Steel Beam Design 2/16

HW - Steel Beam Design

Tower Project

Lab – Steel Columns

Structure II
Section 004

Yifan Ma
yifanma@umich.edu

HW - Steel Beam Design

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	25 FT
Span B	17 FT
Floor Dead Load	14 PSF
Floor Live Load	90 PSF

Given:
bracing type (Lb < Lp zone 1)
load

Goal:
Member Size?

1. Calculate Required Moment

   Determine Mn

   Mu = ϕ_b Mn

   Mn = Mu / ϕ_b

2. Determine the Minimum Zx required

   Mn = Fy*Zx = Mp

   Zx > Mn / Fy

3. Choose a section based on Z from the AISC table.
   Bold-faced sections are lighter

M_u = maximum moment from factored loads
ϕ_b = resistance factor for bending = 0.9
M_n = nominal moment (ultimate capacity)
F_y = yield strength of the steel
Z = plastic section modulus

Z_x Table 3-2 (continued) W-Shapes Selection by Z_x F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.1**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.94**	**13.2**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	425	83.5	125
W10×60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	341	85.7	129
W16×40	**73.0**	**182**	**274**	**113**	**170**	**6.67**	**10.0**	**5.55**	**15.9**	**518**	**97.6**	**146**
W12×50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	391	90.3	135
W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
W14×43	69.6	174	261	109	164	4.88	7.28	6.68	20.0	428	83.6	125
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	**54.6**	**136**	**205**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**
W16×31	**54.0**	**135**	**203**	**82.4**	**124**	**6.86**	**10.3**	**4.13**	**11.8**	**375**	**87.5**	**131**
W12×35	51.2	128	192	79.6	120	4.34	6.45	5.44	16.6	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.67	2.55	7.35	35.2	184	68.0	102
W14×30	**47.3**	**118**	**177**	**73.4**	**110**	**4.63**	**6.95**	**5.26**	**14.9**	**291**	**74.5**	**112**
W10×39	46.8	117	176	73.5	111	2.53	3.78	6.99	24.2	209	62.5	93.7
W16×26[v]	**44.2**	**110**	**166**	**67.1**	**101**	**5.93**	**8.98**	**3.96**	**11.2**	**301**	**70.5**	**106**
W12×30	43.1	108	162	67.4	101	3.97	5.96	5.37	15.6	238	64.0	95.9

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

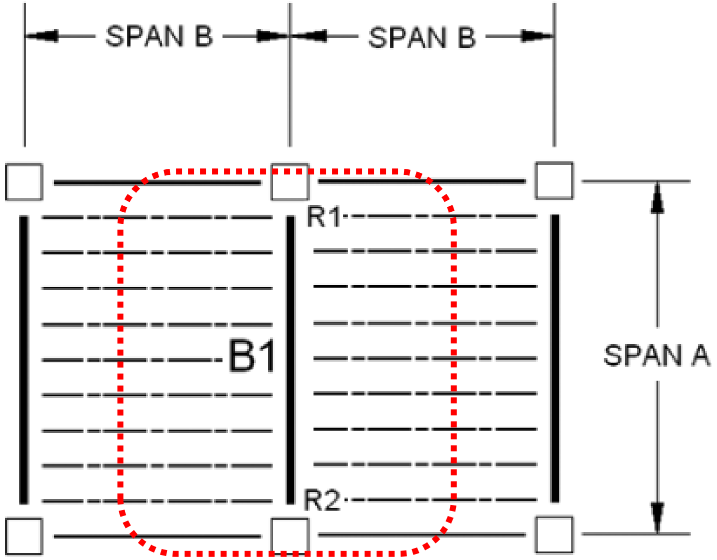
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	25 FT
Span B	17 FT
Floor Dead Load	14 PSF
Floor Live Load	90 PSF



1. The total unfactored floor dead load on the beam B1 (neglecting selfweight), w_DL*

w_DL* = DL * SpanB = 14*17 = 238 plf

2. The total unfactored floor live load on the beam, w_LL

w_LL= LL * SpanB = 90*17 = 1530 plf

3. The total factored design load on the beam (neglecting selfweight), wu*

wu*= 1.2*w_DL* + 1.6*w_LL
= (1.2*238+1.6*1530)/1000
= 2.73 klf

LRFD Analysis

Load & Resistance Factored Design (LRFD)

- Use loads with safety factor γ
- Use forces with strength factor ϕ

$P_{load} = \gamma \cdot P_{applied}$ $P_{load} \le P_{resisting}$ $P_{resisting} = \phi \cdot P_{material}$

Design Strength $P_u \le \phi P_n$ Required (Nominal) Strength

2.3 LOAD COMBINATIONS FOR STRENGTH DESIGN

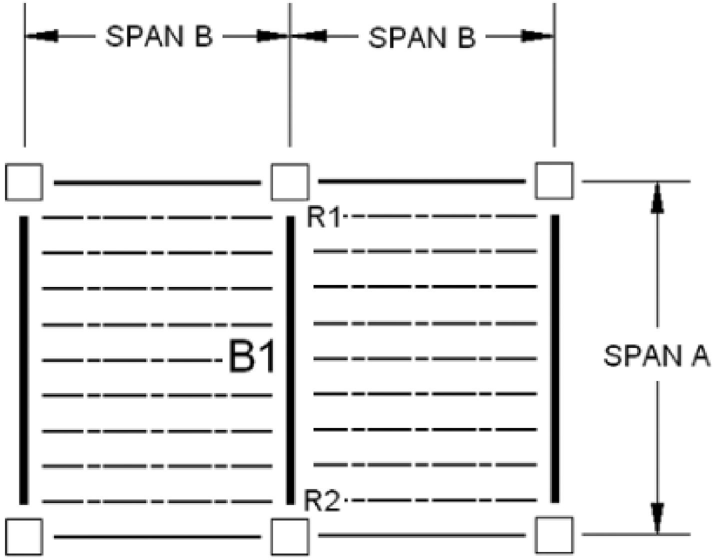
1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $0.9D + 1.0W$

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	25 FT
Span B	17 FT
Floor Dead Load	14 PSF
Floor Live Load	90 PSF

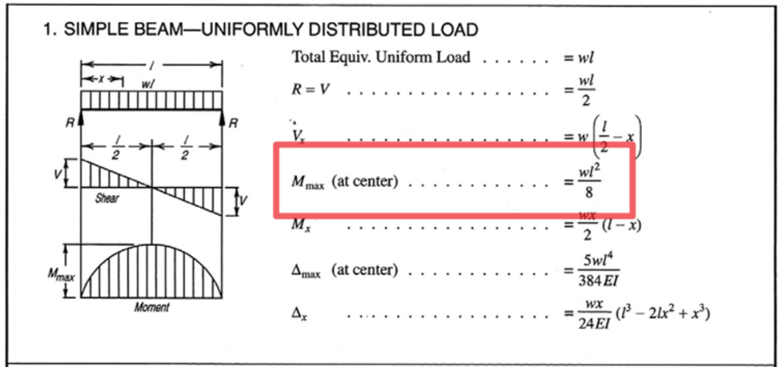


4. The factored design moment (neglecting selfweight), Mu*

Mu* = 1/8 x wu* x SpanA²

= 1/8*2.73*25²

= 213.28 k-ft



5. The nominal bending moment (neglecting selfweight), Mn*

Mn* = Mu* / ϕ

= 213.28*12/0.9

= 2843.73 k-in

Load & Resistance Factored Design (LRFD)

- Use loads with safety factor γ
- Use factor on ultimate strength ϕ

$$P_{load} = \gamma \cdot P_{applied\_load}$$

$$P_{load} \leq P_{resisting}$$

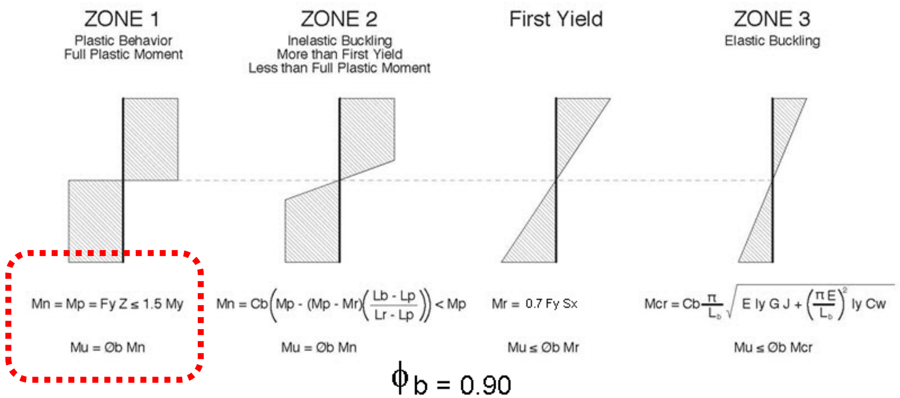
$$P_{resisting} = \phi \cdot P_{material\_strength}$$

6. The Plastic modulus of the section (neglecting selfweight), Zx*

Zx= Mn* / Fy

= 2843.73/50

= 56.87 in³

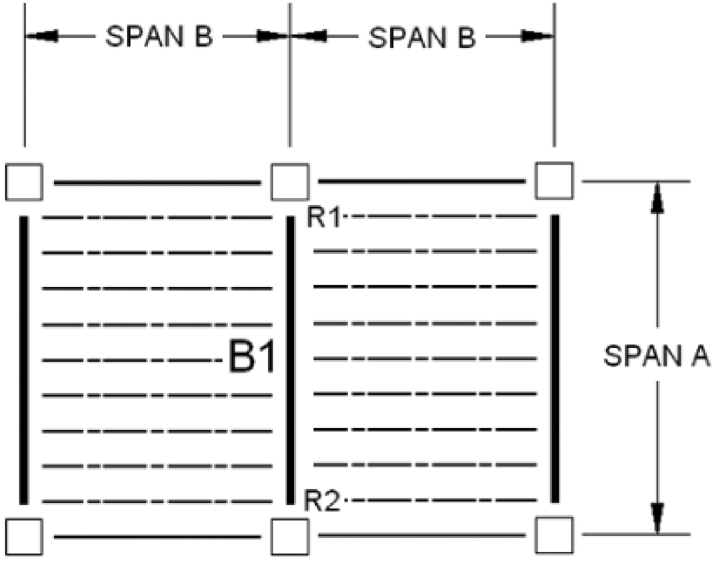


5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	25 FT
Span B	17 FT
Floor Dead Load	14 PSF
Floor Live Load	90 PSF



7. The nominal depth of the lightestpassing W-section from Zx table

Minimum Zx required = 56.87 in³

Choose a section based on Zx
from the AISC table 3-2.
Bold-faced sections are lighter
(Most efficient one_strongest and lightest)

W 18*35, Zx=66.5 in³ > 56.87 in³

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.1**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.94**	**13.2**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	425	83.5	125
W10×60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	341	85.7	129
W16×40	**73.0**	**182**	**274**	**113**	**170**	**6.67**	**10.0**	**5.55**	**15.9**	**518**	**97.6**	**146**
W12×50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	391	90.3	135
W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
W14×43	69.6	174	261	109	164	4.88	7.28	6.68	20.0	428	83.6	125
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	**54.6**	**136**	**205**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**
W16×31	**54.0**	**135**	**203**	**82.4**	**124**	**6.86**	**10.3**	**4.13**	**11.8**	**375**	**87.5**	**131**
W12×35	51.2	128	192	79.6	120	4.34	6.45	5.44	16.6	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.67	2.55	7.35	35.2	184	68.0	102
W14×30	**47.3**	**118**	**177**	**73.4**	**110**	**4.63**	**6.95**	**5.26**	**14.9**	**291**	**74.5**	**112**
W10×39	46.8	117	176	73.5	111	2.53	3.78	6.99	24.2	209	62.5	93.7
W16×26[v]	**44.2**	**110**	**166**	**67.1**	**101**	**5.93**	**8.98**	**3.96**	**11.2**	**301**	**70.5**	**106**
W12×30	43.1	108	162	67.4	101	3.97	5.96	5.37	15.6	238	64.0	95.9

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

8. The weight of the lightest passing W-section from Zx table

W 18*35, Zx=66.5 in

9. The Plastic modulus of the section, Zx

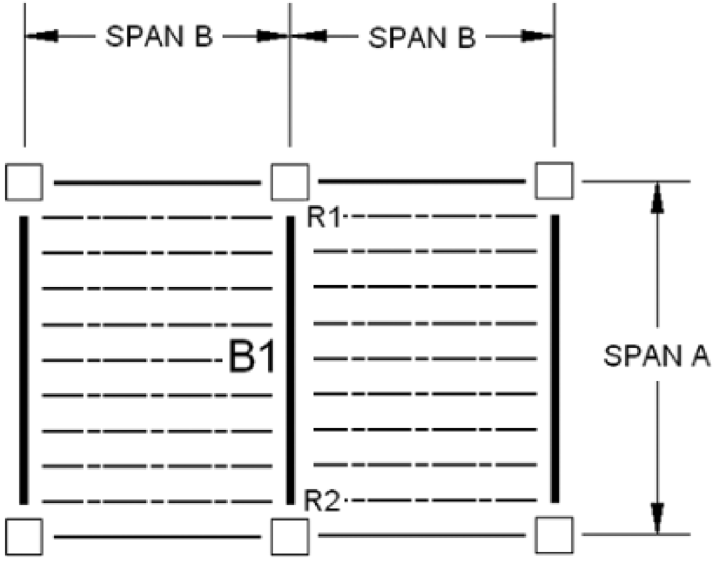
W 18*35, Zx=66.5 in

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	25 FT
Span B	17 FT
Floor Dead Load	14 PSF
Floor Live Load	90 PSF



10. The revised unfactored floor dead load on the beam (including selfweight), w_DL

w_DL= w_DL*+BeamWeight = 238+35 = 273 plf

11. The total factored design load on the beam (including selfweight), wu

wu= 1.2 * w_DL+ 1.6 * w_LL = (1.2*273+1.6*1530)/1000 = 2.7756 klf

12. The factored design moment (including selfweight), Mu in KIP-FT

Mu = 1/8 * wu* SpanA² = 1/8*2.7756*25² = 216.84 k-ft

13. The factored design moment (including selfweight), Mu in KIP-IN

Mu = 216.84*12 = 2602.08 k-in

14. The nominal factored bending moment for the chosen, phi Mn

ϕ *Mn = ϕ *(Zx* Fy) = 0.9*66.5*50 = 2992.5 k-in > Mu = 2602.08 Pass!

Tower Project

Timeline

Due until break - Feb 23

DATE	TOPIC	Text Reading	PROBLEMS (due dates online)
JAN 10	Course Intro	Onouye, Schodek	
JAN 12	Wood Properties	NDS	
JAN 15	**Martin Luther King Day **** No Class **** Martin Luther King Day **** No Class**		
JAN 17	Wood Beam Analysis	Schodek 6.4.2	
JAN 19	Recitation **[1-Wood Beams]**		1. Wood Beam Analysis
JAN 22	Wood Beam Design	Onouye 8	
JAN 24	Column Buckling	Onouye 9.1-9.2 & 9.4, Schodek 7.4.3	
JAN 26	Recitation		2. Wood Beam Design
JAN 29	Wood Columns - Tower Intro	NDS	
JAN 31	Cross Laminated Timbers	CLT Handbook	
FEB 2	Recitation **[2-Wood Columns]**		3. Wood Column Analysis
FEB 5	Steel Properties	AISC, Onouye 8.7	
FEB 7	Steel Beam Analysis	Schodek 6.4.3	
FEB 9	Recitation **[3-Steel Beams]**		4 Steel Beam Analysis
FEB 12	Steel Beam Design	Schodek 6.4.3	
FEB 14	Steel Column Analysis	Onouye 9.3, Schodek 7.4.4	
FEB 16	Recitation **[4-Steel Columns]**		**Prelim. Tower Report Due** 5. Steel Beam Design
FEB 19	Steel Column Design	Onouye 9.3, Schodek 7.4.4	
FEB 21	"Skyscrapers" David Macaulay video		
FEB 32	Recitation		6. Steel Column Analysis
FEB 26	**WINTER RECESS **** NO CLASS **** WINTER RECESS **** NO CLASS ******		
FEB 27	**WINTER RECESS **** NO CLASS **** WINTER RECESS **** NO CLASS ******		
MAR 1	**WINTER RECESS **** NO CLASS **** WINTER RECESS **** NO CLASS ******		
MAR 4	Continuous Beams	I. Engel Ch. 17, Schodek 8	
MAR 6	Gerber Beams	Schodek 8.4.4	
MAR 8	Recitation **[5-Continuous Beams]**		7. Three Moment Theorem
MAR 11	Intro to Concrete – PCA video.		
MAR 13	Concrete Beams	Schodek 6.4.4 – 6.4.6	
MAR 15	Recitation		
MAR 18	**Tower Testing **** Tower Testing **** Tower Testing **** Tower Testing ******		
MAR 20	Concrete Beams	I. Engel Ch.15	
MAR 22	Recitation **[6-Stress vs Strain]**		8. Concrete Beam Analysis
MAR 25	Concrete Beams		
MAR 27	Concrete Columns	Schodek 7.4.5	
MAR 29	Recitation **[7-Concrete Reinforcing]**		9. Concrete Beam Design
APR 1	Composite Sections	TMS 402	
APR 3	Masonry Walls	TMS 402	
APR 5	Recitation **[8-Composite Sections]**		10. Composite Sections
APR 8	Masonry Walls	TMS 402	
APR 10	Shells and Vaults	Schodek 12	
APR 12	Recitation **[9-Lateral Stability]**	**Final Tower Report Due**	11. Masonry Walls
APR 15	Combined Stress	I. Engel Ch. 19	
APR 17	Combined Stress	I. Engel Ch. 19	
APR 19	Recitation **[10-Combined Stress]**		12. Combined Stress
APR 22	Prestress & Post Tension		

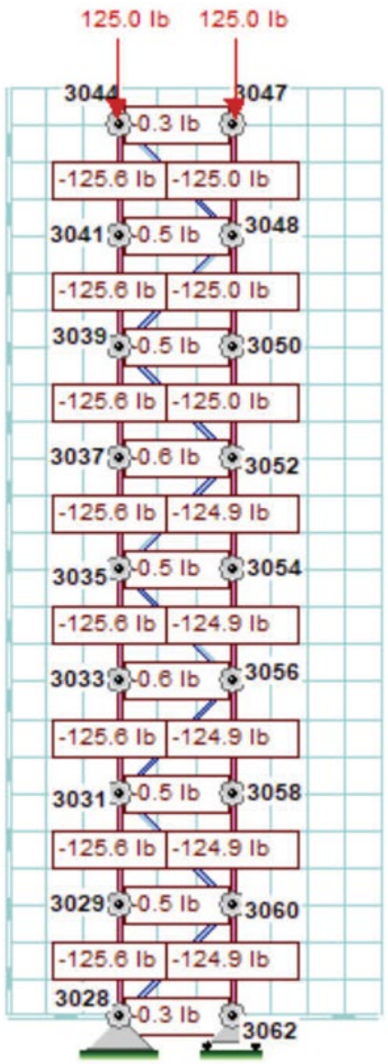
Tower Project

Dr.Frame

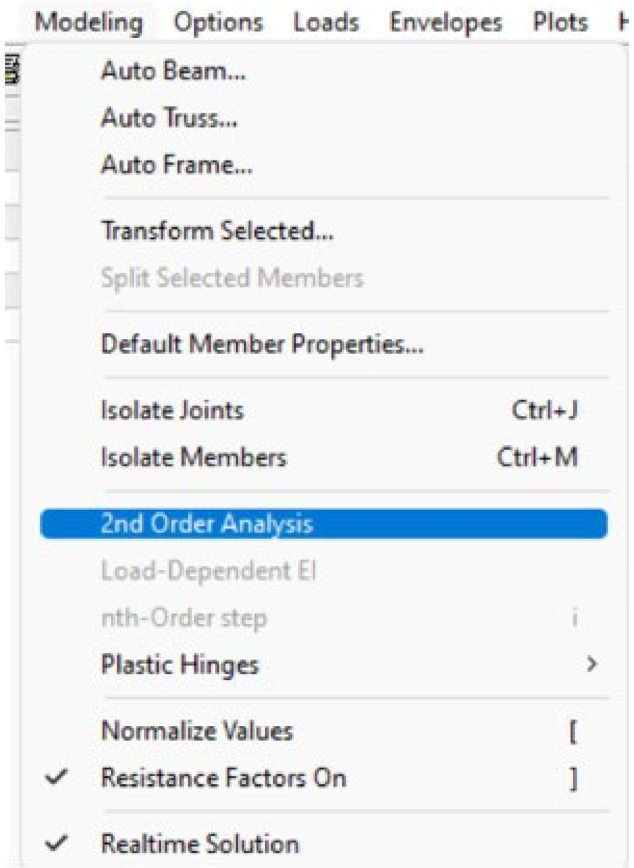
take this as one panel

2 panels totally

load\weight



2nd order Analysis



Tower Project

Dr.Frame

Properties of Basswood: (like in the Media Center)

Density (oven dry)	20 pcf *
E (buckling)	1,650,000 psi **
F (Compression ‖ to grain)	4745 psi *
F (Compression ⊥ to grain)	377 psi *
F (Tension ‖ to grain)	4500 psi (estimate)
F (Tension ⊥ to grain)	348 psi *
F (Shear ‖ to grain)	986 psi *
F (Flexure)	5900 psi *

* from http://www.matweb.com/ ** tested by PvB (small pieces in compression)

Section Data	
Section Type	Custom
Custom Sections	1/4"
Section Subtype	Rectangular
Depth	0.25 in
Width	0.25 in
Properties	Area = 0.0625 in^
In-plane Axis	Strong Axis
Lateral Bracing	No
EI Reduction	1
Material Properties	
Elastic Modulus	1.65e+06 psi
Yield Stress	4745 psi
Density	28 lbs/ft^3
Shear Modulus	6.346e+05 psi
End Conditions	
End 1 Fixity	Hinged
Rotational Stif	0 k-ft/rad
End 2 Fixity	Hinged
Rotational Stif	0 k-ft/rad
Misfit	
Length Misfit	0 in

Tower Project

Tips

overweight

glue and plates take approximately 10% of total members weight

smaller section

less force on the smaller sectional member; and vice versa

narrower spacing

width spacing, more steady; narrow spacing saves weight; if it is too narrow_ tilted

balsa wood: less weight but less strength

use it as plates

use it as connection members in tension (find its material properties online)

think about the handcraft ahead

how to limit the difficulty when building it? less joint? joint detailing?

LAB - Steel Columns

Description

This project gives the opportunity to identify steel sections and determine their properties and strength using the AISC tables.

Goals

To identify a steel section based on dimensions.
To determine the sectional properties using AISC table
To determine the load capacity based on AISC column table.

Procedure

1. Measure the steel column section shown below. (your GSI will tell you which one)
2. Based on the sectional dimensions find the shape in the steel table.
3. Use the column table and the given height to find the load capacity. Both columns are A-36 steel (Fy = 36 ksi).

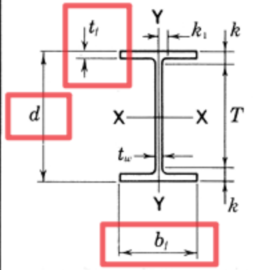
L= 15 ft. – 4 in.

or

CMYK

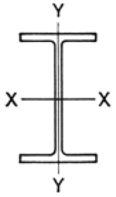
L = 13 ft. 4 in.

Section: W ___ x _____ Design Strength _________ kips



W SHAPES
Dimensions

Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Distance k_1
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.
W 8x 67	19.7	9.00	9	0.570	9/16	5/16	8.280	8 1/4	0.935	15/16	6 1/8	1 7/16	11/16
x 58	17.1	8.75	8 3/4	0.510	1/2	1/4	8.220	8 1/4	0.810	13/16	6 1/8	1 5/16	11/16
x 48	14.1	8.50	8 1/2	0.400	3/8	3/16	8.110	8 1/8	0.685	11/16	6 1/8	1 3/16	5/8
x 40	11.7	8.25	8 1/4	0.360	3/8	3/16	8.070	8 1/8	0.560	9/16	6 1/8	1 1/16	5/8
x 35	10.3	8.12	8 1/8	0.310	5/16	3/16	8.020	8	0.495	1/2	6 1/8	1	9/16
x 31	9.13	8.00	8	0.285	5/16	3/16	7.995	8	0.435	7/16	6 1/8	15/16	9/16
W 8x 28	8.25	8.06	8	0.285	5/16	3/16	6.535	6 1/2	0.465	7/16	6 1/8	15/16	9/16
x 24	7.08	7.93	7 7/8	0.245	1/4	1/8	6.495	6 1/2	0.400	3/8	6 1/8	7/8	9/16
W 8x 21	6.16	8.28	8 1/4	0.250	1/4	1/8	5.270	5 1/4	0.400	3/8	6 5/8	13/16	1/2
x 18	5.26	8.14	8 1/8	0.230	1/4	1/8	5.250	5 1/4	0.330	5/16	6 5/8	3/4	7/16
W 8 x15	4.44	8.11	8 1/8	0.245	1/4	1/8	4.015	4	0.315	5/16	6 5/8	3/4	1/2
x 13	3.84	7.99	8	0.230	1/4	1/8	4.000	4	0.255	1/4	6 5/8	11/16	7/16
x 10	2.96	7.89	7 7/8	0.170	3/16	1/8	3.940	4	0.205	3/16	6 5/8	5/8	7/16
W 6x 25	7.34	6.38	6 3/8	0.320	5/16	3/16	6.080	6 1/8	0.455	7/16	4 3/4	13/16	7/16
x 20	5.87	6.20	6 1/4	0.260	1/4	1/8	6.020	6	0.365	3/8	4 3/4	3/4	7/16
x 15	4.43	5.99	6	0.230	1/4	1/8	5.990	6	0.260	1/4	4 3/4	5/8	3/8
W 6x 16	4.74	6.28	6 1/4	0.260	1/4	1/8	4.030	4	0.405	3/8	4 3/4	3/4	7/16
x 12	3.55	6.03	6	0.230	1/4	1/8	4.000	4	0.280	1/4	4 3/4	5/8	3/8
x 9	2.68	5.90	5 7/8	0.170	3/16	1/8	3.940	4	0.215	3/16	4 3/4	9/16	3/8
W 5x 19	5.54	5.15	5 1/8	0.270	1/4	1/8	5.030	5	0.430	7/16	3 1/2	13/16	7/16
x 16	4.68	5.01	5	0.240	1/4	1/8	5.000	5	0.360	3/8	3 1/2	3/4	7/16
W 4x 13	3.83	4.16	4 1/8	0.280	1/4	1/8	4.060	4	0.345	3/8	2 3/4	11/16	7/16



$F_y = 36$ ksi

$F_y = 50$ ksi

COLUMNS
W shapes
Design axial strength in kips ($\phi = 0.85$)

Designation	W8											
Wt./ft	67		58		48		40		35		31	
F_y	36	50	36	50	36	50	36	50	36	50	36	50
Effective length in ft KL with respect to least radius of gyration r_y												
0	603	837	523	727	431	599	358	497	315	438	279	388
6	567	770	492	667	405	549	335	454	295	399	261	354
7	555	746	481	647	396	532	327	439	288	386	255	342
8	541	721	469	624	386	513	319	423	280	372	248	329
9	526	693	455	599	374	492	309	405	272	356	240	315
10	509	662	441	572	362	470	298	386	262	339	232	300
11	492	631	425	544	349	446	287	366	252	321	223	284
12	478	598	409	515	335	422	275	345	242	303	214	268
13	453	564	391	485	321	397	263	324	231	284	204	251
14	433	529	374	455	306	372	251	303	220	265	194	234
15	412	494	355	425	291	347	238	281	208	246	184	217
16	391	460	337	394	276	321	225	260	197	228	174	200
17	370	425	318	365	260	297	212	239	185	209	163	184
18	349	392	300	335	245	272	198	219	174	191	153	168
19	328	359	281	307	229	249	186	200	162	174	143	153
20	307	328	263	279	214	226	173	180	151	157	133	138
22	267	271	228	231	185	187	148	149	129	130	114	114
24	228	228	194	194	157	157	125	125	109	109	96	96
26	194	194	165	165	134	134	107	107	93	93	82	82
28	167	167	143	143	115	115	92	92	80	80	70	70
30	146	146	124	124	100	100	80	80	70	70	61	61
32	128	128	109	109	88	88	70	70	61	61	54	54
33	120	120	103	103	83	83	66	66	58	58	51	51
34	113	113	97	97	78	78	62	62				
35	107	107	91	91								
Properties												
U	1.33	1.48	1.35	1.49	1.37	1.51	1.39	1.54	1.40	1.55	1.41	1.56
P_{wo} (kips)	147	205	120	167	86	119	69	96	56	78	48	67
P_{wi} (kips/in.)	21	29	18	26	14	20	13	18	11	16	10	14
P_{wb} (kips)	648	764	464	547	224	264	163	192	104	123	81	95
P_{fb} (kips)	177	246	133	185	95	132	64	88	50	69	38	53
L_p (ft)	8.8	7.5	8.8	7.4	8.7	7.4	8.5	7.2	8.5	7.2	8.4	7.1
L_r (ft)	64.0	41.9	56.0	36.8	46.7	31.1	39.1	26.4	35.1	24.1	32.0	22.3
A (in.^{2})	19.7		17.1		14.1		11.7		10.3		9.13	
I_x (in.^{4})	272		228		184		146		127		110	
I_y (in.^{4})	88.6		75.1		60.9		49.1		42.6		37.1	
r_y (in.)	2.12		2.10		2.08		2.04		2.03		2.02	
Ratio r_x/r_y	1.75		1.74		1.74		1.73		1.73		1.72	

†Flange is noncompact; see discussion preceding column load tables.
Note: Heavy line indicates Kl/r of 200.

K=1

Effective Length = 13 ft 4 in

For Example:

if W 8*67

when L=13 ft , capacity=453 kips

when L=14 ft, capacity=433 kips

when L=13 ft 4 in, capacity=x kips?

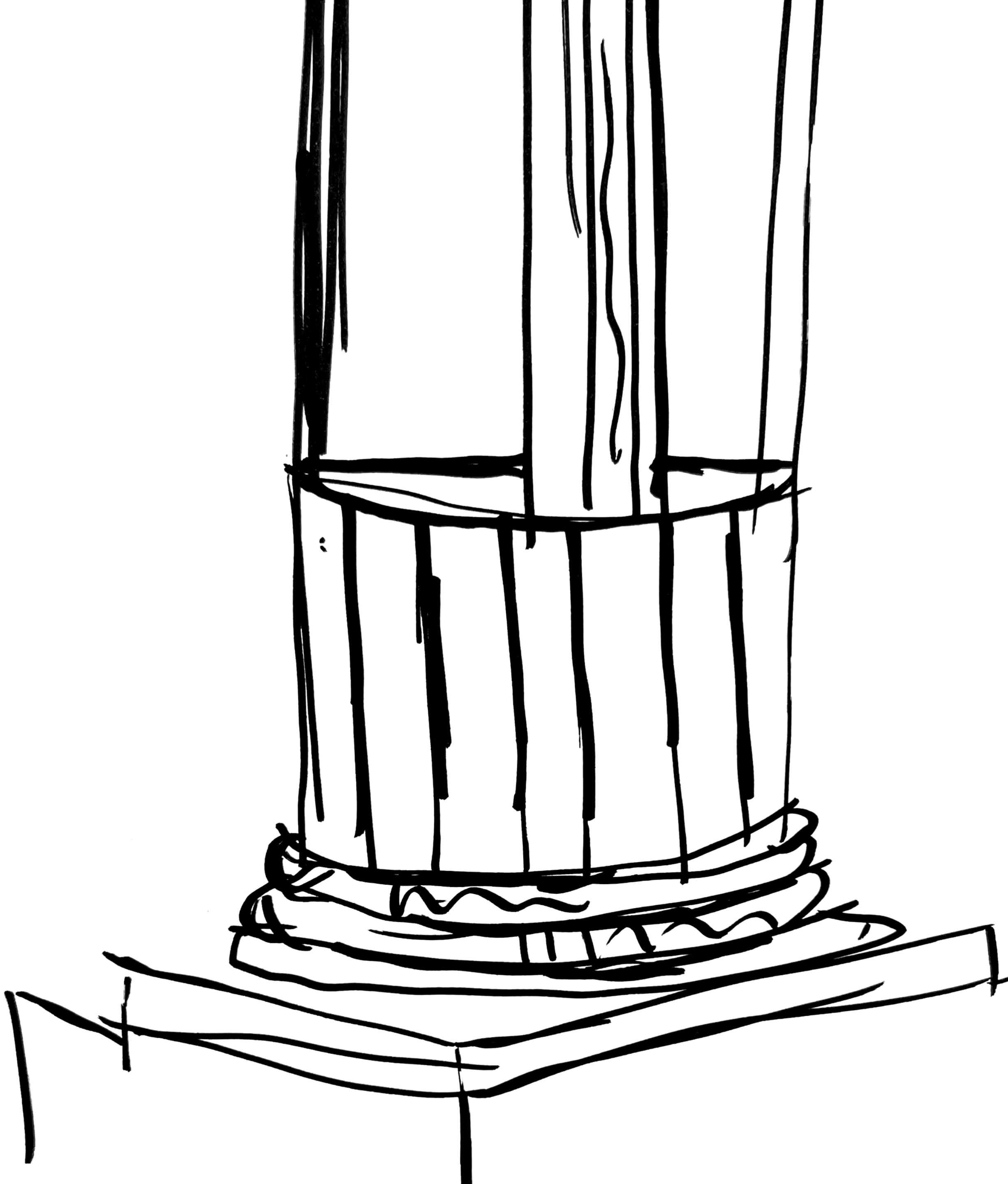
$$\frac{13\ ft\ 4\ in - 13\ ft}{14\ ft - 13\ ft} = \frac{453\ kips - x\ kips}{453\ kips - 433\ kips}$$

$$\frac{4}{12} = \frac{453 - x}{20}$$

x = 446.33 kips

Any Questions?

yifanma@umich.edu



Thank You!