GreyFox CONSULTING, Inc

Structural & Civil
Engineering Solutions

FEASIBILITY REPORT

Date: February 2, 2021

To: Ben Aparo and Nate Greenberg
Summit Ridge Energy

From: Ramin Abbaszadeh, PE

Re: 39 Plymouth Street
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 1980s. 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 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.

This building is divided into two sections. To the east, the building has a 2nd-floor office area. The high atrium entrance in the middle of the building (see pictures3 and 4) separates the two-story from the one-story half. Measurements of the roof framing of this building were difficult. There are dropped ceilings at about 9 feet in the one-story portion to the west that prevented us from reaching the roof structure about 24 feet above the first floor. On the second floor, the finished spaces (see pictures 5 through 9) did not give us a clear sightline to measure the columns' spacing. Pictures 10 and 11 show the

congestion above the dropped ceiling. Our only access to the roof framing was through 2 ft by 4 ft dropped ceiling grid. We did our best to collect enough information to develop a framing plan. The area that we measured is delineated in Picture 1. Other areas framing were extrapolated from our investigation of the measured area.

The building is a rectangular one-story steel structure (see pictures 2 and 3). The building has an approximate length of 340 ft length in the east-west direction and an approximate width of 97 feet in the north-south direction. The roof structure is comprised of a steel deck supported by steel bar joists. In turn, the joists are supported on the cantilever steel beam system. Non-bearing masonry walls surround the perimeter of this building.

The roof structure supports additional loads imposed by the dropped ceilings and additional HVAC ducts because of the office space. This extra load is reflected below under Criteria and Assumptions. This building has a fire suppression system.

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 (ceiling, lights, piping, ductwork, electrical) = 6 psf
  - Dead load: Structural joists = 2 psf from historical data
  - Dead load: Structural 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 = 37 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 in the north-south direction with an approximate span of 32 ft- 3 in. The bar joists are supported on the cantilevered steel beam system.

The approximate spacing between bar joists is 5'-10". Based on our field measurement, the joists are 24 inch-deep with 1.75 inches by 1.75 inches double angles top and bottom chords. The thickness of both chords is 0.19 inches. The web members are comprised of 0.8 inches diameter round bars.

The historical data from SJI designates bar joists with similar cord and web elements as 24H7 joists. This joist has a capacity of 360 plf. The existing dead and snow loads on the joist is 216 plf. As a result, the existing joists have 144 plf or 25 psf reserved capacity.

If this building is selected for the next phase, additional investigation is required with free access to the roof framing to ensure that all joists are of similar size and strength.

ROOF DECK

The roof deck is a 1.5-inch type B wide rib (see picture 16). The roof deck's thickness appears to be approximately 0.04 in or 20 gage. Based on the joist spacing, this deck has a capacity of about 118 psf. 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 BEAMS

Based on our field measurements, we extrapolated that the cantilever steel beam systems comprised of four spans about 40 feet each for the portion to the east of the atrium that we investigated. The steel beams supporting the bar joists are spaced about 32 ft - 3 in apart. The Cantilevered Beams system has two beams that cantilever and two beams simply supported off of the cantilevered beams.

The cantilevered and simply supported beams on the building's exterior are W18x35. The W18x35 beams have no more than 7 psf reserved capacity for the solar panels.

The cantilevered beams on the building's interior frames are W24x68 and W24x55. The simply supported beams are W24x55.

The governing member for the interior frames is W24x68. The interior framing has reserved capacity to support solar panels of 12 psf.

Based on the above paragraphs, this roof could only support solar panels up to 7 psf.

STEEL COLUMNS

The perimeter columns are W10x39, and the interior columns are W10x49. Both of these columns have sufficient capacity to support additional loads of up to 12 psf from solar panels.

Conclusions

Our professional opinion is that this building's roof structure can support solar panels up to 7 psf.

If this building is selected for solar panel installation, the building's entire roof frame must be investigated more accurately by removing the ceiling and giving free access to the roof framing.

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

ONE-STORY OFFICE WITH DROPPED CEILING
HIGH CEILING ATRIUM
INVESTIGATED AREA
NOT INVESTIGATED
ONE-STORY WAREHOUSE WITH NO DROPPED CEILING
NOT INVESTIGATED
EXTRAPOLATED AREA
NOT INVESTIGATED

N

Picture 1

Allied Office Supplies
Office supply store
PSI Pump and
Process, Inc

Picture 2

Picture 3

Picture 4

Picture 5

Picture 6

Picture 7

Picture 8

Picture 9

Picture 10

Picture 11

Picture 12

Picture 13

Picture 14

Picture 15

Picture 16

SUMMIT RIDGE ENERGY

39 Plymouth Street
Fairfield, New Jersey

Roof Evaluation
Structural Calculations

GreyFox CONSULTING, Inc.

Structural & Civil
Engineering Solutions

DATE: 2/2/2021
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 39 Plymouth Street, Fairfield New Jersey				Job Ref.	
	Section Snow Load				Sheet no./rev. 1	
	Calc. by RA	Date 2/2/2021	Chk'd by	Date	App'd by	Date

SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.09

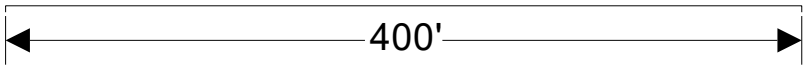
Building details

Roof type	Flat
Width of roof	b = **400.00** ft

Ground snow load

Ground snow load (Figure 7-1)	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 7-2)	Fully exposed
Exposure factor (Table 7-2)	C_e = **0.90**
Thermal condition (Table 7-3)	All
Thermal factor (Table 7-3)	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
(856) 216-9373
Fax (856) 216-9376
Structural & Civil Engineering Solutions

JOB 39 PLYMOUTH ST.
SHEET NO. 1 OF
CALCULATED BY RH DATE 2/2/21
CHECKED BY DATE
SCALE

ROOF DL.

ROOFING	6 PSF
SPRINKLER	3 PSF
MECH./CEILING	6 PSF
ELEC., MISC.	
JOIST	2 PSF
	17 PSF

SNOW = 20 PSF

TOTAL = 17 + 20 = 37

2 H 7 32' 363
33' 352
11

11/12 x 3 = 2.75

363 - 2.75 = 360

37(5.83) = 216

360 - 216 = 144 RESERVED OK
25 PSF

OFFICE ASSUMED LOAD

DL = 70 PSF
LL = 70 PSF
TOTAL 140 PSF

ROOF 37 + 12 = 49 PSF

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JOB 39 PLYMOUTH ST.
SHEET NO. 2 OF
CALCULATED BY KA DATE 2/2/21
CHECKED BY DATE
SCALE

EXT COL.

$49\left(40 \times \frac{32.25}{2}\right) = 31.61\ K$

$140\left(40 \times \frac{32.25}{2}\right) = 90.3\ K$

$121.91\ K$

INT. COL.

$49(40 \times 32.25) = 63.21\ K$

$140(40 \times 32.25) = 180.6\ K$

$243.81\ K$

Ramin Abbaszadeh

From: Ken Charles <kcharles@steeljoist.org>
Sent: Friday, January 29, 2021 12:09 PM
To: Ramin Abbaszadeh
Subject: RE: Joist Investigation Form submitted at Steel Joist Institute

The joist you have presented resembles a 24H7 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: Friday, January 29, 2021 12:06 PM
To: kcharles@steeljoist.org
Subject: Joist Investigation Form submitted at Steel Joist Institute

Ramin Abbaszadeh submitted a Joist Investigation Form.

Case Number	26659
First Name	Ramin
Last Name	Abbaszadeh
Company	Grey Fox Consulting, Inc.
Email Address	ramin@greyfox.biz
Preferred phone	8567451668

Jobsite Location City/State	Fairfield, New Jersey
Project Name	39 Plymouth
Why are you requesting this information?	Evaluation
Other	
What year was the building constructed (or approximate age of the structure)?	1980s
Who was the joist manufacturer?	The Ceco Corp.
Is there a tag on the joist?	Yes
Yes? Provide tag information	15 226A J1 J1
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?	32'-4"
What is the joist spacing?	5'-10"
What is the interior panel point spacing?	24"
What is the joist depth?	24"
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	1.75x1.75
Top chord thickness	0.19
Top chord diameter	

Bottom chord shape	2 Angles
Bottom chord leg size	1.75x1.75
Bottom chord thickness	0.19
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	0.8"
Measurement device	Micrometer
Other device	

For Joist Depths 24" to 30" inclusive

Joist Designation	24H6	24H7	24H8	24H9	24H10	24H11	26H8	26H9	26H10	26H11	28H8	28H9	28H10	28H11	30H8	30H9	30H10	30H11
Nominal *Depth (in.)	24	24	24	24	24	24	26	26	26	26	28	28	28	28	30	30	30	30
Resist. Moment (in.-lbs.)	462,000	576,000	716,000	851,000	957,000	1,106,000	784,000	925,000	1,040,000	1,203,000	846,000	1,000,000	1,124,000	1,300,000	909,000	1,075,000	1,207,000	1,397,000
Max. End React. (lbs.)	5600	5800	6000	7000	7500	8200	6700	7200	7600	8300	6700	7200	7700	8400	6800	7500	8100	8700
†Approx. Wt. (lbs./ft.)	10.3	11.5	12.7	14.0	15.5	17.5	12.8	14.8	16.2	17.9	13.5	15.2	16.8	18.3	14.2	15.4	17.3	18.8
Span in Feet 24	467	483	500	583	625													
25	448	464	480	560	600													
26	431	446	462	538	577	631	515	554	585	638								
27	415 375	430	444	519	556	607	496	533	563	615								
28	393 336	414 406	429	500	536	586	479	514	543	593	479	514	550	600				
29	366 303	400 365	414	483	517	566	462	497	524	572	462	497	531	579				
30	342 273	387 330	400	467 457	500	547	447	480	507	553	447	480	513	560	453	500	540	580
31	320 248	374 299	387 373	452 414	484 465	529	432 418	465	490	535	432	465	497	542	439	484	523	561
32	301 225	363 272	375 339	438 376	469 423	513 482	419 380	450 445	475	519	419	450	481	525	425	469	506	544
33	283 205	352 248	364 309	424 343	455 386	497 440	406 346	436 405	461 456	503	406 404	436	467	509	412	455	491	527
34	266 188	332 227	353 283	412 314	441 353	482 402	394 317	424 371	447 417	488 476	394 370	424	453	494	400	441	476	512
35	251 172	313 208	343 259	400 288	429 323	469 369	383 290	411 340	434 383	474 437	383 339	411 396	440	480	389	429	463	497
36	238 158	296 191	333 238	389 264	417 297	456 339	372 267	400 312	422 352	461 401	372 311	400 364	428 410	467	378 359	417	450	483
37	225 146	280 176	324 219	378 243	405 274	443 312	362 246	389 288	411 324	449 370	362 287	389 336	416 378	454 432	368 330	405 387	438 436	470
38	213 135	266 162	316 202	368 225	395 253	432 288	353 227	379 266	400 299	437 341	353 265	379 310	405 349	442 399	358 305	395 357	426 402	458
39	202 124	252 150	308 187	359 208	385 234	421 266	344 210	369 246	390 276	426 316	344 245	369 287	395 322	431 369	349 282	385 331	415 372	446 426
40	193 115	240 139	298 174	350 193	375 217	410 247	327 194	360 228	380 256	415 292	335 227	360 266	385 299	420 342	340 262	375 306	405 345	435 395
41	183 107	228 129	284 161	337 179	366 201	400 229	311 181	351 211	371 238	405 272	327 211	351 247	376 278	410 318	332 243	366 285	395 320	424 367
42	175 100	218 120	271 150	322 166	357 187	390 213	296 168	343 197	362 221	395 253	319 196	343 229	367 258	400 295	324 226	357 265	386 298	414 341
43	167 93	208 112	258 140	307 155	345 174	381 199	283 156	334 183	353 206	386 235	305 183	335 214	358 241	391 275	316 211	349 247	377 278	405 318
44	159 87	198 105	247 130	293 145	330 163	373 186	270 146	319 171	345 193	377 220	291 171	327 200	350 225	382 257	309 196	341 230	368 259	395 297
45	152 81	190 98	236 122	280 135	315 152	364 173	258 137	305 160	338 180	369 205	279 159	320 187	342 210	373 240	299 184	333 215	360 242	387 278
46	146 76	181 92	226 114	268 127	302 142	348 162	247 128	291 150	328 168	361 192	267 149	313 175	335 197	365 225	286 172	326 202	352 227	378 260
47	139 71	174 86	216 107	257 119	289 133	334 152	237 120	279 140	314 158	353 180	255 140	302 164	328 184	357 211	274 161	319 189	345 213	370 244
48	134 67	167 81	207 100	246 111	277 125	320 143	227 112	268 132	301 148	346 169	245 131	289 154	321 173	350 198	263 151	311 177	338 200	363 229
49							218 106	257 124	289 139	334 159	235 124	278 144	312 163	343 186	252 142	298 167	331 188	355 215
50							209 100	247 117	277 131	321 150	226 116	267 136	300 153	336 175	242 134	287 157	322 177	348 202
51							201 94	237 110	267 124	308 141	217 110	256 128	288 144	329 165	233 126	276 148	309 166	341 191
52							193 88	228 104	256 117	297 133	209 103	247 121	277 136	321 156	224 119	265 139	298 157	335 180
53											201 98	237 114	267 128	309 147	216 112	255 132	286 148	328 170
54											193 92	229 108	257 121	297 139	208 106	246 125	276 140	319 161
55											186 87	220 102	248 115	287 132	200 101	237 118	266 133	308 152
56											180 83	213 97	239 109	276 125	193 95	229 112	257 126	297 144
57															187 90	221 106	248 119	287 137
58															180 86	213 101	239 113	277 130
59															174 81	206 95	231 108	268 123
60															168 77	199 91	224 102	259 117

*Indicates Nominal Depth of steel joists only.

†Approximate Weights per Linear Foot of steel joists only. Accessories and nailer strip not included.

*†See manufacturers' catalog for detailed information on specific joist types.

**Section 5.9 of the "Standard Specifications for Open Web Steel Joists, J- and H-Series" limits the design LIVE load deflection as follows:

FLOORS, 1/360 span. ROOFS, 1/360 of span where a plaster ceiling is attached or suspended; 1/240 of span for all other cases.

Current Date: 1/31/2021 8:30 AM
Units system: English
File name: S:\Projects\20-039 Summit Ridge NJ Roof Evaluation\39 Plymouth Street\Calc\Int Cant Bm.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	40.833	0.00	0.00	0
3	46.833	0.00	0.00	0
4	75.667	0.00	0.00	0
5	81.667	0.00	0.00	0
6	122.50	0.00	0.00	0
7	127.50	0.00	0.00	0
8	163.333	0.00	0.00	0

Restraints

Node	TX	TY	TZ	RX	RY	RZ
1	1	1	1	0	0	0
2	0	1	1	0	0	0
5	0	1	1	0	0	0
6	0	1	1	0	0	0
8	0	1	1	0	0	0

Members

Member	NJ	NK	Description	Section	Material	d0 [in]	dL [in]	lg factor
1	1	2		W 24X68	A36	0.00	0.00	0.00
2	2	3		W 24X68	A36	0.00	0.00	0.00
3	3	4		W 24X55	A36	0.00	0.00	0.00
4	4	5		W 24X55	A36	0.00	0.00	0.00
5	5	6		W 24X55	A36	0.00	0.00	0.00
6	6	7		W 24X55	A36	0.00	0.00	0.00
7	7	8		W 24X68	A36	0.00	0.00	0.00

Hinges

	Node-J				Node-K						
Member	M33	M22	V3	V2	M33	M22	V3	V2	TOR	AXL	Axial rigidity
3	1	0	0	0	1	0	0	0	0	0	Full
7	1	0	0	0	0	0	0	0	0	0	Full

Current Date: 1/31/2021 8:30 AM
Units system: English
File name: S:\Projects\20-039 Summit Ridge NJ Roof Evaluation\39 Plymouth Street\Calc\Int Cant Bm.retx

Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

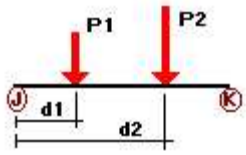
Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SL	Solar Load	No	DL
D2	DL+0.32SL	Yes	

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	2	0.00	-7.00	0.00	0.00	0.00	0.00
	5	0.00	-7.00	0.00	0.00	0.00	0.00
	6	0.00	-7.00	0.00	0.00	0.00	0.00
SL	2	0.00	-7.00	0.00	0.00	0.00	0.00
	5	0.00	-7.00	0.00	0.00	0.00	0.00
	6	0.00	-7.00	0.00	0.00	0.00	0.00

Concentrated forces on members



Condition	Member	Dir1	Value1 [Kip]	Dist1 [ft]	%
DL	1	Y	-7.00	5.8333	No
		Y	-7.00	11.667	No
		Y	-7.00	17.50	No
		Y	-7.00	23.333	No
		Y	-7.00	29.167	No
		Y	-7.00	35.00	No
	2	Y	-7.00	5.833	No
	3	Y	-7.00	5.667	No
		Y	-7.00	11.50	No
		Y	-7.00	17.333	No
		Y	-7.00	23.167	No

	4	Y	-7.00	0.1667	No
	5	Y	-7.00	5.8333	No
		Y	-7.00	11.667	No
		Y	-7.00	17.50	No
		Y	-7.00	23.333	No
		Y	-7.00	29.167	No
		Y	-7.00	35.00	No
	7	Y	-7.00	0.833	No
		Y	-7.00	6.667	No
		Y	-7.00	12.50	No
		Y	-7.00	18.333	No
		Y	-7.00	24.167	No
		Y	-7.00	30.00	No
SL	1	Y	-7.00	5.8333	No
		Y	-7.00	11.667	No
		Y	-7.00	17.50	No
		Y	-7.00	23.333	No
		Y	-7.00	29.167	No
		Y	-7.00	35.00	No
	2	Y	-7.00	5.833	No
	3	Y	-7.00	5.667	No
		Y	-7.00	11.50	No
		Y	-7.00	17.333	No
		Y	-7.00	23.167	No
	4	Y	-7.00	0.1667	No
	5	Y	-7.00	5.8333	No
		Y	-7.00	11.667	No
		Y	-7.00	17.50	No
		Y	-7.00	23.333	No
		Y	-7.00	29.167	No
		Y	-7.00	35.00	No
	7	Y	-7.00	0.833	No
		Y	-7.00	6.667	No
		Y	-7.00	12.50	No
		Y	-7.00	18.333	No
		Y	-7.00	24.167	No
		Y	-7.00	30.00	No

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier			
		Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	0.00	0.00
SL	Solar Load	No	0.00	0.00	0.00
D2	DL+0.32SL	Yes	0.00	0.00	0.00

Earthquake (Dynamic analysis only)

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

Current Date: 1/31/2021 8:31 AM
Units system: English
File name: S:\Projects\20-039 Summit Ridge NJ Roof Evaluation\39 Plymouth Street\Calc\Int Cant Bm.retx

List of materials

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

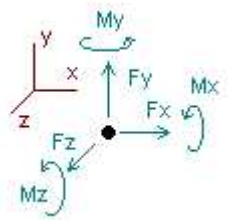
Members:

Profile	Material	Uweight [Kip/ft]	Length [ft]	Weight [Kip]
W 24X55	A36	5.52E-02	80.667	4.454
W 24X68	A36	6.85E-02	82.666	5.663
Total weight [Kip]				10.116

Current Date: 1/31/2021 8:32 AM
Units system: English
File name: S:\Projects\20-039 Summit Ridge NJ Roof Evaluation\39 Plymouth Street\Calc\Int Cant Bm.retx

Analysis result

Reactions



Direction of positive forces and moments

Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **D2=DL+0.32SL**						
1	0.00000	23.68427	0.00000	0.00000	0.00000	0.00000
2	0.00000	68.71605	0.00000	0.00000	0.00000	0.00000
5	0.00000	64.84688	0.00000	0.00000	0.00000	0.00000
6	0.00000	68.38048	0.00000	0.00000	0.00000	0.00000
8	0.00000	23.85231	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	249.48000	0.00000	0.00000	0.00000	0.00000

Maximum forces at members

Condition : **D2=DL+0.32SL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	0.00	23.68	0.00	0.00	0.00	252.02
Min	0.00	-31.76	0.00	0.00	0.00	-164.78
MEMBER **2**						
Max	0.00	27.72	0.00	0.00	0.00	0.00
Min	0.00	18.48	0.00	0.00	0.00	-164.78
MEMBER **3**						
Max	0.00	18.48	0.00	0.00	0.00	158.63
Min	0.00	-18.48	0.00	0.00	0.00	0.00
MEMBER **4**						
Max	0.00	-18.48	0.00	0.00	0.00	0.00
Min	0.00	-27.72	0.00	0.00	0.00	-164.78
MEMBER **5**						
Max	0.00	27.89	0.00	0.00	0.00	162.38
Min	0.00	-27.55	0.00	0.00	0.00	-164.78
MEMBER **6**						
Max	0.00	31.59	0.00	0.00	0.00	0.00
Min	0.00	31.59	0.00	0.00	0.00	-157.94
MEMBER **7**						
Max	0.00	31.59	0.00	0.00	0.00	254.10
Min	0.00	-23.85	0.00	0.00	0.00	0.00

Grey Fox Consulting, Inc. 57 Cooper Avenue Cherry Hill, NJ 08002	Project 39 Plymouth Street, Fairfield New Jersey				Job Ref.	
	Section Int Cant Beam W24x68				Sheet no./rev. 1	
	Calc. by RA	Date 1/31/2021	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

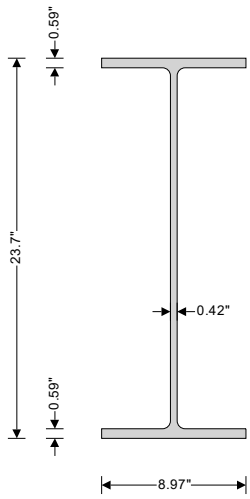
Safety factors

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	W 24x68 (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



W 24x68 (AISC 15th Edn (v15.0))
Section depth, d, 23.7 in
Section breadth, b_f, 8.97 in
Weight of section, Weight, 68 lbf/ft
Flange thickness, t_f, 0.585 in
Web thickness, t_w, 0.415 in
Area of section, A, 20.1 in^2
Radius of gyration about x-axis, r_x, 9.55 in
Radius of gyration about y-axis, r_y, 1.87 in
Elastic section modulus about x-axis, S_x, 154 in^3
Elastic section modulus about y-axis, S_y, 15.7 in^3
Plastic section modulus about x-axis, Z_x, 177 in^3
Plastic section modulus about y-axis, Z_y, 24.5 in^3
Second moment of area about x-axis, I_x, 1830 in^4
Second moment of area about y-axis, I_y, 70.4 in^4

Analysis results

Required flexural strength - Major axis $M_{r,x}$ = **254** kips_ft

Restraint spacing

Major axis lateral restraint	L_x = **40** ft
Minor axis lateral restraint	L_y = **6** ft
Torsional restraint	L_z = **40** ft

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	b_f / (2 * t_f) = **7.67**	
Limiting ratio for compact section	λ_{pff} = 0.38 * √[E / F_y] = **10.79**	
Limiting ratio for non-compact section	λ_{rff} = 1.0 * √[E / F_y] = **28.38**	Compact

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

Width to thickness ratio $(d - 2 * k) / t_w$ = **51.86**

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

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

Section is compact in flexure

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **254** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **531** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **6** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **7.784** ft

Distance between flange centroids h_o = **23.1** in

c = **1**

r_{ts} = **2.3** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **22.943** ft

L_b <= L_p - Lateral-torsional buckling does not apply

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **531** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **318** kips_ft

$M_{r,x} / M_{c,x}$ = **0.799**

PASS - Allowable flexural strength exceeds required flexural strength

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	Section Int Cant Beam W24x55				Sheet no./rev. 1	
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STEEL MEMBER DESIGN (AISC 360)

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

Tedds calculation version 4.4.05

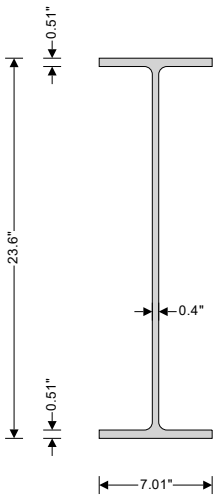
Safety factors

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	W 24x55 (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



W 24x55 (AISC 15th Edn (v15.0))
Section depth, d, 23.6 in
Section breadth, b_f, 7.01 in
Weight of section, Weight, 55 lbf/ft
Flange thickness, t_f, 0.505 in
Web thickness, t_w, 0.395 in
Area of section, A, 16.2 in^2
Radius of gyration about x-axis, r_x, 9.11 in
Radius of gyration about y-axis, r_y, 1.34 in
Elastic section modulus about x-axis, S_x, 114 in^3
Elastic section modulus about y-axis, S_y, 8.3 in^3
Plastic section modulus about x-axis, Z_x, 134 in^3
Plastic section modulus about y-axis, Z_y, 13.3 in^3
Second moment of area about x-axis, I_x, 1350 in^4
Second moment of area about y-axis, I_y, 29.1 in^4

Analysis results

Required flexural strength - Major axis $M_{r,x}$ = **164** kips_ft

Restraint spacing

Major axis lateral restraint	L_x = **40** ft
Minor axis lateral restraint	L_y = **6** ft
Torsional restraint	L_z = **40** ft

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	b_f / (2 * t_f) = **6.94**	
Limiting ratio for compact section	λ_{pff} = 0.38 * √[E / F_y] = **10.79**	
Limiting ratio for non-compact section	λ_{rff} = 1.0 * √[E / F_y] = **28.38**	Compact

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

Width to thickness ratio $(d - 2 * k) / t_w$ = **54.63**

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

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

Section is compact in flexure

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **164** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **402** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **6** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **5.578** ft

Distance between flange centroids h_o = **23.1** in

c = **1**

r_{ts} = **1.72** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **16.872** ft

LTB modification factor C_b = **1.000**

Nominal flexural strength for lateral-torsional buckling - eq F2-2

$M_{n,ltb,x} = \min(C_b * (M_{p,x} - (M_{p,x} - 0.7 * F_y * S_x) * (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **395.9** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **395.9** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **237.1** kips_ft

$M_{r,x} / M_{c,x}$ = **0.692**

PASS - Allowable flexural strength exceeds required flexural strength

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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	40.833	0.00	0.00	0
3	46.833	0.00	0.00	0
4	75.667	0.00	0.00	0
5	81.667	0.00	0.00	0
6	122.50	0.00	0.00	0
7	127.50	0.00	0.00	0
8	163.333	0.00	0.00	0

Restraints

Node	TX	TY	TZ	RX	RY	RZ
1	1	1	1	0	0	0
2	0	1	1	0	0	0
5	0	1	1	0	0	0
6	0	1	1	0	0	0
8	0	1	1	0	0	0

Members

Member	NJ	NK	Description	Section	Material	d0 [in]	dL [in]	lg factor
1	1	2		W 18X35	A36	0.00	0.00	0.00
2	2	3		W 18X35	A36	0.00	0.00	0.00
3	3	4		W 18X35	A36	0.00	0.00	0.00
4	4	5		W 18X35	A36	0.00	0.00	0.00
5	5	6		W 18X35	A36	0.00	0.00	0.00
6	6	7		W 18X35	A36	0.00	0.00	0.00
7	7	8		W 18X35	A36	0.00	0.00	0.00

Hinges

	Node-J				Node-K						
Member	M33	M22	V3	V2	M33	M22	V3	V2	TOR	AXL	Axial rigidity
3	1	0	0	0	1	0	0	0	0	0	Full
7	1	0	0	0	0	0	0	0	0	0	Full

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Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

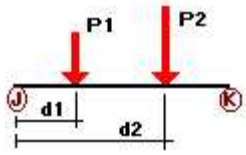
Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SL	Solar Load	No	DL
D2	0.5DL+0.1SL	Yes	

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	2	0.00	-7.00	0.00	0.00	0.00	0.00
	5	0.00	-7.00	0.00	0.00	0.00	0.00
	6	0.00	-7.00	0.00	0.00	0.00	0.00
SL	2	0.00	-7.00	0.00	0.00	0.00	0.00
	5	0.00	-7.00	0.00	0.00	0.00	0.00
	6	0.00	-7.00	0.00	0.00	0.00	0.00

Concentrated forces on members



Condition	Member	Dir1	Value1 [Kip]	Dist1 [ft]	%
DL	1	Y	-7.00	5.8333	No
		Y	-7.00	11.667	No
		Y	-7.00	17.50	No
		Y	-7.00	23.333	No
		Y	-7.00	29.167	No
		Y	-7.00	35.00	No
	2	Y	-7.00	5.833	No
	3	Y	-7.00	5.667	No
		Y	-7.00	11.50	No
		Y	-7.00	17.333	No
		Y	-7.00	23.167	No

	4	Y	-7.00	0.1667	No
	5	Y	-7.00	5.8333	No
		Y	-7.00	11.667	No
		Y	-7.00	17.50	No
		Y	-7.00	23.333	No
		Y	-7.00	29.167	No
		Y	-7.00	35.00	No
	7	Y	-7.00	0.833	No
		Y	-7.00	6.667	No
		Y	-7.00	12.50	No
		Y	-7.00	18.333	No
		Y	-7.00	24.167	No
		Y	-7.00	30.00	No
SL	1	Y	-7.00	5.8333	No
		Y	-7.00	11.667	No
		Y	-7.00	17.50	No
		Y	-7.00	23.333	No
		Y	-7.00	29.167	No
		Y	-7.00	35.00	No
	2	Y	-7.00	5.833	No
	3	Y	-7.00	5.667	No
		Y	-7.00	11.50	No
		Y	-7.00	17.333	No
		Y	-7.00	23.167	No
	4	Y	-7.00	0.1667	No
	5	Y	-7.00	5.8333	No
		Y	-7.00	11.667	No
		Y	-7.00	17.50	No
		Y	-7.00	23.333	No
		Y	-7.00	29.167	No
		Y	-7.00	35.00	No
	7	Y	-7.00	0.833	No
		Y	-7.00	6.667	No
		Y	-7.00	12.50	No
		Y	-7.00	18.333	No
		Y	-7.00	24.167	No
		Y	-7.00	30.00	No

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier			
		Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	0.00	0.00
SL	Solar Load	No	0.00	0.00	0.00
D2	0.5DL+0.1SL	Yes	0.00	0.00	0.00

Earthquake (Dynamic analysis only)

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

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Units system: English
File name: S:\Projects\20-039 Summit Ridge NJ Roof Evaluation\39 Plymouth Street\Calc\ext Cant Bm.retx

List of materials

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

Members:

Profile	Material	Uweight [Kip/ft]	Length [ft]	Weight [Kip]
W 18X35	A36	3.51E-02	163.333	5.733
Total weight [Kip]				5.733

Current Date: 2/1/2021 6:50 AM
Units system: English
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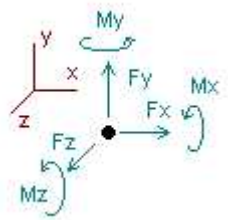
Analysis result

Reactions



Direction of positive forces and moments

Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **D2=0.5DL+0.1SL**						
1	0.00000	10.76558	0.00000	0.00000	0.00000	0.00000
2	0.00000	31.23457	0.00000	0.00000	0.00000	0.00000
5	0.00000	29.47585	0.00000	0.00000	0.00000	0.00000
6	0.00000	31.08204	0.00000	0.00000	0.00000	0.00000
8	0.00000	10.84196	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	113.40000	0.00000	0.00000	0.00000	0.00000

Maximum forces at members

Condition : **D2=0.5DL+0.1SL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	0.00	10.77	0.00	0.00	0.00	114.55
Min	0.00	-14.43	0.00	0.00	0.00	-74.90
MEMBER **2**						
Max	0.00	12.60	0.00	0.00	0.00	0.00
Min	0.00	8.40	0.00	0.00	0.00	-74.90
MEMBER **3**						
Max	0.00	8.40	0.00	0.00	0.00	72.10
Min	0.00	-8.40	0.00	0.00	0.00	0.00
MEMBER **4**						
Max	0.00	-8.40	0.00	0.00	0.00	0.00
Min	0.00	-12.60	0.00	0.00	0.00	-74.90
MEMBER **5**						
Max	0.00	12.68	0.00	0.00	0.00	73.81
Min	0.00	-12.52	0.00	0.00	0.00	-74.90
MEMBER **6**						
Max	0.00	14.36	0.00	0.00	0.00	0.00
Min	0.00	14.36	0.00	0.00	0.00	-71.79
MEMBER **7**						
Max	0.00	14.36	0.00	0.00	0.00	115.50
Min	0.00	-10.84	0.00	0.00	0.00	0.00

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	Section Ext Cant Beam W18x35				Sheet no./rev. 1	
	Calc. by RA	Date 1/31/2021	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

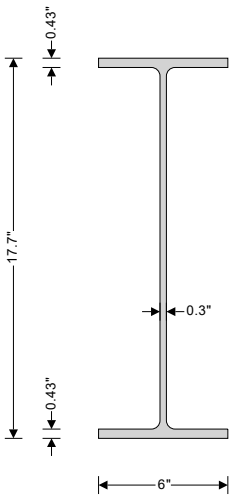
Safety factors

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	W 18x35 (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



W 18x35 (AISC 15th Edn (v15.0))
Section depth, d, 17.7 in
Section breadth, b_f, 6 in
Weight of section, Weight, 35 lbf/ft
Flange thickness, t_f, 0.425 in
Web thickness, t_w, 0.3 in
Area of section, A, 10.3 in^2
Radius of gyration about x-axis, r_x, 7.04 in
Radius of gyration about y-axis, r_y, 1.22 in
Elastic section modulus about x-axis, S_x, 57.6 in^3
Elastic section modulus about y-axis, S_y, 5.12 in^3
Plastic section modulus about x-axis, Z_x, 66.5 in^3
Plastic section modulus about y-axis, Z_y, 8.06 in^3
Second moment of area about x-axis, I_x, 510 in^4
Second moment of area about y-axis, I_y, 15.3 in^4

Analysis results

Required flexural strength - Major axis $M_{r,x}$ = **116** kips_ft

Restraint spacing

Major axis lateral restraint	L_x = **40** ft
Minor axis lateral restraint	L_y = **6** ft
Torsional restraint	L_z = **40** ft

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	b_f / (2 * t_f) = **7.06**	
Limiting ratio for compact section	λ_{pff} = 0.38 * √[E / F_y] = **10.79**	
Limiting ratio for non-compact section	λ_{rff} = 1.0 * √[E / F_y] = **28.38**	Compact

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

Width to thickness ratio $(d - 2 * k) / t_w$ = **53.49**

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

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

Section is compact in flexure

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **116** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **199.5** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **6** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **5.079** ft

Distance between flange centroids h_o = **17.3** in

c = **1**

r_{ts} = **1.51** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **15.004** ft

LTB modification factor C_b = **1.000**

Nominal flexural strength for lateral-torsional buckling - eq F2-2

$M_{n,ltb,x} = \min(C_b * (M_{p,x} - (M_{p,x} - 0.7 * F_y * S_x) * (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **192.2** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **192.2** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **115.1** kips_ft

$M_{r,x} / M_{c,x}$ = **1.008**

FAIL - Required flexural strength exceeds allowable flexural strength

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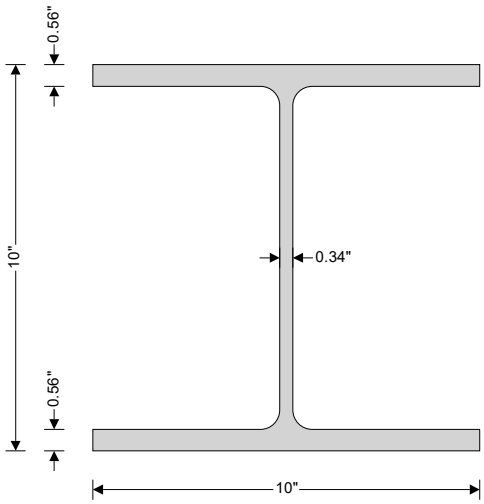
STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section	**W 10x49**

Design loading

Required axial strength	P_r = **244** 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 = **168** in
For buckling about y axis	L_y = **168** in
For torsional buckling	L_z = **168** 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	$L_{cx} = L_x \times K_x$ = **168** in

For buckling about y axis $L_{cy} = L_y \times K_y =$ **168** in

For torsional buckling $L_{cz} = L_z \times K_z =$ **168** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f / 2 =$ **5.000** in

Width to thickness ratio of flange $\lambda_f = b / t_f =$ **8.929**

Depth between root radii $h = d - 2 \times k =$ **7.880** in

Width to thickness ratio of web $\lambda_w = h / t_w =$ **23.176**

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 $SR_x = L_{cx} / r_x =$ **38.6**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **66.1**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **191.9** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) * F_y =$ **33.3** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A =$ **479.3** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **65.4** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) * F_y =$ **28.6** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} * A =$ **411.8** kips

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

Flexural-torsional elastic buckling stress $F_{et} = (\pi^2 * E * C_w / (L_{cz})^2 + G * J) / (I_x + I_y) =$ **100.1** ksi

Flexural-torsional buckling stress $F_{crt} = (0.658^{F_y / F_{et}}) * F_y =$ **31** ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} * A =$ **445.9** kips

Allowable compressive strength (cl. E1)

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

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c =$ **246.6** kips

PASS - The allowable compressive strength exceeds the required compressive strength

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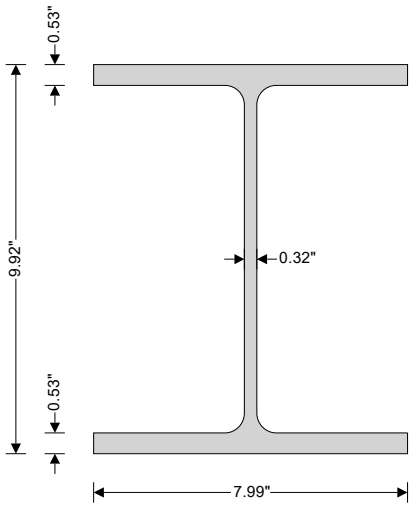
STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section	**W 10x39**

Design loading

Required axial strength	P_r = **122** 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 = **168** in
For buckling about y axis	L_y = **168** in
For torsional buckling	L_z = **168** 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	$L_{cx} = L_x \times K_x$ = **168** in

For buckling about y axis $L_{cy} = L_y \times K_y =$ **168** in

For torsional buckling $L_{cz} = L_z \times K_z =$ **168** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f / 2 =$ **3.995** in

Width to thickness ratio of flange $\lambda_f = b / t_f =$ **7.538**

Depth between root radii $h = d - 2 \times k =$ **7.860** in

Width to thickness ratio of web $\lambda_w = h / t_w =$ **24.952**

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 $SR_x = L_{cx} / r_x =$ **39.3**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **84.8**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **184.9** ksi

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

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A =$ **381.6** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **39.8** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) * F_y =$ **24.6** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} * A =$ **283.4** kips

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

Flexural-torsional elastic buckling stress $F_{et} = (\pi^2 * E * C_w / (L_{cz})^2 + G * J) / (I_x + I_y) =$ **82.6** ksi

Flexural-torsional buckling stress $F_{crt} = (0.658^{F_y / F_{et}}) * F_y =$ **30** ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} * A =$ **345** kips

Allowable compressive strength (cl. E1)

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

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c =$ **169.7** kips

PASS - The allowable compressive strength exceeds the required compressive strength