Steel Beam Analysis 2/9

HW - Steel Beam Analysis

Tower Project

Lab – Steel Beams

Structure II
Section 004

Yifan Ma
yifanma@umich.edu

Happy Lunar Dragon Year!

HW - Steel Beam Analysis

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-

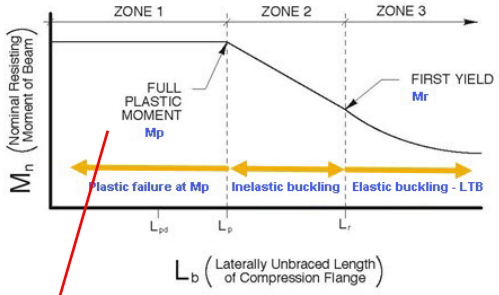
W-section	W10X30
Fy	50 KSI
Span A	18 FT
Span B	12 FT
Floor DL	14 PSF

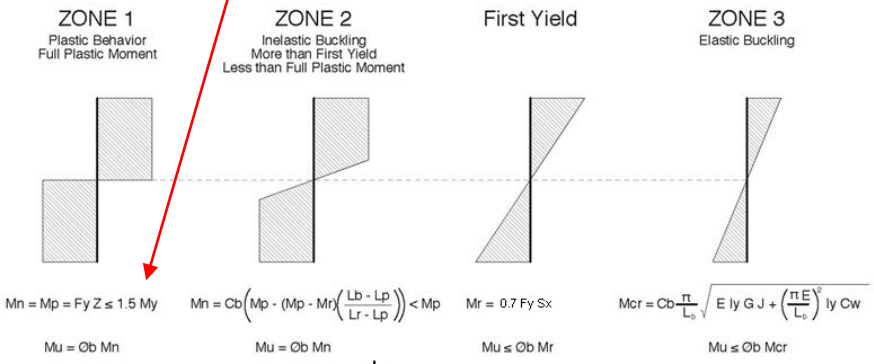
Given:

beam size (yield stress)
bracing type
load

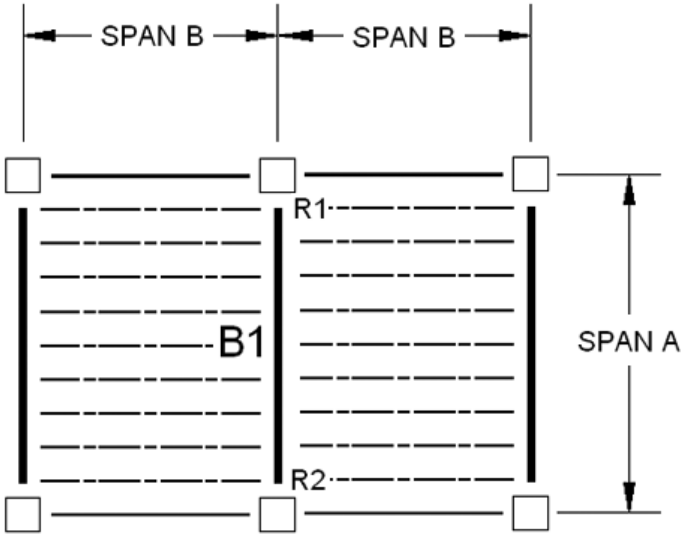
Goal:

load capacity?





$\phi_b = 0.90$ Safety Factor



1. Determine the unbraced length of the compression flange (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 = ϕ_b Mn

Given:
Lb < Lp

Plastic Behavior (zone 1)

Maximum Moment:
Mn = Mp = Fy Zx

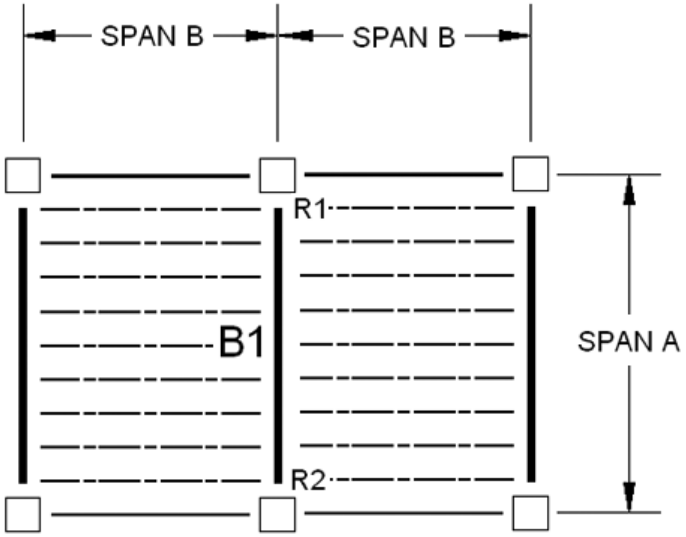
Find the Plastic Modulus (Zx) for the given section from the AISC table 1-1

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).

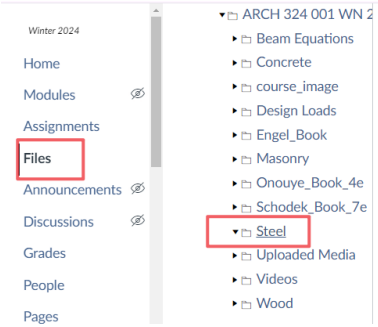
DATASET: 1 -2- -3-

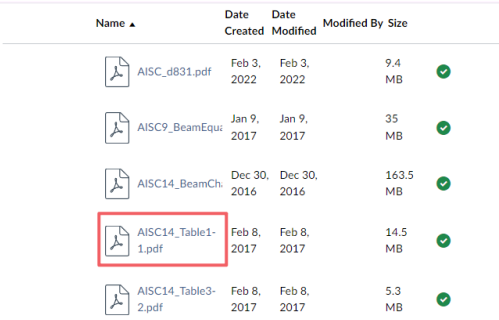
W-section	W10X30
Fy	50 KSI
Span A	18 FT
Span B	12 FT
Floor DL	14 PSF



1. The plastic modulus of the section, Zx = 36.6 in³

AISC table 1-1





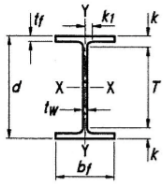


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W12×58	17.0	12.2	12¼	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1½	15/16	9¼	5½
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1⅜	15/16	9¼	5½
W12×50	14.6	12.2	12¼	0.370	3/8	3/16	8.08	8⅛	0.640	5/8	1.14	1½	15/16	9¼	5½
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1⅜	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	½	1.02	1⅜	⅞	↓	↓
W12×35[c]	10.3	12.5	12½	0.300	5/16	3/16	6.56	6½	0.520	½	0.820	1 3/16	¾	10⅛	3½
×30[c]	8.79	12.3	12⅜	0.260	¼	⅛	6.52	6½	0.440	7/16	0.740	1⅛	¾	↓	↓
×26[c]	7.65	12.2	12¼	0.230	¼	⅛	6.49	6½	0.380	⅜	0.680	1 1/16	¾	↓	↓
W12×22[c]	6.48	12.3	12¼	0.260	¼	⅛	4.03	4	0.425	7/16	0.725	15/16	⅝	10⅜	2¼[g]
×19[c]	5.57	12.2	12⅛	0.235	¼	⅛	4.01	4	0.350	⅜	0.650	⅞	9/16	↓	↓
×16[c]	4.71	12.0	12	0.220	¼	⅛	3.99	4	0.265	¼	0.565	13/16	9/16	↓	↓
×14[c,v]	4.16	11.9	11⅞	0.200	3/16	⅛	3.97	4	0.225	¼	0.525	¾	9/16	↓	↓
W10×112	32.9	11.4	11⅜	0.755	¾	⅜	10.4	10⅜	1.25	1¼	1.75	1 15/16	1	7½	5½
×100	29.3	11.1	11⅛	0.680	11/16	⅜	10.3	10⅜	1.12	1⅛	1.62	1 13/16	1	↓	↓
×88	26.0	10.8	10⅞	0.605	⅝	5/16	10.3	10¼	0.990	1	1.49	1 11/16	15/16	↓	↓
×77	22.7	10.6	10⅝	0.530	½	¼	10.2	10¼	0.870	⅞	1.37	1 9/16	⅞	↓	↓
×68	19.9	10.4	10⅜	0.470	½	¼	10.1	10⅛	0.770	¾	1.27	1 7/16	⅞	↓	↓
×60	17.7	10.2	10¼	0.420	7/16	¼	10.1	10⅛	0.680	11/16	1.18	1⅜	13/16	↓	↓
×54	15.8	10.1	10⅛	0.370	⅜	3/16	10.0	10	0.615	⅝	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	1¼	13/16	↓	↓
W10×45	13.3	10.1	10⅛	0.350	⅜	3/16	8.02	8	0.620	⅝	1.12	1 5/16	13/16	7½	5½
×39	11.5	9.92	9⅞	0.315	5/16	3/16	7.99	8	0.530	½	1.03	1 3/16	13/16	↓	↓
×33	9.71	9.73	9¾	0.290	5/16	3/16	7.96	8	0.435	7/16	0.935	1⅛	¾	↓	↓
W10×30	8.84	10.5	10½	0.300	5/16	3/16	5.81	5¾	0.510	½	0.810	1⅛	11/16	8¼	2¾[g]
×26	7.61	10.3	10⅜	0.260	¼	⅛	5.77	5¾	0.440	7/16	0.740	1 1/16	11/16	↓	↓
×22[c]	6.49	10.2	10⅛	0.240	¼	⅛	5.75	5¾	0.360	⅜	0.660	15/16	⅝	↓	↓

Table 1-1 (continued)
W-Shapes
Properties

W12-W10

Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	Torsional Properties J	Torsional Properties C_w
lb/ft			in.⁴	in.³	in.	in.³	in.⁴	in.³	in.	in.³	in.	in.	in.⁴	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, 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

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

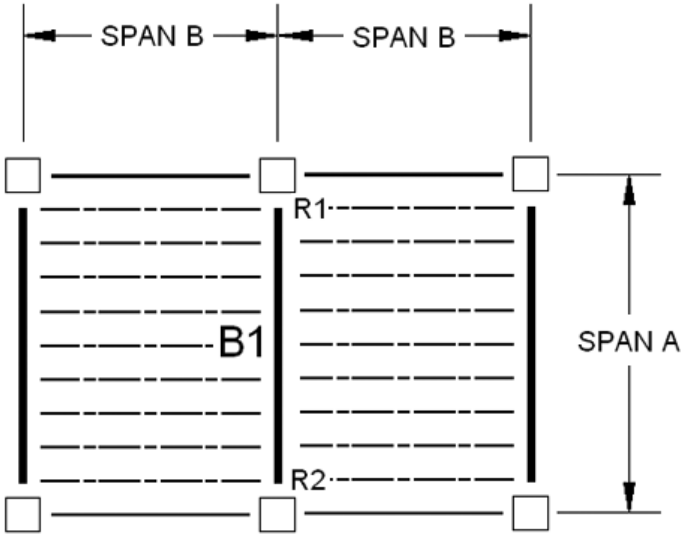
4. The factored design moment, Mu

Mu = ϕ *Mn = 1647/12 = 137.25 k-ft

5. The total factored design load, wu

Mu = wu*SpanA² /8 = 137.25 k-ft

wu= 8 * Mu / SpanA²
= 8*137.25/18²
= 3.39 klf



Load & Resistance Factored Design (LRFD)

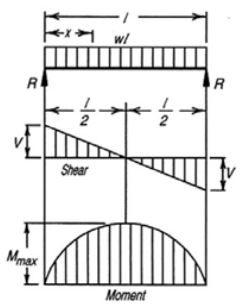
- Use loads with safety factor γ
- Use factor on ultimate strength ϕ

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

$$P_{load} \le P_{resisting}$$

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

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) . . . . . $= \frac{5wl^4}{384EI}$

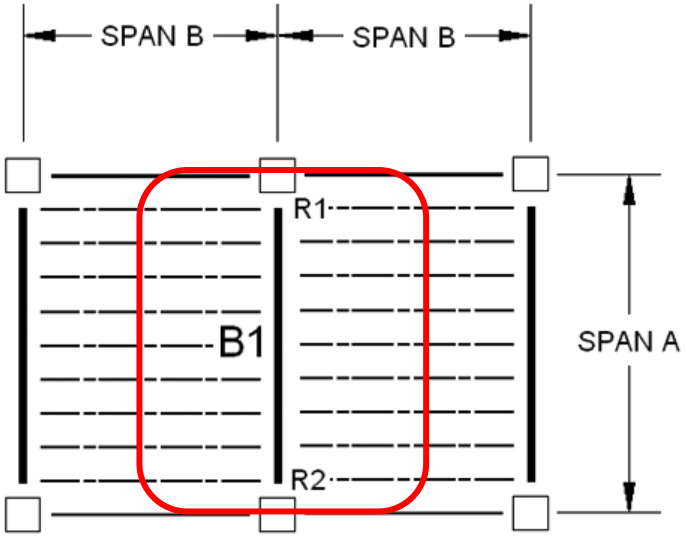
Δ_x . . . . . $= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

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



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

LRFD Analysis

Load & Resistance Factored Design (LRFD)

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

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

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

2.3 LOAD COMBINATIONS FOR STRENGTH DESIGN

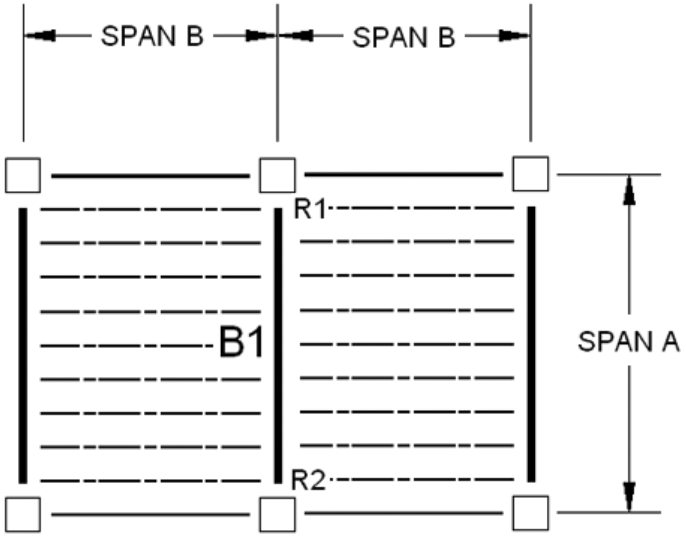
1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $0.9D + 1.0W$

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-

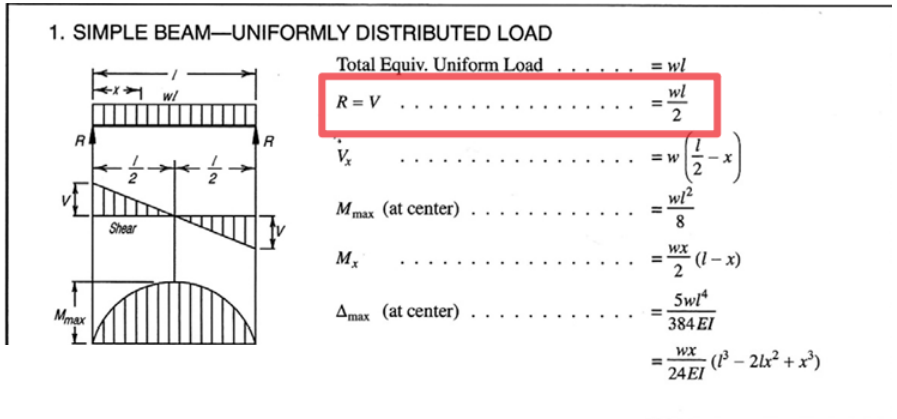
W-section	W10X30
Fy	50 KSI
Span A	18 FT
Span B	12 FT
Floor DL	14 PSF



Check the shear force Vu<=øVn

11. The maximum factored design beam shear force, Vu_max

Vu_max= wu* SpanA/2
= 3.39*18/2
= 30.51 kip



12. The Web Area, Aw

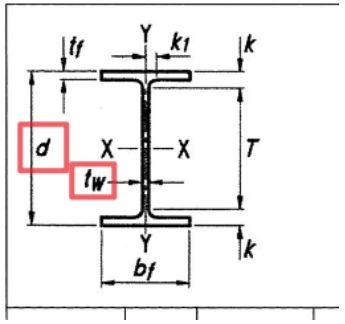
Aw = d * tw = 10.5*0.3 = 3.15 in²

Design for Shear

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

$F_v = V / A_w$

$A_w = d * t_w$



Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Distance Workable Gage
	in.²	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	15/16	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	15/16	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	15/16	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	15/16	↓	↓
×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 3/16	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[g]
×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	5/16	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	11/16	8 1/4	2 3/4[g]
×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	11/16	↓	↓
×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	↓	↓

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

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

Design for Shear

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

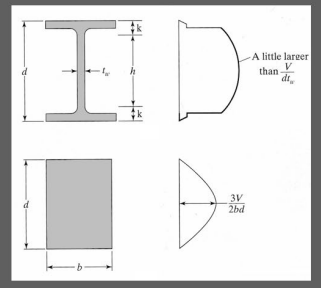
$F_v = V / A_w$

$A_w = d * t_w$

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 $\frac{h}{t_w} \le 2.45\sqrt{E/F_y} = 59$ (for 50 ksi steel)

then: $V_n = 0.6\ F_y\ A_w$

Zone 2:

INELASTIC WEB BUCKLING

if $2.45\sqrt{E/F_y} < \frac{h}{t_w} \le 3.07\sqrt{E/F_y} = 74$ (for 50 ksi steel)

then: $V_n = 0.6\ F_y\ A_w\ (2.45\sqrt{E/F}\ ) / \frac{h}{t_w}$

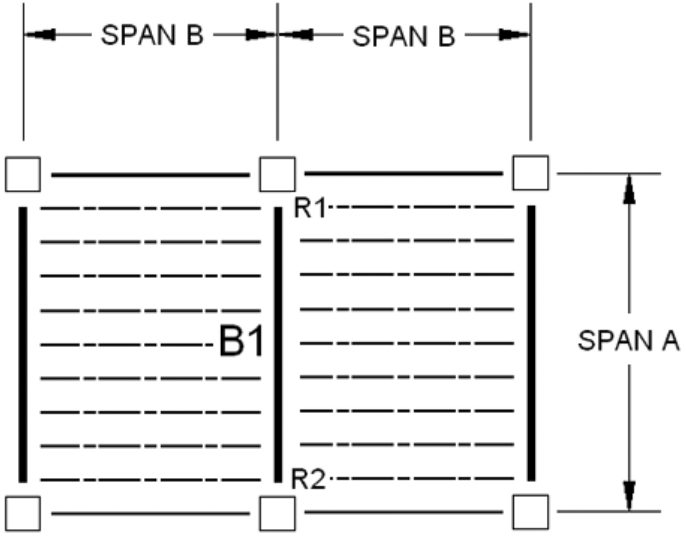
Zone 3:

ELASTIC WEB BUCKLING

if $3.07\sqrt{E/F_y} < \frac{h}{t_w} \le 260$

then: $V_n = A_w \left[\frac{4.25\ E}{\left(\frac{h}{t_w}\right)^2}\right]$

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 < ϕ *Vn Safe!

Shape	Area, A	Depth, d	Web Thickness, t_w	$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	k k_{det}	k_1	T	Workable Gage
	in.^{2}	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	15/16	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	15/16	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	15/16	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	15/16	↓	↓
×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 3/16	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[g]
×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	5/16	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	11/16	8 1/4	2 3/4[g]
×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	11/16	↓	↓
×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	↓	↓

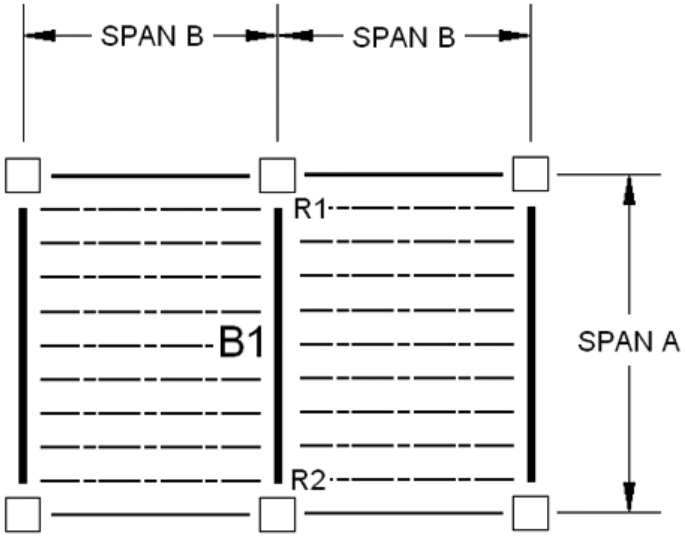
Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	Axis X-X I	S	r	Z	Axis Y-Y I	S	r	Z	r_{ts}	h_o	Torsional Properties J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
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, 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

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

E_steel=29000 ksi

Ix = 170 in^4 (AISCtable1-1)

Shape	Area, A	Depth, d	Web Thickness, t_w	Web $\frac{t_w}{2}$	Flange Width, b_f	Flange Thickness, t_f	Distance k, k_{des}	Distance k, k_{act}	Distance k_1	Distance T	Workable Gage
	in.^{2}	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	15/16	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 1/8	15/16	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	15/16	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 1/8	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 1/8	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 3/16	3/4	10 1/8	3 1/2
×30[c]	8.79	12.3	12 5/8	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 13/16	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	5/16	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 5/8	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	3/4	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 7/16	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[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	11/16	↓	↓
×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	↓	↓

Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	Torsional Properties J	Torsional Properties C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	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	9.48	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

Deflection = 5 * (w_DL+ w_LL) * SpanA^4 / (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

Architecture Project

Contact | Schedule | Lectures | Recitation | Towers 1 | Towers2 | Problems

Tower Project Brief 2024

Prelim Report Guidlines 2024

Final Report Guidlines 2024

Score Sheet 2024

Study of Tower Types

Example Reports

Dr. Frame Software (download)

Dr. Frame Tutorials

STAAD example

Videos of Old Tower Tests

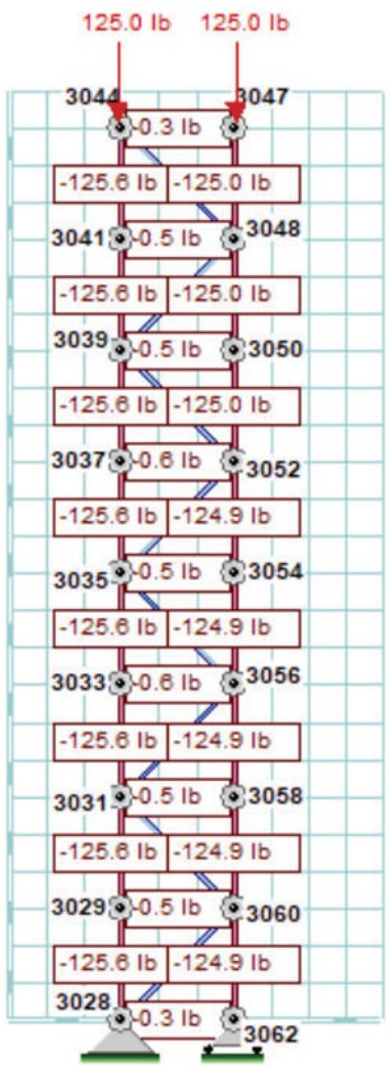
2022 Lectures

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				

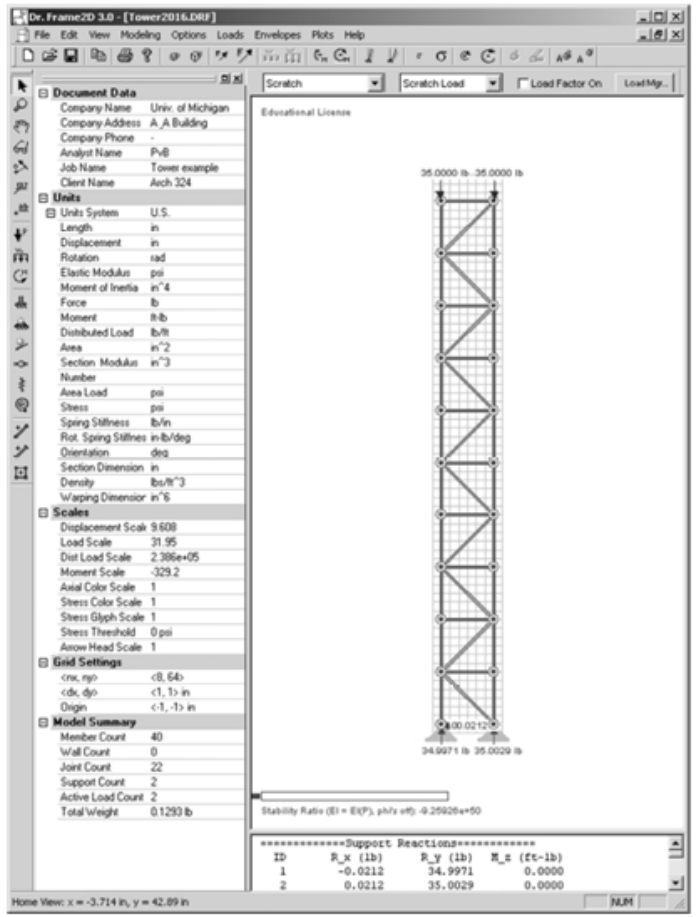
Tower Project

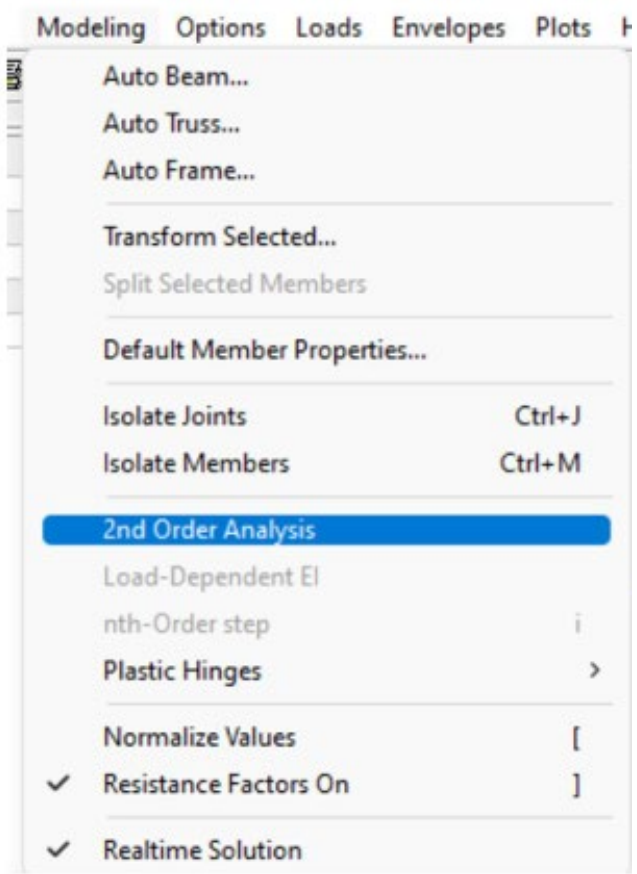
Dr. Frame



Dr. Frame 2D

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





Tower Projec

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_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).

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

where:

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

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

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

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 *

* www.matweb.com
** tested by PvB

PREDICATE CAPACITY

1. Vertical Member Buckling Capacity:

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

$$Fce = \frac{0.822\ Emin}{(Le/d)} = \frac{0.822 \times 1650000}{24^2} = 2355\ PSI$$

$$Fc^* = Fc = 4745\ PSI, \frac{Fce}{Fc^*} = 0.496$$

$$Cp = \frac{1+(\frac{Fce}{F^*c})}{2c} - \sqrt{\left[\frac{1+(\frac{Fce}{F^*c})}{2c}\right]^2 - \frac{(\frac{Fce}{F^*c})}{c}} = \frac{1+0.496}{2\times0.8} - \sqrt{\left[\frac{1+0.496}{2\times0.8}\right]^2 - \frac{0.496}{0.8}} = 0.43$$

$$F'c = Fc.(CD.Cm.Ct.CF.Ci.Cp) = 4745 \times 0.43 = 2040PSI$$

$$P = F'c.A = 2040 \times 0.25^2 = 127\#$$

Pcr_members

2. Buckling Capacity of the Tower as a whole:

$$I = \Sigma I + \Sigma Ad^2 = 4 \times \frac{0.25\times0.25^3}{12} + 4 \times (0.25 \times 0.5) \times (3 - 0.125)^2 = 2.07\ in^4$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2.07}{0.25^2\times4}} = 2.88''$$

$$\frac{Kl}{r} = \frac{1\times48}{2.88} = 16.67$$

$$Pcr = \frac{\pi^2 IE}{(\frac{Kl}{r})^2} = \frac{\pi^2\times2.07\times1650000}{(16.67)^2} = 121306\#$$

Each column: $\frac{Pcr}{4} = 30326.5\#$

Pcr_tower

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.

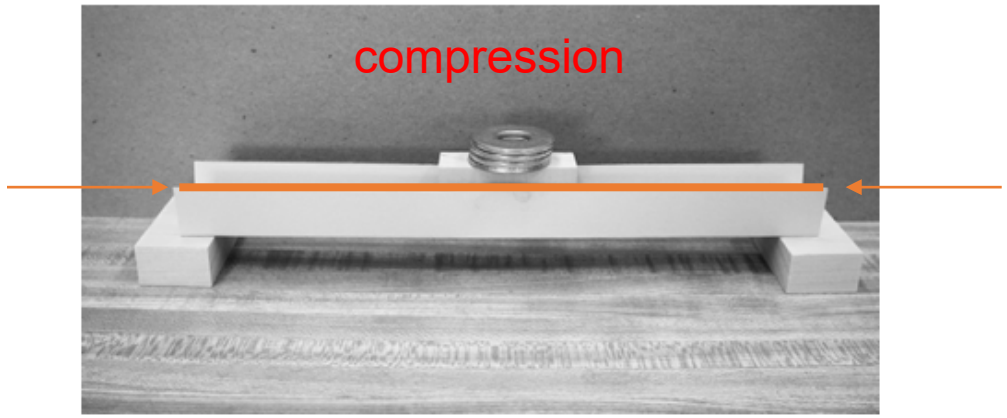
Goals

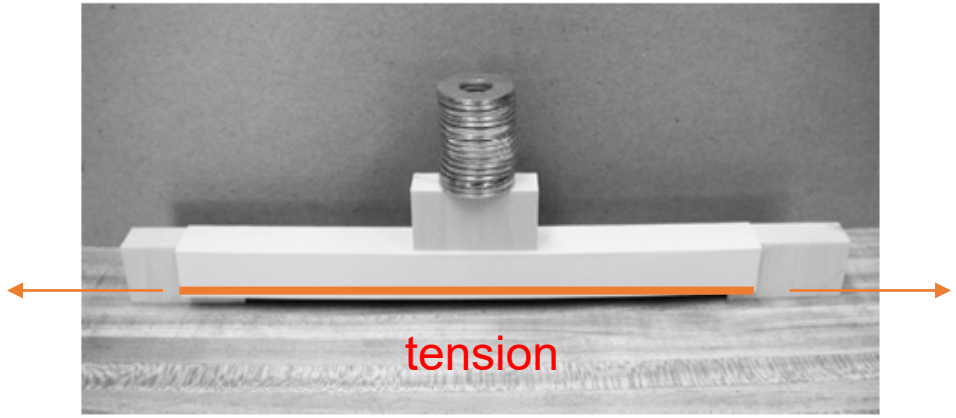
To observe the behavior of unbraced section edges in compression vs tension.

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?

yifanma@umich.edu

Thank You!