Structure II Recitation 2/16

Steel Beam Design

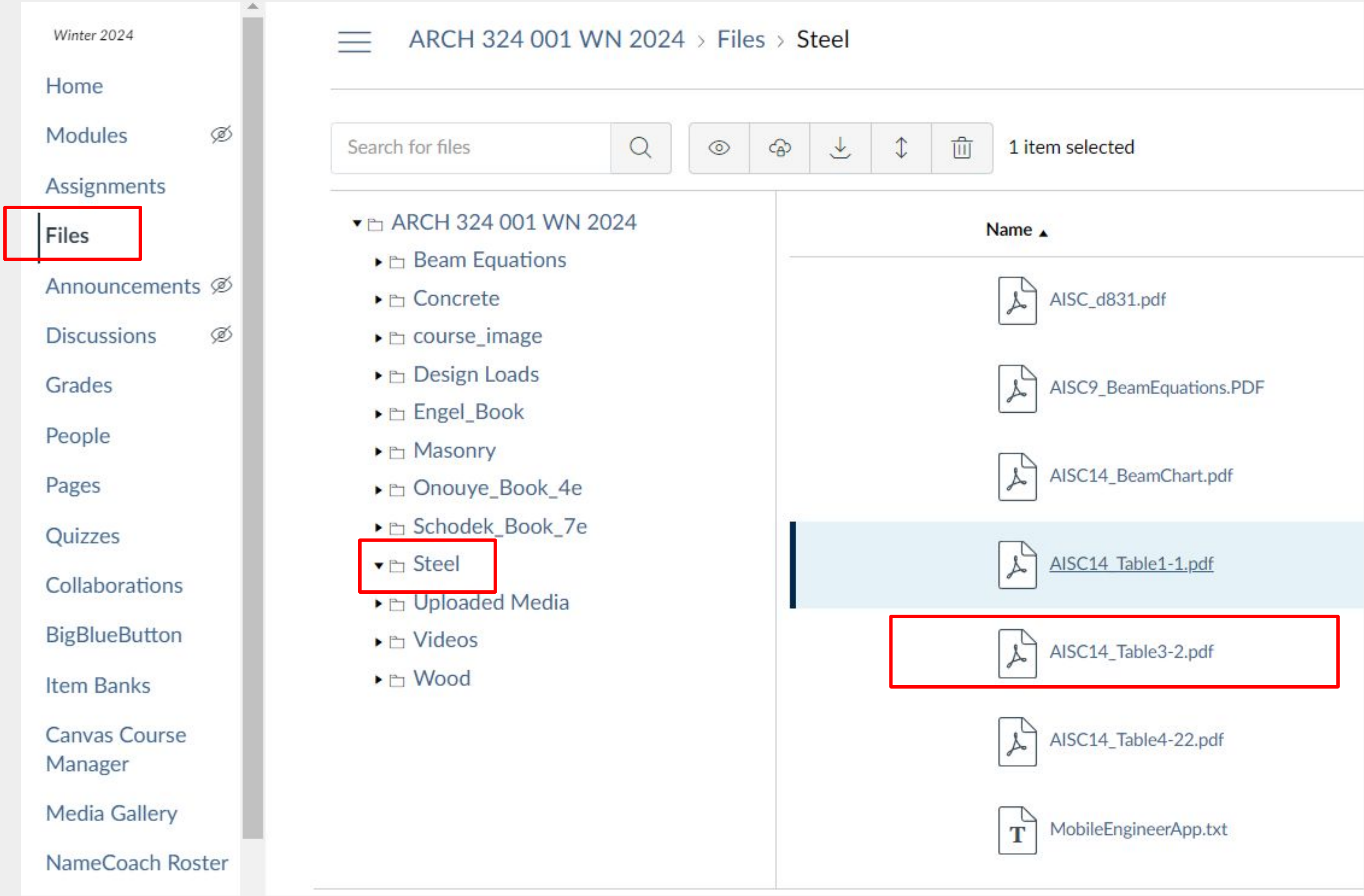
Before we start …

Today’s Tasks

1. Homework Example (Steel Beam Analysis) (14 Questions)
2. Tower Project Explanation (A little bit Dr. Frame)
3. Lab (Steel Column)

Reminder

1. **Submit the report no later than next Friday.**
2. Submit the report on canvas.
3. Scale beside Peter’s office if needed.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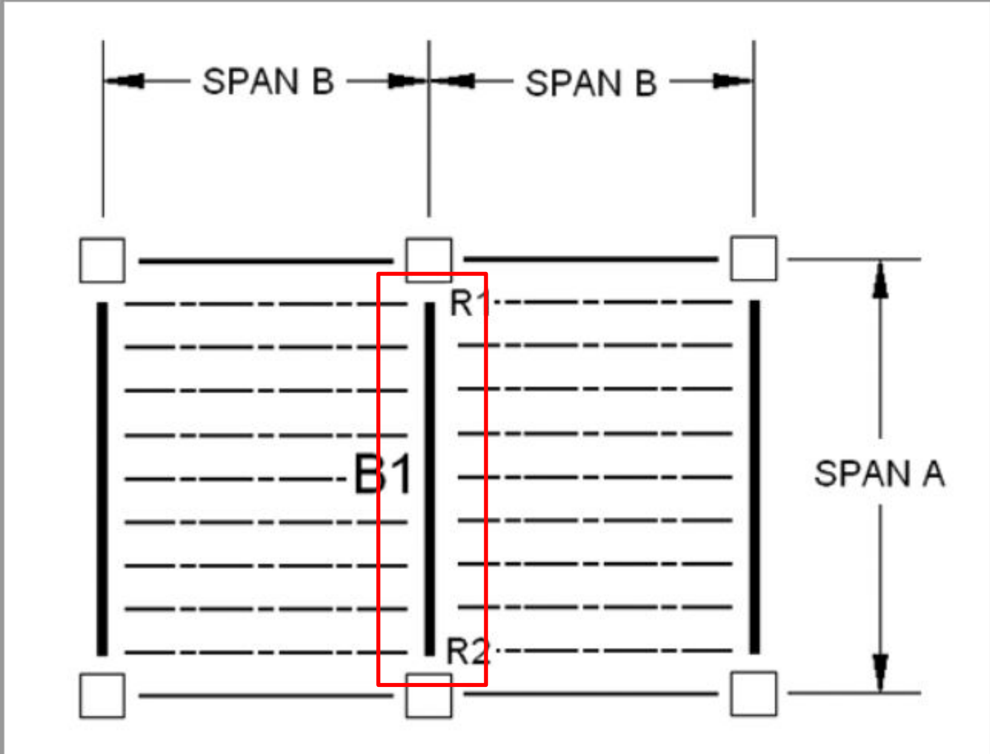
AISC9_BeamEquations.PDF
AISC14_BeamChart.pdf
AISC14_Table1-1.pdf
AISC14_Table3-2.pdf
AISC14_Table4-22.pdf
MobileEngineerApp.txt

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	27 FT
Span B	16 FT
Floor Dead Load	24 PSF
Floor Live Load	95 PSF



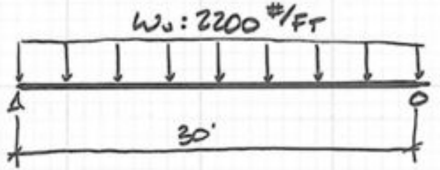
Steps:

Calculate wu, Mu, Mn (Neglect Self Weight) → Get estimated Zx → Find actual Zx → Recalculate wu, Mu, Mn (Consider Self Weight)

Design of Steel Beam – Procedure (zone 1)

1. Use the maximum moment equation, and solve for the ultimate moment, M_u.
2. Set $\phi M_n = M_u$ and solve for M_n
3. Assume Zone 1 to determine Z_x required
4. Select the lightest beam with a Z_x greater than the Z_x required from AISC table
5. Determine if h/tw < 59
   (case 1, most common)
6. Determine A_w:
   $Aw = d\ t_w$
7. Calculate V_n:
   $V_n = 0.6\ F_y\ A_w$
8. Calculate Vu for the given loading
   $V_u = w_u\ L / 2$ (e.g. unif. load)
9. Check $V_u < \phi\ V_n$
   ϕ for V = 1.0
10. Check deflection

GIVEN: $F_y = 50$ KSI
FULLY BRACED



$M_u = \frac{w_u l^2}{8} = \frac{2200 \text{ PLF} \cdot 30 \text{ FT}^2}{8}$

$M_u = 247{,}500$ #-FT $= 247.5$ KFT

$M_n = M_u / \phi_b = \frac{247.5 \text{ KFT}}{0.90} = 275$ KFT

Q1: The Unfactored Floor Dead Load on the Beam (Neglecting Self Weight) (w_DL*)

w_DL*

= Floor DL x Tributary Area / Span A

= Floor DL x Span B

= 24 x 16 = **<u>384 plf</u>**

Q2: The Unfactored Floor Live Load on the Beam (w_LL)

w_LL

= Floor LL x Tributary Area / Span A

= Floor LL x Span B

= 95 x 16 = **<u>1520 plf</u>**

Q3: The Total Factored Design Load on the Beam (Neglecting Self Weight) (LL*)

wu* = 1.2 x (w_DL*) + 1.6 x (w_LL)

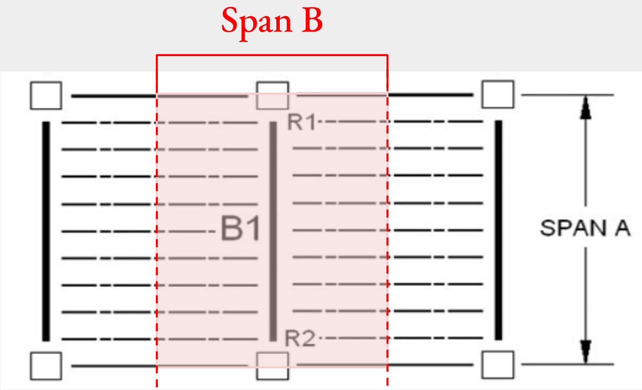
= (1.2 x 384 + 1.6 x 1520) / 1000

= **<u>2.8928 klf</u>**

Convert Unit (Pounds to Kips)

Given from Question

Fy	50 KSI
Span A	27 FT
Span B	16 FT
Floor Dead Load	24 PSF
Floor Live Load	95 PSF



$$w_u = 1.2w_{DL} + 1.6w_{LL}$$

Q4: The Factored Design Moment (Neglecting Self Weight) (Mu*)

$Mu^* = wu^* \times L^2 / 8 = 2.8928 \times 27^2 / 8 =$ **263.6064 k-ft**

from Q3

Q5: The Nominal Bending Moment (Neglecting Self Weight) (Mn*)

We assume $Mu^* = \Phi\ Mn$ in order to find Mn,

Since $\Phi = 0.9$,

$Mn^* = Mu^* / 0.9 = 263.6064 / 0.9 \times 12 =$ **3514.752 k-in**

from Q4 Convert Unit

Q6: The Plastic Modulus of the Section (Neglecting Self Weight) (Zx*)

Since the beam is fully braced (Lb < Lp): Zone 1,

We can use the formula: $Mn = Fy \times Zx^*$

$Zx^* = Mn / Fy = 3514.752 / 50 =$ **70.295 in^3**

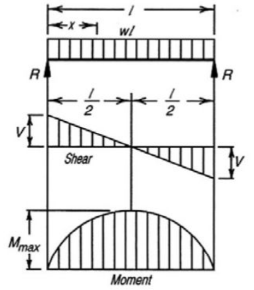
Given from Question

Fy	50 KSI
Span A	27 FT
Span B	16 FT
Floor Dead Load	24 PSF
Floor Live Load	95 PSF

$$\phi M_n = 0.90\ M_n$$

$$Mn = F_y\ Z_x$$

1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . . . $= wl$

$R = V$ . . . . $= \frac{wl}{2}$

V_x . . . . $= w\left(\frac{l}{2} - x\right)$

M_{max} (at center) . . . . $= \frac{wl^2}{8}$

M_x . . . . $= \frac{wx}{2}(l - x)$

Δ_{max} (at center) . . . . $= \frac{5wl^4}{384EI}$

Δ_x . . . . $= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

Q7: The Nominal Depth of the Lightest Passing W-Section from the Zx table (Include Self Weight)

Check AISC 14, Table 3-2, for me: p.5 (PDF), Find the smallest Zx that is bigger than the Zx* we calculated previously (Look at the ones with **bold stroke** and ignore the ones with a ~~strikethrough~~)

For my situation: W18x40

Nominal Depth = **18in**

Q8: The Weight of the Lightest Passing W-Section from Zx table (Include Self Weight)

Weight = **40 plf**

Q9: The Weight of the Lightest Passing W-Section from Zx table (Include Self Weight)

Zx = **78.4 in**^{3}

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
~~**W21×55**~~	**126**	**314**	473	**192**	289	**10.8**	16.3	**6.11**	**17.4**	**1140**	**156**	234
W14×74	126	314	473	196	294	5.31	8.05	8.76	31.0	795	128	192
W18×60	123	307	461	189	284	9.62	14.4	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.78	5.67	10.8	39.9	662	117	175
W14×68	115	287	431	180	270	5.19	7.81	8.69	29.3	722	116	174
W10×88	113	282	424	172	259	2.62	3.94	9.29	51.2	534	131	196
W18×55	**112**	**279**	420	**172**	258	**9.15**	13.8	**5.90**	**17.6**	**890**	**141**	212
W21×50	**110**	**274**	413	**165**	248	**12.1**	18.3	**4.59**	**13.6**	**984**	**158**	237
W12×72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159
W21×48[f]	**107**	**265**	398	**162**	244	**9.89**	14.8	**6.09**	**16.5**	**959**	**144**	216
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.93	7.48	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192
W10×77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169
W12×65[f]	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142
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

Q10: The Revised Unfactored Dead Load on the Beam (Including Self Weight) (w_DL)

w_DL = Dead Load + Self Weight = 384 + 40 = **424 plf**

From Q1 From Q8

Q11: The Total Factored Design Load on the Beam (Including Self Weight) (wu)

wu = 1.2 x (w_DL) + 1.6 x (w_LL)

= (1.2 x 424 + 1.6 x 1520) / 1000 = **2.9408 klf**

From Q10 From Q2

Q12: The Factored Design Moment (Including Self Weight) (Mu) (in k-ft)

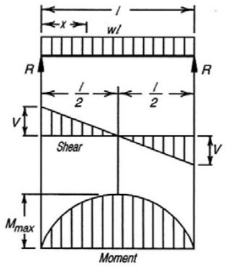
Mu = wu x L^2 / 8 = 2.9408 x 27^2 / 8 = **267.9804 k-ft**

Given from Question

Fy	50 KSI
Span A	27 FT
Span B	16 FT
Floor Dead Load	24 PSF
Floor Live Load	95 PSF

$$w_u = 1.2w_{DL} + 1.6w_{LL}$$

1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load	$= wl$
$R = V$	$= \frac{wl}{2}$
V_x	$= w\left(\frac{l}{2} - x\right)$
M_{max} (at center)	$\boxed{= \frac{wl^2}{8}}$
M_x	$= \frac{wx}{2}(l - x)$
Δ_{max} (at center)	$= \frac{5wl^4}{384EI}$
Δ_x	$= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

Q13: The Factored Design Moment (Including Self Weight) (Mu) (in k-in)

Mu (k-in) = Mu(k-ft) x 12 = 267.9804 x 12 = **<u>3215.7648 k-in</u>**

From Q12

Given from Question

Fy	50 KSI
Span A	27 FT
Span B	16 FT
Floor Dead Load	24 PSF
Floor Live Load	95 PSF

Q14: The Nominal Factored Bending Moment for the chosen section (Φ Mn)

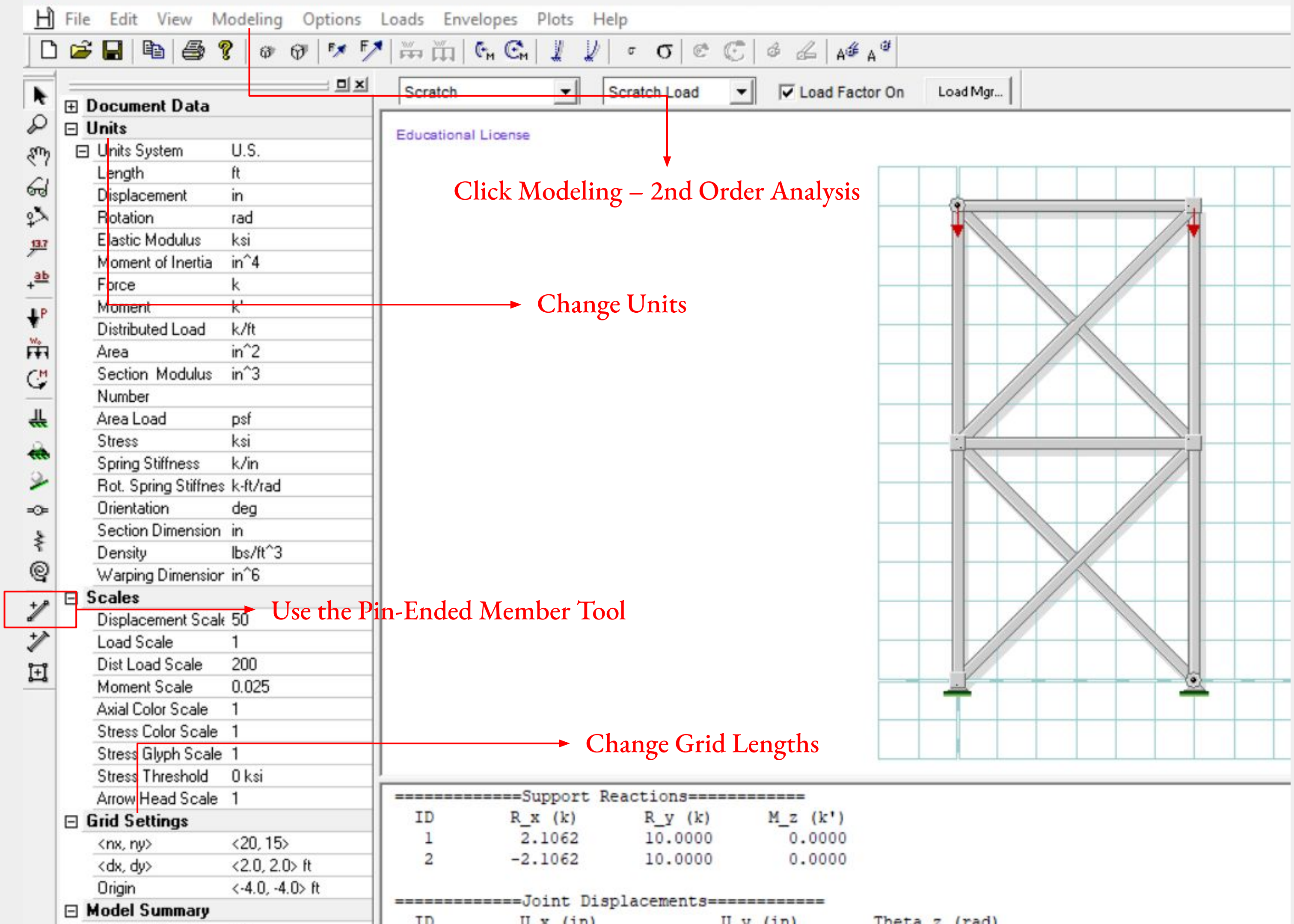
Φ Mn = 0.9 x Fy x Zx = 0.9 x 50 x 78.4 = **<u>3528 k-in</u>**

From Q9

$$\o M_n = 0.90\ M_n$$

$$Mn = F_y Z_x$$


IT'S OVER.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Length ft
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Rotation rad
Elastic Modulus ksi
Moment of Inertia in^4
Force k
Moment k'
Distributed Load k/ft
Area in^2
Section Modulus in^3
Number
Area Load psf
Stress ksi
Spring Stiffness k/in
Rot. Spring Stiffnes k-ft/rad
Orientation deg
Section Dimension in
Density lbs/ft^3
Warping Dimension in^6
Scales
Displacement Scale 50
Load Scale 1
Dist Load Scale 200
Moment Scale 0.025
Axial Color Scale 1
Stress Color Scale 1
Stress Glyph Scale 1
Stress Threshold 0 ksi
Arrow Head Scale 1
Grid Settings
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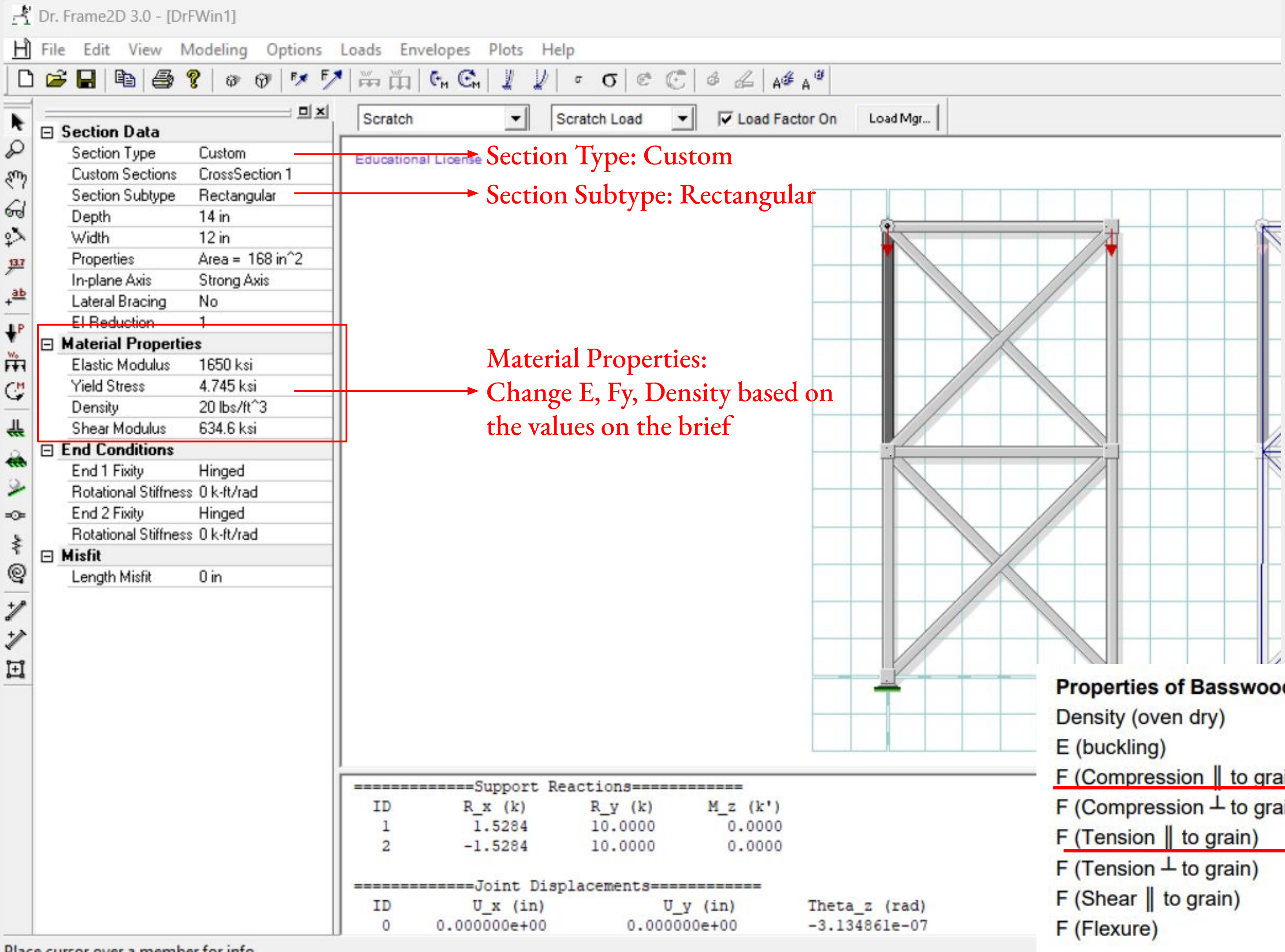
Model Summary
Educational License
Click Modeling – 2nd Order Analysis
Change Units
Use the Pin-Ended Member Tool
Change Grid Lengths
=============Support Reactions============
ID R_x (k) R_y (k) M_z (k')
1 2.1062 10.0000 0.0000
2 -2.1062 10.0000 0.0000
=============Joint Displacements============



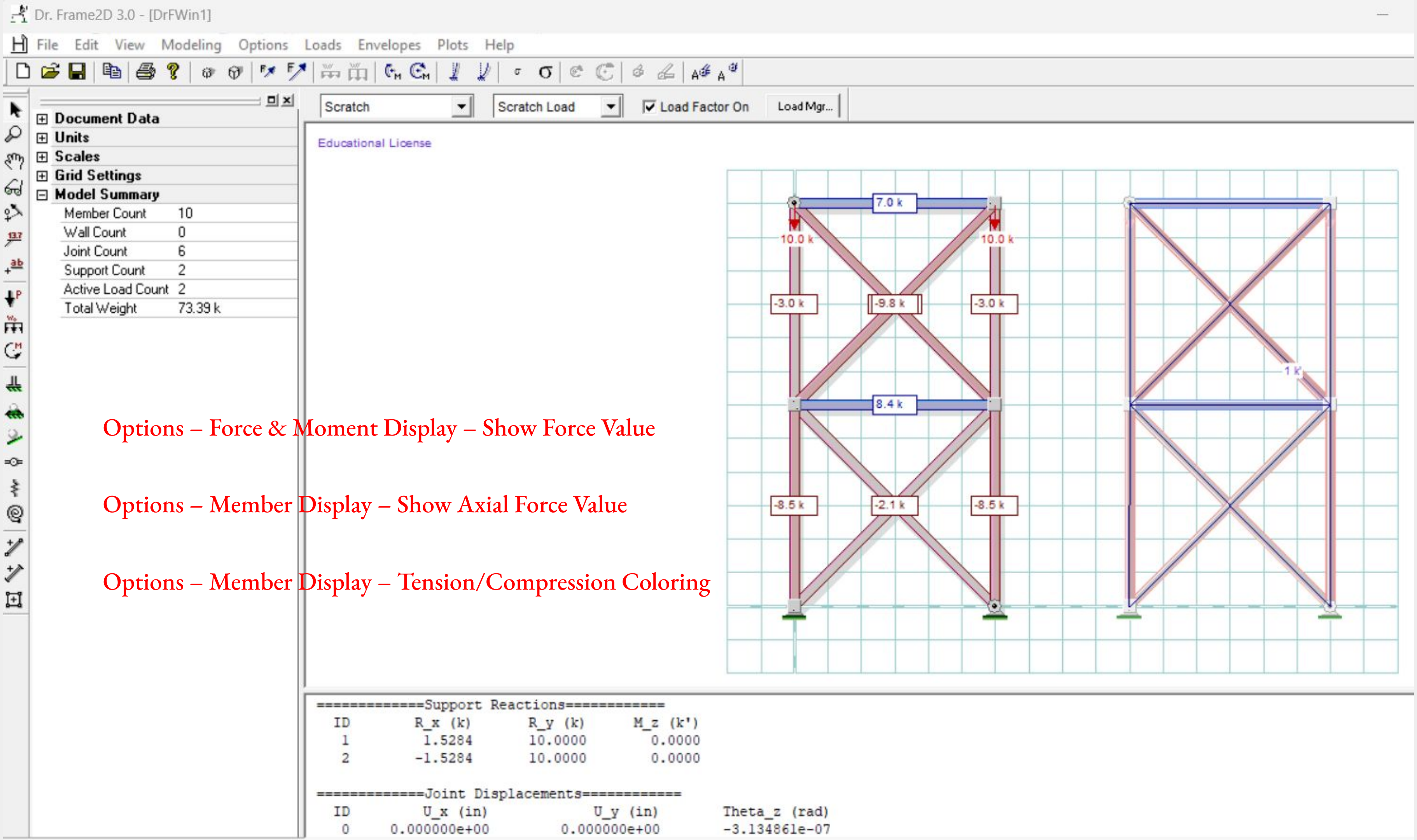
Section Type: Custom

Section Subtype: Rectangular

Material Properties:
Change E, Fy, Density based on the values on the brief

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 *



Options – Force & Moment Display – Show Force Value

Options – Member Display – Show Axial Force Value

Options – Member Display – Tension/Compression Coloring

Tower Preliminary Report

Explanation – describe how the design was developed, the basis of the structural concept, and how the principles of column behavior influenced the design decisions.

Illustration – include diagrams/drawings that describe the structure in its entirety. **At least a horizontal cross-section and an elevation of the tower are required**. Dimensions are to be included and the member sizes labeled.

Analysis – the report should include the following:

- **Choose wood type and stress properties**. Either use values below for typical model grade Basswood or use values in the NDS or find test values online. Indicate in the report which values you choose.
- **Determine the cross-sectional area of each member**. Find the axial force P and the allowable stress F'c. The force P can be determined either by a hand calculated truss analysis or as a second order analysis in Dr. Frame or STAAD.Pro. The stress F'c should be found using the NDS equations for C_P and F'c. Other NDS stress adjustment factors (C_D, C_M, C_t, C_F and C_i) can be taken equal to 1.0. Size members based on the predicted load, P and the allowable stress F'c. Target (or predict) some total capacity load for the tower. A minimum of 50 LBS is required. Then size the members based on the force in each member.
- **Predict the total weight of the tower**. Provide a table with each member type showing, length, section and weight for each. Make an estimate of the weight added by glue joints and/or gusset plates. The total weight should be under 4 OZ.
- **Predict Capacity**. Predict the ultimate capacity in pounds that the entire tower can carry based on the actual cross-sections chosen. Produce a utilization table to show for each member type (e.g. main vertical, horizontal tie, diagonal brace) the utilization ratio fc/F'c based on the predicted total capacity load. This ratio should be below 1.0 for all members.
- **Calculate the buckling capacity of the tower as a whole**. This is done by treating the tower as one column loaded at the top, made up in cross section of multiple columns. Show the moment of inertia of the tower cross-section, and use it to calculate the critical buckling load using the Euler equation. An example of this calculation is given in the slides from the class lecture. The ultimate capacity is the lower of the two capacities (critical member or tower as a whole).

Note: If an excel spreadsheet is used to make calculations, show the equations being used for each cell or column in the table. If STAAD.Pro or Dr. Frame is used to do any of the above, include print-outs showing the applied loads and resulting member forces.

Properties of Basswood:

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 *

Determine the cross sectional area of each member

Method 1: Use predicted forces to find the cross sectional areas

1. Use Dr. Frame to find the predicted Ps (Axial Force) for each member.

2. Calculate Allowable Stress Fc' using Fc' = Fc (Given) x Cp (Calculated).

3. Use A = P / Fc' to find the estimated A.

4. Pick the dimension for your material that has the cross sectional area bigger than the estimated A.

5. Recalculate the capacities for each members (Buckling + Crushing) (for deciding the critical member, look at the vertical member)

Allowable Stress:

$F_c' = F_c$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	C_P	-

$$C_P = \frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \quad (3.7\text{-}1)$$

Fc* = Fc (Given)

Since all the factors except Cp = 1

where:

F_c^* = reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_P (see 2.3), psi

$$F_{cE} = \frac{0.822\, E_{min}'}{(\ell_e/d)^2}$$

Le = K x L

Since K = 1 (Pin-Pin), so Le = L

c = 0.8 for sawn lumber

c = 0.85 for round timber poles and piles

c = 0.9 for structural glued laminated timber or structural composite lumber

Buckling Load:

$$P_{cr} = \frac{\pi^2 AE}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 IE}{(KL)^2}$$

Crushing Load:

$$P_{max} = F_c \times A$$

Analysis	Capacity	Design
$f_c = \frac{P}{A} \le F'_c$	$P = F'_c A$	$A = \frac{P}{F'_c}$

Method 2: Decide cross-sectional area first to calculate the maximum Ps, then compare the values with the predicted forces to make sure the decision

1. First design the cross sectional areas of your material.

2. Calculate maximum P using P = Fc’ x A for each member.

2. Use Dr. Frame to find the predicted Ps (Axial Force) for each member.

4. Compare the predicted forces with calculated maximum Ps, make sure the predicted forces are not bigger than the calculated Ps.

5. If pass, calculate the buckling & crushing capacities to decide which one is the critical load.

Analysis

$$f_c = \frac{P}{A} \leq F'_c$$

Capacity

$$P = F'_c\, A$$

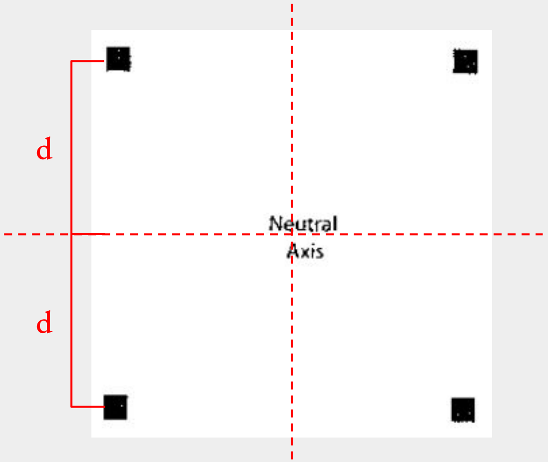
Design

$$A = \frac{P}{F'_c}$$

Calculate the buckling capacity of the tower as a whole

1. After deciding the dimensions, look at the cross-section of your tower, calculate the moment of inertia.

2. Use the Euler Buckling Equation to calculate the buckling capacity of the whole tower.



Total Cross Sectional Area

$$I_x = \frac{bh^3}{12}$$

Cross Sectional Area

$$I = \sum I + \sum A d^2$$

Euler Buckling (elastic buckling)

$$P_{cr} = \frac{\pi^2 AE}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 IE}{(KL)^2}$$

Tower Height

$$r = \sqrt{\frac{I}{A}}$$

$$I = Ar^2$$

Predict the total weight of the tower

Limit = 4 oz !!!!!!!!!!

Member Weight = Volume x Density = <u>Cross Sectional Area x Length x Density</u>

Design Design Given

Reminders:

1. Include the estimated glue weight in your predicted weight
2. Make a table that includes member weight, area, and length (label all the members)

	SIZE (in)	TOTAL LENGTH (in)	BASSWOOD PROPERTIES DENSITY (lb/ft^3)	CROSS SECTIONAL AREA (in^2)	WEIGHT(oz)	VOLUME (in^3)
VERTICAL PILLARS	1/4 X 1/4	192	20	0.063	2.222	12
DIAGONAL BRACES	3/32 X 1/8	500	20	0.012	1.085	5.858
HORIZONTAL BRACE	1/8 X1/8	57	20	0.016	0.165	0.891
GLUE					0.3472	
TOTAL					3.891	18.748


WHEN YOUR TO-DO LIST IS DONE!!

Lab Session: Steel Columns

Goal: Find the load capacity

Steps:

1. Measure the dimensions of the column **(d, bf, tf)**

2. Use the dimensions to find the size (First Chart)

3. Use the size and length to find the capacity (Second Chart) (Assume K = 1)

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

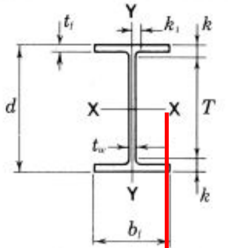
1. Measure the steel column section shown below. (your GSI will tell you which one)
2. Based on the sectional dimensions find the shape in the steel table.
3. Use the column table and the given height to find the load capacity. Both columns are A-36 steel (Fy = 36 ksi).

L= 15 ft. – 4 in.

or

L = 13 ft. 4 in.

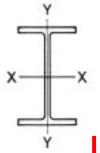
Section: W ___ x _____ Design Strength _________ kips



W SHAPES
Dimensions

Assume picking W8X67

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	8¼	0.935	15/16	6⅛	1 7/16	11/16
x 58	17.1	8.75	8¾	0.510	½	¼	8.220	8¼	0.810	13/16	6⅛	1 5/16	11/16
x 48	14.1	8.50	8½	0.400	⅜	3/16	8.110	8⅛	0.685	11/16	6⅛	1 3/16	⅝
x 40	11.7	8.25	8¼	0.360	⅜	3/16	8.070	8⅛	0.560	9/16	6⅛	1 1/16	⅝
x 35	10.3	8.12	8⅛	0.310	5/16	3/16	8.020	8	0.495	½	6⅛	1	9/16
x 31	9.13	8.00	8	0.285	5/16	3/16	7.995	8	0.435	7/16	6⅛	15/16	9/16
W 8x 28	8.25	8.06	8	0.285	5/16	3/16	6.535	6½	0.465	7/16	6⅛	15/16	9/16
x 24	7.08	7.93	7⅞	0.245	¼	⅛	6.495	6½	0.400	⅜	6⅛	⅞	9/16
W 8x 21	6.16	8.28	†8¼	0.250	¼	⅛	5.270	5¼	0.400	⅜	6⅝	13/16	½
x 18	5.26	8.14	8⅛	0.230	¼	⅛	5.250	5¼	0.330	5/16	6⅝	¾	7/16
W 8 x15	4.44	8.11	8⅛	0.245	¼	⅛	4.015	4	0.315	5/16	6⅝	¾	½
x 13	3.84	7.99	8	0.230	¼	⅛	4.000	4	0.255	¼	6⅝	11/16	7/16
x 10	2.96	7.89	7⅞	0.170	3/16	⅛	3.940	4	0.205	3/16	6⅝	⅝	7/16
W 6x 25	7.34	6.38	6⅜	0.320	5/16	3/16	6.080	6⅛	0.455	7/16	4¾	13/16	7/16
x 20	5.87	6.20	6¼	0.260	¼	⅛	6.020	6	0.365	⅜	4¾	¾	7/16
x 15	4.43	5.99	6	0.230	¼	⅛	5.990	6	0.260	¼	4¾	⅝	⅜
W 6x 16	4.74	6.28	6¼	0.260	¼	⅛	4.030	4	0.405	⅜	4¾	¾	7/16
x 12	3.55	6.03	6	0.230	¼	⅛	4.000	4	0.280	¼	4¾	⅝	⅜
x 9	2.68	5.90	5⅞	0.170	3/16	⅛	3.940	4	0.215	3/16	4¾	9/16	⅜
W 5x 19	5.54	5.15	5⅛	0.270	¼	⅛	5.030	5	0.430	7/16	3½	13/16	7/16
x 16	4.68	5.01	5	0.240	¼	⅛	5.000	5	0.360	⅜	3½	¾	7/16
W 4x 13	3.83	4.16	4⅛	0.280	¼	⅛	4.060	4	0.345	⅜	2¾	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	473	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	$\boxed{412}$	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.