

Steel Beam Analysis 2/9

HW - Steel Beam Analysis

Tower Project

Lab – Steel Beams

Structure II
Section 004

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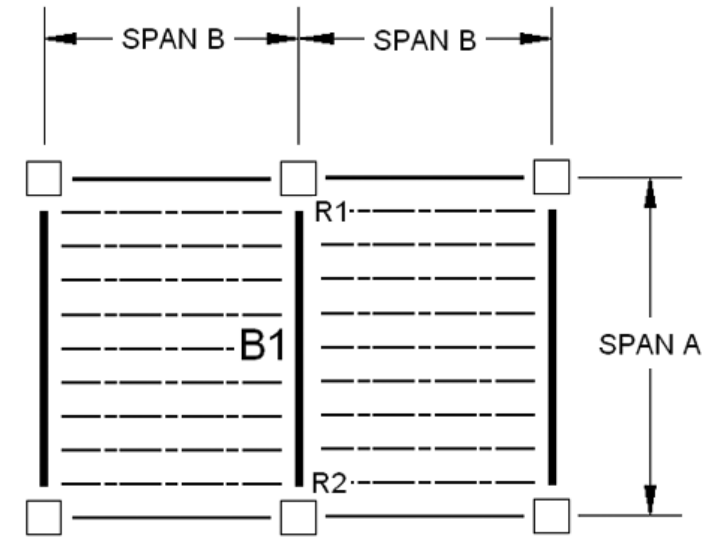
Happy Lunar Dragon Year!



4. Steel Beam Analysis

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, $L_b < L_p$ (zone 1).

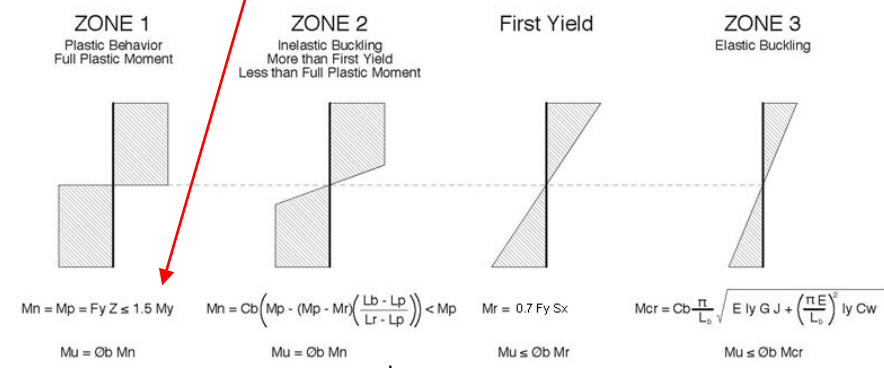
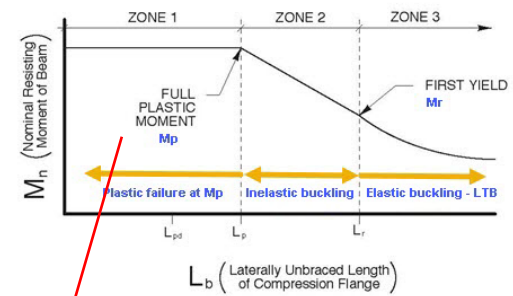
DATASET: 1	-2-	-3-
W-section	W10X30	
Fy	50 KSI	
Span A	18 FT	
Span B	12 FT	
Floor DL	14 PSF	



HW - Steel Beam Analysis

Given:
beam size (yield stress)
bracing type
load

Goal:
load capacity?



$\phi_b = 0.90$ Safety Factor

1. Determine the unbraced length of the compression flange (L_b).
2. Find the L_p and L_r values from the AISC properties table 3-6
3. Compare L_b to L_p and L_r and determine which equation for M_n or M_{cr} to be used.
4. Determine the beam load equation for maximum moment in the beam. Solve for M_n .
5. Calculate load based on maximum moment. $M_u = \phi_b M_n$

Given:
 $L_b < L_p$

Plastic Behavior (zone 1)

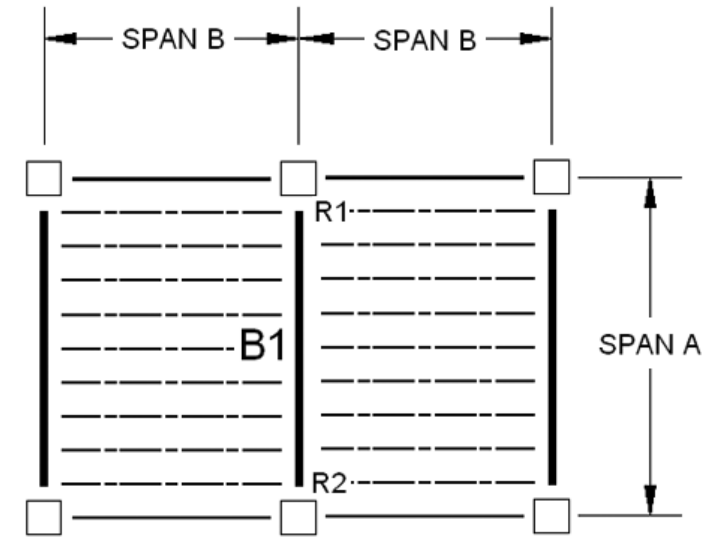
Maximum Moment:
 $M_n = M_p = F_y Z_x$

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

4. Steel Beam Analysis

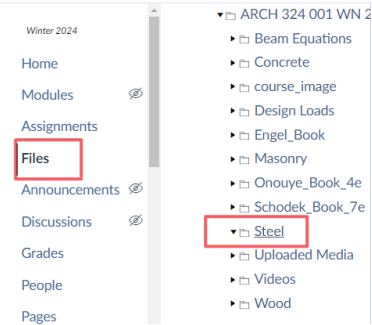
Analyze the given W-section for beam B1, to determine the maximum live load capacity the floor can carry. Determine the shear and bending forces and check the maximum deflection against an allowable of L/180. Assume the beam is fully braced, $L_b < L_p$ (zone 1).

DATASET: 1 -2- -3-
W-section W10X30
 F_y 50 KSI
Span A 18 FT
Span B 12 FT
Floor DL 14 PSF



1. The plastic modulus of the section, $Z_x = 36.6 \text{ in}^3$


AISC table 1-1



Name	Date Created	Date Modified	Modified By	Size
AISC_d831.pdf	Feb 3, 2022	Feb 3, 2022		9.4 MB
AISC9_BeamEqu	Jan 9, 2017	Jan 9, 2017		35 MB
AISC14_BeamCh	Dec 30, 2016	Dec 30, 2016		163.5 MB
AISC14_Table1-1.pdf	Feb 8, 2017	Feb 8, 2017		14.5 MB
AISC14_Table3-2.pdf	Feb 8, 2017	Feb 8, 2017		5.3 MB

Table 1-1 (continued)
W-Shapes
Dimensions

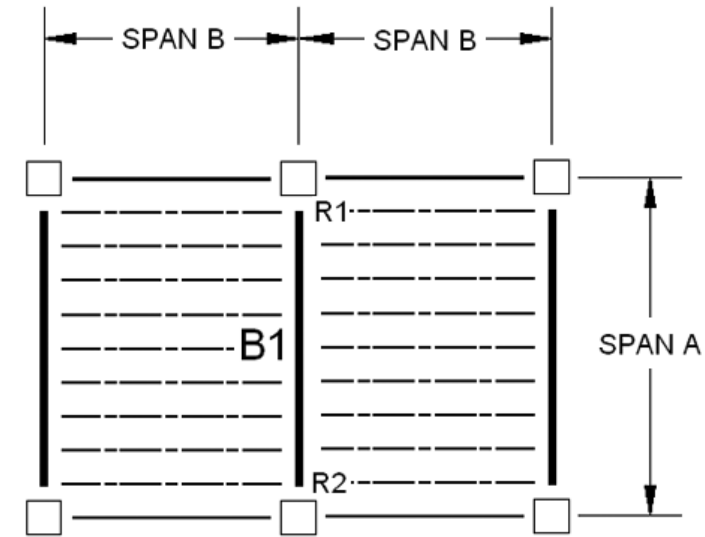
Shape	Area, A	Depth, d		Web		Flange		Distance					Work- able Gage		
				Thickness, t _w	t _w 2	Width, b _f	Thickness, t _f	k		k ₁	T				
								k _{des}	k _{det}						
	in. ²	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.			
W12×58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1 1/2	15 1/16	9 1/4	5 1/2
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15 1/16	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/8	1.14	1 1/2	15 1/16	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 3/8	15 1/16		
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 3/8	7/8		
W12×35 ^c	10.3	12.5	12 1/2	0.300	5/16	3/16	6.56	6 1/2	0.520	1/2	0.820	1 3/8	3/4	10 1/8	3 1/2
×30 ^c	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1 1/8	3/4		
×26 ^c	7.65	12.2	12 1/4	0.230	1/4	1/8	6.49	6 1/2	0.380	3/8	0.680	1 1/16	3/4		
W12×22 ^c	6.48	12.3	12 1/4	0.260	1/4	1/8	4.03	4	0.425	7/16	0.725	15/16	5/8	10 3/8	2 1/4 ^d
×19 ^c	5.57	12.2	12 1/8	0.235	1/4	1/8	4.01	4	0.350	3/8	0.650	7/8	9/16		
×16 ^c	4.71	12.0	12	0.220	1/4	1/8	3.99	4	0.265	1/4	0.565	13/16	9/16		
×14 ^{c,v}	4.16	11.9	11 7/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	3/4	9/16		
W10×112	32.9	11.4	11 3/8	0.755	3/4	3/8	10.4	10 3/8	1.25	1 1/4	1.75	1 15/16	1	7 1/2	5 1/2
×100	29.3	11.1	11 1/8	0.680	11/16	3/8	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	1		
×88	26.0	10.8	10 7/8	0.605	5/8	3/16	10.3	10 1/4	0.990	1	1.49	1 11/16	15/16		
×77	22.7	10.6	10 5/8	0.530	1/2	1/4	10.2	10 1/4	0.870	7/8	1.37	1 9/16	7/8		
×68	19.9	10.4	10 3/8	0.470	1/2	1/4	10.1	10 1/8	0.770	3/4	1.27	1 7/16	7/8		
×60	17.7	10.2	10 1/4	0.420	7/16	1/4	10.1	10 1/8	0.680	11/16	1.18	1 3/8	13/16		
×54	15.8	10.1	10 1/8	0.370	3/8	3/16	10.0	10	0.615	5/8	1.12	1 5/16	13/16		
×49	14.4	10.0	10	0.340	5/16	3/16	10.0	10	0.560	9/16	1.06	1 1/4	13/16		
W10×45	13.3	10.1	10 1/8	0.350	3/8	3/16	8.02	8	0.620	5/8	1.12	1 5/16	13/16	7 1/2	5 1/2
×39	11.5	9.92	9 7/8	0.315	5/16	3/16	7.99	8	0.530	1/2	1.03	1 3/16	13/16		
×33	9.71	9.73	9 3/4	0.290	5/16	3/16	7.96	8	0.435	7/16	0.935	1 1/8	3/4		
W10×30	8.84	10.5	10 1/2	0.300	5/16	3/16	5.81	5 3/4	0.510	1/2	0.810	1 1/8	1 1/16	8 1/4	2 3/4 ^d
×26	7.61	10.3	10 3/8	0.260	1/4	1/8	5.77	5 3/4	0.440	7/16	0.740	1 1/16	1 1/16		
×22 ^c	6.49	10.2	10 1/8	0.240	1/4	1/8	5.75	5 3/4	0.360	3/8	0.660	15/16	5/8		

Table 1-1 (continued) W-Shapes Properties															 W12-W10	
Nom- inal Wt. lb/ft	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties			
			I in. ⁴	S in. ³	r in.	Z in. ³	I in. ⁴	S in. ³	r in.	Z in. ³			in.	in.	J in. ⁴	C_w in. ⁶
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570		
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		
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	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	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		
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	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	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		
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	5.72	46.2	130	21.3	4.82	24.7	3.76	1.88	0.822	2.98	1.02	11.9	0.180	131		
16	7.53	49.4	103	17.1	4.67	20.1	2.82	1.41	0.773	2.26	0.983	11.7	0.103	96.9		
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		
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	4.62	11.6	623	112	4.60	130	207	40.0	2.65	61.0	3.04	10.0	10.9	5150		
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	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	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	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	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	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		
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	7.53	25.0	209	42.1	4.27	46.8	45.0	11.3	1.98	17.2	2.24	9.39	0.976	992		
33	9.15	27.1	171	35.0	4.19	38.8	36.6	9.20	1.94	14.0	2.20	9.30	0.583	791		
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	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		
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		

4. Steel Beam Analysis

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, $L_b < L_p$ (zone 1).

DATASET: 1	-2-	-3-
W section	W10X30	
Fy	50 KSI	
Span A	18 FT	
Span B	12 FT	
Floor DL	14 PSF	



2. The nominal bending moment, Mn

$M_n = F_y \cdot Z_x = 50 \cdot 36.6 = 1830 \text{ k-in}$

3. The factored bending resistance, phi Mn

$\phi \cdot M_n = 0.9 \cdot 1830 = 1647 \text{ k-in}$

4. The factored design moment, Mu

$M_u = \phi \cdot M_n = 1647 / 12 = 137.25 \text{ k-ft}$

5. The total factored design load, wu

$M_u = w_u \cdot \text{SpanA}^2 / 8 = 137.25 \text{ k-ft}$
 $w_u = 8 \cdot M_u / \text{SpanA}^2$
 $= 8 \cdot 137.25 / 18^2$
 $= 3.39 \text{ klf}$

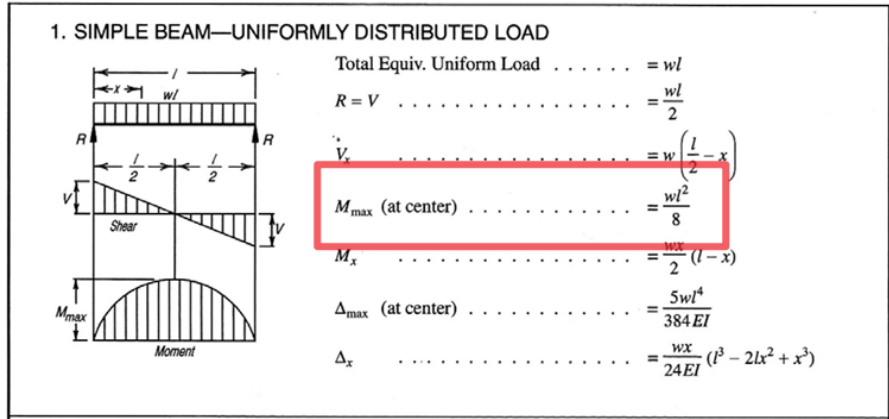
Load & Resistance Factored Design (LRFD)

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

$P_{load} = \gamma \cdot P_{applied_load}$

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

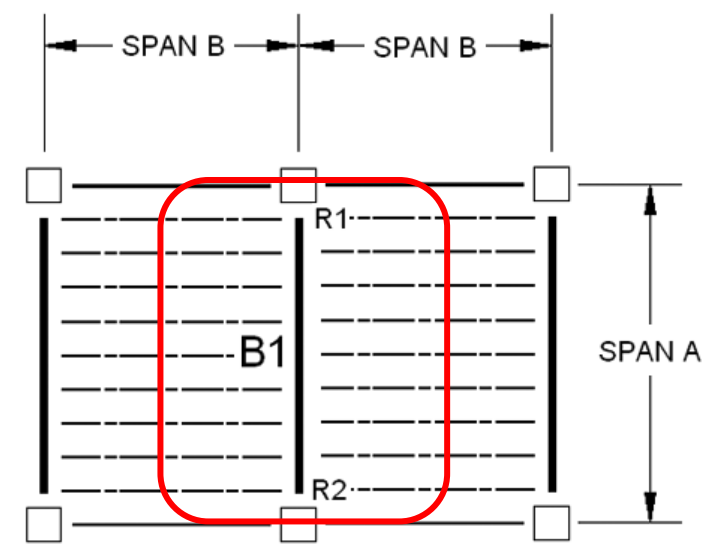
$P_{resisting} = \phi \cdot P_{material_strength}$



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DATASET: 1	-2-	-3-
W-section	W10X30	
Fy	50 KSI	
Span A	18 FT	
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Floor DL	14 PSF	



6. The total unfactored dead load on the beam, w_{DL}

$$w_{Floor\ DL} = \text{Tributary area} \cdot \text{Floor DL} = \text{SpanB} \cdot \text{Floor DL}$$
$$= 12 \cdot 14 = 168 \text{ plf}$$

$$w_{Beam\ DL} = 30 \text{ plf}$$

$$w_{DL} = w_{Floor\ DL} + w_{Beam\ DL} = (168 + 30) / 1000 = 0.198 \text{ klf}$$

7. The total factored dead load on the beam, w_{uDL}

$$w_{uDL} = 1.2 \cdot w_{DL} = 1.2 \cdot 0.198 = 0.2376 \text{ klf}$$

8. The factored beam live load, w_{uLL}

$$w_{uDL} + w_{uLL} = w_u$$

$$w_{uLL} = w_u - w_{uDL} = 3.39 - 0.2376 = 3.1524 \text{ klf}$$

9. The actual beam live load(capacity), w_{LL}

$$w_{uLL} = w_{LL} \cdot 1.6$$

$$w_{LL} = w_{uLL} / 1.6 = 3.1524 / 1.6 = 1.97 \text{ klf}$$

10. The actual floor live load(floor capacity), LL

$$LL = w_{LL} / \text{SpanB} = (1.97 / 12) \cdot 1000 = 164.17 \text{ psf}$$

LRFD Analysis

Load & Resistance Factored Design (LRFD)

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

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

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

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

Design Strength

$$\frac{P_u}{\phi} \leq P_n$$

Required (Nominal) Strength

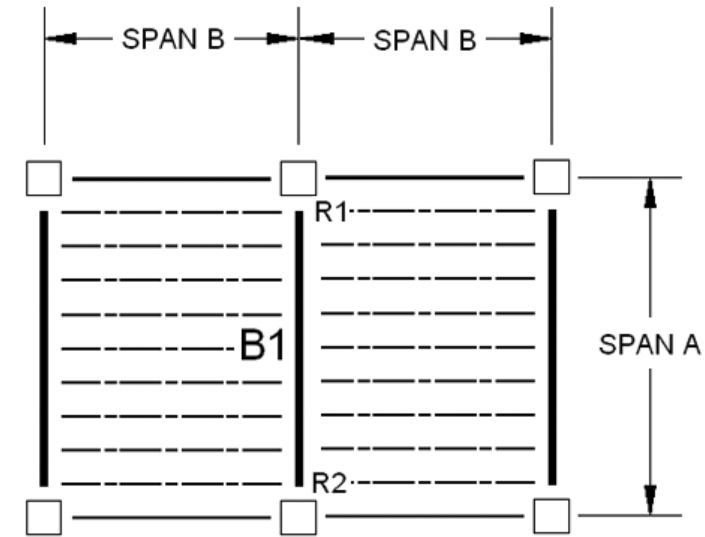
2.3 LOAD COMBINATIONS FOR STRENGTH DESIGN

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. $0.9D + 1.0W$

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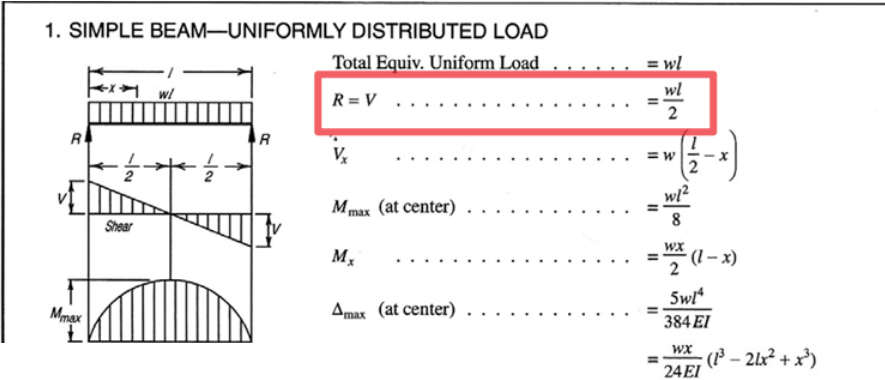
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Check the shear force $V_u \leq \phi V_n$

11. The maximum factored design beam shear force, V_{u_max}

$$V_{u_max} = w_u \cdot \text{Span} / 2$$
$$= 3.39 \cdot 18 / 2$$
$$= 30.51 \text{ kip}$$

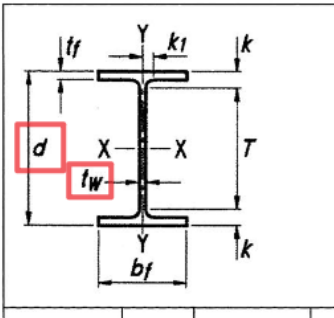


12. The Web Area, A_w

$$A_w = d \cdot t_w = 10.5 \cdot 0.3 = 3.15 \text{ in}^2$$

Design for Shear

Shear stress in steel sections is approximated by averaging the stress in the web:

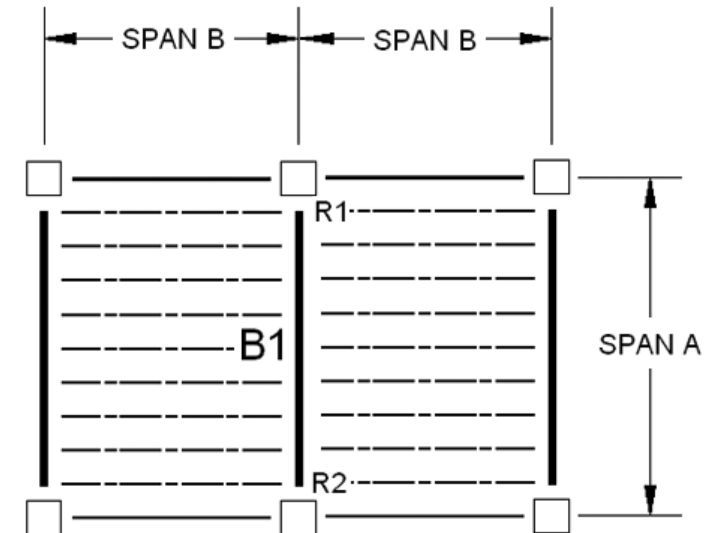
$$F_v = V / A_w$$
$$A_w = d \cdot t_w$$


Shape	Area, A	Depth, d	Web				Flange				Distance					
			Thickness, t _w		t _w 2	Width, b _f		Thickness, t _f		k		k ₁	T	Work- able Gage		
			in. ²	in.		in.	in.	in.	in.	k _{des}	k _{det}				in.	in.
W12×58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1 1/2	15/16	9/4	5 1/2	
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	×30 ^c	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1/8	3/4	↓	↓
	×26 ^c	7.65	12.2	12 1/4	0.230	1/4	1/8	6.49	6 1/2	0.380	3/8	0.680	1 1/16	3/4	↓	↓
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×49	14.4	10.0	10	0.340	5/16	3/16	10.0	10	0.560	9/16	1.06	1 1/4	13/16	↓	↓	
W10×45	13.3	10.1	10 1/8	0.350	3/8	3/16	8.02	8	0.620	5/8	1.12	1 5/16	13/16	7 1/2	5 1/2	
	×39	11.5	9.92	9 7/8	0.315	5/16	3/16	7.99	8	0.530	1/2	1.03	1 3/16	13/16	↓	↓
	×33	9.71	9.73	9 3/4	0.290	5/16	3/16	7.96	8	0.435	7/16	0.935	1 1/8	3/4	↓	↓
W10×30	8.84	10.5	10 1/2	0.300	5/16	3/16	5.81	5 3/4	0.510	1/2	0.810	1 1/8	11/16	8 1/4	2 3/4 ^a	
	×26	7.61	10.3	10 3/8	0.260	1/4	1/8	5.77	5 3/4	0.440	7/16	0.740	1 1/16	5/8	↓	↓
	×22 ^c	6.49	10.2	10 1/8	0.240	1/4	1/8	5.75	5 3/4	0.360	3/8	0.660	15/16	5/8	↓	↓

4. Steel Beam Analysis

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	-2-	-3-
W-section	W10X30	
Fy	50 KSI	
Span A	18 FT	
Span B	12 FT	
Floor DL	14 PSF	



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

$\phi * V_n = 1 * 94.5 = 94.5 \text{ kip}$
 $V_{u_max} = 30.51$
 $V_{u_max} < \phi * V_n \text{ Safe!}$

Check the shear force $V_u \leq \phi V_n$

13. The factored shear resistance, phi Vn

$h / t_w = 29.5 < 59$

So Zone 1
 $V_n = 0.6 * F_y * A_w$
 $= 0.6 * 50 * 3.15 = 94.5 \text{ kip}$

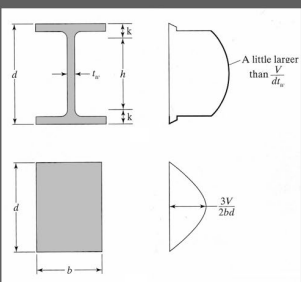
Design for Shear

Shear stress in steel sections is approximated by averaging the stress in the web:

$F_v = V / A_w$
 $A_w = d * t_w$

To adjust the stress a reduction factor of 0.6 is applied to F_y

$F_v = 0.6 F_y$
so, $V_n = 0.6 F_y A_w$ (Zone 1)



The equations for the 3 stress zones:
(ϕ in all cases = 1.0)

Zone 1:
WEB YIELDING (Most beam sections fall into this category)
if $\frac{h}{t_w} \leq 2.45 \sqrt{E/F_y} = 59$ (for 50 ksi steel)
then: $V_n = 0.6 F_y A_w$

Zone 2:
INELASTIC WEB BUCKLING
if $2.45 \sqrt{E/F_y} < \frac{h}{t_w} \leq 3.07 \sqrt{E/F_y} = 74$ (for 50 ksi steel)
then: $V_n = 0.6 F_y A_w (2.45 \sqrt{E/F_y}) / \frac{h}{t_w}$

Zone 3:
ELASTIC WEB BUCKLING
if $3.07 \sqrt{E/F_y} < \frac{h}{t_w} \leq 260$
then: $V_n = A_w \left[\frac{4.25 E}{\left(\frac{h}{t_w} \right)^2} \right]$

Shape	Area, A	Depth, d	Web		Flange		Distance						Nominal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r _{ts}	h _o	Torsional Properties					
			Thickness, t _w	t _w Z	Width, b _f	Thickness, t _f	K				T	Work- able Gage		I	S	r	Z	I	S	r	Z	J	C _w								
							k _{des}	k _{des}	k ₁	T																					
																										in.	in.	in.	in.		
in. ²	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	lb/ft	b _f 2t _w	h t _w	I	S	r	Z	I	S	r	Z	in. ⁶	in.	in. ⁴	in. ⁶					
W12x58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/16	1.24	1 1/2	15 1/16	9 1/4	5 1/2	58	782	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570	
	x53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 1/8	15 1/16	9 1/4	5 1/2	53	869	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
W12x50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/16	1.14	1 1/2	15 1/16	9 1/4	5 1/2	50	631	26.8	391	64.2	5.18	71.9	95.3	13.9	1.96	21.3	2.25	11.6	1.71	1880	
	x45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 1/8	15 1/16		5	45	700	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
x40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.510	1/2	1.02	1 1/2	13 1/8	7/8		40	777	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440	
W12x35 ^c	10.3	12.5	12 1/2	0.300	3/16	3/16	6.56	6 1/2	0.520	1/2	0.820	1 1/8	13 1/8	3/4	10 1/8	3 1/2	35	631	36.2	285	45.6	5.25	51.2	24.5	7.47	1.54	11.5	1.79	12.0	0.741	879
	x30 ^c	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1 1/8	13 1/8	3/4		30	741	41.8	238	38.6	5.21	43.1	20.3	6.24	1.52	9.56	1.77	11.9	0.457	720
	x26 ^c	7.65	12.2	12 1/4	0.230	1/4	1/8	6.49	6 1/2	0.380	3/8	0.680	1 1/8	13 1/8	3/4		26	854	47.2	204	33.4	5.17	37.2	17.3	5.34	1.51	8.17	1.75	11.8	0.300	607
W12x22 ^c	6.48	12.3	12 1/4	0.260	1/4	1/8	4.03	4	0.425	7/16	0.725	1 1/8	13 1/8	5/8	10 1/8	2 1/4	22	474	41.8	156	25.4	4.91	29.3	4.66	2.31	0.848	3.66	1.04	11.9	0.293	164
	x19 ^c	5.57	12.2	12 1/8	0.235	1/4	1/8	4.01	4	0.350	3/8	0.650	1 1/8	13 1/8	5/8		19	572	46.2	130	21.3	4.82	24.7	3.76	1.88	0.822	2.98	1.02	11.9	0.180	131
	x16 ^c	4.71	12.0	12	0.220	1/4	1/8	3.99	4	0.265	1/4	0.565	1 1/8	13 1/8	5/8		16	753	49.4	103	17.1	4.67	20.1	2.82	1.41	0.773	2.26	0.983	11.7	0.103	96.9
x14 ^c	4.16	11.9	11 1/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	1 1/8	13 1/8	5/8		14	882	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	
W10x112	32.9	11.4	11 3/8	0.755	3/4	3/8	10.4	10 1/2	1.25	1 1/4	1.75	1 15/16	1	7 1/2	5 1/2	112	417	10.4	716	126	4.66	147	236	45.3	2.68	69.2	3.08	10.2	15.1	6020	
	x100	29.3	11.1	11 1/8	0.680	11/16	3/8	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	1		100	462	11.6	623	112	4.60	130	207	40.0	2.65	61.0	3.04	10.0	10.9	5150	
	x88	26.0	10.8	10 7/8	0.605	5/8	3/8	10.3	10 1/4	0.990	1	1.49	1 11/16	15 1/16		88	518	13.0	534	98.5	4.54	113	179	34.8	2.63	53.1	2.99	9.81	7.53	4330	
	x77	22.7	10.6	10 5/8	0.530	1/2	1/4	10.2	10 1/4	0.870	7/8	1.37	1 9/16	13 1/8	7/8		77	586	14.8	455	85.9	4.49	97.6	154	30.1	2.60	45.9	2.95	9.73	5.11	3630
	x68	19.9	10.4	10 3/8	0.470	1/2	1/4	10.1	10 1/8	0.770	3/4	1.27	1 7/8	13 1/8	7/8		68	658	16.7	394	75.7	4.44	85.3	134	26.4	2.59	40.1	2.92	9.63	3.56	3100
	x60	17.7	10.2	10 1/4	0.420	7/16	1/4	10.1	10 1/8	0.680	11/16	1.18	1 3/8	13 1/8	13/16		60	741	18.7	341	66.7	4.39	74.6	116	23.0	2.57	35.0	2.88	9.52	2.48	2640
	x54	15.8	10.1	10 1/8	0.370	3/8	3/16	10.0	10	0.615	5/8	1.12	1 5/8	13 1/8	13/16		54	815	21.2	303	60.0	4.37	66.6	103	20.6	2.56	31.3	2.85	9.49	1.82	2320
	x49	14.4	10.0	10	0.340	5/16	3/16	10.0	10	0.560	9/16	1.06	1 1/4	13 1/8			49	893	23.1	272	54.6	4.35	60.4	93.4	18.7	2.54	28.3	2.84	9.44	1.39	2070
	W10x45	13.3	10.1	10 1/8	0.350	3/8	3/16	8.02	8	0.620	5/8	1.12	1 5/8	13 1/8	13/16	5 1/2	45	647	22.5	248	49.1	4.32	54.9	53.4	13.3	2.01	20.3	2.27	9.48	1.51	1200
		x39	11.5	9.92	9 7/8	0.315	5/16	7.99	8	0.530	1/2	1.03	1 3/8	13 1/8	13/16		39	753	25.0	209	42.1	4.27	46.8	45.0	11.3	1.98	17.2	2.24	9.39	0.976	992
x33		9.71	9.73	9 1/4	0.290	5/16	7.96	8	0.435	7/16	0.935	1 1/8	13 1/8	13/16		33	915	27.1	171	35.0	4.19	38.6	36.6	9.20	1.94	14.0	2.20	9.30	0.583	791	
W10x30	8.84	10.5	10 1/2	0.300	5/16	3/16	5.81	5 1/4	0.510	1/2	0.810	1 1/8	13 1/8	8 1/4	2 1/4	30	570	29.5	170	32.4	4.38	36.6	16.7	5.75	1.37	8.84	1.60	10.0	0.622	414	
x26	7.61	10.3	10 3/8	0.260	1/4	1/8	5.77	5 1/4	0.440	7/16	0.740	1 1/8	13 1/8	13/16		26	656	34.0	144	27.9	4.35	31.3	14.1	4.89	1.36	7.50	1.58	9.86	0.402	345	
x22 ^c	6.49	10.2	10 1/8	0.240	1/4	1/8	5.75	5 1/4	0.360	3/8	0.660	1 1/8	13 1/8	13/16		22	799	36.9	118	23.2	4.27	26.0	11.4	3.97	1.33	6.10	1.55	9.84	0.239	275	

4. Steel Beam Analysis

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

-2-

-3-

W-section

W10X30

Fy

50 KSI

Span A

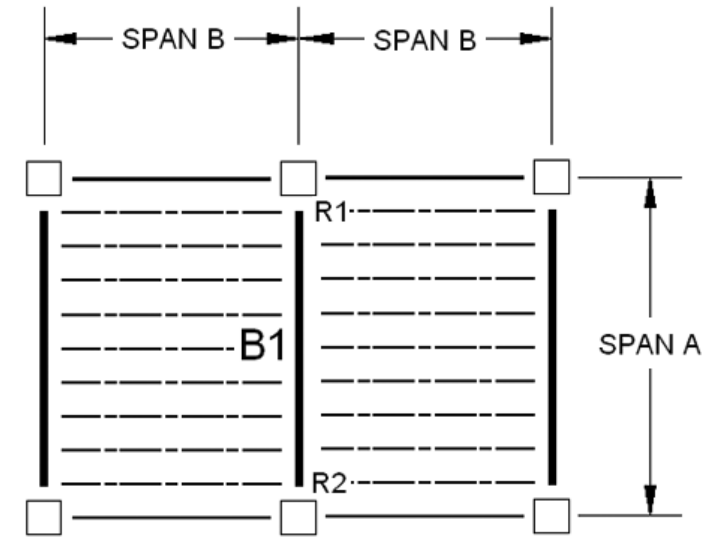
18 FT

Span B

12 FT

Floor DL

14 PSF



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

$$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI}$$

E_steel=29000 ksi
Ix = 170 in⁴ (AISCtable1-1)

Shape	Area, A	Depth, d	Web		Flange		Distance						Work- able Gage			
			Thickness, t _w	t _w	Width, b _f	Thickness, t _f	k	k _{out}	k _{in}	r ₁						
											I _x	S _x		I _y	S _y	r _x
in. ²	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in. ⁴	in. ⁶
W12x58	17.0	12.2	12	0.360	1/4	10.0	10	0.640	1/4	1.24	1 1/2	9 1/4	5 1/2	10.0	120	3570
>53	15.6	12.1	12	0.345	3/8	10.0	10	0.575	3/8	1.18	1 1/4	9 1/4	5 1/2	9.5	118	3160
W12x50	14.6	12.2	12 1/4	0.370	3/8	8.05	8 1/4	0.640	3/8	1.14	1 1/4	9 1/4	5 1/2	9.3	120	2790
>45	13.1	12.1	12	0.355	3/8	8.05	8	0.575	3/8	1.08	1 1/4	9 1/4	5 1/2	8.8	116	2170
>40	11.7	11.9	12	0.295	3/8	8.01	8	0.515	3/8	1.02	1 1/4	9 1/4	5 1/2	8.0	112	1660
W12x35	10.3	12.5	12 1/4	0.300	3/8	6.56	6 1/2	0.520	3/8	0.820	1 1/4	10 1/4	3 1/2	7.1	114	879
>20	8.79	12.3	12 1/4	0.260	1/4	6.52	6 1/2	0.440	3/8	0.740	1 1/4	10 1/4	3 1/2	6.5	109	720
>26	7.65	12.2	12 1/4	0.230	1/4	6.49	6 1/2	0.380	3/8	0.680	1 1/4	10 1/4	3 1/2	6.0	107	607
W12x22	6.48	12.3	12 1/4	0.260	1/4	4.03	4	0.425	3/8	0.725	1 1/4	10 1/4	2 1/4	4.0	104	457
>19	5.57	12.2	12 1/4	0.235	1/4	4.01	4	0.350	3/8	0.650	1 1/4	10 1/4	2 1/4	3.8	103	364
>16	4.71	12.0	12	0.220	1/4	3.99	4	0.265	3/8	0.565	1 1/4	10 1/4	2 1/4	3.5	102	269
>14	4.16	11.9	11 3/4	0.200	3/8	3.97	4	0.225	3/8	0.525	1 1/4	10 1/4	2 1/4	3.2	101	210
W10x112	32.9	11.4	11 1/4	0.755	3/4	10.4	10 1/4	1.25	1 1/4	1.75	1 1/4	7 1/2	5 1/2	11.2	417	6020
>100	29.3	11.1	11 1/4	0.680	1/4	10.3	10 1/4	1.12	1 1/4	1.62	1 1/4	7 1/2	5 1/2	10.6	402	5150
>88	26.0	10.8	10 7/8	0.605	3/8	10.3	10 1/4	0.990	1	1.49	1 1/4	7 1/2	5 1/2	9.9	391	4330
>77	22.7	10.6	10 7/8	0.530	1/2	10.2	10 1/4	0.870	3/4	1.37	1 1/4	7 1/2	5 1/2	9.3	381	3630
>68	19.9	10.4	10 7/8	0.470	1/2	10.1	10 1/4	0.770	3/4	1.27	1 1/4	7 1/2	5 1/2	8.8	369	3100
>60	17.7	10.2	10 7/8	0.420	3/4	10.1	10 1/4	0.680	3/4	1.18	1 1/4	7 1/2	5 1/2	8.3	358	2640
>54	15.8	10.1	10 7/8	0.370	3/4	10.0	10	0.615	3/4	1.12	1 1/4	7 1/2	5 1/2	7.9	348	2320
>49	14.4	10.0	10	0.340	3/4	10.0	10	0.560	3/4	1.06	1 1/4	7 1/2	5 1/2	7.4	338	2070
W10x45	13.3	10.1	10 7/8	0.350	3/8	8.02	8	0.620	3/4	1.12	1 1/4	7 1/2	5 1/2	6.8	327	1800
>39	11.5	9.92	9 7/8	0.315	3/8	7.98	8	0.530	3/4	1.03	1 1/4	7 1/2	5 1/2	6.3	316	1520
>33	9.71	9.73	9 7/8	0.290	3/8	7.96	8	0.435	3/4	0.930	1 1/4	7 1/2	5 1/2	5.9	306	1220
W10x30	8.84	10.5	10 7/8	0.300	3/8	5.81	5 1/2	0.510	3/4	0.810	1 1/4	7 1/2	5 1/2	5.0	295	992
>26	7.61	10.3	10 7/8	0.260	1/4	5.77	5 1/2	0.440	3/4	0.740	1 1/4	7 1/2	5 1/2	4.6	284	825
>22	6.49	10.2	10 7/8	0.240	1/4	5.75	5 1/2	0.360	3/4	0.660	1 1/4	7 1/2	5 1/2	4.2	274	692

Steel I-Beam WT	Compact Section Criteria		Axis X-X				Axis Y-Y				r _x	r _y	Torsional Properties		
	h _x	h _y	I _x	S _x	r _x	Z _x	I _y	S _y	r _y	Z _y			J	C _w	
lb/ft	in.	in.	in. ⁴	in. ³	in.	in. ³	in. ³	in.	in. ³	in.	in. ³	in.	in.	in. ⁴	in. ⁶
58	7.82	27.0	475	76.0	5.28	86.4	107	21.4	2.51	32.5	28.1	11.6	2.10	3570	1000
59	8.69	28.1	425	70.6	5.23	77.9	95.9	19.2	2.48	29.1	27.9	11.5	1.58	3160	800
60	9.31	28.9	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	22.5	11.6	1.71	2890	600
45	7.00	29.6	348	57.7	5.13	64.2	50.0	12.4	1.95	19.0	22.3	11.5	1.26	2650	450
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	22.1	11.4	0.906	2440	350
35	6.31	36.2	285	45.6	5.25	51.2	24.5	7.47	1.54	11.5	17.9	12.0	0.741	1879	250
30	7.41	41.8	238	38.6	5.21	43.1	20.3	6.24	1.52	9.56	17.7	11.9	0.457	1720	200
26	6.54	47.2	204	33.4	5.17	37.2	17.3	5.34	1.51	8.17	17.5	11.8	0.300	1607	150
22	4.74	41.8	156	25.4	4.91	29.3	4.06	2.31	0.949	3.66	10.4	11.9	0.263	164	100
19	5.72	46.2	130	21.3	4.82	24.7	3.76	1.88	0.822	2.98	10.2	11.9	0.180	131	75
16	7.53	49.4	103	17.1	4.67	20.1	2.82	1.41	0.773	2.26	0.983	11.7	0.103	96.9	45
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	30
112	4.17	10.4	716	126	4.66	147	236	45.3	2.68	69.2	3.08	10.2	15.1	6020	1000
100	4.62	11.6	623	112	4.60	130	207	40.0	2.65	61.0	3.04	10.0	10.9	5150	800
88	5.18	13.0	534	96.5	4.54	113	179	34.8	2.63	53.1	2.99	9.81	7.53	4330	600
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	450
68	6.38	16.7	394	75.7	4.44	85.3	134	26.4	2.59	40.1	2.92	9.63	3.56	3100	350
60	7.41	18.7	341	66.7	4.39	74.6	116	23.0	2.57	35.0	2.89	9.52	2.46	2640	250
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	200
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	150
45	6.47	22.5	248	49.1	4.32	54.9	53.4	13.3	2.01	20.3	2.27	9.49	1.51	1200	100
39	7.53	25.0	209	42.1	4.27	46.8	45.0	11.3	1.98	17.2	2.24	9.39	0.976	992	75
33	9.15	27.1	171	35.0	4.19	38.6	36.6	9.30	1.94	14.0	2.20	9.30	0.593	791	50
30	7.50	29.5	170	32.4	4.36	36.6	16.7	5.75	1.37	8.84	1.60	10.0	0.622	414	30
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	20
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	10

Deflection = 5 * (w_DL+ w_LL) * SpanA⁴ / (384 * E* I)
= 5 * [(0.198+1.97) * 1000 * $\frac{1}{12}$] * (18*12)⁴ / (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, 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



























Resources





2022 Lectures 



Canvas 

Date	Lectures	Video w/Quiz	Video	Slides	Notes
Jan 10	Course Intro				
Jan 12	Wood Properties		 		
Jan 17	Wood Beam Analysis				
Jan 22	Wood Beam Design				
Jan 24	Wood Column Analysis				
Jan 29	Wood Column Design				
Jan 29	Tower Project				
Jan 31	Wood - CLT				
Feb 5	Properties of Steel				
Feb 7	Steel Beam Analysis				


Tower Project Brief 2024 


Prelim Report Guidelines 2024 

Final Report Guidelines 2024 


Score Sheet 2024 

Study of Tower Types 

Example Reports 

Dr. Frame Software (download) 

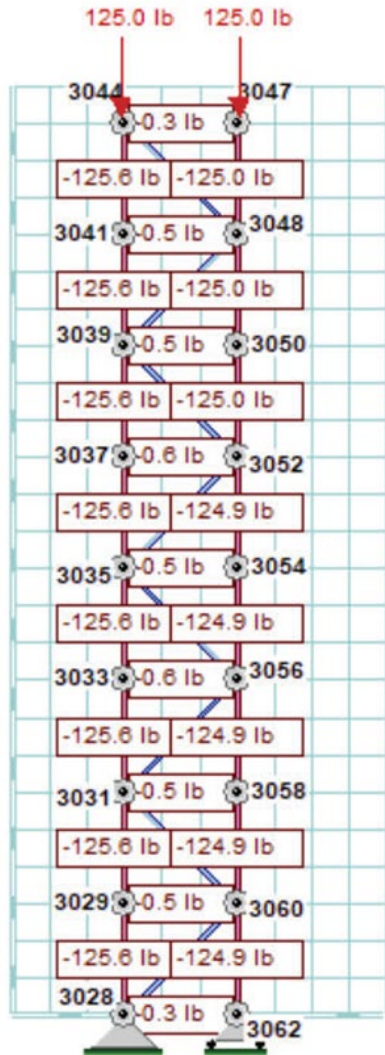
Dr. Frame Tutorials   

STAAD example 

Videos of Old Tower Tests 

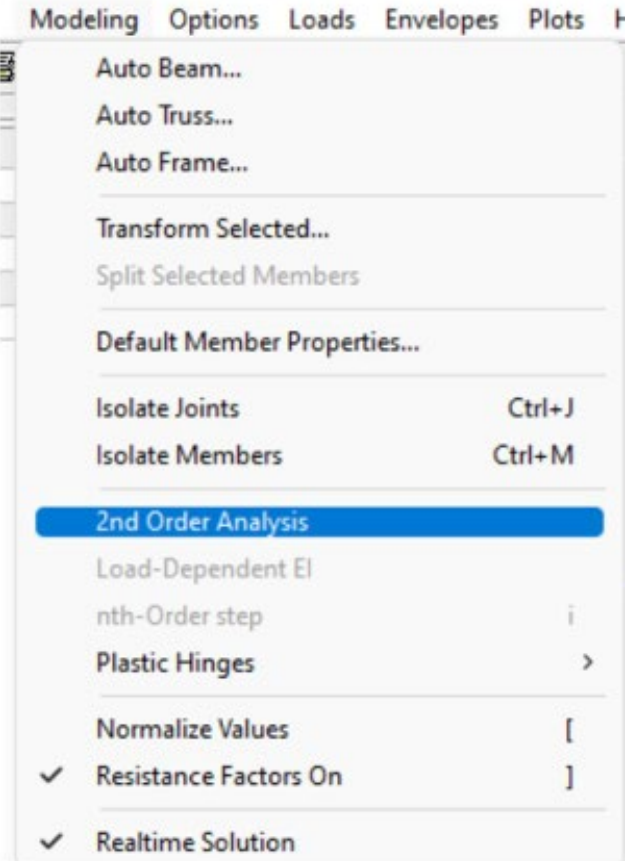
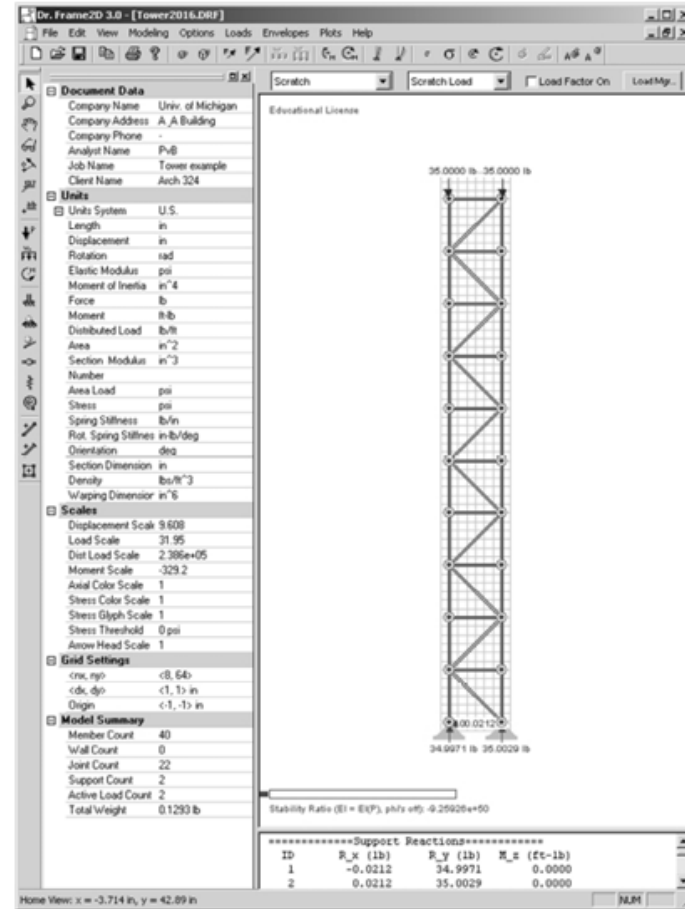
Tower Project

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



Tower Project

Resources

Analysis

Use NDS approach

Find load P and stress F'_c for each member

Use 1.0 for all factors except C_p

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_u) 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 f_c/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).

$$C_p = \frac{1 + (F_{ce}/F'_c)}{2c} - \sqrt{\left[\frac{1 + (F_{ce}/F'_c)}{2c} \right]^2 - \frac{F_{ce}/F'_c}{c}} \quad (3.7-1)$$

where:

F'_c = reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_p , (see 2.3), psi

$$F_{ce} = \frac{0.822 E_{min}}{(l_e/d)^2}$$

$c = 0.8$ for sawn lumber

$c = 0.85$ for round timber poles and piles

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

Analysis

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

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 *

Capacity

$$P = F'_c A$$

Design

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

* www.matweb.com
** tested by PvB

PREDICATE CAPACITY

1. Vertical Member Buckling Capacity:

$$\text{If } K = 1 \text{ then } \frac{L_e}{d} = \frac{L \cdot K}{d} = \frac{(6)' \cdot 1}{0.25'} = 24 < 50$$

$$F_{ce} = \frac{0.822 E_{min}}{(L_e/d)^2} = \frac{0.822 \times 1650000}{24^2} = 2355 \text{ PSI}$$

$$F_c^* = F_c = 4745 \text{ PSI}, \frac{F_{ce}}{F_c^*} = 0.496$$

$$C_p = \frac{1 + (F_{ce}/F'_c)}{2c} - \sqrt{\left[\frac{1 + (F_{ce}/F'_c)}{2c} \right]^2 - \frac{F_{ce}/F'_c}{c}} = \frac{1 + 0.496}{2 \times 0.8} - \sqrt{\left[\frac{1 + 0.496}{2 \times 0.8} \right]^2 - \frac{0.496}{0.8}} = 0.43$$

$$F'_c = F_c \cdot (C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_p) = 4745 \times 0.43 = 2040 \text{ PSI}$$

$$P = F'_c \cdot A = 2040 \times 0.25^2 = 127 \#$$

$P_{cr_members}$

2. Buckling Capacity of the Tower as a whole:

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

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

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

$$P_{cr} = \frac{\pi^2 EI}{(Kl/r)^2} = \frac{\pi^2 \times 2.07 \times 1650000}{(16.67)^2} = 121306 \#$$

$$\text{Each column: } \frac{P_{cr}}{4} = 30326.5 \#$$

P_{cr_tower}

3. Crushing Capacity of Vertical Members:

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

$P_{compression \text{ max}}$

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.

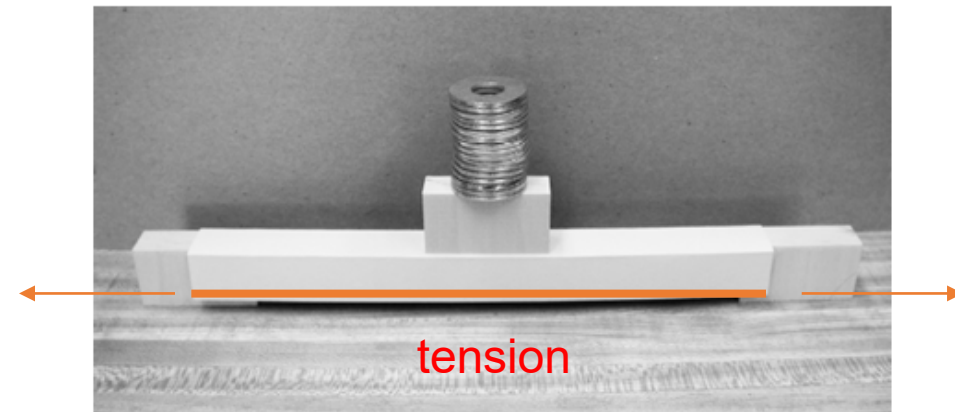
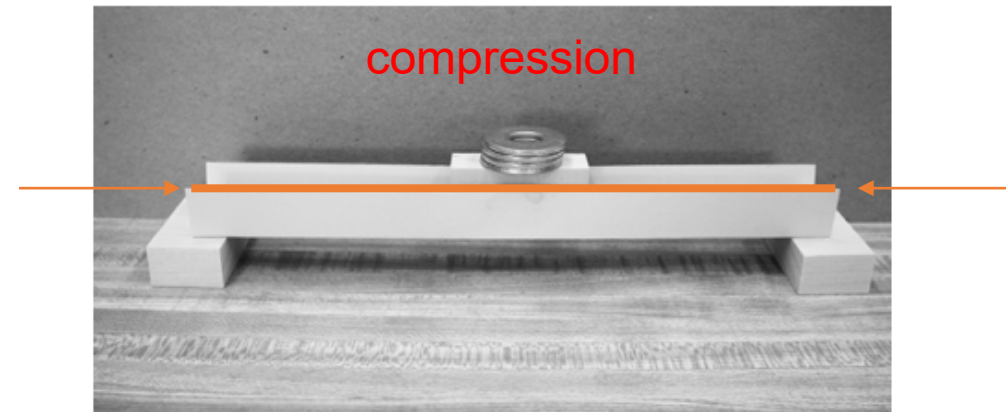
Goals

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!

