GreyFox
CONSULTING, Inc.

Structural & Civil
Engineering Solutions

FEASIBILITY REPORT

Date: November 27, 2020

To: Ben Aparo and Nate Greenberg
Summit Ridge Energy

From: Ramin Abbaszadeh, PE

Re: 150 Mid Atalnatic Parkway
West Deptford, New Jersey

Subject: Roof Capacity Evaluation

Project No.: 20-039

General

As requested, Grey Fox Consulting Engineering (GF) evaluated the above-referenced building's roof structural capacity. The purpose of this evaluation was to determine if the existing roof could support the additional load from solar arrays. The building was constructed in the mid-1960s to early 1970s. There are no drawings available that we could rely on for the structural framing information. We conducted a site visit to measure the roof deck, steel bar joists, steel joist girders, and columns that constitute this building's roof framing.

We used the following documents for our analysis:

- International Building Code 2018 edition (IBC 108) is the current code adopted by New Jersey.
- IBC 2018 references American Society of Civil Engineers Standard 7-16, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-16).
- We referred to the Steel Joist Institutes (SJI) Historical Data to obtain the existing steel bar joists' capacity.

The subject building is a rectangular one-story, high-roof, steel structure (see pictures 1 through 5), with a small one-story office area with a lower roof to the east of the high-roof building. The high-roof building has approximately 168 ft length in the north-south direction and 160 ft width in the east-west direction. The roof structure is comprised of a steel deck supported by steel bar joists. The joists, in turn, are supported on steel girders and columns. Non-bearing masonry walls surround the perimeter of this

building. We field measured only the high roof structure, as this roof is only being considered for installing the solar panels.

As shown in the pictures from inside the building, the roof structure supports very little mechanical and electrical equipment. The building has a sprinkler system for fire protection.

Criteria and Assumptions:

We based our structural analysis on the following criteria and assumptions:

- Structural steel beams and columns with a yield stress of 36 ksi per historical data
- Roof loads:
  - Dead Load: Steel decking, insulation & membrane (TPO) roofing = 5 pounds per square foot (psf) was assumed
  - Dead Load: Sprinkler system = 2 psf was assumed
  - Dead Load: Collateral (lights, piping, ductwork, electrical) = 3 psf was assumed for future application
  - Dead load: Structural joists = 1 psf from historical data
  - Dead load: Structural girders—as calculated in our modeling of the truss
  - No ceilings in the area that we measured the roof framing
  - Roof Live Load: 20 psf per Code
  - Snow Load: 20 psf per Code.
  - Building code prescribes to use Live Load or Snow Load, but not both simultaneously
  - Building code prescribes to use Live Load or Solar Panel load, but not both simultaneously
  - Existing total load = 31 psf + steel weight
- Solar Array Dead Load (including accessories, attachments & ballast blocks) assumed to range between 2 to 12 psf.

Snowdrift Loads

We have not considered snowdrift for this roof because of the roof composition.

Analysis of Roof Structure

BAR JOISTS

The steel bar joists are running from north to south with an approximate span of 24 ft. The bar joists are supported on the steel joists girders, except at the north wall, where steel beams spanning approximately 20 feet between steel columns and supporting the roof bar joists.

The approximate spacing between bar joists is 6'-8". Based on our field measurement, the joists are 14-inch-deep with 1.4 inches by 1.4 inches double angles bottom chords and 1.5 inches by 1.5 inches double angles top chords. The web members are comprised of 0.6" diameter round bars.

The historical data from SJI designates bar joists with similar cord and web elements as 14J6 or 14H6 joists. The J series was in effect from 1961 to 1978 with the steel strength of 22,000 psi. The H series was fabricated between 1961 and 1986 with the steel strength of 30,000 psi. Because of the difference in steel strength, 14J6 has a lower capacity compared to 14H6. The table below shows the total load for each joist taken from the historical table:

Joist Designation	Total Uniform Load (Pounds per Linear Foot plf)
14J6	184
14H6	245

Based on the assumption of 31 psf dead and snow load and the joist spacing of 6'-8", each joist's uniform load would be 206.77 plf. From the table above, we can conclude that 14J6 has no capacity left for solar panels and would already be overstressed under the assumed load. In comparison, 14H6 is stronger and can support about 5.7 psf load imposed from solar panels. Based on this comparison, it is most likely that the current roof joists are 14H6 with a steel strength of 30,000 psi. Samples of existing joists must be tested to confirm our assumption.

ROOF DECK

The roof deck is a 1.5-inch type F narrow rib (see picture 5). We were not able to measure the thickness of the deck. Based on the joist spacing, it is most likely that the deck is 20 gage with a minimum of three continuous spans. Under these assumptions, the deck can carry more than 50 psf uniform loads. The deck's total uniform load (deck, roofing, insulation, and snow load), plus 12 psf for solar panel load, is approximately 37 psf. The deck capacity can support the additional load from solar panels.

STEEL GIRDERS

The steel girders supporting the roof joists comprise of steel "WT" section for top and bottom chords. Double or single angles are used for the web members. The panel points are at 6'-8", matching the joist spacing. The joist girders span 40 feet from column to column. Based on our field measurements, we determined that the joist girders constructed of the following steel members:

Top Chord	Continuous WT7x17
Bottom Chord	Continuous WT6x13.5
Web 1 at each end	(2) L 2 ½ x 2 x ¼
Web 2 vertical	L 2 ½ x 2 x ¼ or (2) L 2 ½ x 2 x ¼ varies
Web 3 interior diagonals	(2) L 3 x 2 x ¼
Web 4 diagonals at mid-point	(2) L 2 x 1 ½ x 1/8

In our analysis of the steel girder based on the above configuration, we have determined that the girders have sufficient capacity to support solar panels up to 12 pounds per square foot uniform load. Detailed calculations of our analysis are attached to this report.

STEEL COLUMNS

The majority of the columns are W8x31 with an approximate length of 26 feet. These columns have sufficient capacity to support the solar panel weight up to 12 psf.

Along the east and west wall of the building, there are columns that their dimensions represent W8x28. These columns are also capable of supporting additional load from solar panels up to 12 psf.

STEEL BEAMS

Along the northern exterior wall, there is a line of steel beams supported on the steel columns with a span of 20 feet. Our measurements indicate that these beams are W10x15. These beams also have sufficient capacity to support an additional load of 12 psf from solar panels.

Conclusions

Our professional opinion is that this building's roof structure can support solar panels up to 5 psf, if the roof joists are fabricated of 30,000 psi steel. If this building is selected for further investigation, samples of roof joists' steel must be collected and tested to determine the steel's strength.

The existing column footings must also be investigated to ensure they are of the size that could support additional loads from the solar panels. A geotechnical engineer should be consulted for the best way to explore the columns' foundation size.

GF did not evaluate the overall structure to determine if the building systems, foundations, and lateral resisting elements conform to the current building codes' requirements.

If we can be of further assistance in this matter, please contact us.

Sincerely,
Grey Fox Consulting, Inc.

Ramin Abbaszadeh, PE, SE
Principal

Picture 1

Picture 2

Picture 3

Picture 4

Picture 5

SUMMIT RIDGE ENERGY

150 Mid Atlantic Parkway
West Deptford, New Jersey

Roof Evaluation
Structural Calculations

GreyFox CONSULTING, Inc.

Structural & Civil
Engineering Solutions

DATE: 11/27/2020
Project No. 20-035
Prepared by: Ramin Abbaszadeh, PE, SE
NJ Lic. No. 34277

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JOB 150 MID ATLANTIC PKWY
SHEET NO. 1 OF
CALCULATED BY RA DATE 11/19/2020
CHECKED BY DATE
SCALE

BUILDING LENGTH 24' x 7 = 168'
BUILDING WIDTH 40' x 4 = 140'

ROOF DL

DECK/INSUL/ROOFING	=	5 PSF
SPRINKLER	=	2 PSF
MISC.	=	3 PSF
JOIST	=	1 PSF
TOTAL		11 PSF

ROOF SNOW = 20 PSF

JOIST SPACING = 6'-8"

TOTAL LOAD = 11 + 20 = 31 PSF

31(6.67) = 206.77 PLF

JOIST DESIGNATION 14J6/14H6

SEE EMAIL FROM SJI

Ramin Abbaszadeh

From: Ken Charles <kcharles@steeljoist.org>
Sent: Wednesday, November 18, 2020 3:13 PM
To: Ramin Abbaszadeh
Subject: RE: Joist Investigation Form submitted at Steel Joist Institute

The joist you have presented resembles a 14J4 / 14H4 if manufactured by an SJI member company.

1929-1958	**SJ-Series**
1958-1961	**S-Series**
1961-1978	**J-Series**
1961-1986	**H-Series**
1986-Present	**K-Series**

All SJI load tables (1928 – present) are available here: TABLES

J. Kenneth Charles, III
Managing Director
Steel Joist Institute
140 West Evans Street, Suite 203, Florence, SC 29501
T 843-407-4091
steeljoist.org

From: Steel Joist Institute <no-reply@steeljoist.org>
Sent: Wednesday, November 18, 2020 2:59 PM
To: kcharles@steeljoist.org
Subject: Joist Investigation Form submitted at Steel Joist Institute

Ramin Abbaszadeh submitted a Joist Investigation Form.

Case Number	25891
First Name	Ramin
Last Name	Abbaszadeh
Company	Grey Fox Consulting, Inc.
Email Address	ramin@greyfox.biz
Preferred phone	8567451668

Jobsite Location City/State	Cherry Hill
Project Name	150 Mid Atlantic Pkway
Why are you requesting this information?	Evaluation
Other	
What year was the building constructed (or approximate age of the structure)?	1965-1970
Who was the joist manufacturer?	Hancock
Is there a tag on the joist?	Yes
Yes? Provide tag information	Not legible
What type of trusses are the joists?	Warren
Other	
What were the joists used for?	Roof loading
What type of bridging is used?	Horizontal
What is the joist span or length of joist?	24 feet
What is the joist spacing?	6'- 8"
What is the interior panel point spacing?	20" +/-
What is the joist depth?	14 in.
What is the height of the joist seat?	2-1/2"
Other, describe or attach sketch	
Top chord shape	2 Angles
Top chord leg size	1.5x1.5
Top chord thickness	0.17
Top chord diameter	

Bottom chord shape	2 Angles
Bottom chord leg size	1.4x1.4
Bottom chord thickness	0.14
Bottom chord diameter	
Vertical web shape	
Vertical web leg size	
Vertical web thickness	
Vertical web diameter	
Diagonal web shape	1 Round
Diagonal web leg size	
Diagonal web thickness	
Diagonal web diameter	0.6
Measurement device	Micrometer
Other device	

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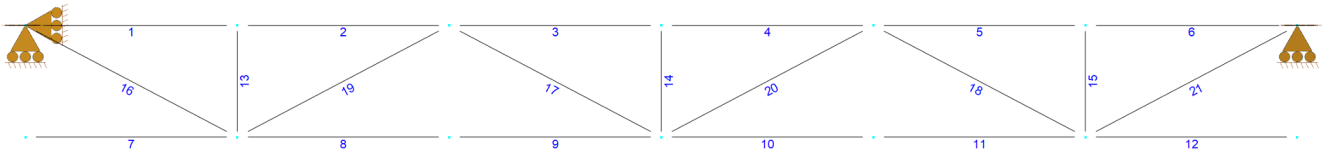

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Geometry data

GLOSSARY

Cb22, Cb33	: Moment gradient coefficients
Cm22, Cm33	: Coefficients applied to bending term in interaction formula
d0	: Tapered member section depth at J end of member
DJX	: Rigid end offset distance measured from J node in axis X
DJY	: Rigid end offset distance measured from J node in axis Y
DJZ	: Rigid end offset distance measured from J node in axis Z
DKX	: Rigid end offset distance measured from K node in axis X
DKY	: Rigid end offset distance measured from K node in axis Y
DKZ	: Rigid end offset distance measured from K node in axis Z
dL	: Tapered member section depth at K end of member
Ig factor	: Inertia reduction factor (Effective Inertia/Gross Inertia) for reinforced concrete members
K22	: Effective length factor about axis 2
K33	: Effective length factor about axis 3
L22	: Member length for calculation of axial capacity
L33	: Member length for calculation of axial capacity
LB pos	: Lateral unbraced length of the compression flange in the positive side of local axis 2
LB neg	: Lateral unbraced length of the compression flange in the negative side of local axis 2
RX	: Rotation about X
RY	: Rotation about Y
RZ	: Rotation about Z
TO	: 1 = Tension only member 0 = Normal member
TX	: Translation in X
TY	: Translation in Y
TZ	: Translation in Z

Nodes

Node	X [ft]	Y [ft]	Z [ft]	Rigid Floor
1	0.00	0.00	0.00	0
2	6.667	0.00	0.00	0
3	13.33	0.00	0.00	0
4	20.00	0.00	0.00	0
5	26.667	0.00	0.00	0
6	33.335	0.00	0.00	0
7	40.00	0.00	0.00	0
8	0.00	3.50	0.00	0
9	6.667	3.50	0.00	0
11	20.00	3.50	0.00	0
12	26.667	3.50	0.00	0
10	13.33	3.50	0.00	0
13	33.335	3.50	0.00	0
14	40.00	3.50	0.00	0

Restraints

Node	TX	TY	TZ	RX	RY	RZ
8	1	1	1	0	0	0
14	0	1	1	0	0	0

Members

Member	NJ	NK	Description	Section	Material	d0 [in]	dL [in]	lg factor
1	8	9		WT 8X18	A36	0.00	0.00	0.00
2	9	10		WT 8X18	A36	0.00	0.00	0.00
3	10	11		WT 8X18	A36	0.00	0.00	0.00
4	11	12		WT 8X18	A36	0.00	0.00	0.00
5	12	13		WT 8X18	A36	0.00	0.00	0.00
6	13	14		WT 8X18	A36	0.00	0.00	0.00
7	1	2		WT 6X15	A36	0.00	0.00	0.00
8	2	3		WT 6X15	A36	0.00	0.00	0.00
9	3	4		WT 6X15	A36	0.00	0.00	0.00
10	4	5		WT 6X15	A36	0.00	0.00	0.00
11	5	6		WT 6X15	A36	0.00	0.00	0.00
12	6	7		WT 6X15	A36	0.00	0.00	0.00
13	2	9		LU 2-1_2X2X1_4	A36	0.00	0.00	0.00
14	4	11		LU 2-1_2X2X1_4	A36	0.00	0.00	0.00
15	6	13		LU 2-1_2X2X1_4	A36	0.00	0.00	0.00
16	8	2		T2LU 2-1_2X2X1_4LLBB	A36	0.00	0.00	0.00
17	10	4		T2L 2X2X1_8	A36	0.00	0.00	0.00
18	12	6		T2LU 3X2X1_4LLBB	A36	0.00	0.00	0.00
19	2	10		T2LU 3X2X1_4LLBB	A36	0.00	0.00	0.00
20	4	12		T2L 2X2X1_8	A36	0.00	0.00	0.00
21	6	14		T2LU 2-1_2X2X1_4LLBB	A36	0.00	0.00	0.00

Orientation of local axes

Member	Rotation [Deg]	Axes23	NX	NY	NZ
7	180.00	0	0.00	0.00	0.00
8	180.00	0	0.00	0.00	0.00
9	180.00	0	0.00	0.00	0.00
10	180.00	0	0.00	0.00	0.00
11	180.00	0	0.00	0.00	0.00
12	180.00	0	0.00	0.00	0.00

Hinges

Member	Node-J M33	Node-J M22	Node-J V3	Node-J V2	Node-K M33	Node-K M22	Node-K V3	Node-K V2	TOR	AXL	Axial rigidity
13	1	0	0	0	1	0	0	0	0	0	Full
14	1	0	0	0	1	0	0	0	0	0	Full
15	1	0	0	0	1	0	0	0	0	0	Full
16	1	0	0	0	1	0	0	0	0	0	Full
17	1	0	0	0	1	0	0	0	0	0	Full
18	1	0	0	0	1	0	0	0	0	0	Full
19	1	0	0	0	1	0	0	0	0	0	Full
20	1	0	0	0	1	0	0	0	0	0	Full
21	1	0	0	0	1	0	0	0	0	0	Full

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Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

Condition	Description	Comb.	Category
D1	DL	Yes	
D2	DL+SP	Yes	

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier			
		Comb.	MultX	MultY	MultZ
D1	DL	Yes	0.00	0.00	0.00
D2	DL+SP	Yes	0.00	0.00	0.00

Earthquake (Dynamic analysis only)

Condition	a/g	Ang. [Deg]	Damp. [%]
D1	0.00	0.00	0.00
D2	0.00	0.00	0.00

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List of materials

Note.- Only the graphically selected members and shells are listed

Members:

Profile	Material	Uweight [Kip/ft]	Length [ft]	Weight [Kip]
LU 2-1_2X2X1_4	A36	3.61E-03	10.500	0.038
T2L 2X2X1_8	A36	3.35E-03	15.062	0.050
T2LU 2-1_2X2X1_4LLBB	A36	7.29E-03	15.058	0.110
T2LU 3X2X1_4LLBB	A36	8.18E-03	15.057	0.123
WT 6X15	A36	1.50E-02	40.000	0.600
WT 8X18	A36	1.80E-02	40.000	0.721
Total weight [Kip]				1.642

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Analysis result

Maximum forces at members

Condition : **D1=DL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-23.83	0.24	0.00	0.00	0.00	1.55
Min	-23.83	0.24	0.00	0.00	0.00	-0.03
MEMBER **2**						
Max	-23.84	-0.14	0.00	0.00	0.00	1.54
Min	-23.84	-0.14	0.00	0.00	0.00	0.63
MEMBER **3**						
Max	-42.72	0.28	0.00	0.00	0.00	2.48
Min	-42.72	0.28	0.00	0.00	0.00	0.62
MEMBER **4**						
Max	-42.72	-0.28	0.00	0.00	0.00	2.48
Min	-42.72	-0.28	0.00	0.00	0.00	0.62
MEMBER **5**						
Max	-23.83	0.13	0.00	0.00	0.00	1.53
Min	-23.83	0.13	0.00	0.00	0.00	0.63
MEMBER **6**						
Max	-23.82	-0.24	0.00	0.00	0.00	1.55
Min	-23.82	-0.24	0.00	0.00	0.00	-0.03
MEMBER **7**						
Max	0.00	0.00	0.00	0.00	0.00	0.00
Min	0.00	0.00	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	38.53	-0.07	0.00	0.00	0.00	-0.04
Min	38.53	-0.07	0.00	0.00	0.00	-0.50
MEMBER **9**						
Max	38.53	-0.07	0.00	0.00	0.00	-0.50
Min	38.53	-0.07	0.00	0.00	0.00	-0.96
MEMBER **10**						
Max	38.53	0.07	0.00	0.00	0.00	-0.50
Min	38.53	0.07	0.00	0.00	0.00	-0.96
MEMBER **11**						
Max	38.53	0.07	0.00	0.00	0.00	-0.04
Min	38.53	0.07	0.00	0.00	0.00	-0.50
MEMBER **12**						
Max	0.00	0.00	0.00	0.00	0.00	0.00
Min	0.00	0.00	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	-4.73	0.00	-0.01	0.00	0.03	0.00
Min	-4.73	0.00	-0.01	0.00	-0.02	0.00
MEMBER **14**						
Max	-4.54	0.00	0.00	0.00	0.00	0.00
Min	-4.54	0.00	0.00	0.00	0.00	0.00
MEMBER **15**						
Max	-4.73	0.00	0.01	0.00	0.02	0.00
Min	-4.73	0.00	0.01	0.00	-0.03	0.00
MEMBER **16**						
Max	26.91	0.00	0.00	0.00	-0.03	0.00
Min	26.91	0.00	0.00	0.00	-0.05	0.00
MEMBER **17**						

Max	4.74	0.00	0.00	0.00	-0.02	0.00
Min	4.74	0.00	0.00	0.00	-0.02	0.00
MEMBER **18**						
Max	-16.60	0.00	0.00	0.00	0.00	0.00
Min	-16.60	0.00	0.00	0.00	0.00	0.00
MEMBER **19**						
Max	-16.59	0.00	0.00	0.00	0.00	0.00
Min	-16.59	0.00	0.00	0.00	0.00	0.00
MEMBER **20**						
Max	4.74	0.00	0.00	0.00	-0.02	0.00
Min	4.74	0.00	0.00	0.00	-0.02	0.00
MEMBER **21**						
Max	26.91	0.00	0.00	0.00	-0.03	0.00
Min	26.91	0.00	0.00	0.00	-0.05	0.00

Condition : **D2=DL+SP**

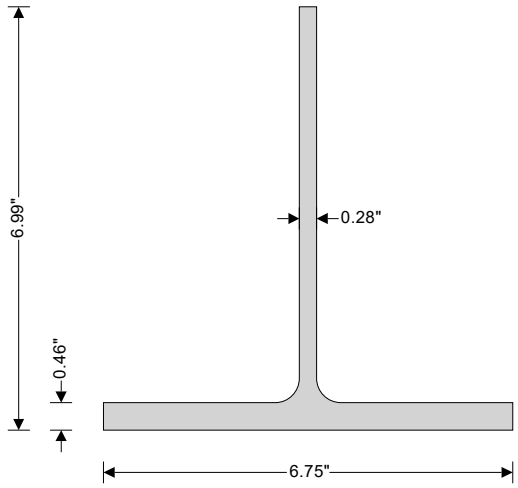
	Axial	**Shear V2**	**Shear V3**	**Torsion**	**M22**	**M33**
	[Kip]	[Kip]	[Kip]	[Kip*ft]	[Kip*ft]	[Kip*ft]
MEMBER **1**						
Max	-32.80	0.33	0.00	0.00	0.00	2.13
Min	-32.80	0.33	0.00	0.00	0.00	-0.05
MEMBER **2**						
Max	-32.81	-0.19	0.00	0.00	0.00	2.11
Min	-32.81	-0.19	0.00	0.00	0.00	0.87
MEMBER **3**						
Max	-58.81	0.39	0.00	0.00	0.00	3.42
Min	-58.81	0.39	0.00	0.00	0.00	0.85
MEMBER **4**						
Max	-58.81	-0.39	0.00	0.00	0.00	3.42
Min	-58.81	-0.39	0.00	0.00	0.00	0.85
MEMBER **5**						
Max	-32.80	0.19	0.00	0.00	0.00	2.11
Min	-32.80	0.19	0.00	0.00	0.00	0.87
MEMBER **6**						
Max	-32.79	-0.33	0.00	0.00	0.00	2.13
Min	-32.79	-0.33	0.00	0.00	0.00	-0.05
MEMBER **7**						
Max	0.00	0.00	0.00	0.00	0.00	0.00
Min	0.00	0.00	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	53.03	-0.09	0.00	0.00	0.00	-0.06
Min	53.03	-0.09	0.00	0.00	0.00	-0.69
MEMBER **9**						
Max	53.03	-0.09	0.00	0.00	0.00	-0.69
Min	53.03	-0.09	0.00	0.00	0.00	-1.32
MEMBER **10**						
Max	53.04	0.09	0.00	0.00	0.00	-0.69
Min	53.04	0.09	0.00	0.00	0.00	-1.32
MEMBER **11**						
Max	53.04	0.09	0.00	0.00	0.00	-0.06
Min	53.04	0.09	0.00	0.00	0.00	-0.69
MEMBER **12**						
Max	0.00	0.00	0.00	0.00	0.00	0.00
Min	0.00	0.00	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	-6.51	0.00	-0.02	0.00	0.04	0.00
Min	-6.51	0.00	-0.02	0.00	-0.03	0.00

MEMBER **14**						
Max	-6.25	0.00	0.00	0.00	0.00	0.00
Min	-6.25	0.00	0.00	0.00	0.00	0.00
MEMBER **15**						
Max	-6.51	0.00	0.02	0.00	0.03	0.00
Min	-6.51	0.00	0.02	0.00	-0.04	0.00
MEMBER **16**						
Max	37.05	0.00	0.00	0.00	-0.05	0.00
Min	37.05	0.00	0.00	0.00	-0.07	0.00
MEMBER **17**						
Max	6.52	0.00	0.00	0.00	-0.02	0.00
Min	6.52	0.00	0.00	0.00	-0.02	0.00
MEMBER **18**						
Max	-22.85	0.00	0.00	0.00	0.00	0.00
Min	-22.85	0.00	0.00	0.00	0.00	0.00
MEMBER **19**						
Max	-22.84	0.00	0.00	0.00	0.00	0.00
Min	-22.84	0.00	0.00	0.00	0.00	0.00
MEMBER **20**						
Max	6.52	0.00	0.00	0.00	-0.02	0.00
Min	6.52	0.00	0.00	0.00	-0.02	0.00
MEMBER **21**						
Max	37.04	0.00	0.00	0.00	-0.05	0.00
Min	37.04	0.00	0.00	0.00	-0.07	0.00

STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section	**WT 7x17**

Design loading

Required axial strength	P_r = **21** kips (Compression)

Material details

Steel grade	**A36**
Yield strength	F_y = **36** ksi
Ultimate strength	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **80** in
For buckling about y axis	L_y = **80** in
For torsional buckling	L_z = **80** in

Effective length factors

For buckling about x axis	K_x = **1.00**
For buckling about y axis	K_y = **1.00**
For torsional buckling	K_z = **1.00**

Effective unbraced lengths

For buckling about x axis	$L_{cx} = L_x \times K_x$ = **80** in
For buckling about y axis	$L_{cy} = L_y \times K_y$ = **80** in
For torsional buckling	$L_{cz} = L_z \times K_z$ = **80** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f / 2 =$ **3.375** in

Width to thickness ratio of flange $\lambda_f = b / t_f =$ **7.418**

Width to thickness ratio of stem $\lambda_w = d / t_w =$ **24.526**

Compression

Limit for nonslender flange $\lambda_{rf_c} = 0.45 \times \sqrt{(E / F_y)} =$ **12.772**

The flange is nonslender in compression

Limit for nonslender stem $\lambda_{rw_c} = 0.75 \times \sqrt{(E / F_y)} =$ **21.287**

The stem is slender in compression

The section is slender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x =$ **39.2**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **52.3**

Compressive strength

Flexural buckling about x axis (cl. E7)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2 =$ **185.9** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **33.2** ksi

Width of compression element 1 $b =$ **3.375** in

Imperfection Adjustment Factors $c_{1,b} =$ **0.22**

$c_{2,b} =$ **1.49**

Elastic local buckling stress $F_{el,b} = (c_{2,b} \times \lambda_{rf_c} / (\lambda_f))^2 \times F_y =$ **237** ksi

Reduced effective width $b_e = b =$ **3.4** in

Width of compression element 2 $d =$ **6.99** in

Imperfection Adjustment Factors $c_{1,d} =$ **0.22**

$c_{2,d} =$ **1.49**

Elastic local buckling stress $F_{el,d} = (c_{2,d} \times \lambda_{rw_c} / (\lambda_w))^2 \times F_y =$ **60.2** ksi

Reduced effective width $d_e = d \times (1 - c_{1,d} \times \sqrt{(F_{el,d} / F_{crx})}) \times \sqrt{(F_{el,d} / F_{crx})} =$ **6.6** in

Effective area of cross-section $A_e = A - 2 \times (b - b_e) \times t_f - (d - d_e) \times t_w =$ **4.896** in^2

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A_e =$ **162.5** kips

Flexural buckling about y axis (cl. E7)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **104.6** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y =$ **31.2** ksi

Width of compression element 1 $b =$ **3.375** in

Imperfection Adjustment Factors $c_{1,b} =$ **0.22**

$c_{2,b} =$ **1.49**

Elastic local buckling stress $F_{el,b} = (c_{2,b} \times \lambda_{rf_c} / (\lambda_f))^2 \times F_y =$ **237** ksi

Reduced effective width $b_e = b =$ **3.4** in

Width of compression element 2	d = **6.99** in
Imperfection Adjustment Factors	$c_{1,d}$ = **0.22**
	$c_{2,d}$ = **1.49**
Elastic local buckling stress	$F_{el,d} = (c_{2,d} \times \lambda_{rw_c} / (\lambda_w))^2 \times F_y$ = **60.2** ksi
Reduced effective width	$d_e = d \times (1 - c_{1,d} \times \sqrt{(F_{el,d} / F_{cry})}) \times \sqrt{(F_{el,d} / F_{cry})}$ = **6.7** in
Effective area of cross-section	$A_e = A - 2 \times (b - b_e) \times t_f - (d - d_e) \times t_w$ = **4.93** in^2
Nominal compressive strength for flexural buckling	$P_{ny} = F_{cry} \times A_e$ = **153.7** kips

Torsional and torsional-flexural buckling of members with slender elements (cl. E7)

Elastic torsional buckling stress	$F_{ez} = (\pi^2 \times E \times C_w / (L_{cz})^2 + G \times J) / (A \times r_o^2)$ = **77.7** ksi
Flexural-torsional elastic buckling stress	$F_{et} = (F_{ey} + F_{ez}) / (2 \times H) \times (1 - \sqrt{(1 - 4 \times F_{ey} \times F_{ez} \times H / (F_{ey} + F_{ez})^2)})$ = **60.4** ksi
Flexural-torsional buckling stress	$F_{crt} = (0.658^{F_y / F_{et}}) \times F_y$ = **28.1** ksi
Width of compression element 1	b = **3.375** in
Imperfection Adjustment Factors	$c_{1,b}$ = **0.22**
	$c_{2,b}$ = **1.49**
Elastic local buckling stress	$F_{el,b} = (c_{2,b} \times \lambda_{rf_c} / (\lambda_f))^2 \times F_y$ = **237** ksi
Reduced effective width	$b_e = b$ = **3.4** in
Width of compression element 2	d = **6.99** in
Imperfection Adjustment Factors	$c_{1,d}$ = **0.22**
	$c_{2,d}$ = **1.49**
Elastic local buckling stress	$F_{el,d} = (c_{2,d} \times \lambda_{rw_c} / (\lambda_w))^2 \times F_y$ = **60.2** ksi
Reduced effective width	$d_e = d \times (1 - c_{1,d} \times \sqrt{(F_{el,d} / F_{crt})}) \times \sqrt{(F_{el,d} / F_{crt})}$ = **6.9** in
Effective area of cross-section	$A_e = A - 2 \times (b - b_e) \times t_f - (d - d_e) \times t_w$ = **4.986** in^2
Nominal compressive strength for torsional and flexural-torsional buckling	
	$P_{nt} = F_{crt} \times A_e$ = **139.9** kips

Allowable compressive strength (cl. E1)

Safety factor for compression	Ω_c = **1.67**
Allowable compressive strength	$P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c$ = **83.8** kips

PASS - The allowable compressive strength exceeds the required compressive strength

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STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 15th Edition published 2016 using the ASD method

Tedds calculation version 4.4.01

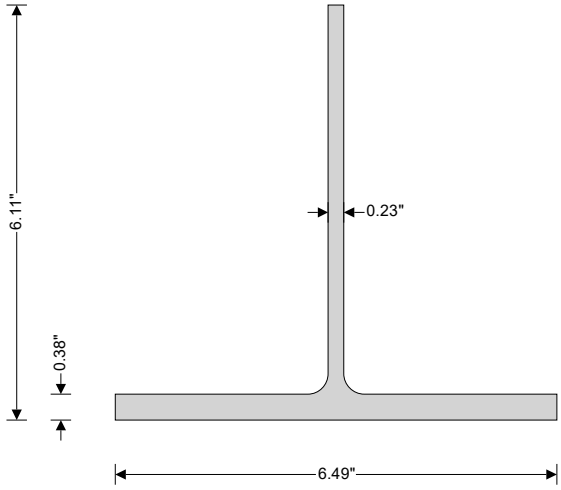
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Design section 1

Section details

Section type	WT 6x13 (AISC 15th Edn (v15.0))
ASTM steel designation	A36
Steel yield stress	F_y = **36** ksi
Steel tensile stress	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi



WT 6x13 (AISC 15th Edn (v15.0))
Section depth, d, 6.11 in
Section breadth, b_f, 6.49 in
Weight of section, Weight, 13 lbf/ft
Flange thickness, t_f, 0.38 in
Web thickness, t_w, 0.23 in
Area of section, A, 3.8 in²
Radius of gyration about x-axis, r_x, 1.75 in
Radius of gyration about y-axis, r_y, 1.51 in
Elastic section modulus about x-axis, S_x, 2.4 in³
Elastic section modulus about y-axis, S_y, 2.67 in³
Plastic section modulus about x-axis, Z_x, 4.2 in³
Plastic section modulus about y-axis, Z_y, 4.08 in³
Second moment of area about x-axis, I_x, 11.7 in⁴
Second moment of area about y-axis, I_y, 8.66 in⁴

Analysis results

Required tensile strength P_r = **18.89** kips

Restraint spacing

Major axis lateral restraint	L_x = **40** ft
Minor axis lateral restraint	L_y = **40** ft
Torsional restraint	L_z = **40** ft

Design of members for tension - Chapter D

Required tensile strength P_r = **18.89** kips

Slenderness limitations - Section D1

Slenderness ratio $\lambda = \max(L_x / r_x, L_y / r_y)$ = **317.881**

WARNING - Tension member slenderness ratio exceeds recommended limit of 300

Allowable tensile strength - D2

Nominal tensile yielding strength $P_{n,yld} = F_y \times A =$ **137.52** kips

Nominal tensile rupture strength $P_{n,r} = F_u \times A_e =$ **221.56** kips

Allowable tensile strength $P_c = \min(P_{n,yld} / \Omega_{t,y}, P_{n,r} / \Omega_{t,r}) =$ **82.347** kips

$P_r / P_c =$ **0.229**

PASS - Nominal tensile strength exceeds required tensile strength

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	Section Joist Girder Web 16 & 21				Sheet no./rev. 1	
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STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.4.01

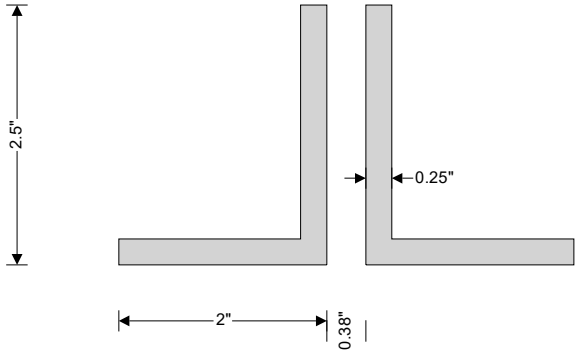
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Design section 1

Section details

Section type	2L 2-1/2x2x1/4x3/8 (AISC 15th Edn (v15.0))
ASTM steel designation	A36
Steel yield stress	F_y = **36** ksi
Steel tensile stress	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi



2L 2-1/2x2x1/4x3/8 (AISC 15th Edn (v15.0))
Section depth, L_1, 2.5 in
Section breadth, L_2, 2 in
Weight of section, Weight, 7.2 lbf/ft
Section thickness, t, 0.25 in
Area of section, A, 2.1 in^2
Radius of gyration about x-axis, r_x, 0.782 in
Radius of gyration about y-axis, r_y, 0.93 in
Elastic section modulus about x-axis, S_x, 0.762 in^3
Second moment of area about x-axis, I_x, 1.31 in^4
Second moment of area about y-axis, I_y, 1.85 in^4

Analysis results

Required tensile strength P_r = **13.19** kips

Restraint spacing

Major axis lateral restraint	L_x = **7.5** ft
Minor axis lateral restraint	L_y = **7.5** ft
Torsional restraint	L_z = **7.5** ft

Design of members for tension - Chapter D

Required tensile strength P_r = **13.19** kips

Slenderness limitations - Section D1

Slenderness ratio $\lambda = \max(L_x / r_x, L_y / r_y)$ = **115.09**

Tension member slenderness ratio does not exceed recommended limit of 300

Allowable tensile strength - D2

Nominal tensile yielding strength	$P_{n,yld} = F_y \times A$ = **77.04** kips
Nominal tensile rupture strength	$P_{n,r} = F_u \times A_e$ = **124.12** kips
Allowable tensile strength	$P_c = \min(P_{n,yld} / \Omega_{t,y}, P_{n,r} / \Omega_{t,r})$ = **46.132** kips

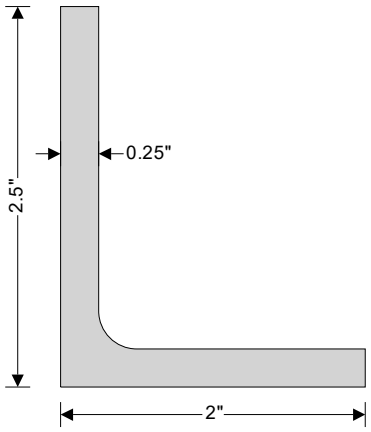
P_r / P_c = **0.286**

PASS - Nominal tensile strength exceeds required tensile strength

STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section	**L 2-1/2x2x1/4**

Design loading

Required axial strength	P_r = **2** kips (Compression)

Material details

Steel grade	**A36**
Yield strength	F_y = **36** ksi
Ultimate strength	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Angle details

Length between work points	L = **42.0** in
Connected leg	**Long**
Type of member	**Individual**

Section classification

Section classification for local buckling (cl. B4)

Critical width	$b = \max(L_1, L_2)$ = **2.500** in
Maximum width to thickness ratio	$\lambda = b / t$ = **10.000**

Compression

Limit for nonslender section	$\lambda_{r_c} = 0.45 \times \sqrt{(E / F_y)}$ = **12.772**

The section is nonslender in compression

Slenderness

Slenderness (cl. E5)

Slenderness ratio $SR = 72 + 0.75 \times L / r_y =$ **125.5**

Compressive strength

Flexural buckling about axis (cl. E3)

Elastic critical buckling stress $F_e = \pi^2 \times E / (SR)^2 =$ **18.2** ksi

Flexural buckling stress $F_{cr} = (0.658^{F_y / F_e}) \times F_y =$ **15.7** ksi

Nominal compressive strength for flexural buckling $P_n = F_{cr} \times A =$ **16.8** kips

Allowable compressive strength (cl. E1)

Safety factor for compression $\Omega_c =$ **1.67**

Allowable compressive strength $P_c = P_n / \Omega_c =$ **10.1** kips

PASS - The allowable compressive strength exceeds the required compressive strength

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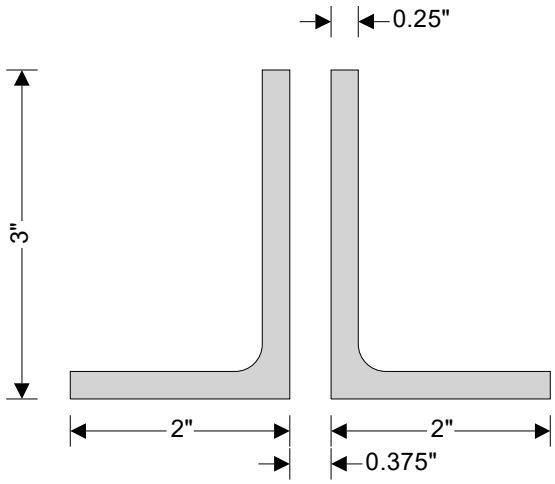
STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section	**2L 3x2x1/4x3/8**

Design loading

Required axial strength	P_r = **8** kips (Compression)

Material details

Steel grade	**A36**
Yield strength	F_y = **36** ksi
Ultimate strength	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **90** in
For buckling about y axis	L_y = **90** in
For torsional buckling	L_z = **90** in

Effective length factors

For buckling about x axis	K_x = **1.00**
For buckling about y axis	K_y = **1.00**
For torsional buckling	K_z = **1.00**

Effective unbraced lengths

For buckling about x axis	$L_{cx} = L_x \times K_x$ = **90** in
For buckling about y axis	$L_{cy} = L_y \times K_y$ = **90** in
For torsional buckling	$L_{cz} = L_z \times K_z$ = **90** in
Type of connector fixing angles together	**Welded**
Distance between connectors	a = **40.0** in

Section classification

Section classification for local buckling (cl. B4)

Critical width $b = \max(L_1, L_2) = \mathbf{3.000}$ in

Maximum width to thickness ratio $\lambda = b / t = \mathbf{12.000}$

Compression

Limit for nonslender section $\lambda_{r_c} = 0.45 \times \sqrt{(E / F_y)} = \mathbf{12.772}$

The section is nonslender in compression

Slenderness

Effective slenderness (cl. E6)

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) = \mathbf{0.43}$ in

Buckling about axis parallel to unconnected leg $SR_x = L_{cx} / r_x = \mathbf{94.4}$

Buckling about axis parallel to connected leg $SR_y = \sqrt{((L_{cy} / r_y)^2 + (0.5 \times a / r_i)^2)} = \mathbf{112.0}$

Check slenderness between connectors (cl. E6.2)

Effective length factor between connectors $K = \mathbf{1.0}$

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) = \mathbf{0.43}$ in

Slenderness between connectors $SR_{con} = K \times a / r_i = \mathbf{92.8}$

Maximum allowable slenderness $SR_{allow} = 0.75 \times \max(SR_x, SR_y) = \mathbf{84.0}$

FAIL - The slenderness between connectors exceeds 3/4 of governing slenderness of built-up member

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2 = \mathbf{32.1}$ ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y = \mathbf{22.5}$ ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A = \mathbf{54}$ kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2 = \mathbf{22.8}$ ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y = \mathbf{18.6}$ ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} \times A = \mathbf{44.6}$ kips

Torsional and torsional-flexural buckling of members with slender elements (cl. E7)

Elastic torsional buckling stress $F_{ez} = (\pi^2 \times E \times C_w / (L_{cz})^2 + G \times J) / (A \times r_o^2) = \mathbf{103.6}$ ksi

Flexural-torsional elastic buckling stress $F_{et} = (F_{ey} + F_{ez}) / (2 \times H) \times (1 - \sqrt{(1 - 4 \times F_{ey} \times F_{ez} \times H / (F_{ey} + F_{ez})^2)}) = \mathbf{21.2}$ ksi

Flexural-torsional buckling stress $F_{crt} = (0.658^{F_y / F_{et}}) \times F_y = \mathbf{17.7}$ ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} \times A = \mathbf{42.4}$ kips

Allowable compressive strength (cl. E1)

Safety factor for compression $\Omega_c = \mathbf{1.67}$

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c = \mathbf{25.4}$ kips

PASS - The allowable compressive strength exceeds the required compressive strength

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.4.01

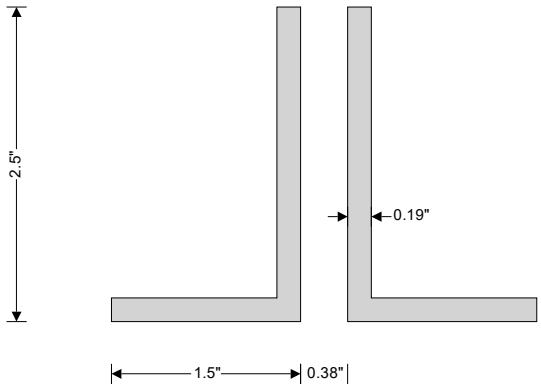
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Design section 1

Section details

Section type	2L 2-1/2x1-1/2x3/16x3/8 (AISC 15th Edn (v15.0))
ASTM steel designation	A36
Steel yield stress	F_y = **36** ksi
Steel tensile stress	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi



2L 2-1/2x1-1/2x3/16x3/8 (AISC 15th Edn (v15.0))
Section depth, L_1, 2.5 in
Section breadth, L_2, 1.5 in
Weight of section, Weight, 4.9 lbf/ft
Section thickness, t, 0.188 in
Area of section, A, 1.5 in^2
Radius of gyration about x-axis, r_x, 0.8 in
Radius of gyration about y-axis, r_y, 0.677 in
Elastic section modulus about x-axis, S_x, 0.56 in^3
Second moment of area about x-axis, I_x, 0.928 in^4
Second moment of area about y-axis, I_y, 0.665 in^4

Analysis results

Required tensile strength P_r = **2.32** kips

Restraint spacing

Major axis lateral restraint	L_x = **7.5** ft
Minor axis lateral restraint	L_y = **7.5** ft
Torsional restraint	L_z = **7.5** ft

Design of members for tension - Chapter D

Required tensile strength P_r = **2.32** kips

Slenderness limitations - Section D1

Slenderness ratio $\lambda = \max(L_x / r_x, L_y / r_y)$ = **132.939**

Tension member slenderness ratio does not exceed recommended limit of 300

Allowable tensile strength - D2

Nominal tensile yielding strength	$P_{n,yld} = F_y \times A$ = **52.2** kips
Nominal tensile rupture strength	$P_{n,r} = F_u \times A_e$ = **84.1** kips

Allowable tensile strength

$P_c = \min(P_{n,yld} / \Omega_{t,y}, P_{n,r} / \Omega_{t,r})$ = **31.257** kips

P_r / P_c = **0.074**

PASS - Nominal tensile strength exceeds required tensile strength

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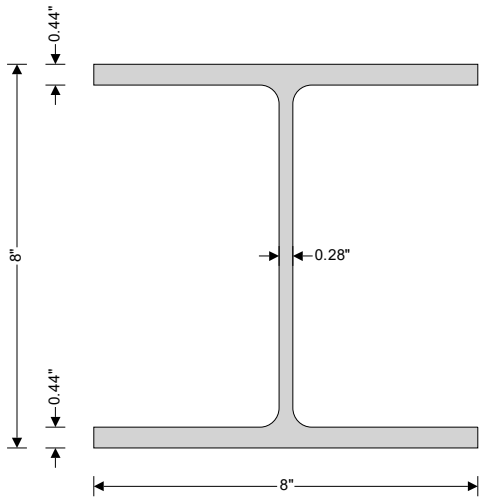
STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section	**W 8x31**

Design loading

Required axial strength	P_r = **57** kips (Compression)
Maximum moment about x axis	M_x = **0.0** kips_ft
Maximum moment about y axis	M_y = **0.0** kips_ft
Maximum shear force parallel to y axis	V_{ry} = **0.0** kips
Maximum shear force parallel to x axis	V_{rx} = **0.0** kips

Material details

Steel grade	**A36**
Yield strength	F_y = **36** ksi
Ultimate strength	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **312** in
For buckling about y axis	L_y = **312** in
For torsional buckling	L_z = **312** in

Effective length factors

For buckling about x axis	K_x = **1.00**
For buckling about y axis	K_y = **1.00**
For torsional buckling	K_z = **1.00**

Effective unbraced lengths

For buckling about x axis	$L_{cx} = L_x \times K_x$ = **312** in

For buckling about y axis $L_{cy} = L_y \times K_y =$ **312** in

For torsional buckling $L_{cz} = L_z \times K_z =$ **312** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f / 2 =$ **4.000** in

Width to thickness ratio of flange $\lambda_f = b / t_f =$ **9.195**

Depth between root radii $h = d - 2 \times k =$ **6.342** in

Width to thickness ratio of web $\lambda_w = h / t_w =$ **22.253**

Compression

Limit for nonslender flange $\lambda_{rf_c} = 0.56 \times \sqrt{(E / F_y)} =$ **15.894**

The flange is nonslender in compression

Limit for nonslender web $\lambda_{rw_c} = 1.49 \times \sqrt{(E / F_y)} =$ **42.290**

The web is nonslender in compression

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x =$ **89.9**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **154.5**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2 =$ **35.4** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **23.5** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A =$ **214.8** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **12** ksi

Flexural buckling stress $F_{cry} = 0.877 \times F_{ey} =$ **10.5** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} \times A =$ **96.1** kips

Torsional and torsional-flexural buckling of members with slender elements (cl. E7)

Flexural-torsional elastic buckling stress $F_{et} = (\pi^2 \times E \times C_w / (L_{cz})^2 + G \times J) / (I_x + I_y) =$ **51.4** ksi

Flexural-torsional buckling stress $F_{crt} = 0.877 \times F_{et} =$ **45.1** ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} \times A =$ **411.6** kips

Allowable compressive strength (cl. E1)

Safety factor for compression $\Omega_c =$ **1.67**

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c =$ **57.5** kips

PASS - The allowable compressive strength exceeds the required compressive strength

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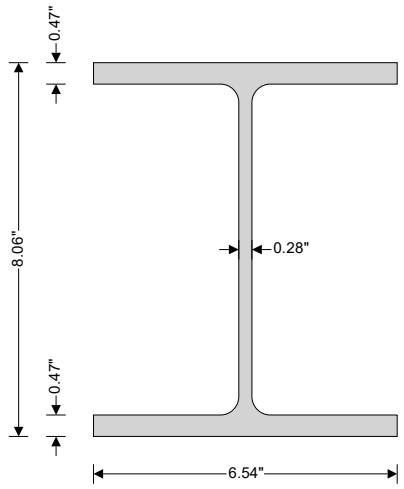
STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section	**W 8x28**

Design loading

Required axial strength	P_r = **29** kips (Compression)
Maximum moment about x axis	M_x = **0.0** kips_ft
Maximum moment about y axis	M_y = **0.0** kips_ft
Maximum shear force parallel to y axis	V_{ry} = **0.0** kips
Maximum shear force parallel to x axis	V_{rx} = **0.0** kips

Material details

Steel grade	**A36**
Yield strength	F_y = **36** ksi
Ultimate strength	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **312** in
For buckling about y axis	L_y = **312** in
For torsional buckling	L_z = **312** in

Effective length factors

For buckling about x axis	K_x = **1.00**
For buckling about y axis	K_y = **1.00**
For torsional buckling	K_z = **1.00**

Effective unbraced lengths

For buckling about x axis	$L_{cx} = L_x \times K_x$ = **312** in

For buckling about y axis $L_{cy} = L_y \times K_y =$ **312** in

For torsional buckling $L_{cz} = L_z \times K_z =$ **312** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f / 2 =$ **3.270** in

Width to thickness ratio of flange $\lambda_f = b / t_f =$ **7.032**

Depth between root radii $h = d - 2 \times k =$ **6.342** in

Width to thickness ratio of web $\lambda_w = h / t_w =$ **22.253**

Compression

Limit for nonslender flange $\lambda_{rf_c} = 0.56 \times \sqrt{(E / F_y)} =$ **15.894**

The flange is nonslender in compression

Limit for nonslender web $\lambda_{rw_c} = 1.49 \times \sqrt{(E / F_y)} =$ **42.290**

The web is nonslender in compression

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x =$ **90.4**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **192.6**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2 =$ **35** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **23.4** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A =$ **193.1** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **7.7** ksi

Flexural buckling stress $F_{cry} = 0.877 \times F_{ey} =$ **6.8** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} \times A =$ **55.8** kips

Torsional and torsional-flexural buckling of members with slender elements (cl. E7)

Flexural-torsional elastic buckling stress $F_{et} = (\pi^2 \times E \times C_w / (L_{cz})^2 + G \times J) / (I_x + I_y) =$ **57.9** ksi

Flexural-torsional buckling stress $F_{crt} = 0.877 \times F_{et} =$ **50.8** ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} \times A =$ **419** kips

Allowable compressive strength (cl. E1)

Safety factor for compression $\Omega_c =$ **1.67**

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c =$ **33.4** kips

PASS - The allowable compressive strength exceeds the required compressive strength

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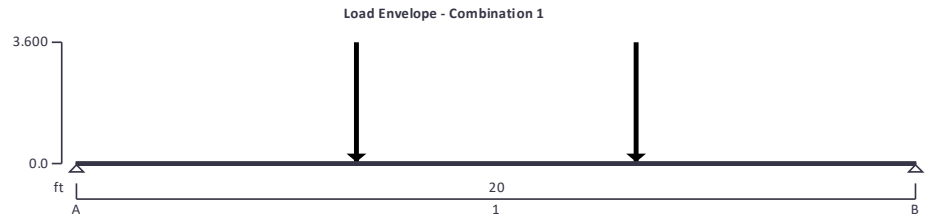
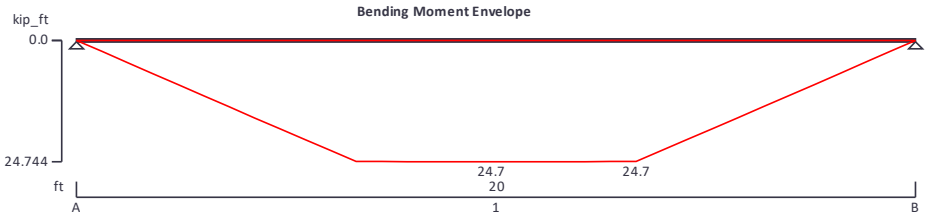
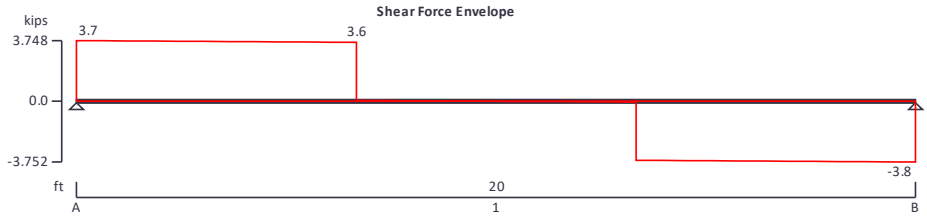
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360-10 using the ASD method

Tedds calculation version 3.0.15







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
Span 1 loads	Dead point load 3.6 kips at 80.04 in
	Dead point load 3.6 kips at 160.08 in

Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00

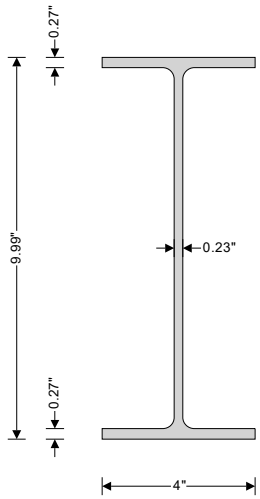
		Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **24.7** kips_ft	M_{min} = **0** kips_ft
Maximum moment span 1 segment 1	$M_{s1_seg1_max}$ = **24.7** kips_ft	$M_{s1_seg1_min}$ = **0** kips_ft
Maximum moment span 1 segment 2	$M_{s1_seg2_max}$ = **24.7** kips_ft	$M_{s1_seg2_min}$ = **0** kips_ft
Maximum moment span 1 segment 3	$M_{s1_seg3_max}$ = **24.7** kips_ft	$M_{s1_seg3_min}$ = **0** kips_ft
Maximum shear	V_{max} = **3.7** kips	V_{min} = **-3.8** kips
Maximum shear span 1 segment 1	$V_{s1_seg1_max}$ = **3.7** kips	$V_{s1_seg1_min}$ = **0** kips
Maximum shear span 1 segment 2	$V_{s1_seg2_max}$ = **3.6** kips	$V_{s1_seg2_min}$ = **-0.1** kips
Maximum shear span 1 segment 3	$V_{s1_seg3_max}$ = **0** kips	$V_{s1_seg3_min}$ = **-3.8** kips
Deflection segment 4	δ_{max} = **0.9** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **3.7** kips	R_{A_min} = **3.7** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **3.7** kips	
Maximum reaction at support B	R_{B_max} = **3.8** kips	R_{B_min} = **3.8** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **3.8** kips	

Section details

Section type	**W 10x15 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A36**
Steel yield stress	F_y = **36** ksi
Steel tensile stress	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding Ω_{ty} = **1.67**

Safety factor for tensile rupture Ω_{tr} = **2.00**

Safety factor for compression Ω_c = **1.67**

Safety factor for flexure Ω_b = **1.67**

Lateral bracing

Span 1 has lateral bracing at supports plus third points

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 \times t_f)$ = **7.41**

Limiting ratio for compact section $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **10.79**

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **28.38** Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 \times k) / t_w$ = **38.48**

Limiting ratio for compact section $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **106.72**

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **161.78** Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **3.752** kips

Web area $A_w = d \times t_w$ = **2.298** in^2

Web plate buckling coefficient k_v = **5**

Web shear coefficient - eq G2-3 C_v = **1**

Nominal shear strength – eq G2-1 $V_n = 0.6 \times F_y \times A_w \times C_v$ = **49.630** kips

Safety factor for shear Ω_v = **1.50**

Allowable shear strength $V_c = V_n / \Omega_v$ = **33.087** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 segment 2 - Chapter F

Required flexural strength $M_r = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min}))$ = **24.744** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **48** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{s1_seg2}$ = **80** in

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{[E / F_y]}$ = **40.462** in

Distance between flange centroids $h_o = d - t_f$ = **9.72** in

c = **1**

$r_{ts} = \sqrt{[\sqrt{(I_y \times C_w)} / S_x]}$ = **1.009** in

Limiting unbraced length for inelastic LTB - eq F2-6

$$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)}]} = \mathbf{127.362} \text{ in}$$

Cross-section mono-symmetry parameter R_m = **1.000**

Moment at quarter point of segment M_A = **24.726** kips_ft

Moment at center-line of segment M_B = **24.744** kips_ft

Moment at three quarter point of segment M_C = **24.720** kips_ft

Maximum moment in segment M_{abs} = **24.744** kips_ft

Lateral torsional buckling modification factor - eq F1-1 $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C]$ = **1.000**

Nominal flexural strength for lateral torsional buckling - eq F2-2 $M_{nltb} = C_b \times [M_p - (M_p - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)]$ = **39.362** kips_ft

Nominal flexural strength $M_n = \min(M_{nyld}, M_{nltb})$ = **39.362** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b$ = **23.570** kips_ft

FAIL - Required flexural strength exceeds allowable flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240$ = **1** in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.911** in

PASS - Maximum deflection does not exceed deflection limit