GreyFox CONSULTING, Inc.

Structural & Civil
Engineering Solutions

FEASIBILITY REPORT

Date: December 23, 2020

To: Ben Aparo and Nate Greenberg
Summit Ridge Energy

From: Ramin Abbaszadeh, PE

Re: 67 Route 46
Fairfield, 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 late 1990s. 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 steel structure (see pictures 1 and 2). The building has approximately 200 ft width in the north-south direction and 360 ft length in the east-west direction. The roof structure is comprised of a steel deck supported by steel bar joists. Steel joists girders support the bar joists. The columns are Hollow Steel Tubes. The outside perimeter walls are precast concrete panels.

A tire warehouse and distribution center occupy the building. Because their operation is the busiest during the week, we were given access only on a Saturday during their operation hours of 8:30 to 1:00 pm.

As shown in pictures 3 to 7, the warehouse is tightly packed with racking extending to the roof framing. Between the limited investigation time and the racking layout, we only checked two locations for joists and joist girders. Measurement of joist girders takes more time compared to wide flange steel girder sytem.

As shown in pictures 3 through 12, the roof structure in the storage area supports minimal mechanical and lighting 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 & Builtup Roof = 6 pounds per square foot (psf) was assumed
  - Dead Load: Sprinkler system = 3 psf was assumed
  - Dead Load: Collateral (lights, piping, ductwork, electrical) = 3 psf was assumed for future application
  - Dead load: Structural joists = 3 psf from historical data
  - Dead load: Structural joist girders—as calculated in our modeling of the framing
  - 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 at storage area = 35 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 41 ft - 4 in. The bar joists are supported on the steel joist girders.

The approximate spacing between bar joists is 5'-8". Based on our field measurement, the joists are 28-inch-deep with 2 inches by 2 inches double angles top chord and 1.75 inches by 1.75 inches double angles bottom chords. The thickness of the top chords is 0.21 inches. The bottom chords have a thickness of 0.16 inches. The web members are comprised of 0.93 inches diameter round bars and various size single angle web members.

The historical data from SJI designates bar joists with similar cord and web elements as 28K9 joists. At 41 ft – 4 in span and spacing of 5'-8", the 28K9 joist can support a maximum of 335 pounds per linear foot (plf) or 59 psf uniform roof load. The total dead and snow loads over the storage area are 35 psf. The existing joists over the storage area can support solar panel systems up to 24 psf uniform loads, beyond the assumed dead and snow load.

ROOF DECK

The roof deck is a 1.5-inch type B wide rib (see picture 13). We were not able to measure the thickness of the deck. Based on the joist spacing, it is most likely that the deck is 22 gage with a minimum of three continuous spans. Under these assumptions, the deck can carry more than 70 psf uniform loads. The deck's total uniform load (deck, roofing, insulation, and snow load), plus 12 psf for solar panel load, is approximately 38 psf. The deck capacity can support the additional load from solar panels.

STEEL JOIST GIRDERS

The 36-inch deep steel joist girders, which support the roof joists, are comprised of double angles for top and bottom chords. Double or single angles are used for the web members. The panel points are about 2 ft – 10 in at the top chord and approximately 5'- 8" at the bottom chord. The joist girders span 45 feet from column to column. Based on our field measurements, we determined that the joist girders constructed of the following steel members:

Top Chord	(2) L 4x4x 0.53
Bottom Chord	(2) L 4x4x 0.54
Web 1 at each end	(2) L 2x2x 0.33
Web 2 vertical	(1) L 1x1x 0.11
Web 3 diagonals compression	(2) L 3x3x 0.31
Web 4 diagonals tension	(2) L 2x2x 0.23
Web 5 vertical	(1) L 1x1x 0.11
Web 6 diagonal compression	(2) L 2 ½ x2 ½ x 0.24
Web 7 diagonal tention	(2) L 1 ¾ x 1 ¾ x 0.14
Web 8 vertical	(1) L 1 ½ x 1 ½ x 0.14
Web 9 diagonal compression	(2) L 2x2x 0.26
Web 10 diagonal tension	(1) L 2x2x 0.17
Web 11 vertical	(1) L 1 ½ x 1 ½ x 0.17
Web 12 diagonal compression	(2) 1 ¾ x 1 ¾ x 0.15

This building was constructed when the steel joist manuafacturer was transitioning from 36 ksi steel to 50 ksi steel. If the steel grade is 36 ksi, this building cannot support any solar panels. If the steel is of higher grade, the joist girder has less than 5 psf capacity. We have to gather samples for testing to determine the steel's strength.

It is typical to find joist girders with no to little reserve capacity because the joist girders are designed specifically for this building.

STEEL COLUMNS

The interior columns are 8 in x 8in Hollow Structural Section tubes; thus, we could not measure the columns' thickness. Typically, columns have sufficient capacity for additional loads. The interior columns require a minimum capacity of 88,000 pounds, and the perimeter columns need a capacity of 21,700 pounds or larger to be able to support the additional load from solar panels. The interior columns must have a thickness of at least 3/8 in to meet the above requirements. The perimeter columns must have a minimum of 1/4 in thick to support the dead, snow, and solar loads.

If this building is selected for installing the solar panels, an ultrasound instrument must be used to verify the thickness of the columns.

Conclusions

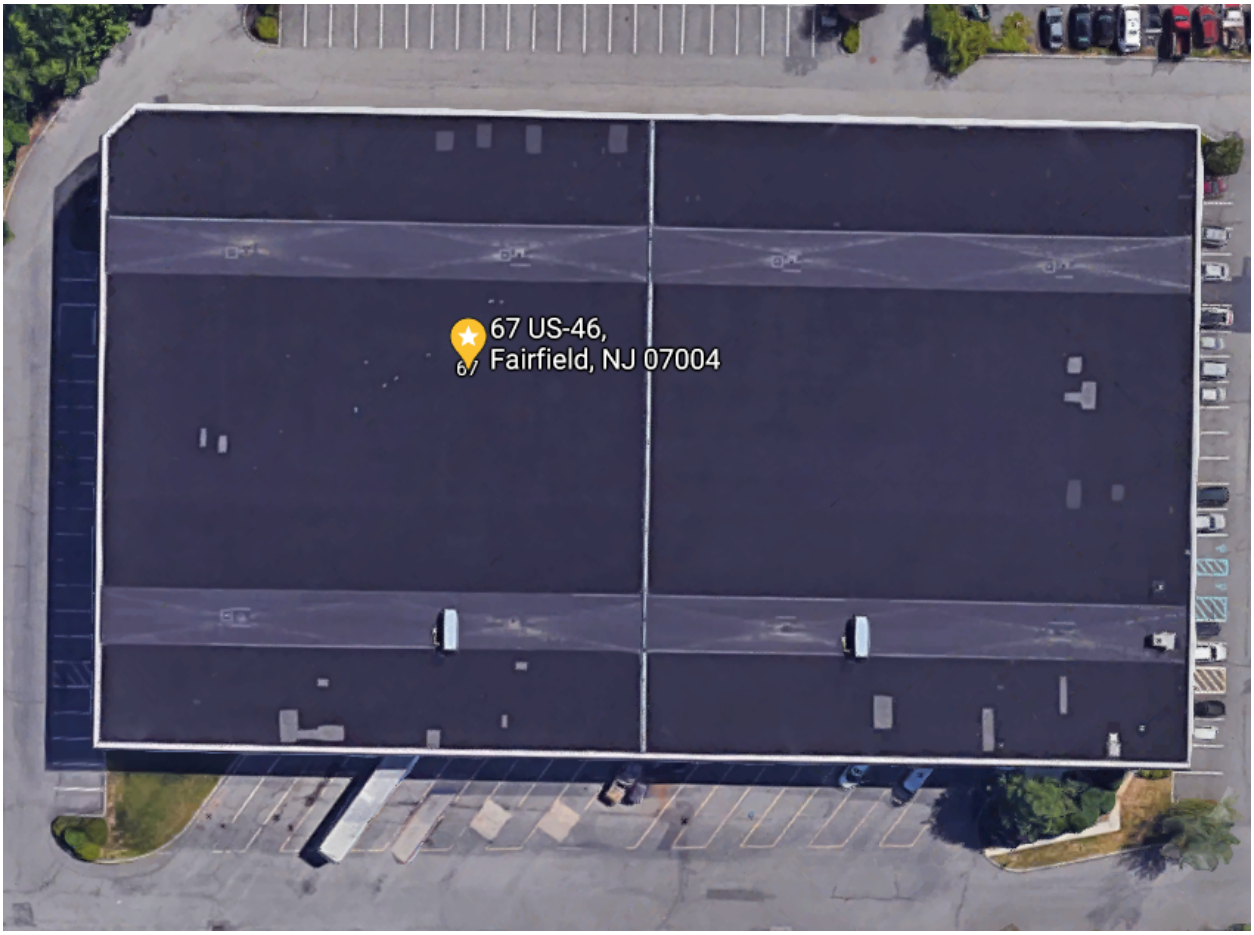
The weakest member in this building is the steel joist girder. If through testing, we find that the steel grade is 50 ksi, a solar panel with 2 psf load that is mechanically attached to the roof deck and joist framing. After obtaining the final layout and attachment loads, further analysis the required to ensure that the roof framing can support the wind uplift loads.

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.

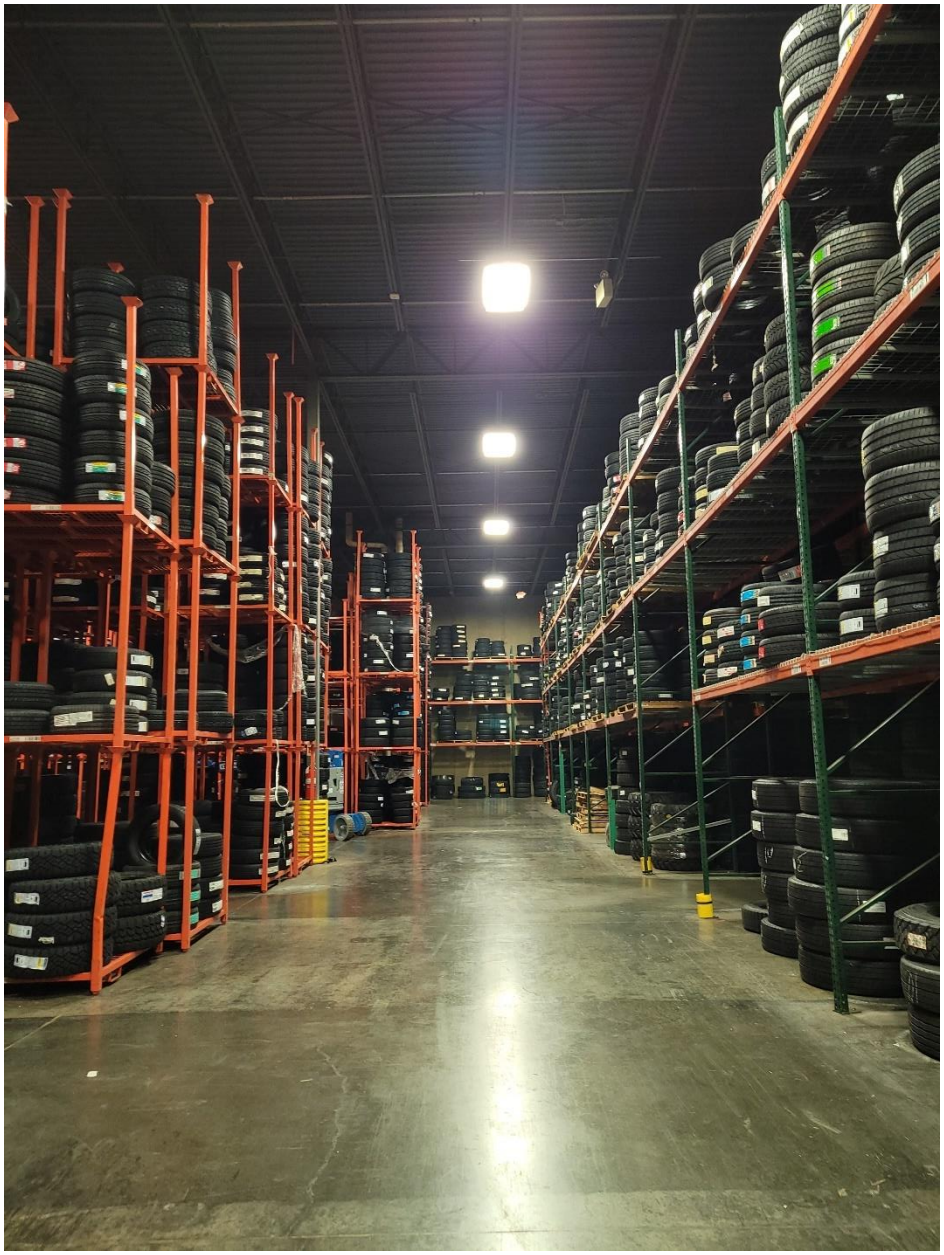
Sincerely,
Grey Fox Consulting, Inc.

Ramin Abbaszadeh, PE, SE
Principal

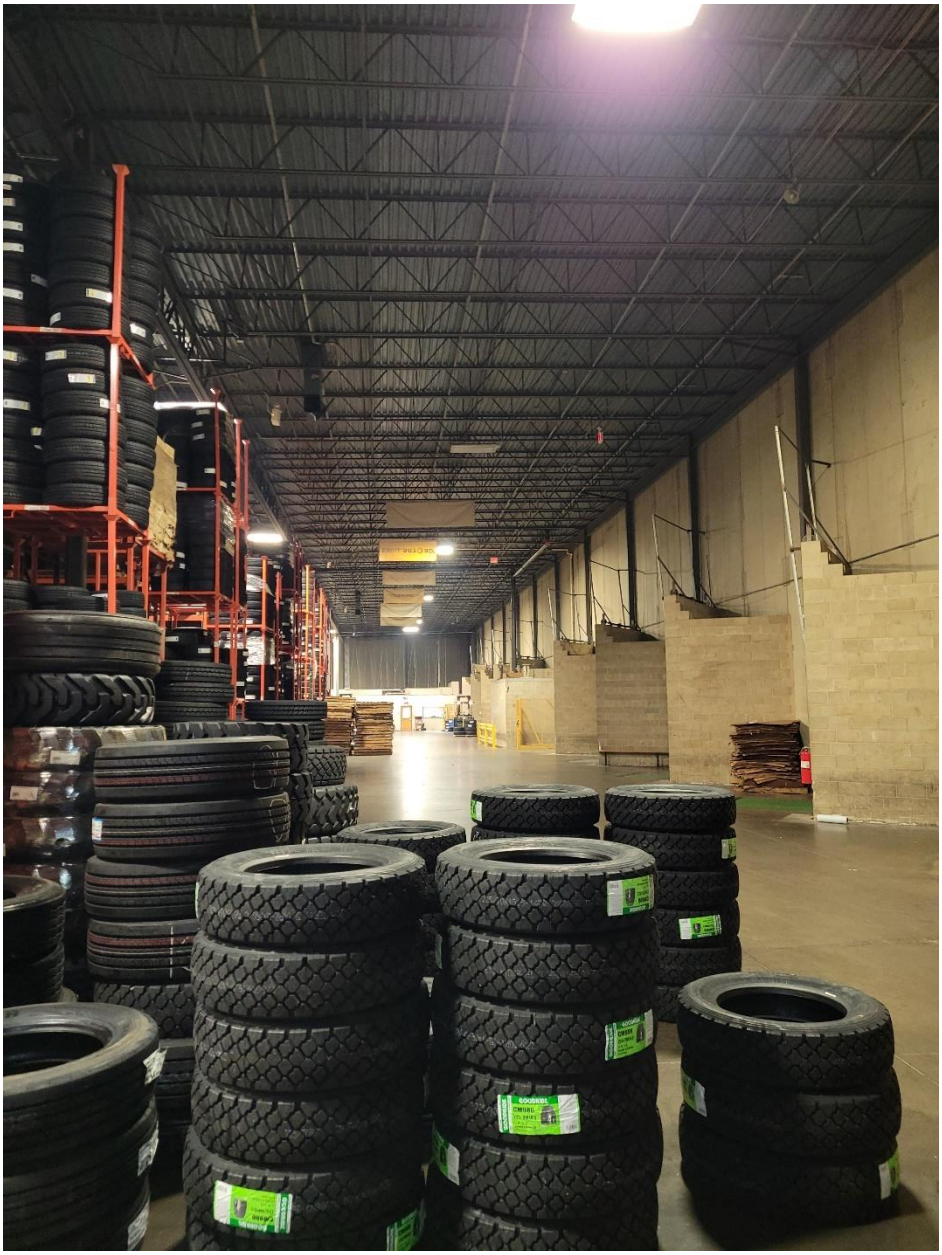


Picture 1

Picture 2



Picture 3



Picture 4

Picture 5

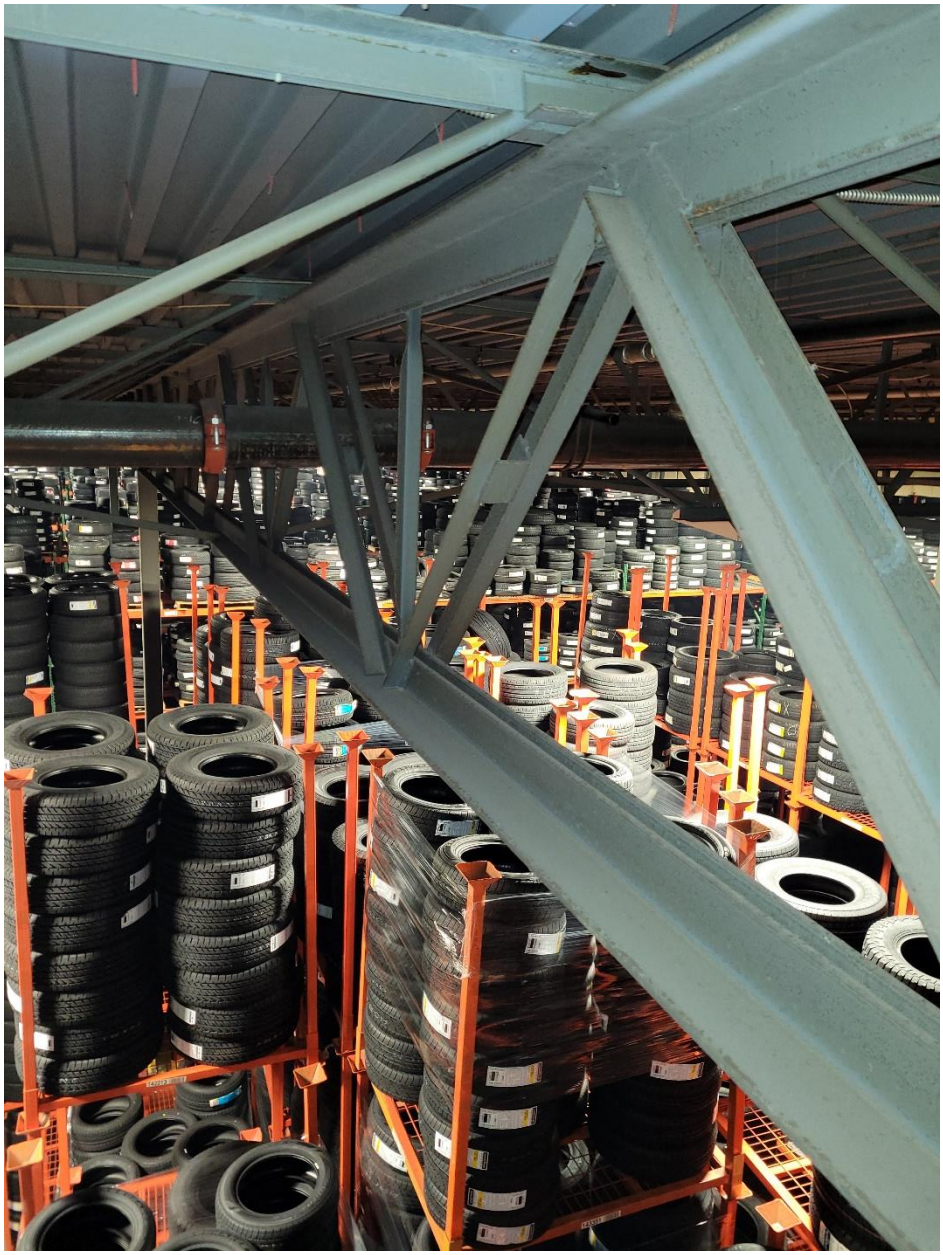
Picture 6

Picture 7

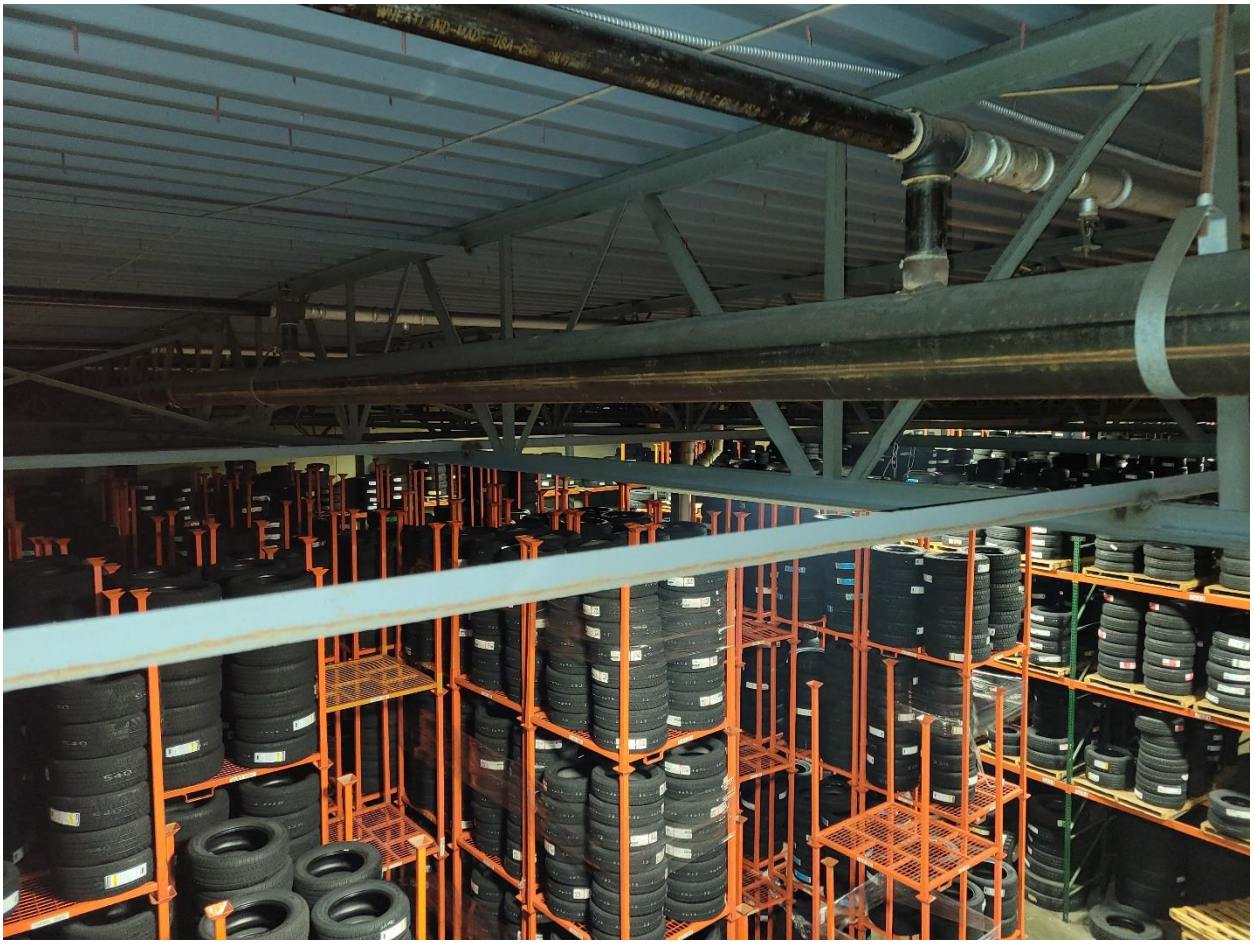
Picture 8

Picture 9

Picture 10



Picture 11



Picture 12

Picture 13

SUMMIT RIDGE ENERGY

67 ROUTE 46
Fairfield, New Jersey

Roof Evaluation
Structural Calculations

GreyFox CONSULTING, Inc.

Structural & Civil
Engineering Solutions

DATE: 12/23/2020
Project No. 20-035
Prepared by: Ramin Abbaszadeh, PE, SE
NJ Lic. No. 34277

Grey Fox Consulting, Inc. 57 Cooper Avenue Cherry Hill, NJ 08002	Project 67 Route 46, Fairfield, New Jersey				Job Ref.	
	Section Snow Load				Sheet no./rev. 1	
	Calc. by RA	Date 12/17/2020	Chk'd by	Date	App'd by	Date

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

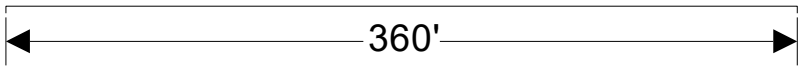
Building details

Roof type	Flat
Width of roof	b = **360.00** ft

Ground snow load

Ground snow load (Figure 7.2-1)	p_g = **30.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 * p_g / 1ft + 14lb/ft^3, 30lb/ft^3)$ = **17.90** lb/ft^3
Terrain typeSect. 26.7	B
Exposure condition (Table 7.3-1)	Fully exposed
Exposure factor (Table 7.3-1)	C_e = **0.90**
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)	$p_{f_min} = I_s * 20$ lb/ft^2 = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 * C_e * C_t * I_s * p_g$ = **18.90** lb/ft^2

Balanced load 20.0 psf



Roof elevation

GreyFox CONSULTING, Inc.
1415 Route 70 E., Ste. 201
Cherry Hill, NJ 08034
Structural & Civil Engineering Solutions
(856) 216-9372
Fax (856) 216-9376

JOB 67 ROUTE 46
SHEET NO. 1 OF
CALCULATED BY KA DATE 12/17/20
CHECKED BY DATE
SCALE

ROOF DEAD LOAD

ROOF DECK/INSUL.	= 6 PSF
SPRINKLER	= 3 PSF
MISC.	= 3 PSF
JOIST	= 3 PSF
	15 PSF
SNOW LOAD	= 20 PSF
TOTAL LOAD	35 PSF

JOIST 28K9 SPAN. ≈ 41'-4"

41' 340 PLF 16/3 = 5.33 PLF

42' 324 PLF

16 PLF 340 − 5.33 = 334.67 PLF

38K9 → 334.67 PLF ⇒ 59 PSF

35 + 12 = 47 < 59 PSF o.k.
DL+SNOW SOLAR

INIT COLS.

(35 + 12)(41.33)(45.33) = 88 K

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JOB 67 ROUTE 46
SHEET NO. 2 OF
CALCULATED BY KA DATE 12/17/20
CHECKED BY DATE
SCALE

EXT COLS.

$$(35+12)\left(\frac{41.33}{2}\right)(22.33) = 21.7 \text{ Kips}$$

Ramin Abbaszadeh

From: Ken Charles <kcharles@steeljoist.org>
Sent: Monday, December 14, 2020 4:36 PM
To: Ramin Abbaszadeh
Subject: RE: Joist Investigation Form submitted at Steel Joist Institute

The joist you have presented resembles a 28K9 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: Monday, December 14, 2020 3:40 PM
To: kcharles@steeljoist.org
Subject: Joist Investigation Form submitted at Steel Joist Institute

Ramin Abbaszadeh submitted a Joist Investigation Form.

Case Number	26165
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	67 Route 46
Why are you requesting this information?	Evaluation
Other	
What year was the building constructed (or approximate age of the structure)?	1995 to 2002
Who was the joist manufacturer?	NCJ Corp. (New Columbia Joist?)
Is there a tag on the joist?	Yes
Yes? Provide tag information	0K0 14AJ1
What type of trusses are the joists?	Modified 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?	41' 4"
What is the joist spacing?	5'-10"
What is the interior panel point spacing?	24" AT TOP, 48" AT BOTTOM
What is the joist depth?	28"
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	2x2
Top chord thickness	0.21"

Top chord diameter	
Bottom chord shape	2 Angles
Bottom chord leg size	1.75x1.75
Bottom chord thickness	0.16
Bottom chord diameter	
Vertical web shape	1 Angle
Vertical web leg size	1X1
Vertical web thickness	0.11 TO 0.15
Vertical web diameter	
Diagonal web shape	1 Angle
Diagonal web leg size	1.5X1.5
Diagonal web thickness	0.15
Diagonal web diameter	
Measurement device	Micrometer
Other device	

STANDARD LOAD TABLE/OPEN WEB STEEL JOISTS, K-SERIES

Based on a Maximum Allowable Tensile Stress of 30 ksi

Joist Designation	24K4	24K5	24K6	24K7	24K8	24K9	24K10	24K12	26K5	26K6	26K7	26K8	26K9	26K10	26K12	28K6	28K7	28K8	28K9	28K10	28K12
Depth (In.)	24	24	24	24	24	24	24	24	26	26	26	26	26	26	26	28	28	28	28	28	28
Approx. Wt. (lbs./ft.)	8.4	9.3	9.7	10.1	11.5	12.0	13.1	16.0	9.8	10.6	10.9	12.1	12.2	13.8	16.6	11.4	11.8	12.7	13.0	14.3	17.1
Span (ft.) ↓																					
24	520 516	550 544	550 544	550 544	550 544	550 544	550 544	550 544													
25	479 456	540 511	550 520	550 520	550 520	550 520	550 520	550 520													
26	442 405	499 453	543 493	550 499	550 499	550 499	550 499	550 499	542 535	550 541	550 541	550 541	550 541	550 541	550 541						
27	410 361	462 404	503 439	550 479	550 479	550 479	550 479	550 479	502 477	547 519	550 522	550 522	550 522	550 522	550 522						
28	381 323	429 362	467 393	521 436	550 456	550 456	550 456	550 456	466 427	508 464	550 501	550 501	550 501	550 501	550 501	548 541	550 543	550 543	550 543	550 543	550 543
29	354 290	400 325	435 354	485 392	536 429	550 436	550 436	550 436	434 384	473 417	527 463	550 479	550 479	550 479	550 479	511 486	550 522	550 522	550 522	550 522	550 522
30	331 262	373 293	406 319	453 353	500 387	544 419	550 422	550 422	405 346	441 377	492 417	544 457	550 459	550 459	550 459	477 439	531 486	550 500	550 500	550 500	550 500
31	310 237	349 266	380 289	424 320	468 350	510 379	550 410	550 410	379 314	413 341	460 378	509 413	550 444	550 444	550 444	446 397	497 440	550 480	550 480	550 480	550 480
32	290 215	327 241	357 262	397 290	439 318	478 344	549 393	549 393	356 285	387 309	432 343	477 375	519 407	549 431	549 431	418 361	466 400	515 438	549 463	549 463	549 463
33	273 196	308 220	335 239	373 265	413 289	449 313	532 368	532 368	334 259	364 282	406 312	448 342	488 370	532 404	532 404	393 329	438 364	484 399	527 432	532 435	532 435
34	257 179	290 201	315 218	351 242	388 264	423 286	502 337	516 344	315 237	343 257	382 285	422 312	459 338	516 378	516 378	370 300	412 333	456 364	496 395	516 410	516 410
35	242 164	273 184	297 200	331 221	366 242	399 262	473 308	501 324	297 217	323 236	360 261	398 286	433 310	501 356	501 356	349 275	389 305	430 333	468 361	501 389	501 389
36	229 150	258 169	281 183	313 203	346 222	377 241	447 283	487 306	280 199	305 216	340 240	376 263	409 284	486 334	487 334	330 252	367 280	406 306	442 332	487 366	487 366
37	216 138	244 155	266 169	296 187	327 205	356 222	423 260	474 290	265 183	289 199	322 221	356 242	387 262	460 308	474 315	312 232	348 257	384 282	418 305	474 344	474 344
38	205 128	231 143	252 156	281 172	310 189	338 204	401 240	461 275	251 169	274 184	305 204	337 223	367 241	436 284	461 299	296 214	329 237	364 260	396 282	461 325	461 325
39	195 118	219 132	239 144	266 159	294 174	320 189	380 222	449 261	238 156	260 170	289 188	320 206	348 223	413 262	449 283	280 198	313 219	346 240	376 260	447 306	449 308
40	185 109	208 122	227 133	253 148	280 161	304 175	361 206	438 247	227 145	247 157	275 174	304 191	331 207	393 243	438 269	266 183	297 203	328 222	357 241	424 284	438 291
41	176 101	198 114	216 124	241 137	266 150	290 162	344 191	427 235	215 134	235 146	262 162	289 177	315 192	374 225	427 256	253 170	283 189	312 206	340 224	404 263	427 277
42	168 94	189 106	206 115	229 127	253 139	276 151	327 177	417 224	205 125	224 136	249 150	275 164	300 178	356 210	417 244	241 158	269 175	297 192	324 208	384 245	417 264
43	160 88	180 98	196 107	219 118	242 130	263 140	312 165	406 213	196 116	213 126	238 140	263 153	286 166	339 195	407 232	230 147	257 163	284 179	309 194	367 228	407 252
44	153 82	172 92	187 100	209 110	231 121	251 131	298 154	387 199	187 108	204 118	227 131	251 143	273 155	324 182	398 222	220 137	245 152	271 167	295 181	350 212	398 240
45	146 76	164 86	179 93	199 103	220 113	240 122	285 144	370 185	179 101	194 110	217 122	240 133	261 145	310 170	389 212	210 128	234 142	259 156	282 169	334 198	389 229
46	139 71	157 80	171 87	191 97	211 106	230 114	272 135	354 174	171 95	186 103	207 114	229 125	250 135	296 159	380 203	201 120	224 133	248 146	270 158	320 186	380 219
47	133 67	150 75	164 82	183 90	202 99	220 107	261 126	339 163	164 89	178 96	199 107	219 117	239 127	284 149	369 192	192 112	214 125	237 136	258 148	306 174	372 210
48	128 63	144 70	157 77	175 85	194 93	211 101	250 118	325 153	157 83	171 90	190 100	210 110	229 119	272 140	353 180	184 105	206 117	227 128	247 139	294 163	365 201
49									150 78	164 85	183 94	202 103	220 112	261 131	339 169	177 99	197 110	218 120	237 130	282 153	357 193
50									144 73	157 80	175 89	194 97	211 105	250 124	325 159	170 93	189 103	209 113	228 123	270 144	350 185
51									139 69	151 75	168 83	186 91	203 99	241 116	313 150	163 88	182 97	201 106	219 115	260 136	338 175
52									133 65	145 71	162 79	179 86	195 93	231 110	301 142	157 83	175 92	193 100	210 109	250 128	325 165
53																151 78	168 87	186 95	203 103	240 121	313 156
54																145 74	162 82	179 89	195 97	232 114	301 147
55																140 70	156 77	173 85	188 92	223 108	290 139
56																135 66	151 73	166 80	181 87	215 102	280 132

Current Date: 12/24/2020 11:41 AM
Units system: English
File name: S:\Projects\20-039 Summit Ridge NJ Roof Evaluation\67 Rt 46\GIRDER 50 ksi.retx

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	3.333	0.00	0.00	0
3	8.17	0.00	0.00	0
4	13.83	0.00	0.00	0
5	19.50	0.00	0.00	0
6	25.17	0.00	0.00	0
7	30.83	0.00	0.00	0
8	36.50	0.00	0.00	0
9	41.333	0.00	0.00	0
10	44.67	0.00	0.00	0
11	0.00	3.00	0.00	0
12	3.333	3.00	0.00	0
13	5.3333	3.00	0.00	0
14	8.17	3.00	0.00	0
15	11.00	3.00	0.00	0
16	13.83	3.00	0.00	0
17	16.67	3.00	0.00	0
18	19.50	3.00	0.00	0
19	22.333	3.00	0.00	0
20	25.17	3.00	0.00	0
21	28.00	3.00	0.00	0
22	30.83	3.00	0.00	0

23	33.67	3.00	0.00	0
24	36.50	3.00	0.00	0
25	39.333	3.00	0.00	0
26	41.333	3.00	0.00	0
27	44.67	3.00	0.00	0

Restraints

Node	TX	TY	TZ	RX	RY	RZ
11	1	1	1	0	0	0
27	0	1	1	0	0	0

Members

Member	NJ	NK	Description	Section	Material	d0 [in]	dL [in]	lg factor
1	11	12		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
2	12	13		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
3	13	14		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
4	14	15		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
5	15	16		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
6	16	17		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
7	17	18		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
8	18	19		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
9	19	20		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
10	20	21		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
11	21	22		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
12	22	23		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
13	23	24		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
14	24	25		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
15	25	26		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
16	26	27		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
17	1	2		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
18	2	3		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
19	3	4		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
20	4	5		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
21	5	6		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
22	6	7		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
23	7	8		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
24	8	9		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
25	9	10		T2L 4X4X1_2X3_4	A572 Gr50	0.00	0.00	0.00
26	2	12		L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
27	3	14		L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
28	4	16		L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
29	5	18		L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
30	6	20		L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
31	7	22		L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
32	8	24		L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
33	9	26		L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
34	11	2		T2L 2X2X5_16X3_4	A572 Gr50	0.00	0.00	0.00
35	13	3		T2L 2X2X1_4X3_4	A572 Gr50	0.00	0.00	0.00

36	15	4	T2L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
37	17	5	L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
38	19	6	T2L 2X2X1_8X3_4	A572 Gr50	0.00	0.00	0.00
39	21	7	T2L 2X2X1_4X3_4	A572 Gr50	0.00	0.00	0.00
40	23	8	T2L 2-1_2X2-1_2X3_16...	A572 Gr50	0.00	0.00	0.00
41	25	9	T2L 3X3X5_16X3_4	A572 Gr50	0.00	0.00	0.00
42	2	13	T2L 3X3X5_16X3_4	A572 Gr50	0.00	0.00	0.00
43	3	15	T2L 2-1_2X2-1_2X3_16...	A572 Gr50	0.00	0.00	0.00
44	4	17	T2L 2X2X1_4X3_4	A572 Gr50	0.00	0.00	0.00
45	5	19	T2L 2X2X1_8X3_4	A572 Gr50	0.00	0.00	0.00
46	6	21	L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
47	7	23	T2L 2X2X1_8	A572 Gr50	0.00	0.00	0.00
48	8	25	T2L 2X2X1_4X3_4	A572 Gr50	0.00	0.00	0.00
49	9	27	T2L 2X2X5_16X3_4	A572 Gr50	0.00	0.00	0.00

Orientation of local axes

Member	Rotation [Deg]	Axes23	NX	NY	NZ
17	180.00	0	0.00	0.00	0.00
18	180.00	0	0.00	0.00	0.00
19	180.00	0	0.00	0.00	0.00
20	180.00	0	0.00	0.00	0.00
21	180.00	0	0.00	0.00	0.00
22	180.00	0	0.00	0.00	0.00
23	180.00	0	0.00	0.00	0.00
24	180.00	0	0.00	0.00	0.00
25	180.00	0	0.00	0.00	0.00

Hinges

	Node-J				Node-K						
Member	M33	M22	V3	V2	M33	M22	V3	V2	TOR	AXL	Axial rigidity
26	1	0	0	0	1	0	0	0	0	0	Full
27	1	0	0	0	1	0	0	0	0	0	Full
28	1	0	0	0	1	0	0	0	0	0	Full
29	1	0	0	0	1	0	0	0	0	0	Full
30	1	0	0	0	1	0	0	0	0	0	Full
31	1	0	0	0	1	0	0	0	0	0	Full
32	1	0	0	0	1	0	0	0	0	0	Full
33	1	0	0	0	1	0	0	0	0	0	Full
34	1	0	0	0	1	0	0	0	0	0	Full
35	1	0	0	0	1	0	0	0	0	0	Full
36	1	0	0	0	1	0	0	0	0	0	Full
37	1	0	0	0	1	0	0	0	0	0	Full
38	1	0	0	0	1	0	0	0	0	0	Full
39	1	0	0	0	1	0	0	0	0	0	Full
40	1	0	0	0	1	0	0	0	0	0	Full
41	1	0	0	0	1	0	0	0	0	0	Full
42	1	0	0	0	1	0	0	0	0	0	Full
43	1	0	0	0	1	0	0	0	0	0	Full

44	1	0	0	0	1	0	0	0	0	0	Full
45	1	0	0	0	1	0	0	0	0	0	Full
46	1	0	0	0	1	0	0	0	0	0	Full
47	1	0	0	0	1	0	0	0	0	0	Full
48	1	0	0	0	1	0	0	0	0	0	Full
49	1	0	0	0	1	0	0	0	0	0	Full

Current Date: 12/24/2020 11:42 AM
Units system: English
File name: S:\Projects\20-039 Summit Ridge NJ Roof Evaluation\67 Rt 46\GIRDER 50 ksi.retx

Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SOLAR	Solar Load	No	DL
D1	DL	Yes	
D2	DL+SOLAR	Yes	

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	13	0.00	-8.23	0.00	0.00	0.00	0.00
	15	0.00	-8.23	0.00	0.00	0.00	0.00
	17	0.00	-8.23	0.00	0.00	0.00	0.00
	19	0.00	-8.23	0.00	0.00	0.00	0.00
	21	0.00	-8.23	0.00	0.00	0.00	0.00
	23	0.00	-8.23	0.00	0.00	0.00	0.00
	25	0.00	-8.23	0.00	0.00	0.00	0.00
SOLAR	13	0.00	-2.82	0.00	0.00	0.00	0.00
	15	0.00	-2.82	0.00	0.00	0.00	0.00
	17	0.00	-2.82	0.00	0.00	0.00	0.00
	19	0.00	-2.82	0.00	0.00	0.00	0.00
	21	0.00	-2.82	0.00	0.00	0.00	0.00
	23	0.00	-2.82	0.00	0.00	0.00	0.00
	25	0.00	-2.82	0.00	0.00	0.00	0.00

Self weight multipliers for load conditions

Condition	Description	Comb.	Self weight multiplier MultX	MultY	MultZ
DL	Dead Load	No	0.00	0.00	0.00
SOLAR	Solar Load	No	0.00	0.00	0.00
D1	DL	Yes	0.00	0.00	0.00
D2	DL+SOLAR	Yes	0.00	0.00	0.00

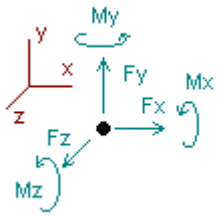
Earthquake (Dynamic analysis only)

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

Current Date: 12/24/2020 11:43 AM
Units system: English
File name: S:\Projects\20-039 Summit Ridge NJ Roof Evaluation\67 Rt 46\GIRDER 50 ksi.retx

Analysis result

Reactions



Direction of positive forces and moments

Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **D1=DL**						
11	0.00000	28.80605	0.00000	0.00000	0.00000	0.00000
27	0.00000	28.80395	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	57.61000	0.00000	0.00000	0.00000	0.00000
Condition **D2=DL+SOLAR**						
11	0.00000	38.67641	0.00000	0.00000	0.00000	0.00000
27	0.00000	38.67359	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	77.35000	0.00000	0.00000	0.00000	0.00000

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	-31.72	0.00	0.28	0.00	0.08	0.00
Min	-31.72	0.00	0.28	0.00	-0.85	0.00
MEMBER **2**						
Max	-31.73	0.00	-0.32	0.00	-0.21	0.00
Min	-31.73	0.00	-0.32	0.00	-0.84	0.00
MEMBER **3**						
Max	-70.33	0.00	0.27	0.00	0.26	0.00
Min	-70.33	0.00	0.27	0.00	-0.52	0.00
MEMBER **4**						
Max	-70.34	0.00	0.02	0.00	-0.50	0.00
Min	-70.34	0.00	0.02	0.00	-0.57	0.00
MEMBER **5**						
Max	-100.96	0.00	0.34	0.00	-0.35	0.00
Min	-100.96	0.00	0.34	0.00	-1.31	0.00
MEMBER **6**						
Max	-100.97	0.00	-0.27	0.00	-0.51	0.00
Min	-100.97	0.00	-0.27	0.00	-1.29	0.00

MEMBER **7**						
Max	-116.55	0.00	0.23	0.00	-0.45	0.00
Min	-116.55	0.00	0.23	0.00	-1.11	0.00
MEMBER **8**						
Max	-116.55	0.00	0.17	0.00	-1.10	0.00
Min	-116.55	0.00	0.17	0.00	-1.58	0.00
MEMBER **9**						
Max	-116.55	0.00	-0.17	0.00	-1.10	0.00
Min	-116.55	0.00	-0.17	0.00	-1.58	0.00
MEMBER **10**						
Max	-116.55	0.00	-0.23	0.00	-0.45	0.00
Min	-116.55	0.00	-0.23	0.00	-1.11	0.00
MEMBER **11**						
Max	-101.01	0.00	0.28	0.00	-0.51	0.00
Min	-101.01	0.00	0.28	0.00	-1.30	0.00
MEMBER **12**						
Max	-101.00	0.00	-0.34	0.00	-0.35	0.00
Min	-101.00	0.00	-0.34	0.00	-1.31	0.00
MEMBER **13**						
Max	-70.35	0.00	-0.02	0.00	-0.50	0.00
Min	-70.35	0.00	-0.02	0.00	-0.57	0.00
MEMBER **14**						
Max	-70.33	0.00	-0.27	0.00	0.26	0.00
Min	-70.33	0.00	-0.27	0.00	-0.52	0.00
MEMBER **15**						
Max	-31.76	0.00	0.32	0.00	-0.21	0.00
Min	-31.76	0.00	0.32	0.00	-0.84	0.00
MEMBER **16**						
Max	-31.75	0.00	-0.28	0.00	0.08	0.00
Min	-31.75	0.00	-0.28	0.00	-0.85	0.00
MEMBER **17**						
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 **18**						
Max	51.26	0.00	-0.20	0.00	0.57	0.00
Min	51.26	0.00	-0.20	0.00	-0.38	0.00
MEMBER **19**						
Max	89.72	0.00	-0.13	0.00	1.02	0.00
Min	89.72	0.00	-0.13	0.00	0.27	0.00
MEMBER **20**						
Max	112.93	0.00	-0.05	0.00	1.13	0.00
Min	112.93	0.00	-0.05	0.00	0.83	0.00
MEMBER **21**						
Max	120.28	0.00	0.00	0.00	1.10	0.00
Min	120.28	0.00	0.00	0.00	1.09	0.00
MEMBER **22**						
Max	112.93	0.00	0.05	0.00	1.13	0.00
Min	112.93	0.00	0.05	0.00	0.84	0.00
MEMBER **23**						
Max	89.72	0.00	0.13	0.00	1.02	0.00
Min	89.72	0.00	0.13	0.00	0.27	0.00
MEMBER **24**						
Max	51.29	0.00	0.20	0.00	0.57	0.00
Min	51.29	0.00	0.20	0.00	-0.38	0.00
MEMBER **25**						
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 **26**						
Max	0.59	0.00	-0.01	0.00	0.02	0.00

Min	0.59	0.00	-0.01	0.00	-0.01	0.00
MEMBER **27**						
Max	0.25	0.00	-0.02	0.00	0.02	0.00
Min	0.25	0.00	-0.02	0.00	-0.02	0.00
MEMBER **28**						
Max	0.61	0.00	-0.01	0.00	0.02	0.00
Min	0.61	0.00	-0.01	0.00	-0.02	0.00
MEMBER **29**						
Max	0.06	0.00	-0.01	0.00	0.01	0.00
Min	0.06	0.00	-0.01	0.00	-0.01	0.00
MEMBER **30**						
Max	0.06	0.00	0.01	0.00	0.01	0.00
Min	0.06	0.00	0.01	0.00	-0.01	0.00
MEMBER **31**						
Max	0.61	0.00	0.01	0.00	0.02	0.00
Min	0.61	0.00	0.01	0.00	-0.02	0.00
MEMBER **32**						
Max	0.25	0.00	0.02	0.00	0.02	0.00
Min	0.25	0.00	0.02	0.00	-0.02	0.00
MEMBER **33**						
Max	0.59	0.00	0.01	0.00	0.01	0.00
Min	0.59	0.00	0.01	0.00	-0.02	0.00
MEMBER **34**						
Max	42.66	0.00	-0.02	0.00	0.00	0.00
Min	42.66	0.00	-0.02	0.00	-0.08	0.00
MEMBER **35**						
Max	27.71	0.00	-0.04	0.00	0.07	0.00
Min	27.71	0.00	-0.04	0.00	-0.09	0.00
MEMBER **36**						
Max	16.35	0.00	-0.03	0.00	0.03	0.00
Min	16.35	0.00	-0.03	0.00	-0.07	0.00
MEMBER **37**						
Max	5.27	0.00	0.00	0.00	0.00	0.00
Min	5.27	0.00	0.00	0.00	-0.01	0.00
MEMBER **38**						
Max	-5.43	0.00	0.00	0.00	-0.03	0.00
Min	-5.43	0.00	0.00	0.00	-0.03	0.00
MEMBER **39**						
Max	-17.33	0.00	0.05	0.00	0.06	0.00
Min	-17.33	0.00	0.05	0.00	-0.14	0.00
MEMBER **40**						
Max	-28.15	0.00	0.08	0.00	0.14	0.00
Min	-28.15	0.00	0.08	0.00	-0.21	0.00
MEMBER **41**						
Max	-34.90	0.00	0.21	0.00	0.38	0.00
Min	-34.90	0.00	0.21	0.00	-0.37	0.00
MEMBER **42**						
Max	-34.90	0.00	-0.21	0.00	0.38	0.00
Min	-34.90	0.00	-0.21	0.00	-0.37	0.00
MEMBER **43**						
Max	-28.15	0.00	-0.08	0.00	0.14	0.00
Min	-28.15	0.00	-0.08	0.00	-0.21	0.00
MEMBER **44**						
Max	-17.35	0.00	-0.05	0.00	0.06	0.00
Min	-17.35	0.00	-0.05	0.00	-0.14	0.00
MEMBER **45**						
Max	-5.43	0.00	0.00	0.00	-0.03	0.00
Min	-5.43	0.00	0.00	0.00	-0.03	0.00
MEMBER **46**						

Max	5.27	0.00	0.00	0.00	0.00	0.00
Min	5.27	0.00	0.00	0.00	-0.01	0.00
MEMBER **47**						
Max	16.37	0.00	0.03	0.00	0.03	0.00
Min	16.37	0.00	0.03	0.00	-0.07	0.00
MEMBER **48**						
Max	27.69	0.00	0.04	0.00	0.07	0.00
Min	27.69	0.00	0.04	0.00	-0.09	0.00
MEMBER **49**						
Max	42.69	0.00	0.02	0.00	0.00	0.00
Min	42.69	0.00	0.02	0.00	-0.08	0.00

Condition : **D2=DL+SOLAR**

	Axial	**Shear V2**	**Shear V3**	**Torsion**	**M22**	**M33**
	[Kip]	[Kip]	[Kip]	[Kip*ft]	[Kip*ft]	[Kip*ft]
MEMBER **1**						
Max	-42.59	0.00	0.37	0.00	0.10	0.00
Min	-42.59	0.00	0.37	0.00	-1.14	0.00
MEMBER **2**						
Max	-42.60	0.00	-0.42	0.00	-0.28	0.00
Min	-42.60	0.00	-0.42	0.00	-1.13	0.00
MEMBER **3**						
Max	-94.43	0.00	0.37	0.00	0.35	0.00
Min	-94.43	0.00	0.37	0.00	-0.70	0.00
MEMBER **4**						
Max	-94.44	0.00	0.03	0.00	-0.67	0.00
Min	-94.44	0.00	0.03	0.00	-0.76	0.00
MEMBER **5**						
Max	-135.55	0.00	0.45	0.00	-0.48	0.00
Min	-135.55	0.00	0.45	0.00	-1.75	0.00
MEMBER **6**						
Max	-135.56	0.00	-0.37	0.00	-0.69	0.00
Min	-135.56	0.00	-0.37	0.00	-1.73	0.00
MEMBER **7**						
Max	-156.48	0.00	0.31	0.00	-0.60	0.00
Min	-156.48	0.00	0.31	0.00	-1.49	0.00
MEMBER **8**						
Max	-156.49	0.00	0.23	0.00	-1.47	0.00
Min	-156.49	0.00	0.23	0.00	-2.12	0.00
MEMBER **9**						
Max	-156.49	0.00	-0.23	0.00	-1.47	0.00
Min	-156.49	0.00	-0.23	0.00	-2.12	0.00
MEMBER **10**						
Max	-156.48	0.00	-0.31	0.00	-0.60	0.00
Min	-156.48	0.00	-0.31	0.00	-1.49	0.00
MEMBER **11**						
Max	-135.62	0.00	0.37	0.00	-0.69	0.00
Min	-135.62	0.00	0.37	0.00	-1.74	0.00
MEMBER **12**						
Max	-135.61	0.00	-0.45	0.00	-0.47	0.00
Min	-135.61	0.00	-0.45	0.00	-1.76	0.00
MEMBER **13**						
Max	-94.45	0.00	-0.03	0.00	-0.67	0.00
Min	-94.45	0.00	-0.03	0.00	-0.76	0.00
MEMBER **14**						
Max	-94.43	0.00	-0.37	0.00	0.34	0.00
Min	-94.43	0.00	-0.37	0.00	-0.70	0.00

MEMBER **15**						
Max	-42.65	0.00	0.42	0.00	-0.28	0.00
Min	-42.65	0.00	0.42	0.00	-1.13	0.00
MEMBER **16**						
Max	-42.64	0.00	-0.37	0.00	0.10	0.00
Min	-42.64	0.00	-0.37	0.00	-1.15	0.00
MEMBER **17**						
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 **18**						
Max	68.83	0.00	-0.26	0.00	0.76	0.00
Min	68.83	0.00	-0.26	0.00	-0.51	0.00
MEMBER **19**						
Max	120.46	0.00	-0.18	0.00	1.37	0.00
Min	120.46	0.00	-0.18	0.00	0.36	0.00
MEMBER **20**						
Max	151.63	0.00	-0.07	0.00	1.51	0.00
Min	151.63	0.00	-0.07	0.00	1.12	0.00
MEMBER **21**						
Max	161.50	0.00	0.00	0.00	1.47	0.00
Min	161.50	0.00	0.00	0.00	1.47	0.00
MEMBER **22**						
Max	151.63	0.00	0.07	0.00	1.51	0.00
Min	151.63	0.00	0.07	0.00	1.12	0.00
MEMBER **23**						
Max	120.47	0.00	0.18	0.00	1.38	0.00
Min	120.47	0.00	0.18	0.00	0.36	0.00
MEMBER **24**						
Max	68.87	0.00	0.26	0.00	0.76	0.00
Min	68.87	0.00	0.26	0.00	-0.51	0.00
MEMBER **25**						
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 **26**						
Max	0.80	0.00	-0.01	0.00	0.02	0.00
Min	0.80	0.00	-0.01	0.00	-0.02	0.00
MEMBER **27**						
Max	0.34	0.00	-0.02	0.00	0.03	0.00
Min	0.34	0.00	-0.02	0.00	-0.03	0.00
MEMBER **28**						
Max	0.82	0.00	-0.02	0.00	0.03	0.00
Min	0.82	0.00	-0.02	0.00	-0.03	0.00
MEMBER **29**						
Max	0.09	0.00	-0.01	0.00	0.02	0.00
Min	0.09	0.00	-0.01	0.00	-0.01	0.00
MEMBER **30**						
Max	0.09	0.00	0.01	0.00	0.01	0.00
Min	0.09	0.00	0.01	0.00	-0.02	0.00
MEMBER **31**						
Max	0.82	0.00	0.02	0.00	0.03	0.00
Min	0.82	0.00	0.02	0.00	-0.03	0.00
MEMBER **32**						
Max	0.34	0.00	0.02	0.00	0.03	0.00
Min	0.34	0.00	0.02	0.00	-0.03	0.00
MEMBER **33**						
Max	0.80	0.00	0.01	0.00	0.02	0.00
Min	0.80	0.00	0.01	0.00	-0.02	0.00
MEMBER **34**						
Max	57.28	0.00	-0.02	0.00	0.00	0.00

Min	57.28	0.00	-0.02	0.00	-0.10	0.00
MEMBER **35**						
Max	37.20	0.00	-0.05	0.00	0.10	0.00
Min	37.20	0.00	-0.05	0.00	-0.12	0.00
MEMBER **36**						
Max	21.95	0.00	-0.03	0.00	0.05	0.00
Min	21.95	0.00	-0.03	0.00	-0.10	0.00
MEMBER **37**						
Max	7.07	0.00	0.00	0.00	0.00	0.00
Min	7.07	0.00	0.00	0.00	-0.01	0.00
MEMBER **38**						
Max	-7.29	0.00	0.00	0.00	-0.03	0.00
Min	-7.29	0.00	0.00	0.00	-0.04	0.00
MEMBER **39**						
Max	-23.26	0.00	0.06	0.00	0.08	0.00
Min	-23.26	0.00	0.06	0.00	-0.19	0.00
MEMBER **40**						
Max	-37.79	0.00	0.11	0.00	0.19	0.00
Min	-37.79	0.00	0.11	0.00	-0.28	0.00
MEMBER **41**						
Max	-46.86	0.00	0.28	0.00	0.51	0.00
Min	-46.86	0.00	0.28	0.00	-0.50	0.00
MEMBER **42**						
Max	-46.86	0.00	-0.28	0.00	0.51	0.00
Min	-46.86	0.00	-0.28	0.00	-0.50	0.00
MEMBER **43**						
Max	-37.80	0.00	-0.11	0.00	0.19	0.00
Min	-37.80	0.00	-0.11	0.00	-0.28	0.00
MEMBER **44**						
Max	-23.30	0.00	-0.06	0.00	0.08	0.00
Min	-23.30	0.00	-0.06	0.00	-0.19	0.00
MEMBER **45**						
Max	-7.29	0.00	0.00	0.00	-0.03	0.00
Min	-7.29	0.00	0.00	0.00	-0.03	0.00
MEMBER **46**						
Max	7.07	0.00	0.00	0.00	0.00	0.00
Min	7.07	0.00	0.00	0.00	-0.01	0.00
MEMBER **47**						
Max	21.98	0.00	0.03	0.00	0.05	0.00
Min	21.98	0.00	0.03	0.00	-0.10	0.00
MEMBER **48**						
Max	37.18	0.00	0.05	0.00	0.10	0.00
Min	37.18	0.00	0.05	0.00	-0.12	0.00
MEMBER **49**						
Max	57.31	0.00	0.02	0.00	0.00	0.00
Min	57.31	0.00	0.02	0.00	-0.10	0.00

Grey Fox Consulting, Inc. 57 Cooper Avenue Cherry Hill, NJ 08002	Project 67 Route 46, Fairfield, New Jersey				Job Ref.	
	Section Girder Bot Chord				Sheet no./rev. 1	
	Calc. by RA	Date 12/17/2020	Chk'd by	Date	App'd by	Date

STEEL MEMBER DESIGN (AISC 360)

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

Tedds calculation version 4.4.05

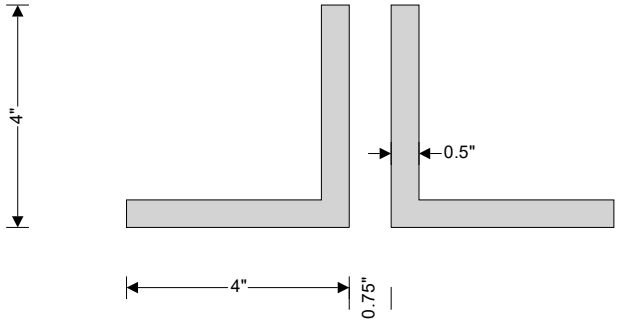
Safety factors

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 4x4x1/2x3/4 (AISC 15th Edn (v15.0))
ASTM steel designation	A572 Gr.50
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



2L 4x4x1/2x3/4 (AISC 15th Edn (v15.0))
Section depth, L_1, 4 in
Section breadth, L_2, 4 in
Weight of section, Weight, 25.6 lbf/ft
Section thickness, t, 0.5 in
Area of section, A, 7.5 in^2
Radius of gyration about x-axis, r_x, 1.21 in
Radius of gyration about y-axis, r_y, 1.97 in
Elastic section modulus about x-axis, S_x, 3.92 in^3
Second moment of area about x-axis, I_x, 11 in^4
Second moment of area about y-axis, I_y, 29.1 in^4

Analysis results

Required tensile strength P_r = **161.05** kips

Restraint spacing

Major axis lateral restraint	L_x = **6** ft
Minor axis lateral restraint	L_y = **45** ft
Torsional restraint	L_z = **6** ft

Design of members for tension - Chapter D

Required tensile strength P_r = **161.05** kips

Slenderness limitations - Section D1

Slenderness ratio $\lambda = \max(L_x / r_x, L_y / r_y)$ = **274.112**

Tension member slenderness ratio does not exceed recommended limit of 300

Allowable tensile strength - D2

Nominal tensile yielding strength	$P_{n,yld} = F_y * A$ = **375** kips
Nominal tensile rupture strength	$P_{n,r} = F_u * A_e$ = **487.5** kips
Allowable tensile strength	$P_c = \min(P_{n,yld} / \Omega_{t,y}, P_{n,r} / \Omega_{t,r})$ = **224.551** kips
	P_r / P_c = **0.717**

PASS - Nominal tensile strength exceeds required tensile strength

Grey Fox Consulting, Inc. 57 Cooper Avenue Cherry Hill, NJ 08002	Project 67 Route 46, Fairfield, New Jersey				Job Ref.	
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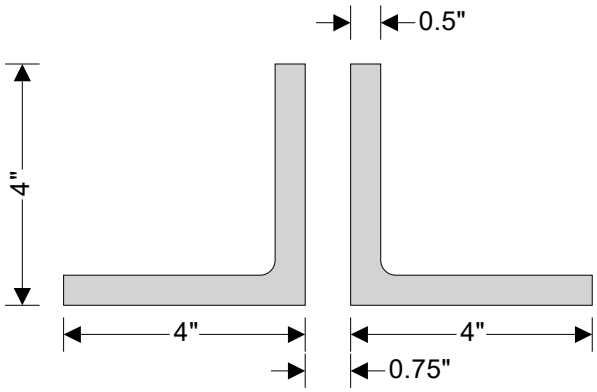
	Calc. by RA	Date 12/17/2020	Chk'd by	Date	App'd by	Date

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 4x4x1/2x3/4**

Design loading

Required axial strength	P_r = **157** kips (Compression)

Material details

Steel grade	**A572 Gr. 50**
Yield strength	F_y = **50** ksi
Ultimate strength	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **72** in
For buckling about y axis	L_y = **72** in
For torsional buckling	L_z = **72** 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	$L_{cx} = L_x \times K_x$ = **72** in
For buckling about y axis	$L_{cy} = L_y \times K_y$ = **72** in
For torsional buckling	$L_{cz} = L_z \times K_z$ = **72** in
Type of connector fixing angles together	**Welded**
Distance between connectors	a = **3.0** in

Section classification

Section classification for local buckling (cl. B4)

Critical width $b = \max(L_1, L_2) =$ **4.000** in

Maximum width to thickness ratio $\lambda = b / t =$ **8.000**

Compression

Limit for nonslender section $\lambda_{r_c} = 0.45 \times \sqrt{(E / F_y)} =$ **10.837**

The section is nonslender in compression

Slenderness

Effective slenderness (cl. E6)

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) =$ **0.78** in

Buckling about axis parallel to unconnected leg $SR_x = L_{cx} / r_x =$ **59.5**

Buckling about axis parallel to connected leg $SR_y = L_{cy} / r_y =$ **36.5**

Check slenderness between connectors (cl. E6.2)

Effective length factor between connectors $K =$ **1.0**

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) =$ **0.78** in

Slenderness between connectors $SR_{con} = K \times a / r_i =$ **3.9**

Maximum allowable slenderness $SR_{allow} = 0.75 \times \max(SR_x, SR_y) =$ **44.6**

PASS - The slenderness between connectors is less than 3/4 of governing slenderness of built-up member

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **80.8** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) * F_y =$ **38.6** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A =$ **289.5** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **214.3** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) * F_y =$ **45.3** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} * A =$ **340.1** kips

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

Elastic torsional buckling stress $F_{ez} = (\pi^2 * E * C_w / (L_{cz})^2 + G * J) / (A * r_o^2) =$ **155.1** ksi

Flexural-torsional elastic buckling stress $F_{et} = (F_{ey} + F_{ez}) / (2 * H) * (1 - \sqrt{(1 - 4 * F_{ey} * F_{ez} * H / (F_{ey} + F_{ez})^2)}) =$ **128.5** ksi

Flexural-torsional buckling stress $F_{crt} = (0.658^{F_y / F_{et}}) * F_y =$ **42.5** ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} * A =$ **318.7** kips

Allowable compressive strength (cl. E1)

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

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c =$ **173.3** kips

PASS - The allowable compressive strength exceeds the required compressive strength

STEEL MEMBER DESIGN (AISC 360)

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

Tedds calculation version 4.4.05

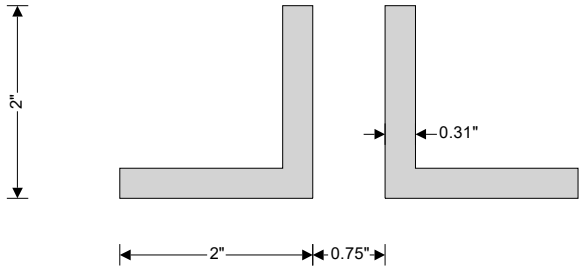
Safety factors

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 2x2x5/16x3/4 (AISC 15th Edn (v15.0))
ASTM steel designation	A572 Gr.50
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



2L 2x2x5/16x3/4 (AISC 15th Edn (v15.0))
Section depth, L_1, 2 in
Section breadth, L_2, 2 in
Weight of section, Weight, 7.8 lbf/ft
Section thickness, t, 0.313 in
Area of section, A, 2.3 in^2
Radius of gyration about x-axis, r_x, 0.598 in
Radius of gyration about y-axis, r_y, 1.15 in
Elastic section modulus about x-axis, S_x, 0.596 in^3
Second moment of area about x-axis, I_x, 0.828 in^4
Second moment of area about y-axis, I_y, 3.07 in^4

Analysis results

Required tensile strength P_r = **57.24** kips

Restraint spacing

Major axis lateral restraint	L_x = **5** ft
Minor axis lateral restraint	L_y = **5** ft
Torsional restraint	L_z = **5** ft

Design of members for tension - Chapter D

Required tensile strength P_r = **57.24** kips

Slenderness limitations - Section D1

Slenderness ratio $\lambda = \max(L_x / r_x, L_y / r_y)$ = **100.334**

Tension member slenderness ratio does not exceed recommended limit of 300

Allowable tensile strength - D2

Nominal tensile yielding strength $P_{n,yld} = F_y * A$ = **116** kips

Nominal tensile rupture strength $P_{n,r} = F_u * A_e$ = **150.8** kips

Allowable tensile strength $P_c = \min(P_{n,yld} / \Omega_{t,y}, P_{n,r} / \Omega_{t,r})$ = **69.461** kips

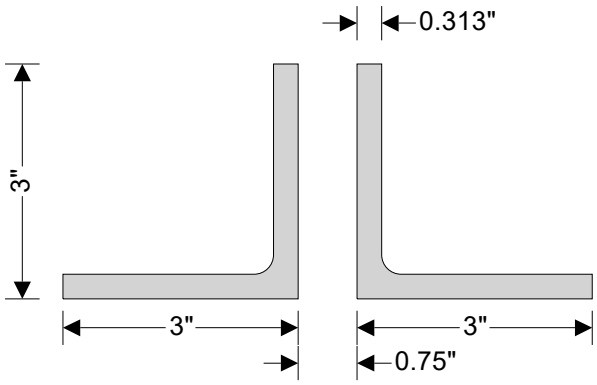
P_r / P_c = **0.824**

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	**2L 3x3x5/16x3/4**

Design loading

Required axial strength	P_r = **47** kips (Compression)

Material details

Steel grade	**A572 Gr. 50**
Yield strength	F_y = **50** ksi
Ultimate strength	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **51** in
For buckling about y axis	L_y = **51** in
For torsional buckling	L_z = **51** 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	$L_{cx} = L_x \times K_x$ = **51** in
For buckling about y axis	$L_{cy} = L_y \times K_y$ = **51** in
For torsional buckling	$L_{cz} = L_z \times K_z$ = **51** in
Type of connector fixing angles together	**Welded**
Distance between connectors	a = **24.0** in

Section classification

Section classification for local buckling (cl. B4)

Critical width $b = \max(L_1, L_2) =$ **3.000** in

Maximum width to thickness ratio $\lambda = b / t =$ **9.585**

Compression

Limit for nonslender section $\lambda_{r_c} = 0.45 \times \sqrt{(E / F_y)} =$ **10.837**

The section is nonslender in compression

Slenderness

Effective slenderness (cl. E6)

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) =$ **0.58** in

Buckling about axis parallel to unconnected leg $SR_x = L_{cx} / r_x =$ **55.6**

Buckling about axis parallel to connected leg $SR_y = \sqrt{((L_{cy} / r_y)^2 + (0.5 * a / r_i)^2)} =$ **39.0**

Check slenderness between connectors (cl. E6.2)

Effective length factor between connectors $K =$ **1.0**

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) =$ **0.58** in

Slenderness between connectors $SR_{con} = K \times a / r_i =$ **41.2**

Maximum allowable slenderness $SR_{allow} = 0.75 \times \max(SR_x, SR_y) =$ **41.7**

PASS - The slenderness between connectors is less than 3/4 of governing slenderness of built-up member

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **92.7** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) * F_y =$ **39.9** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A =$ **142** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **188.3** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) * F_y =$ **44.7** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} * A =$ **159.3** kips

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

Elastic torsional buckling stress $F_{ez} = (\pi^2 * E * C_w / (L_{cz})^2 + G * J) / (A * r_o^2) =$ **100.8** ksi

Flexural-torsional elastic buckling stress $F_{et} = (F_{ey} + F_{ez}) / (2 * H) * (1 - \sqrt{(1 - 4 * F_{ey} * F_{ez} * H / (F_{ey} + F_{ez})^2)}) =$ **89.9** ksi

Flexural-torsional buckling stress $F_{crt} = (0.658^{F_y / F_{et}}) * F_y =$ **39.6** ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} * A =$ **141** kips

Allowable compressive strength (cl. E1)

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

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c =$ **84.4** kips

PASS - The allowable compressive strength exceeds the required compressive strength

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	Section Web 4 Tension				Sheet no./rev. 1	
	Calc. by RA	Date 12/17/2020	Chk'd by	Date	App'd by	Date

STEEL MEMBER DESIGN (AISC 360)

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

Tedds calculation version 4.4.05

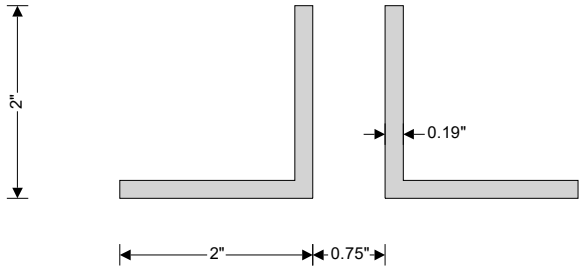
Safety factors

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 2x2x3/16x3/4 (AISC 15th Edn (v15.0))
ASTM steel designation	A572 Gr.50
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



2L 2x2x3/16x3/4 (AISC 15th Edn (v15.0))
Section depth, L_1, 2 in
Section breadth, L_2, 2 in
Weight of section, Weight, 4.9 lbf/ft
Section thickness, t, 0.188 in
Area of section, A, 1.4 in^2
Radius of gyration about x-axis, r_x, 0.612 in
Radius of gyration about y-axis, r_y, 1.12 in
Elastic section modulus about x-axis, S_x, 0.376 in^3
Second moment of area about x-axis, I_x, 0.542 in^4
Second moment of area about y-axis, I_y, 1.81 in^4

Analysis results

Required tensile strength P_r = **36.34** kips

Restraint spacing

Major axis lateral restraint	L_x = **5** ft
Minor axis lateral restraint	L_y = **5** ft
Torsional restraint	L_z = **5** ft

Design of members for tension - Chapter D

Required tensile strength P_r = **36.34** kips

Slenderness limitations - Section D1

Slenderness ratio $\lambda = \max(L_x / r_x, L_y / r_y)$ = **98.039**

Tension member slenderness ratio does not exceed recommended limit of 300

Allowable tensile strength - D2

Nominal tensile yielding strength $P_{n,yld} = F_y * A$ = **72** kips

Nominal tensile rupture strength $P_{n,r} = F_u * A_e$ = **93.6** kips

Allowable tensile strength $P_c = \min(P_{n,yld} / \Omega_{t,y}, P_{n,r} / \Omega_{t,r})$ = **43.114** kips

P_r / P_c = **0.843**

PASS - Nominal tensile strength exceeds required tensile strength

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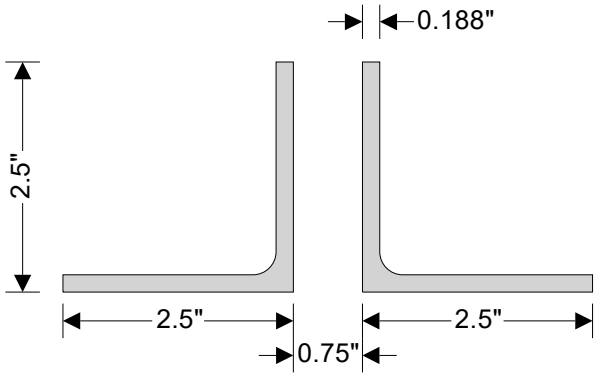
	Section Web 6 Compression				Sheet no./rev. 1	
	Calc. by RA	Date 12/24/2020	Chk'd by	Date	App'd by	Date

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 2-1/2x2-1/2x3/16x3/4**

Design loading

Required axial strength	P_r = **39** kips (Compression)

Material details

Steel grade	**A572 Gr. 50**
Yield strength	F_y = **50** ksi
Ultimate strength	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **51** in
For buckling about y axis	L_y = **51** in
For torsional buckling	L_z = **51** 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	$L_{cx} = L_x \times K_x$ = **51** in
For buckling about y axis	$L_{cy} = L_y \times K_y$ = **51** in
For torsional buckling	$L_{cz} = L_z \times K_z$ = **51** in
Type of connector fixing angles together	**Snug-tight bolts**
Distance between connectors	a = **24.0** in

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 $\lambda = b / t =$ **13.298**

Compression

Limit for nonslender section $\lambda_{r_c} = 0.45 \times \sqrt{(E / F_y)} =$ **10.837**

The section is slender in compression

Slenderness

Effective slenderness (cl. E6)

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) =$ **0.48** in

Buckling about axis parallel to unconnected leg $SR_x = \sqrt{[(L_{cx} / r_x)^2 + (a / r_i)^2]} =$ **82.8**

Buckling about axis parallel to connected leg $SR_y = \sqrt{[(L_{cy} / r_y)^2 + (a / r_i)^2]} =$ **63.2**

Check slenderness between connectors (cl. E6.2)

Effective length factor between connectors $K =$ **1.0**

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) =$ **0.48** in

Slenderness between connectors $SR_{con} = K \times a / r_i =$ **49.8**

Maximum allowable slenderness $SR_{allow} = 0.75 \times \max(SR_x, SR_y) =$ **62.1**

PASS - The slenderness between connectors is less than 3/4 of governing slenderness of built-up member

Compressive strength

Flexural buckling about x axis (cl. E7)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **41.8** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) * F_y =$ **30.3** ksi

Width of compression element 1 $b_1 =$ **2.5** in

Imperfection Adjustment Factors $c_{1,b1} =$ **0.22**

$c_{2,b1} =$ **1.49**

Elastic local buckling stress $F_{el,b1} = (c_{2,b1} * \lambda_{r_c} / (L_1 / t))^2 * F_y =$ **73.7** ksi

Reduced effective width $b_{1e} = b_1 =$ **2.5** in

Width of compression element 2 $b_2 =$ **2.5** in

Imperfection Adjustment Factors $c_{1,b2} =$ **0.22**

$c_{2,b2} =$ **1.49**

Elastic local buckling stress $F_{el,b2} = (c_{2,b2} * \lambda_{r_c} / (L_2 / t))^2 * F_y =$ **73.7** ksi

Reduced effective width $b_{2e} = b_2 =$ **2.5** in

Effective area of cross-section $A_e = A - 2 * (b_1 - b_{1e}) * t - 2 * (b_2 - b_{2e}) * t =$ **1.8** in^2

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A_e =$ **54.5** kips

Flexural buckling about y axis (cl. E7)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **71.6** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) * F_y =$ **37.3** ksi

Width of compression element 1 $b_1 =$ **2.5** in

Imperfection Adjustment Factors $c_{1,b1} =$ **0.22**

	$c_{2,b1}$ = **1.49**
Elastic local buckling stress	$F_{el,b1} = (c_{2,b1} * \lambda_{r_c} / (L_1 / t))^2 * F_y$ = **73.7** ksi
Reduced effective width	$b_{1e} = b_1 * (1 - c_{1,b1} * \sqrt{(F_{el,b1} / F_{cry})}) * \sqrt{(F_{el,b1} / F_{cry})}$ = **2.4** in
Width of compression element 2	b_2 = **2.5** in
Imperfection Adjustment Factors	$c_{1,b2}$ = **0.22**
	$c_{2,b2}$ = **1.49**
Elastic local buckling stress	$F_{el,b2} = (c_{2,b2} * \lambda_{r_c} / (L_2 / t))^2 * F_y$ = **73.7** ksi
Reduced effective width	$b_{2e} = b_2 * (1 - c_{1,b2} * \sqrt{(F_{el,b2} / F_{cry})}) * \sqrt{(F_{el,b2} / F_{cry})}$ = **2.4** in
Effective area of cross-section	$A_e = A - 2 * (b_1 - b_{1e}) * t - 2 * (b_2 - b_{2e}) * t$ = **1.745** in^2
Nominal compressive strength for flexural buckling	$P_{ny} = F_{cry} * A_e$ = **65.2** kips

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

Elastic torsional buckling stress	$F_{ez} = (\pi^2 * E * C_w / (L_{cz})^2 + G * J) / (A * r_o^2)$ = **46.8** ksi
Flexural-torsional elastic buckling stress	$F_{et} = (F_{ey} + F_{ez}) / (2 * H) * (1 - \sqrt{(1 - 4 * F_{ey} * F_{ez} * H / (F_{ey} + F_{ez})^2)})$ = **40.1** ksi
Flexural-torsional buckling stress	$F_{crt} = (0.658^{F_y / F_{et}}) * F_y$ = **29.7** ksi
Width of compression element 1	b_1 = **2.5** in
Imperfection Adjustment Factors	$c_{1,b1}$ = **0.22**
	$c_{2,b1}$ = **1.49**
Elastic local buckling stress	$F_{el,b1} = (c_{2,b1} * \lambda_{r_c} / (L_1 / t))^2 * F_y$ = **73.7** ksi
Reduced effective width	$b_{1e} = b_1$ = **2.5** in
Width of compression element 2	b_2 = **2.5** in
Imperfection Adjustment Factors	$c_{1,b2}$ = **0.22**
	$c_{2,b2}$ = **1.49**
Elastic local buckling stress	$F_{el,b2} = (c_{2,b2} * \lambda_{r_c} / (L_2 / t))^2 * F_y$ = **73.7** ksi
Reduced effective width	$b_{2e} = b_2$ = **2.5** in
Effective area of cross-section	$A_e = A - 2 * (b_1 - b_{1e}) * t - 2 * (b_2 - b_{2e}) * t$ = **1.8** in^2
Nominal compressive strength for torsional and flexural-torsional buckling	
	$P_{nt} = F_{crt} * A_e$ = **53.4** kips

Allowable compressive strength (cl. E1)

Safety factor for compression	Ω_c = **1.67**
Allowable compressive strength	$P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c$ = **32.0** kips

FAIL - The required compressive strength exceeds the allowable compressive strength

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	Section Web 7 Tension				Sheet no./rev. 1	
	Calc. by RA	Date 12/17/2020	Chk'd by	Date	App'd by	Date

STEEL MEMBER DESIGN (AISC 360)

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

Tedds calculation version 4.4.05

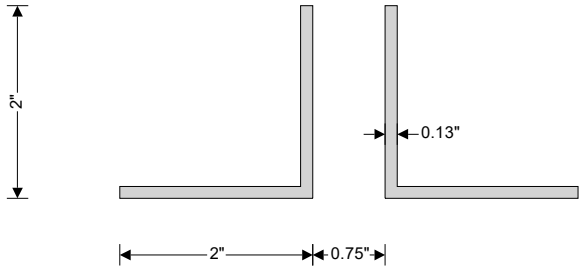
Safety factors

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 2x2x1/8x3/4 (AISC 15th Edn (v15.0))
ASTM steel designation	A572 Gr.50
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



2L 2x2x1/8x3/4 (AISC 15th Edn (v15.0))
Section depth, L_1, 2 in
Section breadth, L_2, 2 in
Weight of section, Weight, 3.3 lbf/ft
Section thickness, t, 0.125 in
Area of section, A, 1 in^2
Radius of gyration about x-axis, r_x, 0.62 in
Radius of gyration about y-axis, r_y, 1.1 in
Elastic section modulus about x-axis, S_x, 0.258 in^3
Second moment of area about x-axis, I_x, 0.378 in^4
Second moment of area about y-axis, I_y, 1.19 in^4

Analysis results

Required tensile strength P_r = **21.8** kips

Restraint spacing

Major axis lateral restraint	L_x = **5** ft
Minor axis lateral restraint	L_y = **5** ft
Torsional restraint	L_z = **5** ft

Design of members for tension - Chapter D

Required tensile strength P_r = **21.8** kips

Slenderness limitations - Section D1

Slenderness ratio $\lambda = \max(L_x / r_x, L_y / r_y)$ = **96.774**

Tension member slenderness ratio does not exceed recommended limit of 300

Allowable tensile strength - D2

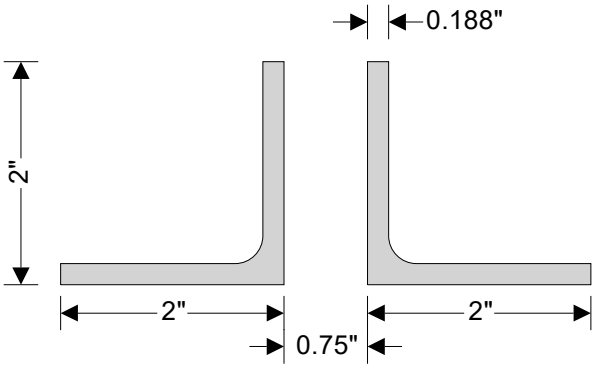
Nominal tensile yielding strength	$P_{n,yld} = F_y * A$ = **49.1** kips
Nominal tensile rupture strength	$P_{n,r} = F_u * A_e$ = **63.83** kips
Allowable tensile strength	$P_c = \min(P_{n,yld} / \Omega_{t,y}, P_{n,r} / \Omega_{t,r})$ = **29.401** kips
	P_r / P_c = **0.741**

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	**2L 2x2x3/16x3/4**

Design loading

Required axial strength	P_r = **23** kips (Compression)

Material details

Steel grade	**A572 Gr. 50**
Yield strength	F_y = **50** ksi
Ultimate strength	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **51** in
For buckling about y axis	L_y = **51** in
For torsional buckling	L_z = **51** 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	$L_{cx} = L_x \times K_x$ = **51** in
For buckling about y axis	$L_{cy} = L_y \times K_y$ = **51** in
For torsional buckling	$L_{cz} = L_z \times K_z$ = **51** in
Type of connector fixing angles together	**Snug-tight bolts**
Distance between connectors	a = **24.0** in

Section classification

Section classification for local buckling (cl. B4)

Critical width $b = \max(L_1, L_2) =$ **2.000** in

Maximum width to thickness ratio $\lambda = b / t =$ **10.638**

Compression

Limit for nonslender section $\lambda_{r_c} = 0.45 \times \sqrt{(E / F_y)} =$ **10.837**

The section is nonslender in compression

Slenderness

Effective slenderness (cl. E6)

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) =$ **0.39** in

Buckling about axis parallel to unconnected leg $SR_x = \sqrt{[(L_{cx} / r_x)^2 + (a / r_i)^2]} =$ **103.7**

Buckling about axis parallel to connected leg $SR_y = \sqrt{[(L_{cy} / r_y)^2 + (a / r_i)^2]} =$ **76.7**

Check slenderness between connectors (cl. E6.2)

Effective length factor between connectors $K =$ **1.0**

Minimum radius of gyration of indiv. angle $r_i = \min(r_{z_single}, r_{y_single}, r_{x_single}) =$ **0.39** in

Slenderness between connectors $SR_{con} = K \times a / r_i =$ **61.7**

Maximum allowable slenderness $SR_{allow} = 0.75 \times \max(SR_x, SR_y) =$ **77.8**

PASS - The slenderness between connectors is less than 3/4 of governing slenderness of built-up member

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **26.6** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) * F_y =$ **22.8** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A =$ **32.8** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **48.7** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) * F_y =$ **32.5** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} * A =$ **46.8** kips

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

Elastic torsional buckling stress $F_{ez} = (\pi^2 * E * C_w / (L_{cz})^2 + G * J) / (A * r_o^2) =$ **77.4** ksi

Flexural-torsional elastic buckling stress $F_{et} = (F_{ey} + F_{ez}) / (2 * H) * (1 - \sqrt{(1 - 4 * F_{ey} * F_{ez} * H / (F_{ey} + F_{ez})^2)}) =$ **42.5** ksi

Flexural-torsional buckling stress $F_{crt} = (0.658^{F_y / F_{et}}) * F_y =$ **30.6** ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} * A =$ **44** kips

Allowable compressive strength (cl. E1)

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

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c =$ **19.6** kips

FAIL - The required compressive strength exceeds the allowable compressive strength

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	Section Web 10 Tension				Sheet no./rev. 1	
	Calc. by RA	Date 12/24/2020	Chk'd by	Date	App'd by	Date

STEEL MEMBER DESIGN (AISC 360)

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

Tedds calculation version 4.4.05

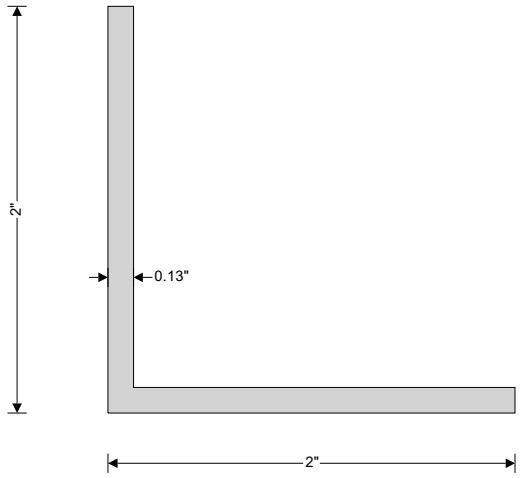
Safety factors

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	L 2x2x1/8 (AISC 15th Edn (v15.0))
ASTM steel designation	A572 Gr.50
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



L 2x2x1/8 (AISC 15th Edn (v15.0))
Section depth, L_1, 2 in
Section breadth, L_2, 2 in
Weight of section, Weight, 1.7 lbf/ft
Section thickness, t, 0.125 in
Area of section, A, 0.5 in^2
Radius of gyration about x-axis, r_x, 0.62 in
Radius of gyration about y-axis, r_y, 0.62 in
Elastic section modulus about x-axis, S_x, 0.129 in^3
Second moment of area about x-axis, I_x, 0.189 in^4
Second moment of area about y-axis, I_y, 0.189 in^4

Analysis results

Required tensile strength P_r = **7.02** kips

Restraint spacing

Major axis lateral restraint	L_x = **5** ft
Minor axis lateral restraint	L_y = **5** ft
Torsional restraint	L_z = **5** ft

Design of members for tension - Chapter D

Required tensile strength P_r = **7.02** kips

Slenderness limitations - Section D1

Slenderness ratio $\lambda = \max(L_x / r_x, L_y / r_y)$ = **96.774**

Tension member slenderness ratio does not exceed recommended limit of 300

Allowable tensile strength - D2

Nominal tensile yielding strength	$P_{n,yld} = F_y * A =$ **24.55** kips
Nominal tensile rupture strength	$P_{n,r} = F_u * A_e =$ **31.915** kips
Allowable tensile strength	$P_c = \min(P_{n,yld} / \Omega_{t,y}, P_{n,r} / \Omega_{t,r}) =$ **14.701** kips
	$P_r / P_c =$ **0.478**

PASS - Nominal tensile strength exceeds required tensile strength

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JOB 67 RT. 46
SHEET NO. WEB 12/A OF
CALCULATED BY KA DATE 12/24/20
CHECKED BY DATE
SCALE

WEB 12 (2) ∠ 1 3/4 x 1 3/4 x 1/8 $h = 24''$

$F_y = 50$ KSi

$L_x = L_y = L_z = 51$ in $K_x = K_y = K_z = 1$

$b = 1.75$ $\lambda = \frac{1.75}{1/8} = 14$ $0.45\sqrt{\frac{29000}{50}} = 10.84$

MEMBER IS SLENDER

$r_{min} = 0.347$ in

$$S_{rx} = \sqrt{\left(\frac{51}{0.546}\right)^2 + \left(\frac{24}{0.347}\right)^2} = 116.23$$

$$S_{ry} = \sqrt{\left(\frac{51}{1.02}\right)^2 + \left(\frac{24}{0.347}\right)^2} = 85.34$$

$$SR_{CONN} = K \times \frac{24}{0.347} = 69.16$$

$SR_{ALLOW} = 0.75(116.23) = 87.17 > 69.16$ O.K

$$F_{ex} = \frac{\pi^2(29000)}{(116.23)^2} = 21.19 \text{ KSi}$$

$$F_{crx} = \left((0.658)^{\frac{50}{21.19}}\right) 50 = 18.62 \text{ KSi}$$

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JOB 67 RT. 46
SHEET NO. WEB 12/B OF
CALCULATED BY RA DATE 12/24/20
CHECKED BY DATE
SCALE

$b = 1.75''$

$c_1 b_1 = 0.22$

$c_1 b_2 = 1.49$

$$F_e b_1 = \left(1.49 \times \frac{10.837}{\left(\frac{1.75}{1/8}\right)^2}\right)^2 \times 50 = 66.51$$

$$b_{1e} = 1.75\left(1 - 0.22 \times \sqrt{\frac{66.51}{18.62}}\right) * \sqrt{\frac{66.51}{18.62}} = 1.93$$

$b_{1e} = 1.75$

$b_2 = 1.75$

$c_1 b_1 = 0.22$

$c_1 b_2 = 1.49$

$$F_e b_2 = \left(1.49 \times \frac{10.837}{\frac{1.75}{1/8}}\right)^2 \times 50 = 66.51$$

$$b_{2e} = 1.75\left(1 - 0.22 \times \sqrt{\frac{66.51}{18.62}}\right) * \sqrt{\frac{66.51}{18.62}} = 1.93$$

$b_{2e} = 1.75$

$$A_e = 0.844 - 2\,(\cancel{1.75 - 1.75}^{\,\emptyset}) \times \tfrac{1}{8} - 2 \times (\cancel{1.75 - 1.75}^{\,\emptyset}) \times \tfrac{1}{8}$$

$A = 0.844$

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SCALE

$P_n = 18.62(0.844) = 15.72$ Kips.

$P_u = \frac{15.72}{1.67} = 9.41 > 5.43$ Kips & 7.28 Kips

O.K.

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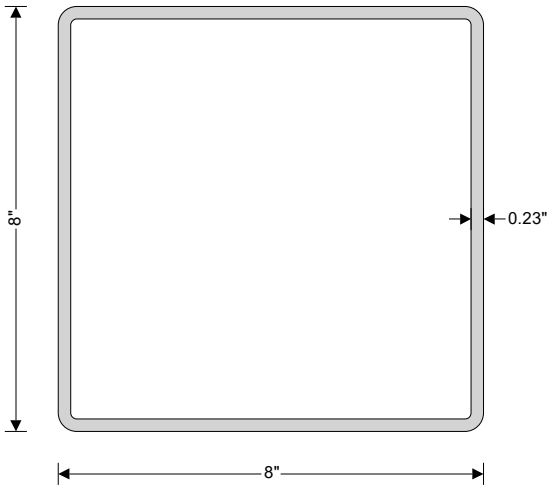
	Section Perimeter Column HSS 8x8x 0.25				Sheet no./rev. 1	
	Calc. by RA	Date 12/17/2020	Chk'd by	Date	App'd by	Date

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	**HSS 8x8x1/4**

Design loading

Required axial strength	P_r = **66** 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	**A500 Gr. B**
Yield strength	F_y = **46** 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 = **384** in
For buckling about y axis	L_y = **384** in
For torsional buckling	L_z = **384** 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	$L_{cx} = L_x \times K_x$ = **384** in

For buckling about y axis $L_{cy} = L_y \times K_y =$ **384** in

For torsional buckling $L_{cz} = L_z \times K_z =$ **384** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f - 3 \times t =$ **7.301** in

Critical web width $h = d - 3 \times t =$ **7.301** in

Width to thickness ratio of flange (compression) $\lambda_{f_c} = b / t =$ **31.335**

Width to thickness ratio of web (compression) $\lambda_{w_c} = h / t =$ **31.335**

Width to thickness ratio of flange (major flexure) $\lambda_{f_fx} = b / t =$ **31.335**

Width to thickness ratio of web (major flexure) $\lambda_{w_fx} = h / t =$ **31.335**

Width to thickness ratio of flange (minor flexure) $\lambda_{f_fy} = h / t =$ **31.335**

Width to thickness ratio of web (minor flexure) $\lambda_{w_fy} = b / t =$ **31.335**

Compression

Limit for nonslender section $\lambda_{r_c} = 1.40 \times \sqrt{(E / F_y)} =$ **35.152**

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x =$ **121.9**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **121.9**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **19.3** ksi

Flexural buckling stress $F_{crx} = 0.877 * F_{ex} =$ **16.9** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A =$ **119.9** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **19.3** ksi

Flexural buckling stress $F_{cry} = 0.877 * F_{ey} =$ **16.9** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} * A =$ **119.9** kips

Allowable compressive strength (cl. E1)

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

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}) / \Omega_c =$ **71.8** kips

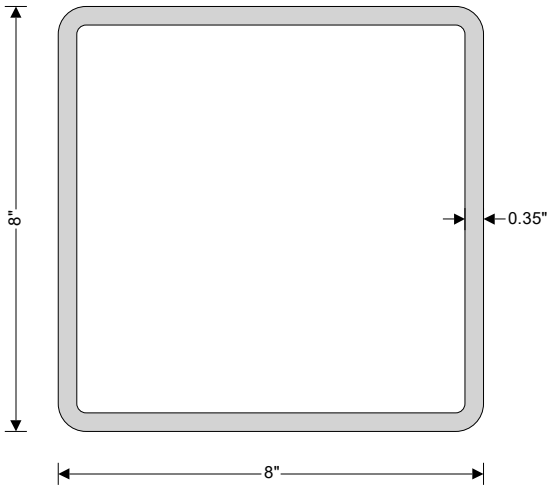
PASS - The allowable compressive strength exceeds the required compressive strength

Grey Fox Consulting, Inc. 57 Cooper Avenue Cherry Hill, NJ 08002	Project 67 Route 46, Fairfield, New Jersey				Job Ref.	
	Section Interior Column 8x8x0.375				Sheet no./rev. 1	
	Calc. by RA	Date 12/17/2020	Chk'd by	Date	App'd by	Date

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	**HSS 8x8x3/8**

Design loading

Required axial strength	P_r = **66** 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	**A500 Gr. B**
Yield strength	F_y = **46** 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 = **384** in
For buckling about y axis	L_y = **384** in
For torsional buckling	L_z = **384** 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	$L_{cx} = L_x \times K_x$ = **384** in

For buckling about y axis $L_{cy} = L_y \times K_y =$ **384** in

For torsional buckling $L_{cz} = L_z \times K_z =$ **384** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f - 3 \times t =$ **6.953** in

Critical web width $h = d - 3 \times t =$ **6.953** in

Width to thickness ratio of flange (compression) $\lambda_{f_c} = b / t =$ **19.923**

Width to thickness ratio of web (compression) $\lambda_{w_c} = h / t =$ **19.923**

Width to thickness ratio of flange (major flexure) $\lambda_{f_fx} = b / t =$ **19.923**

Width to thickness ratio of web (major flexure) $\lambda_{w_fx} = h / t =$ **19.923**

Width to thickness ratio of flange (minor flexure) $\lambda_{f_fy} = h / t =$ **19.923**

Width to thickness ratio of web (minor flexure) $\lambda_{w_fy} = b / t =$ **19.923**

Compression

Limit for nonslender section $\lambda_{r_c} = 1.40 \times \sqrt{(E / F_y)} =$ **35.152**

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x =$ **123.9**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **123.9**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **18.7** ksi

Flexural buckling stress $F_{crx} = 0.877 * F_{ex} =$ **16.4** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A =$ **170.1** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **18.7** ksi

Flexural buckling stress $F_{cry} = 0.877 * F_{ey} =$ **16.4** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} * A =$ **170.1** kips

Allowable compressive strength (cl. E1)

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

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}) / \Omega_c =$ **101.9** kips

PASS - The allowable compressive strength exceeds the required compressive strength