Steel Beam Analysis 2/9

HW - Steel Beam Analysis

**Tower Project** 

Lab – Steel Beams

Structure II Section 004

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# Happy Lunar Dragon Year!



Analyze the given W-section for beam B1, to determine the maximum live load capacity the floor can carry. Determine the shear and bending forces and check the maximum deflection against an allowable of L/180. Assume the beam is fully braced, Lb < Lp (zone 1).

DATASET: 1 -23-	
W-section	W10X30
Fy	50 KSI
Span A	18 FT
Span B	12 FT
Floor DL	14 PSF

# HW - Steel Beam Analysis

**Given:** beam size (yield stress) bracing type load

### Goal:

load capacity?

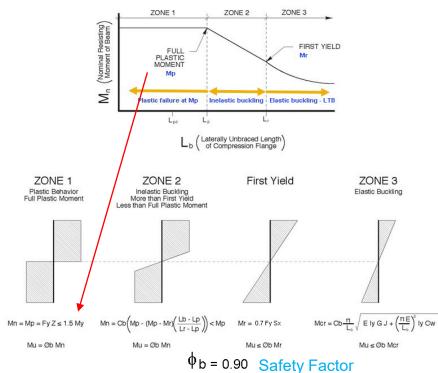
1. Determine the unbraced length of the compression flange (Lb).

2. Find the Lp and Lr values from the AISC properties table 3-6

3. Compare Lb to Lp and Lr and determine which equation for Mn or Mcr to be used.

4. Determine the beam load equation for maximum moment in the beam. Solve for Mn.

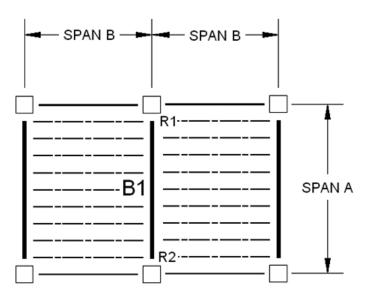
5. Calculate load based on maximum moment. Mu =  $\phi_{b}$  Mn



Given: Lb < Lp Plastic Behavior (zone 1)

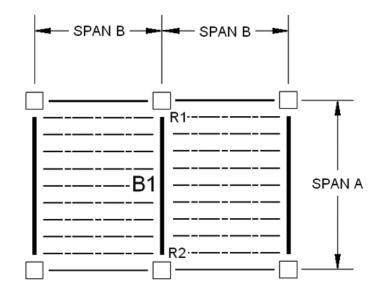
Maximum Moment: Mn = Mp = Fy Zx

Find the Plastic Modulus (Zx) for the given section from the AISC table 1-1



Analyze the given W-section for beam B1, to determine the maximum live load capacity the floor can carry. Determine the shear and bending forces and check the maximum deflection against an allowable of L/180. Assume the beam is fully braced, Lb < Lp (zone 1).

DATASET: 1	-23-	
W-section		W10X30
Fy		50 KSI
Span A		18 FT
Span B		12 FT
Floor DL		14 PSF



#### **1. The plastic modulus of the section, Zx** = 36.6 in<sup>3</sup>

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				tw	2		b <sub>f</sub>		t <sub>f</sub>	<i>k</i> <sub>des</sub>	k <sub>det</sub>	-		Gage			b <sub>1</sub> h	1	S	r	Z	1	S	r	Z			J	Cw
W12×58	in. <sup>2</sup> 17.0	ir 12.2	1	in. 0.360 <sup>3</sup> /8	in. <sup>3/16</sup>	10.0	<b>n.</b>	0.640	n.	in. 1.24	in.	in.	in. 9 <sup>1</sup> /4	in. 5 <sup>1</sup> /2		Ib/ft 58	2t <sub>f</sub> t <sub>w</sub> 7.82 27.0	in. <sup>4</sup> 475	in. <sup>3</sup> 78.0	in. 5.28	in. <sup>3</sup> 86.4	in. <sup>4</sup> 107	in. <sup>3</sup> 21.4	in. 2.51	in. <sup>3</sup> 32.5	in. 2.81	in. 11.6	in. <sup>4</sup> 2.10	in. <sup>6</sup>
×53	15.6	12.1	12	0.345 3/8	3/16	10.0	10	0.575			13/8	15/16		51/2		53	8.69 28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
W12×50	14.6	12.2	12 <sup>1</sup> /4	0.370 3/8	3/16	8.08	81/8	0.640	5/8	1.14	11/2	15/16	91/4	51/2		50	6.31 26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
×45		12.1	12	0.335 5/16	3/16	8.05		0.575		1.08	13/8	15/16	↓	↓		45	7.00 29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
×40		11.9	12	0.295 5/16	3/16	8.01		0.515		1.02	13/8	7/8	'			40	7.77 33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440
W12×35 <sup>c</sup> ×30 <sup>c</sup>	10.3	12.5		0.300 <sup>5</sup> / <sub>16</sub> 0.260 <sup>1</sup> / <sub>4</sub>	<sup>3</sup> /16 1/8		6 <sup>1</sup> /2 6 <sup>1</sup> /2	0.520	1/2 7/16	0.820		3/4 3/4	101/8	31/2		35 30	6.31 36.2 7.41 41.8	285 238	45.6 38.6	5.25 5.21	51.2 43.1	24.5 20.3		1.54 1.52	11.5 9.56	1.79	12.0 11.9	0.741 0.457	879 720
×26°		12.2		0.230 1/4	1/8		61/2	0.380		0.680		3/4	1	♥		26	8.54 47.2	204	33.4	5.17	37.2	17.3		1.51		1.75	11.8	0.300	607
W12×22 <sup>c</sup>	6.48	12.3	121/4	0.260 1/4	1/8	4.03	4	0.425	7/16	0.725	15/16		103/8	2 <sup>1</sup> /4 <sup>9</sup>		22	4.74 41.8	156	25.4	4.91	29.3	4.66	2.31	0.848	3.66	1.04	11.9	0.293	164
×19°		12.2		0.235 1/4	1/8	4.01		0.350	3/8	0.650		9/16				19	5.72 46.2	130	21.3	4.82	24.7	3.76		0.822		1.02	11.9	0.180	131
×16 <sup>c</sup> ×14 <sup>c,v</sup>		12.0 11.9	12 11 <sup>7</sup> /8	0.220 <sup>1</sup> / <sub>4</sub> 0.200 <sup>3</sup> / <sub>16</sub>	1/8 1/8	3.99		0.265		0.565	<sup>13</sup> /16 3/4	<sup>9</sup> /16 <sup>9</sup> /16	•	↓		16 14	7.53 49.4 8.82 54.3	103 88.6	17.1 14.9	4.67 4.62	20.1 17.4	2.82 2.36	1.41	0.773 0.753	2.26 1.90		11.7 11.7	0.103 0.0704	96.9 80.4
W10×112	32.9			0.755 3/4	3/8	10.4	103/8	1.25	11/4	1.75	115/16	1	71/2	51/2		112	4.17 10.4	716	126	4.66	147	236	45.3	2.68	69.2	3.08	10.2	15.1	6020
×100	29.3	11.1	111/8	0.680 11/16	3/8	10.3	103/8	1.12	11/8	1.62	<b>1</b> <sup>13</sup> /16	1				100	4.62 11.6	623	112	4.60	130	207	40.0	2.65	61.0	3.04	10.0	10.9	5150
×88	26.0 22.7	10.8		0.605 5/8	5/16	10.3	10 <sup>1</sup> /4	0.990		1.49	111/16					88	5.18 13.0	534	98.5	4.54	113	179	34.8	2.63	53.1	2.99	9.81	7.53	4330
×77 ×68	19.9	10.6 10.4		0.530 <sup>1</sup> / <sub>2</sub> 0.470 <sup>1</sup> / <sub>2</sub>	1/4 1/4	10.2	101/8	0.870		1.37	1 <sup>9</sup> /16 1 <sup>7</sup> /16	7/8 7/8				77 68	5.86 14.8 6.58 16.7	455 394	85.9 75.7	4.49 4.44	97.6 85.3	154 134	30.1 26.4	2.60 2.59	45.9 40.1	2.95 2.92	9.73 9.63	5.11 3.56	3630 3100
×60	17.7	10.2		0.420 7/16	1/4	10.1	101/8	0.680	11/16	1.18	13/8	13/16				60	7.41 18.7	341	66.7	4.39	74.6	116	23.0	2.57	35.0	2.88	9.52	2.48	2640
×54 ×49	15.8 14.4	10.1		0.370 <sup>3</sup> / <sub>8</sub> 0.340 <sup>5</sup> / <sub>16</sub>	<sup>3</sup> /16 <sup>3</sup> /16	10.0	10	0.615		1.12	1 <sup>5</sup> /16 1 <sup>1</sup> /4	<sup>13</sup> /16				54 49	8.15 21.2 8.93 23.1	303 272	60.0 54.6	4.37 4.35	66.6 60.4	103 93.4	20.6	2.56 2.54	31.3 28.3	2.85 2.84	9.49 9.44	1.82 1.39	2320 2070
W10×45	13.3	10.0		0.350 3/8	3/16	8.02		0.620	5/8	1.12	15/16	13/16		51/2		49	6.47 22.5	248	49.1		54.9	53.4	13.3		20.3		9.44		1200
×39	11.5	9.92		0.315 5/16	3/16	7.99		0.620		1.03	1 <sup>3</sup> /16	13/16		5.12		45 39	6.47 22.5 7.53 25.0	248	49.1	4.32 4.27	54.9 46.8	53.4 45.0	13.3	2.01 1.98	20.3	2.27	9.48	1.51 0.976	992
×33	9.71	9.73	9 <sup>3</sup> /4	0.290 5/16	3/16	7.96	8	0.435	7/16	0.935	11/8	3/4	♥	V V		33	9.15 27.1	171	35.0	4.19	38.8	36.6		1.94	14.0	2.20	9.30	0.583	791
W10×30		10.5		0.300 5/16	3/16		5 <sup>3</sup> /4	0.510		0.810		11/16		2 <sup>3</sup> /4 <sup>9</sup>		30	5.70 29.5	170	32.4	4.38	36.6	16.7		1.37		1.60	10.0	0.622	414
×26 ×22℃		10.3		0.260 1/4 0.240 1/4	1/8 1/8		5 <sup>3</sup> /4 5 <sup>3</sup> /4	0.440			1 1/16 15/16	<sup>11/</sup> 16 5/8	•	V		26 22	6.56 34.0 7.99 36.9	144 118	27.9 23.2	4.35 4.27	31.3 26.0	14.1 11.4		1.36 1.33		1.58 1.55	9.86 9.84	0.402 0.239	345 275
	0.10		1.0.70		1	0.70	1		1 10	0.000		1 1	I '	I '			1.00 00.0		20.2	7.61	20.0	1.1.4	0.01		0.10	1.00	0.04	0.200	215

Analyze the given W-section for beam B1, to determine the maximum live load capacity the floor can carry. Determine the shear and bending forces and check the maximum deflection against an allowable of L/180. Assume the beam is fully braced, Lb < Lp (zone 1).

DATASET: 1 -23-	
W section	W10X30
Fy	50 KSI
Span A	18 FT
Span B	12 FT
Floor DL	14 PSF

### 2. The nominal bending moment, Mn

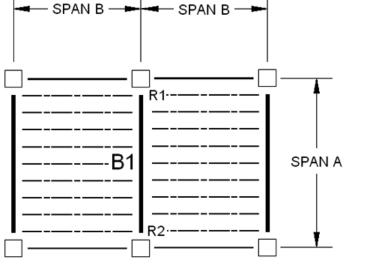
Mn = Fy\*Zx = 50\*36.6 = 1830 k-in

3. The factored bending resistance, phi Mn

φ \*Mn = 0.9\*1830 = 1647 k-in

4. The factored design moment, Mu

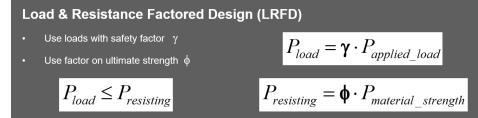
Mu = φ \*Mn = 1647/12 = 137.25 k-ft

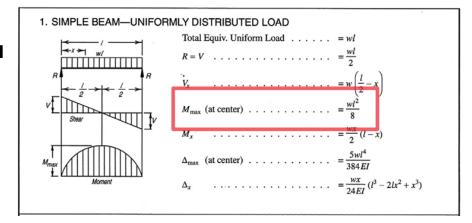


5. The total factored design load, wu

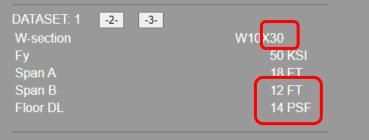
Mu = wu\*SpanA<sup>2</sup> /8 = 137.25 k-ft

wu= 8 \* Mu / SpanA<sup>2</sup> = 8\*137.25/18<sup>2</sup> = 3.39 klf





Analyze the given W-section for beam B1, to determine the maximum live load capacity the floor can carry. Determine the shear and bending forces and check the maximum deflection against an allowable of L/180. Assume the beam is fully braced, Lb < Lp (zone 1).



#### 6. The total unfactored dead load on the beam, w\_DL

w\_Floor DL = Tributary area \* Floor DL = SpanB \* Floor DL = 12\*14 = 168 plf

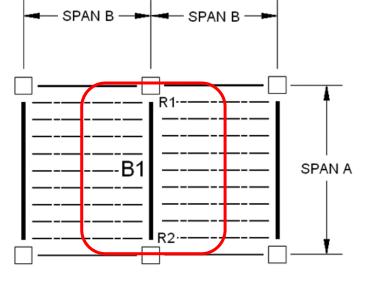
w\_Beam DL = 30 plf

w\_DL= w\_Floor DL + w\_Beam DL = (168 +30)/1000 = 0.198 klf

### 7. The total factored dead load on the beam, wu\_DL

wu\_DL = 1.2 \* w\_DL = 1.2\*0.198 = 0.2376 klf

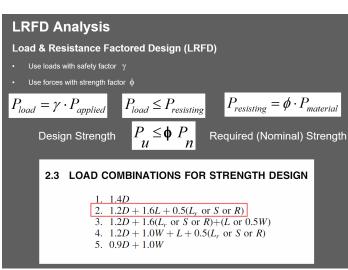
8. The factored beam live load, wu\_LL wu\_DL + wu\_LL = wu wu\_LL= wu-wu\_DL = 3.39-0.2376 = 3.1524 klf



9. The actual beam live load(capacity),w\_LL

wu\_LL = w\_LL \* 1.6 w\_LL = wu\_LL /1.6 = 3.1524/1.6 = 1.97 klf

**10. The actual floor live load(floor capacity), LL** LL= w\_LL/ SpanB = (1.97/12)\*1000 = 164.17 psf

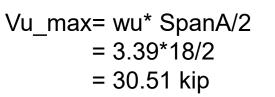


Analyze the given W-section for beam B1, to determine the maximum live load capacity the floor can carry Determine the shear and bending forces and check the maximum deflection against an allowable of L/180. Assume the beam is fully braced, Lb < Lp (zone 1).

W10X30
50 KSI
18 FT
12 FT
14 PSF

#### Check the shear force Vu<=øVn

### **11.** The maximum factored design beam shear force, Vu\_max



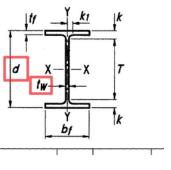
#### 12. The Web Area, Aw

 $Aw = d * tw = 10.5*0.3 = 3.15 in^{2}$ 

### **Design for Shear**

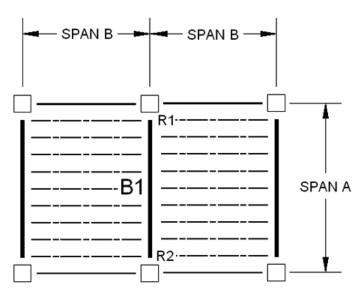
Shear stress in steel sections is approximated by averaging the stress in the web:  $F_v = V / A_w$ 

 $A_w = d * t_w$ 



1. SIMPLE BEAM-UNIFORM	ILY DISTRIBUTED LOAD	
<b>★</b> / → ★	Total Equiv. Uniform Load	= wl
	R = V	$=\frac{wl}{2}$
$R \xrightarrow{l} \frac{1}{2} \xrightarrow{l} \frac{1}{2} \xrightarrow{R}$	$V_x$	(- )
v Shear	$M_{\rm max}$ (at center)	
	<i>M<sub>x</sub></i>	2
	$\Delta_{max}$ (at center)	$=\frac{5wl^4}{384EI}$
		$=\frac{wx}{24EI}\left(l^3-2lx^2+x^3\right)$

					Web			Fla	nge			1	Distand	e	
Shape	Area, A	Dep	oth, d	Thick		$\frac{t_w}{2}$	0000	ith, Dr	Thick	,		k	<b>k</b> 1	т	Work able
									t <sub>1</sub> in.		<i>k</i> <sub>des</sub>	<b>k</b> <sub>det</sub>			Gage
	in. <sup>2</sup>	ir		in		in.		1.			in.	in.	in.	in.	in.
W12×58	17.0	12.2	121/4		3/8	3/16	10.0	10	0.640		1.24	11/2	<sup>15</sup> /16	91/4	5 <sup>1</sup> /2
$\times 53$	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 <sup>3</sup> /8	<sup>15</sup> /16	91/4	5 <sup>1</sup> /2
W12×50	14.6	12.2	121/4	0.370	3/8	3/16	8.08	8 <sup>1</sup> /8	0.640	5/8	1.14	11/2	15/16	9 <sup>1</sup> /4	5 <sup>1</sup> /2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 <sup>3</sup> /8	<sup>15</sup> /16		
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 <sup>3</sup> /8	7/8	۲	۲
W12×35 <sup>c</sup>	10.3	12.5	121/2	0.300	5/16	3/16	6.56	6 <sup>1</sup> /2	0.520	1/2	0.820	1 <sup>3</sup> /16	3/4	10 <sup>1</sup> /8	31/2
×30 <sup>c</sup>	8.79	12.3	12 <sup>3</sup> /8	0.260	1/4	1/8	6.52	6 <sup>1</sup> /2	0.440	7/16	0.740	11/8	3/4		1
×26 <sup>c</sup>	7.65	12.2	121/4	0.230	1/4	1/8	6.49	61/2	0.380	3/8	0.680	<b>1</b> <sup>1</sup> /16	3/4	۲	۷
W12×22 <sup>c</sup>	6.48	12.3	121/4	0.260	1/4	1/8	4.03	4	0.425	7/16	0.725	15/16	5/8	10 <sup>3</sup> /8	2 <sup>1</sup> /4 <sup>9</sup>
×19 <sup>c</sup>	5.57	12.2	121/8	0.235	1/4	1/8	4.01	4	0.350	3/8	0.650	7/8	9/16	T.	1
×16 <sup>c</sup>	4.71	12.0	12	0.220	1/4	1/8	3.99	4	0.265	1/4	0.565	13/16	<sup>9</sup> /16		
×14 <sup>c,v</sup>	4.16	11.9	117/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	3/4	<sup>9</sup> /16	۲	۷
W10×112	32.9	11.4	113/8	0.755	3/4	3/8	10.4	10 <sup>3</sup> /8	1.25	11/4	1.75	<b>1</b> <sup>15</sup> /16	1	7 <sup>1</sup> /2	5½
×100	29.3	11.1	111/8	0.680	11/16	3/8	10.3	10 <sup>3</sup> /8	1.12	11/8	1.62	1 <sup>13</sup> /16	1		
$\times 88$	26.0	10.8	107/8	0.605	5/8	5/16	10.3	101/4	0.990	1	1.49	<b>1</b> <sup>11</sup> / <sub>16</sub>	15/16		
×77	22.7	10.6	105/8	0.530	1/2	1/4	10.2	101/4	0.870	7/8	1.37	1 <sup>9</sup> /16	7/8		
×68	19.9	10.4	103/8	0.470	1/2	1/4	10.1	10 <sup>1</sup> /8	0.770	3/4	1.27	17/16	7/8		
$\times 60$	17.7	10.2		0.420	7/16	1/4	10.1	101/8	0.680	11/16	1.18	1 <sup>3</sup> /8	<sup>13</sup> /16		
×54	15.8	10.1		0.370	3/8	<sup>3</sup> /16	10.0	10	0.615	5/8	1.12	<b>1</b> <sup>5</sup> /16	13/16		
×49	14.4	10.0	10	0.340	5/16	<sup>3</sup> /16	10.0	10	0.560	9/16	1.06	11/4	<sup>13</sup> /16	V	Y
W10×45	13.3	10.1		0.350	3/8	<sup>3</sup> /16	8.02	8	0.620	5/ <sub>8</sub>	1.12	<b>1</b> 5/16	<sup>13</sup> /16	71/2	5 <sup>1</sup> /2
×39	11.5	9.92		0.315	5/16	3/16	7.99	8	0.530	1/2	1.03	<b>1</b> <sup>3</sup> /16	<sup>13</sup> /16		
×33	9.71	9.73	9 <sup>3</sup> /4	0.290	5/16	3/16	7.96	8	0.435	7/16	0.935	11/8	3/4	۲	۷
W10×30				0.300	<sup>5</sup> /16	<sup>3</sup> /16	5.81		0.510	1/2	0.810		11/16	81/4	2 <sup>3</sup> /4 <sup>9</sup>
×26				0.260	1/4	1/8	5.77	5 <sup>3</sup> /4	0.440	7/16	0.740		11/16	$\downarrow$	L ⊥
×22 <sup>c</sup>	6.49	10.2	101/8	0.240	1/4	1/8	5.75	5 <sup>3</sup> /4	0.360	3/8	0.660	15/16	5/8	V	



Analyze the given W-section for beam B1, to determine the maximum live load capacity the floor can carry Determine the shear and bending forces and check the maximum deflection against an allowable of L/180. Assume the beam is fully braced, Lb < Lp (zone 1).

DATASET: 1 -23-	
W-section	W10X30
Fy	50 KSI
Span A	18 FT
Span B	12 FT
Floor DL	14 PSF

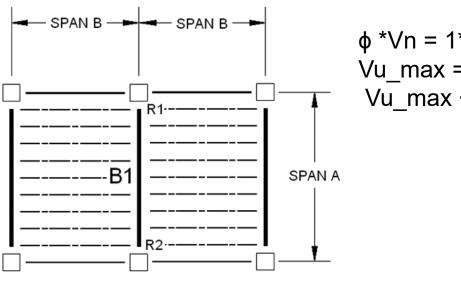
#### Check the shear force Vu<=øVn

### 13. The factored shear resistance, phi Vn

h / tw = 29.5 < 59

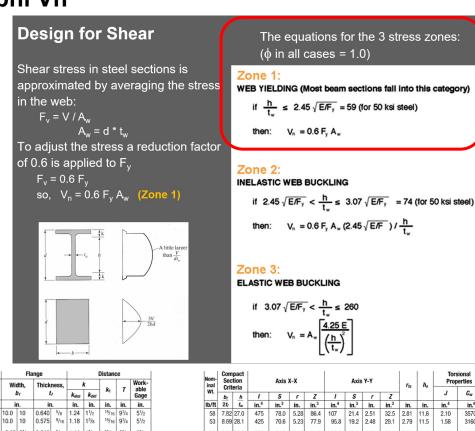
So Zone 1 Vn = 0.6 \* Fy\* Aw= 0.6\*50\*3.15 = 94.5 kip

#### 14. Is the section safe for shear?(1 = yes, 0 = no)



φ \*Vn = 1\*94.5 = 94.5 kip Vu\_max = 30.51

 $Vu\_max < \phi *Vn Safe! \\ shape k A d Area, A d Bepth, A$ 



Torsional Properties

22 7.99 36.9 118 23.2 4.27 26.0 11.4 3.97 1.33 6.10 1.55 9.84 0.239 275

				$t_w$		2		b,	t,	r	k <sub>des</sub>	<b>k</b> det		•	Gage	w		b, h	1	S	r	Z	1	S	r	7	1	1 1	J	Cw
ł	in. <sup>2</sup>	in		in.		in.	i	n.	in		in.	in.	in.	in.	in.	lb/	/ft	$\frac{b_f}{2t_f}$ $\frac{h}{t_s}$		in. <sup>3</sup>	in.	in.3	in.4	in. <sup>3</sup>	in.	in.3	in.	in.	in.4	in.6
W12×58				0.360				10	0.640	5/8		11/2	15/16		51/2			7.82 27.		78.0	5.28		107		2.51	32.5		11.6	2.10	3570
×53					3/8			10	0.575		1.18		15/16		51/2			8.69 28.		70.6	5.23		95.8						1.58	3160
															512	3	20	5.09 20.	423	70.0	5.25	11.5	55.0	19.2	2.40	29.1	2.19	11.5	1.30	3100
W12×50		12.2				3/16	8.08		0.640		1.14		15/16		51/2	5		6.31 26.		64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
×45		12.1				3/16	8.05		0.575		1.08		15/16			4		7.00 29.		57.7	5.15	64.2	50.0	12.4	1.95	19.0		11.5	1.26	1650
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	13/8	7/8	,	•	4	40	7.77 33.	6 307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440
W12×35°	10.3	12.5	121/2	0.300	5/16	3/16	6.56	61/2	0.520	1/2	0.820	13/16	3/4	10 <sup>1</sup> /8	31/2	3	35	6.31 36.	2 285	45.6	5.25	51.2	24.5	7.47	1.54	11.5	1.79	12.0	0.741	879
×30°		12.3			1/4	1/8	6.52		0.440	7/16	0.740	11/8	3/4		1	-		7.41 41.		38.6	5.21	43.1	20.3		1.52		1.77		0.457	720
×26°					1/4	1/8			0.380		0.680		3/4	۲	*	-		8.54 47.		33.4	5.17	37.2	17.3		1.51		1.75		0.300	607
													E.	1021	01/ 0			1												
W12×22°				0.000	1/4	1/8	4.03		0.425		0.725			103/8	21/49			4.74 41.		25.4	4.91	29.3	4.66		0.848		1.04		0.293	164
×19 <sup>c</sup>					1/4	1/8	4.01	1.1	0.350		0.650		9/16					5.72 46.		21.3	4.82		3.76		0.822		1.02		0.180	131
×16 <sup>c</sup>		12.0			1/4	1/8	3.99		0.265		0.565		9/16	4	1			7.53 49.		17.1	4.67	20.1	2.82		0.773		0.983		0.103	96.9
×14 <sup>c,v</sup>	4.16	11.9	11 1/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	3/4	9/16	'	•	1	14	8.82 54.	3 88.6	14.9	4.62	17.4	2.36	1.19	0.753	1.90	0.961	11.7	0.0704	80.4
W10×112	32.9	11.4	113/8	0.755	3/4	3/8	10.4	10 <sup>3</sup> /8	1.25	11/4	1.75	1 <sup>15</sup> /16	1	71/2	5 <sup>1</sup> /2	11	12	4.17 10.	4 716	126	4.66	147	236	45.3	2.68	69.2	3.08	10.2	15.1	6020
×100	29.3	11.1	111/8	0.680	11/16	3/8	10.3	103/8	1.12	11/8	1.62	113/16	1			10	00	4.62 11.	6 623	112	4.60	130	207	40.0	2.65	61.0	3.04	10.0	10.9	5150
×88	26.0	10.8	107/8	0.605	5/8	5/16	10.3	101/4	0.990	1	1.49	111/16	15/16			8	88	5.18 13.	0 534	98.5	4.54	113	179	34.8	2.63	53.1	2.99	9.81	7.53	4330
×77	22.7	10.6	105/8	0.530	1/2	1/4	10.2	101/4	0.870	7/8	1.37	19/16	7/8			7	77	5.86 14.	8 455	85.9	4.49	97.6	154	30.1	2.60	45.9	2.95	9.73	5.11	3630
×68	19.9	10.4	103/8	0.470	1/2	1/4	10.1	101/8	0.770	3/4	1.27	17/16	7/8			6	68	6.58 16.	7 394	75.7	4.44	85.3	134	26.4	2.59	40.1	2.92	9.63	3.56	3100
×60	17.7	10.2	101/4	0.420	7/16	1/4	10.1	101/8	0.680	11/16	1.18	13/8	13/16			6	60	7.41 18.	7 341	66.7	4.39	74.6	116	23.0	2.57	35.0	2.88	9.52	2.48	2640
×54	15.8	10.1	101/8	0.370	3/8	3/16	10.0	10	0.615	5/8	1.12	15/16	13/16			5	54	8.15 21.	2 303	60.0	4.37	66.6	103	20.6	2.56	31.3	2.85	9.49	1.82	2320
×49	14.4	10.0	10	0.340	5/16	3/16	10.0	10	0.560	9/16	1.06	11/4	13/16	¥	¥	4	49	8.93 23.	1 272	54.6	4.35	60.4	93.4	18.7	2.54	28.3	2.84	9.44	1.39	2070
W10×45	133	10.1	101/0	0.350	3/0	3/16	8.02	8	0.620	5/8	1.12	15/10	13/16	71/2	5 <sup>1</sup> /2		45	6.47 22.	5 248	49.1	4.32	54.9	53.4	13.3	2.01	20.3	2.27	9.48	1.51	1200
×39	11.5			0.315		3/16	7.99		0.530			13/16	13/16	1 12	1			7.53 25.		43.1	4.32	46.8	45.0		1.98	17.2	2.24	9.39		992
×33	9.71					3/16	7.96		0.435		0.935		3/4	¥	•			9.15 27.		35.0	4.19		36.6				2.24	9.30	0.583	791
	0.71																													
W10×30				0.300		3/16			0.510		0.810		11/16		2 <sup>3</sup> /4 <sup>g</sup>			5.70 29.		32.4	4.38		16.7		1.37			10.0	0.622	414
×26	7.61	10.3	103/8	0.260	1/4	1/8	5.77	53/4	0.440	1/16	0.740	1/16	11/16			2	26	6.56 34.	0 144	27.9	4.35	31.3	14.1	4.89	1.36	7.50	1.58	9.86	0.402	345

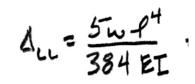
6.49 10.2 10<sup>1</sup>/<sub>8</sub> 0.240 <sup>1</sup>/<sub>4</sub> <sup>1</sup>/<sub>8</sub> 5.75 5<sup>3</sup>/<sub>4</sub> 0.360 <sup>3</sup>/<sub>8</sub> 0.660 <sup>15</sup>/<sub>16</sub> <sup>5</sup>/<sub>8</sub> ¥ ¥

×22°

Analyze the given W-section for beam B1, to determine the maximum live load capacity the floor can carry. Determine the shear and bending forces and check the maximum deflection against an allowable of L/180. Assume the beam is fully braced, Lb < Lp (zone 1).

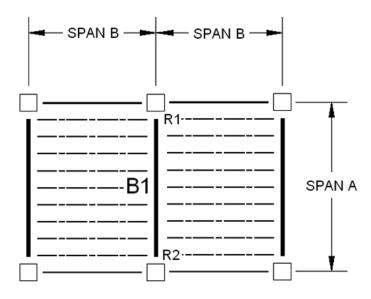
DATASET: 1 -23-	
W-section	W10X30
Fy	50 KSI
Span A	18 FT
Span B	12 FT
Floor DL	14 PSF

#### 15. The actual (unfactored) deflection due to total DL+LL



E\_steel=29000 ksi Ix = 170 in<sup>4</sup> (AISCtable1-1)

-																															1
	Area,	De		Thick	Web ness	t.	w	Fla dth.	nge Thick	ness.		k	Distan	ce	Work	Non	m-	Comp Sect Crite	ion		Axis 2	(-X			Axis	Y-Y		The	ha		ional erties
Shape	A		d	t,		2		b <sub>f</sub>	1		<b>k</b> des	<b>k</b> det	<i>k</i> 1	T	able Gage	Wt		b,	na h	1	\$	r	z	1	\$	r	Z			J	C <sub>w</sub>
	in.2	i		in		in.	1	in.	ir		in.	in.	in.	in.	in.	Ib/I	/ft	24	t <sub>er</sub>	in.4	in. <sup>3</sup>	in.	in.3	in.4	in.3	in.	in. <sup>3</sup>	in.	in.	in.4	in.6
W12×58	17.0			0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	11/2	15/16		5½	5	58	7.82		475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	<sup>9</sup> /16	1.18	13/8	15/16	91/4	51/2	5	53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
W12×50	14.6	12.2	121/4	0.370	3/8	3/16	8.08	81/8	0.640	5/8	1.14	11/2	15/18	91/4	51/2	5	50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
×45	13.1	12.1	12	0.335	5/16	3/16	8.05		0.575	9/16	1.08	13/8	15/16	1.		4	45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
×40	11.7	11.9	12	0.295	\$/16	3/16	8.01	8	0.515	1/2	1.02	13/8	7/8	V	V 1	4	40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440
W12×35°	10.3	12.5	121/2	0.300	\$/16	3/16	6.56	61/2	0.520	1/2	0.820	13/16	3/4	101/8	31/2	3	35	6.31	36.2	285	45.6	5.25	51.2	24.5	7.47	1.54	11.5	1.79	12.0	0.741	879
×30°	8.79	12.3	12 <sup>3</sup> /8	0.260	1/4	1/8	6.52	61/2	0.440	7/16	0.740	11/8	$3_{4}$	1	1.	3	30	7.41	41.8	238	38.6	5.21	43.1	20.3	6.24	1.52	9.56	1.77	11.9	0.457	720
×26 <sup>c</sup>	7.65	12.2	121/4	0.230	1/4	1/8	6.49	61/2	0.380	3/8	0.680	11/16	<sup>3</sup> /4	V.	V V	2	26	8.54	47.2	204	33.4	5.17	37.2	17.3	5.34	1.51	8.17	1.75	11.8	0.300	607
W12×22°	6.48	12.3	121/4	0.260	1/4	1/8	4.03	4	0.425	7/16	0.725	15/16	5/8	10 <sup>3</sup> /a	21/48	2	22	4.74	41.8	156	25.4	4.91	29.3	4.66	2.31	0.848	3.66	1.04	11.9	0.293	164
×19 <sup>c</sup>	5.57	12.2	121/8	0.235	1/4	1/8	4.01	4	0.350	3/8	0.650		9/16		1.1	1	19	5.72	46.2	130	21.3	4.82	24.7	3.76		0.822	2.98	1.02	11.9	0.180	131
×16 <sup>c</sup>		12.0		0.220	1/4	1/8	3.99		0.265	1/4	0.565					1		7.53		103	17.1	4.67	20.1	2.82		0.773		0.983		0.103	96.9
×14 <sup>c,y</sup>	4.16	11.9	117/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	3/4	P/16			1.	14	8.82	54.3	88.6	14.9	4.62	17.4	2.36	1.19	0.753	1.90	0.961	11.7	0.0704	80.4
W10×112	32.9	11.4	113/8	0.755	3/4	3/8	10.4	103/8	1.25	11/4	1.75	115/16	1	71/2	51/2	11:	12	4.17	10.4	716	126	4.66	147	236	45.3	2.68	69.2	3.08	10.2	15.1	6020
×100				0.680	11/16	3/8	10.3	10 <sup>\$</sup> /8		11/8	1.62	113/16				10		4.62		623	112	4.60	130	207	40.0	2.65	61.0	3.04	10.0	10.9	5150
×88	26.0			0.605	5/8	\$/16	10.3	101/4	0.990	1	1.49	111/16						5.18		534	98.5		113	179	34.8	2.63	53.1	2.99	9.81	7.53	4330
×77 ×68	22.7 19.9		105/8		1/2	1/4 1/4	10.2	10 <sup>1</sup> /4 10 <sup>1</sup> /8	0.870	7/8 3/4	1.37	1%16 17/16	7/8			7		5.86		455	85.9	4.49		154	30.1	2.60	45.9	2.95	9.73	5.11	3630
×00 ×60	17.7			0.470	1/2 7/16	74 1/4	10.1	10%	0.680	11/16	1.27	13/8	7/8 13/16				68 60	6.58 7.41		394 341	75.7 66.7	4.44	85.3 74.6	134 116	26.4 23.0	2.59 2.57	40.1 35.0	2.92	9.63 9.52	3.56 2.48	3100 2640
×54	15.8			0.370	3/8	3/16	10.0	1078	0.615	5/8	1.12	15/16	13/16					8.15		303	60.0	4.37	66.6	103	20.6	2.57	31.3	2.00	9.32	1.82	2320
×49	14.4			0.340	5/16	3/16	10.0	10	0.560	9/16		11/4	13/16		1			8.93		272	54.6	4.35	60.4	93.4	18.7	2.54	28.3	2.84	9.44	1.39	2070
W10×45	13.3	10.1	101/0	0.350	3/8	3/16	8.02	8	0.620	5y <sub>8</sub>	1.12	15/16	13/16	71/2	51/2			6.47		248	49.1	4.32		53.4	13.3	2.01	20.3	2.27	9.48	1.51	1200
×39	11.5	9.92		0.315	5/16	3/16	7.99		0.530	1/2	1.03	13/16	13/16		1 1			7.53		240	42.1	4.27	46.8	45.0	11.3	1.98	17.2	2.24	9.40	0.976	992
×33	9.71			0.290	5/16	3/16	7.96		0.435		0.935		3/4	1	1 *			9.15		171	35.0	4.19	38.8	36.6		1.94	14.0	2.20	9.30	0.583	791
W10×30	8.84	10.5	101/2	0.300	5/16	3/16	5.81	53/4	0.510	1/2	0.810	1%	11/16	81/4	23/49	3	30	5.70	29.5	170	32.4	4.38	36.6	16.7	5.75	1.37	8.84	1.60	10.0	0.622	414
×26				0.260	1/4	1/8	5.77	53/4	0.440	7/16	0.740	11/18	11/16					6.56		144	27.9	4.35	31.3	14.1		1.36		1.58	9.86	0.402	345
×22 <sup>s</sup>	6.49	10.2	101/8	0.240	1/4	1/8	5.75	53/4	0.360	3/8	0.660	15/16	5/8	₩	♥	2	22	7.99	36.9	118	23.2	4.27	26.0	11.4	3.97	1.33	6.10	1.55	9.84	0.239	275



Deflection = 5 \* (w\_DL+ w\_LL) \* SpanA<sup>4</sup> / (384 \* E'\* I)  
= 5 \* [(0.198+1.97) \*1000 \* 
$$\frac{1}{12}$$
]\* (18\*12)<sup>4</sup> / (384 \* 29000000 \* 170)  
= 1.03868 in

16. The deflection limit L/180

SpanA /180 = 18\*12/180 = 1.2 in

**17.** Is the deflection less than the Limit L/180?(1=yes,0=no) 1.03868 in <1.2 in , Pass! = 1

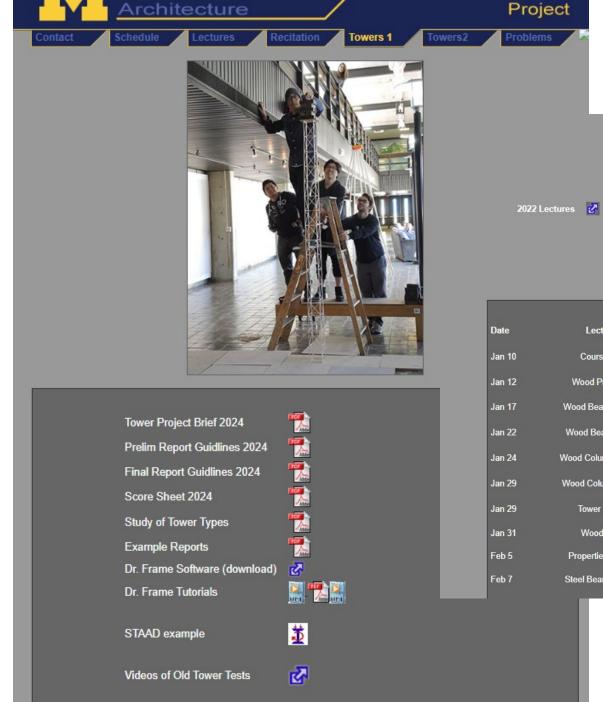
# **Tower Project**

# Timeline Sign up team

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, Scho	dek 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]		
	Otrail Decention	100 00000 07	<ol><li>Wood Column Analysis</li></ol>
FEB 5	Steel Properties	AISC, Onouye 8.7 Schodek 6.4.3	
FEB 7 FEB 9	Steel Beam Analysis Recitation [3-Steel Beams]	Schodek 0.4.5	
TED 5	Neckator [5-Steel Beams]		4 Steel Beam Analysis
FEB 12	Steel Beam Design	Schodek 6.4.3	4 eteel Bearry haryone
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 FEB 32	"Skyscrapers" David Macaulay Recitation	video	
FEB 32	Recitation		6. Steel Column Analysis
FEB 26	WINTER RECESS **** NO CL	ASS **** WINTER RECESS *	
FEB 27	WINTER RECESS **** NO CLA		
MAR 1	WINTER RECESS **** NO CLA		
MAR 4	Continuous Beams	I. Engel Ch. 17, Schodek 8	
MAR 6	Gerber Beams	Schodek 8.4.4	
MAR 8	Recitation [5-Continuous Bear	nsj	7. Three Moment Theorem
MAR 11	Intro to Concrete – PCA video.		7. Three Moment Theorem
MAR 13	Concrete Beams	Schodek 6.4.4 – 6.4.6	
MAR 15	Recitation		
MAR 18	Tower Testing **** Tower Tes		Tower Testing ****
MAR 20		I. Engel Ch.15	
MAR 22	Recitation [6-Stress vs Strain]		9 Concrete Ream Analysis
MAR 25	Concrete Beams		8. Concrete Beam Analysis
MAR 27	Concrete Columns	Schodek 7.4.5	
MAR 29	Recitation [7-Concrete Reinfor		
APR 1			9. Concrete Beam Design
741111	Composite Sections	TMS 402	9. Concrete Beam Design
APR 3	Masonry Walls	TMS 402	9. Concrete Beam Design
		TMS 402	-
APR 3 APR 5	Masonry Walls Recitation [8-Composite Secti	TMS 402 ons]	9. Concrete Beam Design 10. Composite Sections
APR 3 APR 5 APR 8	Masonry Walls Recitation [8-Composite Secti Masonry Walls	TMS 402 ons] TMS 402	-
APR 3 APR 5 APR 8 APR 10	Masonry Walls Recitation [8-Composite Secti Masonry Walls Shells and Vaults	TMS 402 ons] TMS 402 Schodek 12	-
APR 3 APR 5 APR 8	Masonry Walls Recitation [8-Composite Secti Masonry Walls	TMS 402 ons] TMS 402 Schodek 12	10. Composite Sections
APR 3 APR 5 APR 8 APR 10	Masonry Walls Recitation [8-Composite Secti Masonry Walls Shells and Vaults	TMS 402 ons] TMS 402 Schodek 12 Final Tower Report Due	-
APR 3 APR 5 APR 8 APR 10 APR 12	Masonry Walls Recitation [8-Composite Secti Masonry Walls Shells and Vaults Recitation [9-Lateral Stability] Combined Stress Combined Stress	TMS 402 ons] TMS 402 Schodek 12 Final Tower Report Due I. Engel Ch. 19 I. Engel Ch. 19 I. Engel Ch. 19	10. Composite Sections
APR 3 APR 5 APR 8 APR 10 APR 12 APR 15	Masonry Walls Recitation [8-Composite Secti Masonry Walls Shells and Vaults Recitation [9-Lateral Stability] Combined Stress	TMS 402 ons] TMS 402 Schodek 12 Final Tower Report Due I. Engel Ch. 19 I. Engel Ch. 19 I. Engel Ch. 19	10. Composite Sections
APR 3 APR 5 APR 10 APR 10 APR 12 APR 15 APR 17 APR 19	Masonry Walls Recitation [8-Composite Secti Masonry Walls Shells and Vaults Recitation [9-Lateral Stability] Combined Stress Combined Stress Recitation [10-Combined Stress	TMS 402 ons] TMS 402 Schodek 12 Final Tower Report Due I. Engel Ch. 19 I. Engel Ch. 19 I. Engel Ch. 19	10. Composite Sections
APR 3 APR 5 APR 10 APR 10 APR 12 APR 15 APR 17	Masonry Walls Recitation [8-Composite Secti Masonry Walls Shells and Vaults Recitation [9-Lateral Stability] Combined Stress Combined Stress	TMS 402 ons] TMS 402 Schodek 12 Final Tower Report Due I. Engel Ch. 19 I. Engel Ch. 19 I. Engel Ch. 19	10. Composite Sections 11. Masonry Walls

# **Tower Project**

## Resources



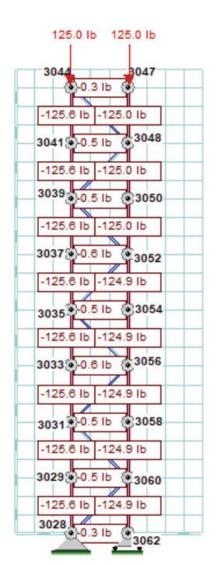


Canvas 🛃

)	Lectures	Video w/Quiz	Video	Slides	Notes
10	Course Intro		MP4		1
12	Wood Properties	NP4			<b>*</b>
17	Wood Beam Analysis		MP4	1	1
22	Wood Beam Design	NP4	MP4	<b>***</b>	
24	Wood Column Analysis	MP4	MP4	-	-
29	Wood Column Design	NP4		1	1
29	Tower Project		MP4	-	
31	Wood - CLT				
5	Properties of Steel				
7	Steel Beam Analysis				

# **Tower Project**

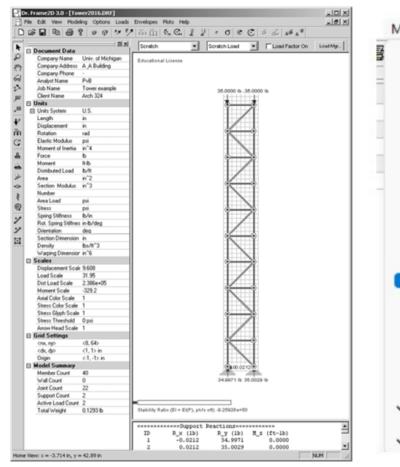
# Dr. Frame



### Dr. Frame 2D

- Material properties
  - Provided test values
  - NDS
  - Other (online)
- Member dimensions
  - Use actual dimensions
- Connections
  - Use pinned connections

Second order analysis



lod	leling	Options	Loads	Envelopes	Plots	1			
	Auto	Beam							
	Auto	Truss							
	Auto								
	Trans	form Selec							
	Split								
	Defa	Default Member Properties							
	Isolate Joints Isolate Members			Ctrl+J Ctrl+M					
	2nd (	Order Anal	ysis		3				
	Load	-Depender	nt El						
	nth-(	Order step			î.				
	Plast	ic Hinges			>				
	Norn	nalize Value	es		[				
/	Resis	tance Facto	ors On		]				
/	Realt	ime Solutio	on						

## **Tower Projec**

## Resources

Analysis

Use NDS approach

Find load P and stress F'c for each member

Use 1.0 for all factors except C<sub>p</sub>

where:

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 CP and F'c. Other NDS stress adjustment factors (CD, CM, CL, CF and CI) 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).

F'

Analysis Capacity Design  $C_{p} = \frac{1 + (F_{cE}/F_{c}^{*})}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_{c}^{*})}{2c}\right]^{2} - \frac{F_{cE}/F_{c}^{*}}{c}} (3.7-1)$  $f_a = \frac{P}{M} \leq F'_a$  $P = F'_{c} A$ A = -F. = reference compression design value paral-Properties of Basswood: lel to grain multiplied by all applicable adjustment factors except C, (see 2.3), psi Density (oven dry) 20 pcf \*  $F_{cE} = \frac{0.822 E_{min}}{(\ell_e/d)^2}$ E (buckling) 1,650,000 psi \*\* F (Compression || to grain) 4745 psi ' F (Compression ⊥ to grain) 377 psi \* c = 0.8 for sawn lumber F (Tension || to grain) 4500 psi (estimate) c = 0.85 for round timber poles and piles F (Tension ⊥ to grain) 348 psi \* c = 0.9 for structural glued laminated timber or www.matweb.com F (Shear || to grain) 986 psi \* structural composite lumber tested by PvB F (Flexure) 5900 psi \* University of Michigan, TCAUP Structures II Slide 4 of 17

## PREDICATE CAPACITY

1. Vertical Member Buckling Capacity:

If K = 1 then  $\frac{le}{d} = \frac{l.K}{d} = \frac{(6)^{\circ}}{0.25^{\circ}} = 24 < 50$  $Fce = \frac{0.822 Emin}{(le/d)} = \frac{0.822 \times 1650000}{24^2} = 2355 PSI$  $Fc *= Fc = 4745 PSI, \frac{Fce}{Ec^*} = 0.496$  $Cp = \frac{1 + \left(\frac{fct}{Pc}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{fct}{Pc}\right)}{2c}\right]^2 - \frac{\left(\frac{fct}{Pc}\right)}{c}} = \frac{1 + 0.496}{2\times0.8} - \sqrt{\left[\frac{1 + 0.496}{2\times0.8}\right]^2 - \frac{0.496}{0.8}}$ = 0.43F'c = Fc. (CD. Cm. Ct. CF. Ci. Cp) = 4745 × 0.43 = 2040PSI Pcr\_members  $P = F'c A = 2040 \times 0.25^2 = 127#$ 

2. Buckling Capacity of the Tower as a whole:

$$I = \Sigma I + \Sigma A d^{2} = 4 \times \frac{0.25 \times 0.25^{3}}{12} + 4 \times (0.25 \times 0.5) \times (3 - 0.125)^{2} = 2.07 in^{4}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2.07}{0.25^{2} \times 4}} = 2.88''$$

$$\frac{Kl}{r} = \frac{1 \times 48}{2.88} = 16.67$$

$$Pcr = \frac{\pi^{2} IE}{\left(\frac{KL}{r}\right)^{2}} = \frac{\pi^{2} \times 2.07 \times 1650000}{(16.67)^{2}} = 121306\#$$
Each column:  $\frac{Pcr}{4} = 30326.5\#$ 

3. Crushing Capacity of Vertical Members:

 $P = Fc A = 4745 \times 0.25^2 = 296\#$ 

## P compression max

## LAB - Steel Beams

#### Description

This project uses observation to understand how unbraced compression edges and lateral torsional buckling reduce the ultimate load capacity of steel beams.

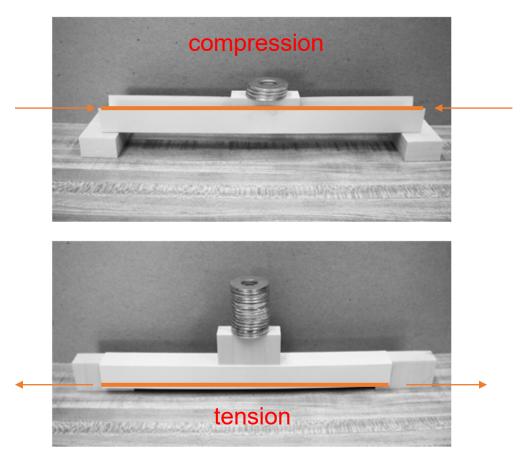
### Goals

To observe the behavior of unbraced section edges in compression vs tension.

To measure capacity loss due to lateral torsional buckling.

#### Procedure

- 1. Position the U shaped section with the free edges on the upper side of the span.
- 2. Test how many washers the section can support at mid span. Use a wood block to position the load. Observe the mode (how) it fails.
- 3. Repeat the procedure with the section inverted and the free edges downward.
- 4. Compare the load level carried by each orientation of the paper beam and describe the behavior under load.



Any Questions?

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# Thank You!

