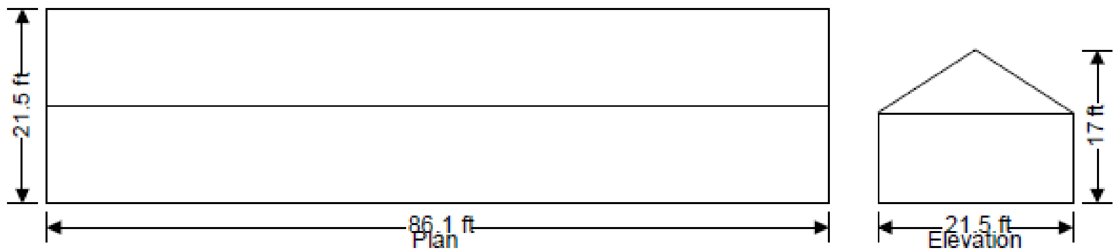
Plan view - Gable roof


A3
A2
A1
10 ft
C
10 ft
B
10 ft
11.9 ft
35 ft
Side face
Leeward face

Wind load on block D

In accordance with ASCE7-16

Using the directional design method



Building data

Type of roof	Gable
Length of building	b = **86.10** ft
Width of building	d = **21.50** ft
Height to eaves	H = **10.00** ft
Pitch of roof	α_0 = **33.0** deg
Mean height	h = **13.49** ft

General wind load requirements

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

Topography

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

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
10.00	0.85	22.38
13.49	0.85	22.38
15.00	0.85	22.38
16.98	0.87	22.90

Peak velocity pressure for internal pressure

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

Pressures and forces

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

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	13.49	-0.21	22.38	-8.03	1103.62	-8.86
B (-ve)	13.49	-0.60	22.38	-15.44	1103.62	-17.04

Total vertical net force $F_{w,v}$ = **-21.72** kips

Total horizontal net force $F_{w,h}$ = **4.46** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	22.38	11.19	861.00	9.63
B	13.49	-0.50	22.38	-13.54	861.00	-11.66
C	13.49	-0.70	22.38	-17.34	290.05	-5.03
D	13.49	-0.70	22.38	-17.34	290.05	-5.03

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H = \mathbf{861.00}$ ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) = \mathbf{601.08}$ ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{18.58}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-11.7}$ kips

Windward net force $F_w = F_{w,wA} = \mathbf{9.6}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{25.7}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	13.49	0.24	22.38	8.68	1103.62	9.58
B (+ve)	13.49	-0.60	22.38	-7.39	1103.62	-8.15

Total vertical net force $F_{w,v} = \mathbf{1.20}$ kips

Total horizontal net force $F_{w,h} = \mathbf{9.66}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	22.38	19.25	861.00	16.57
B	13.49	-0.50	22.38	-5.48	861.00	-4.72
C	13.49	-0.70	22.38	-9.29	290.05	-2.69
D	13.49	-0.70	22.38	-9.29	290.05	-2.69

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H = \mathbf{861.00}$ ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) = \mathbf{601.08}$ ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{18.58}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-4.7}$ kips

Windward net force $F_w = F_{w,wA} = \mathbf{16.6}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{30.9}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	13.49	-0.90	22.38	-21.15	172.92	-3.66
B (-ve)	13.49	-0.90	22.38	-21.15	172.92	-3.66
C (-ve)	13.49	-0.50	22.38	-13.54	345.84	-4.68
D (-ve)	13.49	-0.30	22.38	-9.74	1515.56	-14.75

Total vertical net force $F_{w,v}$ = **-22.44** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	22.38	11.19	284.00	3.18
A_2	16.98	0.80	22.90	11.54	6.04	0.07
B	13.49	-0.20	22.38	-7.83	290.05	-2.27
C	13.49	-0.70	22.38	-17.34	861.00	-14.93
D	13.49	-0.70	22.38	-17.34	861.00	-14.93

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4$ = **290.05** ft^2

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

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **4.64** kips

Leeward net force $F_l = F_{w,wB}$ = **-2.3** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2}$ = **3.2** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **5.5** 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)	13.49	-0.18	22.38	0.60	172.92	0.10
B (+ve)	13.49	-0.18	22.38	0.60	172.92	0.10
C (+ve)	13.49	-0.18	22.38	0.60	345.84	0.21
D (+ve)	13.49	-0.18	22.38	0.60	1515.56	0.92

Total vertical net force $F_{w,v}$ = **1.12** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	22.38	19.25	284.00	5.47
A_2	16.98	0.80	22.90	19.60	6.04	0.12
B	13.49	-0.20	22.38	0.22	290.05	0.06
C	13.49	-0.70	22.38	-9.29	861.00	-8.00
D	13.49	-0.70	22.38	-9.29	861.00	-8.00

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 =$ **290.05** ft^2

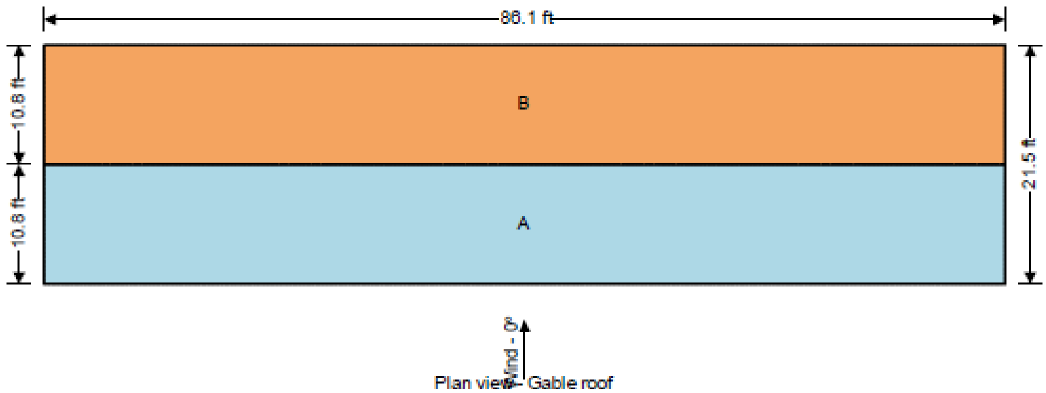
Projected vertical area of roof $A_{vert\_r\_90} =$ **0.00** ft^2

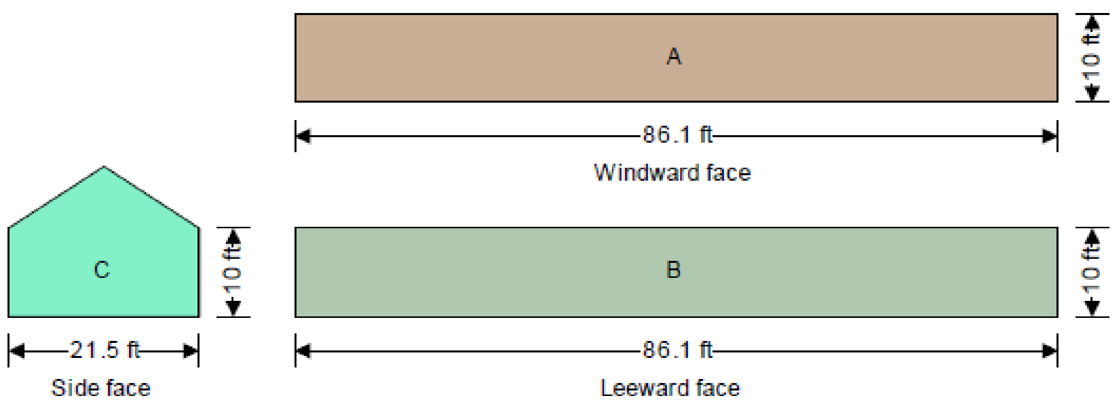
Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **4.64** kips

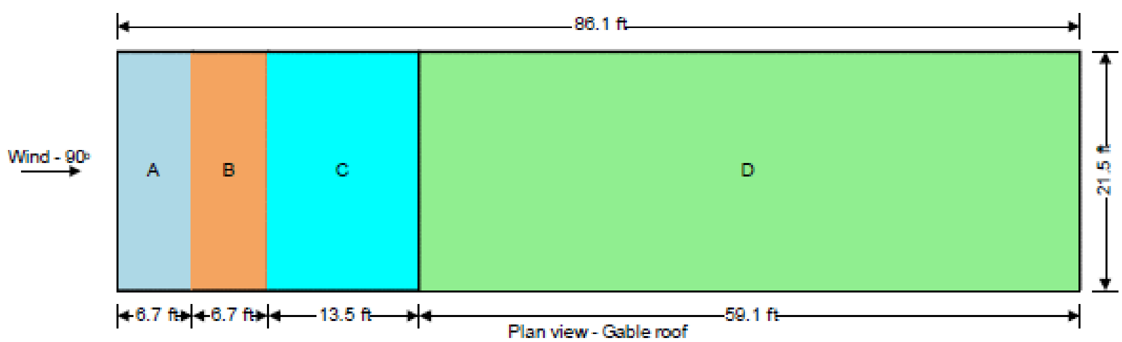
Leeward net force $F_l = F_{w,wB} =$ **0.1** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **5.6** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **5.5** kips

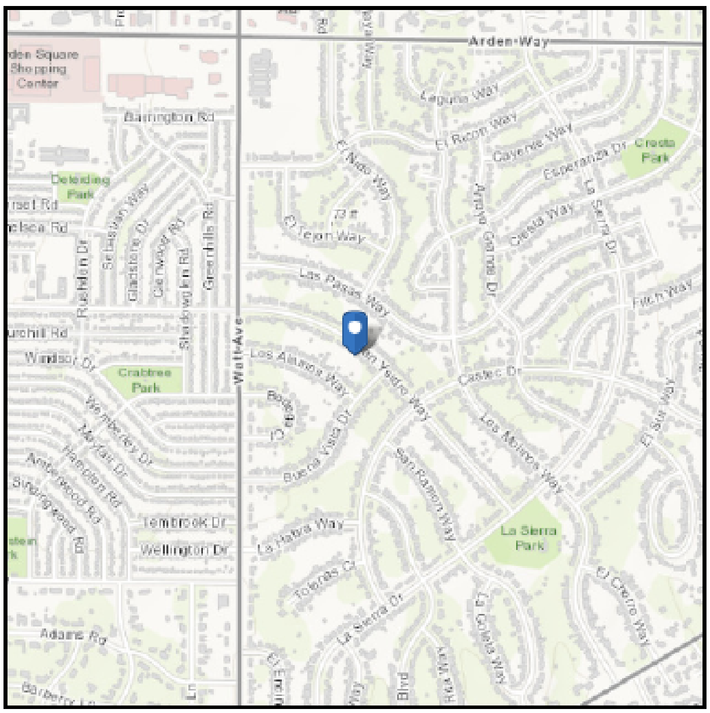

86.1 ft
B
A
10.8 ft
10.8 ft
21.5 ft
Wind - 0°
Plan view - Gable roof

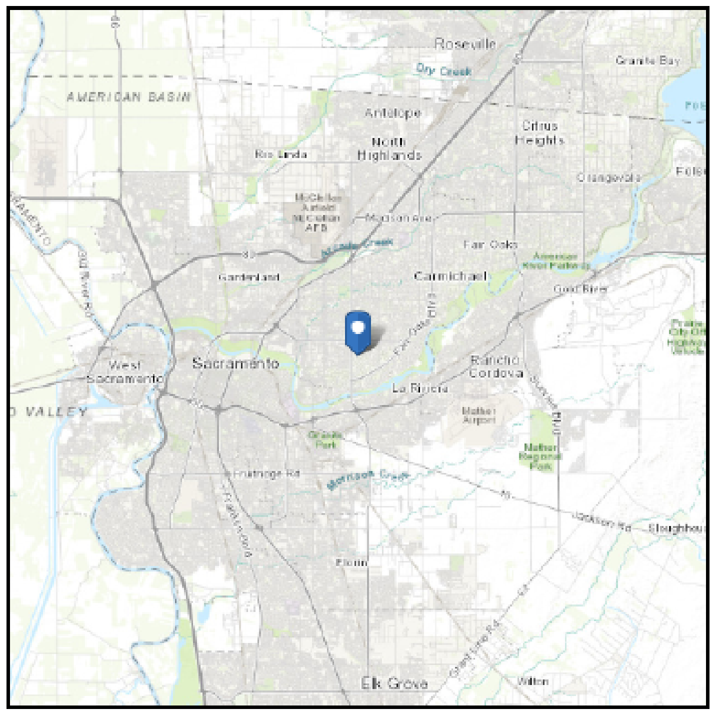

A
10 ft
86.1 ft
Windward face
C
10 ft
21.5 ft
Side face
B
10 ft
86.1 ft
Leeward face


86.1 ft
Wind - 90°
A
B
C
D
21.5 ft
6.7 ft
6.7 ft
13.5 ft
59.1 ft
Plan view - Gable roof

Seismic loads.

Address:	3660 SAN YSIDRO WAY SACRAMENTO, CA 95864
Coordinates:	38.5873414, -121.3790251
Elevation:	78 ft
Timestamp:	2023-04-05T09:32:56.131Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default





Basic Parameters

Name	Value	Description
S_S	0.496	MCE_R ground motion (period=0.2s)
S_1	0.235	MCE_R ground motion (period=1.0s)
S_{MS}	0.696	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.464	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.403	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.956	Coefficient of risk (0.2s)
CR_1	0.944	Coefficient of risk (1.0s)
PGA	0.209	MCE_G peak ground acceleration
F_{PGA}	1.391	Site amplification factor at PGA
PGA_M	0.291	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	0.496	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.519	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.235	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.249	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

2. STRUCTURAL ANALYSIS

2.1 BUILDING DESCRIPTION

The building is 86’ 4 1/2” long and 62’ 8” feet wide. It consists of AAC walls and LGS roof and floor joists. All openings are reinforced by vertical rebars from sides and by horizontal reinforced U-block lintel. Foundations is a slab-on-grade and Strip foundations.

The design of the structure is based on the requirements of the 2022 California Building Code and reference codes.

The analytical model is presented as a spatial model.

The structural rigidity of the containers in the longitudinal and transverse directions is achieved due to the steel walls of the containers.

Structural analysis is done in SCIA Engineer 20 software. This software allows automatic determination of the load combination that causes highest forces in structural members for further analysis and cross section selection. Governing load cases are shown in the sections "CHECKING STEEL ELEMENTS".

Seismic Force-Resisting System refers to Steel systems not specifically detailed for seismic resistance.

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

Modal Mode:

This approach allows the modal analysis of the structure, setting the first n values and eigenvectors of the structure.

The available analysis methods: subspace iteration, Lanczos method and the basis reduction method.

Iterations will be completed if the following condition is met: where:

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

i = 1,2,...,n vibration modes, k - number of iterations.

Upper limit is the period value (pulsation, frequency), which describes that in the range, (0, upper limit) the following values and eigenvectors will be set. Sturm check, which allows finding the skipped pulsations, is possible.

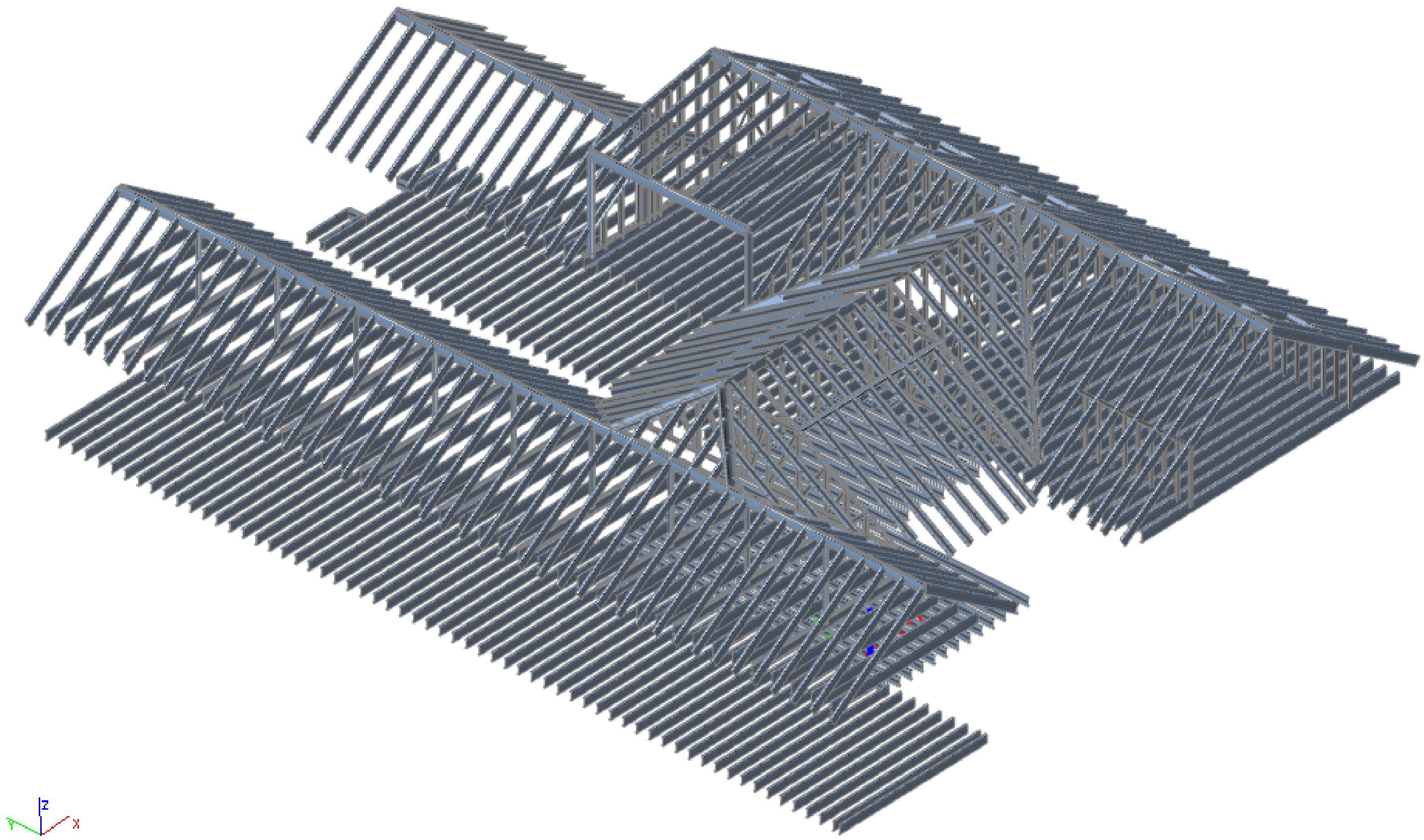
Response Spectrum Method:
Seismic analysis is based on the response spectrum method. All data is defined the same way as in modal analysis. Additionally, parameters required by a specific national code to establish the response spectrum shape must be specified. Calculations and results are the same as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

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

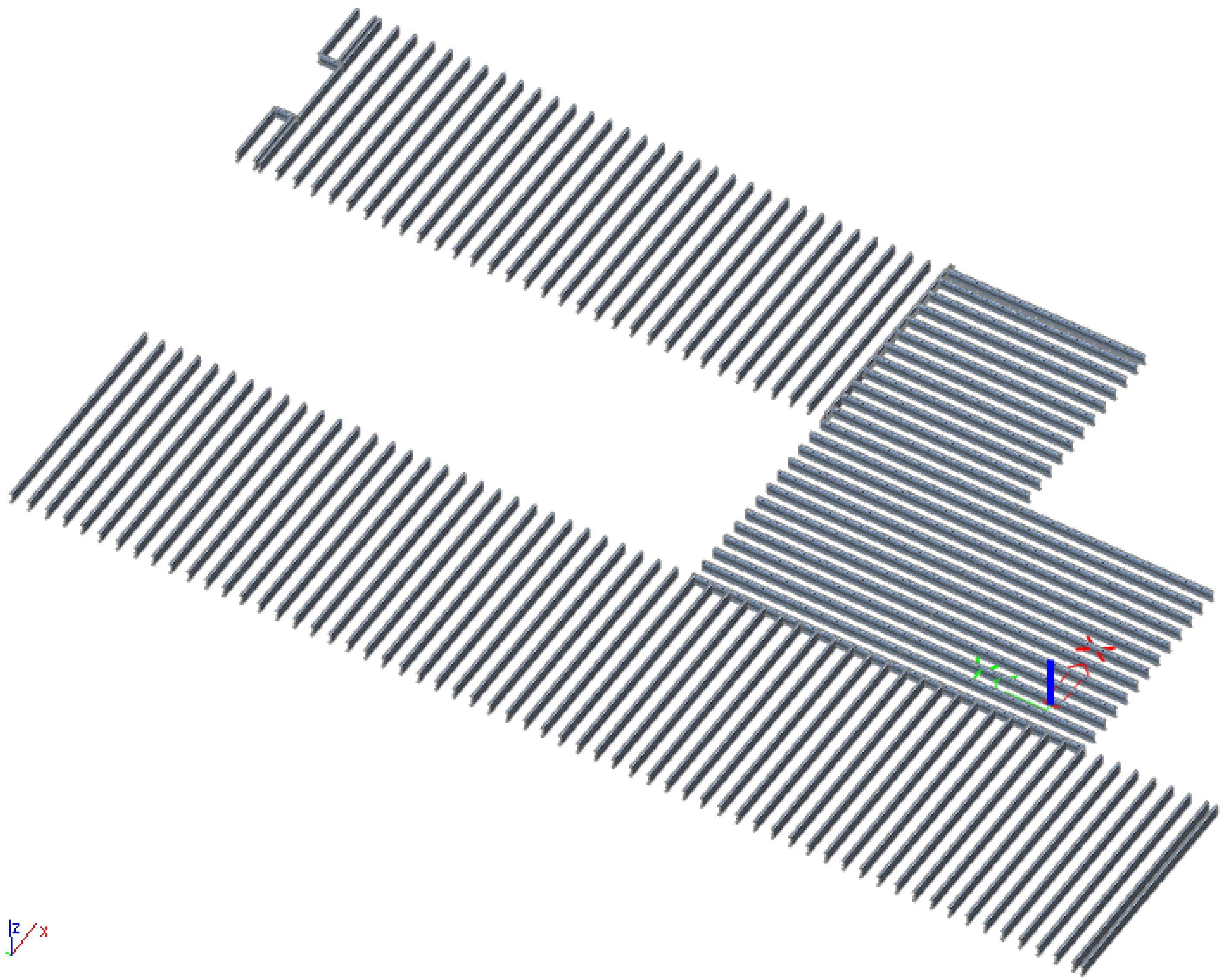
For seismic analysis, the same quadratic combination methods as those for spectral analysis are available.

2.2 GENERAL SCHEME

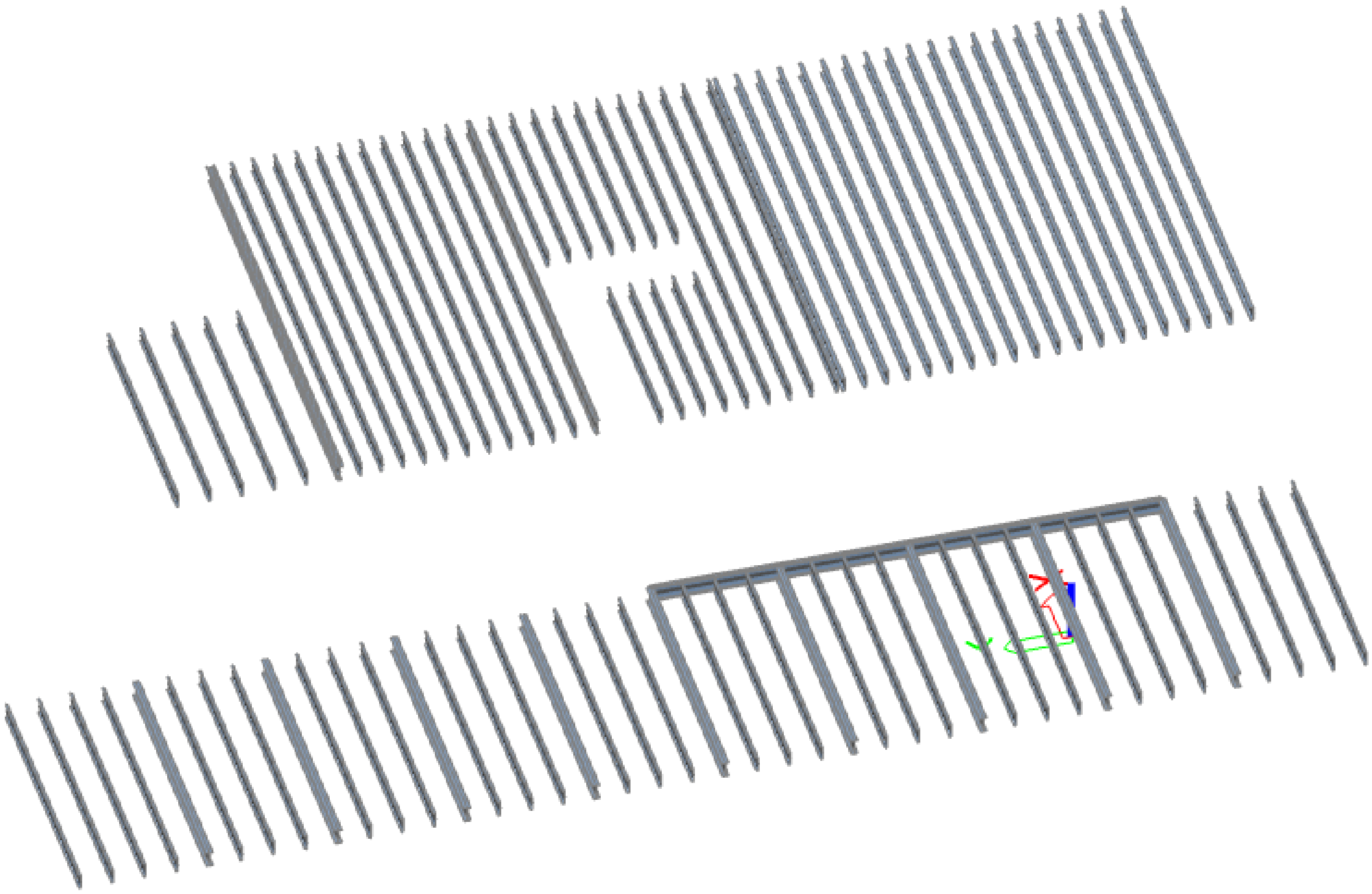
General views



1-st floor view

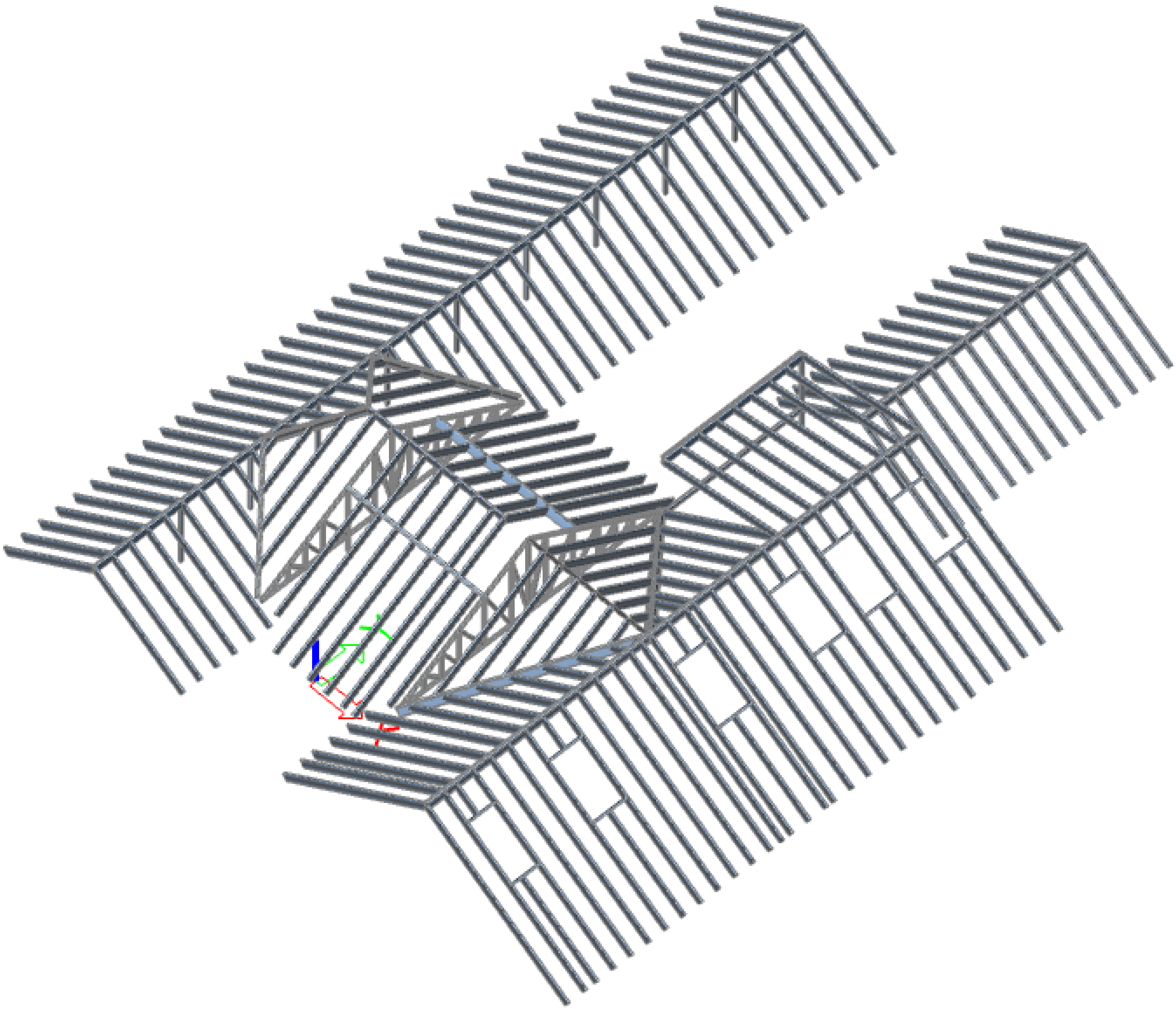


2-nd floor view

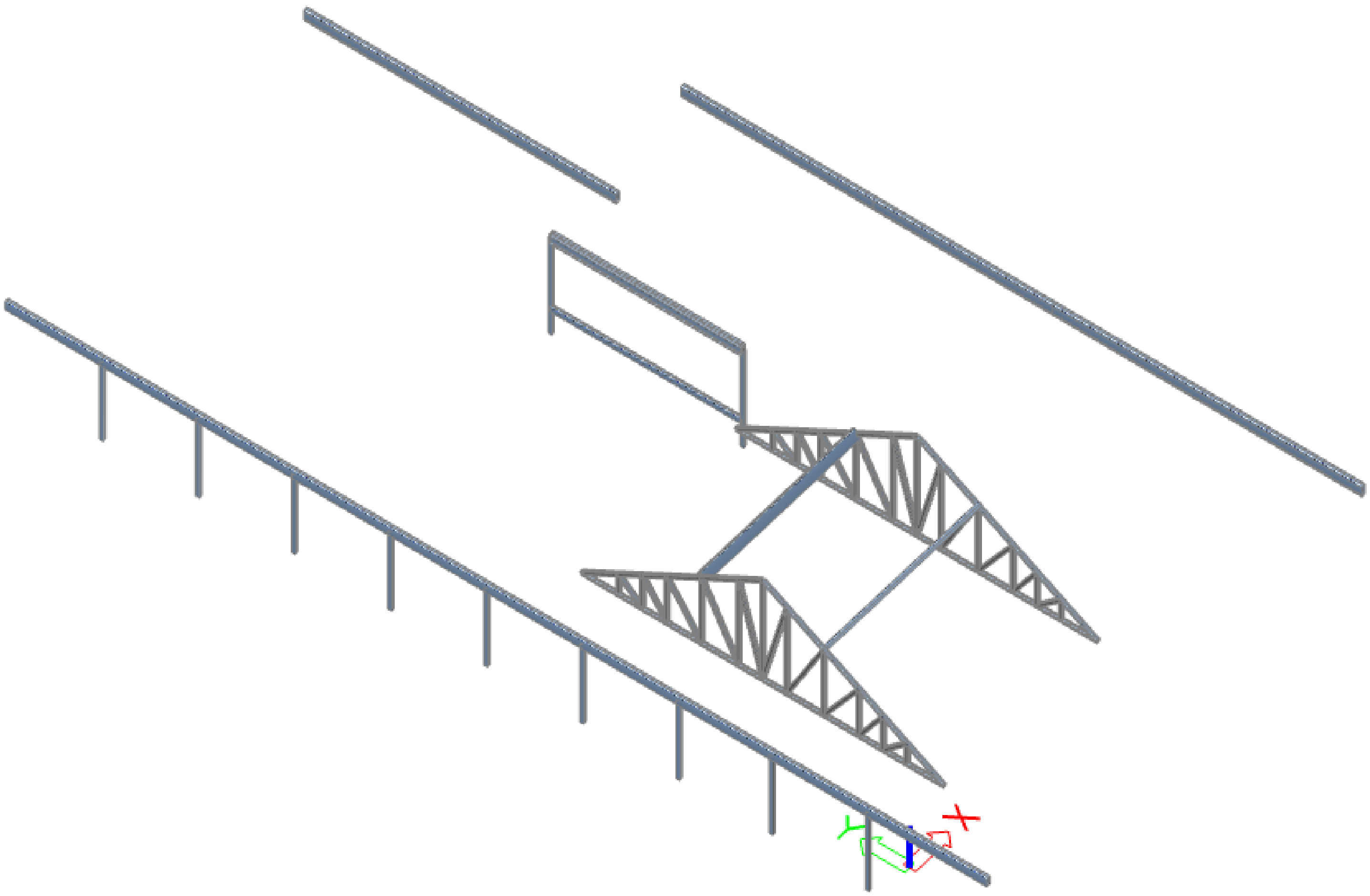


Wall view

Roof view



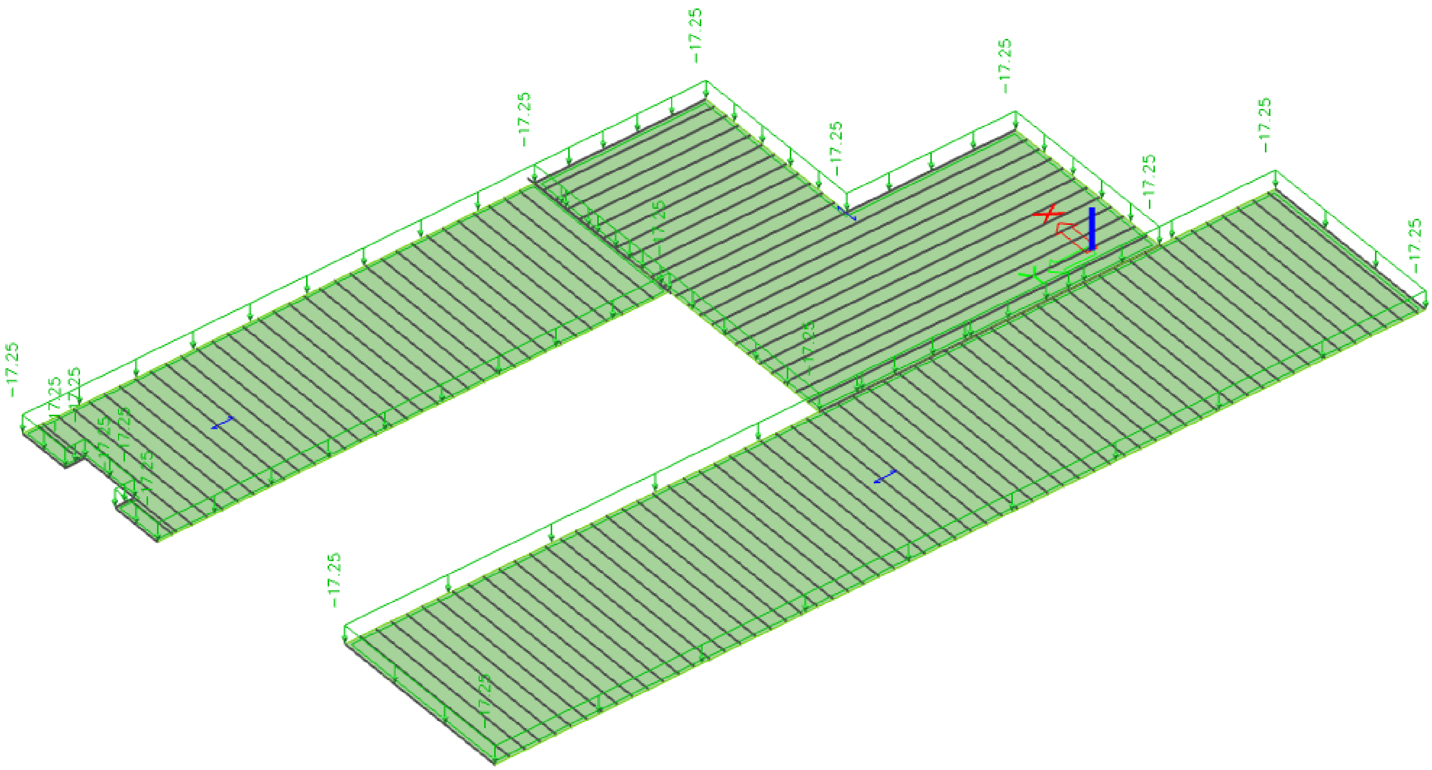
Truss & beam view



2.3 APPLIED LOADS

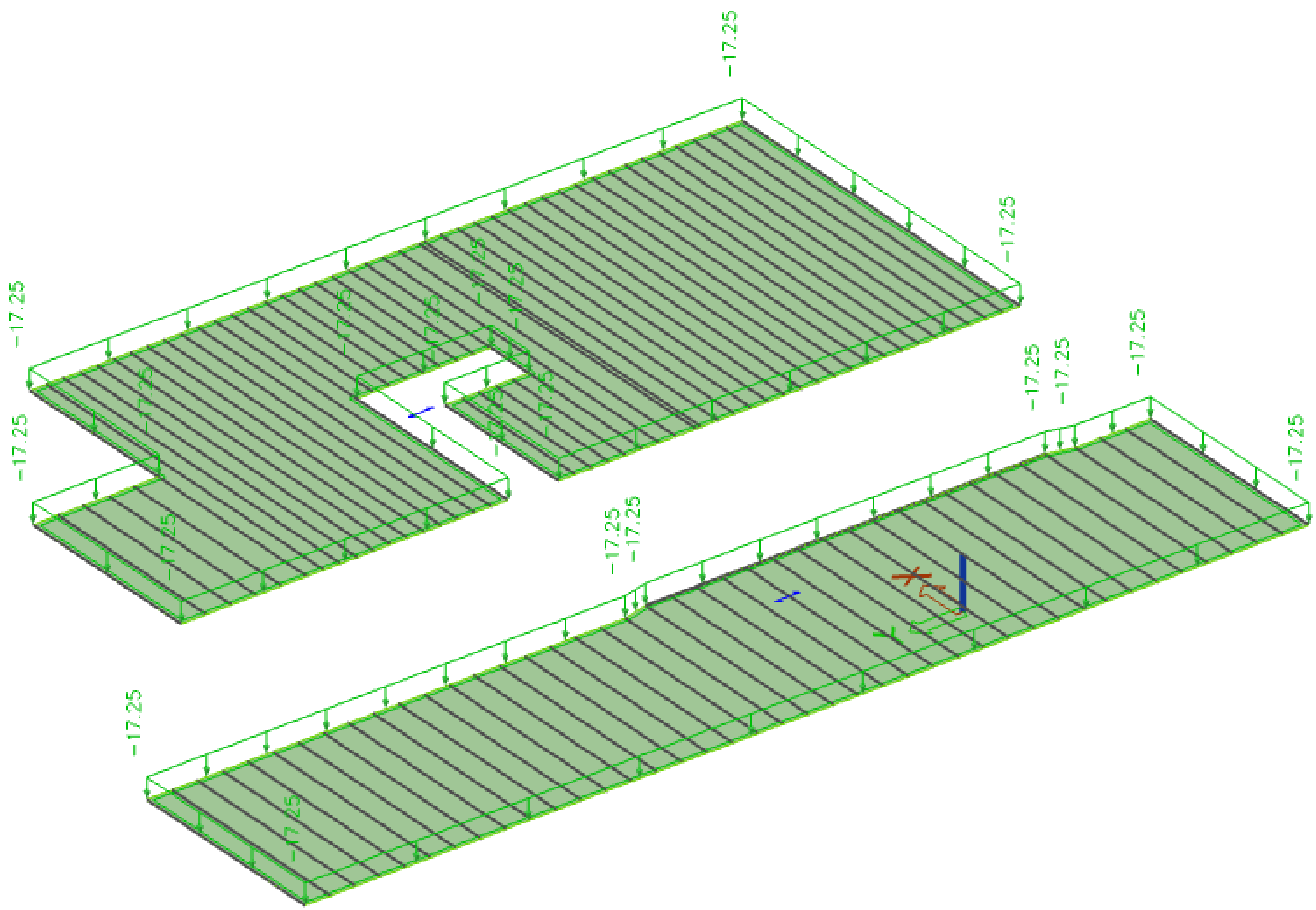
1. **DL1 - Dead weight of steel structures (automatically)**

2. **DL2 Self weight_Floor**
   Dead loads distribution
   1-ST FLOOR
   DL = 17.25 psf



2-ND FLOOR

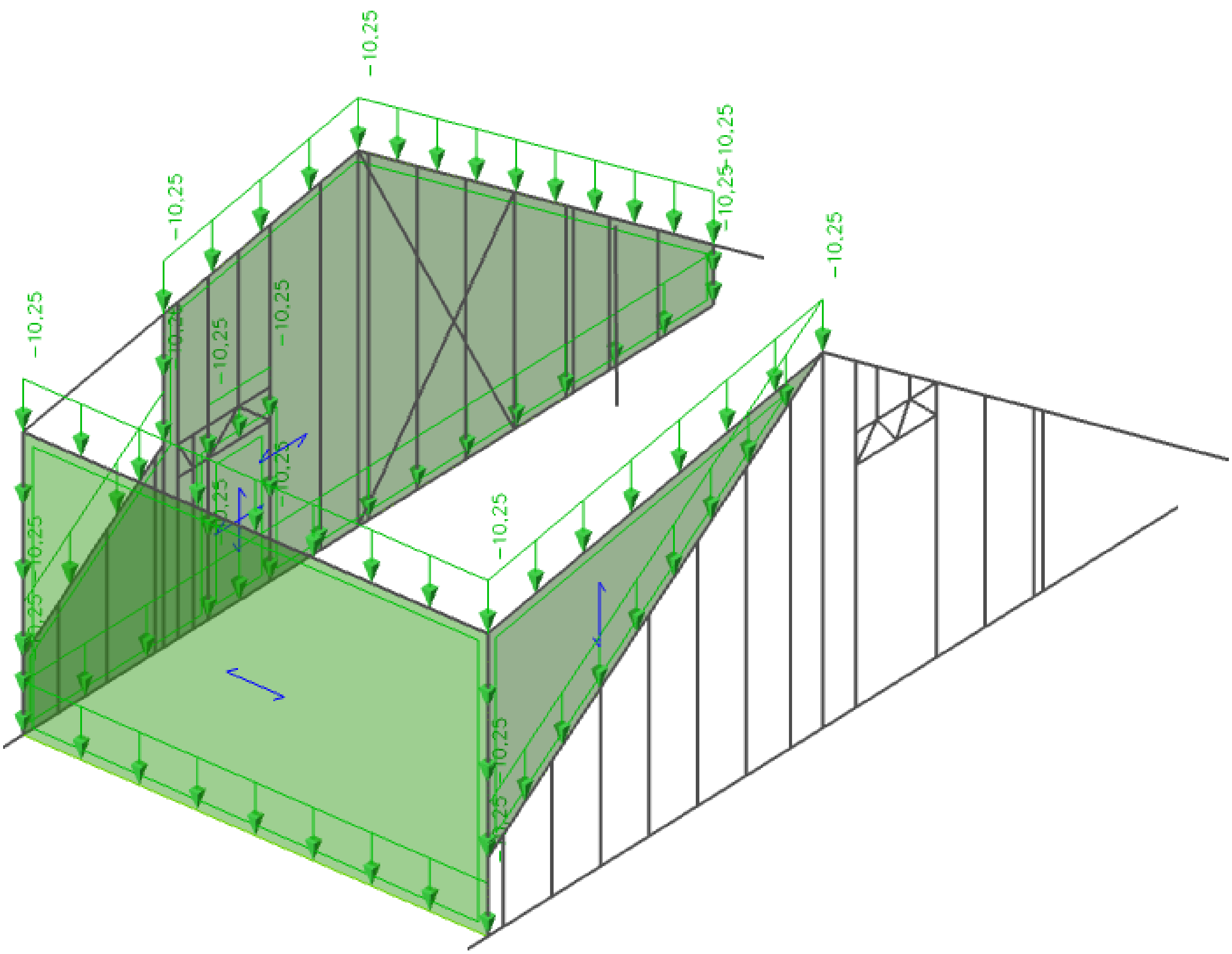
DL = 17.25 psf



3. DL3 Self weight Wall

Dead loads distribution

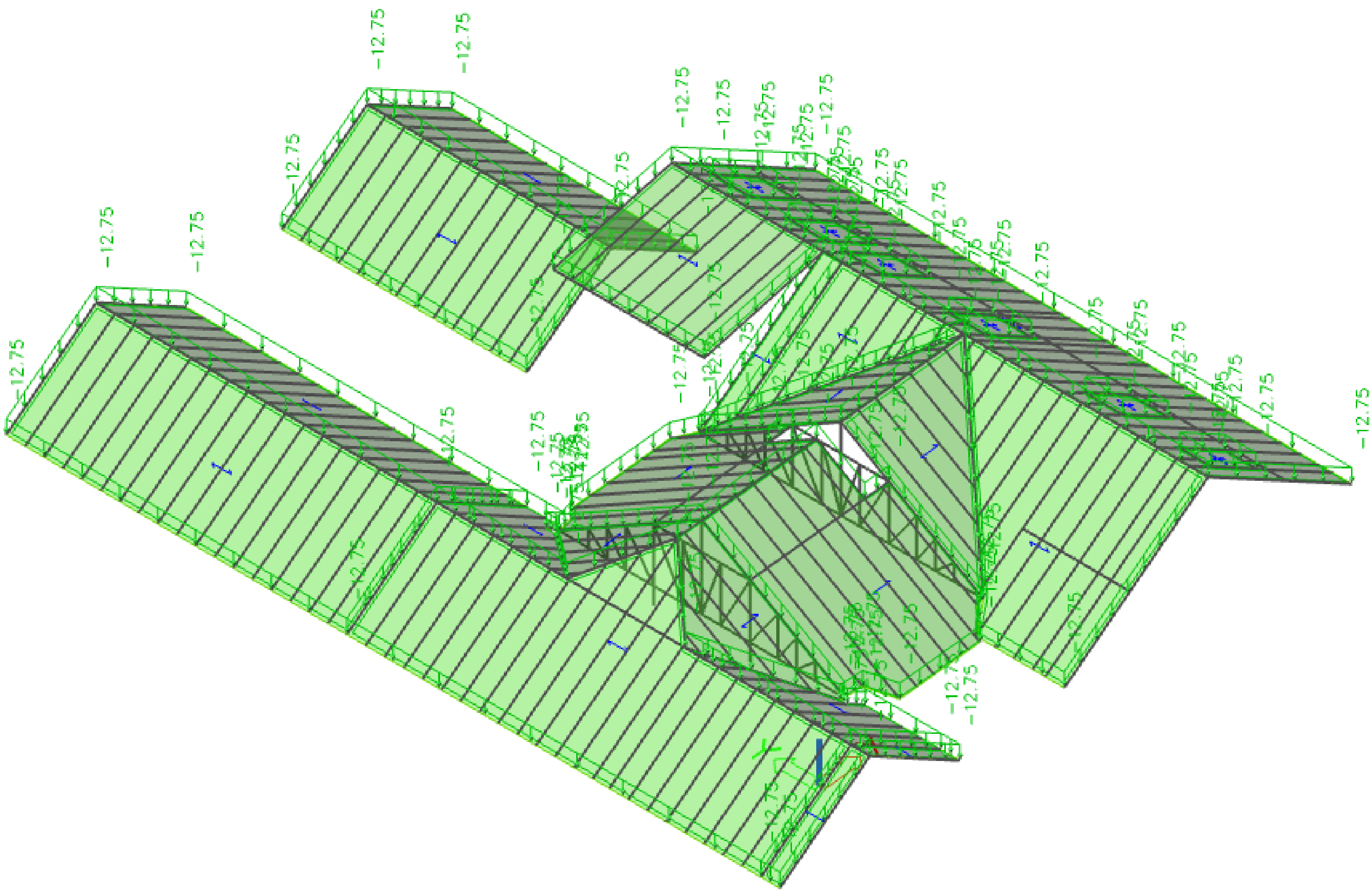
DL = 10.25 psf



4. DL4 Self weight Roof

Dead loads distribution

DL = 12.75 psf

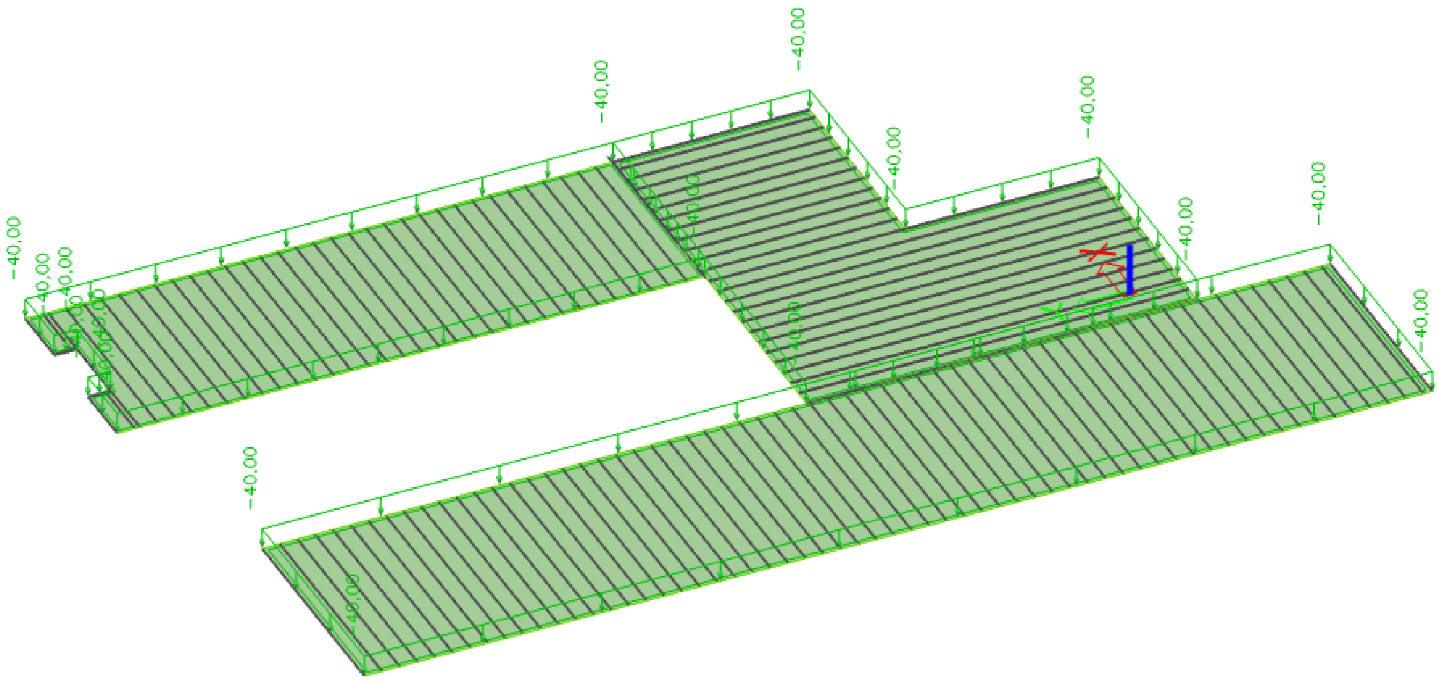


5. L Live load

Live loads distribution

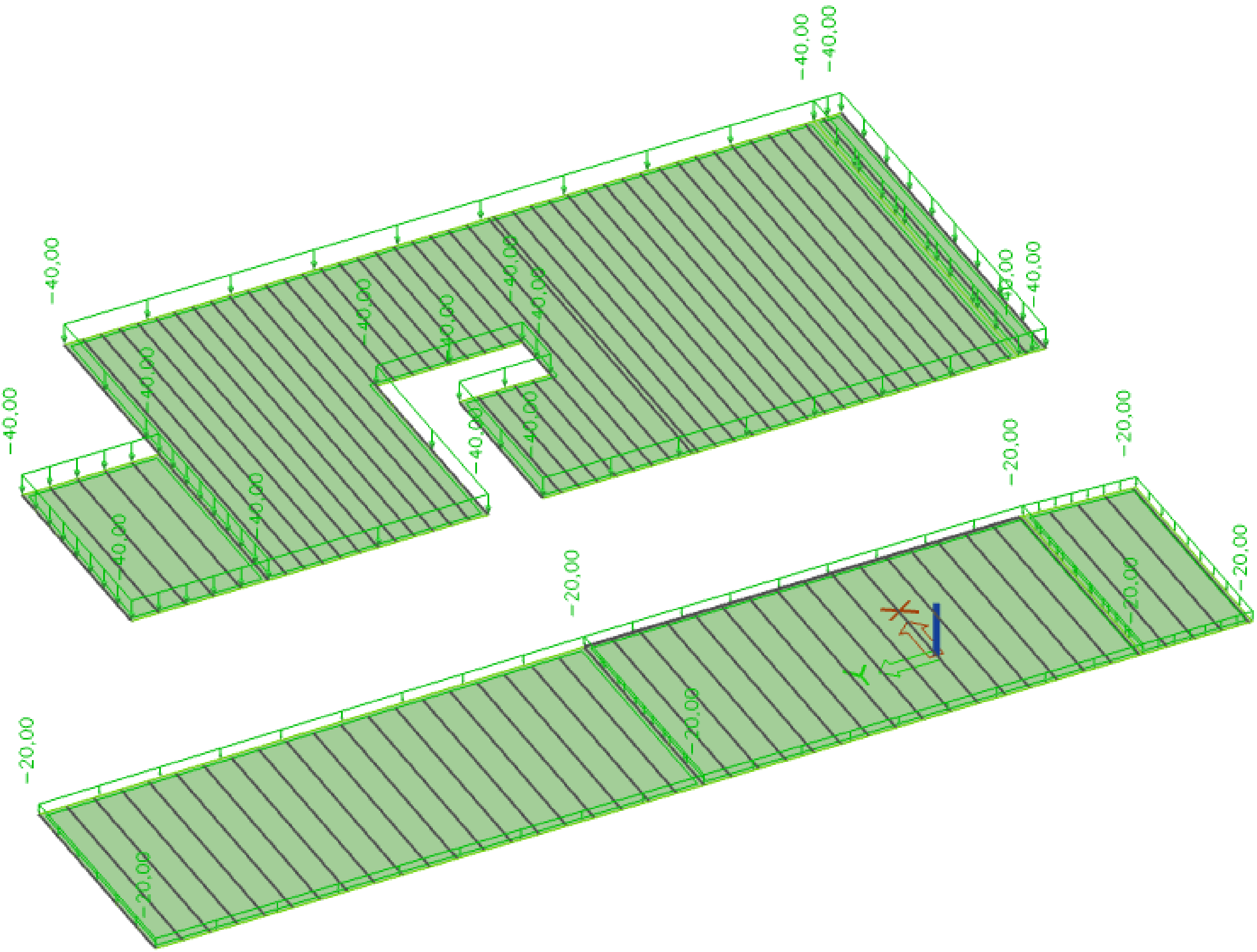
1-ST FLOOR

L = 40 psf



2-ND FLOOR

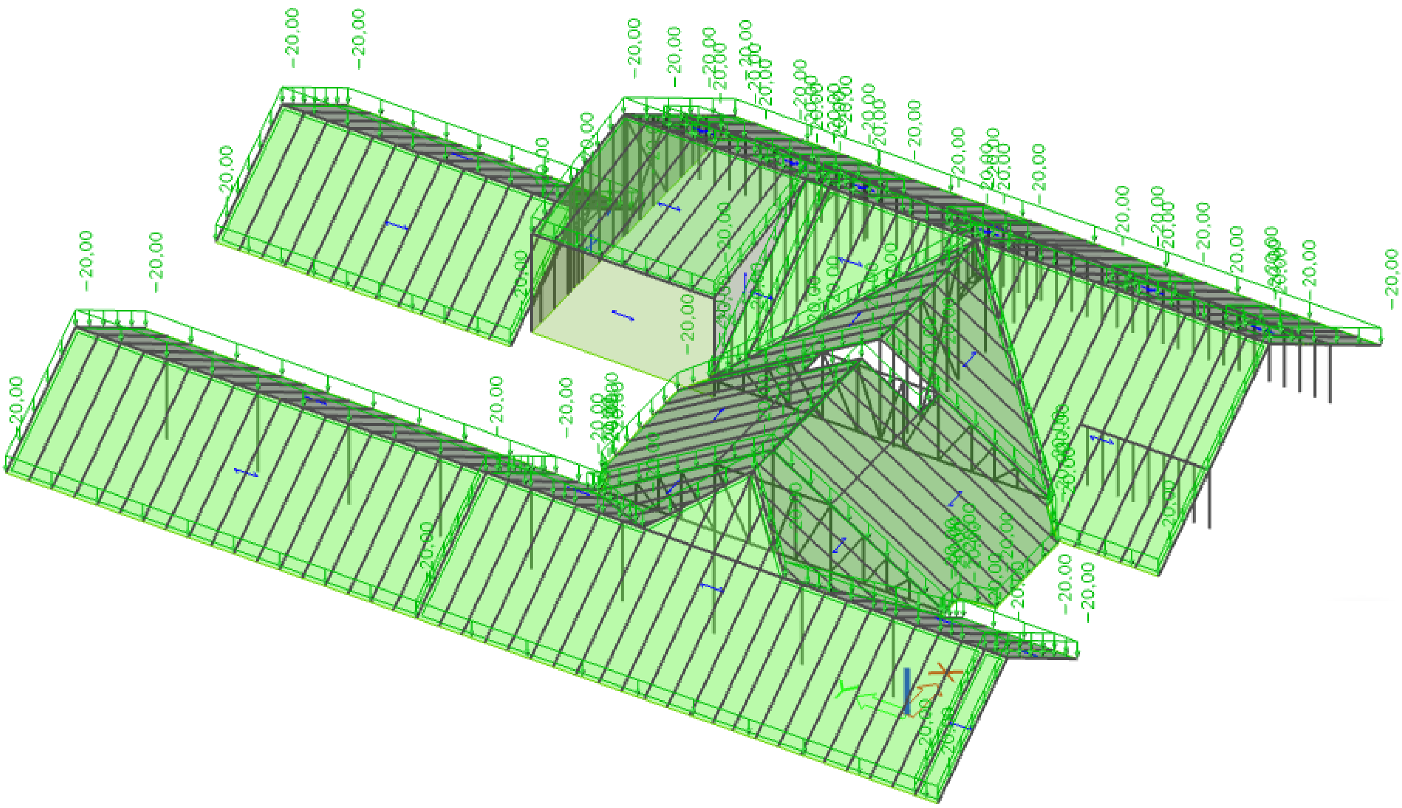
L = 20 psf; 40 psf;



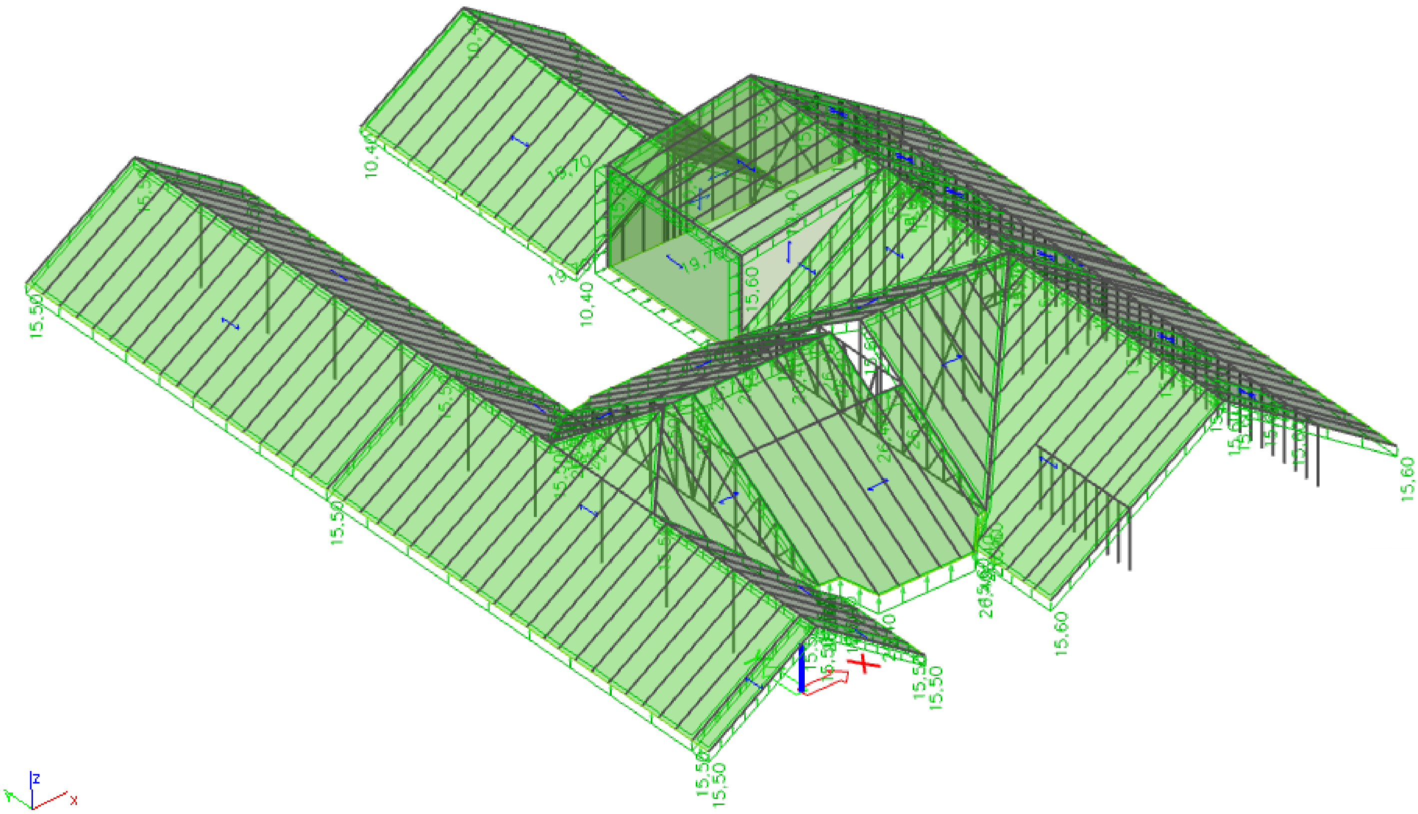
6. LR Roof live load

Live loads distribution

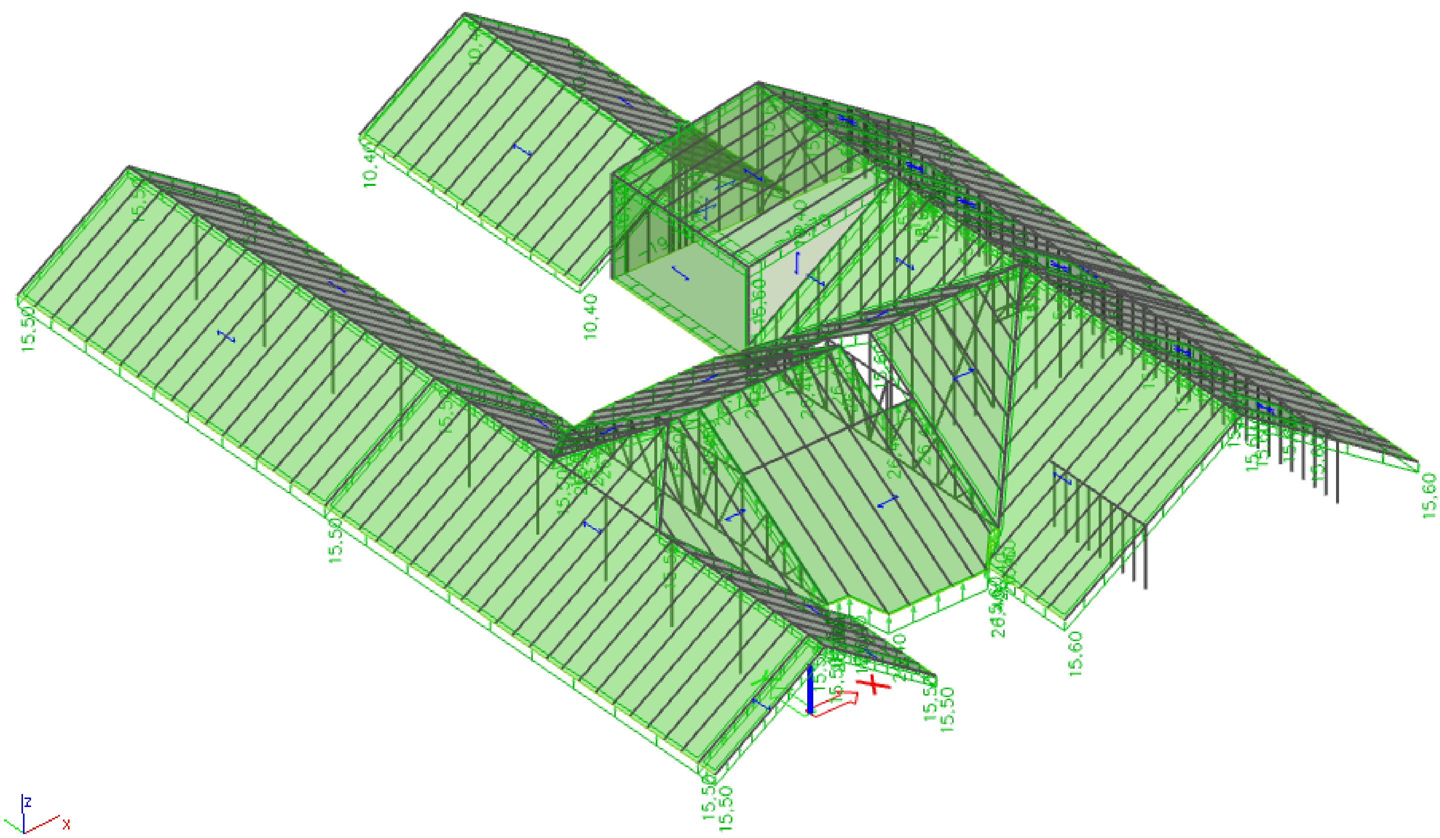
Lr = 20 psf



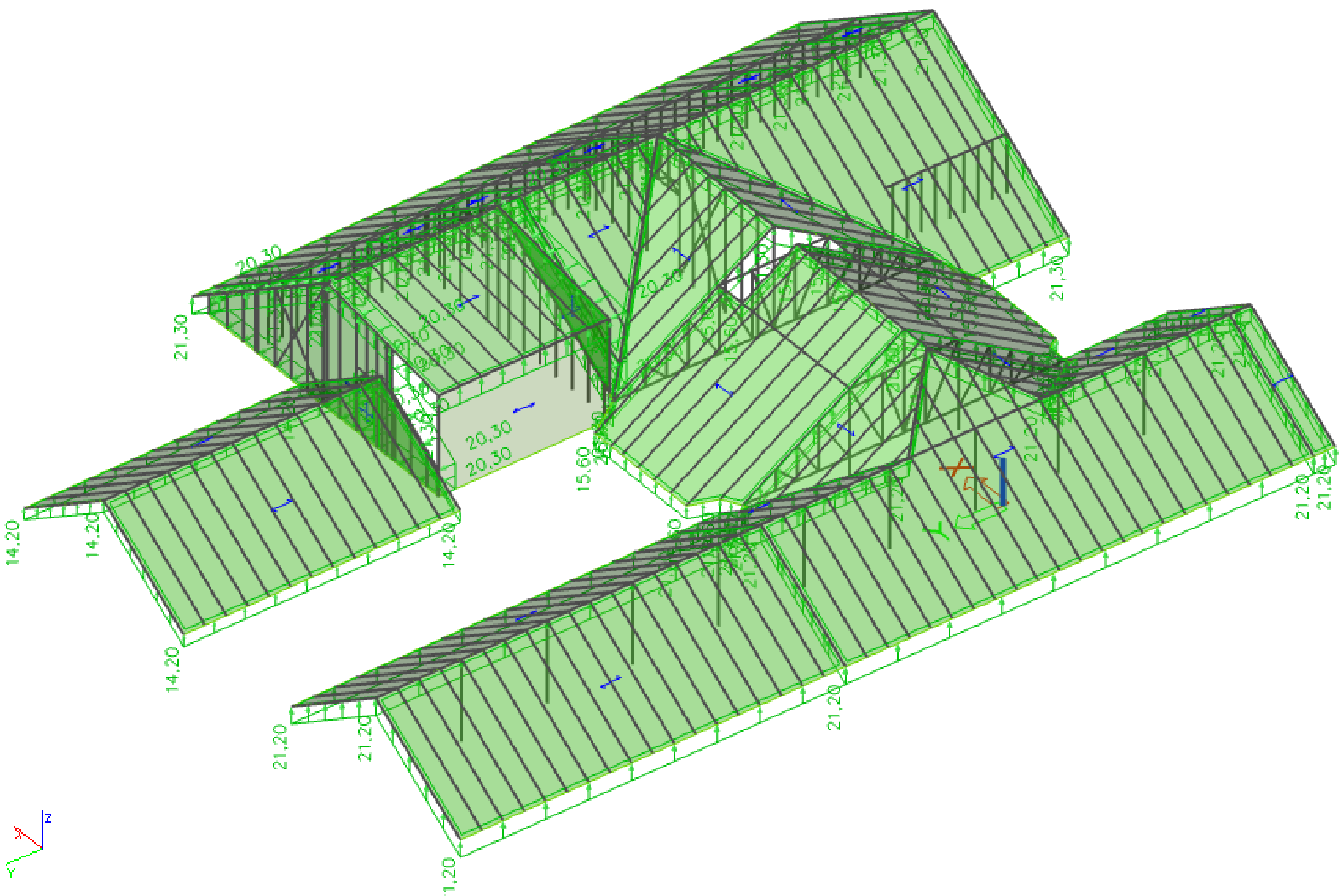
7. Wx+ Wind load



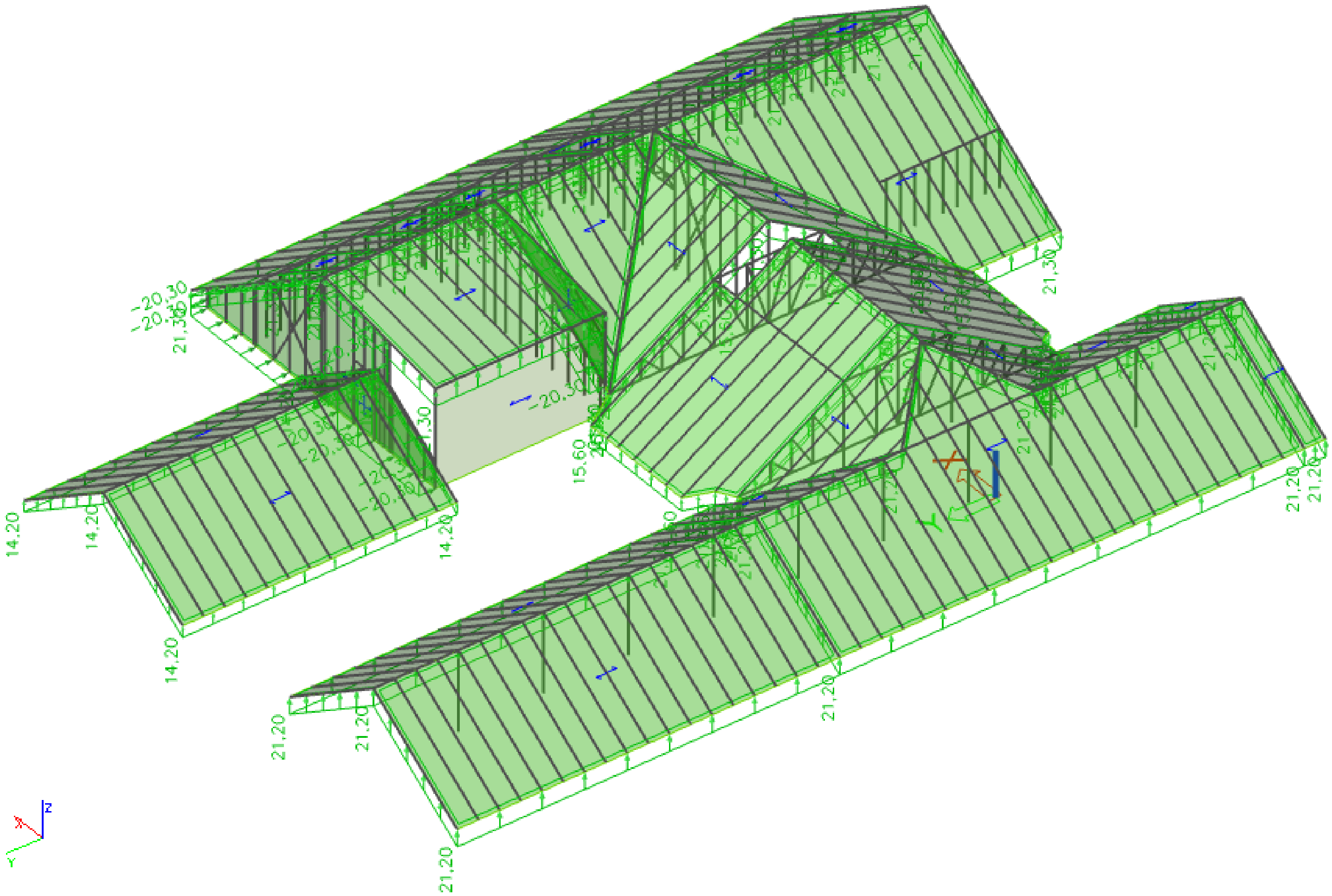
8. Wx- Wind load



9. Wy+ Wind load



10. Wy- Wind load



Load cases

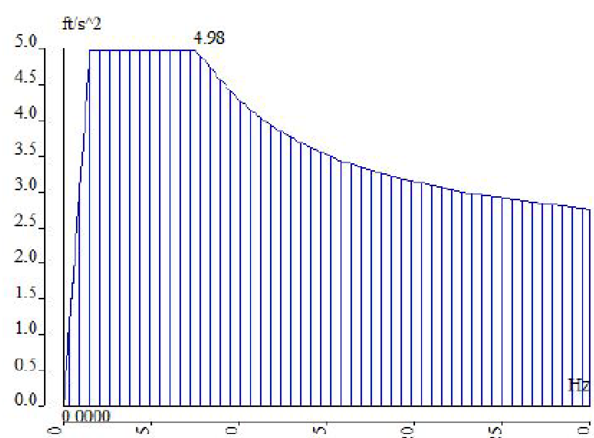
Case # **Case name**

Name	Description Spec	Action type Load type	Load group	Direction	Duration	Master load case
DL1	Self weight	Permanent	DL	-Z		
		Self weight				
DL2	Self weight floor	Permanent	DL			
		Standard				
DL3	Self weight wall	Permanent	DL			
		Self weight				
DL4	Self weight roof	Permanent	DL			
		Standard				
L	Floor live load Standard	Variable	L		Short	None
		Static				
Lr	Roof live load Standard	Variable	Lr		Short	None
		Static				
Wx+	Wind load Static wind	Variable	W			None
		Static				
Wx-	Wind load Static wind	Variable	W			None
		Static				
Wy+	Wind load Static wind	Variable	W			None
		Static				
Wy-	Wind load Static wind	Variable	W			None
		Static				

Ex	Seismic load Seismicity	Variable	E			None
		Static equivalent				
Ey	Seismic load Seismicity	Variable	E			None
		Static equivalent				

For seismic calculations, we use - Equivalent Lateral Forces (ELF). The ELF method is a static analysis method. ELF is calculated in the background after a model analysis. Calculation method for ELF forces based on "Approximate fundamental period". Calculated equivalent lateral forces are applied as one concentrated force at the mass centre of each story of the building and linear analysis is performed. The calculated story forces are applied to the structure using the reduced system. The transformation matrices of the IRS method make it possible to "smear" the concentrated story forces in such a way that the resultant of each story force is applied at the mass centre of the corresponding story.
For the ASCE 7 spectrum, the following variables are required to be specified in the program:

- site class
- response modification factor (R)
- seismic risk
- category of the structure (I to IV)
- short period acceleration parameter (Ss)
- second period acceleration parameter (S1)
- long period (TL)

Name UniqueID	Type drawing	Info	Drawing
FS1	Frequency	Type code - ASCE/SEI 7-16 – IBC US standard Type spectrum - DCE Coeff accel. Ag -1.000 g - 32.19 ft/s^2 Site class - D R - 3 Seismic risk category -II I_e - 1 Seismic design category - D Ss - 0.496 S1 - 0.235 TL - 12.00 s	

Load combinations

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

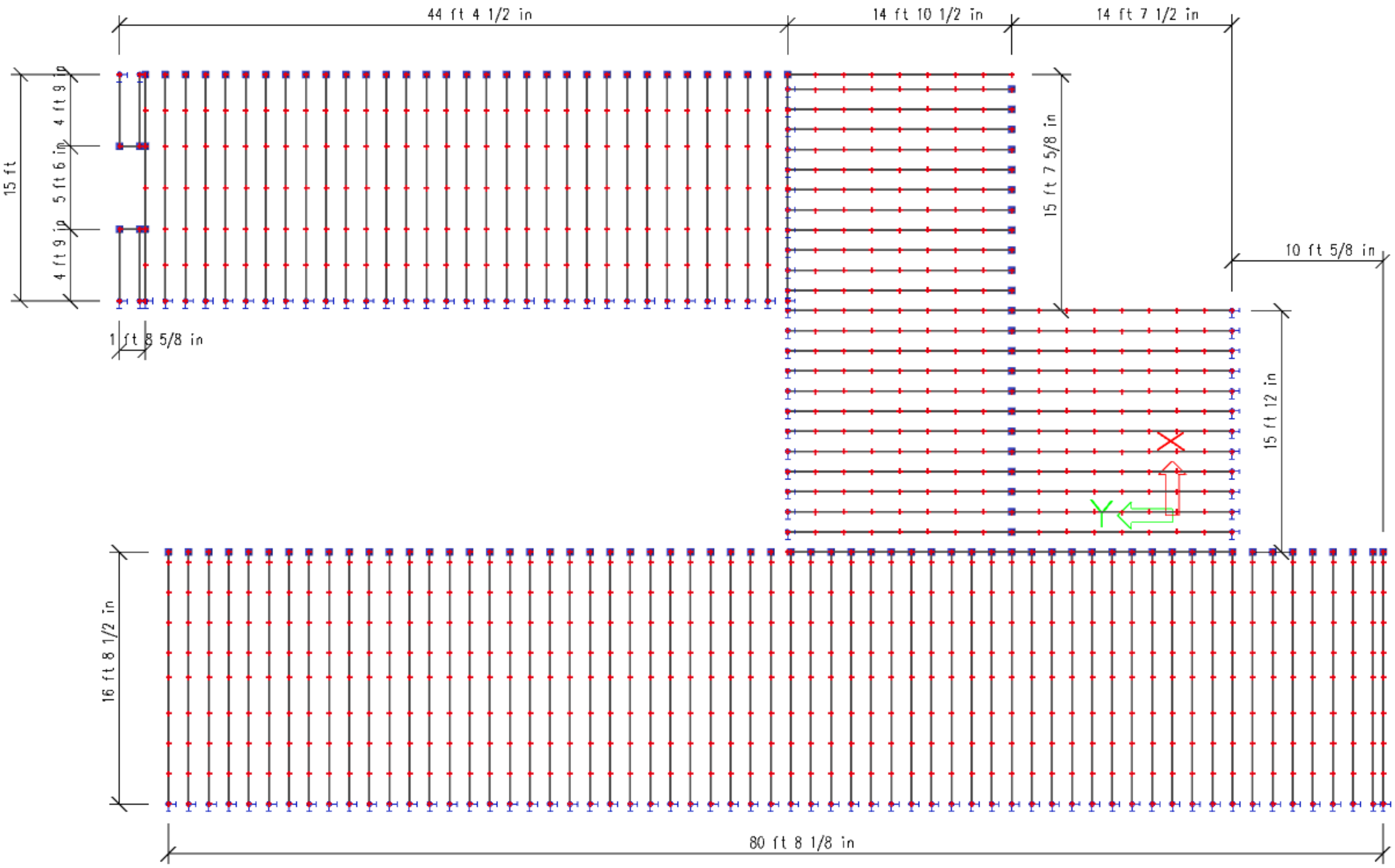
22	LRFD-Ult (auto) 22	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + Wx+
23	LRFD-Ult (auto) 23	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + Wx+
24	LRFD-Ult (auto) 24	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + Wx-
25	LRFD-Ult (auto) 25	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + Wx-
26	LRFD-Ult (auto) 26	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + Wy+
27	LRFD-Ult (auto) 27	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + Wy-
28	LRFD-Ult (auto) 28	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + Wy+
29	LRFD-Ult (auto) 29	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + Wy-
30	LRFD-Ult (auto) 30	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + Wx+
31	LRFD-Ult (auto) 31	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + Wx-
32	LRFD-Ult (auto) 32	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + Wy+
33	LRFD-Ult (auto)33	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + Wy-
34	LRFD-Ult (auto) 34	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4
35	LRFD-Ult (auto) 35	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx+
36	LRFD-Ult (auto) 36	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx-
37	LRFD-Ult (auto) 37	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy+
38	LRFD-Ult (auto) 38	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy-
39	LRFD-Ult (auto) 39	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + (-) Ex
40	LRFD-Ult (auto) 40	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + Ey
41	LRFD-Ult (auto) 41	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + (-) Ey
42	LRFD-Ult (auto) 42	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4
43	LRFD-Ult (auto) 43	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx+

44	LRFD-Ult (auto) 44	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx-
45	LRFD-Ult (auto) 45	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy+
46	LRFD-Ult (auto) 46	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy-
47	LRFD-Ult (auto) 47	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Ex
48	LRFD-Ult (auto) 48	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + (-) Ex
49	LRFD-Ult (auto) 49	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Ey
50	LRFD-Ult (auto) 50	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + (-) Ey

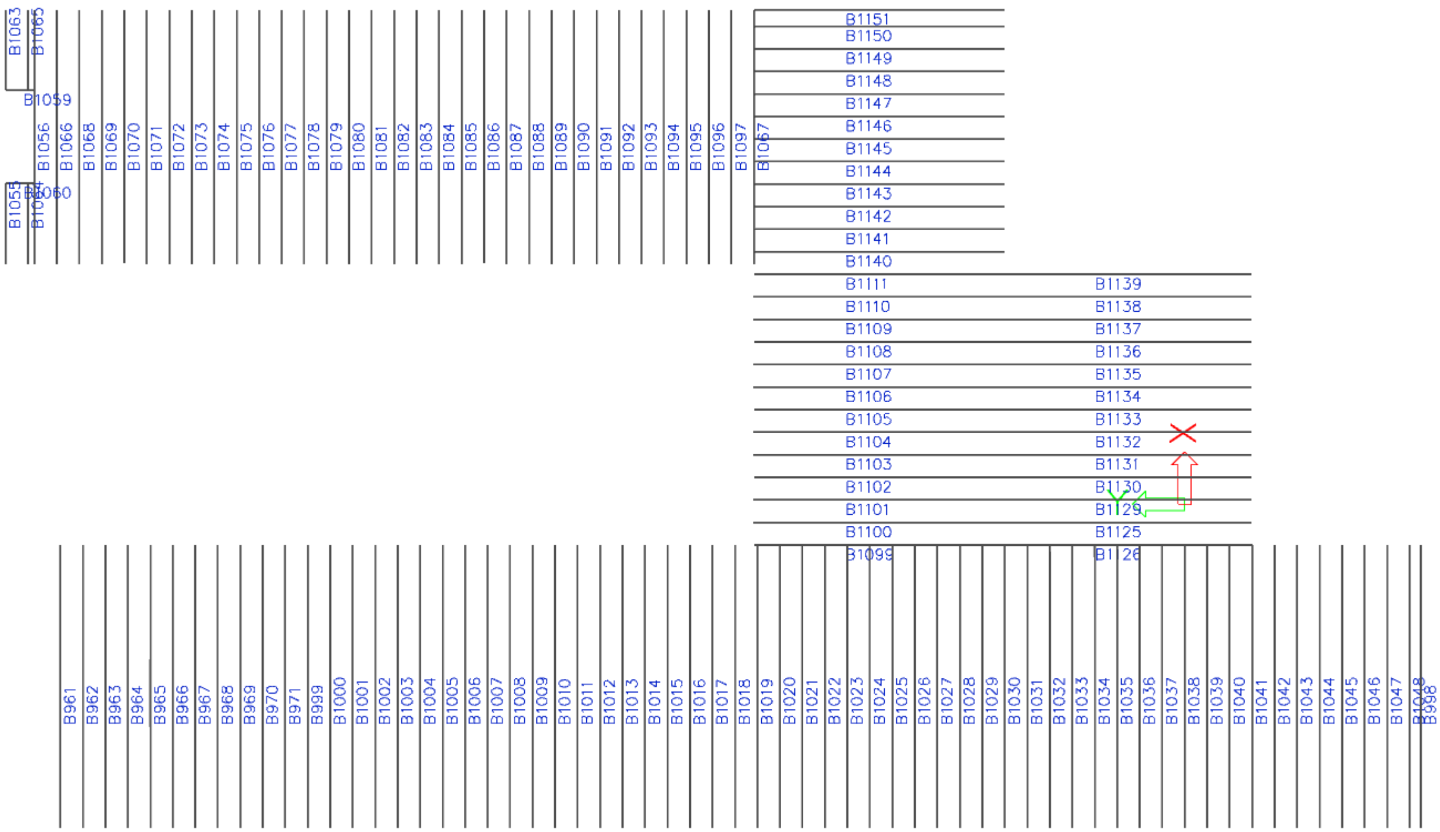
2.4 STEEL STRUCTURE CHECK
2.4.1 1-SR FLOOR JOIST STRUCTURAL ANALYSIS

General scheme

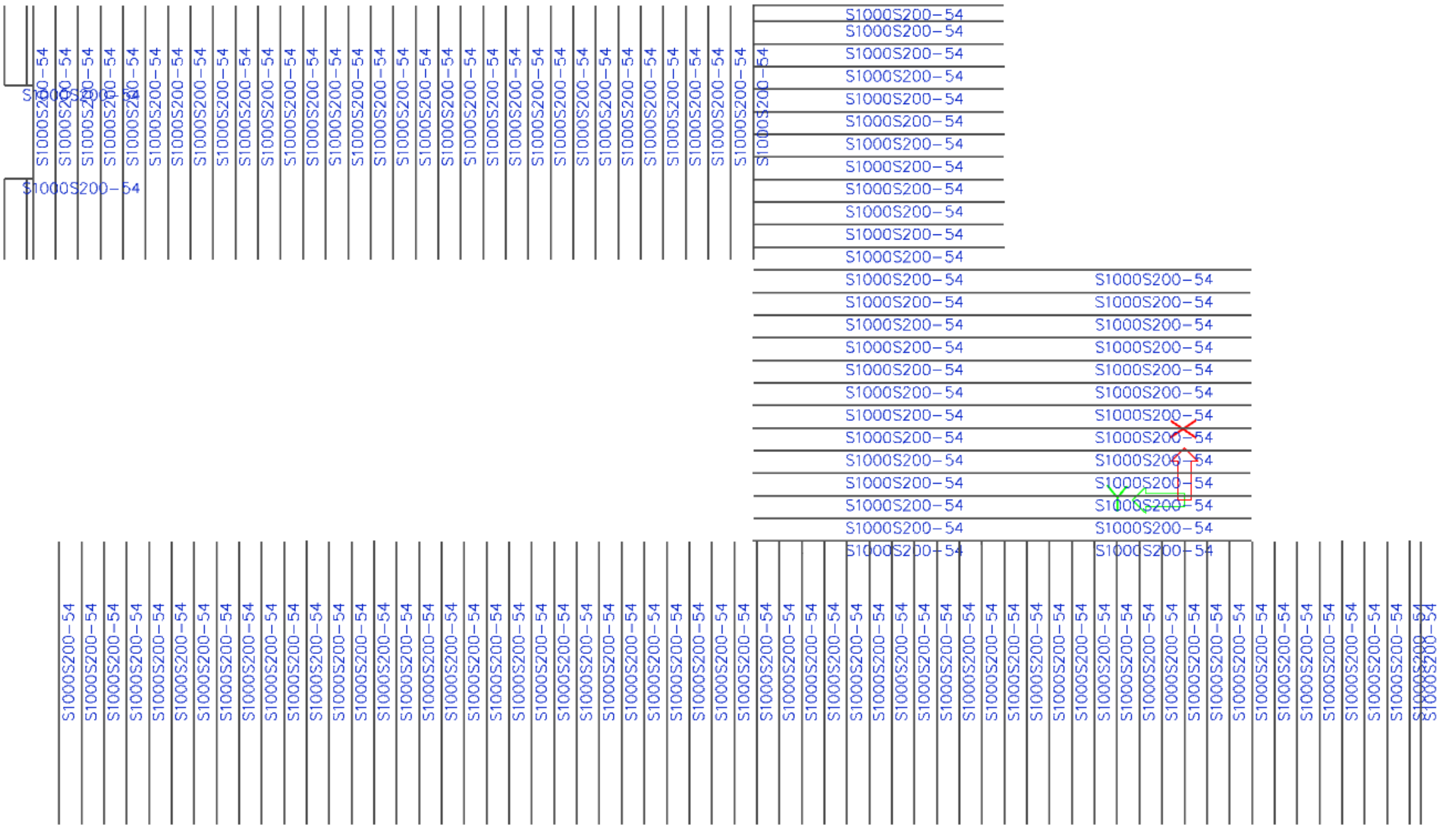
Floor joist spacing - 16 inch.



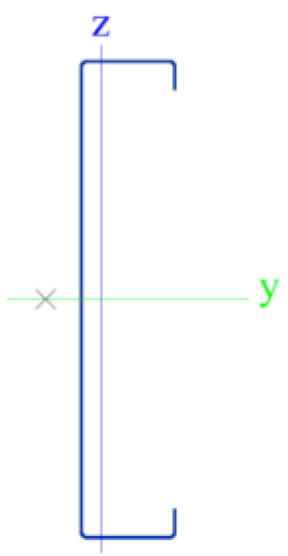
Member numbers



Cross-sections of members



Cross-sections properties

CS3		
Type	S1000S200-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.842	
A_y [inch^2], A_z [inch^2]	0.229	0.566
A_L [inch^2/inch], A_D [inch^2/inch]	2.98e+01	2.98e+01
$c_{Y.ucs}$ [inch], $c_{Z.ucs}$ [inch]	0.427	5.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	11.350	0.378
i_y [inch], i_z [inch]	3.671	0.670
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	2.255	0.240
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.753	0.340
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.38e+02	1.38e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.70e+01	1.70e+01
d_y [inch], d_z [inch]	-1.143	0.000
I_t [inch^4], I_w [inch^6]	0.001	7.665
$β_y$ [inch], $β_z$ [inch]	0.000	12.209
Picture		

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

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

Maximum force diagram
Shear force diagram Vz ,
LRFD-Ult (auto)4 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L), lbf.

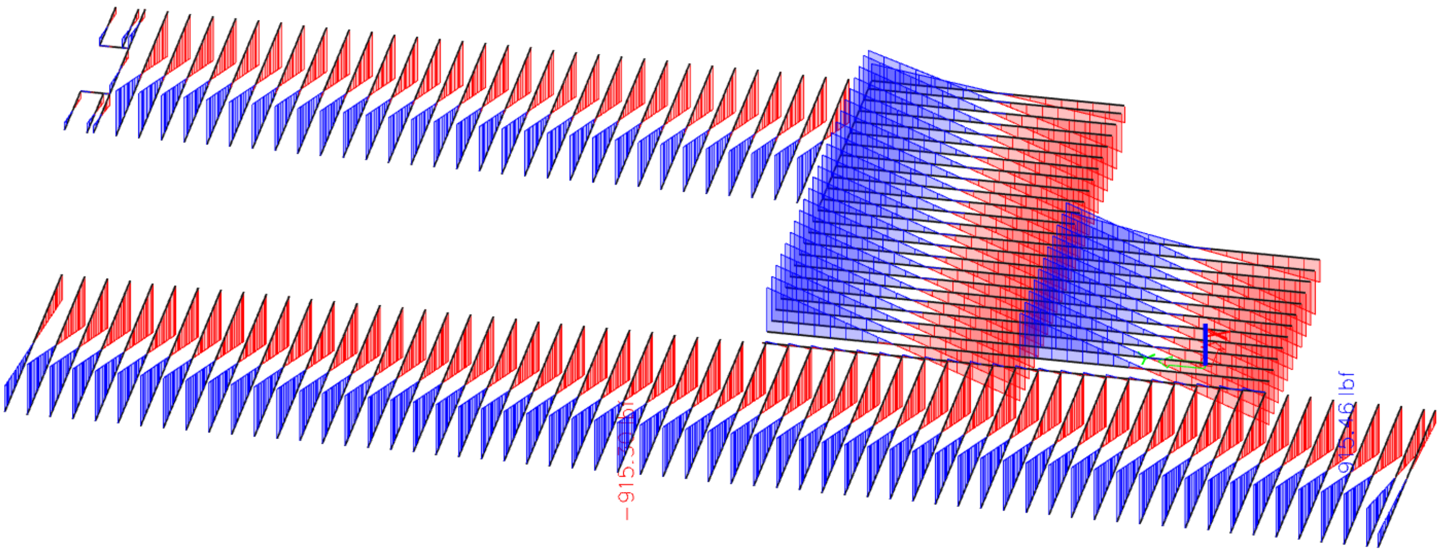
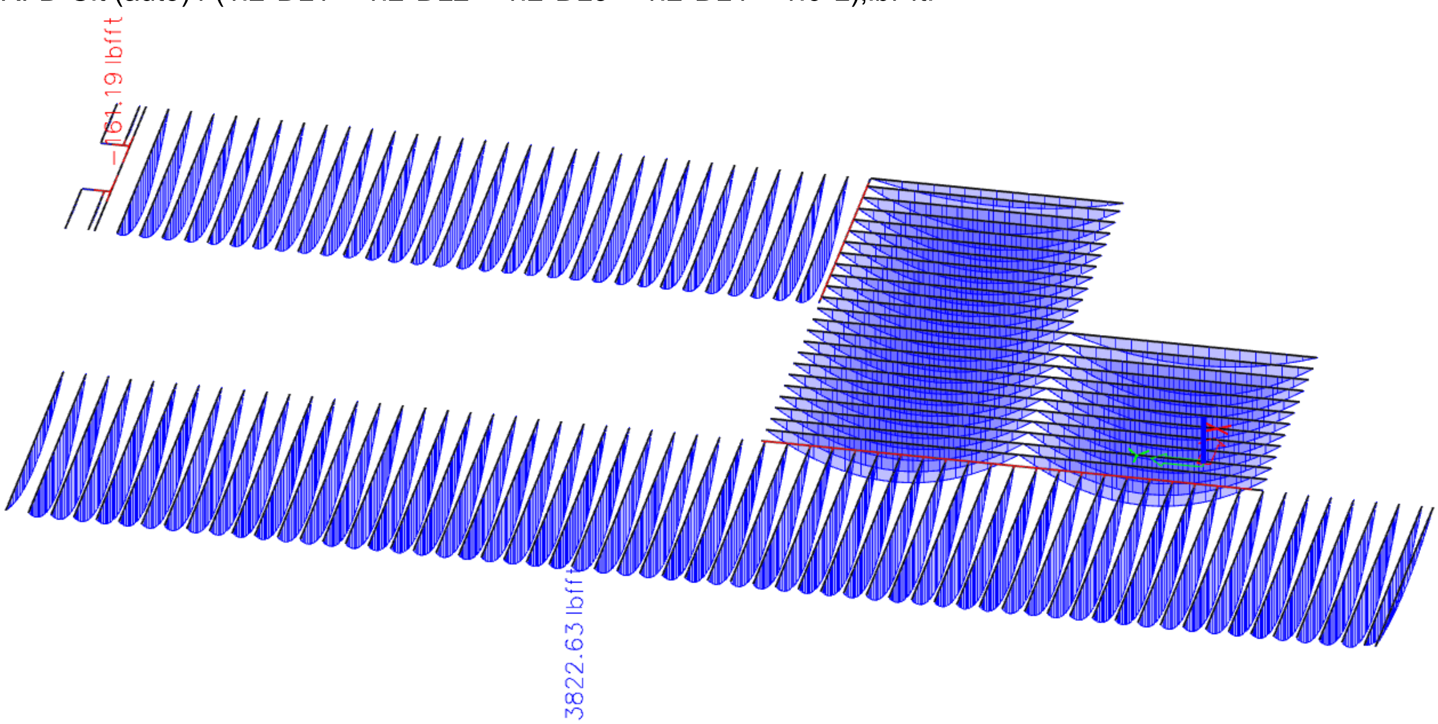
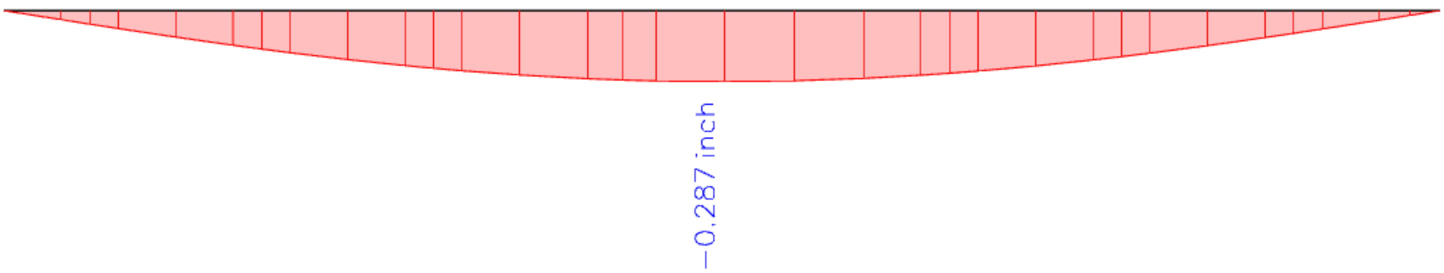


Diagram of moment My,
LRFD-Ult (auto)4 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L),lbf*ft.



Displacement of elements

Value: Uz - L, (inch) .



The maximum deflection is 0.287" according to table 1604.3 the code IBC 2021 - the deflection limits L/360. L = 16'-8.5"= 16'*12"+8.5"=200.5". 200.5"/360 =0.556"
0.287" < 0.556" <u>Deflection is OK!</u>

STEEL MEMBER B1031 CHECK (FLOOR JOIST)

AISI S100-16 LRFD Check

Member B1013	S1000S200-54	A1008 grade 54	LRFD-Ult (auto)	0.65

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

Internal forces		
Pu	0.00	lbf
Vux	0.00	lbf
Vuy	-3.70	lbf
Mut	-0.21	lbfft
Mux	3822.45	lbfft
Muy	0.00	lbfft

The critical check is on position 8.39 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

...::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	0.484	-32.657 -36.932	-	-	-	-	-	- -	- -	-	- -	-
3	1.717	-37.933 -37.933	-	-	-	-	-	- -	- -	-	- -	-
5	9.717	48.999 -36.932	0.75	18.295	16.274	1.735	0.503	- 4.890	1.303 1.486	-	- -	-
7	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
9	0.484	48.999 44.723	0.91	0.461	165.766	0.544	1.000	0.223 -	- -	-	- -	-

Table of values		
Sxe	1.690	inch^3
Mnxo	7041.71	lbfft
Resistance factor	0.90	
Unity check	0.60	-

...::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	0.484	-32.657 -36.932	-	-	-	-	-	- -	- -	-	- -	-
3	1.717	-37.933 -37.933	-	-	-	-	-	- -	- -	-	- -	-
5	9.717	48.999 -36.932	0.75	18.295	16.274	1.735	0.503	- 4.890	1.303 1.486	-	- -	-
7	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
9	0.484	48.999 44.723	0.91	0.461	165.766	0.544	1.000	0.223 -	- -	-	- -	-

Lateral-Torsional Buckling Strength

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

Table of values		
Lltb	1 ft 7.366 in	ft
Sigma,ey	342.428	ksi
Kt	1.00	
Lt	1 ft 7.366 in	ft
Sigma,t	456.944	ksi
Cb	1.01	
Sfx	2.270	inch^3
Fcre	578.686	ksi

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

Distortional Buckling Strength

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

Table of values		
Sfy	2.270	inch^3
My	9458.42	lbfft
L	1 ft 7.366 in	ft
Beta	1.04	
k,phi,fe	210.56	lbf
k,phi,we	186.02	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	inch^2
k,phi,wg	0.004	inch^2
Fd	36.551	ksi
Sf	2.270	inch^3

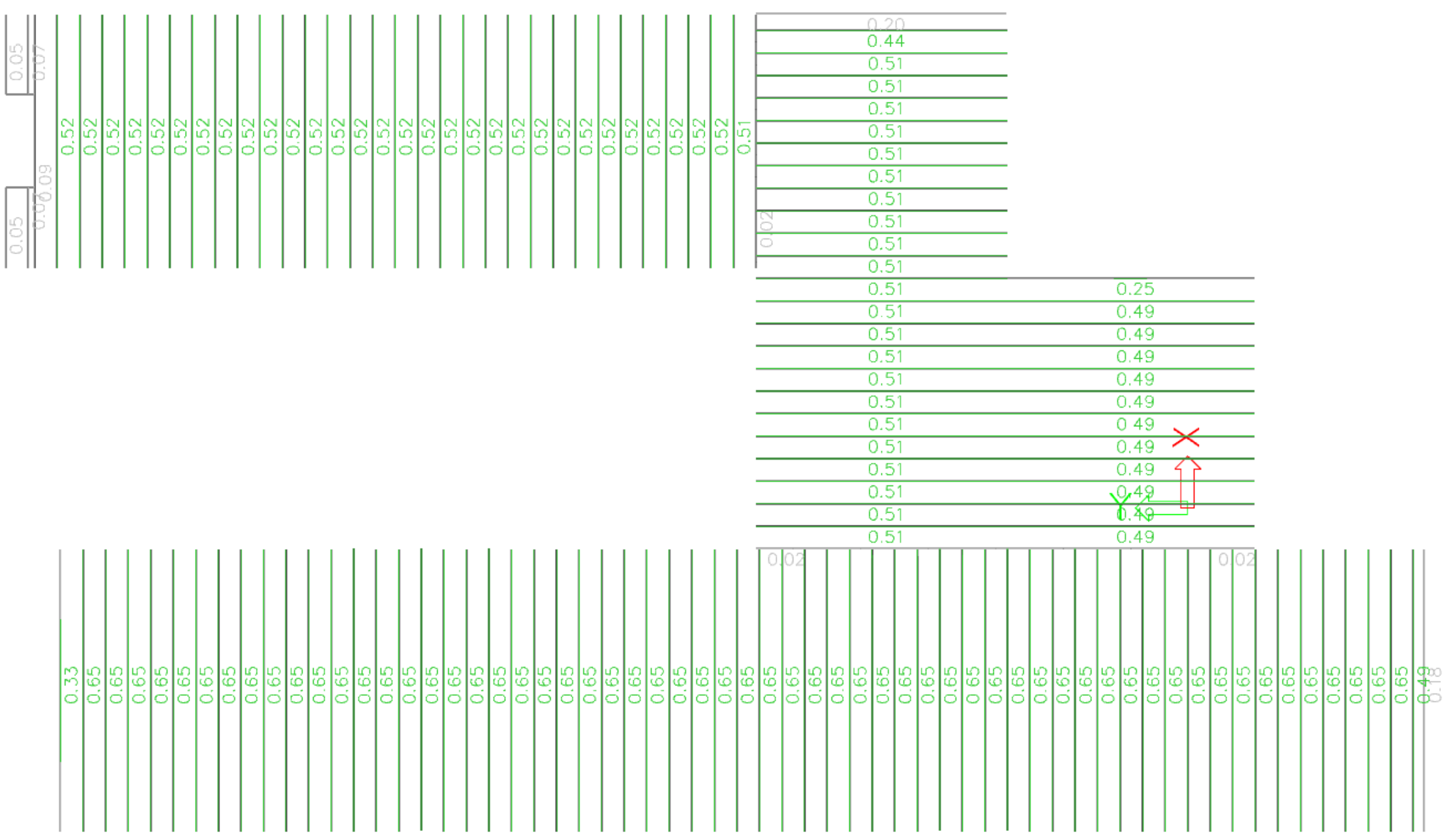
Table of values		
Mcrd	6914.30	lbfft
Lambda,d	1.17	
Mn	6565.78	lbfft
Resistance factor	0.90	
Unity check	0.65	-

Data		
Lm	1 ft 7.366 in	ft
Lcr	1 ft 8.136 in	ft
h0	10.000	inch
Ixf	0.003	$inch^4$
Iyf	0.049	$inch^4$
Ixyf	-0.007	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.691	inch
hxf	-1.239	inch
Af	0.135	$inch^2$
y0f	0.068	inch
Ksi,web	2.00	

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

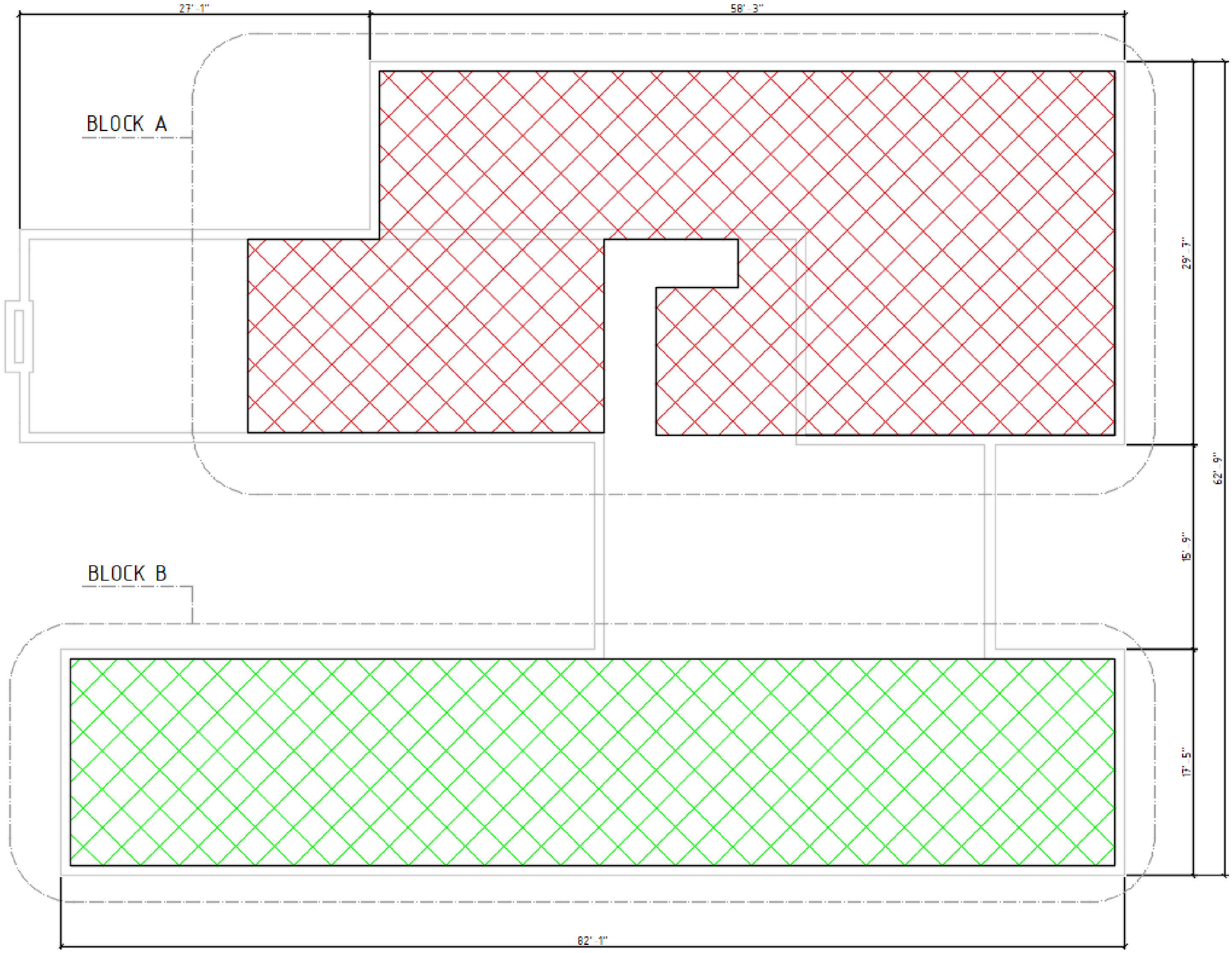
The member satisfies the check !

Unity check

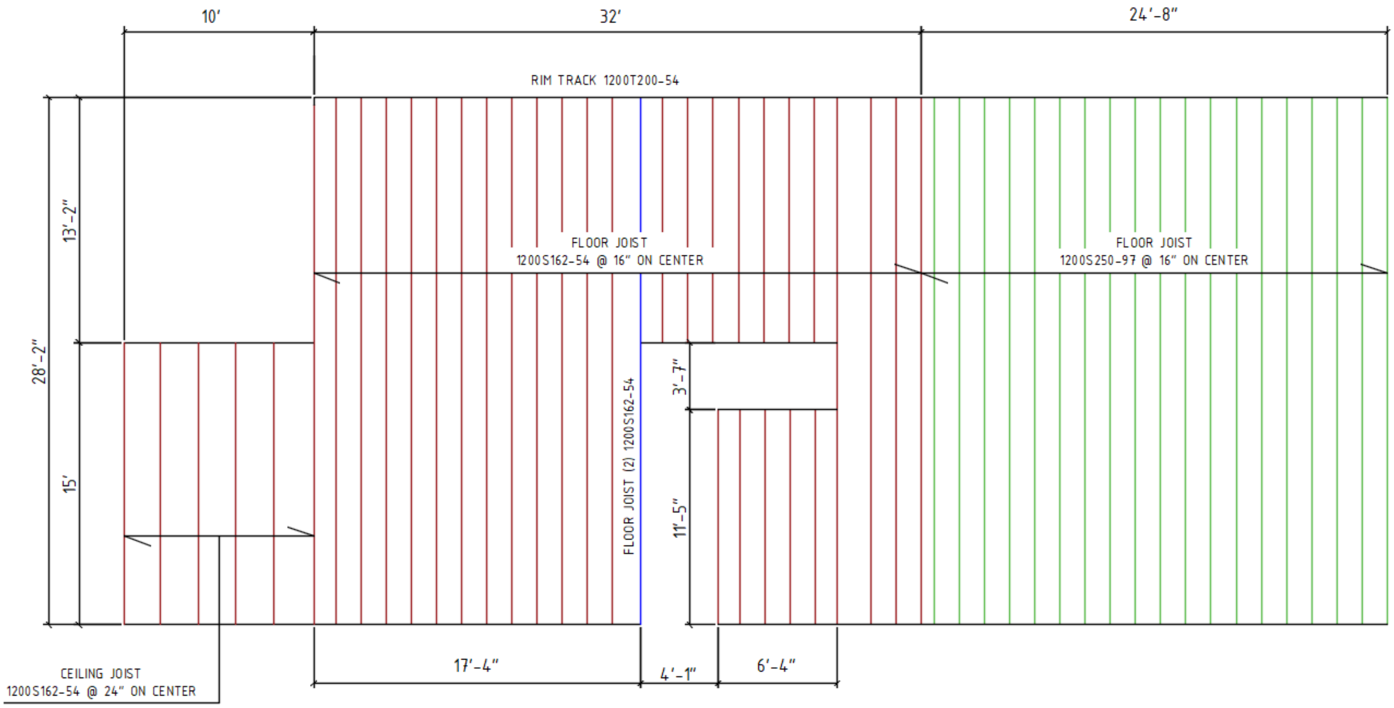


2.4.2 2-ND FLOOR JOIST & CEILING JOIST STRUCTURAL ANALYSIS

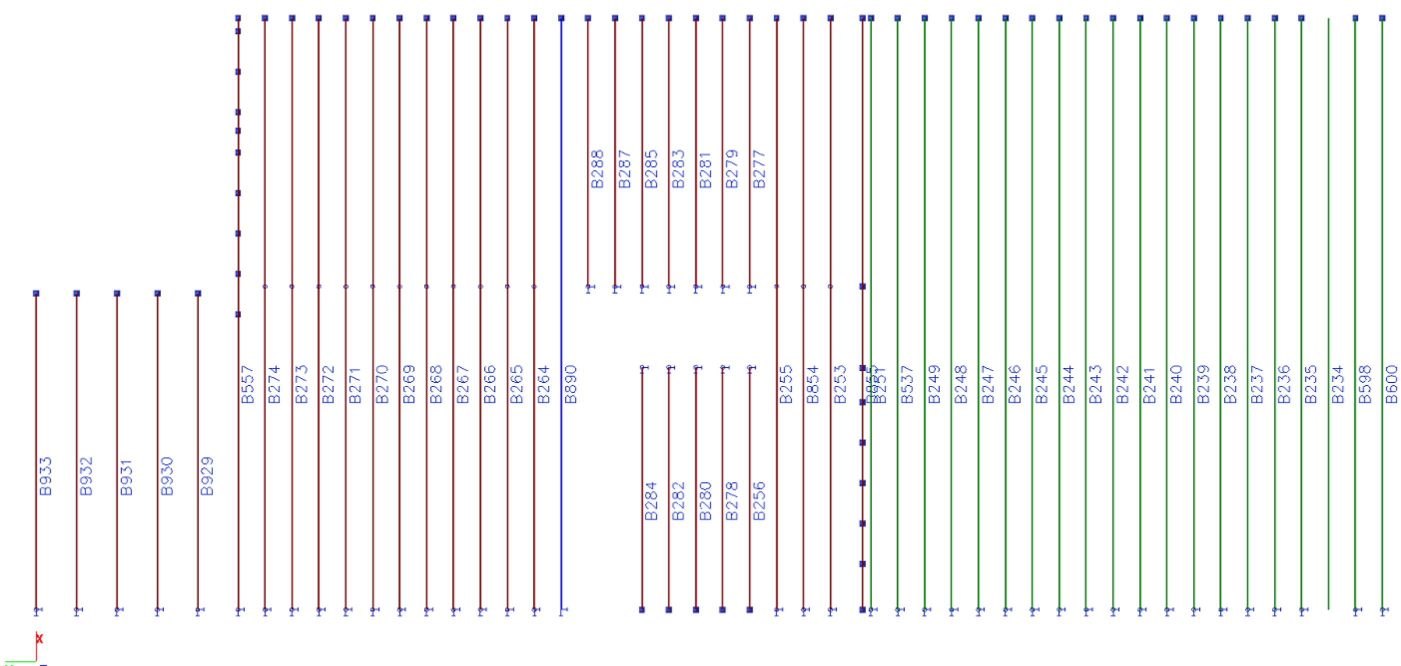
General scheme



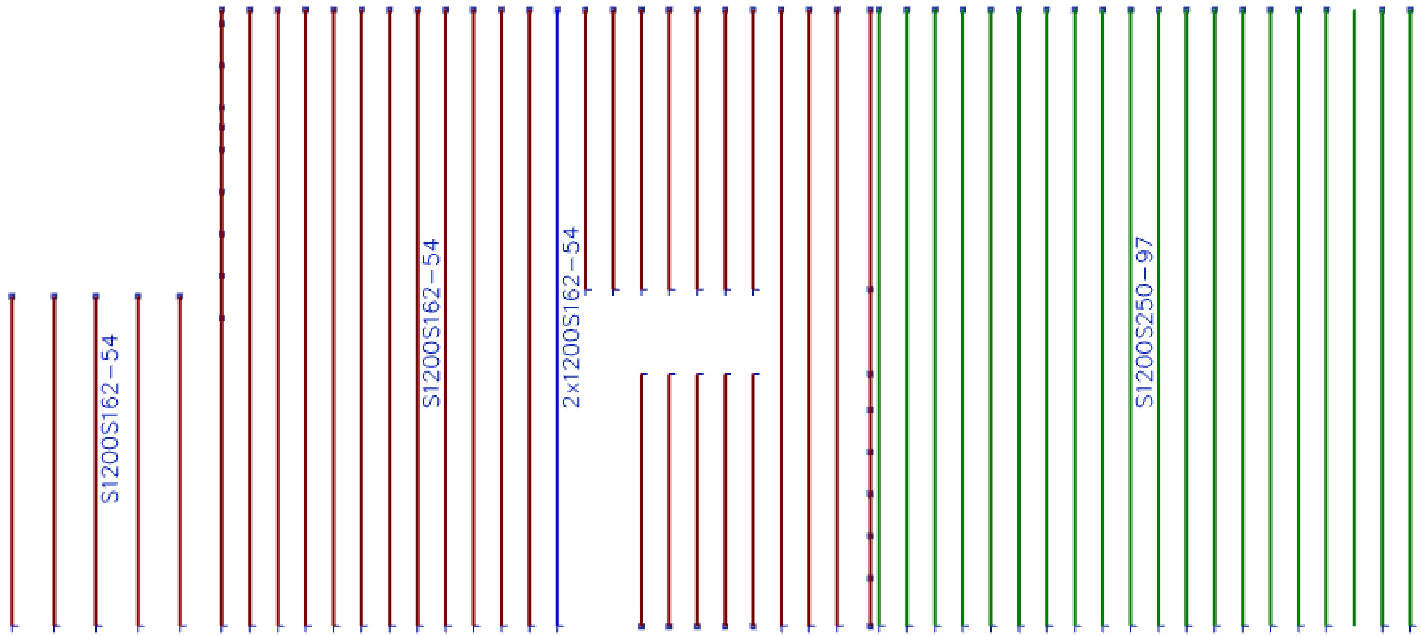
BLOCK A



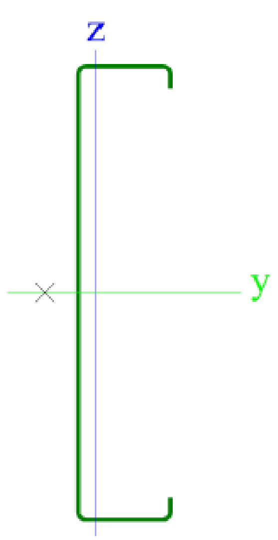
Member numbers

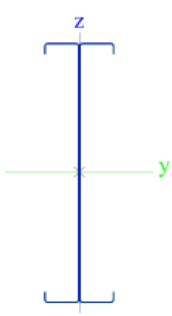


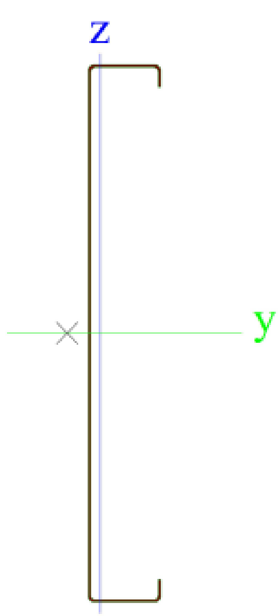
Cross-sections of the block A



Cross-sections properties

CS1		
Type	S1200S250-97	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	1.788	
A_y [inch^2], A_z [inch^2]	0.505	1.207
A_L [inch^2/inch], A_D [inch^2/inch]	3.52e+01	3.52e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.513	6.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	34.346	1.123
i_y [inch], i_z [inch]	4.383	0.792
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	5.666	0.565
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	6.947	0.844
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3.47e+02	3.47e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	4.22e+01	4.22e+01
d_y [inch], d_z [inch]	-1.343	0.000
I_t [inch^4], I_w [inch^6]	0.006	32.734
β_y [inch], β_z [inch]	0.000	14.929
Picture		

CS2		
Type	2x1200S162-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	1.791	
A_y [inch^2], A_z [inch^2]	0.376	1.317
A_L [inch^2/inch], A_D [inch^2/inch]	4.01e+01	4.01e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-0.581	0.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	31.447	0.552
i_y [inch], i_z [inch]	4.190	0.555
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	5.241	0.340
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	6.636	0.479
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3.32e+02	3.32e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.40e+01	2.40e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.006	21.452
β_y [inch], β_z [inch]	0.000	0.000
Picture		

CS3		
Type	S1200S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [$inch^2$]	0.899	
A_y [$inch^2$], A_z [$inch^2$]	0.188	0.659
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	3.18e+01	3.18e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.267	6.000
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	15.835	0.212
i_y [inch], i_z [inch]	4.198	0.486
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	2.621	0.156
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	3.318	0.223
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.66e+02	1.66e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.11e+01	1.11e+01
d_y [inch], d_z [inch]	-0.739	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.001	6.340
$β_y$ [inch], $β_z$ [inch]	0.000	19.591
Picture		

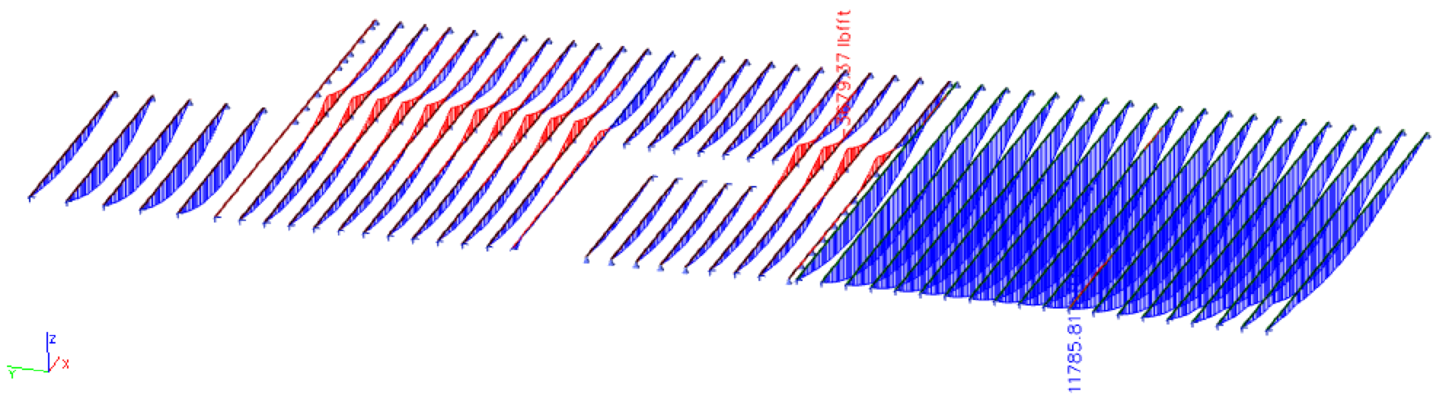
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction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

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

Maximum force diagram

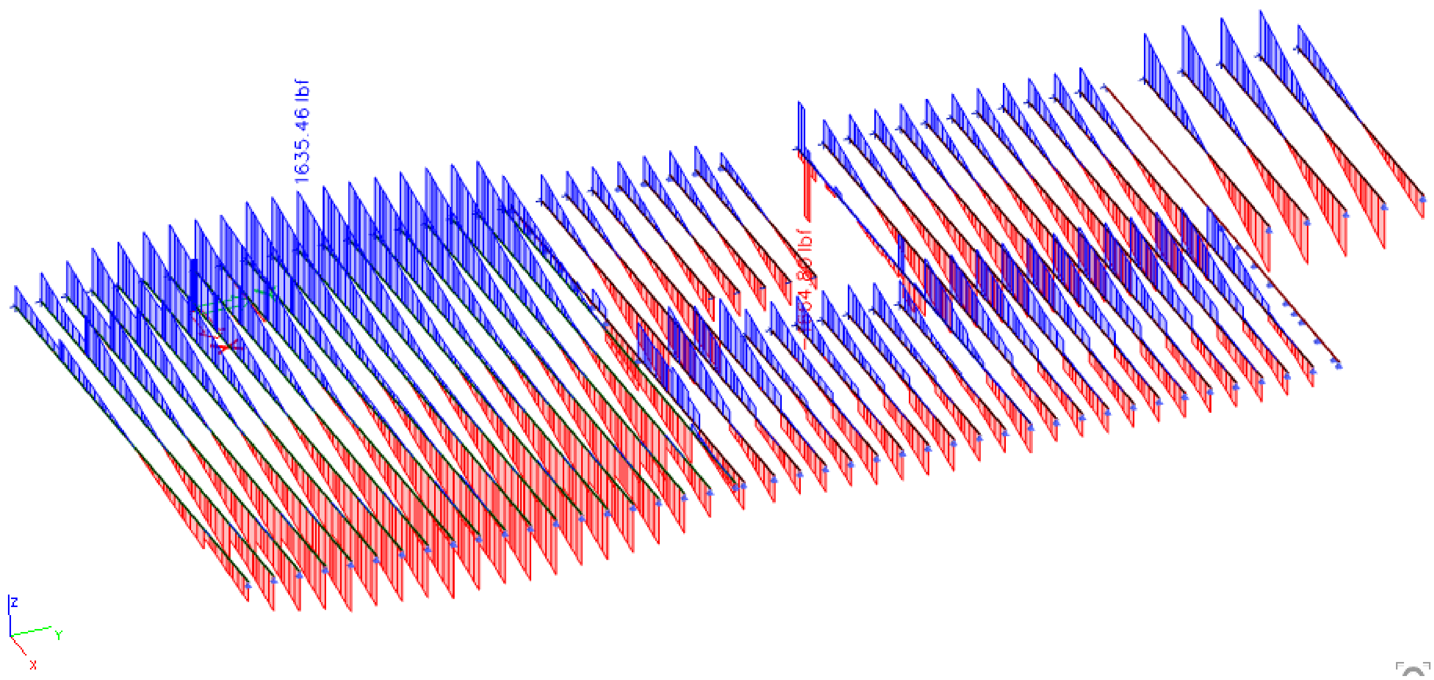
Diagram of moment My,

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



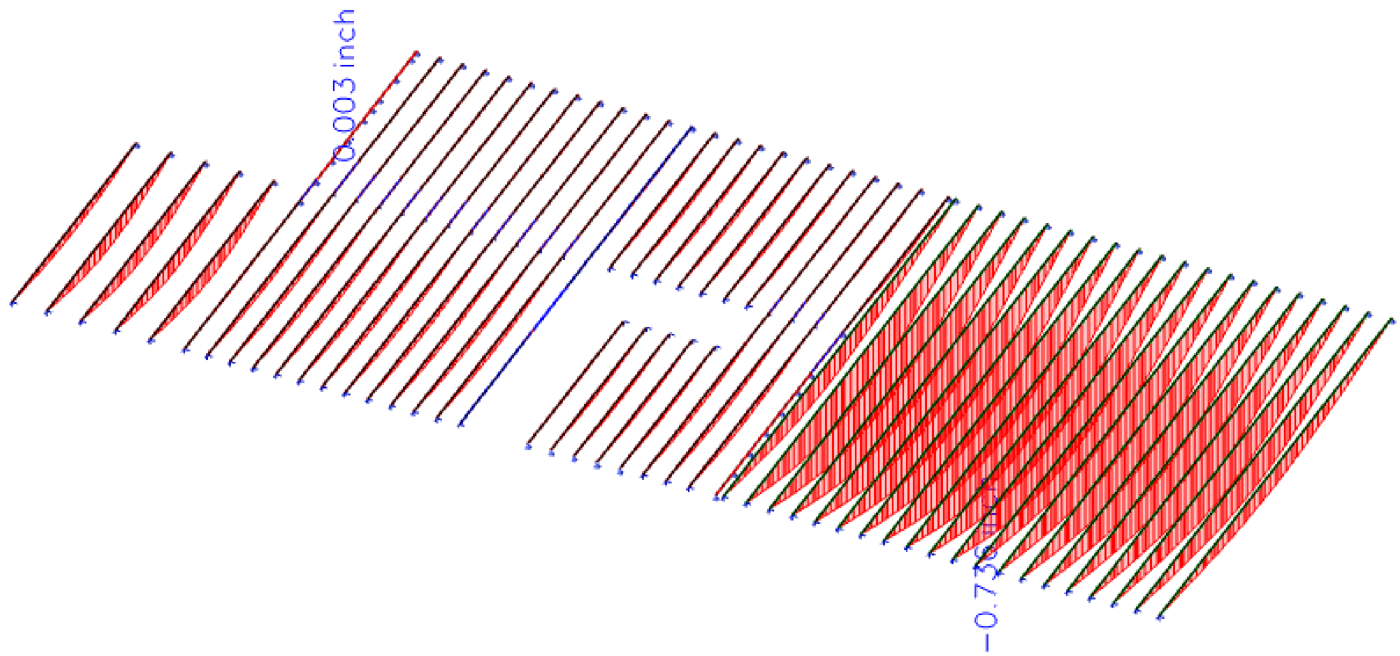
Shear force diagram Vz ,

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



Displacement of elements

Value: Uz - L, (inch) .



The maximum deflection is 0.736" according to table 1604.3 the code IBC 2019 - the deflection limits L/360. L = 28' 2"= 28'*12"+2"=338"/360 =0.938"

0.736" < 0.938" **<u>Deflection is OK!</u>**

STEEL MEMBER B241 CHECK (FLOOR JOIST)

AISI S100-16 LRFD Check

Member B241	S1200S250-97	A1008 grade 54	LRFD-Ult (auto)	0.69

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

The critical check is on position 14.67 ft

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

Internal forces		
Pu	0.00	lbf
Vux	0.00	lbf
Vuy	-31.62	lbf
Mut	-0.00	lbfft
Mux	11737.23	lbfft
Muy	0.01	lbfft

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.371	-39.865 -42.807	-	-	-	-	-	- -	-	-	- -	-
3	1.992	-44.421 -44.421	-	-	-	-	-	- -	-	-	- -	-
5	11.492	48.386 42.807	0.88	21.159	43.447	1.055	0.750	- 8.619	2.219 2.354	-	- -	-
7	1.992	50.000 50.000	1.00	2.513	171.783	0.540	1.000	1.992 -	0.348 1.644	30.83	0.001 0.000	0.129
9	0.371	48.386 45.444	0.94	0.452	891.137	0.233	1.000	0.129 -	-	-	- -	-

Table of values		
Sxe	5.016	$inch^3$
Mnxo	20898.61	lbfft
Resistance factor	0.90	
Unity check	0.62	-

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

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

Table of values		
Lltb	2 ft 0.000 in	ft
Sigma,ey	312.051	ksi
Kt	1.00	
Lt	2 ft 0.000 in	ft
Sigma,t	422.265	ksi
Cb	1.00	
Sfx	5.724	$inch^3$
Fcre	528.460	ksi

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

Table of values		
Sfy	5.724	$inch^3$
My	23851.58	lbfft
L	1 ft 5.371 in	ft
Beta	1.01	
k,phi,fe	1031.33	lbf
k,phi,we	1067.36	lbf
k,phi	0.00	lbf
k,phi,fg	0.026	$inch^2$
k,phi,wg	0.015	$inch^2$
Fd	52.525	ksi
Sf	5.724	$inch^3$

Table of values		
Mcrd	25056.05	lbfft
Lambda,d	0.98	
Mn	18934.07	lbfft
Resistance factor	0.90	
Unity check	0.69	-

Data		
Lm	2 ft 0.000 in	ft
Lcr	1 ft 5.371 in	ft
h0	12.000	inch
Ixf	0.006	$inch^4$
Iyf	0.138	$inch^4$
Ixyf	-0.015	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.001	$inch^4$
x0f	0.870	inch
hxf	-1.499	inch
Af	0.273	$inch^2$
y0f	0.051	inch
Ksi,web	2.00	

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

The member satisfies the check !

STEEL MEMBER B269 CHECK (FLOOR JOIST)

AISI S100-16 LRFD Check

Member B269	S1200S162-54	A1008 grade 54	LRFD-Ult (auto)	0.73

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

The critical check is on position 16.00 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	-0.00	lbf
Vuy	1148.21	lbf
Mut	0.00	lbft
Mux	-3402.64	lbft
Muy	-0.00	lbft

...::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	0.359	49.179 46.577	0.95	0.449	293.467	0.409	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	11.717	49.179 35.851	0.73	17.795	10.887	2.125	0.422	- 4.942	1.325 1.533	-	- -	-
7	1.342	-36.673 -36.673	-	-	-	-	-	- -	- -	-	- -	-
9	0.359	-33.249 -35.851	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	1.903	$inch^3$
Mnxo	7928.51	lbft
Resistance factor	0.90	
Unity check	0.48	-

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

Table of values		
Lltb	2 ft 0.000 in	ft
Sigma,ey	117.198	ksi
Kt	1.00	
Lt	2 ft 0.000 in	ft
Sigma,t	191.216	ksi
Cb	1.34	
Sfx	2.639	$inch^3$
Fcre	293.296	ksi

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

Distortional Buckling Strength

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

Table of values		
Sfy	2.639	$inch^3$
My	10996.39	lbfft
L	1 ft 3.985 in	ft
Beta	1.21	
k,phi,fe	165.43	lbf
k,phi,we	196.28	lbf
k,phi	0.00	lbf
k,phi,fg	0.005	$inch^2$
k,phi,wg	0.010	$inch^2$
Fd	29.017	ksi

Table of values		
Sf	2.639	$inch^3$
Mcrd	6381.61	lbfft
Lambda,d	1.31	
Mn	6973.08	lbfft
Resistance factor	0.90	
Unity check	0.54	-

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

Number of compressed flanges: 1

Critical flange contains Initial shape parts: 2, 3, 1

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

Shear Strength

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

Shear force Vy

Element ID	Aw [inch²]	Vn [lbf]
3	0.000	0.00
5	0.663	2166.97
7	0.000	0.00

Table of values		
Vn,y	2166.97	lbf
Resistance factor	0.95	
Unity check	0.56	-

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

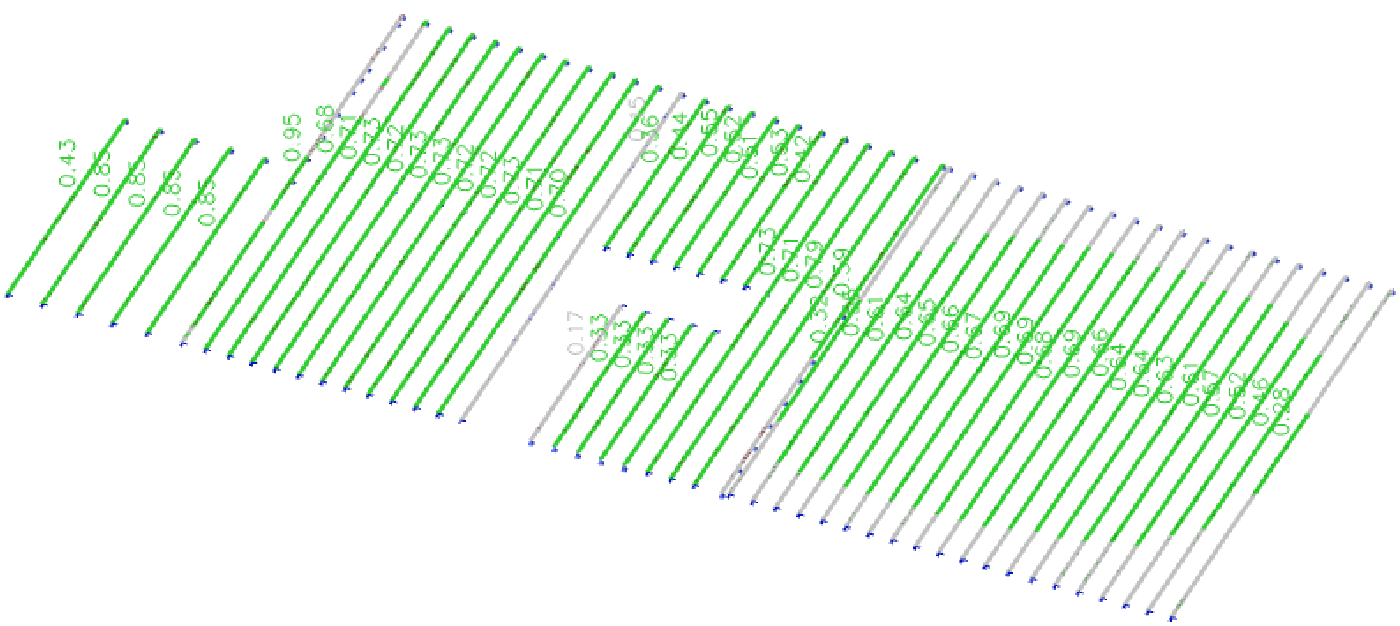
Table of values		
Mnxo	7928.51	lbfft
Vny	2166.97	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.23+0.31) = 0.73

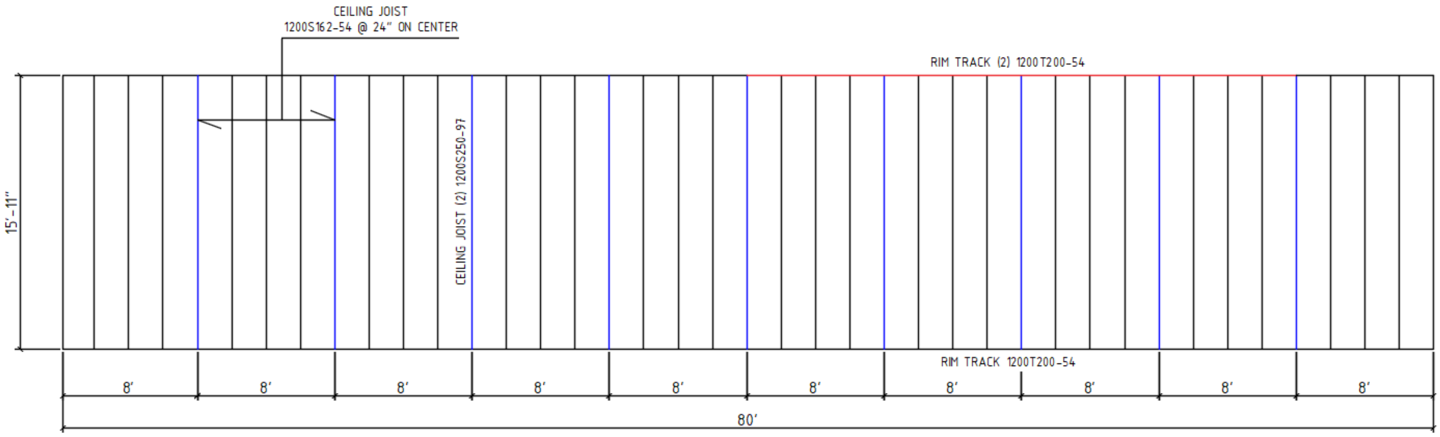
Note: Web Crippling has been ignored due to user input

The member satisfies the check !

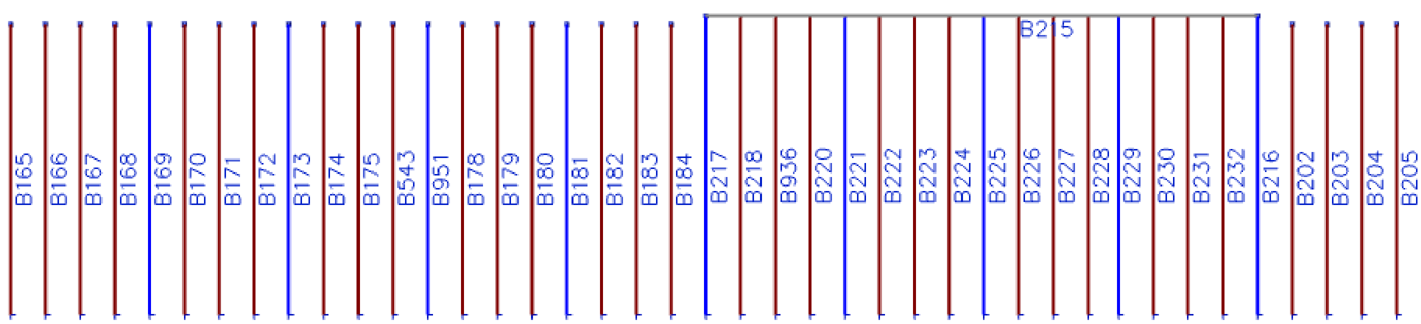
Unity check



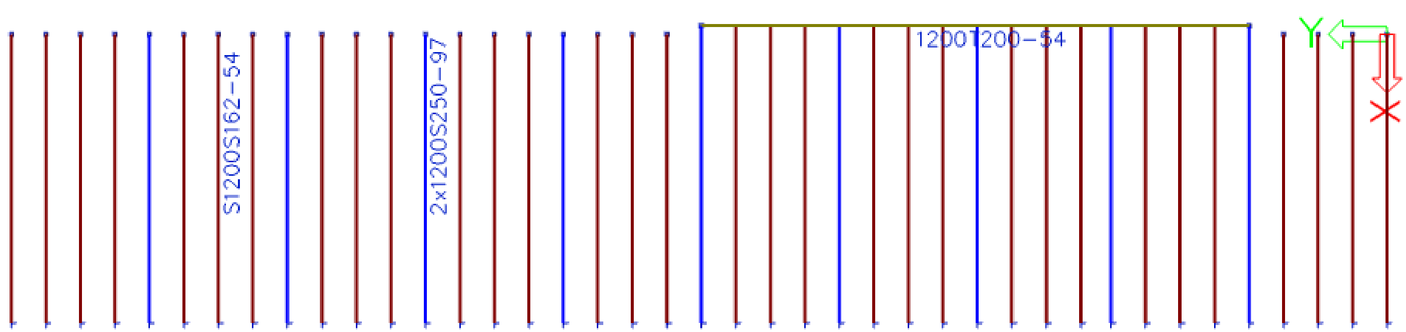
BLOCK B



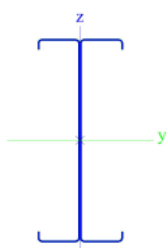
Member numbers

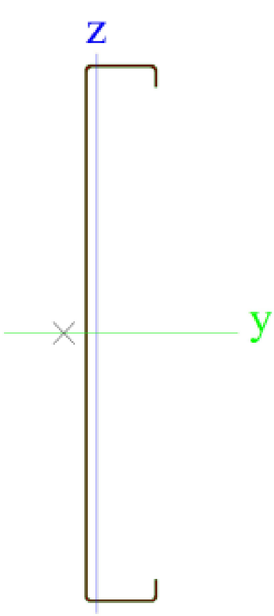


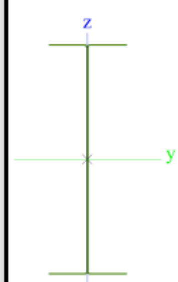
Cross-sections of members



Cross-sections properties

CS7		
Type	2x1200S250-97	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	3.557	
A_y [inch^2], A_z [inch^2]	1.023	2.413
A_L [inch^2/inch], A_D [inch^2/inch]	4.74e+01	4.74e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-3.521	0.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	67.996	3.181
i_y [inch], i_z [inch]	4.372	0.946
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	11.333	1.272
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	13.893	1.824
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	6.95e+02	6.95e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.12e+01	9.12e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.038	125.028
β_y [inch], β_z [inch]	0.000	0.000
Picture		

CS3		
Type	S1200S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.899	
A_y [inch^2], A_z [inch^2]	0.188	0.659
A_L [inch^2/inch], A_D [inch^2/inch]	3.18e+01	3.18e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.267	6.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	15.835	0.212
i_y [inch], i_z [inch]	4.198	0.486
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	2.621	0.156
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	3.318	0.223
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.66e+02	1.66e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.11e+01	1.11e+01
d_y [inch], d_z [inch]	-0.739	0.000
I_t [inch^4], I_w [inch^6]	0.001	6.340
β_y [inch], β_z [inch]	0.000	19.591
Picture		

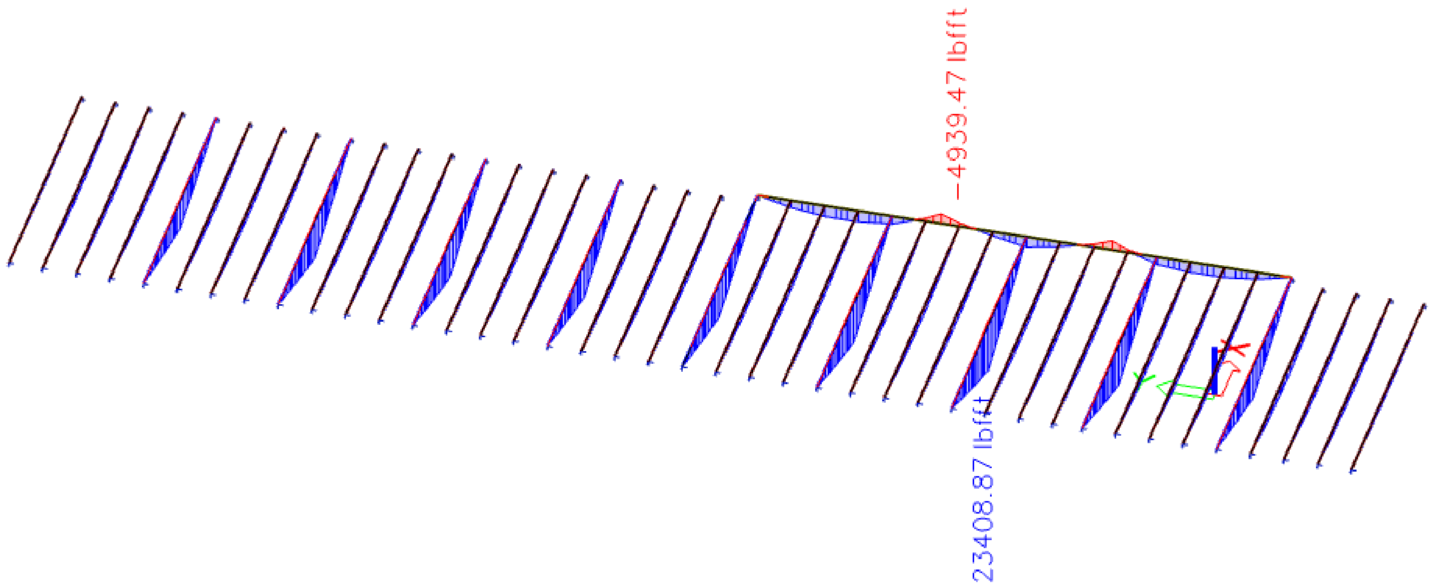
CS4		
Type	(2) 1200T200-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour	■	
A [$inch^2$]	1.787	
A_y [$inch^2$], A_z [$inch^2$]	0.418	1.324
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	4.00e+01	4.00e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-0.025	-0.001
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	31.589	0.605
i_y [inch], i_z [inch]	4.204	0.582
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	5.264	0.303
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	6.636	0.491
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3.32e+02	3.32e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.45e+01	2.45e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.006	21.529
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

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

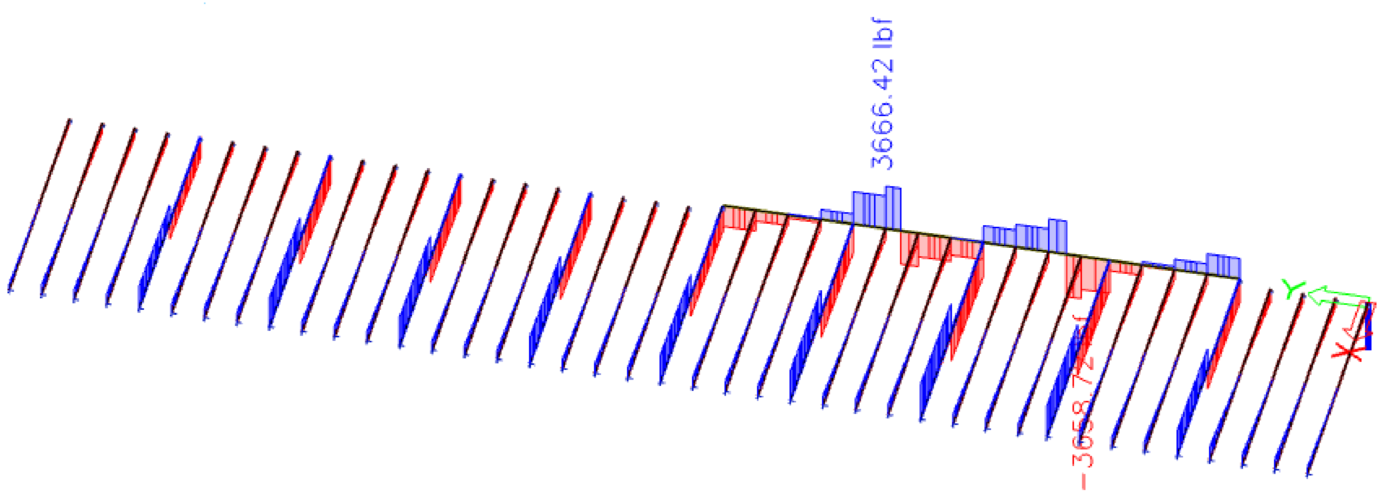
Explanations of symbols	
i	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the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

Maximum force diagram

Diagram of moment My,
LRFD-Ult (auto)4 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L), lbf.

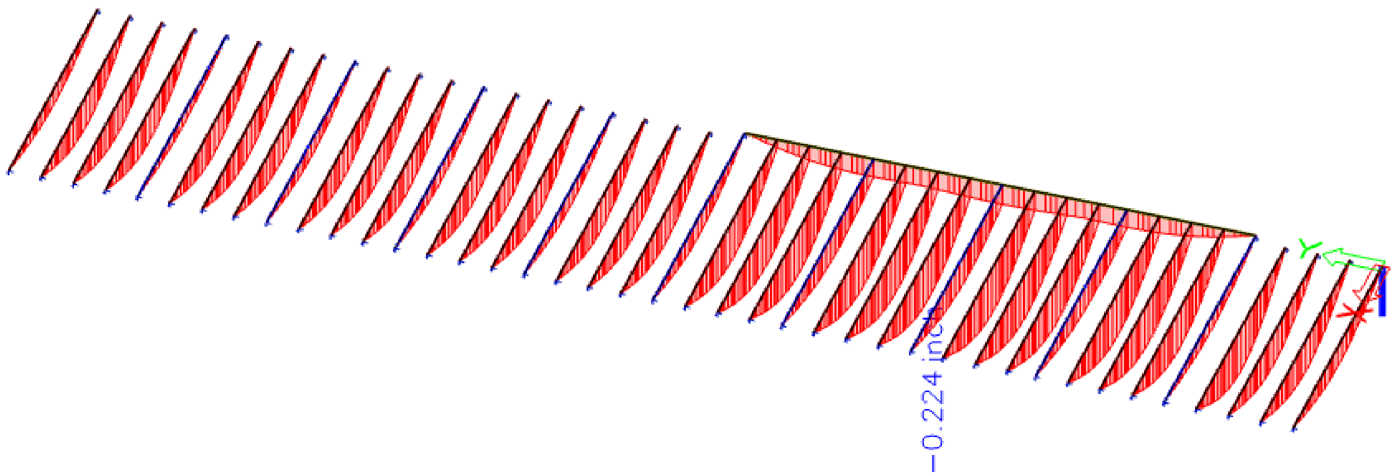


Shear force diagram Vz ,
LRFD-Ult (auto)4 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L), lbf.



Displacement of elements

Value: Uz - L, (inch) .



The maximum deflection is 0.224" according to table 1604.3 the code IBC 2019 - the deflection limits L/360. L = 16'= 16'*12"=192"/360 =0.533"

0.224" < 0.533" **<u>Deflection is OK!</u>**

STEEL MEMBER B224 CHECK (CEILING JOIST)

AISI S100-16 LRFD Check

Member B224	S1200S162-54	A1008 grade 54	LRFD-Ult (auto)	0.62

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

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

The critical check is on position 8.39 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	20.84	lbf
Mut	-0.21	lbfft
Mux	3651.43	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	0.359	-33.249 -35.851	-	-	-	-	-	- -	- -	-	- -	-
3	1.342	-36.673 -36.673	-	-	-	-	-	- -	- -	-	- -	-
5	11.717	49.179 35.851	0.73	17.795	10.887	2.125	0.422	- 4.942	1.325 1.533	-	- -	-
7	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
9	0.359	49.179 46.577	0.95	0.449	293.467	0.409	1.000	0.222 -	- -	-	- -	-

Table of values		
Sxe	1.903	$inch^3$
Mnxo	7928.51	lbfft
Resistance factor	0.90	
Unity check	0.51	-

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

Table of values		
Lltb	1 ft 7.366 in	ft
Sigma,ey	179.994	ksi
Kt	1.00	
Lt	1 ft 7.366 in	ft
Sigma,t	293.322	ksi
Cb	1.01	
Sfx	2.639	$inch^3$
Fcre	337.487	ksi

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

Distortional Buckling Strength

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

Table of values		
Sfy	2.639	$inch^3$
My	10996.39	lbfft
L	1 ft 3.985 in	ft
Beta	1.03	
k,phi,fe	165.43	lbf
k,phi,we	196.28	lbf
k,phi	0.00	lbf
k,phi,fg	0.005	$inch^2$
k,phi,wg	0.010	$inch^2$
Fd	24.572	ksi

Table of values		
Sf	2.639	inch
Mcrd	5404.13	lbfft
Lambda,d	1.43	
Mn	6519.91	lbfft
Resistance factor	0.90	
Unity check	0.62	-

Data		
Lm	1 ft 7.366 in	ft
Lcr	1 ft 3.985 in	ft
h0	12.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch
Ksi,web	2.00	

Number of compressed flanges: 1

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

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

Shear Strength

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

Shear force Vy

Element ID	Aw [inch²]	Vn [lbf]
3	0.000	0.00
5	0.663	2166.97
7	0.000	0.00

Table of values		
Vn,y	2166.97	lbf
Resistance factor	0.95	
Unity check	0.01	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

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

Unity check (Mx, Vy) = sqrt(0.26+0.00) = 0.51

Note: Web Crippling has been ignored due to user input

The member satisfies the check !

STEEL MEMBER B215 CHECK (CEILING RIM TRACK)

AISI S100-16 LRFD Check

Member B215	1200T200-54	A1008 grade 54	LRFD-Ult (auto)	0.45

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

The critical check is on position 20.83 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	0.06	lbf
Vux	-0.01	lbf
Vuy	3681.53	lbf
Mut	0.01	lbfft
Mux	-5028.61	lbfft
Muy	-0.32	lbfft

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	1.972	-36.238 -36.238	-		-	-	-	- -	-	-	- -	-
2	0.003	50.000 49.982	1.00	4.001	51328150.136	0.001	1.000	- 0.003	0.001 0.001	-	- -	-
3	1.972	50.000 50.000	1.00	0.430	9.290	2.320	0.390	0.769 -	-	-	- -	-
4	1.972	49.982 49.982	1.00	0.431	9.319	2.316	0.391	0.770 -	-	-	- -	-
5	0.003	-36.238 -36.257	-	-	-	-	-	- -	-	-	- -	-
6	1.972	-36.257 -36.257	-	-	-	-	-	- -	- -	-	- -	-
7	11.941	49.982 -36.238	0.73	17.717	41.744	1.094	0.730	- 8.719	2.341 2.714	-	- -	-

Table of values		
Sxe	3.486	inch^3
Mnxo	14523.89	lbfft
Resistance factor	0.90	
Unity check	0.38	-

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

Table of values		
Lltb	2 ft 0.000 in	ft
Sigma,ey	168.304	ksi
Kt	1.00	
Lt	2 ft 0.000 in	ft
Sigma,t	334.547	ksi
Cb	1.16	
Sfx	5.264	$inch^3$
Fcre	396.974	ksi

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

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

Element ID	Aw [$inch^2$]	Vn [lbf]
1	0.000	0.00
2	0.000	4.34
3	0.000	0.00
4	0.000	0.00
5	0.000	4.34
6	0.000	0.00
7	1.352	17010.81

Table of values		
Vn,y	17019.50	lbf
Resistance factor	0.95	
Unity check	0.23	-

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

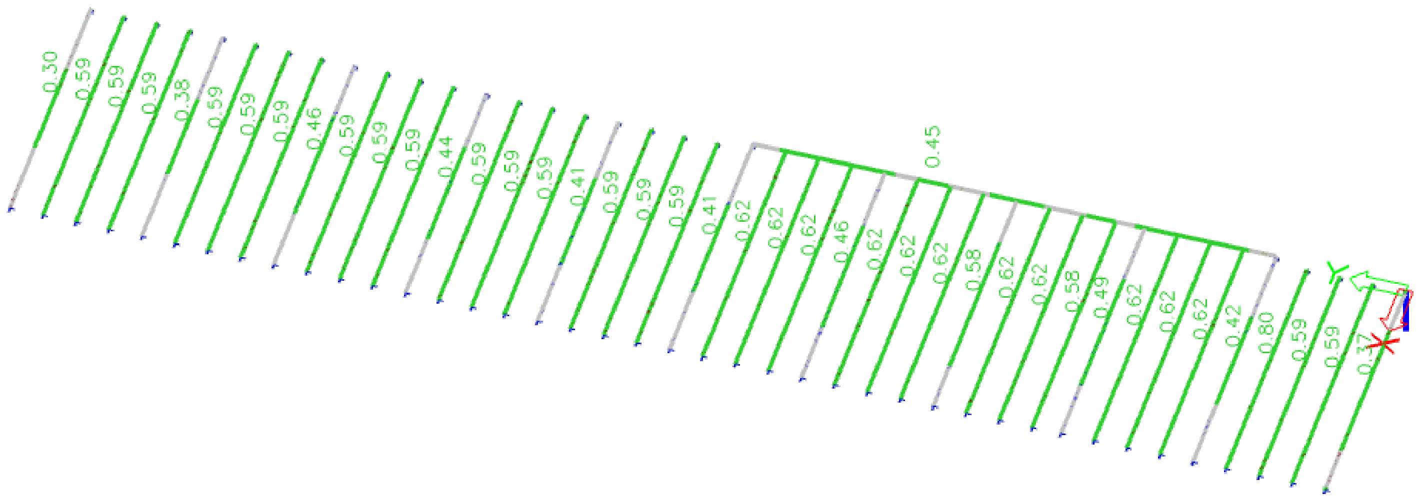
Table of values		
Mnxo	14523.89	lbfft
Vny	17019.50	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.15+0.05) = 0.45

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

The member satisfies the check !

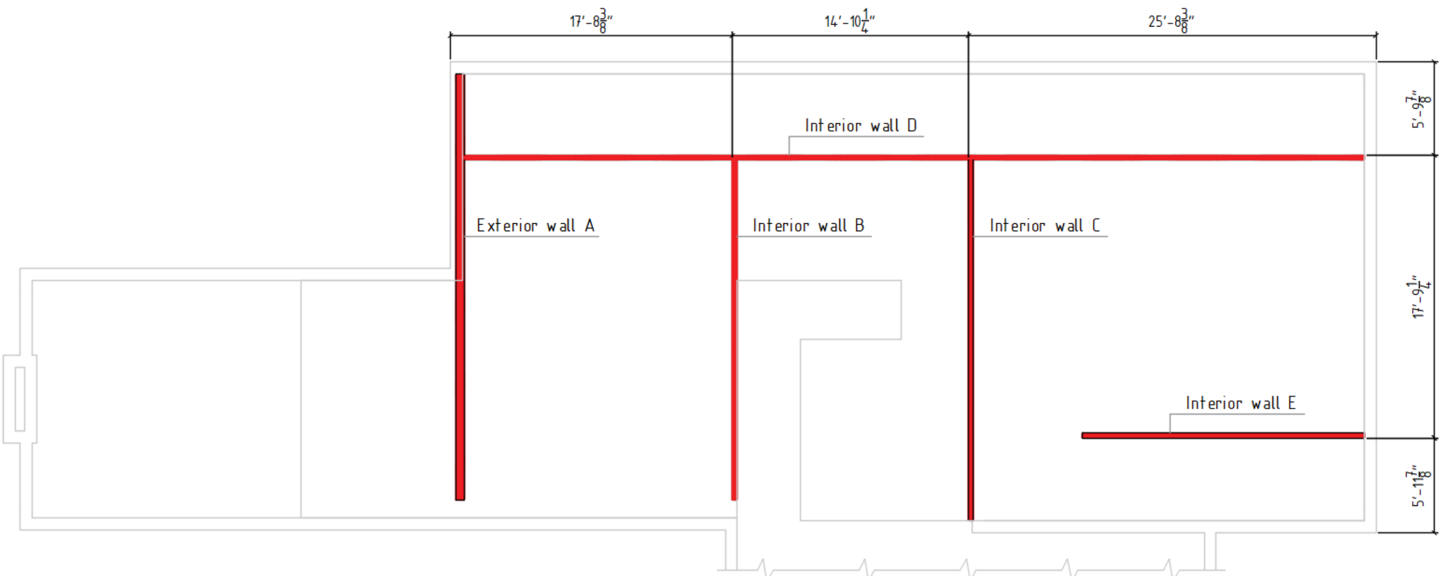
Unity check



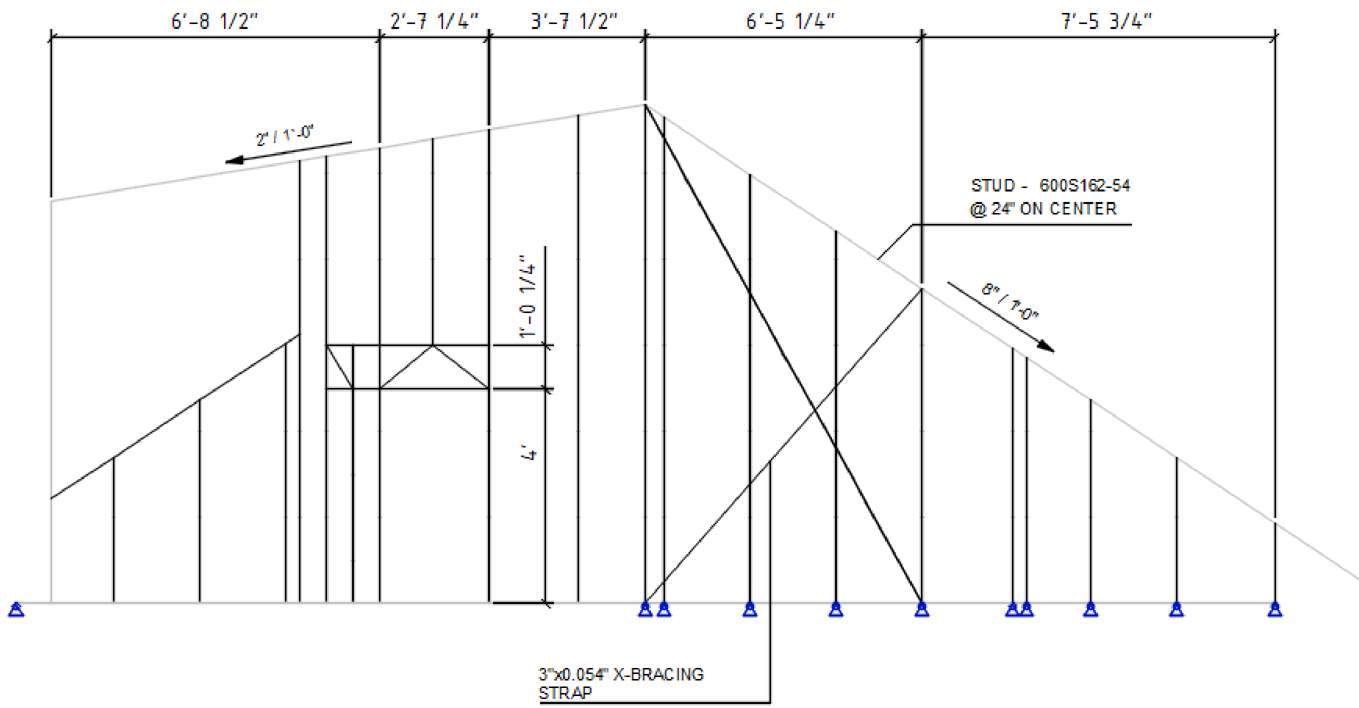
0.30
0.59
0.59
0.59
0.38
0.59
0.59
0.59
0.46
0.59
0.59
0.59
0.44
0.59
0.59
0.59
0.41
0.59
0.59
0.59
0.41
0.62
0.62
0.62
0.46
0.62
0.62
0.62
0.58
0.62
0.62
0.58
0.49
0.62
0.62
0.62
0.42
0.80
0.59
0.59
0.37
0.45

2.4.3 LIGHT GAUGE STEEL WALL STRUCTURAL ANALYSIS

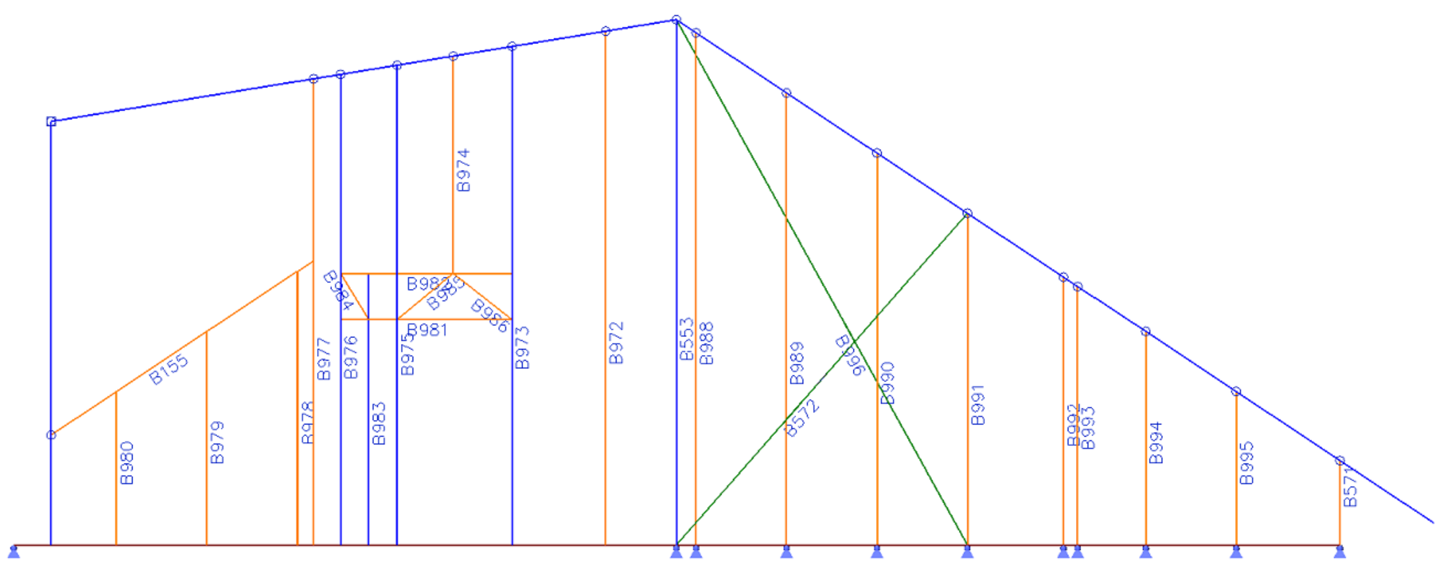
General scheme



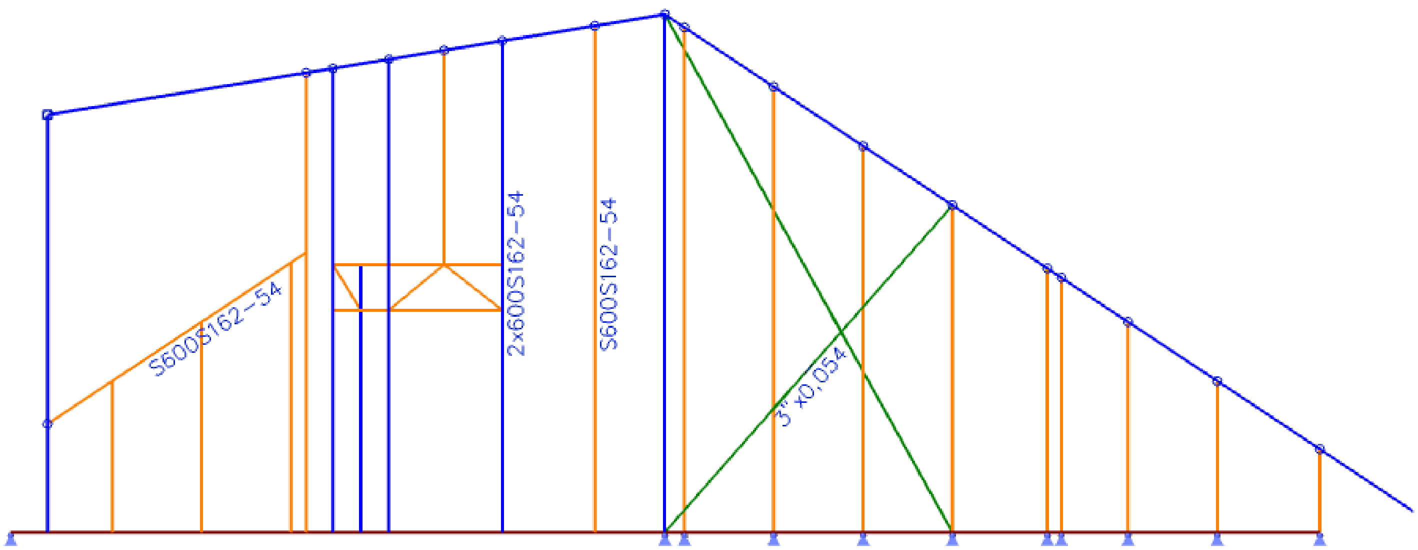
EXTERIOR WALL A



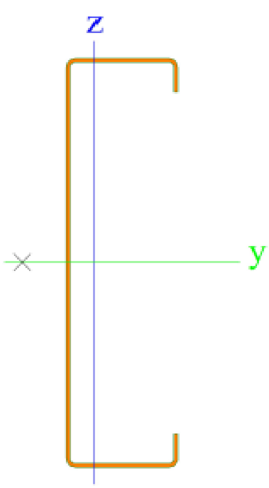
Member numbers

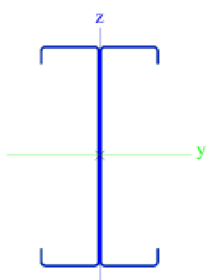


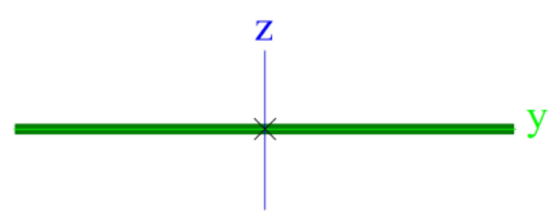
Cross-section walls element.



Section Properties:

CS17		
Type	S600S162-54	
Formcode	114- Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.559	
A_y [inch^2], A_z [inch^2]	0.186	0.346
A_L [inch^2/inch], A_D [inch^2/inch]	1.98e+01	1.98e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.413	3.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.886	0.181
i_y [inch], i_z [inch]	2.272	0.568
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.953	0.149
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.140	0.217
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.70e+01	5.70e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.08e+01	1.08e+01
d_y [inch], d_z [inch]	-1.056	0.000
I_t [inch^4], I_w [inch^6]	0.001	1.337
$β_y$ [inch], $β_z$ [inch]	0.000	6.508
Picture		

CS38		
Type	2x600S162-54	
Shape type	Thin walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [$inch^2$]	1.112	
A_y [$inch^2$], A_z [$inch^2$]	0.375	0.692
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	2.81e+01	2.81e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-0.436	0.000
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	5.718	0.551
i_y [inch], i_z [inch]	2.267	0.704
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	1.906	0.339
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	2.280	0.460
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.14e+02	1.14e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.30e+01	2.30e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.003	5.519
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

CS10		
Type	3"x0,054	
Shape type	Thin walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.162	
A_y [inch^2], A_z [inch^2]	0.135	0.135
A_L [inch^2/inch], A_D [inch^2/inch]	6.11e+00	6.11e+00
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1.500	0.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.000	0.122
i_y [inch], i_z [inch]	0.016	0.866
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.001	0.081
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.002	0.121
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.09e-01	1.09e-01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	6.08e+00	6.08e+00
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.000
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

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

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

Maximum force diagram

Axial force diagram N,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr), lbf.

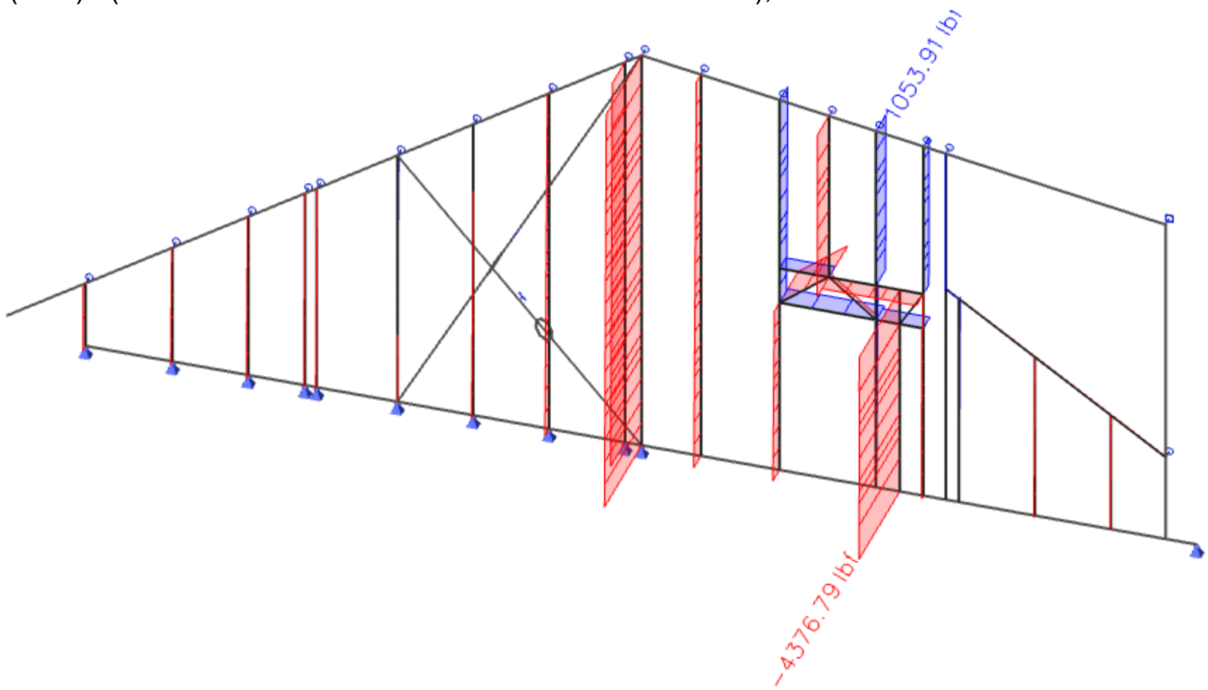
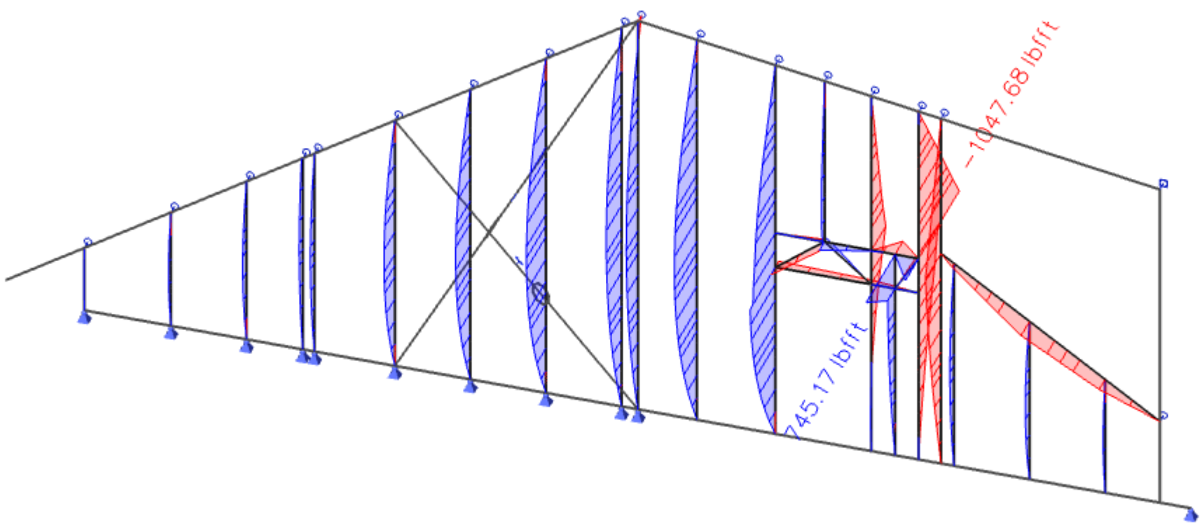
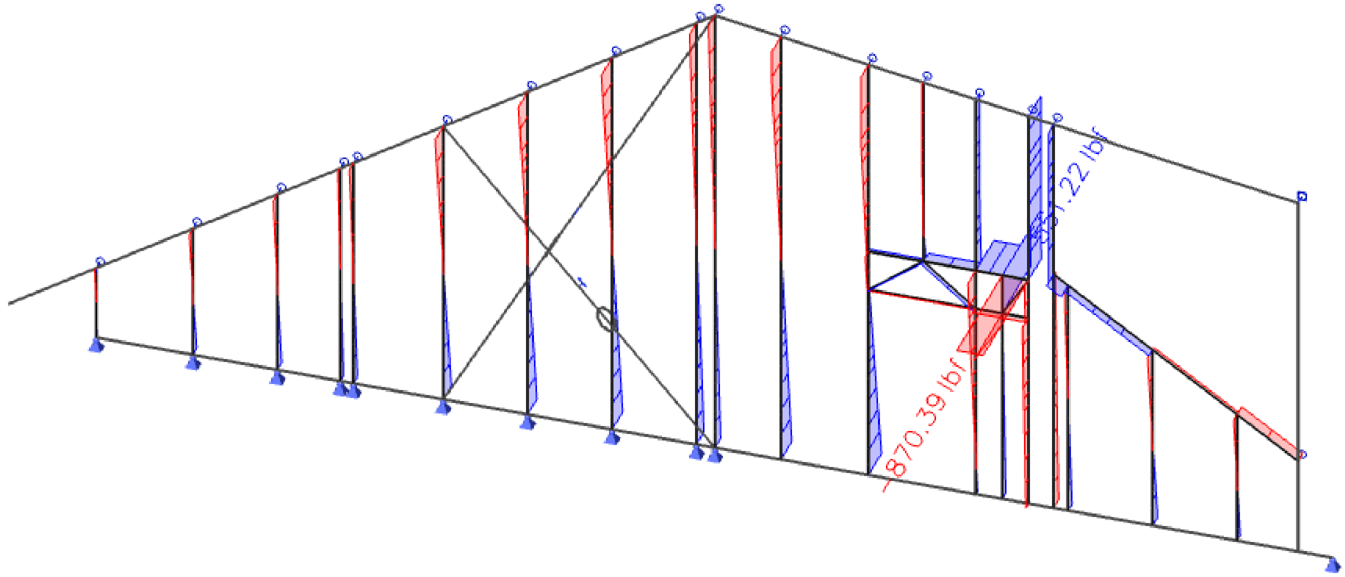


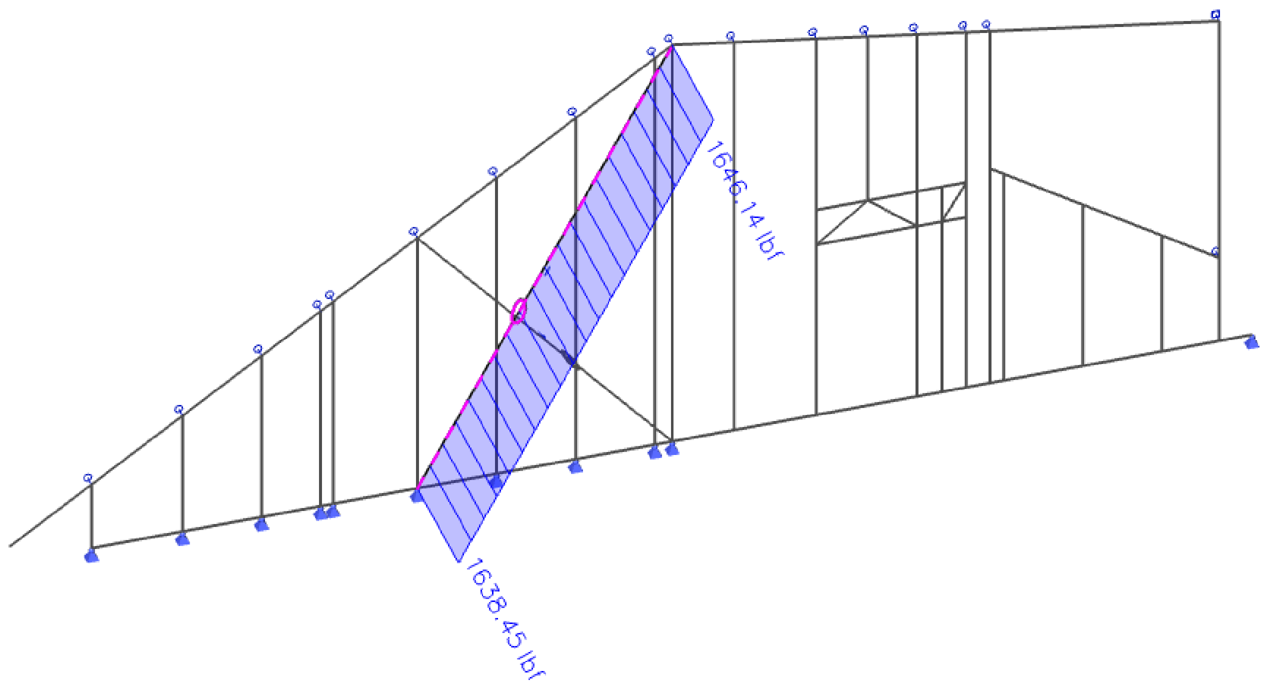
Diagram of moment My,
LRFD-Ult (auto)32 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + Wy+), lbf.



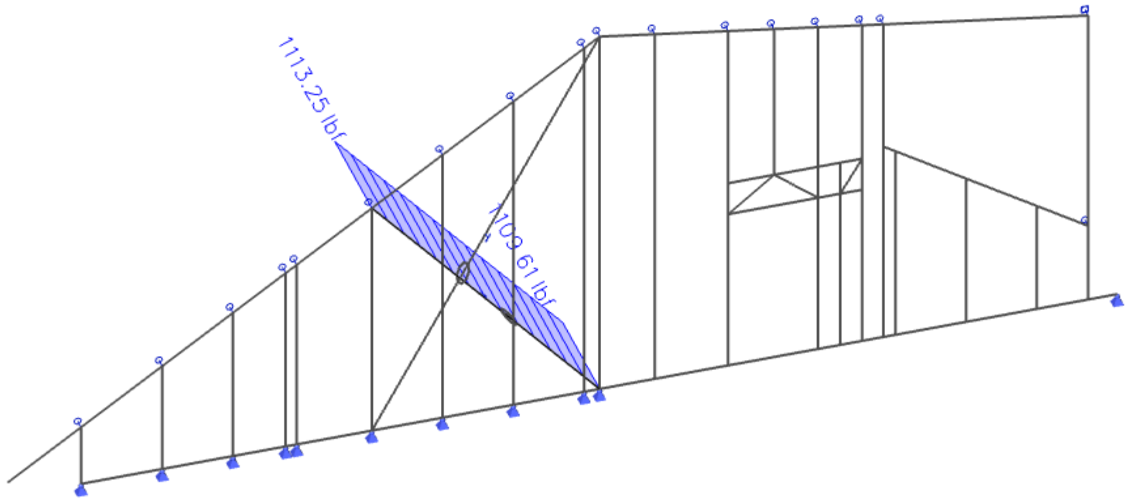
Shear force diagram Vz,
LRFD-Ult (auto)32 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + Wy+), lbf.



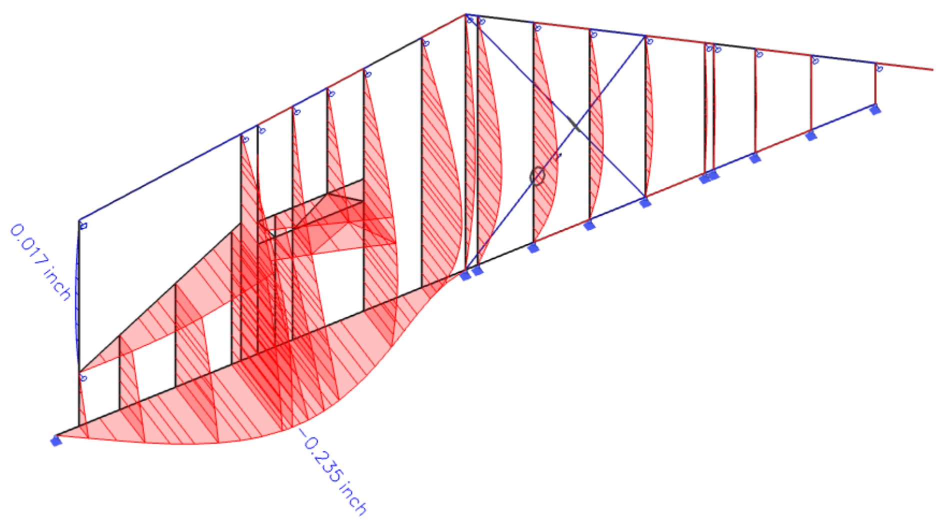
Axial force diagram for X-bracing N,
LRFD-Ult (auto)31 1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + Wx-, lbf.



Axial force diagram for X-bracing N,
LRFD-Ult (auto)35 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx+), lbf.



Displacement of elements
Value: Uy - Wy+, (inch) .



The maximum deflection is 0.235" according to table 1604.3 the code IBC 2019 - the deflection limits L/360. L = 14' 8"= 14' 8"*12"=176"/360 =0.488"
0.235" < 0.488" **<u>Deflection is OK!</u>**

STEEL MEMBER B976 CHECK (STUD)

AISI S100-16 LRFD Check

Member B976	2x600S162-54	A1008 grade 54	LRFD-Ult (auto)	0.30

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

The critical check is on position 6.00 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	0.51	kip
Vux	0.06	kip
Vuy	0.12	kip
Mut	0.00	kipft
Mux	-0.52	kipft
Muy	-0.27	kipft

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

Table of values		
Tn	55.61	kip
Resistance factor	0.90	
Unity check	0.01	-

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	5.943	50.000 -50.000	1.00	24.000	228.256	0.468	1.000	- 5.943	1.486 2.972	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 42.063	0.84	0.489	184.700	0.520	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 42.063	0.84	0.489	184.700	0.520	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	- 1.568	0.784 0.784	-	- -	-
8	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
9	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	1.906	inch^3
Mnxo	7.94	kipft
Resistance factor	0.90	
Unity check	0.07	-

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

Table of values		
Lltb	2.000	ft
Sigma,ey	246.310	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	443.555	ksi
Cb	1.22	
Sfx	1.906	inch^3
Fcre	560.865	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.
...::Flexural Strength about Y-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	-0.886 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	5.943	- -	-	-	-	-	-	- -	- -	-	- -	-
4	1.568	-0.886 -50.000	-	-	-	-	-	- -	- -	-	- -	-
5	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
6	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 0.886	0.02	7.860	268.370	0.432	1.000	- 1.568	0.526 1.042	-	- -	-
8	1.568	50.000 0.886	0.02	7.860	268.370	0.432	1.000	- 1.568	0.526 1.042	-	- -	-
9	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-

Table of values		
Sye	0.339	inch^3
Mnyo	1.41	kipft
Resistance factor	0.90	
Unity check	0.22	-

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

Table of values		
Sigma,ex	94.591	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	443.555	ksi
Cb	1.00	
Sfy	0.339	$inch^3$
Fcre	1594.636	ksi

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

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

Element ID	Aw [$inch^2$]	Vn [kip]
1	0.000	0.00
2	0.089	2.66
3	0.000	0.00
4	0.089	2.66
5	0.000	0.00
6	0.000	0.00
7	0.089	2.66
8	0.089	2.66
9	0.000	0.00

Table of values		
Vn,x	10.65	kip
Resistance factor	0.95	
Unity check	0.01	-

Shear force Vy

Element ID	Aw [$inch^2$]	Vn [kip]
1	0.027	0.80
2	0.000	0.00
3	0.673	20.18
4	0.000	0.00
5	0.027	0.80
6	0.027	0.80
7	0.000	0.00
8	0.000	0.00
9	0.027	0.80

Table of values		
Vn,y	23.39	kip
Resistance factor	0.95	
Unity check	0.01	-

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

Table of values		
Mnxo	7.94	kipft
Vny	23.39	kip
Mnyo	1.41	kipft
Vnx	10.65	kip
Resistance factor shear	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check (Mx, Vy) = sqrt(0.01+0.00) = 0.07
Unity check (My, Vx) = sqrt(0.05+0.00) = 0.22

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

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

Table of values		
Sftx	1.906	$inch^3$
Sfty	0.339	$inch^3$
Mnxt	7.94	kipft
Mnyt	1.41	kipft
Mnx	7.94	kipft
Mny	1.41	kipft
Tn	55.61	kip
Resistance factor tension	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.07+0.22+0.01 = 0.30 - (H1.1-1)
Unity check = 0.07+0.22-0.01 = 0.28 - (H1.1-2)

The member satisfies the check !

STEEL MEMBER B981 CHECK (HEADER)

AISI S100-16 LRFD Check

Member B982	S600S162-54	A1008 grade 54	LRFD-Ult (auto)	0.96

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

The critical check is on position 0.62 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	-0.75	kip
Vux	-0.92	kip
Vuy	0.00	kip
Mut	0.00	kipft
Mux	-0.00	kipft
Muy	0.36	kipft

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

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

Table of values		
Sye	0.149	inch^3
Mnyo	0.62	kipft
Resistance factor	0.90	
Unity check	0.64	-

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

Table of values		
Sigma,ex	713.829	ksi
Kt	1.00	
Lt	1.866	ft
Sigma,t	208.767	ksi
Cs	-1.00	
CTF	0.61	
Sfy	0.149	$inch^3$
j	3.407	inch
Fcre	1197.248	ksi

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

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

Table of values		
Sfy	0.149	$inch^3$
My	0.62	kipft
L	1.109	ft
Beta	1.25	
k,phi,fe	0.31	kip
k,phi,we	0.29	kip
k,phi	0.00	kip
k,phi,fg	0.008	$inch^2$

Table of values		
k,phi,wg	0.011	$inch^2$
Fd	39.226	ksi
Sf	0.149	$inch^3$
Mcrd	0.49	kipft
Lambda,d	1.13	
Mn	0.44	kipft
Resistance factor	0.90	
Unity check	0.90	-

Data		
Lm	1.866	ft
Lcr	1.109	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch
Ksi,web	0.00	

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

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

Element ID	Aw [inch^2]	Vn [kip]
3	0.076	2.28
5	0.000	0.00
7	0.076	2.28

Table of values		
Vn,x	4.56	kip
Resistance factor	0.95	
Unity check	0.21	-

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

Table of values		
Mnyo	0.62	kipft
Vnx	4.56	kip
Resistance factor shear	0.95	
Resistance factor bending y	0.90	

Unity check (My, Vx) = sqrt(0.41+0.04) = 0.68

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

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

Table of values		
Fn	50.000	ksi
Ae	0.349	inch^2
Pno	17.47	kip
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	3 7/8	1 7/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	3 7/8	1 7/8	ft
Slenderness	20.03	39.40	
Flexural Buckling stress Fcre	713.829	184.416	ksi

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

Table of values		
Sigma,ex	713.829	ksi
Sigma,ey	184.416	ksi
Kt	1.00	
Lt	1 7/8	ft
Sigma,t	208.767	ksi
Sigma,TF	184.416	ksi
Torsional (-Flexural) buckling stress Fcre	184.416	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	44.636 44.636	1.00	0.430	281.003	0.399	1.000	0.300 -	- -	-	- -	-
3	1.342	44.636 44.636	1.00	3.185	148.528	0.548	1.000	1.342 -	0.562 0.780	32.63	0.000 0.000	0.300
5	5.717	44.636 44.636	1.00	4.000	10.279	2.084	0.429	2.454 -	- -	-	- -	-
7	1.342	44.636 44.636	1.00	3.185	148.528	0.548	1.000	1.342 -	0.562 0.780	32.63	0.000 0.000	0.300
9	0.359	44.636 44.636	1.00	0.430	281.003	0.399	1.000	0.300 -	- -	-	- -	-

Table of values		
Fe	184.416	ksi
lambda, c	0.52	
Fn	44.636	ksi
Ae	0.365	inch^2
Pn	16.29	kip
Resistance factor	0.85	
Unity check	0.05	-

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

Table of values		
Py	27.95	kip
L	1.226	ft
k,phi,fe	0.22	kip
k,phi,we	0.16	kip
k,phi	0.00	kip
k,phi,fg	0.006	inch^2
k,phi,wg	0.009	inch^2
Fd	24.162	ksi
Pcrd	13.51	kip
Lambda,d	1.44	
Pn	15.15	kip
Resistance factor	0.85	
Unity check	0.06	-

Data		
Lm	1.866	ft
Lcr	1.226	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch

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

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	1.344 1.344	1.00	0.430	281.003	0.069	1.000	0.359 -	- -	-	- -	-
3	1.342	1.344 1.344	1.00	4.000	186.542	0.085	1.000	1.342 -	0.671 0.671	188.02	- 0.000	0.359
5	5.717	1.344 1.344	1.00	4.000	10.279	0.362	1.000	5.717 -	- -	-	- -	-
7	1.342	1.344 1.344	1.00	4.000	186.542	0.085	1.000	1.342 -	0.671 0.671	188.02	- 0.000	0.359
9	0.359	1.344 1.344	1.00	0.430	281.003	0.069	1.000	0.359 -	- -	-	- -	-

Table of values		
Mny	0.44	kipft
Pn	15.15	kip
Resistance factor compression	0.85	
Resistance factor bending y	0.90	

Unity check = 0.06+0.00+0.90 = 0.96 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B996 CHECK (X-BRACING)

AISI S100-16 LRFD Check

Member B996	3"x0,054	A1008 grade 54	NC_LRFD-Ult (au	0.23

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

The critical check is on position 13.27 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

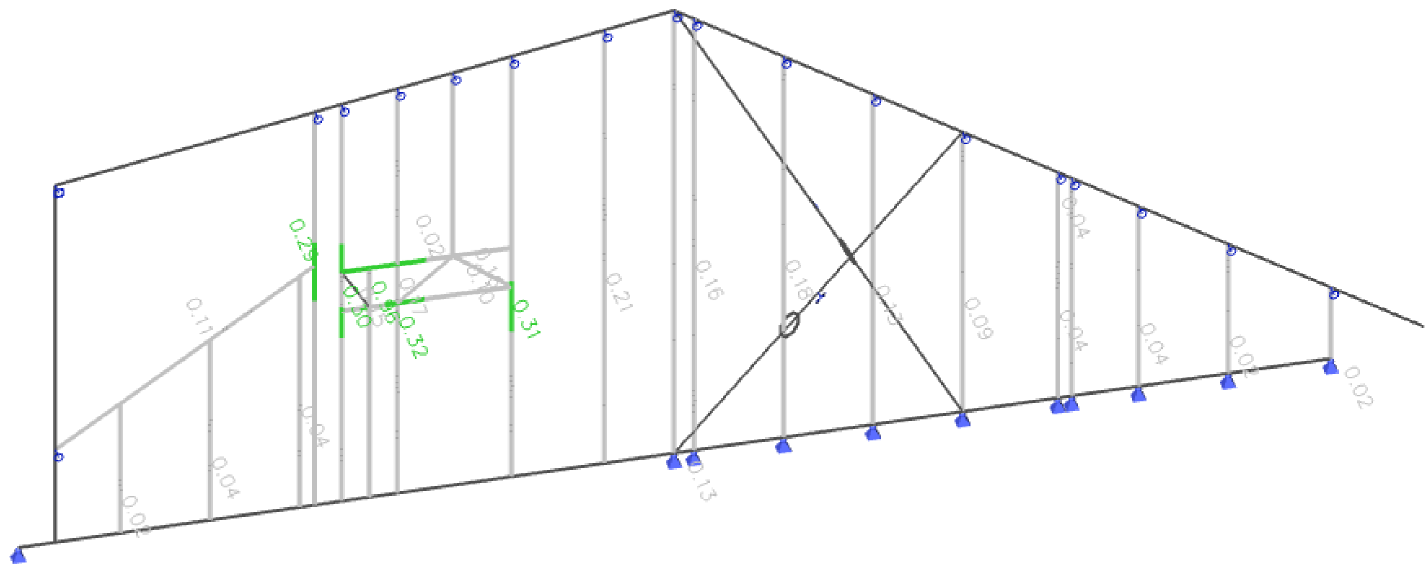
Internal forces		
Pu	1.65	kip
Vux	-0.00	kip
Vuy	-0.00	kip
Mut	-0.00	kipft
Mux	0.00	kipft
Muy	0.00	kipft

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

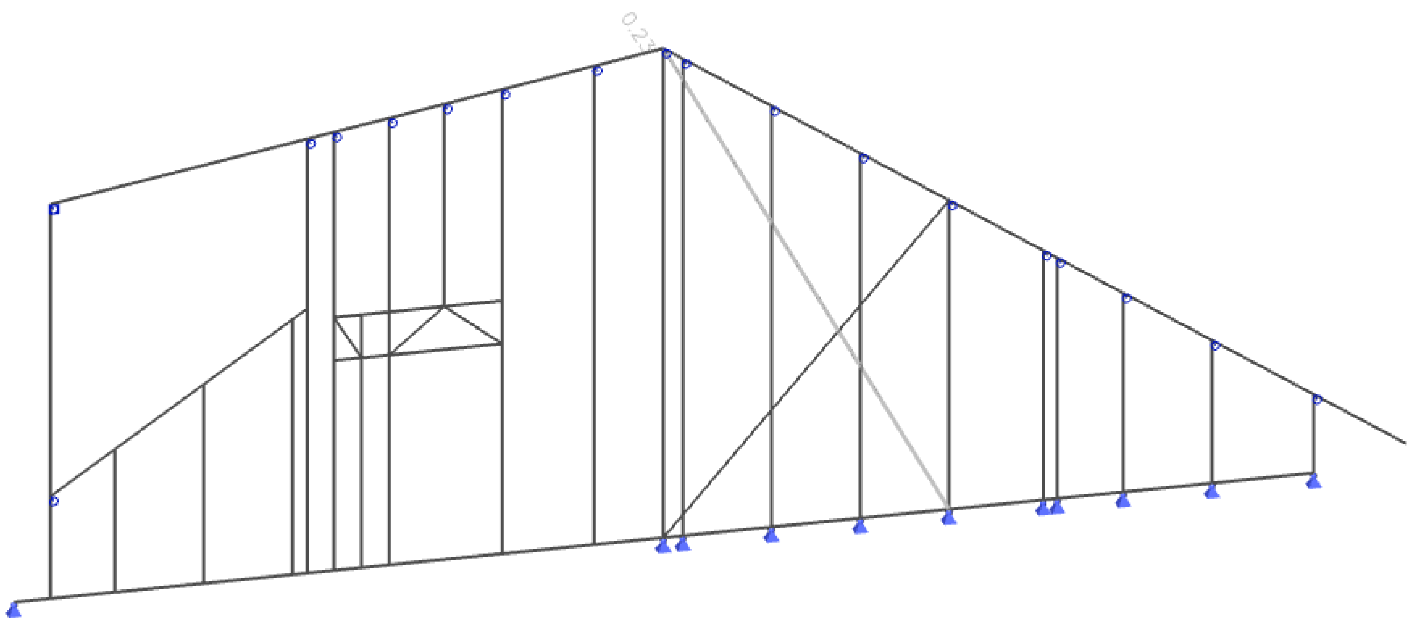
Table of values		
Tn	8.10	kip
Resistance factor	0.90	
Unity check	0.23	-

The member satisfies the check !

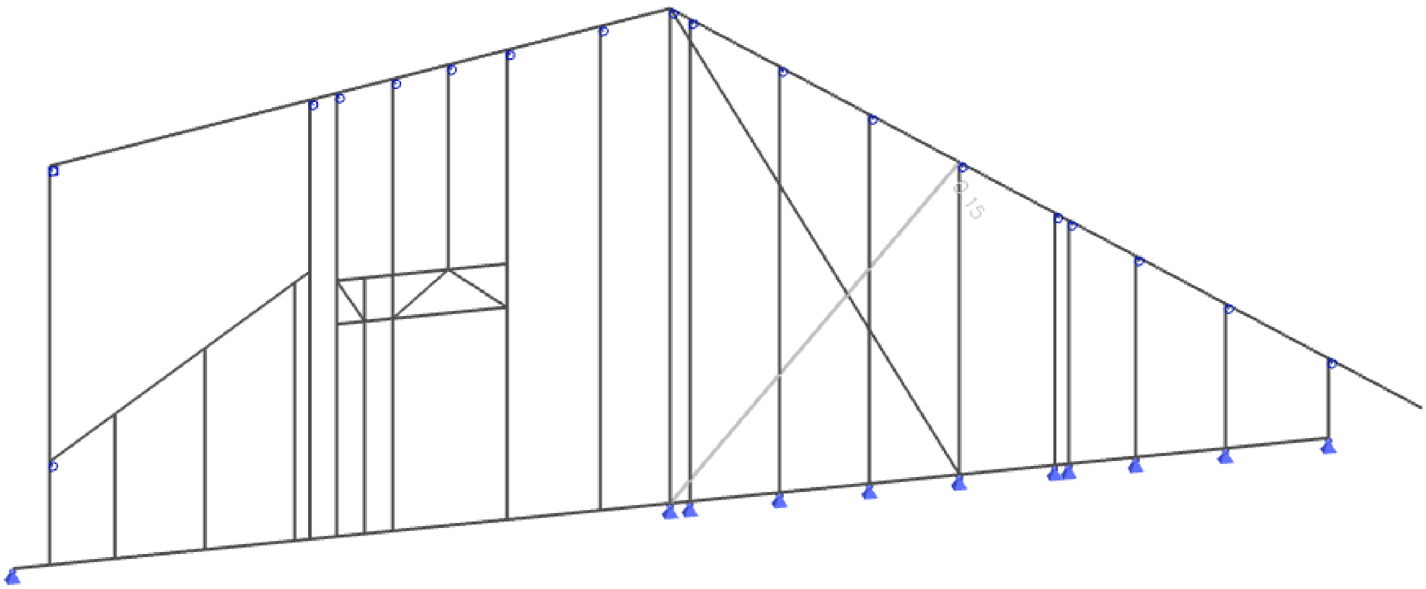
Unity check



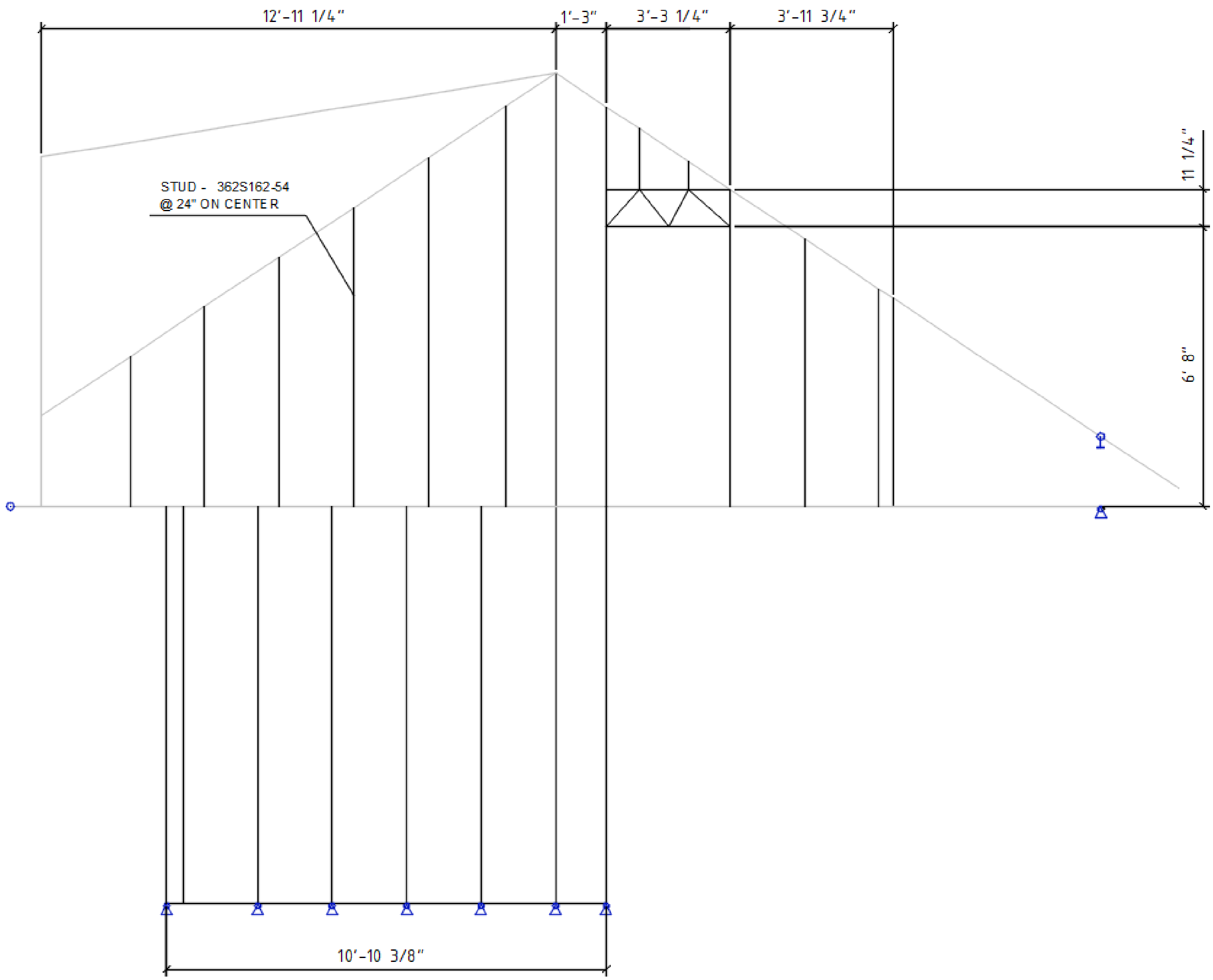
X-Bracing Unity check



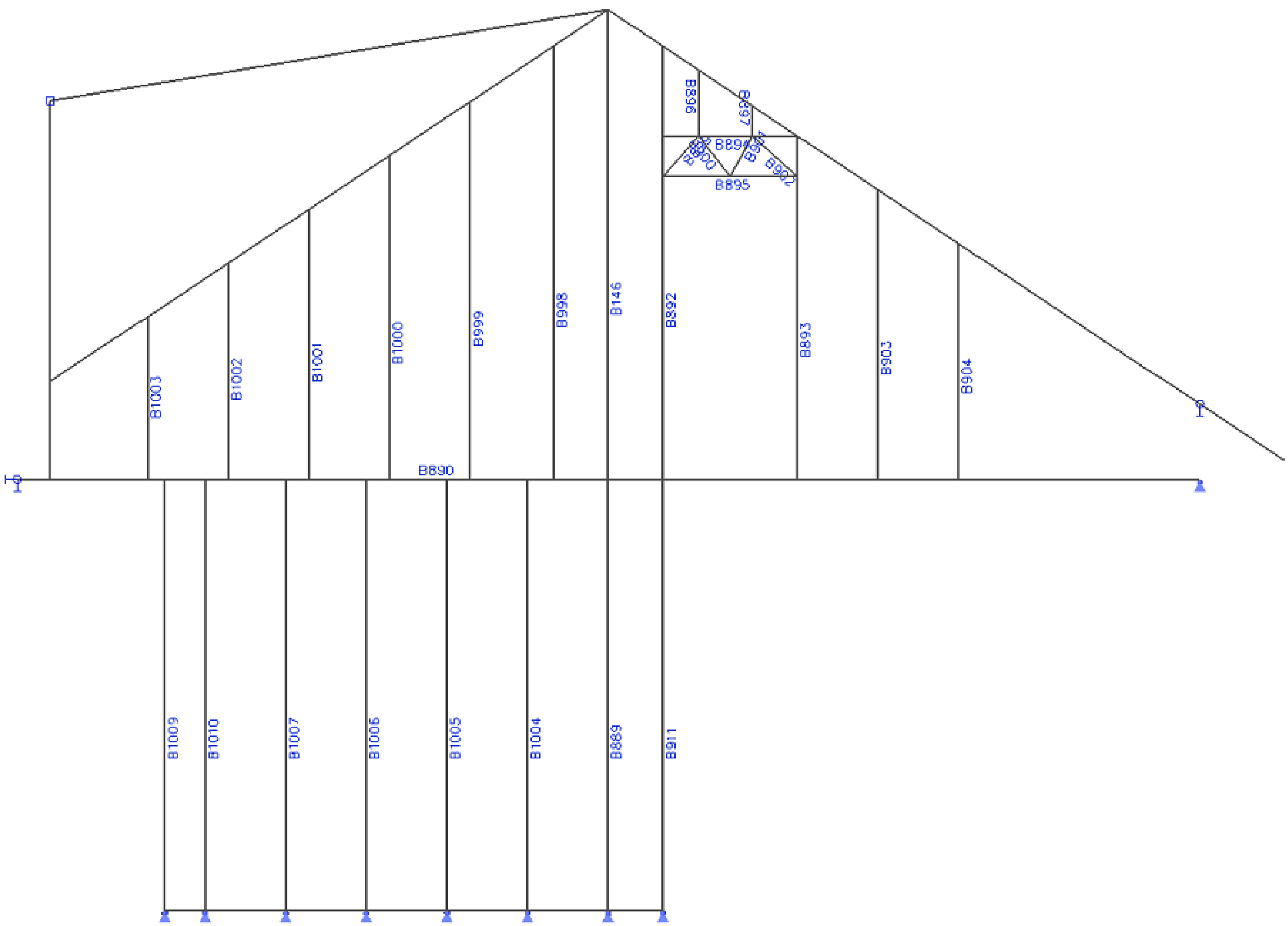
X-Bracing Unity check



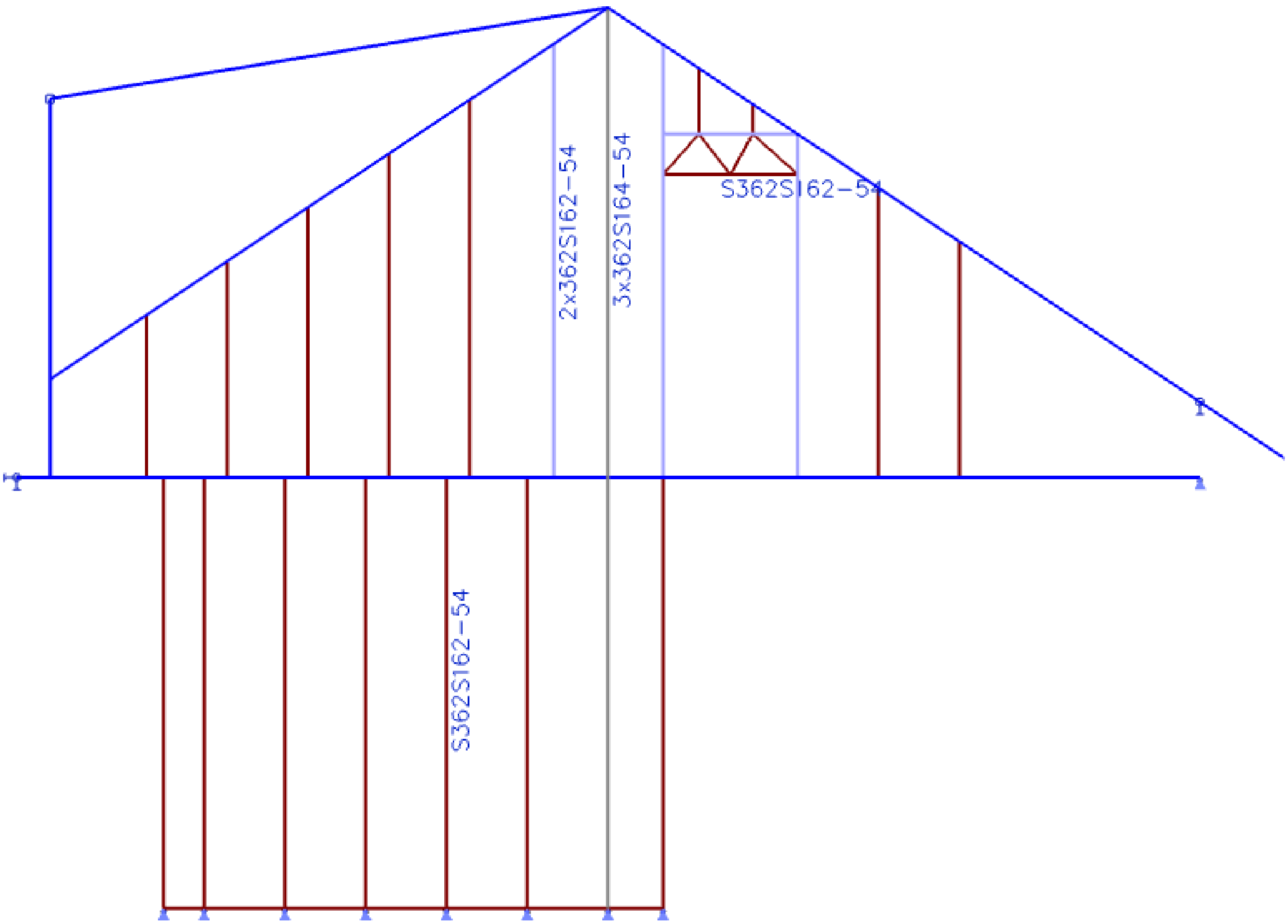
INTERIOR WALL B



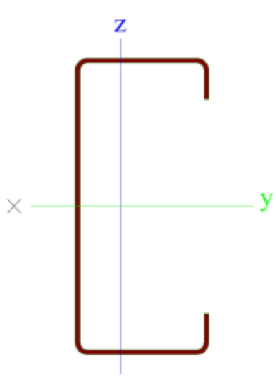
Member numbers

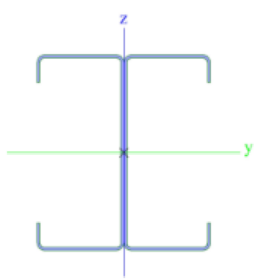


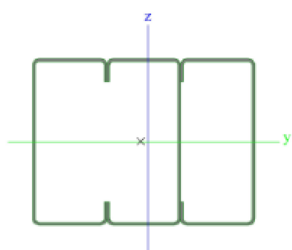
Cross-section walls element.



Section Properties:

CS11		
Type	S362S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.425	
A_y [inch^2], A_z [inch^2]	0.184	0.220
A_L [inch^2/inch], A_D [inch^2/inch]	1.50e+01	1.50e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.536	1.812
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.882	0.154
i_y [inch], i_z [inch]	1.441	0.603
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.481	0.142
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.559	0.212
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.80e+01	2.80e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.06e+01	1.06e+01
d_y [inch], d_z [inch]	-1.290	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.457
β_y [inch], β_z [inch]	0.000	4.130
Picture		

CS25		
Type	2x362S162-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.843	
A_y [inch^2], A_z [inch^2]	0.374	0.439
A_L [inch^2/inch], A_D [inch^2/inch]	2.34e+01	2.34e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-0.313	0.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	1.745	0.551
i_y [inch], i_z [inch]	1.438	0.808
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.963	0.339
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.119	0.452
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.59e+01	5.59e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.26e+01	2.26e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.002	2.094
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

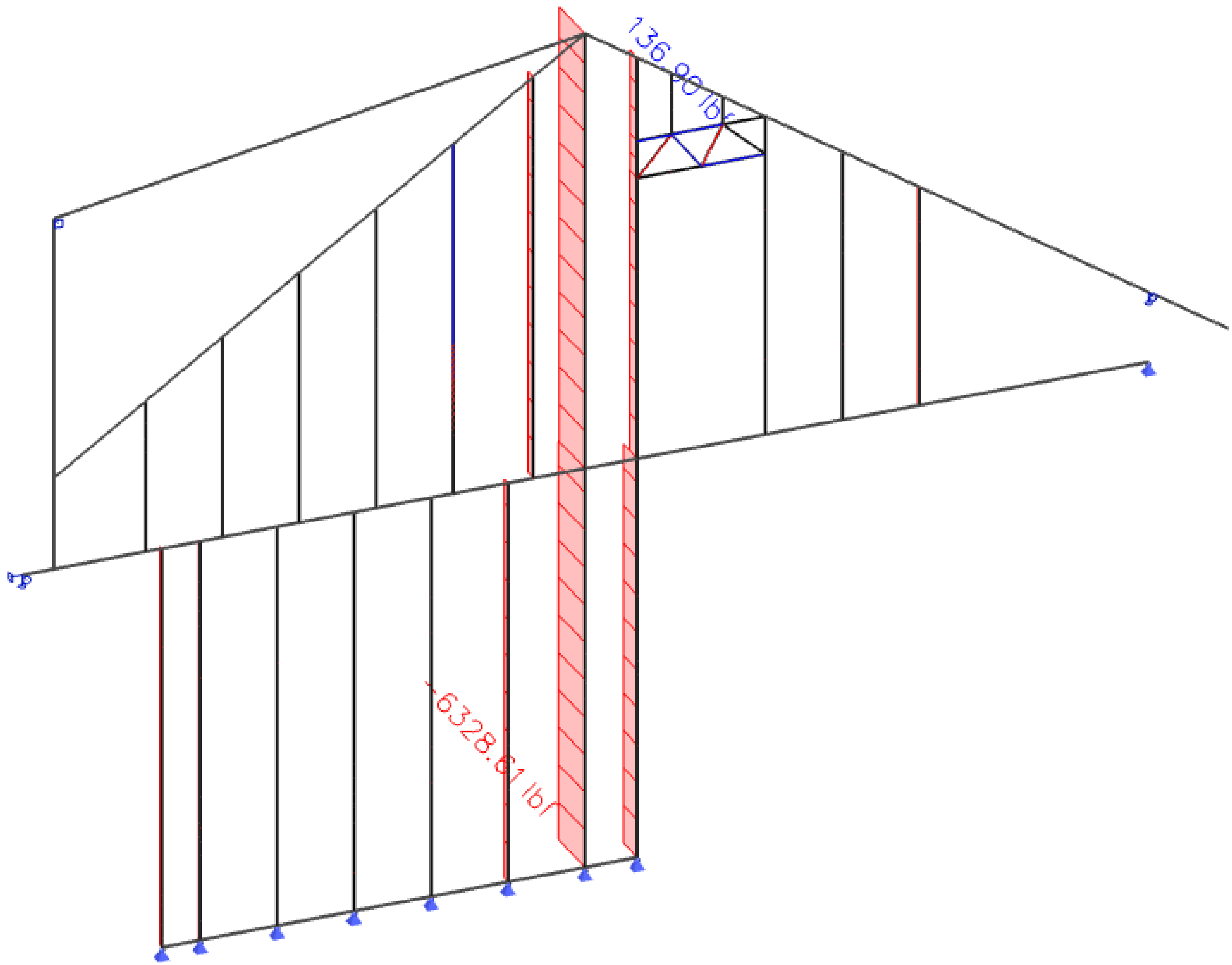
CS32		
Type	3x362S164-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	1.265	
A_y [inch^2], A_z [inch^2]	0.592	0.659
A_L [inch^2/inch], A_D [inch^2/inch]	1.74e+01	4.22e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-1.279	0.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.617	3.533
i_y [inch], i_z [inch]	1.438	1.671
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.444	1.397
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.678	1.816
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	8.39e+01	8.39e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.08e+01	9.08e+01
d_y [inch], d_z [inch]	-0.165	0.000
I_t [inch^4], I_w [inch^6]	0.200	14.316
$β_y$ [inch], $β_z$ [inch]	0.000	0.025
Picture		

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

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

Maximum force diagram

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



STEEL MEMBER B911 CHECK (STUD)

AISI S100-16 LRFD Check

Member B911	S362S162-54	A1008 grade 54	LRFD-Ult (auto)	0.39

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

The critical check is on position 10.00 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	-3320.39	lbf
Vux	-0.00	lbf
Vuy	0.43	lbf
Mut	-0.00	lbfft
Mux	4.34	lbfft
Muy	-0.00	lbfft

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

Table of values		
Lltb	2.000	ft
Sigma,ey	180.489	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	133.313	ksi
Cb	1.09	
Sfx	0.487	$inch^3$
Fcre	298.068	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	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	3.342	50.000 50.000	1.00	4.000	30.079	1.289	0.643	2.150 -	- -	-	- -	-
7	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
9	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.339	$inch^2$
Pno	16945.16	lbf
Resistance factor	0.85	
Unity check	0.23	-

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	10 3/4	2	ft
Effective Length factor K	1.00	1.00	
Effective Length	10 3/4	2	ft
Slenderness	88.63	39.83	
Flexural Buckling stress Fcre	36.447	180.489	ksi

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

Table of values		
Sigma,ex	36.447	ksi
Sigma,ey	180.489	ksi
Kt	1.00	
Lt	2	ft
Sigma,t	133.313	ksi
Sigma,TF	32.270	ksi
Torsional (-Flexural) buckling stress Fcre	32.270	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	26.141 26.141	1.00	0.430	281.003	0.305	1.000	0.359 -	- -	-	- -	-
3	1.342	26.141 26.141	1.00	3.387	157.959	0.407	1.000	1.342 -	0.671 0.671	42.64	0.000 0.000	0.359
5	3.342	26.141 26.141	1.00	4.000	30.079	0.932	0.820	2.739 -	- -	-	- -	-
7	1.342	26.141 26.141	1.00	3.387	157.959	0.407	1.000	1.342 -	0.671 0.671	42.64	0.000 0.000	0.359
9	0.359	26.141 26.141	1.00	0.430	281.003	0.305	1.000	0.359 -	- -	-	- -	-

Table of values		
Fe	32.270	ksi
lambda, c	1.24	
Fn	26.141	ksi
Ae	0.388	$inch^2$
Pn	10135.63	lbf
Resistance factor	0.85	
Unity check	0.39	-

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

Table of values		
Py	21232.26	lbf
L	1.081	ft
k,phi,fe	341.95	lbf
k,phi,we	265.74	lbf
k,phi	0.00	lbf
k,phi,fg	0.008	$inch^2$
k,phi,wg	0.003	$inch^2$
Fd	55.567	ksi
Pcrd	23596.26	lbf
Lambda,d	0.95	
Pn	16595.67	lbf
Resistance factor	0.85	
Unity check	0.24	-

Data		
Lm	2.000	ft
Lcr	1.081	ft
h0	3.625	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch

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

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

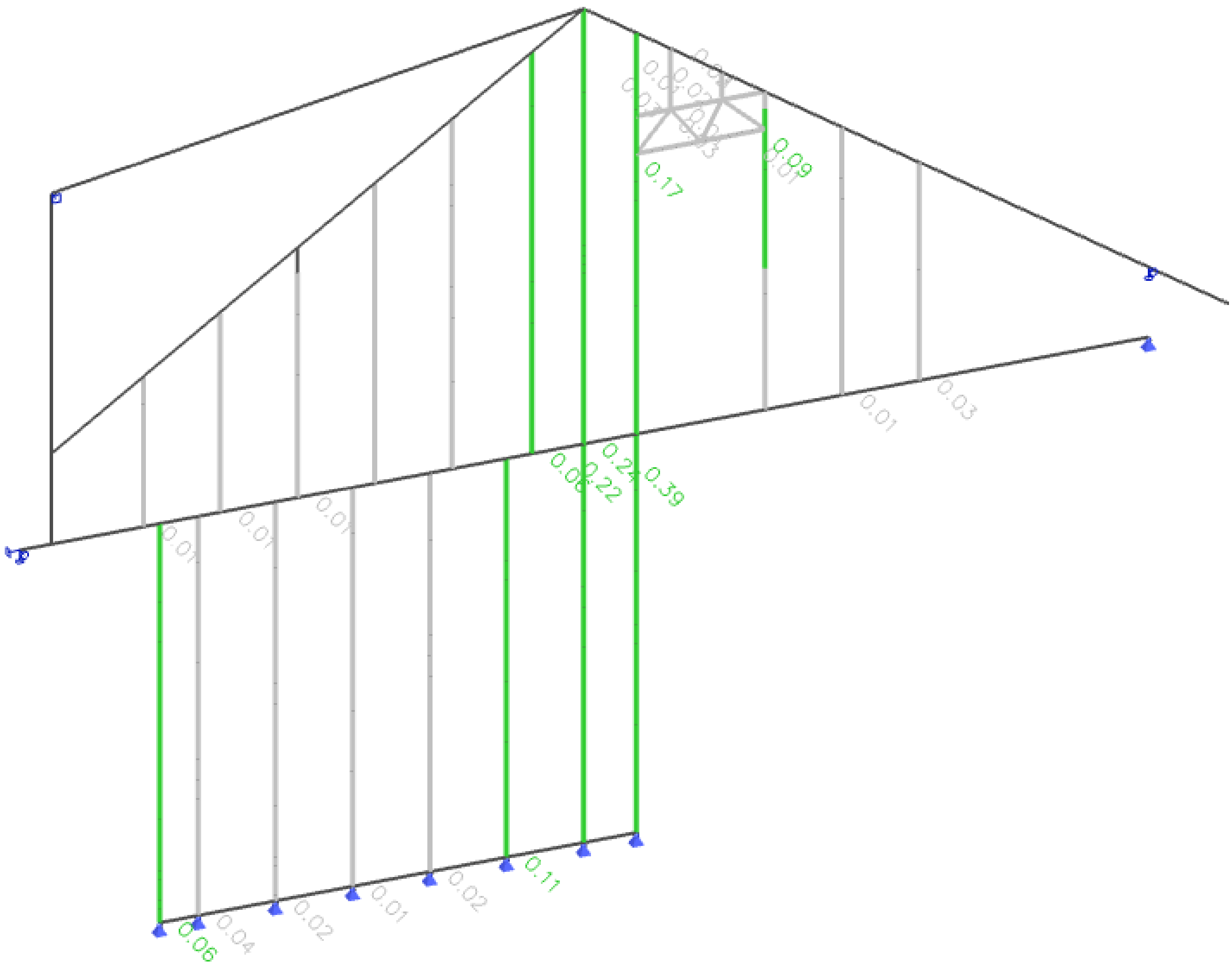
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	7.819 7.819	1.00	0.430	281.003	0.167	1.000	0.359 -	- -	-	- -	-
3	1.342	7.819 7.819	1.00	4.000	186.542	0.205	1.000	1.342 -	0.671 0.671	77.96	- 0.000	0.359
5	3.342	7.819 7.819	1.00	4.000	30.079	0.510	1.000	3.342 -	- -	-	- -	-
7	1.342	7.819 7.819	1.00	4.000	186.542	0.205	1.000	1.342 -	0.671 0.671	77.96	- 0.000	0.359
9	0.359	7.819 7.819	1.00	0.430	281.003	0.167	1.000	0.359 -	- -	-	- -	-

Table of values		
Mnx	1939.13	lbfft
PEx	15477.23	lbf
Alfa x	0.79	
Cmx	0.85	
Pn	10135.63	lbf
Pno	16945.16	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.39+0.00+0.00 = 0.39 - (H1.2-1)
Unity check = 0.23+0.00+0.00 = 0.23 -

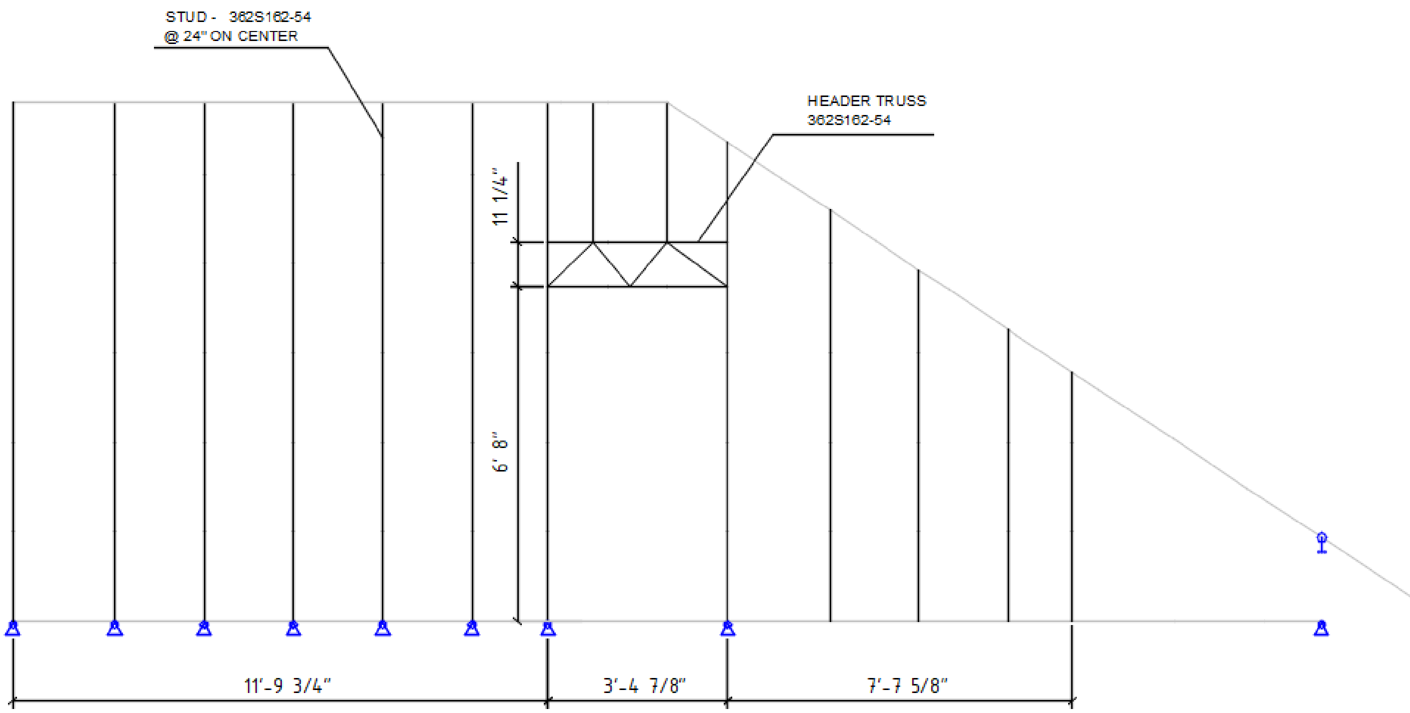
The member satisfies the check !

Unity check

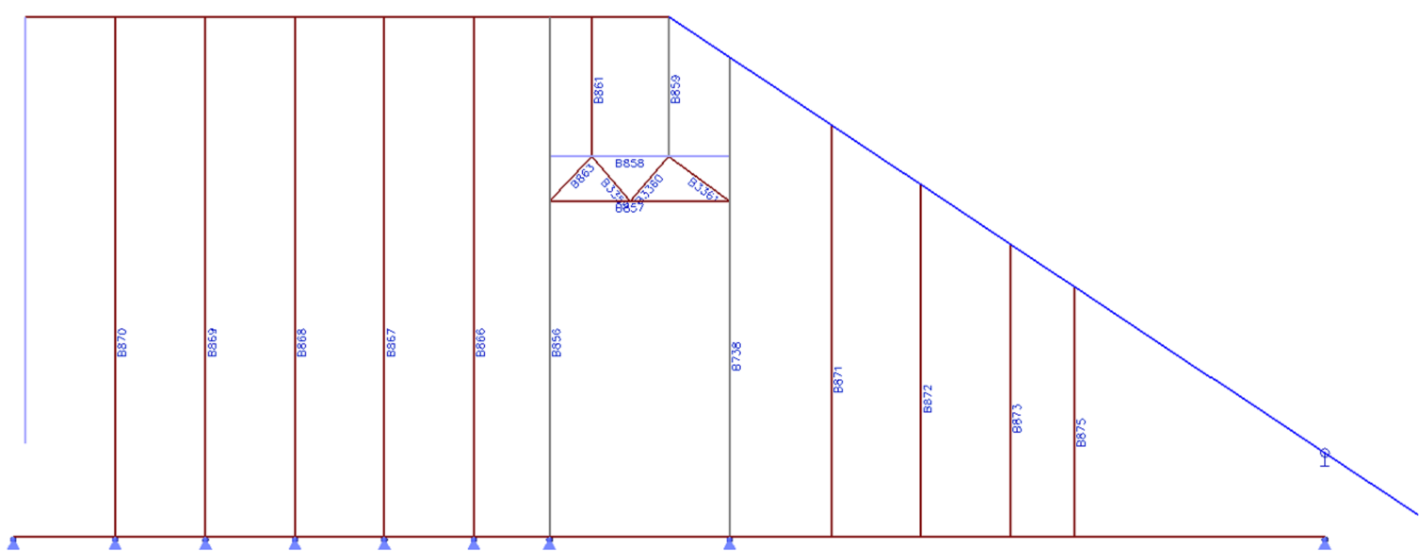


0.17
0.09
0.01
0.03
0.06
0.24
0.22
0.39
0.01
0.01
0.01
0.11
0.06
0.04
0.02
0.01
0.02

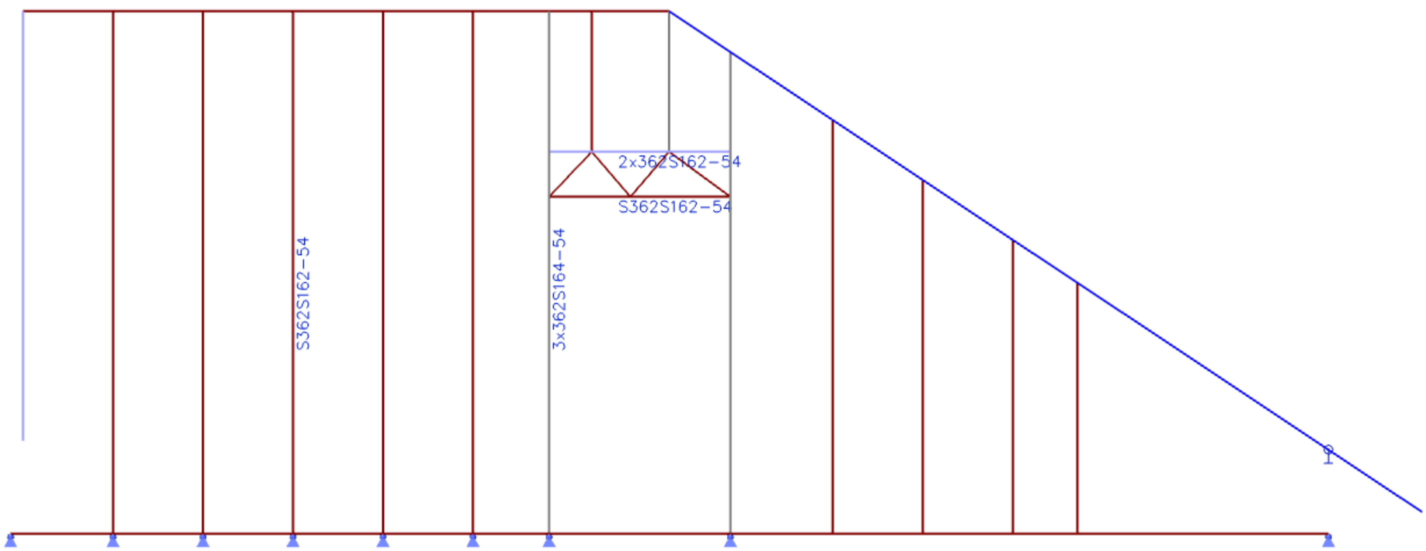
INTERIOR WALL C



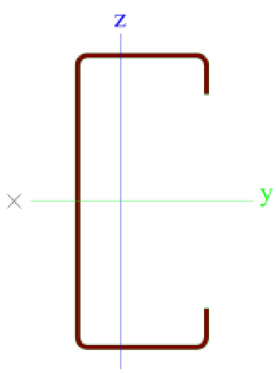
Member numbers

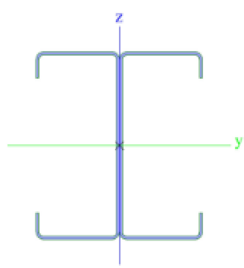


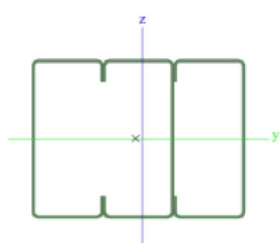
Cross-section walls element.



Section Properties:

CS11		
Type	S362S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [$inch^2$]	0.425	
A_y [$inch^2$], A_z [$inch^2$]	0.184	0.220
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	1.50e+01	1.50e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.536	1.812
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	0.882	0.154
i_y [inch], i_z [inch]	1.441	0.603
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.481	0.142
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	0.559	0.212
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.80e+01	2.80e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.06e+01	1.06e+01
d_y [inch], d_z [inch]	-1.290	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.000	0.457
$β_y$ [inch], $β_z$ [inch]	0.000	4.130
Picture		

CS25		
Type	2x362S162-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.843	
A_y [inch^2], A_z [inch^2]	0.374	0.439
A_L [inch^2/inch], A_D [inch^2/inch]	2.34e+01	2.34e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-0.313	0.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	1.745	0.551
i_y [inch], i_z [inch]	1.438	0.808
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.963	0.339
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1.119	0.452
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.59e+01	5.59e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.26e+01	2.26e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.002	2.094
β_y [inch], β_z [inch]	0.000	0.000
Picture		

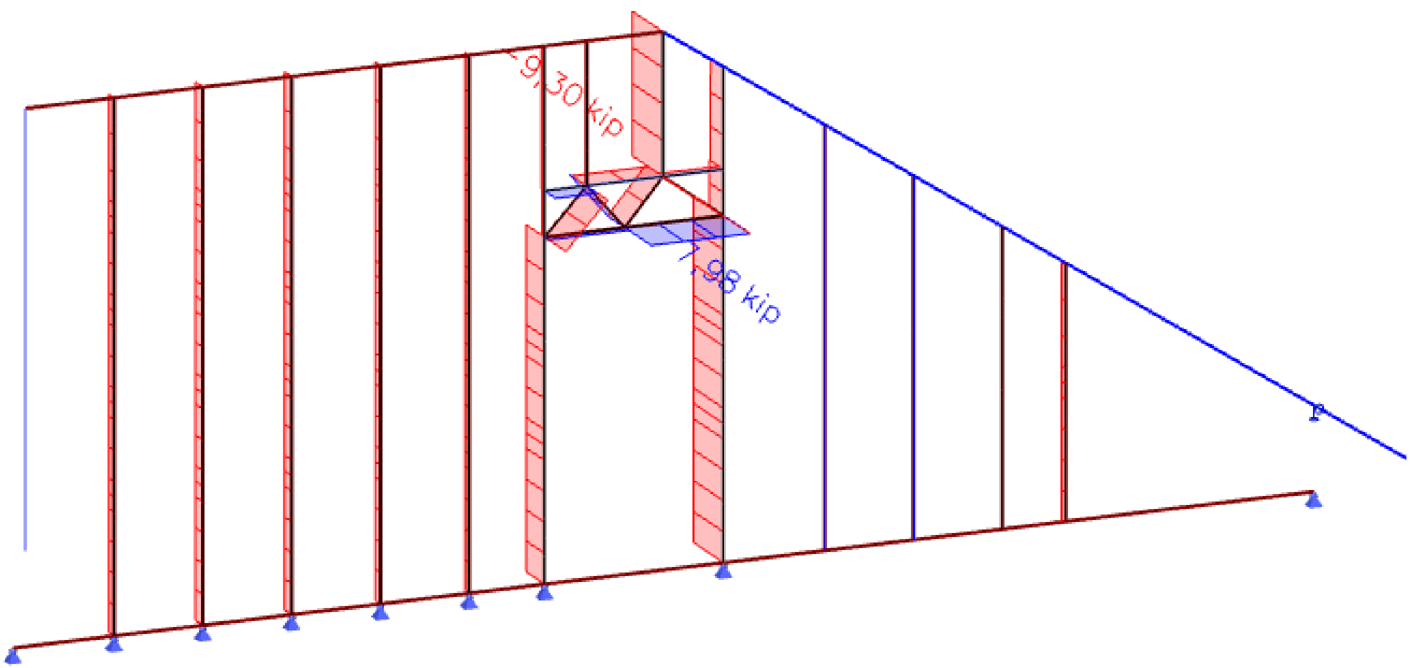
CS32		
Type	3x362S164-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch ^{2}]	1.265	
A_y [inch ^{2}], A_z [inch ^{2}]	0.592	0.659
A_L [inch ^{2}/inch], A_D [inch ^{2}/inch]	1.74e+01	4.22e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-1.279	0.000
α [deg]	0.00	
I_y [inch ^{4}], I_z [inch ^{4}]	2.617	3.533
i_y [inch], i_z [inch]	1.438	1.671
$W_{el.y}$ [inch ^{3}], $W_{el.z}$ [inch ^{3}]	1.444	1.397
$W_{pl.y}$ [inch ^{3}], $W_{pl.z}$ [inch ^{3}]	1.678	1.816
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	8.39e+01	8.39e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	9.08e+01	9.08e+01
d_y [inch], d_z [inch]	-0.165	0.000
I_t [inch ^{4}], I_w [inch ^{6}]	0.200	14.316
β_y [inch], β_z [inch]	0.000	0.025
Picture		

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

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

Maximum force diagram

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



STEEL MEMBER B738 CHECK (STUD)

AISI S100-16 LRFD Check

Member B738	3x362S164-54	A1008 grade 54	LRFD-Ult (auto)	0.49

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

The critical check is on position 7.48 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	-9.10	kip
Vux	-0.00	kip
Vuy	-0.16	kip
Mut	0.00	kipft
Mux	-1.17	kipft
Muy	-0.01	kipft

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

Table of values		
Critical fibre	11	
Sigma Mx	-8.026	ksi
Sigma My	-0.089	ksi
f bending	-8.116	ksi
Tau t	0.033	ksi
f torsion	0.033	ksi
Composed Stress	8.116	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 [inch⁴]	ds [inch]
1	0.472	-15.245 -15.245	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	-15.818 -47.581	-	-	-	-	-	- -	- -	-	- -	-
3	3.568	-47.581 -47.581	-	-	-	-	-	- -	- -	-	- -	-
4	1.568	-15.818 -47.581	-	-	-	-	-	- -	- -	-	- -	-

5	0.472	-15.245 -15.245	-	-	-	-	-	- -	- -	-	- -	-
6	1.568	17.091 -14.672	0.86	20.555	701.821	0.156	1.000	- 1.568	0.406 0.784	-	- -	-
7	2.625	17.091 17.091	1.00	4.000	48.755	0.592	1.000	2.625 -	- -	-	- -	-
8	1.568	17.091 -14.672	0.86	20.555	701.821	0.156	1.000	- 1.568	0.406 0.784	-	- -	-
9	0.472	17.664 17.664	1.00	4.000	6039.613	0.054	1.000	- 0.472	0.236 0.236	-	- -	-
10	1.568	50.000 18.237	0.36	5.783	197.460	0.503	1.000	- 1.568	0.595 0.973	-	- -	-
11	3.568	50.000 50.000	1.00	4.000	26.384	1.377	0.610	2.178 -	- -	-	- -	-
12	1.568	50.000 18.237	0.36	5.783	197.460	0.503	1.000	- 1.568	0.595 0.973	-	- -	-
13	0.472	17.664 17.664	1.00	4.000	6039.613	0.054	1.000	0.472 -	- -	-	- -	-

Table of values		
Sxe	1.611	$inch^3$
Mnxo	6.71	kipft
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	1.484	ft
Sigma,ey	1438.712	ksi
Kt	1.00	
Lt	1.484	ft
Sigma,t	2450.875	ksi
Cb	1.09	
Sfx	1.444	$inch^3$
Fcre	3951.154	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	2520.348	ksi
Kt	1.00	
Lt	1.484	ft
Sigma,t	2450.875	ksi
Cb	1.00	
Sfy	1.397	$inch^3$
Fcre	4977.725	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 [inch²]	Vn [kip]
1	0.053	1.60
2	0.000	0.00
3	0.202	5.35
4	0.000	0.00
5	0.053	1.60
6	0.000	0.00
7	0.149	4.46
8	0.000	0.00
9	0.053	1.60
10	0.000	0.00
11	0.202	5.35
12	0.000	0.00
13	0.053	1.60

Table of values		
Vn,y	21.57	kip
Resistance factor	0.95	
Unity check	0.01	-

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

Table of values		
Mnxo	6.71	kipft
Vny	21.57	kip
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

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

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

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	50.000 50.000	1.00	0.430	649.258	0.278	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	3.568	50.000 50.000	1.00	4.000	26.384	1.377	0.610	2.178 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 50.000	1.00	0.430	649.258	0.278	1.000	0.472 -	- -	-	- -	-
6	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
7	2.625	50.000 50.000	1.00	4.000	48.755	1.013	0.773	2.029 -	- -	-	- -	-
8	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
9	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-

10	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
11	3.568	50.000 50.000	1.00	4.000	26.384	1.377	0.610	2.178 -	- -	-	- -	-
12	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
13	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	1.109	$inch^2$
Pno	55.44	kip
Resistance factor	0.85	
Unity check	0.19	-

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	1 1/2	10 3/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	1 1/2	10 3/4	ft
Slenderness	10.66	89.27	
Flexural Buckling stress Fcre	2520.348	35.928	ksi

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

Table of values		
Sigma,ex	2520.348	ksi
Sigma,ey	35.928	ksi
Kt	1.00	
Lt	1 1/2	ft
Sigma,t	2450.875	ksi
Sigma,TF	35.925	ksi
Torsional (-Flexural) buckling stress Fcre	35.925	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.472	27.924 27.924	1.00	0.430	649.258	0.207	1.000	0.472 -	- -	-	- -	-
2	1.568	27.924 27.924	1.00	4.000	136.574	0.452	1.000	1.568 -	- -	-	- -	-
3	3.568	27.924 27.924	1.00	4.000	26.384	1.029	0.764	2.727 -	- -	-	- -	-
4	1.568	27.924 27.924	1.00	4.000	136.574	0.452	1.000	1.568 -	- -	-	- -	-
5	0.472	27.924 27.924	1.00	0.430	649.258	0.207	1.000	0.472 -	- -	-	- -	-
6	1.568	27.924 27.924	1.00	4.000	136.574	0.452	1.000	1.568 -	- -	-	- -	-
7	2.625	27.924 27.924	1.00	4.000	48.755	0.757	0.937	2.460 -	- -	-	- -	-
8	1.568	27.924 27.924	1.00	4.000	136.574	0.452	1.000	1.568 -	- -	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
9	0.472	27.924 27.924	1.00	4.000	6039.613	0.068	1.000	0.472 -	- -	-	- -	-
10	1.568	27.924 27.924	1.00	4.000	136.574	0.452	1.000	1.568 -	- -	-	- -	-
11	3.568	27.924 27.924	1.00	4.000	26.384	1.029	0.764	2.727 -	- -	-	- -	-
12	1.568	27.924 27.924	1.00	4.000	136.574	0.452	1.000	1.568 -	- -	-	- -	-
13	0.472	27.924 27.924	1.00	4.000	6039.613	0.068	1.000	0.472 -	- -	-	- -	-

Table of values		
Fe	35.925	ksi
lambda, c	1.18	
Fn	27.924	ksi
Ae	1.195	inch^2
Pn	33.38	kip
Resistance factor	0.85	
Unity check	0.32	-

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	0.472	7.190 7.190	1.00	0.430	649.258	0.105	1.000	0.472 -	- -	-	- -	-
2	1.568	7.190 7.190	1.00	4.000	136.574	0.229	1.000	1.568 -	- -	-	- -	-
3	3.568	7.190 7.190	1.00	4.000	26.384	0.522	1.000	3.568 -	- -	-	- -	-
4	1.568	7.190 7.190	1.00	4.000	136.574	0.229	1.000	1.568 -	- -	-	- -	-
5	0.472	7.190 7.190	1.00	0.430	649.258	0.105	1.000	0.472 -	- -	-	- -	-
6	1.568	7.190 7.190	1.00	4.000	136.574	0.229	1.000	1.568 -	- -	-	- -	-
7	2.625	7.190 7.190	1.00	4.000	48.755	0.384	1.000	2.625 -	- -	-	- -	-
8	1.568	7.190 7.190	1.00	4.000	136.574	0.229	1.000	1.568 -	- -	-	- -	-
9	0.472	7.190 7.190	1.00	4.000	6039.613	0.035	1.000	0.472 -	- -	-	- -	-
10	1.568	7.190 7.190	1.00	4.000	136.574	0.229	1.000	1.568 -	- -	-	- -	-
11	3.568	7.190 7.190	1.00	4.000	26.384	0.522	1.000	3.568 -	- -	-	- -	-
12	1.568	7.190 7.190	1.00	4.000	136.574	0.229	1.000	1.568 -	- -	-	- -	-
13	0.472	7.190 7.190	1.00	4.000	6039.613	0.035	1.000	0.472 -	- -	-	- -	-

Table of values		
Mnx	6.71	kipft
Mny	4.31	kipft
PEx	3188.55	kip
PEy	45.45	kip
Alfa x	1.00	
Alfa y	0.80	
Cmx	0.85	
Cmy	0.85	
Pn	33.38	kip
Pno	55.44	kip
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.32+0.16+0.00 = 0.49 - (H1.2-1)
Unity check = 0.19+0.19+0.00 = 0.39 -

The member satisfies the check !

STEEL MEMBER B858 CHECK (HEADER)

AISI S100-16 LRFD Check

Member B858	2x362S162-54	A1008 grade 54	LRFD-Ult (auto)	0.48

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

The critical check is on position 2.67 ft

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

Internal forces		
Pu	-4.45	kip
Vux	0.27	kip
Vuy	-0.08	kip
Mut	-0.00	kipft
Mux	-0.10	kipft
Muy	0.41	kipft

...::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	0.472	-36.781 -50.000	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	3.568	50.000 -50.000	1.00	24.000	633.206	0.281	1.000	- 3.568	0.892 1.784	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 36.781	0.74	0.537	202.841	0.496	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 36.781	0.74	0.537	202.841	0.496	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
8	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
9	0.472	-36.781 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	0.963	$inch^3$
Mnxo	4.01	kipft
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		
Lltb	1.688	ft
Sigma,ey	456.028	ksi
Kt	1.00	
Lt	1.688	ft
Sigma,t	647.613	ksi
Cb	2.14	
Sfx	0.963	inch³
Fcre	1684.281	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 [inch⁴]	ds [inch]
1	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 0.886	0.02	7.860	268.370	0.432	1.000	- 1.568	0.526 1.042	-	- -	-
3	3.568	- -	-	4.000	105.534	-	1.000	3.568 -	- -	-	- -	-
4	1.568	50.000 0.886	0.02	7.860	268.370	0.432	1.000	- 1.568	0.526 1.042	-	- -	-
5	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
6	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	1.568	-0.886 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	1.568	-0.886 -50.000	-	-	-	-	-	- -	- -	-	- -	-
9	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.339	inch³
Mnyo	1.41	kipft
Resistance factor	0.90	
Unity check	0.32	-

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

Table of values		
Sigma,ex	253.237	ksi
Kt	1.00	
Lt	1.688	ft
Sigma,t	647.613	ksi
Cb	1.00	
Sfy	0.339	inch³
Fcre	1662.156	ksi

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

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

Shear Strength

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

Shear force Vx

Element ID	Aw [inch ^{2}]	Vn [kip]
1	0.000	0.00
2	0.089	2.66
3	0.000	0.00
4	0.089	2.66
5	0.000	0.00
6	0.000	0.00
7	0.089	2.66
8	0.089	2.66
9	0.000	0.00

Table of values		
Vn,x	10.65	kip
Resistance factor	0.95	
Unity check	0.03	-

Shear force Vy

Element ID	Aw [inch ^{2}]	Vn [kip]
1	0.027	0.80
2	0.000	0.00
3	0.404	12.12
4	0.000	0.00
5	0.027	0.80
6	0.027	0.80
7	0.000	0.00
8	0.000	0.00
9	0.027	0.80

Table of values		
Vn,y	15.32	kip
Resistance factor	0.95	
Unity check	0.01	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	4.01	kipft
Vny	15.32	kip
Mnyo	1.41	kipft
Vnx	10.65	kip
Resistance factor shear	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check (Mx, Vy) = sqrt(0.00+0.00) = 0.03

Unity check (My, Vx) = sqrt(0.10+0.00) = 0.32

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

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

Nominal Axial Strength

According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	3.568	50.000 50.000	1.00	4.000	105.534	0.688	0.988	3.527 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
8	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
9	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.843	inch²
Pno	42.17	kip
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	4 1/8	1 3/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	4 1/8	1 3/4	ft
Slenderness	33.62	25.06	
Flexural Buckling stress Fcre	253.237	456.028	ksi

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

Table of values		
Sigma,ex	253.237	ksi
Sigma,ey	456.028	ksi
Kt	1.00	
Lt	1 3/4	ft
Sigma,t	647.613	ksi
Sigma,TF	253.237	ksi
Torsional (-Flexural) buckling stress Fcre	253.237	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	46.034 46.034	1.00	0.430	162.315	0.533	1.000	0.472 -	- -	-	- -	-
2	1.568	46.034 46.034	1.00	4.000	136.574	0.581	1.000	1.568 -	- -	-	- -	-
3	3.568	46.034 46.034	1.00	4.000	105.534	0.660	1.000	3.568 -	- -	-	- -	-
4	1.568	46.034 46.034	1.00	4.000	136.574	0.581	1.000	1.568 -	- -	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
5	0.472	46.034 46.034	1.00	0.430	162.315	0.533	1.000	0.472 -	- -	-	- -	-
6	0.472	46.034 46.034	1.00	0.430	162.315	0.533	1.000	0.472 -	- -	-	- -	-
7	1.568	46.034 46.034	1.00	4.000	136.574	0.581	1.000	1.568 -	- -	-	- -	-
8	1.568	46.034 46.034	1.00	4.000	136.574	0.581	1.000	1.568 -	- -	-	- -	-
9	0.472	46.034 46.034	1.00	0.430	162.315	0.533	1.000	0.472 -	- -	-	- -	-

Table of values		
Fe	253.237	ksi
lambda, c	0.44	
Fn	46.034	ksi
Ae	0.843	inch^2
Pn	38.83	kip
Resistance factor	0.85	
Unity check	0.13	-

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

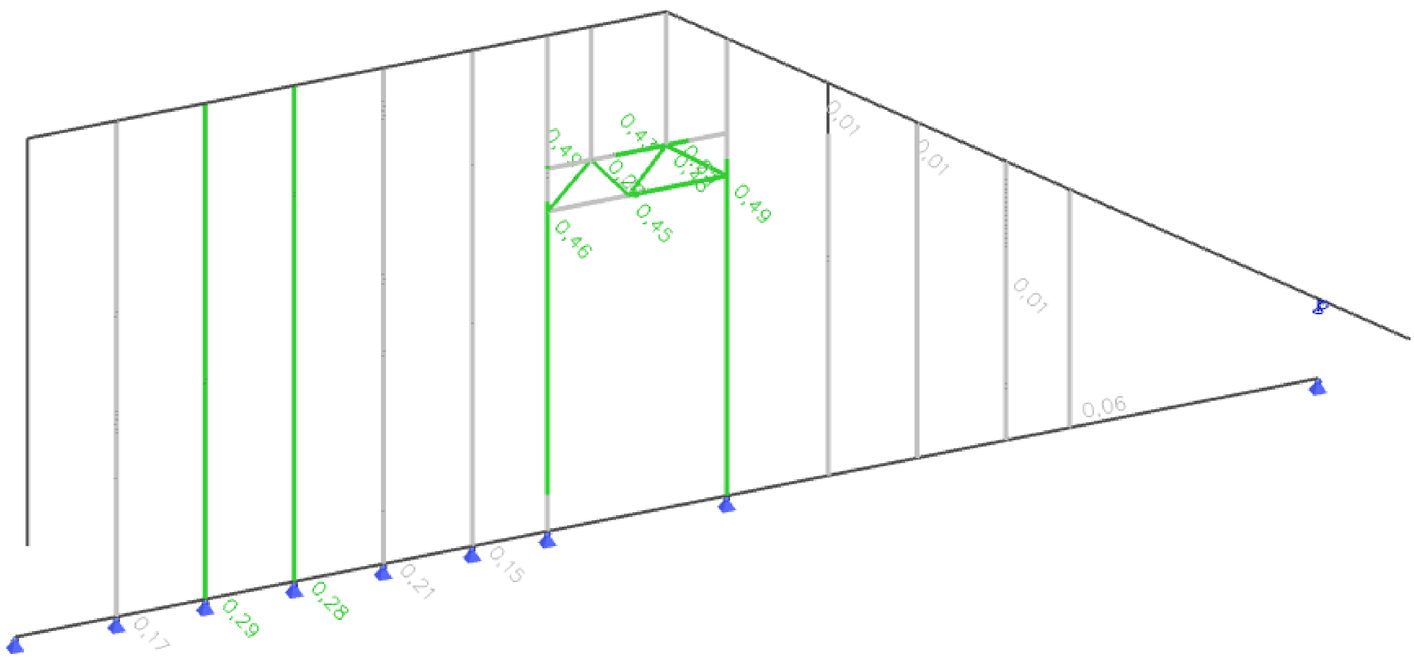
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	5.278 5.278	1.00	0.430	162.315	0.180	1.000	0.472 -	- -	-	- -	-
2	1.568	5.278 5.278	1.00	4.000	136.574	0.197	1.000	1.568 -	- -	-	- -	-
3	3.568	5.278 5.278	1.00	4.000	105.534	0.224	1.000	3.568 -	- -	-	- -	-
4	1.568	5.278 5.278	1.00	4.000	136.574	0.197	1.000	1.568 -	- -	-	- -	-
5	0.472	5.278 5.278	1.00	0.430	162.315	0.180	1.000	0.472 -	- -	-	- -	-
6	0.472	5.278 5.278	1.00	0.430	162.315	0.180	1.000	0.472 -	- -	-	- -	-
7	1.568	5.278 5.278	1.00	4.000	136.574	0.197	1.000	1.568 -	- -	-	- -	-
8	1.568	5.278 5.278	1.00	4.000	136.574	0.197	1.000	1.568 -	- -	-	- -	-
9	0.472	5.278 5.278	1.00	0.430	162.315	0.180	1.000	0.472 -	- -	-	- -	-

Table of values		
Mnx	4.01	kipft
Mny	1.41	kipft
Pn	38.83	kip
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

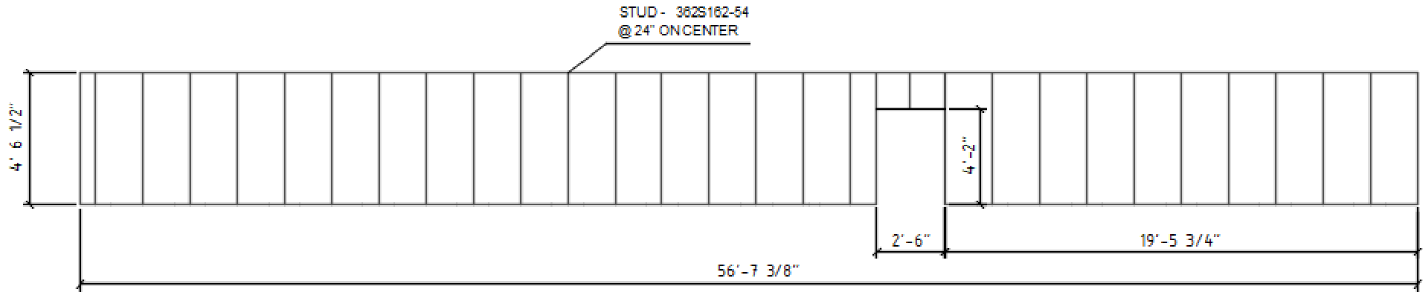
Unity check = 0.13+0.03+0.32 = 0.48 - (C5.2.1-3)

The member satisfies the check !

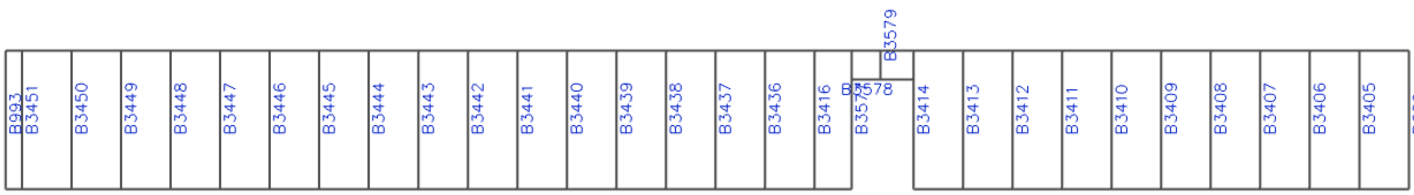
Unity check



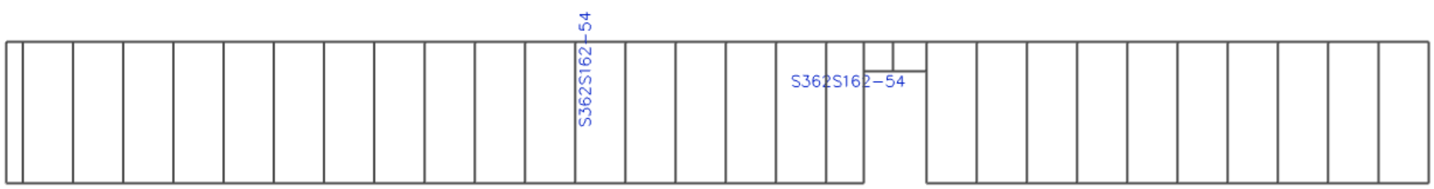
INTERIOR WALL D



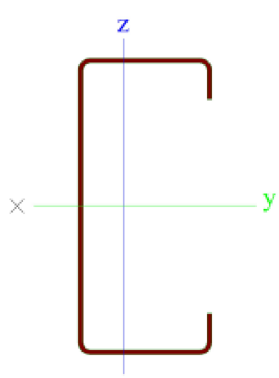
Member numbers



Cross-section walls element.



Section Properties:

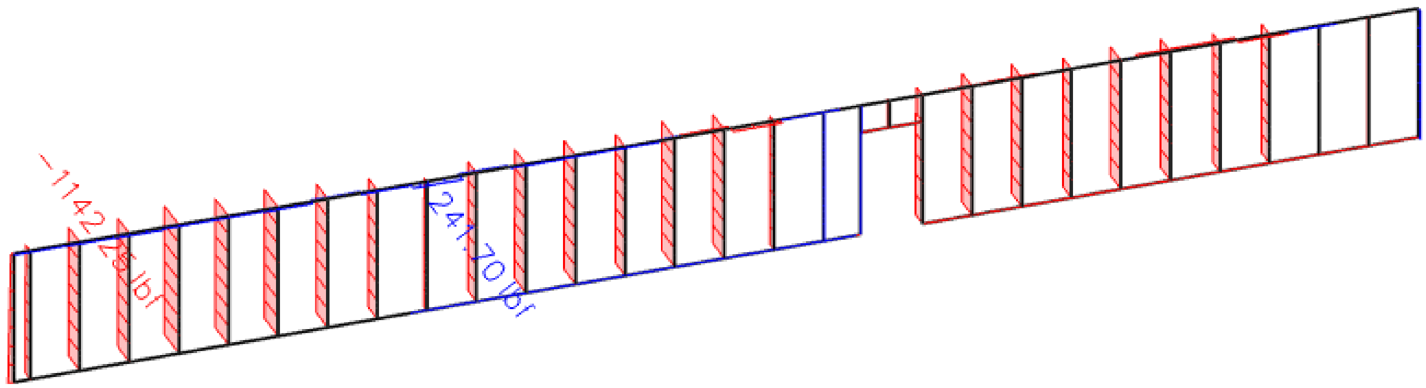
CS11		
Type	S362S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [$inch^2$]	0.425	
A_y [$inch^2$], A_z [$inch^2$]	0.184	0.220
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	1.50e+01	1.50e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.536	1.812
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	0.882	0.154
i_y [inch], i_z [inch]	1.441	0.603
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.481	0.142
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	0.559	0.212
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.80e+01	2.80e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.06e+01	1.06e+01
d_y [inch], d_z [inch]	-1.290	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.000	0.457
$β_y$ [inch], $β_z$ [inch]	0.000	4.130
Picture		

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

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

Maximum force diagram

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



STEEL MEMBER B3448 CHECK (STUD)

AISI S100-16 LRFD Check

Member B3448	S362S162-54	A1008 grade 54	LRFD-Ult (auto)	0.20

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	-1.16	kip
Vux	0.00	kip
Vuy	0.00	kip
Mut	-0.00	kipft
Mux	-0.00	kipft
Muy	0.00	kipft

...::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	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	3.342	50.000 50.000	1.00	4.000	30.079	1.289	0.643	2.150 -	- -	-	- -	-
7	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
9	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.339	$inch^2$
Pno	16.95	kip
Resistance factor	0.85	
Unity check	0.08	-

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	5 5/8	5 5/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	5 5/8	5 5/8	ft
Slenderness	46.42	111.02	
Flexural Buckling stress Fcre	132.844	23.227	ksi

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

Table of values		
Sigma,ex	132.844	ksi
Sigma,ey	23.227	ksi
Kt	1.00	
Lt	5 5/8	ft
Sigma,t	19.735	ksi
Sigma,TF	18.519	ksi
Torsional (-Flexural) buckling stress Fcre	18.519	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.359	16.242 16.242	1.00	0.430	281.003	0.240	1.000	0.359 -	- -	-	- -	-
3	1.342	16.242 16.242	1.00	3.387	157.959	0.321	1.000	1.342 -	0.671 0.671	54.09	0.000 0.000	0.359
5	3.342	16.242 16.242	1.00	4.000	30.079	0.735	0.953	3.186 -	- -	-	- -	-
7	1.342	16.242 16.242	1.00	3.387	157.959	0.321	1.000	1.342 -	0.671 0.671	54.09	0.000 0.000	0.359
9	0.359	16.242 16.242	1.00	0.430	281.003	0.240	1.000	0.359 -	- -	-	- -	-

Table of values		
Fe	18.519	ksi
lambda, c	1.64	
Fn	16.242	ksi
Ae	0.413	inch²
Pn	6.71	kip
Resistance factor	0.85	
Unity check	0.20	-

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

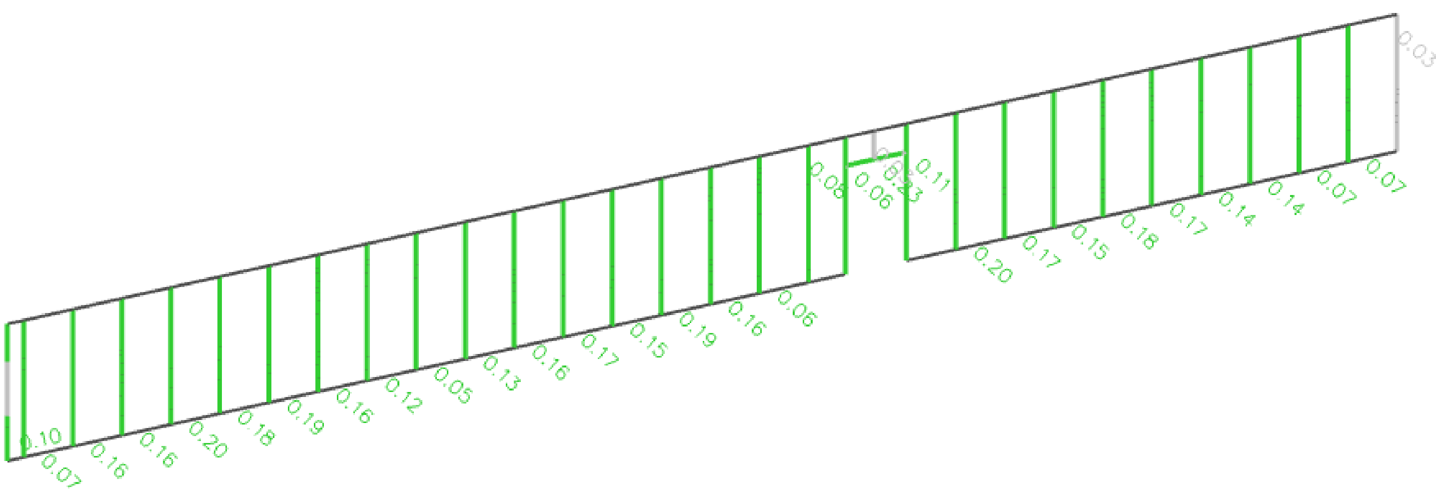
Table of values		
Py	21.23	kip
L	1.081	ft
k,phi,fe	0.34	kip
k,phi,we	0.27	kip
k,phi	0.00	kip
k,phi,fg	0.008	inch²
k,phi,wg	0.003	inch²
Fd	55.567	ksi
Pcrd	23.60	kip
Lambda,d	0.95	
Pn	16.60	kip
Resistance factor	0.85	
Unity check	0.08	-

Data		
Lm	5.575	ft
Lcr	1.081	ft
h0	3.625	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch

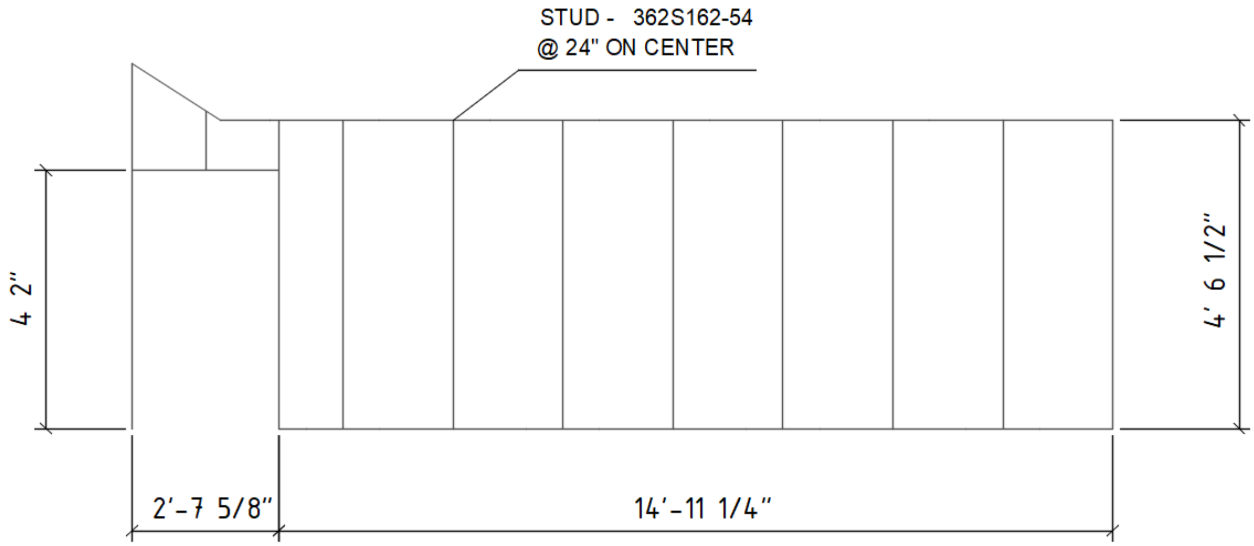
Number of compressed flanges: 2
Critical flange contains Initial shape parts: 8, 7, 9

The member satisfies the check !

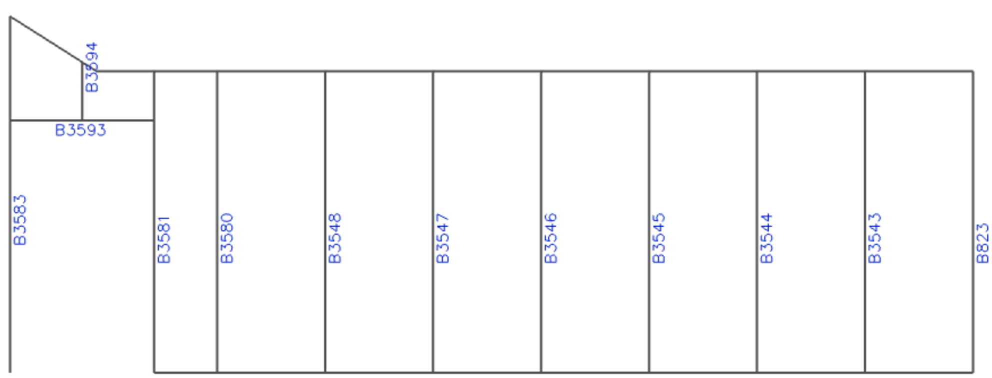
Unity check



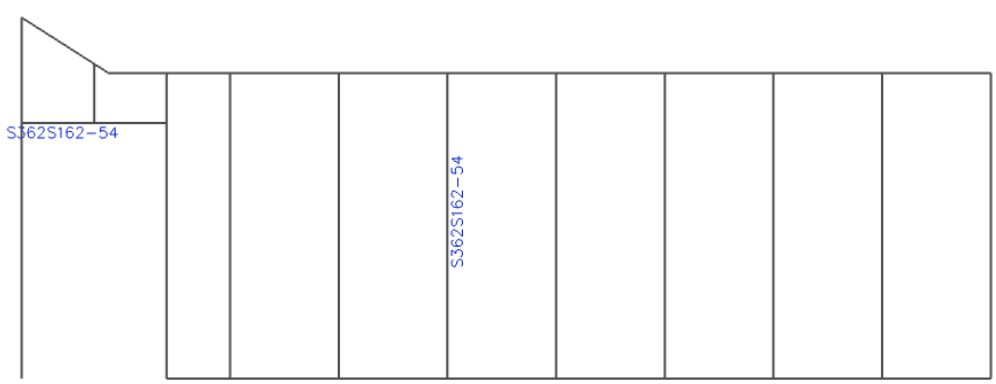
INTERIOR WALL E



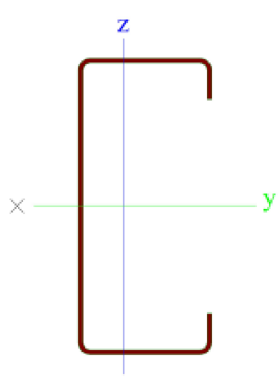
Member numbers



Cross-section walls element.



Section Properties:

CS11		
Type	S362S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.425	
A_y [inch^2], A_z [inch^2]	0.184	0.220
A_L [inch^2/inch], A_D [inch^2/inch]	1.50e+01	1.50e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.536	1.812
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.882	0.154
i_y [inch], i_z [inch]	1.441	0.603
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.481	0.142
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.559	0.212
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.80e+01	2.80e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.06e+01	1.06e+01
d_y [inch], d_z [inch]	-1.290	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.457
β_y [inch], β_z [inch]	0.000	4.130
Picture		

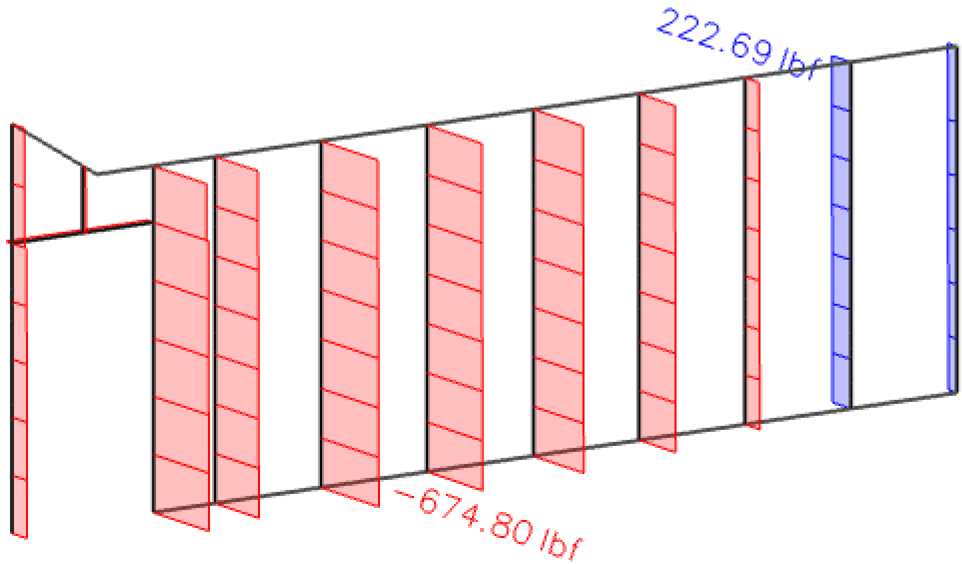
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction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

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

Maximum force diagram

Axial force diagram N,

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



STEEL MEMBER B3583 CHECK (STUD)

AISI S100-16 LRFD Check

Member B3583	S362S162-54	A1008 grade 54	LRFD-Ult (auto)	0.33

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

The critical check is on position 4.67 ft

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

Internal forces		
Pu	-492.82	lbf
Vux	18.93	lbf
Vuy	0.02	lbf
Mut	-0.01	lbfft
Mux	0.09	lbfft
Muy	88.32	lbfft

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.359 -	- -	-	- -	-
3	1.342	44.662 -18.616	0.42	12.522	583.964	0.277	1.000	- 1.342	0.393 0.671	-	- -	-
5	3.342	-23.954 -23.954	-	-	-	-	-	- -	- -	-	- -	-
7	1.342	44.662 -18.616	0.42	12.522	583.964	0.277	1.000	- 1.342	0.393 0.671	-	- -	-
9	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.359 -	- -	-	- -	-

Table of values		
Sye	0.142	$inch^3$
Mnyo	590.17	lbfft
Resistance factor	0.90	
Unity check	0.17	-

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.359	37.387 37.387	1.00	0.430	281.003	0.365	1.000	0.359 -	- -	-	- -	-
3	1.342	33.396 -13.920	0.42	12.522	583.964	0.239	1.000	- 1.342	0.393 0.671	-	- -	-
5	3.342	-17.911 -17.911	-	-	-	-	-	- -	- -	-	- -	-
7	1.342	33.396 -13.920	0.42	12.522	583.964	0.239	1.000	- 1.342	0.393 0.671	-	- -	-
9	0.359	37.387 37.387	1.00	0.430	281.003	0.365	1.000	0.359 -	- -	-	- -	-

Table of values		
Sigma,ex	189.603	ksi
Kt	1.00	
Lt	6.588	ft
Sigma,t	14.974	ksi
Cs	-1.00	
CTF	1.00	
Sfy	0.142	inch³
j	2.131	inch
Fcre	42.469	ksi
Fc	37.387	ksi

Table of values		
Scy	0.142	inch³
Mny	441.29	lbfft
Resistance factor	0.90	
Unity check	0.22	-

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

Table of values		
Sfy	0.142	inch³
My	590.11	lbfft
L	0.977	ft
Beta	1.00	
k,phi,fe	491.40	lbf
k,phi,we	438.80	lbf
k,phi	0.00	lbf
k,phi,fg	0.010	inch²
k,phi,wg	0.003	inch²
Fd	68.212	ksi
Sf	0.142	inch³
Mcrd	805.06	lbfft
Lambda,d	0.86	
Mn	512.14	lbfft
Resistance factor	0.90	
Unity check	0.19	-

Data		
Lm	6.588	ft
Lcr	0.977	ft
h0	3.625	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch
Ksi,web	0.00	

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

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	3.342	50.000 50.000	1.00	4.000	30.079	1.289	0.643	2.150 -	- -	-	- -	-
7	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
9	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.339	$inch^2$
Pno	16945.16	lbf
Resistance factor	0.85	
Unity check	0.03	-

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	4 3/4	6 5/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	4 3/4	6 5/8	ft
Slenderness	38.86	131.19	
Flexural Buckling stress Fcre	189.603	16.634	ksi

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

Table of values		
Sigma,ex	189.603	ksi
Sigma,ey	16.634	ksi
Kt	1.00	
Lt	6 5/8	ft
Sigma,t	14.974	ksi
Sigma,TF	14.488	ksi
Torsional (-Flexural) buckling stress Fcre	14.488	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	12.706 12.706	1.00	0.430	281.003	0.213	1.000	0.359 -	- -	-	- -	-
3	1.342	12.706 12.706	1.00	3.387	157.959	0.284	1.000	1.342 -	0.671 0.671	61.16	0.000 0.000	0.359
5	3.342	12.706 12.706	1.00	4.000	30.079	0.650	1.000	3.342 -	- -	-	- -	-
7	1.342	12.706 12.706	1.00	3.387	157.959	0.284	1.000	1.342 -	0.671 0.671	61.16	0.000 0.000	0.359
9	0.359	12.706 12.706	1.00	0.430	281.003	0.213	1.000	0.359 -	- -	-	- -	-

Table of values		
Fe	14.488	ksi
lambda, c	1.86	
Fn	12.706	ksi
Ae	0.422	inch^2
Pn	5360.08	lbf
Resistance factor	0.85	
Unity check	0.11	-

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

Table of values		
Py	21232.26	lbf
L	1.081	ft
k,phi,fe	341.95	lbf
k,phi,we	265.74	lbf
k,phi	0.00	lbf
k,phi,fg	0.008	inch^2
k,phi,wg	0.003	inch^2
Fd	55.567	ksi
Pcrd	23596.26	lbf
Lambda,d	0.95	
Pn	16595.67	lbf
Resistance factor	0.85	
Unity check	0.03	-

Data		
Lm	6.588	ft
Lcr	1.081	ft
h0	3.625	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch

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

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

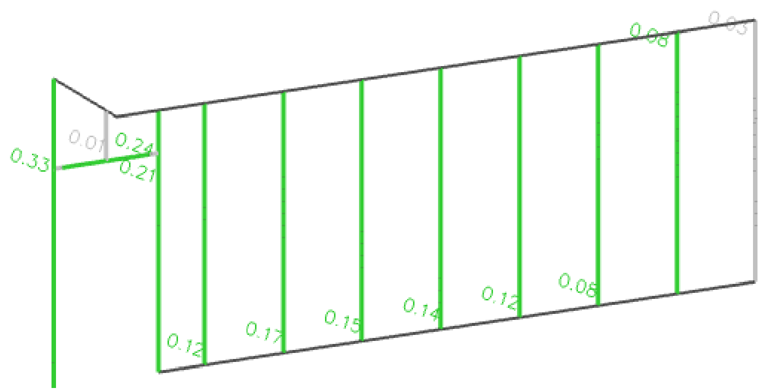
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	1.161 1.161	1.00	0.430	281.003	0.064	1.000	0.359 -	- -	-	- -	-
3	1.342	1.161 1.161	1.00	4.000	186.542	0.079	1.000	1.342 -	0.671 0.671	202.36	- 0.000	0.359
5	3.342	1.161 1.161	1.00	4.000	30.079	0.196	1.000	3.342 -	- -	-	- -	-
7	1.342	1.161 1.161	1.00	4.000	186.542	0.079	1.000	1.342 -	0.671 0.671	202.36	- 0.000	0.359
9	0.359	1.161 1.161	1.00	0.430	281.003	0.064	1.000	0.359 -	- -	-	- -	-

Table of values		
Mny	441.29	lbfft
Pn	5360.08	lbf
Resistance factor compression	0.85	
Resistance factor bending y	0.90	

Unity check = 0.11+0.00+0.22 = 0.33 - (C5.2.1-3)

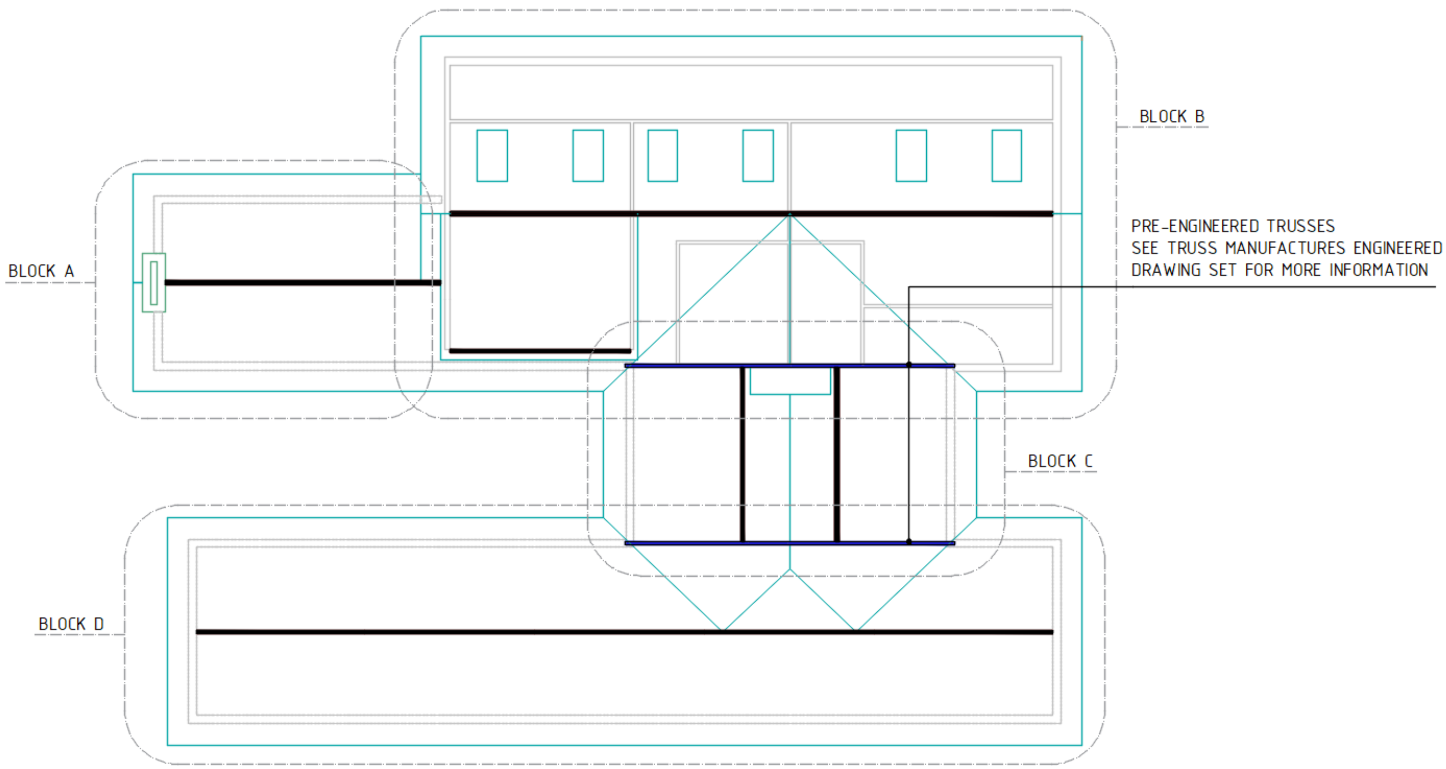
The member satisfies the check !

Unity check



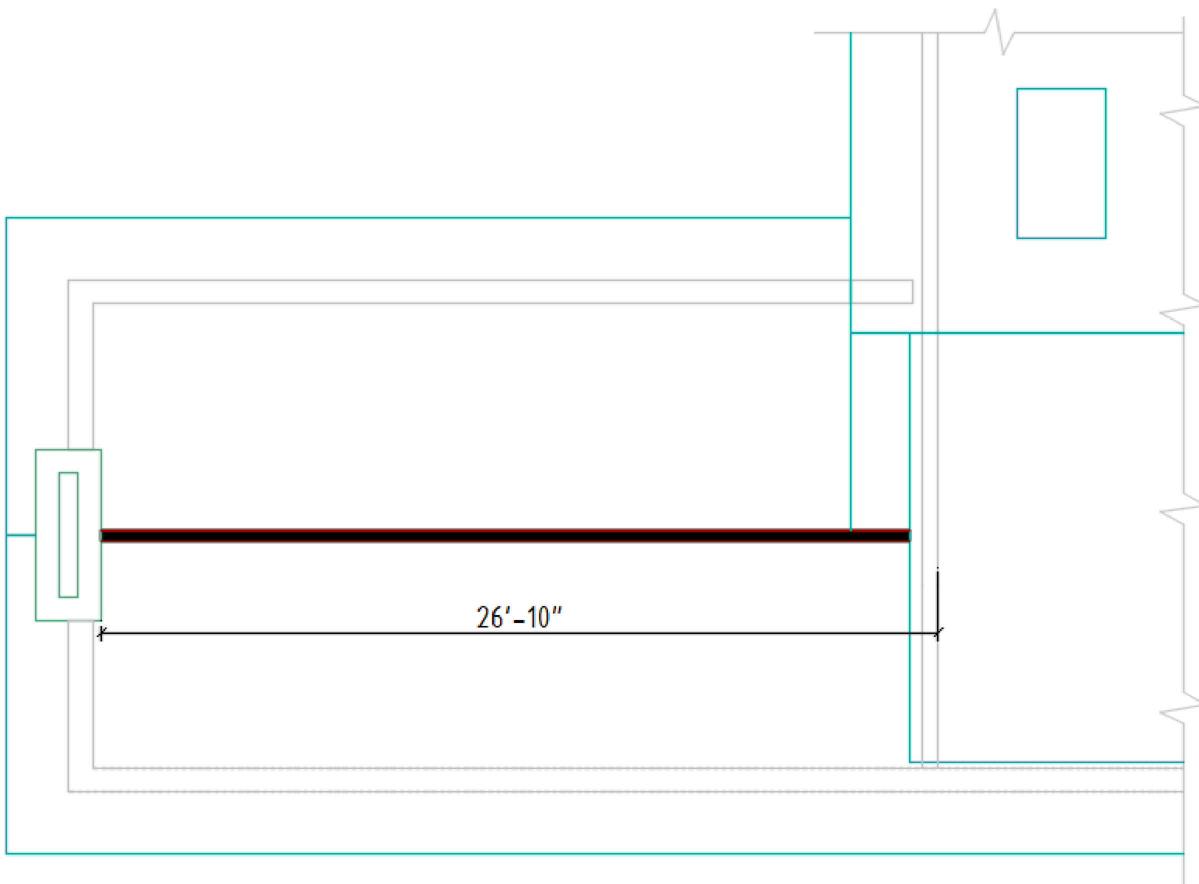
2.4.4 ROOF STRUCTURAL ANALYSIS

General scheme



BLOCK A. RIDGE BEAM STRUCTURAL ANALYSIS

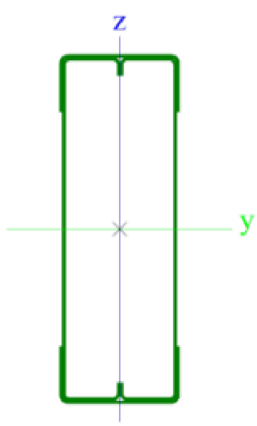
General scheme



-
Member numbers

Cross-sections of

Cross-sections properties

CS30		
Type	Box1200S200-97/400T200-97	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [$inch^2$]	4,864	
A_y [$inch^2$], A_z [$inch^2$]	1,703	2,661
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	3,27e+01	7,13e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1,623	0,000
α [deg]	0,00	
I_y [$inch^4$], I_z [$inch^4$]	108,080	14,173
i_y [inch], i_z [inch]	4,714	1,707
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	17,727	6,759
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	21,082	7,709
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1,05e+03	1,05e+03
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3,85e+02	3,85e+02
d_y [inch], d_z [inch]	0,000	0,000
I_t [$inch^4$], I_w [$inch^6$]	0,701	51,924
$β_y$ [inch], $β_z$ [inch]	0,000	0,000
Picture		

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

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

Maximum force diagram

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

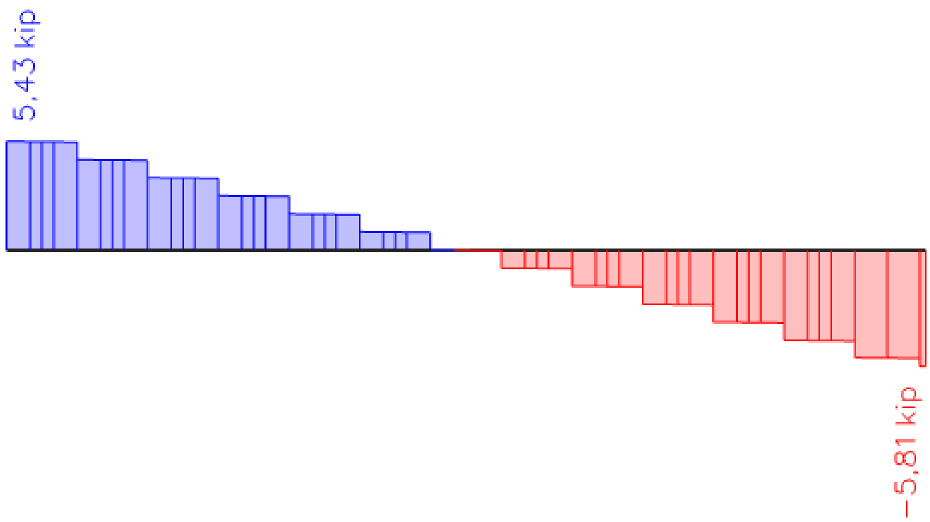
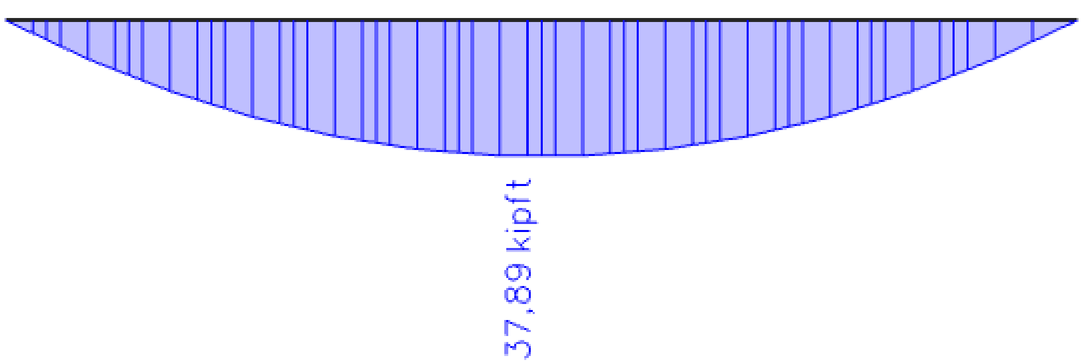
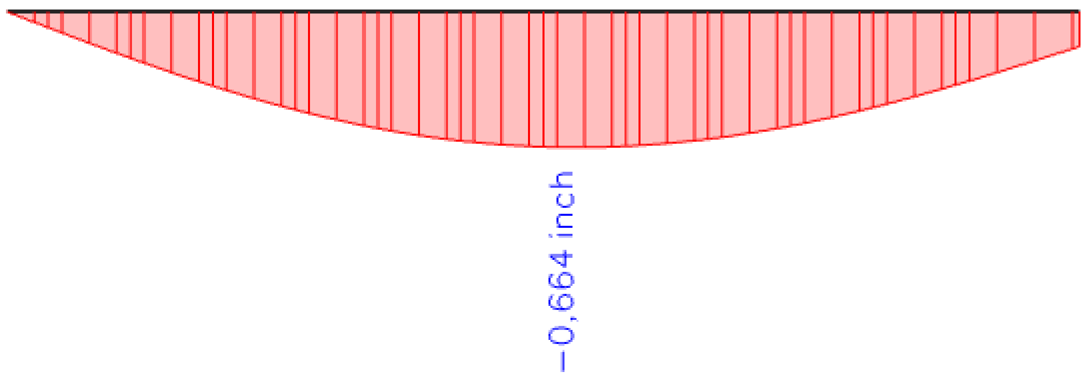


Diagram of moment My,
LRFD-Ult (auto)8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*L), lbf.



Displacement of elements

Value: Uz - Lr, (inch) .



The maximum deflection is 0.664" according to table 1604.3 the code IBC 2021 - the deflection limits L/360. L = 26'-10"= 26'*12"+10"=322". 322"/360 =0.894"
0.664" < 0.894" Deflection is OK!

STEEL MEMBER B377 CHECK (RIDGE BEAM)

AISI S100-16 LRFD Check

Member B377	Box1200S200-97/400T200-97	A1008 grade 54	LRFD-Ult (auto)	0.57

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

The critical check is on position 13.00 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	kip
Vux	0.00	kip
Vuy	-0.00	kip
Mut	-0.00	kipft
Mux	37.89	kipft
Muy	0.00	kipft

...::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	0.574	49.179 44.433	0.90	0.465	1529.412	0.179	1.000	0.574 -	- -	-	- -	-
2	1.898	49.580 49.580	1.00	4.000	1148.984	0.208	1.000	- 1.898	0.949 0.949	-	- -	-
3	8.194	33.868 -33.868	1.00	24.000	96.928	0.591	1.000	- 8.194	2.048 2.048	-	- -	-
4	1.898	-49.580 -49.580	-	-	-	-	-	- -	- -	-	- -	-
5	0.574	-44.433 -49.179	-	-	-	-	-	- -	- -	-	- -	-
6	1.898	-49.580 -49.580	-	-	-	-	-	- -	- -	-	- -	-
7	8.194	33.868 -33.868	1.00	24.000	96.928	0.591	1.000	- 8.194	2.049 2.049	-	- -	-
8	1.898	49.580 49.580	1.00	4.000	1148.984	0.208	1.000	- 1.898	0.949 0.949	-	- -	-
9	0.099	50.000 49.179	0.98	4.033	100788.016	0.022	1.000	- 0.099	0.049 0.050	-	- -	-
10	0.099	50.000 50.000	1.00	4.000	99966.789	0.022	1.000	- 0.099	0.050 0.050	-	- -	-
11	0.099	50.000 49.179	0.98	4.033	100788.016	0.022	1.000	- 0.099	0.049 0.050	-	- -	-
12	0.099	-49.179 -50.000	-	-	-	-	-	- -	- -	-	- -	-
13	0.099	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
14	0.099	-49.179 -50.000	-	-	-	-	-	- -	- -	-	- -	-
17	0.102	50.000 50.000	1.00	4.000	95400.265	0.023	1.000	- 0.102	0.051 0.051	-	- -	-
22	1.852	49.179 33.868	0.69	4.683	1413.048	0.187	1.000	- 1.852	0.801 1.051	-	- -	-
26	1.852	-33.868 -49.179	-	-	-	-	-	- -	- -	-	- -	-
30	0.099	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
37	0.102	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
42	1.852	49.179 33.868	0.69	4.683	1413.048	0.187	1.000	- 1.852	0.801 1.051	-	- -	-
46	1.852	-33.868 -49.179	-	-	-	-	-	- -	- -	-	- -	-
50	0.099	50.000 50.000	1.00	4.000	99966.789	0.022	1.000	- 0.099	0.050 0.050	-	- -	-

Table of values		
Sxe	17.727	$inch^3$
Mnxo	73.86	kipft
Resistance factor	0.90	
Unity check	0.57	-

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

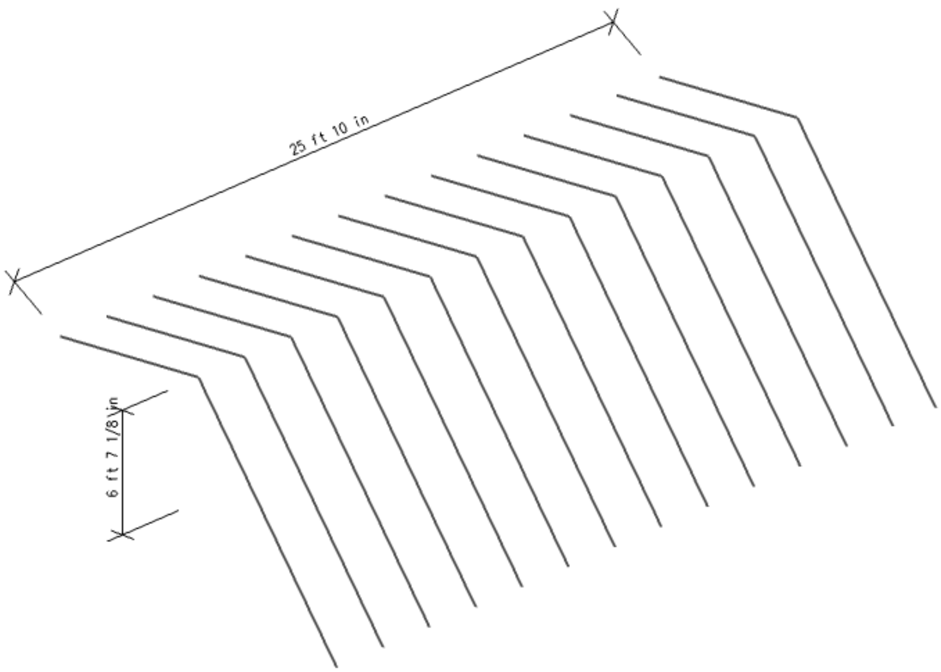
Table of values		
Lltb	2.000	ft
Sigma,ey	1448.136	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	275.061	ksi
Cb	1.00	
Sfx	17.727	$inch^3$
Fcre	868.268	ksi

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

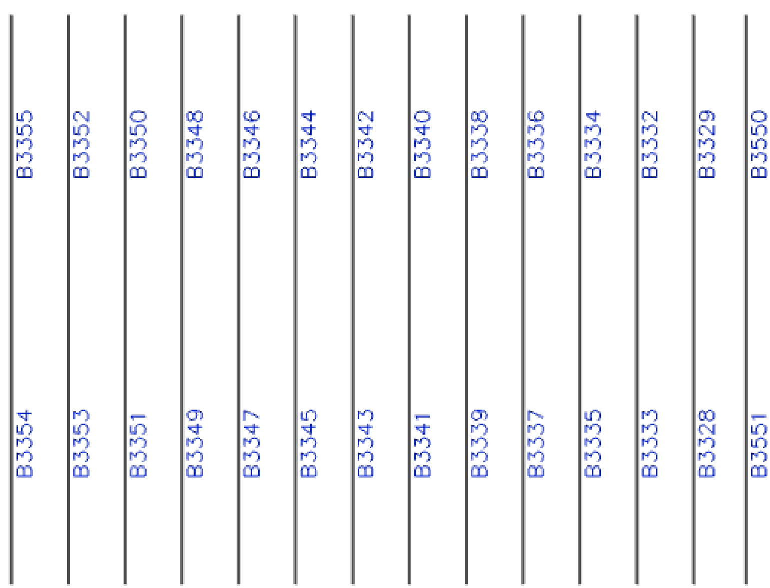
The member satisfies the check !

BLOCK A. ROOF RAFTER STRUCTURAL ANALYSIS

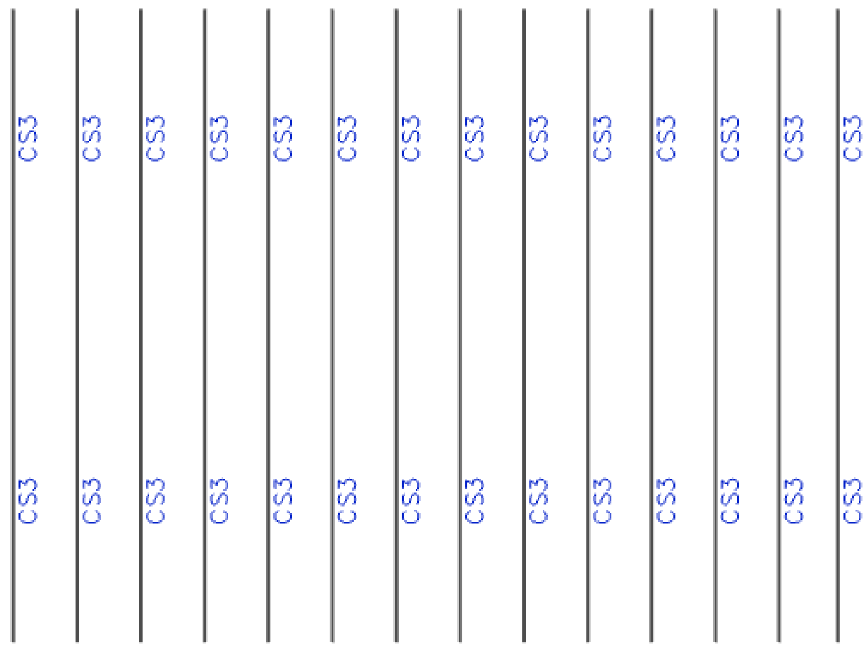
General scheme



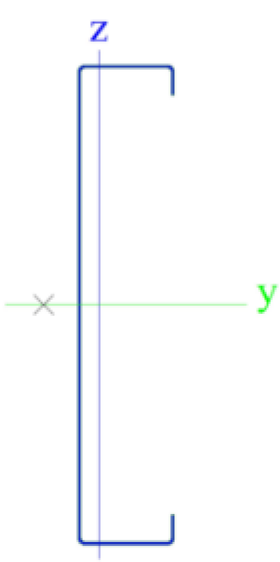
Member numbers



Cross-sections of



CS3 Cross-sections properties

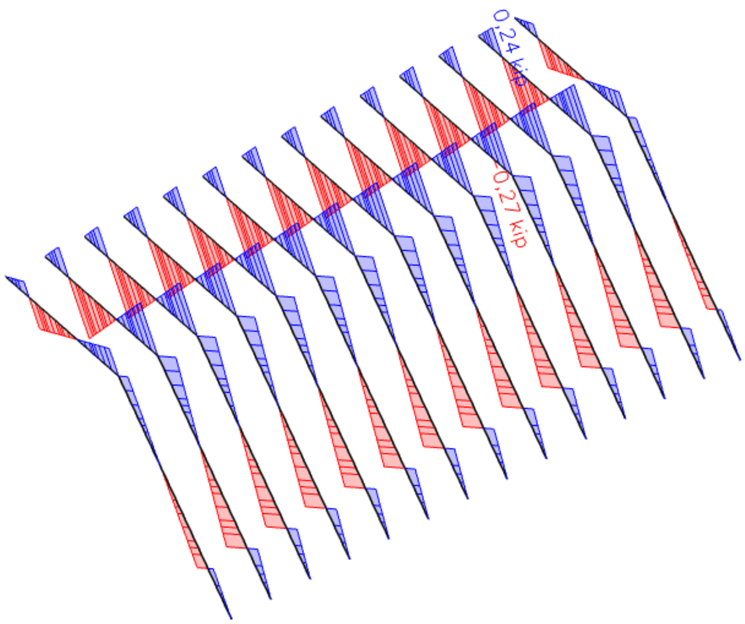
CS3		
Type	S1000S200-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.842	
A_y [inch^2], A_z [inch^2]	0.229	0.566
A_L [inch^2/inch], A_D [inch^2/inch]	2.98e+01	2.98e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.427	5.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	11.350	0.378
i_y [inch], i_z [inch]	3.671	0.670
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	2.255	0.240
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.753	0.340
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.38e+02	1.38e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.70e+01	1.70e+01
d_y [inch], d_z [inch]	-1.143	0.000
I_t [inch^4], I_w [inch^6]	0.001	7.665
β_y [inch], β_z [inch]	0.000	12.209
Picture		

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

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

Maximum force diagram

Axial force diagram N,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr), lbf.



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

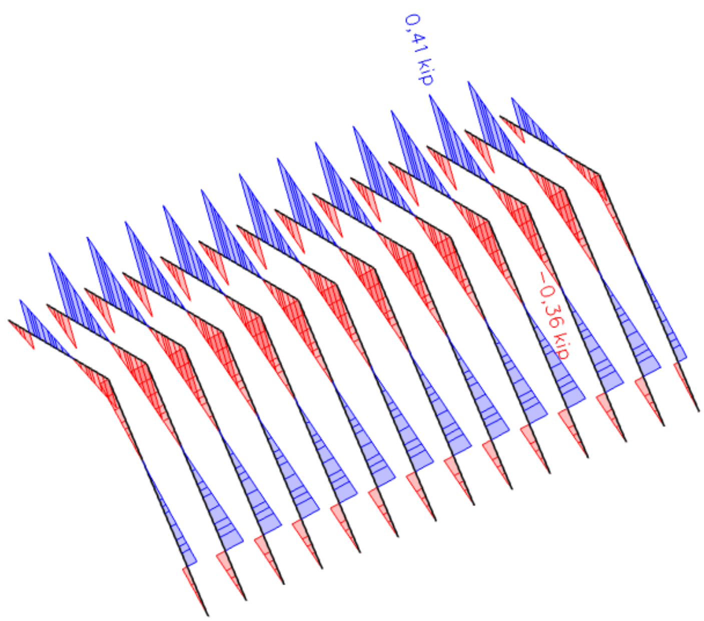
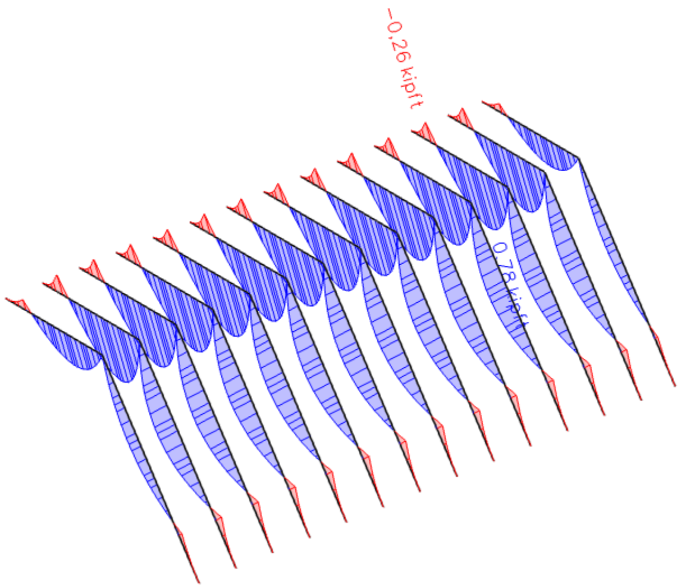
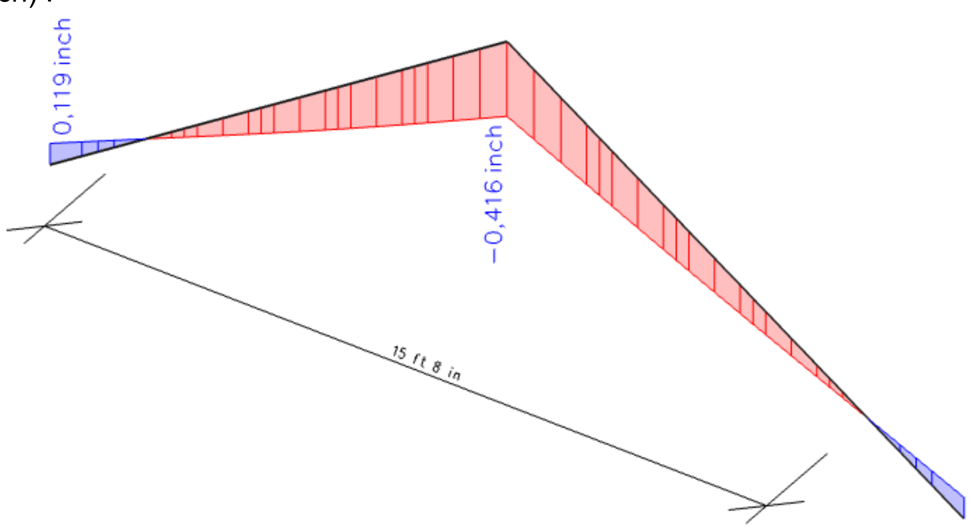


Diagram of moment My,
LRFD-Ult (auto)8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*L), lbf.



Displacement of elements
Value: Uz - Lr, (inch) .



The maximum deflection is 0.416" according to table 1604.3 the code IBC 2021 - the deflection limits L/360. L = 15'-8"= 16'*12"+8"=188". 188"/360 =0.52"
0.416" < 0.52" <u>Deflection is OK!</u>

STEEL MEMBER B3343 CHECK (RAFTER)

AISI S100-16 LRFD Check

Member Light B3343	S1000S200-54	A1008 grade 54	LRFD-Ult (auto)	0.17

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

The critical check is on position 2.50 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	-0.27	kip
Vux	-0.00	kip
Vuy	0.41	kip
Mut	-0.00	kipft
Mux	-0.26	kipft
Muy	0.00	kipft

...::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	0.484	48.999 44.723	0.91	0.461	165.766	0.544	1.000	0.223 -	- -	-	- -	-
3	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
5	9.717	48.999 -36.932	0.75	18.295	16.274	1.735	0.503	- 4.890	1.303 1.486	-	- -	-
7	1.717	-37.933 -37.933	-	-	-	-	-	- -	- -	-	- -	-
9	0.484	-32.657 -36.932	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	1.690	$inch^3$
Mnxo	7.04	kipft
Resistance factor	0.90	
Unity check	0.04	-

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

Table of values		
Lltb	2.000	ft
Sigma,ey	222.962	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	297.800	ksi
Cb	2.05	
Sfx	2.270	$inch^3$
Fcre	765.880	ksi

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

Table of values		
Sfy	2.270	$inch^3$
My	9.46	kipft
L	1.678	ft
Beta	1.30	
k,phi,fe	0.18	kip
k,phi,we	0.18	kip
k,phi	0.00	kip

Table of values		
k,phi,fg	0.007	$inch^2$
k,phi,wg	0.004	$inch^2$
Fd	45.435	ksi
Sf	2.270	$inch^3$
Mcrd	8.59	kipft
Lambda,d	1.05	
Mn	7.13	kipft
Resistance factor	0.90	
Unity check	0.04	-

Data		
Lm	2.000	ft
Lcr	1.678	ft
h0	10.000	inch
Ixf	0.003	$inch^4$
Iyf	0.049	$inch^4$
Ixyf	0.007	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.691	inch
hxf	-1.239	inch
Af	0.135	$inch^2$
y0f	-0.068	inch
Ksi,web	2.00	

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

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

Element ID	Aw [inch²]	Vn [kip]
3	0.000	0.00
5	0.550	2.61
7	0.000	0.00

Table of values		
Vn,y	2.61	kip
Resistance factor	0.95	
Unity check	0.17	-

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

Table of values		
Mnxo	7.04	kipft
Vny	2.61	kip
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.00+0.03) = 0.17

...::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	0.484	50.000 50.000	1.00	0.430	154.489	0.569	1.000	0.223 -	- -	-	- -	-
3	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
5	9.717	50.000 50.000	1.00	4.000	3.558	3.749	0.251	2.440 -	- -	-	- -	-
7	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
9	0.484	50.000 50.000	1.00	0.430	154.489	0.569	1.000	0.223 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.380	inch²
Pno	18.99	kip
Resistance factor	0.85	
Unity check	0.02	-

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	9 1/2	2	ft
Effective Length factor K	1.00	1.00	
Effective Length	9 1/2	2	ft
Slenderness	30.74	35.83	
Flexural Buckling stress Fcre	302.945	222.962	ksi

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

Table of values		
Sigma,ex	302.945	ksi
Sigma,ey	222.962	ksi
Kt	1.00	
Lt	2	ft
Sigma,t	297.800	ksi
Sigma,TF	222.962	ksi
Torsional (-Flexural) buckling stress Fcre	222.962	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.484	45.520 45.520	1.00	0.430	154.489	0.543	1.000	0.276 -	- -	-	- -	-
3	1.717	45.520 45.520	1.00	2.900	82.612	0.742	0.948	1.628 -	0.465 1.163	32.31	0.001 0.001	0.276
5	9.717	45.520 45.520	1.00	4.000	3.558	3.577	0.262	2.550 -	- -	-	- -	-
7	1.717	45.520 45.520	1.00	2.900	82.612	0.742	0.948	1.628 -	0.465 1.163	32.31	0.001 0.001	0.276
9	0.484	45.520 45.520	1.00	0.430	154.489	0.543	1.000	0.276 -	- -	-	- -	-

Table of values		
Fe	222.962	ksi
lambda, c	0.47	
Fn	45.520	ksi
Ae	0.400	$inch^2$
Pn	18.21	kip
Resistance factor	0.85	
Unity check	0.02	-

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

Table of values		
Py	42.10	kip
L	1.853	ft
k,phi,fe	0.13	kip
k,phi,we	0.10	kip
k,phi	0.00	kip
k,phi,fg	0.005	$inch^2$
k,phi,wg	0.019	$inch^2$
Fd	9.266	ksi
Pcrd	7.80	kip
Lambda,d	2.32	
Pn	13.92	kip
Resistance factor	0.85	
Unity check	0.02	-

Data		
Lm	2.000	ft
Lcr	1.853	ft
h0	10.000	inch
Ixf	0.003	$inch^4$
Iyf	0.049	$inch^4$
Ixyf	-0.007	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.691	inch
hxf	-1.239	inch
Af	0.135	$inch^2$
y0f	0.068	inch

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

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

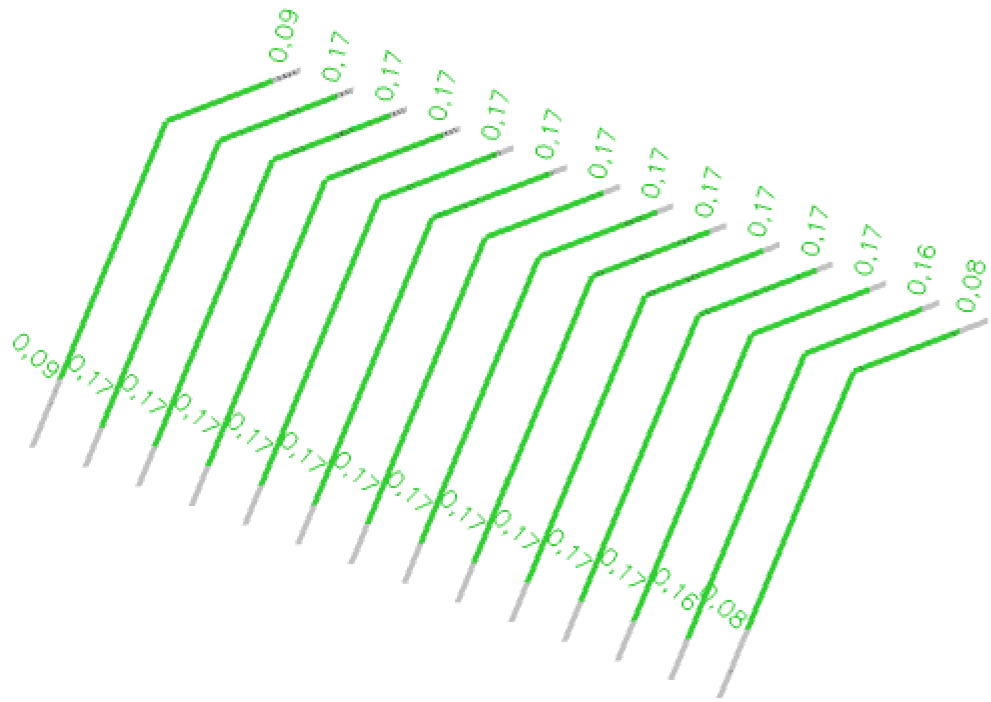
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.484	0.326 0.326	1.00	0.430	154.489	0.046	1.000	0.484 -	- -	-	- -	-
3	1.717	0.326 0.326	1.00	4.000	113.957	0.054	1.000	1.717 -	0.858 0.858	381.57	- 0.001	0.484
5	9.717	0.326 0.326	1.00	4.000	3.558	0.303	1.000	9.717 -	- -	-	- -	-
7	1.717	0.326 0.326	1.00	4.000	113.957	0.054	1.000	1.717 -	0.858 0.858	381.57	- 0.001	0.484
9	0.484	0.326 0.326	1.00	0.430	154.489	0.046	1.000	0.484 -	- -	-	- -	-

Table of values		
Mnx	7.04	kipft
Pn	13.92	kip
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.02+0.04+0.00 = 0.06 - (C5.2.1-3)

The member satisfies the check !

Unity check


0,09
0,17
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0,17
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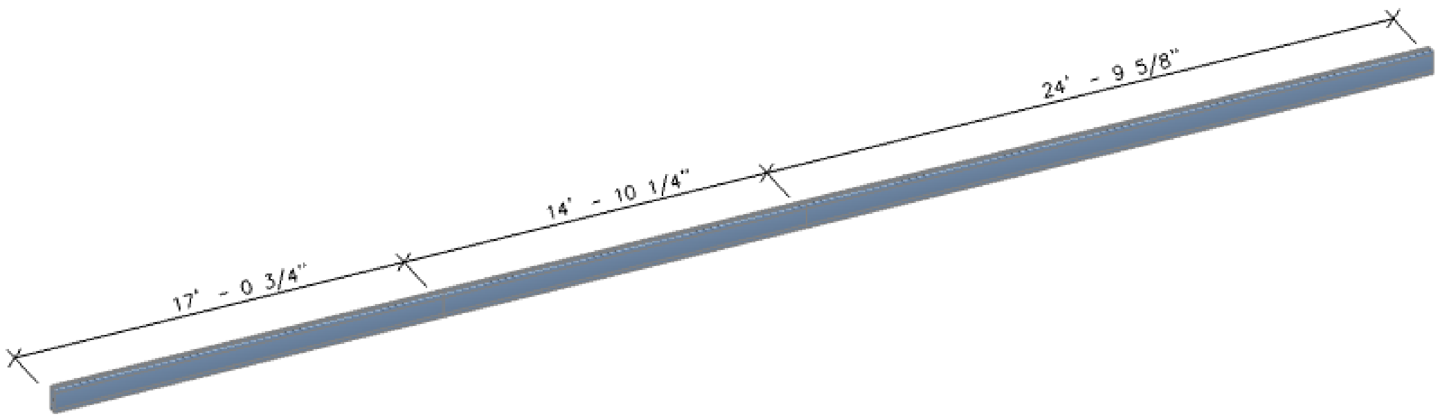
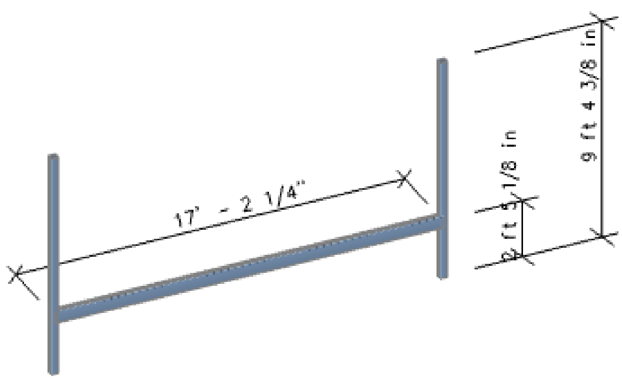
0,17
0,16
0,08

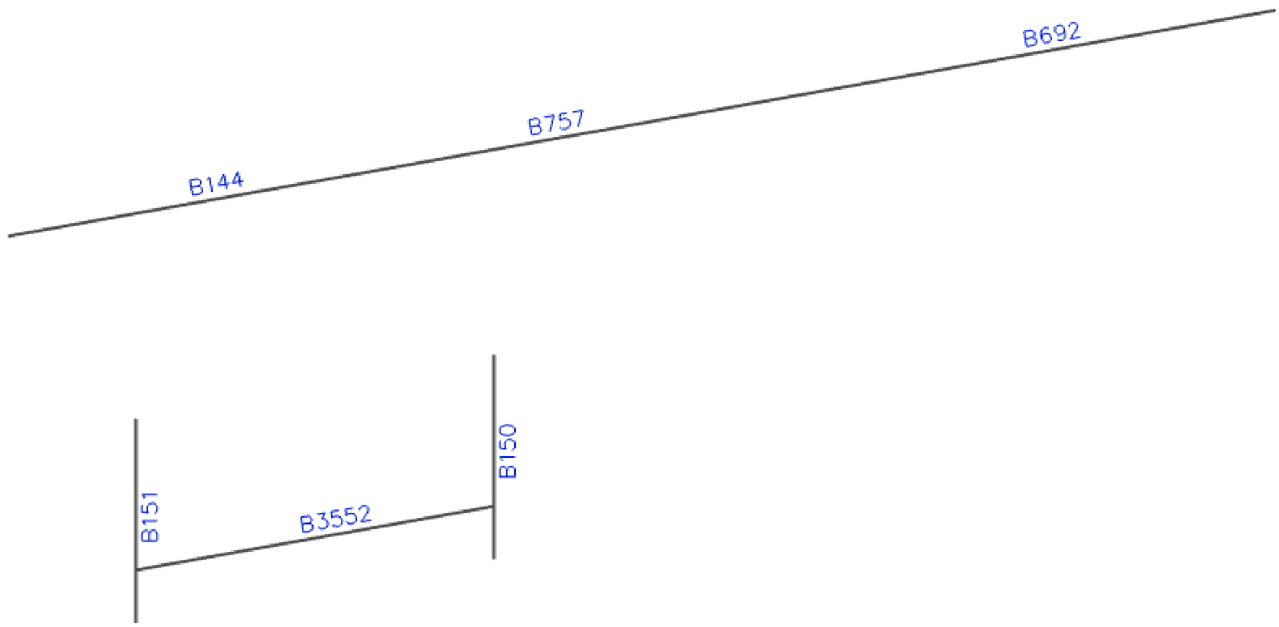
BLOCK B. RIDGE BEAM & COLUMN STRUCTURAL ANALYSIS

General scheme

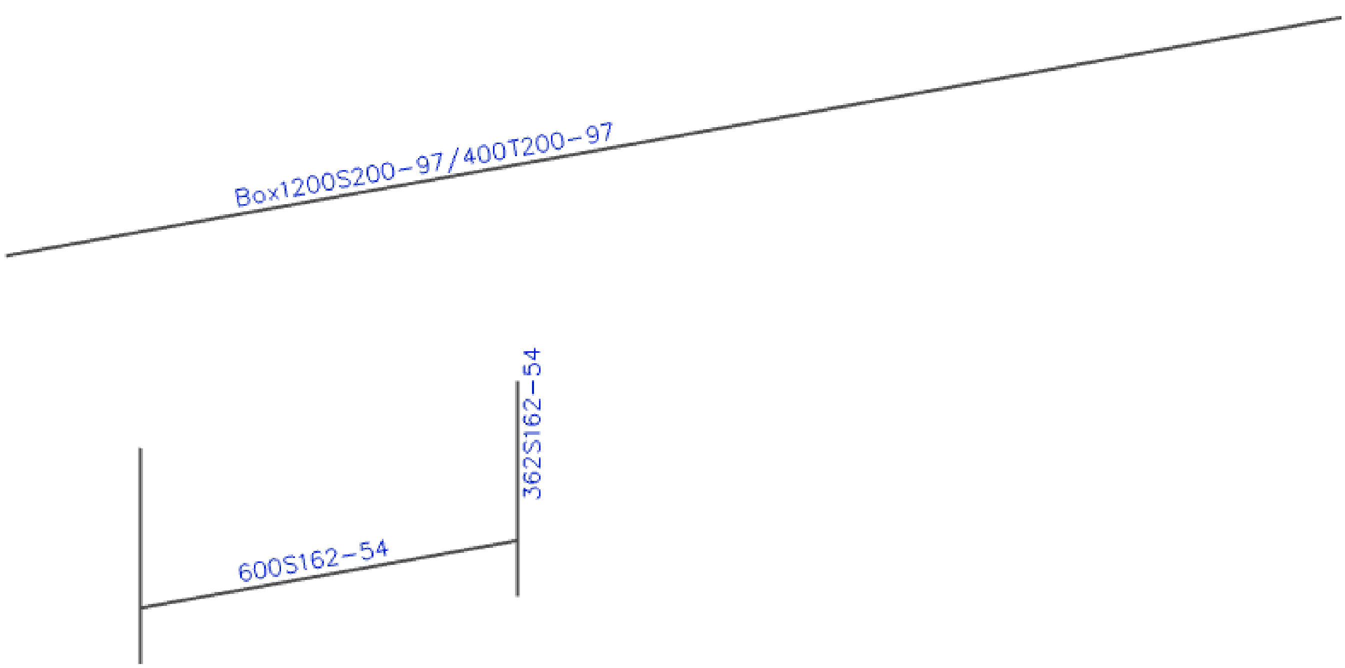




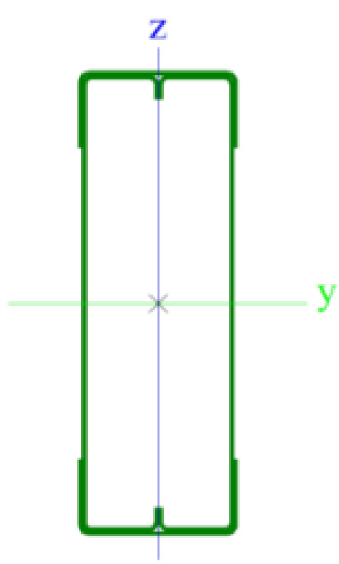
Member numbers

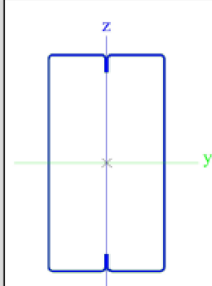


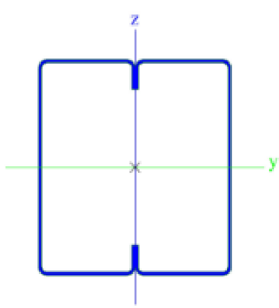
Cross-sections of



Cross-sections properties

CS30		
Type	Box1200S200-97/400T200-97	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	4,864	
A_y [inch^2], A_z [inch^2]	1,703	2,661
A_L [inch^2/inch], A_D [inch^2/inch]	3,27e+01	7,13e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1,623	0,000
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	108,080	14,173
i_y [inch], i_z [inch]	4,714	1,707
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	17,727	6,759
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	21,082	7,709
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1,05e+03	1,05e+03
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3,85e+02	3,85e+02
d_y [inch], d_z [inch]	0,000	0,000
I_t [inch^4], I_w [inch^6]	0,701	51,924
$β_y$ [inch], $β_z$ [inch]	0,000	0,000
Picture		

CS39		
Type	600S162-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	1,112	
A_y [inch^2], A_z [inch^2]	0,394	0,692
A_L [inch^2/inch], A_D [inch^2/inch]	1,86e+01	3,81e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-2,061	0,000
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	5,718	1,994
i_y [inch], i_z [inch]	2,267	1,339
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1,906	1,227
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2,280	1,348
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1,14e+02	1,14e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	6,74e+01	6,74e+01
d_y [inch], d_z [inch]	0,000	0,000
I_t [inch^4], I_w [inch^6]	0,452	18,401
$β_y$ [inch], $β_z$ [inch]	0,000	0,000
Picture		

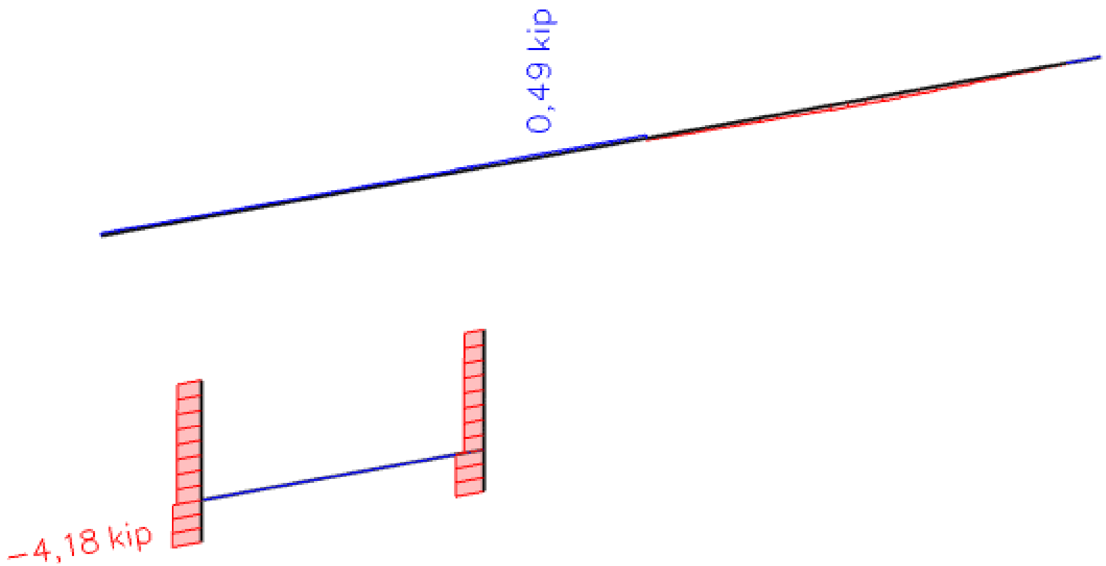
CS42		
Type	362S162-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0,843	
A_y [inch^2], A_z [inch^2]	0,391	0,439
A_L [inch^2/inch], A_D [inch^2/inch]	1,38e+01	2,86e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-2,184	0,000
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	1,745	1,308
i_y [inch], i_z [inch]	1,438	1,245
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0,963	0,805
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	1,119	0,918
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5,59e+01	5,59e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	4,59e+01	4,59e+01
d_y [inch], d_z [inch]	0,000	0,000
I_t [inch^4], I_w [inch^6]	0,168	6,567
$β_y$ [inch], $β_z$ [inch]	0,000	0,000
Picture		

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

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

Maximum force diagram

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



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

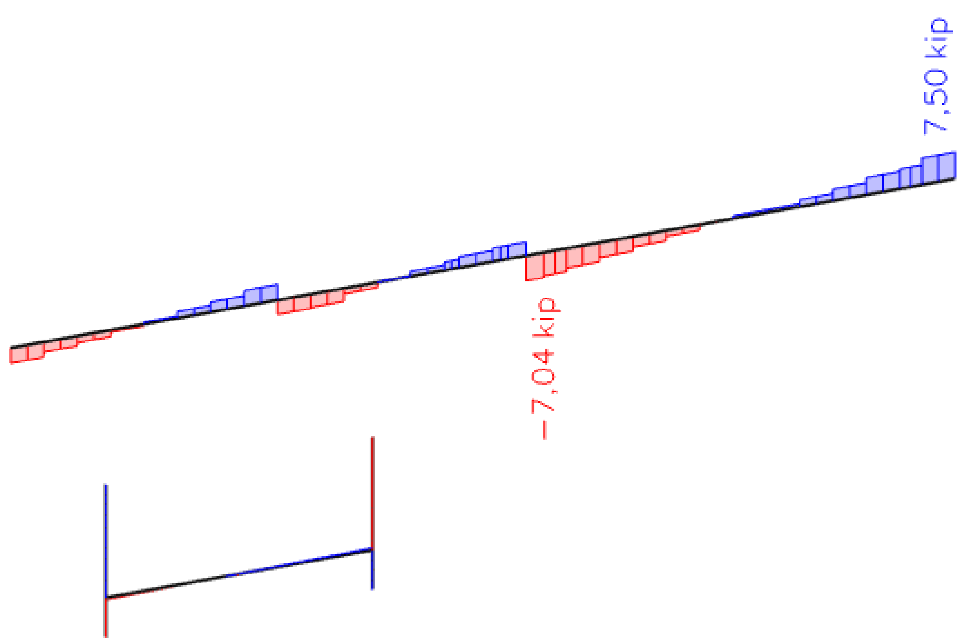
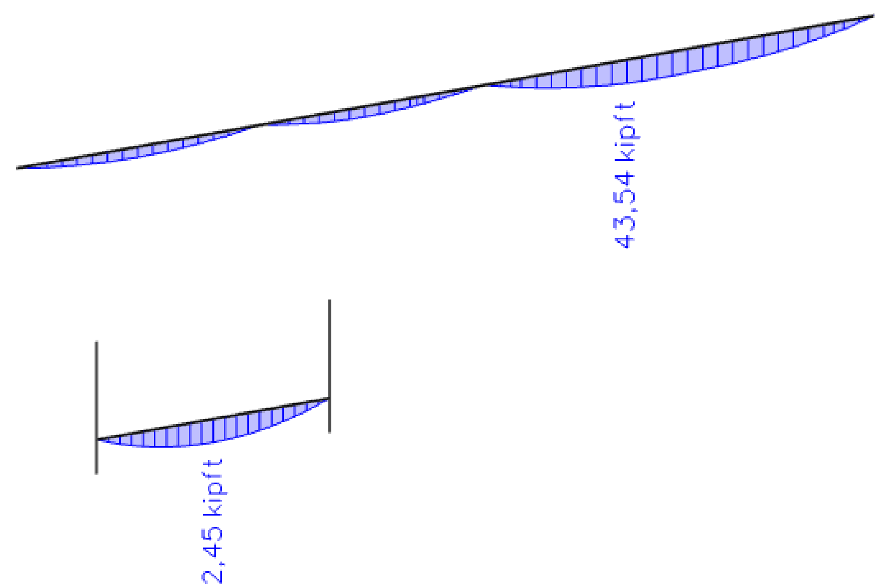
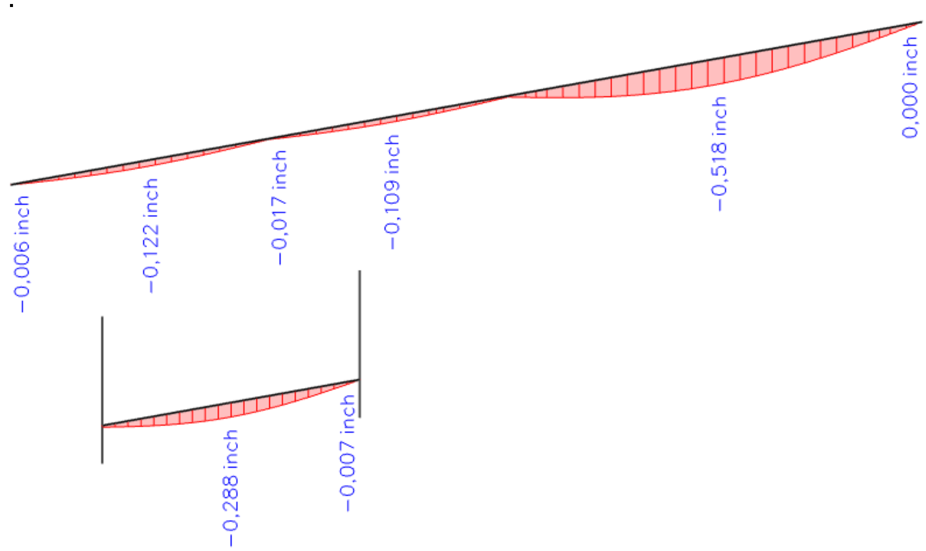


Diagram of moment My,
LRFD-Ult (auto)8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*L), lbf.



Displacement of elements
Value: Uz - Lr, (inch) .



The maximum deflection is 0.518" according to table 1604.3 the code IBC 2021 - the deflection limits L/360. L = 24'-9 5/8"= 24'*12"+9 5/8"=288". 288 5/8"/360 =0.801"
0.518" < 0.801" <u>Deflection is OK!</u>

STEEL MEMBER B692 CHECK (RIDGE BEAM)

AISI S100-16 LRFD Check

Member B692	Box1200S200-97/400T200-97	A1008 grade 54	LRFD-Ult (auto)	0.80

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

The critical check is on position 11.33 ft

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

Internal forces		
Pu	-663.21	lbf
Vux	281.70	lbf
Vuy	789.77	lbf
Mut	0.00	lbfft
Mux	41990.91	lbfft
Muy	3518.65	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	0.574	49.179 44.433	0.90	0.465	1529.412	0.179	1.000	0.574 -	- -	-	- -	-
2	1.898	49.580 49.580	1.00	4.000	1148.984	0.208	1.000	- 1.898	0.949 0.949	-	- -	-
3	8.194	33.868 -33.868	1.00	24.000	96.928	0.591	1.000	- 8.194	2.048 2.048	-	- -	-
4	1.898	-49.580 -49.580	-	-	-	-	-	- -	- -	-	- -	-
5	0.574	-44.433 -49.179	-	-	-	-	-	- -	- -	-	- -	-
6	1.898	-49.580 -49.580	-	-	-	-	-	- -	- -	-	- -	-
7	8.194	33.868 -33.868	1.00	24.000	96.928	0.591	1.000	- 8.194	2.049 2.049	-	- -	-
8	1.898	49.580 49.580	1.00	4.000	1148.984	0.208	1.000	- 1.898	0.949 0.949	-	- -	-
9	0.099	50.000 49.179	0.98	4.033	100788.016	0.022	1.000	- 0.099	0.049 0.050	-	- -	-
10	0.099	50.000 50.000	1.00	4.000	99966.789	0.022	1.000	- 0.099	0.050 0.050	-	- -	-
11	0.099	50.000 49.179	0.98	4.033	100788.016	0.022	1.000	- 0.099	0.049 0.050	-	- -	-
12	0.099	-49.179 -50.000	-	-	-	-	-	- -	- -	-	- -	-

13	0.099	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
14	0.099	-49.179 -50.000	-	-	-	-	-	- -	- -	-	- -	-
17	0.102	50.000 50.000	1.00	4.000	95400.265	0.023	1.000	- 0.102	0.051 0.051	-	- -	-
22	1.852	49.179 33.868	0.69	4.683	1413.048	0.187	1.000	- 1.852	0.801 1.051	-	- -	-
26	1.852	-33.868 -49.179	-	-	-	-	-	- -	- -	-	- -	-
30	0.099	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
37	0.102	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
42	1.852	49.179 33.868	0.69	4.683	1413.048	0.187	1.000	- 1.852	0.801 1.051	-	- -	-
46	1.852	-33.868 -49.179	-	-	-	-	-	- -	- -	-	- -	-
50	0.099	50.000 50.000	1.00	4.000	99966.789	0.022	1.000	- 0.099	0.050 0.050	-	- -	-

Table of values		
Sxe	17.727	$inch^3$
Mnxo	73.86	kipft
Resistance factor	0.90	
Unity check	0.63	-

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

Table of values		
Lltb	2.000	ft
Sigma,ey	1448.136	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	275.061	ksi
Cb	1.02	
Sfx	17.727	$inch^3$
Fcre	881.717	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	0.574	3.929 3.929	1.00	0.430	1414.841	0.053	1.000	0.574 -	- -	-	- -	-
2	1.898	2.785 -39.907	14.33	7235.866	2078473.900	0.001	1.000	- 1.898	0.110 0.014	-	- -	-
3	8.194	-39.907 -39.907	-	-	-	-	-	- -	- -	-	- -	-
4	1.898	2.785 -39.907	14.33	7235.866	2078473.900	0.001	1.000	- 1.898	0.110 0.014	-	- -	-
5	0.574	3.929 3.929	1.00	0.430	1414.841	0.053	1.000	0.574 -	- -	-	- -	-

6	1.898	47.766 5.073	0.11	7.216	2072.674	0.152	1.000	- 1.898	0.656 1.242	-	- -	-
7	8.194	47.766 47.766	1.00	4.000	16.155	1.720	0.507	- 4.156	2.078 2.078	-	- -	-
8	1.898	47.766 5.073	0.11	7.216	2072.674	0.152	1.000	- 1.898	0.656 1.242	-	- -	-
9	0.099	-42.142 -42.142	-	-	-	-	-	- -	- -	-	- -	-
10	0.099	-39.907 -42.142	-	-	-	-	-	- -	- -	-	- -	-
11	0.099	50.000 50.000	1.00	4.000	99966.789	0.022	1.000	- 0.099	0.050 0.050	-	- -	-
12	0.099	50.000 50.000	1.00	4.000	99966.789	0.022	1.000	- 0.099	0.050 0.050	-	- -	-
13	0.099	50.000 47.766	0.96	4.090	102204.896	0.022	1.000	- 0.099	0.049 0.051	-	- -	-
14	0.099	-42.142 -42.142	-	-	-	-	-	- -	- -	-	- -	-
17	0.102	5.073 2.785	0.55	5.085	121280.441	0.006	1.000	- 0.102	0.041 0.060	-	- -	-
22	1.852	-40.998 -40.998	-	-	-	-	-	- -	- -	-	- -	-
26	1.852	-40.998 -40.998	-	-	-	-	-	- -	- -	-	- -	-
30	0.099	-39.907 -42.142	-	-	-	-	-	- -	- -	-	- -	-
37	0.102	5.073 2.785	0.55	5.085	121280.441	0.006	1.000	- 0.102	0.041 0.060	-	- -	-
42	1.852	48.856 48.856	1.00	4.000	1206.956	0.201	1.000	- 1.852	0.926 0.926	-	- -	-
46	1.852	48.856 48.856	1.00	4.000	1206.956	0.201	1.000	- 1.852	0.926 0.926	-	- -	-
50	0.099	50.000 47.766	0.96	4.090	102204.896	0.022	1.000	- 0.099	0.049 0.051	-	- -	-

Table of values		
Sye	5.636	$inch^3$
Mnyo	23.48	kipft
Resistance factor	0.90	
Unity check	0.17	-

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

Table of values		
Sigma,ex	66.620	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	275.061	ksi
Cb	1.00	
Sfy	6.759	$inch^3$
Fcre	488.436	ksi

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

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

Shear Strength

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

Shear force Vx

Element ID	Aw [inch ^{2}]	Vn [kip]
1	0.000	0.00
2	0.377	11.32
3	0.000	0.00
4	0.377	11.32
5	0.000	0.00
6	0.377	11.32
7	0.000	0.00
8	0.377	11.32
9	0.000	0.00
10	0.010	0.29
11	0.000	0.00
12	0.000	0.00
13	0.010	0.29
14	0.000	0.00
17	0.010	0.30
22	0.000	0.00
26	0.000	0.00
30	0.010	0.29
37	0.010	0.30
42	0.000	0.00
46	0.000	0.00
50	0.010	0.29

Table of values		
Vn,x	47.01	kip
Resistance factor	0.95	
Unity check	0.01	-

Shear force Vy

Element ID	Aw [inch ^{2}]	Vn [kip]
1	0.117	3.50
2	0.000	0.00
3	0.833	17.28
4	0.000	0.00
5	0.117	3.50
6	0.000	0.00
7	0.833	17.28
8	0.000	0.00
9	0.010	0.29
10	0.000	0.00
11	0.010	0.29
12	0.010	0.29
13	0.000	0.00
14	0.010	0.29
17	0.000	0.00
22	0.368	11.04
26	0.368	11.04
30	0.000	0.00
37	0.000	0.00
42	0.368	11.04
46	0.368	11.04
50	0.000	0.00

Table of values		
Vn,y	86.88	kip
Resistance factor	0.95	
Unity check	0.01	-

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

Table of values		
Mnxo	73.86	kipft
Vny	86.88	kip
Mnyo	23.48	kipft
Vnx	47.01	kip
Resistance factor shear	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check (Mx, Vy) = sqrt(0.40+0.00) = 0.63
Unity check (My, Vx) = sqrt(0.03+0.00) = 0.17

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

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	25 7/8	2	ft
Effective Length factor K	1.00	1.00	
Effective Length	25 7/8	2	ft
Slenderness	65.55	14.06	
Flexural Buckling stress Fcre	66.620	1448.136	ksi

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

Table of values		
Sigma,ex	66.620	ksi
Sigma,ey	1448.136	ksi
Kt	1.00	
Lt	2	ft
Sigma,t	275.061	ksi
Sigma,TF	66.620	ksi
Torsional (-Flexural) buckling stress Fcre	66.620	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.574	36.521 36.521	1.00	0.430	1414.841	0.161	1.000	0.574 -	- -	-	- -	-
2	1.898	36.521 36.521	1.00	4.000	1148.984	0.178	1.000	1.898 -	- -	-	- -	-
3	8.194	36.521 36.521	1.00	4.000	16.155	1.504	0.568	4.652 -	- -	-	- -	-
4	1.898	36.521 36.521	1.00	4.000	1148.984	0.178	1.000	1.898 -	- -	-	- -	-
5	0.574	36.521 36.521	1.00	0.430	1414.841	0.161	1.000	0.574 -	- -	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
6	1.898	36.521 36.521	1.00	4.000	1148.984	0.178	1.000	1.898 -	- -	-	- -	-
7	8.194	36.521 36.521	1.00	4.000	16.155	1.504	0.568	4.652 -	- -	-	- -	-
8	1.898	36.521 36.521	1.00	4.000	1148.984	0.178	1.000	1.898 -	- -	-	- -	-
9	0.099	36.521 36.521	1.00	4.000	99966.789	0.019	1.000	0.099 -	- -	-	- -	-
10	0.099	36.521 36.521	1.00	4.000	99966.789	0.019	1.000	0.099 -	- -	-	- -	-
11	0.099	36.521 36.521	1.00	4.000	99966.789	0.019	1.000	0.099 -	- -	-	- -	-
12	0.099	36.521 36.521	1.00	4.000	99966.789	0.019	1.000	0.099 -	- -	-	- -	-
13	0.099	36.521 36.521	1.00	4.000	99966.789	0.019	1.000	0.099 -	- -	-	- -	-
14	0.099	36.521 36.521	1.00	4.000	99966.789	0.019	1.000	0.099 -	- -	-	- -	-
17	0.102	36.521 36.521	1.00	4.000	95400.265	0.020	1.000	0.102 -	- -	-	- -	-
22	1.852	36.521 36.521	1.00	4.000	1206.956	0.174	1.000	1.852 -	- -	-	- -	-
26	1.852	36.521 36.521	1.00	4.000	1206.956	0.174	1.000	1.852 -	- -	-	- -	-
30	0.099	36.521 36.521	1.00	4.000	99966.789	0.019	1.000	0.099 -	- -	-	- -	-
37	0.102	36.521 36.521	1.00	4.000	95400.265	0.020	1.000	0.102 -	- -	-	- -	-
42	1.852	36.521 36.521	1.00	4.000	1206.956	0.174	1.000	1.852 -	- -	-	- -	-
46	1.852	36.521 36.521	1.00	4.000	1206.956	0.174	1.000	1.852 -	- -	-	- -	-
50	0.099	36.521 36.521	1.00	4.000	99966.789	0.019	1.000	0.099 -	- -	-	- -	-

Table of values		
Fe	66.620	ksi
lambda, c	0.87	
Fn	36.521	ksi
Ae	4.264	inch^2
Pn	155.72	kip
Resistance factor	0.85	
Unity check	0.01	-

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.574	0.136 0.136	1.00	0.430	1414.841	0.010	1.000	0.574 -	- -	-	- -	-
2	1.898	0.136 0.136	1.00	4.000	1148.984	0.011	1.000	1.898 -	- -	-	- -	-
3	8.194	0.136 0.136	1.00	4.000	16.155	0.092	1.000	8.194 -	- -	-	- -	-
4	1.898	0.136 0.136	1.00	4.000	1148.984	0.011	1.000	1.898 -	- -	-	- -	-
5	0.574	0.136 0.136	1.00	0.430	1414.841	0.010	1.000	0.574 -	- -	-	- -	-
6	1.898	0.136 0.136	1.00	4.000	1148.984	0.011	1.000	1.898 -	- -	-	- -	-
7	8.194	0.136 0.136	1.00	4.000	16.155	0.092	1.000	8.194 -	- -	-	- -	-
8	1.898	0.136 0.136	1.00	4.000	1148.984	0.011	1.000	1.898 -	- -	-	- -	-
9	0.099	0.136 0.136	1.00	4.000	99966.789	0.001	1.000	0.099 -	- -	-	- -	-
10	0.099	0.136 0.136	1.00	4.000	99966.789	0.001	1.000	0.099 -	- -	-	- -	-
11	0.099	0.136 0.136	1.00	4.000	99966.789	0.001	1.000	0.099 -	- -	-	- -	-
12	0.099	0.136 0.136	1.00	4.000	99966.789	0.001	1.000	0.099 -	- -	-	- -	-
13	0.099	0.136 0.136	1.00	4.000	99966.789	0.001	1.000	0.099 -	- -	-	- -	-
14	0.099	0.136 0.136	1.00	4.000	99966.789	0.001	1.000	0.099 -	- -	-	- -	-
17	0.102	0.136 0.136	1.00	4.000	95400.265	0.001	1.000	0.102 -	- -	-	- -	-
22	1.852	0.136 0.136	1.00	4.000	1206.956	0.011	1.000	1.852 -	- -	-	- -	-
26	1.852	0.136 0.136	1.00	4.000	1206.956	0.011	1.000	1.852 -	- -	-	- -	-
30	0.099	0.136 0.136	1.00	4.000	99966.789	0.001	1.000	0.099 -	- -	-	- -	-
37	0.102	0.136 0.136	1.00	4.000	95400.265	0.001	1.000	0.102 -	- -	-	- -	-
42	1.852	0.136 0.136	1.00	4.000	1206.956	0.011	1.000	1.852 -	- -	-	- -	-
46	1.852	0.136 0.136	1.00	4.000	1206.956	0.011	1.000	1.852 -	- -	-	- -	-
50	0.099	0.136 0.136	1.00	4.000	99966.789	0.001	1.000	0.099 -	- -	-	- -	-

Table of values		
Mnx	73.86	kipft
Mny	23.48	kipft
Pn	155.72	kip
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.01+0.63+0.17 = 0.80 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B150 CHECK (COLUMN)

AISI S100-16 LRFD Check

Member B150	362S162-54	A1008 grade 54	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 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	-3371.63	lbf
Vux	-911.64	lbf
Vuy	0.52	lbf
Mut	-3.03	lbfft
Mux	0.00	lbfft
Muy	1542.87	lbfft

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

Table of values		
Critical fibre	39	
Sigma Mx	0.000	ksi
Sigma My	-26.946	ksi
f bending	-26.946	ksi
Tau t	-0.040	ksi
f torsion	-0.040	ksi
Composed Stress	26.946	ksi
R	1.00	-

...::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	0.472	4.543 4.543	1.00	0.430	649.258	0.084	1.000	0.472 -	- -	-	- -	-
2	1.568	3.737 -40.915	10.95	3439.589	117439.386	0.006	1.000	- 1.568	0.112 0.784	-	- -	-
3	3.568	-40.915 -40.915	-	-	-	-	-	- -	- -	-	- -	-
4	1.568	3.737 -40.915	10.95	3439.589	117439.386	0.006	1.000	- 1.568	0.112 0.784	-	- -	-
5	0.472	4.543 4.543	1.00	0.430	649.258	0.084	1.000	0.472 -	- -	-	- -	-
6	1.568	50.000 5.348	0.11	7.210	246.190	0.451	1.000	- 1.568	0.542 1.026	-	- -	-
7	3.568	50.000 50.000	1.00	4.000	26.384	1.377	0.610	2.178 -	- -	-	- -	-
8	1.568	50.000 5.348	0.11	7.210	246.190	0.451	1.000	- 1.568	0.542 1.026	-	- -	-

Table of values		
Sye	0.626	$inch^3$
Mnyo	2.61	kipft
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).

Table of values		
Sigma,ex	698.464	ksi
Kt	1.00	
Lt	2.427	ft
Sigma,t	1340.756	ksi
Cb	1.00	
Sfy	0.805	$inch^3$
Fcre	1928.842	ksi

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

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

Element ID	Aw [$inch^2$]	Vn [kip]
1	0.000	0.00
2	0.089	2.66
3	0.000	0.00
4	0.089	2.66
5	0.000	0.00
6	0.089	2.66
7	0.000	0.00
8	0.089	2.66

Table of values		
Vn,x	10.65	kip
Resistance factor	0.95	
Unity check	0.09	-

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

Table of values		
Mnyo	2.61	kipft
Vnx	10.65	kip
Resistance factor shear	0.95	
Resistance factor bending y	0.90	

Unity check (My, Vx) = sqrt(0.43+0.01) = 0.66

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

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	50.000 50.000	1.00	0.430	649.258	0.278	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	3.568	50.000 50.000	1.00	4.000	26.384	1.377	0.610	2.178 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 50.000	1.00	0.430	649.258	0.278	1.000	0.472 -	- -	-	- -	-
6	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
7	3.568	50.000 50.000	1.00	4.000	26.384	1.377	0.610	2.178 -	- -	-	- -	-
8	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.709	inch^2
Pno	35.44	kip
Resistance factor	0.85	
Unity check	0.11	-

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	2 1/2	2 1/2	ft
Effective Length factor K	1.00	1.00	
Effective Length	2 1/2	2 1/2	ft
Slenderness	20.25	23.38	
Flexural Buckling stress Fcre	698.464	523.719	ksi

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

Table of values		
Sigma,ex	698.464	ksi
Sigma,ey	523.719	ksi
Kt	1.00	
Lt	2 1/2	ft
Sigma,t	1340.756	ksi
Sigma,TF	523.719	ksi
Torsional (-Flexural) buckling stress Fcre	523.719	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	48.041 48.041	1.00	0.430	649.258	0.272	1.000	0.472 -	- -	-	- -	-
2	1.568	48.041 48.041	1.00	4.000	136.574	0.593	1.000	1.568 -	- -	-	- -	-
3	3.568	48.041 48.041	1.00	4.000	26.384	1.349	0.620	2.213 -	- -	-	- -	-
4	1.568	48.041 48.041	1.00	4.000	136.574	0.593	1.000	1.568 -	- -	-	- -	-
5	0.472	48.041 48.041	1.00	0.430	649.258	0.272	1.000	0.472 -	- -	-	- -	-
6	1.568	48.041 48.041	1.00	4.000	136.574	0.593	1.000	1.568 -	- -	-	- -	-
7	3.568	48.041 48.041	1.00	4.000	26.384	1.349	0.620	2.213 -	- -	-	- -	-
8	1.568	48.041 48.041	1.00	4.000	136.574	0.593	1.000	1.568 -	- -	-	- -	-

Table of values		
Fe	523.719	ksi
lambda, c	0.31	
Fn	48.041	ksi
Ae	0.713	inch²
Pn	34.25	kip
Resistance factor	0.85	
Unity check	0.12	-

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

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	3.998 3.998	1.00	0.430	649.258	0.078	1.000	0.472 -	- -	-	- -	-
2	1.568	3.998 3.998	1.00	4.000	136.574	0.171	1.000	1.568 -	- -	-	- -	-
3	3.568	3.998 3.998	1.00	4.000	26.384	0.389	1.000	3.568 -	- -	-	- -	-
4	1.568	3.998 3.998	1.00	4.000	136.574	0.171	1.000	1.568 -	- -	-	- -	-
5	0.472	3.998 3.998	1.00	0.430	649.258	0.078	1.000	0.472 -	- -	-	- -	-
6	1.568	3.998 3.998	1.00	4.000	136.574	0.171	1.000	1.568 -	- -	-	- -	-
7	3.568	3.998 3.998	1.00	4.000	26.384	0.389	1.000	3.568 -	- -	-	- -	-
8	1.568	3.998 3.998	1.00	4.000	136.574	0.171	1.000	1.568 -	- -	-	- -	-

Table of values		
Mny	2.61	kipft
Pn	34.25	kip
Resistance factor compression	0.85	
Resistance factor bending y	0.90	

Unity check = 0.12+0.00+0.66 = 0.77 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B3552 CHECK (BEAM)

Member B3552	600S162-54	A1008 grade 54	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 8.00 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	-46.05	lbf
Vux	-0.39	lbf
Vuy	0.00	lbf
Mut	-5.20	lbfft
Mux	2449.43	lbfft
Muy	-6.51	lbfft

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

Table of values		
Critical fibre	5	
Sigma Mx	-15.421	ksi
Sigma My	-0.083	ksi
f bending	-15.505	ksi
Tau t	-0.037	ksi
f torsion	-0.037	ksi
Composed Stress	15.505	ksi
R	1.00	-

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	50.000 42.063	0.84	0.489	738.802	0.260	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	5.943	50.000 -50.000	1.00	24.000	57.064	0.936	0.817	- 4.857	1.214 2.429	-	- -	-
4	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
5	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
6	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	5.943	50.000 -50.000	1.00	24.000	57.064	0.936	0.817	- 4.857	1.214 2.429	-	- -	-
8	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	- 1.568	0.784 0.784	-	- -	-

Table of values		
Sxe	1.906	inch^3
Mnxo	7.94	kipft
Resistance factor	0.90	
Unity check	0.34	-

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

Table of values		
Lltb	16.000	ft
Sigma,ey	13.921	ksi
Kt	1.00	
Lt	16.000	ft
Sigma,t	672.999	ksi
Cb	1.14	

Table of values		
Sfx	1.906	inch^3
Fcre	169.937	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	39.925	ksi
Kt	1.00	
Lt	16.000	ft
Sigma,t	672.999	ksi
Cb	1.00	
Sfy	1.227	inch^3
Fcre	391.291	ksi

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

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	16	16	ft
Effective Length factor K	1.00	1.00	
Effective Length	16	16	ft
Slenderness	84.68	143.41	
Flexural Buckling stress Fcre	39.925	13.921	ksi

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

Table of values		
Sigma,ex	39.925	ksi
Sigma,ey	13.921	ksi
Kt	1.00	
Lt	16	ft
Sigma,t	672.999	ksi
Sigma,TF	13.921	ksi
Torsional (-Flexural) buckling stress Fcre	13.921	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	12.209 12.209	1.00	0.430	649.258	0.137	1.000	0.472 -	- -	-	- -	-
2	1.568	12.209 12.209	1.00	4.000	136.574	0.299	1.000	1.568 -	- -	-	- -	-
3	5.943	12.209 12.209	1.00	4.000	9.511	1.133	0.711	4.227 -	- -	-	- -	-
4	1.568	12.209 12.209	1.00	4.000	136.574	0.299	1.000	1.568 -	- -	-	- -	-
5	0.472	12.209 12.209	1.00	0.430	649.258	0.137	1.000	0.472 -	- -	-	- -	-
6	1.568	12.209 12.209	1.00	4.000	136.574	0.299	1.000	1.568 -	- -	-	- -	-
7	5.943	12.209 12.209	1.00	4.000	9.511	1.133	0.711	4.227 -	- -	-	- -	-
8	1.568	12.209 12.209	1.00	4.000	136.574	0.299	1.000	1.568 -	- -	-	- -	-

Table of values		
Fe	13.921	ksi
lambda, c	1.90	
Fn	12.209	ksi
Ae	0.941	inch^2
Pn	11.49	kip
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)

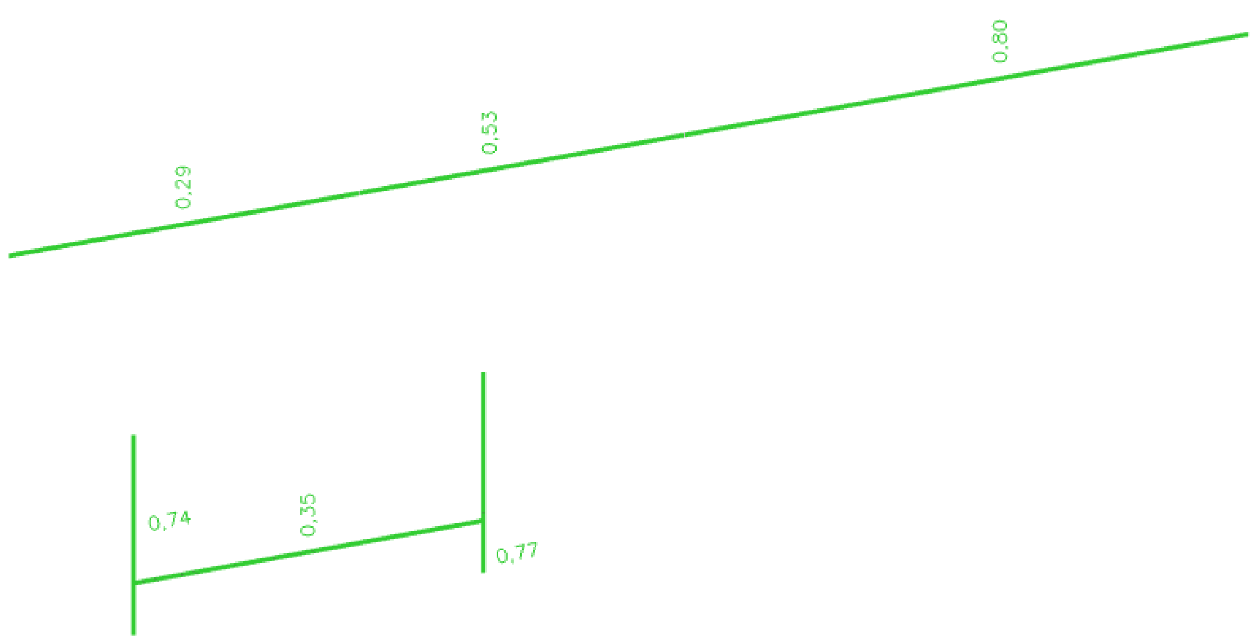
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	0.041 0.041	1.00	0.430	649.258	0.008	1.000	0.472 -	- -	-	- -	-
2	1.568	0.041 0.041	1.00	4.000	136.574	0.017	1.000	1.568 -	- -	-	- -	-
3	5.943	0.041 0.041	1.00	4.000	9.511	0.066	1.000	5.943 -	- -	-	- -	-
4	1.568	0.041 0.041	1.00	4.000	136.574	0.017	1.000	1.568 -	- -	-	- -	-
5	0.472	0.041 0.041	1.00	0.430	649.258	0.008	1.000	0.472 -	- -	-	- -	-
6	1.568	0.041 0.041	1.00	4.000	136.574	0.017	1.000	1.568 -	- -	-	- -	-
7	5.943	0.041 0.041	1.00	4.000	9.511	0.066	1.000	5.943 -	- -	-	- -	-
8	1.568	0.041 0.041	1.00	4.000	136.574	0.017	1.000	1.568 -	- -	-	- -	-

Table of values		
Mnx	7.94	kipft
Mny	2.93	kipft
Pn	11.49	kip
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.00+0.34+0.00 = 0.35 - (C5.2.1-3)

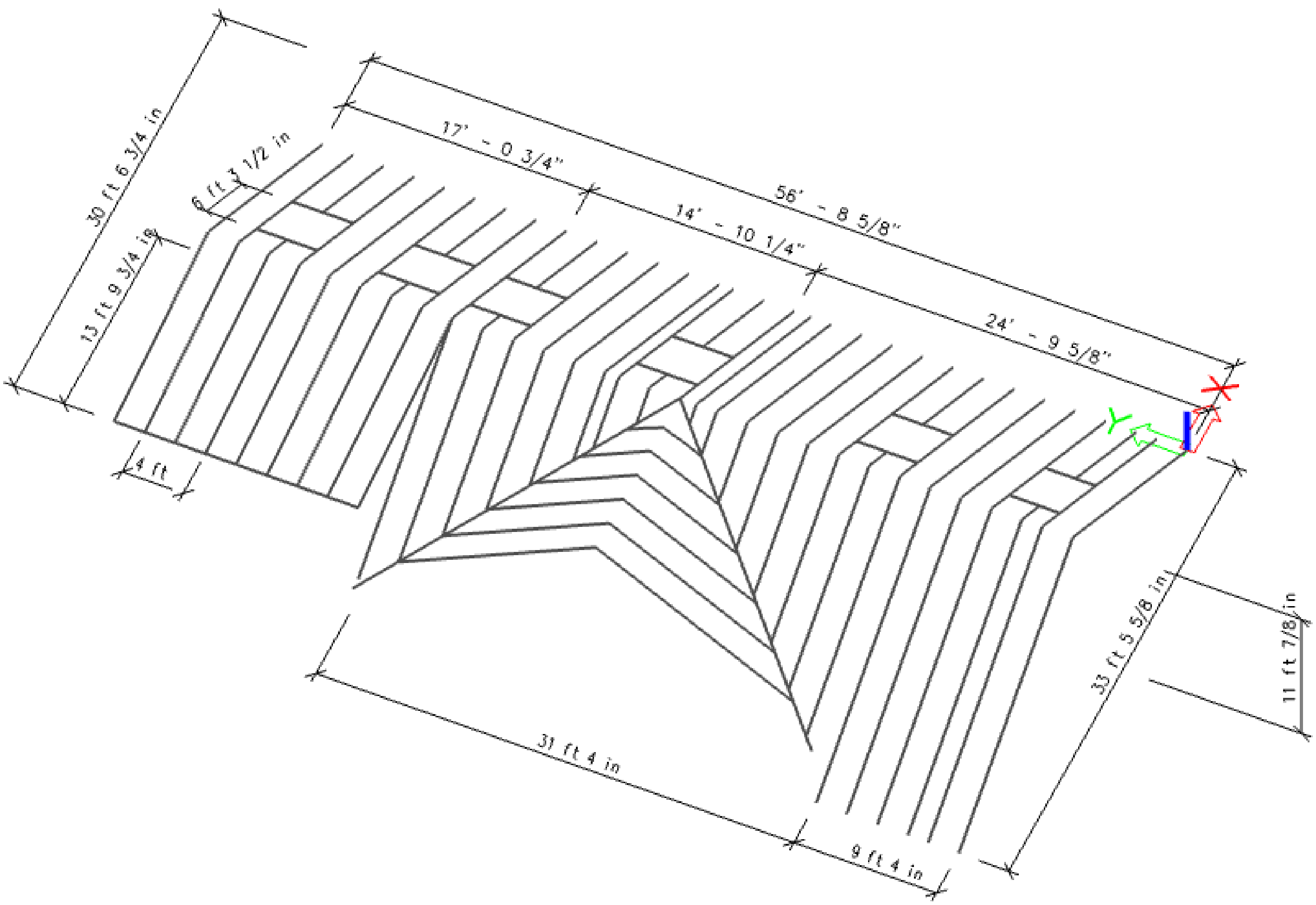
The member satisfies the check !

Unity check

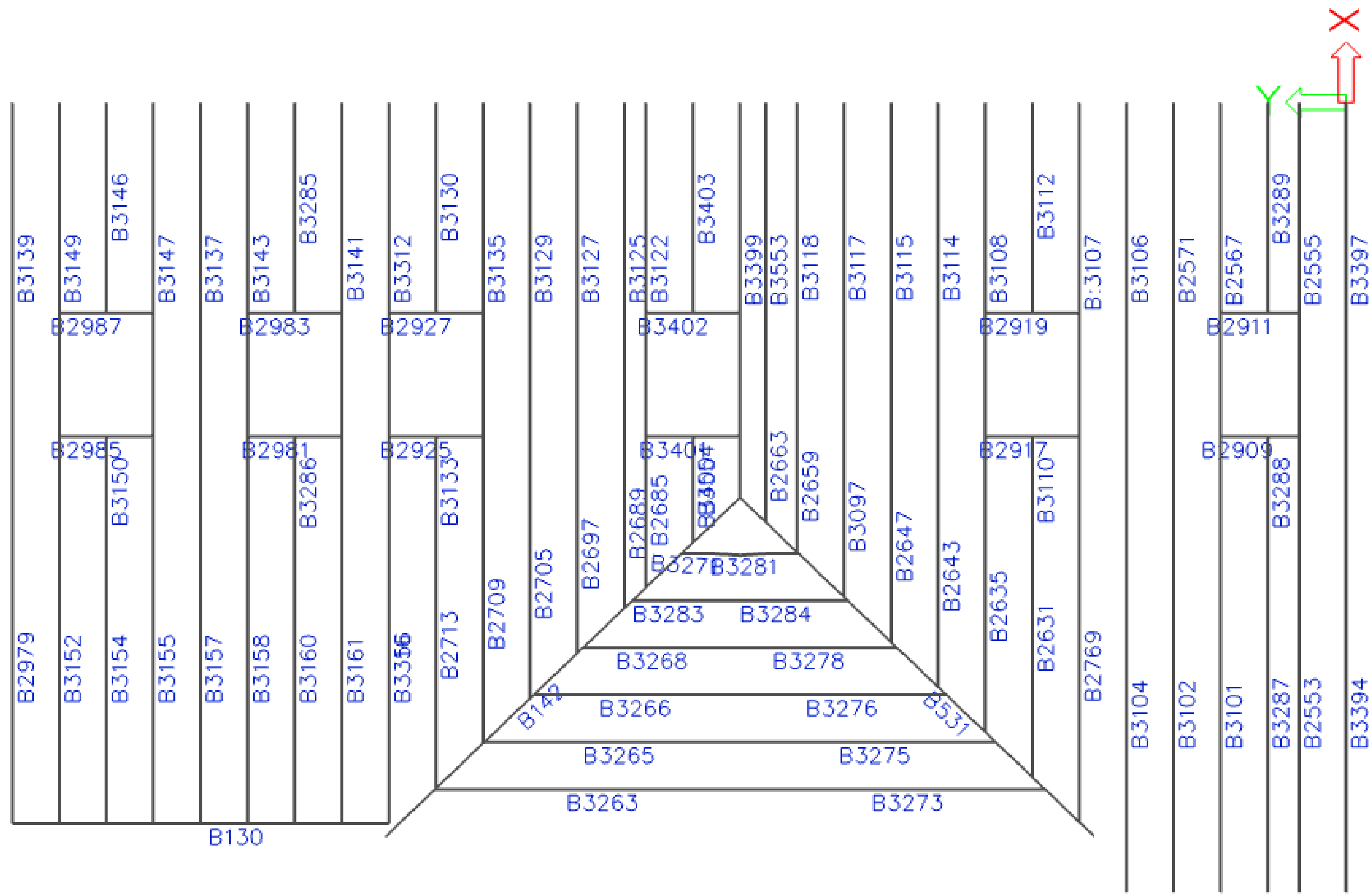


BLOCK B. ROOF RAFTER STRUCTURAL ANALYSIS

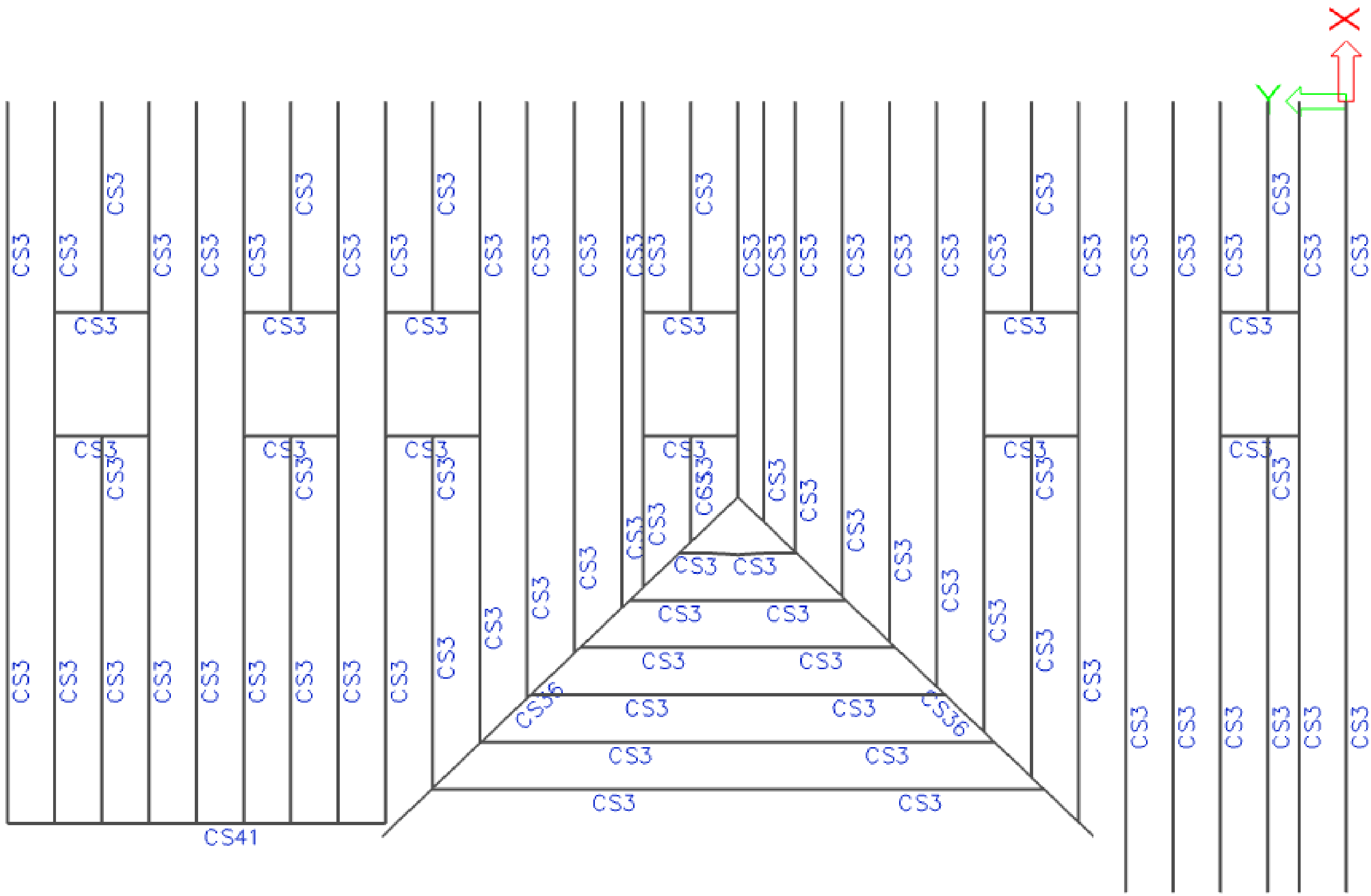
General scheme



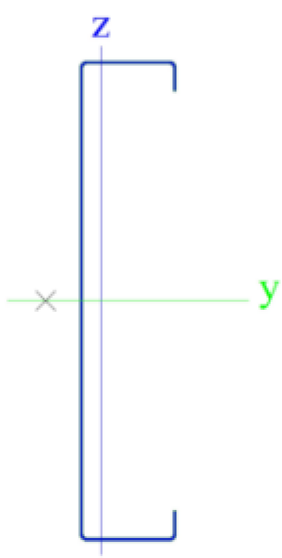
Member numbers



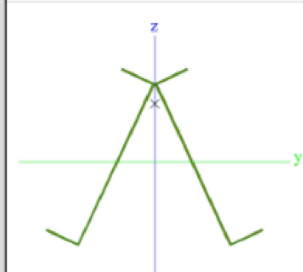
Cross-sections of



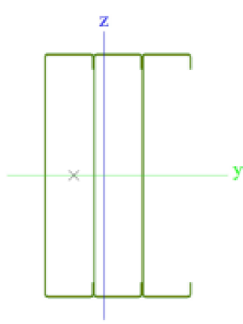
CS3 Cross-sections properties

CS3		
Type	S1000S200-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.842	
A_y [inch^2], A_z [inch^2]	0.229	0.566
A_L [inch^2/inch], A_D [inch^2/inch]	2.98e+01	2.98e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.427	5.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	11.350	0.378
i_y [inch], i_z [inch]	3.671	0.670
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	2.255	0.240
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.753	0.340
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.38e+02	1.38e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.70e+01	1.70e+01
d_y [inch], d_z [inch]	-1.143	0.000
I_t [inch^4], I_w [inch^6]	0.001	7.665
β_y [inch], β_z [inch]	0.000	12.209
Picture		

CS36 Cross-sections properties

CS36		
Type	1000T200-97	
Shape type	Thick-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	2,678	
A_y [inch^2], A_z [inch^2]	1,849	0,673
A_L [inch^2/inch], A_D [inch^2/inch]	5,56e+01	5,56e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-2,242	-0,075
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	28,285	22,325
i_y [inch], i_z [inch]	3,250	2,887
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	5,398	3,694
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	7,709	6,460
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3,85e+02	3,85e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3,23e+02	3,23e+02
d_y [inch], d_z [inch]	0,000	3,272
I_t [inch^4], I_w [inch^6]	0,008	26,529
β_y [inch], β_z [inch]	-8,592	0,000
Picture		

CS41 Cross-sections properties

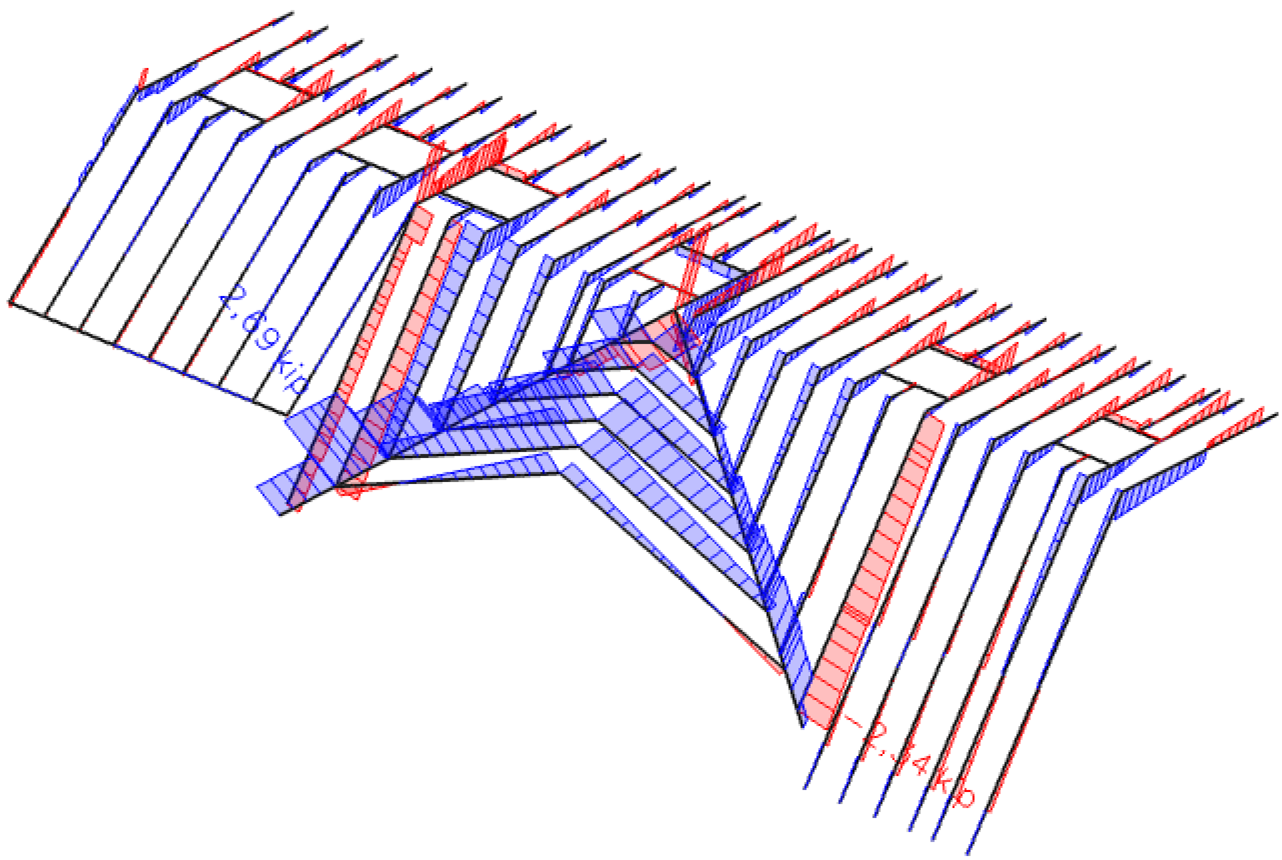
CS41		
Type	1000S200-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	2,517	
A_y [inch^2], A_z [inch^2]	0,652	1,699
A_L [inch^2/inch], A_D [inch^2/inch]	3,84e+01	8,54e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	2,003	0,000
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	33,821	7,846
i_y [inch], i_z [inch]	3,665	1,765
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	6,764	2,196
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	8,258	3,697
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4,13e+02	4,13e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1,85e+02	1,85e+02
d_y [inch], d_z [inch]	-1,233	0,000
I_t [inch^4], I_w [inch^6]	1,135	55,372
$β_y$ [inch], $β_z$ [inch]	0,000	3,900
Picture		

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

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

Maximum force diagram

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



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

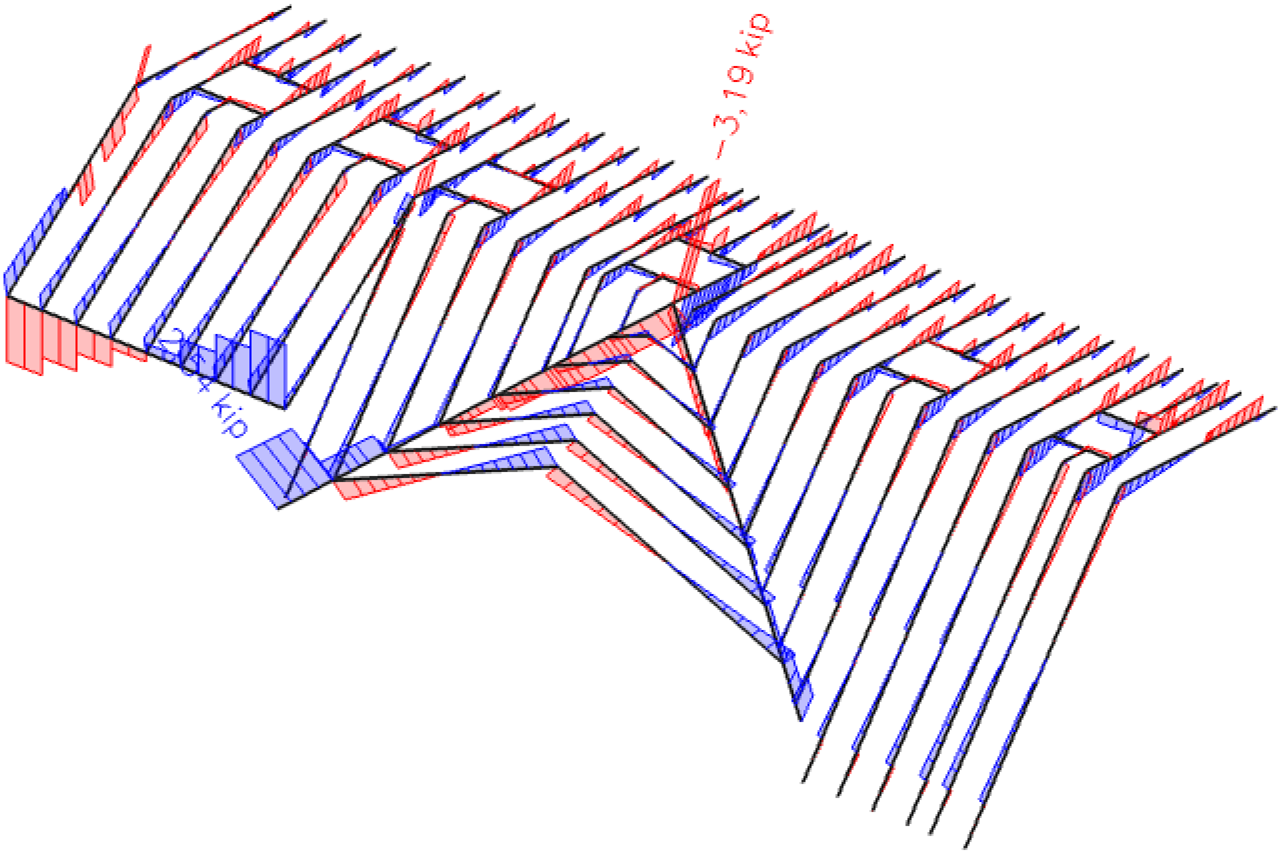
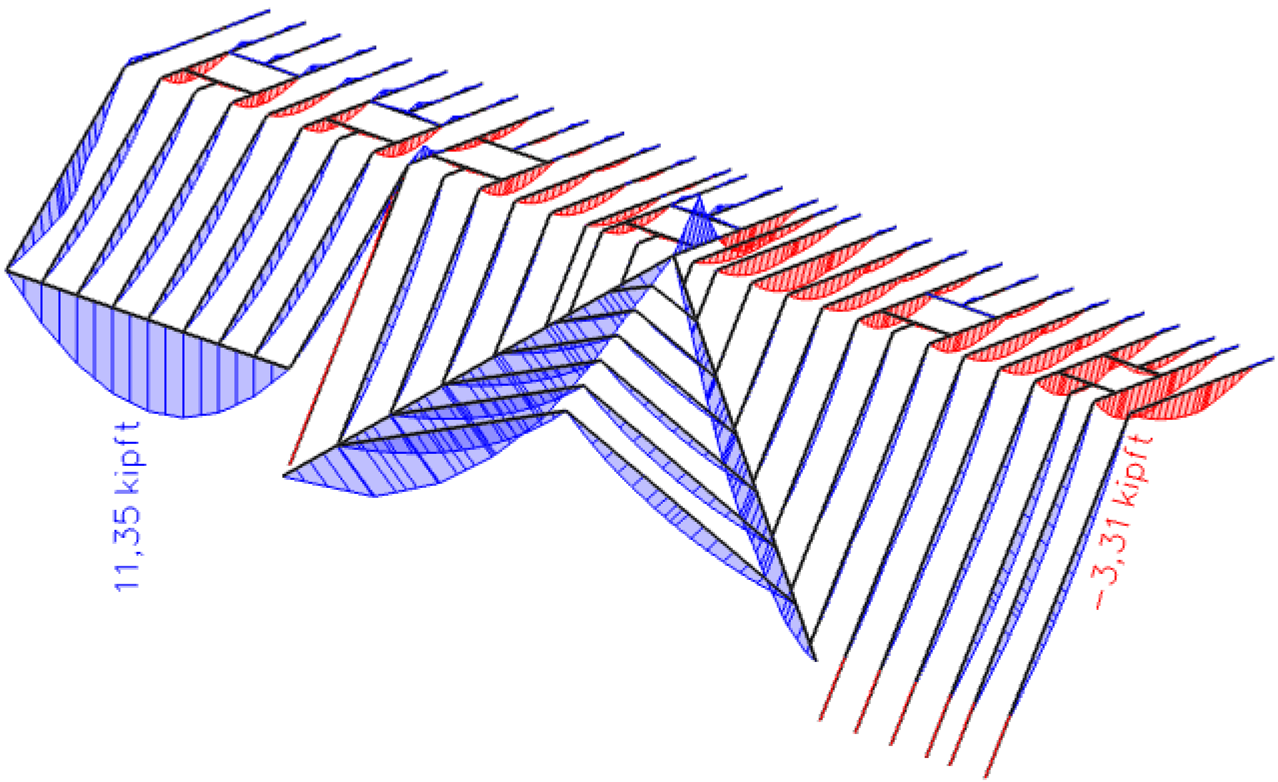
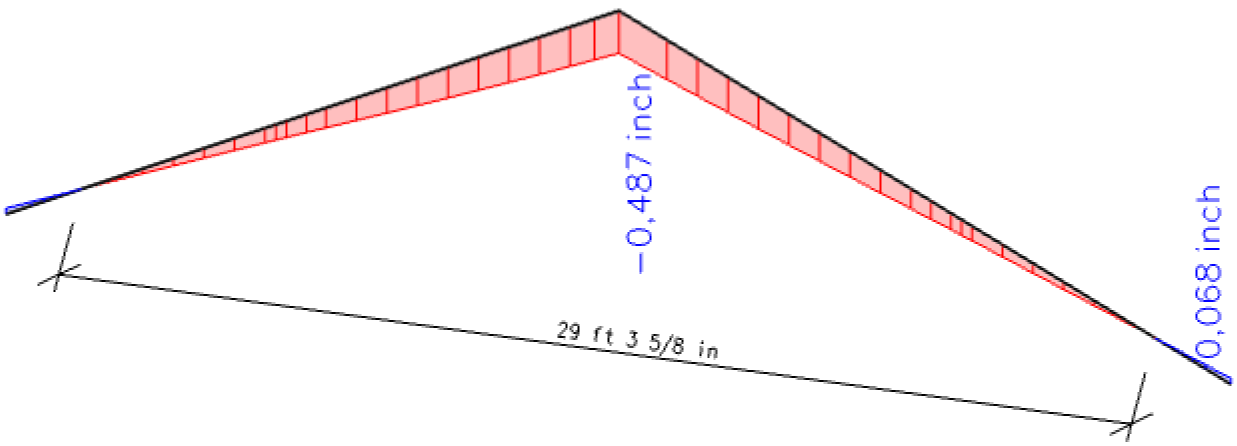


Diagram of moment My,
LRFD-Ult (auto)8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*L), lbf.



Displacement of elements
Value: Uz - Lr, (inch) .



The maximum deflection is 0.487" according to table 1604.3 the code IBC 2021 - the deflection limits L/360. L = 29'-3 5/8"= 29'*12"+8.5"=351 5/8". 351 5/8"/360 =0.976"
0.487" < 0.976" <u>Deflection is OK!</u>

STEEL MEMBER B3101 CHECK (RAFTER)

AISI S100-16 LRFD Check

Member B3101	S1000S200-54	A1008 grade 54	LRFD-Ult (auto)	0.94

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

The critical check is on position 9.18 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	0.11	kip
Vux	0.03	kip
Vuy	-0.07	kip
Mut	0.00	kipft
Mux	3.51	kipft
Muy	-0.30	kipft

...::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	0.484	-32.657 -36.932	-	-	-	-	-	- -	- -	-	- -	-
3	1.717	-37.933 -37.933	-	-	-	-	-	- -	- -	-	- -	-
5	9.717	48.999 -36.932	0.75	18.295	16.274	1.735	0.503	- 4.890	1.303 1.486	-	- -	-
7	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
9	0.484	48.999 44.723	0.91	0.461	165.766	0.544	1.000	0.223 -	- -	-	- -	-

Table of values		
Sxe	1.690	inch ^{3}
Mnxo	7.04	kipft
Resistance factor	0.90	
Unity check	0.55	-

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

Table of values		
Lltb	1.321	ft
Sigma,ey	510.717	ksi
Kt	1.00	
Lt	1.321	ft
Sigma,t	681.123	ksi
Cb	1.01	
Sfx	2.270	$inch^3$
Fcre	866.163	ksi

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

Distortional Buckling Strength

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

Table of values		
Sfy	2.270	$inch^3$
My	9.46	kipft
L	1.321	ft
Beta	1.04	
k,phi,fe	0.44	kip
k,phi,we	0.21	kip
k,phi	0.00	kip
k,phi,fg	0.011	$inch^2$
k,phi,wg	0.006	$inch^2$
Fd	40.282	ksi

Table of values		
Sf	2.270	$inch^3$
Mcrd	7.62	kipft
Lambda,d	1.11	
Mn	6.81	kipft
Resistance factor	0.90	
Unity check	0.57	-

Data		
Lm	1.321	ft
Lcr	1.678	ft
h0	10.000	inch
Ixf	0.003	$inch^4$
Iyf	0.049	$inch^4$
Ixyf	-0.007	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.691	inch
hxf	-1.239	inch
Af	0.135	$inch^2$
y0f	0.068	inch
Ksi,web	2.00	

Number of compressed flanges: 1

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

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

Nominal Flexural Strength

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.484	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	1.717	24.451 -45.395	1.86	56.331	1604.838	0.123	1.000	- 1.717	0.354 0.248	-	- -	-
5	9.717	29.056 29.056	1.00	4.000	3.558	2.858	0.323	3.139 -	- -	-	- -	-
7	1.717	24.451 -45.395	1.86	56.331	1604.838	0.123	1.000	- 1.717	0.354 0.248	-	- -	-
9	0.484	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.216	$inch^3$
Mnyo	0.90	kipft
Resistance factor	0.90	
Unity check	0.37	-

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

Table of values		
Sigma,ex	225.752	ksi
Kt	1.00	
Lt	1.321	ft
Sigma,t	681.123	ksi
Cs	1.00	
CTF	0.95	
Sfy	0.886	$inch^3$
j	6.363	inch
Fcre	3532.750	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 [kip]
3	0.000	0.00
5	0.550	2.61
7	0.000	0.00

Table of values		
Vn,y	2.61	kip
Resistance factor	0.95	
Unity check	0.03	-

Table of values		
Mnxo	7.04	kipft
Vny	2.61	kip
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.31+0.00) = 0.55

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

Table of values		
Sftx	2.270	$inch^3$
Sfty	0.240	$inch^3$
Mnxt	9.46	kipft
Mnyt	1.00	kipft
Mnx	6.81	kipft
Mny	0.90	kipft
Tn	42.10	kip
Resistance factor tension	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.41+0.33+0.00 = 0.75 - (H1.1-1)
Unity check = 0.57+0.37-0.00 = 0.94 - (H1.1-2)

The member satisfies the check !

STEEL MEMBER B142 CHECK (RIM TRACK)

AISI S100-16 LRFD Check

Member B142	1000T200-97	A1008 grade 54	LRFD-Ult (auto)	0.88

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

The critical check is on position 9.56 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	870.56	lbf
Vux	48.77	lbf
Vuy	-340.78	lbf
Mut	0.14	lbfft
Mux	11268.47	lbfft
Muy	2063.52	lbfft

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

Table of values		
Tn	138503.26	lbf
Resistance factor	0.90	
Unity check	0.01	-

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	1.949	-30.444 -37.831	-	-	-	-	-	- -	- -	-	- -	-
2	9.898	42.613 -37.831	0.89	21.231	58.759	0.852	0.871	- 8.620	2.217 2.349	-	- -	-
3	1.949	50.000 42.613	0.85	0.442	31.538	1.259	0.655	1.278 -	- -	-	- -	-
4	1.949	50.000 42.613	0.85	0.442	31.538	1.259	0.655	1.278 -	- -	-	- -	-
5	9.898	42.613 -37.831	0.89	21.231	58.759	0.852	0.871	- 8.620	2.217 2.349	-	- -	-
6	1.949	-30.444 -37.831	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	4.473	$inch^3$
Mnxo	18638.41	lbfft
Resistance factor	0.90	
Unity check	0.67	-

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

Table of values		
Lltb	2.812	ft
Sigma,ey	1991.164	ksi
Kt	1.00	
Lt	2.812	ft
Sigma,t	88.256	ksi
Cb	1.04	
Sfx	5.511	$inch^3$
Fcre	1172.002	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	1.949	-27.737 -41.329	-	-	-	-	-	- -	- -	-	- -	-
2	9.898	4.450 -27.737	6.23	775.374	2145.952	0.046	1.000	- 9.898	1.072 0.296	-	- -	-
3	1.949	4.450 -9.142	2.05	1.297	92.568	0.219	1.000	1.949 -	- -	-	- -	-
4	1.949	17.813 4.221	0.24	0.524	37.412	0.690	0.987	1.924 -	- -	-	- -	-
5	9.898	36.408 4.221	0.12	7.150	19.789	1.356	0.618	- 6.114	2.120 3.994	-	- -	-
6	1.949	50.000 36.408	0.73	0.454	32.418	1.242	0.663	1.291 -	- -	-	- -	-

Table of values		
Sye	2.593	$inch^3$
Mnyo	10805.37	lbfft
Resistance factor	0.90	
Unity check	0.21	-

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

Table of values		
Sigma,ex	39.533	ksi
Kt	1.00	
Lt	2.812	ft
Sigma,t	88.256	ksi
Cb	1.00	
Sfy	3.682	$inch^3$
Fcre	238.559	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	0.035	1062.15
2	0.827	12222.90
3	0.035	1062.15
4	0.035	1062.15
5	0.827	12222.90
6	0.035	1062.15

Table of values		
Vn,y	28694.39	lbf
Resistance factor	0.95	
Unity check	0.01	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	18638.41	lbfft
Vny	28694.39	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.45+0.00) = 0.67

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

Combined Tensile Axial Load and Bending

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

Table of values		
Sftx	6.313	$inch^3$
Sfty	3.682	$inch^3$
Mnxt	26302.86	lbfft
Mnyt	15341.07	lbfft
Mnx	18638.41	lbfft
Mny	10805.37	lbfft
Tn	138503.26	lbf
Resistance factor tension	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.48+0.15+0.01 = 0.63 - (H1.1-1)

Unity check = 0.67+0.21-0.01 = 0.88 - (H1.1-2)

The member satisfies the check !

STEEL MEMBER B130 CHECK (ROOF BEAM)

AISI S100-16 LRFD Check

Member B130	1000S200-54	A1008 grade 54	LRFD-Ult (auto)	0.53

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

The critical check is on position 8.00 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	kip
Vux	-0.01	kip
Vuy	0.35	kip
Mut	0.00	kipft
Mux	11.35	kipft
Muy	0.13	kipft

...::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	0.597	50.000 44.459	0.89	4.224	3985.942	0.112	1.000	- 0.597	0.283 0.314	-	- -	-
2	1.943	50.000 50.000	1.00	4.000	88.952	0.750	0.942	1.831 -	- -	-	- -	-
3	9.943	50.000 -42.339	0.85	20.291	17.237	1.703	0.511	- 5.084	1.322 1.431	-	- -	-
4	1.943	-42.339 -42.339	-	-	-	-	-	- -	- -	-	- -	-
5	0.597	-36.798 -42.339	-	-	-	-	-	- -	- -	-	- -	-
6	0.597	50.000 44.459	0.89	4.224	3985.942	0.112	1.000	- 0.597	0.283 0.314	-	- -	-
7	1.943	50.000 50.000	1.00	4.000	88.952	0.750	0.942	1.831 -	- -	-	- -	-
8	8.750	44.459 -36.798	0.83	19.866	21.793	1.428	0.592	- 5.183	1.354 1.482	-	- -	-
9	1.943	-42.339 -42.339	-	-	-	-	-	- -	- -	-	- -	-
10	0.597	-36.798 -42.339	-	-	-	-	-	- -	- -	-	- -	-
11	0.597	50.000 44.459	0.89	0.470	110.923	0.671	1.000	0.597 -	- -	-	- -	-
12	1.943	50.000 50.000	1.00	4.000	88.952	0.750	0.942	1.831 -	- -	-	- -	-
13	8.750	44.459 -36.798	0.83	19.866	21.793	1.428	0.592	- 5.183	1.354 1.482	-	- -	-
14	1.943	-42.339 -42.339	-	-	-	-	-	- -	- -	-	- -	-
15	0.597	-36.798 -42.339	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	5.903	$inch^3$
Mnxo	24.60	kipft
Resistance factor	0.90	
Unity check	0.51	-

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

Table of values		
Lltb	2.000	ft
Sigma,ey	1549.173	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	883.328	ksi
Cb	1.03	
Sfx	6.764	$inch^3$
Fcre	1898.174	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	0.597	-2.884 -2.884	-	-	-	-	-	- -	- -	-	- -	-
2	1.943	-3.261 -29.137	-	-	-	-	-	- -	- -	-	- -	-
3	9.943	-29.137 -29.137	-	-	-	-	-	- -	- -	-	- -	-
4	1.943	-3.261 -29.137	-	-	-	-	-	- -	- -	-	- -	-
5	0.597	-2.884 -2.884	-	-	-	-	-	- -	- -	-	- -	-
6	0.597	23.746 23.746	1.00	4.000	3774.234	0.079	1.000	0.597 -	- -	-	- -	-
7	1.943	23.370 -2.507	0.11	8.930	198.580	0.343	1.000	- 1.943	0.625 1.130	-	- -	-
8	8.750	-2.507 -2.507	-	-	-	-	-	- -	- -	-	- -	-
9	1.943	23.370 -2.507	0.11	8.930	198.580	0.343	1.000	- 1.943	0.625 1.130	-	- -	-
10	0.597	23.746 23.746	1.00	4.000	3774.234	0.079	1.000	0.597 -	- -	-	- -	-
11	0.597	50.000 50.000	1.00	0.430	101.433	0.702	0.978	0.584 -	- -	-	- -	-
12	1.943	50.000 24.123	0.48	5.312	118.135	0.651	1.000	- 1.943	0.772 1.171	-	- -	-
13	8.750	24.123 24.123	1.00	4.000	4.388	2.345	0.386	3.382 -	- -	-	- -	-
14	1.943	50.000 24.123	0.48	5.312	118.135	0.651	1.000	- 1.943	0.772 1.171	-	- -	-
15	0.597	50.000 50.000	1.00	0.430	101.433	0.702	0.978	0.584 -	- -	-	- -	-

Table of values		
Sye	1.872	$inch^3$
Mnyo	7.80	kipft
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	104.338	ksi
Kt	1.00	
Lt	2.000	ft
Sigma,t	883.328	ksi
Cb	1.00	
Sfy	2.196	$inch^3$
Fcre	1479.720	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 [kip]
1	0.068	2.03
2	0.000	0.00
3	0.563	2.55
4	0.000	0.00
5	0.068	2.03
6	0.068	2.03
7	0.000	0.00

Element ID	Aw [$inch^2$]	Vn [kip]
8	0.495	2.90
9	0.000	0.00
10	0.068	2.03
11	0.034	1.01
12	0.000	0.00
13	0.495	2.90
14	0.000	0.00
15	0.034	1.01

Table of values		
Vn,y	18.49	kip
Resistance factor	0.95	
Unity check	0.02	-

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

Table of values		
Mnxo	24.60	kipft
Vny	18.49	kip
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.26+0.00) = 0.51

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

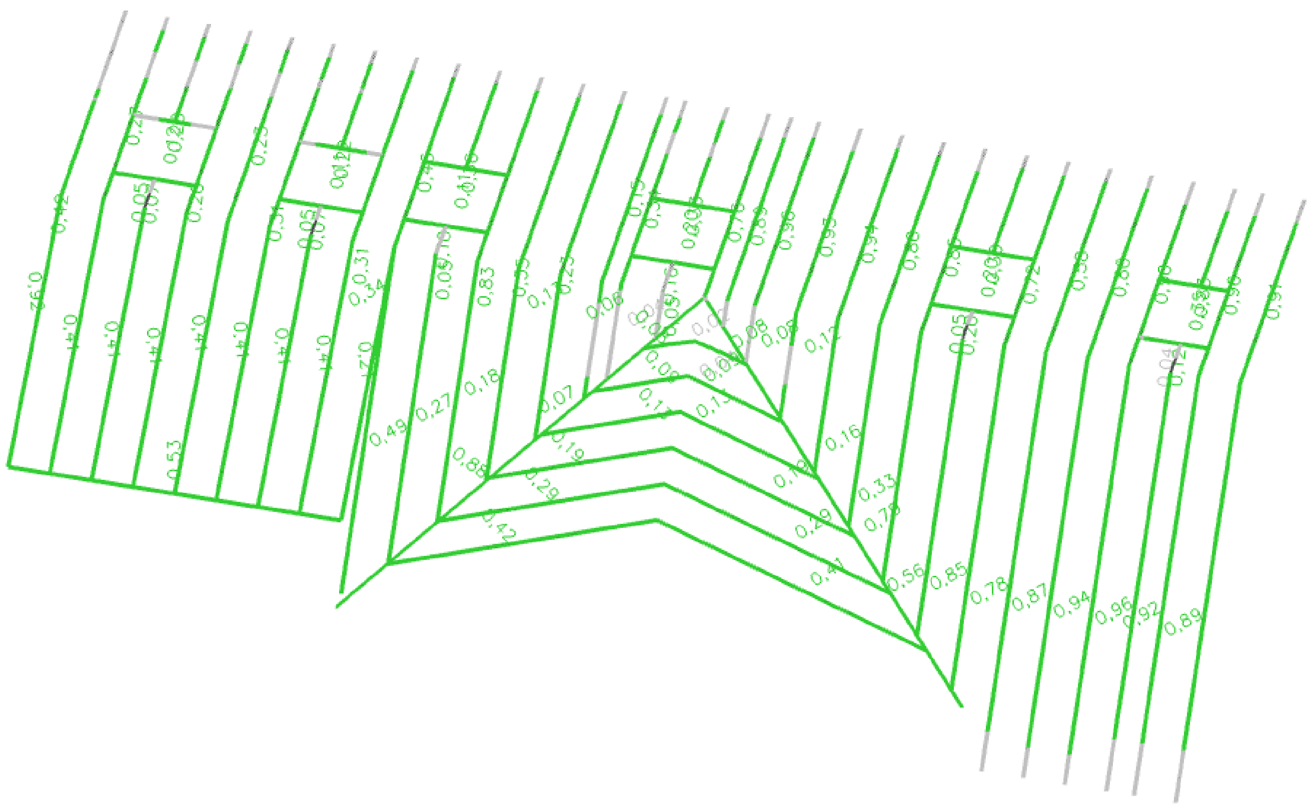
Combined Tensile Axial Load and Bending
According to article H1.1 and formulas (H1.1-1), (H1.1-2)

Table of values		
Sftx	6.764	inch³
Sfty	3.234	inch³
Mnxt	28.18	kipft
Mnyt	13.47	kipft
Mnx	24.60	kipft
Mny	7.80	kipft
Tn	125.87	kip
Resistance factor tension	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.45+0.01+0.00 = 0.46 - (H1.1-1)
Unity check = 0.51+0.02-0.00 = 0.53 - (H1.1-2)

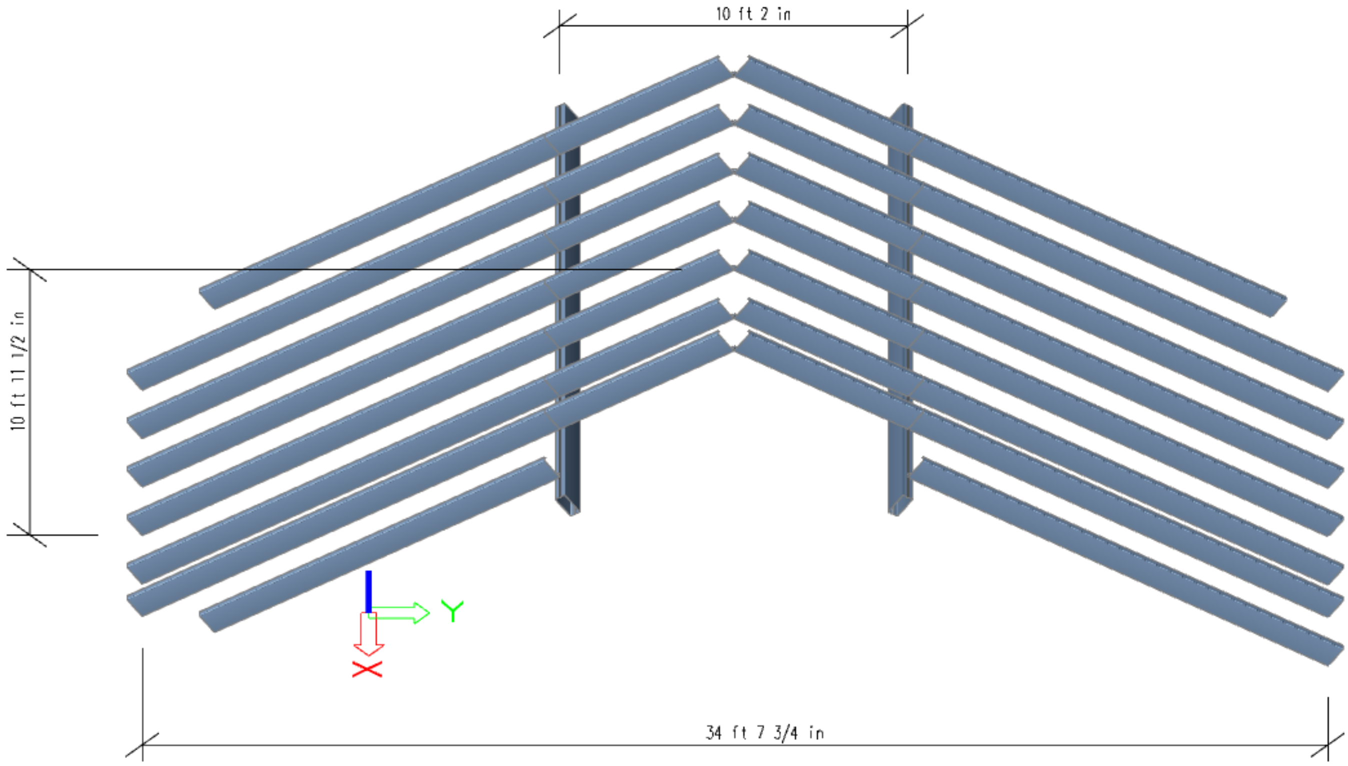
The member satisfies the check !

Unity check

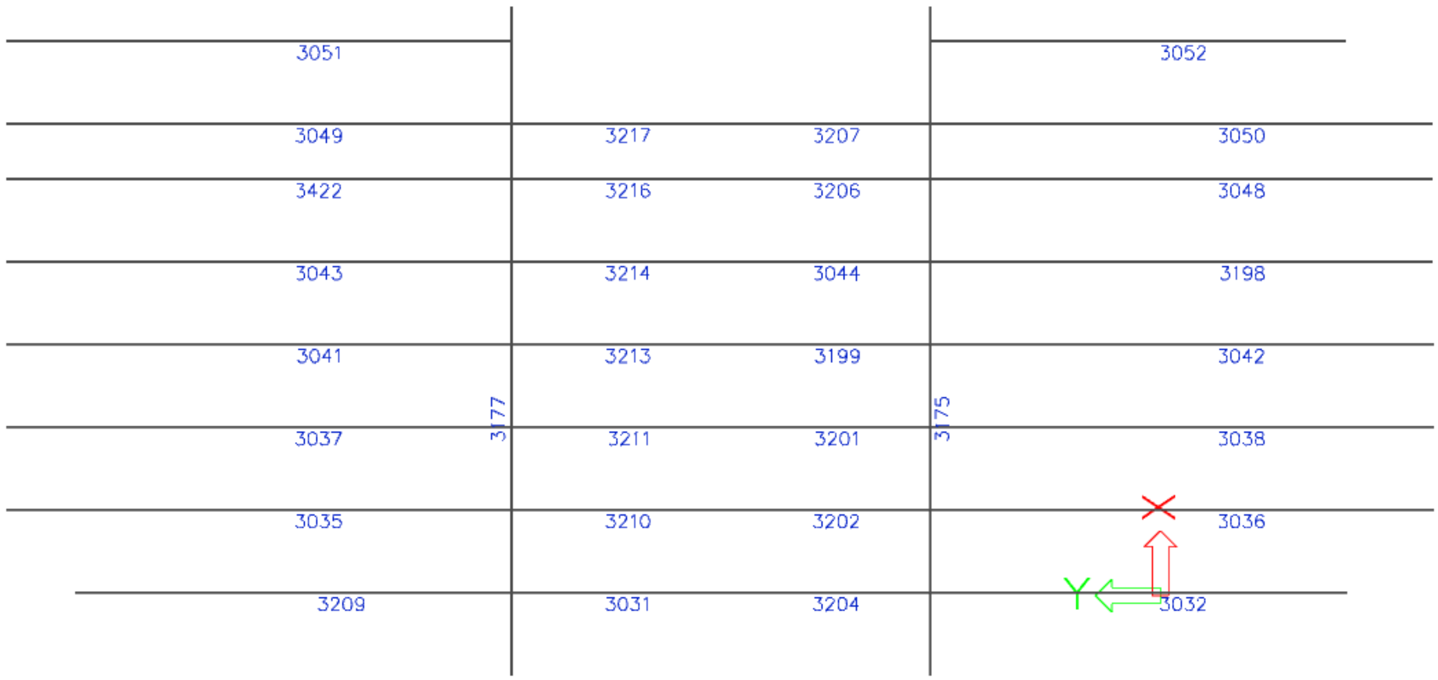


BLOCK C. BEAMS & RAFTERS STRUCTURAL ANALYSIS

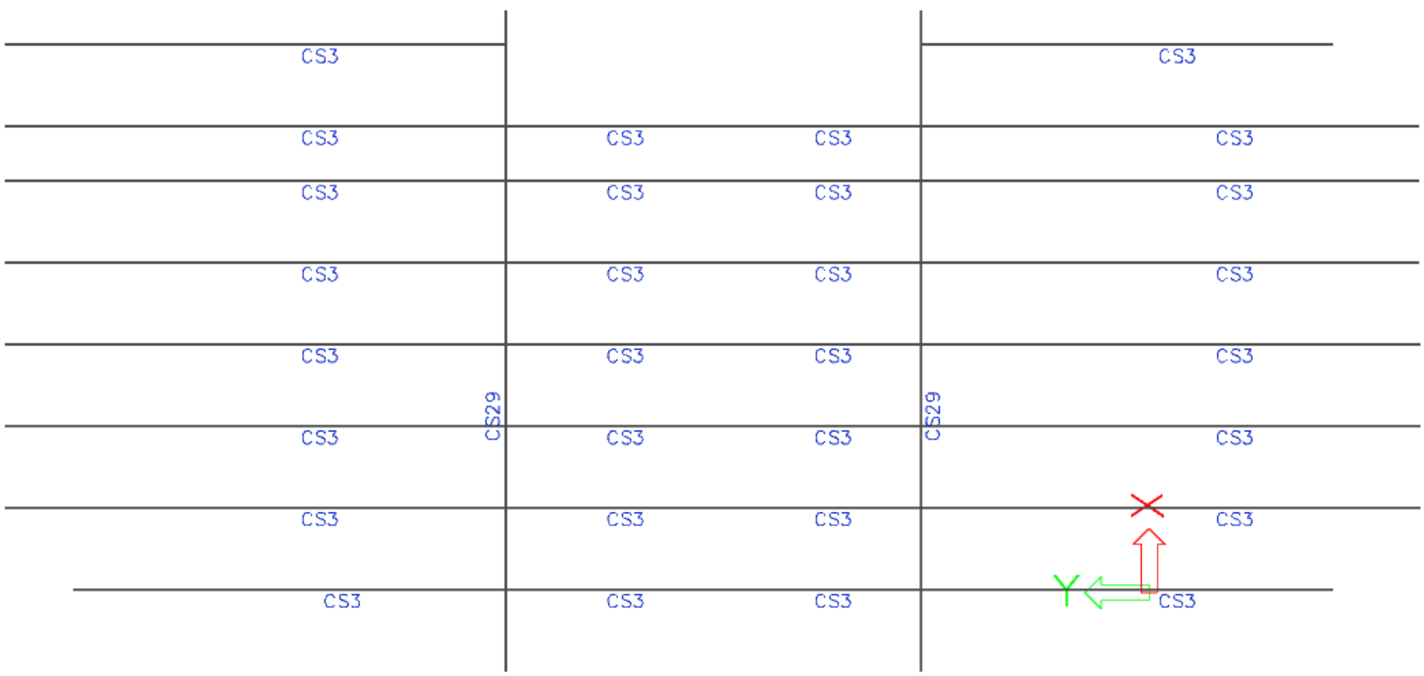
General scheme



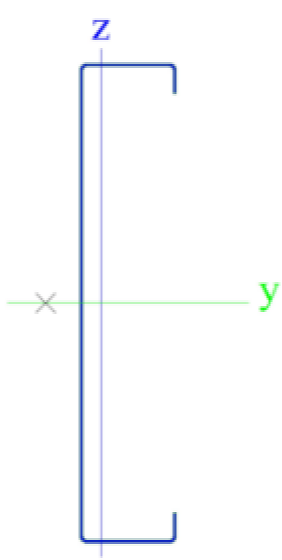
Member numbers



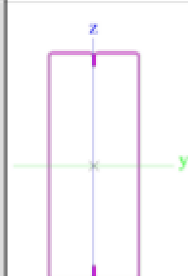
Cross-sections of members



CS31 Cross-sections properties

CS3		
Type	S1000S200-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.842	
A_y [inch^2], A_z [inch^2]	0.229	0.566
A_L [inch^2/inch], A_D [inch^2/inch]	2.98e+01	2.98e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.427	5.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	11.350	0.378
i_y [inch], i_z [inch]	3.671	0.670
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	2.255	0.240
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.753	0.340
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.38e+02	1.38e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.70e+01	1.70e+01
d_y [inch], d_z [inch]	-1.143	0.000
I_t [inch^4], I_w [inch^6]	0.001	7.665
$β_y$ [inch], $β_z$ [inch]	0.000	12.209
Picture		

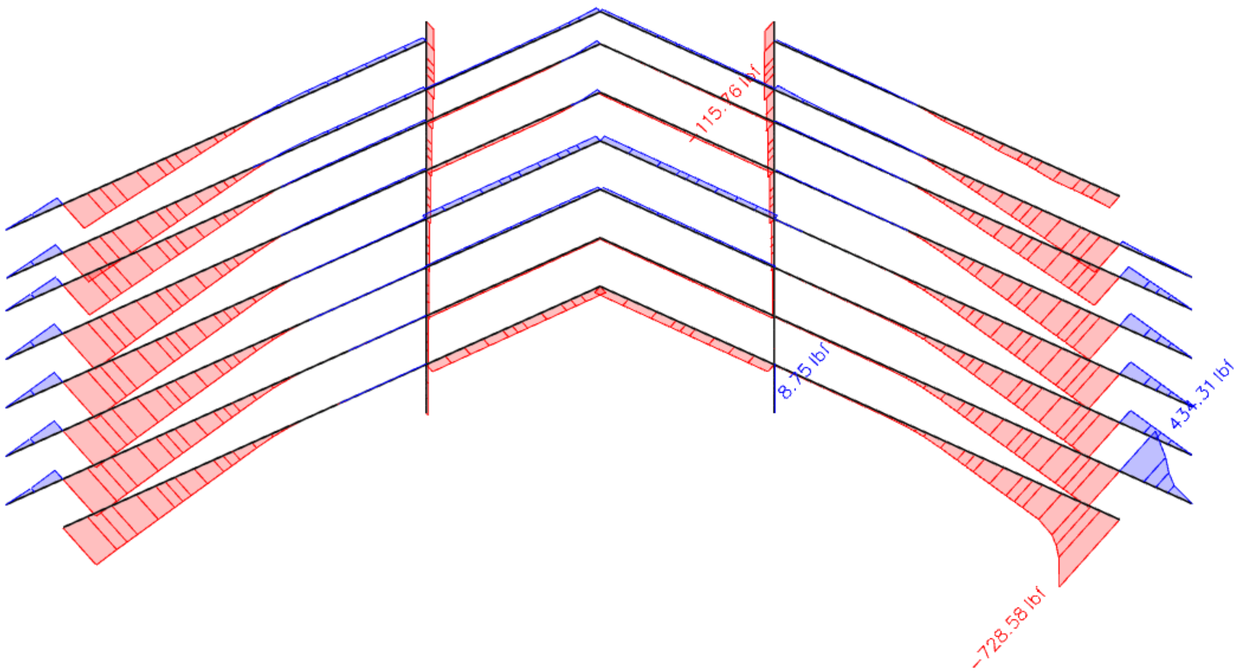
CS29 Cross-sections properties

CS29		
Type	Box1000S200-68	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	2.098	
A_y [inch^2], A_z [inch^2]	0.597	1.389
A_L [inch^2/inch], A_D [inch^2/inch]	2.81e+01	5.74e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1.577	0.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	27.976	6.115
i_y [inch], i_z [inch]	3.651	1.707
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	5.595	3.057
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	6.853	3.302
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3.43e+02	3.43e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.65e+02	1.65e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	14.862	10.941
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

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

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

Maximum force diagram
Axial force diagram N,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr), lbf.



Shear force diagram Vz ,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr), lbf.

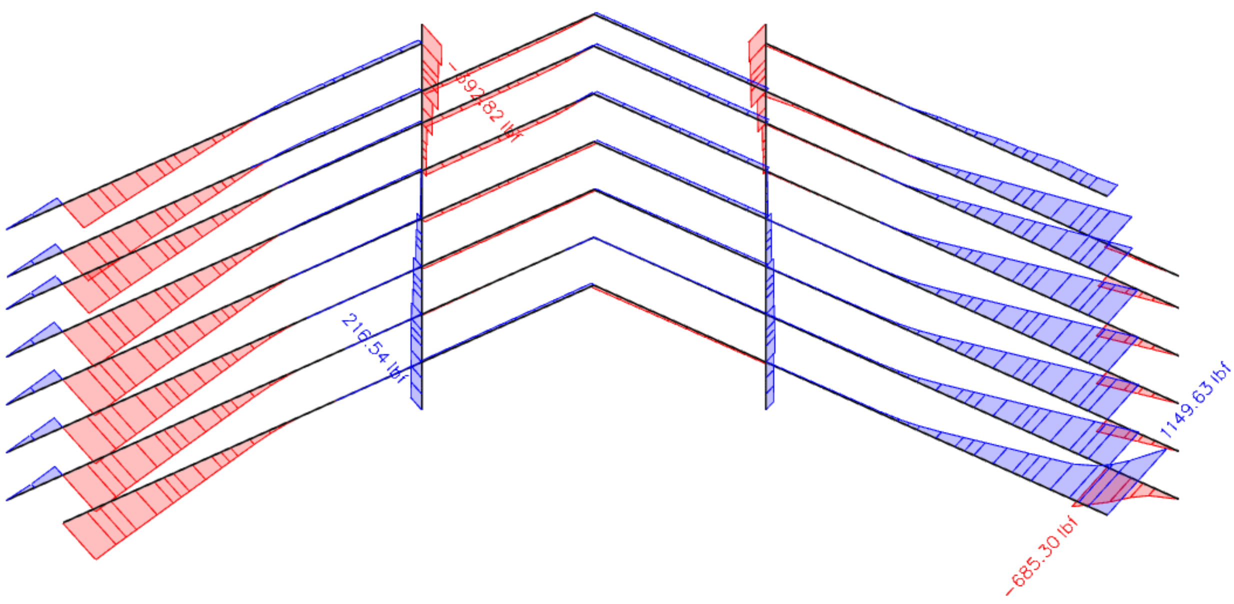
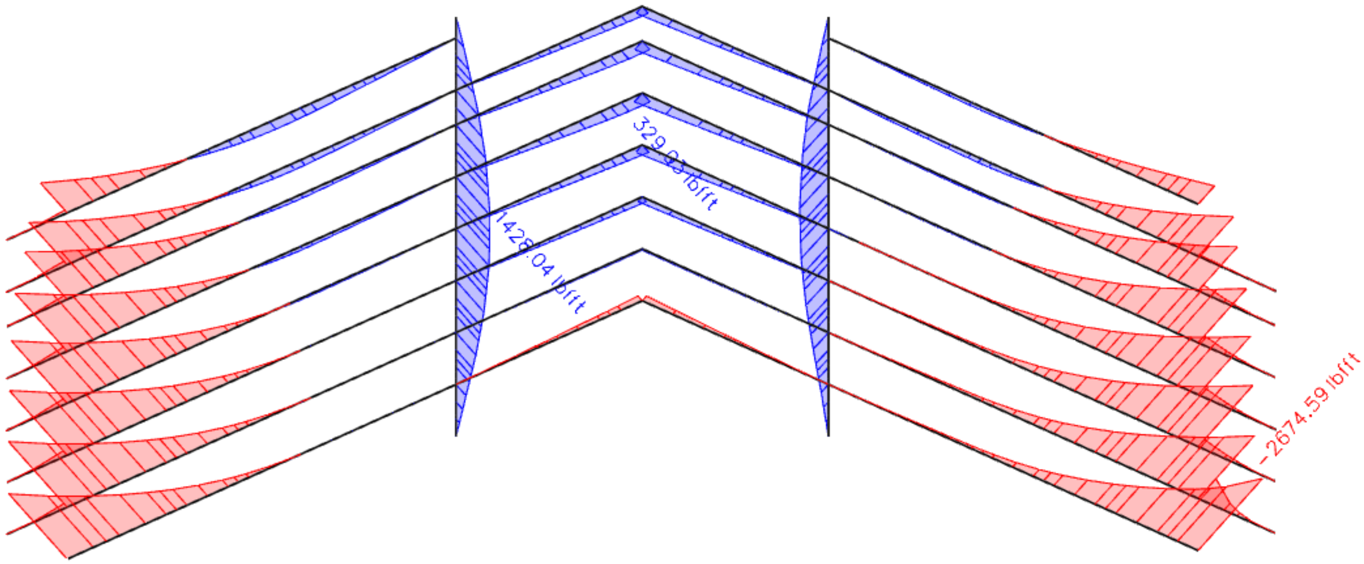
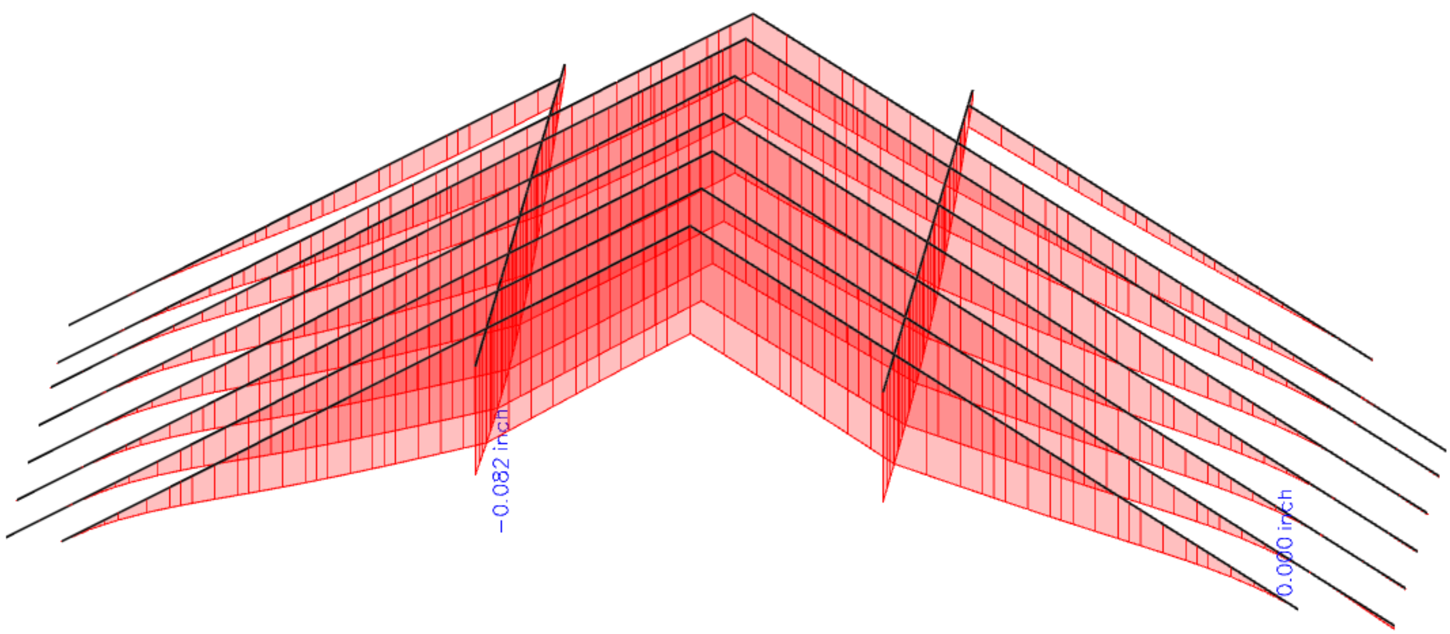


Diagram of moment My,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr),lbf*ft.



Displacement of elements
Value: Uz - Lr, (inch) .



The maximum deflection is 0.082" according to table 1604.3 the code IBC 2021 - the deflection limits L/360. L = 16'-2"= 16'*12"+2"=194". 194"/360 =0.538"
0.082" < 0.538" <u>Deflection is OK!</u>

STEEL MEMBER B3177 CHECK (BEAM)

AISI S100-16 LRFD Check

Member 3177	Box1000S200-68	A1008 grade 54	LRFD-Ult (auto)	0.15

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

The critical check is on position 10.00 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	-64.47	lbf
Vux	67.68	lbf
Vuy	-87.65	lbf
Mut	0.00	lbfft
Mux	1428.04	lbfft
Muy	-382.86	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	0.589	50.000 44.469	0.89	0.470	721.648	0.263	1.000	0.589 -	- -	-	- -	-
2	1.929	50.000 50.000	1.00	4.000	143.317	0.591	1.000	1.929 -	- -	-	- -	-
3	9.929	50.000 -43.188	0.86	20.675	27.954	1.337	0.625	- 6.203	1.605 1.723	-	- -	-
4	1.929	-43.188 -43.188	-	-	-	-	-	- -	- -	-	- -	-
5	0.589	-37.657 -43.188	-	-	-	-	-	- -	- -	-	- -	-
6	1.929	-43.188 -43.188	-	-	-	-	-	- -	- -	-	- -	-
7	9.929	50.000 -43.188	0.86	20.675	27.954	1.337	0.625	- 6.203	1.605 1.723	-	- -	-
8	1.929	50.000 50.000	1.00	4.000	143.317	0.591	1.000	- 1.929	0.964 0.964	-	- -	-

Table of values		
Sxe	5.024	$inch^3$
Mnxo	20931.35	lbfft
Resistance factor	0.90	
Unity check	0.08	-

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

Table of values		
Lltb	1 ft 12.000 in	ft
Sigma,ey	1448.402	ksi
Kt	1.00	
Lt	1 ft 12.000 in	ft
Sigma,t	5023.403	ksi
Cb	1.05	
Sfx	5.595	$inch^3$
Fcre	4299.124	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	0.589	11.523 11.523	1.00	0.430	660.008	0.132	1.000	0.589 -	- -	-	- -	-
2	1.929	50.000 12.222	0.24	6.374	228.369	0.468	1.000	- 1.929	0.700 1.229	-	- -	-
3	9.929	50.000 50.000	1.00	4.000	5.408	3.041	0.305	3.029 -	- -	-	- -	-
4	1.929	50.000 12.222	0.24	6.374	228.369	0.468	1.000	- 1.929	0.700 1.229	-	- -	-
5	0.589	11.523 11.523	1.00	0.431	662.069	0.132	1.000	0.589 -	- -	-	- -	-
6	1.929	10.825 -26.953	2.49	95.989	3439.200	0.056	1.000	- 1.929	0.351 0.201	-	- -	-
7	9.929	-26.953 -26.953	-	-	-	-	-	- -	- -	-	- -	-
8	1.929	10.825 -26.953	2.49	95.989	3439.200	0.056	1.000	- 1.929	0.351 0.201	-	- -	-

Table of values		
Sye	1.436	$inch^3$
Mnyo	5984.84	lbfft
Resistance factor	0.90	
Unity check	0.07	-

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

Table of values		
Sigma,ex	101.413	ksi
Kt	1.00	
Lt	1 ft 12.000 in	ft
Sigma,t	5023.403	ksi
Cb	1.00	
Sfy	3.057	$inch^3$
Fcre	1974.446	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	0.084	2521.24
2	0.000	0.00
3	0.708	5112.07
4	0.000	0.00
5	0.084	2521.24
6	0.000	0.00
7	0.708	5112.07
8	0.000	0.00

Table of values		
Vn,y	15266.62	lbf
Resistance factor	0.95	
Unity check	0.01	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	20931.35	lbfft
Vny	15266.62	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.01+0.00) = 0.08

Buckling check

According to article E2 and formula (E2-1)

Flexural Buckling Strength

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	16 1/4	2	ft
Effective Length factor K	1.00	1.00	
Effective Length	16 1/4	2	ft
Slenderness	53.13	14.06	
Flexural Buckling stress Fcre	101.413	1448.402	ksi

Torsional (-Flexural) Buckling Strength

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

Table of values		
Sigma,ex	101.413	ksi
Sigma,ey	1448.402	ksi
Kt	1.00	
Lt	2	ft
Sigma,t	5023.403	ksi
Sigma,TF	101.413	ksi
Torsional (-Flexural) buckling stress Fcre	101.413	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	0.589	40.677 40.677	1.00	0.430	660.008	0.248	1.000	0.589 -	- -	-	- -	-
2	1.929	40.677 40.677	1.00	4.000	143.317	0.533	1.000	1.929 -	- -	-	- -	-
3	9.929	40.677 40.677	1.00	4.000	5.408	2.743	0.335	3.330 -	- -	-	- -	-
4	1.929	40.677 40.677	1.00	4.000	143.317	0.533	1.000	1.929 -	- -	-	- -	-
5	0.589	40.677 40.677	1.00	0.430	660.008	0.248	1.000	0.589 -	- -	-	- -	-
6	1.929	40.677 40.677	1.00	4.000	143.317	0.533	1.000	1.929 -	- -	-	- -	-
7	9.929	40.677 40.677	1.00	4.000	5.408	2.743	0.335	3.330 -	- -	-	- -	-
8	1.929	40.677 40.677	1.00	4.000	143.317	0.533	1.000	1.929 -	- -	-	- -	-

Table of values		
Fe	101.413	ksi
lambda, c	0.70	
Fn	40.677	ksi
Ae	1.194	inch ^{2}
Pn	48549.88	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	0.589	0.031 0.031	1.00	0.430	660.008	0.007	1.000	0.589 -	- -	-	- -	-
2	1.929	0.031 0.031	1.00	4.000	143.317	0.015	1.000	1.929 -	- -	-	- -	-
3	9.929	0.031 0.031	1.00	4.000	5.408	0.075	1.000	9.929 -	- -	-	- -	-
4	1.929	0.031 0.031	1.00	4.000	143.317	0.015	1.000	1.929 -	- -	-	- -	-
5	0.589	0.031 0.031	1.00	0.430	660.008	0.007	1.000	0.589 -	- -	-	- -	-
6	1.929	0.031 0.031	1.00	4.000	143.317	0.015	1.000	1.929 -	- -	-	- -	-
7	9.929	0.031 0.031	1.00	4.000	5.408	0.075	1.000	9.929 -	- -	-	- -	-
8	1.929	0.031 0.031	1.00	4.000	143.317	0.015	1.000	1.929 -	- -	-	- -	-

Table of values		
Mnx	20931.35	lbfft
Mny	5984.84	lbfft
Pn	48549.88	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.00+0.08+0.07 = 0.15 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B3032 CHECK (RAFTER)

AISI S100-16 LRFD Check

Member 3032	S1000S200-54	A1008 grade 54	LRFD-Ult (auto)	0.65

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	-736.18	lbf
Vux	2.78	lbf
Vuy	1161.62	lbf
Mut	-0.00	lbfft
Mux	-2817.70	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	0.484	48.999 44.723	0.91	0.461	165.766	0.544	1.000	0.223 -	- -	-	- -	-
3	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
5	9.717	48.999 -36.932	0.75	18.295	16.274	1.735	0.503	- 4.890	1.303 1.486	-	- -	-
7	1.717	-37.933 -37.933	-	-	-	-	-	- -	- -	-	- -	-
9	0.484	-32.657 -36.932	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	1.690	$inch^3$
Mnxo	7041.71	lbfft
Resistance factor	0.90	
Unity check	0.44	-

Lateral-Torsional Buckling Strength

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

Table of values		
Lltb	2 ft 0.000 in	ft
Sigma,ey	222.962	ksi
Kt	1.00	
Lt	2 ft 0.000 in	ft
Sigma,t	297.800	ksi
Cb	1.32	
Sfx	2.270	$inch^3$
Fcre	493.927	ksi

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

Distortional Buckling Strength

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

Table of values		
Sfy	2.270	$inch^3$
My	9458.42	lbfft
L	1 ft 8.136 in	ft
Beta	1.22	
k,phi,fe	183.14	lbf
k,phi,we	182.81	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	$inch^2$
k,phi,wg	0.004	$inch^2$
Fd	42.668	ksi
Sf	2.270	$inch^3$

Table of values		
Mcrd	8071.40	lbfft
Lambda,d	1.08	
Mn	6961.72	lbfft
Resistance factor	0.90	
Unity check	0.45	-

Data		
Lm	2 ft 0.000 in	ft
Lcr	1 ft 8.136 in	ft
h0	10.000	inch
Ixf	0.003	$inch^4$
Iyf	0.049	$inch^4$
Ixyf	0.007	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.691	inch
hxf	-1.239	inch
Af	0.135	$inch^2$
y0f	-0.068	inch
Ksi,web	2.00	

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

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

Shear Strength

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

Shear force Vy

Element ID	Aw [inch ^{2}]	Vn [lbf]
3	0.000	0.00
5	0.550	2612.99
7	0.000	0.00

Table of values		
Vn,y	2612.99	lbf
Resistance factor	0.95	
Unity check	0.47	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	7041.71	lbfft
Vny	2612.99	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.20+0.22) = 0.65

...::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	0.484	50.000 50.000	1.00	0.430	154.489	0.569	1.000	0.223 -	- -	-	- -	-
3	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
5	9.717	50.000 50.000	1.00	4.000	3.558	3.749	0.251	2.440 -	- -	-	- -	-
7	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
9	0.484	50.000 50.000	1.00	0.430	154.489	0.569	1.000	0.223 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.380	inch ^{2}
Pno	18985.89	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	2	ft
Effective Length factor K	1.00	1.00	
Effective Length	12	2	ft
Slenderness	39.04	35.83	
Flexural Buckling stress Fcre	187.869	222.962	ksi

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

Table of values		
Sigma,ex	187.869	ksi
Sigma,ey	222.962	ksi
Kt	1.00	
Lt	2	ft
Sigma,t	297.800	ksi
Sigma,TF	168.899	ksi
Torsional (-Flexural) buckling stress Fcre	168.899	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	0.484	44.173 44.173	1.00	0.430	154.489	0.535	1.000	0.296 -	- -	-	- -	-
3	1.717	44.173 44.173	1.00	2.956	84.209	0.724	0.961	1.651 -	0.505 1.145	32.80	0.001 0.001	0.296
5	9.717	44.173 44.173	1.00	4.000	3.558	3.523	0.266	2.586 -	- -	-	- -	-
7	1.717	44.173 44.173	1.00	2.956	84.209	0.724	0.961	1.651 -	0.505 1.145	32.80	0.001 0.001	0.296
9	0.484	44.173 44.173	1.00	0.430	154.489	0.535	1.000	0.296 -	- -	-	- -	-

Table of values		
Fe	168.899	ksi
lambda, c	0.54	
Fn	44.173	ksi
Ae	0.407	inch^2
Pn	17974.53	lbf
Resistance factor	0.85	
Unity check	0.05	-

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

Table of values		
Py	42103.51	lbf
L	1 ft 10.238 in	ft
k,phi,fe	128.99	lbf
k,phi,we	96.33	lbf
k,phi	0.00	lbf
k,phi,fg	0.005	$inch^2$
k,phi,wg	0.019	$inch^2$
Fd	9.266	ksi
Pcrd	7802.49	lbf
Lambda,d	2.32	
Pn	13920.87	lbf
Resistance factor	0.85	
Unity check	0.06	-

Data		
Lm	2 ft 0.000 in	ft
Lcr	1 ft 10.238 in	ft
h0	10.000	inch
Ixf	0.003	$inch^4$
Iyf	0.049	$inch^4$
Ixyf	-0.007	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.691	inch
hxf	-1.239	inch
Af	0.135	$inch^2$
y0f	0.068	inch

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

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

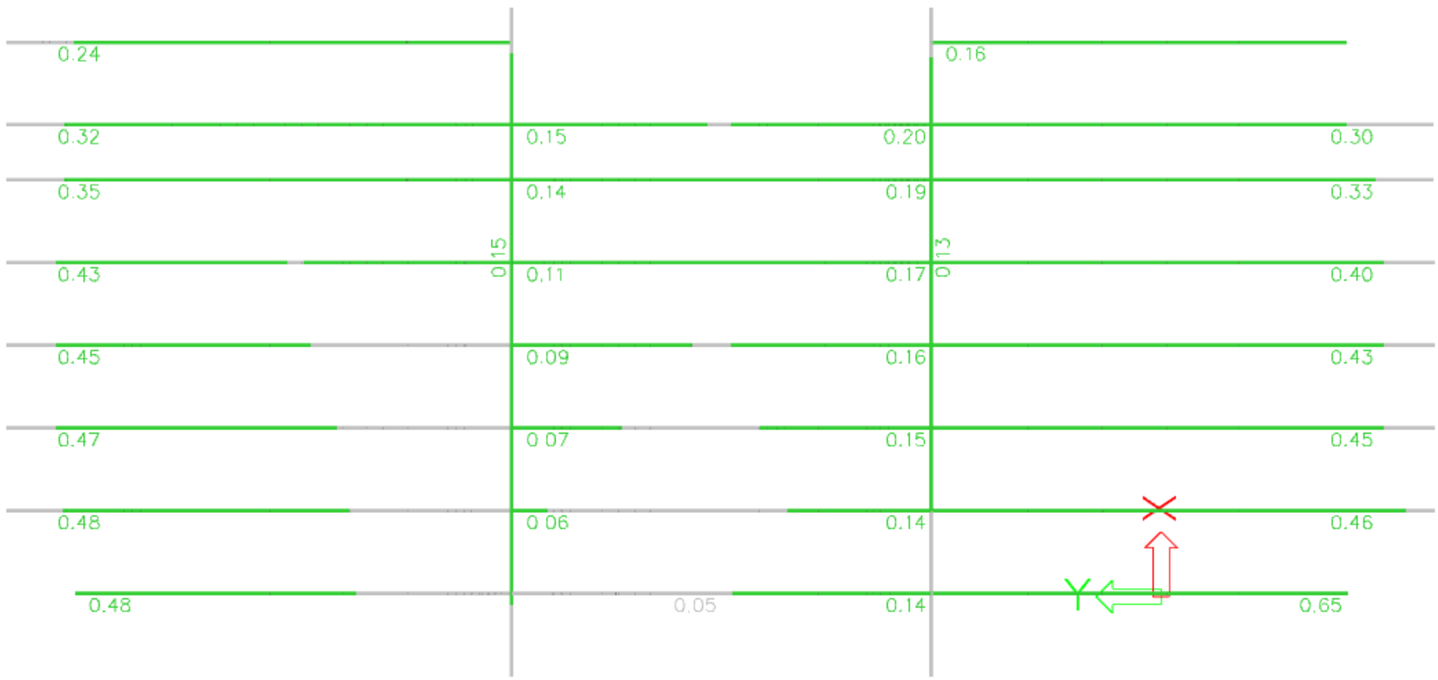
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.484	0.874 0.874	1.00	0.430	154.489	0.075	1.000	0.484 -	- -	-	- -	-
3	1.717	0.874 0.874	1.00	4.000	113.957	0.088	1.000	1.717 -	0.858 0.858	233.16	- 0.001	0.484
5	9.717	0.874 0.874	1.00	4.000	3.558	0.496	1.000	9.717 -	- -	-	- -	-
7	1.717	0.874 0.874	1.00	4.000	113.957	0.088	1.000	1.717 -	0.858 0.858	233.16	- 0.001	0.484
9	0.484	0.874 0.874	1.00	0.430	154.489	0.075	1.000	0.484 -	- -	-	- -	-

Table of values		
Mnx	6961.72	lbfft
Pn	13920.87	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.06+0.45+0.00 = 0.51 - (C5.2.1-3)

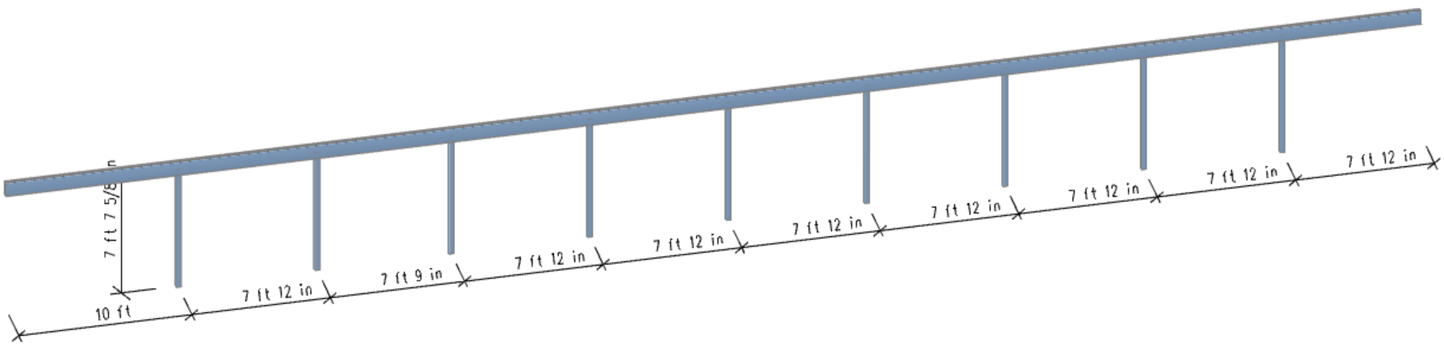
The member satisfies the check !

Unity check

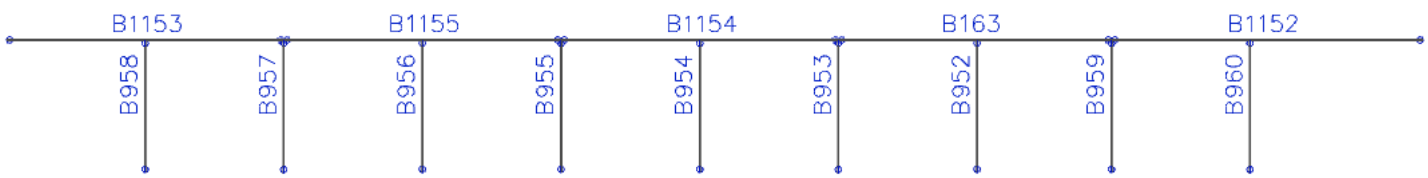


BLOCK D. RIDGE BEAM & COLUMN STRUCTURAL ANALYSIS

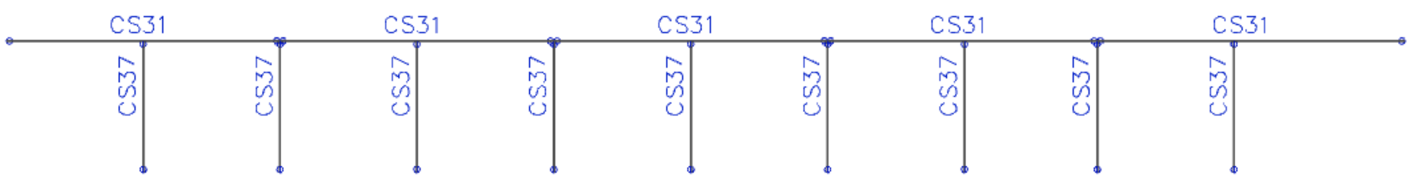
General scheme



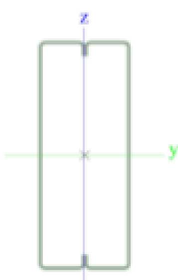
Member numbers



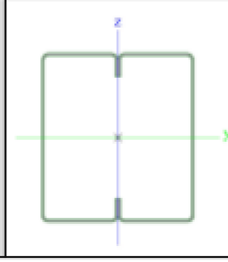
Cross-sections of



CS31 Cross-sections properties

CS31		
Type	Box1000S200-97	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [$inch^2$]	2.947	
A_y [$inch^2$], A_z [$inch^2$]	0.865	1.971
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	2.81e+01	5.69e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1.578	0.000
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	38.649	8.515
i_y [inch], i_z [inch]	3.622	1.700
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	7.730	4.258
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	9.533	4.636
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	4.77e+02	4.77e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.32e+02	2.32e+02
d_y [inch], d_z [inch]	0.000	0.000
I_t [$inch^4$], I_w [$inch^6$]	20.771	14.786
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

CS37 Cross-sections properties

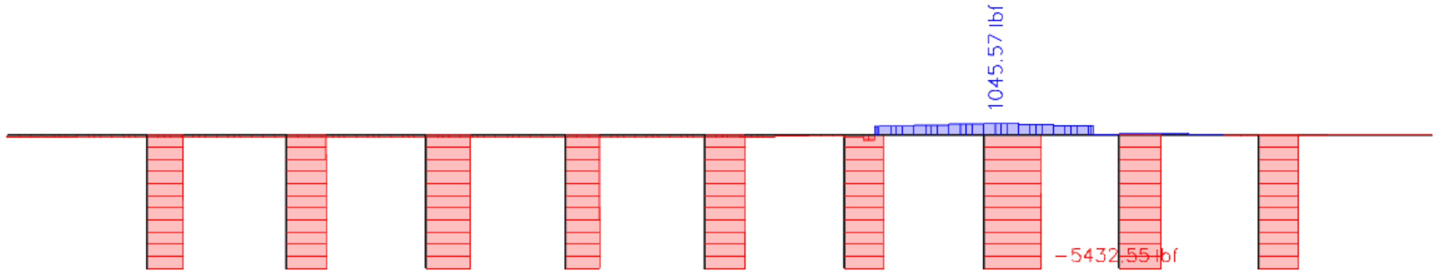
CS37		
Type	Box362S162-54	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [$inch^2$]	0.843	
A_y [$inch^2$], A_z [$inch^2$]	0.391	0.439
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	1.38e+01	2.86e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	1.091	0.000
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	1.745	1.308
i_y [inch], i_z [inch]	1.438	1.245
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.963	0.805
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	1.119	0.918
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.59e+01	5.59e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	4.59e+01	4.59e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.168	6.567
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

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

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

Maximum force diagram

Axial force diagram N,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr), lbf.



Shear force diagram Vz ,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr), lbf.

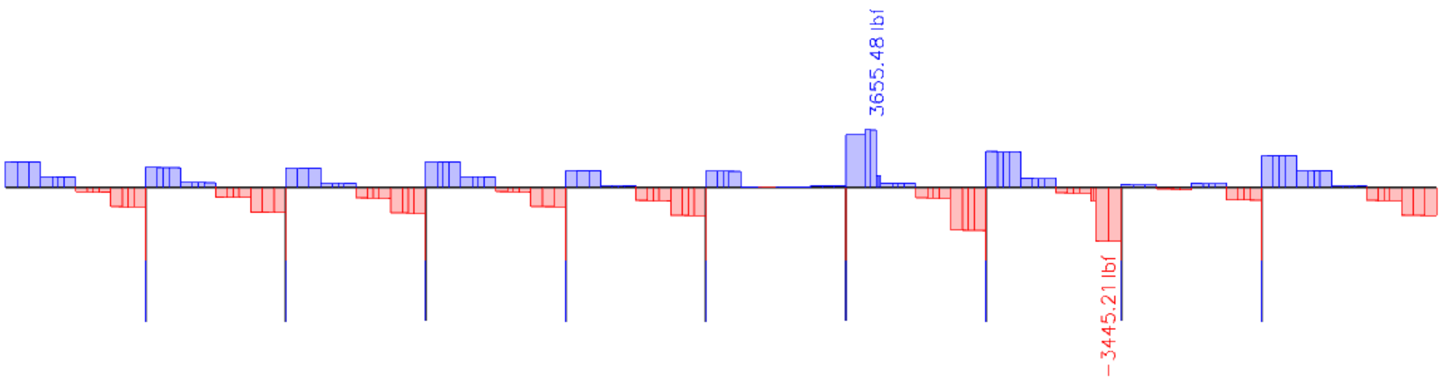
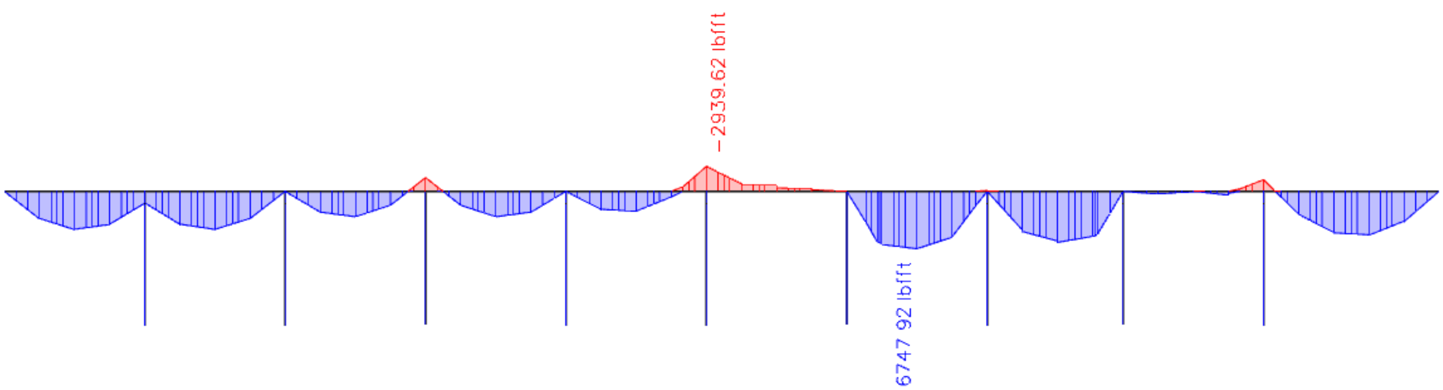
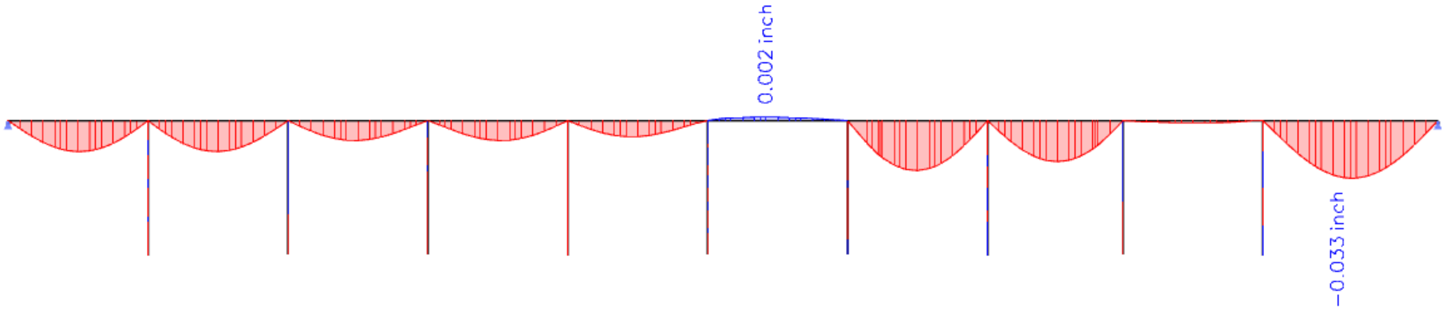


Diagram of moment My,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr),lbf*ft.



Displacement of elements

Value: Uz - Lr, (inch) .



The maximum deflection is 0.033" according to table 1604.3 the code IBC 2021 - the deflection limits L/360. L = 10'= 10'*12"=120". 120"/360 =0.334"
0.033" < 0.334" <u>Deflection is OK!</u>

STEEL MEMBER B163 CHECK (RIDGE BEAM)

AISI S100-16 LRFD Check

Member B163	Box1000S200-97	A1008 grade 54	LRFD-Ult (auto)	0.90

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

The critical check is on position 14.27 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	32.01	lbf
Vux	2130.13	lbf
Vuy	-3427.34	lbf
Mut	-0.00	lbfft
Mux	5098.88	lbfft
Muy	-6456.11	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	0.574	50.000 44.354	0.89	0.471	1549.855	0.180	1.000	0.574 -	- -	-	- -	-
2	1.898	50.000 50.000	1.00	4.000	300.996	0.408	1.000	1.898 -	- -	-	- -	-
3	9.898	50.000 -47.329	0.95	22.645	62.674	0.893	0.844	- 8.352	2.116 2.174	-	- -	-
4	1.898	-47.329 -47.329	-	-	-	-	-	- -	- -	-	- -	-
5	0.574	-41.684 -47.329	-	-	-	-	-	- -	- -	-	- -	-
6	1.898	-47.329 -47.329	-	-	-	-	-	- -	- -	-	- -	-
7	9.898	50.000 -47.329	0.95	22.645	62.674	0.893	0.844	- 8.352	2.116 2.174	-	- -	-
8	1.898	50.000 50.000	1.00	4.000	300.996	0.408	1.000	- 1.898	0.949 0.949	-	- -	-

Table of values		
Sxe	7.523	inch³
Mnxo	31344.30	lbfft
Resistance factor	0.90	
Unity check	0.18	-

Lateral-Torsional Buckling Strength

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

Table of values		
Lltb	1 ft 5.806 in	ft
Sigma,ey	2609.230	ksi
Kt	1.00	
Lt	1 ft 5.806 in	ft
Sigma,t	5196.603	ksi
Cb	1.67	
Sfx	7.730	inch³
Fcre	9353.550	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 [inch⁴]	ds [inch]
1	0.574	9.636 9.636	1.00	0.430	1414.841	0.083	1.000	0.574 -	- -	-	- -	-
2	1.898	50.000 10.689	0.21	6.544	492.458	0.319	1.000	- 1.898	0.681 1.217	-	- -	-
3	9.898	50.000 50.000	1.00	4.000	11.071	2.125	0.422	4.175 -	- -	-	- -	-

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]
4	1.898	50.000 10.689	0.21	6.544	492.458	0.319	1.000	- 1.898	0.681 1.217	-	- -	-
5	0.574	9.636 9.636	1.00	0.430	1414.841	0.083	1.000	0.574 -	- -	-	- -	-
6	1.898	8.583 -30.727	3.58	205.291	15447.897	0.024	1.000	- 1.898	0.288 0.126	-	- -	-
7	9.898	-30.727 -30.727	-	-	-	-	-	- -	- -	-	- -	-
8	1.898	8.583 -30.727	3.58	205.291	15447.897	0.024	1.000	- 1.898	0.288 0.126	-	- -	-

Table of values		
Sye	2.400	inch^3
Mnyo	9998.70	lbfft
Resistance factor	0.90	
Unity check	0.72	-

Lateral-Torsional Buckling Strength

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

Table of values		
Sigma,ex	434.141	ksi
Kt	1.00	
Lt	1 ft 5.806 in	ft
Sigma,t	5196.603	ksi
Cb	1.00	
Sfy	4.258	inch^3
Fcre	4159.080	ksi

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

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

Shear Strength

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

Shear force Vx

Element ID	Aw [inch^2]	Vn [lbf]
1	0.000	0.00
2	0.193	5791.71
3	0.000	0.00
4	0.193	5791.71
5	0.000	0.00
6	0.193	5791.71
7	0.000	0.00
8	0.193	5791.71

Table of values		
Vn,x	23166.86	lbf
Resistance factor	0.95	
Unity check	0.10	-

Shear force Vy

Element ID	Aw [inch²]	Vn [lbf]
1	0.117	3503.46
2	0.000	0.00
3	1.007	14880.68
4	0.000	0.00
5	0.117	3503.46
6	0.000	0.00
7	1.007	14880.68
8	0.000	0.00

Table of values		
Vn,y	36768.30	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	31344.30	lbfft
Vny	36768.30	lbf
Mnyo	9998.70	lbfft
Vnx	23166.86	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check (Mx, Vy) = sqrt(0.03+0.01) = 0.21
Unity check (My, Vx) = sqrt(0.51+0.01) = 0.72

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

Table of values		
Sftx	7.730	inch³
Sfty	4.258	inch³
Mnxt	32207.18	lbfft
Mnyt	17739.62	lbfft
Mnx	31344.30	lbfft
Mny	9998.70	lbfft
Tn	147339.00	lbf
Resistance factor tension	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.18+0.40+0.00 = 0.58 - (H1.1-1)
Unity check = 0.18+0.72-0.00 = 0.90 - (H1.1-2)

The member satisfies the check !

STEEL MEMBER B952 CHECK (COLUMN)

AISI S100-16 LRFD Check

Member B952	Box362S162-54	A1008 grade 54	LRFD-Ult (auto)	0.25

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

The critical check is on position 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	-5432.55	lbf
Vux	0.00	lbf
Vuy	0.00	lbf
Mut	-12.24	lbfft
Mux	-0.00	lbfft
Muy	-0.00	lbfft

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	50.000 50.000	1.00	0.430	649.258	0.278	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	3.568	50.000 50.000	1.00	4.000	26.384	1.377	0.610	2.178 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 50.000	1.00	0.430	649.258	0.278	1.000	0.472 -	- -	-	- -	-
6	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
7	3.568	50.000 50.000	1.00	4.000	26.384	1.377	0.610	2.178 -	- -	-	- -	-
8	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.709	inch^2
Pno	35442.99	lbf
Resistance factor	0.85	
Unity check	0.18	-

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	7 3/4	7 3/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	7 3/4	7 3/4	ft
Slenderness	63.69	73.56	
Flexural Buckling stress Fcre	70.571	52.915	ksi

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

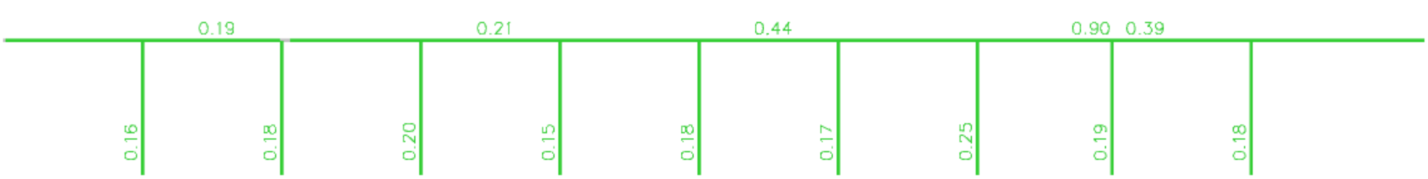
Table of values		
Sigma,ex	70.571	ksi
Sigma,ey	52.915	ksi
Kt	1.00	
Lt	7 3/4	ft
Sigma,t	687.904	ksi
Sigma,TF	52.915	ksi
Torsional (-Flexural) buckling stress Fcre	52.915	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.472	33.667 33.667	1.00	0.430	649.258	0.228	1.000	0.472 -	- -	-	- -	-
2	1.568	33.667 33.667	1.00	4.000	136.574	0.497	1.000	1.568 -	- -	-	- -	-
3	3.568	33.667 33.667	1.00	4.000	26.384	1.130	0.713	2.544 -	- -	-	- -	-
4	1.568	33.667 33.667	1.00	4.000	136.574	0.497	1.000	1.568 -	- -	-	- -	-
5	0.472	33.667 33.667	1.00	0.430	649.258	0.228	1.000	0.472 -	- -	-	- -	-
6	1.568	33.667 33.667	1.00	4.000	136.574	0.497	1.000	1.568 -	- -	-	- -	-
7	3.568	33.667 33.667	1.00	4.000	26.384	1.130	0.713	2.544 -	- -	-	- -	-
8	1.568	33.667 33.667	1.00	4.000	136.574	0.497	1.000	1.568 -	- -	-	- -	-

Table of values		
Fe	52.915	ksi
lambda, c	0.97	
Fn	33.667	ksi
Ae	0.750	$inch^2$
Pn	25259.67	lbf
Resistance factor	0.85	
Unity check	0.25	-

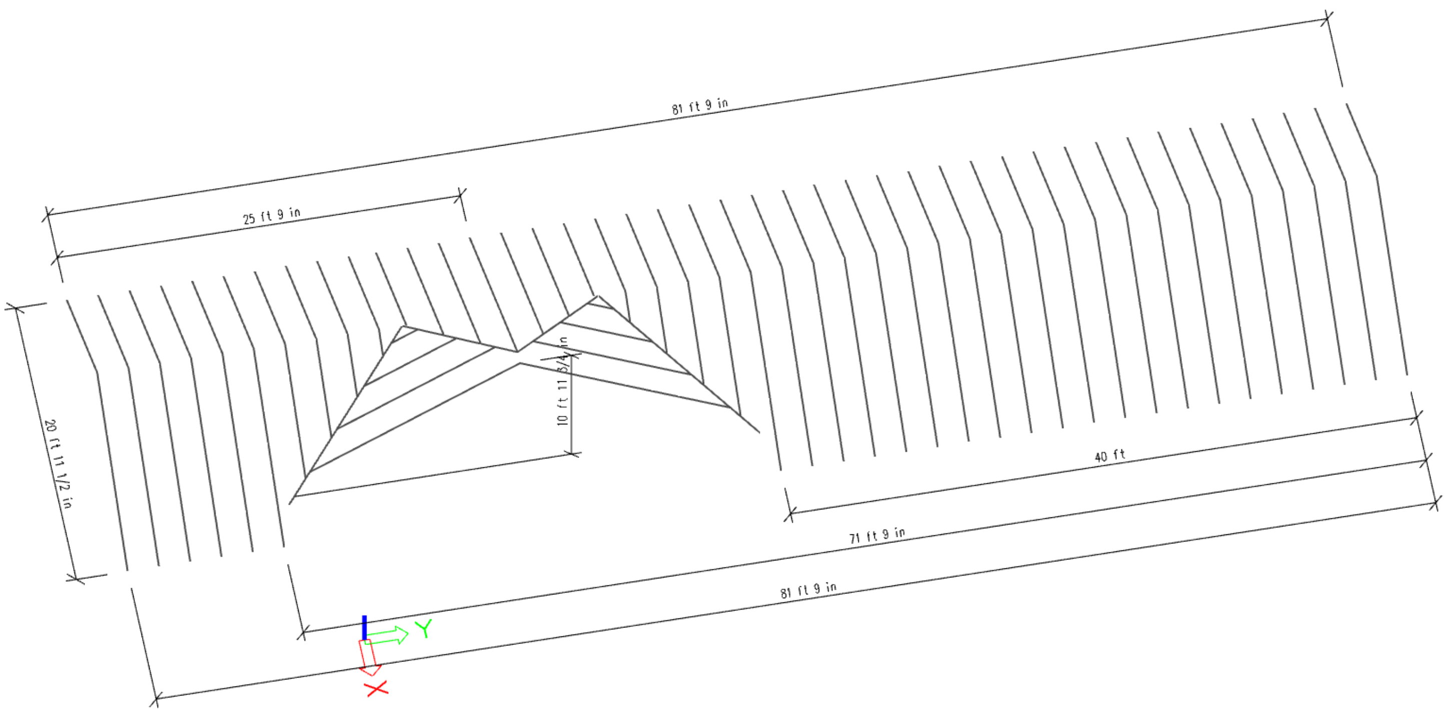
The member satisfies the check !

Unity check

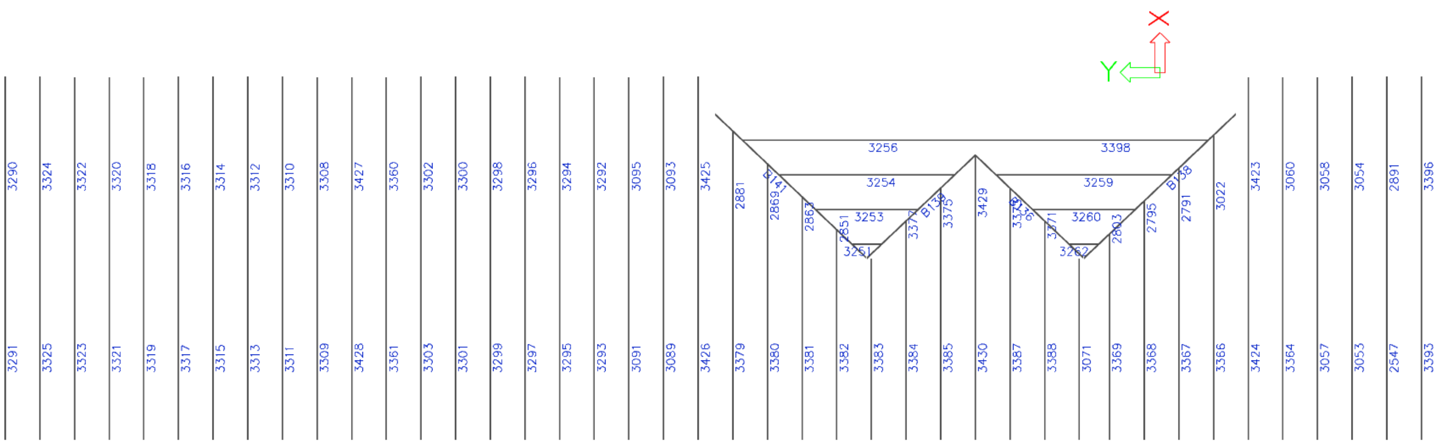


BLOCK D. ROOF RAFTER STRUCTURAL ANALYSIS

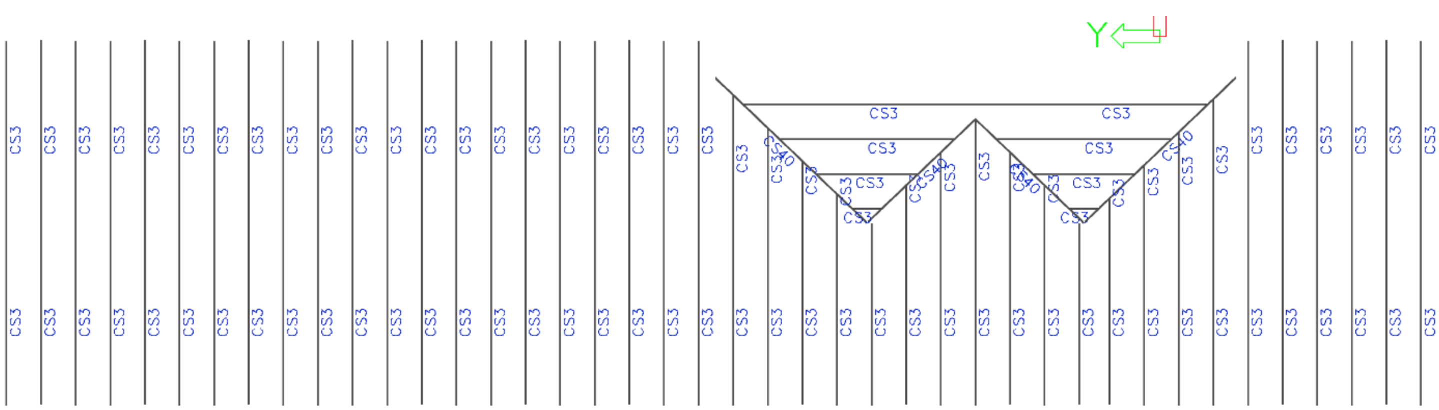
General scheme



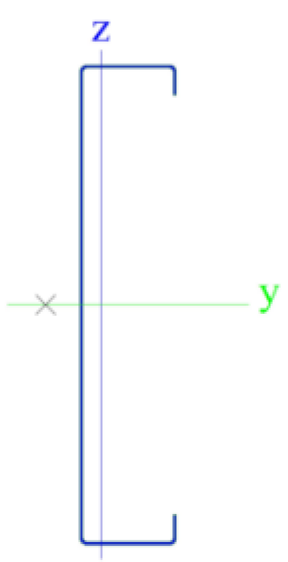
Member numbers



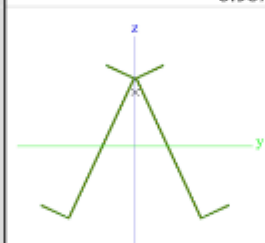
Cross-sections of



CS3 Cross-sections properties

CS3		
Type	S1000S200-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	0.842	
A_y [inch^2], A_z [inch^2]	0.229	0.566
A_L [inch^2/inch], A_D [inch^2/inch]	2.98e+01	2.98e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.427	5.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	11.350	0.378
i_y [inch], i_z [inch]	3.671	0.670
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	2.255	0.240
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.753	0.340
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.38e+02	1.38e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.70e+01	1.70e+01
d_y [inch], d_z [inch]	-1.143	0.000
I_t [inch^4], I_w [inch^6]	0.001	7.665
$β_y$ [inch], $β_z$ [inch]	0.000	12.209
Picture		

CS40 Cross-sections properties

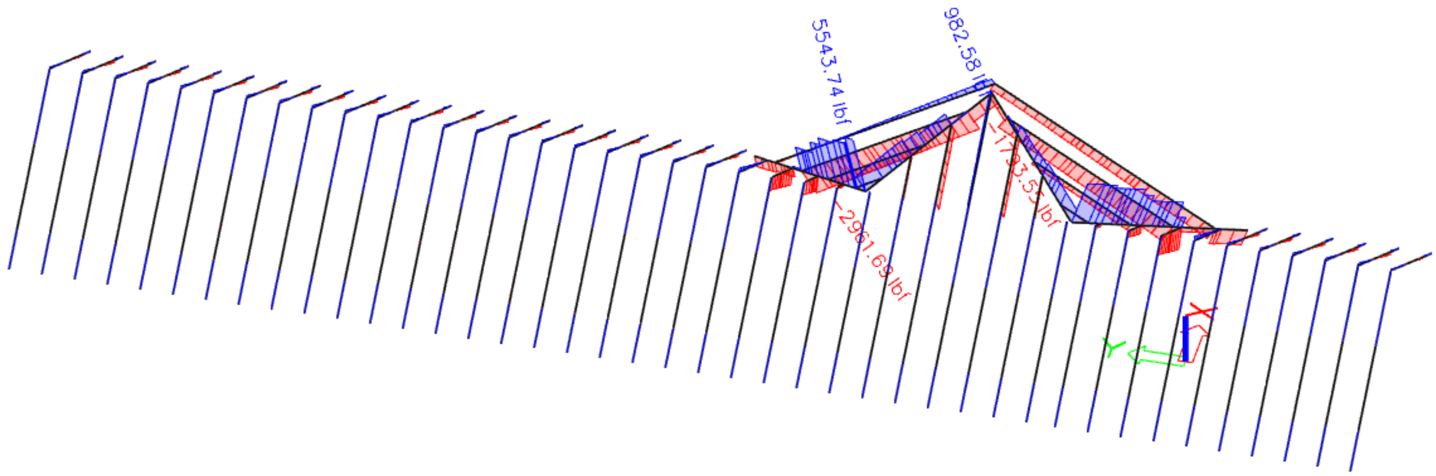
CS40		
Type	1000T200-97/	
Shape type	Thick-walled	
Item material	A1008 grade 54	
Fabrication	cold formed	
Colour		
A [inch^2]	2.678	
A_y [inch^2], A_z [inch^2]	1.849	0.673
A_L [inch^2/inch], A_D [inch^2/inch]	5.56e+01	5.56e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-2.246	-0.075
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	28.285	22.277
i_y [inch], i_z [inch]	3.250	2.884
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	5.398	3.688
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	7.709	6.450
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	3.85e+02	3.85e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.22e+02	3.22e+02
d_y [inch], d_z [inch]	0.000	3.471
I_t [inch^4], I_w [inch^6]	0.008	28.518
$β_y$ [inch], $β_z$ [inch]	-8.987	0.000
Picture		

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

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

Maximum force diagram

Axial force diagram N,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr), lbf.



Shear force diagram Vz ,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr), lbf.

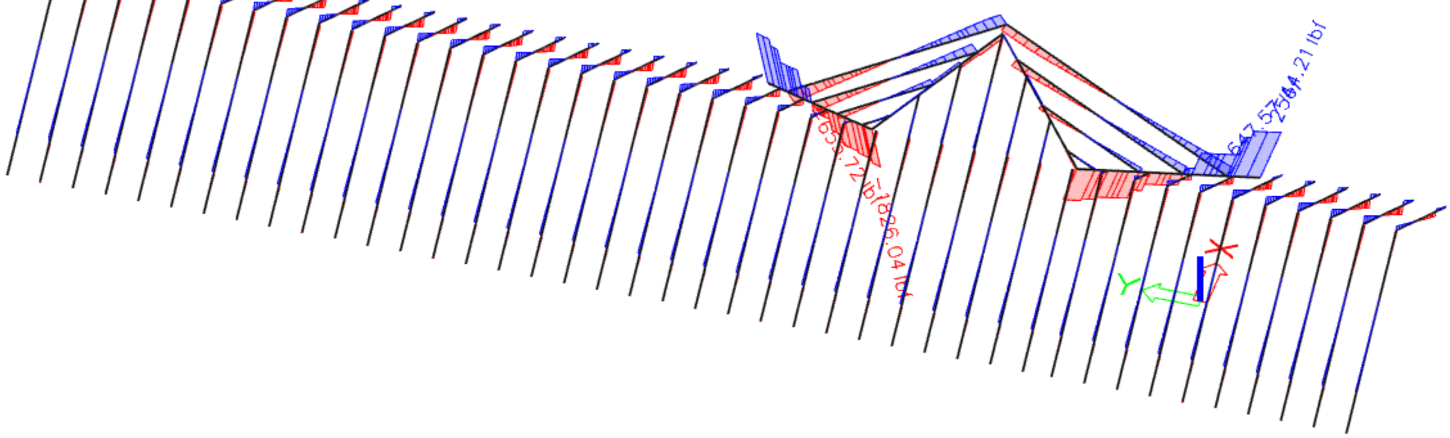
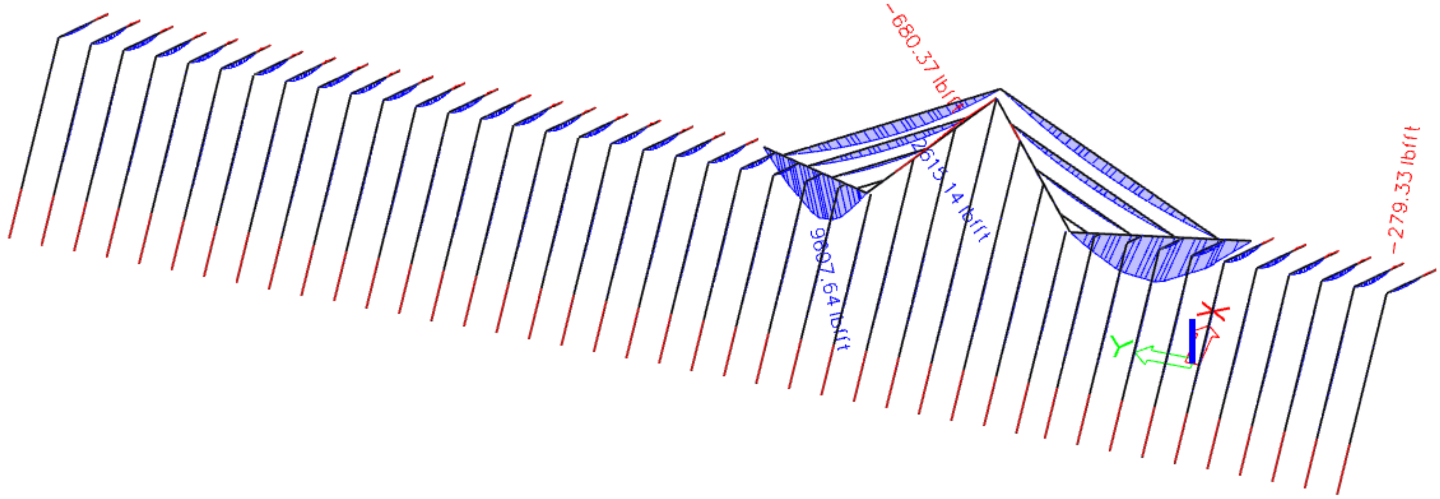
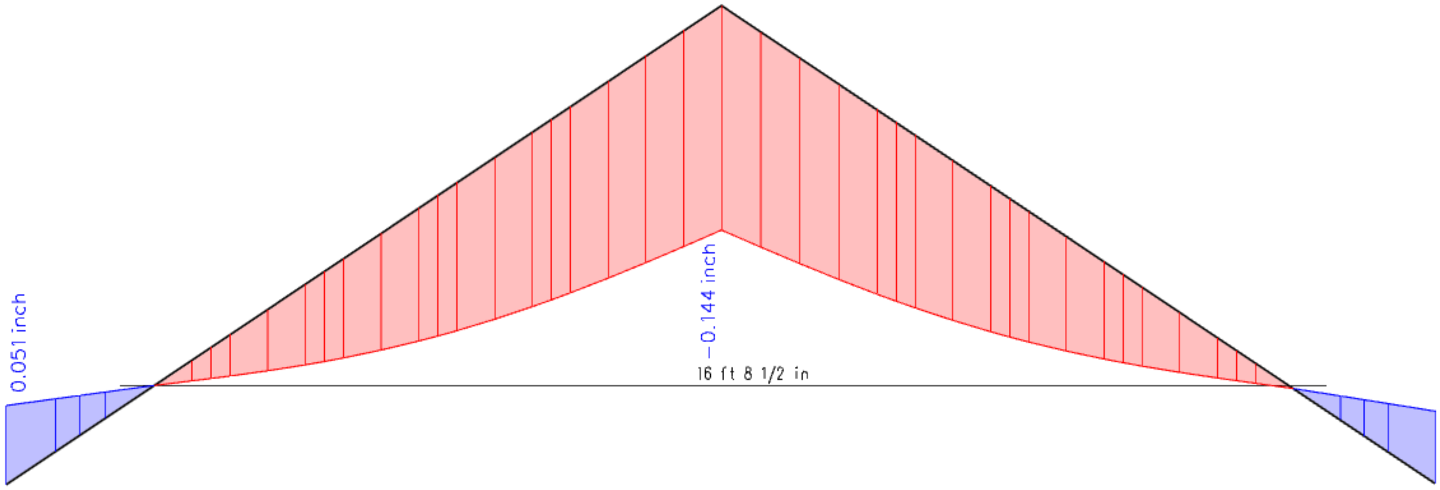


Diagram of moment My,
LRFD-Ult (auto)7 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr),lbf*ft.



Displacement of elements
Value: Uz - Lr, (inch) .



The maximum deflection is 0.144" according to table 1604.3 the code IBC 2021 - the deflection limits L/360. L = 16’-8.5”= 16’*12”+8.5”=200.5”. 200.5”/360 =0.556”
0.144” < 0.556” **<u>Deflection is OK!</u>**

STEEL MEMBER B132 CHECK

AISI S100-16 LRFD Check

Member B138	1000T200-97/	A913 grade 50	LRFD-Ult (auto)	0.64

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

The critical check is on position 4.97 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	-84.47	lbf
Vux	-381.87	lbf
Vuy	419.25	lbf
Mut	-0.13	lbfft
Mux	6588.28	lbfft
Muy	-2354.69	lbfft

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

Nominal Flexural Strength

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	1.949	50.000 42.613	0.85	0.442	31.538	1.259	0.655	1.278 -	- -	-	- -	-
2	9.898	42.613 -37.831	0.89	21.231	58.759	0.852	0.871	- 8.620	2.217 2.349	-	- -	-
3	1.949	-30.444 -37.831	-	-	-	-	-	- -	- -	-	- -	-
4	1.949	-30.444 -37.831	-	-	-	-	-	- -	- -	-	- -	-
5	9.898	42.613 -37.831	0.89	21.231	58.759	0.852	0.871	- 8.620	2.217 2.349	-	- -	-
6	1.949	50.000 42.613	0.85	0.442	31.538	1.259	0.655	1.278 -	- -	-	- -	-

Table of values		
Sxe	4.473	$inch^3$
Mnxo	18638.41	lbfft
Resistance factor	0.90	
Unity check	0.39	-

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

Table of values		
Lltb	2 ft 6.675 in	ft
Sigma,ey	2408.851	ksi
Kt	1.00	
Lt	2 ft 6.675 in	ft
Sigma,t	106.431	ksi
Cb	1.29	
Sfx	5.511	$inch^3$
Fcre	1766.040	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	1.949	4.449 -9.143	2.05	1.297	92.575	0.219	1.000	1.949 -	- -	-	- -	-
2	9.898	4.449 -27.737	6.23	775.527	2146.375	0.046	1.000	- 9.898	1.072 0.296	-	- -	-
3	1.949	-27.737 -41.329	-	-	-	-	-	- -	- -	-	- -	-
4	1.949	50.000 36.408	0.73	0.454	32.418	1.242	0.663	1.291 -	- -	-	- -	-
5	9.898	36.408 4.222	0.12	7.150	19.788	1.356	0.618	- 6.114	2.120 3.994	-	- -	-
6	1.949	17.814 4.222	0.24	0.524	37.412	0.690	0.987	1.924 -	- -	-	- -	-

Table of values		
Sye	2.593	$inch^3$
Mnyo	10805.58	lbfft
Resistance factor	0.90	
Unity check	0.24	-

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

Table of values		
Sigma,ex	117.225	ksi
Kt	1.00	
Lt	2 ft 6.675 in	ft
Sigma,t	106.431	ksi
Cb	1.00	
Sfy	3.682	$inch^3$
Fcre	450.943	ksi

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

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

Shear Strength

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

Shear force Vx

Element ID	Aw [inch ^{2}]	Vn [lbf]
1	0.163	4884.71
2	0.180	2657.78
3	0.163	4884.71
4	0.163	4884.71
5	0.180	2657.78
6	0.163	4884.71

Table of values		
Vn,x	24854.41	lbf
Resistance factor	0.95	
Unity check	0.02	-

Shear force Vy

Element ID	Aw [inch ^{2}]	Vn [lbf]
1	0.035	1062.15
2	0.827	12222.90
3	0.035	1062.15
4	0.035	1062.15
5	0.827	12222.90
6	0.035	1062.15

Table of values		
Vn,y	28694.39	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	18638.41	lbfft
Vny	28694.39	lbf
Mnyo	10805.58	lbfft
Vnx	24854.41	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check (Mx, Vy) = sqrt(0.15+0.00) = 0.39

Unity check (My, Vx) = sqrt(0.06+0.00) = 0.24

Buckling check

According to article E2 and formula (E2-1)

Flexural Buckling Strength

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

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	13 3/8	2 5/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	13 3/8	2 5/8	ft
Slenderness	49.42	10.90	
Flexural Buckling stress Fcre	117.225	2408.851	ksi

Torsional (-Flexural) Buckling Strength

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

Table of values		
Sigma,ex	117.225	ksi
Sigma,ey	2408.851	ksi
Kt	1.00	
Lt	2 5/8	ft
Sigma,t	106.431	ksi
Sigma,TF	104.703	ksi
Torsional (-Flexural) buckling stress Fcre	104.703	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	1.949	40.942 40.942	1.00	0.430	30.691	1.155	0.701	1.366 -	- -	-	- -	-
2	9.898	40.942 40.942	1.00	4.000	11.071	1.923	0.461	4.558 -	- -	-	- -	-
3	1.949	40.942 40.942	1.00	0.430	30.691	1.155	0.701	1.366 -	- -	-	- -	-
4	1.949	40.942 40.942	1.00	0.430	30.691	1.155	0.701	1.366 -	- -	-	- -	-
5	9.898	40.942 40.942	1.00	4.000	11.071	1.923	0.461	4.558 -	- -	-	- -	-
6	1.949	40.942 40.942	1.00	0.430	30.691	1.155	0.701	1.366 -	- -	-	- -	-

Table of values		
Fe	104.703	ksi
lambda, c	0.69	
Fn	40.942	ksi
Ae	1.483	inch ^{2}
Pn	60712.28	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	1.949	0.030 0.030	1.00	0.430	30.691	0.032	1.000	1.949 -	- -	-	- -	-
2	9.898	0.030 0.030	1.00	4.000	11.071	0.052	1.000	9.898 -	- -	-	- -	-
3	1.949	0.030 0.030	1.00	0.430	30.691	0.032	1.000	1.949 -	- -	-	- -	-
4	1.949	0.030 0.030	1.00	0.430	30.691	0.032	1.000	1.949 -	- -	-	- -	-
5	9.898	0.030 0.030	1.00	4.000	11.071	0.052	1.000	9.898 -	- -	-	- -	-
6	1.949	0.030 0.030	1.00	0.430	30.691	0.032	1.000	1.949 -	- -	-	- -	-

Table of values		
Mnx	18638.41	lbfft
Mny	10805.58	lbfft
Pn	60712.28	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.00+0.39+0.24 = 0.64 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER 3254 CHECK

AISI S100-16 LRFD Check

Member 3254	S1000S200-54	A1008 grade 54	LRFD-Ult (auto)	0.70

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

The critical check is on position 6.24 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	-6078.20	lbf
Vux	0.00	lbf
Vuy	-20.60	lbf
Mut	0.03	lbfft
Mux	1499.66	lbfft
Muy	-73.22	lbfft

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

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.484	-32.657 -36.932	-	-	-	-	-	- -	- -	-	- -	-
3	1.717	-37.933 -37.933	-	-	-	-	-	- -	- -	-	- -	-
5	9.717	48.999 -36.932	0.75	18.295	16.274	1.735	0.503	- 4.890	1.303 1.486	-	- -	-
7	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
9	0.484	48.999 44.723	0.91	0.461	165.766	0.544	1.000	0.223 -	- -	-	- -	-

Table of values		
Sxe	1.690	inch ^{3}
Mnxo	7041.71	lbfft
Resistance factor	0.90	
Unity check	0.24	-

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

Table of values		
Lltb	2 ft 0.000 in	ft
Sigma,ey	inf	ksi
Kt	1.00	
Lt	2 ft 0.000 in	ft
Sigma,t	297.800	ksi
Cb	1.02	
Sfx	2.270	$inch^3$
Fcre	inf	ksi

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

Distortional Buckling Strength
According to article F4 and formula F4.3-1.

Table of values		
Sfy	2.270	$inch^3$
My	9458.42	lbfft
L	0.000 in	ft
Beta	1.00	
k,phi,fe	inf	lbf
k,phi,we	inf	lbf
k,phi	0.00	lbf
k,phi,fg	inf	$inch^2$
k,phi,wg	inf	$inch^2$

Table of values		
Fd	-nan(ind)	ksi
Sf	2.270	$inch^3$
Mcrd	-nan(ind)	lbfft
Lambda,d	-nan(ind)	
Cyd	3.00	
Mp	11469.12	lbfft
cyt	3.00	
Myc	9458.42	lbfft
Myt3	0.00	lbfft
Mn	11245.71	lbfft
Resistance factor	0.90	
Unity check	0.15	-

Data		
Lm	0.000 in	ft
Lcr	1 ft 8.136 in	ft
h0	10.000	inch
Ixf	0.003	$inch^4$
Iyf	0.049	$inch^4$
Ixyf	-0.007	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.691	inch
hxf	-1.239	inch
Af	0.135	$inch^2$
y0f	0.068	inch
Ksi,web	2.00	

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

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

Nominal Flexural Strength

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.484	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	1.717	24.451 -45.395	1.86	56.331	1604.838	0.123	1.000	- 1.717	0.354 0.248	-	- -	-
5	9.717	29.056 29.056	1.00	4.000	3.558	2.858	0.323	3.139 -	- -	-	- -	-
7	1.717	24.451 -45.395	1.86	56.331	1604.838	0.123	1.000	- 1.717	0.354 0.248	-	- -	-
9	0.484	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.216	inch ^{3}
Mnyo	901.35	lbfft
Resistance factor	0.90	
Unity check	0.09	-

Lateral-Torsional Buckling Strength

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

Table of values		
Sigma,ex	186.523	ksi
Kt	1.00	
Lt	2 ft 0.000 in	ft
Sigma,t	297.800	ksi
Cs	1.00	
CTF	1.00	
Sfy	0.886	inch ^{3}
j	6.363	inch
Fcre	2556.038	ksi

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

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

Shear Strength

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

Shear force Vy

Element ID	Aw [inch ^{2}]	Vn [lbf]
3	0.000	0.00
5	0.550	2612.99
7	0.000	0.00

Table of values		
Vn,y	2612.99	lbf
Resistance factor	0.95	
Unity check	0.01	-

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

Table of values		
Mnxo	7041.71	lbfft
Vny	2612.99	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.06+0.00) = 0.24

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

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.484	50.000 50.000	1.00	0.430	154.489	0.569	1.000	0.223 -	- -	-	- -	-
3	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
5	9.717	50.000 50.000	1.00	4.000	3.558	3.749	0.251	2.440 -	- -	-	- -	-
7	1.717	50.000 50.000	1.00	2.743	78.151	0.800	0.906	1.556 -	0.359 1.197	30.83	0.001 0.001	0.223
9	0.484	50.000 50.000	1.00	0.430	154.489	0.569	1.000	0.223 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.380	inch ^{2}
Pno	18985.89	lbf
Resistance factor	0.85	
Unity check	0.38	-

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	2	ft
Effective Length factor K	1.00	1.00	
Effective Length	12	2	ft
Slenderness	39.18	35.83	
Flexural Buckling stress Fcre	186.523	222.962	ksi

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

Table of values		
Sigma,ex	186.523	ksi
Sigma,ey	222.962	ksi
Kt	1.00	
Lt	2	ft
Sigma,t	297.800	ksi
Sigma,TF	167.916	ksi
Torsional (-Flexural) buckling stress Fcre	167.916	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	0.484	44.141 44.141	1.00	0.430	154.489	0.535	1.000	0.297 -	- -	-	- -	-
3	1.717	44.141 44.141	1.00	2.957	84.248	0.724	0.962	1.651 -	0.506 1.145	32.81	0.001 0.001	0.297
5	9.717	44.141 44.141	1.00	4.000	3.558	3.522	0.266	2.586 -	- -	-	- -	-
7	1.717	44.141 44.141	1.00	2.957	84.248	0.724	0.962	1.651 -	0.506 1.145	32.81	0.001 0.001	0.297
9	0.484	44.141 44.141	1.00	0.430	154.489	0.535	1.000	0.297 -	- -	-	- -	-

Table of values		
Fe	167.916	ksi
lambda, c	0.55	
Fn	44.141	ksi
Ae	0.407	inch^2
Pn	17968.95	lbf
Resistance factor	0.85	
Unity check	0.40	-

Distortional Buckling Strength

According to article E4 and formula (E4.1-1).

Table of values		
Py	42103.51	lbf
L	0.000 in	ft
k,phi,fe	inf	lbf
k,phi,we	96.33	lbf
k,phi	0.00	lbf
k,phi,fg	inf	inch^2
k,phi,wg	inf	inch^2
Fd	-nan(ind)	ksi
Pcrd	-nan(ind)	lbf
Lambda,d	-nan(ind)	
Pn	42103.51	lbf
Resistance factor	0.85	
Unity check	0.17	-

Data		
Lm	0.000 in	ft
Lcr	1 ft 10.238 in	ft
h0	10.000	inch
Ixf	0.003	inch^4
Iyf	0.049	inch^4
Ixyf	0.007	inch^4
Cwf	0.000	inch^6
Jf	0.000	inch^4
x0f	0.691	inch
hxf	-1.239	inch
Af	0.135	inch^2
y0f	-0.068	inch

Number of compressed flanges: 2

Critical flange contains Initial shape parts: 2, 3, 1

Combined Compressive Axial Load and Bending

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

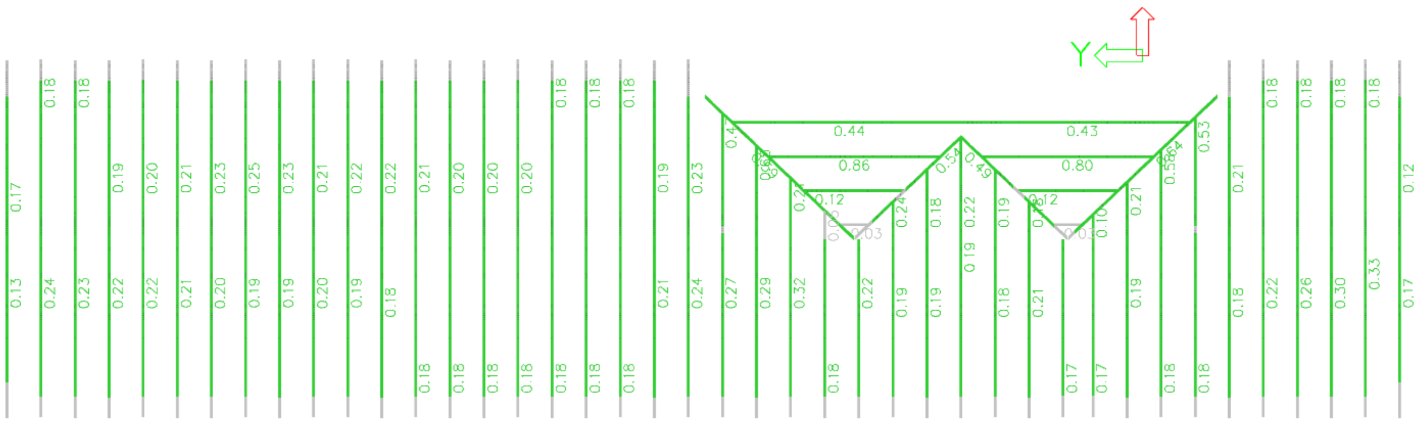
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.484	7.218 7.218	1.00	0.430	154.489	0.216	1.000	0.484 -	- -	-	- -	-
3	1.717	7.218 7.218	1.00	3.430	97.717	0.272	1.000	1.717 -	0.858 0.858	81.14	0.000 0.001	0.484
5	9.717	7.218 7.218	1.00	4.000	3.558	1.424	0.594	5.768 -	- -	-	- -	-
7	1.717	7.218 7.218	1.00	3.430	97.717	0.272	1.000	1.717 -	0.858 0.858	81.14	0.000 0.001	0.484
9	0.484	7.218 7.218	1.00	0.430	154.489	0.216	1.000	0.484 -	- -	-	- -	-

Table of values		
Centerline shift ex	0.145	inch
Centerline shift ey	0.000	inch
Additional moment Mx	0.00	lbfft
Additional moment My	-73.22	lbfft
Mnx	7041.71	lbfft
Mny	901.35	lbfft
PEx	157065.66	lbf
PEy	187749.43	lbf
Alfa x	0.96	
Alfa y	0.97	
Cmx	0.85	
Cmy	0.85	
Pn	17968.95	lbf
Pno	18985.89	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

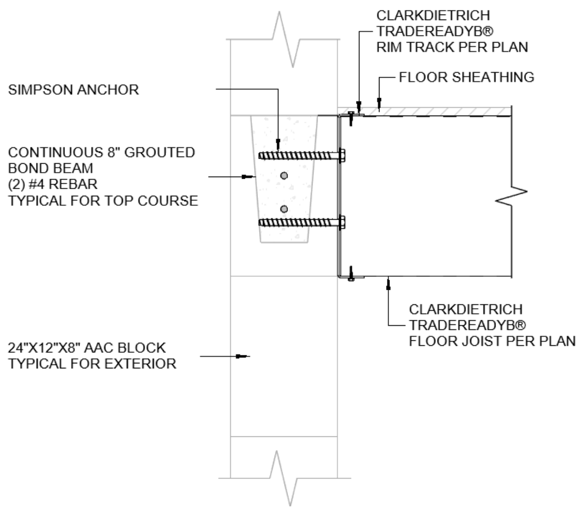
Unity check = 0.40+0.21+0.08 = 0.69 - (H1.2-1)
Unity check = 0.38+0.24+0.09 = 0.70 -

The member satisfies the check !

Unity check



2.5 CONNECTIONS DESIGN
2.5.1 FLOOR JOIST TO AAC WALL CONNECTION DESIGN



Floor joist to AAC wall connection sketch

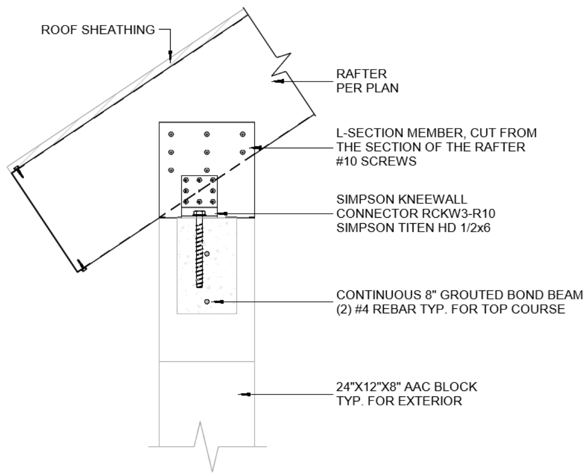
Maximum support force is 2865 lb. For connections use (2) Mechanical Anchors Simpson Titen HD 1/2"X6"

Titen HD Tension and Shear Strength Design Data for
the Soffit of Normal-Weight or Sand-Lightweight Concrete over Steel Deck[1,6,7]

Characteristic	Symbol	Units	Nominal Anchor Diameter, d_a (in.)									
			Lower Flute						Upper Flute			
			Figure 2		Figure 1				Figure 2		Figure 1	
			¼		⅜		½		¼		⅜	½
Nominal Embedment Depth	h_{nom}	in.	1⅝	2½	1⅞	2½	2	3½	1⅝	2½	1⅞	2
Effective Embedment Depth	h_{ef}	in.	1.19	1.94	1.23	1.77	1.29	2.56	1.19	1.94	1.23	1.29
Pullout Resistance, concrete on steel deck (cracked)[2,3,4]	$N_{p,deck,cr}$	lb.	420	535	375	870	905	2,040	655	1,195	500	1,700
Pullout Resistance, concrete on steel deck (uncracked)[2,3,4]	$N_{p,deck,uncr}$	lb.	995	1,275	825	1,905	1,295	2,910	1,555	2,850	1,095	2,430
Steel Strength in Shear, concrete on steel deck[5]	$V_{sa,deck}$	lb.	1,335	1,745	2,240	2,395	2,435	4,430	2,010	2,420	4,180	7,145
Steel Strength in Shear, Seismic	$V_{sa,deck,eq}$	lb.	870	1,135	1,434	1,533	1,565	2,846	1,305	1,575	2,676	4,591

2 * 2435 lb = 4780 lb 4780 lb > 2865 lb **Connection is OK!**

2.5.2 ROOF RAFTER TO AAC WALL CONNECTION DESIGN



Floor joist to AAC wall connection sketch

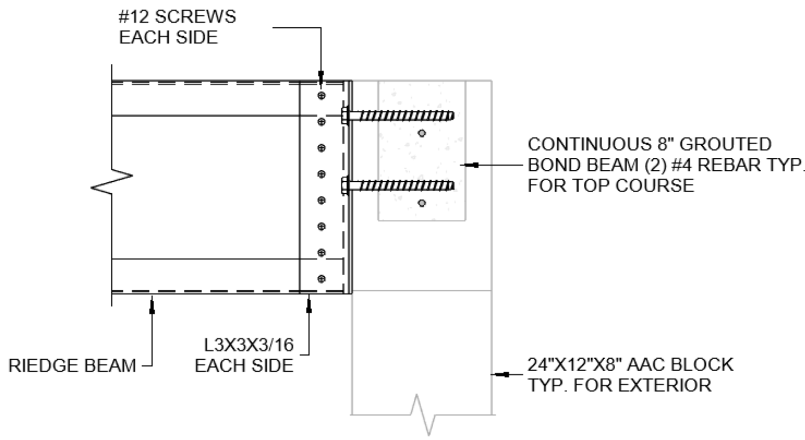
Maximum support force is 1550 lb. For connections use Simpson Self-Drilling X Metal Screw #10

X Metal Screw — Cold-Formed Steel Connection Loads

Size (in.)	Model No.	Nominal Dia. (in.)[7]	Load Description	Reference Shear (lb.)						Reference Pull-Over (lb.)						Reference Pull-Out (lb.)					
				Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]					
				27	33	43	54	68	97	27	33	43	54	68	97	27	33	43	54	68	97
				(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)
#10-16 x ¾	X34B1016	0.190	ASD	175	235	360	540	540	540	330	400	475	645	925	975	71	87	129	200	270	445
#10-16 x 1	XQ1S1016 X1S1016		LRFD	280	375	570	810	810	810	525	640	755	1,035	1,465	1,465	114	139	205	320	430	715
			Nominal strength	400	535	815	1,290	1,290	1,290	805	990	1,160	1,585	2,260	2,695	174	215	315	490	660	1,095
#12-14 x 1	XQ1S1214 X1S1214	0.216	ASD	176	235	385	595	840	840	295	375	525	785	1,045	1,210	74	96	147	215	325	500
			LRFD	280	375	610	950	1,265	1,265	470	600	835	1,255	1,670	1,875	117	154	235	340	520	795
			Nominal strength	400	535	870	1,350	2,135	2,135	720	920	1,285	1,925	2,565	2,965	180	235	360	520	800	1,220

Minimum number of screw
1550 lb /810 lb = 1.91 pcs = **2** pcs

2.5.3 ROOF BEAM TO AAC WALL CONNECTION DESIGN



Floor joist to AAC wall connection sketch

Maximum support force is 8500 lb. For connections use (4) Mechanical Anchors Simpson Titen HD 1/2"X6"

Titen HD Tension and Shear Strength Design Data for
the Soffit of Normal-Weight or Sand-Lightweight Concrete over Steel Deck[1,6,7]

Characteristic	Symbol	Units	Nominal Anchor Diameter, d_a (in.)									
			Lower Flute						Upper Flute			
			Figure 2		Figure 1				Figure 2		Figure 1	
			¼		⅜		½		¼		⅜	½
Nominal Embedment Depth	h_{nom}	in.	1⅝	2½	1⅞	2½	2	3½	1⅝	2½	1⅞	2
Effective Embedment Depth	h_{ef}	in.	1.19	1.94	1.23	1.77	1.29	2.56	1.19	1.94	1.23	1.29
Pullout Resistance, concrete on steel deck (cracked)[2,3,4]	$N_{p,deck,cr}$	lb.	420	535	375	870	905	2,040	655	1,195	500	1,700
Pullout Resistance, concrete on steel deck (uncracked)[2,3,4]	$N_{p,deck,uncr}$	lb.	995	1,275	825	1,905	1,295	2,910	1,555	2,850	1,095	2,430
Steel Strength in Shear, concrete on steel deck[5]	$V_{sa,deck}$	lb.	1,335	1,745	2,240	2,395	2,435	4,430	2,010	2,420	4,180	7,145
Steel Strength in Shear, Seismic	$V_{sa,deck,eq}$	lb.	870	1,135	1,434	1,533	1,565	2,846	1,305	1,575	2,676	4,591

4 * 2435 lb = 9800 lb   9800 lb > 8500 lb **Connection is OK!**

Screw number calculation

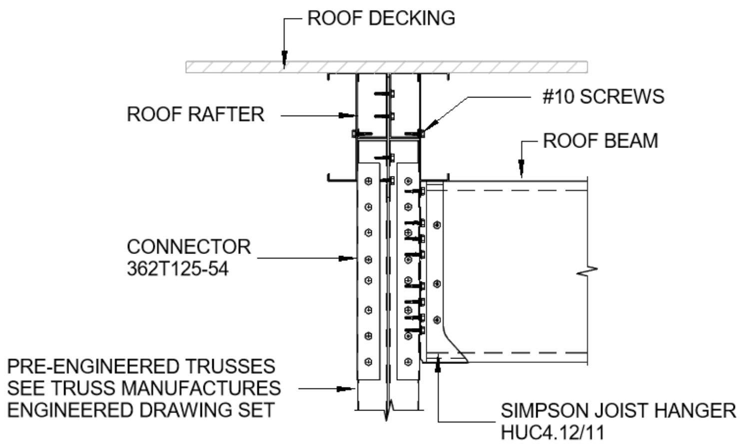
X Metal Screw — Cold-Formed Steel Connection Loads

Size (in.)	Model No.	Nominal Dia. (in.)[7]	Load Description	Reference Shear (lb.)						Reference Pull-Over (lb.)						Reference Pull-Out (lb.)					
				Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]					
				27	33	43	54	68	97	27	33	43	54	68	97	27	33	43	54	68	97
				(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)
#10-16 x ¾	X34B1016	0.190	ASD	175	235	360	540	540	540	330	400	475	645	925	975	71	87	129	200	270	445
#10-16 x 1	XQ1S1016 X1S1016		LRFD	280	375	570	810	810	810	525	640	755	1,035	1,465	1,465	114	139	205	320	430	715
			Nominal strength	400	535	815	1,290	1,290	1,290	805	990	1,160	1,585	2,260	2,695	174	215	315	490	660	1,095
#12-14 x 1	XQ1S1214 X1S1214	0.216	ASD	176	235	385	595	840	840	295	375	525	785	1,045	1,210	74	96	147	215	325	500
			LRFD	280	375	610	950	1,265	$\boxed{1,265}$	470	600	835	1,255	1,670	1,875	117	154	235	340	520	795
			Nominal strength	400	535	870	1,350	2,135	2,135	720	920	1,285	1,925	2,565	2,965	180	235	360	520	800	1,220

Minimum number of screw

9800 lb /1265 lb = 7.77 pcs = **8** pcs.

2.5.4 ROOF RAFTER TO AAC WALL CONNECTION DESIGN



Floor joist to AAC wall connection sketch

Maximum support force is 1210 lb. For connections use Simpson Self-Drilling X Metal Screw #10
For connection use Simpson joist hanger HUC4.12/11

X Metal Screw — Cold-Formed Steel Connection Loads

Size (in.)	Model No.	Nominal Dia. (in.)[7]	Load Description	Reference Shear (lb.)						Reference Pull-Over (lb.)						Reference Pull-Out (lb.)					
				Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]					
				27	33	43	54	68	97	27	33	43	54	68	97	27	33	43	54	68	97
				(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)
#10-16 x ¾	X34B1016	0.190	ASD	175	235	360	540	540	540	330	400	475	645	925	975	71	87	129	200	270	445
#10-16 x 1	XQ1S1016 X1S1016		LRFD	280	375	570	810	810	810	525	640	755	1,035	1,465	1,465	114	139	205	320	430	715
			Nominal strength	400	535	815	1,290	1,290	1,290	805	990	1,160	1,585	2,260	2,695	174	215	315	490	660	1,095
#12-14 x 1	XQ1S1214 X1S1214	0.216	ASD	176	235	385	595	840	840	295	375	525	785	1,045	1,210	74	96	147	215	325	500
			LRFD	280	375	610	950	1,265	1,265	470	600	835	1,255	1,670	1,875	117	154	235	340	520	795
			Nominal strength	400	535	870	1,350	2,135	2,135	720	920	1,285	1,925	2,565	2,965	180	235	360	520	800	1,220

Minimum number of screw
1210 lb /810 lb = 1.49 pcs = **2** pcs

2.6 LINTEL DESIGN
2.6.1 LINTEL DESIGN FOR GARAGE DOOR

The design forces in the lintel are determined in the structural analysis soft SCIA

MASONRY LINTEL ANALYSIS AND DESIGN TO MSJC 2013

Clear span of opening, L		12	ft
Dead load of wall		45	lb/ft²
Dead load of roof		40	lb/ft
Roof Live load		60	lb/ft
Load combination	1*D+1*L		

Shear force diagram Vz

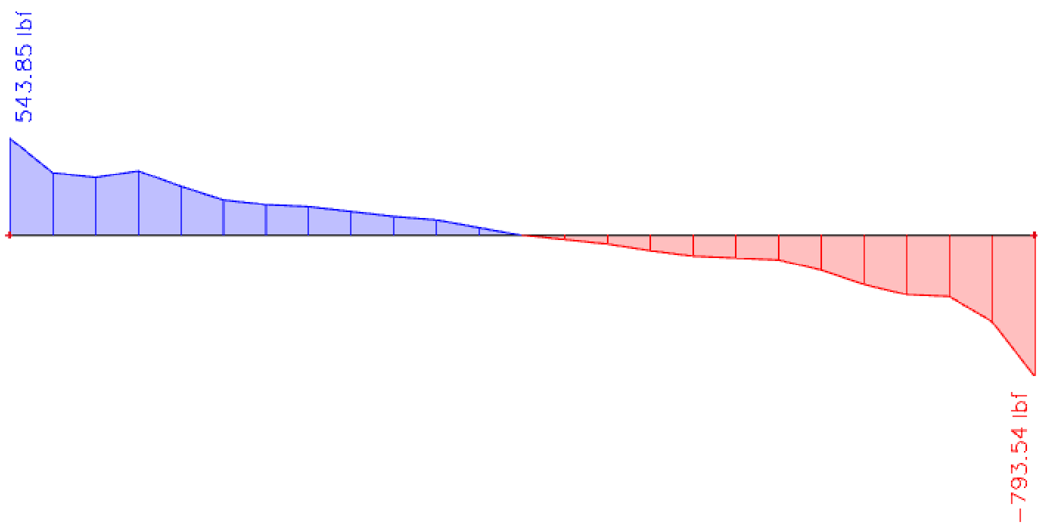
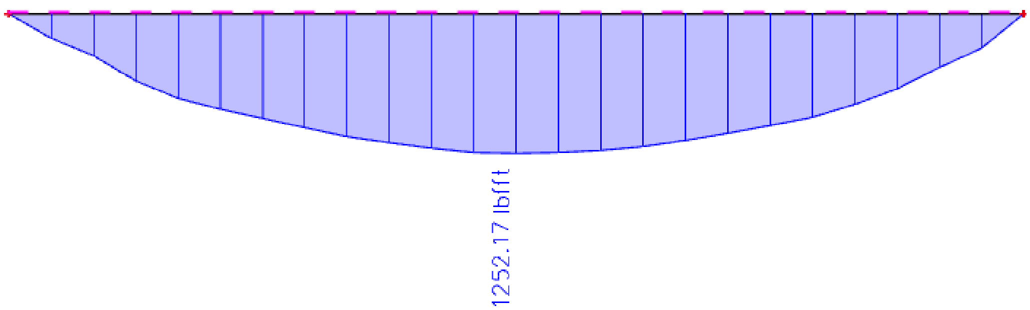


Diagram of moment My



Design forces

Reactions

R =Ra = Rb R = 800 lb **Mmax =** 1250 lb*ft **Vmax =** 800 lb

Masonry details

Masonry type;		AAC	
Density of masonry unit;	γ =	44.95 lb/ft3	
Compressive strength of mas. ur	f'_{AAC} =	1500 psi	
Net compressive strength of masonry grout (Table 2)	f'_m =	1900 psi	
Modulus of elasticity			(4.2.2.2.1)
$E_m = 900 * f'_m$	E_m =	1710000 psi	
Allowable flexural tensile stress	F_t =	106 psi	(8.2.4.2)

Section properties

Modular ratio	$n = E_s / E_m$ =	16.96
Section width	b =	4 in
Section depth;	h =	8 in
Net shear area;	$A_{nv} = b * h$ =	32.00 in²
Section modulus;	$S = bh^2/6$ =	42.67 in³
Depth to tension reinforcement	d =	4.5 in
Moment of inertia of net section		
	$I_n = bh^3/12$ =	170.67 in^4
Cracked moment of inertia		
	$B = b/ (n*A_s)$ =	0.590
	$kd = (\sqrt{(2d*B+1)}-1)/B$ =	2.563
	$I_{cr} = b*(k*d^3)/3 + n * A_s * (d - k*d)^2$ =	47.900 in^4

Flexure design (Chapter 8)

Tension reinforcement		2	x #4 Bar	
Area of tension reinforcement	A_s =	0.4	in²	
Reinforcement ratio;	$\rho_{ratio} = A_s / (b*_B)$ =	0.02222		
Neutral axis facto				
	$k = \sqrt{(2*\rho_{ratio}*n + (\rho_{ratio}*n)^2)} - \rho_{ratio}*n$ =	0.56958		
Lever arm factor;	$j = 1- k/3$=	0.8101		
Cracking moment;	$M_{cr} = 2.5* F_t * S$=	0.94	kip*ft	
Design bending moment	M =	1.25	kip*ft	
Tensile stress in reinforcement;				
	$f_s = M / (A_s * j * d)$ =	10286.29	psi	
Allowable tensile stress in reinf.	F_s =	32000	psi	(8.3.3.1)
Reinforcement stress ratio	f_s / F_s =	**0.32**	**PASS - Allowable tensile stress exceeds tensile stress due to flexure**	

Compressive stress in masonry

	$f_b = 2 * M / (j * k * b * d^2)$ =	802.64	psi	
Allowable stress in masonry;				(8.3.4.2.2)
	$F_b = 0.45 * f'_m$ =	855	psi	
Masonry stress ratio;	f_b / F_b =	**0.939**	**PASS - Allowable compressive stress exceeds compressive stress due to flexure**	

Shear design (Chapter 8)

Design shear force;	V =	800	lb	
Depth of shear area	d_v =	4.5	in	
Moment shear relationship, M/Vd;				
	Assume; M/Vdratio =	1		
Shear stress;	$f_v = V / A_{nv}$ =	25.00	psi	(8-24)
Allowable masonry shear stress				
	$F_v = 1/2[(4-1.75 (M/V_{dratio}))* \sqrt{f'_m}]$ =	49.04	psi	(8-29)
Masonry shear stress ratio	f_v / F_v =	**0.51**	**PASS - Allowable shear stress exceeds shear stress in masonry**	

Deflection (5.2.1.4)

Deflection (5.2.1.4) Dead + Live

Moment at midspan under deflection loads;

M_a = 1250 lb*ft

Effective moment of inertia $I_{eff} \leq I_n$

$I_{eff} = I_n*(M_{cr}/M_a)^3+I_{cr}(1-(M_{cr}/M_a)^3),I_n)$= 100.48 in^4 (5-1)

$I_{eff} \leq I_n$ 100.47895

$\delta_{inst} = (5M\ h^2)/(48\ E_m\ I_n)$= **0.00034** in (9-29)

Deflection limit; L/600 **0.24** in

PASS - Deflection limit exceeds deflection of lintel

2.6.2 LINTEL DESIGN FOR EXTERIOR WALL DOOR

MASONRY LINTEL ANALYSIS AND DESIGN TO MSJC 2013

Clear span of opening, L 16 ft

Dead load 370 lb/ft

Live load 160 lb/ft

Load combination 1*D+1*L

Total load, q 530 lb/ft

Analysis results

Reactions

$R = R_a = R_b$ R = 4240 lb **M_{max} =** 16960 lb*ft **V_{max} =** 4240 lb

	x=0		x=L/2		x=L	
M =	0	lb*ft	16960	lb*ft	0	lb*ft
V =	4240	lb	0	lb	-4240	lb

Masonry details

Masonry type; AAC

Density of masonry unit; γ = 140 lb/ft3

Compressive strength of mas. ur f'_{AAC} = 870 psi

Net compressive strength

of masonry grout (Table 2) f'_m = 1900 psi

Modulus of elasticity (4.2.2.2.1)

$E_m = 900 * f'_m$ E_m = 1710000 psi

Allowable flexural tensile stress F_t = 106 psi (8.2.4.2)

Reinforcement details

Allowable tensile stress;	F_s =	32000	psi
Modulus of elasticity of steel	E_s =	29000000	psi

Section properties

Modular ratio	$n = E_s / E_m$ =	16.96	
Section width	b =	4	in
Section depth;	h =	32	in
Net shear area;	$A_{nv} = b * h$ =	128.00	in²
Section modulus;	$S = bh^2/6$ =	682.67	in³
Depth to tension reinforcement	d =	28.5	in
Moment of inertia of net section			
	$I_n = bh^3/12$ =	10922.667	in^4
Cracked moment of inertia			
	$B = b/ (n*A_s)$ =	0.590	
	$kd = (\sqrt{(2d*B+1)}-1)/B$ =	8.281	
	$I_{cr} = b*(k*d^3)/3 + n * A_s * (d - k*d)^2$ =	3530.359	in^4

Flexure design (Chapter 8)

Tension reinforcement		2	x #4 Bar	
Area of tension reinforcement	A_s =	0.4	in²	
Reinforcement ratio;	$\rho_{ratio} = A_s / (b*B)$ =	0.00351		
Neutral axis facto				
	$k = \sqrt{(2*\rho_{ratio}*n + (\rho_{ratio}*n)^2)}-\rho_{ratio}*n$ =	0.29057		
Lever arm factor;	$j = 1- k/3$=	0.9031		
Cracking moment;	$M_{cr} = 2.5* F_t * S$=	15.075556	kip*ft	
Design bending moment	M =	16.96	kip*ft	
Tensile stress in reinforcement;				
	$f_s = M / (A_s * j * d)$ =	19767.21	psi	
Allowable tensile stress in reinf.	F_s =	32000	psi	(8.3.3.1)
Reinforcement stress ratio	f_s / F_s =	**0.62**	**PASS - Allowable tensile stress exceeds tensile stress due to flexure**	

Compressive stress in masonry

	$f_b = 2 * M / (j * k * b * d^2)$ =	477.40	psi	
Allowable stress in masonry;				(8.3.4.2.2)
	$F_b = 0.45 * f'_m$ =	855	psi	
Masonry stress ratio;	f_b / F_b =	**0.558**	**PASS - Allowable compressive stress exceeds compressive stress due to flexure**	

Shear design (Chapter 8)

Design shear force;	$V =$	4240	lb	
Depth of shear area	$d_v =$	28.5	in	
Moment shear relationship, M/Vd;				
	Assume; M/Vdratio =	1		
Shear stress;	$f_v = V / A_{nv} =$	33.13	psi	(8-24)
Allowable masonry shear stress				
	$F_v = 1/2[(4-1.75\ (M/V_{dratio}))^* \sqrt{f'_m}] =$	49.04	psi	(8-29)
Masonry shear stress ratio	$f_v / F_v =$	**0.68**	**PASS - Allowable shear stress exceeds shear stress in masonry**	

Deflection (5.2.1.4)

Deflection (5.2.1.4)	Dead + Live			
Moment at midspan under deflection loads;				
	Ma =	16960	lb*ft	
Effective moment of inertia	$I_{eff} \leq I_n$			
	$I_{eff} = I_n^*(M_{cr}/M_a)^3+I_{cr}(1-(M_{cr}/M_a)^3),I_n)=$	8722.21	in^4	(5-1)
	Ieff ≤ In	8722.213		
	$\delta_{inst} = (5M\ h^2)/(48\ E_m\ I_n)=$	**0.00116**	in	(9-29)
Deflection limit;	L/600	**0.32**	in	
			PASS - Deflection limit exceeds deflection of lintel	

2.7 LATERAL ANALYSIS

2.7.1 BUILDING SEISMIC WEIGHT

Building seismic weight:

	Area, ft2	Weight, pcf	Weight, kip
Exterior AAC wall	2924	35.2	102.92
Exterior LGS wall	146	10.25	1.50
Interior LGS wall	483	10.25	4.95
Roof	5784	12.75	73.75
Floor	3035	17.25	52.35
Total seismic weight:			235.47

2.7.2 BASE SHEAR CALCULATION

Site parameters

Site class — D, Soil properties not known

Mapped acceleration parameters (Section 11.4.2)

at short period — S_S = **0.496**

at 1 sec period — S_1 = **0.235**

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} = **0.750**

at 1 sec period (Sect 21.4) — S_{D1alt} = **0.750**

Spectral response acceleration parameters

at short period (Sect 21.4) — $S_{MS} = 1.5 \times S_{DSalt}$ = **1.125**

at 1 sec period (Sect 21.4) — $S_{M1} = 1.5 \times S_{D1alt}$ = **1.125**

Seismic design category

Occupancy category (Table 1-1) — II

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

D

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

D

Seismic design category — D

Approximate fundamental period

Height above base to highest level of building — h_n = **12** ft

From Table 12.8-2:

Structure type	All other systems
Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**
Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$= **0.129** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.129** sec
Long-period transition period	T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	B_BUILDING_FRAME_SYSTEMS 3. Ordinary steel concentrically braced frames
Response modification factor (Table 12.2-1)	R = **3.25**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2)	$C_{s\_calc} = S_{DSalt} / (R / I_e)$ = **0.2308**
Maximum ((Eq 12.8-3))	$C_{s\_max} = S_a / (R / I_e)$ = **0.2308**
Minimum (Eq 9.5.5.2.1-3)	$C_{s\_min} = \max(0.044 \times S_{DSalt} \times I_e, 0.01)$ = **0.0330**
Seismic response coefficient	C_s = **0.2308**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure	W = **235.5** kips
Seismic response coefficient	C_s = **0.2308**
Seismic base shear (Eq 12.8-1)	$V = C_s \times W$ = **54.3** kips

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12)	$C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k)$
Lateral force induced at level i (Eq 12.8-11)	$F_x = C_{vx} \times V$

Minimum diaphragm forces (Section 12.10.1.1)

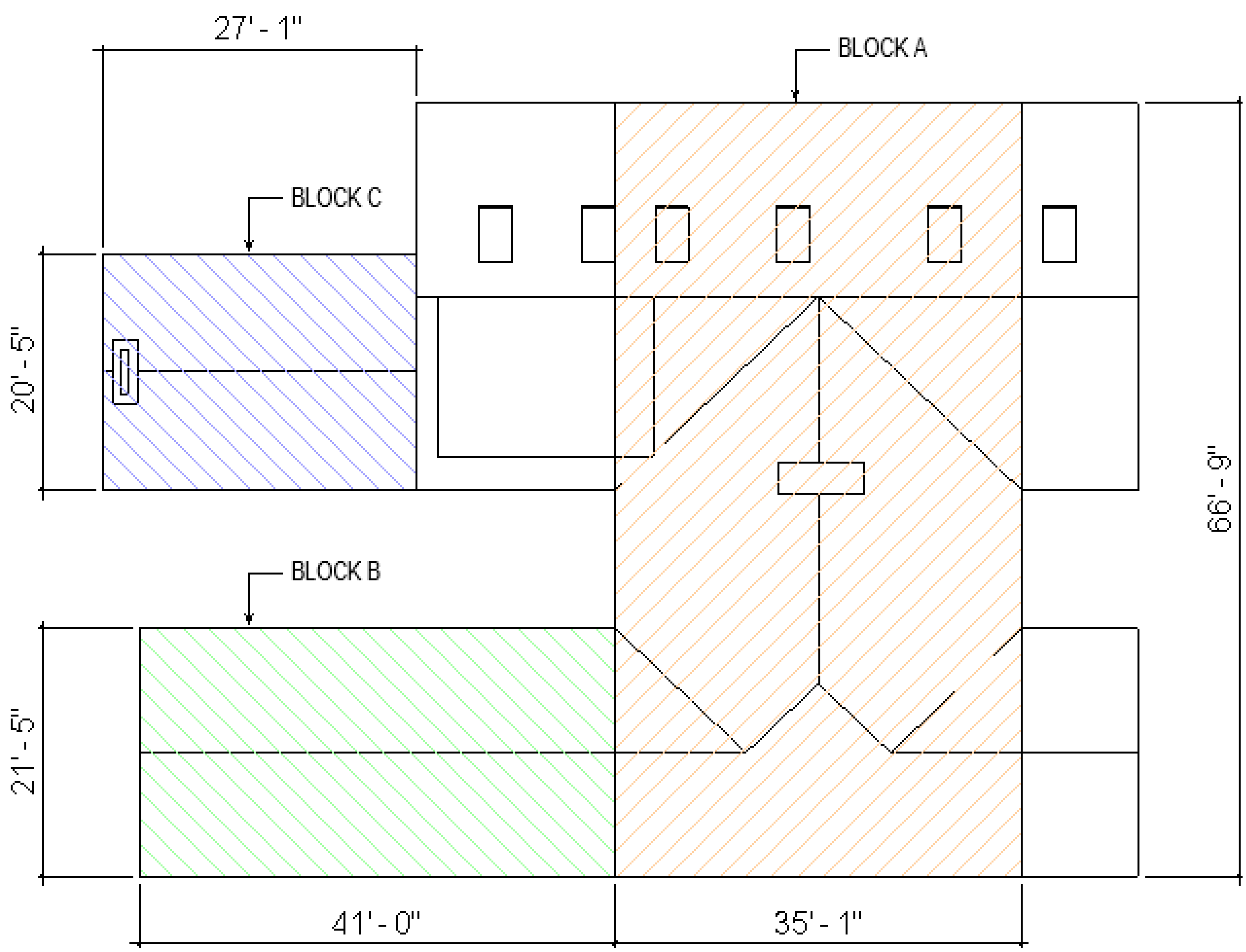
Calculated min. diaphragm force (Eq 12.10-1)	$F_{px} = \Sigma F_i \times w_{px} / \Sigma w_i$,(i=x to n)
	$F_{pxmin} = 0.2 \times S_{DS} \times I_e \times w_{px}$
	$F_{pxmax} = 0.4 \times S_{DS} \times I_e \times w_{px}$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x	Weight tributary to the diaphragm at Level i (kips), w_{px}	Minimum diaphragm force at Level i (kips), F_{px}
1	12.0;	235.5;	1.00;	1.000;	54.3	235.5	54.3

2.7.3 ROOF DIAPHRAGM CALCULATION

General scheme



2.7.3.1 BLOCK A

Considered part of the building's seismic weight:

	Area, ft2	Weight, pcf	Weight, kip
Exterior AAC wall	1726	35.2	60.76
Exterior LGS wall	146	10.25	1.50
Interior LGS wall	483	10.25	4.95
Roof	4096	12.75	52.22
Floor	2234	17.25	38.54
Total seismic weight:			157.96

Seismic base shear: V=Cs x W = 0.2308 x 157.96 = 36.45 kip

ROK-ON ROOF DIAPHRAGM CALCULATION

Considered part of the roof

Building length	$L =$	66.75	ft
Building width	$B =$	35.1	ft
Wall height	$h_w =$	12	ft
Roof height	$h_r =$	12	ft
Max wind pressure	$q_w =$	20	psf
Seismic diaphragm force	$S =$	36.45	kip

From ROK-ON™ Structural Insulated Sheathing Structural Properties

Nominal unit shear capacity for wind design	$V_{nw} =$	831	lb/ft
Nominal unit shear capacity for seismic design	$V_{ss} =$	943	lb/ft

The uniformly distributed wind load acting on the roof diaphragm by tributary area is:

Transverse direction

$W_t = q_w*(h_r + h_w/2) =$ 360 lb/ft

Longitudinal direction

$W_l = q_w*(h_r/2 + h_w/2) =$ 240 lb/ft

The maximum diaphragm wind shear in the roof diaphragm is:

Transverse direction

$V_t = (W_t * L) / 2B =$ 342.3077 lb/ft

Longitudinal direction

$V_L = (W_L * B) / 2L =$ 63.10112 lb/ft

Shear capacity for wind loading;

$V_w / \max(V_t; V_L) \leq 1$

$\max(V_t; V_L) =$ 342.31 lb/ft

$V_w = V_{nw} =$ 831.00 lb/ft

$V_w / \max(V_t; V_L) =$ **0.41** **PASS!**

The uniformly distributed seismic load acting on the roof diaphragm is:

Transverse direction

$q_{st} = S/L =$ 546.07 lb/ft

Longitudinal direction

$q_{sL} = S/B =$ 1038.46 lb/ft

The maximum diaphragm seismic shear in the roof diaphragm is:

Transverse direction

$S_t = S / 2B =$ 519.23 lb/ft

Longitudinal direction

$S_l = S / 2L =$ 273.03 lb/ft

Shear capacity for seismic loading;

$V_s / \max(S_t; S_l) \leq 1$

$\max(St; Sl) =$ 519.23 lb/ft

$V_s = V_{ss} =$ 943.00 lb/ft

$V_w / \max(V_t; V_l; T_t; T_l) =$ **0.55** **PASS!**

2.7.3.2 BLOCK B

Considered part of the building's seismic weight:

	Area, ft2	Weight, pcf	Weight, kip
Exterior AAC wall	486	35.2	17.11
Roof	1072	12.75	13.67
Floor	651	17.25	11.23
			42.00

Seismic base shear: V=Cs x W = 0.2308 x 42 = 9.69 kip

ROK-ON ROOF DIAPHRAGM CALCULATION

Considered part of the roof

Building length	$L =$	41	ft
Building width	$B =$	21.42	ft
Wall height	$h_w =$	12	ft
Roof height	$h_r =$	8	ft
Max wind pressure	$q_w =$	20	psf
Seismic diaphragm force	$S =$	9.69	kip

From ROK-ON™ Structural Insulated Sheathing Structural Properties

Nominal unit shear capacity for wind design	$V_{nw} =$	831	lb/ft
Nominal unit shear capacity for seismic design	$V_{ss} =$	943	lb/ft

The uniformly distributed wind load acting on the roof diaphragm by tributary area is:

Transverse direction

$W_t = q_w*(h_r + h_w/2) =$ 280 lb/ft

Longitudinal direction

$W_l = q_w*(h_r/2 + h_w/2) =$ 200 lb/ft

The maximum diaphragm wind shear in the roof diaphragm is:

Transverse direction

$V_t = (W_t * L) / 2B =$ 267.9739 lb/ft

Longitudinal direction

$V_L = (W_L * B) / 2L =$ 52.2439 lb/ft

Shear capacity for wind loading;

$V_w / \max(V_t; V_L) \leq 1$

$\max(V_t; V_L) =$ 267.97 lb/ft

$V_w = V_{nw} =$ 831.00 lb/ft

$V_w / \max(V_t; V_L) =$ **0.32** **PASS!**

The uniformly distributed seismic load acting on the roof diaphragm is:

Transverse direction

$q_{st} = S/L =$ 236.34 lb/ft

Longitudinal direction

$q_{sL} = S/B =$ 452.38 lb/ft

The maximum diaphragm seismic shear in the roof diaphragm is:

Transverse direction

$S_t = S / 2B =$ 226.19 lb/ft

Longitudinal direction

$S_l = S / 2L =$ 118.17 lb/ft

Shear capacity for seismic loading;

$V_s / \max(S_t; S_l) \leq 1$

$\max(St; Sl) =$ 226.19 lb/ft

$V_s = V_{ss} =$ 943.00 lb/ft

$V_w / \max(V_t; V_l; T_t; T_l) =$ **0.24** **PASS!**

2.7.3.3 BLOCK C

Considered part of the building's seismic weight:

	Area, ft2	Weight, pcf	Weight, kip
Exterior AAC wall	712	35.2	25.06
Roof	616	12.75	7.85
Floor	150	17.25	2.59
			35.50

Seismic base shear: V=Cs x W = 0.2308 x 35.5 = 8.19 kip

ROK-ON ROOF DIAPHRAGM CALCULATION

Considered part of the roof

Building length	L =	27.1	ft
Building width	B =	20.42	ft
Wall height	h_w =	12	ft
Roof height	h_r =	8	ft
Max wind pressure	q_w =	20	psf
Seismic diaphragm force	S =	8.19	kip

From ROK-ON™ Structural Insulated Sheathing Structural Properties

Nominal unit shear capacity for wind design	V_{nw} =	831	lb/ft
Nominal unit shear capacity for seismic design	V_{ss} =	943	lb/ft

The uniformly distributed wind load acting on the roof diaphragm by tributary area is:

Transverse direction

$W_t = q_w*(h_r + h_w/2) =$ 280 lb/ft

Longitudinal direction

$W_l = q_w*(h_r/2 + h_w/2) =$ 200 lb/ft

The maximum diaphragm wind shear in the roof diaphragm is:

Transverse direction

$V_t = (W_t * L) / 2B =$ 185.7982 lb/ft

Longitudinal direction

$V_L = (W_L * B) / 2L =$ 75.35055 lb/ft

Shear capacity for wind loading;

$V_w / \max(V_t; V_L) \leq 1$

$\max(V_t; V_L) =$ 185.80 lb/ft

$V_w = V_{nw} =$ 831.00 lb/ft

$V_w / \max(V_t; V_L) =$ **0.22** **PASS!**

The uniformly distributed seismic load acting on the roof diaphragm is:

Transverse direction

$q_{st} = S/L =$ 302.21 lb/ft

Longitudinal direction

$q_{sL} = S/B =$ 401.08 lb/ft

The maximum diaphragm seismic shear in the roof diaphragm is:

Transverse direction

$S_t = S / 2B =$ 200.54 lb/ft

Longitudinal direction

$S_l = S / 2L =$ 151.11 lb/ft

Shear capacity for seismic loading;

$V_s / \max(S_t; S_l) \leq 1$

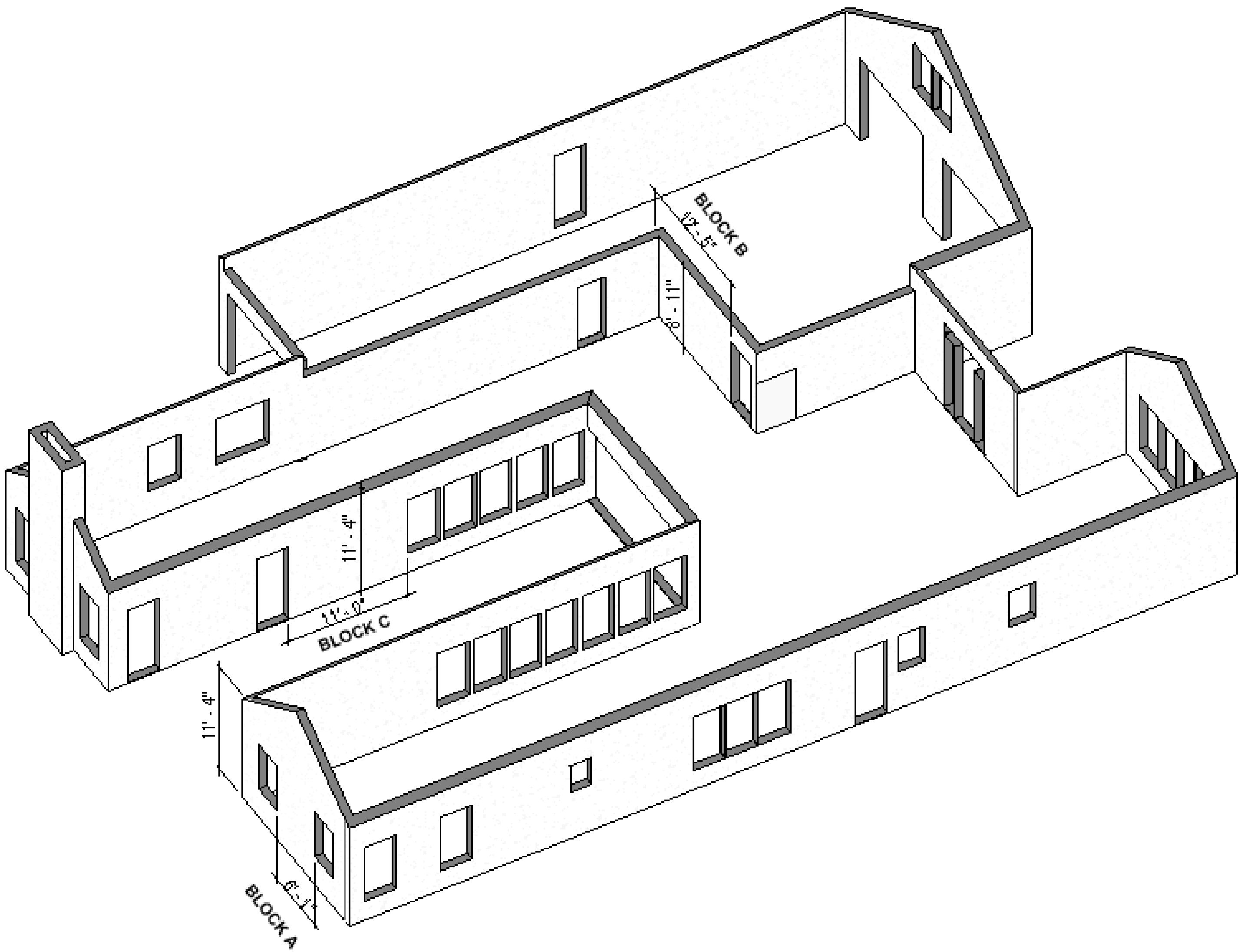
$\max(St; Sl) =$ 200.54 lb/ft

$V_s = V_{ss} =$ 943.00 lb/ft

$V_w / \max(V_t; V_l; T_t; T_l) =$ **0.21** **PASS!**

2.8 AAC WALLS CHECK

General scheme



2.8.1 BLOCK A

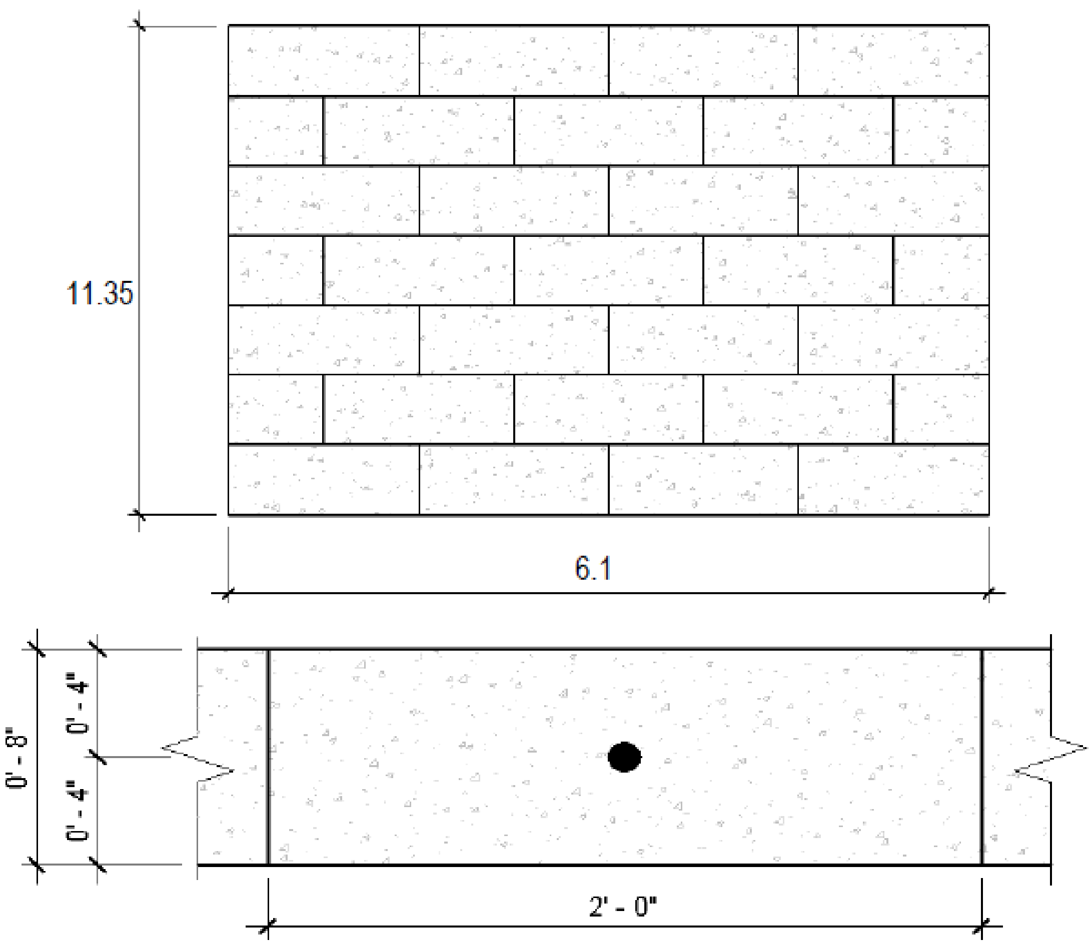
MASONRY AAC WALL PANEL DESIGN TO MSJC

Using the allowable stress design method

Masonry wall panel details:

Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads. The wall is fixed at the bottom and free at the top for in plane loads

Panel length	6.1	ft
Panel height	11.35	ft



Seismic properties:

Seismic design category	D		
Seismic importance factor (ASCE7 Table 1.5-2)	Ie	1	
Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)	SDS	0.464	
Redundancy factor, on out-of-plane load	ρE	1	

Masonry details:

Compressive strength of unit	f' AAC	870	psi
Density of masonry unit	ϒ block	30	lb/ft3
Width of masonry units	d b	8	in
Height of masonry units	hb	12	in
Length of masonry units	l b	24	in

From MSJC Clause 4.2.2 Elastic moduli

Modulus of elasticity for masonry (Eq 4.2.2.3.1)	E AAC	377246.2	psi
Shear modulus of masonry	Ev	150898.5	psi

Reinforcement details:

Yield strength of reinforcement	fy	60000	psi
Allowable tensile stress in reinforcement	Fs	32000	psi
Modulus of elasticity for reinforcement	Es	29000000	psi
Vertical reinforcement provided	No.4 bars at 96 in center		
Area of vertical reinforcement	As	0.2	in2

Lateral out-of-plane loads:

Wind load on panel	W	20	psf
Seismic load factor (ASCE7 12.11.1)	Fp	0.4	
Seismic load from wall	Ewall	28.2	psf

Lateral in-plane loads:

Dead load from above	DLabove	208	lb/ft
Live load from above	LLabove	320	lb/ft

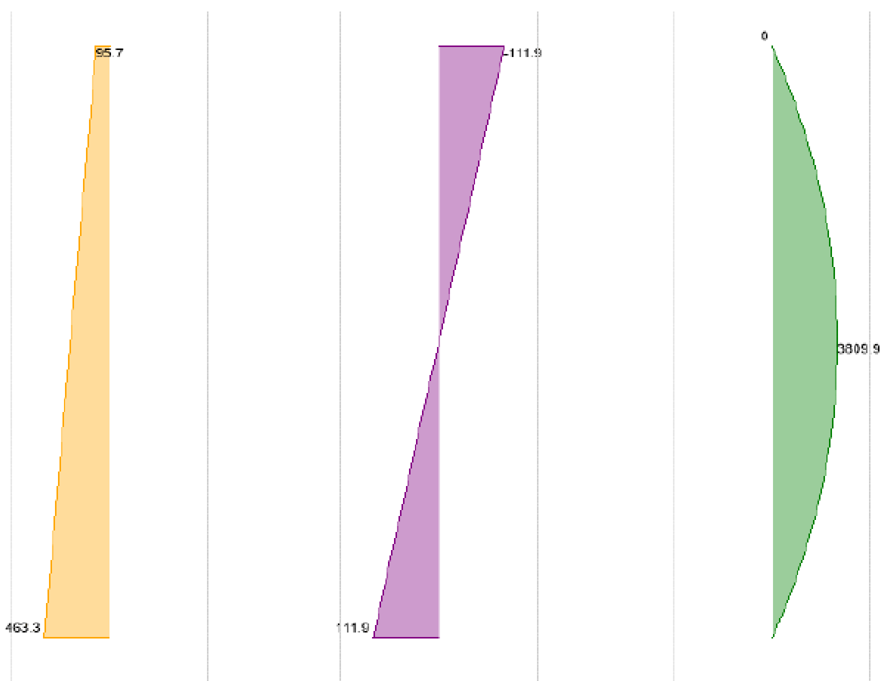
From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1	DL	0.034
Load combination no.2	DL + LL	0.045
Load combination no.3	DL + (LLr or SL or RL)	0.034
Load combination no.4	DL + 0.75 x LL + 0.75 x (LLr or SL or RL)	0.042
Load combination no.5	DL + 0.6 x W	0.127
Load combination no.6	DL + 0.7 x Eh + 0.7 x Ev	0.207
Load combination no.7	DL + 0.75 x LL + 0.45 x W + 0.75 x (LLr or SL or RL)	0.093
Load combination no.8	DL + 0.75 x LL + 0.525 x Eh + 0.525 x Ev + 0.75 x SL	0.152
Load combination no.9	0.6 x DL + 0.6 x W	0.13
Load combination no.10	0.6 x DL + 0.7 x Eh - 0.7 x Ev	0.216

Properties of masonry section:

Cross-sectional area	A	96	in2/ft
Moment of inertia	I	512	in4/ft
Section modulus	S	128	in3/ft
Radius of gyration	r	2.309	in

Consider wall at maximum moment location under load combination no.10



Maximum moment location		5.68	ft
Axial load at max moment loc. of panel	P	280	lb/ft
Compressive stress due to axial load	fa	2.92	psi
Slenderness ratio	h/r	58.976	<99
Allowable compressive stress due to axial load (Eq. 8-16)	Fa	178.903	psi
From MSJC Clause 8.2.3 - Axial compression and flexure			
	fa/Fa	0.02	<1
PASS - Allowable compressive stress exceeds compressive stress due to axial loads			

Bending moment at max. moment loc. of panel	M	3810	lb-in/ft
Compressive stress due to flexural load	fb	29.77	psi
Allowable compressive stress due to flexural load (Eq. 8-18)	Fb	290.00	psi
From MSJC Clause 8.2.3 - Axial compression and flexure			
	fb/Fb	0.10	<1
PASS - Allowable compressive stress exceeds compressive stress due to flexure			

From MSJC Clause 8.2.3 - Axial compression and flexure			
Axial compression and flexure	fa / Fa + fb / Fb	0.12	<1
PASS - Combined axial and flexural stresses are acceptable			

Net flexural tension	ft	26.85	psi
Allowable tensile stress due to flexural load	Ft	32	psi
From MSJC Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry			
	ft / Ft	0.84	<1
PASS - Allowable tensile stress exceeds tensile stress due to flexure			

Factored moment (Eq. 11-18)	Mu	2837.11	lb·in/ft
Factored axial load (Eq. 11-19)	Pu	3776	lb
Factored load from tributary floor areas	Puf	980	lb
Factored load from tributary roof areas	Pur	980	lb
Factored weight of wall area tributary to wall section under consideration	Puw	1816	lb
Deflection due to factored load	δu	0.31	in
Depth of an equivalent compression stress block at nominal strength	a	2.667	in
Nominal moment strength (Eq. 11-29)	Mn	3505.78	
	Mu/ϕMn	0.73	<1
PASS - Allowable nominal moment exceeds applied factored moment			
Seismic response coefficient	Cs	0.2308	kip
Effective seismic weight of the structure	W	235.5	kip
Total perimeter of the walls is parallel to the considered wall	p	75.5	ft
Total seismic base shear	Vut	54.35	kip
Local seismic base shear	Vul	4.39	kip
Total wind shear force	Vtw	19.03	kip
Local wind shear force	Vlw	1.54	kip
Local seismic base shear is biger than local wind shear force		Vul>Vlw	
Factored moment	Mu	57.70	k·ft
Factored axial load	Pu	24.60	kip
Nominal shear strength provided by AAC masonry (Eq. 11-13a)	VnAAC	16.41	kip
Nominal shear strength provided by shear reinforcement (Eq. 11-15)	Vns	4.575	kip
Nominal shear strength (Eq. 11-9)	Vn	20.99	kip
	Vul/Vn>1	0.21	<1
Allowable nominal shear strength exceeds applied local seismic base shear			
	Mu/Vu·dv	2.15	
From MSJC Clause 11.3.4.1.2 (d)	6.00 ·An·√f´AAC	103.64	>Vn
PASS - Allowable shear stress exceeds applied shear stress			

2.8.2 BLOCK B

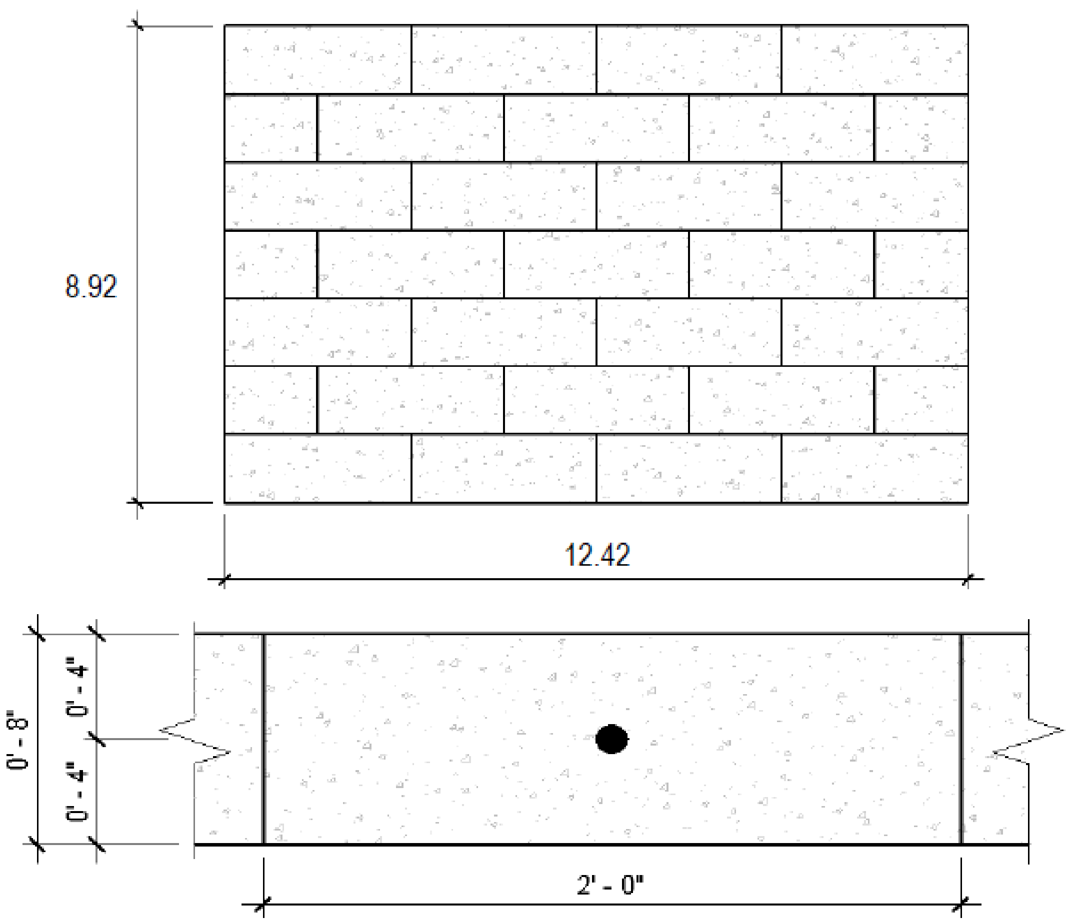
MASONRY AAC WALL PANEL DESIGN TO MSJC

Using the allowable stress design method

Masonry wall panel details:

Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads. The wall is fixed at the bottom and free at the top for in plane loads

Panel length	12.42	ft
Panel height	8.92	ft



Seismic properties:

Seismic design category	D		
Seismic importance factor (ASCE7 Table 1.5-2)	Ie	1	
Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)	SDS	0.464	
Redundancy factor, on out-of-plane load	ρE	1	

Masonry details:

Compressive strength of unit	f' AAC	870	psi
Density of masonry unit	ϒ block	30	lb/ft3
Width of masonry units	d b	8	in
Height of masonry units	hb	12	in
Length of masonry units	l b	24	in
From MSJC Clause 4.2.2 Elastic moduli			
Modulus of elasticity for masonry (Eq 4.2.2.3.1)	E AAC	377246.2	psi
Shear modulus of masonry	Ev	150898.5	psi

Reinforcement details:

Yield strength of reinforcement	fy	60000	psi
Allowable tensile stress in reinforcement	Fs	32000	psi
Modulus of elasticity for reinforcement	Es	29000000	psi
Vertical reinforcement provided	No.4 bars at 96 in center		
Area of vertical reinforcement	As	0.2	in2

Lateral out-of-plane loads:

Wind load on panel	W	20	psf
Seismic load factor (ASCE7 12.11.1)	Fp	0.4	
Seismic load from wall	Ewall	28.2	psf

Lateral in-plane loads:

Dead load from above	DLabove	208	lb/ft
Live load from above	LLabove	320	lb/ft

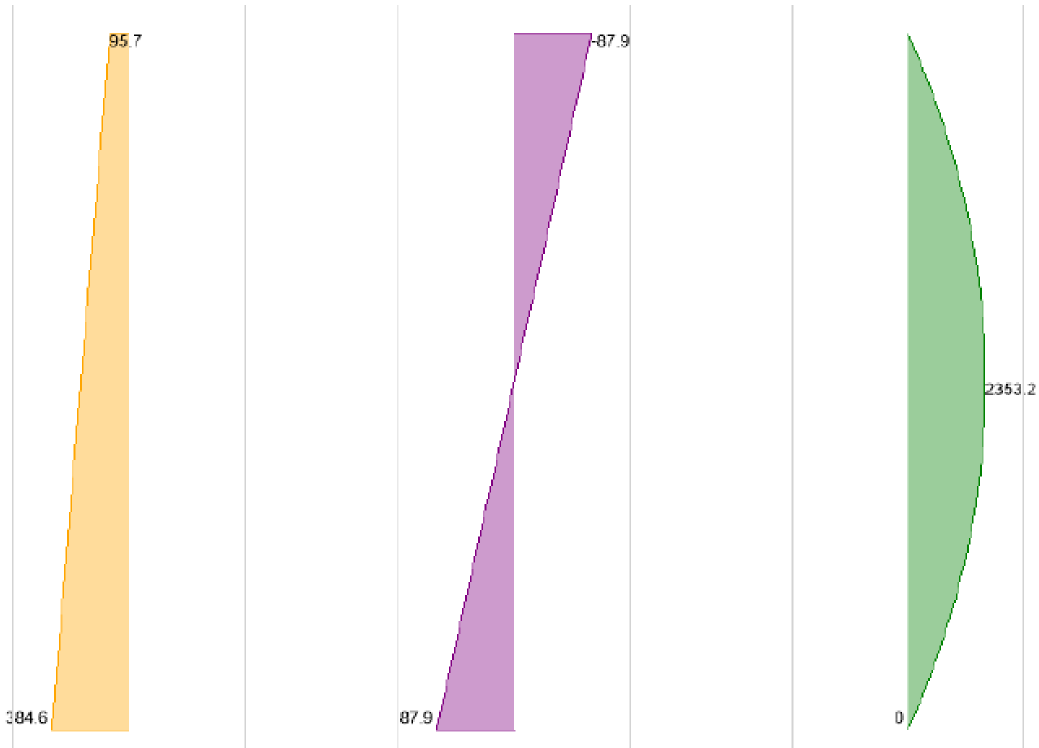
From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1	DL	0.026
Load combination no.2	DL + LL	0.036
Load combination no.3	DL + (LLr or SL or RL)	0.026
Load combination no.4	DL + 0.75 x LL + 0.75 x (LLr or SL or RL)	0.034
Load combination no.5	DL + 0.6 x W	0.079
Load combination no.6	DL + 0.7 x Eh + 0.7 x Ev	0.129
Load combination no.7	DL + 0.75 x LL + 0.45 x W + 0.75 x (LLr or SL or RL)	0.058
Load combination no.8	DL + 0.75 x LL + 0.525 x Eh + 0.525 x Ev + 0.75 x SL	0.095
Load combination no.9	0.6 x DL + 0.6 x W	0.081
Load combination no.10	0.6 x DL + 0.7 x Eh - 0.7 x Ev	0.134

Properties of masonry section:

Cross-sectional area	A	96	in2/ft
Moment of inertia	I	512	in4/ft
Section modulus	S	128	in3/ft
Radius of gyration	r	2.309	in

Consider wall at maximum moment location under load combination no.10



Maximum moment location		4.46	ft
Axial load at max moment loc. of panel	P	240	lb/ft
Compressive stress due to axial load	fa	2.50	psi
Slenderness ratio	h/r	46.350	<99
Allowable compressive stress due to axial load (Eq. 8-16)	Fa	193.661	psi
From MSJC Clause 8.2.3 - Axial compression and flexure			
	fa/Fa	0.01	<1
PASS - Allowable compressive stress exceeds compressive stress due to axial loads			
Bending moment at max. moment loc. of panel	M	2353	lb-in/ft
Compressive stress due to flexural load	fb	18.38	psi
Allowable compressive stress due to flexural load (Eq. 8-18)	Fb	290.00	psi
From MSJC Clause 8.2.3 - Axial compression and flexure			
	fb/Fb	0.06	<1
PASS - Allowable compressive stress exceeds compressive stress due to flexure			
From MSJC Clause 8.2.3 - Axial compression and flexure			
Axial compression and flexure	fa / Fa + fb / Fb	0.08	<1
PASS - Combined axial and flexural stresses are acceptable			
Net flexural tension	ft	15.88	psi
Allowable tensile stress due to flexural load	Ft	32	psi
From MSJC Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry			
	ft / Ft	0.50	<1
PASS - Allowable tensile stress exceeds tensile stress due to flexure			

Factored moment (Eq. 11-18)	Mu	1976.16	lb·in/ft
Factored axial load (Eq. 11-19)	Pu	5407.2	lb
Factored load from tributary floor areas	Puf	1990	lb
Factored load from tributary roof areas	Pur	1990	lb
Factored weight of wall area tributary to wall section under consideration	Puw	1427.2	lb
Deflection due to factored load	δu	0.12	in
Depth of an equivalent compression stress block at nominal strength	a	2.942	in
Nominal moment strength (Eq. 11-29)	Mn	3668.28	
	Mu/ϕMn	0.48	<1
PASS - Allowable nominal moment exceeds applied factored moment			
Seismic response coefficient	Cs	0.2308	kip
Effective seismic weight of the structure	W	235.5	kip
Total perimeter of the walls is parallel to the considered wall	p	75.5	ft
Total seismic base shear	Vut	54.35	kip
Local seismic base shear	Vul	8.94	kip
Total wind shear force	Vtw	19.03	kip
Local wind shear force	Vlw	3.13	kip
Local seismic base shear is biger than local wind shear force		Vul>Vlw	
Factored moment	Mu	89.64	k·ft
Factored axial load	Pu	50.08	kip
Nominal shear strength provided by AAC masonry (Eq. 11-13a)	VnAAC	33.42	kip
Nominal shear strength provided by shear reinforcement (Eq. 11-15)	Vns	9.315	kip
Nominal shear strength (Eq. 11-9)	Vn	42.73	kip
	Vul/Vn>1	0.21	<1
Allowable nominal shear strength exceeds applied local seismic base shear			
	Mu/Vu·dv	0.81	
From MSJC Clause 11.3.4.1.2 (d)	5.49 ·An·√f'AAC	192.93	>Vn
PASS - Allowable shear stress exceeds applied shear stress			

2.8.3 BLOCK C

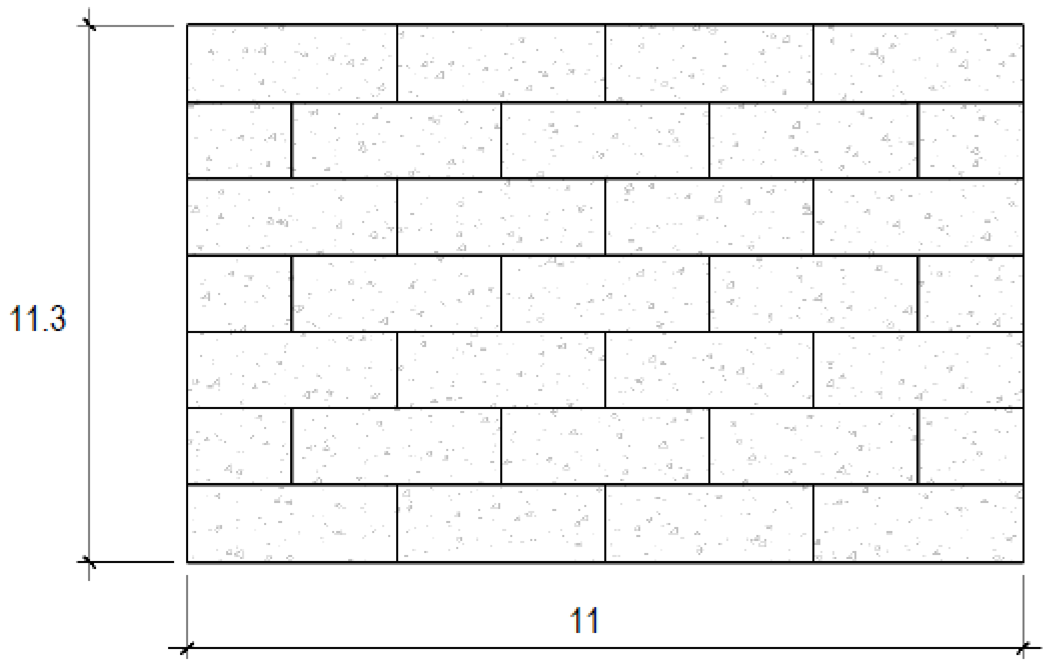
MASONRY AAC WALL PANEL DESIGN TO MSJC

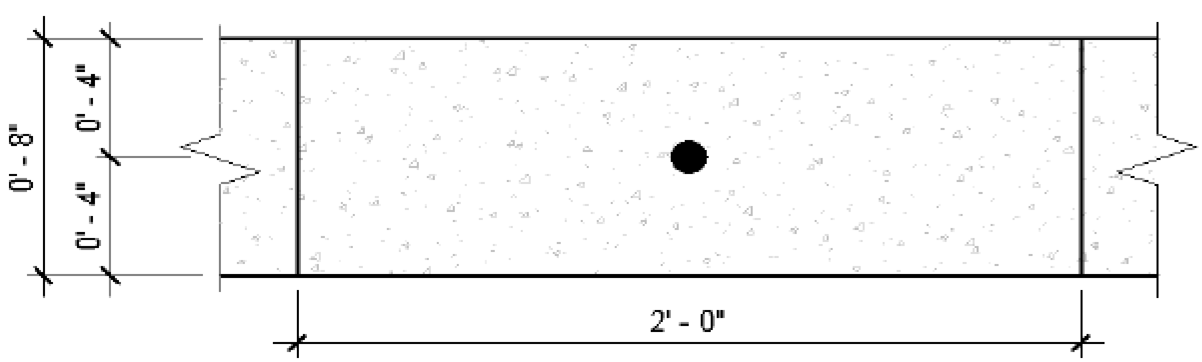
Using the allowable stress design method

Masonry wall panel details:

Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads. The wall is fixed at the bottom and free at the top for in plane loads

Panel length	11	ft
Panel height	11.3	ft





Seismic properties:

Seismic design category	D		
Seismic importance factor (ASCE7 Table 1.5-2)	Ie	1	
Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)	SDS	0.464	
Redundancy factor, on out-of-plane load	ρE	1	

Masonry details:

Compressive strength of unit	f' AAC	870	psi
Density of masonry unit	ϒ block	30	lb/ft3
Width of masonry units	d b	8	in
Height of masonry units	hb	12	in
Length of masonry units	l b	24	in
From MSJC Clause 4.2.2 Elastic moduli			
Modulus of elasticity for masonry (Eq 4.2.2.3.1)	E AAC	377246.2	psi
Shear modulus of masonry	Ev	150898.5	psi

Reinforcement details:

Yield strength of reinforcement	fy	60000	psi
Allowable tensile stress in reinforcement	Fs	32000	psi
Modulus of elasticity for reinforcement	Es	29000000	psi
Vertical reinforcement provided	No.4 bars at 96 in center		
Area of vertical reinforcement	As	0.2	in2

Lateral out-of-plane loads:

Wind load on panel	W	20	psf
Seismic load factor (ASCE7 12.11.1)	Fp	0.4	
Seismic load from wall	Ewall	28.2	psf

Lateral in-plane loads:

Dead load from above	DLabove	208	lb/ft
Live load from above	LLabove	320	lb/ft

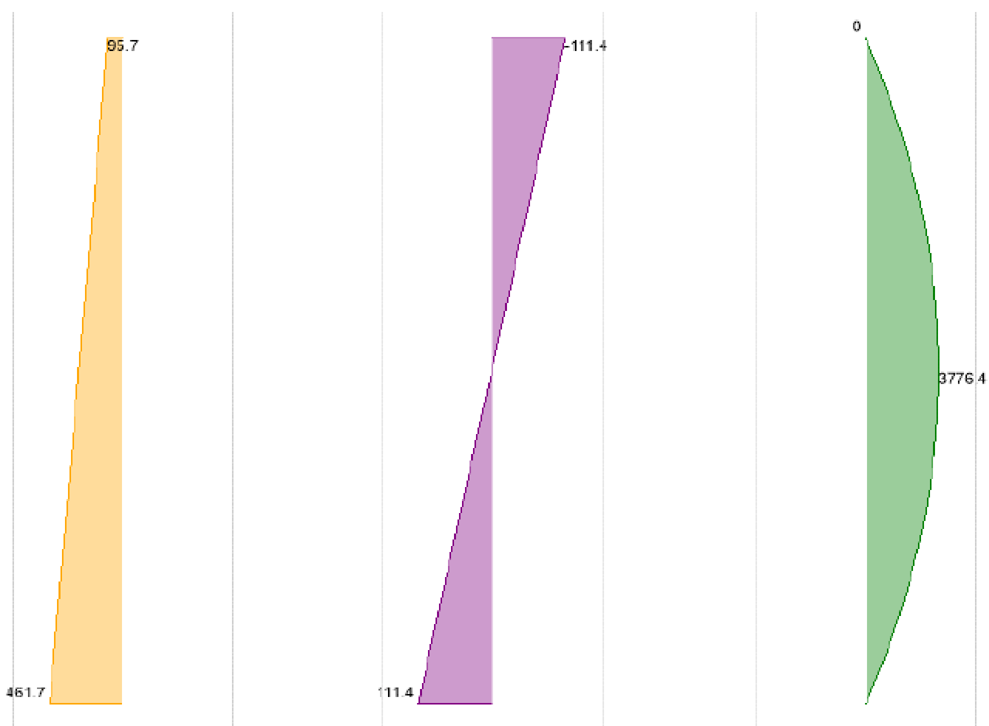
From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1	DL	0.034
Load combination no.2	DL + LL	0.045
Load combination no.3	DL + (LLr or SL or RL)	0.034
Load combination no.4	DL + 0.75 x LL + 0.75 x (LLr or SL or RL)	0.042
Load combination no.5	DL + 0.6 x W	0.126
Load combination no.6	DL + 0.7 x Eh + 0.7 x Ev	0.205
Load combination no.7	DL + 0.75 x LL + 0.45 x W + 0.75 x (LLr or SL or RL)	0.092
Load combination no.8	DL + 0.75 x LL + 0.525 x Eh + 0.525 x Ev + 0.75 x SL	0.15
Load combination no.9	0.6 x DL + 0.6 x W	0.129
Load combination no.10	0.6 x DL + 0.7 x Eh - 0.7 x Ev	0.214

Properties of masonry section:

Cross-sectional area	A	96	in2/ft
Moment of inertia	I	512	in4/ft
Section modulus	S	128	in3/ft
Radius of gyration	r	2.309	in

Consider wall at maximum moment location under load combination no.10



Maximum moment location		5.65	ft
Axial load at max moment loc. of panel	P	279	lb/ft
Compressive stress due to axial load	fa	2.91	psi
Slenderness ratio	h/r	58.717	<99
Allowable compressive stress due to axial load (Eq. 8-16)	Fa	179.242	psi
From MSJC Clause 8.2.3 - Axial compression and flexure			
	fa/Fa	0.02	<1
PASS - Allowable compressive stress exceeds compressive stress due to axial loads			

Bending moment at max. moment loc. of panel	M	3776	lb-in/ft
Compressive stress due to flexural load	fb	29.50	psi
Allowable compressive stress due to flexural load (Eq. 8-18)	Fb	290.00	psi
From MSJC Clause 8.2.3 - Axial compression and flexure			
	fb/Fb	0.10	<1
PASS - Allowable compressive stress exceeds compressive stress due to flexure			

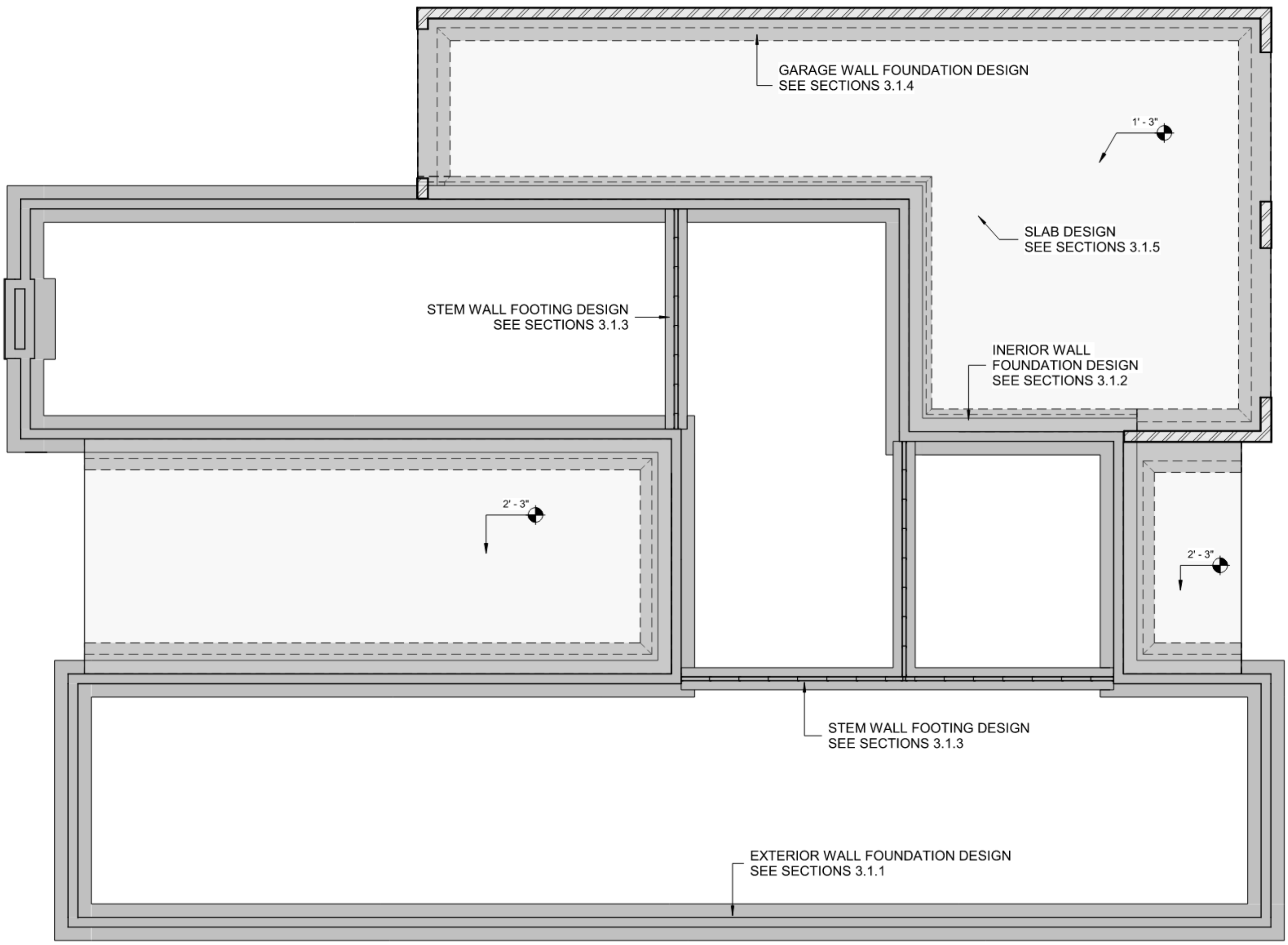
From MSJC Clause 8.2.3 - Axial compression and flexure			
Axial compression and flexure	fa / Fa + fb / Fb	0.12	<1
PASS - Combined axial and flexural stresses are acceptable			

Net flexural tension	ft	26.59	psi
Allowable tensile stress due to flexural load	Ft	32	psi
From MSJC Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry			
	ft / Ft	0.83	<1
PASS - Allowable tensile stress exceeds tensile stress due to flexure			

Factored moment (Eq. 11-18)	Mu	2982.06	lb·in/ft
Factored axial load (Eq. 11-19)	Pu	5328	lb
Factored load from tributary floor areas	Puf	1760	lb
Factored load from tributary roof areas	Pur	1760	lb
Factored weight of wall area tributary to wall section under consideration	Puw	1808	lb
Deflection due to factored load	δu	0.30	in
Depth of an equivalent compression stress block at nominal strength	a	2.929	in
Nominal moment strength (Eq. 11-29)	Mn	3661.26	
	Mu/ϕMn	0.73	<1
PASS - Allowable nominal moment exceeds applied factored moment			

Seismic response coefficient	Cs	0.2308	kip
Effective seismic weight of the structure	W	235.5	kip
Total perimeter of the walls is parallel to the considered wall	p	242.3	ft
Total seismic base shear	Vut	54.35	kip
Local seismic base shear	Vul	2.47	kip
Total wind shear force	Vtw	13.82	kip
Local wind shear force	Vlw	0.63	kip
Local seismic base shear is biger than local wind shear force		Vul>Vlw	
Factored moment	Mu	41.93	k·ft
Factored axial load	Pu	13.82	kip
Nominal shear strength provided by AAC masonry (Eq. 11-13a)	VnAAC	29.59	kip
Nominal shear strength provided by shear reinforcement (Eq. 11-15)	Vns	8.25	kip
Nominal shear strength (Eq. 11-9)	Vn	37.84	kip
	Vul/Vn>1	0.07	<1
Allowable nominal shear strength exceeds applied local seismic base shear			
	Mu/Vu·dv	1.54	
From MSJC Clause 11.3.4.1.2 (d)	6.00 ·An·√f'AAC	186.89	>Vn
PASS - Allowable shear stress exceeds applied shear stress			

3 FOUNDATION ANALYSIS
3.1 FOUNDATION PLAN



FOUNDATION PLAN OF RESIDENTIAL

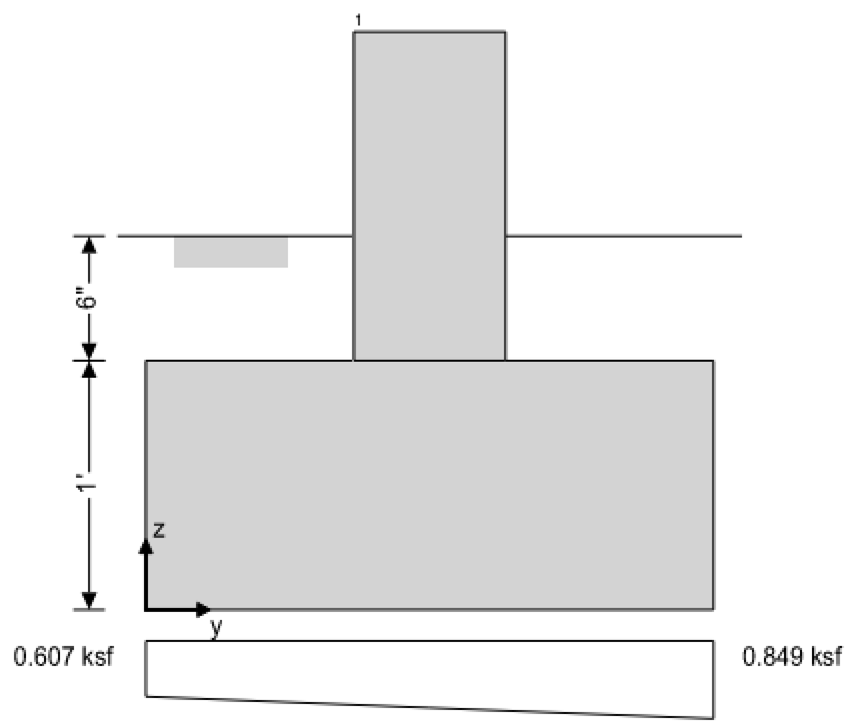
3.1.1 EXTERIOR WALL FOUNDATION DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **1** ft
Width of foundation	L_y = **2.5** ft
Foundation area	$A = L_x \times L_y$ = **2.5** ft^2
Depth of foundation	h = **12** in
Depth of soil over foundation	h_{soil} = **6** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall	l_{y1} = **8** in
position in y-axis	y_1 = **15** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **1.5** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Self weight	$F_{swt} = h * \gamma_{conc}$ = **150** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **60** psf

Wall no.1 loads per linear foot

Dead load in z	F_{Dz1} = **0.7** kips
Live load in z	F_{Lz1} = **0.3** kips
Live roof load in z	F_{Lrz1} = **0.3** kips
Wind load in z	F_{Wz1} = **0.2** kips
Seismic load in z	F_{Ez1} = **0.9** kips
Wind load in y	F_{Wy1} = **0.3** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.327)

1.0D + 1.0L (0.415)

1.0D + 1.0Lr (0.415)

1.0D + 1.0S (0.327)

1.0D + 1.0R (0.327)

1.0D + 0.75L + 0.75Lr (0.459)

1.0D + 0.75L + 0.75S (0.393)

1.0D + 0.75L + 0.75R (0.393)

1.0D + 0.6W (0.469)

$(1.0 + 0.14 * S_{DS})$D + 0.7E (0.516)

1.0D + 0.75L + 0.75Lr + 0.45W (0.566)

1.0D + 0.75L + 0.75S + 0.45W (0.500)

1.0D + 0.75L + 0.75R + 0.45W (0.500)

$(1.0 + 0.10 * S_{DS})$D + 0.75L + 0.75S + 0.525E (0.534)

0.6D + 0.6W (0.339)

$(0.6 - 0.14 * S_{DS})$D + 0.7E (0.343)

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

Forces on foundation per linear foot

Force in y-axis — $F_{dy} = \gamma_W * F_{Wy1} =$ **0.1** kips

Force in z-axis — $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_{Lr} * F_{Lrz1} + \gamma_W * F_{Wz1} =$ **1.8** kips

Moments on foundation per linear foot

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1) + \gamma_W * (F_{Wz1} * y_1 + F_{Wy1} * h) =$ **2.4** kip_ft

Uplift verification

Vertical force — $F_{dz} =$ **1.819** kips

PASS - Foundation is not subject to uplift

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

Overturning moment — $M_{OTyL} = \gamma_W * (F_{Wy1} * h) =$ **0.13** kip_ft

Resisting moment — $M_{RyL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2)) + \gamma_D * (F_{Dz1} * (y_1 - L_y)) + \gamma_L * (F_{Lz1} * (y_1 - L_y)) + \gamma_{Lr} * (F_{Lrz1} * (y_1 - L_y)) + \gamma_W * (F_{Wz1} * (y_1 - L_y)) =$ **-2.27** kip_ft

Factor of safety — $abs(M_{RyL} / M_{OTyL}) =$ **18.046**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction — $F_{RFriction} = max(F_{dz}, 0 kN) * tan(\delta_{bb}) =$ **1.05** kips

Stability against sliding in y direction

Total sliding resistance — $F_{Ry} = F_{RFriction} =$ **1.05** kips

Factor of safety — $abs(F_{Ry} / F_{dy}) =$ **8.33**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0.831** in

Strip base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dy} / L_y) / (L_y * 1 ft) =$ **0.607** ksf

$q_2 = F_{dz} * (1 + 6 * e_{dy} / L_y) / (L_y * 1 ft) =$ **0.849** ksf

Minimum base pressure — $q_{min} = min(q_1,q_2) =$ **0.607** ksf

Maximum base pressure — $q_{max} = max(q_1,q_2) =$ **0.849** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} =$ **1.5** ksf

$q_{max} / q_{allow} =$ **0.566**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete	f'_c = **3000** psi
Yield strength of reinforcement	f_y = **60000** psi
Cover to reinforcement	c_{nom} = **3** in
Concrete type	Normal weight
Concrete modification factor	λ = **1.00**

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.023)
1.2D + 1.6L + 0.5Lr (0.035)
1.2D + 1.6L + 0.5S (0.032)
1.2D + 1.6L + 0.5R (0.032)
1.2D + 1.0L + 1.6Lr (0.039)
1.2D + 1.0L + 1.6S (0.027)
1.2D + 1.0L + 1.6R (0.027)
1.2D + 1.6Lr + 0.5W (0.040)
1.2D + 1.6S + 0.5W (0.027)
1.2D + 1.6R + 0.5W (0.027)
1.2D + 1.0L + 0.5Lr + 1.0W (0.047)
1.2D + 1.0L + 0.5S + 1.0W (0.043)
1.2D + 1.0L + 0.5R + 1.0W (0.043)

$(1.2 + 0.2 * S_{DS})$D + 1.0L + 0.2S + 1.0E (0.049)
0.9D + 1.0W (0.031)
$(0.9 - 0.2 * S_{DS})$D + 1.0E (0.034)

Combination 14 results: $(1.2 + 0.2 * S_{DS})$D + 1.0L + 0.2S + 1.0E

Forces on foundation per linear foot

Ultimate force in z-axis	$F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_E * F_{Ez1}$ = **2.8** kips

Moments on foundation per linear foot

Ultimate moment in y-axis, about y is 0	$M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_E * (F_{Ez1} * y_1)$ = **3.5** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in y-axis	$e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0.000** in

Strip base pressures

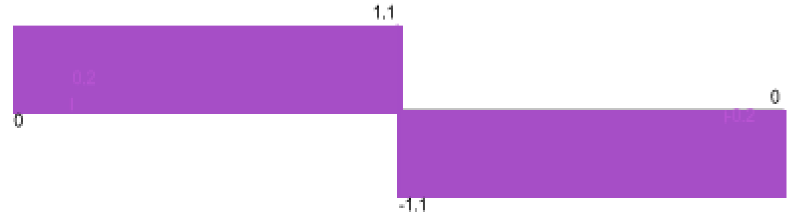
$q_{u1} = F_{uz} * (1 - 6 * e_{uy} / L_y) / (L_y * 1\text{ ft}) = \mathbf{1.125}$ ksf

$q_{u2} = F_{uz} * (1 + 6 * e_{uy} / L_y) / (L_y * 1\text{ ft}) = \mathbf{1.125}$ ksf

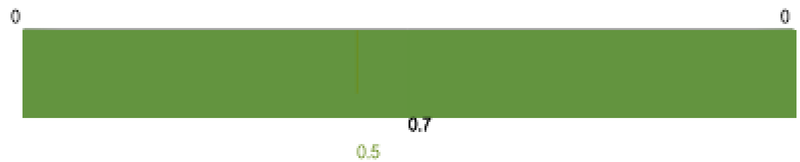
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2}) = \mathbf{1.125}$ ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2}) = \mathbf{1.125}$ ksf

Shear diagram (kips)



Moment diagram (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max} = \mathbf{0.501}$ kip_ft

Tension reinforcement provided — No.4 bars at 9.0 in c/c bottom

Area of tension reinforcement provided — $A_{sy.bot.prov} = \mathbf{0.267}$ in^2

Minimum area of reinforcement (7.6.1.1) — $A_{s.min} = 0.0018 * L_x * h = \mathbf{0.259}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3) — $s_{max} = \min(3 * h, 18\text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

---

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{y.bot} / 2 = \mathbf{8.750}$ in

Depth of compression block — $a = A_{sy.bot.prov} * f_y / (0.85 * f'_c * L_x) = \mathbf{0.523}$ in

Neutral axis factor — $\beta_1 = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{0.615}$ in

Strain in tensile reinforcement (7.3.3.1) — $\varepsilon_t = 0.003 * d / c - 0.003 = \mathbf{0.03967}$

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity — $M_n = A_{sy.bot.prov} * f_y * (d - a / 2) = \mathbf{11.318}$ kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9) = \mathbf{0.900}$

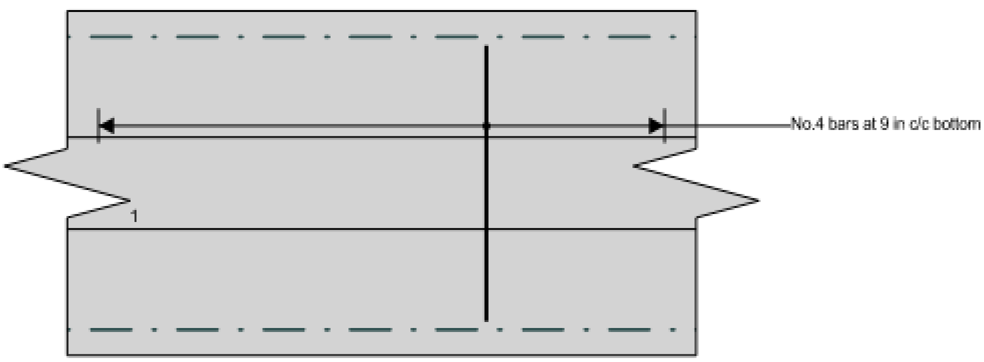
Design moment capacity	$\phi M_n = \phi_f * M_n =$ **10.186** kip_ft
	$M_{u.y.max} / \phi M_n =$ **0.049**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y} =$ **0.16** kips
Depth to reinforcement	$d_v = h - c_{nom} - \phi_{y.bot} / 2 =$ **8.75** in
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 * \lambda * \surd(f'_c * 1 \text{ psi}) * L_x * d_v =$ **11.502** kips
Design shear capacity	$\phi V_n = \phi_v * V_n =$ **8.627** kips
	$V_{u.y} / \phi V_n =$ **0.019**

PASS - Design shear capacity exceeds ultimate shear load



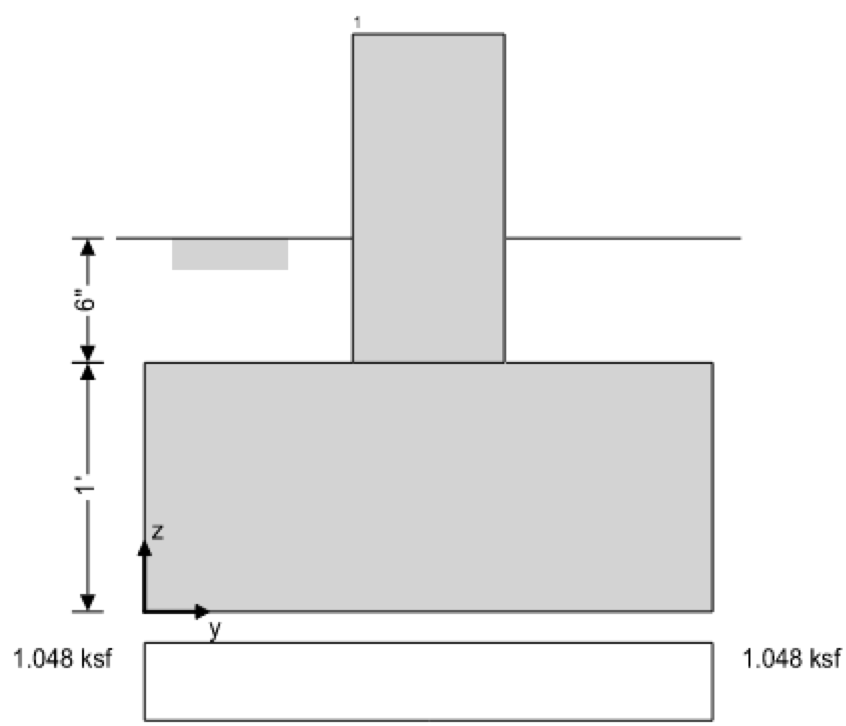
3.1.2 INTERIOR WALL FOUNDATION DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **1** ft
Width of foundation	L_y = **2.5** ft
Foundation area	$A = L_x \times L_y$ = **2.5** ft^2
Depth of foundation	h = **12** in
Depth of soil over foundation	h_{soil} = **6** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall	l_{y1} = **8** in
position in y-axis	y_1 = **15** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **1.5** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**

Foundation loads

Self weight	$F_{swt} = h * \gamma_{conc}$ = **150** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **60** psf

Wall no.1 loads per linear foot

Dead load in z	F_{Dz1} = **1.0** kips
Live load in z	F_{Lz1} = **0.6** kips
Live roof load in z	F_{Lrz1} = **0.3** kips
Seismic load in z	F_{Ez1} = **1.2** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.393)

1.0D + 1.0L (0.548)

1.0D + 1.0Lr (0.471)

1.0D + 1.0S (0.393)

1.0D + 1.0R (0.393)

1.0D + 0.75L + 0.75Lr (0.567)

1.0D + 0.75L + 0.75S (0.509)

1.0D + 0.75L + 0.75R (0.509)

1.0D + 0.6W (0.393)

$(1.0 + 0.14 * S_{DS})$D + 0.7E (0.647)

1.0D + 0.75L + 0.75Lr + 0.45W (0.567)

1.0D + 0.75L + 0.75S + 0.45W (0.509)

1.0D + 0.75L + 0.75R + 0.45W (0.509)

$(1.0 + 0.10 * S_{DS})$D + 0.75L + 0.75S + 0.525E (0.698)

0.6D + 0.6W (0.236)

$(0.6 - 0.14 * S_{DS})$D + 0.7E (0.438)

Combination 14 results: (1.0 + 0.10 * S_{DS})D + 0.75L + 0.75S + 0.525E

Forces on foundation per linear foot

Force in z-axis — $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_E * F_{Ez1}$ = **2.6** kips

Moments on foundation per linear foot

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_E * (F_{Ez1} * y_1)$ = **3.3** kip_ft

Uplift verification

Vertical force — F_{dz} = **2.619** kips

PASS - Foundation is not subject to uplift

Stability against sliding

Resistance due to base friction — $F_{RFriction}$ = max(F_{dz}, 0 kN) * tan(δ_{bb}) = **1.512** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dy} / L_y) / (L_y * 1\ ft)$ = **1.048** ksf

$q_2 = F_{dz} * (1 + 6 * e_{dy} / L_y) / (L_y * 1\ ft)$ = **1.048** ksf

Minimum base pressure — q_{min} = min(q_1,q_2) = **1.048** ksf

Maximum base pressure — q_{max} = max(q_1,q_2) = **1.048** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross}$ = **1.5** ksf

q_{max} / q_{allow} = **0.698**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete — f'_c = **3000** psi

Yield strength of reinforcement — f_y = **60000** psi

Cover to reinforcement — c_{nom} = **3** in

Concrete type — Normal weight

Concrete modification factor — λ = **1.00**

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.031)

1.2D + 1.6L + 0.5Lr (0.051)

1.2D + 1.6L + 0.5S (0.048)

1.2D + 1.6L + 0.5R (0.048)

1.2D + 1.0L + 1.6Lr (0.050)

1.2D + 1.0L + 1.6S (0.040)

1.2D + 1.0L + 1.6R (0.040)

1.2D + 1.6Lr + 0.5W (0.037)

1.2D + 1.6S + 0.5W (0.026)

1.2D + 1.6R + 0.5W (0.026)

1.2D + 1.0L + 0.5Lr + 1.0W (0.043)

1.2D + 1.0L + 0.5S + 1.0W (0.040)

1.2D + 1.0L + 0.5R + 1.0W (0.040)

$(1.2 + 0.2 * S_{DS})D + 1.0L + 0.2S + 1.0E$ (0.070)

0.9D + 1.0W (0.020)

$(0.9 - 0.2 * S_{DS})D + 1.0E$ (0.046)

Combination 14 results: $(1.2 + 0.2 * S_{DS})D + 1.0L + 0.2S + 1.0E$

Forces on foundation per linear foot

Ultimate force in z-axis	$F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_E * F_{Ez1}$ = **3.7** kips

Moments on foundation per linear foot

Ultimate moment in y-axis, about y is 0	$M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_E * (F_{Ez1} * y_1)$ = **4.6** kip_ft

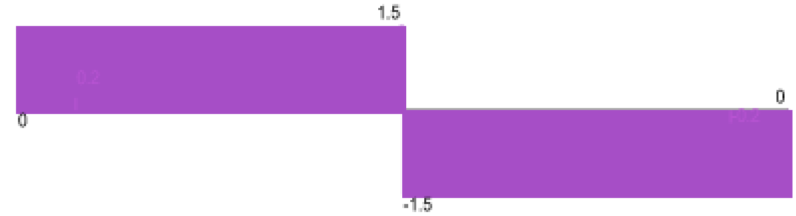
Eccentricity of base reaction

Eccentricity of base reaction in y-axis	$e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0.000** in

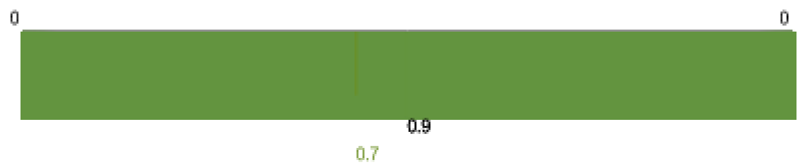
Strip base pressures

	$q_{u1} = F_{uz} * (1 - 6 * e_{uy} / L_y) / (L_y * 1 \text{ ft})$ = **1.483** ksf
	$q_{u2} = F_{uz} * (1 + 6 * e_{uy} / L_y) / (L_y * 1 \text{ ft})$ = **1.483** ksf
Minimum ultimate base pressure	$q_{umin} = \min(q_{u1},q_{u2})$ = **1.483** ksf
Maximum ultimate base pressure	$q_{umax} = \max(q_{u1},q_{u2})$ = **1.483** ksf

Shear diagram (kips)



Moment diagram (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **0.711** kip_ft

Tension reinforcement provided No.4 bars at 9.0 in c/c bottom

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **0.267** in^2

Minimum area of reinforcement (7.6.1.1) $A_{s.min} = 0.0018 * L_x * h$ = **0.259** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3) s_{max} = min(3 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{y.bot} / 2$ = **8.750** in

Depth of compression block $a = A_{sy.bot.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.523** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.615** in

Strain in tensile reinforcement (7.3.3.1) $\varepsilon_t = 0.003 * d / c - 0.003$ = **0.03967**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity $M_n = A_{sy.bot.prov} * f_y * (d - a / 2)$ = **11.318** kip_ft

Flexural strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) * (250 / 3), 0.65), 0.9) = **0.900**

Design moment capacity $\phi M_n = \phi_f * M_n$ = **10.186** kip_ft

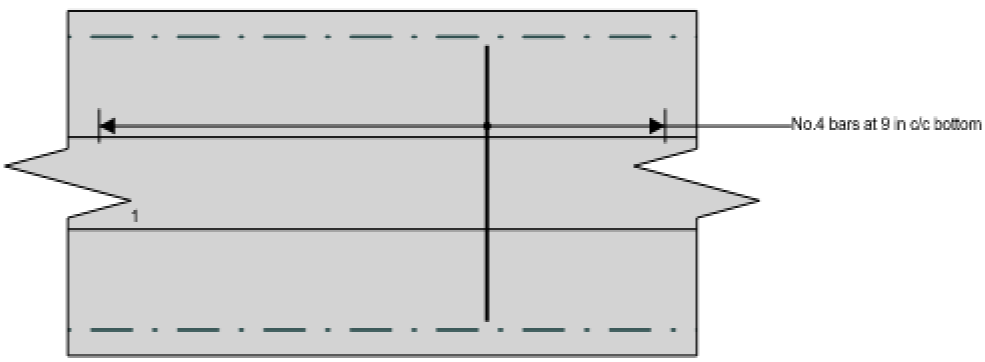
$M_{u.y.max} / \phi M_n$ = **0.070**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **0.228** kips
Depth to reinforcement	$d_v = h - c_{nom} - \phi_{y.bot} / 2$ = **8.75** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 * \lambda * \sqrt{(f'_c * 1 \text{ psi})} * L_x * d_v$ = **11.502** kips
Design shear capacity	$\phi V_n = \phi_v * V_n$ = **8.627** kips
	$V_{u.y} / \phi V_n$ = **0.026**

PASS - Design shear capacity exceeds ultimate shear load



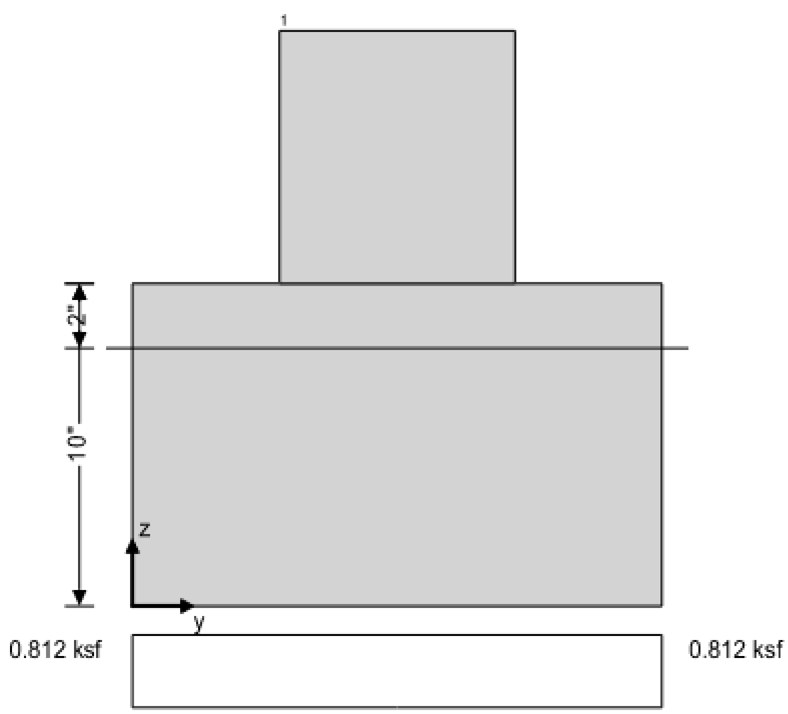
3.1.3 STEM WALL FOOTING DESIGN

FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation	L_x = **1** ft
Width of foundation	L_y = **1.5** ft
Foundation area	$A = L_x \times L_y$ = **1.5** ft^2
Depth of foundation	h = **10** in
Depth of soil over foundation	h_{soil} = **-2** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall	l_{y1} = **8** in
position in y-axis	y_1 = **9** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **1.5** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**

Foundation loads

Self weight	$F_{swt} = h * \gamma_{conc}$ = **125** psf

Wall no.1 loads per linear foot

Dead load in z	F_{Dz1} = **0.5** kips
Live load in z	F_{Lz1} = **0.6** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.274)

1.0D + 1.0L (0.541)

1.0D + 1.0Lr (0.274)

1.0D + 1.0S (0.274)

1.0D + 1.0R (0.274)

1.0D + 0.75L + 0.75Lr (0.474)

1.0D + 0.75L + 0.75S (0.474)

1.0D + 0.75L + 0.75R (0.474)

1.0D + 0.6W (0.274)

$(1.0 + 0.14 * S_{DS})$D + 0.7E (0.292)

1.0D + 0.75L + 0.75Lr + 0.45W (0.474)

1.0D + 0.75L + 0.75S + 0.45W (0.474)

1.0D + 0.75L + 0.75R + 0.45W (0.474)

$(1.0 + 0.10 * S_{DS})$D + 0.75L + 0.75S + 0.525E (0.487)

0.6D + 0.6W (0.165)

$(0.6 - 0.14 * S_{DS})$D + 0.7E (0.147)

Combination 2 results: 1.0D + 1.0L

Forces on foundation per linear foot

Force in z-axis	$F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1}$ = **1.2** kips

Moments on foundation per linear foot

Moment in y-axis, about y is 0	$M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1)$ = **0.9** kip_ft

Uplift verification

Vertical force F_{dz} = **1.218** kips

PASS - Foundation is not subject to uplift

Stability against sliding

Resistance due to base friction $F_{RFriction} = max(F_{dz}, 0 kN) * tan(\delta_{bb})$ = **0.703** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dy} / L_y) / (L_y * 1 ft)$ = **0.812** ksf

$q_2 = F_{dz} * (1 + 6 * e_{dy} / L_y) / (L_y * 1 ft)$ = **0.812** ksf

Minimum base pressure $q_{min} = min(q_1,q_2)$ = **0.812** ksf

Maximum base pressure $q_{max} = max(q_1,q_2)$ = **0.812** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **1.5** ksf

q_{max} / q_{allow} = **0.541**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete f'_c = **3000** psi

Yield strength of reinforcement f_y = **60000** psi

Cover to reinforcement c_{nom} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.009)

1.2D + 1.6L + 0.5Lr (0.022)

1.2D + 1.6L + 0.5S (0.022)

1.2D + 1.6L + 0.5R (0.022)

1.2D + 1.0L + 1.6Lr (0.017)

1.2D + 1.0L + 1.6S (0.017)

1.2D + 1.0L + 1.6R (0.017)

1.2D + 1.6Lr + 0.5W (0.008)

1.2D + 1.6S + 0.5W (0.008)

1.2D + 1.6R + 0.5W (0.008)

1.2D + 1.0L + 0.5Lr + 1.0W (0.017)

1.2D + 1.0L + 0.5S + 1.0W (0.017)

1.2D + 1.0L + 0.5R + 1.0W (0.017)

$(1.2 + 0.2 * S_{DS})D + 1.0L + 0.2S + 1.0E$ (0.017)

0.9D + 1.0W (0.006)

$(0.9 - 0.2 * S_{DS})D + 1.0E$ (0.005)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation per linear foot

Ultimate force in z-axis — $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1}$ = **1.7** kips

Moments on foundation per linear foot

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1)$ = **1.3** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0.000** in

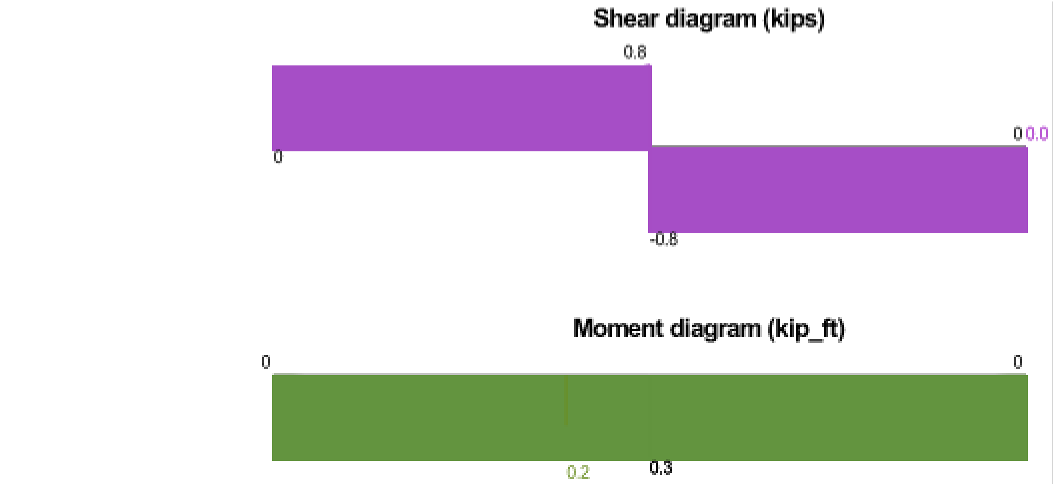
Strip base pressures

$q_{u1} = F_{uz} * (1 - 6 * e_{uy} / L_y) / (L_y * 1 \text{ ft})$ = **1.134** ksf

$q_{u2} = F_{uz} * (1 + 6 * e_{uy} / L_y) / (L_y * 1 \text{ ft})$ = **1.134** ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1}, q_{u2})$ = **1.134** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1}, q_{u2})$ = **1.134** ksf



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **0.172** kip_ft
Tension reinforcement provided	No.4 bars at 9.0 in c/c bottom
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **0.267** in^2
Minimum area of reinforcement (7.6.1.1)	$A_{s.min} = 0.0018 * L_x * h$ = **0.216** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3)	s_{max} = min(3 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom} - \phi_{y.bot} / 2$ = **6.750** in
Depth of compression block	$a = A_{sy.bot.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.523** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.615** in
Strain in tensile reinforcement (7.3.3.1)	$\varepsilon_t = 0.003 * d / c - 0.003$ = **0.02992**

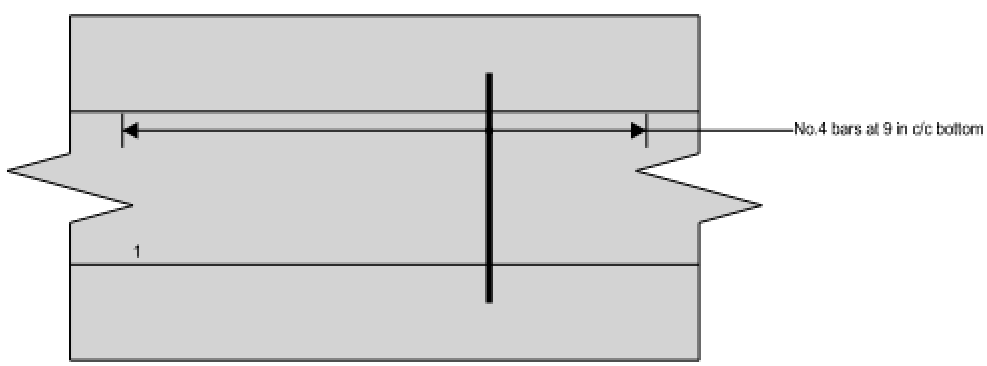
PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	$M_n = A_{sy.bot.prov} * f_y * (d - a / 2)$ = **8.651** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) * (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f * M_n$ = **7.786** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.022**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.



3.1.4 GARAGE WALL FOUNDATION DESIGN

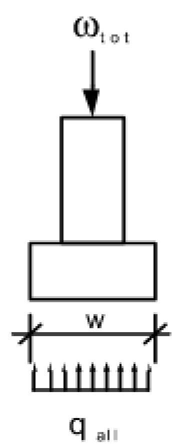
FOUNDATION WIDTH CALCULATION

Description of Variables:			DL	LL	
	1r = 1 ft of roof trib	Roof	19	20	psf
	2f = 2 ft of floor trib	Floor	22.25	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	26.25	0	psf

Conventional Footing Design Parameters:

Allowable Bearing Pressure 1500 psf

FT	TRIB	Wdl	Wll
	(ft)	(plf)	(plf)
F	14.5 r	276	290
	7.8 f	172	310
	0.0 g	0	0
	0.0 d	0	0
	13.8 w	361	0
	TL	**809**	**600**



w_{tot} 1409 plf

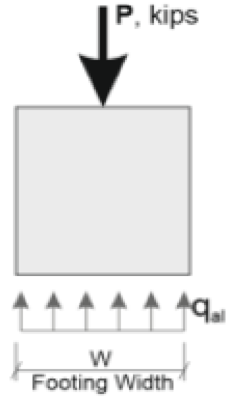
Try Footing Width, w = 16 inches

q_{bear} 1057 psf

q_{allow} 1500 psf

q_{allow}/q_{bear} 1.42 Footing OK

FOUNDATION REINFORCEMENT CALCULATION



Reactions on Footing		D	L	E	W	
Pt Load from	F1	0.8	0.6	0.0	0.2	kips
	Total	0.8	0.6	0.0	0.2	kips
				U =	**2.0**	kips
				P =	**1.4**	kips

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

Conventional Footing Design Parameters:

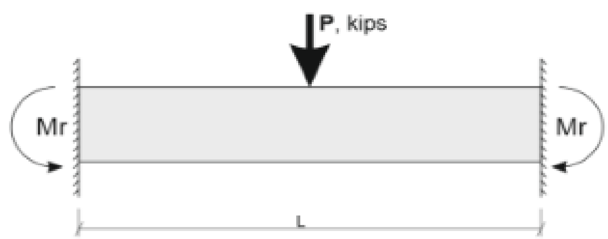
Allowable Bearing Pressure			1500	psf
Seismic or Wind Loading?	N	**--->**	1500	psf

Footing Width	16	inches
A_{req}	0.9	ft^2
L_{req}	**0.7**	ft

Model Foundation as Fixed/Fixed Member

Eqn. 1: $M_u = PL/8$

$M_u =$ **0.2** k-ft



Steel Reinforcing Design:

Steel Depth	12	in
Bar size (#)	4	bar
# of bars	1	

$\text{ø}M_n$ =	10.4	kip-ft	*(Design Cap.)*
M_u=	0.2	kip-ft	*(From Eqn. 1)*
øMn/Mu =	**59.1**	OK	

Therefore use 16 inch x 15 inch footing, with (1) #4 bar top and bottom. See Details.

3.1.5 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 = 28 ft, w = 40 psf and fs = 2/3 fy where fy is 60,000 psi.

As = 0.021 sq.in./ft of slab width, required each way.

Follows ACI's structural slab limitation of 5h or 18 in. (whichever is less), then #3 bars at 18 in. are selected, As = 0.055 sq.in./ft.