LEIGH & O'KANE L.L.C. – STRUCTURAL CALCULATIONS FOR

Express Stop – View High

3394 NW Village Park Drive, Lee's Summit, MO 64081

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June 16, 2025

Leigh & O'Kane
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GENERAL DESIGN CONSIDERATIONS AND LOADING

PROJECT DESCRIPTION

The calculations provided are for a new convenience store in Lee's Summit, Missouri. The structural design is based on the architectural design given by Collins-Webb Architecture. The building is constructed with prefabricated wood roof trusses bearing on wood load bearing walls and steel frames. Foundations consist of concrete trench footings and isolated concrete footings at columns. The lateral forces will be resisted by wood shear walls and steel moment frames.

REFERENCED DESIGN STANDARDS

- 2018 IBC - International Building Code
- ASCE 7-16 - American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures
- AISC 360-16 - American Institute of Steel Construction Fifteenth Edition
- ACI 318-14 - American Concrete Institute Building Code Requirements for Structural Concrete
- NDS 2018 - National Design Specification for Wood Construction

COMPILED DESIGN LOADING

The following items are listed on the construction documents per IBC 2018.

Overall Building Classifications	
Risk Category	II
Snow Importance Factor, I_s	1.00
Ice Importance Factor-Wind, I_w	1.00
Seismic Importance Factor, I_e	1.00

Slab on Grade Floor Loads	
Live Load	100 psf
Concentrated Load	3000 lbs acting on an area 4.5 in by 4.5 in

Roof Dead Loads and Live Loads	
Dead Load Top Chord	20 psf
Dead Load Bottom Chord	5 psf
Live Load Top Chord	20 psf
Live Load Bottom Chord	0 psf (Unless Noted Otherwise)

Roof Snow Loads	
Ground Snow Load, P_g	20 psf
Flat Roof Snow Load, P_f	14 psf
Snow Exposure Factor, C_e	1.0
Thermal Factor, C_t	1.0
Slope Factor, C_s	1.0

Drifting and Unbalanced Loading per ASCE 7-16.

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Wind Loads

Basic Wind Speed (3 second gust), V	109 mph
Exposure Category	C
Internal Pressure Coefficient, GC_{pi}	+/- 0.18

Components and Cladding per ASCE 7-16.

Seismic Loads

S_s	0.099 g
S_1	0.068 g
Site Class	D – Default
S_{DS}	0.105 g
S_{D1}	0.109 g
Seismic Design Category	B
Seismic Force Resisting System	Wood Walls Sheathed with Wood Structural Panels Rated for Shear and Ordinary Steel Moment Frames
Design Base Shear	C_s X W
Design Response Coefficient, C_s	0.016 & 0.035
Response Modification Coefficient, R	6.5 & 3
Analysis Procedure Used	Equivalent Lateral Force (ELF) Procedure

Roof Rain Loads

60-min duration/100 year Rain Intensity, I	3.53 in/hr
15-min duration/100 year Rain Intensity, I	7.51 in/hr

SITE DESIGN LOADING

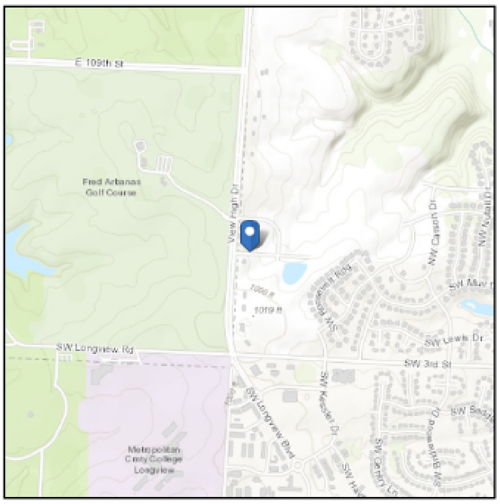
ASCE AMERICAN SOCIETY OF CIVIL ENGINEERS

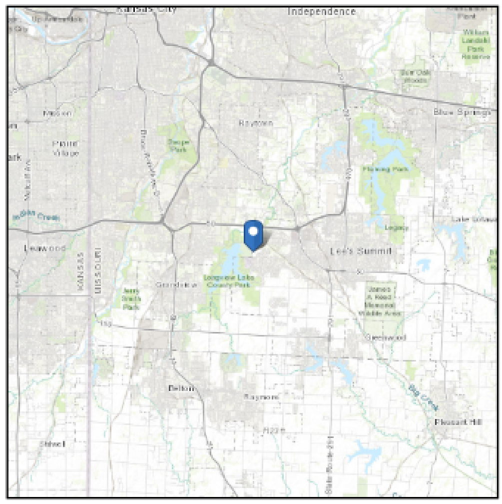
ASCE Hazards Report

Address:
3394 NW Village Park Dr
Lees Summit, Missouri
64081

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Latitude: 38.917023
Longitude: -94.449893
Elevation: 983.9491270441138 ft (NAVD 88)





Wind

Results:

Wind Speed	109 Vmph
10-year MRI	76 Vmph
25-year MRI	83 Vmph
50-year MRI	88 Vmph
100-year MRI	94 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Tue Apr 29 2025

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

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

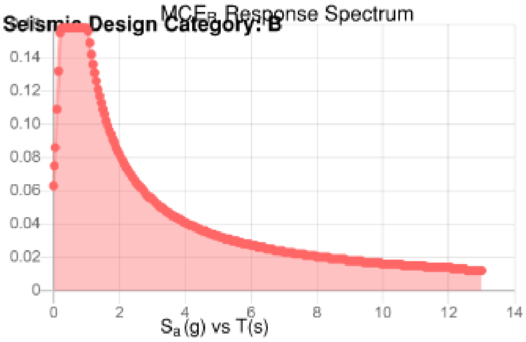
Seismic

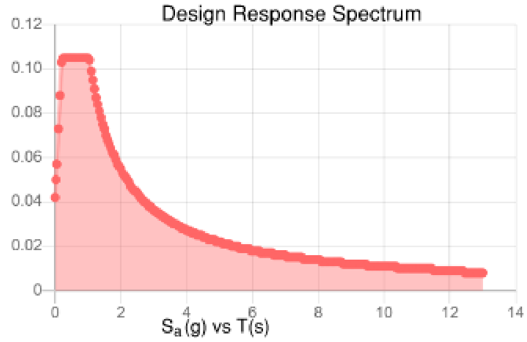
Site Soil Class: D - Default (see Section 11.4.3)

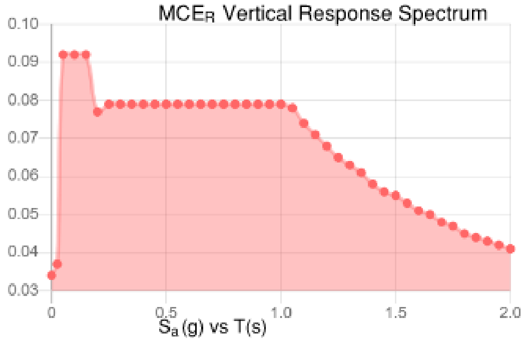
Results:

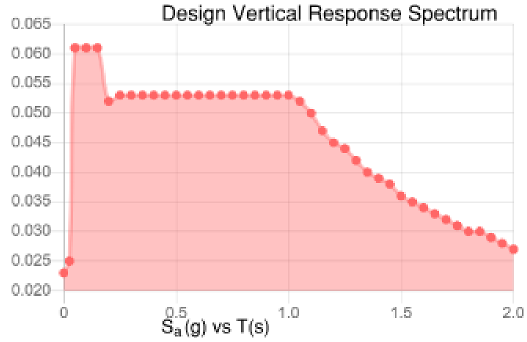
S_S :	0.099	S_{D1} :	0.109
S_1 :	0.068	T_L :	12
F_a :	1.6	PGA :	0.047
F_v :	2.4	PGA_M :	0.075
S_{MS} :	0.158	F_{PGA} :	1.6
S_{M1} :	0.164	I_e :	1
S_{DS} :	0.105	C_v :	0.7

Seismic Design Category: B









Data Accessed: **Tue Apr 29 2025**

Date Source:

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

Ice

Results:

Ice Thickness:	1.50 in.
Concurrent Temperature:	5 F
Gust Speed	40 mph
Data Source:	Standard ASCE/SEI 7-16, Figs. 10-2 through 10-8
Date Accessed:	Tue Apr 29 2025

Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Values provided are equivalent radial ice thicknesses due to freezing rain with concurrent 3-second gust speeds, for a 500-year mean recurrence interval, and temperatures concurrent with ice thicknesses due to freezing rain. Thicknesses for ice accretions caused by other sources shall be obtained from local meteorological studies. Ice thicknesses in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Snow

Results:

Ground Snow Load, p_g :	20 lb/ft^2
Mapped Elevation:	983.9 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Tue Apr 29 2025

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

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

Rain

Results:

15-minute Precipitation Intensity: 7.51 in./h

60-minute Precipitation Intensity: 3.53 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

Date Accessed: Tue Apr 29 2025

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SNOW LOADING

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.12

Building details

Roof type; Flat

Width of roof; b = **55.00** ft

Ground snow load

Ground snow load (Figure 7.2-1); p_g = **20.00** lb/ft^2

Density of snow; $\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3

Surface roughness category (Sect. 26.7); C

Exposure condition (Table 7.3-1); Partially exposed

Exposure factor (Table 7.3-1); C_e = **1.00**

Thermal condition (Table 7.3-2); All

Thermal factor (Table 7.3-2); C_t = **1.00**

Importance category (Table 1.5-1); II

Importance factor (Table 1.5-2); I_s = **1.00**

Min snow load for low slope roofs (Sect 7.3.4); $p_{f\_min} = I_s \times p_g$ = **20.00** lb/ft^2

Flat roof snow load (Sect 7.3); $p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **14.00** lb/ft^2

Left parapet

Balanced snow load height; $h_b = p_f / \gamma$ = **0.84** ft

Height of left parapet; h_{pptL} = **6.00** ft

Height from balance load to top of left parapet; $h_{c\_pptL} = h_{pptL} - h_b$ = **5.16** ft

Length of roof - left parapet; $l_{u\_pptL} = b$ = **55.00** ft

Drift height windward drift - left parpet; $h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.75** ft

Drift height - left parapet; $h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **1.75** ft

Drift width; $W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **6.98** ft

Drift surcharge load - left parapet; $p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **28.97** lb/ft^2

Right parapet

Height of right parapet; h_{pptR} = **6.00** ft

Height from balance load to top of right parapet; $h_{c\_pptR} = h_{pptR} - h_b$ = **5.16** ft

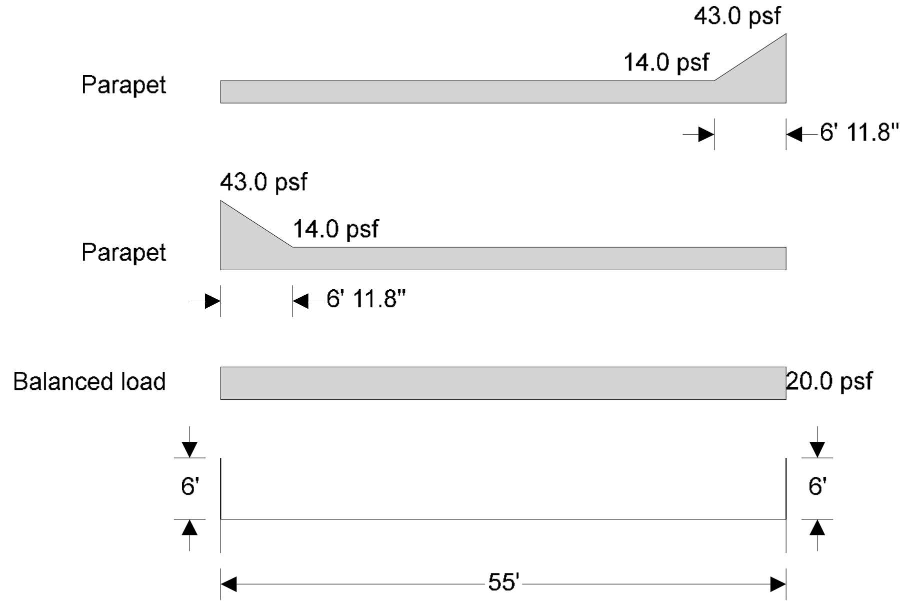
Length of roof - right parapet; $l_{u\_pptR} = b$ = **55.00** ft

Drift height windward drift - right parpet; $h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.75** ft

Drift height - right parapet; $h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **1.75** ft

Drift width; $W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **6.98** ft

Drift surcharge load - right parapet; $p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **28.97** lb/ft^2



Roof elevation

Drift calculations

Balanced snow load height; $h_b = p_f / \gamma =$ **0.84** ft

Length of upper roof; $l_u =$ **0.00** ft

Length of lower roof; $l_l =$ **42.00** ft

Height diff between uppper and lower roofs; $h_{diff} =$ **6.00** ft

Height from balance load to top of upper roof; $h_c = h_{diff} - h_b =$ **5.16** ft

Drift height leeward drift; $h_{d\_l} = \min(\sqrt{(I_s)} \times (0.43 \times (\max(20\text{ ft}, l_u) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} -$ $1.5\text{ft}), 0.6 \times l_l, \sqrt{(I_s \times p_g \times l_u / (4 \times \gamma))}) =$ **0.00** ft

Drift height windward drift; $h_{d\_w} = 0.75 \times \sqrt{(I_s)} \times (0.43 \times (\max(20\text{ ft}, l_l) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} -$ $1.5\text{ft}) =$ **1.50** ft

Maximum lw/ww drift height; $h_{d\_max} = \max(h_{d\_w}, h_{d\_l}) =$ **1.50** ft

Drift height; $h_d = \min(h_{d\_max}, h_c) =$ **1.50** ft

Drift width; $W_d = \min(4 \times h_{d\_max}, 8 \times h_c) =$ **5.99** ft

Drift surcharge load; $p_d = h_d \times \gamma =$ **24.88** lb/ft^2

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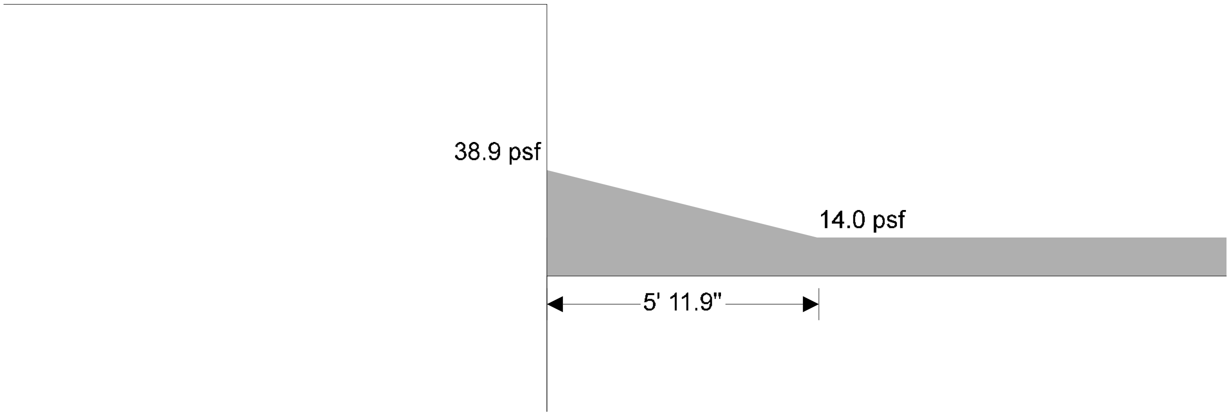
Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 9
Project Name: Express Stop – View High		



Elevation on snow drift

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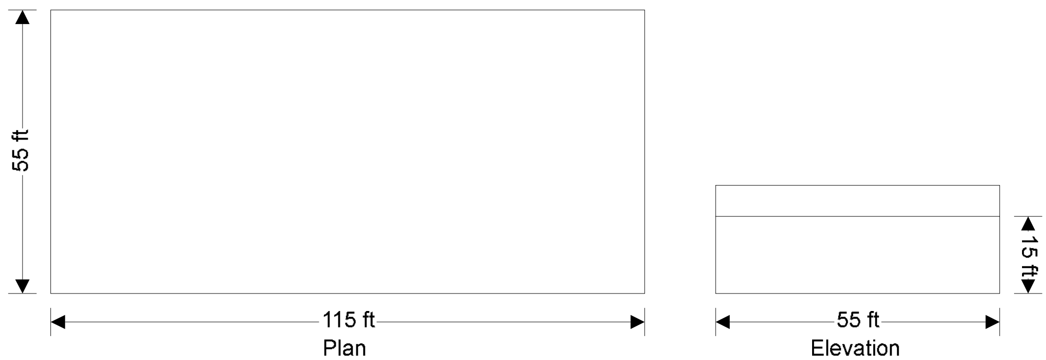
WIND LOADING

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.19



Building data

Type of roof;	Flat
Length of building;	b = **115.00** ft
Width of building;	d = **55.00** ft
Height to eaves;	H = **15.00** ft
Height of parapet;	h_p = **6.00** ft
Mean height;	h = **15.00** ft

General wind load requirements

Basic wind speed;	V = **109.0** mph
Risk category;	II
Wind directionality factor (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 roof design wind load (cl.27.1.5);	p_{min_r} = **8** psf
Minimum wall design wind load (cl.27.1.5);	p_{min_w} = **16** psf

Topography

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

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
15.00	0.85	21.98

z (ft)	K_z (Table 26.10-1)	q_z (psf)
21.00	0.91	23.47

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); q_i = **21.98** psf

Parapet pressures and forces

Velocity pressure at top of parapet; q_p = **23.47** psf

Combined net pressure coefficient, leeward; GC_{pnl} = **-1.0**

Combined net parapet pressure, leeward; $p_{pl} = q_p \times GC_{pnl}$ = **-23.47** psf

Combined net pressure coefficient, windward; GC_{pnw} = **1.5**

Combined net parapet pressure, windward; $p_{pw} = q_p \times GC_{pnw}$ = **35.21** psf

Wind direction 0 deg:

Leeward parapet force; $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-16.2** kips

Windward parapet force; $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **24.3** kips

Wind direction 90 deg:

Leeward parapet force; $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-7.7** kips

Windward parapet force; $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **11.6** kips

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)	15.00	-0.90	21.98	-20.77	862.50	-17.91
B (-ve)	15.00	-0.90	21.98	-20.77	862.50	-17.91
C (-ve)	15.00	-0.50	21.98	-13.29	1725.00	-22.93
D (-ve)	15.00	-0.30	21.98	-9.56	2875.00	-27.48

Total vertical net force; $F_{w,v}$ = **-86.24** kips

Total horizontal net force; $F_{w,h}$ = **0.00** 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	15.00	0.80	21.98	10.99	1725.00	18.95
B	15.00	-0.50	21.98	-13.29	1725.00	-22.93
C	15.00	-0.70	21.98	-17.03	825.00	-14.05
D	15.00	-0.70	21.98	-17.03	825.00	-14.05

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \times (H + h_p)$ = **2415.00** ft^2

Projected vertical area of roof; $A_{vert\_r\_0}$ = **0.00** ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **38.64** kips

Leeward net force; $F_l = F_{w,wB} + F_{w,wpl\_0} =$ **-39.1** kips
Windward net force; $F_w = F_{w,wA} + F_{w,wpw\_0} =$ **43.2** kips
Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **82.4** kips

Roof load case 2 - 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)	15.00	-0.18	21.98	0.59	862.50	0.51
B (+ve)	15.00	-0.18	21.98	0.59	862.50	0.51
C (+ve)	15.00	-0.18	21.98	0.59	1725.00	1.02
D (+ve)	15.00	-0.18	21.98	0.59	2875.00	1.71

Total vertical net force; $F_{w,v} =$ **3.75** kips
Total horizontal net force; $F_{w,h} =$ **0.00** kips

Walls load case 2 - 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	15.00	0.80	21.98	18.90	1725.00	32.60
B	15.00	-0.50	21.98	-5.38	1725.00	-9.29
C	15.00	-0.70	21.98	-9.12	825.00	-7.52
D	15.00	-0.70	21.98	-9.12	825.00	-7.52

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \times (H + h_p) =$ **2415.00** ft^2
Projected vertical area of roof; $A_{vert\_r\_0} =$ **0.00** ft^2
Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **38.64** kips
Leeward net force; $F_l = F_{w,wB} + F_{w,wpl\_0} =$ **-25.5** kips
Windward net force; $F_w = F_{w,wA} + F_{w,wpw\_0} =$ **56.9** kips
Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **82.4** 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)	15.00	-0.90	21.98	-20.77	412.50	-8.57
B (-ve)	15.00	-0.90	21.98	-20.77	412.50	-8.57
C (-ve)	15.00	-0.50	21.98	-13.29	825.00	-10.97
D (-ve)	15.00	-0.30	21.98	-9.56	4675.00	-44.69

Total vertical net force; $F_{w,v} =$ **-72.79** 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	15.00	0.80	21.98	10.99	825.00	9.06
B	15.00	-0.30	21.98	-9.47	825.00	-7.82
C	15.00	-0.70	21.98	-17.03	1725.00	-29.38
D	15.00	-0.70	21.98	-17.03	1725.00	-29.38

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \times (H + h_p) = \mathbf{1155.00}$ ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{18.48}$ kips

Leeward net force; $F_l = F_{w,wB} + F_{w,wpl\_90} = \mathbf{-15.6}$ kips

Windward net force; $F_w = F_{w,wA} + F_{w,wpw\_90} = \mathbf{20.7}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{36.2}$ 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)	15.00	-0.18	21.98	0.59	412.50	0.24
B (+ve)	15.00	-0.18	21.98	0.59	412.50	0.24
C (+ve)	15.00	-0.18	21.98	0.59	825.00	0.49
D (+ve)	15.00	-0.18	21.98	0.59	4675.00	2.77

Total vertical net force; $F_{w,v} = \mathbf{3.75}$ 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	15.00	0.80	21.98	18.90	825.00	15.59
B	15.00	-0.30	21.98	-1.56	825.00	-1.29
C	15.00	-0.70	21.98	-9.12	1725.00	-15.73
D	15.00	-0.70	21.98	-9.12	1725.00	-15.73

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \times (H + h_p) = \mathbf{1155.00}$ ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{18.48}$ kips

Leeward net force; $F_l = F_{w,wB} + F_{w,wpl\_90} = \mathbf{-9.0}$ kips

Windward net force; $F_w = F_{w,wA} + F_{w,wpw\_90} = \mathbf{27.2}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{36.2}$ kips

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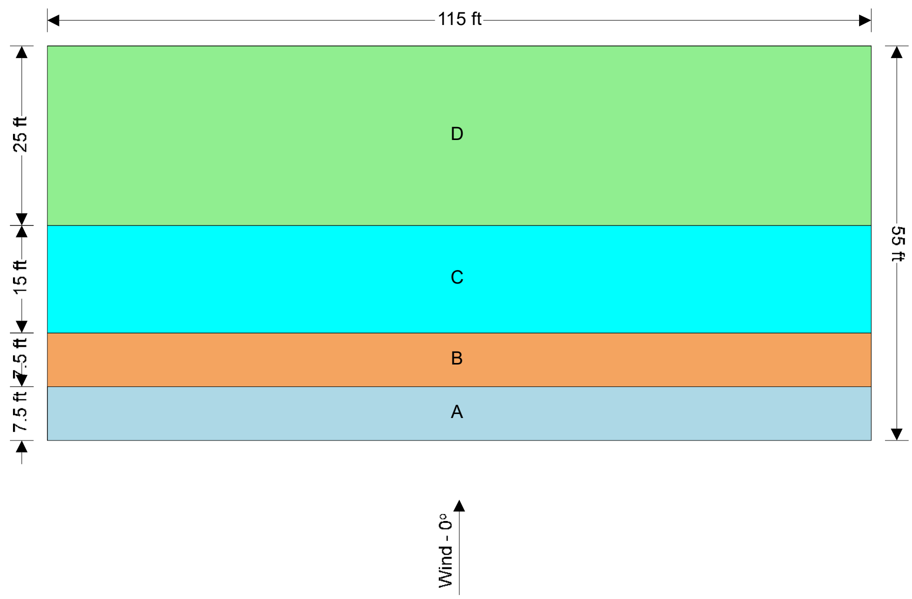
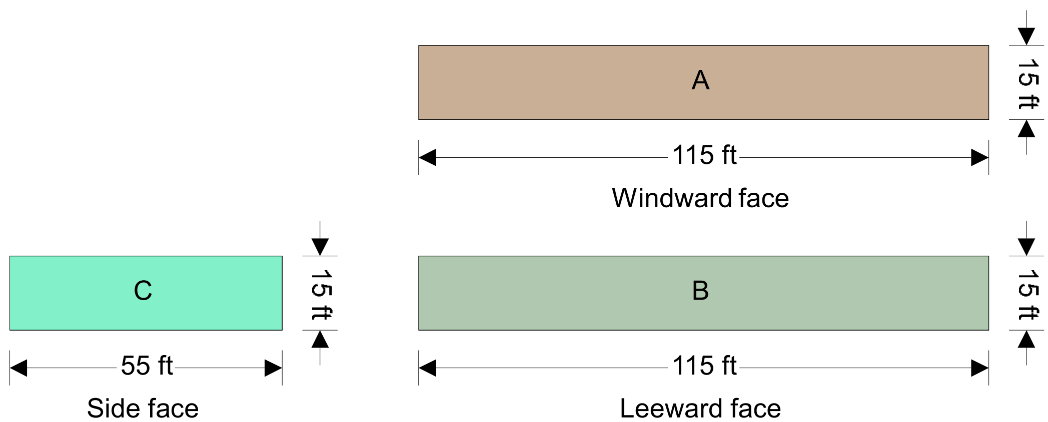
Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 14
Project Name: Express Stop – View High		



Plan view - Flat roof



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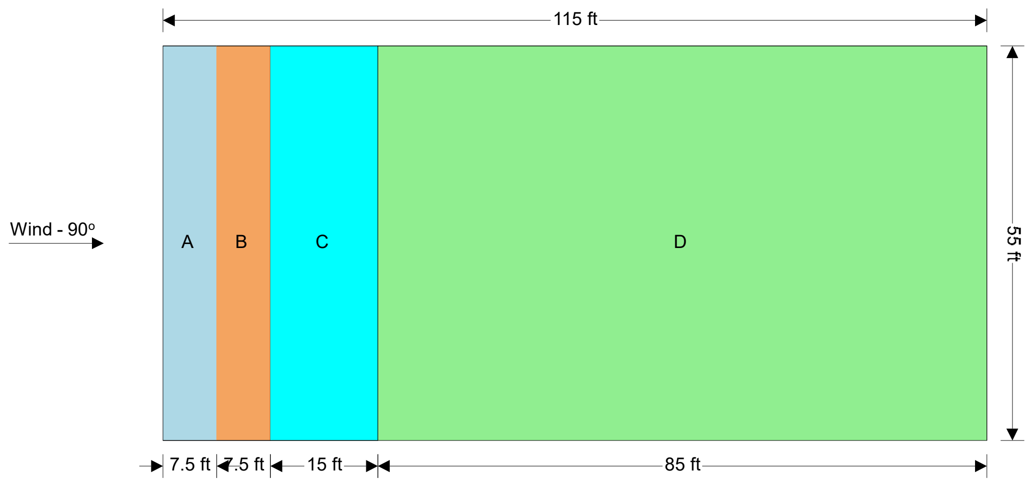
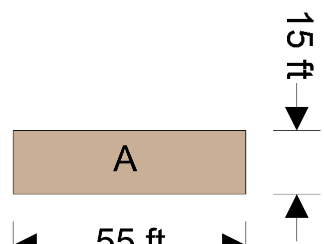
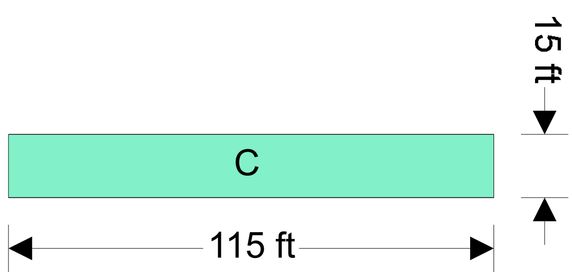
Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 15
Project Name: Express Stop – View High		



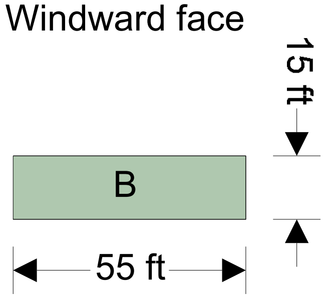
Plan view - Flat roof



Windward face



Side face



Leeward face

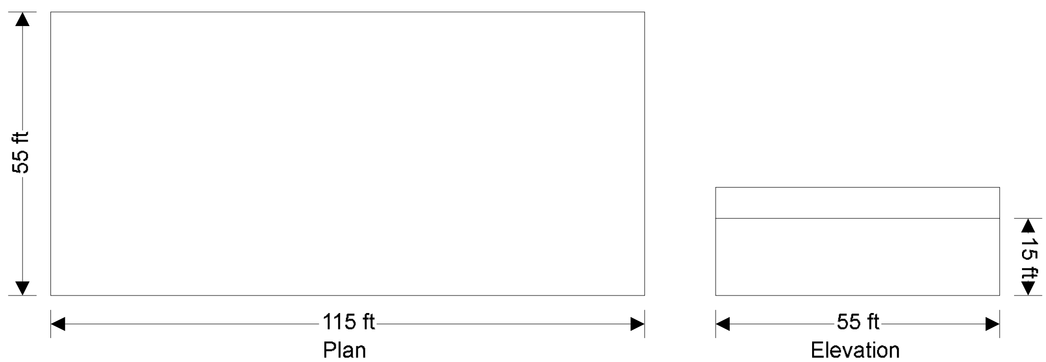
COMPONENTS AND CLADDING WIND LOADING

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.19



Building data

Type of roof;	Flat
Length of building;	b = **115.00** ft
Width of building;	d = **55.00** ft
Height to eaves;	H = **15.00** ft
Height of parapet;	h_p = **6.00** ft
Mean height;	h = **15.00** ft
End zone width;	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **5.50** ft

General wind load requirements

Basic wind speed;	V = **109.0** mph
Risk category;	II
Wind directionality factor (Table 26.6-1);	K_d = **0.85**
Ground elevation above sea level;	z_{gl} = **0** ft
Ground elevation factor;	K_e = exp(-0.0000362 × z_{gl}/1ft) = **1.00**
Exposure category (cl 26.7.3);	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**
Parapet enclosure classification;	Enclosed
Parapet internal pressure coef +ve (Table 26.13-1);	GC_{pi_pp} = **0.18**
Parapet internal pressure coef –ve (Table 26.13-1);	GC_{pi_np} = **-0.18**
Gust effect factor;	G_f = **0.85**

Topography

Topography factor not significant;	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1);	K_z = **0.85**

Velocity pressure; $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2 = \mathbf{22.0}$ psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1); $K_z = \mathbf{0.91}$

Velocity pressure; $q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2 = \mathbf{23.5}$ psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); $q_i = \mathbf{21.98}$ psf

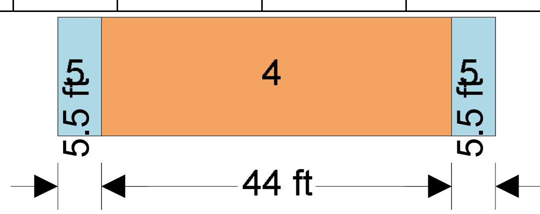
Equations used in tables

Net pressure; $p = q_h \times (GC_p - GC_{pi})$

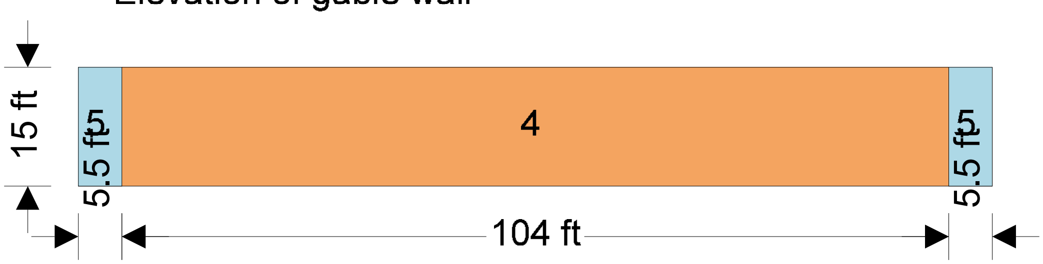
Parapet net pressure; $p = q_p \text{ x } (GC_p - GC_{pi\_p})$;

Components and cladding pressures - Wall (Table 30.3-1 and (Figure 30.3-2A))

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	0.90	-0.99	23.7	-25.7
20 sf	4	-	-	20.0	0.85	-0.94	22.7	-24.7
50 sf	4	-	-	50.0	0.79	-0.88	21.3	-23.3
>100 sf	4	-	-	100.1	0.74	-0.83	20.2	-22.2
<=10 sf	5	-	-	10.0	0.90	-1.26	23.7	-31.6
20 sf	5	-	-	20.0	0.85	-1.16	22.7	-29.5
50 sf	5	-	-	50.0	0.79	-1.04	21.3	-26.8
>100 sf	5	-	-	100.1	0.74	-0.94	20.2	-24.7



Elevation of gable wall



Elevation of side wall

Components and cladding pressures - Roof (Figure 30.3-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.30	-1.70	10.5 #	-41.3
20 sf	1	-	-	20.0	0.27	-1.58	9.9 #	-38.6

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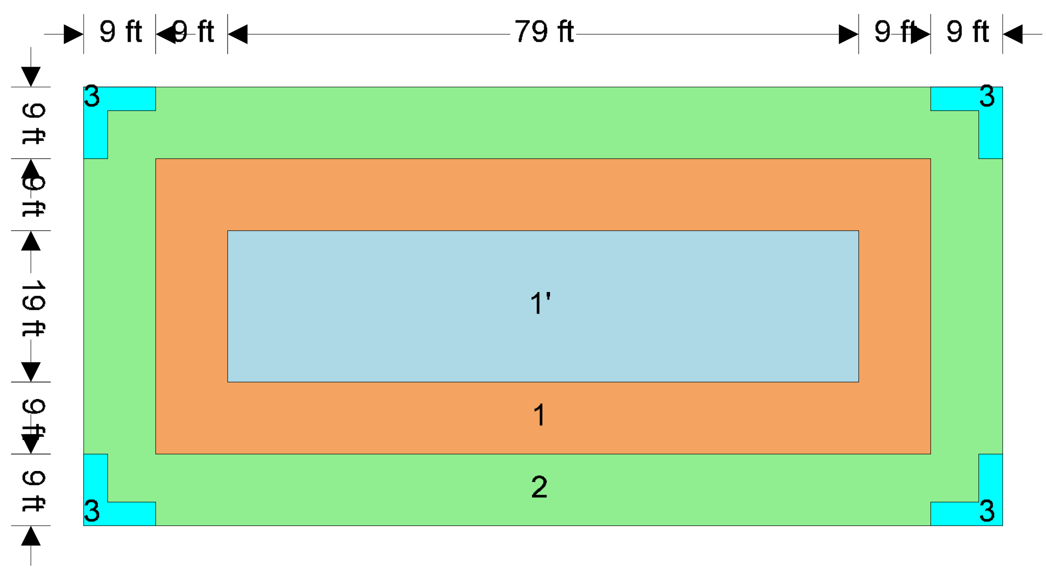
Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
50 sf	1	-	-	50.0	0.23	-1.41	9.0 #	-35.0
>100 sf	1	-	-	100.1	0.20	-1.29	8.4 #	-32.3
<=10 sf	1'	-	-	10.0	0.30	-0.90	10.5 #	-23.7
20 sf	1'	-	-	20.0	0.27	-0.90	9.9 #	-23.7
50 sf	1'	-	-	50.0	0.23	-0.90	9.0 #	-23.7
>100 sf	1'	-	-	100.1	0.20	-0.90	8.4 #	-23.7
<=10 sf	2	-	-	10.0	0.90	-2.30	23.7	-54.5
20 sf	2	-	-	20.0	0.85	-2.14	22.7	-51.0
50 sf	2	-	-	50.0	0.79	-1.93	21.3	-46.4
>100 sf	2	-	-	100.1	0.74	-1.77	20.2	-42.9
<=10 sf	3	-	-	10.0	0.90	-2.30	23.7	-54.5
20 sf	3	-	-	20.0	0.85	-2.14	22.7	-51.0
50 sf	3	-	-	50.0	0.79	-1.93	21.3	-46.4
>100 sf	3	-	-	100.1	0.74	-1.77	20.2	-42.9

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Plan on roof

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SEISMIC LOADING

SEISMIC FORCES

In accordance with ASCE 7-16

Tedds calculation version 3.1.05

Site parameters

Site class; D, Soil properties not known

Mapped acceleration parameters (Section 11.4.2)

at short period; S_S = **0.099**

at 1 sec period; S_1 = **0.068**

Site coefficient;

at short period (Table 11.4-1); F_a = **1.600**

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

Spectral response acceleration parameters

at short period (Eq. 11.4-1); $S_{MS} = F_a \times S_S =$ **0.158**

at 1 sec period (Eq. 11.4-2); $S_{M1} = F_v \times S_1 =$ **0.163**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3); $S_{DS} = 2 / 3 \times S_{MS} =$ **0.106**

at 1 sec period (Eq. 11.4-4); $S_{D1} = 2 / 3 \times S_{M1} =$ **0.109**

Seismic design category

Risk category (Table 1.5-1); II

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

A

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

B

Seismic design category; B

Approximate fundamental period

Height above base to highest level of building; h_n = **21** 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); $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x =$ **0.196** sec

Building fundamental period (Sect 12.8.2); $T = T_a =$ **0.196** sec

Long-period transition period; T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1); A. Bearing_Wall_Systems

15. Light-frame (wood) walls sheathed with wood structural panels

Response modification factor (Table 12.2-1); R = **6.5**

Seismic importance factor (Table 1.5-2);	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2);	$C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0162**
Maximum (Eq 12.8-3);	$C_{s\_max} = S_{D1} / ((T / 1\ sec) \times (R / I_e))$ = **0.0853**
Minimum (Eq.12.8-5);	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.0100**
Seismic response coefficient;	C_s = **0.0162**

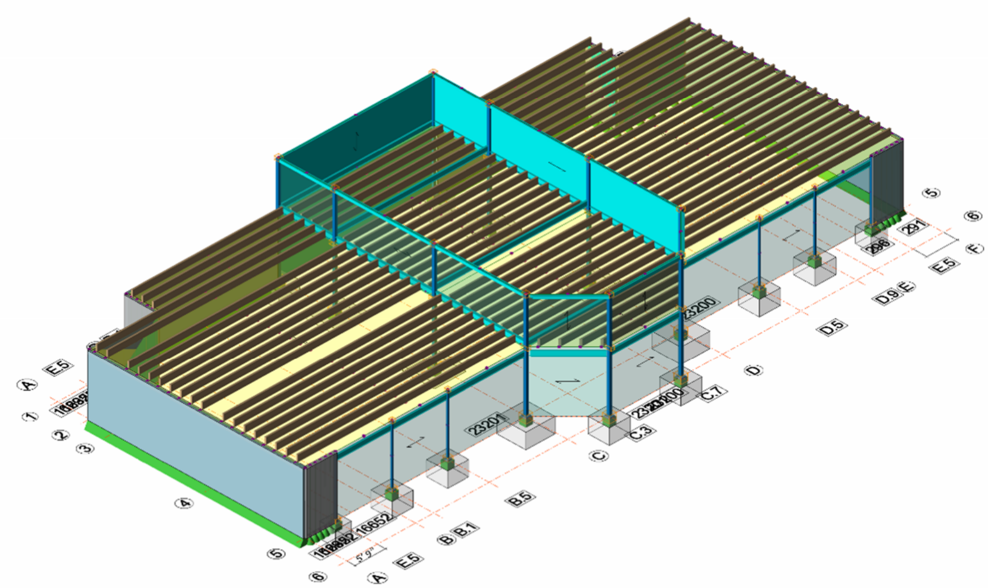
Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure;	W = **300.0** kips
Seismic response coefficient;	C_s = **0.0162**
Seismic base shear (Eq 12.8-1);	$V = C_s \times W$ = **4.9** kips

;

TEKLA STRUCTURAL DESIGNER OUTPUT

OVERALL 3D VIEW



ACTION CODES

General Loading	ASCE7 (2016)
Wind Loading	ASCE7 (2016)
Snow Loading	ASCE7 (2016)
Seismic Loading	ASCE7 (2016)
Combinations	ASCE7 (2016)

RESISTANCE CODES

Steel Design	AISC 360/341 LRFD (2016)
Concrete Design	ACI 318 (2014)
Composite Design	AISC 360/341 LRFD (2016)
Timber Design	NDS LRFD (2018)
Masonry Design	MSJC ASD (2013)
Foundation Design	No Design Code
Seismic Design and Detailing	ASCE7 (2016)
Steel Fire Design	No Design Code

COMBINATIONS

Name	Class	Active	Strength	Service
1 $LRFD_{1}$-1.4D	Gravity	●	●	●
2 $LRFD_{2}$-1.2D+1.6L	Gravity	●	●	●
3 $LRFD_{3}$-1.2D+1.6L+0.5Lr	Gravity	●	●	●
4 $LRFD_{4.1}$-1.2D+1.6L+0.5S	Gravity	●	●	●
5 $LRFD_{4.2}$-1.2D+1.6L+0.5S	Gravity	●	●	●
6 $LRFD_{4.3}$-1.2D+1.6L+0.5S	Gravity	●	●	●
7 $LRFD_{5}$-1.2D+L+1.6Lr	Gravity	●	●	●
8 $LRFD_{6.1}$-1.2D+L+1.6S	Gravity	●	●	●
9 $LRFD_{6.2}$-1.2D+L+1.6S	Gravity	●	●	●
10 $LRFD_{6.3}$-1.2D+L+1.6S	Gravity	●	●	●
11 $LRFD_{7.1}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
12 $LRFD_{7.2}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
13 $LRFD_{7.3}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
14 $LRFD_{7.4}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
15 $LRFD_{7.5}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
16 $LRFD_{7.6}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
17 $LRFD_{7.7}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
18 $LRFD_{7.8}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
19 $LRFD_{7.9}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
20 $LRFD_{7.10}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
21 $LRFD_{7.11}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
22 $LRFD_{7.12}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
23 $LRFD_{7.13}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
24 $LRFD_{7.14}$-1.2D+1.6Lr+0.5W	Lateral	●	●	●
25 $LRFD_{8.1}$-1.2D+1.6S+0.5W	Lateral	●	●	●
26 $LRFD_{8.2}$-1.2D+1.6S+0.5W	Lateral	●	●	●
27 $LRFD_{8.3}$-1.2D+1.6S+0.5W	Lateral	●	●	●
28 $LRFD_{8.4}$-1.2D+1.6S+0.5W	Lateral	●	●	●
29 $LRFD_{8.5}$-1.2D+1.6S+0.5W	Lateral	●	●	●
30 $LRFD_{8.6}$-1.2D+1.6S+0.5W	Lateral	●	●	●

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Name	Class	Active	Strength	Service
31 $LRFD_{8.7}$-1.2D+1.6S+0.5W	Lateral	●	●	●
32 $LRFD_{8.8}$-1.2D+1.6S+0.5W	Lateral	●	●	●
33 $LRFD_{8.9}$-1.2D+1.6S+0.5W	Lateral	●	●	●
34 $LRFD_{8.10}$-1.2D+1.6S+0.5W	Lateral	●	●	●
35 $LRFD_{8.11}$-1.2D+1.6S+0.5W	Lateral	●	●	●
36 $LRFD_{8.12}$-1.2D+1.6S+0.5W	Lateral	●	●	●
37 $LRFD_{8.13}$-1.2D+1.6S+0.5W	Lateral	●	●	●
38 $LRFD_{8.14}$-1.2D+1.6S+0.5W	Lateral	●	●	●
39 $LRFD_{8.15}$-1.2D+1.6S+0.5W	Lateral	●	●	●
40 $LRFD_{8.16}$-1.2D+1.6S+0.5W	Lateral	●	●	●
41 $LRFD_{8.17}$-1.2D+1.6S+0.5W	Lateral	●	●	●
42 $LRFD_{8.18}$-1.2D+1.6S+0.5W	Lateral	●	●	●
43 $LRFD_{8.19}$-1.2D+1.6S+0.5W	Lateral	●	●	●
44 $LRFD_{8.20}$-1.2D+1.6S+0.5W	Lateral	●	●	●
45 $LRFD_{8.21}$-1.2D+1.6S+0.5W	Lateral	●	●	●
46 $LRFD_{8.22}$-1.2D+1.6S+0.5W	Lateral	●	●	●
47 $LRFD_{8.23}$-1.2D+1.6S+0.5W	Lateral	●	●	●
48 $LRFD_{8.24}$-1.2D+1.6S+0.5W	Lateral	●	●	●
49 $LRFD_{8.25}$-1.2D+1.6S+0.5W	Lateral	●	●	●
50 $LRFD_{8.26}$-1.2D+1.6S+0.5W	Lateral	●	●	●
51 $LRFD_{8.27}$-1.2D+1.6S+0.5W	Lateral	●	●	●
52 $LRFD_{8.28}$-1.2D+1.6S+0.5W	Lateral	●	●	●
53 $LRFD_{8.29}$-1.2D+1.6S+0.5W	Lateral	●	●	●
54 $LRFD_{8.30}$-1.2D+1.6S+0.5W	Lateral	●	●	●
55 $LRFD_{8.31}$-1.2D+1.6S+0.5W	Lateral	●	●	●
56 $LRFD_{8.32}$-1.2D+1.6S+0.5W	Lateral	●	●	●
57 $LRFD_{8.33}$-1.2D+1.6S+0.5W	Lateral	●	●	●
58 $LRFD_{8.34}$-1.2D+1.6S+0.5W	Lateral	●	●	●
59 $LRFD_{8.35}$-1.2D+1.6S+0.5W	Lateral	●	●	●
60 $LRFD_{8.36}$-1.2D+1.6S+0.5W	Lateral	●	●	●
61 $LRFD_{8.37}$-1.2D+1.6S+0.5W	Lateral	●	●	●
62 $LRFD_{8.38}$-1.2D+1.6S+0.5W	Lateral	●	●	●
63 $LRFD_{8.39}$-1.2D+1.6S+0.5W	Lateral	●	●	●
64 $LRFD_{8.40}$-1.2D+1.6S+0.5W	Lateral	●	●	●
65 $LRFD_{8.41}$-1.2D+1.6S+0.5W	Lateral	●	●	●
66 $LRFD_{8.42}$-1.2D+1.6S+0.5W	Lateral	●	●	●
67 $LRFD_{9.1}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
69 $LRFD_{9.3}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
70 $LRFD_{9.4}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
71 $LRFD_{9.5}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
72 $LRFD_{9.6}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
73 $LRFD_{9.7}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
74 $LRFD_{9.8}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
75 $LRFD_{9.9}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
76 $LRFD_{9.10}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
77 $LRFD_{9.11}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
78 $LRFD_{9.12}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
79 $LRFD_{9.13}$-1.2D+L+0.5Lr+W	Lateral	●	●	●
80 $LRFD_{9.14}$-1.2D+L+0.5Lr+W	Lateral	●	●	●

Name	Class	Active	Strength	Service
81 $LRFD_{10.1}$-1.2D+L+0.5S+W	Lateral	●	●	●
82 $LRFD_{10.2}$-1.2D+L+0.5S+W	Lateral	●	●	●
83 $LRFD_{10.3}$-1.2D+L+0.5S+W	Lateral	●	●	●
84 $LRFD_{10.4}$-1.2D+L+0.5S+W	Lateral	●	●	●
85 $LRFD_{10.5}$-1.2D+L+0.5S+W	Lateral	●	●	●
86 $LRFD_{10.6}$-1.2D+L+0.5S+W	Lateral	●	●	●
87 $LRFD_{10.7}$-1.2D+L+0.5S+W	Lateral	●	●	●
88 $LRFD_{10.8}$-1.2D+L+0.5S+W	Lateral	●	●	●
89 $LRFD_{10.9}$-1.2D+L+0.5S+W	Lateral	●	●	●
90 $LRFD_{10.10}$-1.2D+L+0.5S+W	Lateral	●	●	●
91 $LRFD_{10.11}$-1.2D+L+0.5S+W	Lateral	●	●	●
92 $LRFD_{10.12}$-1.2D+L+0.5S+W	Lateral	●	●	●
93 $LRFD_{10.13}$-1.2D+L+0.5S+W	Lateral	●	●	●
94 $LRFD_{10.14}$-1.2D+L+0.5S+W	Lateral	●	●	●
95 $LRFD_{10.15}$-1.2D+L+0.5S+W	Lateral	●	●	●
96 $LRFD_{10.16}$-1.2D+L+0.5S+W	Lateral	●	●	●
97 $LRFD_{10.17}$-1.2D+L+0.5S+W	Lateral	●	●	●
98 $LRFD_{10.18}$-1.2D+L+0.5S+W	Lateral	●	●	●
99 $LRFD_{10.19}$-1.2D+L+0.5S+W	Lateral	●	●	●
100 $LRFD_{10.20}$-1.2D+L+0.5S+W	Lateral	●	●	●
101 $LRFD_{10.21}$-1.2D+L+0.5S+W	Lateral	●	●	●
102 $LRFD_{10.22}$-1.2D+L+0.5S+W	Lateral	●	●	●
103 $LRFD_{10.23}$-1.2D+L+0.5S+W	Lateral	●	●	●
104 $LRFD_{10.24}$-1.2D+L+0.5S+W	Lateral	●	●	●
105 $LRFD_{10.25}$-1.2D+L+0.5S+W	Lateral	●	●	●
106 $LRFD_{10.26}$-1.2D+L+0.5S+W	Lateral	●	●	●
107 $LRFD_{10.27}$-1.2D+L+0.5S+W	Lateral	●	●	●
108 $LRFD_{10.28}$-1.2D+L+0.5S+W	Lateral	●	●	●
109 $LRFD_{10.29}$-1.2D+L+0.5S+W	Lateral	●	●	●
110 $LRFD_{10.30}$-1.2D+L+0.5S+W	Lateral	●	●	●
111 $LRFD_{10.31}$-1.2D+L+0.5S+W	Lateral	●	●	●
112 $LRFD_{10.32}$-1.2D+L+0.5S+W	Lateral	●	●	●
113 $LRFD_{10.33}$-1.2D+L+0.5S+W	Lateral	●	●	●
114 $LRFD_{10.34}$-1.2D+L+0.5S+W	Lateral	●	●	●
115 $LRFD_{10.35}$-1.2D+L+0.5S+W	Lateral	●	●	●
116 $LRFD_{10.36}$-1.2D+L+0.5S+W	Lateral	●	●	●
117 $LRFD_{10.37}$-1.2D+L+0.5S+W	Lateral	●	●	●
118 $LRFD_{10.38}$-1.2D+L+0.5S+W	Lateral	●	●	●
119 $LRFD_{10.39}$-1.2D+L+0.5S+W	Lateral	●	●	●
120 $LRFD_{10.40}$-1.2D+L+0.5S+W	Lateral	●	●	●
121 $LRFD_{10.41}$-1.2D+L+0.5S+W	Lateral	●	●	●
122 $LRFD_{10.42}$-1.2D+L+0.5S+W	Lateral	●	●	●
123 $LRFD_{12.1}$-0.9D+W	Lateral	●	●	●
124 $LRFD_{12.2}$-0.9D+W	Lateral	●	●	●
125 $LRFD_{12.3}$-0.9D+W	Lateral	●	●	●
126 $LRFD_{12.4}$-0.9D+W	Lateral	●	●	●
127 $LRFD_{12.5}$-0.9D+W	Lateral	●	●	●
128 $LRFD_{12.6}$-0.9D+W	Lateral	●	●	●
129 $LRFD_{12.7}$-0.9D+W	Lateral	●	●	●
130 $LRFD_{12.8}$-0.9D+W	Lateral	●	●	●

Name	Class	Active	Strength	Service
131 LRFD$_{12.9}$-0.9D+W	Lateral	●	●	●
132 LRFD$_{12.10}$-0.9D+W	Lateral	●	●	●
133 LRFD$_{12.11}$-0.9D+W	Lateral	●	●	●
134 LRFD$_{12.12}$-0.9D+W	Lateral	●	●	●
135 LRFD$_{12.13}$-0.9D+W	Lateral	●	●	●
136 LRFD$_{12.14}$-0.9D+W	Lateral	●	●	●
137 Effective Seismic Weight	Modal Mass	●	●	
138 LRFD$_{11.1}$-1.2D+L+0.2S+E	Seismic	●	●	●
139 LRFD$_{11.2}$-1.2D+L+0.2S+E	Seismic	●	●	●
140 LRFD$_{11.3}$-1.2D+L+0.2S+E	Seismic	●	●	●
141 LRFD$_{11.4}$-1.2D+L+0.2S+E	Seismic	●	●	●
142 LRFD$_{13.1}$-0.9D+E	Seismic	●	●	●
143 LRFD$_{13.2}$-0.9D+E	Seismic	●	●	●
144 LRFD$_{13.3}$-0.9D+E	Seismic	●	●	●
145 LRFD$_{13.4}$-0.9D+E	Seismic	●	●	●

1 LRFD$_1$-1.4D

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.400	1.000
2 Slab self weight	1.400	1.000
3 Dead	1.400	1.000

2 LRFD$_2$-1.2D+1.6L

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000

3 LRFD$_3$-1.2D+1.6L+0.5Lr

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
5 Roof Live	0.500	0.750

4 LRFD$_{4.1}$-1.2D+1.6L+0.5S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
20 Minimum Snow Load	0.500	0.750

5 LRFD$_{4.2}$-1.2D+1.6L+0.5S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
21 Balanced Snow Load	0.500	0.750

6 LRFD$_{4.3}$-1.2D+1.6L+0.5S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
22 Drift Snow Load 1	0.500	0.750

7 $LRFD_{5}$-1.2D+L+1.6Lr

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
5 Roof Live	1.600	1.000

8 $LRFD_{6.1}$-1.2D+L+1.6S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
20 Minimum Snow Load	1.600	1.000

9 $LRFD_{6.2}$-1.2D+L+1.6S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
21 Balanced Snow Load	1.600	1.000

10 $LRFD_{6.3}$-1.2D+L+1.6S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
22 Drift Snow Load 1	1.600	1.000

11 $LRFD_{7.1}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 Wind +X',GCpi 0.18,-Cp	0.500	0.500
5 Roof Live	1.600	1.000

12 $LRFD_{7.2}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
7 Wind +X',GCpi 0.18,+Cp	0.500	0.500
5 Roof Live	1.600	1.000

13 LRFD$_{7.3}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
8 Wind +X'+Y',GCpi 0.18	0.500	0.500
5 Roof Live	1.600	1.000

14 LRFD$_{7.4}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
9 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
5 Roof Live	1.600	1.000

15 LRFD$_{7.5}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
10 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
5 Roof Live	1.600	1.000

16 LRFD$_{7.6}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
11 Wind -X'+Y',GCpi 0.18	0.500	0.500
5 Roof Live	1.600	1.000

17 LRFD$_{7.7}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
12 Wind -X',GCpi 0.18,-Cp	0.500	0.500
5 Roof Live	1.600	1.000

18 LRFD$_{7.8}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
13 Wind -X',GCpi 0.18,+Cp	0.500	0.500
5 Roof Live	1.600	1.000

19 LRFD$_{7.9}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000

Loadcase Title	Strength	Service
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
14 Wind -X'-Y',GCpi 0.18	0.500	0.500
5 Roof Live	1.600	1.000

20 $LRFD_{7.10}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
15 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
5 Roof Live	1.600	1.000

21 $LRFD_{7.11}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
16 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
5 Roof Live	1.600	1.000

22 $LRFD_{7.12}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
17 Wind +X'-Y',GCpi 0.18	0.500	0.500
5 Roof Live	1.600	1.000

23 $LRFD_{7.13}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
18 Wind +X' - Min Design Load	0.500	0.500
5 Roof Live	1.600	1.000

24 $LRFD_{7.14}$-1.2D+1.6Lr+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
19 Wind +Y' - Min Design Load	0.500	0.500
5 Roof Live	1.600	1.000

25 $LRFD_{8.1}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000

Loadcase Title	Strength	Service
6 Wind +X',GCpi 0.18,-Cp	0.500	0.500
20 Minimum Snow Load	1.600	1.000

26 LRFD$_{8.2}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
7 Wind +X',GCpi 0.18,+Cp	0.500	0.500
20 Minimum Snow Load	1.600	1.000

27 LRFD$_{8.3}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
8 Wind +X'+Y',GCpi 0.18	0.500	0.500
20 Minimum Snow Load	1.600	1.000

28 LRFD$_{8.4}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
9 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
20 Minimum Snow Load	1.600	1.000

29 LRFD$_{8.5}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
10 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
20 Minimum Snow Load	1.600	1.000

30 LRFD$_{8.6}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
11 Wind -X'+Y',GCpi 0.18	0.500	0.500
20 Minimum Snow Load	1.600	1.000

31 LRFD$_{8.7}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
12 Wind -X',GCpi 0.18,-Cp	0.500	0.500
20 Minimum Snow Load	1.600	1.000

32 LRFD$_{8.8}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
13 Wind -X',GCpi 0.18,+Cp	0.500	0.500
20 Minimum Snow Load	1.600	1.000

33 LRFD$_{8.9}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
14 Wind -X'-Y',GCpi 0.18	0.500	0.500
20 Minimum Snow Load	1.600	1.000

34 LRFD$_{8.10}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
15 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
20 Minimum Snow Load	1.600	1.000

35 LRFD$_{8.11}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
16 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
20 Minimum Snow Load	1.600	1.000

36 LRFD$_{8.12}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
17 Wind +X'-Y',GCpi 0.18	0.500	0.500
20 Minimum Snow Load	1.600	1.000

37 LRFD$_{8.13}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
18 Wind +X' - Min Design Load	0.500	0.500
20 Minimum Snow Load	1.600	1.000

38 LRFD$_{8.14}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
19 Wind +Y' - Min Design Load	0.500	0.500
20 Minimum Snow Load	1.600	1.000

39 $LRFD_{8.15}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 Wind +X',GCpi 0.18,-Cp	0.500	0.500
21 Balanced Snow Load	1.600	1.000

40 $LRFD_{8.16}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
7 Wind +X',GCpi 0.18,+Cp	0.500	0.500
21 Balanced Snow Load	1.600	1.000

41 $LRFD_{8.17}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
8 Wind +X'+Y',GCpi 0.18	0.500	0.500
21 Balanced Snow Load	1.600	1.000

42 $LRFD_{8.18}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
9 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
21 Balanced Snow Load	1.600	1.000

43 $LRFD_{8.19}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
10 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
21 Balanced Snow Load	1.600	1.000

44 $LRFD_{8.20}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000

Loadcase Title	Strength	Service
3 Dead	1.200	1.000
11 Wind -X'+Y',GCpi 0.18	0.500	0.500
21 Balanced Snow Load	1.600	1.000

45 $LRFD_{8.21}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
12 Wind -X',GCpi 0.18,-Cp	0.500	0.500
21 Balanced Snow Load	1.600	1.000

46 $LRFD_{8.22}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
13 Wind -X',GCpi 0.18,+Cp	0.500	0.500
21 Balanced Snow Load	1.600	1.000

47 $LRFD_{8.23}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
14 Wind -X'-Y',GCpi 0.18	0.500	0.500
21 Balanced Snow Load	1.600	1.000

48 $LRFD_{8.24}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
15 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
21 Balanced Snow Load	1.600	1.000

49 $LRFD_{8.25}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
16 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
21 Balanced Snow Load	1.600	1.000

50 $LRFD_{8.26}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
17 Wind +X'-Y',GCpi 0.18	0.500	0.500

Loadcase Title	Strength	Service
21 Balanced Snow Load	1.600	1.000

51 LRFD$_{8.27}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
18 Wind +X' - Min Design Load	0.500	0.500
21 Balanced Snow Load	1.600	1.000

52 LRFD$_{8.28}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
19 Wind +Y' - Min Design Load	0.500	0.500
21 Balanced Snow Load	1.600	1.000

53 LRFD$_{8.29}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 Wind +X',GCpi 0.18,-Cp	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

54 LRFD$_{8.30}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
7 Wind +X',GCpi 0.18,+Cp	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

55 LRFD$_{8.31}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
8 Wind +X'+Y',GCpi 0.18	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

56 LRFD$_{8.32}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
9 Wind +Y',GCpi 0.18,-Cp	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

57 LRFD$_{8.33}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
10 Wind +Y',GCpi 0.18,+Cp	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

58 LRFD$_{8.34}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
11 Wind -X'+Y',GCpi 0.18	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

59 LRFD$_{8.35}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
12 Wind -X',GCpi 0.18,-Cp	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

60 LRFD$_{8.36}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
13 Wind -X',GCpi 0.18,+Cp	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

61 LRFD$_{8.37}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
14 Wind -X'-Y',GCpi 0.18	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

62 LRFD$_{8.38}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
15 Wind -Y',GCpi 0.18,-Cp	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

63 LRFD$_{8.39}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000

Loadcase Title	Strength	Service
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
16 Wind -Y',GCpi 0.18,+Cp	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

64 LRFD$_{8.40}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
17 Wind +X'-Y',GCpi 0.18	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

65 LRFD$_{8.41}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
18 Wind +X' - Min Design Load	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

66 LRFD$_{8.42}$-1.2D+1.6S+0.5W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
19 Wind +Y' - Min Design Load	0.500	0.500
22 Drift Snow Load 1	1.600	1.000

67 LRFD$_{9.1}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 Wind +X',GCpi 0.18,-Cp	1.000	0.450
5 Roof Live	0.500	0.750

68 LRFD$_{9.2}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
7 Wind +X',GCpi 0.18,+Cp	1.000	0.450
5 Roof Live	0.500	0.750

69 LRFD$_{9.3}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000

Loadcase Title	Strength	Service
8 Wind +X'+Y',GCpi 0.18	1.000	0.450
5 Roof Live	0.500	0.750

70 $LRFD_{9.4}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
9 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
5 Roof Live	0.500	0.750

71 $LRFD_{9.5}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
10 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
5 Roof Live	0.500	0.750

72 $LRFD_{9.6}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
11 Wind -X'+Y',GCpi 0.18	1.000	0.450
5 Roof Live	0.500	0.750

73 $LRFD_{9.7}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
12 Wind -X',GCpi 0.18,-Cp	1.000	0.450
5 Roof Live	0.500	0.750

74 $LRFD_{9.8}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
13 Wind -X',GCpi 0.18,+Cp	1.000	0.450
5 Roof Live	0.500	0.750

75 $LRFD_{9.9}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
14 Wind -X'-Y',GCpi 0.18	1.000	0.450
5 Roof Live	0.500	0.750

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76 $LRFD_{9.10}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
15 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
5 Roof Live	0.500	0.750

77 $LRFD_{9.11}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
16 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
5 Roof Live	0.500	0.750

78 $LRFD_{9.12}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
17 Wind +X'-Y',GCpi 0.18	1.000	0.450
5 Roof Live	0.500	0.750

79 $LRFD_{9.13}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
18 Wind +X' - Min Design Load	1.000	0.450
5 Roof Live	0.500	0.750

80 $LRFD_{9.14}$-1.2D+L+0.5Lr+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
19 Wind +Y' - Min Design Load	1.000	0.450
5 Roof Live	0.500	0.750

81 $LRFD_{10.1}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 Wind +X',GCpi 0.18,-Cp	1.000	0.450
20 Minimum Snow Load	0.500	0.750

82 $LRFD_{10.2}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
7 Wind +X',GCpi 0.18,+Cp	1.000	0.450
20 Minimum Snow Load	0.500	0.750

83 $LRFD_{10.3}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
8 Wind +X'+Y',GCpi 0.18	1.000	0.450
20 Minimum Snow Load	0.500	0.750

84 $LRFD_{10.4}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
9 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
20 Minimum Snow Load	0.500	0.750

85 $LRFD_{10.5}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
10 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
20 Minimum Snow Load	0.500	0.750

86 $LRFD_{10.6}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
11 Wind -X'+Y',GCpi 0.18	1.000	0.450
20 Minimum Snow Load	0.500	0.750

87 $LRFD_{10.7}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
12 Wind -X',GCpi 0.18,-Cp	1.000	0.450
20 Minimum Snow Load	0.500	0.750

88 $LRFD_{10.8}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000

Loadcase Title	Strength	Service
3 Dead	1.200	1.000
13 Wind -X',GCpi 0.18,+Cp	1.000	0.450
20 Minimum Snow Load	0.500	0.750

89 $LRFD_{10.9}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
14 Wind -X'-Y',GCpi 0.18	1.000	0.450
20 Minimum Snow Load	0.500	0.750

90 $LRFD_{10.10}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
15 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
20 Minimum Snow Load	0.500	0.750

91 $LRFD_{10.11}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
16 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
20 Minimum Snow Load	0.500	0.750

92 $LRFD_{10.12}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
17 Wind +X'-Y',GCpi 0.18	1.000	0.450
20 Minimum Snow Load	0.500	0.750

93 $LRFD_{10.13}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
18 Wind +X' - Min Design Load	1.000	0.450
20 Minimum Snow Load	0.500	0.750

94 $LRFD_{10.14}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
19 Wind +Y' - Min Design Load	1.000	0.450

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Loadcase Title	Strength	Service
20 Minimum Snow Load	0.500	0.750

95 $LRFD_{10.15}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 Wind +X',GCpi 0.18,-Cp	1.000	0.450
21 Balanced Snow Load	0.500	0.750

96 $LRFD_{10.16}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
7 Wind +X',GCpi 0.18,+Cp	1.000	0.450
21 Balanced Snow Load	0.500	0.750

97 $LRFD_{10.17}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
8 Wind +X'+Y',GCpi 0.18	1.000	0.450
21 Balanced Snow Load	0.500	0.750

98 $LRFD_{10.18}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
9 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
21 Balanced Snow Load	0.500	0.750

99 $LRFD_{10.19}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
10 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
21 Balanced Snow Load	0.500	0.750

100 $LRFD_{10.20}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
11 Wind -X'+Y',GCpi 0.18	1.000	0.450
21 Balanced Snow Load	0.500	0.750

101 $LRFD_{10.21}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
12 Wind -X',GCpi 0.18,-Cp	1.000	0.450
21 Balanced Snow Load	0.500	0.750

102 $LRFD_{10.22}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
13 Wind -X',GCpi 0.18,+Cp	1.000	0.450
21 Balanced Snow Load	0.500	0.750

103 $LRFD_{10.23}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
14 Wind -X'-Y',GCpi 0.18	1.000	0.450
21 Balanced Snow Load	0.500	0.750

104 $LRFD_{10.24}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
15 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
21 Balanced Snow Load	0.500	0.750

105 $LRFD_{10.25}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
16 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
21 Balanced Snow Load	0.500	0.750

106 $LRFD_{10.26}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
17 Wind +X'-Y',GCpi 0.18	1.000	0.450
21 Balanced Snow Load	0.500	0.750

107 $LRFD_{10.27}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000

Loadcase Title	Strength	Service
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
18 Wind +X' - Min Design Load	1.000	0.450
21 Balanced Snow Load	0.500	0.750

108 $LRFD_{10.28}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
19 Wind +Y' - Min Design Load	1.000	0.450
21 Balanced Snow Load	0.500	0.750

109 $LRFD_{10.29}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
6 Wind +X',GCpi 0.18,-Cp	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

110 $LRFD_{10.30}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
7 Wind +X',GCpi 0.18,+Cp	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

111 $LRFD_{10.31}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
8 Wind +X'+Y',GCpi 0.18	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

112 $LRFD_{10.32}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
9 Wind +Y',GCpi 0.18,-Cp	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

113 $LRFD_{10.33}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000

Loadcase Title	Strength	Service
10 Wind +Y',GCpi 0.18,+Cp	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

114 $LRFD_{10.34}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
11 Wind -X'+Y',GCpi 0.18	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

115 $LRFD_{10.35}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
12 Wind -X',GCpi 0.18,-Cp	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

116 $LRFD_{10.36}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
13 Wind -X',GCpi 0.18,+Cp	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

117 $LRFD_{10.37}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
14 Wind -X'-Y',GCpi 0.18	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

118 $LRFD_{10.38}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
15 Wind -Y',GCpi 0.18,-Cp	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

119 $LRFD_{10.39}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
16 Wind -Y',GCpi 0.18,+Cp	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

120 $LRFD_{10.40}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
17 Wind +X'-Y',GCpi 0.18	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

121 $LRFD_{10.41}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
18 Wind +X' - Min Design Load	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

122 $LRFD_{10.42}$-1.2D+L+0.5S+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
19 Wind +Y' - Min Design Load	1.000	0.450
22 Drift Snow Load 1	0.500	0.750

123 $LRFD_{12.1}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
6 Wind +X',GCpi 0.18,-Cp	1.000	0.600

124 $LRFD_{12.2}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
7 Wind +X',GCpi 0.18,+Cp	1.000	0.600

125 $LRFD_{12.3}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
8 Wind +X'+Y',GCpi 0.18	1.000	0.600

126 $LRFD_{12.4}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600

Loadcase Title	Strength	Service
3 Dead	0.900	0.600
9 Wind +Y',GCpi 0.18,-Cp	1.000	0.600

127 $LRFD_{12.5}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
10 Wind +Y',GCpi 0.18,+Cp	1.000	0.600

128 $LRFD_{12.6}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
11 Wind -X'+Y',GCpi 0.18	1.000	0.600

129 $LRFD_{12.7}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
12 Wind -X',GCpi 0.18,-Cp	1.000	0.600

130 $LRFD_{12.8}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
13 Wind -X',GCpi 0.18,+Cp	1.000	0.600

131 $LRFD_{12.9}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
14 Wind -X'-Y',GCpi 0.18	1.000	0.600

132 $LRFD_{12.10}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
15 Wind -Y',GCpi 0.18,-Cp	1.000	0.600

133 $LRFD_{12.11}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600

Loadcase Title	Strength	Service
3 Dead	0.900	0.600
16 Wind -Y',GCpi 0.18,+Cp	1.000	0.600

134 LRFD$_{12.12}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
17 Wind +X'-Y',GCpi 0.18	1.000	0.600

135 LRFD$_{12.13}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
18 Wind +X' - Min Design Load	1.000	0.600

136 LRFD$_{12.14}$-0.9D+W

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.900	0.600
2 Slab self weight	0.900	0.600
3 Dead	0.900	0.600
19 Wind +Y' - Min Design Load	1.000	0.600

137 Effective Seismic Weight

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.000	1.000
2 Slab self weight	1.000	1.000
3 Dead	1.000	1.000

138 LRFD$_{11.1}$-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
21 Balanced Snow Load	0.200	0.000
23 Seismic Dir1	1.000	0.700

139 LRFD$_{11.2}$-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
21 Balanced Snow Load	0.200	0.000
23 Seismic Dir1	-1.000	-0.700

140 LRFD$_{11.3}$-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015

Loadcase Title	Strength	Service
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
21 Balanced Snow Load	0.200	0.000
24 Seismic Dir2	1.000	0.700

141 $LRFD_{11.4}$-1.2D+L+0.2S+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.221	1.015
2 Slab self weight	1.221	1.015
3 Dead	1.221	1.015
21 Balanced Snow Load	0.200	0.000
24 Seismic Dir2	-1.000	-0.700

142 $LRFD_{13.1}$-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
23 Seismic Dir1	1.000	0.700

143 $LRFD_{13.2}$-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
23 Seismic Dir1	-1.000	-0.700

144 $LRFD_{13.3}$-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
24 Seismic Dir2	1.000	0.700

145 $LRFD_{13.4}$-0.9D+E

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	0.879	0.585
2 Slab self weight	0.879	0.585
3 Dead	0.879	0.585
24 Seismic Dir2	-1.000	-0.700

209 SE Douglas St.
Lee's Summit, MO 64063
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WIND DATA

General

Method
ASCE/SEI 7-16 : Directional Procedure Part 1

Site Details

Site Ground Level	0″	ft, in
Orientation of Principal Axes relative to Global axes	0.0000	Â°
Mean Roof Height, h	12′ 0″	ft, in
Level of Highest Opening in Building (Use Mean Roof Height)	12′ 0″	ft, in
Overall Building X' Dimension	113′ 0″	ft, in
Overall Building Y' Dimension	54′ 9 1/8″	ft, in
Torsion Design Pressure Factor	75.00	%
Torsion Eccentricity	15.00	%
Wind Loading		
Basic Wind Speed, V	109.00	mph
Directionality Factor, K_d	0.850	
Ground Elevation above Sea Level, z_g	0	ft
Ground Elevation Factor, K_e	1.000	
Enclosure Classification	Enclosed	
Gust Effect factor, G	0.850	
Principal Axis	+X'	
Exposure Category	B	
Topographic Feature	None	
Principal Axis	+Y'	
Exposure Category	B	
Topographic Feature	None	
Principal Axis	-X'	
Exposure Category	B	
Topographic Feature	None	
Principal Axis	-Y'	
Exposure Category	B	
Topographic Feature	None	

Intermediate Factors

+X'

Height above ground, z	12′ 0″	ft, in
Exposure Coefficient, K_z	0.575	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	15′ 0″	ft, in
Exposure Coefficient, K_z	0.575	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	21′ 0″	ft, in
Exposure Coefficient, K_z	0.633	
Topographic Factor, K_{zt}	1.000	

+Y'

Height above ground, z	12′ 0″	ft, in
Exposure Coefficient, K_z	0.575	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	15′ 0″	ft, in

Exposure Coefficient, K_z 0.575
Topographic Factor, K_{zt} 1.000
Height above ground, z 21' 0" ft, in
Exposure Coefficient, K_z 0.633
Topographic Factor, K_{zt} 1.000

-X'

Height above ground, z 12' 0" ft, in
Exposure Coefficient, K_z 0.575
Topographic Factor, K_{zt} 1.000
Height above ground, z 15' 0" ft, in
Exposure Coefficient, K_z 0.575
Topographic Factor, K_{zt} 1.000
Height above ground, z 21' 0" ft, in
Exposure Coefficient, K_z 0.633
Topographic Factor, K_{zt} 1.000

-Y'

Height above ground, z 12' 0" ft, in
Exposure Coefficient, K_z 0.575
Topographic Factor, K_{zt} 1.000
Height above ground, z 15' 0" ft, in
Exposure Coefficient, K_z 0.575
Topographic Factor, K_{zt} 1.000
Height above ground, z 21' 0" ft, in
Exposure Coefficient, K_z 0.633
Topographic Factor, K_{zt} 1.000

Velocity Pressures

+X'

Height above ground, z 12' 0" ft, in
Velocity Pressure, q_z 14.8 psf
Height above ground, z 15' 0" ft, in
Velocity Pressure, q_z 14.8 psf
Height above ground, z 21' 0" ft, in
Velocity Pressure, q_z 16.3 psf

+Y'

Height above ground, z 12' 0" ft, in
Velocity Pressure, q_z 14.8 psf
Height above ground, z 15' 0" ft, in
Velocity Pressure, q_z 14.8 psf
Height above ground, z 21' 0" ft, in
Velocity Pressure, q_z 16.3 psf

-X'

Height above ground, z 12' 0" ft, in
Velocity Pressure, q_z 14.8 psf
Height above ground, z 15' 0" ft, in
Velocity Pressure, q_z 14.8 psf
Height above ground, z 21' 0" ft, in

Velocity Pressure, q_z	16.3	psf

-Y'

Height above ground, z	12' 0"	ft, in
Velocity Pressure, q_z	14.8	psf
Height above ground, z	15' 0"	ft, in
Velocity Pressure, q_z	14.8	psf
Height above ground, z	21' 0"	ft, in
Velocity Pressure, q_z	16.3	psf

Minimum Design Wind Loads

Roof panels: All windward roof panels have 8 psf applied to them. The load is applied to the area of the panel projected at right angles to the wind direction.

Wall panels: All windward wall panels have 16 psf applied to them. The load is applied to the area of the panel projected at right angles to the wind direction.

Generated Loadcases

11 Wind -X'+Y',GCpi 0.18

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load [kip]
WI 4	WX12	7.4	134	1.0
WI 3	WY12	5.6	502	2.8
WI 5	WY12	5.6	162	0.9
WI 6	Zero	0.0	134	0.0
WI 7	WY12	5.6	502	2.8
WI 8	LX	-4.8	490	-2.4
WI 9	LY	-6.7	156	-1.1
WI 10	LX	-4.8	72	-0.3
WI 11	LY	-6.7	1044	-7.0
WI 12	WX12	5.6	72	0.4
WI 13	LY	-6.7	156	-1.1
WI 14	WX12	5.6	490	2.7
WI 15	Zero	0.0	100	0.0
WI 16	WPY	18.4	122	2.2
WI 17	WPX	24.5	100	2.5
WI 18	LPX	-12.3	422	-5.2
WI 19	LPY	-12.3	264	-3.2
WI 20	LPX	-12.3	422	-5.2

Average Wall Loads

Windward	4.1	psf
Leeward	-7.8	psf
Left	0.0	psf
Right	0.0	psf

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	-5.0	13.2	57' 2 129/256"	41' 157/256"
St. 2 (Upper T.O.S.)	-3.2	10.3	58' 1 29/64"	100' 9 189/256"
Total	**-8.3**	**23.6**	-	-

Total Loads

X	-8.3	kip
Y	23.6	kip
Z	0.0	kip

10 Wind +Y',GCpi 0.18,+Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load [kip]
WI 4	W12	7.4	134	1.0
WI 3	W12	7.4	502	3.7
WI 5	W12	7.4	162	1.2
WI 6	W12	7.4	134	1.0
WI 7	W12	7.4	502	3.7
WI 8	S	-11.5	490	-5.6
WI 9	L	-9.0	156	-1.4
WI 10	S	-11.5	72	-0.8
WI 11	L	-9.0	1044	-9.4
WI 12	S	-11.5	72	-0.8
WI 13	L	-9.0	156	-1.4
WI 14	S	-11.5	490	-5.6
WI 15	WP	24.5	100	2.5
WI 16	WP	24.5	122	3.0
WI 17	WP	24.5	100	2.5
WI 18	Zero	0.0	422	0.0
WI 19	LP	-16.3	264	-4.3
WI 20	Zero	0.0	422	0.0

Average Wall Loads

Windward	10.6	psf
Leeward	-10.2	psf
Left	-6.6	psf
Right	-6.6	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load [kip]
RI 2	1	-4.9	251	-1.2
	2	-4.9	251	-1.2
	3	-4.9	502	-2.5
	4	-4.9	879	-4.3
RI 3	1	-4.9	176	-0.9
	2	-4.9	176	-0.9
	3	-4.9	352	-1.7
	4	-4.9	670	-3.3
RI 4	1	-4.9	251	-1.2
	2	-4.9	251	-1.2
	3	-4.9	502	-2.5
	4	-4.9	879	-4.3
RI 8	1	-4.9	117	-0.6
	2	-4.9	52	-0.3

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	0.0	18.2	56' 5 63/64"	-
St. 2 (Upper T.O.S.)	0.0	14.8	56' 5 119/128"	-
Total	**0.0**	**33.0**	-	-

Total Loads

X	0.0	kip
Y	33.0	kip
Z	-26.3	kip

9 Wind +Y',GCpi 0.18,-Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	W12	7.4	134	1.0
WI 3	W12	7.4	502	3.7
WI 5	W12	7.4	162	1.2
WI 6	W12	7.4	134	1.0
WI 7	W12	7.4	502	3.7
WI 8	S	-11.5	490	-5.6
WI 9	L	-9.0	156	-1.4
WI 10	S	-11.5	72	-0.8
WI 11	L	-9.0	1044	-9.4
WI 12	S	-11.5	72	-0.8
WI 13	L	-9.0	156	-1.4
WI 14	S	-11.5	490	-5.6
WI 15	WP	24.5	100	2.5
WI 16	WP	24.5	122	3.0
WI 17	WP	24.5	100	2.5
WI 18	Zero	0.0	422	0.0
WI 19	LP	-16.3	264	-4.3
WI 20	Zero	0.0	422	0.0

Average Wall Loads

Windward	10.6	psf
Leeward	-10.2	psf
Left	-6.6	psf
Right	-6.6	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 2	1	-14.0	251	-3.5
	2	-14.0	251	-3.5
	3	-9.0	502	-4.5
	4	-6.5	879	-5.7
RI 3	1	-14.0	176	-2.5
	2	-14.0	176	-2.5
	3	-9.0	352	-3.2
	4	-6.5	670	-4.3

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load [kip]
RI 4	1	-14.0	251	-3.5
	2	-14.0	251	-3.5
	3	-9.0	502	-4.5
	4	-6.5	879	-5.7
RI 8	1	-14.0	117	-1.6
	2	-14.0	52	-0.7

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	0.0	18.2	56' 5 63/64"	-
St. 2 (Upper T.O.S.)	0.0	14.8	56' 5 119/128"	-
Total	**0.0**	**33.0**	-	-

Total Loads

X	0.0	kip
Y	33.0	kip
Z	-49.3	kip

8 Wind +X'+Y',GCpi 0.18

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load [kip]
WI 4	Zero	0.0	134	0.0
WI 3	WY12	5.6	502	2.8
WI 5	WY12	5.6	162	0.9
WI 6	WY12	7.4	134	1.0
WI 7	WY12	5.6	502	2.8
WI 8	WX12	5.6	490	2.7
WI 9	LY	-6.7	156	-1.1
WI 10	WX12	5.6	72	0.4
WI 11	LY	-6.7	1044	-7.0
WI 12	LX	-4.8	72	-0.3
WI 13	LY	-6.7	156	-1.1
WI 14	LX	-4.8	490	-2.4
WI 15	WPY	24.5	100	2.5
WI 16	WPY	18.4	122	2.2
WI 17	Zero	0.0	100	0.0
WI 18	WPX	18.4	422	7.8
WI 19	LPY	-12.3	264	-3.2
WI 20	WPX	18.4	422	7.8

Average Wall Loads

Windward	9.2	psf
Leeward	-2.8	psf
Left	0.0	psf
Right	0.0	psf

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	5.0	13.2	55' 9 15/32"	48' 3 67/128"
St. 2 (Upper T.O.S.)	3.2	10.3	54' 10 53/128"	142' 2 45/128"
Total	**8.3**	**23.6**	-	-

Total Loads

X	8.3	kip
Y	23.6	kip
Z	0.0	kip

7 Wind +X',GCpi 0.18,+Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	L	-6.4	134	-0.9
WI 3	S	-11.5	502	-5.8
WI 5	S	-11.5	162	-1.9
WI 6	W12	7.4	134	1.0
WI 7	S	-11.5	502	-5.8
WI 8	W12	7.4	490	3.6
WI 9	S	-11.5	156	-1.8
WI 10	W12	7.4	72	0.5
WI 11	S	-11.5	1044	-12.0
WI 12	L	-6.4	72	-0.5
WI 13	S	-11.5	156	-1.8
WI 14	L	-6.4	490	-3.1
WI 15	WP	24.5	100	2.5
WI 16	Zero	0.0	122	0.0
WI 17	LP	-16.3	100	-1.6
WI 18	WP	24.5	422	10.3
WI 19	Zero	0.0	264	0.0
WI 20	WP	24.5	422	10.3

Average Wall Loads

Windward	14.8	psf
Leeward	3.5	psf
Left	-10.4	psf
Right	-9.6	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 2	1	-4.9	245	-1.2
	2	-4.9	245	-1.2
	3	-4.9	490	-2.4
	4	-4.9	729	-3.6
	1	-4.9	36	-0.2
	2	-4.9	36	-0.2
	3	-4.9	72	-0.4
	4	-4.9	29	-0.1

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 3	1	-4.9	281	-1.4
	2	-4.9	281	-1.4
	3	-4.9	562	-2.8
	4	-4.9	249	-1.2
RI 4	1	-4.9	281	-1.4
	2	-4.9	281	-1.4
	3	-4.9	562	-2.8
	4	-4.9	759	-3.8
RI 8	1	-4.9	67	-0.3
	2	-4.9	64	-0.3
	3	-4.9	38	-0.2

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	7.1	1.8	46′ 11 101/256″	45′ 10 67/256″
St. 2 (Upper T.O.S.)	4.9	1.1	30′ 9 51/128″	126′ 2 119/128″
Total	**12.0**	**2.9**	-	-

Total Loads

X	12.0	kip
Y	2.9	kip
Z	-26.3	kip

6 Wind +X',GCpi 0.18,-Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	L	-6.4	134	-0.9
WI 3	S	-11.5	502	-5.8
WI 5	S	-11.5	162	-1.9
WI 6	W12	7.4	134	1.0
WI 7	S	-11.5	502	-5.8
WI 8	W12	7.4	490	3.6
WI 9	S	-11.5	156	-1.8
WI 10	W12	7.4	72	0.5
WI 11	S	-11.5	1044	-12.0
WI 12	L	-6.4	72	-0.5
WI 13	S	-11.5	156	-1.8
WI 14	L	-6.4	490	-3.1
WI 15	WP	24.5	100	2.5
WI 16	Zero	0.0	122	0.0
WI 17	LP	-16.3	100	-1.6
WI 18	WP	24.5	422	10.3
WI 19	Zero	0.0	264	0.0
WI 20	WP	24.5	422	10.3

Average Wall Loads

Windward	14.8	psf
Leeward	3.5	psf

Left -10.4 psf
Right -9.6 psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 2	1	-14.0	245	-3.4
	2	-14.0	245	-3.4
	3	-9.0	490	-4.4
	4	-6.5	729	-4.7
	1	-14.0	36	-0.5
	2	-14.0	36	-0.5
	3	-9.0	72	-0.6
	4	-6.5	29	-0.2
RI 3	1	-14.0	281	-3.9
	2	-14.0	281	-3.9
	3	-9.0	562	-5.1
	4	-6.5	249	-1.6
RI 4	1	-14.0	281	-3.9
	2	-14.0	281	-3.9
	3	-9.0	562	-5.1
	4	-6.5	759	-4.9
RI 8	1	-14.0	67	-0.9
	2	-14.0	64	-0.9
	3	-9.0	38	-0.3

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	7.1	1.8	46' 11 101/256"	45' 10 67/256"
St. 2 (Upper T.O.S.)	4.9	1.1	30' 9 51/128"	126' 2 119/128"
Total	**12.0**	**2.9**	-	-

Total Loads

X 12.0 kip
Y 2.9 kip
Z -52.4 kip

12 Wind -X',GCpi 0.18,-Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	W12	7.4	134	1.0
WI 3	S	-11.5	502	-5.8
WI 5	S	-11.5	162	-1.9
WI 6	L	-6.4	134	-0.9
WI 7	S	-11.5	502	-5.8
WI 8	L	-6.4	490	-3.1
WI 9	S	-11.5	156	-1.8
WI 10	L	-6.4	72	-0.5
WI 11	S	-11.5	1044	-12.0
WI 12	W12	7.4	72	0.5

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 13	S	-11.5	156	-1.8
WI 14	W12	7.4	490	3.6
WI 15	LP	-16.3	100	-1.6
WI 16	Zero	0.0	122	0.0
WI 17	WP	24.5	100	2.5
WI 18	LP	-16.3	422	-6.9
WI 19	Zero	0.0	264	0.0
WI 20	LP	-16.3	422	-6.9

Average Wall Loads

Windward	0.6	psf
Leeward	-10.7	psf
Left	-9.6	psf
Right	-10.4	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 2	1	-14.0	281	-3.9
	2	-14.0	281	-3.9
	3	-9.0	562	-5.1
	4	-6.5	758	-4.9
RI 3	1	-14.0	281	-3.9
	2	-14.0	281	-3.9
	3	-9.0	562	-5.1
	4	-6.5	249	-1.6
RI 4	1	-14.0	36	-0.5
	2	-14.0	36	-0.5
	3	-9.0	72	-0.6
	4	-6.5	29	-0.2
	1	-14.0	245	-3.4
	2	-14.0	245	-3.4
	3	-9.0	490	-4.4
	4	-6.5	730	-4.7
RI 8	1	-14.0	67	-0.9
	2	-14.0	64	-0.9
	3	-9.0	38	-0.3

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	-7.1	1.8	66' 91/256"	39' 33/256"
St. 2 (Upper T.O.S.)	-4.9	1.1	82' 2 45/128"	89' 7 157/256"
Total	**-12.0**	**2.9**	-	-

Total Loads

X	-12.0	kip
Y	2.9	kip
Z	-52.4	kip

13 Wind -X',GCpi 0.18,+Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	W12	7.4	134	1.0
WI 3	S	-11.5	502	-5.8
WI 5	S	-11.5	162	-1.9
WI 6	L	-6.4	134	-0.9
WI 7	S	-11.5	502	-5.8
WI 8	L	-6.4	490	-3.1
WI 9	S	-11.5	156	-1.8
WI 10	L	-6.4	72	-0.5
WI 11	S	-11.5	1044	-12.0
WI 12	W12	7.4	72	0.5
WI 13	S	-11.5	156	-1.8
WI 14	W12	7.4	490	3.6
WI 15	LP	-16.3	100	-1.6
WI 16	Zero	0.0	122	0.0
WI 17	WP	24.5	100	2.5
WI 18	LP	-16.3	422	-6.9
WI 19	Zero	0.0	264	0.0
WI 20	LP	-16.3	422	-6.9

Average Wall Loads

Windward	0.6	psf
Leeward	-10.7	psf
Left	-9.6	psf
Right	-10.4	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 2	1	-4.9	281	-1.4
	2	-4.9	281	-1.4
	3	-4.9	562	-2.8
	4	-4.9	758	-3.7
RI 3	1	-4.9	281	-1.4
	2	-4.9	281	-1.4
	3	-4.9	562	-2.8
	4	-4.9	249	-1.2
RI 4	1	-4.9	36	-0.2
	2	-4.9	36	-0.2
	3	-4.9	72	-0.4
	4	-4.9	29	-0.1
	1	-4.9	245	-1.2
	2	-4.9	245	-1.2
	3	-4.9	490	-2.4
	4	-4.9	730	-3.6
RI 8	1	-4.9	67	-0.3
	2	-4.9	64	-0.3
	3	-4.9	38	-0.2

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	-7.1	1.8	66′ 91/256″	39′ 33/256″
St. 2 (Upper T.O.S.)	-4.9	1.1	82′ 2 45/128″	89′ 7 157/256″
Total	**-12.0**	**2.9**	-	-

Total Loads

X	-12.0	kip
Y	2.9	kip
Z	-26.3	kip

14 Wind -X'-Y',GCpi 0.18

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	Zero	0.0	134	0.0
WI 3	LY	-6.7	502	-3.4
WI 5	LY	-6.7	162	-1.1
WI 6	LY	-9.0	134	-1.2
WI 7	LY	-6.7	502	-3.4
WI 8	LX	-4.8	490	-2.4
WI 9	WY12	5.6	156	0.9
WI 10	LX	-4.8	72	-0.3
WI 11	WY12	5.6	1044	5.8
WI 12	WX12	5.6	72	0.4
WI 13	WY12	5.6	156	0.9
WI 14	WX12	5.6	490	2.7
WI 15	LPY	-16.3	100	-1.6
WI 16	LPY	-12.3	122	-1.5
WI 17	Zero	0.0	100	0.0
WI 18	LPX	-12.3	422	-5.2
WI 19	WPY	18.4	264	4.9
WI 20	LPX	-12.3	422	-5.2

Average Wall Loads

Windward	4.0	psf
Leeward	-8.0	psf
Left	0.0	psf
Right	0.0	psf

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	-5.0	-13.2	55′ 9 169/256″	41′ 2 37/128″
St. 2 (Upper T.O.S.)	-2.8	-10.5	55′ 3 27/32″	115′ 55/256″
Total	**-7.9**	**-23.8**	-	-

Total Loads

X	-7.9	kip
Y	-23.8	kip
Z	0.0	kip

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15 Wind -Y',GCpi 0.18,-Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	L	-9.0	134	-1.2
WI 3	L	-9.0	502	-4.5
WI 5	L	-9.0	162	-1.5
WI 6	L	-9.0	134	-1.2
WI 7	L	-9.0	502	-4.5
WI 8	S	-11.5	490	-5.6
WI 9	W12	7.4	156	1.2
WI 10	S	-11.5	72	-0.8
WI 11	W12	7.4	1044	7.8
WI 12	S	-11.5	72	-0.8
WI 13	W12	7.4	156	1.2
WI 14	S	-11.5	490	-5.6
WI 15	LP	-16.3	100	-1.6
WI 16	LP	-16.3	122	-2.0
WI 17	LP	-16.3	100	-1.6
WI 18	Zero	0.0	422	0.0
WI 19	WP	24.5	264	6.5
WI 20	Zero	0.0	422	0.0

Average Wall Loads

Windward	10.2	psf
Leeward	-10.3	psf
Left	-6.6	psf
Right	-6.6	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 2	1	-14.0	78	-1.1
	2	-14.0	78	-1.1
	3	-9.0	156	-1.4
	4	-6.5	219	-1.4
	1	-14.0	173	-2.4
	2	-14.0	173	-2.4
	3	-9.0	346	-3.1
	4	-6.5	659	-4.3
RI 3	1	-14.0	176	-2.5
	2	-14.0	176	-2.5
	3	-9.0	352	-3.2
	4	-6.5	670	-4.3
RI 4	1	-14.0	173	-2.4
	2	-14.0	173	-2.4
	3	-9.0	346	-3.1
	4	-6.5	660	-4.3
	1	-14.0	78	-1.1
	2	-14.0	78	-1.1
	3	-9.0	156	-1.4
	4	-6.5	219	-1.4

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 8	1	-14.0	140	-2.0
	2	-14.0	29	-0.4

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	0.0	-18.2	56' 5 63/64"	-
St. 2 (Upper T.O.S.)	0.0	-14.8	56' 5 119/128"	-
Total	**0.0**	**-33.0**	-	-

Total Loads

X	0.0	kip
Y	-33.0	kip
Z	-49.3	kip

16 Wind -Y',GCpi 0.18,+Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	L	-9.0	134	-1.2
WI 3	L	-9.0	502	-4.5
WI 5	L	-9.0	162	-1.5
WI 6	L	-9.0	134	-1.2
WI 7	L	-9.0	502	-4.5
WI 8	S	-11.5	490	-5.6
WI 9	W12	7.4	156	1.2
WI 10	S	-11.5	72	-0.8
WI 11	W12	7.4	1044	7.8
WI 12	S	-11.5	72	-0.8
WI 13	W12	7.4	156	1.2
WI 14	S	-11.5	490	-5.6
WI 15	LP	-16.3	100	-1.6
WI 16	LP	-16.3	122	-2.0
WI 17	LP	-16.3	100	-1.6
WI 18	Zero	0.0	422	0.0
WI 19	WP	24.5	264	6.5
WI 20	Zero	0.0	422	0.0

Average Wall Loads

Windward	10.2	psf
Leeward	-10.3	psf
Left	-6.6	psf
Right	-6.6	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
RI 2	1	-4.9	78	-0.4
	2	-4.9	78	-0.4
	3	-4.9	156	-0.8

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
	4	-4.9	219	-1.1
	1	-4.9	173	-0.9
	2	-4.9	173	-0.9
	3	-4.9	346	-1.7
	4	-4.9	659	-3.3
RI 3	1	-4.9	176	-0.9
	2	-4.9	176	-0.9
	3	-4.9	352	-1.7
	4	-4.9	670	-3.3
RI 4	1	-4.9	173	-0.9
	2	-4.9	173	-0.9
	3	-4.9	346	-1.7
	4	-4.9	660	-3.3
	1	-4.9	78	-0.4
	2	-4.9	78	-0.4
	3	-4.9	156	-0.8
	4	-4.9	219	-1.1
RI 8	1	-4.9	140	-0.7
	2	-4.9	29	-0.1

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	0.0	-18.2	56' 5 63/64"	-
St. 2 (Upper T.O.S.)	0.0	-14.8	56' 5 119/128"	-
Total	**0.0**	**-33.0**	-	-

Total Loads

X	0.0	kip
Y	-33.0	kip
Z	-26.3	kip

17 Wind +X'-Y',GCpi 0.18

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 4	LX	-6.4	134	-0.9
WI 3	LY	-6.7	502	-3.4
WI 5	LY	-6.7	162	-1.1
WI 6	Zero	0.0	134	0.0
WI 7	LY	-6.7	502	-3.4
WI 8	WX12	5.6	490	2.7
WI 9	WY12	5.6	156	0.9
WI 10	WX12	5.6	72	0.4
WI 11	WY12	5.6	1044	5.8
WI 12	LX	-4.8	72	-0.3
WI 13	WY12	5.6	156	0.9
WI 14	LX	-4.8	490	-2.4
WI 15	Zero	0.0	100	0.0
WI 16	LPY	-12.3	122	-1.5
WI 17	LPX	-16.3	100	-1.6

Reference	Zone	Design Pressure [psf]	Area [ft²]	Applied Load [kip]
WI 18	WPX	18.4	422	7.8
WI 19	WPY	18.4	264	4.9
WI 20	WPX	18.4	422	7.8

Average Wall Loads

Windward	8.9	psf
Leeward	-2.7	psf
Left	0.0	psf
Right	0.0	psf

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	4.9	-13.1	57′ 23/32″	50′ 81/128″
St. 2 (Upper T.O.S.)	2.8	-10.5	57′ 7 67/256″	166′ 5 153/256″
Total	**7.6**	**-23.5**	-	-

Total Loads

X	7.6	kip
Y	-23.5	kip
Z	0.0	kip

18 Wind +X' - Min Design Load

Wind Walls

Reference	Effective Area [ft²]	Area Load [psf]	Wind Force [kip]
WI 6	94.8	16.0	1.5
WI 8	490.4	16.0	7.8
WI 10	72.0	16.0	1.2
WI 15	71.1	16.0	1.1
WI 18	421.8	16.0	6.7

Total Loads

X	18.4	kip
Y	0.0	kip
Z	0.0	kip

19 Wind +Y' - Min Design Load

Wind Walls

Reference	Effective Area [ft²]	Area Load [psf]	Wind Force [kip]
WI 4	94.8	16.0	1.5
WI 3	502.3	16.0	8.0
WI 5	162.3	16.0	2.6
WI 6	94.8	16.0	1.5
WI 7	502.0	16.0	8.0
WI 15	71.1	16.0	1.1
WI 16	121.7	16.0	1.9
WI 17	71.1	16.0	1.1

Total Loads

X	0.0	kip
Y	25.9	kip
Z	0.0	kip

SNOW DATA

General

Method ASCE7

Basic data

Ground Snow Load, p_g	20.0	psf	ASCE7-16 Clause 7.2
Terrain Category	B (urban, suburban, wooded)		
Exposure	Partially exposed		
Exposure Factor, C_e	1.000		ASCE7-16 Clause 7.3.1
Thermal Condition	All except below		
Thermal Factor, C_t	1.000		ASCE7-16 Clause 7.3.2
Risk Category	II		
Snow Importance Factor, I_s	1.000		
Flat Roof Snow Load, p_f	14.0	psf	ASCE7-16 Clause 7.3

Snow load cases

Minimum Snow Load	Yes		
Minimum snow load, p_m	20.0	psf	ASCE7-16 Clause 7.3.4
Balanced Snow Load	Yes		
Unbalanced Snow Load	No		
Drift Snow Load	Yes		
Rain on Snow Surcharge	No		

SEISMIC LOADING SUMMARY

General

Method	ASCE7		
Structure details			
Height to highest level	21' 0"	ft, in	
Ignore seismic in floor (and below)	St. Base (Base)		
Number of storeys	1		
Two-Period Spectrum			
Max earthquake spectral response acceleration			
S_S short (0.2 s) period (mapped)	9.90	% g	
S_1 1.0 s period (mapped)	6.80	% g	
Site Class	D - Stiff soil		ASCE7-16 Table 20.3-1
Risk Category	II		ASCE7-16 Table 1.5-1
Design spectral response acceleration			
S_{DS} short (0.2 s) period	10.56	% g	ASCE7-16 Clause 11.4-3
S_{D1} 1.0 s period	10.88	% g	ASCE7-16 Clause 11.4-4
Seismic Importance Factor, I_e	1.000		ASCE7-16 Table 1.5-2
Seismic Design Category	B		ASCE7-16 Clause 11.6
T_L long period transition period	12.000	sec	

T_S (S_{D1}/S_{DS})	1.030	sec	
Effective Seismic Weight, W	277.1	kip	ASCE7-16 Clause 12.7.2
Structure Type			ASCE7-16 Table 12.8-2
Direction Dir1	All other structural systems		
Direction Dir2	All other structural systems		
Basic seismic force resisting system			ASCE7-16 Table 12.2-1
Direction Dir1	H. Steel Systems not Specifically detailed for Seismic		
Direction Dir2	H. Steel Systems not Specifically detailed for Seismic		

	Direction Dir1	**Direction Dir2**	
Response Modification Factor, R	3.000	3.000	
System Over-Strength Factor, Ω_0	3.000	3.000	
Deflection Amplification Factor, Cd	3.000	3.000	
Redundancy Factor, ρ	1.000	1.000	ASCE7-16 Clause 12.3.4.2
Approximate fundamental period, T_a [sec]	0.196	0.196	ASCE7-16 Clause 12.8.2.1
Exponent related to structural period k	1.000	1.000	
Seismic response coefficient, C_S	0.035	0.035	ASCE7-16 Clause 12.8.1.1
Seismic base shear, V [kip]	**9.8**	**9.8**	
Scaling Factor for Forces (1.00 V/V_t)	-	-	ASCE7-16 Clause 12.9.1.4.1
Scaling Factor for Drifts	-	-	ASCE7-16 Clause 12.9.1.4.2

Structure Plan Irregularities - User Defined		ASCE7-16 Table 12.3-1
Plan irreg 1a - torsion	Yes	
Plan irreg 1b - extreme torsion	No	
Plan irreg 2 - re-entrant corners	Yes	
Plan irreg 3 - diaphragm discontinuity	No	
Plan irreg 4 - out of plane	No	
Plan irreg 5 - Non parallel systems	No	

Structure Vertical Irregularities - User Defined		ASCE7-16 Table 12.3-2
Vert irreg 1a - soft story	No	
Vert irreg 1b - extreme soft story	No	
Vert irreg 2 - weight mass	No	
Vert irreg 3 - geometric	No	
Vert irreg 4 - in plane	No	
Vert irreg 5a - weak story	No	
Vert irreg 5b - extreme weak story	No	

Equivalent Lateral Force Procedure is permitted (ASCE7-16 12.8).

Reference	**Level [ft, in]**	**Weight [kip]**	**Direction Dir1**			**Direction Dir2**		
			C_V	**F [kip]**	**Ecc [ft, in]**	**C_V**	**F [kip]**	**Ecc [ft, in]**
St. 2 (Upper T.O.S.)	21′ 0″	10.6	0.065	0.6	2′ 8 219/256″	0.065	0.6	5′ 7 205/256″
St. 1 (Lower T.O.S.)	12′ 0″	266.5	0.935	9.1	2′ 8 219/256″	0.935	9.1	5′ 7 205/256″

Analysis procedure to be used:

Equivalent Lateral Force Procedure

PAD BASE DESIGN

Pad Base Group Design Summary

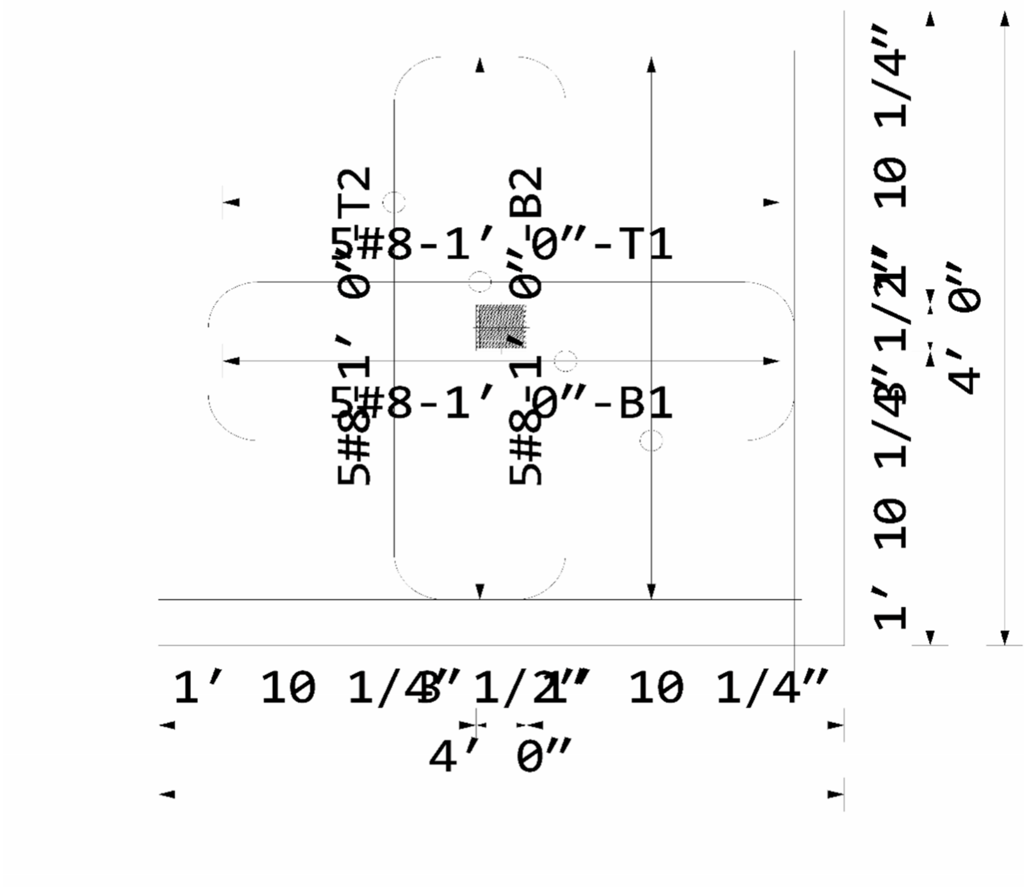
Static

Reference	No. in Group	Critical Member	Size [ft, in]	Grade	Max. UR	Status
PBG2	2	-	No critical member set			
PBG3	4	-	No critical member set			
PBG6	2	-	No critical member set			
PBG7	2	-	No critical member set			
PBG8	2	-	No critical member set			
PBG16	2	-	No critical member set			

Reference	Size [ft, in]	Grade	Max. UR	Status
PB 13	4′ 0″ Ã— 4′ 0″ (Depth = 3′ 0″)	4000 psi	0.513	✔ Pass
PB 6	4′ 0″ Ã— 4′ 0″ (Depth = 3′ 0″)	4000 psi	0.513	✔ Pass

PB 13

4′ 0″ × 4′ 0″ (Depth = 3′ 0″)



PB 13

Static & RSA Design Summary

Applied Loads Summary

	Strength	**Service**
Analysis	3D Building Analysis	3D Building Analysis
Combination	94 $LRFD_{10.14}$-1.2D+L+0.5S+W	136 $LRFD_{12.14}$-0.9D+W
P	1.7 kip	0.6 kip
$F_{x,sup}$	0.0 kip	0.0 kip
$F_{y,sup}$	-1.4 kip	-0.8 kip
$M_{x,sup}$	3.6 kip ft	2.0 kip ft
$M_{y,sup}$	0.0 kip ft	0.0 kip ft
	Added Loads	
F_{swt}	7.2 kip	
F_{soil}	0.0 kip	
$F_{dl,sur}$	0.0 kip	
$F_{ll,sur}$	0.0 kip	

Foundation Details

Foundation Type	Pad Base
Concrete Class	4000 psi
Size	4′ 0″ × 4′ 0″
Overall Depth	3′ 0″ ft, in
Top Cover	3″ ft, in
Bottom Cover	3″ ft, in
Side Cover	3″ ft, in
Depth From Surface	0″ ft, in

Bearing Capacity - Critical

Summary

Size	4′ 0″ × 4′ 0″
Analysis	3D Building Analysis
Combination	38 $LRFD_{8.14}$-1.2D+1.6S+0.5W
q_{max}	1.00 ksf
$q_{a,g}$	1.95 ksf
Ratio	**0.513**

Bending Capacity

Summary

Direction	X-Bot	Y-Bot	X-Top	Y-Top
Analysis	3D Building Analysis	3D Building Analysis	3D Building Analysis	3D Building Analysis
Combination	94 $LRFD_{10.14}$-1.2D+L+0.5S+W	94 $LRFD_{10.14}$-1.2D+L+0.5S+W	11 $LRFD_{7.1}$-1.2D+1.6Lr+0.5W	11 $LRFD_{7.1}$-1.2D+1.6Lr+0.5W
m_u	1.4 kip-ft/ft	1.4 kip-ft/ft	0.9 kip-ft/ft	0.9 kip-ft/ft
d	2′ 7 1/2″ ft, in	2′ 8 1/2″ ft, in	2′ 7 1/2″ ft, in	2′ 8 1/2″ ft, in
R_n / R_{nt}	**0.002**	**0.002**	**0.001**	**0.001**
$A_{s,reqd}$	0.01 in^2/ft	0.01 in^2/ft	0.01 in^2/ft	0.01 in^2/ft
$A_{s,prov}$	0.79 in^2/ft	0.79 in^2/ft	0.79 in^2/ft	0.79 in^2/ft
$A_{s,min}$	0.78 in^2/ft	0.78 in^2/ft	0.78 in^2/ft	0.78 in^2/ft
Ratio	**0.013**	**0.012**	**0.008**	**0.008**
Reinforcement	Deformed # 8 @ 1′ 0″-B2	Deformed # 8 @ 1′ 0″-B1	Deformed # 8 @ 1′ 0″-T2	Deformed # 8 @ 1′ 0″-T1

Shear Capacity

Summary

Direction	X	Y
Analysis	3D Building Analysis	3D Building Analysis
Combination	1 LRFD$_1$-1.4D	1 LRFD$_1$-1.4D
v_u	0.000 ksi	0.000 ksi
$\phi_{shear} \times v_n$	0.095 ksi	0.095 ksi
Ratio	**0.000**	**0.000**

Punching Shear

Summary

Analysis	Combination	b_o [ft, in]	v_u [ksi]	v_n [ksi]	Ratio	Status
3D Building Analysis	30 LRFD$_{8.6}$-1.2D+1.6S+0.5W	6′ 11 1/2″	0.001	0.190	0.006	✔ Pass

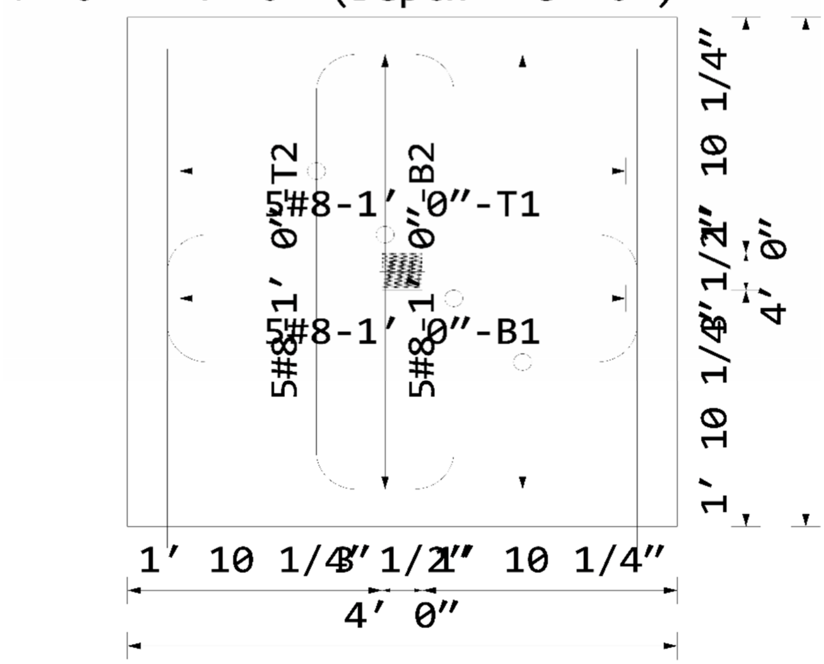
Uplift

Summary

Analysis	3D Building Analysis
Combination	1 LRFD$_1$-1.4D
P	0.0 kip
$F_{dl,stb}$	0.0 kip
Ratio	**0.000**

PB 6

4′ 0″ × 4′ 0″ (Depth = 3′ 0″)



PB 6

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Static & RSA Design Summary

Applied Loads Summary

	Strength	**Service**
Analysis	3D Building Analysis	3D Building Analysis
Combination	94 $LRFD_{10.14}$-1.2D+L+0.5S+W	136 $LRFD_{12.14}$-0.9D+W
P	1.7 kip	0.6 kip
$F_{x,sup}$	-0.0 kip	-0.0 kip
$F_{y,sup}$	-1.4 kip	-0.8 kip
$M_{x,sup}$	3.6 kip ft	2.0 kip ft
$M_{y,sup}$	-0.0 kip ft	-0.0 kip ft
	Added Loads	
F_{swt}	7.2 kip	
F_{soil}	0.0 kip	
$F_{dl,sur}$	0.0 kip	
$F_{ll,sur}$	0.0 kip	

Foundation Details

Foundation Type	Pad Base
Concrete Class	4000 psi
Size	4′ 0″ × 4′ 0″
Overall Depth	3′ 0″ ft, in
Top Cover	3″ ft, in
Bottom Cover	3″ ft, in
Side Cover	3″ ft, in
Depth From Surface	0″ ft, in

Bearing Capacity - Critical

Summary

Size	4′ 0″ × 4′ 0″
Analysis	3D Building Analysis
Combination	38 $LRFD_{8.14}$-1.2D+1.6S+0.5W
q_{max}	1.00 ksf
$q_{a,g}$	1.95 ksf
Ratio	**0.513**

Bending Capacity

Summary

Direction	X-Bot	Y-Bot	X-Top	Y-Top
Analysis	3D Building Analysis	3D Building Analysis	3D Building Analysis	3D Building Analysis
Combination	94 $LRFD_{10.14}$-1.2D+L+0.5S+W	94 $LRFD_{10.14}$-1.2D+L+0.5S+W	11 $LRFD_{7.1}$-1.2D+1.6Lr+0.5W	11 $LRFD_{7.1}$-1.2D+1.6Lr+0.5W
m_u	1.4 kip-ft/ft	1.4 kip-ft/ft	0.9 kip-ft/ft	0.9 kip-ft/ft
d	2′ 7 1/2″ ft, in	2′ 8 1/2″ ft, in	2′ 7 1/2″ ft, in	2′ 8 1/2″ ft, in
R_n / R_{nt}	**0.002**	**0.002**	**0.001**	**0.001**
$A_{s,reqd}$	0.01 in^2/ft	0.01 in^2/ft	0.01 in^2/ft	0.01 in^2/ft
$A_{s,prov}$	0.79 in^2/ft	0.79 in^2/ft	0.79 in^2/ft	0.79 in^2/ft
$A_{s,min}$	0.78 in^2/ft	0.78 in^2/ft	0.78 in^2/ft	0.78 in^2/ft
Ratio	**0.013**	**0.012**	**0.008**	**0.008**
Reinforcement	Deformed # 8 @ 1′ 0″-B2	Deformed # 8 @ 1′ 0″-B1	Deformed # 8 @ 1′ 0″-T2	Deformed # 8 @ 1′ 0″-T1

Shear Capacity

Summary

Direction	X	Y
Analysis	3D Building Analysis	3D Building Analysis
Combination	1 $LRFD_1$-1.4D	1 $LRFD_1$-1.4D
v_u	0.000 ksi	0.000 ksi
$\phi_{shear} \times v_n$	0.095 ksi	0.095 ksi
Ratio	**0.000**	**0.000**

Punching Shear

Summary

Analysis	Combination	b_o [ft, in]	v_u [ksi]	v_n [ksi]	Ratio	Status
3D Building Analysis	37 $LRFD_{8.13}$-1.2D+1.6S+0.5W	6′ 11 1/2″	0.001	0.190	0.006	✔ Pass

Uplift

Summary

Analysis	3D Building Analysis
Combination	1 $LRFD_1$-1.4D
P	0.0 kip
$F_{dl,stb}$	0.0 kip
Ratio	**0.000**

STEEL BEAM DESIGN

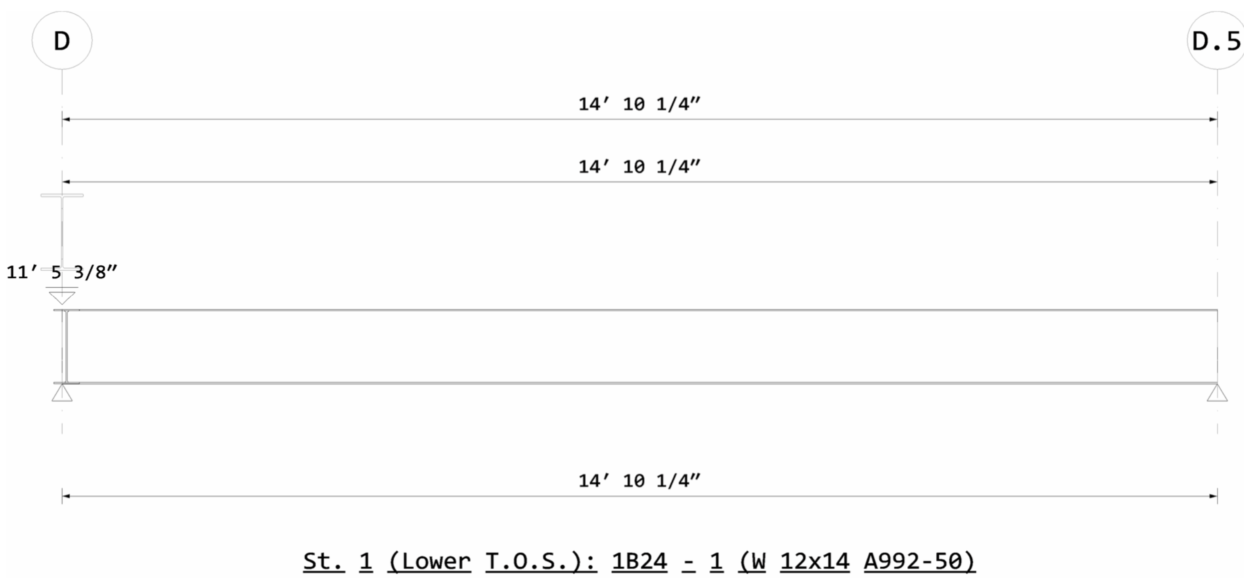
Beam Group Design Summary

Static

Reference	Spans	Critical Span	Section	Grade	Max. UR	Status
1B24	1	1	W 12x14	A992-50	0.279	✔ Pass
1B36	1	1	W 12x14	A992-50	0.695	✔ Pass
1B34	1	1	W 12x26	A992-50	0.755	✔ Pass
2B1	1	1	HSS 6x6x1/4	A500B-46	0.183	! Beyond Scope
1B33	1	1	W 12x26	A992-50	0.749	✔ Pass
1B32	1	1	W 12x26	A992-50	0.472	✔ Pass
2B2	1	1	HSS 6x6x1/4	A500B-46	0.163	✔ Pass
2B4	1	1	HSS 6x6x1/4	A500B-46	0.256	! Beyond Scope
2B5	1	1	HSS 6x6x1/4	A500B-46	0.217	! Beyond Scope
2B6	1	1	HSS 6x6x1/4	A500B-46	0.282	! Beyond Scope
1B23	1	1	W 12x14	A992-50	0.206	✔ Pass
2B7	1	1	HSS 6x6x3/8	A500B-46	0.384	! Beyond Scope
1B28	1	1	W 12x26	A992-50	0.775	✔ Pass
1B29	1	1	W 12x26	A992-50	0.760	✔ Pass
1B30	1	1	W 12x26	A992-50	0.490	✔ Pass
2B9	1	1	HSS 6x6x1/4	A500B-46	0.208	! Beyond Scope
2B10	1	1	HSS 6x6x1/4	A500B-46	0.240	! Beyond Scope

Reference	Spans	Critical Span	Section	Grade	Max. UR	Status
2B8	1	1	HSS 6x6x1/4	A500B-46	0.248	Beyond Scope
1B27	1	1	W 12x26	A992-50	0.239	Pass
1B18	1	1	W 12x14	A992-50	0.087	Pass
1B19	1	1	W 12x14	A992-50	0.067	Pass
1B22	1	1	W 12x14	A992-50	0.407	Pass
1B20	1	1	W 12x14	A992-50	0.278	Pass
1B26	1	1	W 12x14	A992-50	0.086	Pass
1B25	1	1	W 12x14	A992-50	0.070	Pass
1B31	1	1	W 12x26	A992-50	0.197	Pass
1B35	1	1	W 12x14	A992-50	0.695	Pass
1B21	1	1	W 12x14	A992-50	0.207	Pass
2B3	1	1	HSS 6x6x1/4	A500B-46	0.230	Beyond Scope

1B24



Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	●		●		●		●	
sub-beam	14' 10 1/4"	●	1.000		1.000		1.000		1.000
support	14' 10 1/4"	●		●		●		●	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass

209 SE Douglas St.
Lee's Summit, MO 64063
816.444.3144
www.leok.com

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Shear Major	32	2.1	64.3	kip	0.033	✔ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	74	-8.1	36.2	kip ft	0.225	✔ Pass
Flexure Minor	No Significant Forces			kip ft	-	Not required
Axial Tension	133	-4.1	187.2	kip	0.022	✔ Pass
Axial Compression	111	3.0	16.8	kip	0.182	✔ Pass
Combined Forces	83	-	-	-	0.279	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	138	0.002	0.495	in	0.004	✔ Pass
Deflection Live	7	-0.002	0.495	in	0.003	✔ Pass
Deflection Wind	123	0.024	0.891	in	0.027	✔ Pass
Deflection Total	26	0.029	0.500	in	0.058	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

32 $LRFD_{8.8}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 0" - Critical

Position of V_{ry}	=0" ft, in	
Required major axis shear strength, V_{ry}	=2.1 kip	
Design shear strength	=64.3 kip	AISC 360 G2
Ratio	**=0.033**	

✔ Pass

Flexure Major

3D Building Analysis - Critical

74 $LRFD_{9.8}$-1.2D+L+0.5Lr+W - Critical

Span 1 W 12x14 A992-50 - Critical

LTB Flange Btm: 0" - 14' 10 1/4" - Critical

Dist of M_{rx1} along member	=0" ft, in	
Required flexural strength, M_{rx1}	=-8.1 kip ft	
Design flexural strength	=36.2 kip ft	AISC 360 F1, F2, F3 and F4
Ratio	**=0.225**	

✔ Pass

Axial Tension

3D Building Analysis - Critical

133 $LRFD_{12.11}$-0.9D+W - Critical

Span 1 W 12x14 A992-50 - Critical

Yielding: Position 0″ - Critical

Distance of P_r along member	=0″ ft, in	
Required tensile strength, P_r	=-4.1 kip	
Design yield strength	=187.2 kip	AISC 360 D2
Ratio	**=0.022**	

Pass

Axial Compression

3D Building Analysis - Critical

111 $LRFD_{10.31}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Flexural - out-of-plane: 0″ - 14′ 10 1/4″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0″ - 14′ 10 1/4″	Flexural - out-of-plane	14′ 10 1/4″	3.0	16.8	kip	0.182	Pass

Combined Forces

3D Building Analysis - Critical

83 $LRFD_{10.3}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Flange Btm LTB 0″ - 14′ 10 1/4″ - Critical

Axial Comp: I.P. 0″ - 14′ 10 1/4″ , O.o.P.0″ - 14′ 10 1/4″ , Tors. 0″ - 14′ 10 1/4″ - Critical

Mode	Ratio	Status
In-plane instability	0.064	Pass
Out-of-plane instability	0.279	Pass

Deflection

3D Building Analysis - Critical

26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical

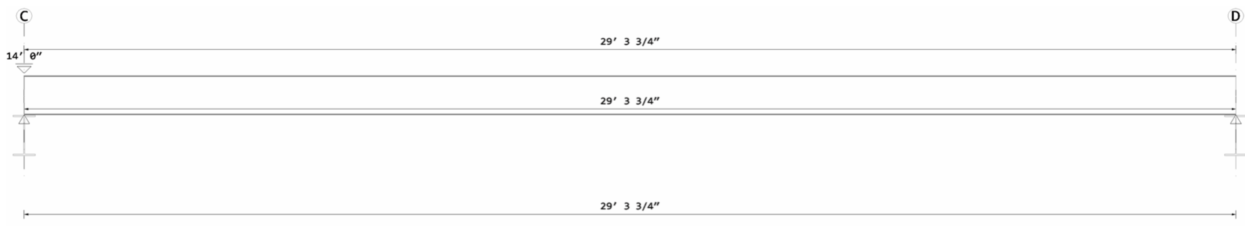
Span 1 W 12x14 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=6′ 10 91/128″ ft, in
Max. Total load deflection	=0.029 in
Span over limit	=0.743 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.058**

Pass

1B36



St. 1 (Lower T.O.S.): 1B36 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0”	•		•		•		•	
sub-beam	29’ 3 3/4”	•	1.000		1.000		1.000		1.000
support	29’ 3 3/4”	•		•		•		•	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	11	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	22	-3.0	64.3	kip	0.046	✔ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	23	22.0	65.2	kip ft	0.337	✔ Pass
Flexure Minor	No Significant Forces			kip ft	-	Not required
Axial Tension	88	-0.9	187.2	kip	0.005	✔ Pass
Axial Compression	68	1.1	4.3	kip	0.264	✔ Pass
Combined Forces	68	-	-	-	0.423	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.095	-	in	-	-
Deflection Dead	138	0.671	0.977	in	0.687	✔ Pass
Deflection Live	7	0.265	0.977	in	0.271	✔ Pass
Deflection Wind	129	-0.084	1.759	in	0.048	✔ Pass
Deflection Total	13	1.019	1.466	in	0.695	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

11 $LRFD_{7.1}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0” - Critical

Axial Force, P	=0.5 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major
3D Building Analysis - Critical
22 $LRFD_{7.12}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Position 29' 3 3/4" - Critical

Position of V_{ry} =29' 3 3/4" ft, in
Required major axis shear strength, V_{ry} =-3.0 kip
Design shear strength =64.3 kip AISC 360 G2
Ratio **=0.046**
Pass

Flexure Major
3D Building Analysis - Critical
23 $LRFD_{7.13}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: 14' 7 7/8" - Critical

Dist of M_{rx} along member =14' 7 7/8" ft, in
Required flexural strength, M_{rx1} =22.0 kip ft
Design flexural strength =65.2 kip ft AISC 360 F1, F2 and F3
Ratio **=0.337**
Pass

Axial Tension
3D Building Analysis - Critical
88 $LRFD_{10.8}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: Position 0" - Critical

Distance of P_r along member =0" ft, in
Required tensile strength, P_r =-0.9 kip
Design yield strength =187.2 kip AISC 360 D2
Ratio **=0.005**
Pass

Axial Compression
3D Building Analysis - Critical
68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flexural - out-of-plane: 0" - 29' 3 3/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 29' 3 3/4"	Flexural - out-of-plane	29' 3 3/4"	1.1	4.3	kip	0.264	Pass

Combined Forces
3D Building Analysis - Critical
68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flange Top LTB 0" - 29' 3 3/4" - Critical
Axial Comp: I.P. 0" - 29' 3 3/4" , O.o.P.0" - 29' 3 3/4" , Tors. 0" - 29' 3 3/4" - Critical

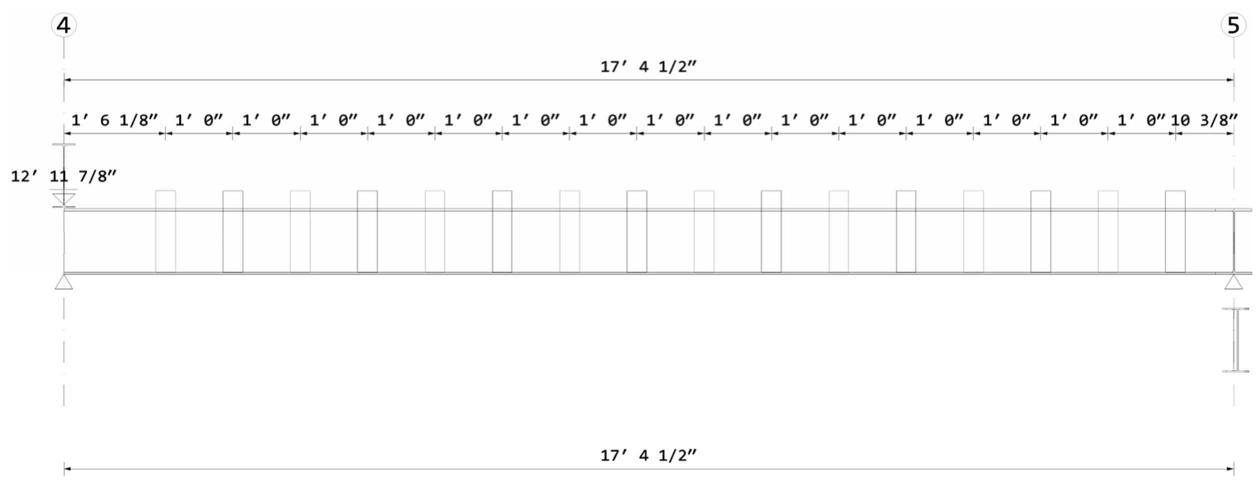
Mode	Ratio	Status
In-plane instability	0.252	Pass
Out-of-plane instability	0.423	Pass

Deflection
3D Building Analysis - Critical
7 LRFD$_5$-1.2D+L+1.6Lr - Critical
Span 1 W 12x14 A992-50 - Critical
Deflection Total - Critical

Position Total load deflection	=14′ 7 7/8″ ft, in
Max. Total load deflection	=1.019 in
Span over limit	=1.466 in
Design limit	=1.466 in
Utilization Ratio	**=0.695**

Pass

1B34



St. 1 (Lower T.O.S.): 1B34 - 1 (W 12x26 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	1′ 6 1/8″	●	1.000		1.000		1.000		1.000
member	1′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	2′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	3′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	4′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	5′ 6 1/8″	●						●	

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	6′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	7′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	8′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	9′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	10′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	11′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	12′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	13′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	14′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	15′ 6 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	16′ 6 1/8″	●						●	
sub-beam	10 3/8″	●	1.000		1.000		1.000		1.000
support	17′ 4 1/2″	●		●		●		●	

Static Design Summary

Summary W 12x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	13	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	16	28.9	84.2	kip	0.344	✔ Pass
Shear Minor	82	0.8	133.2	kip	0.006	✔ Pass
Flexure Major	16	-103.6	139.5	kip ft	0.743	✔ Pass
Flexure Minor	82	1.2	30.6	kip ft	0.039	✔ Pass
Axial Tension	68	-1.6	344.3	kip	0.005	✔ Pass
Axial Compression	136	2.4	183.1	kip	0.013	✔ Pass
Combined Forces	13	-	-	-	0.755	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.025	-	in	-	-
Deflection Dead	138	0.195	0.579	in	0.337	✔ Pass
Deflection Live	7	0.093	0.579	in	0.161	✔ Pass
Deflection Wind	126	-0.036	1.043	in	0.034	✔ Pass
Deflection Total	13	0.315	0.500	in	0.631	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

13 $LRFD_{7.3}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Axial: Position 13' 6 1/8" - Critical

Axial Force, P	=0.1 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

16 $LRFD_{7.6}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 0" - Critical

Position of V_{ry}	=0" ft, in	
Required major axis shear strength, V_{ry}	=28.9 kip	
Design shear strength	=84.2 kip	AISC 360 G2
Ratio	**=0.344**	

✔ Pass

Shear Minor

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 0" - Critical

Position of V_{rx}	=0" ft, in	
Required minor axis shear strength, V_{rx}	=0.8 kip	
Design shear strength	=133.2 kip	AISC 360 G6
Ratio	**=0.006**	

✔ Pass

Flexure Major

3D Building Analysis - Critical

16 $LRFD_{7.6}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: 0" - Critical

Dist of M_{rx} along member	=0" ft, in	
Required flexural strength, M_{rx1}	=-103.6 kip ft	
Design flexural strength	=139.5 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.743**	

✔ Pass

Flexure Minor

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: 1′ 6 1/8″ - Critical

Dist of M_{ry} along member =1′ 6 1/8″ ft, in
Required flexural strength, M_{ry1} =1.2 kip ft
Design flexural strength =30.6 kip ft AISC 360 F6
Ratio **=0.039**

✔ Pass

Axial Tension

3D Building Analysis - Critical

68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: Position 0″ - Critical

Distance of P_r along member =0″ ft, in
Required tensile strength, P_r =-1.6 kip
Design yield strength =344.3 kip AISC 360 D2
Ratio **=0.005**

✔ Pass

Axial Compression

3D Building Analysis - Critical

136 $LRFD_{12.14}$-0.9D+W - Critical

Span 1 W 12x26 A992-50 - Critical

Flexural torsional: 0″ - 17′ 4 1/2″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0″ - 17′ 4 1/2″	Flexural torsional	17′ 4 1/2″	2.4	183.1	kip	0.013	✔ Pass

Combined Forces

3D Building Analysis - Critical

13 $LRFD_{7.3}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Flange Top LTB 0″ - 1′ 6 1/8″ - Critical

Axial Comp: I.P. 0″ - 17′ 4 1/2″ , O.o.P.0″ - 1′ 6 1/8″ , Tors. 0″ - 17′ 4 1/2″ - Critical

Mode	Ratio	Status
Combined buckling	0.755	✔ Pass

Deflection

3D Building Analysis - Critical

13 $LRFD_{7.3}$-1.2D+1.6Lr+0.5W - Critical

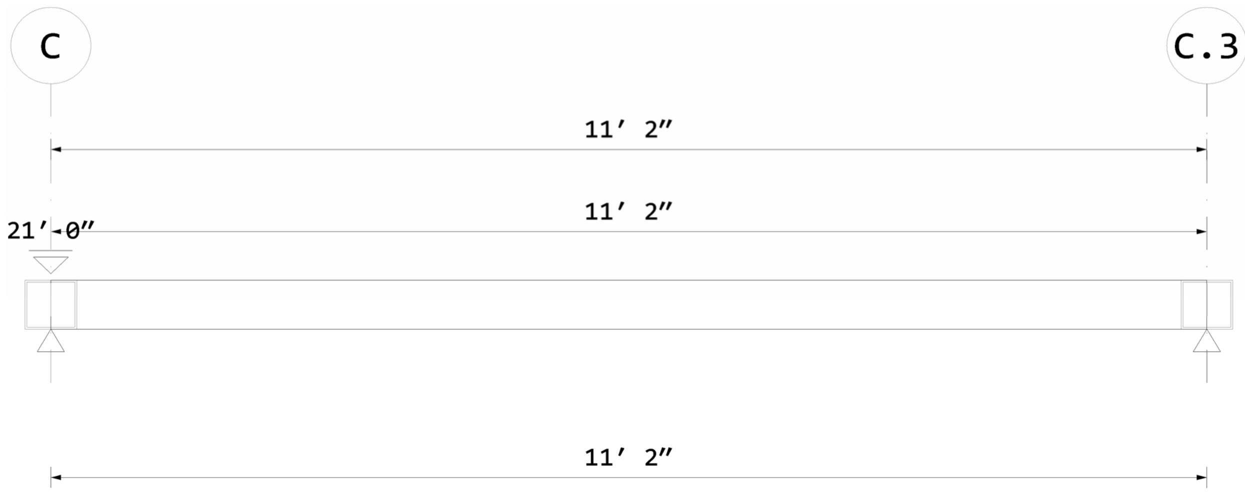
Span 1 W 12x26 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection =9′ 7 125/256″ ft, in
Max. Total load deflection =0.315 in
Span over limit =0.869 in
Abs. limit =0.500 in
Design limit =0.500 in
Utilization Ratio **=0.631**

✔ Pass

2B1



St. 2 (Upper T.O.S.): 2B1 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0”	•		•		•		•	
sub-beam	11’ 1 255/256”	•	1.000		1.000		1.000		1.000
support	11’ 1 255/256”	•		•		•		•	

Static Design Summary

Summary HSS 6x6x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	11	Non-slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	36	-1.3	61.4	kip	0.022	Pass
Shear Minor	123	-0.8	61.4	kip	0.012	Pass
Flexure Major	86	-4.7	38.6	kip ft	0.122	Pass
Flexure Minor	123	2.9	38.6	kip ft	0.076	Pass
Axial Tension	89	-1.5	216.9	kip	0.007	Pass
Axial Compression	125	1.6	174.0	kip	0.009	Pass
Combined Forces	87	-	-	-	0.183	Pass
Torsion	-	-	-	-	-	Beyond Scope
Deflection Self weight	138	0.002	-	in	-	-
Deflection Dead	138	0.005	0.372	in	0.013	Pass
Deflection Live	7	0.004	0.372	in	0.011	Pass
Deflection Wind	124	0.015	0.670	in	0.023	Pass
Deflection Total	54	0.023	0.558	in	0.041	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flexural: Position 0" - Critical

Moment, M_x	=-0.6 kip ft	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Flexural section class	Compact	AISC 360 Table B4.1b

Shear Major

3D Building Analysis - Critical

36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 11' 1 255/256" - Critical

Position of V_{ry}	=11' 1 255/256" ft, in	
Required major axis shear strength, V_{ry}	=-1.3 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.022**	

✔ Pass

Shear Minor

3D Building Analysis - Critical

123 $LRFD_{12.1}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 0" - Critical

Position of V_{rx}	=0" ft, in	
Required minor axis shear strength, V_{rx}	=-0.8 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.012**	

✔ Pass

Flexure Major

3D Building Analysis - Critical

86 $LRFD_{10.6}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: 0" - Critical

Dist of M_{rx} along member	=0" ft, in	
Required flexural strength, M_{rx1}	=-4.7 kip ft	
Design flexural strength	=38.6 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.122**	

✔ Pass

Flexure Minor

3D Building Analysis - Critical

123 $LRFD_{12.1}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: 0" - Critical

Dist of M_{ry} along member =0" ft, in

Required flexural strength, M_{ry1} =2.9 kip ft

Design flexural strength =38.6 kip ft AISC 360 F7

Ratio **=0.076**

Pass

Axial Tension

3D Building Analysis - Critical

89 $LRFD_{10.9}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: Position 0" - Critical

Distance of P_r along member =0" ft, in

Required tensile strength, P_r =-1.5 kip

Design yield strength =216.9 kip AISC 360 D2

Ratio **=0.007**

Pass

Axial Compression

3D Building Analysis - Critical

125 $LRFD_{12.3}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flexural - in-plane: 0" - 11' 1 255/256" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 11' 1 255/256"	Flexural - in-plane	11' 1 255/256"	1.6	174.0	kip	0.009	Pass

Combined Forces

3D Building Analysis - Critical

87 $LRFD_{10.7}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flange Top LTB 0" - 11' 1 255/256" - Critical

Axial Tension: 0" - 11' 1 255/256" - Critical

Mode	Ratio	Status
Combined buckling	0.183	Pass

Torsion

3D Building Analysis - Critical

8 $LRFD_{6.1}$-1.2D+L+1.6S - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Beyond Scope - Critical

Member not pinned at both ends

Beyond Scope

Deflection

3D Building Analysis - Critical

54 $LRFD_{8.30}$-1.2D+1.6S+0.5W - Critical

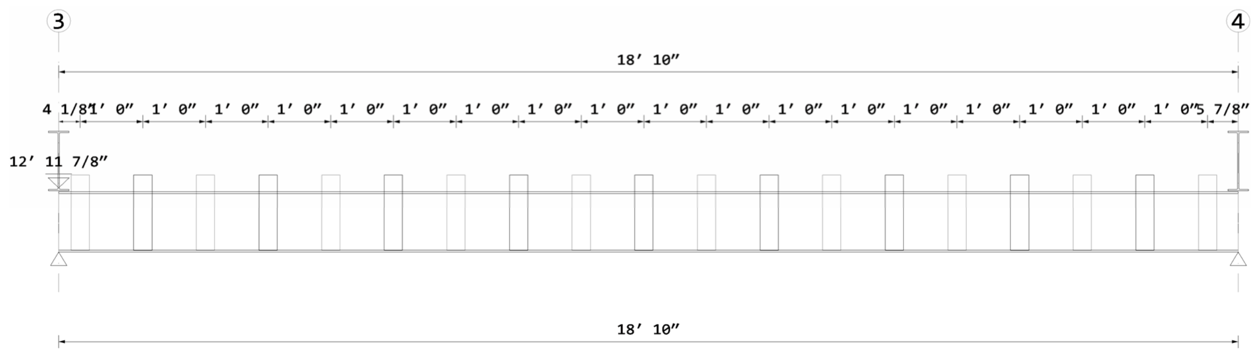
Span 1 HSS 6x6x1/4 A500B-46 - Critical

Deflection Total - Critical

Position Total load deflection	=3′ 8 215/256″ ft, in
Max. Total load deflection	=0.023 in
Span over limit	=0.558 in
Design limit	=0.558 in
Utilization Ratio	**=0.041**

✔ Pass

1B33



St. 1 (Lower T.O.S.): 1B33 - 1 (W 12x26 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	4 1/8″	●	1.000		1.000		1.000		1.000
member	4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	1′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	2′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	3′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	4′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	5′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	6′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	7′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	8′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	9′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	10′ 4 1/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	11′ 4 1/8″	●						●	

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	12′ 4 1/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	13′ 4 1/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	14′ 4 1/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	15′ 4 1/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	16′ 4 1/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	17′ 4 1/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	18′ 4 1/8″	•						•	
sub-beam	5 7/8″	•	1.000		1.000		1.000		1.000
support	18′ 10″	•		•		•		•	

Static Design Summary

Summary W 12x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	24	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	22	-30.8	84.2	kip	0.366	✔ Pass
Shear Minor	82	-1.9	133.2	kip	0.014	✔ Pass
Flexure Major	22	-101.8	138.2	kip ft	0.737	✔ Pass
Flexure Minor	82	1.1	30.6	kip ft	0.035	✔ Pass
Axial Tension	88	-2.2	344.3	kip	0.006	✔ Pass
Axial Compression	136	1.5	173.2	kip	0.009	✔ Pass
Combined Forces	22	-	-	-	0.749	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.026	-	in	-	-
Deflection Dead	138	0.199	0.628	in	0.317	✔ Pass
Deflection Live	7	0.096	0.628	in	0.152	✔ Pass
Deflection Wind	123	-0.029	1.130	in	0.026	✔ Pass
Deflection Total	24	0.318	0.500	in	0.636	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

24 LRFD$_{7.14}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Axial: Position 3′ 4 1/8″ - Critical

Axial Force, P =0.1 kip

Elastic modulus of steel, E =29000 ksi
Minimum yield stress, F_y =50.00 ksi
Axial section class Slender AISC 360 Table B4.1a

Shear Major
3D Building Analysis - Critical
22 $LRFD_{7.12}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Position 18' 10" - Critical
Position of V_{ry} =18' 10" ft, in
Required major axis shear strength, V_{ry} =-30.8 kip
Design shear strength =84.2 kip AISC 360 G2
Ratio **=0.366**
✔ Pass

Shear Minor
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x26 A992-50 - Critical
Position 18' 4 1/8" - Critical
Position of V_{rx} =18' 4 1/8" ft, in
Required minor axis shear strength, V_{rx} =-1.9 kip
Design shear strength =133.2 kip AISC 360 G6
Ratio **=0.014**
✔ Pass

Flexure Major
3D Building Analysis - Critical
22 $LRFD_{7.12}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
LTB Flange Btm: 0" - 18' 10" - Critical
Dist of M_{rx1} along member =18' 10" ft, in
Required flexural strength, M_{rx1} =-101.8 kip ft
Design flexural strength =138.2 kip ft AISC 360 F1, F2, F3 and F4
Ratio **=0.737**
✔ Pass

Flexure Minor
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x26 A992-50 - Critical
Yielding: 17' 4 1/8" - Critical
Dist of M_{ry} along member =17' 4 1/8" ft, in
Required flexural strength, M_{ry1} =1.1 kip ft
Design flexural strength =30.6 kip ft AISC 360 F6
Ratio **=0.035**
✔ Pass

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Axial Tension

3D Building Analysis - Critical

88 LRFD$_{10.8}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: Position 0" - Critical

Distance of P_r along member	=0" ft, in	
Required tensile strength, P_r	=-2.2 kip	
Design yield strength	=344.3 kip	AISC 360 D2
Ratio	**=0.006**	

Pass

Axial Compression

3D Building Analysis - Critical

136 LRFD$_{12.14}$-0.9D+W - Critical

Span 1 W 12x26 A992-50 - Critical

Flexural torsional: 0" - 18' 10" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 18' 10"	Flexural torsional	18' 10"	1.5	173.2	kip	0.009	Pass

Combined Forces

3D Building Analysis - Critical

22 LRFD$_{7.12}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Flange Btm LTB 0" - 18' 10" - Critical

Axial Tension: 0" - 18' 10" - Critical

Mode	Ratio	Status
Combined buckling	0.749	Pass

Deflection

3D Building Analysis - Critical

24 LRFD$_{7.14}$-1.2D+1.6Lr+0.5W - Critical

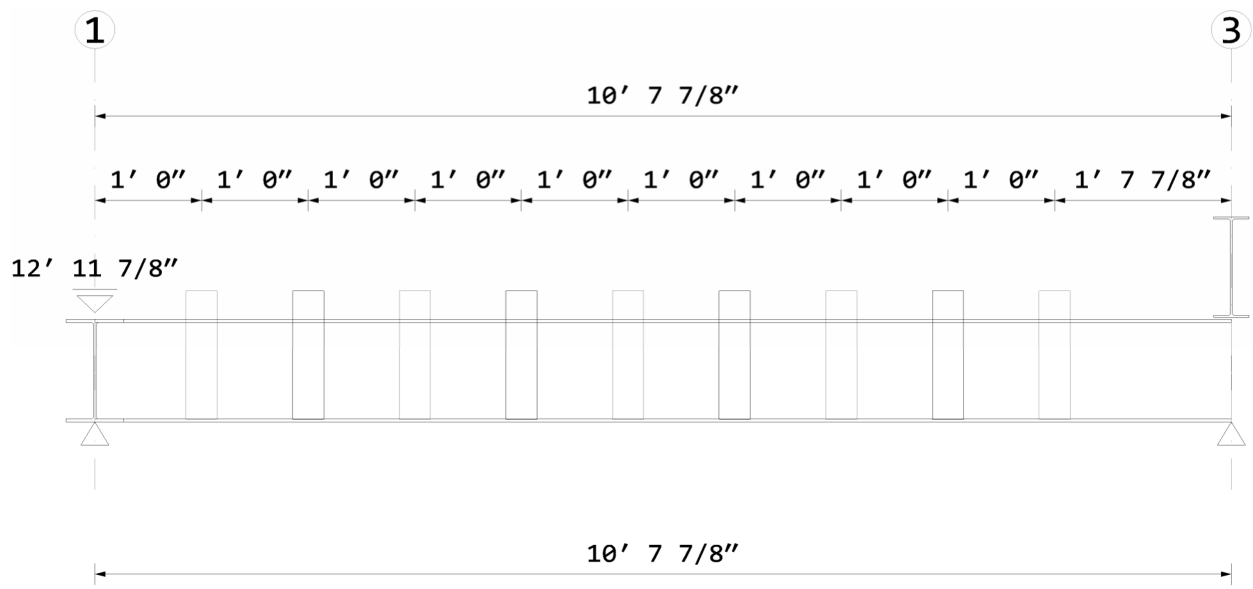
Span 1 W 12x26 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=9' 1 51/256" ft, in
Max. Total load deflection	=0.318 in
Span over limit	=0.942 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.636**

Pass

1B32



St. 1 (Lower T.O.S.): 1B32 - 1 (W 12x26 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	1′ 0″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	2′ 0″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	3′ 0″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	4′ 0″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	5′ 0″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	6′ 0″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	7′ 0″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	8′ 0″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	9′ 0″	•						•	
sub-beam	1′ 7 7/8″	•	1.000		1.000		1.000		1.000
support	10′ 7 7/8″	•		•		•		•	

Static Design Summary

Summary W 12x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	4	Slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	64	-18.3	84.2	kip	0.218	Pass
Shear Minor	110	-2.8	133.2	kip	0.021	Pass
Flexure Major	36	-61.2	139.5	kip ft	0.439	Pass
Flexure Minor	110	-2.8	30.6	kip ft	0.091	Pass
Axial Tension	74	-2.2	344.3	kip	0.007	Pass
Axial Compression	90	1.4	241.1	kip	0.006	Pass
Combined Forces	36	-	-	-	0.472	Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.001	-	in	-	-
Deflection Dead	138	0.011	0.355	in	0.031	Pass
Deflection Live	7	0.005	0.355	in	0.013	Pass
Deflection Wind	126	-0.006	0.639	in	0.009	Pass
Deflection Total	63	0.025	0.500	in	0.051	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

4 LRFD$_{4.1}$-1.2D+1.6L+0.5S - Critical

Span 1 W 12x26 A992-50 - Critical

Axial: Position 8' 0" - Critical

Axial Force, P	=0.1 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

64 LRFD$_{8.40}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 10' 7 7/8" - Critical

Position of V_{ry}	=10' 7 7/8" ft, in	
Required major axis shear strength, V_{ry}	=-18.3 kip	
Design shear strength	=84.2 kip	AISC 360 G2
Ratio	**=0.218**	

Pass

Shear Minor

3D Building Analysis - Critical

110 LRFD$_{10.30}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 0" - Critical

Position of V_{rx} =0" ft, in
Required minor axis shear strength, V_{rx} =-2.8 kip
Design shear strength =133.2 kip AISC 360 G6
Ratio **=0.021**
Pass

Flexure Major
3D Building Analysis - Critical
36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Yielding: 10' 7 7/8" - Critical
Dist of M_{rx} along member =10' 7 7/8" ft, in
Required flexural strength, M_{rx1} =-61.2 kip ft
Design flexural strength =139.5 kip ft AISC 360 F1, F2 and F3
Ratio **=0.439**
Pass

Flexure Minor
3D Building Analysis - Critical
110 $LRFD_{10.30}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x26 A992-50 - Critical
Yielding: 2' 0" - Critical
Dist of M_{ry} along member =2' 0" ft, in
Required flexural strength, M_{ry1} =-2.8 kip ft
Design flexural strength =30.6 kip ft AISC 360 F6
Ratio **=0.091**
Pass

Axial Tension
3D Building Analysis - Critical
74 $LRFD_{9.8}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x26 A992-50 - Critical
Yielding: Position 0" - Critical
Distance of P_r along member =0" ft, in
Required tensile strength, P_r =-2.2 kip
Design yield strength =344.3 kip AISC 360 D2
Ratio **=0.007**
Pass

Axial Compression
3D Building Analysis - Critical
90 $LRFD_{10.10}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x26 A992-50 - Critical
Flexural torsional: 0" - 10' 7 7/8" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 10' 7 7/8"	Flexural torsional	10' 7 7/8"	1.4	241.1	kip	0.006	Pass

Combined Forces
3D Building Analysis - Critical

36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Flange Btm LTB 0" - 10' 7 7/8" - Critical

Axial Comp: I.P. 0" - 10' 7 7/8" , O.o.P.0" - 1' 0" , Tors. 0" - 10' 7 7/8" - Critical

Mode	Ratio	Status
Combined buckling	0.472	Pass

Deflection

3D Building Analysis - Critical

63 $LRFD_{8.39}$-1.2D+1.6S+0.5W - Critical

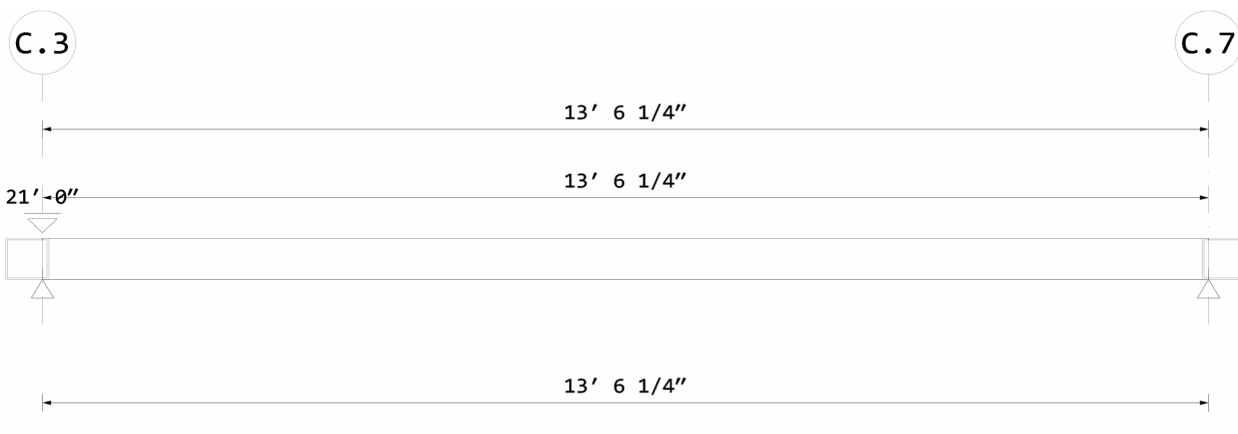
Span 1 W 12x26 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=3' 9 33/128" ft, in
Max. Total load deflection	=0.025 in
Span over limit	=0.533 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.051**

Pass

2B2



St. 2 (Upper T.O.S.): 2B2 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	13' 6 1/4"	•	1.000		1.000		1.000		1.000
support	13' 6 1/4"	•		•		•		•	

Static Design Summary

Summary HSS 6x6x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	13	Non-slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Shear Major	26	-1.5	61.4	kip	0.024	Pass
Shear Minor	126	0.7	61.4	kip	0.012	Pass
Flexure Major	82	-5.0	38.6	kip ft	0.129	Pass
Flexure Minor	69	1.7	38.6	kip ft	0.043	Pass
Axial Tension	90	-1.2	216.9	kip	0.006	Pass
Axial Compression	126	1.5	156.9	kip	0.010	Pass
Combined Forces	82	-	-	-	0.163	Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.004	-	in	-	-
Deflection Dead	138	0.003	0.451	in	0.007	Pass
Deflection Live	7	-0.002	0.451	in	0.003	Pass
Deflection Wind	123	0.013	0.811	in	0.016	Pass
Deflection Total	28	0.027	0.676	in	0.039	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flexural: Position 0" - Critical

Moment, M_x	=-1.3 kip ft	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Flexural section class	Compact	AISC 360 Table B4.1b

Shear Major

3D Building Analysis - Critical

26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 13' 6 1/4" - Critical

Position of V_{ry}	=13' 6 1/4" ft, in	
Required major axis shear strength, V_{ry}	=-1.5 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.024**	

Pass

Shear Minor

3D Building Analysis - Critical

126 $LRFD_{12.4}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 13' 6 1/4" - Critical

Position of V_{rx}	=13' 6 1/4" ft, in	
Required minor axis shear strength, V_{rx}	=0.7 kip	
Design shear strength	=61.4 kip	AISC 360 G4

Ratio =**0.012**

Pass

Flexure Major
3D Building Analysis - Critical
82 LRFD10.2-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 13′ 6 1/4″ - Critical
Dist of M_{rx} along member =13′ 6 1/4″ ft, in
Required flexural strength, M_{rx1} =-5.0 kip ft
Design flexural strength =38.6 kip ft AISC 360 F1, F2 and F3
Ratio **=0.129**

Pass

Flexure Minor
3D Building Analysis - Critical
69 LRFD9.3-1.2D+L+0.5Lr+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 0″ - Critical
Dist of M_{ry} along member =0″ ft, in
Required flexural strength, M_{ry1} =1.7 kip ft
Design flexural strength =38.6 kip ft AISC 360 F7
Ratio **=0.043**

Pass

Axial Tension
3D Building Analysis - Critical
90 LRFD10.10-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: Position 0″ - Critical
Distance of P_r along member =0″ ft, in
Required tensile strength, P_r =-1.2 kip
Design yield strength =216.9 kip AISC 360 D2
Ratio **=0.006**

Pass

Axial Compression
3D Building Analysis - Critical
126 LRFD12.4-0.9D+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Flexural - in-plane: 0″ - 13′ 6 1/4″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0″ - 13′ 6 1/4″	Flexural - in-plane	13′ 6 1/4″	1.5	156.9	kip	0.010	Pass

Combined Forces
3D Building Analysis - Critical
82 LRFD10.2-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flange Top LTB 0" - 13' 6 1/4" - Critical

Axial Comp: I.P. 0" - 13' 6 1/4" , O.o.P.0" - 13' 6 1/4" , Tors. 0" - 13' 6 1/4" - Critical

Mode	Ratio	Status
Combined buckling	0.163	✔ Pass

Deflection

3D Building Analysis - Critical

28 LRFD$_{8.4}$-1.2D+1.6S+0.5W - Critical

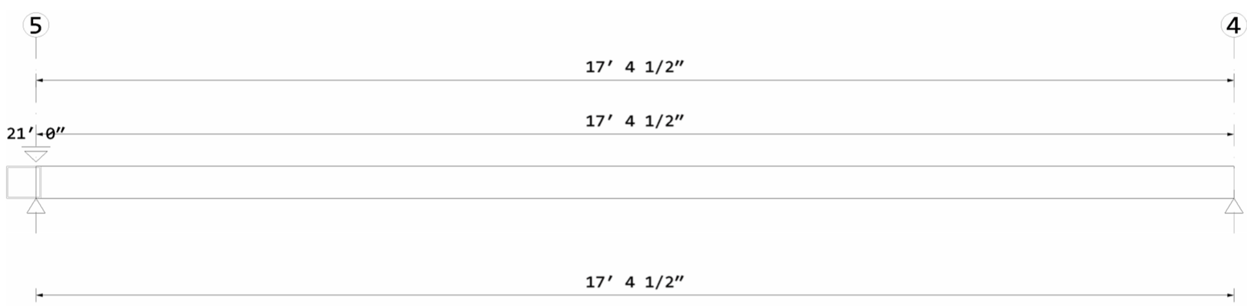
Span 1 HSS 6x6x1/4 A500B-46 - Critical

Deflection Total - Critical

Position Total load deflection	=6' 9 37/256" ft, in
Max. Total load deflection	=0.027 in
Span over limit	=0.676 in
Design limit	=0.676 in
Utilization Ratio	**=0.039**

✔ Pass

2B4



St. 2 (Upper T.O.S.): 2B4 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	●		●		●		●	
sub-beam	17' 4 1/2"	●	1.000		1.000		1.000		1.000
support	17' 4 1/2"	●		●		●		●	

Static Design Summary

Summary HSS 6x6x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	36	1.8	61.4	kip	0.029	✔ Pass
Shear Minor	82	0.5	61.4	kip	0.008	✔ Pass
Flexure Major	36	-6.9	38.6	kip ft	0.179	✔ Pass
Flexure Minor	82	4.6	38.6	kip ft	0.119	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Tension	123	-0.5	216.9	kip	0.003	Pass
Axial Compression	72	2.7	127.1	kip	0.021	Pass
Combined Forces	26	-	-	-	0.256	Pass
Torsion	-	-	-	-	-	Beyond Scope
Deflection Self weight	138	0.008	-	in	-	-
Deflection Dead	138	-0.011	0.579	in	0.020	Pass
Deflection Live	7	-0.011	0.579	in	0.019	Pass
Deflection Wind	129	0.016	1.043	in	0.016	Pass
Deflection Total	31	0.043	0.869	in	0.050	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Axial: Position 0" - Critical

Axial Force, P	=1.4 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Axial section class	Non-slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 0" - Critical

Position of V_{ry}	=0" ft, in	
Required major axis shear strength, V_{ry}	=1.8 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.029**	

Pass

Shear Minor

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 0" - Critical

Position of V_{rx}	=0" ft, in	
Required minor axis shear strength, V_{rx}	=0.5 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.008**	

Pass

Flexure Major

3D Building Analysis - Critical
36 LRFD$_{8.12}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 0" - Critical
Dist of M_{rx} along member =0" ft, in
Required flexural strength, M_{rx1} =-6.9 kip ft
Design flexural strength =38.6 kip ft AISC 360 F1, F2 and F3
Ratio **=0.179**
✔ Pass

Flexure Minor
3D Building Analysis - Critical
82 LRFD$_{10.2}$-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 17' 4 1/2" - Critical
Dist of M_{ry} along member =17' 4 1/2" ft, in
Required flexural strength, M_{ry1} =4.6 kip ft
Design flexural strength =38.6 kip ft AISC 360 F7
Ratio **=0.119**
✔ Pass

Axial Tension
3D Building Analysis - Critical
123 LRFD$_{12.1}$-0.9D+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: Position 0" - Critical
Distance of P_r along member =0" ft, in
Required tensile strength, P_r =-0.5 kip
Design yield strength =216.9 kip AISC 360 D2
Ratio **=0.003**
✔ Pass

Axial Compression
3D Building Analysis - Critical
72 LRFD$_{9.6}$-1.2D+L+0.5Lr+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Flexural - in-plane: 0" - 17' 4 1/2" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 17' 4 1/2"	Flexural - in-plane	17' 4 1/2"	2.7	127.1	kip	0.021	✔ Pass

Combined Forces
3D Building Analysis - Critical
26 LRFD$_{8.2}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Flange Top LTB 0" - 17' 4 1/2" - Critical
Axial Comp: I.P. 0" - 17' 4 1/2" , O.o.P.0" - 17' 4 1/2" , Tors. 0" - 17' 4 1/2" - Critical

Mode	Ratio	Status
Combined buckling	0.256	✔ Pass

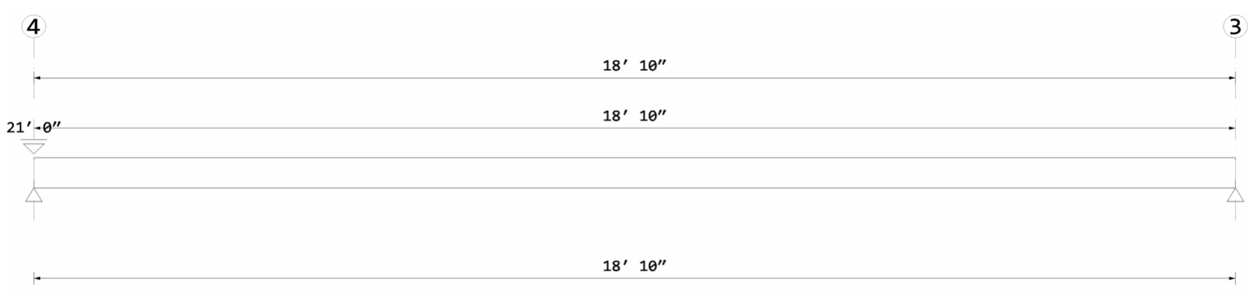
Torsion
3D Building Analysis - Critical
8 LRFD6.1-1.2D+L+1.6S - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Beyond Scope - Critical
Member not pinned at both ends
Beyond Scope

Deflection
3D Building Analysis - Critical
31 LRFD8.7-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Deflection Total - Critical

Position Total load deflection	=8′ 9 57/256″ ft, in
Max. Total load deflection	=0.043 in
Span over limit	=0.869 in
Design limit	=0.869 in
Utilization Ratio	**=0.050**

Pass

2B5



St. 2 (Upper T.O.S.): 2B5 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	18′ 10″	•	1.000		1.000		1.000		1.000
support	18′ 10″	•		•		•		•	

Static Design Summary
Summary HSS 6x6x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	29	-1.8	61.4	kip	0.030	Pass
Shear Minor	No Significant Forces			kip	-	Not required

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Flexure Major	29	-6.2	38.6	kip ft	0.161	Pass
Flexure Minor	82	4.0	38.6	kip ft	0.102	Pass
Axial Tension	123	-0.2	216.9	kip	0.001	Pass
Axial Compression	18	2.2	115.8	kip	0.019	Pass
Combined Forces	27	-	-	-	0.217	Pass
Torsion	-	-	-	-	-	Beyond Scope
Deflection Self weight	138	0.014	-	in	-	-
Deflection Dead	138	0.031	0.628	in	0.050	Pass
Deflection Live	7	-0.004	0.628	in	0.006	Pass
Deflection Wind	133	-0.011	1.130	in	0.010	Pass
Deflection Total	29	0.103	0.942	in	0.110	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Axial: Position 0" - Critical

Axial Force, P	=1.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Axial section class	Non-slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 18' 10" - Critical

Position of V_{ry}	=18' 10" ft, in	
Required major axis shear strength, V_{ry}	=-1.8 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.030**	

Pass

Flexure Major

3D Building Analysis - Critical

29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: 18' 10" - Critical

Dist of M_{rx} along member	=18' 10" ft, in	
Required flexural strength, M_{rx1}	=-6.2 kip ft	
Design flexural strength	=38.6 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.161**	

Pass

Flexure Minor
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 0" - Critical

Dist of M_{ry} along member	=0" ft, in	
Required flexural strength, M_{ry1}	=4.0 kip ft	
Design flexural strength	=38.6 kip ft	AISC 360 F7
Ratio	**=0.102**	

Pass

Axial Tension
3D Building Analysis - Critical
123 $LRFD_{12.1}$-0.9D+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: Position 0" - Critical

Distance of P_r along member	=0" ft, in	
Required tensile strength, P_r	=-0.2 kip	
Design yield strength	=216.9 kip	AISC 360 D2
Ratio	**=0.001**	

Pass

Axial Compression
3D Building Analysis - Critical
18 $LRFD_{7.8}$-1.2D+1.6Lr+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Flexural - in-plane: 0" - 18' 10" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 18' 10"	Flexural - in-plane	18' 10"	2.2	115.8	kip	0.019	Pass

Combined Forces
3D Building Analysis - Critical
27 $LRFD_{8.3}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Flange Top LTB 0" - 18' 10" - Critical
Axial Comp: I.P. 0" - 18' 10" , O.o.P.0" - 18' 10" , Tors. 0" - 18' 10" - Critical

Mode	Ratio	Status
Combined buckling	0.217	Pass

Torsion
3D Building Analysis - Critical
8 $LRFD_{6.1}$-1.2D+L+1.6S - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Beyond Scope - Critical
Member not pinned at both ends
Beyond Scope

Deflection

3D Building Analysis - Critical

29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical

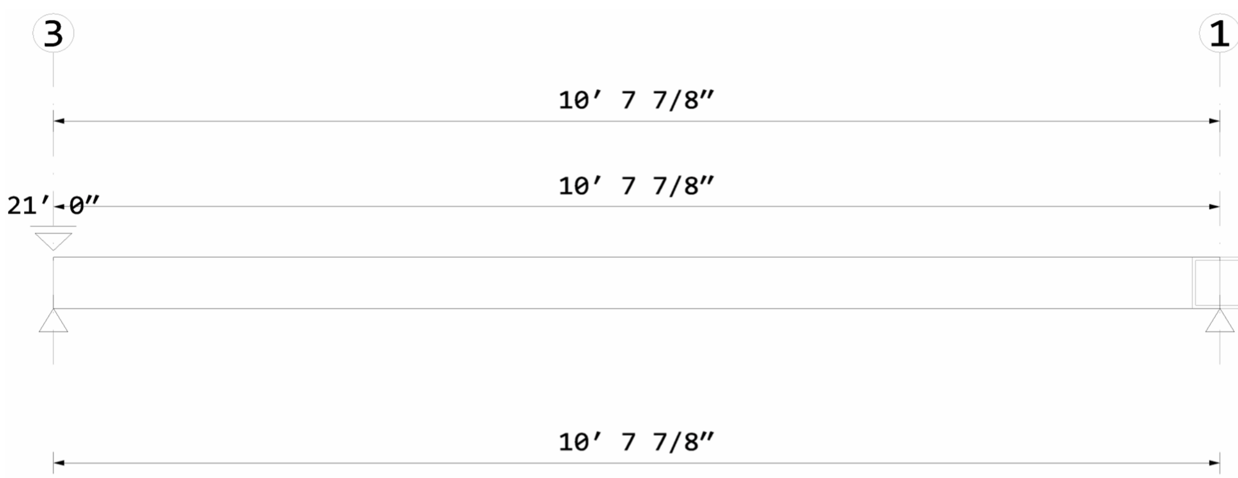
Span 1 HSS 6x6x1/4 A500B-46 - Critical

Deflection Total - Critical

Position Total load deflection	=8′ 8 81/256″ ft, in
Max. Total load deflection	=0.103 in
Span over limit	=0.942 in
Design limit	=0.942 in
Utilization Ratio	**=0.110**

Pass

2B6



St. 2 (Upper T.O.S.): 2B6 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	10′ 7 7/8″	•	1.000		1.000		1.000		1.000
support	10′ 7 7/8″	•		•		•		•	

Static Design Summary

Summary HSS 6x6x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	29	-1.5	61.4	kip	0.024	Pass
Shear Minor	82	-0.9	61.4	kip	0.015	Pass
Flexure Major	29	-4.9	38.6	kip ft	0.126	Pass
Flexure Minor	83	-7.4	38.6	kip ft	0.191	Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Tension	125	-0.7	216.9	kip	0.003	Pass
Axial Compression	117	1.4	177.4	kip	0.008	Pass
Combined Forces	83	-	-	-	0.282	Pass
Torsion	-	-	-	-	-	Beyond Scope
Deflection Self weight	138	0.001	-	in	-	-
Deflection Dead	138	-0.005	0.355	in	0.013	Pass
Deflection Live	7	0.003	0.355	in	0.008	Pass
Deflection Wind	132	0.007	0.639	in	0.011	Pass
Deflection Total	29	-0.011	0.533	in	0.021	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_{1}$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Axial section class	Non-slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 10' 7 7/8" - Critical

Position of V_{ry}	=10' 7 7/8" ft, in	
Required major axis shear strength, V_{ry}	=-1.5 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.024**	

Pass

Shear Minor

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 0" - Critical

Position of V_{rx}	=0" ft, in	
Required minor axis shear strength, V_{rx}	=-0.9 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.015**	

Pass

Flexure Major

3D Building Analysis - Critical

29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: 10′ 7 7/8″ - Critical

Dist of M_{rx} along member	=10′ 7 7/8″ ft, in	
Required flexural strength, M_{rx1}	=-4.9 kip ft	
Design flexural strength	=38.6 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.126**	

Pass

Flexure Minor

3D Building Analysis - Critical

83 $LRFD_{10.3}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: 10′ 7 7/8″ - Critical

Dist of M_{ry} along member	=10′ 7 7/8″ ft, in	
Required flexural strength, M_{ry1}	=-7.4 kip ft	
Design flexural strength	=38.6 kip ft	AISC 360 F7
Ratio	**=0.191**	

Pass

Axial Tension

3D Building Analysis - Critical

125 $LRFD_{12.3}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: Position 0″ - Critical

Distance of P_r along member	=0″ ft, in	
Required tensile strength, P_r	=-0.7 kip	
Design yield strength	=216.9 kip	AISC 360 D2
Ratio	**=0.003**	

Pass

Axial Compression

3D Building Analysis - Critical

117 $LRFD_{10.37}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flexural - in-plane: 0″ - 10′ 7 7/8″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0″ - 10′ 7 7/8″	Flexural - in-plane	10′ 7 7/8″	1.4	177.4	kip	0.008	Pass

Combined Forces

3D Building Analysis - Critical

83 $LRFD_{10.3}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flange Top LTB 0″ - 10′ 7 7/8″ - Critical

Axial Tension: 0″ - 10′ 7 7/8″ - Critical

Mode	Ratio	Status
Combined buckling	0.282	Pass

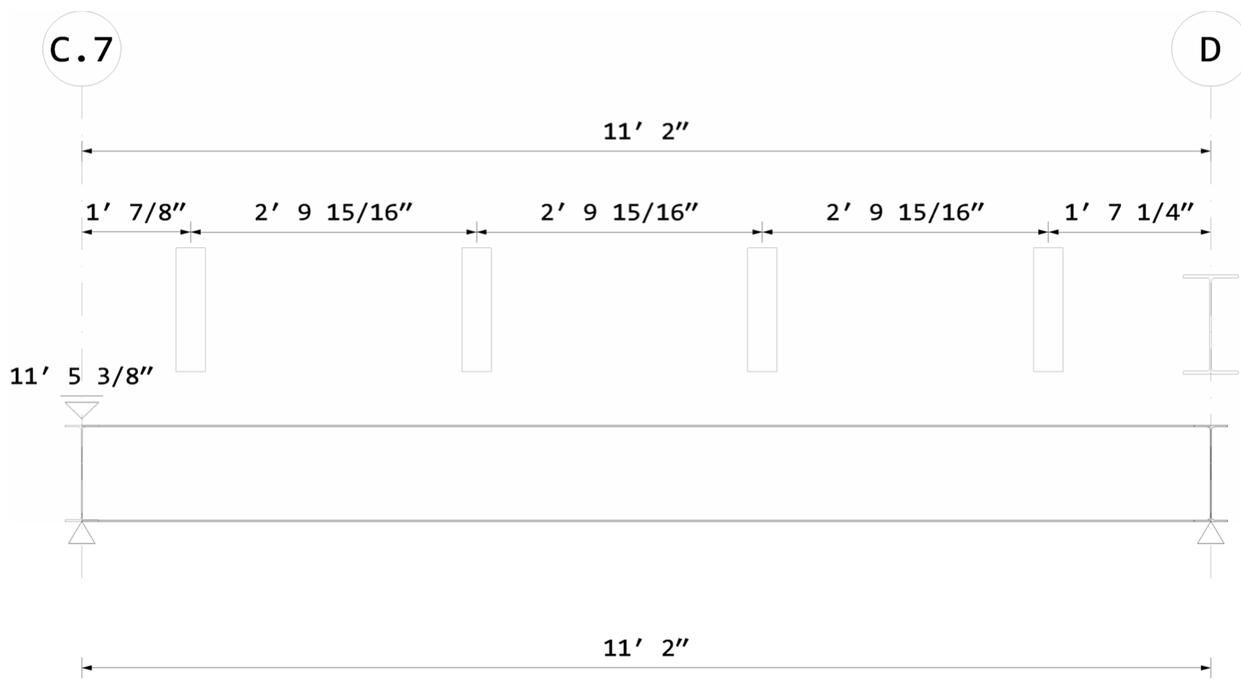
Torsion
3D Building Analysis - Critical
4 $LRFD_{4.1}$-1.2D+1.6L+0.5S - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Beyond Scope - Critical
Member not pinned at both ends
Beyond Scope

Deflection
3D Building Analysis - Critical
29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Deflection Total - Critical

Position Total load deflection	=8' 6 59/128" ft, in
Max. Total load deflection	=-0.011 in
Span over limit	=0.533 in
Design limit	=0.533 in
Utilization Ratio	**=0.021**

Pass

1B23



St. 1 (Lower T.O.S.): 1B23 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	1′ 29/32″	●	1.000		1.000		1.000		1.000
member	1′ 29/32″	●						●	
sub-beam	2′ 9 241/256″	●	1.000		1.000		1.000		1.000
member	3′ 10 217/256″	●						●	
sub-beam	2′ 9 241/256″	●	1.000		1.000		1.000		1.000
member	6′ 8 201/256″	●						●	
sub-beam	2′ 9 241/256″	●	1.000		1.000		1.000		1.000
member	9′ 6 93/128″	●						●	
sub-beam	1′ 7 69/256″	●	1.000		1.000		1.000		1.000
support	11′ 1 255/256″	●		●		●		●	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	36	-4.3	64.3	kip	0.067	✔ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	38	12.4	65.2	kip ft	0.190	✔ Pass
Flexure Minor	129	-0.2	7.1	kip ft	0.024	✔ Pass
Axial Tension	133	-1.5	187.2	kip	0.008	✔ Pass
Axial Compression	94	2.9	74.5	kip	0.039	✔ Pass
Combined Forces	30	-	-	-	0.206	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.007	-	in	-	-
Deflection Dead	138	0.046	0.372	in	0.124	✔ Pass
Deflection Live	7	0.021	0.372	in	0.055	✔ Pass
Deflection Wind	132	-0.009	0.670	in	0.013	✔ Pass
Deflection Total	27	0.083	0.500	in	0.166	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 LRFD$_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0″ - Critical

Axial Force, P	=0.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major
3D Building Analysis - Critical
36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Position 11' 1 255/256" - Critical

Position of V_{ry}	=11' 1 255/256" ft, in	
Required major axis shear strength, V_{ry}	=-4.3 kip	
Design shear strength	=64.3 kip	AISC 360 G2
Ratio	**=0.067**	

Pass

Flexure Major
3D Building Analysis - Critical
38 $LRFD_{8.14}$-1.2D+1.6S+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: 6' 8 201/256" - Critical

Dist of M_{rx} along member	=6' 8 201/256" ft, in	
Required flexural strength, M_{rx1}	=12.4 kip ft	
Design flexural strength	=65.2 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.190**	

Pass

Flexure Minor
3D Building Analysis - Critical
129 $LRFD_{12.7}$-0.9D+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: 8' 3 115/128" - Critical

Dist of M_{ry} along member	=8' 3 115/128" ft, in	
Required flexural strength, M_{ry1}	=-0.2 kip ft	
Design flexural strength	=7.1 kip ft	AISC 360 F6
Ratio	**=0.024**	

Pass

Axial Tension
3D Building Analysis - Critical
133 $LRFD_{12.11}$-0.9D+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: Position 11' 1 255/256" - Critical

Distance of P_r along member	=11' 1 255/256" ft, in	
Required tensile strength, P_r	=-1.5 kip	
Design yield strength	=187.2 kip	AISC 360 D2
Ratio	**=0.008**	

Pass

Axial Compression
3D Building Analysis - Critical
94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flexural torsional: 0" - 11' 1 255/256" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 11' 1 255/256"	Flexural torsional	11' 1 255/256"	2.9	74.5	kip	0.039	✔ Pass

Combined Forces

3D Building Analysis - Critical

30 $LRFD_{8.6}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Flange Top LTB 6' 8 201/256" - 9' 6 93/128" - Critical

Axial Comp: I.P. 0" - 11' 1 255/256" , O.o.P.6' 8 201/256" - 9' 6 93/128" , Tors. 0" - 11' 1 255/256" - Critical

Mode	Ratio	Status
Combined buckling	0.206	✔ Pass

Deflection

3D Building Analysis - Critical

27 $LRFD_{8.3}$-1.2D+1.6S+0.5W - Critical

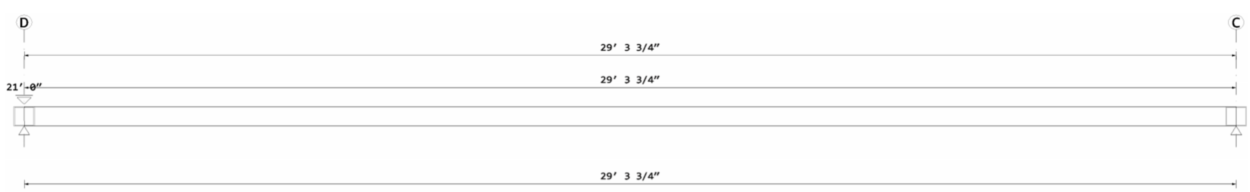
Span 1 W 12x14 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=5' 8 11/256" ft, in
Max. Total load deflection	=0.083 in
Span over limit	=0.558 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.166**

✔ Pass

2B7



St. 2 (Upper T.O.S.): 2B7 - 1 (HSS 6x6x3/8 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	●		●		●		●	
sub-beam	29' 3 3/4"	●	1.000		1.000		1.000		1.000
support	29' 3 3/4"	●		●		●		●	

Static Design Summary

Summary HSS 6x6x3/8

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	26	3.0	85.9	kip	0.034	✔ Pass
Shear Minor	91	-1.6	85.9	kip	0.019	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Flexure Major	26	-14.7	54.5	kip ft	0.270	✓ Pass
Flexure Minor	134	7.9	54.5	kip ft	0.144	✓ Pass
Axial Tension		No Significant Forces		kip	-	Not required
Axial Compression	35	1.5	71.9	kip	0.021	✓ Pass
Combined Forces	92	-	-	-	0.349	✓ Pass
Torsion	-	-	-	-	-	! Beyond Scope
Deflection Self weight	138	0.116	-	in	-	-
Deflection Dead	138	0.157	0.977	in	0.161	✓ Pass
Deflection Live	7	-0.001	0.977	in	0.001	✓ Pass
Deflection Wind	123	0.059	1.759	in	0.034	✓ Pass
Deflection Total	27	0.563	1.466	in	0.384	✓ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_{1}$-1.4D - Critical

Span 1 HSS 6x6x3/8 A500B-46 - Critical

Axial: Position 0″ - Critical

Axial Force, P	=0.7 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Axial section class	Non-slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x3/8 A500B-46 - Critical

Position 0″ - Critical

Position of V_{ry}	=0″ ft, in	
Required major axis shear strength, V_{ry}	=3.0 kip	
Design shear strength	=85.9 kip	AISC 360 G4
Ratio	**=0.034**	

✓ Pass

Shear Minor

3D Building Analysis - Critical

91 $LRFD_{10.11}$-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x3/8 A500B-46 - Critical

Position 0″ - Critical

Position of V_{rx}	=0″ ft, in	
Required minor axis shear strength, V_{rx}	=-1.6 kip	
Design shear strength	=85.9 kip	AISC 360 G4
Ratio	**=0.019**	

✓ Pass

Flexure Major
3D Building Analysis - Critical
26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x3/8 A500B-46 - Critical
Yielding: 0" - Critical

Dist of M_{rx} along member	=0" ft, in	
Required flexural strength, M_{rx1}	=-14.7 kip ft	
Design flexural strength	=54.5 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.270**	

Pass

Flexure Minor
3D Building Analysis - Critical
134 $LRFD_{12.12}$-0.9D+W - Critical
Span 1 HSS 6x6x3/8 A500B-46 - Critical
Yielding: 29' 3 3/4" - Critical

Dist of M_{ry} along member	=29' 3 3/4" ft, in	
Required flexural strength, M_{ry1}	=7.9 kip ft	
Design flexural strength	=54.5 kip ft	AISC 360 F7
Ratio	**=0.144**	

Pass

Axial Compression
3D Building Analysis - Critical
35 $LRFD_{8.11}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x3/8 A500B-46 - Critical
Flexural - in-plane: 0" - 29' 3 3/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 29' 3 3/4"	Flexural - in-plane	29' 3 3/4"	1.5	71.9	kip	0.021	Pass

Combined Forces
3D Building Analysis - Critical
92 $LRFD_{10.12}$-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x3/8 A500B-46 - Critical
Flange Top LTB 0" - 29' 3 3/4" - Critical
Axial Comp: I.P. 0" - 29' 3 3/4" , O.o.P.0" - 29' 3 3/4" , Tors. 0" - 29' 3 3/4" - Critical

Mode	Ratio	Status
Combined buckling	0.349	Pass

Torsion
3D Building Analysis - Critical
1 $LRFD_1$-1.4D - Critical
Span 1 HSS 6x6x3/8 A500B-46 - Critical
Beyond Scope - Critical
Member not pinned at both ends
Beyond Scope

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Deflection

3D Building Analysis - Critical

27 $LRFD_{8.3}$-1.2D+1.6S+0.5W - Critical

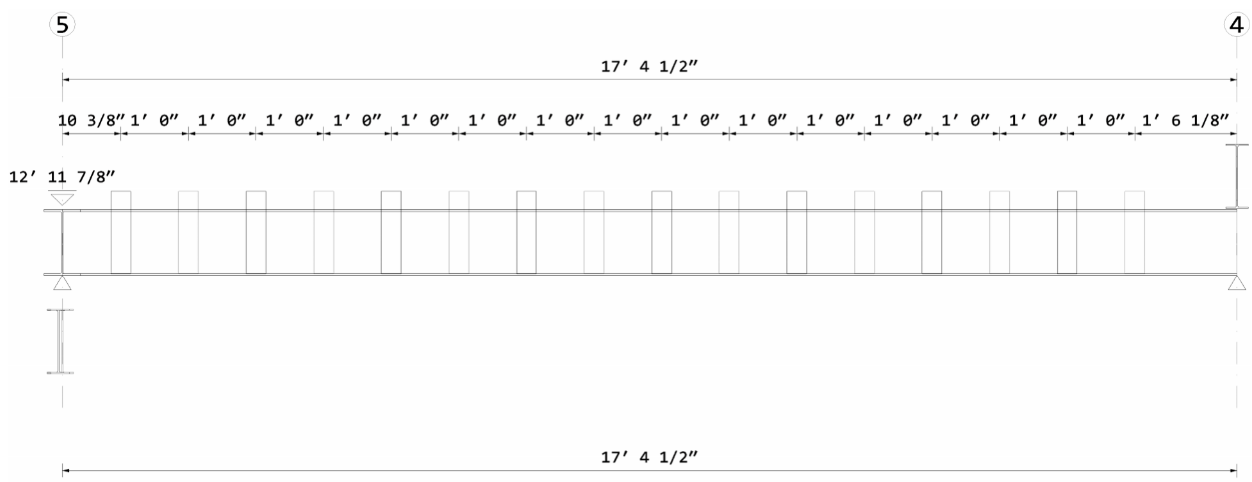
Span 1 HSS 6x6x3/8 A500B-46 - Critical

Deflection Total - Critical

Position Total load deflection	=15′ 4 163/256″ ft, in
Max. Total load deflection	=0.563 in
Span over limit	=1.466 in
Design limit	=1.466 in
Utilization Ratio	**=0.384**

✔ Pass

1B28



St. 1 (Lower T.O.S.): 1B28 - 1 (W 12x26 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	10 3/8″	•	1.000		1.000		1.000		1.000
member	10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	1′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	2′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	3′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	4′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	5′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	6′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	7′ 10 3/8″	•						•	

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	8′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	9′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	10′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	11′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	12′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	13′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	14′ 10 3/8″	•						•	
sub-beam	1′ 0″	•	1.000		1.000		1.000		1.000
member	15′ 10 3/8″	•						•	
sub-beam	1′ 6 1/8″	•	1.000		1.000		1.000		1.000
support	17′ 4 1/2″	•		•		•		•	

Static Design Summary

Summary W 12x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	13	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	13	-30.5	84.2	kip	0.362	✔ Pass
Shear Minor	68	0.9	133.2	kip	0.007	✔ Pass
Flexure Major	13	-105.6	139.5	kip ft	0.757	✔ Pass
Flexure Minor	68	-1.4	30.6	kip ft	0.045	✔ Pass
Axial Tension	74	-1.6	344.3	kip	0.005	✔ Pass
Axial Compression	136	2.4	183.1	kip	0.013	✔ Pass
Combined Forces	13	-	-	-	0.775	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.025	-	in	-	-
Deflection Dead	138	0.196	0.579	in	0.338	✔ Pass
Deflection Live	7	0.094	0.579	in	0.162	✔ Pass
Deflection Wind	126	-0.036	1.043	in	0.034	✔ Pass
Deflection Total	16	0.317	0.500	in	0.633	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

13 $LRFD_{7.3}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.4 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major
3D Building Analysis - Critical
13 $LRFD_{7.3}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Position 17' 4 1/2" - Critical

Position of V_{ry}	=17' 4 1/2" ft, in	
Required major axis shear strength, V_{ry}	=-30.5 kip	
Design shear strength	=84.2 kip	AISC 360 G2
Ratio	**=0.362**	

✓ Pass

Shear Minor
3D Building Analysis - Critical
68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x26 A992-50 - Critical
Position 15' 10 3/8" - Critical

Position of V_{rx}	=15' 10 3/8" ft, in	
Required minor axis shear strength, V_{rx}	=0.9 kip	
Design shear strength	=133.2 kip	AISC 360 G6
Ratio	**=0.007**	

✓ Pass

Flexure Major
3D Building Analysis - Critical
13 $LRFD_{7.3}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Yielding: 17' 4 1/2" - Critical

Dist of M_{rx} along member	=17' 4 1/2" ft, in	
Required flexural strength, M_{rx1}	=-105.6 kip ft	
Design flexural strength	=139.5 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.757**	

✓ Pass

Flexure Minor
3D Building Analysis - Critical
68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x26 A992-50 - Critical
Yielding: 15' 10 3/8" - Critical

Dist of M_{ry} along member	=15' 10 3/8" ft, in	
Required flexural strength, M_{ry1}	=-1.4 kip ft	
Design flexural strength	=30.6 kip ft	AISC 360 F6
Ratio	**=0.045**	

✓ Pass

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Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 111
Project Name: Express Stop – View High		

Axial Tension
3D Building Analysis - Critical
74 $LRFD_{9.8}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x26 A992-50 - Critical
Yielding: Position 17′ 4 1/2″ - Critical

Distance of P_r along member	=17′ 4 1/2″ ft, in	
Required tensile strength, P_r	=-1.6 kip	
Design yield strength	=344.3 kip	AISC 360 D2
Ratio	**=0.005**	

✔ Pass

Axial Compression
3D Building Analysis - Critical
136 $LRFD_{12.14}$-0.9D+W - Critical
Span 1 W 12x26 A992-50 - Critical
Flexural torsional: 0″ - 17′ 4 1/2″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0″ - 17′ 4 1/2″	Flexural torsional	17′ 4 1/2″	2.4	183.1	kip	0.013	✔ Pass

Combined Forces
3D Building Analysis - Critical
13 $LRFD_{7.3}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Flange Top LTB 15′ 10 3/8″ - 17′ 4 1/2″ - Critical
Axial Comp: I.P. 0″ - 17′ 4 1/2″ , O.o.P.15′ 10 3/8″ - 17′ 4 1/2″ , Tors. 0″ - 17′ 4 1/2″ - Critical

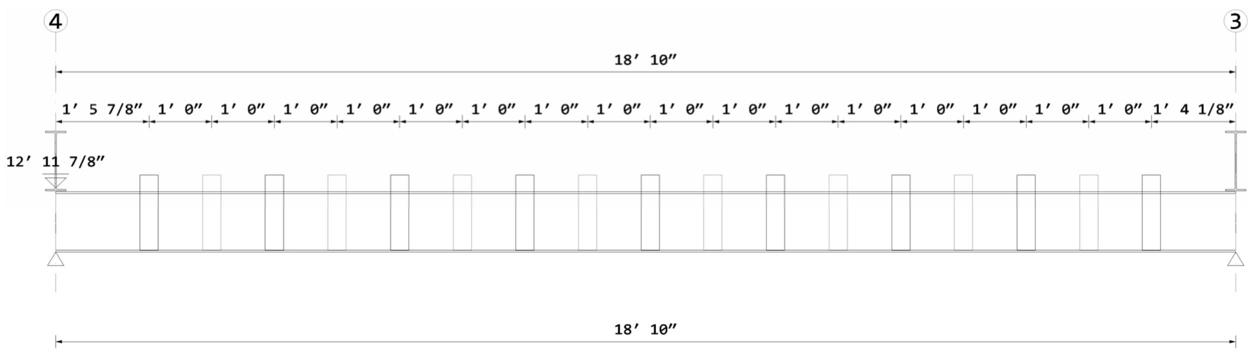
Mode	Ratio	Status
Combined buckling	0.775	✔ Pass

Deflection
3D Building Analysis - Critical
16 $LRFD_{7.6}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Deflection Total - Critical

Position Total load deflection	=7′ 9 19/128″ ft, in
Max. Total load deflection	=0.317 in
Span over limit	=0.869 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.633**

✔ Pass

1B29



St. 1 (Lower T.O.S.): 1B29 - 1 (W 12x26 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	●		●		●		●	
sub-beam	1' 5 7/8"	●	1.000		1.000		1.000		1.000
member	1' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	2' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	3' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	4' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	5' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	6' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	7' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	8' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	9' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	10' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	11' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	12' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	13' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	14' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	15' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000
member	16' 5 7/8"	●						●	
sub-beam	1' 0"	●	1.000		1.000		1.000		1.000

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
member	17′ 5 7/8″	●						●	
sub-beam	1′ 4 1/8″	●	1.000		1.000		1.000		1.000
support	18′ 10″	●		●		●		●	

Static Design Summary

Summary W 12x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	24	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	19	29.0	84.2	kip	0.345	✔ Pass
Shear Minor	68	-0.7	133.2	kip	0.005	✔ Pass
Flexure Major	19	-103.6	138.4	kip ft	0.748	✔ Pass
Flexure Minor	68	-1.1	30.6	kip ft	0.034	✔ Pass
Axial Tension	82	-2.1	344.3	kip	0.006	✔ Pass
Axial Compression	136	1.5	173.2	kip	0.009	✔ Pass
Combined Forces	22	-	-	-	0.760	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.026	-	in	-	-
Deflection Dead	138	0.202	0.628	in	0.321	✔ Pass
Deflection Live	7	0.097	0.628	in	0.155	✔ Pass
Deflection Wind	129	-0.030	1.130	in	0.026	✔ Pass
Deflection Total	24	0.322	0.500	in	0.644	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

24 $LRFD_{7.14}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Axial: Position 0″ - Critical

Axial Force, P	=0.5 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

19 $LRFD_{7.9}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 0″ - Critical

Position of V_{ry}	=0″ ft, in	
Required major axis shear strength, V_{ry}	=29.0 kip	
Design shear strength	=84.2 kip	AISC 360 G2

Ratio =**0.345**

✔ Pass

Shear Minor

3D Building Analysis - Critical

68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 0" - Critical

Position of V_{rx} =0" ft, in

Required minor axis shear strength, V_{rx} =-0.7 kip

Design shear strength =133.2 kip AISC 360 G6

Ratio =**0.005**

✔ Pass

Flexure Major

3D Building Analysis - Critical

19 $LRFD_{7.9}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

LTB Flange Btm: 0" - 18' 10" - Critical

Dist of M_{rx1} along member =0" ft, in

Required flexural strength, M_{rx1} =-103.6 kip ft

Design flexural strength =138.4 kip ft AISC 360 F1, F2, F3 and F4

Ratio =**0.748**

✔ Pass

Flexure Minor

3D Building Analysis - Critical

68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: 2' 5 7/8" - Critical

Dist of M_{ry} along member =2' 5 7/8" ft, in

Required flexural strength, M_{ry1} =-1.1 kip ft

Design flexural strength =30.6 kip ft AISC 360 F6

Ratio =**0.034**

✔ Pass

Axial Tension

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: Position 18' 10" - Critical

Distance of P_r along member =18' 10" ft, in

Required tensile strength, P_r =-2.1 kip

Design yield strength =344.3 kip AISC 360 D2

Ratio =**0.006**

✔ Pass

Axial Compression
3D Building Analysis - Critical
136 LRFD$_{12.14}$-0.9D+W - Critical
Span 1 W 12x26 A992-50 - Critical
Flexural torsional: 0" - 18' 10" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 18' 10"	Flexural torsional	18' 10"	1.5	173.2	kip	0.009	Pass

Combined Forces
3D Building Analysis - Critical
22 LRFD$_{7.12}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Flange Btm LTB 0" - 18' 10" - Critical
Axial Tension: 0" - 18' 10" - Critical

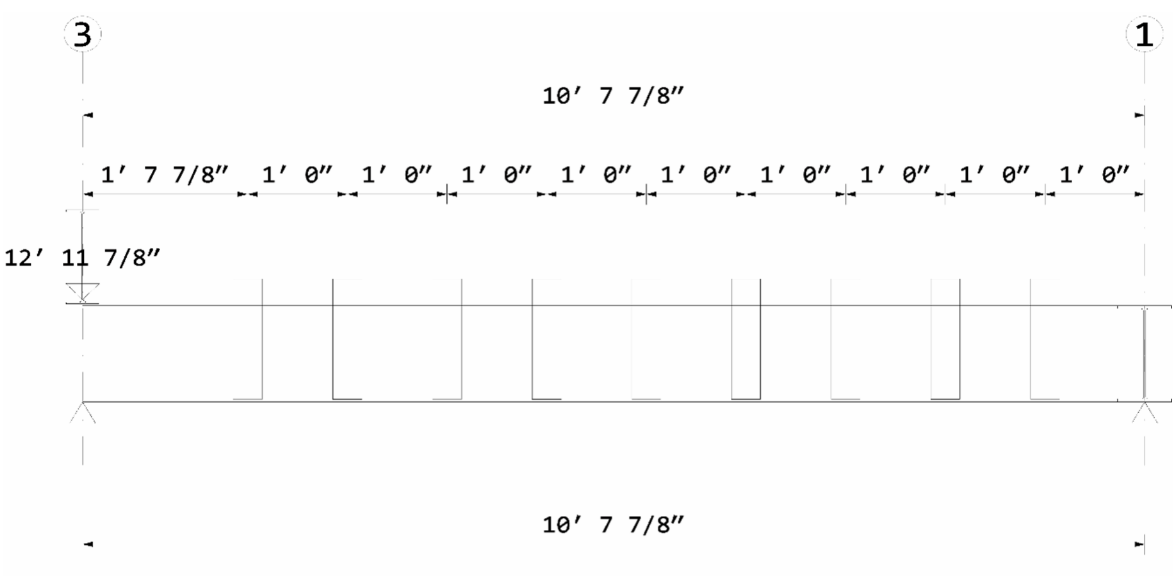
Mode	Ratio	Status
Combined buckling	0.760	Pass

Deflection
3D Building Analysis - Critical
24 LRFD$_{7.14}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Deflection Total - Critical

Position Total load deflection	=9' 8 103/128" ft, in
Max. Total load deflection	=0.322 in
Span over limit	=0.942 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.644**

Pass

1B30



St. 1 (Lower T.O.S.): 1B30 - 1 (W 12x26 A992-50)

LEOK
LEIGH + O'KANE
209 SE Douglas St.
Lee's Summit, MO 64063
816.444.3144
www.leok.com

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	1′ 7 7/8″	●	1.000		1.000		1.000		1.000
member	1′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	2′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	3′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	4′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	5′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	6′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	7′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	8′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
member	9′ 7 7/8″	●						●	
sub-beam	1′ 0″	●	1.000		1.000		1.000		1.000
support	10′ 7 7/8″	●		●		●		●	

Static Design Summary

Summary W 12x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	4	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	33	19.8	84.2	kip	0.235	✔ Pass
Shear Minor	82	-2.8	133.2	kip	0.021	✔ Pass
Flexure Major	33	-63.2	139.5	kip ft	0.453	✔ Pass
Flexure Minor	82	2.8	30.6	kip ft	0.092	✔ Pass
Axial Tension	68	-2.3	344.3	kip	0.007	✔ Pass
Axial Compression	90	1.4	241.1	kip	0.006	✔ Pass
Combined Forces	36	-	-	-	0.490	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.001	-	in	-	-
Deflection Dead	138	0.012	0.355	in	0.033	✔ Pass
Deflection Live	7	0.005	0.355	in	0.014	✔ Pass
Deflection Wind	126	-0.006	0.639	in	0.009	✔ Pass
Deflection Total	63	0.026	0.500	in	0.052	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

4 $LRFD_{4.1}$-1.2D+1.6L+0.5S - Critical

Span 1 W 12x26 A992-50 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.1 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

33 $LRFD_{8.9}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 0" - Critical

Position of V_{ry}	=0" ft, in	
Required major axis shear strength, V_{ry}	=19.8 kip	
Design shear strength	=84.2 kip	AISC 360 G2
Ratio	**=0.235**	

✔ Pass

Shear Minor

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 9' 7 7/8" - Critical

Position of V_{rx}	=9' 7 7/8" ft, in	
Required minor axis shear strength, V_{rx}	=-2.8 kip	
Design shear strength	=133.2 kip	AISC 360 G6
Ratio	**=0.021**	

✔ Pass

Flexure Major

3D Building Analysis - Critical

33 $LRFD_{8.9}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: 0" - Critical

Dist of M_{rx} along member	=0" ft, in	
Required flexural strength, M_{rx1}	=-63.2 kip ft	
Design flexural strength	=139.5 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.453**	

✔ Pass

Flexure Minor

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: 8' 7 7/8" - Critical

Dist of M_{ry} along member =8' 7 7/8" ft, in
Required flexural strength, M_{ry1} =2.8 kip ft
Design flexural strength =30.6 kip ft AISC 360 F6
Ratio **=0.092**
✔ Pass

Axial Tension
3D Building Analysis - Critical
68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x26 A992-50 - Critical
Yielding: Position 10' 7 7/8" - Critical
Distance of P_r along member =10' 7 7/8" ft, in
Required tensile strength, P_r =-2.3 kip
Design yield strength =344.3 kip AISC 360 D2
Ratio **=0.007**
✔ Pass

Axial Compression
3D Building Analysis - Critical
90 $LRFD_{10.10}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x26 A992-50 - Critical
Flexural torsional: 0" - 10' 7 7/8" - Critical

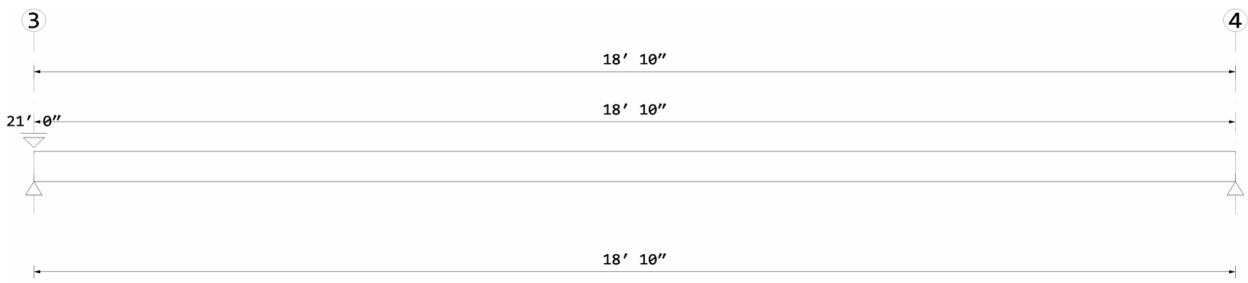
Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 10' 7 7/8"	Flexural torsional	10' 7 7/8"	1.4	241.1	kip	0.006	✔ Pass

Combined Forces
3D Building Analysis - Critical
36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Flange Btm LTB 0" - 10' 7 7/8" - Critical
Axial Comp: I.P. 0" - 10' 7 7/8" , O.o.P.0" - 1' 7 7/8" , Tors. 0" - 10' 7 7/8" - Critical

Mode	Ratio	Status
Combined buckling	0.490	✔ Pass

Deflection
3D Building Analysis - Critical
63 $LRFD_{8.39}$-1.2D+1.6S+0.5W - Critical
Span 1 W 12x26 A992-50 - Critical
Deflection Total - Critical
Position Total load deflection =6' 9 31/32" ft, in
Max. Total load deflection =0.026 in
Span over limit =0.533 in
Abs. limit =0.500 in
Design limit =0.500 in
Utilization Ratio **=0.052**
✔ Pass

2B9



St. 2 (Upper T.O.S.): 2B9 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	18′ 10″	●	1.000		1.000		1.000		1.000
support	18′ 10″	●		●		●		●	

Static Design Summary

Summary HSS 6x6x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	29	1.8	61.4	kip	0.030	Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	29	-6.2	38.6	kip ft	0.161	Pass
Flexure Minor	124	-3.0	38.6	kip ft	0.078	Pass
Axial Tension	No Significant Forces			kip	-	Not required
Axial Compression	12	2.3	115.8	kip	0.020	Pass
Combined Forces	30	-	-	-	0.208	Pass
Torsion	-	-	-	-	-	Beyond Scope
Deflection Self weight	138	0.014	-	in	-	-
Deflection Dead	138	0.031	0.628	in	0.050	Pass
Deflection Live	7	-0.004	0.628	in	0.006	Pass
Deflection Wind	133	-0.011	1.130	in	0.010	Pass
Deflection Total	29	0.104	0.942	in	0.110	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Axial: Position 0″ - Critical

Axial Force, P =1.3 kip
Elastic modulus of steel, E =29000 ksi
Minimum yield stress, F_y =46.00 ksi
Axial section class Non-slender AISC 360 Table B4.1a

Shear Major
3D Building Analysis - Critical
29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Position 0" - Critical
Position of V_{ry} =0" ft, in
Required major axis shear strength, V_{ry} =1.8 kip
Design shear strength =61.4 kip AISC 360 G4
Ratio **=0.030**
Pass

Flexure Major
3D Building Analysis - Critical
29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 0" - Critical
Dist of M_{rx} along member =0" ft, in
Required flexural strength, M_{rx1} =-6.2 kip ft
Design flexural strength =38.6 kip ft AISC 360 F1, F2 and F3
Ratio **=0.161**
Pass

Flexure Minor
3D Building Analysis - Critical
124 $LRFD_{12.2}$-0.9D+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 18' 10" - Critical
Dist of M_{ry} along member =18' 10" ft, in
Required flexural strength, M_{ry1} =-3.0 kip ft
Design flexural strength =38.6 kip ft AISC 360 F7
Ratio **=0.078**
Pass

Axial Compression
3D Building Analysis - Critical
12 $LRFD_{7.2}$-1.2D+1.6Lr+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Flexural - in-plane: 0" - 18' 10" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 18' 10"	Flexural - in-plane	18' 10"	2.3	115.8	kip	0.020	Pass

Combined Forces
3D Building Analysis - Critical
30 $LRFD_{8.6}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flange Top LTB 0″ - 18′ 10″ - Critical

Axial Comp: I.P. 0″ - 18′ 10″ , O.o.P.0″ - 18′ 10″ , Tors. 0″ - 18′ 10″ - Critical

Mode	Ratio	Status
Combined buckling	0.208	Pass

Torsion

3D Building Analysis - Critical

8 LRFD$_{6.1}$-1.2D+L+1.6S - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Beyond Scope - Critical

Member not pinned at both ends

Beyond Scope

Deflection

3D Building Analysis - Critical

29 LRFD$_{8.5}$-1.2D+1.6S+0.5W - Critical

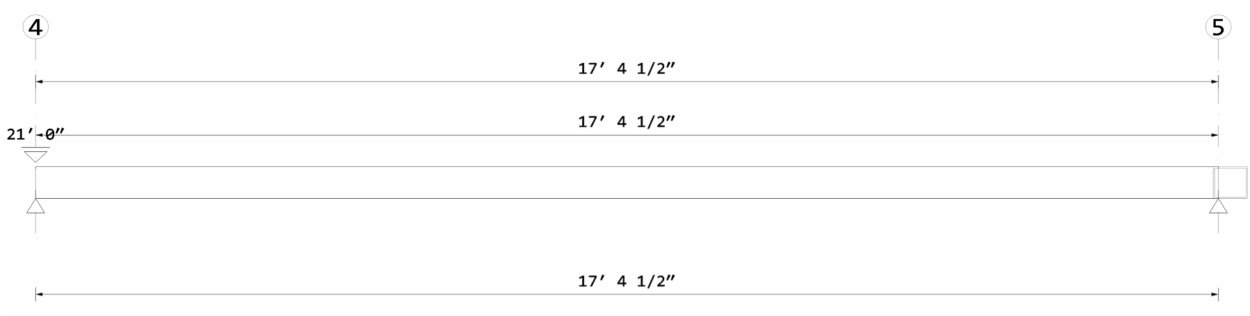
Span 1 HSS 6x6x1/4 A500B-46 - Critical

Deflection Total - Critical

Position Total load deflection	=10′ 1 11/16″ ft, in
Max. Total load deflection	=0.104 in
Span over limit	=0.942 in
Design limit	=0.942 in
Utilization Ratio	**=0.110**

Pass

2B10



St. 2 (Upper T.O.S.): 2B10 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	17′ 4 1/2″	•	1.000		1.000		1.000		1.000
support	17′ 4 1/2″	•		•		•		•	

Static Design Summary

Summary HSS 6x6x1/4

209 SE Douglas St.
Lee's Summit, MO 64063
816.444.3144
www.leok.com

Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 122
Project Name: Express Stop – View High		

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	33	-1.8	61.4	kip	0.029	Pass
Shear Minor	123	0.4	61.4	kip	0.006	Pass
Flexure Major	33	-6.8	38.6	kip ft	0.176	Pass
Flexure Minor	124	-3.5	38.6	kip ft	0.090	Pass
Axial Tension	129	-0.3	216.9	kip	0.001	Pass
Axial Compression	69	2.9	127.1	kip	0.022	Pass
Combined Forces	33	-	-	-	0.240	Pass
Torsion	-	-	-	-	-	Beyond Scope
Deflection Self weight	138	0.008	-	in	-	-
Deflection Dead	138	-0.012	0.579	in	0.020	Pass
Deflection Live	7	-0.011	0.579	in	0.019	Pass
Deflection Wind	123	0.020	1.043	in	0.019	Pass
Deflection Total	25	0.045	0.869	in	0.052	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Axial: Position 0" - Critical

Axial Force, P	=1.4 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Axial section class	Non-slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

33 $LRFD_{8.9}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 17' 4 1/2" - Critical

Position of V_{ry}	=17' 4 1/2" ft, in	
Required major axis shear strength, V_{ry}	=-1.8 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.029**	

Pass

Shear Minor

3D Building Analysis - Critical

123 $LRFD_{12.1}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 0" - Critical

Position of V_{rx} =0" ft, in
Required minor axis shear strength, V_{rx} =0.4 kip
Design shear strength =61.4 kip AISC 360 G4
Ratio **=0.006**
✓ Pass

Flexure Major
3D Building Analysis - Critical
33 $LRFD_{8.9}$-1.2D+1.6S+0.5W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 17' 4 1/2" - Critical
Dist of M_{rx} along member =17' 4 1/2" ft, in
Required flexural strength, M_{rx1} =-6.8 kip ft
Design flexural strength =38.6 kip ft AISC 360 F1, F2 and F3
Ratio **=0.176**
✓ Pass

Flexure Minor
3D Building Analysis - Critical
124 $LRFD_{12.2}$-0.9D+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 0" - Critical
Dist of M_{ry} along member =0" ft, in
Required flexural strength, M_{ry1} =-3.5 kip ft
Design flexural strength =38.6 kip ft AISC 360 F7
Ratio **=0.090**
✓ Pass

Axial Tension
3D Building Analysis - Critical
129 $LRFD_{12.7}$-0.9D+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: Position 0" - Critical
Distance of P_r along member =0" ft, in
Required tensile strength, P_r =-0.3 kip
Design yield strength =216.9 kip AISC 360 D2
Ratio **=0.001**
✓ Pass

Axial Compression
3D Building Analysis - Critical
69 $LRFD_{9.3}$-1.2D+L+0.5Lr+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Flexural - in-plane: 0" - 17' 4 1/2" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 17' 4 1/2"	Flexural - in-plane	17' 4 1/2"	2.9	127.1	kip	0.022	✓ Pass

Combined Forces

3D Building Analysis - Critical

33 $LRFD_{8.9}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flange Top LTB 0" - 17' 4 1/2" - Critical

Axial Comp: I.P. 0" - 17' 4 1/2" , O.o.P.0" - 17' 4 1/2" , Tors. 0" - 17' 4 1/2" - Critical

Mode	Ratio	Status
Combined buckling	0.240	Pass

Torsion

3D Building Analysis - Critical

8 $LRFD_{6.1}$-1.2D+L+1.6S - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Beyond Scope - Critical

Member not pinned at both ends

Beyond Scope

Deflection

3D Building Analysis - Critical

25 $LRFD_{8.1}$-1.2D+1.6S+0.5W - Critical

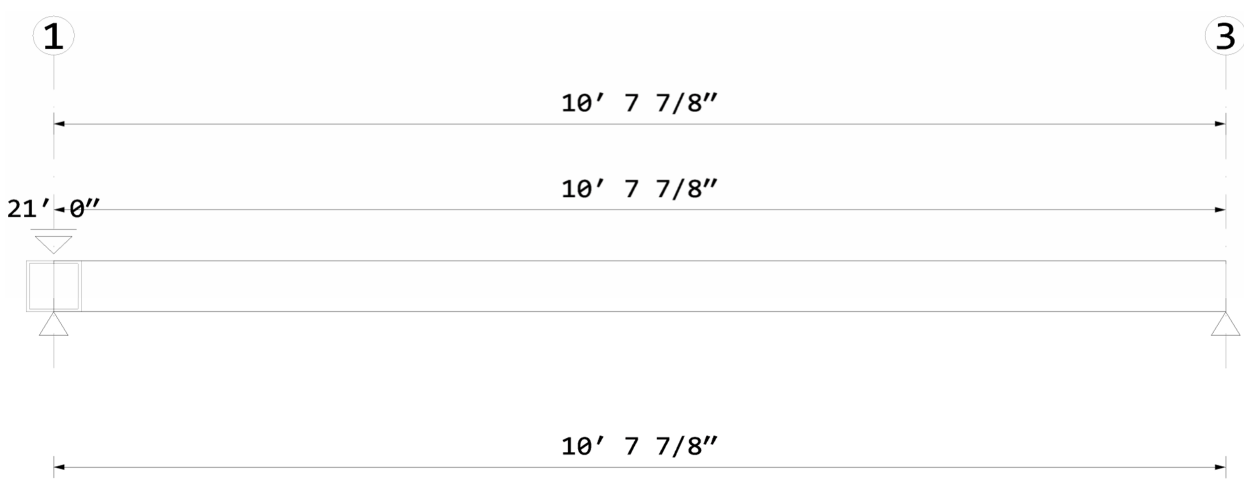
Span 1 HSS 6x6x1/4 A500B-46 - Critical

Deflection Total - Critical

Position Total load deflection	=8' 8 81/128" ft, in
Max. Total load deflection	=0.045 in
Span over limit	=0.869 in
Design limit	=0.869 in
Utilization Ratio	**=0.052**

Pass

2B8



St. 2 (Upper T.O.S.): 2B8 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0”	•		•		•		•	
sub-beam	10’ 7 7/8”	•	1.000		1.000		1.000		1.000
support	10’ 7 7/8”	•		•		•		•	

Static Design Summary

Summary HSS 6x6x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	29	1.5	61.4	kip	0.024	Pass
Shear Minor	134	-0.9	61.4	kip	0.014	Pass
Flexure Major	29	-4.9	38.6	kip ft	0.126	Pass
Flexure Minor	134	7.5	38.6	kip ft	0.195	Pass
Axial Tension	128	-0.6	216.9	kip	0.003	Pass
Axial Compression	120	1.5	177.4	kip	0.009	Pass
Combined Forces	86	-	-	-	0.248	Pass
Torsion	-	-	-	-	-	Beyond Scope
Deflection Self weight	138	0.001	-	in	-	-
Deflection Dead	138	-0.005	0.355	in	0.013	Pass
Deflection Live	7	0.003	0.355	in	0.008	Pass
Deflection Wind	132	0.007	0.639	in	0.011	Pass
Deflection Total	29	-0.011	0.533	in	0.021	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Axial: Position 0” - Critical

Axial Force, P	=0.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Axial section class	Non-slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 0” - Critical

Position of V_{ry}	=0” ft, in	
Required major axis shear strength, V_{ry}	=1.5 kip	
Design shear strength	=61.4 kip	AISC 360 G4

Ratio =**0.024**

✓ Pass

Shear Minor

3D Building Analysis - Critical

134 $LRFD_{12.12}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 0" - Critical

Position of V_{rx}	=0" ft, in	
Required minor axis shear strength, V_{rx}	=-0.9 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.014**	

✓ Pass

Flexure Major

3D Building Analysis - Critical

29 $LRFD_{8.5}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: 0" - Critical

Dist of M_{rx} along member	=0" ft, in	
Required flexural strength, M_{rx1}	=-4.9 kip ft	
Design flexural strength	=38.6 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.126**	

✓ Pass

Flexure Minor

3D Building Analysis - Critical

134 $LRFD_{12.12}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: 0" - Critical

Dist of M_{ry} along member	=0" ft, in	
Required flexural strength, M_{ry1}	=7.5 kip ft	
Design flexural strength	=38.6 kip ft	AISC 360 F7
Ratio	**=0.195**	

✓ Pass

Axial Tension

3D Building Analysis - Critical

128 $LRFD_{12.6}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Yielding: Position 0" - Critical

Distance of P_r along member	=0" ft, in	
Required tensile strength, P_r	=-0.6 kip	
Design yield strength	=216.9 kip	AISC 360 D2
Ratio	**=0.003**	

✓ Pass

Axial Compression

3D Building Analysis - Critical

120 LRFD10.40-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flexural - in-plane: 0" - 10' 7 7/8" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 10' 7 7/8"	Flexural - in-plane	10' 7 7/8"	1.5	177.4	kip	0.009	Pass

Combined Forces

3D Building Analysis - Critical

86 LRFD10.6-1.2D+L+0.5S+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flange Top LTB 0" - 10' 7 7/8" - Critical

Axial Tension: 0" - 10' 7 7/8" - Critical

Mode	Ratio	Status
Combined buckling	0.248	Pass

Torsion

3D Building Analysis - Critical

4 LRFD4.1-1.2D+1.6L+0.5S - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Beyond Scope - Critical

Member not pinned at both ends

Beyond Scope

Deflection

3D Building Analysis - Critical

29 LRFD8.5-1.2D+1.6S+0.5W - Critical

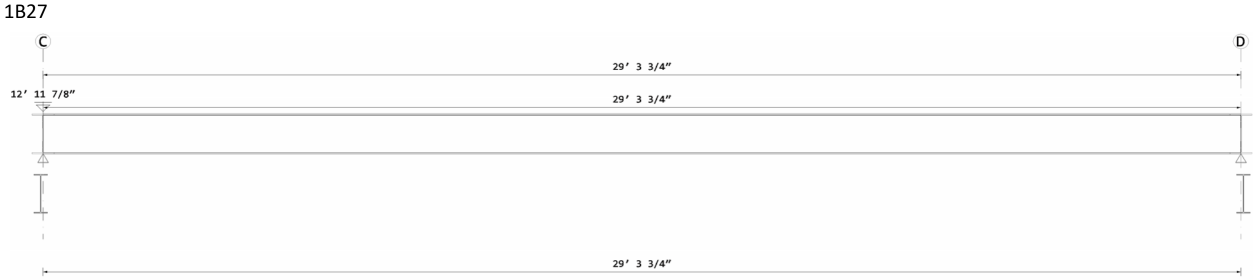
Span 1 HSS 6x6x1/4 A500B-46 - Critical

Deflection Total - Critical

Position Total load deflection	=2' 1 117/256" ft, in
Max. Total load deflection	=-0.011 in
Span over limit	=0.533 in
Design limit	=0.533 in
Utilization Ratio	**=0.021**

Pass

1B27



St. 1 (Lower T.O.S.): 1B27 - 1 (W 12x26 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	29′ 3 3/4″	●	1.000		1.000		1.000		1.000
support	29′ 3 3/4″	●		●		●		●	

Static Design Summary

Summary W 12x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	12	-2.2	84.2	kip	0.026	Pass
Shear Minor		No Significant Forces		kip	-	Not required
Flexure Major	68	-17.0	71.2	kip ft	0.239	Pass
Flexure Minor		No Significant Forces		kip ft	-	Not required
Axial Tension	91	-2.8	344.3	kip	0.008	Pass
Axial Compression	72	1.6	31.8	kip	0.049	Pass
Combined Forces	78	-	-	-	0.198	Pass
Torsion		No Significant Forces		-	-	Not required
Deflection Self weight	138	0.030	-	in	-	-
Deflection Dead	138	0.056	0.977	in	0.057	Pass
Deflection Live	7	0.026	0.977	in	0.027	Pass
Deflection Wind	123	-0.033	1.759	in	0.019	Pass
Deflection Total	22	0.119	0.500	in	0.237	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x26 A992-50 - Critical

Axial: Position 0″ - Critical

Axial Force, P	=0.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

12 $LRFD_{7.2}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 29′ 3 3/4″ - Critical

Position of V_{ry}	=29′ 3 3/4″ ft, in	
Required major axis shear strength, V_{ry}	=-2.2 kip	
Design shear strength	=84.2 kip	AISC 360 G2

Ratio =**0.026**

✔ Pass

Flexure Major

3D Building Analysis - Critical

68 LRFD$_{9.2}$-1.2D+L+0.5Lr+W - Critical

Span 1 W 12x26 A992-50 - Critical

LTB Flange Btm: 0" - 29' 3 3/4" - Critical

Dist of M_{rx1} along member =29' 3 3/4" ft, in

Required flexural strength, M_{rx1} =-17.0 kip ft

Design flexural strength =71.2 kip ft AISC 360 F1, F2, F3 and F4

Ratio **=0.239**

✔ Pass

Axial Tension

3D Building Analysis - Critical

91 LRFD$_{10.11}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: Position 29' 3 3/4" - Critical

Distance of P_r along member =29' 3 3/4" ft, in

Required tensile strength, P_r =-2.8 kip

Design yield strength =344.3 kip AISC 360 D2

Ratio **=0.008**

✔ Pass

Axial Compression

3D Building Analysis - Critical

72 LRFD$_{9.6}$-1.2D+L+0.5Lr+W - Critical

Span 1 W 12x26 A992-50 - Critical

Flexural - out-of-plane: 0" - 29' 3 3/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 29' 3 3/4"	Flexural - out-of-plane	29' 3 3/4"	1.6	31.8	kip	0.049	✔ Pass

Combined Forces

3D Building Analysis - Critical

78 LRFD$_{9.12}$-1.2D+L+0.5Lr+W - Critical

Span 1 W 12x26 A992-50 - Critical

Flange Btm LTB 0" - 29' 3 3/4" - Critical

Axial Tension: 0" - 29' 3 3/4" - Critical

Mode	Ratio	Status
Combined buckling	0.198	✔ Pass

Deflection

3D Building Analysis - Critical

22 LRFD$_{7.12}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

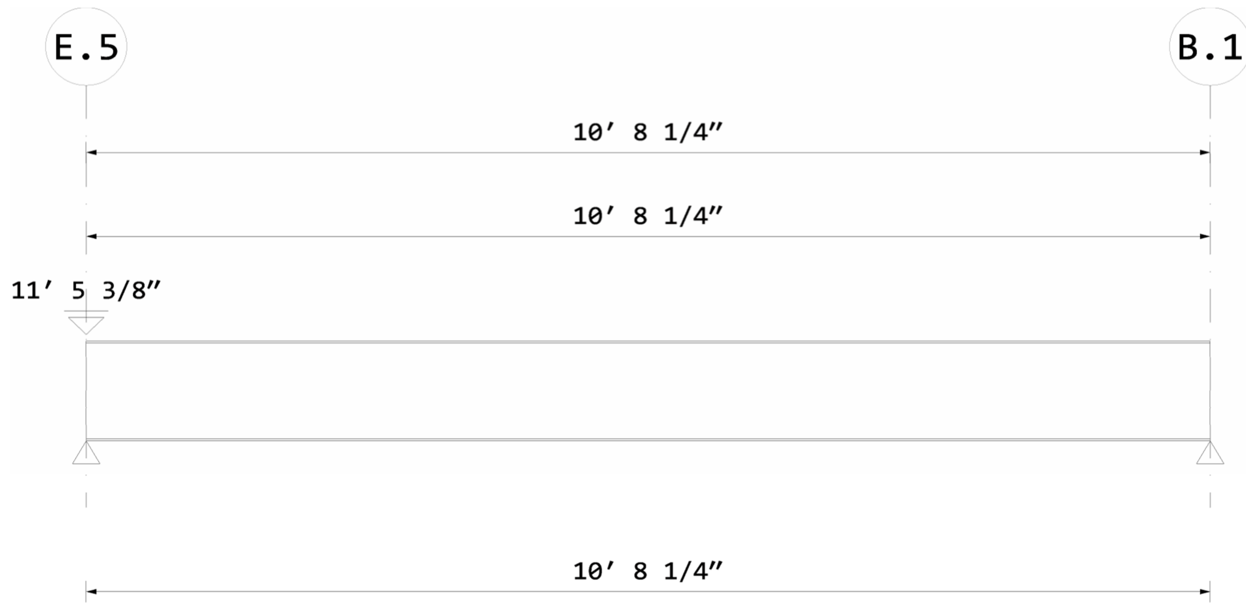
Deflection Total - Critical

Position Total load deflection =13' 253/256" ft, in

Max. Total load deflection	=0.119 in
Span over limit	=1.466 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.237**

✓ Pass

1B18



St. 1 (Lower T.O.S.): 1B18 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	10′ 8 1/4″	•	1.000		1.000		1.000		1.000
support	10′ 8 1/4″	•		•		•		•	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	✓ Pass
Flexural Classification	1	Compact	-	-	-	✓ Pass
Shear Major	26	-1.6	64.3	kip	0.025	✓ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	26	-3.1	37.4	kip ft	0.084	✓ Pass
Flexure Minor	No Significant Forces			kip ft	-	Not required

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Tension	133	-2.7	187.2	kip	0.014	Pass
Axial Compression	111	1.4	32.4	kip	0.045	Pass
Combined Forces	36	-	-	-	0.087	Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.001	-	in	-	-
Deflection Dead	138	0.004	0.356	in	0.012	Pass
Deflection Live	7	0.001	0.356	in	0.003	Pass
Deflection Wind	124	0.002	0.641	in	0.003	Pass
Deflection Total	26	0.012	0.500	in	0.024	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.1 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 10' 8 1/4" - Critical

Position of V_{ry}	=10' 8 1/4" ft, in	
Required major axis shear strength, V_{ry}	=-1.6 kip	
Design shear strength	=64.3 kip	AISC 360 G2
Ratio	**=0.025**	

Pass

Flexure Major

3D Building Analysis - Critical

26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

LTB Flange Btm: 0" - 10' 8 1/4" - Critical

Dist of M_{rx1} along member	=10' 8 1/4" ft, in	
Required flexural strength, M_{rx1}	=-3.1 kip ft	
Design flexural strength	=37.4 kip ft	AISC 360 F1, F2, F3 and F4
Ratio	**=0.084**	

Pass

Axial Tension

3D Building Analysis - Critical

133 $LRFD_{12.11}$-0.9D+W - Critical

Span 1 W 12x14 A992-50 - Critical

Yielding: Position 10' 8 1/4" - Critical

Distance of P_r along member	=10' 8 1/4" ft, in	
Required tensile strength, P_r	=-2.7 kip	
Design yield strength	=187.2 kip	AISC 360 D2
Ratio	**=0.014**	

Pass

Axial Compression

3D Building Analysis - Critical

111 $LRFD_{10.31}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Flexural - out-of-plane: 0" - 10' 8 1/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 10' 8 1/4"	Flexural - out-of-plane	10' 8 1/4"	1.4	32.4	kip	0.045	Pass

Combined Forces

3D Building Analysis - Critical

36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Flange Btm LTB 0" - 10' 8 1/4" - Critical

Axial Tension: 0" - 10' 8 1/4" - Critical

Mode	Ratio	Status
Combined buckling	0.087	Pass

Deflection

3D Building Analysis - Critical

26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=4' 6 27/256" ft, in
Max. Total load deflection	=0.012 in
Span over limit	=0.534 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.024**

Pass

1B19

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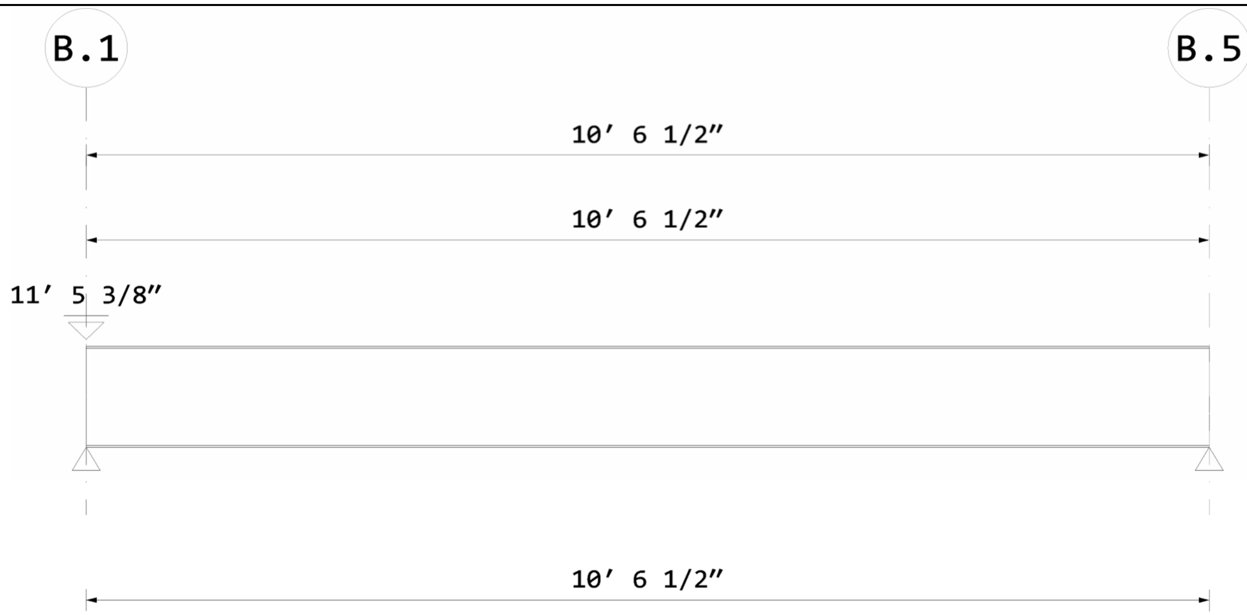


St. 1 (Lower T.O.S.): 1B19 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	10' 6 1/2"	•	1.000		1.000		1.000		1.000
support	10' 6 1/2"	•		•		•		•	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	30	-1.3	64.3	kip	0.021	✔ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	30	-3.3	65.2	kip ft	0.051	✔ Pass
Flexure Minor	No Significant Forces			kip ft	-	Not required
Axial Tension	91	-3.4	187.2	kip	0.018	✔ Pass
Axial Compression	114	2.1	33.3	kip	0.063	✔ Pass
Combined Forces	86	-	-	-	0.067	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.001	-	in	-	-
Deflection Dead	138	0.001	0.351	in	0.003	✔ Pass
Deflection Live	7	0.000	0.351	in	0.001	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Deflection Wind	124	0.003	0.633	in	0.004	Pass
Deflection Total	12	0.004	0.500	in	0.008	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.2 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

30 $LRFD_{8.6}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 10' 6 1/2" - Critical

Position of V_{ry}	=10' 6 1/2" ft, in	
Required major axis shear strength, V_{ry}	=-1.3 kip	
Design shear strength	=64.3 kip	AISC 360 G2
Ratio	**=0.021**	

Pass

Flexure Major

3D Building Analysis - Critical

30 $LRFD_{8.6}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Yielding: 10' 6 1/2" - Critical

Dist of M_{rx} along member	=10' 6 1/2" ft, in	
Required flexural strength, M_{rx1}	=-3.3 kip ft	
Design flexural strength	=65.2 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.051**	

Pass

Axial Tension

3D Building Analysis - Critical

91 $LRFD_{10.11}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Yielding: Position 10' 6 1/2" - Critical

Distance of P_r along member	=10' 6 1/2" ft, in	
Required tensile strength, P_r	=-3.4 kip	
Design yield strength	=187.2 kip	AISC 360 D2
Ratio	**=0.018**	

Pass

Axial Compression
3D Building Analysis - Critical
114 $LRFD_{10.34}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flexural - out-of-plane: 0" - 10' 6 1/2" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 10' 6 1/2"	Flexural - out-of-plane	10' 6 1/2"	2.1	33.3	kip	0.063	✔ Pass

Combined Forces
3D Building Analysis - Critical
86 $LRFD_{10.6}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flange Top LTB 0" - 10' 6 1/2" - Critical
Axial Comp: I.P. 0" - 10' 6 1/2" , O.o.P.0" - 10' 6 1/2" , Tors. 0" - 10' 6 1/2" - Critical

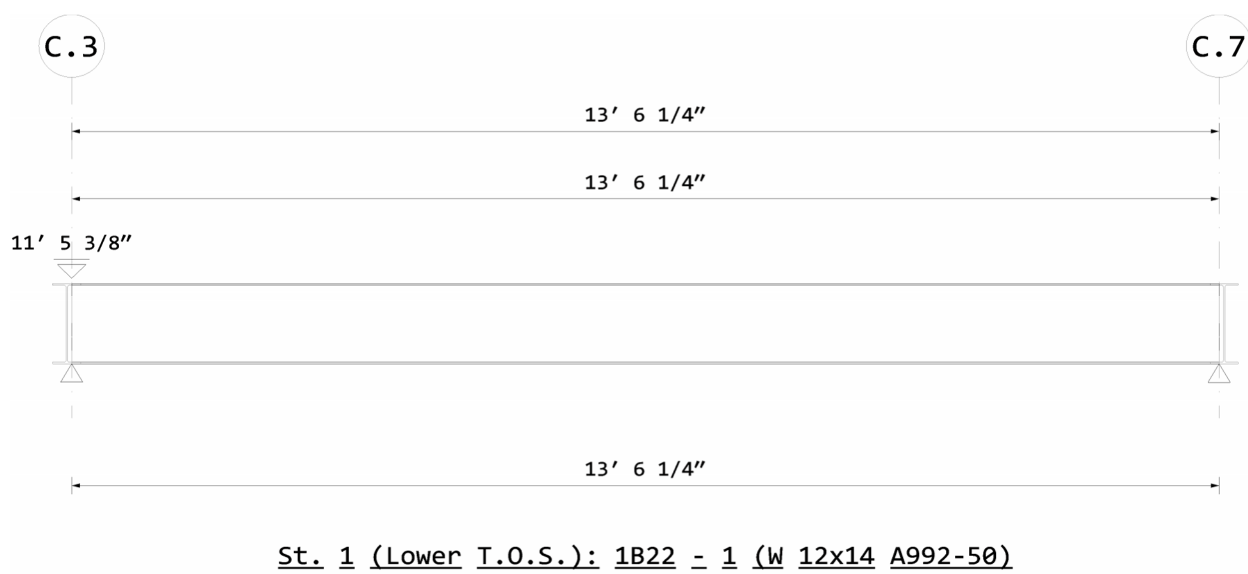
Mode	Ratio	Status
Combined buckling	0.067	✔ Pass

Deflection
3D Building Analysis - Critical
12 $LRFD_{7.2}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Deflection Total - Critical

Position Total load deflection	=5' 4 63/128" ft, in
Max. Total load deflection	=0.004 in
Span over limit	=0.527 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.008**

✔ Pass

1B22



Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	13′ 6 1/4″	●	1.000		1.000		1.000		1.000
support	13′ 6 1/4″	●		●		●		●	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	82	-2.6	64.3	kip	0.041	Pass
Shear Minor	84	-0.4	48.2	kip	0.009	Pass
Flexure Major	82	-13.9	34.1	kip ft	0.407	Pass
Flexure Minor	84	0.5	7.1	kip ft	0.072	Pass
Axial Tension	133	-0.5	187.2	kip	0.003	Pass
Axial Compression	94	1.3	20.2	kip	0.067	Pass
Combined Forces	82	-	-	-	0.405	Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.002	-	in	-	-
Deflection Dead	138	0.008	0.451	in	0.017	Pass
Deflection Live	7	0.001	0.451	in	0.003	Pass
Deflection Wind	123	-0.016	0.811	in	0.020	Pass
Deflection Total	26	0.028	0.500	in	0.055	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0″ - Critical

Axial Force, P	=0.2 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 13′ 6 1/4″ - Critical

Position of V_{ry}	=13′ 6 1/4″ ft, in
Required major axis shear strength, V_{ry}	=-2.6 kip

Design shear strength =64.3 kip AISC 360 G2
Ratio **=0.041**
✔ Pass

Shear Minor
3D Building Analysis - Critical
84 LRFD$_{10.4}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Position 6′ 9 1/8″ - Critical
Position of V_{rx} =6′ 9 1/8″ ft, in
Required minor axis shear strength, V_{rx} =-0.4 kip
Design shear strength =48.2 kip AISC 360 G6
Ratio **=0.009**
✔ Pass

Flexure Major
3D Building Analysis - Critical
82 LRFD$_{10.2}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
LTB Flange Btm: 0″ - 13′ 6 1/4″ - Critical
Dist of M_{rx1} along member =13′ 6 1/4″ ft, in
Required flexural strength, M_{rx1} =-13.9 kip ft
Design flexural strength =34.1 kip ft AISC 360 F1, F2, F3 and F4
Ratio **=0.407**
✔ Pass

Flexure Minor
3D Building Analysis - Critical
84 LRFD$_{10.4}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: 6′ 9 1/8″ - Critical
Dist of M_{ry} along member =6′ 9 1/8″ ft, in
Required flexural strength, M_{ry1} =0.5 kip ft
Design flexural strength =7.1 kip ft AISC 360 F6
Ratio **=0.072**
✔ Pass

Axial Tension
3D Building Analysis - Critical
133 LRFD$_{12.11}$-0.9D+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: Position 13′ 6 1/4″ - Critical
Distance of P_r along member =13′ 6 1/4″ ft, in
Required tensile strength, P_r =-0.5 kip
Design yield strength =187.2 kip AISC 360 D2
Ratio **=0.003**
✔ Pass

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Axial Compression

3D Building Analysis - Critical

94 LRFD$_{10.14}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Flexural - out-of-plane: 0" - 13' 6 1/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 13' 6 1/4"	Flexural - out-of-plane	13' 6 1/4"	1.3	20.2	kip	0.067	✔ Pass

Combined Forces

3D Building Analysis - Critical

82 LRFD$_{10.2}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Flange Btm LTB 0" - 13' 6 1/4" - Critical

Axial Tension: 0" - 13' 6 1/4" - Critical

Mode	Ratio	Status
Combined buckling	0.405	✔ Pass

Deflection

3D Building Analysis - Critical

26 LRFD$_{8.2}$-1.2D+1.6S+0.5W - Critical

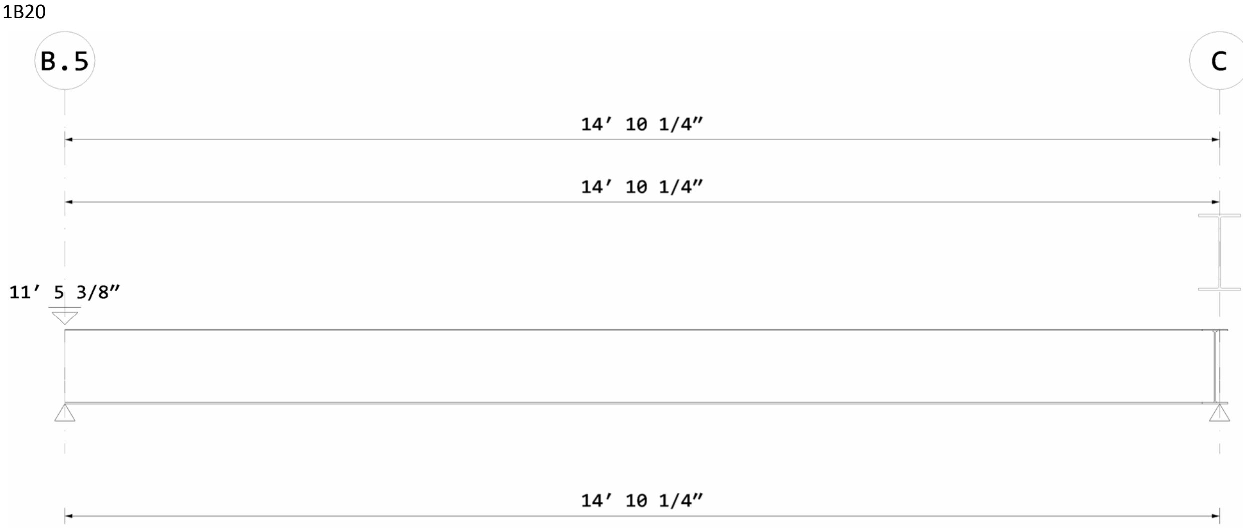
Span 1 W 12x14 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=4' 3 223/256" ft, in
Max. Total load deflection	=0.028 in
Span over limit	=0.676 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.055**

✔ Pass

1B20



St. 1 (Lower T.O.S.): 1B20 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	14′ 10 1/4″	•	1.000		1.000		1.000		1.000
support	14′ 10 1/4″	•		•		•		•	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	26	-2.2	64.3	kip	0.034	✔ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	68	-9.6	34.9	kip ft	0.275	✔ Pass
Flexure Minor	No Significant Forces			kip ft	-	Not required
Axial Tension	133	-4.1	187.2	kip	0.022	✔ Pass
Axial Compression	114	3.0	16.8	kip	0.178	✔ Pass
Combined Forces	68	-	-	-	0.278	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Dead	138	0.002	0.495	in	0.004	✔ Pass
Deflection Live	7	-0.002	0.495	in	0.003	✔ Pass
Deflection Wind	124	-0.023	0.891	in	0.026	✔ Pass
Deflection Total	124	-0.025	0.500	in	0.050	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0″ - Critical

Axial Force, P	=0.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 14′ 10 1/4″ - Critical

Position of V_{ry}	=14′ 10 1/4″ ft, in
Required major axis shear strength, V_{ry}	=-2.2 kip

Design shear strength =64.3 kip AISC 360 G2
Ratio **=0.034**
Pass

Flexure Major
3D Building Analysis - Critical
68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x14 A992-50 - Critical
LTB Flange Btm: 0" - 14' 10 1/4" - Critical
Dist of M_{rx1} along member =14' 10 1/4" ft, in
Required flexural strength, M_{rx1} =-9.6 kip ft
Design flexural strength =34.9 kip ft AISC 360 F1, F2, F3 and F4
Ratio **=0.275**
Pass

Axial Tension
3D Building Analysis - Critical
133 $LRFD_{12.11}$-0.9D+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: Position 14' 10 1/4" - Critical
Distance of P_r along member =14' 10 1/4" ft, in
Required tensile strength, P_r =-4.1 kip
Design yield strength =187.2 kip AISC 360 D2
Ratio **=0.022**
Pass

Axial Compression
3D Building Analysis - Critical
114 $LRFD_{10.34}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flexural - out-of-plane: 0" - 14' 10 1/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 14' 10 1/4"	Flexural - out-of-plane	14' 10 1/4"	3.0	16.8	kip	0.178	Pass

Combined Forces
3D Building Analysis - Critical
68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flange Btm LTB 0" - 14' 10 1/4" - Critical
Axial Tension: 0" - 14' 10 1/4" - Critical

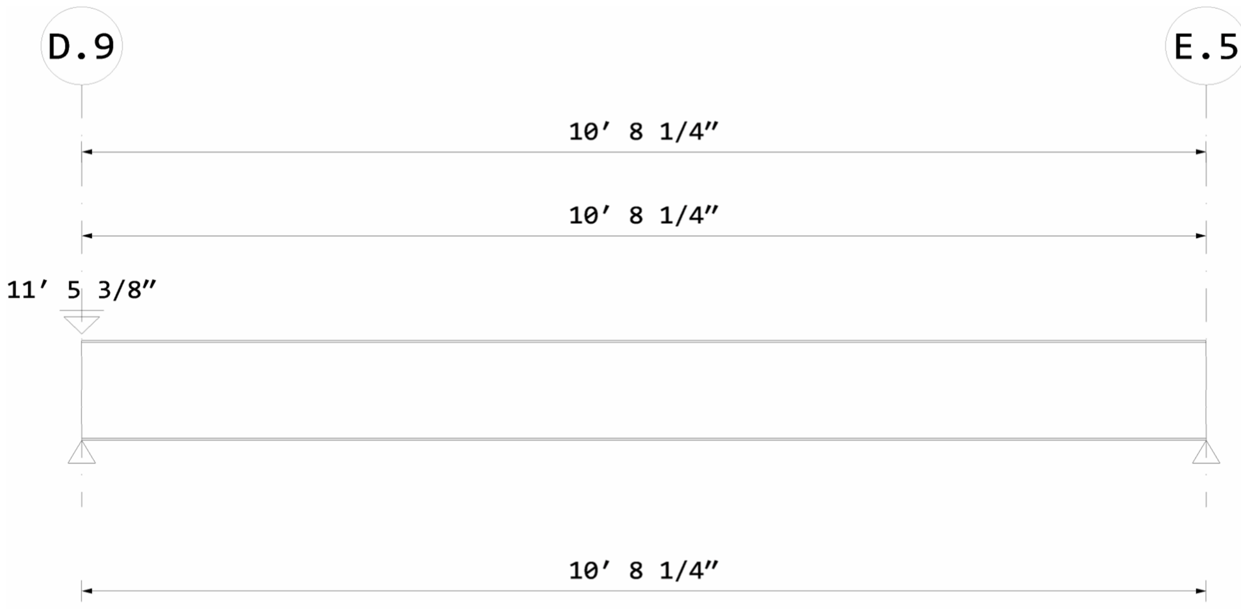
Mode	Ratio	Status
Combined buckling	0.278	Pass

Deflection
3D Building Analysis - Critical
124 $LRFD_{12.2}$-0.9D+W - Critical
Span 1 W 12x14 A992-50 - Critical
Deflection Total - Critical

Position Total load deflection	=9' 11 177/256" ft, in
Max. Total load deflection	=-0.025 in
Span over limit	=0.743 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.050**

✔ Pass

1B26



St. 1 (Lower T.O.S.): 1B26 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	10' 8 1/4"	•	1.000		1.000		1.000		1.000
support	10' 8 1/4"	•		•		•		•	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	32	1.6	64.3	kip	0.024	✔ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	32	-3.0	36.5	kip ft	0.082	✔ Pass
Flexure Minor	No Significant Forces			kip ft	-	Not required

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Tension	132	-2.7	187.2	kip	0.014	✔ Pass
Axial Compression	114	1.5	32.4	kip	0.047	✔ Pass
Combined Forces	33	-	-	-	0.086	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.001	-	in	-	-
Deflection Dead	138	0.004	0.356	in	0.012	✔ Pass
Deflection Live	7	0.001	0.356	in	0.003	✔ Pass
Deflection Wind	123	-0.002	0.641	in	0.003	✔ Pass
Deflection Total	30	0.012	0.500	in	0.023	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.2 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

32 $LRFD_{8.8}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 0" - Critical

Position of V_{ry}	=0" ft, in	
Required major axis shear strength, V_{ry}	=1.6 kip	
Design shear strength	=64.3 kip	AISC 360 G2
Ratio	**=0.024**	

✔ Pass

Flexure Major

3D Building Analysis - Critical

32 $LRFD_{8.8}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

LTB Flange Btm: 0" - 10' 8 1/4" - Critical

Dist of M_{rx1} along member	=0" ft, in	
Required flexural strength, M_{rx1}	=-3.0 kip ft	
Design flexural strength	=36.5 kip ft	AISC 360 F1, F2, F3 and F4
Ratio	**=0.082**	

✔ Pass

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Axial Tension

3D Building Analysis - Critical

132 LRFD$_{12.10}$-0.9D+W - Critical

Span 1 W 12x14 A992-50 - Critical

Yielding: Position 0" - Critical

Distance of P_r along member	=0" ft, in	
Required tensile strength, P_r	=-2.7 kip	
Design yield strength	=187.2 kip	AISC 360 D2
Ratio	**=0.014**	

Pass

Axial Compression

3D Building Analysis - Critical

114 LRFD$_{10.34}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Flexural - out-of-plane: 0" - 10' 8 1/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 10' 8 1/4"	Flexural - out-of-plane	10' 8 1/4"	1.5	32.4	kip	0.047	Pass

Combined Forces

3D Building Analysis - Critical

33 LRFD$_{8.9}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Flange Btm LTB 0" - 10' 8 1/4" - Critical

Axial Tension: 0" - 10' 8 1/4" - Critical

Mode	Ratio	Status
Combined buckling	0.086	Pass

Deflection

3D Building Analysis - Critical

30 LRFD$_{8.6}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=6' 31/128" ft, in
Max. Total load deflection	=0.012 in
Span over limit	=0.534 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.023**

Pass

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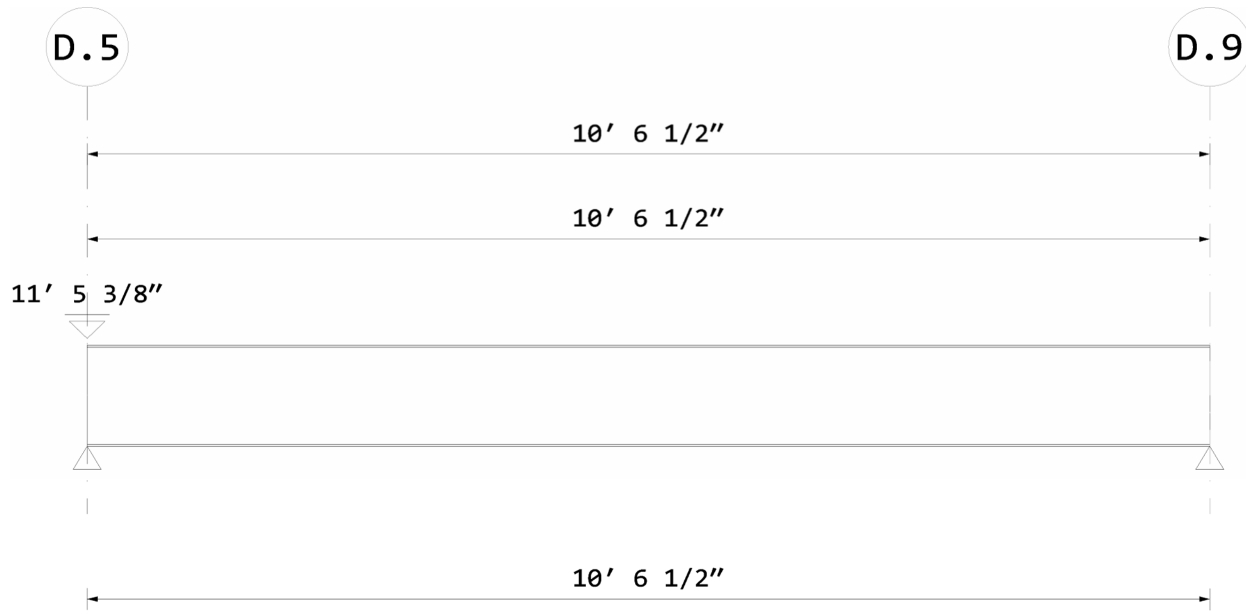
Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 144
Project Name: Express Stop – View High		

1B25



St. 1 (Lower T.O.S.): 1B25 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	10′ 6 1/2″	●	1.000		1.000		1.000		1.000
support	10′ 6 1/2″	●		●		●		●	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	27	1.3	64.3	kip	0.021	✔ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	27	-3.4	65.2	kip ft	0.052	✔ Pass
Flexure Minor	No Significant Forces			kip ft	-	Not required
Axial Tension	91	-3.4	187.2	kip	0.018	✔ Pass
Axial Compression	111	2.2	33.3	kip	0.066	✔ Pass
Combined Forces	83	-	-	-	0.070	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.001	-	in	-	-
Deflection Dead	138	0.001	0.351	in	0.003	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Deflection Live	7	0.000	0.351	in	0.001	✔ Pass
Deflection Wind	123	-0.003	0.633	in	0.004	✔ Pass
Deflection Total	18	0.004	0.500	in	0.007	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.2 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

27 $LRFD_{8.3}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 0" - Critical

Position of V_{ry}	=0" ft, in	
Required major axis shear strength, V_{ry}	=1.3 kip	
Design shear strength	=64.3 kip	AISC 360 G2
Ratio	**=0.021**	

✔ Pass

Flexure Major

3D Building Analysis - Critical

27 $LRFD_{8.3}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Yielding: 0" - Critical

Dist of M_{rx} along member	=0" ft, in	
Required flexural strength, M_{rx1}	=-3.4 kip ft	
Design flexural strength	=65.2 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.052**	

✔ Pass

Axial Tension

3D Building Analysis - Critical

91 $LRFD_{10.11}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Yielding: Position 0" - Critical

Distance of P_r along member	=0" ft, in	
Required tensile strength, P_r	=-3.4 kip	
Design yield strength	=187.2 kip	AISC 360 D2
Ratio	**=0.018**	

✔ Pass

Axial Compression

3D Building Analysis - Critical

111 $LRFD_{10.31}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Flexural - out-of-plane: 0" - 10' 6 1/2" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 10' 6 1/2"	Flexural - out-of-plane	10' 6 1/2"	2.2	33.3	kip	0.066	✔ Pass

Combined Forces

3D Building Analysis - Critical

83 $LRFD_{10.3}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x14 A992-50 - Critical

Flange Top LTB 0" - 10' 6 1/2" - Critical

Axial Comp: I.P. 0" - 10' 6 1/2" , O.o.P.0" - 10' 6 1/2" , Tors. 0" - 10' 6 1/2" - Critical

Mode	Ratio	Status
Combined buckling	0.070	✔ Pass

Deflection

3D Building Analysis - Critical

18 $LRFD_{7.8}$-1.2D+1.6Lr+0.5W - Critical

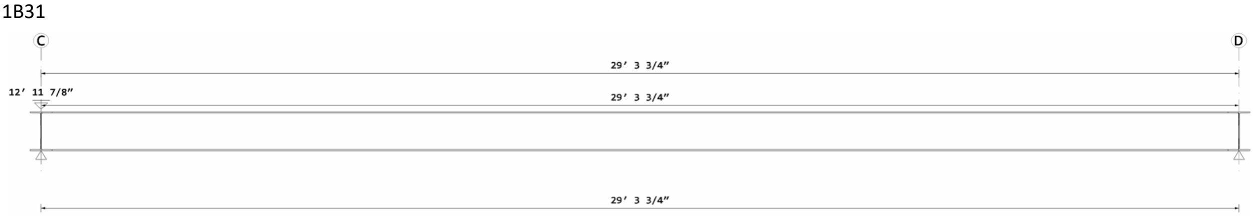
Span 1 W 12x14 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=5' 2 11/64" ft, in
Max. Total load deflection	=0.004 in
Span over limit	=0.527 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.007**

✔ Pass

1B31



St. 1 (Lower T.O.S.): 1B31 - 1 (W 12x26 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	29' 3 3/4"	•	1.000		1.000		1.000		1.000
support	29' 3 3/4"	•		•		•		•	

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Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 147
Project Name: Express Stop – View High		

Static Design Summary

Summary W 12x26

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	11	Slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	55	-2.3	84.2	kip	0.027	Pass
Shear Minor	133	1.0	133.2	kip	0.007	Pass
Flexure Major	55	-11.3	74.2	kip ft	0.152	Pass
Flexure Minor	133	-2.6	30.6	kip ft	0.084	Pass
Axial Tension	83	-2.4	344.3	kip	0.007	Pass
Axial Compression	134	1.4	31.8	kip	0.043	Pass
Combined Forces	120	-	-	-	0.197	Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.016	-	in	-	-
Deflection Dead	138	0.036	0.977	in	0.037	Pass
Deflection Live	7	0.014	0.977	in	0.015	Pass
Deflection Wind	123	-0.007	1.759	in	0.004	Pass
Deflection Total	55	0.079	0.500	in	0.157	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

11 $LRFD_{7.1}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Axial: Position 29' 3 3/4" - Critical

Axial Force, P	=0.4 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

55 $LRFD_{8.31}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 29' 3 3/4" - Critical

Position of V_{ry}	=29' 3 3/4" ft, in	
Required major axis shear strength, V_{ry}	=-2.3 kip	
Design shear strength	=84.2 kip	AISC 360 G2
Ratio	**=0.027**	

Pass

Shear Minor

3D Building Analysis - Critical

133 $LRFD_{12.11}$-0.9D+W - Critical

Span 1 W 12x26 A992-50 - Critical

Position 14' 7 7/8" - Critical

Position of V_{rx} =14' 7 7/8" ft, in

Required minor axis shear strength, V_{rx} =1.0 kip

Design shear strength =133.2 kip AISC 360 G6

Ratio **=0.007**

✔ Pass

Flexure Major

3D Building Analysis - Critical

55 $LRFD_{8.31}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x26 A992-50 - Critical

LTB Flange Btm: 0" - 29' 3 3/4" - Critical

Dist of M_{rx1} along member =29' 3 3/4" ft, in

Required flexural strength, M_{rx1} =-11.3 kip ft

Design flexural strength =74.2 kip ft AISC 360 F1, F2, F3 and F4

Ratio **=0.152**

✔ Pass

Flexure Minor

3D Building Analysis - Critical

133 $LRFD_{12.11}$-0.9D+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: 14' 7 7/8" - Critical

Dist of M_{ry} along member =14' 7 7/8" ft, in

Required flexural strength, M_{ry1} =-2.6 kip ft

Design flexural strength =30.6 kip ft AISC 360 F6

Ratio **=0.084**

✔ Pass

Axial Tension

3D Building Analysis - Critical

83 $LRFD_{10.3}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Yielding: Position 0" - Critical

Distance of P_r along member =0" ft, in

Required tensile strength, P_r =-2.4 kip

Design yield strength =344.3 kip AISC 360 D2

Ratio **=0.007**

✔ Pass

Axial Compression

3D Building Analysis - Critical

134 $LRFD_{12.12}$-0.9D+W - Critical

Span 1 W 12x26 A992-50 - Critical

Flexural - out-of-plane: 0" - 29' 3 3/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 29' 3 3/4"	Flexural - out-of-plane	29' 3 3/4"	1.4	31.8	kip	0.043	✔ Pass

Combined Forces

3D Building Analysis - Critical

120 LRFD$_{10.40}$-1.2D+L+0.5S+W - Critical

Span 1 W 12x26 A992-50 - Critical

Flange Btm LTB 0” - 29’ 3 3/4” - Critical

Axial Comp: I.P. 0” - 29’ 3 3/4” , O.o.P.0” - 29’ 3 3/4” , Tors. 0” - 29’ 3 3/4” - Critical

Mode	Ratio	Status
Combined buckling	0.197	✔ Pass

Deflection

3D Building Analysis - Critical

55 LRFD$_{8.31}$-1.2D+1.6S+0.5W - Critical

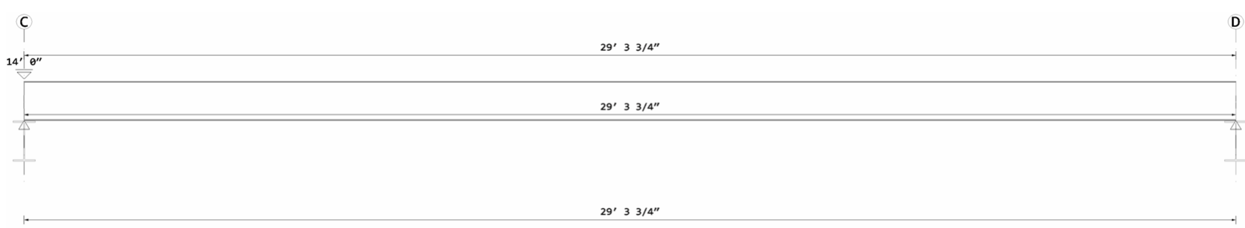
Span 1 W 12x26 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=14’ 4 7/64” ft, in
Max. Total load deflection	=0.079 in
Span over limit	=1.466 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.157**

✔ Pass

1B35



St. 1 (Lower T.O.S.): 1B35 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0”	●		●		●		●	
sub-beam	29’ 3 3/4”	●	1.000		1.000		1.000		1.000
support	29’ 3 3/4”	●		●		●		●	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	11	Slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	13	-3.0	64.3	kip	0.046	✔ Pass
Shear Minor	No Significant Forces			kip	-	Not required
Flexure Major	23	22.0	65.2	kip ft	0.337	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Flexure Minor	No Significant Forces			kip ft	-	Not required
Axial Tension	88	-0.7	187.2	kip	0.004	Pass
Axial Compression	124	0.7	4.3	kip	0.171	Pass
Combined Forces	36	-	-	-	0.338	Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.095	-	in	-	-
Deflection Dead	138	0.671	0.977	in	0.687	Pass
Deflection Live	7	0.265	0.977	in	0.271	Pass
Deflection Wind	132	-0.111	1.759	in	0.063	Pass
Deflection Total	36	1.019	1.466	in	0.695	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

11 LRFD$_{7.1}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0” - Critical

Axial Force, P	=0.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

13 LRFD$_{7.3}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 29’ 3 3/4” - Critical

Position of V_{ry}	=29’ 3 3/4” ft, in	
Required major axis shear strength, V_{ry}	=-3.0 kip	
Design shear strength	=64.3 kip	AISC 360 G2
Ratio	**=0.046**	

Pass

Flexure Major

3D Building Analysis - Critical

23 LRFD$_{7.13}$-1.2D+1.6Lr+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Yielding: 14’ 7 7/8” - Critical

Dist of M_{rx} along member	=14’ 7 7/8” ft, in	
Required flexural strength, M_{rx1}	=22.0 kip ft	
Design flexural strength	=65.2 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.337**	

Pass

Axial Tension
3D Building Analysis - Critical
88 $LRFD_{10.8}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: Position 0" - Critical

Distance of P_r along member	=0" ft, in	
Required tensile strength, P_r	=-0.7 kip	
Design yield strength	=187.2 kip	AISC 360 D2
Ratio	**=0.004**	

Pass

Axial Compression
3D Building Analysis - Critical
124 $LRFD_{12.2}$-0.9D+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flexural - out-of-plane: 0" - 29' 3 3/4" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0" - 29' 3 3/4"	Flexural - out-of-plane	29' 3 3/4"	0.7	4.3	kip	0.171	Pass

Combined Forces
3D Building Analysis - Critical
36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Flange Top LTB 0" - 29' 3 3/4" - Critical
Axial Tension: 0" - 29' 3 3/4" - Critical

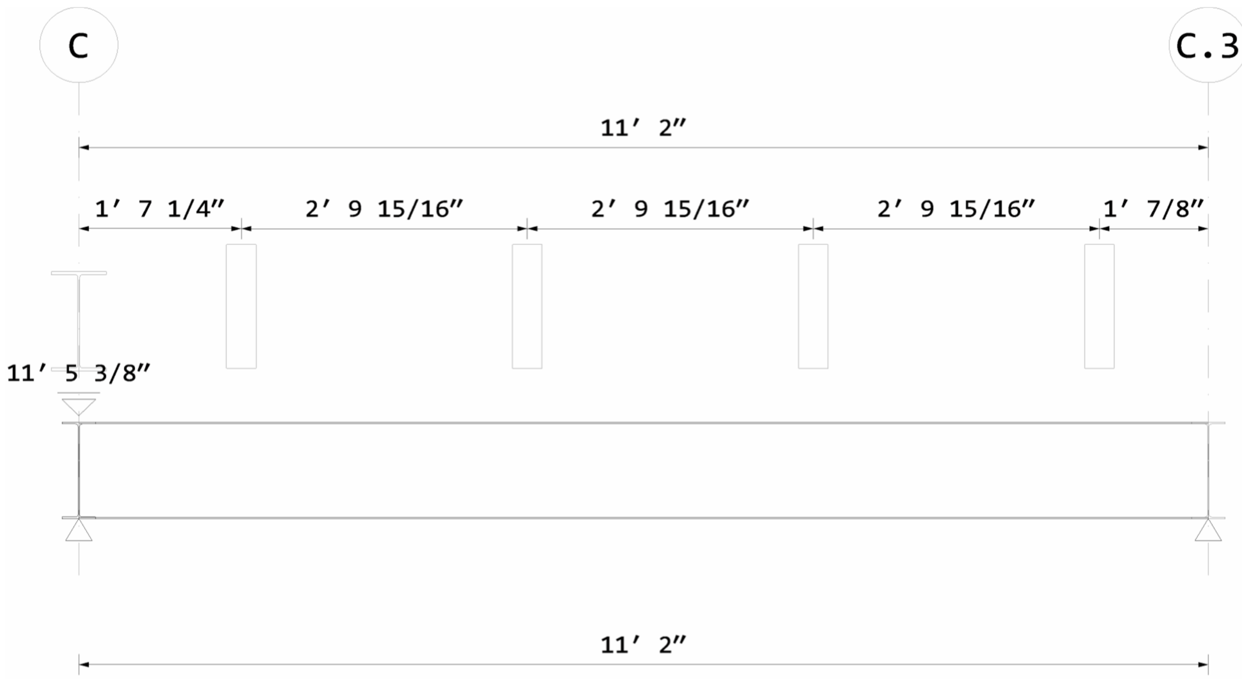
Mode	Ratio	Status
Combined buckling	0.338	Pass

Deflection
3D Building Analysis - Critical
22 $LRFD_{7.12}$-1.2D+1.6Lr+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Deflection Total - Critical

Position Total load deflection	=14' 7 7/8" ft, in
Max. Total load deflection	=1.019 in
Span over limit	=1.466 in
Design limit	=1.466 in
Utilization Ratio	**=0.695**

Pass

1B21



St. 1 (Lower T.O.S.): 1B21 - 1 (W 12x14 A992-50)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0"	•		•		•		•	
sub-beam	1' 7 69/256"	•	1.000		1.000		1.000		1.000
member	1' 7 69/256"	•						•	
sub-beam	2' 9 241/256"	•	1.000		1.000		1.000		1.000
member	4' 5 27/128"	•						•	
sub-beam	2' 9 241/256"	•	1.000		1.000		1.000		1.000
member	7' 3 39/256"	•						•	
sub-beam	2' 9 241/256"	•	1.000		1.000		1.000		1.000
member	10' 1 3/32"	•						•	
sub-beam	1' 29/32"	•	1.000		1.000		1.000		1.000
support	11' 1 255/256"	•		•		•		•	

Static Design Summary

Summary W 12x14

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Slender	-	-	-	Pass
Flexural Classification	1	Compact	-	-	-	Pass
Shear Major	33	4.3	64.3	kip	0.067	Pass
Shear Minor	68	-0.2	48.2	kip	0.004	Pass
Flexure Major	38	12.4	65.2	kip ft	0.190	Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Flexure Minor	116	0.2	7.1	kip ft	0.023	Pass
Axial Tension	133	-1.5	187.2	kip	0.008	Pass
Axial Compression	94	2.9	74.5	kip	0.039	Pass
Combined Forces	27	-	-	-	0.207	Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	138	0.007	-	in	-	-
Deflection Dead	138	0.046	0.372	in	0.124	Pass
Deflection Live	7	0.021	0.372	in	0.055	Pass
Deflection Wind	132	-0.009	0.670	in	0.013	Pass
Deflection Total	30	0.083	0.500	in	0.166	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Axial: Position 0" - Critical

Axial Force, P	=0.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=50.00 ksi	
Axial section class	Slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

33 $LRFD_{8.9}$-1.2D+1.6S+0.5W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 0" - Critical

Position of V_{ry}	=0" ft, in	
Required major axis shear strength, V_{ry}	=4.3 kip	
Design shear strength	=64.3 kip	AISC 360 G2
Ratio	**=0.067**	

Pass

Shear Minor

3D Building Analysis - Critical

68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical

Span 1 W 12x14 A992-50 - Critical

Position 7' 3 39/256" - Critical

Position of V_{rx}	=7' 3 39/256" ft, in	
Required minor axis shear strength, V_{rx}	=-0.2 kip	
Design shear strength	=48.2 kip	AISC 360 G6
Ratio	**=0.004**	

Pass

Flexure Major
3D Building Analysis - Critical
38 $LRFD_{8.14}$-1.2D+1.6S+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: 4′ 5 27/128″ - Critical

Dist of M_{rx} along member	=4′ 5 27/128″ ft, in	
Required flexural strength, M_{rx1}	=12.4 kip ft	
Design flexural strength	=65.2 kip ft	AISC 360 F1, F2 and F3
Ratio	**=0.190**	

Pass

Flexure Minor
3D Building Analysis - Critical
116 $LRFD_{10.36}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: 2′ 8 131/256″ - Critical

Dist of M_{ry} along member	=2′ 8 131/256″ ft, in	
Required flexural strength, M_{ry1}	=0.2 kip ft	
Design flexural strength	=7.1 kip ft	AISC 360 F6
Ratio	**=0.023**	

Pass

Axial Tension
3D Building Analysis - Critical
133 $LRFD_{12.11}$-0.9D+W - Critical
Span 1 W 12x14 A992-50 - Critical
Yielding: Position 0″ - Critical

Distance of P_r along member	=0″ ft, in	
Required tensile strength, P_r	=-1.5 kip	
Design yield strength	=187.2 kip	AISC 360 D2
Ratio	**=0.008**	

Pass

Axial Compression
3D Building Analysis - Critical
94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical
Span 1 W 12x14 A992-50 - Critical
Flexural torsional: 0″ - 11′ 1 255/256″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0″ - 11′ 1 255/256″	Flexural torsional	11′ 1 255/256″	2.9	74.5	kip	0.039	Pass

Combined Forces
3D Building Analysis - Critical
27 $LRFD_{8.3}$-1.2D+1.6S+0.5W - Critical
Span 1 W 12x14 A992-50 - Critical
Flange Btm LTB 0″ - 11′ 1 255/256″ - Critical
Axial Comp: I.P. 0″ - 11′ 1 255/256″ , O.o.P.0″ - 1′ 7 69/256″ , Tors. 0″ - 11′ 1 255/256″ - Critical

Mode	Ratio	Status
Combined buckling	0.207	✔ Pass

Deflection

3D Building Analysis - Critical

30 LRFD$_{8.6}$-1.2D+1.6S+0.5W - Critical

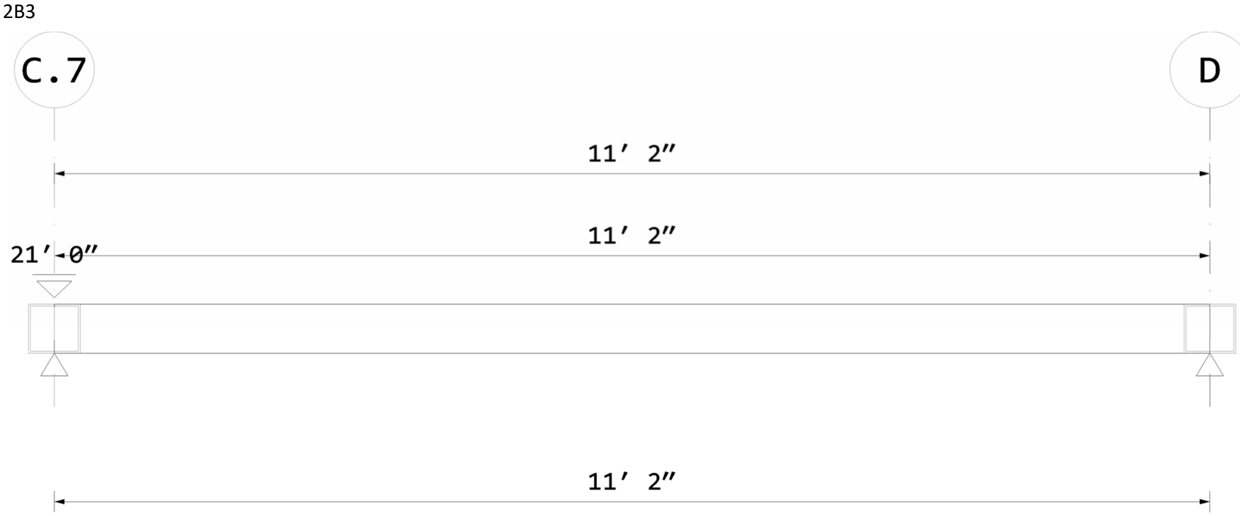
Span 1 W 12x14 A992-50 - Critical

Deflection Total - Critical

Position Total load deflection	=5′ 5 61/64″ ft, in
Max. Total load deflection	=0.083 in
Span over limit	=0.558 in
Abs. limit	=0.500 in
Design limit	=0.500 in
Utilization Ratio	**=0.166**

✔ Pass

2B3



St. 2 (Upper T.O.S.): 2B3 - 1 (HSS 6x6x1/4 A500B-46)

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub-Beam	LTB Top Factor	LTB Btm / Sub-Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	11′ 1 255/256″	•	1.000		1.000		1.000		1.000
support	11′ 1 255/256″	•		•		•		•	

Static Design Summary

Summary HSS 6x6x1/4

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	14	Non-slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Shear Major	27	-1.4	61.4	kip	0.023	Pass
Shear Minor	129	0.7	61.4	kip	0.012	Pass
Flexure Major	83	-5.5	38.6	kip ft	0.141	Pass
Flexure Minor	82	-3.6	38.6	kip ft	0.092	Pass
Axial Tension	92	-1.7	216.9	kip	0.008	Pass
Axial Compression	128	1.4	174.0	kip	0.008	Pass
Combined Forces	81	-	-	-	0.230	Pass
Torsion	-	-	-	-	-	Beyond Scope
Deflection Self weight	138	0.002	-	in	-	-
Deflection Dead	138	0.005	0.372	in	0.013	Pass
Deflection Live	7	0.004	0.372	in	0.011	Pass
Deflection Wind	123	-0.017	0.670	in	0.026	Pass
Deflection Total	61	0.020	0.558	in	0.035	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flexural: Position 0" - Critical

Moment, M_x	=-1.0 kip ft	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Flexural section class	Compact	AISC 360 Table B4.1b

Shear Major

3D Building Analysis - Critical

27 $LRFD_{8.3}$-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 11' 1 255/256" - Critical

Position of V_{ry}	=11' 1 255/256" ft, in	
Required major axis shear strength, V_{ry}	=-1.4 kip	
Design shear strength	=61.4 kip	AISC 360 G4
Ratio	**=0.023**	

Pass

Shear Minor

3D Building Analysis - Critical

129 $LRFD_{12.7}$-0.9D+W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Position 11' 1 255/256" - Critical

Position of V_{rx}	=11' 1 255/256" ft, in	
Required minor axis shear strength, V_{rx}	=0.7 kip	
Design shear strength	=61.4 kip	AISC 360 G4

Ratio =**0.012**

✔ Pass

Flexure Major
3D Building Analysis - Critical
83 $LRFD_{10.3}$-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 11′ 1 255/256″ - Critical
Dist of M_{rx} along member =11′ 1 255/256″ ft, in
Required flexural strength, M_{rx1} =-5.5 kip ft
Design flexural strength =38.6 kip ft AISC 360 F1, F2 and F3
Ratio **=0.141**

✔ Pass

Flexure Minor
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: 11′ 1 255/256″ - Critical
Dist of M_{ry} along member =11′ 1 255/256″ ft, in
Required flexural strength, M_{ry1} =-3.6 kip ft
Design flexural strength =38.6 kip ft AISC 360 F7
Ratio **=0.092**

✔ Pass

Axial Tension
3D Building Analysis - Critical
92 $LRFD_{10.12}$-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Yielding: Position 0″ - Critical
Distance of P_r along member =0″ ft, in
Required tensile strength, P_r =-1.7 kip
Design yield strength =216.9 kip AISC 360 D2
Ratio **=0.008**

✔ Pass

Axial Compression
3D Building Analysis - Critical
128 $LRFD_{12.6}$-0.9D+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical
Flexural - in-plane: 0″ - 11′ 1 255/256″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
0″ - 11′ 1 255/256″	Flexural - in-plane	11′ 1 255/256″	1.4	174.0	kip	0.008	✔ Pass

Combined Forces
3D Building Analysis - Critical
81 $LRFD_{10.1}$-1.2D+L+0.5S+W - Critical
Span 1 HSS 6x6x1/4 A500B-46 - Critical

Flange Top LTB 0" - 11' 1 255/256" - Critical

Axial Tension: 0" - 11' 1 255/256" - Critical

Mode	Ratio	Status
Combined buckling	0.230	Pass

Torsion

3D Building Analysis - Critical

8 LRFD6.1-1.2D+L+1.6S - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Beyond Scope - Critical

Member not pinned at both ends

Beyond Scope

Deflection

3D Building Analysis - Critical

61 LRFD8.37-1.2D+1.6S+0.5W - Critical

Span 1 HSS 6x6x1/4 A500B-46 - Critical

Deflection Total - Critical

Position Total load deflection	=7' 5 33/64" ft, in
Max. Total load deflection	=0.020 in
Span over limit	=0.558 in
Design limit	=0.558 in
Utilization Ratio	**=0.035**

Pass

STEEL COLUMN DESIGN

Column Group Design Summary

Static

Reference	No. in Group	Critical Member	Stacks	Critical Stack	Section	Grade	Max. UR	Status
SCR1	4	C237	2	1	HSS 6x6x3/8	A500B-46	0.457	Pass
SCR2	4	C235	2	1	HSS 6x6x3/8	A500B-46	0.251	Pass
SCR3	4	C243	1	1	HSS 3-1/2x3-1/2x1/4	A500B-46	0.374	Pass
SCR4	2	C236	2	1	HSS 6x6x3/8	A500B-46	0.458	Pass

Reference	Stacks	Critical Stack	Section	Grade	Max. UR	Status
C245	1	1	HSS 3-1/2x3-1/2x1/4	A500B-46	0.173	Pass
C240	1	1	HSS 3-1/2x3-1/2x1/4	A500B-46	0.173	Pass

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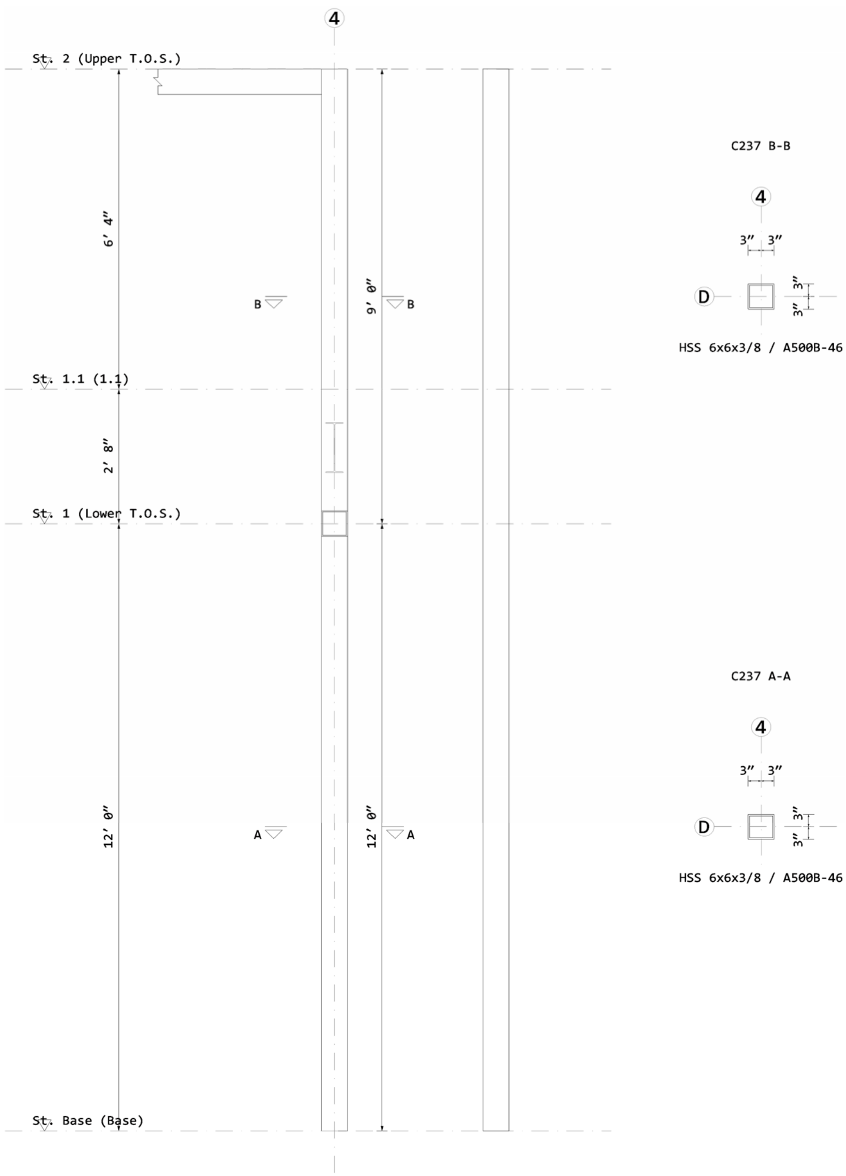
Revision:	Date:	Sheet No. 159
Project Name: Express Stop – View High		

C237



C237

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
3	floor	21' 0"	Yes		Yes	
	sub-beam	9' 0"	No	1.000	No	1.000
2	floor	12' 0"	Yes		Yes	
	sub-beam	12' 0"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
3	floor	21′ 0″	No		Yes	
	sub-beam	9′ 0″	No	1.000	No	1.000
2	floor	12′ 0″	Yes		Yes	
	sub-beam	12′ 0″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 6x6x3/8 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	123	-3.4	85.9	kip	0.040	✔ Pass
Shear Minor	127	1.1	85.9	kip	0.013	✔ Pass
Flexure Major	81	12.8	54.5	kip ft	0.235	✔ Pass
Flexure Minor	113	-7.4	54.5	kip ft	0.136	✔ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	30	66.1	239.9	kip	0.275	✔ Pass
Combined Forces	83	-	-	-	0.457	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Stack 2 (9′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Position 0″ - Critical

Moment, M_x	=1.3 kip ft	
Moment, M_y	=-0.4 kip ft	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Flexural section class	Compact	AISC 360 Table B4.1b

Shear Major

3D Building Analysis - Critical

123 $LRFD_{12.1}$-0.9D+W - Critical

Stack 2 (9′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Position 0″ - Critical

Position of V_{ry}	=0″ ft, in	
Required major axis shear strength, V_{ry}	=-3.4 kip	
Design shear strength	=85.9 kip	AISC 360 G4
Ratio	**=0.040**	

✔ Pass

Shear Minor

3D Building Analysis - Critical

127 $LRFD_{12.5}$-0.9D+W - Critical

Stack 1 (12′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Position 0″ - Critical

Position of V_{rx} =0″ ft, in

Required minor axis shear strength, V_{rx} =1.1 kip

Design shear strength =85.9 kip AISC 360 G4

Ratio **=0.013**

✔ Pass

Flexure Major

3D Building Analysis - Critical

81 $LRFD_{10.1}$-1.2D+L+0.5S+W - Critical

Stack 2 (9′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Yielding: 12′ 0″ - Critical

Dist of M_{rx} along member =12′ 0″ ft, in

Required flexural strength, M_{rx1} =12.8 kip ft

Design flexural strength =54.5 kip ft AISC 360 F1, F2 and F3

Ratio **=0.235**

✔ Pass

Flexure Minor

3D Building Analysis - Critical

113 $LRFD_{10.33}$-1.2D+L+0.5S+W - Critical

Stack 1 (12′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Yielding 0″ - Critical

Dist of M_{ry} along member =0″ ft, in

Required flexural strength, M_{ry1} =-7.4 kip ft

Design flexural strength =54.5 kip ft AISC 360 F7

Ratio **=0.136**

✔ Pass

Axial Compression

3D Building Analysis - Critical

30 $LRFD_{8.6}$-1.2D+1.6S+0.5W - Critical

Stack 1 (12′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Flexural - in-plane: 12′ 0″ - 0″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
12′ 0″ - 0″	Flexural - in-plane	12′ 0″	66.1	239.9	kip	0.275	✔ Pass

Combined Forces

3D Building Analysis - Critical

83 $LRFD_{10.3}$-1.2D+L+0.5S+W - Critical

Stack 1 (12′ 0″) HSS 6x6x3/8 A500B-46 - Critical

LTB Flange A 12′ 0″ - 0″ - Critical

Axial Comp: I.P. 12′ 0″ - 0″, O.P. 12′ 0″ - 0″, Tors. 12′ 0″ - 0″ - Critical

Mode	Ratio	Status
Combined buckling	0.457	✔ Pass

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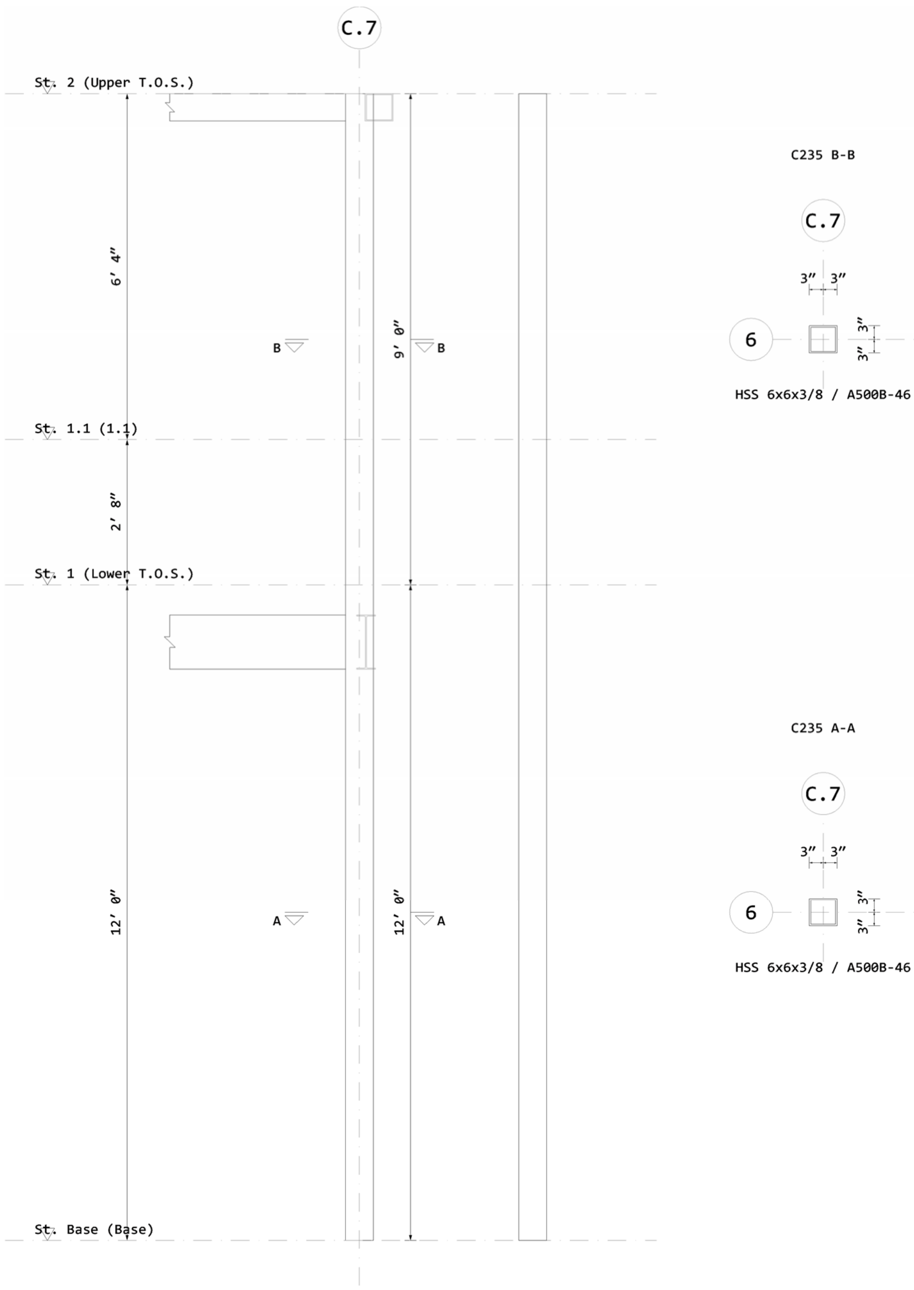
Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 162
Project Name: Express Stop – View High		

C235



C235

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
3	floor	21' 0"	Yes		Yes	
	sub-beam	9' 0"	No	1.000	No	1.000
2	floor	12' 0"	Yes		Yes	
	sub-beam	12' 0"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

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Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
3	floor	21′ 0″	No		Yes	
	sub-beam	9′ 0″	No	1.000	No	1.000
2	floor	12′ 0″	No		Yes	
	sub-beam	12′ 0″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 6x6x3/8 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	✔ Pass
Flexural Classification	1	Compact	-	-	-	✔ Pass
Shear Major	94	1.4	85.9	kip	0.016	✔ Pass
Shear Minor	82	1.6	85.9	kip	0.018	✔ Pass
Flexure Major	80	-5.0	54.5	kip ft	0.092	✔ Pass
Flexure Minor	82	-10.6	54.5	kip ft	0.195	✔ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	26	8.9	137.9	kip	0.064	✔ Pass
Combined Forces	82	-	-	-	0.251	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 LRFD$_1$-1.4D - Critical

Stack 2 (9′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Position 0″ - Critical

Moment, M_x	=-0.4 kip ft	
Moment, M_y	=-0.4 kip ft	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Flexural section class	Compact	AISC 360 Table B4.1b

Shear Major

3D Building Analysis - Critical

94 LRFD$_{10.14}$-1.2D+L+0.5S+W - Critical

Stack 1 (12′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Position 0″ - Critical

Position of V_{ry}	=0″ ft, in	
Required major axis shear strength, V_{ry}	=1.4 kip	
Design shear strength	=85.9 kip	AISC 360 G4
Ratio	**=0.016**	

✔ Pass

Shear Minor

3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Stack 1 (12' 0") HSS 6x6x3/8 A500B-46 - Critical
Position 0" - Critical
Position of V_{rx} =0" ft, in
Required minor axis shear strength, V_{rx} =1.6 kip
Design shear strength =85.9 kip AISC 360 G4
Ratio **=0.018**
Pass

Flexure Major
3D Building Analysis - Critical
80 $LRFD_{9.14}$-1.2D+L+0.5Lr+W - Critical
Stack 1 (12' 0") HSS 6x6x3/8 A500B-46 - Critical
Yielding: 0" - Critical
Dist of M_{rx} along member =0" ft, in
Required flexural strength, M_{rx1} =-5.0 kip ft
Design flexural strength =54.5 kip ft AISC 360 F1, F2 and F3
Ratio **=0.092**
Pass

Flexure Minor
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Stack 1 (12' 0") HSS 6x6x3/8 A500B-46 - Critical
Yielding 0" - Critical
Dist of M_{ry} along member =0" ft, in
Required flexural strength, M_{ry1} =-10.6 kip ft
Design flexural strength =54.5 kip ft AISC 360 F7
Ratio **=0.195**
Pass

Axial Compression
3D Building Analysis - Critical
26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical
Stack 2 (9' 0") HSS 6x6x3/8 A500B-46 - Critical
Flexural - in-plane: 21' 0" - 0" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
21' 0" - 0"	Flexural - in-plane	21' 0"	8.9	137.9	kip	0.064	Pass

Combined Forces
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Stack 1 (12' 0") HSS 6x6x3/8 A500B-46 - Critical
LTB Flange A 12' 0" - 0" - Critical
Axial Comp: I.P. 21' 0" - 0", O.P. 12' 0" - 0", Tors. 21' 0" - 0" - Critical

Mode	Ratio	Status
Combined buckling	0.251	Pass

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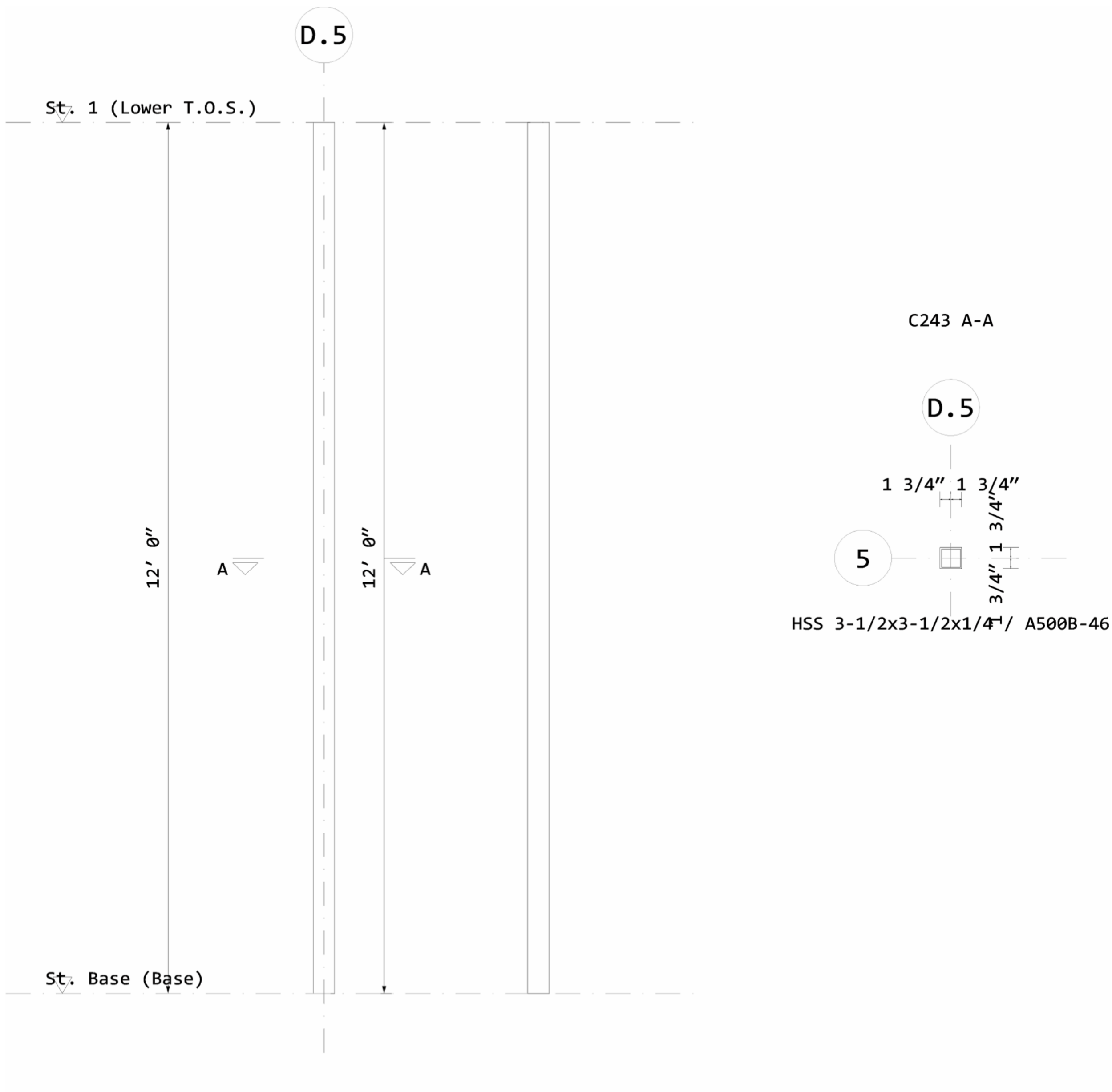
Revision:	Date:	Sheet No. 165
Project Name: Express Stop – View High		

C243



C243

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
2	floor	12' 0"	Yes		Yes	
	sub-beam	12' 0"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
2	floor	12′ 0″	No		Yes	
	sub-beam	12′ 0″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 3-1/2x3-1/2x1/4 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	✓ Pass
Flexural Classification	4	Compact	-	-	-	✓ Pass
Shear Major	94	1.6	32.4	kip	0.050	✓ Pass
Shear Minor	82	0.3	32.4	kip	0.008	✓ Pass
Flexure Major	94	-4.3	12.1	kip ft	0.355	✓ Pass
Flexure Minor	82	1.7	12.1	kip ft	0.139	✓ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	26	3.3	54.1	kip	0.061	✓ Pass
Combined Forces	94	-	-	-	0.374	✓ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Position 0″ - Critical

Axial Force, P	=1.3 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Axial section class	Non-slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical

Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Position 0″ - Critical

Position of V_{ry}	=0″ ft, in	
Required major axis shear strength, V_{ry}	=1.6 kip	
Design shear strength	=32.4 kip	AISC 360 G4
Ratio	**=0.050**	

✓ Pass

Shear Minor

3D Building Analysis - Critical

82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical

Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Position 0″ - Critical

Position of V_{rx} =0" ft, in
Required minor axis shear strength, V_{rx} =0.3 kip
Design shear strength =32.4 kip AISC 360 G4
Ratio **=0.008**
Pass

Flexure Major
3D Building Analysis - Critical
94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical
Stack 1 (12' 0") HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
Yielding: 0" - Critical
Dist of M_{rx} along member =0" ft, in
Required flexural strength, M_{rx1} =-4.3 kip ft
Design flexural strength =12.1 kip ft AISC 360 F1, F2 and F3
Ratio **=0.355**
Pass

Flexure Minor
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Stack 1 (12' 0") HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
Yielding 12' 0" - Critical
Dist of M_{ry} along member =12' 0" ft, in
Required flexural strength, M_{ry1} =1.7 kip ft
Design flexural strength =12.1 kip ft AISC 360 F7
Ratio **=0.139**
Pass

Axial Compression
3D Building Analysis - Critical
26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical
Stack 1 (12' 0") HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
Flexural - in-plane: 12' 0" - 0" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
12' 0" - 0"	Flexural - in-plane	12' 0"	3.3	54.1	kip	0.061	Pass

Combined Forces
3D Building Analysis - Critical
94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical
Stack 1 (12' 0") HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
LTB Flange A 12' 0" - 0" - Critical
Axial Comp: I.P. 12' 0" - 0", O.P. 12' 0" - 0", Tors. 12' 0" - 0" - Critical

Mode	Ratio	Status
Combined buckling	0.374	Pass

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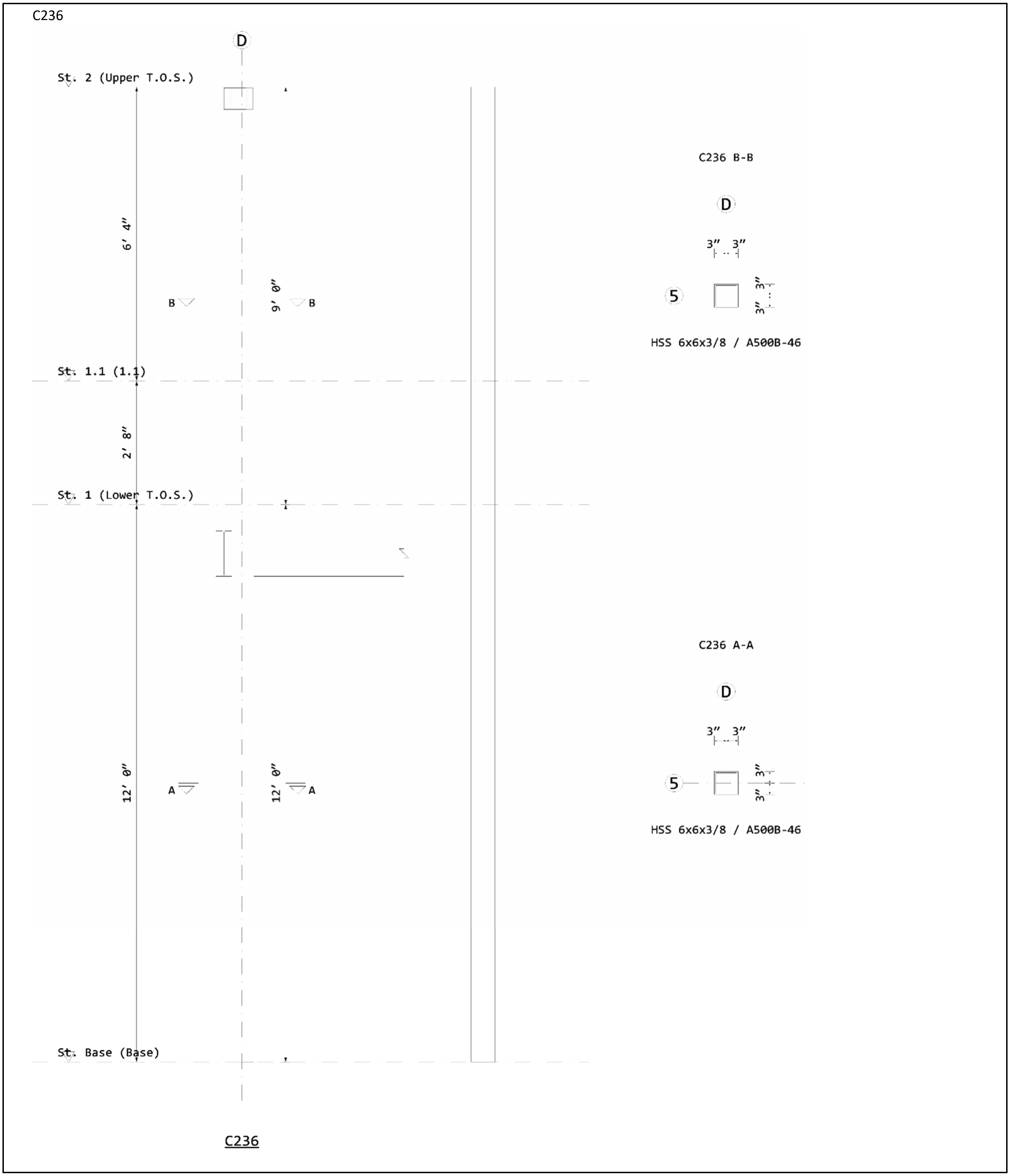

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
3	floor	21′ 0″	Yes		Yes	
	sub-beam	9′ 0″	No	1.000	No	1.000
2	floor	12′ 0″	Yes		Yes	
	sub-beam	12′ 0″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
3	floor	21′ 0″	Yes		Yes	
	sub-beam	9′ 0″	No	1.000	No	1.000
2	floor	12′ 0″	Yes		Yes	
	sub-beam	12′ 0″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 6x6x3/8 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	✓ Pass
Flexural Classification	1	Compact	-	-	-	✓ Pass
Shear Major	35	-2.4	85.9	kip	0.027	✓ Pass
Shear Minor	124	2.0	85.9	kip	0.023	✓ Pass
Flexure Major	36	16.3	54.5	kip ft	0.299	✓ Pass
Flexure Minor	68	-11.3	54.5	kip ft	0.207	✓ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	36	32.9	239.9	kip	0.137	✓ Pass
Combined Forces	78	-	-	-	0.458	✓ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Stack 2 (9′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Position 0″ - Critical

Moment, M_x	=11.4 kip ft	
Moment, M_y	=-1.0 kip ft	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Flexural section class	Compact	AISC 360 Table B4.1b

Shear Major

3D Building Analysis - Critical

35 $LRFD_{8.11}$-1.2D+1.6S+0.5W - Critical

Stack 2 (9′ 0″) HSS 6x6x3/8 A500B-46 - Critical

Position 0" - Critical

Position of V_{ry} =0" ft, in

Required major axis shear strength, V_{ry} =-2.4 kip

Design shear strength =85.9 kip AISC 360 G4

Ratio **=0.027**

✔ Pass

Shear Minor

3D Building Analysis - Critical

124 $LRFD_{12.2}$-0.9D+W - Critical

Stack 2 (9' 0") HSS 6x6x3/8 A500B-46 - Critical

Position 0" - Critical

Position of V_{rx} =0" ft, in

Required minor axis shear strength, V_{rx} =2.0 kip

Design shear strength =85.9 kip AISC 360 G4

Ratio **=0.023**

✔ Pass

Flexure Major

3D Building Analysis - Critical

36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical

Stack 2 (9' 0") HSS 6x6x3/8 A500B-46 - Critical

Yielding: 12' 0" - Critical

Dist of M_{rx} along member =12' 0" ft, in

Required flexural strength, M_{rx1} =16.3 kip ft

Design flexural strength =54.5 kip ft AISC 360 F1, F2 and F3

Ratio **=0.299**

✔ Pass

Flexure Minor

3D Building Analysis - Critical

68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical

Stack 1 (12' 0") HSS 6x6x3/8 A500B-46 - Critical

Yielding 0" - Critical

Dist of M_{ry} along member =0" ft, in

Required flexural strength, M_{ry1} =-11.3 kip ft

Design flexural strength =54.5 kip ft AISC 360 F7

Ratio **=0.207**

✔ Pass

Axial Compression

3D Building Analysis - Critical

36 $LRFD_{8.12}$-1.2D+1.6S+0.5W - Critical

Stack 1 (12' 0") HSS 6x6x3/8 A500B-46 - Critical

Flexural - in-plane: 12' 0" - 0" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
12' 0" - 0"	Flexural - in-plane	12' 0"	32.9	239.9	kip	0.137	✔ Pass

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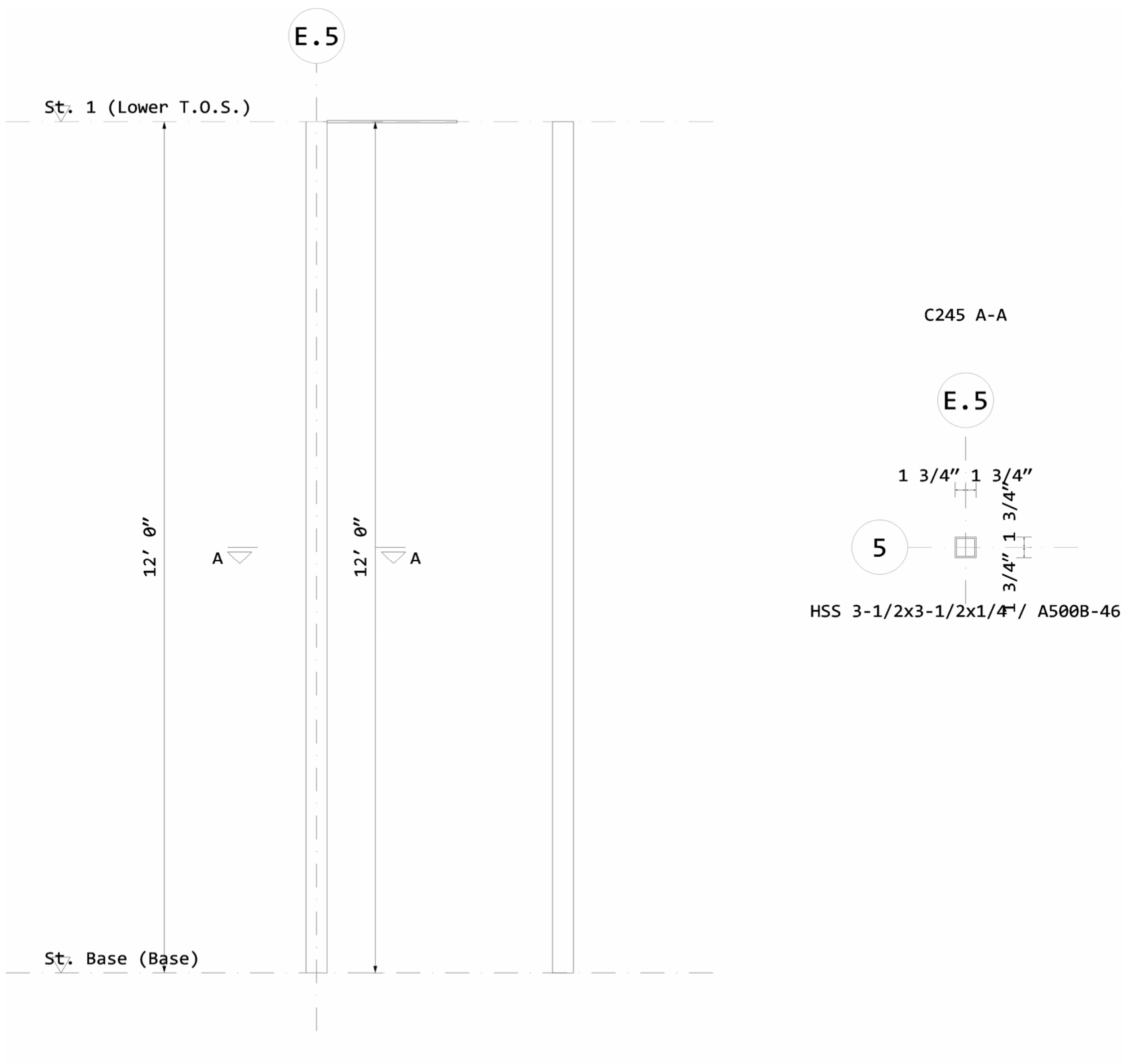
Combined Forces
3D Building Analysis - Critical
78 LRFD9.12-1.2D+L+0.5Lr+W - Critical
Stack 1 (12′ 0″) HSS 6x6x3/8 A500B-46 - Critical
LTB Flange A 12′ 0″ - 0″ - Critical
Axial Comp: I.P. 12′ 0″ - 0″, O.P. 12′ 0″ - 0″, Tors. 12′ 0″ - 0″ - Critical

Mode	Ratio	Status
Combined buckling	0.458	Pass

C245



C245

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
2	floor	12′ 0″	Yes		Yes	
	sub-beam	12′ 0″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
2	floor	12′ 0″	No		Yes	
	sub-beam	12′ 0″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 3-1/2x3-1/2x1/4 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	Pass
Flexural Classification	4	Compact	-	-	-	Pass
Shear Major	94	0.7	32.4	kip	0.023	Pass
Shear Minor	82	0.3	32.4	kip	0.008	Pass
Flexure Major	94	-1.9	12.1	kip ft	0.160	Pass
Flexure Minor	82	1.6	12.1	kip ft	0.134	Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	26	1.4	54.1	kip	0.027	Pass
Combined Forces	94	-	-	-	0.173	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Position 0″ - Critical

Axial Force, P	=0.8 kip	
Elastic modulus of steel, E	=29000 ksi	
Minimum yield stress, F_y	=46.00 ksi	
Axial section class	Non-slender	AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical

Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Position 0″ - Critical

Position of V_{ry}	=0″ ft, in	
Required major axis shear strength, V_{ry}	=0.7 kip	
Design shear strength	=32.4 kip	AISC 360 G4
Ratio	**=0.023**	

Pass

Shear Minor
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
Position 0″ - Critical
Position of V_{rx} =0″ ft, in
Required minor axis shear strength, V_{rx} =0.3 kip
Design shear strength =32.4 kip AISC 360 G4
Ratio **=0.008**
Pass

Flexure Major
3D Building Analysis - Critical
94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical
Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
Yielding: 0″ - Critical
Dist of M_{rx} along member =0″ ft, in
Required flexural strength, M_{rx1} =-1.9 kip ft
Design flexural strength =12.1 kip ft AISC 360 F1, F2 and F3
Ratio **=0.160**
Pass

Flexure Minor
3D Building Analysis - Critical
82 $LRFD_{10.2}$-1.2D+L+0.5S+W - Critical
Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
Yielding 12′ 0″ - Critical
Dist of M_{ry} along member =12′ 0″ ft, in
Required flexural strength, M_{ry1} =1.6 kip ft
Design flexural strength =12.1 kip ft AISC 360 F7
Ratio **=0.134**
Pass

Axial Compression
3D Building Analysis - Critical
26 $LRFD_{8.2}$-1.2D+1.6S+0.5W - Critical
Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
Flexural - in-plane: 12′ 0″ - 0″ - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
12′ 0″ - 0″	Flexural - in-plane	12′ 0″	1.4	54.1	kip	0.027	Pass

Combined Forces
3D Building Analysis - Critical
94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical
Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical
LTB Flange A 12′ 0″ - 0″ - Critical
Axial Comp: I.P. 12′ 0″ - 0″, O.P. 12′ 0″ - 0″, Tors. 12′ 0″ - 0″ - Critical

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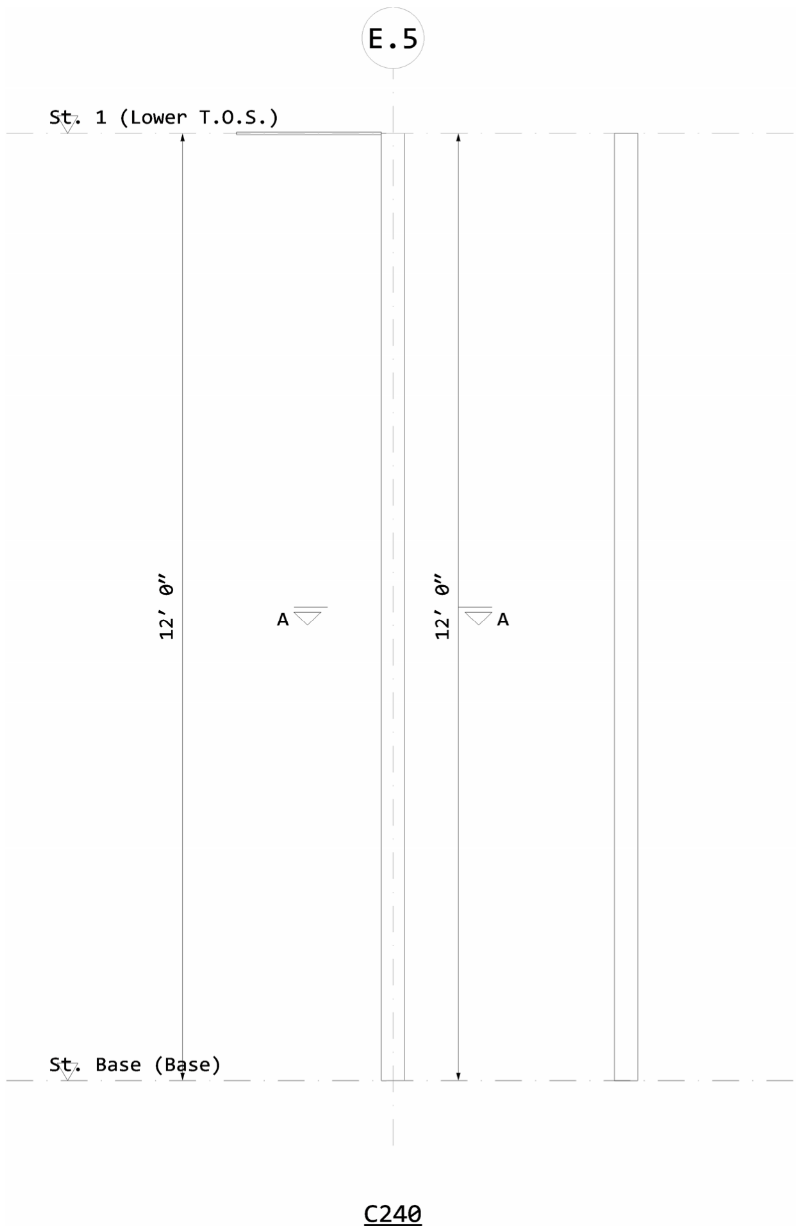
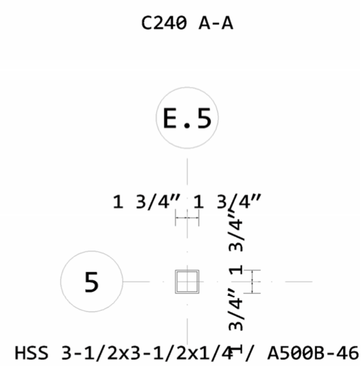
Mode	Ratio	Status
Combined buckling	0.173	Pass

C240





Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack continuous	Face A factor	Face C restrained/ Sub-stack continuous	Face C factor
2	floor	12' 0"	Yes		Yes	
	sub-beam	12' 0"	No	1.000	No	1.000
1	floor	0"	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack continuous	Major factor	Minor restrained / Sub-stack continuous	Minor factor
2	floor	12′ 0″	No		Yes	
	sub-beam	12′ 0″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 3-1/2x3-1/2x1/4 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	1	Non-slender	-	-	-	Pass
Flexural Classification	4	Compact	-	-	-	Pass
Shear Major	94	0.7	32.4	kip	0.023	Pass
Shear Minor	68	0.3	32.4	kip	0.008	Pass
Flexure Major	94	-1.9	12.1	kip ft	0.160	Pass
Flexure Minor	68	-1.6	12.1	kip ft	0.129	Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	32	1.4	54.1	kip	0.026	Pass
Combined Forces	94	-	-	-	0.173	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

1 $LRFD_1$-1.4D - Critical

Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Position 0″ - Critical

Axial Force, P	=0.8 kip
Elastic modulus of steel, E	=29000 ksi
Minimum yield stress, F_y	=46.00 ksi
Axial section class	Non-slender AISC 360 Table B4.1a

Shear Major

3D Building Analysis - Critical

94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical

Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Position 0″ - Critical

Position of V_{ry}	=0″ ft, in
Required major axis shear strength, V_{ry}	=0.7 kip
Design shear strength	=32.4 kip AISC 360 G4
Ratio	**=0.023**

Pass

Shear Minor

3D Building Analysis - Critical

68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical

Stack 1 (12′ 0″) HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Position 0" - Critical

Position of V_{rx} =0" ft, in

Required minor axis shear strength, V_{rx} =0.3 kip

Design shear strength =32.4 kip AISC 360 G4

Ratio **=0.008**

✔ Pass

Flexure Major

3D Building Analysis - Critical

94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical

Stack 1 (12' 0") HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Yielding: 0" - Critical

Dist of M_{rx} along member =0" ft, in

Required flexural strength, M_{rx1} =-1.9 kip ft

Design flexural strength =12.1 kip ft AISC 360 F1, F2 and F3

Ratio **=0.160**

✔ Pass

Flexure Minor

3D Building Analysis - Critical

68 $LRFD_{9.2}$-1.2D+L+0.5Lr+W - Critical

Stack 1 (12' 0") HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Yielding 0" - Critical

Dist of M_{ry} along member =0" ft, in

Required flexural strength, M_{ry1} =-1.6 kip ft

Design flexural strength =12.1 kip ft AISC 360 F7

Ratio **=0.129**

✔ Pass

Axial Compression

3D Building Analysis - Critical

32 $LRFD_{8.8}$-1.2D+1.6S+0.5W - Critical

Stack 1 (12' 0") HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

Flexural - in-plane: 12' 0" - 0" - Critical

Start - End	Mode	Length	Value	Limit	Units	Ratio	Status
12' 0" - 0"	Flexural - in-plane	12' 0"	1.4	54.1	kip	0.026	✔ Pass

Combined Forces

3D Building Analysis - Critical

94 $LRFD_{10.14}$-1.2D+L+0.5S+W - Critical

Stack 1 (12' 0") HSS 3-1/2x3-1/2x1/4 A500B-46 - Critical

LTB Flange A 12' 0" - 0" - Critical

Axial Comp: I.P. 12' 0" - 0", O.P. 12' 0" - 0", Tors. 12' 0" - 0" - Critical

Mode	Ratio	Status
Combined buckling	0.173	✔ Pass

BUILDING CALCULATIONS

SHEAR WALLS

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.15

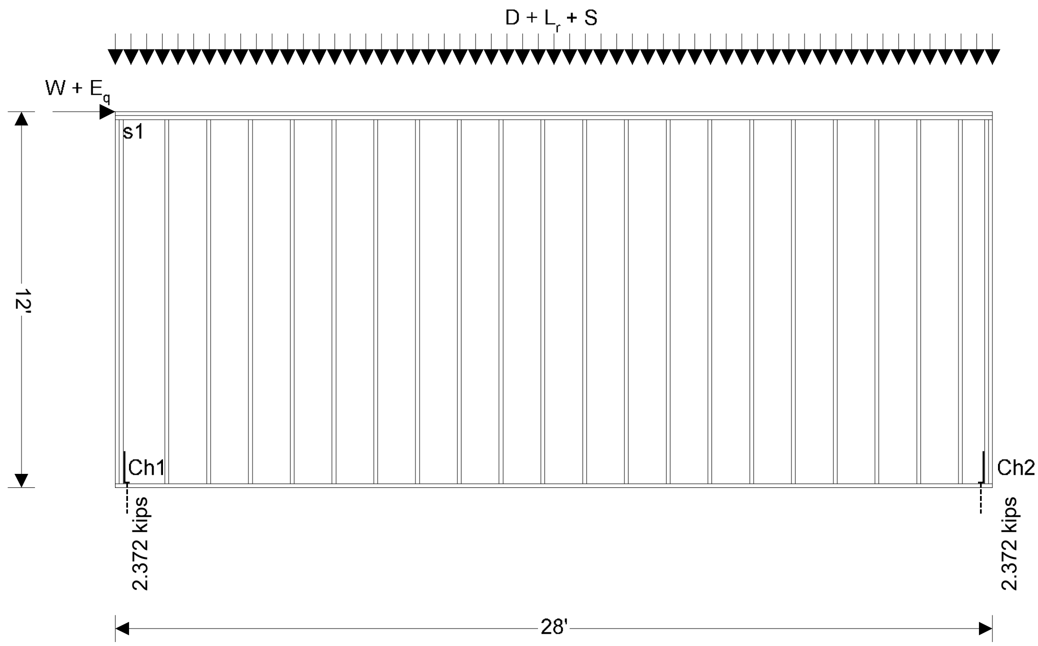
Panel details

Structural I wood panel sheathing on one side

Panel height; h = **12** ft

Panel length; b = **28** ft

Total area of wall; A = h × b = **336** ft^2



Panel construction

Nominal stud size; 2" x 8"

Dressed stud size; 1.5" x 7.25"

Cross-sectional area of studs; A_s = **10.875** in^2

Stud spacing; s = **16** in

Nominal end post size; 2 x 2" x 8"

Dressed end post size; 2 x 1.5" x 7.25"

Cross-sectional area of end posts; A_e = **21.75** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **18.75** in^2

Nominal collector size; 2 x 2" x 8"

Dressed collector size; 2 x 1.5" x 7.25"

Service condition;	Dry
Temperature;	100 degF or less
Anchor location;	Inside face
Anchor offset;	e_{anchor} = **0** in
Vertical anchor stiffness;	k_a = **25000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification;	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity;	G = **0.42**
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material;	15/32" wood panel structural I oriented strandboard sheathing
Fastener type;	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity;	v_n = 785 plf × min[1 - (0.5 - G), 1] = **722.2** lb/ft
Apparent shear wall shear stiffness;	G_a = **14** kips/in

Loading details

Dead load acting on top of panel;	D = **900** lb/ft
Roof live load acting on top of panel;	L_r = **750** lb/ft
Snow load acting on top of panel;	S = **750** lb/ft
Self weight of panel;	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel;	W = **9100** lbs
Wind load serviceability factor;	f_{Wserv} = **0.60**
In plane seismic load acting at head of panel;	E_q = **2500** lbs
Design spectral response accel. par., short periods;	S_{DS} = **0.105**

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1;	D + 0.6W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4;	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5;	0.6D + 0.6W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.20**
Size factor for compression – Table 4A;	C_{Fc} = **1.05**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3; C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8; C_i = **1.00**

Buckling stiffness factor – cl.4.4.2; C_T = **1.00**

Bearing area factor - cl. 3.10.4; C_b = **1.0**

Adjusted modulus of elasticity; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **510000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1063** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **1932** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.47**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **28** ft

Shear wall aspect ratio; h / b = **0.429**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **5.46** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2.0$ = **10.111** kips

V_{w_max} / V_w = **0.54**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **1.75** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2.8$ = **7.222** kips

V_{s_max} / V_s = **0.242**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; h / b = **0.429**

Effective length for chord forces; $b_{eff} = b - 3 / 2 \times b_{EndPost} - e_{anchor}$ = **27.62** ft

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.46** kips

Axial force for maximum tension; P = 0 kips = **0** kips

Maximum tensile force in chord; $T = V \times h / b_{eff} - P$ = **2.372** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **126** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **864** lb/in^2

f_t / F_t' = **0.146**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **5.46** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.68** kips

Maximum compressive force in chord; $C = V \times h / b_{eff} + P$ = **3.052** kips

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Maximum applied compressive stress; $f_c = C / A_e =$ **140** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **904** lb/in^2

$f_c / F_c' =$ **0.155**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **425** lb/in^2

$f_c / F_{c\_perp}' =$ **0.330**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1; $T_1 =$ **2.372** kips

Chord 2; $T_2 =$ **2.372** kips

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv}$ ´ $W =$ **5.46** kips

Deflection limit; $\Delta_{w\_allow} = h / 600 =$ **0.24** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **195** lb/ft

Anchor tension force; $T_\delta = \max(0$ kips$, v_{\delta w}$ ´ h ´ $b / b_{eff}) =$ **2.372** kips

Chord compression force; $C_\delta = \max(0$ kips$, v_{\delta w}$ ´ h ´ $b / b_{eff}) =$ **2.372** kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a =$ **0.095** in

Vertical compression at chord; $\Delta_C = 0.04$ in ´ $C_\delta / (A_e$ ´ $F_{c\_perp}) =$ **0.010** in

Total vertical deflection; $\Delta_a = (\Delta_T + \Delta_C)$ ´ $(b / b_{eff}) =$ **0.107** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2$ ´ $v_{\delta w}$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b) + v_{\delta w}$ ´ $h / (G_a) + h$ ´ $\Delta_a / b =$ **0.216** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.9**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **2.5** kips

Deflection limit; $\Delta_{s\_allow} = 0.010$ ´ $h =$ **1.44** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **89.29** lb/ft

Anchor tension force; $T_\delta = \max(0$ kips$, v_{\delta s}$ ´ h ´ $b / b_{eff}) =$ **1.086** kips

Chord compression force; $C_\delta = \max(0$ kips$, v_{\delta s}$ ´ h ´ $b / b_{eff}) =$ **1.086** kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a =$ **0.043** in

Vertical compression at chord; $\Delta_C = 0.04$ in ´ $C_\delta / (A_e$ ´ $F_{c\_perp}) =$ **0.005** in

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Total vertical deflection; $\Delta_a = (\Delta_T + \Delta_C) \times (b / b_{eff})$ = **0.049** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b$ = **0.099** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Amp. seis. deflection – ASCE 7-16, Eqn.12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e$ = **0.396** in

$\delta_{sws} / \Delta_{s\_allow}$ = **0.275**

PASS - Shear wall deflection is less than deflection limit

SHEAR WALL HOLDOWN ANCHOR

Anchor Designer™ for Concrete Software
Version 3.3.2410.2

Company:		Date:	5/29/2025
Engineer:		Page:	1
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Project description:
Location:
Design name: Design

Comment:

2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 3.500
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 4.88
c_{ac} (inch): 5.43
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 10.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ 5/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057

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Revision:	Date:	Sheet No. 183
Project Name: Express Stop – View High		

SIMPSON Strong-Tie
Anchor Designer™ for Concrete Software
Version 3.3.2410.2

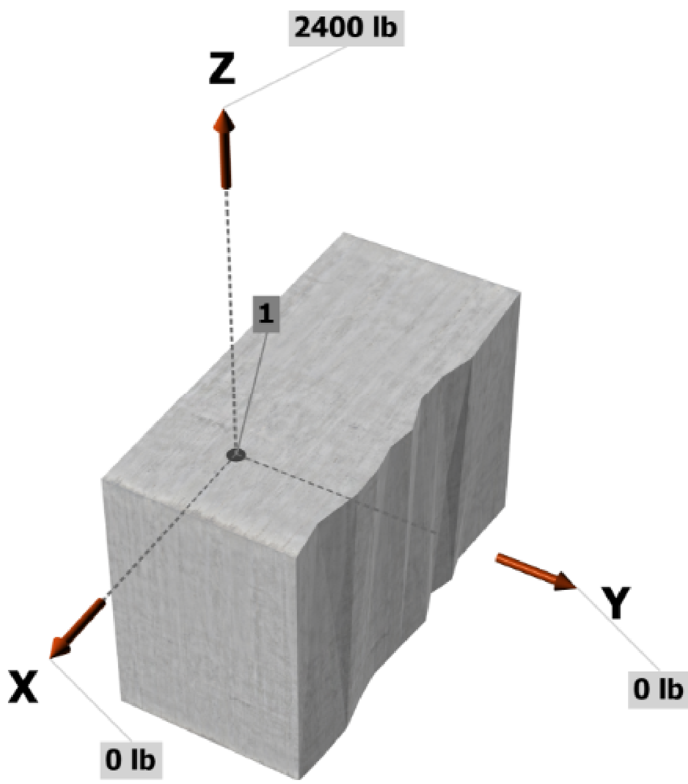
Company:		Date:	5/29/2025
Engineer:		Page:	2
Project:			
Address:			
Phone:			
E-mail:			

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

Strength level loads:

N_{ua} [lb]:	2400
V_{uax} [lb]:	0
V_{uay} [lb]:	0

<Figure 1>



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Anchor Designer™ for Concrete Software
Version 3.3.2410.2

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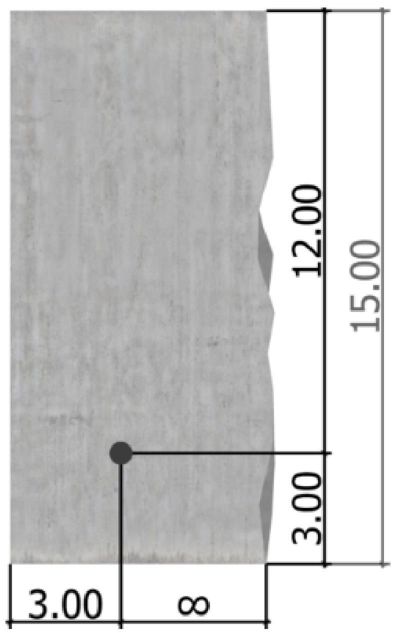
<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	2400.0	0.0	0.0	0.0
Sum	2400.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 2400
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

Anchor Designer™ for Concrete Software
Version 3.3.2410.2

Company:		Date:	5/29/2025
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E-mail:			

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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

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

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	3.500	7040

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

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
68.06	110.25	3.00	0.871	1.00	1.000	7040	0.65	2462

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

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

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	f'_c (psi)	n	$\tau_{k,cr}$ (psi)
1356	1.00	1.00	4000	0.24	1518

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

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1518	0.63	3.500	10431

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

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
138.35	307.10	8.76	3.00	0.803	1.000	10431	0.65	2452

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	2400	9833	0.24	Pass
Concrete breakout	2400	2462	0.97	Pass
Adhesive	**2400**	**2452**	**0.98**	**Pass (Governs)**

SET-3G w/ 5/8"Ø F1554 Gr. 36 with hef = 3.500 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

STUD WALL

WOOD MEMBER DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Design summary

Overall design utilisation; 0.734

Overall design status; PASS

24 inch stud spacing results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	1389	639	0.460	PASS
Shear stress	lb/in^2	155	41	0.267	PASS
Compressive stress	lb/in^2	823	276	0.335	PASS
Bending and axial force				0.734	PASS

24 inch stud spacing

Member details

Service condition; Dry

Sawn lumber section details

Number of sections in member; N = **1**

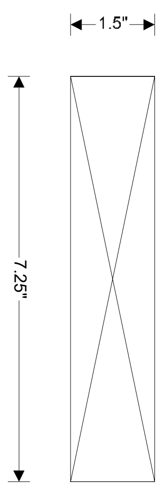
Nominal breadth of sections; b_{nom} = **2** in

Breadth of sections; b = **1.5** in

Nominal depth of sections; d_{nom} = **8** in

Depth of sections; d = **7.25** in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



2"x8" sawn lumber section
Cross-sectional area, A, 10.875 in^2
Section modulus, S_x, 13.1 in^3
Section modulus, S_y, 2.7 in^3
Second moment of area, I_x, 47.6 in^4
Second moment of area, I_y, 2 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 0.433 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Unbraced length - Major axis; L_x = **12** ft

Effective bending length - Major axis; $L_{e,x} = 1.63 \times L_x + 3 \times b$ = **19.935** ft

Column buckling length - Major axis; $L_{b,x} = L_x$ = **12** ft

Unbraced length - Minor axis; L_y = **0** ft
Bearing length; L_b = **4** in

Analysis results

Design bending moment - Major axis; M_x = **700** lb_ft
Design shear force - Major axis; V_x = **300** lb
Design axial compression force; P = **3000** lb

Adjustment factors - Table 4.3.1

Two months load duration factor - Table 2.3.2; C_D = **1.15**
Size factor for bending - Table 4A; C_{Fb} = **1.2**
Size factor for compression - Table 4A; C_{Fc} = **1.05**
Repetitive member factor - Table 4.3.9; C_r = **1.15**
Reference compression design value; $F_c^* = F_c \times C_D \times C_{Fc}$ = **1389** lb/in^2
Adjusted modulus of elasticity; $E_{min}' = E_{min}$ = **510000** lb/in^2
Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (L_{b,x} / d)^2$ = **1063** lb/in^2
Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8}$ = **0.593**

Compression members - General - cl.3.6

Design axial compression force; P = **3000** lb
Design compression parallel to grain - Table 4.3.1; $F_c' = F_c \times C_D \times C_{Fc} \times C_P$ = **823** lb/in^2
Actual compression parallel to grain; $f_c = P / (b \times d)$ = **276** lb/in^2
f_c / F_c' = **0.335**

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **700** lb_ft
Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r$ = **1389** lb/in^2
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **639** lb/in^2
$f_{b,x} / F_{b,x}'$ = **0.460**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; V_x = **300** lb
Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D$ = **155** lb/in^2
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d)$ = **41** lb/in^2
$f_{v,x} / F_{v,x}'$ = **0.267**

PASS - Design shear stress exceeds actual shear stress

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis; $F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2$ = **1063** lb/in^2
Critical buckling design value in y-axis; $F_{cE2} = 0.822 \times E_{min}'$ = **419220** lb/in^2
Bending and compression check - eqs.3.9-3 and 3.9-4

$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2}))$ = **0.734**; < 1.0

PASS - Combined bending and compressive stresses are within permissible limits

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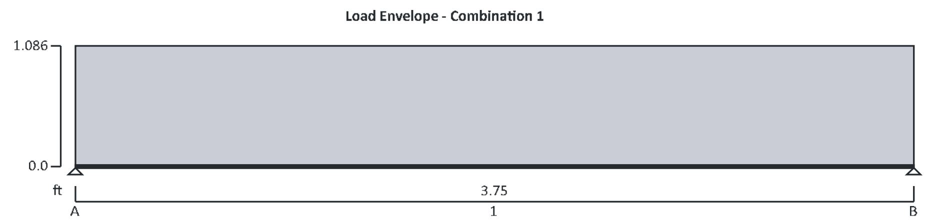
Revision:	Date:	Sheet No. 189
Project Name: Express Stop – View High		

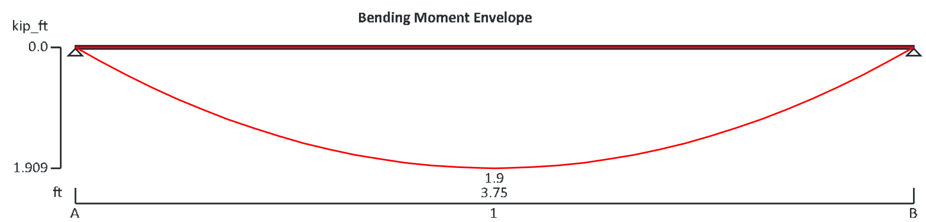
TYPICAL HEADER

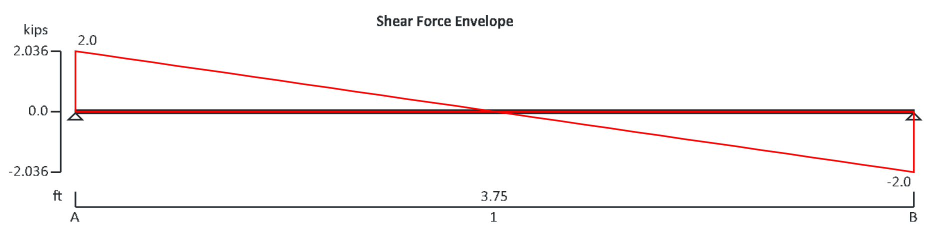
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 540 lb/ft
Snow full UDL 540 lb/ft

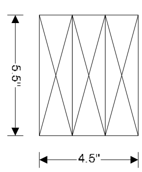
Load combinations

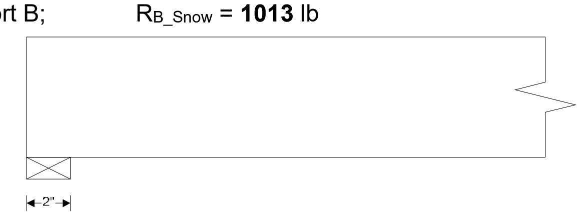
Load combination 1	Support A	Dead × 1.00
		Live × 0.00
		Roof Live × 0.00
		Snow × 1.00
		Wind × 0.00

		Seismic × 0.00
	Span 1	Dead × 1.00
		Live × 0.00
		Roof Live × 0.00
		Snow × 1.00
		Wind × 0.00
		Seismic × 0.00
	Support B	Dead × 1.00
		Live × 0.00
		Roof Live × 0.00
		Snow × 1.00
		Wind × 0.00
		Seismic × 0.00

Analysis results

Maximum moment;	M_{max} = **1909** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **1909** lb_ft	
Maximum shear;	F_{max} = **2036** lb;	F_{min} = **-2036** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **2036** lb	
Total load on member;	W_{tot} = **4073** lb	
Reaction at support A;	R_{A_max} = **2036** lb;	R_{A_min} = **2036** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **1024** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **1013** lb	
Reaction at support B;	R_{B_max} = **2036** lb;	R_{B_min} = **2036** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **1024** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **1013** lb	





Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **2** in
Dressed breadth of sections;	b = **1.5** in
Nominal depth of sections;	d_{nom} = **6** in
Dressed depth of sections;	d = **5.5** in
Number of sections in member;	N = **3**
Overall breadth of member;	b_b = N × b = **4.5** in
Species, grade and size classification;	Spruce-Pine-Fir, No.2 grade, 2" & wider
Bending parallel to grain;	F_b = **875** lb/in^2
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2

Shear parallel to grain; F_v = **135** lb/in^2
Modulus of elasticity; E = **1400000** lb/in^2
Modulus of elasticity, stability calculations; E_{min} = **510000** lb/in^2
Mean shear modulus; G_{def} = E / 16 = **87500** lb/in^2

Member details

Service condition; **Dry**
Length of span; L_{s1} = **3.75** ft
Length of bearing; L_b = **2** in
Load duration; **Ten years**

Section properties

Cross sectional area of member; A = N × b × d = **24.75** in^2
Section modulus; S_x = N × b × d^2 / 6 = **22.69** in^3
S_y = d × (N × b)^2 / 6 = **18.56** in^3
Second moment of area; I_x = N × b × d^3 / 12 = **62.39** in^4
I_y = d × (N × b)^3 / 12 = **41.77** in^4

Adjustment factors

Load duration factor - Table 2.3.2; C_D = **1.00**
Temperature factor - Table 2.3.3; C_t = **1.00**
Size factor for bending - Table 4A; C_{Fb} = **1.30**
Size factor for tension - Table 4A; C_{Ft} = **1.30**
Size factor for compression - Table 4A; C_{Fc} = **1.10**
Flat use factor - Table 4A; C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8
C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8
C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9; C_r = **1.15**
Bearing area factor - cl.3.10.4; C_b = **1.00**
Depth-to-breadth ratio; d_{nom} / (N × b_{nom}) = **1.00**
- Beam is fully restrained
Beam stability factor - cl.3.3.3; C_L = **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain; F_{c_perp}' = F_{c_perp} × C_t × C_{ic_perp} × C_b = **425** lb/in^2
Applied compression stress perpendicular to grain; f_{c_perp} = R_{B_max} / (N × b × L_b) = **226** lb/in^2
f_{c_perp} / F_{c_perp}' = **0.532**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; F_b' = F_b × C_D × C_t × C_L × C_{Fb} × C_i × C_r = **1308** lb/in^2
Actual bending stress; f_b = M / S_x = **1010** lb/in^2
f_b / F_b' = **0.772**

PASS - Design bending stress exceeds actual bending stress

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Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i =$ **135** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **123** lb/in^2

$f_v / F_v' =$ **0.914**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1400000** lb/in^2

Design deflection; $\delta_{adm} = 0.003 \times L_{s1} =$ **0.135** in

Total deflection; $\delta_{b\_s1} =$ **0.055** in

$\delta_{b\_s1} / \delta_{adm} =$ **0.410**

PASS - Total deflection is less than design deflection

TOP PLATE

WOOD MEMBER DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.24

Design summary

Overall design utilisation; 0.977

Overall design status; PASS

Top Plate results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	1389	1214	0.874	PASS
Shear stress	lb/in^2	155	152	0.977	PASS
Bearing stress	lb/in^2	425	202	0.476	PASS

Top Plate

Member details

Service condition; Dry

Sawn lumber section details

Number of sections in member; N = **2**

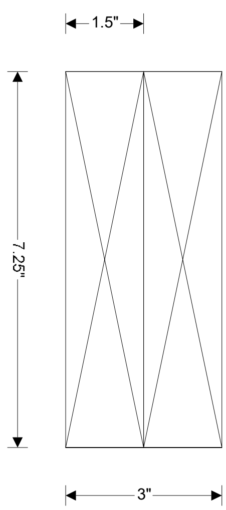
Nominal breadth of sections; b_{nom} = **2** in

Breadth of sections; b = **1.5** in

Nominal depth of sections; d_{nom} = **8** in

Depth of sections; d = **7.25** in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



2/2"x8" sawn lumber sections
Cross-sectional area, A, 21.75 in^2
Section modulus, S_x, 26.3 in^3
Section modulus, S_y, 10.9 in^3
Second moment of area, I_x, 95.3 in^4
Second moment of area, I_y, 16.3 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 0.866 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Unbraced length - Major axis; L_x = **0** ft

Unbraced length - Minor axis; L_y = **2** ft

Effective bending length - Minor axis; $L_{e,y} = 1.63 \times L_y + 3 \times d$ = **5.072** ft

Column buckling length - Minor axis; $L_{b,y} = L_y$ = **2** ft

Bearing length; L_b = **1.5** in

Analysis results

Design bending moment - Minor axis; M_y = **1100** lb_ft

Design shear force - Minor axis; V_y = **2200** lb

Design perpendicular compression - Minor axis; R_y = **2200** lb

Adjustment factors - Table 4.3.1

Two months load duration factor - Table 2.3.2; C_D = **1.15**

Size factor for bending - Table 4A; C_{Fb} = **1.2**

Flat use factor - Table 4A; C_{fu} = **1.15**

Bending members - Flexure - cl.3.3

Design bending moment; M_y = **1100** lb_ft

Design bending stress - Table 4.3.1; $F_{b,y}' = F_b \times C_D \times C_{Fb} \times C_{fu}$ = **1389** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,y} = M_y / S_y$ = **1214** lb/in^2

$f_{b,y} / F_{b,y}'$ = **0.874**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; V_y = **2200** lb

Design shear stress - Table 4.3.1; $F_{v,y}' = F_v \times C_D$ = **155** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,y} = 3 \times V_y / (2 \times N \times b \times d)$ = **152** lb/in^2

$f_{v,y} / F_{v,y}'$ = **0.977**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_y = **2200** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,y}' = F_{c\_perp}$ = **425** lb/in^2

Actual bearing stress; $f_{c\_perp,y} = R_y / (d \times L_b)$ = **202** lb/in^2

$f_{c\_perp,y} / F_{c\_perp,y}'$ = **0.476**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

TRENCH FOOTING REINFORCEMENT

RC BEAM ANALYSIS & DESIGN (ACI318-2014)

In accordance with ACI318-2014

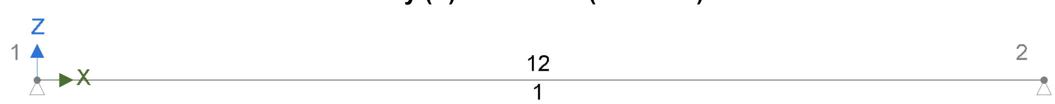
Tedds calculation version 3.3.21

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Concrete (4000 150) - R 24x36

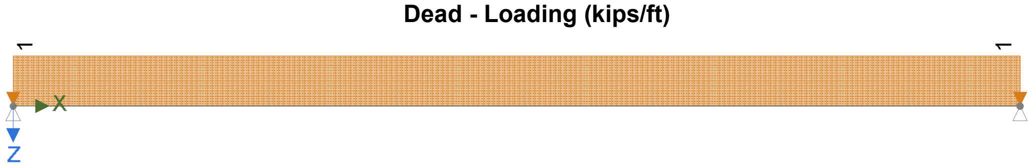


Span	Length (ft)	Section	Start Support	End Support
1	12	R 24x36	Pinned	Pinned
R 24x36: Area 864 in^2, Inertia Major 93312 in^4, Inertia Minor 41472 in^4, Shear area parallel to Minor 720 in^2, Shear area parallel to Major 720 in^2				
Concrete (4000 150): Density 150 lbm/ft^3, Youngs 3834 ksi, Shear 1750 ksi, Thermal 0.00001 °C^{-1}				

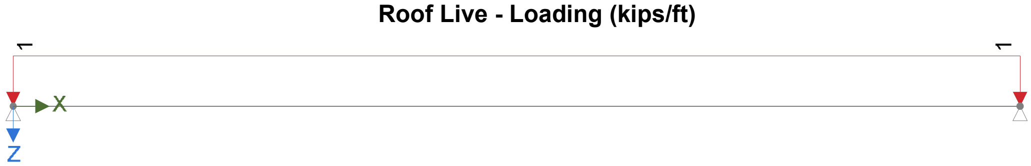
Loading

Self weight included

Dead - Loading (kips/ft)



Roof Live - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Roof Live
1.4D (Strength)	1.40	1.40	
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00

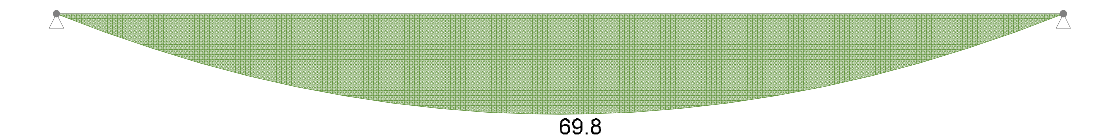
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	1 kips/ft
Beam	Roof Live	UDL	GlobalZ	1 kips/ft

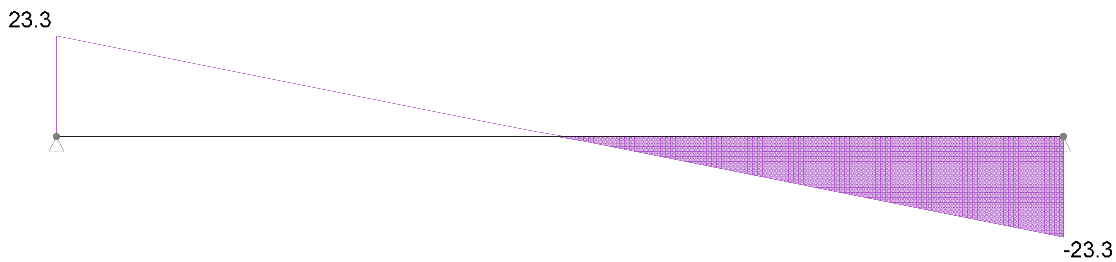
Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



;

Concrete details

Compressive strength of concrete; f'_c = **4000** psi

Density of reinforced concrete; w_c = **150** lb / ft^3

Concrete type; Normal weight

Modulus of elasticity of concrete (cl.19.2.2.1); $E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **3834254** psi

Strength reduction factor for shear; ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement; f_y = **60000** psi

Compression-controlled strain limit (cl.21.2.2.1); ε_{ty} = **0.00200**

Nominal cover to reinforcement

Cover to top reinforcement; c_{nom_t} = **3** in

Cover to bottom reinforcement; c_{nom_b} = **3** in

Cover to side reinforcement; c_{nom_s} = **3** in

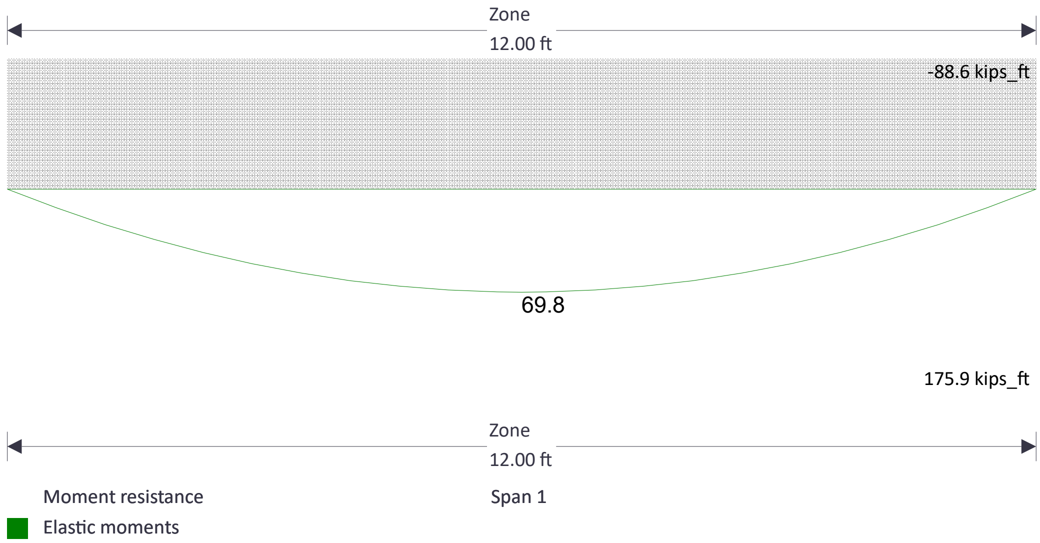
Beam - Span 1

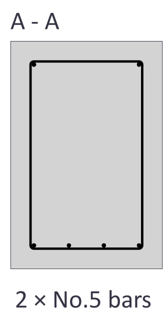
Rectangular section details

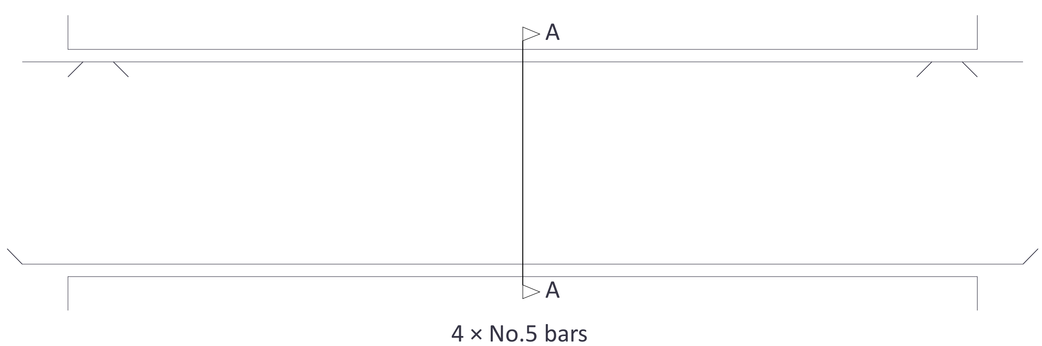
Section width; b = **24** in

Section depth; h = **36** in

Moment design







Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section; $M_u = abs(M_{m1\_s1\_z2\_max\_red}) =$ **69.840** kip_ft

Effective depth to tension reinforcement; d = **32.313** in

Tension reinforcement provided; 4 × No.5 bars

Area of tension reinforcement provided; $A_{s,prov} =$ **1.227** in^2

Minimum area of reinforcement (cl.9.6.1.2); $A_{s,min} = min(max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y, 4/3 \times A_{s,des}) =$ **0.643** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

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Stress block depth factor (cl.22.2.2.4.3); β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth of equivalent rectangular stress block; a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **0.902** in

Depth to neutral axis; c = a / β_1 = **1.062** in

Net tensile strain in extreme tension fibers; ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.08831**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1); ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.90**

Nominal moment strength; M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **195.499** kip_ft

Design moment strength; ϕM_n = M_n × ϕ_f = **175.949** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.; $s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z2\_b\_L1}$ = **5.542** in

Service load stress in reinforcement (cl.24.3.2); f_s = 2/3 × f_y = **40000** psi

Clear cover of reinforcement; c_c = c_{nom_b} + ϕ_v = **3.375** in

Maximum allowable bot bar spacing (Table 24.3.2); s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **6.563** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Control of deflections (Section 24.2)

Concrete density factor; K_w = **1.00**

Reinforcement yield strength factor; K_f = 0.4 + f_y / 100000 psi = **1.00**

Minimum thickness of beam (Table 9.3.1.1); h_{min} = (L_{m1_s1} / 16.0) × K_w × K_f = **9** in

PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing; s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z2\_v}$) + $\phi_{m1\_s1\_z2\_t\_L1}$ × $N_{m1\_s1\_z2\_t\_L1}$)) / ($N_{m1\_s1\_z2\_t\_L1}$ - 1) = **16.000** in

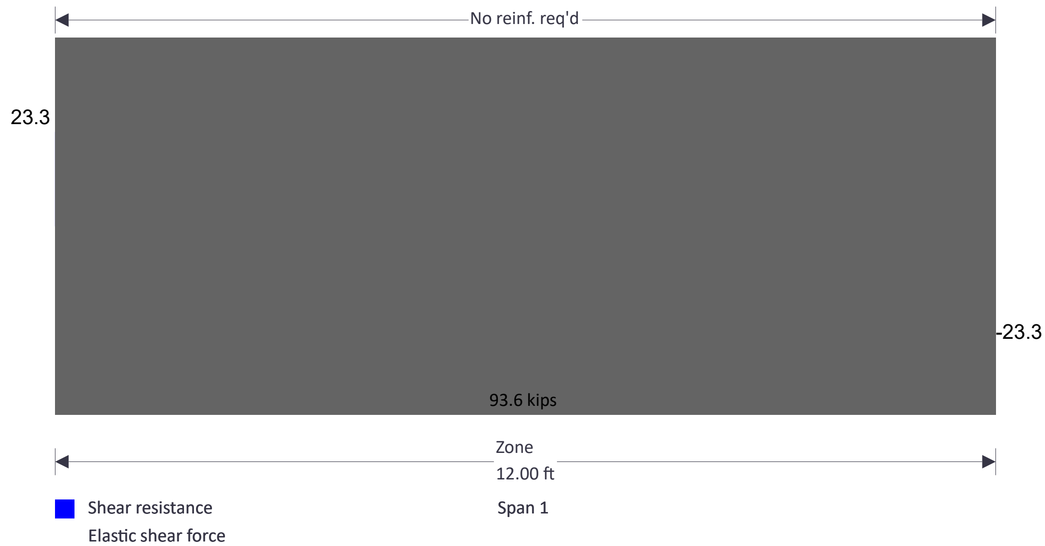
Min. allowable top bar clear spacing (cl.25.2.1); $s_{top,min}$ = **1.000** in

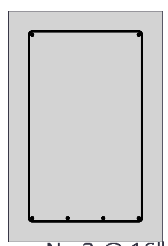
Bottom bar clear spacing; s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z2\_v}$) + $\phi_{m1\_s1\_z2\_b\_L1}$ × $N_{m1\_s1\_z2\_b\_L1}$)) / ($N_{m1\_s1\_z2\_b\_L1}$ - 1) = **4.917** in

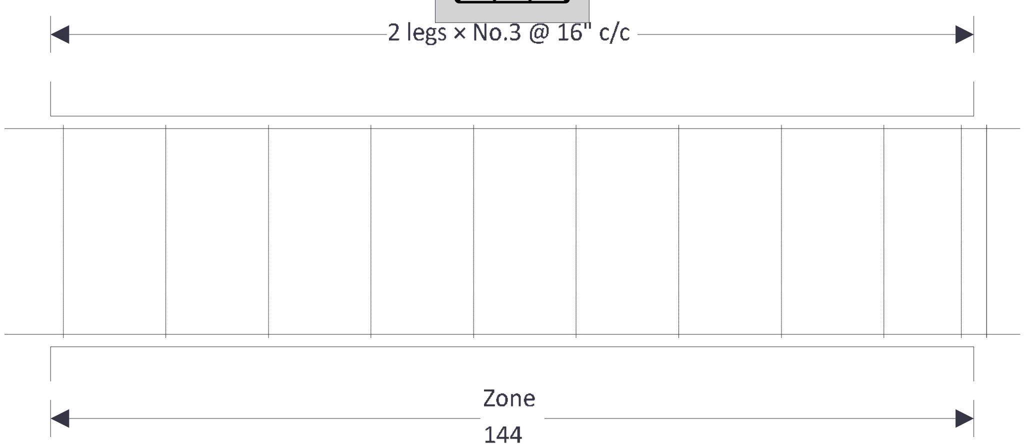
Min. allowable bottom bar clear spacing (cl.25.2.1); $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Shear design







Rectangular section in shear

Concrete weight modification factor;	λ = **1.00**
Location where min. reinf. is req'd (V_u less ϕV_c);	N/A no reinf. required along full length
Location where no reinf. is req'd (V_u less $\phi V_{s,lim}$);	Between 0.00 ft and 12.00 ft
Maximum reinforcement shear strength;	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1 \text{ psi})} \times b \times d$ = **294.282** kips
Minimum area of shear reinf. (Table 9.6.3.3);	$A_{sv,min}$ = max(50 psi, 0.75 psi $\times \sqrt{(f'_c / 1 \text{ psi})}) \times b$ / min(f_y, 60000 psi) = **0.240** in^2/ft
No reinforcement shear strength limit;	$\phi V_{s,lim} = \phi V_c / 2$ = **36.785** kips

Zone 1

Effective depth of long. reinf. used for shear zone;	d = **32.313** in
Concrete shear strength (eqn. 22.5.5.1);	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **73.570** kips
Design shear force within zone;	V_u = **23.280** kips
Reinf. shear strength required (eqn. 22.5.1.1);	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3);	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.000** in^2/ft
Area of shear reinforcement required;	$A_{sv,req}$ = 0mm^2/ft = **0.000** in^2/ft
Shear reinforcement provided;	2 legs × No.3 @ 16" c/c
Area of shear reinforcement provided;	$A_{sv,prov}$ = **0.166** in^2/ft

PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)

MISCELLANEOUS CALCULATIONS

BASE PLATES

SIMPSON STRONG-TIE COMPANY INC.
5956 W. Las Positas Blvd., Pleasanton, CA 94588
(800) 999-5099
www.strongtie.com

Calculation Report for

Prepared for

Calculation Report

Date of Completion: Jun 12, 2025

Prepared by:

Project Id

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LEIGH + O'KANE
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Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 202
Project Name: Express Stop – View High		

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.3.2410.2

Company:		Date:	6/12/2025
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Address:			
Phone:			
E-mail:			

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Version 3.3.2410.2

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Phone:			
E-mail:			

1.Project information

Project description:
Location:
Design name: Edge HSS

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.50
C_{min} (inch): 4.50
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 10.00 x 14.00 x 0.75
Yield stress: 36000 psi

Profile type/size: 6X6X3/8

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 3/4"Ø Heavy Hex Bolt, F1554 Gr. 36

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
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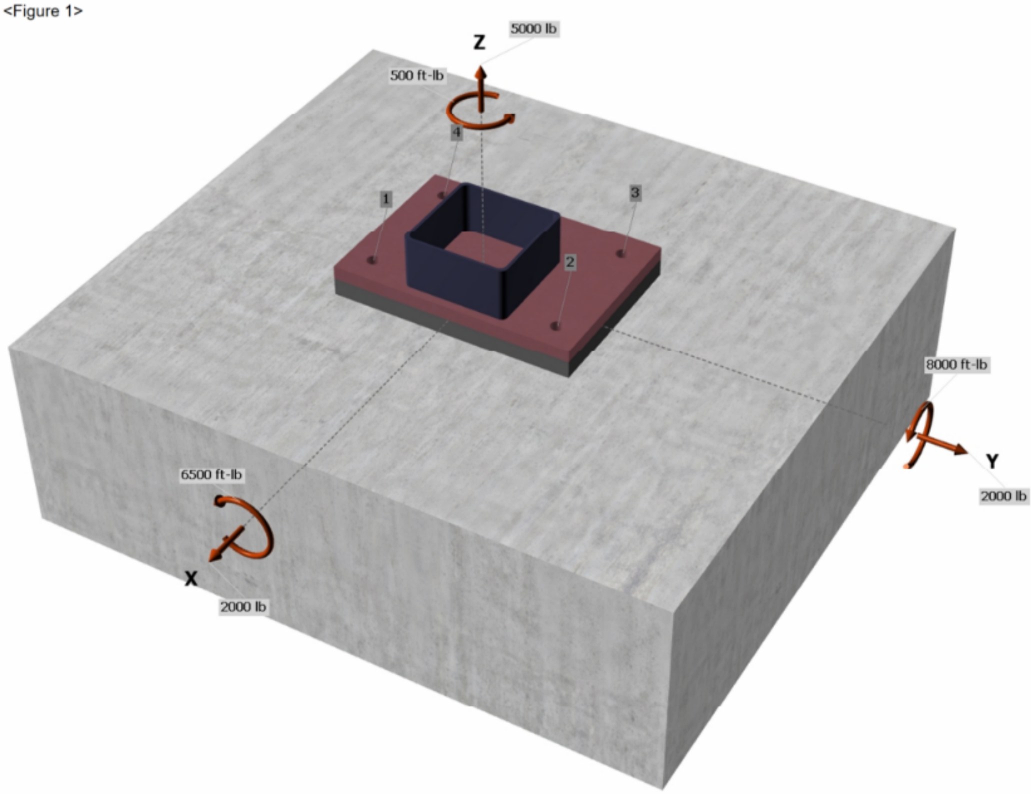
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]:	5000
V_{uax} [lb]:	2000
V_{uay} [lb]:	2000
M_{ux} [ft-lb]:	6500
M_{uy} [ft-lb]:	8000
M_{uz} [ft-lb]:	500

<Figure 1>



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Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 205
Project Name: Express Stop – View High		

SIMPSON Strong-Tie Anchor Designer™ for Concrete Software
Version 3.3.2410.2

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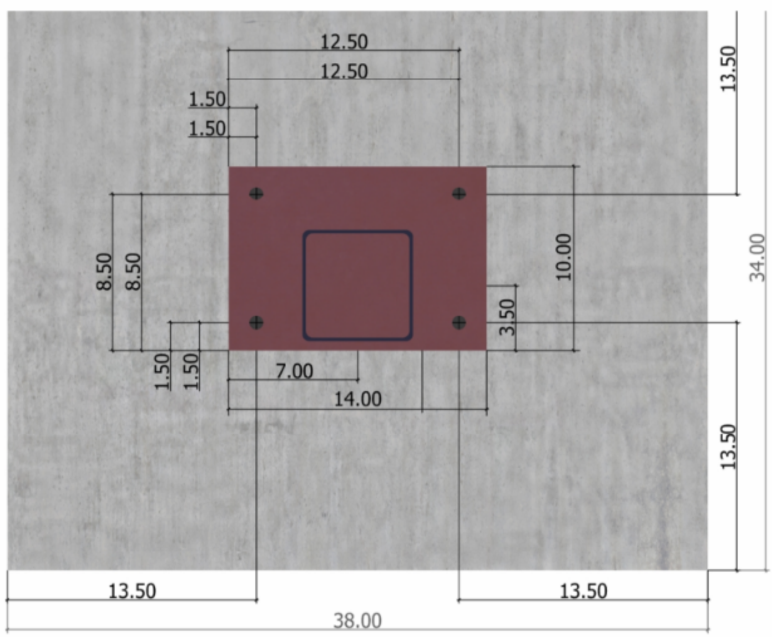
<Figure 2>

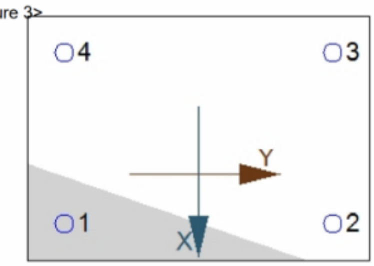


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	791.2	685.3	1046.7
2	1900.8	208.8	685.3	716.4
3	9082.7	208.8	314.7	377.7
4	5225.9	791.2	314.7	851.5
Sum	16209.5	2000.0	2000.0	2992.3

Maximum concrete compression strain (‰): 0.34
Maximum concrete compression stress (psi): 1497
Resultant tension force (lb): 16210
Resultant compression force (lb): 11210
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.12
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 1.51
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 2.25
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 2.25

<Figure 3>



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Made by: RAH	Date: 06/16/2025	Job No. 25-011
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SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.3.2410.2

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4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	8.000	34346

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
1008.00	576.00	13.50	0.879	1.000	1.00	1.000	34346	0.70	36994

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.91	4000	0.70	20406

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8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
11625	0.8	0.65	6045

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	4000	9.00	15369

$\phi V_{cbgy} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
408.00	364.50	0.912	1.000	1.000	1.061	15369	0.70	11653

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	4000	9.00	15369

$\phi V_{cbgx} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
456.00	364.50	0.808	1.000	1.000	1.061	15369	0.70	11538

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	4000	9.00	15369

$\phi V_{cbgx} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
456.00	364.50	1.000	1.000	1.000	1.061	15369	0.70	28550

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	4000	9.00	15369

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
408.00	364.50	1.000	1.000	1.000	1.061	15369	0.70	25545

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	1085.00	576.00	0.709	1.000	1.000	1.000	34346	0.70	64231

11. Results

Interaction of Tensile and Shear Forces (Sec. R17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	**9083**	**14528**	**0.63**	**Pass (Governs)**
Concrete breakout	16210	36994	0.44	Pass

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Pullout	9083	20406	0.45	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	1047	6045	0.17	Pass
T Concrete breakout y+	2000	11653	0.17	Pass
T Concrete breakout x+	2000	11538	0.17	Pass
|| Concrete breakout x+	1371	28550	0.05	Pass
|| Concrete breakout y-	1582	25545	0.06	Pass
Concrete breakout, combined	-	-	**0.24**	**Pass (Governs)**
Pryout	2828	64231	0.04	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Utilization Ratio	Permissible	Status
Sec. R17.6	0.46	0.10	55.2%	1.0	Pass

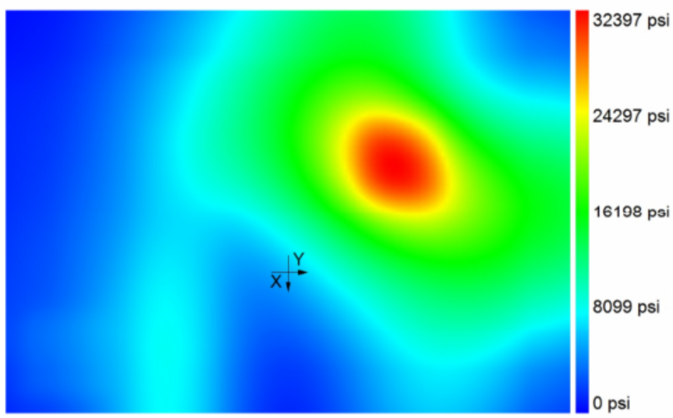
3/4"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.5 inches

Steel	**36000 psi**
Maximum stress	**32397 psi**
Calculated plate thickness	**1.707 inch**

Stress distribution



For ACI and CSA design methods, maximum base plate stress is limited to 0.9 times yield stress.
For ETAG design method, maximum base plate stress is limited to yield stress divide by 1.5.
Plate stress is derived using Von Mises theory.

$$\sigma_{xx} = \frac{F_{xx}}{t} + \frac{6M_{xx}}{t^2} \text{ (@ bottom) or } \sigma_{xx} = \frac{F_{xx}}{t} - \frac{6M_{xx}}{t^2} \text{ (@ top)}$$

$$\sigma_{yy} = \frac{F_{yy}}{t} + \frac{6M_{yy}}{t^2} \text{ (@bottom) or } \sigma_{yy} = \frac{F_{yy}}{t} - \frac{6M_{yy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xy} = \frac{F_{xy}}{t} + \frac{6M_{xy}}{t^2} \text{ (@bottom) or } \sigma_{xy} = \frac{F_{xy}}{t} - \frac{6M_{xy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xz} = \frac{V_x}{t}$$

$$\sigma_{yz} = \frac{V_y}{t}$$

$\sigma_{xx}, \sigma_{yy}, \sigma_{xy}$ as follows:

$$S_1 = \frac{\sigma_{xx} + \sigma_{yy}}{2} + \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \sigma_{xy}^2}$$

$$S_2 = \frac{\sigma_{xx} + \sigma_{yy}}{2} - \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \sigma_{xy}^2}$$

$$S_3 = 0$$

$$\sigma_{Von\,Mises} = \sqrt{\frac{(S_1 - S_2)^2 + (S_1 - S_3)^2 + (S_2 - S_3)^2}{2}}$$

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

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1.Project information

Project description:
Location:
Design name: Storefront HSS

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.50
C_{min} (inch): 4.50
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 8.00 x 10.00 x 0.75
Yield stress: 36000 psi

Profile type/size: 3-1/2X3-1/2X1/4

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 3/4"Ø Heavy Hex Bolt, F1554 Gr. 36

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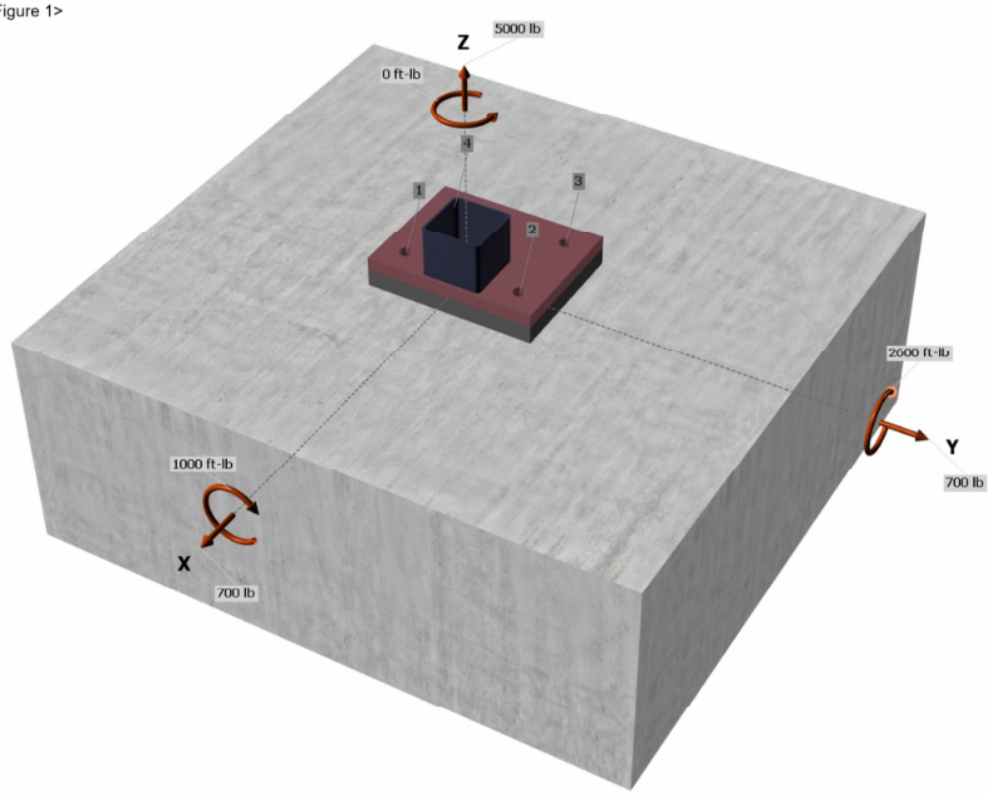
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]:	5000
V_{uax} [lb]:	700
V_{uay} [lb]:	700
M_{ux} [ft-lb]:	-1000
M_{uy} [ft-lb]:	-2600
M_{uz} [ft-lb]:	0

<Figure 1>



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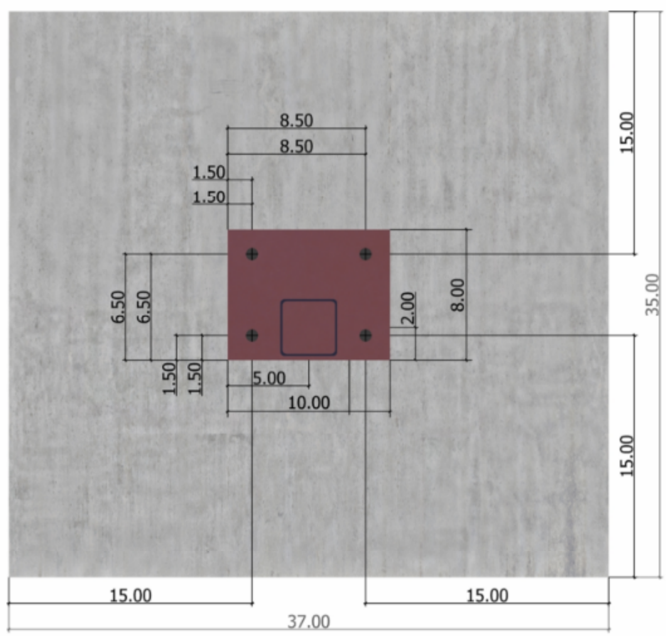
<Figure 2>

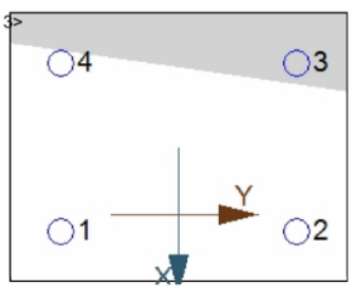


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	5404.2	241.2	222.3	328.0
2	4404.4	108.8	222.3	247.5
3	0.0	108.8	127.7	167.8
4	383.8	241.2	127.7	272.9
Sum	10192.3	700.0	700.0	1016.2

Maximum concrete compression strain (‰): 0.20
Maximum concrete compression stress (psi): 870
Resultant tension force (lb): 10192
Resultant compression force (lb): 5192
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.69
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 1.48
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 1.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 1.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	8.000	34346

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
864.00	576.00	15.00	0.842	1.000	1.00	1.000	34346	0.70	30358

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.91	4000	0.70	20406

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
11625	0.8	0.65	6045

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	4000	22.00	58736

$\phi V_{cbgy} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
1155.00	2178.00	0.980	0.836	1.000	1.000	58736	0.70	17870

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	4000	20.00	50912

$\phi V_{cbgx} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
1110.00	1800.00	0.958	0.850	1.000	1.000	50912	0.70	17891

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	4000	15.00	33068

$\phi V_{cbgx} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
832.50	1012.50	1.000	1.000	1.000	1.000	33068	0.70	38065

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	4000	15.00	33068

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
787.50	1012.50	1.000	1.000	1.000	1.000	33068	0.70	36007

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	899.00	576.00	0.852	1.000	1.000	1.000	34346	0.70	63947

11. Results

Interaction of Tensile and Shear Forces (Sec. R17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	**5404**	**14528**	**0.37**	**Pass (Governs)**
Concrete breakout	10192	30358	0.34	Pass

Pullout	5404	20406	0.26	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	328	6045	0.05	Pass
T Concrete breakout y+	700	17870	0.04	Pass
T Concrete breakout x+	700	17891	0.04	Pass
|| Concrete breakout x+	445	38065	0.01	Pass
|| Concrete breakout y-	482	36007	0.01	Pass
Concrete breakout, combined	-	-	**0.06**	**Pass (Governs)**
Pryout	990	63947	0.02	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Utilization Ratio	Permissible	Status
Sec. R17.6	0.19	0.01	20.0%	1.0	Pass

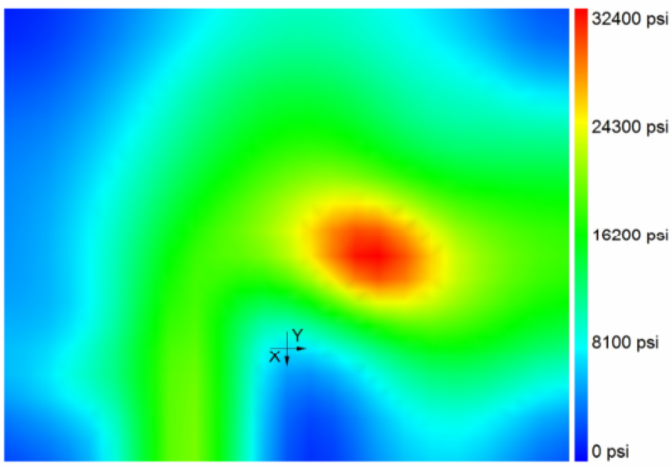
3/4"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.5 inches

Steel	**36000 psi**
Maximum stress	**32400 psi**
Calculated plate thickness	**1.030 inch**

Stress distribution



For ACI and CSA design methods, maximum base plate stress is limited to 0.9 times yield stress.
For ETAG design method, maximum base plate stress is limited to yield stress divide by 1.5.
Plate stress is derived using Von Mises theory.

$$\sigma_{xx} = \frac{F_{xx}}{t} + \frac{6M_{xx}}{t^2} \text{ (@ bottom) or } \sigma_{xx} = \frac{F_{xx}}{t} - \frac{6M_{xx}}{t^2} \text{ (@ top)}$$

$$\sigma_{yy} = \frac{F_{yy}}{t} + \frac{6M_{yy}}{t^2} \text{ (@bottom) or } \sigma_{yy} = \frac{F_{yy}}{t} - \frac{6M_{yy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xy} = \frac{F_{xy}}{t} + \frac{6M_{xy}}{t^2} \text{ (@bottom) or } \sigma_{xy} = \frac{F_{xy}}{t} - \frac{6M_{xy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xz} = \frac{V_x}{t}$$

$$\sigma_{yz} = \frac{V_y}{t}$$

$\sigma_{xx}, \sigma_{yy}, \sigma_{xy}$ as follows:

$$S_1 = \frac{\sigma_{xx} + \sigma_{yy}}{2} + \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \sigma_{xy}^{\ 2}}$$

$$S_2 = \frac{\sigma_{xx} + \sigma_{yy}}{2} - \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \sigma_{xy}^{\ 2}}$$

$$S_3 = 0$$

$$\sigma_{Von\,Mises} = \sqrt{\frac{(S_1 - S_2)^2 + (S_1 - S_3)^2 + (S_2 - S_3)^2}{2}}$$

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

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Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 217
Project Name: Express Stop – View High		

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E-mail:			

1.Project information

Project description:
Location:
Design name: Corner HSS

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.50
C_{min} (inch): 4.50
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 10.00 x 10.00 x 1.00
Yield stress: 36000 psi

Profile type/size: 6X6X3/8

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 3/4"Ø Heavy Hex Bolt, F1554 Gr. 36

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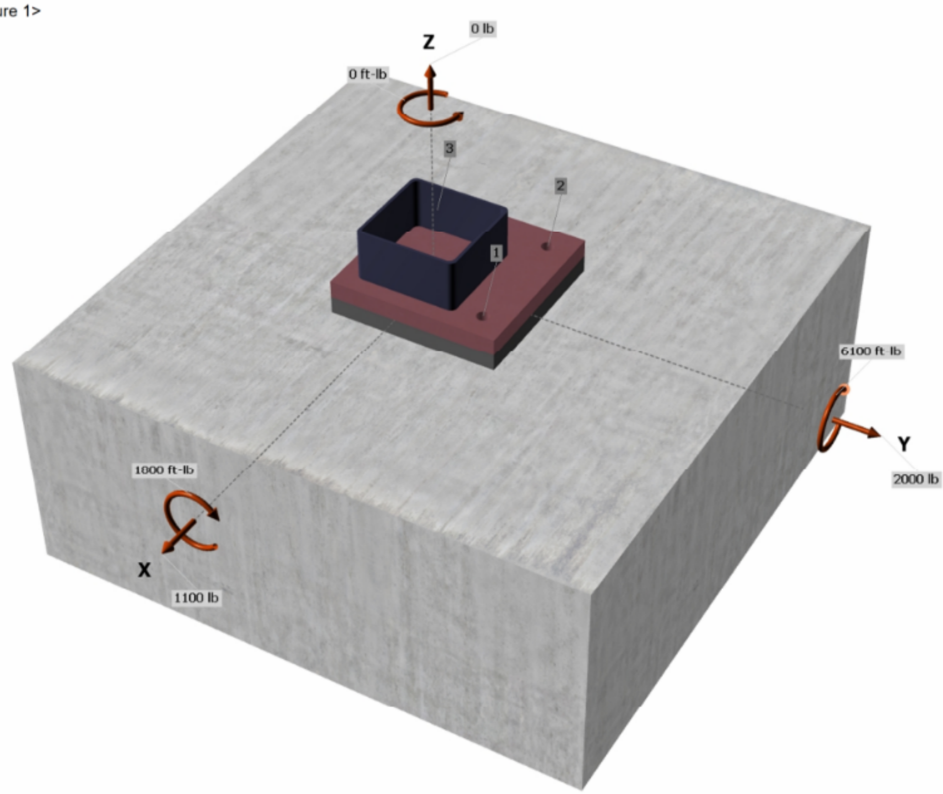
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]:	0
V_{uax} [lb]:	1100
V_{uay} [lb]:	2000
M_{ux} [ft-lb]:	-1800
M_{uy} [ft-lb]:	-6100
M_{uz} [ft-lb]:	0

<Figure 1>



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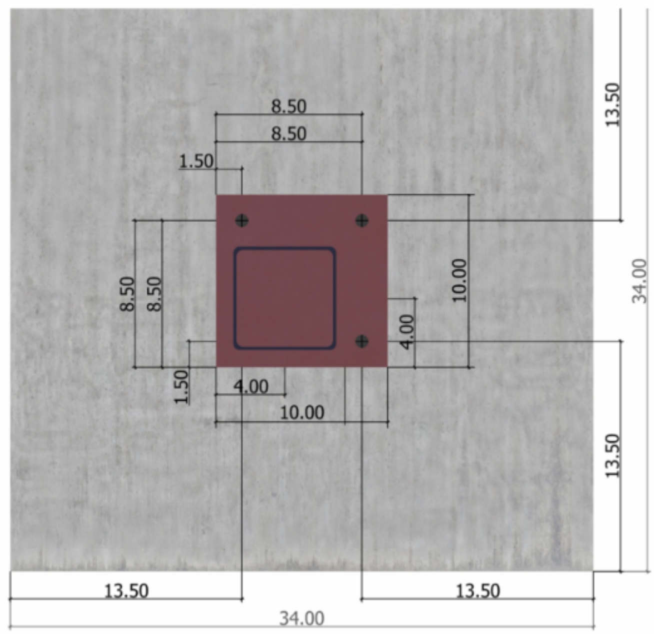
<Figure 2>

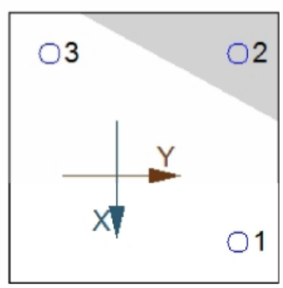


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	9514.6	126.8	1146.4	1153.4
2	0.0	126.8	426.8	445.2
3	3753.4	846.4	426.8	947.9
Sum	13268.1	1100.0	2000.0	2546.6

Maximum concrete compression strain (‰): 0.62
Maximum concrete compression stress (psi): 2679
Resultant tension force (lb): 13268
Resultant compression force (lb): 13268
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 1.52
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 1.52
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 1.42
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 2.58

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	8.000	34346

$\phi N_{cbg} = \phi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
863.00	576.00	13.50	0.788	1.000	1.00	1.000	34346	0.70	28378

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.91	4000	0.70	20406

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8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
11625	0.8	0.65	6045

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	4000	13.50	28234

$\phi V_{cbgy} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
688.50	820.13	0.975	0.900	1.000	1.000	28234	0.70	14564

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	4000	13.50	28234

$\phi V_{cbx} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbx} (lb)
683.44	820.13	0.900	1.000	1.000	28234	0.70	14823

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	4000	20.50	52833

$\phi V_{cbx} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
1045.50	1891.13	1.000	1.000	1.000	52833	0.70	40892

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	4000	20.50	52833

$\phi V_{cby} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
1045.50	1891.13	1.000	1.000	1.000	52833	0.70	40892

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	912.00	576.00	0.736	1.000	1.000	1.000	34346	0.70	56045

11. Results

Interaction of Tensile and Shear Forces (Sec. R17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	**9515**	**14528**	**0.65**	**Pass (Governs)**
Concrete breakout	13268	28378	0.47	Pass

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Pullout	9515	20406	0.47	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	**1153**	**6045**	**0.19**	**Pass (Governs)**
T Concrete breakout y+	2000	14564	0.14	Pass
T Concrete breakout x+	1100	14823	0.07	Pass
|| Concrete breakout x-	2000	40892	0.05	Pass
|| Concrete breakout y+	1100	40892	0.03	Pass
Concrete breakout, combined	-	-	0.16	Pass
Pryout	2283	56045	0.04	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Utilization Ratio	Permissible	Status
Sec. R17.6	0.49	0.06	55.7%	1.0	Pass

3/4"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

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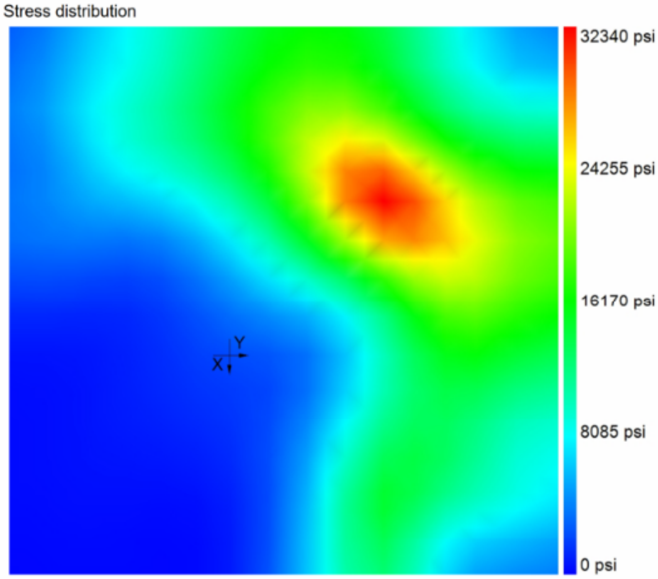
Base Plate Thickness

Required base plate thickness: 0.5 inches

Steel	**36000 psi**
Maximum stress	**32340 psi**
Calculated plate thickness	**0.989 inch**

Stress distribution



For ACI and CSA design methods, maximum base plate stress is limited to 0.9 times yield stress.
For ETAG design method, maximum base plate stress is limited to yield stress divide by 1.5.
Plate stress is derived using Von Mises theory.

$$\sigma_{xx} = \frac{F_{xx}}{t} + \frac{6M_{xx}}{t^2} \text{ (@ bottom) or } \sigma_{xx} = \frac{F_{xx}}{t} - \frac{6M_{xx}}{t^2} \text{ (@ top)}$$

$$\sigma_{yy} = \frac{F_{yy}}{t} + \frac{6M_{yy}}{t^2} \text{ (@bottom) or } \sigma_{yy} = \frac{F_{yy}}{t} - \frac{6M_{yy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xy} = \frac{F_{xy}}{t} + \frac{6M_{xy}}{t^2} \text{ (@bottom) or } \sigma_{xy} = \frac{F_{xy}}{t} - \frac{6M_{xy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xz} = \frac{V_x}{t}$$

$$\sigma_{yz} = \frac{V_y}{t}$$

$\sigma_{xx}, \sigma_{yy}, \sigma_{xy}$ as follows:

$$S_1 = \frac{\sigma_{xx} + \sigma_{yy}}{2} + \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \sigma_{xy}^{\ 2}}$$

$$S_2 = \frac{\sigma_{xx} + \sigma_{yy}}{2} - \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \sigma_{xy}^{\ 2}}$$

$$S_3 = 0$$

$$\sigma_{Von\,Mises} = \sqrt{\frac{(S_1 - S_2)^2 + (S_1 - S_3)^2 + (S_2 - S_3)^2}{2}}$$

12. Warnings

- Calculated concrete compression stress exceeds the permissible bearing stress of Φ 0.85f'c per ACI 318 Section 10.14.

- For irregular anchor patterns, the designer must consider sizing of base plate holes to ensure shear loads are distributed to anchors as designed.

- Designer must exercise own judgement to determine if this design is suitable.

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1.Project information

Project description:
Location:
Design name: Center HSS

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 105
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.50
C_{min} (inch): 4.50
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 12.00 x 12.00 x 0.72
Yield stress: 36000 psi

Profile type/size: 6X6X3/8

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 3/4"Ø Heavy Hex Bolt, F1554 Gr. 105

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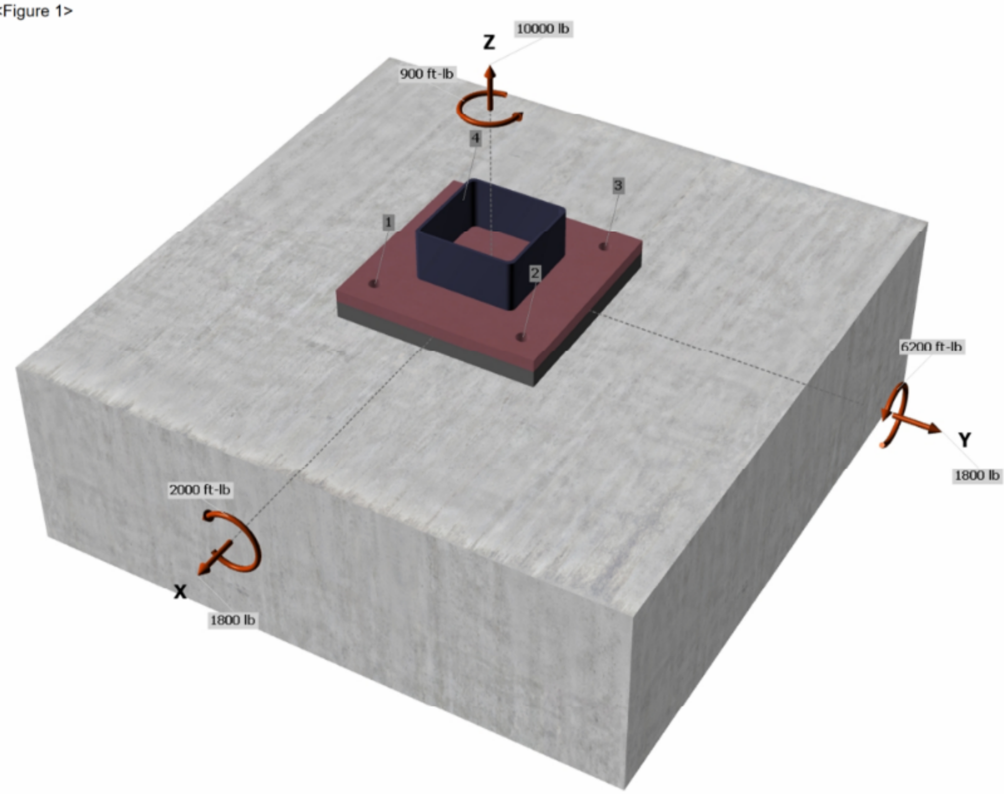
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]:	10000
V_{uax} [lb]:	1800
V_{uay} [lb]:	1800
M_{ux} [ft-lb]:	2000
M_{uy} [ft-lb]:	6200
M_{uz} [ft-lb]:	900

<Figure 1>



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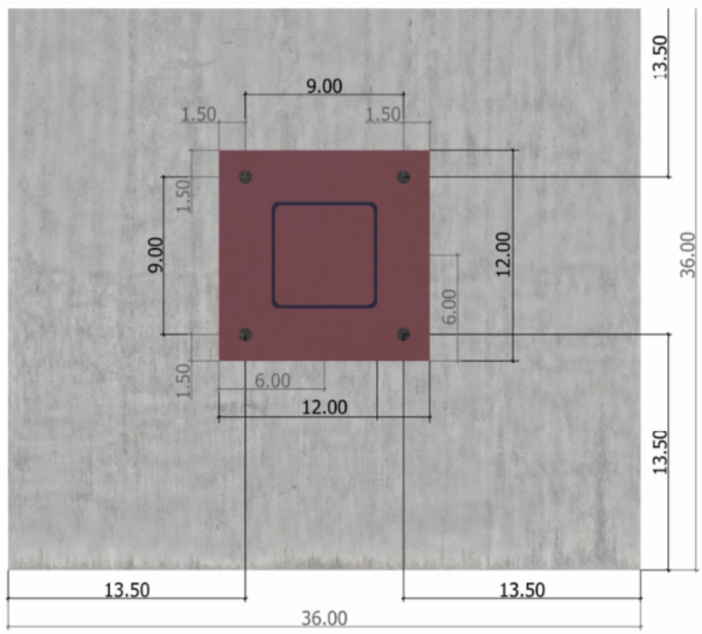
<Figure 2>

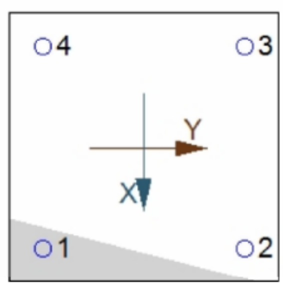


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	750.0	750.0	1060.6
2	946.8	150.0	750.0	764.8
3	7268.4	150.0	150.0	212.1
4	5656.2	750.0	150.0	764.8
Sum	13871.4	1800.0	1800.0	2802.5

Maximum concrete compression strain (‰): 0.17
Maximum concrete compression stress (psi): 740
Resultant tension force (lb): 13871
Resultant compression force (lb): 3872
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.67
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 2.39
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 3.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 3.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
41750	0.75	31313

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	8.000	34346

$\phi N_{cbg} = \phi\ (A_{Nc} / A_{Nco})\ \Psi_{ec,N}\ \Psi_{ed,N}\ \Psi_{c,N}\ \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
1008.00	576.00	13.50	0.790	1.000	1.00	1.000	34346	0.70	33241

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.91	4000	0.70	20406

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8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
25050	0.8	0.65	13026

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	4000	9.00	15369

$\phi V_{cbgy} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
432.00	364.50	0.818	1.000	1.000	1.061	15369	0.70	11065

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	4000	9.00	15369

$\phi V_{cbgx} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
432.00	364.50	0.818	1.000	1.000	1.061	15369	0.70	11065

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	4000	9.00	15369

$\phi V_{cbgx} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
432.00	364.50	1.000	1.000	1.000	1.061	15369	0.70	27047

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	4000	9.00	15369

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
432.00	364.50	1.000	1.000	1.000	1.061	15369	0.70	27047

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	1089.00	576.00	0.640	1.000	1.000	1.000	34346	0.70	58183

11. Results

Interaction of Tensile and Shear Forces (Sec. R17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	7268	31313	0.23	Pass
Concrete breakout	**13871**	**33241**	**0.42**	**Pass (Governs)**

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.3.2410.2

Company:		Date:	6/12/2025
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Project:			
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Phone:			
E-mail:			

Pullout	7268	20406	0.36	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	1061	13026	0.08	Pass
T Concrete breakout y+	1800	11065	0.16	Pass
T Concrete breakout x+	1800	11065	0.16	Pass
|| Concrete breakout x+	1500	27047	0.06	Pass
|| Concrete breakout y-	1500	27047	0.06	Pass
Concrete breakout, combined	**-**	**-**	**0.23**	**Pass (Governs)**
Pryout	2546	58183	0.04	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Utilization Ratio	Permissible	Status
Sec. R17.6	0.23	0.09	31.9%	1.0	Pass

3/4"Ø Heavy Hex Bolt, F1554 Gr. 105 with hef = 8.000 inch meets the selected design criteria.

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Version 3.3.2410.2

Company:		Date:	6/12/2025
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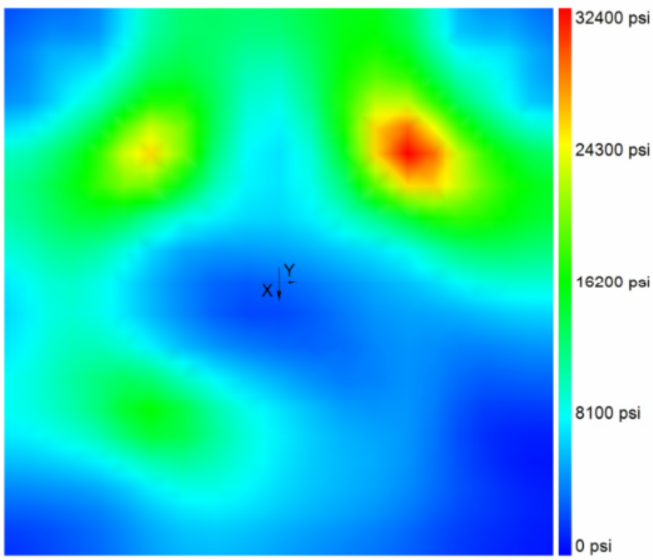
Phone:			
E-mail:			

Base Plate Thickness

Required base plate thickness: 0.5 inches

Steel	**36000 psi**
Maximum stress	**32400 psi**
Calculated plate thickness	**0.737 inch**

Stress distribution



For ACI and CSA design methods, maximum base plate stress is limited to 0.9 times yield stress.
For ETAG design method, maximum base plate stress is limited to yield stress divide by 1.5.
Plate stress is derived using Von Mises theory.

$$\sigma_{xx} = \frac{F_{xx}}{t} + \frac{6M_{xx}}{t^2} \text{ (@ bottom) or } \sigma_{xx} = \frac{F_{xx}}{t} - \frac{6M_{xx}}{t^2} \text{ (@ top)}$$

$$\sigma_{yy} = \frac{F_{yy}}{t} + \frac{6M_{yy}}{t^2} \text{ (@bottom) or } \sigma_{yy} = \frac{F_{yy}}{t} - \frac{6M_{yy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xy} = \frac{F_{xy}}{t} + \frac{6M_{xy}}{t^2} \text{ (@bottom) or } \sigma_{xy} = \frac{F_{xy}}{t} - \frac{6M_{xy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xz} = \frac{V_x}{t}$$

$$\sigma_{yz} = \frac{V_y}{t}$$

$\sigma_{xx}, \sigma_{yy}, \sigma_{xy}$ as follows:

$$S_1 = \frac{\sigma_{xx} + \sigma_{yy}}{2} + \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \sigma_{xy}^2}$$

$$S_2 = \frac{\sigma_{xx} + \sigma_{yy}}{2} - \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \sigma_{xy}^2}$$

$$S_3 = 0$$

$$\sigma_{Von\,Mises} = \sqrt{\frac{(S_1 - S_2)^2 + (S_1 - S_3)^2 + (S_2 - S_3)^2}{2}}$$

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

MOMENT FRAME CONNECTIONS

Project item Mid Storefront Moment

Design

Name	Mid Storefront Moment
Description	
Analysis	Stress, strain/ simplified loading
Design code	AISC 360-16 (LRFD) / ACI 318-14

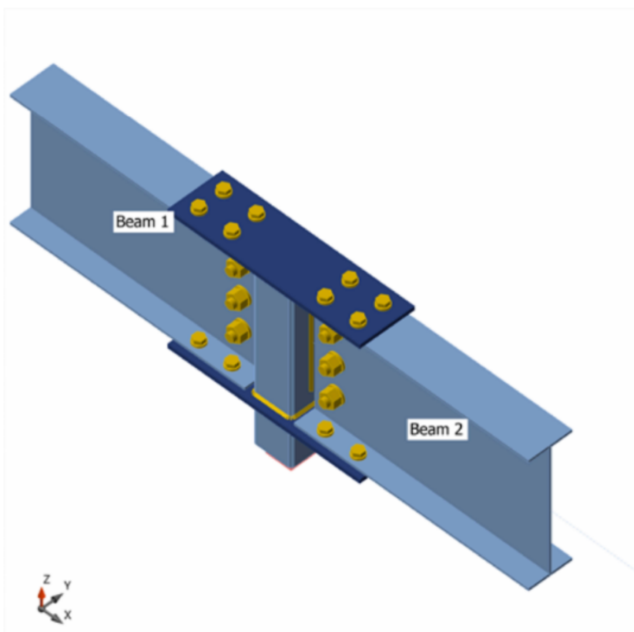
Members

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
Column	379 - HSS 3-1/2x3-1/2x1/4	0.0	90.0	-90.0	0.00	0.00	0.00
Beam 1	107 - W 12x14	0.0	0.0	0.0	0.00	0.00	-12.58
Beam 2	107 - W 12x14	0.0	0.0	0.0	0.00	0.00	-12.58

Supports and forces

Name	Support	Forces in	X [in]
Column / begin	N-Vy-Vz-Mx-My-Mz	Position	0.00
Beam 1 / begin		Position	0.00
Beam 2 / end		Position	0.00



Cross-sections

Name	Material
379 - HSS 3-1/2x3-1/2x1/4	A500B-46
107 - W 12x14	A992-50

Bolts

Name	Diameter [in]	f_y [ksi]	f_u [ksi]	Gross area [in^2]
3/4 A325	0.75	92.0	120.0	0.44
1/2 A325	0.50	92.0	120.0	0.20

Load effects (Equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	Beam 1 / Begin	-0.53	0.01	-1.23	0.81	-2.41	0.00
	Beam 2 / End	0.28	0.01	-1.55	0.85	2.88	0.00
123 LRFD_{12.1}-0.9D+W(2)	Beam 1 / Begin	0.18	0.02	-0.32	-0.01	-1.07	0.00
	Beam 2 / End	0.19	0.02	-0.12	0.00	-0.61	0.00
124 LRFD_{12.2}-0.9D+W(3)	Beam 1 / Begin	0.19	0.02	-0.33	-0.01	-1.09	0.00
	Beam 2 / End	0.18	0.02	-0.13	0.00	-0.59	0.00
88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	Beam 1 / Begin	0.02	0.02	-0.72	0.24	-1.17	0.00
	Beam 2 / End	-0.31	0.02	-1.05	0.27	2.47	0.00
132 LRFD_{12.10}-0.9D+W(9)	Beam 1 / Begin	-2.99	0.01	-0.37	-0.01	-0.76	0.00
	Beam 2 / End	2.60	0.01	-0.40	0.00	0.73	0.00
28 LRFD_{8.4}-1.2D+1.6S+0.5W(7)	Beam 1 / Begin	0.43	0.00	-1.16	0.81	-2.38	0.00
	Beam 2 / End	-0.18	0.00	-1.43	0.85	2.43	0.00
122 LRFD_{10.42}-1.2D+L+0.5S+W(8)	Beam 1 / Begin	1.50	-0.02	-0.58	-0.02	-1.18	0.00
	Beam 2 / End	-1.18	-0.01	-0.70	0.00	1.21	0.00
111 LRFD_{10.31}-1.2D+L+0.5S+W(11)	Beam 1 / Begin	1.61	-0.01	-0.57	-0.02	-1.45	0.00
	Beam 2 / End	-1.02	0.00	-0.50	0.00	0.26	0.00
82 LRFD_{10.2}-1.2D+L+0.5S+W(14)	Beam 1 / Begin	0.21	0.02	-0.68	0.24	-1.80	0.00
	Beam 2 / End	0.16	0.02	-0.56	0.27	0.14	0.00
27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	Beam 1 / Begin	0.80	0.00	-1.19	0.81	-2.59	0.00
	Beam 2 / End	-0.51	0.00	-1.37	0.85	2.03	0.00
26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	Beam 1 / Begin	0.12	0.02	-1.19	0.81	-2.64	0.00
	Beam 2 / End	0.06	0.01	-1.32	0.85	1.84	0.00
114 LRFD_{10.34}-1.2D+L+0.5S+W(20)	Beam 1 / Begin	1.60	-0.01	-0.59	-0.02	-0.98	0.00
	Beam 2 / End	-1.47	-0.01	-0.85	0.00	1.94	0.00
32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	Beam 1 / Begin	0.03	0.02	-1.21	0.81	-2.32	0.00
	Beam 2 / End	-0.18	0.01	-1.57	0.85	3.01	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Loc. deformation	0.0 < 3%	OK
Bolts	34.1 < 100%	OK
Welds	75.1 < 100%	OK
Buckling	Not calculated	
GMNA	Not calculated	

Plates

Name	Material	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
Column	A500B-46	1/4	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	14.9	0.0	0.0	OK
Beam 1-bfl 1	A992-50	1/4	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	36.5	0.0	1.8	OK
Beam 1-tfl 1	A992-50	1/4	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	42.1	0.0	1.9	OK
Beam 1-w 1	A992-50	3/16	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	29.9	0.0	2.4	OK
Beam 2-bfl 1	A992-50	1/4	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	38.6	0.0	1.9	OK
Beam 2-tfl 1	A992-50	1/4	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	43.4	0.0	1.9	OK
Beam 2-w 1	A992-50	3/16	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	31.0	0.0	2.5	OK
Bottom FP	A36	3/8	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	18.6	0.0	1.9	OK
Top FP	A36	3/8	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	16.6	0.0	1.9	OK
SP B1	A500B-46	1/2	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	12.2	0.0	6.6	OK
SP B2	A500B-46	1/2	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	12.4	0.0	6.9	OK

Design data

Material	F_y [ksi]	ε_{lim} [%]
A500B-46	46.0	5.0
A992-50	50.0	5.0
A36	36.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain

Detailed result for Beam 2-tfl 1

Design values used in the analysis

$\phi F_{y} = \space$ 45.0 ksi

Where:
$F_{y} = \space$50.0 ksi – characteristic yield strength
$\phi = \space$0.90 – resistance factor for steel material AISC 360-16 – B3.1

Loc. deformation

Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
Column	3.50	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.00	0.11	0.0	OK

Symbol explanation

d_0	Cross-section size
δ	Local cross-section deformation
δ_{lim}	Allowed deformation

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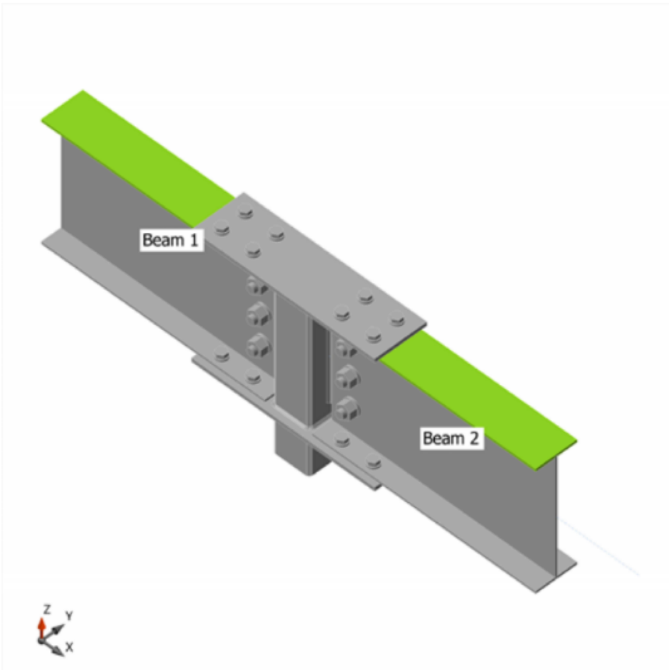
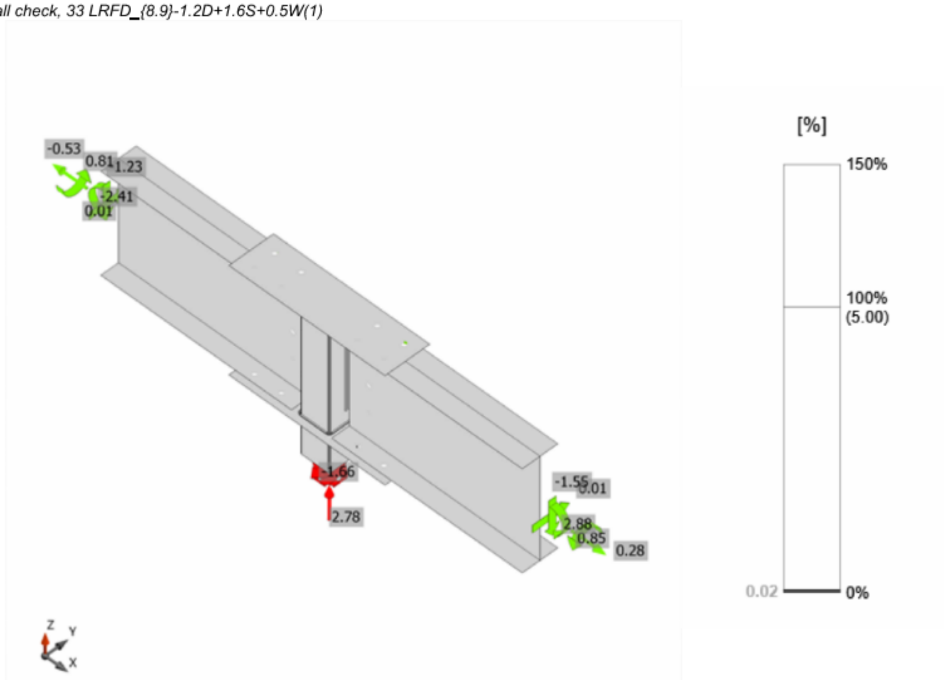
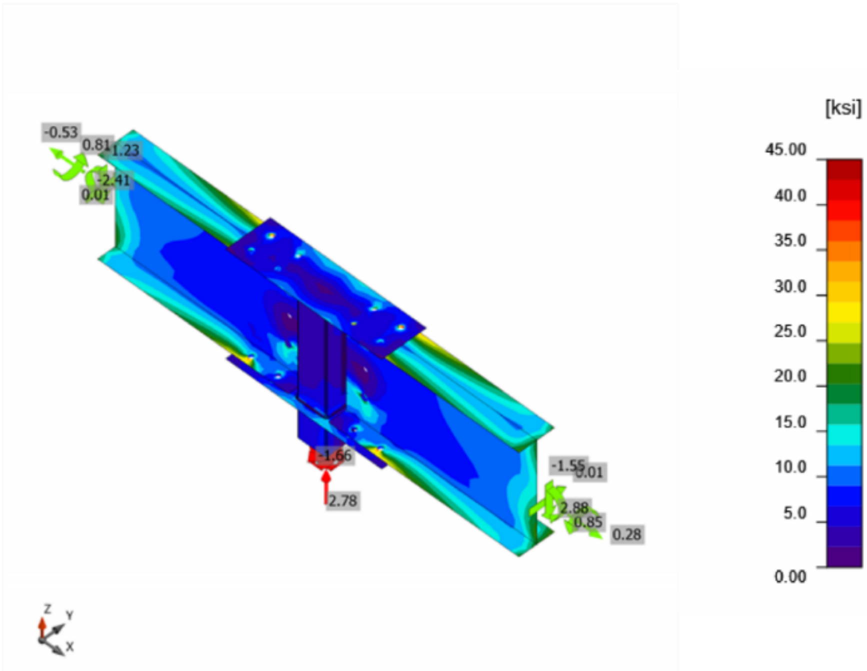
Revision:	Date:	Sheet No. 234
Project Name: Express Stop – View High		



Overall check, 33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)



Strain check, 33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)



Equivalent stress, 33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)

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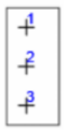
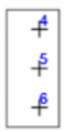
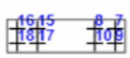
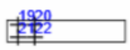
Revision:	Date:	Sheet No. 236
Project Name: Express Stop – View High		

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B1	3/4 A325 - 1	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	1.47	0.36	17.55	4.9	2.0	0.0	OK	OK
	B2	3/4 A325 - 1	132 LRFD_{12.10}-0.9D+W(9)	0.07	0.27	17.55	0.2	1.6	0.0	OK	OK
	B3	3/4 A325 - 1	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	1.97	0.40	17.55	6.6	2.3	0.0	OK	OK
	B4	3/4 A325 - 1	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	1.56	0.40	17.55	5.2	2.3	0.0	OK	OK
	B5	3/4 A325 - 1	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.18	0.25	17.55	0.6	1.4	0.0	OK	OK
	B6	3/4 A325 - 1	28 LRFD_{8.4}-1.2D+1.6S+0.5W(7)	2.04	0.41	17.55	6.8	2.3	0.0	OK	OK
	B7	1/2 A325 - 2	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.37	2.47	11.90	2.8	31.1	0.0	OK	OK
	B8	1/2 A325 - 2	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.75	1.98	13.16	5.6	24.9	0.0	OK	OK
	B9	1/2 A325 - 2	28 LRFD_{8.4}-1.2D+1.6S+0.5W(7)	0.67	1.93	13.16	5.0	24.3	0.0	OK	OK
	B10	1/2 A325 - 2	28 LRFD_{8.4}-1.2D+1.6S+0.5W(7)	0.24	1.28	9.60	1.8	16.1	0.0	OK	OK
	B11	1/2 A325 - 2	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	0.66	2.64	13.16	4.9	33.2	0.0	OK	OK
	B12	1/2 A325 - 2	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	0.37	2.32	13.16	2.8	29.2	0.0	OK	OK
	B13	1/2 A325 - 2	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.39	2.02	8.98	2.9	25.5	0.0	OK	OK
	B14	1/2 A325 - 2	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.32	1.67	13.16	2.5	21.0	0.0	OK	OK
	B15	1/2 A325 - 2	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.77	2.09	13.16	5.8	26.3	0.0	OK	OK
	B16	1/2 A325 - 2	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.39	2.59	12.07	2.9	32.6	0.0	OK	OK
	B17	1/2 A325 - 2	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	0.24	1.40	10.18	1.8	17.7	0.0	OK	OK
	B18	1/2 A325 - 2	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	0.69	2.06	13.16	5.2	25.9	0.0	OK	OK

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B19	1/2 A325 - 2	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.37	2.38	13.16	2.8	30.0	0.0	OK	OK
	B20	1/2 A325 - 2	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.68	2.71	13.16	5.1	34.1	0.0	OK	OK
	B21	1/2 A325 - 2	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.35	1.77	13.16	2.7	22.3	0.0	OK	OK
	B22	1/2 A325 - 2	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.41	2.13	9.27	3.1	26.8	0.0	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
3/4 A325 - 1	29.79	17.88
1/2 A325 - 2	13.24	7.95

Symbol explanation

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

Detailed result for B20

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

\(\phi R_n = \phi \cdot F_{nt} \cdot A_b =\space \) 13.24 kip ≥ \(F_t =\space \) 0.68 kip

Where:
\(F_{nt} =\space \)89.9 ksi – nominal tensile stress AISC 360-16 – Table J3.2
\(A_b =\space \)0.20 in^2 – gross bolt cross-sectional area
\(\phi =\space \)0.75 – resistance factor

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

\(\phi R_n = \phi \cdot F_{nv} \cdot A_b =\space \) 7.95 kip ≥ \(V =\space \) 2.71 kip

Where:
\(F_{nv} =\space \)54.0 ksi – nominal shear stress AISC 360-16 – Table J3.2
\(A_b =\space \)0.20 in^2 – gross bolt cross-sectional area
\(\phi =\space \)0.75 – resistance factor

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

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

$\phi R_n = \space$ 13.16 kip ≥ $V = \space$ 2.71 kip

Where:

$l_c = \space$ 4.24 in	– clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
$t = \space$ 0.23 in	– thickness of the plate
$d = \space$ 0.50 in	– diameter of a bolt
$F_u = \space$ 65.0 ksi	– specified minimum tensile strength of the connected material
$\phi = \space$ 0.75	– resistance factor for bearing at bolt holes

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

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

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Ut_c [%]	Detailing	Status
Column-arc 7	Bottom FP	E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.10	0.37	26.9	13.4	OK	OK
		E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.17	0.36	46.0	28.0	OK	OK
Column-arc 8	Bottom FP	E70xx	1/8	3/16	0.08	0.08	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	0.22	0.43	52.3	33.0	OK	OK
		E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.31	0.41	75.0	50.8	OK	OK
Column-arc 9	Bottom FP	E70xx	1/8	3/16	0.08	0.08	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	0.26	0.47	55.7	35.6	OK	OK
		E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.34	0.46	74.7	50.5	OK	OK
Column-w 4	Bottom FP	E70xx	1/8	3/16	2.94	0.33	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.40	1.78	22.5	9.9	OK	OK
		E70xx	1/8	3/16	2.94	0.33	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.62	1.86	33.5	16.7	OK	OK
Column-arc 10	Bottom FP	E70xx	1/8	3/16	0.08	0.08	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.20	0.47	43.6	26.1	OK	OK
		E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.15	0.46	32.4	17.6	OK	OK
Column-arc 11	Bottom FP	E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.18	0.40	43.9	26.4	OK	OK
		E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.12	0.41	29.5	15.3	OK	OK
Column-arc 12	Bottom FP	E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.11	0.35	30.7	16.2	OK	OK
		E70xx	1/8	3/16	0.08	0.08	111 LRFD_{10.31}-1.2D+L+0.5S+W(11)	0.06	0.37	16.5	7.0	OK	OK
Column-w 1	Bottom FP	E70xx	1/8	3/16	2.94	0.33	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.12	1.42	8.7	0.0	OK	OK
		E70xx	1/8	3/16	2.94	0.33	111 LRFD_{10.31}-1.2D+L+0.5S+W(11)	0.08	1.55	5.4	0.0	OK	OK
Column-arc 1	Bottom FP	E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.12	0.34	33.8	18.6	OK	OK
		E70xx	1/8	3/16	0.08	0.08	114 LRFD_{10.34}-1.2D+L+0.5S+W(20)	0.07	0.39	19.1	8.6	OK	OK
Column-arc 2	Bottom FP	E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.20	0.42	46.3	28.2	OK	OK
		E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.14	0.41	34.8	19.4	OK	OK

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Ut_c [%]	Detailing	Status
Column-arc 3	Bottom FP	E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.21	0.48	45.2	27.3	OK	OK
		E70xx	1/8	3/16	0.08	0.08	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.17	0.47	35.9	20.2	OK	OK
Column-w 2	Bottom FP	E70xx	1/8	3/16	2.94	0.33	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	0.37	1.78	20.6	7.4	OK	OK
		E70xx	1/8	3/16	2.94	0.33	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.64	1.86	34.6	15.1	OK	OK
Column-arc 4	Bottom FP	E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.26	0.47	54.5	34.7	OK	OK
		E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.35	0.46	75.0	50.9	OK	OK
Column-arc 5	Bottom FP	E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.22	0.43	51.9	32.6	OK	OK
		E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.31	0.41	75.1	51.0	OK	OK
Column-arc 6	Bottom FP	E70xx	1/8	3/16	0.08	0.08	27 LRFD_{8.3}-1.2D+1.6S+0.5W(12)	0.10	0.35	28.4	14.5	OK	OK
		E70xx	1/8	3/16	0.08	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.18	0.35	51.3	32.2	OK	OK
Column-w 3	Bottom FP	E70xx	1/8	3/16	2.94	0.33	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.12	1.51	7.8	0.0	OK	OK
		E70xx	1/8	3/16	2.94	0.33	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.19	1.41	13.5	0.0	OK	OK
Column-w 4	SP B1	E70xx	1/8	3/16	8.94	0.31	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.49	1.93	25.6	10.2	OK	OK
		E70xx	1/8	3/16	8.94	0.31	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.52	1.44	36.2	14.8	OK	OK
Column-w 2	SP B2	E70xx	1/8	3/16	8.94	0.31	33 LRFD_{8.9}-1.2D+1.6S+0.5W(1)	0.51	1.93	26.3	11.9	OK	OK
		E70xx	1/8	3/16	8.94	0.31	26 LRFD_{8.2}-1.2D+1.6S+0.5W(17)	0.53	1.47	36.1	17.6	OK	OK
Top FP	Column	E70xx	1/8	3/16	12.72	0.08	32 LRFD_{8.8}-1.2D+1.6S+0.5W(21)	0.15	0.34	44.7	25.0	OK	OK

Design data

Material	F_{exx} [ksi]
E70xx	70.0

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
Ut_c	Weld capacity estimation
◢	Fillet weld
F_{exx}	Ultimate strength as rated by electrode classification number

Detailed result for Column-arc 5 / Bottom FP

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

\(\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =\space \) 0.41 kip ≥ \(F_n =\space \) 0.31 kip

Where:

\(F_{nw} =\space \)51.3 ksi – nominal stress of weld material:

- \(F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 +0.5 \cdot sin^{1.5} \theta)\space \) , where:
  - \(F_{EXX} =\space \) 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - \(\theta =\space \) 35.5° – angle of loading measured from the weld longitudinal axis

\(A_{we} =\space \)0.01 in^2 – effective area of weld critical element

\(\phi =\space \)0.75 – resistance factor for welded connections

Buckling

Buckling analysis was not calculated.

Project data

Project name	
Project number	
Author	
Description	
Date	5/28/2025
Code	AISC/ACI

Material

Steel	A500B-46, A992-50, A53B-35, A572 Gr.50, A36
Concrete	4000 psi

Project item Moment Frame Middle

Design

Name	Moment Frame Middle
Description	
Analysis	Stress, strain/ simplified loading
Design code	AISC 360-16 (LRFD) / ACI 318-14

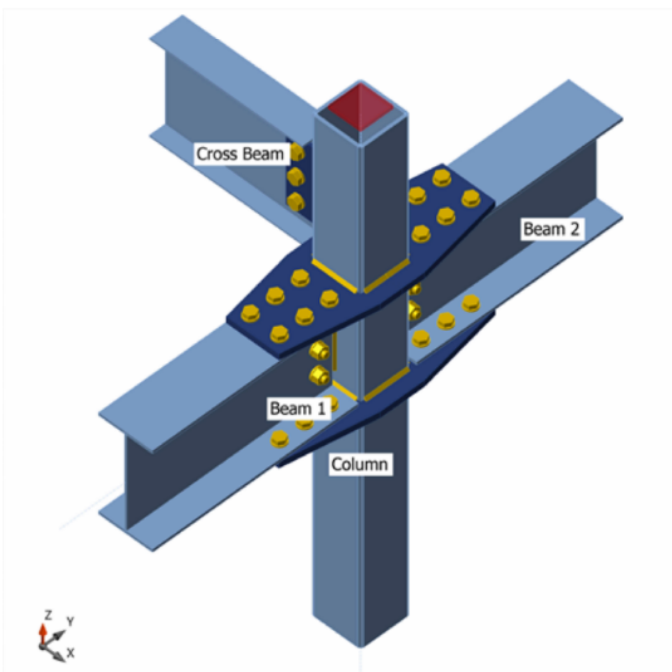
Members

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
Column	9 - HSS 6x6x3/8	0.0	90.0	0.0	0.00	0.00	0.00
Cross Beam	107 - W 12x14	0.0	0.0	0.0	0.00	0.00	18.05
Beam 1	22 - W 12x26	-90.0	0.0	0.0	0.00	0.00	5.78
Beam 2	22 - W 12x26	-90.0	0.0	0.0	0.00	0.00	5.78

Supports and forces

Name	Support	Forces in	X [in]
Column / begin	N-Vy-Vz-Mx-My-Mz	Bolts	0.00
Column / end	N-Vy-Vz-Mx-My-Mz	Bolts	0.00
Cross Beam / begin		Bolts	7.00
Beam 1 / end		Bolts	7.90
Beam 2 / begin		Bolts	7.90



Cross-sections

Name	Material
9 - HSS 6x6x3/8	A500B-46
107 - W 12x14	A992-50
22 - W 12x26	A992-50

Bolts

Name	Diameter [in]	f_y [ksi]	f_u [ksi]	Gross area [in^2]
3/4 A325	0.75	92.0	120.0	0.44

Load effects (Equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
30 LRFD_{8.6}-1.2D+1.6S+0.5W(1)	Cross Beam / Begin	-0.07	0.00	-2.97	-0.01	0.00	0.00
	Beam 1 / End	-0.05	0.19	-28.79	0.00	102.84	0.00
	Beam 2 / Begin	-0.18	0.44	-30.45	0.00	-98.46	0.00
19 LRFD_{7.9}-1.2D+1.6Lr+0.5W(21)	Cross Beam / Begin	0.04	0.00	-2.97	-0.01	0.00	0.00
	Beam 1 / End	0.37	0.21	-28.34	0.00	99.17	0.00
	Beam 2 / Begin	-0.15	0.46	-30.75	0.00	-101.38	0.00
36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	Cross Beam / Begin	-0.39	0.00	-2.97	-0.01	0.00	0.00
	Beam 1 / End	0.36	-0.30	-28.16	0.01	97.98	0.00
	Beam 2 / Begin	-0.39	-0.69	-30.79	0.00	-101.80	0.00
16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	Cross Beam / Begin	0.03	0.00	-2.97	-0.01	0.00	0.00
	Beam 1 / End	0.06	0.19	-28.94	0.00	103.68	0.00
	Beam 2 / Begin	-0.21	0.45	-30.45	0.00	-98.46	0.00
22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	Cross Beam / Begin	-0.29	0.00	-2.97	-0.01	0.00	0.00
	Beam 1 / End	0.46	-0.30	-28.31	0.01	98.82	0.00
	Beam 2 / Begin	-0.41	-0.68	-30.80	0.00	-101.80	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.8 < 5.0%	OK
Loc. deformation	0.2 < 3%	OK
Bolts	95.7 < 100%	OK
Welds	82.5 < 100%	OK
Buckling	Not calculated	
GMNA	Not calculated	

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Plates

Name	Material	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
Column	A500B-46	3/8	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	41.5	0.3	0.0	OK
Cross Beam-bfl 1	A992-50	1/4	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	3.5	0.0	0.0	OK
Cross Beam-tfl 1	A992-50	1/4	19 LRFD_{7.9}-1.2D+1.6Lr+0.5W(21)	3.5	0.0	0.0	OK
Cross Beam-w 1	A992-50	3/16	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	7.2	0.0	0.5	OK
Beam 1-bfl 1	A992-50	3/8	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	45.1	0.4	3.6	OK
Beam 1-tfl 1	A992-50	3/8	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	45.1	0.5	8.0	OK
Beam 1-w 1	A992-50	1/4	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	34.1	0.0	3.4	OK
Beam 2-bfl 1	A992-50	3/8	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	45.1	0.4	3.5	OK
Beam 2-tfl 1	A992-50	3/8	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	45.1	0.4	7.5	OK
Beam 2-w 1	A992-50	1/4	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	33.9	0.0	3.3	OK
SP Cross Beam	A36	3/8	19 LRFD_{7.9}-1.2D+1.6Lr+0.5W(21)	12.0	0.0	0.6	OK
SP Beam 1	A36	1/2	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	20.5	0.0	3.0	OK
SP Beam 2	A36	1/2	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	21.0	0.0	2.9	OK
TFP Beam 1	A36	3/4	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	32.6	0.8	8.0	OK
BFP	A36	3/4	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	32.5	0.2	3.5	OK
TFP Beam 2	A36	3/4	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	32.6	0.8	7.5	OK

Design data

Material	F_y [ksi]	ε_{lim} [%]
A500B-46	46.0	5.0
A992-50	50.0	5.0
A36	36.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain

Detailed result for TFP Beam 1

Design values used in the analysis

\(\phi F_{y} = \space \) 32.4 ksi

Where:

\(F_{y} = \space \)36.0 ksi – characteristic yield strength

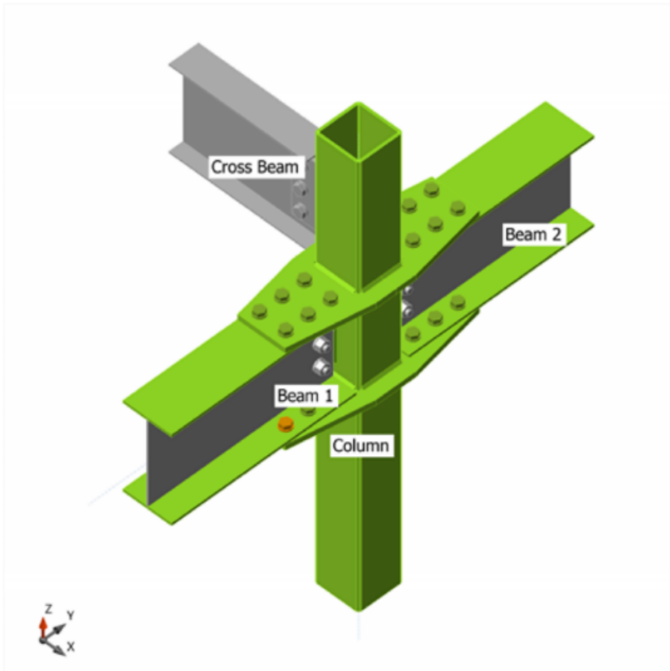
\(\phi = \space \)0.90 – resistance factor for steel material AISC 360-16 – B3.1

Loc. deformation

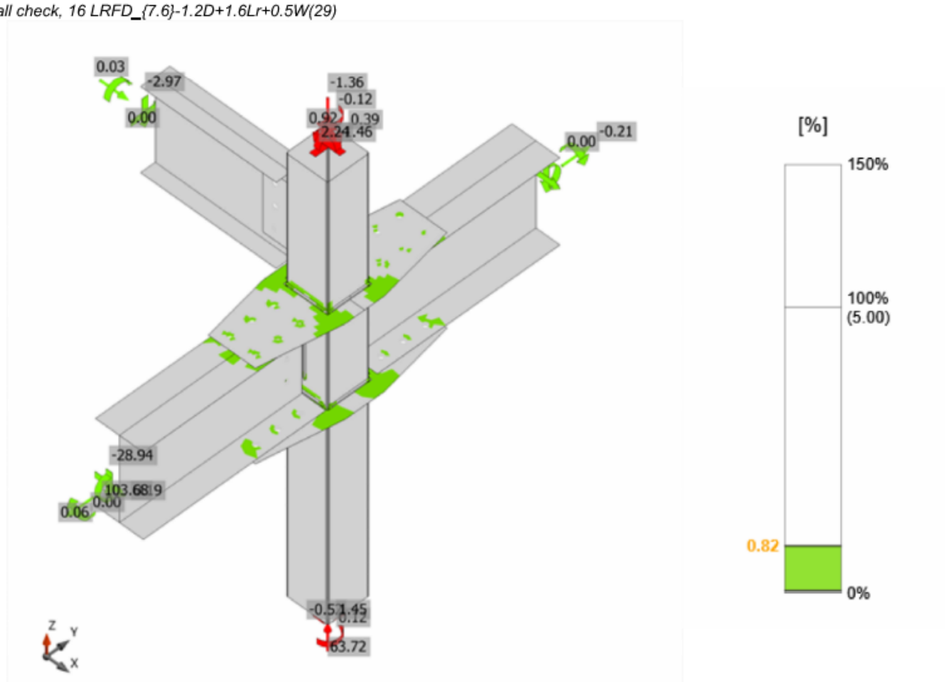
Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
Column	6.00	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	0.01	0.18	0.2	OK

Symbol explanation

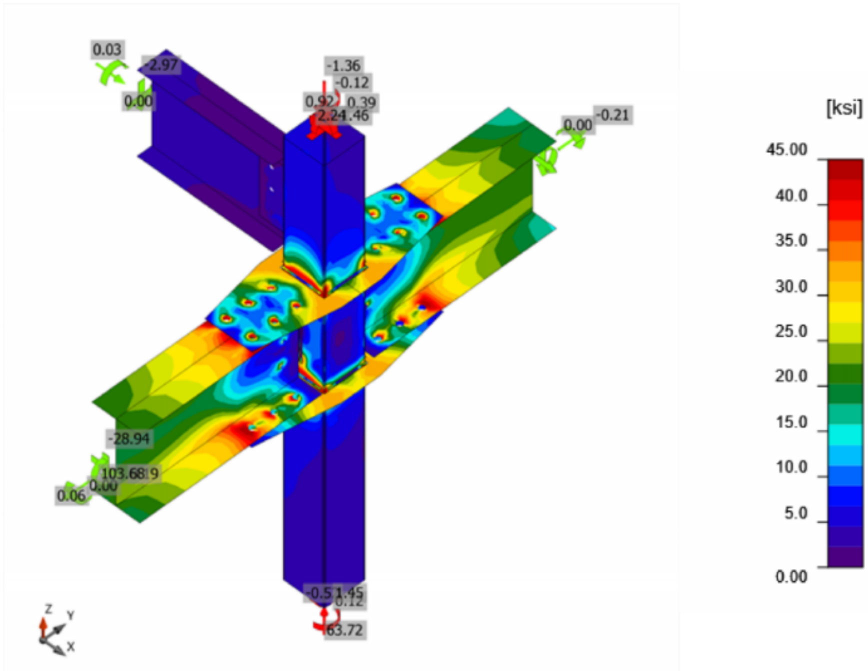
d_0	Cross-section size
δ	Local cross-section deformation
δ_{lim}	Allowed deformation



Overall check, 16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)



Strain check, 16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)



Equivalent stress, 16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)

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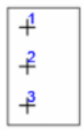
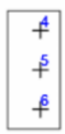
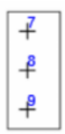
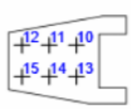
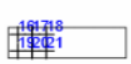
Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B1	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	0.33	0.99	17.55	1.1	5.6	0.0	OK	OK
	B2	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	0.14	1.01	17.55	0.5	5.8	0.0	OK	OK
	B3	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	0.14	1.00	17.55	0.5	5.7	0.0	OK	OK
	B4	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	1.33	5.83	20.18	4.5	32.6	0.0	OK	OK
	B5	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	0.63	3.30	20.18	2.1	18.5	0.0	OK	OK
	B6	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	0.93	5.31	20.18	3.1	29.7	0.0	OK	OK
	B7	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	1.28	5.85	20.18	4.3	32.7	0.0	OK	OK
	B8	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	0.66	3.49	20.18	2.2	19.5	0.0	OK	OK
	B9	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	0.93	5.34	20.18	3.1	29.9	0.0	OK	OK
	B10	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	3.89	16.35	33.34	13.1	91.5	0.0	OK	OK
	B11	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.58	16.56	33.34	8.6	92.6	0.0	OK	OK
	B12	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	4.49	16.84	33.34	15.1	94.2	0.0	OK	OK
	B13	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	4.01	16.33	33.34	13.5	91.4	0.0	OK	OK
	B14	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.48	16.53	33.34	8.3	92.4	0.0	OK	OK
	B15	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	4.39	16.81	33.34	14.7	94.0	0.0	OK	OK
	B16	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	1.93	16.40	33.34	6.5	91.7	0.0	OK	OK
	B17	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	1.91	16.66	33.34	6.4	93.2	0.0	OK	OK
	B18	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.53	17.10	33.34	8.5	95.7	0.0	OK	OK
	B19	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	1.94	16.38	33.34	6.5	91.6	0.0	OK	OK
	B20	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	1.88	16.65	33.34	6.3	93.1	0.0	OK	OK
	B21	3/4 A325 - 1	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.50	17.09	33.34	8.4	95.6	0.0	OK	OK

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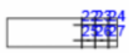
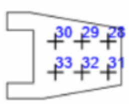
Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B22	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2.49	16.86	33.34	8.4	94.3	0.0	OK	OK
	B23	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	1.86	16.43	33.34	6.2	91.9	0.0	OK	OK
	B24	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	1.92	16.19	33.34	6.4	90.5	0.0	OK	OK
	B25	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2.49	16.82	33.34	8.4	94.1	0.0	OK	OK
	B26	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	1.80	16.42	33.34	6.0	91.8	0.0	OK	OK
	B27	3/4 A325 - 1	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	1.85	16.18	33.34	6.2	90.5	0.0	OK	OK
	B28	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	4.34	16.66	33.34	14.6	93.2	0.0	OK	OK
	B29	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	2.59	16.38	33.34	8.7	91.6	0.0	OK	OK
	B30	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	4.15	16.17	33.34	13.9	90.5	0.0	OK	OK
	B31	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	4.35	16.63	33.34	14.6	93.1	0.0	OK	OK
	B32	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	2.42	16.37	33.34	8.1	91.6	0.0	OK	OK
	B33	3/4 A325 - 1	22 LRFD_{7.12}-1.2D+1.6Lr+0.5W(33)	4.20	16.16	33.34	14.1	90.4	0.0	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
3/4 A325 - 1	29.79	17.88

Symbol explanation

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

Detailed result for B18

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

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 29.79 kip ≥ $F_t =$ 2.53 kip

Where:

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

$A_b =$ 0.44 in^2 – gross bolt cross-sectional area

$\phi =$ 0.75 – resistance factor

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

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 17.88 kip $\geq$ $V =$ 17.10 kip

Where:

$F_{nv} =$ 54.0 ksi – nominal shear stress AISC 360-16 – Table J3.2
$A_b =$ 0.44 in^2 – gross bolt cross-sectional area
$\phi =$ 0.75 – resistance factor

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

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

$\phi R_n =$ 33.34 kip $\geq$ $V =$ 17.10 kip

Where:

$l_c =$ 2.19 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
$t =$ 0.38 in – thickness of the plate
$d =$ 0.75 in – diameter of a bolt
$F_u =$ 65.0 ksi – specified minimum tensile strength of the connected material
$\phi =$ 0.75 – resistance factor for bearing at bolt holes

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

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

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Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Ut_c [%]	Detailing	Status
Column-w 3	SP Cross Beam	E70xx	◢ 3/16 ◣	◢ 1/4 ◣	8.95	0.37	19 LRFD_{7.9}-1.2D+1.6Lr+0.5W(21)	1.47	2.94	50.0	29.9	OK	OK
		E70xx	◢ 3/16 ◣	◢ 1/4 ◣	8.95	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	1.33	2.88	46.1	27.1	OK	OK
Column-w 4	SP Beam 1	E70xx	◢ 3/16 ◣	◢ 1/4 ◣	8.95	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.22	2.92	76.1	57.4	OK	OK
		E70xx	◢ 3/16 ◣	◢ 1/4 ◣	8.95	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.18	2.87	76.0	55.8	OK	OK
Column-w 2	SP Beam 2	E70xx	◢ 3/16 ◣	◢ 1/4 ◣	8.95	0.37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2.22	2.91	76.2	57.7	OK	OK
		E70xx	◢ 3/16 ◣	◢ 1/4 ◣	8.95	0.37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2.18	2.87	76.0	55.1	OK	OK
Column-w 1	TFP Beam 1	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.60	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.02	2.61	77.3	55.6	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.60	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.20	2.92	75.4	50.0	OK	OK
Column-w 3	TFP Beam 1	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.60	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.04	2.64	77.3	52.0	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.60	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.20	2.91	75.5	45.1	OK	OK
Column-w 4	TFP Beam 1	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	3.11	3.77	82.5	77.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.46	3.11	78.9	68.0	OK	OK
Column-w 1	BFP	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.22	2.86	77.4	58.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.16	2.68	80.5	71.4	OK	OK
Column-w 2	BFP	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2.38	3.03	78.4	66.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2.97	3.69	80.5	73.2	OK	OK
Column-w 3	BFP	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.20	2.84	77.6	59.1	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.16	2.68	80.6	71.4	OK	OK
Column-w 4	BFP	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.41	3.06	78.6	67.3	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	16 LRFD_{7.6}-1.2D+1.6Lr+0.5W(29)	2.97	3.68	80.7	73.6	OK	OK

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Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Ut_c [%]	Detailing	Status
Column-w 1	TFP Beam 2	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2,60	0,37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2,01	2,60	77,2	52,3	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2,60	0,37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2,18	2,90	75,3	43,7	OK	OK
Column-w 2	TFP Beam 2	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	3.10	3.78	82.1	75.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.20	0.37	19 LRFD_{7.9}-1.2D+1.6Lr+0.5W(21)	2.42	3.08	78.7	67.4	OK	OK
Column-w 3	TFP Beam 2	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.60	0.37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2.03	2.63	77.3	55.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.60	0.37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(23)	2.19	2.90	75.4	50.3	OK	OK
TFP Beam 2	TFP Beam 1	E70xx	-	-	1.75	-	-	-	-	-	-	OK	OK
TFP Beam 2	TFP Beam 1	E70xx	-	-	2,00	-	-	-	-	-	-	OK	OK

Design data

Material	F_{exx} [ksi]
E70xx	70,0

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
Ut_c	Weld capacity estimation
◢	Fillet weld
F_{exx}	Ultimate strength as rated by electrode classification number

Detailed result for Column-w 4 / TFP Beam 1

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

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.77 kip ≥ $F_n =$ 3.11 kip

Where:

$F_{nw} =$ 61.3 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot \sin^{1.5} \theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 70.7° – angle of loading measured from the weld longitudinal axis

$A_{we} =$ 0.08 in^2 – effective area of weld critical element

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

Buckling

Buckling analysis was not calculated.

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Project data

Project name	
Project number	
Author	
Description	
Date	5/28/2025
Code	AISC/ACI

Material

Steel	A500B-46, A992-50, A53B-35, A572 Gr.50, A36
Concrete	4000 psi

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Project item HSS6x6 on GL5

Design

Name	HSS6x6 on GL5
Description	
Analysis	Stress, strain/ simplified loading
Design code	AISC 360-16 (LRFD) / ACI 318-14

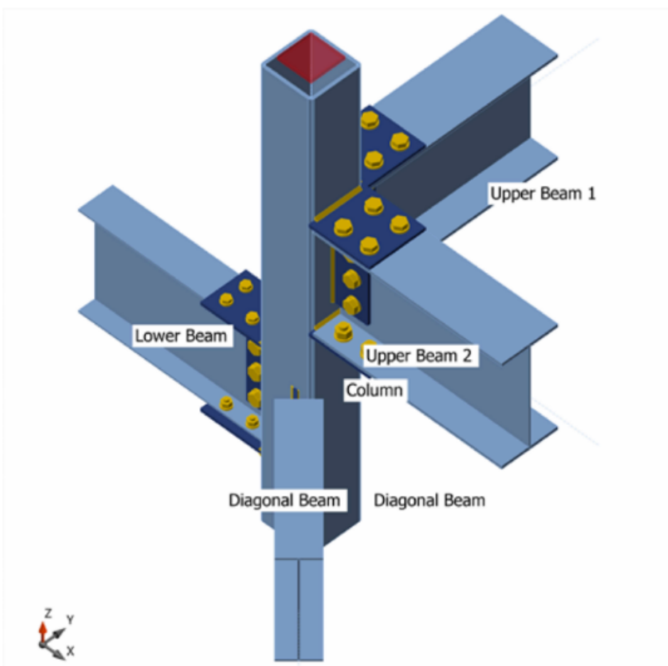
Members

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
Upper Beam 1	22 - W 12x26	90.0	0.0	0.0	0.00	0.00	5.78
Upper Beam 2	22 - W 12x26	0.0	0.0	0.0	0.00	0.00	5.78
Diagonal Beam	107 - W 12x14	-45.0	0.0	0.0	0.00	-1.00	-12.58
Column	9 - HSS 6x6x3/8	0.0	90.0	-90.0	0.00	0.00	0.00
Lower Beam	107 - W 12x14	0.0	0.0	0.0	0.00	0.00	-12.58

Supports and forces

Name	Support	Forces in	X [in]
Upper Beam 1 / end		Position	0.00
Upper Beam 2 / end		Position	0.00
Diagonal Beam / end		Position	0.00
Column / begin	N-Vy-Vz-Mx-My-Mz	Position	0.00
Column / end	N-Vy-Vz-Mx-My-Mz	Position	0.00
Lower Beam / begin		Position	0.00



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Cross-sections

Name	Material
22 - W 12x26	A992-50
107 - W 12x14	A992-50
9 - HSS 6x6x3/8	A500B-46

Bolts

Name	Diameter [in]	f_y [ksi]	f_u [ksi]	Gross area [in^2]
3/4 A325	0.75	92.0	120.0	0.44
1/2 A325	0.50	92.0	120.0	0.20

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Load effects (Equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
136 LRFD_{12.14}-0.9D+W(1)	Upper Beam 1 / End	-2.34	-0.04	-9.63	0.00	11.05	0.00
	Upper Beam 2 / End	0.18	0.00	-1.00	-0.02	4.82	0.00
	Diagonal Beam / End	-2.65	0.02	-1.81	0.01	0.00	0.00
	Lower Beam / Begin	1.89	0.00	-0.56	0.00	-1.68	0.00
74 LRFD_{9.8}-1.2D+L+0.5Lr+W(2)	Upper Beam 1 / End	1.54	-0.08	-14.46	-0.01	20.03	0.00
	Upper Beam 2 / End	-1.20	0.00	-2.01	-0.02	14.63	0.00
	Diagonal Beam / End	0.32	-0.14	-2.61	0.01	0.00	0.00
	Lower Beam / Begin	0.99	0.00	-0.47	0.00	0.96	0.00
126 LRFD_{12.4}-0.9D+W(10)	Upper Beam 1 / End	-1.49	-0.05	-5.72	0.00	0.60	0.00
	Upper Beam 2 / End	-0.32	0.01	-0.78	-0.01	3.99	0.00
	Diagonal Beam / End	-2.29	0.07	-1.26	0.00	0.00	0.00
	Lower Beam / Begin	1.95	0.00	-0.42	0.00	-0.99	0.00
61 LRFD_{8.37}-1.2D+1.6S+0.5W(11)	Upper Beam 1 / End	0.67	-0.01	-21.56	0.00	31.94	0.00
	Upper Beam 2 / End	0.18	0.00	-2.05	-0.03	10.28	0.00
	Diagonal Beam / End	0.28	-0.03	-4.11	0.01	0.00	0.00
	Lower Beam / Begin	-0.59	0.00	-1.20	0.00	-3.70	0.00
81 LRFD_{10.1}-1.2D+L+0.5S+W(3)	Upper Beam 1 / End	0.47	0.08	-13.23	0.01	17.65	0.00
	Upper Beam 2 / End	-0.73	0.00	-0.50	-0.02	-5.85	0.00
	Diagonal Beam / End	0.03	0.14	-2.47	0.28	0.00	0.00
	Lower Beam / Begin	-0.02	0.01	-1.70	0.37	-10.04	0.00
35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	Upper Beam 1 / End	0.63	-0.03	-20.76	0.00	33.97	0.00
	Upper Beam 2 / End	1.22	0.00	-1.80	-0.03	7.93	0.00
	Diagonal Beam / End	0.18	-0.02	-4.19	0.89	0.00	0.00
	Lower Beam / Begin	-1.76	0.00	-1.94	1.19	-6.31	0.00
129 LRFD_{12.7}-0.9D+W(8)	Upper Beam 1 / End	1.46	-0.08	-7.57	-0.01	10.43	0.00
	Upper Beam 2 / End	-1.05	0.00	-1.43	-0.01	12.13	0.00
	Diagonal Beam / End	0.38	-0.14	-1.36	0.00	0.00	0.00
	Lower Beam / Begin	0.88	0.00	-0.09	0.00	2.52	0.00
88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	Upper Beam 1 / End	1.49	-0.08	-14.50	-0.01	20.57	0.00
	Upper Beam 2 / End	-1.12	0.00	-2.01	-0.02	14.83	0.00
	Diagonal Beam / End	0.21	-0.14	-2.81	0.28	0.00	0.00
	Lower Beam / Begin	0.96	0.01	-0.71	0.37	0.57	0.00
123 LRFD_{12.1}-0.9D+W(5)	Upper Beam 1 / End	0.46	0.08	-7.52	0.01	9.23	0.00
	Upper Beam 2 / End	-0.69	0.00	-0.01	-0.01	-8.11	0.00
	Diagonal Beam / End	0.20	0.14	-1.28	0.01	0.00	0.00
	Lower Beam / Begin	-0.08	0.00	-1.15	0.00	-8.37	0.00
132 LRFD_{12.10}-0.9D+W(14)	Upper Beam 1 / End	1.22	-0.04	-9.32	0.00	20.12	0.00
	Upper Beam 2 / End	2.64	0.00	-0.83	-0.02	2.99	0.00
	Diagonal Beam / End	1.35	-0.03	-1.26	0.00	0.00	0.00
	Lower Beam / Begin	-3.85	0.00	-0.64	0.00	-2.98	0.00
72 LRFD_{9.6}-1.2D+L+0.5Lr+W(15)	Upper Beam 1 / End	-0.48	-0.05	-15.08	-0.01	16.40	0.00
	Upper Beam 2 / End	-1.32	0.00	-1.92	-0.02	12.98	0.00
	Diagonal Beam / End	-1.50	-0.06	-2.81	0.01	0.00	0.00
	Lower Beam / Begin	2.56	0.00	-0.57	0.00	0.21	0.00
133 LRFD_{12.11}-0.9D+W(18)	Upper Beam 1 / End	1.24	-0.04	-9.63	0.00	20.50	0.00
	Upper Beam 2 / End	2.64	0.00	-0.92	-0.02	3.34	0.00
	Diagonal Beam / End	1.36	-0.03	-1.62	0.01	0.00	0.00
	Lower Beam / Begin	-3.85	0.00	-0.66	0.00	-3.16	0.00
18 LRFD_{7.8}-1.2D+1.6Lr+0.5W(19)	Upper Beam 1 / End	0.92	-0.03	-20.20	0.00	28.08	0.00
	Upper Beam 2 / End	-0.88	0.00	-2.11	-0.03	11.91	0.00

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
	Diagonal Beam / End	0.04	-0.07	-3.57	0.01	0.00	0.00
	Lower Beam / Begin	0.72	0.00	-0.90	0.00	-2.34	0.00
82 LRFD_{10.2}-1.2D+L+0.5S+W(21)	Upper Beam 1 / End	0.49	0.08	-14.33	0.01	19.16	0.00
	Upper Beam 2 / End	-0.76	0.00	-0.63	-0.02	-5.42	0.00
	Diagonal Beam / End	0.03	0.14	-2.81	0.28	0.00	0.00
	Lower Beam / Begin	0.00	0.01	-1.72	0.37	-10.25	0.00
94 LRFD_{10.14}-1.2D+L+0.5S+W(23)	Upper Beam 1 / End	-2.33	-0.04	-15.34	0.00	19.47	0.00
	Upper Beam 2 / End	0.14	0.00	-1.50	-0.03	7.09	0.00
	Diagonal Beam / End	-2.82	0.02	-3.00	0.28	0.00	0.00
	Lower Beam / Begin	1.96	0.00	-1.11	0.37	-3.35	0.00
33 LRFD_{8.9}-1.2D+1.6S+0.5W(26)	Upper Beam 1 / End	0.52	-0.02	-21.23	0.00	33.71	0.00
	Upper Beam 2 / End	0.41	0.00	-2.04	-0.03	11.06	0.00
	Diagonal Beam / End	-0.07	-0.03	-4.29	0.89	0.00	0.00
	Lower Beam / Begin	-0.71	0.00	-1.81	1.19	-4.76	0.00
36 LRFD_{8.12}-1.2D+1.6S+0.5W(28)	Upper Beam 1 / End	0.23	0.00	-21.19	0.00	33.42	0.00
	Upper Beam 2 / End	0.41	0.00	-1.57	-0.03	4.21	0.00
	Diagonal Beam / End	-0.27	0.01	-4.29	0.89	0.00	0.00
	Lower Beam / Begin	-0.80	0.00	-2.16	1.19	-8.42	0.00
89 LRFD_{10.9}-1.2D+L+0.5S+W(25)	Upper Beam 1 / End	1.03	-0.03	-16.29	-0.01	28.40	0.00
	Upper Beam 2 / End	0.98	0.00	-1.89	-0.03	11.86	0.00
	Diagonal Beam / End	0.70	-0.07	-3.00	0.28	0.00	0.00
	Lower Beam / Begin	-1.68	0.00	-0.95	0.37	-1.74	0.00
77 LRFD_{9.11}-1.2D+L+0.5Lr+W(34)	Upper Beam 1 / End	1.30	-0.04	-15.29	0.00	28.39	0.00
	Upper Beam 2 / End	2.52	0.00	-1.42	-0.02	5.40	0.00
	Diagonal Beam / End	1.30	-0.03	-2.61	0.01	0.00	0.00
	Lower Beam / Begin	-3.75	0.00	-0.96	0.00	-4.43	0.00
86 LRFD_{10.6}-1.2D+L+0.5S+W(39)	Upper Beam 1 / End	-0.53	-0.05	-15.12	-0.01	16.94	0.00
	Upper Beam 2 / End	-1.25	0.00	-1.92	-0.03	13.18	0.00
	Diagonal Beam / End	-1.61	-0.06	-3.00	0.28	0.00	0.00
	Lower Beam / Begin	2.52	0.00	-0.81	0.37	-0.18	0.00
134 LRFD_{12.12}-0.9D+W(40)	Upper Beam 1 / End	0.44	0.01	-10.50	0.01	19.40	0.00
	Upper Beam 2 / End	1.03	0.00	-0.46	-0.02	-4.10	0.00
	Diagonal Beam / End	0.47	0.02	-1.81	0.01	0.00	0.00
	Lower Beam / Begin	-1.93	0.00	-1.08	0.00	-7.40	0.00
21 LRFD_{7.11}-1.2D+1.6Lr+0.5W(32)	Upper Beam 1 / End	0.80	-0.01	-20.62	0.00	32.26	0.00
	Upper Beam 2 / End	0.98	0.00	-1.81	-0.03	7.30	0.00
	Diagonal Beam / End	0.54	-0.02	-3.57	0.01	0.00	0.00
	Lower Beam / Begin	-1.65	0.00	-1.15	0.00	-5.04	0.00
31 LRFD_{8.7}-1.2D+1.6S+0.5W(30)	Upper Beam 1 / End	0.74	-0.04	-19.73	0.00	28.93	0.00
	Upper Beam 2 / End	-0.62	0.00	-2.06	-0.03	12.33	0.00
	Diagonal Beam / End	-0.32	-0.07	-4.06	0.89	0.00	0.00
	Lower Beam / Begin	0.60	0.00	-1.66	1.19	-3.47	0.00
130 LRFD_{12.8}-0.9D+W(35)	Upper Beam 1 / End	1.48	-0.08	-8.79	-0.01	12.15	0.00
	Upper Beam 2 / End	-1.08	0.00	-1.52	-0.02	12.57	0.00
	Diagonal Beam / End	0.38	-0.14	-1.62	0.00	0.00	0.00
	Lower Beam / Begin	0.90	0.00	-0.16	0.00	2.24	0.00
92 LRFD_{10.12}-1.2D+L+0.5S+W(33)	Upper Beam 1 / End	0.45	0.00	-16.21	0.01	27.82	0.00
	Upper Beam 2 / End	0.98	0.00	-0.96	-0.03	-1.84	0.00
	Diagonal Beam / End	0.30	0.02	-3.00	0.29	0.00	0.00
	Lower Beam / Begin	-1.87	0.00	-1.64	0.37	-9.06	0.00
114 LRFD_{10.34}-1.2D+L+0.5S+W(41)	Upper Beam 1 / End	-0.48	-0.05	-15.23	-0.01	16.38	0.00

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Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 257
Project Name: Express Stop – View High		

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
	Upper Beam 2 / End	-1.32	0.00	-1.92	-0.02	12.94	0.00
	Diagonal Beam / End	-1.50	-0.06	-2.95	0.01	0.00	0.00
	Lower Beam / Begin	2.56	0.00	-0.62	0.00	0.15	0.00
26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	Upper Beam 1 / End	0.25	0.04	-20.25	0.01	29.09	0.00
	Upper Beam 2 / End	-0.46	0.00	-1.41	-0.03	2.42	0.00
	Diagonal Beam / End	-0.40	0.07	-4.19	0.89	0.00	0.00
	Lower Beam / Begin	0.13	0.00	-2.20	1.19	-9.02	0.00
38 LRFD_{8.14}-1.2D+1.6S+0.5W(47)	Upper Beam 1 / End	-1.16	-0.02	-20.76	0.00	29.24	0.00
	Upper Beam 2 / End	-0.01	0.00	-1.84	-0.03	8.67	0.00
	Diagonal Beam / End	-1.83	0.01	-4.29	0.89	0.00	0.00
	Lower Beam / Begin	1.11	0.00	-1.89	1.19	-5.57	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	1.2 < 5.0%	OK
Loc. deformation	0.2 < 3%	OK
Bolts	52.9 < 100%	OK
Welds	84.8 < 100%	OK
Buckling	Not calculated	
GMNA	Not calculated	

Plates

Name	Material	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
Upper Beam 1-bfl 1	A992-50	3/8	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	21.9	0.0	7.8	OK
Upper Beam 1-tfl 1	A992-50	3/8	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	23.1	0.0	2.7	OK
Upper Beam 1-w 1	A992-50	1/4	61 LRFD_{8.37}-1.2D+1.6S+0.5W(11)	34.5	0.0	8.3	OK
Upper Beam 2-bfl 1	A992-50	3/8	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	11.5	0.0	0.2	OK
Upper Beam 2-tfl 1	A992-50	3/8	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	12.0	0.0	0.9	OK
Upper Beam 2-w 1	A992-50	1/4	82 LRFD_{10.2}-1.2D+L+0.5S+W(21)	6.2	0.0	1.5	OK
Diagonal Beam-bfl 1	A992-50	1/4	38 LRFD_{8.14}-1.2D+1.6S+0.5W(47)	33.9	0.0	0.0	OK
Diagonal Beam-tfl 1	A992-50	1/4	38 LRFD_{8.14}-1.2D+1.6S+0.5W(47)	44.2	0.1	0.0	OK
Diagonal Beam-w 1	A992-50	3/16	38 LRFD_{8.14}-1.2D+1.6S+0.5W(47)	45.0	0.1	12.1	OK
Column	A500B-46	3/8	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	41.5	0.3	0.0	OK
Lower Beam-bfl 1	A992-50	1/4	31 LRFD_{8.7}-1.2D+1.6S+0.5W(30)	45.0	0.0	2.1	OK
Lower Beam-tfl 1	A992-50	1/4	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	45.0	0.1	2.7	OK
Lower Beam-w 1	A992-50	3/16	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	34.5	0.0	3.1	OK
Diagonal SP	A36	3/8	38 LRFD_{8.14}-1.2D+1.6S+0.5W(47)	32.5	0.4	25.1	OK
Upper 1 SP	A36	3/8	33 LRFD_{8.9}-1.2D+1.6S+0.5W(26)	18.3	0.0	1.1	OK
Lower SP	A36	3/8	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	27.7	0.0	9.6	OK
Upper 2 SP	A36	3/8	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	4.3	0.0	0.5	OK
Lower MP bfl	A36	3/8	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	32.6	0.8	3.7	OK
Lower MP tfl	A36	3/8	36 LRFD_{8.12}-1.2D+1.6S+0.5W(28)	32.6	0.7	3.9	OK
Upper 2 T-fl	A36	3/8	36 LRFD_{8.12}-1.2D+1.6S+0.5W(28)	32.5	0.4	0.2	OK
Upper 1 T-fl	A36	3/8	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	32.7	1.2	3.5	OK
Upper 2 B-fl	A36	3/8	36 LRFD_{8.12}-1.2D+1.6S+0.5W(28)	32.5	0.4	0.9	OK
Upper 1 B-fl	A36	3/8	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	32.7	1.2	12.9	OK

Design data

Material	F_y [ksi]	ε_{lim} [%]
A992-50	50.0	5.0
A500B-46	46.0	5.0
A36	36.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain

Detailed result for Upper 1 T-fl

Design values used in the analysis

\(\phi F_{y} = \space \) 32.4 ksi

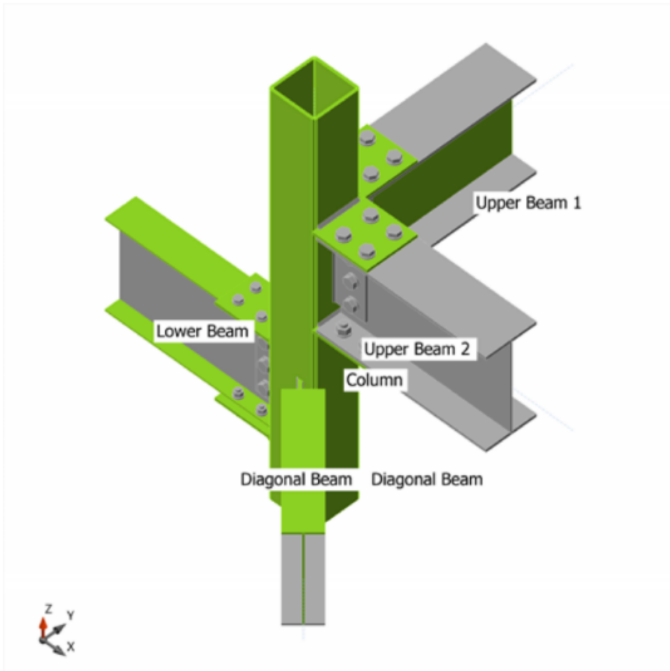
Where:
\(F_{y} = \space \)36.0 ksi – characteristic yield strength
\(\phi = \space \)0.90 – resistance factor for steel material AISC 360-16 – B3.1

Loc. deformation

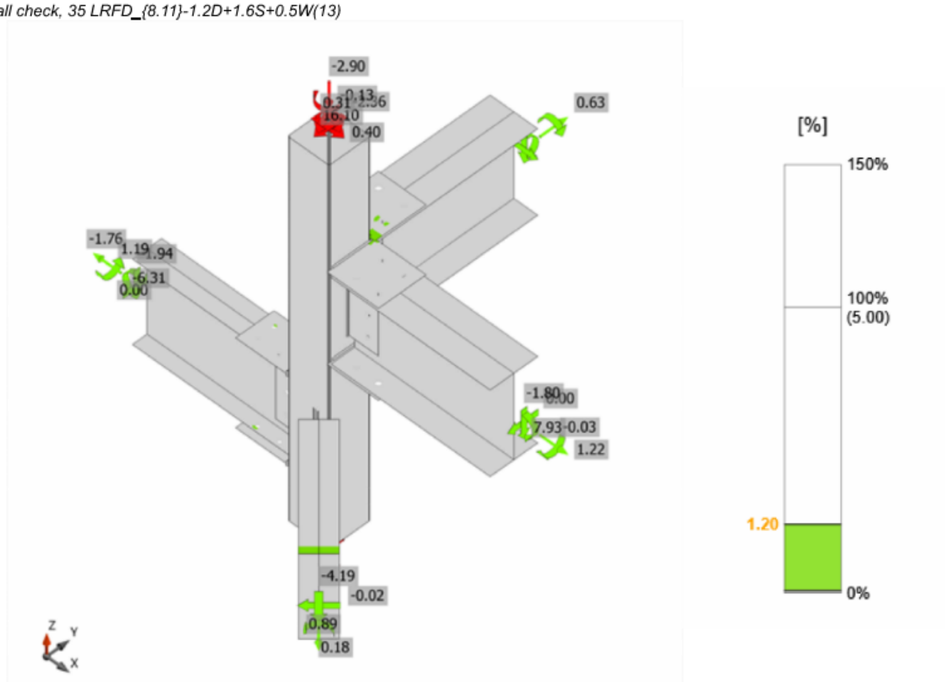
Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
Column	6.00	36 LRFD_{8.12}-1.2D+1.6S+0.5W(28)	0.01	0.18	0.2	OK

Symbol explanation

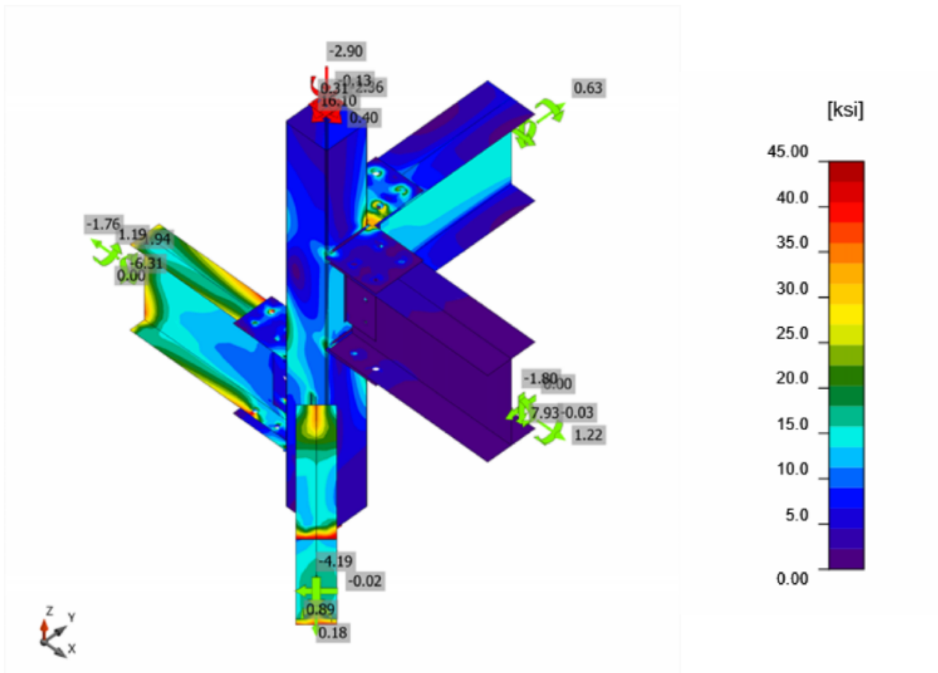
d_0	Cross-section size
δ	Local cross-section deformation
δ_{lim}	Allowed deformation



Overall check, 35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)



Strain check, 35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)



Equivalent stress, 35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)

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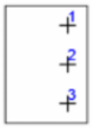
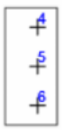
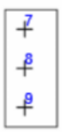
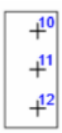
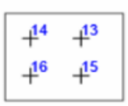
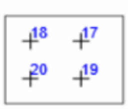
Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B1	3/4 A325 - 1	38 LRFD_{8.14}-1.2D+1.6S+0.5W(47)	8.27	6.13	17.55	27.8	34.9	0.0	OK	OK
	B2	3/4 A325 - 1	38 LRFD_{8.14}-1.2D+1.6S+0.5W(47)	1.22	1.51	17.55	4.1	8.6	0.0	OK	OK
	B3	3/4 A325 - 1	61 LRFD_{8.37}-1.2D+1.6S+0.5W(11)	1.10	5.52	17.55	3.7	31.4	0.0	OK	OK
	B4	3/4 A325 - 1	33 LRFD_{8.9}-1.2D+1.6S+0.5W(26)	0.45	3.21	20.18	1.5	18.0	0.0	OK	OK
	B5	3/4 A325 - 1	61 LRFD_{8.37}-1.2D+1.6S+0.5W(11)	0.35	2.84	20.18	1.2	15.9	0.0	OK	OK
	B6	3/4 A325 - 1	33 LRFD_{8.9}-1.2D+1.6S+0.5W(26)	0.49	2.98	20.18	1.6	16.7	0.0	OK	OK
	B7	3/4 A325 - 1	31 LRFD_{8.7}-1.2D+1.6S+0.5W(30)	2.61	0.37	17.55	8.8	2.1	0.0	OK	OK
	B8	3/4 A325 - 1	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	0.25	0.51	17.55	0.8	2.9	0.0	OK	OK
	B9	3/4 A325 - 1	82 LRFD_{10.2}-1.2D+L+0.5S+W(21)	0.65	1.38	17.55	2.2	7.8	0.0	OK	OK
	B10	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.25	1.04	20.18	0.8	5.8	0.0	OK	OK
	B11	3/4 A325 - 1	33 LRFD_{8.9}-1.2D+1.6S+0.5W(26)	0.06	0.28	20.18	0.2	1.6	0.0	OK	OK
	B12	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.14	1.03	20.18	0.5	5.8	0.0	OK	OK
	B13	1/2 A325 - 2	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	0.48	3.65	13.16	3.6	45.9	0.0	OK	OK
	B14	1/2 A325 - 2	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	0.90	4.20	13.16	6.8	52.9	0.0	OK	OK
	B15	1/2 A325 - 2	123 LRFD_{12.1}-0.9D+W(5)	0.22	1.68	13.16	1.6	21.2	0.0	OK	OK
	B16	1/2 A325 - 2	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	0.56	2.36	5.79	4.2	40.8	0.0	OK	OK
	B17	1/2 A325 - 2	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	0.79	3.81	13.16	6.0	48.0	0.0	OK	OK
	B18	1/2 A325 - 2	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	0.62	4.07	13.16	4.7	51.2	0.0	OK	OK
	B19	1/2 A325 - 2	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	0.42	1.88	5.81	3.2	32.4	0.0	OK	OK
	B20	1/2 A325 - 2	31 LRFD_{8.7}-1.2D+1.6S+0.5W(30)	0.95	2.61	13.16	7.1	32.9	0.0	OK	OK

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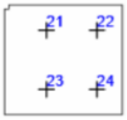
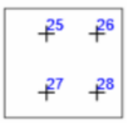
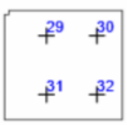
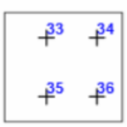
Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 262
Project Name: Express Stop – View High		

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B21	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.39	3.16	29.39	1.3	17.7	0.0	OK	OK
	B22	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.68	3.14	21.45	2.3	17.6	0.0	OK	OK
	B23	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.27	2.98	29.39	0.9	16.7	0.0	OK	OK
	B24	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.60	2.96	21.43	2.0	16.6	0.0	OK	OK
	B25	3/4 A325 - 1	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	2.87	5.92	29.39	9.6	33.1	0.0	OK	OK
	B26	3/4 A325 - 1	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	0.98	5.70	21.43	3.3	31.9	0.0	OK	OK
	B27	3/4 A325 - 1	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	3.13	6.47	29.39	10.5	36.2	0.0	OK	OK
	B28	3/4 A325 - 1	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	1.15	6.28	21.48	3.9	35.2	0.0	OK	OK
	B29	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.02	3.43	29.39	0.1	19.2	0.0	OK	OK
	B30	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.10	3.42	29.39	0.3	19.1	0.0	OK	OK
	B31	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.03	3.27	29.39	0.1	18.3	0.0	OK	OK
	B32	3/4 A325 - 1	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.09	3.26	29.39	0.3	18.2	0.0	OK	OK
	B33	3/4 A325 - 1	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	0.15	5.80	29.39	0.5	32.4	0.0	OK	OK
	B34	3/4 A325 - 1	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	0.15	5.50	29.39	0.5	30.8	0.0	OK	OK
	B35	3/4 A325 - 1	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	0.12	6.52	29.39	0.4	36.5	0.0	OK	OK
	B36	3/4 A325 - 1	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	0.17	6.28	29.39	0.6	35.1	0.0	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
3/4 A325 - 1	29.79	17.88
1/2 A325 - 2	13.24	7.95

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Made by: RAH	Date: 06/16/2025	Job No. 25-011
Revision:	Date:	Sheet No. 263
Project Name: Express Stop – View High		

Symbol explanation

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

Detailed result for B14

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

$\phi R_n = \phi \cdot F_{nt} \cdot A_b = $ 13.24 kip ≥ $F_t = $ 0.90 kip

Where:

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

$A_b = $ 0.20 in^2 – gross bolt cross-sectional area

$\phi = $ 0.75 – resistance factor

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

$\phi R_n = \phi \cdot F_{nv} \cdot A_b = $ 7.95 kip ≥ $V = $ 4.20 kip

Where:

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

$A_b = $ 0.20 in^2 – gross bolt cross-sectional area

$\phi = $ 0.75 – resistance factor

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

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

$\phi R_n = $ 13.16 kip ≥ $V = $ 4.20 kip

Where:

$l_c = $ 4.93 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

$t = $ 0.23 in – thickness of the plate

$d = $ 0.50 in – diameter of a bolt

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

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

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

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

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Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Ut_c [%]	Detailing	Status
Column-w 1	Diagonal SP	E70xx	◢ 1/8 ◣	◢ 3/16 ◣	8.95	0.37	38 LRFD_{8.14}-1.2D+1.6S+0.5W(47)	0.86	2.26	38.2	21.5	OK	OK
		E70xx	◢ 1/8 ◣	◢ 3/16 ◣	8.95	0.37	36 LRFD_{8.12}-1.2D+1.6S+0.5W(28)	0.86	1.77	48.6	28.5	OK	OK
Column-w 3	Upper 1 SP	E70xx	◢ 1/8 ◣	◢ 3/16 ◣	8.95	0.37	33 LRFD_{8.9}-1.2D+1.6S+0.5W(26)	1.57	2.08	75.5	51.9	OK	OK
		E70xx	◢ 1/8 ◣	◢ 3/16 ◣	8.95	0.37	61 LRFD_{8.37}-1.2D+1.6S+0.5W(11)	1.59	2.11	75.3	50.4	OK	OK
Column-w 4	Lower SP	E70xx	◢ 1/8 ◣	◢ 3/16 ◣	8.95	0.37	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	1.46	2.33	62.4	38.5	OK	OK
		E70xx	◢ 1/8 ◣	◢ 3/16 ◣	8.95	0.37	92 LRFD_{10.12}-1.2D+L+0.5S+W(33)	0.70	2.08	33.9	14.4	OK	OK
Column-w 2	Upper 2 SP	E70xx	◢ 1/8 ◣	◢ 3/16 ◣	8.95	0.37	123 LRFD_{12.1}-0.9D+W(5)	0.43	1.97	21.5	8.7	OK	OK
		E70xx	◢ 1/8 ◣	◢ 3/16 ◣	8.95	0.37	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	0.44	1.91	22.9	8.3	OK	OK
Column-w 4	Lower MP bfl	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.22	0.35	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	2.57	3.39	75.7	47.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.22	0.35	26 LRFD_{8.2}-1.2D+1.6S+0.5W(44)	2.55	3.36	75.8	48.8	OK	OK
Column-w 4	Lower MP tfl	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.22	0.35	36 LRFD_{8.12}-1.2D+1.6S+0.5W(28)	2.52	3.33	75.7	47.1	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.22	0.35	36 LRFD_{8.12}-1.2D+1.6S+0.5W(28)	2.54	3.36	75.7	47.2	OK	OK
Column-w 2	Upper 2 T-fl	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.19	0.35	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	2.65	3.50	75.9	44.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.19	0.35	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	2.63	3.49	75.4	42.0	OK	OK
Column-w 3	Upper 1 T-fl	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.19	0.35	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	2.98	3.52	84.8	80.6	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.19	0.35	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	2.87	3.50	82.0	73.8	OK	OK
Column-w 2	Upper 2 B-fl	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.19	0.35	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	2.64	3.49	75.8	43.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.19	0.35	88 LRFD_{10.8}-1.2D+L+0.5S+W(4)	2.66	3.49	76.4	46.1	OK	OK
Column-w 3	Upper 1 B-fl	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.19	0.35	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	2.88	3.51	81.9	73.9	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.19	0.35	35 LRFD_{8.11}-1.2D+1.6S+0.5W(13)	2.97	3.51	84.8	81.5	OK	OK

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Ut_c [%]	Detailing	Status
Upper 1 T-fl	Upper 2 T-fl	E70xx	-	-	0.25	-	-	-	-	-	-	OK	OK
Upper 1 T-fl	Upper 2 T-fl	E70xx	-	-	0.26	-	-	-	-	-	-	OK	OK
Upper 1 B-fl	Upper 2 B-fl	E70xx	-	-	0.25	-	-	-	-	-	-	OK	OK
Upper 1 B-fl	Upper 2 B-fl	E70xx	-	-	0.26	-	-	-	-	-	-	OK	OK

Design data

Material	F_{exx} [ksi]
E70xx	70.0

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
Ut_c	Weld capacity estimation
◢	Fillet weld
F_{exx}	Ultimate strength as rated by electrode classification number

Detailed result for Column-w 3 / Upper 1 T-fl

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

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.52 kip ≥ $F_n =$ 2.98 kip

Where:

$F_{nw} =$ 61.3 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot \sin^{1.5} \theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 70.8° – angle of loading measured from the weld longitudinal axis

$A_{we} =$ 0.08 in^2 – effective area of weld critical element

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

Buckling

Buckling analysis was not calculated.