GreyFox CONSULTING, Inc.

Structural & Civil
Engineering Solutions

FEASIBILITY REPORT

Date: December 9, 2020

To: Ben Aparo and Nate Greenberg
Summit Ridge Energy

From: Ramin Abbaszadeh, PE

Re: 4 Gardner Road
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 early 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.

The subject building is a rectangular one-story steel structure (see pictures 1 through 5). The building has approximately 379 ft length in the north-south direction. The building has two widths. On the north side, the width is about 148 ft. The width of the building to the south is about 116 ft. The roof structure is comprised of a steel deck supported by steel bar joists. The joists, in turn, are supported on the cantilever steel beam system. Non-bearing masonry walls surround the perimeter of this building.

The building is occupied by 5 to 8 different tenants. For our preliminary investigation, we only measured one of the units in the middle of the building occupied by Lennox Industries. This unit has a large area with clear access to the roof structure. Offices with drop ceilings are located on the front of the building. Hence, we could not inspect the roof framing over the office area, as we could not reach the roof framing above the ceiling.

As shown in the pictures from inside the building, the roof structure in the storage area supports very little mechanical and electrical equipment. The building has a sprinkler system for fire protection. For the office area, we added additional dead loads to account for drop ceilings and HVAC ducts.

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 = 2 psf was assumed
  - Dead Load: Collateral (lights, piping, ductwork, electrical) = 3 psf was assumed for future application
  - Additional Load at the office area = 5 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 = 33 psf + steel weight
  - Existing total load at office area = 38 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 east to west with an approximate span of 45 ft. The bar joists are supported on the cantilevered steel beam system.

The approximate spacing between bar joists is 5'-8". Based on our field measurement, the joists are 28-inch-deep with 1.75 inches by 1.75 inches double angles top and bottom chords. The thickness of the top chords is 0.23 inches. The bottom chords have a thickness of 0.18 inches. The web members are comprised of 0.89 inches diameter round bars.

The historical data from SJI designates bar joists with similar cord and web elements as 28K7 joists. At 45 feet span and spacing of 5'-8", the 28k7 joist can support a maximum of 234 pounds per linear foot (plf) or 41 psf uniform roof load. The existing load over the storage area 33 psf. The existing joists over the storage area can support solar panel systems up to 8 psf uniform loads, beyond the assumed dead and snow load. The assumed dead, and snow load over the office area is 38 psf. That leaves only 4 psf for solar panels in this area.

ROOF DECK

The roof deck is a 1.5-inch type B wide rib (see picture 5). We were not able to measure the thickness of the deck. Based on the joist spacing, it is most likely that the deck is 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 37 psf. The deck capacity can support the additional load from solar panels.

STEEL BEAMS

The steel beams supporting the bar joists are spaced about 45 feet apart. A 4-span Cantilevered Beams system was designed for the support of the roof joists. The Cantilevered Beams frame has two beams that cantilever and two beams simply supported off of the cantilevered beams. One of these beams is over the office area, which we could not verify. We made assumptions for the beam that we could not measure based on our analysis of the system. The framing over the office area must be further investigated and verified at a later date.

The cantilevered beams are most likely W21x55, and the simply supported beams are W21x44. Our calculations show that the beams above the office area do not have sufficient capacity to support the existing dead plus snow load and the solar panels. But, solar panels with a maximum of 8 psf weight could be installed over the storage area.

STEEL COLUMNS

The columns are most likely W10x49 with an approximate length of 23 feet. These columns have sufficient capacity to support the solar panel weight up to 12 psf.

Conclusions

Our professional opinion is that this building's roof structure can support solar panels up to 8 psf over areas that do not have a drop ceiling and additional loads from HVAC ducts.

Further investigation of the areas that we could not access is necessary to ensure that the roof system is similar. Some of the columns and beams are hidden in the demising walls. Selective demolition is required to verify the hidden members. If this building is selected for the next phase, a more comprehensive investigation of the entire building is necessary. During this phase, provisions shall be made to access the framing above the office areas.

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

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

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

Sincerely,
Grey Fox Consulting, Inc.

Ramin Abbaszadeh, PE, SE
Principal

Picture 1

Picture 2

Picture 3

Picture 4

Picture 5

SUMMIT RIDGE ENERGY

4 Gardner Road
Farifield, New Jersey

Roof Evaluation
Structural Calculations

GreyFox CONSULTING, Inc.

Structural & Civil
Engineering Solutions

DATE: 12/9/2020
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 4 Gardner Rd., Fairfield, New Jersey				Job Ref.	
	Section Snow Load				Sheet no./rev. 1	
	Calc. by RA	Date 12/7/2020	Chk'd by	Date	App'd by	Date

SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.09

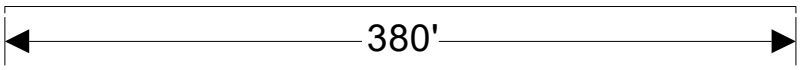
Building details

Roof type	Flat
Width of roof	b = **380.00** 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
Structural & Civil Engineering Solutions
(856) 216-9373
Fax (856) 216-9376

JOB 4 GARDNER RD.
SHEET NO. 1 OF
CALCULATED BY RA DATE 12/7/2020
CHECKED BY DATE
SCALE

BLDG. LENGTH 379' +/-
BLDG. WIDTH 148' +/-
AND 116' +/-

ROOF DL (STORAGE)

DECK / INSUL / BUR	=	6 PSF
SPRINKLER	=	2 PSF
MISC.	=	3 PSF
JOIST	=	2 PSF
TOTAL		13 PSF

OFFICE ADDITIONAL DL 5 PSF
OFFICE TOTAL DL 18 PSF
SNOW 20 PSF

JOIST SPACING 5'-8"
STORAGE (13 + 20)(5.67') = 187 PLF
OFFICE (18 + 20)(5.67') = 215 PLF

JOIST 28K7 CAPACITY = 234 PLF

Ramin Abbaszadeh

From: Ken Charles <kcharles@steeljoist.org>
Sent: Monday, December 7, 2020 2:11 PM
To: Ramin Abbaszadeh
Subject: RE: Joist Investigation Form submitted at Steel Joist Institute

The joist you have presented resembles a 28K7 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 7, 2020 11:39 AM
To: kcharles@steeljoist.org
Subject: Joist Investigation Form submitted at Steel Joist Institute

Ramin Abbaszadeh submitted a Joist Investigation Form.

Case Number	26062
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	4 Gardner Rd
Why are you requesting this information?	Evaluation
Other	
What year was the building constructed (or approximate age of the structure)?	1980 to 1987
Who was the joist manufacturer?	The Ceco Corp.
Is there a tag on the joist?	Yes
Yes? Provide tag information	9 -15 -791 SJ3J1
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?	44' - 6" to 45'-0"
What is the joist spacing?	5' -5"
What is the interior panel point spacing?	24" +/-
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	1.75x1.75
Top chord thickness	0.23
Top chord diameter	

Bottom chord shape	2 Angles
Bottom chord leg size	1.75 x 1.75
Bottom chord thickness	0.18
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	0.89 +/-
Measurement device	Micrometer
Other device	

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

Based on a Maximum Allowable Tensile Stress of 30,000 psi

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

STEEL MEMBER ANALYSIS & DESIGN (AISC 360)

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

Tedds calculation version 4.4.05

ANALYSIS

Tedds calculation version 1.0.35

Geometry

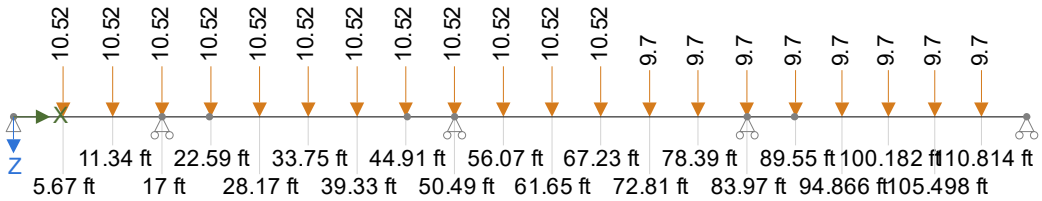
Geometry (ft) - Steel (AISC)

Span	Length (ft)	Section	Start Support	End Support
1	17	W 21x55	Pinned	Roller Pin X
2	5.42	W 21x55	Roller Pin X	Free
3	22.67	W 21x44	Free	Free
4	5.42	W 21x55	Free	Roller Pin X
5	33.5	W 21x55	Roller Pin X	Roller Pin X
6	5.42	W 21x55	Roller Pin X	Free
7	26.58	W 21x44	Free	Roller Pin X
W 21x44: Area 13 in^2, Inertia Major 843 in^4, Inertia Minor 21 in^4, Shear area parallel to Minor 7 in^2, Shear area parallel to Major 5 in^2				
W 21x55: Area 16 in^2, Inertia Major 1140 in^4, Inertia Minor 48 in^4, Shear area parallel to Minor 8 in^2, Shear area parallel to Major 8 in^2				
Steel (AISC): Density 490 lbm/ft^3, Youngs 29000 ksi, Shear 11200 ksi, Thermal 0.000012 $°C^{-1}$				

Loading

Self weight included

Dead - Loading (kips)



Load combination factors

Load combination	Self Weight	Dead	Live
1.0 SW + 3 DL (Strength)	1.00	1.00	1.00
1.0 SW + 3dl (Service)	1.00	1.00	1.00

Member Loads

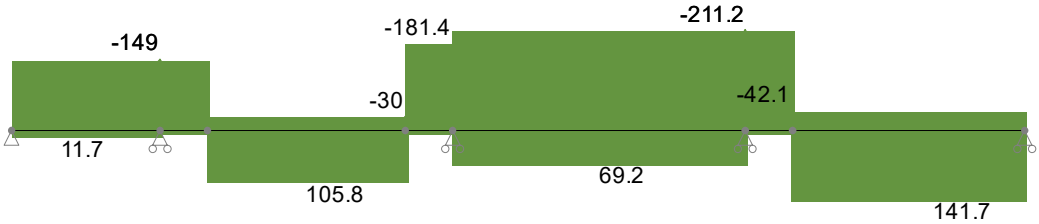
Grey Fox Consulting, Inc. 57 Cooper Avenue Cherry Hill, NJ 08002	Project 4 Gardner Rd., Fairfield, New Jersey				Job Ref.	
	Section Cantilever Beam System Analysis				Sheet no./rev. 2	
	Calc. by RA	Date 12/7/2020	Chk'd by	Date	App'd by	Date

Member	Load case	Load Type	Orientation	Description
Beam	Dead	Point load	GlobalZ	10.52 kips at 5.67 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 11.34 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 17 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 22.59 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 28.17 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 33.75 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 39.33 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 44.91 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 50.49 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 56.07 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 61.65 ft
Beam	Dead	Point load	GlobalZ	10.52 kips at 67.23 ft
Beam	Dead	Point load	GlobalZ	9.7 kips at 72.81 ft
Beam	Dead	Point load	GlobalZ	9.7 kips at 78.39 ft
Beam	Dead	Point load	GlobalZ	9.7 kips at 83.97 ft
Beam	Dead	Point load	GlobalZ	9.7 kips at 89.55 ft
Beam	Dead	Point load	GlobalZ	9.7 kips at 94.866 ft
Beam	Dead	Point load	GlobalZ	9.7 kips at 100.182 ft
Beam	Dead	Point load	GlobalZ	9.7 kips at 105.498 ft
Beam	Dead	Point load	GlobalZ	9.7 kips at 110.814 ft

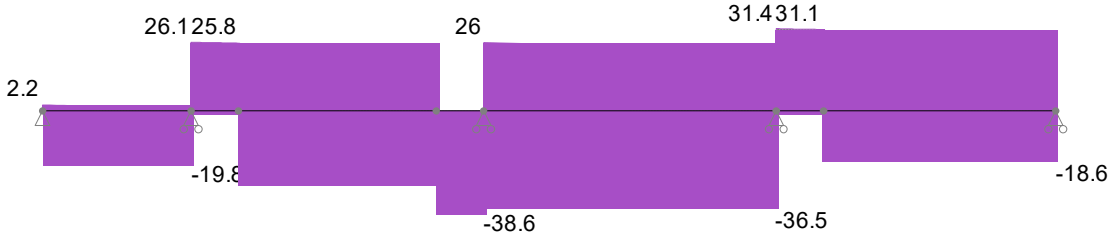
Results

Forces

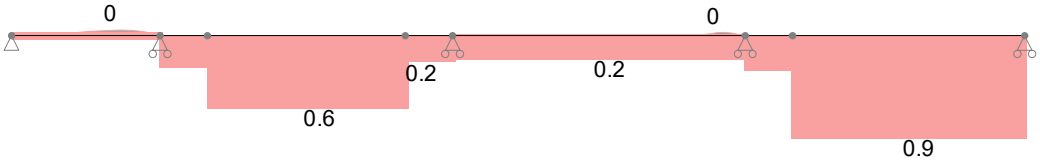
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)



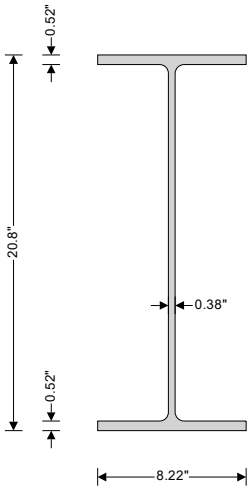
Safety factors

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**

Beam - Span 1 design

Section details

Section type	W 21x55 (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 21x55 (AISC 15th Edn (v15.0))
Section depth, d, 20.8 in
Section breadth, b_f, 8.22 in
Weight of section, Weight, 55 lbf/ft
Flange thickness, t_f, 0.522 in
Web thickness, t_w, 0.375 in
Area of section, A, 16.2 in^2
Radius of gyration about x-axis, r_x, 8.4 in
Radius of gyration about y-axis, r_y, 1.73 in
Elastic section modulus about x-axis, S_x, 110 in^3
Elastic section modulus about y-axis, S_y, 11.8 in^3
Plastic section modulus about x-axis, Z_x, 126 in^3
Plastic section modulus about y-axis, Z_y, 18.4 in^3
Second moment of area about x-axis, I_x, 1140 in^4
Second moment of area about y-axis, I_y, 48.4 in^4

Lateral restraint

Top flange has lateral restraint at supports plus 5ft 8in & 11ft 4in
Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0 SW + 3 DL (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio $b_f / (2 * t_f)$ = **7.87**

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

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

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

Width to thickness ratio $(d - 2 * k) / t_w =$ **50.03**

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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x} =$ **2.2** kips

Web area $A_w = d * t_w =$ **7.8** in^2

Web plate buckling coefficient $k_v =$ **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 $C_{v1} =$ **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1} =$ **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v =$ **112.3** kips

$V_{r,x} / V_{c,x} =$ **0.02**

PASS - Allowable shear strength exceeds required shear strength

Check design 5ft 8.04in along span

Design of members for shear - Chapter G

Required shear strength $V_{r,x} =$ **8.6** kips

Web area $A_w = d * t_w =$ **7.8** in^2

Web plate buckling coefficient $k_v =$ **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 $C_{v1} =$ **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1} =$ **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v =$ **112.3** kips

$V_{r,x} / V_{c,x} =$ **0.077**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x} =$ **11.7** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x =$ **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b =$ **5.667** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)} =$ **7.202** ft

Distance between flange centroids $h_o =$ **20.3** in

$c =$ **1**

$r_{ts} =$ **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **21.182** ft

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

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **378** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **226.3** kips_ft

$M_{r,x} / M_{c,x}$ = **0.052**

PASS - Allowable flexural strength exceeds required flexural strength

Check design at end of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **19.8** kips

Web area $A_w = d * t_w$ = **7.8** in^2

Web plate buckling coefficient k_v = **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_{v1} = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **112.3** kips

$V_{r,x} / V_{c,x}$ = **0.176**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **149** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **17** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **7.202** ft

Distance between flange centroids h_o = **20.3** in

c = **1**

r_{ts} = **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **21.182** ft

Moment at quarter point of segment M_A = **8.9** kips_ft

Moment at center-line of segment M_B = **12.9** kips_ft

Moment at three quarter point of segment M_C = **65.6** kips_ft

Maximum moment in segment M_{max} = **149** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$ = **2.876**

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

$M_{n,ltb,x} = \min(C_b * (M_{p,x} - (M_{p,x} - 0.7 * F_y * S_x) * (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **378** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **378** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **226.3** kips_ft

$M_{r,x}$ / $M_{c,x}$ = **0.658**

PASS - Allowable flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0 SW + 3dl (Service)

Check design 12ft 6.824in along span

Design of members for x-x axis deflection

Maximum deflection — δ_x = **0.047** in

Allowable deflection — $\delta_{x,Allowable} = L_{m1_s1} / 240$ = **0.85** in

$\delta_x / \delta_{x,Allowable}$ = **0.055**

PASS - Allowable deflection exceeds design deflection

Beam - Span 2 design

Section details

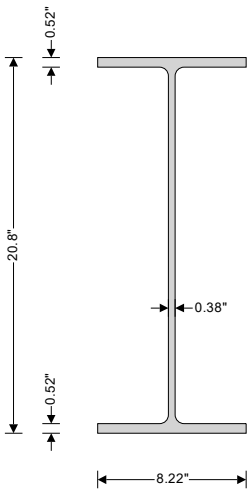
Section type — W 21x55 (AISC 15th Edn (v15.0))

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 21x55 (AISC 15th Edn (v15.0))
Section depth, d, 20.8 in
Section breadth, b_f, 8.22 in
Weight of section, Weight, 55 lbf/ft
Flange thickness, t_f, 0.522 in
Web thickness, t_w, 0.375 in
Area of section, A, 16.2 in^2
Radius of gyration about x-axis, r_x, 8.4 in
Radius of gyration about y-axis, r_y, 1.73 in
Elastic section modulus about x-axis, S_x, 110 in^3
Elastic section modulus about y-axis, S_y, 11.8 in^3
Plastic section modulus about x-axis, Z_x, 126 in^3
Plastic section modulus about y-axis, Z_y, 18.4 in^3
Second moment of area about x-axis, I_x, 1140 in^4
Second moment of area about y-axis, I_y, 48.4 in^4

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.0 SW + 3 DL (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio — $b_f / (2 * t_f)$ = **7.87**

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

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

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

Width to thickness ratio — $(d - 2 * k) / t_w$ = **50.03**

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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x} =$ **26.1** kips

Web area $A_w = d * t_w =$ **7.8** in^2

Web plate buckling coefficient $k_v =$ **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 $C_{v1} =$ **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1} =$ **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v =$ **112.3** kips

$V_{r,x} / V_{c,x} =$ **0.233**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x} =$ **149** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x =$ **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b =$ **5.42** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)} =$ **7.202** ft

Distance between flange centroids $h_o =$ **20.3** in

$c =$ **1**

$r_{ts} =$ **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}) =$ **21.182** ft

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

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x} =$ **378** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b =$ **226.3** kips_ft

$M_{r,x} / M_{c,x} =$ **0.658**

PASS - Allowable flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0 SW + 3dl (Service)

Check design at end of span

Design of members for x-x axis deflection

Maximum deflection $\delta_x =$ **0.245** in

Allowable deflection $\delta_{x,Allowable} = L_{m1_s2} / 240 =$ **0.271** in

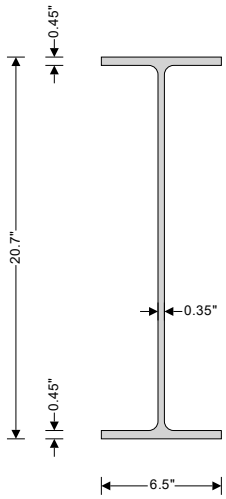
$\delta_x / \delta_{x,Allowable} =$ **0.902**

PASS - Allowable deflection exceeds design deflection

Beam - Span 3 design

Section details

Section type	W 21x44 (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 21x44 (AISC 15th Edn (v15.0))
Section depth, d, 20.7 in
Section breadth, b_f, 6.5 in
Weight of section, Weight, 44 lbf/ft
Flange thickness, t_f, 0.45 in
Web thickness, t_w, 0.35 in
Area of section, A, 13 in^2
Radius of gyration about x-axis, r_x, 8.06 in
Radius of gyration about y-axis, r_y, 1.26 in
Elastic section modulus about x-axis, S_x, 81.6 in^3
Elastic section modulus about y-axis, S_y, 6.37 in^3
Plastic section modulus about x-axis, Z_x, 95.4 in^3
Plastic section modulus about y-axis, Z_y, 10.2 in^3
Second moment of area about x-axis, I_x, 843 in^4
Second moment of area about y-axis, I_y, 20.7 in^4

Lateral restraint

Top flange has lateral restraint at supports plus 5ft 8in, 11ft 4in & 17ft
Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0 SW + 3 DL (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	$b_f / (2 * t_f)$ = **7.22**	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 * \sqrt{[E / F_y]}$ = **10.79**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 * \sqrt{[E / F_y]}$ = **28.38**	Compact

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

Width to thickness ratio	$(d - 2 * k) / t_w$ = **53.71**	
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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength	$V_{r,x}$ = **25.8** kips
Web area	$A_w = d * t_w$ = **7.245** in^2
Web plate buckling coefficient	k_v = **5.34**
	$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2	C_{v1} = **1.000**
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **156.5** kips
Safety factor	$\Omega_v = 1.50$
Allowable shear strength	$V_{c,x} = V_{n,x} / \Omega_v$ = **104.3** kips
	$V_{r,x} / V_{c,x}$ = **0.248**
	PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength	$M_{r,x}$ = **8.1** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1	$M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **286.2** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length	L_b = **22.67** ft
Limiting unbraced length for yielding - eq F2-5	$L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **5.245** ft
Distance between flange centroids	h_o = **20.3** in
	c = **1**
	r_{ts} = **1.6** in
Limiting unbraced length for inelastic LTB - eq F2-6	$L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **15.752** ft
Moment at quarter point of segment	M_A = **79.8** kips_ft
Moment at center-line of segment	M_B = **105.8** kips_ft
Moment at three quarter point of segment	M_C = **68.9** kips_ft
Maximum moment in segment	M_{max} = **105.8** kips_ft
LTB modification factor - eq F1-1	$C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$ = **1.167**
Critical flexural stress - eq F2-4	$F_{cr} = (C_b * \pi^2 * E / (L_b / r_{ts})^2) * \sqrt{(1 + 0.078 * (J * c / (S_x * h_o)) * (L_b / r_{ts})^2)}$ = **16.5** ksi
Nominal flexural strength for lateral-torsional buckling - eq F2-3	
	$M_{n,ltb,x} = \min(F_{cr} * S_x, M_{p,x})$ = **112.4** kips_ft

Allowable flexural strength - F1

Nominal flexural strength	$M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **112.4** kips_ft
Allowable flexural strength	$M_{c,x} = M_{n,x} / \Omega_b$ = **67.3** kips_ft
	$M_{r,x} / M_{c,x}$ = **0.120**
	PASS - Allowable flexural strength exceeds required flexural strength

<u>Check design 11ft 3.96in along span</u>

Design of members for shear - Chapter G

Required shear strength	$V_{r,x}$ = **6.2** kips
Web area	$A_w = d * t_w$ = **7.245** in^2
Web plate buckling coefficient	k_v = **5.34**
	$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$
Web shear coefficient - eq G2-2	C_{v1} = **1.000**
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **156.5** kips
Safety factor	$\Omega_v = 1.50$
Allowable shear strength	$V_{c,x} = V_{n,x} / \Omega_v$ = **104.3** kips
	$V_{r,x} / V_{c,x}$ = **0.06**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **105.8** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **286.2** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **5.668** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **5.245** ft

Distance between flange centroids h_o = **20.3** in

c = **1**

r_{ts} = **1.6** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **15.752** ft

Moment at quarter point of segment M_A = **87.1** kips_ft

Moment at center-line of segment M_B = **93.4** kips_ft

Moment at three quarter point of segment M_C = **99.7** kips_ft

Maximum moment in segment M_{max} = **105.8** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$ = **1.103**

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

$M_{n,ltb,x} = \min(C_b * (M_{p,x} - (M_{p,x} - 0.7 * F_y * S_x) * (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **286.2** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **286.2** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **171.4** kips_ft

$M_{r,x} / M_{c,x}$ = **0.617**

PASS - Allowable flexural strength exceeds required flexural strength

<u>Check design 11ft 4.02in along span</u>

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **6.2** kips

Web area $A_w = d * t_w$ = **7.245** in^2

Web plate buckling coefficient k_v = **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_{v1} = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **156.5** kips

Safety factor Ω_v = 1.50

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **104.3** kips

$V_{r,x} / V_{c,x}$ = **0.06**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **105.8** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **286.2** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **5.668** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **5.245** ft

Distance between flange centroids h_o = **20.3** in

c = **1**

r_{ts} = **1.6** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **15.752** ft

Moment at quarter point of segment M_A = **87.1** kips_ft

Moment at center-line of segment M_B = **93.4** kips_ft

Moment at three quarter point of segment M_C = **99.7** kips_ft

Maximum moment in segment M_{max} = **105.8** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$ = **1.103**

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

$M_{n,ltb,x} = \min(C_b * (M_{p,x} - (M_{p,x} - 0.7 * F_y * S_x) * (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **286.2** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **286.2** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **171.4** kips_ft

$M_{r,x} / M_{c,x}$ = **0.617**

PASS - Allowable flexural strength exceeds required flexural strength

<u>Check design at end of span</u>

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **27.8** kips

Web area $A_w = d * t_w$ = **7.245** in^2

Web plate buckling coefficient k_v = **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_{v1} = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **156.5** kips

Safety factor Ω_v = 1.50

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **104.3** kips

$V_{r,x} / V_{c,x}$ = **0.266**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **30** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **286.2** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **22.67** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **5.245** ft

Distance between flange centroids h_o = **20.3** in

c = **1**

r_{ts} = **1.6** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **15.752** ft

Moment at quarter point of segment M_A = **79.8** kips_ft

Moment at center-line of segment M_B = **105.8** kips_ft

Moment at three quarter point of segment M_C = **68.9** kips_ft

Maximum moment in segment M_{max} = **105.8** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$ = **1.167**

Critical flexural stress - eq F2-4 $F_{cr} = (C_b * \pi^2 * E / (L_b / r_{ts})^2) * \sqrt{(1 + 0.078 * (J * c / (S_x * h_o)) * (L_b / r_{ts})^2)}$ = **16.5** ksi

Nominal flexural strength for lateral-torsional buckling - eq F2-3

$M_{n,ltb,x} = \min(F_{cr} * S_x, M_{p,x})$ = **112.4** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **112.4** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **67.3** kips_ft

$M_{r,x} / M_{c,x}$ = **0.446**

PASS - Allowable flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0 SW + 3dl (Service)

Check design 11ft 0.472in along span

Design of members for x-x axis deflection

Maximum deflection δ_x = **0.61** in

Allowable deflection $\delta_{x,Allowable} = L_{m1_s3} / 240$ = **1.134** in

$\delta_x / \delta_{x,Allowable}$ = **0.539**

PASS - Allowable deflection exceeds design deflection

Beam - Span 4 design

Section details

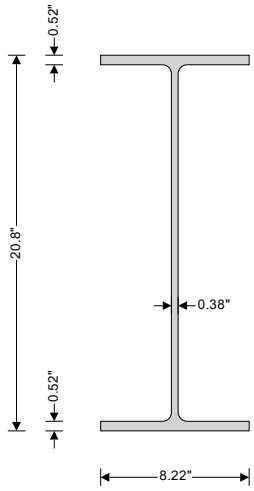
Section type W 21x55 (AISC 15th Edn (v15.0))

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 21x55 (AISC 15th Edn (v15.0))
Section depth, d, 20.8 in
Section breadth, b_f, 8.22 in
Weight of section, Weight, 55 lbf/ft
Flange thickness, t_f, 0.522 in
Web thickness, t_w, 0.375 in
Area of section, A, 16.2 in^2
Radius of gyration about x-axis, r_x, 8.4 in
Radius of gyration about y-axis, r_y, 1.73 in
Elastic section modulus about x-axis, S_x, 110 in^3
Elastic section modulus about y-axis, S_y, 11.8 in^3
Plastic section modulus about x-axis, Z_x, 126 in^3
Plastic section modulus about y-axis, Z_y, 18.4 in^3
Second moment of area about x-axis, I_x, 1140 in^4
Second moment of area about y-axis, I_y, 48.4 in^4

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.0 SW + 3 DL (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio $b_f / (2 * t_f)$ = **7.87**

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

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

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

Width to thickness ratio $(d - 2 * k) / t_w$ = **50.03**

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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **27.8** kips

Web area $A_w = d * t_w$ = **7.8** in^2

Web plate buckling coefficient k_v = **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_{v1} = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **112.3** kips

$V_{r,x} / V_{c,x}$ = **0.247**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **30** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x =$ **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b =$ **5.42** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)} =$ **7.202** ft

Distance between flange centroids $h_o =$ **20.3** in

$c =$ **1**

$r_{ts} =$ **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **21.182** ft

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

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x} =$ **378** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b =$ **226.3** kips_ft

$M_{r,x} / M_{c,x} =$ **0.133**

PASS - Allowable flexural strength exceeds required flexural strength

<u>Check design at end of span</u>

Design of members for shear - Chapter G

Required shear strength $V_{r,x} =$ **38.6** kips

Web area $A_w = d * t_w =$ **7.8** in^2

Web plate buckling coefficient $k_v =$ **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 $C_{v1} =$ **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1} =$ **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v =$ **112.3** kips

$V_{r,x} / V_{c,x} =$ **0.343**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x} =$ **181.4** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x =$ **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b =$ **5.42** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)} =$ **7.202** ft

Distance between flange centroids $h_o =$ **20.3** in

$c =$ **1**

$r_{ts} =$ **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **21.182** ft

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

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **378** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **226.3** kips_ft

$M_{r,x} / M_{c,x}$ = **0.801**

PASS - Allowable flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0 SW + 3dl (Service)

Check design at start of span

Design of members for x-x axis deflection

Maximum deflection δ_x = **0.21** in

Allowable deflection $\delta_{x,Allowable} = L_{m1_s4} / 240$ = **0.271** in

$\delta_x / \delta_{x,Allowable}$ = **0.775**

PASS - Allowable deflection exceeds design deflection

Beam - Span 5 design

Section details

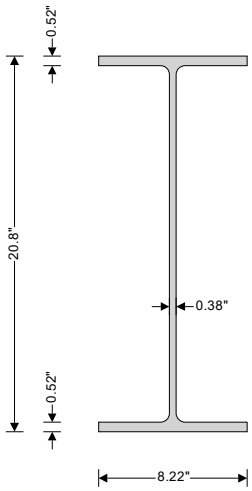
Section type W 21x55 (AISC 15th Edn (v15.0))

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 21x55 (AISC 15th Edn (v15.0))
Section depth, d, 20.8 in
Section breadth, b_f, 8.22 in
Weight of section, Weight, 55 lbf/ft
Flange thickness, t_f, 0.522 in
Web thickness, t_w, 0.375 in
Area of section, A, 16.2 in^2
Radius of gyration about x-axis, r_x, 8.4 in
Radius of gyration about y-axis, r_y, 1.73 in
Elastic section modulus about x-axis, S_x, 110 in^3
Elastic section modulus about y-axis, S_y, 11.8 in^3
Plastic section modulus about x-axis, Z_x, 126 in^3
Plastic section modulus about y-axis, Z_y, 18.4 in^3
Second moment of area about x-axis, I_x, 1140 in^4
Second moment of area about y-axis, I_y, 48.4 in^4

Lateral restraint

Top flange has lateral restraint at supports plus 5ft 7in, 11ft 1.9in, 16ft 8.9in, 22ft 3.8in, 27ft 10.8in & 33ft 5.8in

Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0 SW + 3 DL (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio $b_f / (2 * t_f)$ = **7.87**

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

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

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

Width to thickness ratio $(d - 2 * k) / t_w =$ **50.03**

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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x} =$ **26** kips

Web area $A_w = d * t_w =$ **7.8** in^2

Web plate buckling coefficient $k_v =$ **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 $C_{v1} =$ **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1} =$ **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v =$ **112.3** kips

$V_{r,x} / V_{c,x} =$ **0.231**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x} =$ **181.4** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x =$ **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b =$ **33.5** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)} =$ **7.202** ft

Distance between flange centroids $h_o =$ **20.3** in

$c =$ **1**

$r_{ts} =$ **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}) =$ **21.182** ft

Moment at quarter point of segment $M_A =$ **4.6** kips_ft

Moment at center-line of segment $M_B =$ **69** kips_ft

Moment at three quarter point of segment $M_C =$ **14.8** kips_ft

Maximum moment in segment $M_{max} =$ **211.2** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C) =$ **3.062**

Critical flexural stress - eq F2-4 $F_{cr} = (C_b * \pi^2 * E / (L_b / r_{ts})^2) * \sqrt{(1 + 0.078 * (J * c / (S_x * h_o)) * (L_b / r_{ts})^2)} =$ **38.7** ksi

Nominal flexural strength for lateral-torsional buckling - eq F2-3

$M_{n,ltb,x} = \min(F_{cr} * S_x, M_{p,x}) =$ **355** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x}) =$ **355** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b =$ **212.6** kips_ft

$M_{r,x} / M_{c,x} =$ **0.853**

PASS - Allowable flexural strength exceeds required flexural strength

Check design 16ft 8.64in along span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **6.5** kips

Web area $A_w = d * t_w$ = **7.8** in^2

Web plate buckling coefficient k_v = **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_{v1} = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **112.3** kips

$V_{r,x} / V_{c,x}$ = **0.058**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **69.2** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **5.58** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **7.202** ft

Distance between flange centroids h_o = **20.3** in

c = **1**

r_{ts} = **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **21.182** ft

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

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **378** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **226.3** kips_ft

$M_{r,x} / M_{c,x}$ = **0.306**

PASS - Allowable flexural strength exceeds required flexural strength

Check design at end of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **36.5** kips

Web area $A_w = d * t_w$ = **7.8** in^2

Web plate buckling coefficient k_v = **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_{v1} = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **168.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **112.3** kips

$V_{r,x} / V_{c,x}$ = **0.325**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **211.2** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x$ = **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **33.5** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)}$ = **7.202** ft

Distance between flange centroids h_o = **20.3** in

c = **1**

r_{ts} = **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **21.182** ft

Moment at quarter point of segment M_A = **4.6** kips_ft

Moment at center-line of segment M_B = **69** kips_ft

Moment at three quarter point of segment M_C = **14.8** kips_ft

Maximum moment in segment M_{max} = **211.2** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$ = **3.062**

Critical flexural stress - eq F2-4 $F_{cr} = (C_b * \pi^2 * E / (L_b / r_{ts})^2) * \sqrt{(1 + 0.078 * (J * c / (S_x * h_o)) * (L_b / r_{ts})^2)}$ = **38.7** ksi

Nominal flexural strength for lateral-torsional buckling - eq F2-3

$M_{n,ltb,x} = \min(F_{cr} * S_x, M_{p,x})$ = **355** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **355** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **212.6** kips_ft

$M_{r,x} / M_{c,x}$ = **0.994**

PASS - Allowable flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0 SW + 3dl (Service)

Check design 16ft 1.68in along span

Design of members for x-x axis deflection

Maximum deflection δ_x = **0.181** in

Allowable deflection $\delta_{x,Allowable} = L_{m1_s5} / 240$ = **1.675** in

$\delta_x / \delta_{x,Allowable}$ = **0.108**

PASS - Allowable deflection exceeds design deflection

Beam - Span 6 design

Section details

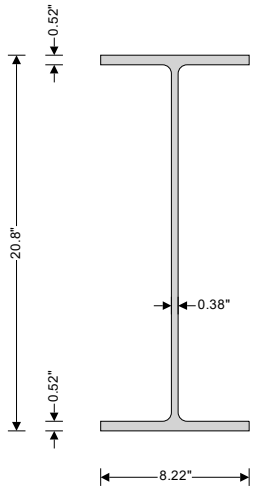
Section type W 21x55 (AISC 15th Edn (v15.0))

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 21x55 (AISC 15th Edn (v15.0))
Section depth, d, 20.8 in
Section breadth, b_f, 8.22 in
Weight of section, Weight, 55 lbf/ft
Flange thickness, t_f, 0.522 in
Web thickness, t_w, 0.375 in
Area of section, A, 16.2 in^2
Radius of gyration about x-axis, r_x, 8.4 in
Radius of gyration about y-axis, r_y, 1.73 in
Elastic section modulus about x-axis, S_x, 110 in^3
Elastic section modulus about y-axis, S_y, 11.8 in^3
Plastic section modulus about x-axis, Z_x, 126 in^3
Plastic section modulus about y-axis, Z_y, 18.4 in^3
Second moment of area about x-axis, I_x, 1140 in^4
Second moment of area about y-axis, I_y, 48.4 in^4

Lateral restraint

Top flange has lateral restraint at supports plus 5ft 4in

Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0 SW + 3 DL (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	$b_f / (2 * t_f)$ = **7.87**	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 * \sqrt{[E / F_y]}$ = **10.79**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 * \sqrt{[E / F_y]}$ = **28.38**	Compact

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

Width to thickness ratio	$(d - 2 * k) / t_w$ = **50.03**	
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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength	$V_{r,x}$ = **31.4** kips
Web area	$A_w = d * t_w$ = **7.8** in^2
Web plate buckling coefficient	k_v = **5.34**
	$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$
Web shear coefficient - eq G2-2	C_{v1} = **1.000**
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **168.5** kips
Safety factor	$\Omega_v = 1.50$
Allowable shear strength	$V_{c,x} = V_{n,x} / \Omega_v$ = **112.3** kips
	$V_{r,x} / V_{c,x}$ = **0.279**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength	$M_{r,x}$ = **211.2** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x =$ **378** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b =$ **5.42** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)} =$ **7.202** ft

Distance between flange centroids $h_o =$ **20.3** in

$c =$ **1**

$r_{ts} =$ **2.11** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **21.182** ft

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

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x} =$ **378** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b =$ **226.3** kips_ft

$M_{r,x} / M_{c,x} =$ **0.933**

PASS - Allowable flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0 SW + 3dl (Service)

Check design at end of span

Design of members for x-x axis deflection

Maximum deflection $\delta_x =$ **0.286** in

Allowable deflection $\delta_{x,Allowable} = L_{m1_s6} / 240 =$ **0.271** in

$\delta_x / \delta_{x,Allowable} =$ **1.057**

FAIL - Design deflection exceeds allowable deflection

Beam - Span 7 design

Section details

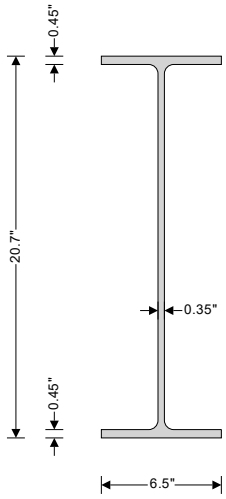
Section type W 21x44 (AISC 15th Edn (v15.0))

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 21x44 (AISC 15th Edn (v15.0))
Section depth, d, 20.7 in
Section breadth, b_f, 6.5 in
Weight of section, Weight, 44 lbf/ft
Flange thickness, t_f, 0.45 in
Web thickness, t_w, 0.35 in
Area of section, A, 13 in^2
Radius of gyration about x-axis, r_x, 8.06 in
Radius of gyration about y-axis, r_y, 1.26 in
Elastic section modulus about x-axis, S_x, 81.6 in^3
Elastic section modulus about y-axis, S_y, 6.37 in^3
Plastic section modulus about x-axis, Z_x, 95.4 in^3
Plastic section modulus about y-axis, Z_y, 10.2 in^3
Second moment of area about x-axis, I_x, 843 in^4
Second moment of area about y-axis, I_y, 20.7 in^4

Lateral restraint

Top flange has lateral restraint at supports plus 5ft 3.8in, 10ft 7.6in, 15ft 11.4in & 21ft 3.2in
Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0 SW + 3 DL (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	$b_f / (2 * t_f)$ = **7.22**	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 * \sqrt{[E / F_y]}$ = **10.79**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 * \sqrt{[E / F_y]}$ = **28.38**	Compact

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

Width to thickness ratio	$(d - 2 * k) / t_w$ = **53.71**	
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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength	$V_{r,x}$ = **31.1** kips
Web area	$A_w = d * t_w$ = **7.245** in^2
Web plate buckling coefficient	k_v = **5.34**
	$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$
Web shear coefficient - eq G2-2	C_{v1} = **1.000**
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 * F_y * A_w * C_{v1}$ = **156.5** kips
Safety factor	$\Omega_v = 1.50$
Allowable shear strength	$V_{c,x} = V_{n,x} / \Omega_v$ = **104.3** kips
	$V_{r,x} / V_{c,x}$ = **0.298**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength	$M_{r,x}$ = **42.1** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x =$ **286.2** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b =$ **26.58** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)} =$ **5.245** ft

Distance between flange centroids $h_o =$ **20.3** in

$c =$ **1**

$r_{ts} =$ **1.6** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **15.752** ft

Moment at quarter point of segment $M_A =$ **88.2** kips_ft

Moment at center-line of segment $M_B =$ **138.1** kips_ft

Moment at three quarter point of segment $M_C =$ **108.7** kips_ft

Maximum moment in segment $M_{max} =$ **141.7** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C) =$ **1.183**

Critical flexural stress - eq F2-4 $F_{cr} = (C_b * \pi^2 * E / (L_b / r_{ts})^2) * \sqrt{(1 + 0.078 * (J * c / (S_x * h_o)) * (L_b / r_{ts})^2)} =$ **13.3** ksi

Nominal flexural strength for lateral-torsional buckling - eq F2-3

$M_{n,ltb,x} = \min(F_{cr} * S_x, M_{p,x}) =$ **90.5** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x}) =$ **90.5** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b =$ **54.2** kips_ft

$M_{r,x} / M_{c,x} =$ **0.777**

PASS - Allowable flexural strength exceeds required flexural strength

<u>Check design 16ft 0.816in along span</u>

Design of members for shear - Chapter G

Required shear strength $V_{r,x} =$ **8.5** kips

Web area $A_w = d * t_w =$ **7.245** in^2

Web plate buckling coefficient $k_v =$ **5.34**

$(d - 2 * k) / t_w <= 2.24 * \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 $C_{v1} =$ **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 * F_y * A_w * C_{v1} =$ **156.5** kips

Safety factor $\Omega_v = 1.50$

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v =$ **104.3** kips

$V_{r,x} / V_{c,x} =$ **0.081**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x} =$ **141.7** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y * Z_x =$ **286.2** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b =$ **5.316** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 * r_y * \sqrt{(E / F_y)} =$ **5.245** ft

Distance between flange centroids h_o = **20.3** in

c = **1**

r_{ts} = **1.6** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{((J * c / (S_x * h_o)) + \sqrt{((J * c / (S_x * h_o))^2 + 6.76 * (0.7 * F_y / E)^2)}}$ = **15.752** ft

Moment at quarter point of segment M_A = **131.5** kips_ft

Moment at center-line of segment M_B = **120.1** kips_ft

Moment at three quarter point of segment M_C = **108.7** kips_ft

Maximum moment in segment M_{max} = **141.7** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$ = **1.139**

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

$M_{n,ltb,x} = \min(C_b * (M_{p,x} - (M_{p,x} - 0.7 * F_y * S_x) * (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **286.2** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **286.2** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **171.4** kips_ft

$M_{r,x} / M_{c,x}$ = **0.827**

PASS - Allowable flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0 SW + 3dl (Service)

Check design 12ft 6.195in along span

Design of members for x-x axis deflection

Maximum deflection δ_x = **0.873** in

Allowable deflection $\delta_{x,Allowable} = L_{m1_s7} / 240$ = **1.329** in

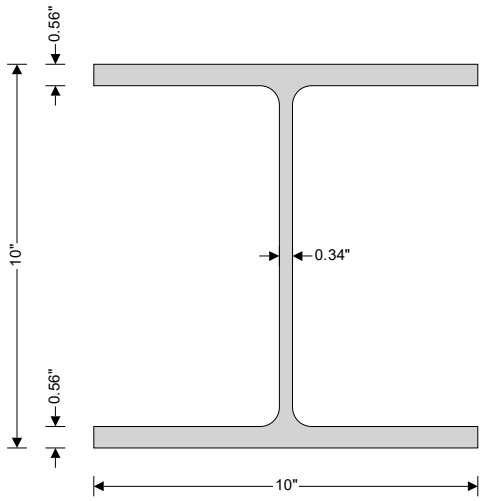
$\delta_x / \delta_{x,Allowable}$ = **0.657**

PASS - Allowable deflection exceeds design deflection

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 = **50** 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 = **288** in
For buckling about y axis	L_y = **288** in
For torsional buckling	L_z = **288** 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	$L_{cx} = L_x \times K_x$ = **288** in

For buckling about y axis $L_{cy} = L_y \times K_y =$ **288** in

For torsional buckling $L_{cz} = L_z \times K_z =$ **288** 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 $SR_x = L_{cx} / r_x =$ **66.2**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **113.4**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 * E / (SR_x)^2 =$ **65.3** ksi

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

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} * A =$ **411.6** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 * E / (SR_y)^2 =$ **22.3** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) * F_y =$ **18.3** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} * A =$ **263.5** kips

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

Flexural-torsional elastic buckling stress $F_{et} = (\pi^2 * E * C_w / (L_{cz})^2 + G * J) / (I_x + I_y) =$ **62.2** ksi

Flexural-torsional buckling stress $F_{crt} = (0.658^{F_y / F_{et}}) * F_y =$ **28.2** ksi

Nominal compressive strength for torsional and flexural-torsional buckling

$P_{nt} = F_{crt} * A =$ **406.8** kips

Allowable compressive strength (cl. E1)

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

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}, P_{nt}) / \Omega_c =$ **157.8** kips

PASS - The allowable compressive strength exceeds the required compressive strength