ARCHITECTURE 324
Structures II

Recitation 05

Sections 04&05

Instructor
Peter von Buelow

GSI
Alireza Fazel
Feb 14, 2025

Office Hours

→ Office Hours

→ Day: Fridays, 12:00 PM - 1:00 PM

→ Location Options:

- In-person meetings: [2223B]
- Virtual meetings via Zoom

Please make sure to sign up at least 24 hours in advance to allow for proper scheduling via this link:

https://docs.google.com/forms/d/e/1FAIpQLSdOb4gAc6SoCdsMAZP4zKrn3ecPyGt6dwVahVcOD3EqXGG-oA/viewform?usp=dialog

If the slots are fully booked or if you have a time conflict, please email me directly to find an alternative time (arfazel@umich.edu)

Contents

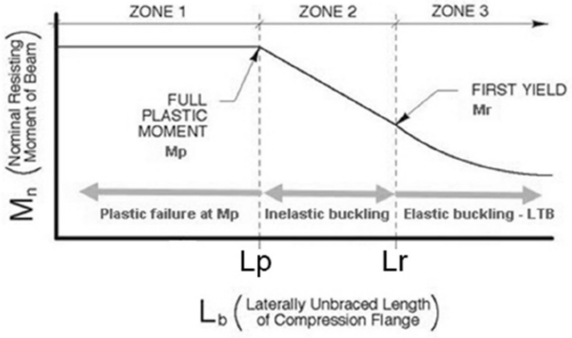
Steel Beam Analysis

Steel Beams by LRFD

Yield Stress Values

- A36 Carbon Steel $F_y = 36$ ksi
- A992 High Strength $F_y = 50$ ksi

Elastic Analysis for Bending



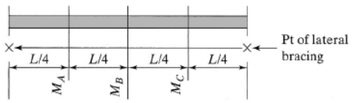
- **Plastic Behavior (zone 1)**
  $M_n = M_p = F_y Z < 1.5 M_y$
  - Braced against LTB ($L_b < L_p$)
- **Inelastic Buckling "Decreased" (zone 2)**
  $M_n = C_b(M_p-(M_p-M_r)[(L_b-L_p)/(L_r-L_p)] < M_p$
  - $L_p < L_b < L_r$
- **Elastic Buckling "Decreased Further" (zone 3)**
  $M_{cr} = C_b * \pi/L_b \sqrt{(E*I_y*G*J + (\pi*E/L_b)^2 * I_yC_w)}$
  - $L_b > L_r$

$L_p = 1.76\ r_y \sqrt{E/Fy}$

$M_p = F_y Z_x$

$M_r = 0.7\ F_y S_x$

C_b is LTB modification factor

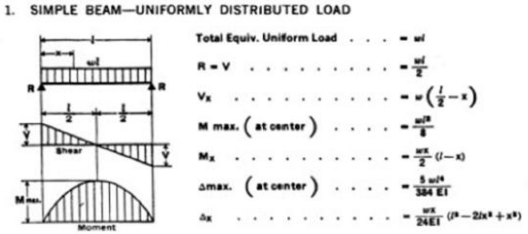


$$C_b = \frac{12.5\ \text{Mmax}}{2.5\ \text{Mmax} + 3\ \text{MA} + 4\text{MB} + 3\text{MC}}$$

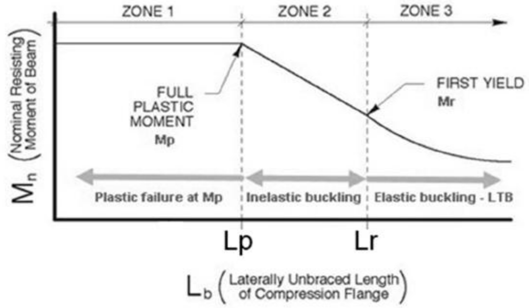
Procedure - Analysis of Steel Beams – **for Zone 1** $L_b < L_p$

Pass/Fail

Given: yield stress, steel section, loading, bracing (L_b)
Find: pass/fail of section



1. Calculate the factored design load w_u
   $w_u = 1.2w_{DL} + 1.6w_{LL}$
2. Determine the design moment M_u.
   M_u will be the maximum beam moment using the factored loads
3. Insure that $L_b < L_p$ (zone 1)
   $L_p = 1.76\ r_y \sqrt{E/Fy}$
4. Determine the nominal moment, Mn
   $M_n = F_y Z_x$ (look up Z_x for section)
5. Factor the nominal moment
   $\phi M_n = 0.90\ M_n$
6. Check that $M_u < \phi M_n$
7. Check shear
8. Check deflection



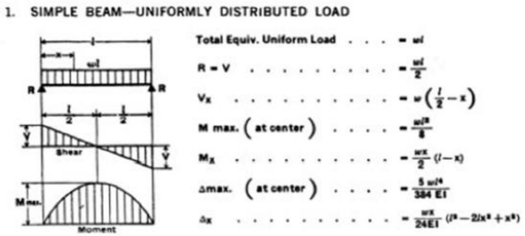
Steel Beam Analysis

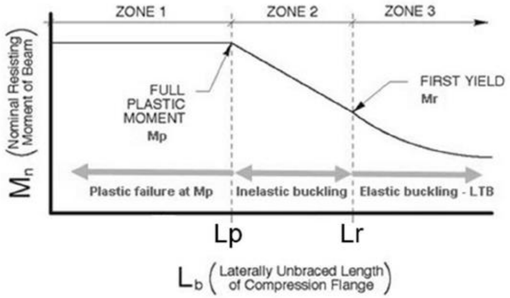
Procedure - Analysis of Steel Beam - **Capacity**

Given: yield stress, steel section, bracing

Find: moment or load capacity

1. Determine the unbraced length of the compression flange (L_b).
2. Find the L_p and L_r values from the AISC Z_x Table 3-2
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.
5. Calculate load based on maximum moment. $M_u = \phi_b \ M_n$



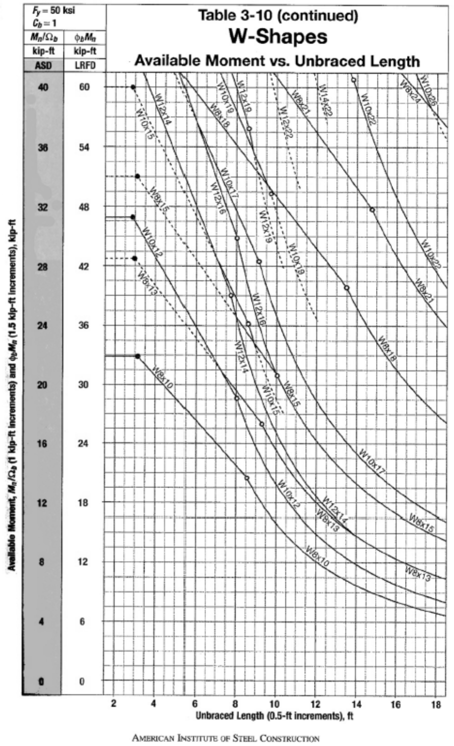


Steel Beams by LRFD

Moment Capacity with L_b Graphs

Analysis for Bending

- Plastic Behavior (zone 1)
  $M_n = M_p$
  Braced against LTB ($L_b < L_p$)
- Inelastic Buckling "Decreased" (zone 2)
  $M_n < M_p$
  $L_p < L_b < L_r$
- Elastic Buckling "Decreased Further" (zone 3)
  $M_n = M_{cr}$
  $L_b > L_r$



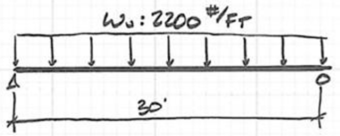
AISC 15th ed.

Steel Beam Design

Design of Steel Beam – Procedure (zone 1)

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

GIVEN: $F_y = 50$ KSI
FULLY BRACED



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

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

$M_N = M_U / \phi_b = \frac{247.5\ \text{KFT}}{0.90} = 275$ KFT

Example - Design of Steel Beam
Check Shear

5. Determine if $h/t_w < 59$
   (case 1, most common)
6. Determine A_w:
   $A_w = d * t_w = 16.0" \times 0.305"$
   $A_w = 4.88\ in^2$

FIND h/t_w FROM TABLES FOR A
W16x40
$h/t_w = 46.5 < 59$

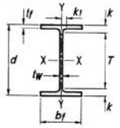


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$t_w/2$	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.
W16×100	29.4	17.0	17	0.585	9/16	5/16	10.4	10 3/8	0.985	1	1.39	1 7/8	1 1/8	13 1/4	5 1/2
×89	26.2	16.8	16 3/4	0.525	1/2	1/4	10.4	10 3/8	0.875	7/8	1.28	1 3/4	1 1/16	↓	↓
×77	22.6	16.5	16 1/2	0.455	7/16	1/4	10.3	10 1/4	0.760	3/4	1.16	1 5/8	1 1/16	↓	↓
×67[c]	19.6	16.3	16 3/8	0.395	3/8	3/16	10.2	10 1/4	0.665	11/16	1.07	1 9/16	1	↓	↓
W16×57	16.8	16.4	16 3/8	0.430	7/16	1/4	7.12	7 1/8	0.715	11/16	1.12	1 3/8	7/8	13 5/8	3 1/2[g]
×50[c]	14.7	16.3	16 1/4	0.380	3/8	3/16	7.07	7 1/8	0.630	5/8	1.03	1 5/16	13/16	↓	↓
×45[c]	13.3	16.1	16 1/8	0.345	3/8	3/16	7.04	7	0.565	9/16	0.967	1 1/4	13/16	↓	↓
×40[c]	11.8	$\boxed{16.0}$	16	$\boxed{0.305}$	5/16	3/16	7.00	7	0.505	1/2	0.907	1 3/16	13/16	↓	↓
×36[c]	10.6	15.9	15 7/8	0.295	5/16	3/16	6.99	7	0.430	7/16	0.832	1 1/8	3/4	↓	↓

Table 1-1 (continued)
W-Shapes
Properties
W16-W14

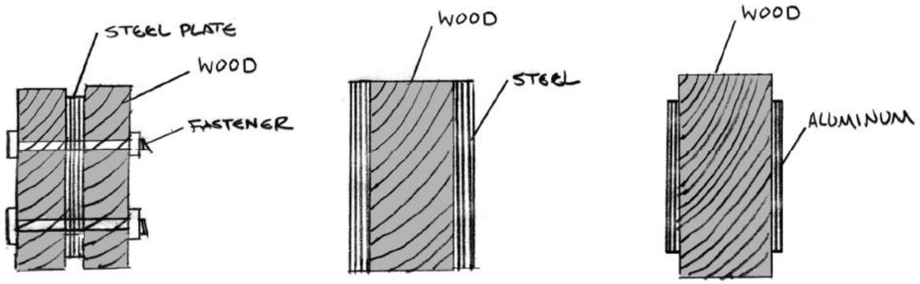
Nominal Wt.	Compact Section Criteria $b_f/2t_f$	h/t_w	Axis X-X I	S	r	Z	Axis Y-Y I	S	r	Z	r_{ts}	h_o	Torsional Properties J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
100	5.29	24.3	1490	175	7.10	198	186	35.7	2.51	54.9	2.92	16.0	7.73	11900
89	5.92	27.0	1300	155	7.05	175	163	31.4	2.49	48.1	2.88	15.9	5.45	10200
77	6.77	31.2	1110	134	7.00	150	138	26.9	2.47	41.1	2.85	15.7	3.57	8590
67	7.70	35.9	954	117	6.96	130	119	23.2	2.46	35.5	2.82	15.6	2.39	7300
57	4.98	33.0	758	92.2	6.72	105	43.1	12.1	1.60	18.9	1.92	15.7	2.22	2660
50	5.61	37.4	659	81.0	6.68	92.0	37.2	10.5	1.59	16.3	1.89	15.7	1.52	2270
45	6.23	41.1	586	72.7	6.65	82.3	32.8	9.34	1.57	14.5	1.87	15.5	1.11	1990
40	6.9[illegible]	$\boxed{46.5}$	518	64.7	6.63	73.0	28.9	8.25	1.57	12.7	1.86	15.5	0.794	1730
36	8.12	48.1	448	56.5	6.51	64.0	24.5	7.00	1.52	10.8	1.83	15.5	0.545	1460

Flitched Beams

Flitched Beams & Scab Plates

Advantages

- Compatible with the wood structure,
  i.e. can be nailed
- Easy to retrofit to existing structure
- Lighter weight than a steel section
- Stronger than wood alone
  - Less deep than wood alone
  - Allow longer spans
- The section can vary over the length of the span to optimize the member (e.g. scab plates)
- The wood stabilizes the thin steel plate

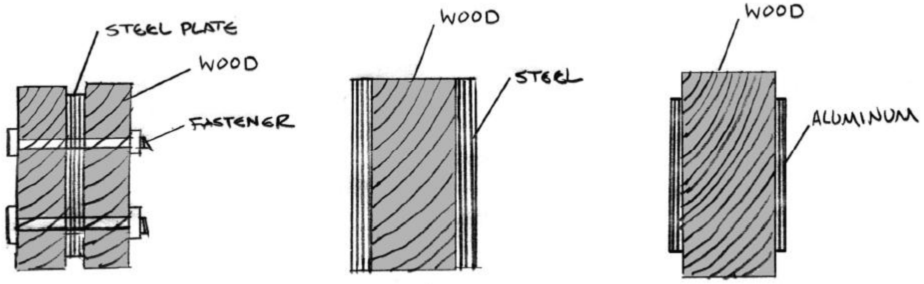


University of Michigan, TCAUP   Structures II   Slide 13 of 24

Flitched Beams & Scab Plates

Disadvantages

- More labor to make – expense. Flitched beams require shop fabrication or field bolting.
- Often replaced by Composite Lumber which is simply cut to length – less labor
  - Glulam
  - LVL
  - PSL
- Flitched Beams are generally heavier than Composite Lumber



University of Michigan, TCAUP   Structures II   Slide 14 of 24

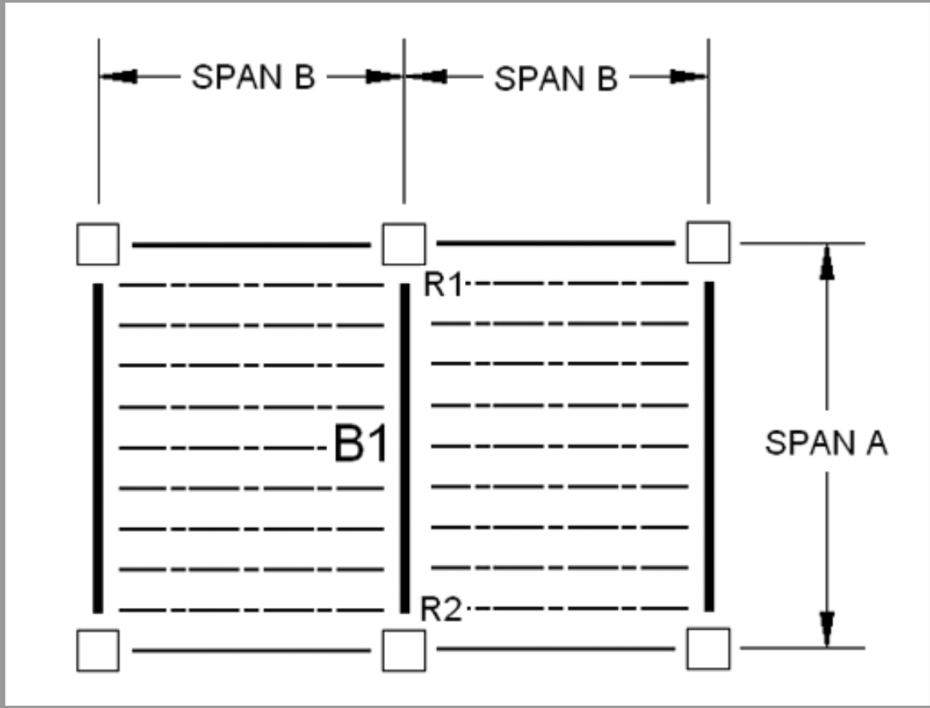
Problem Set 04

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	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF



Problem Set 04

#Q1: The plastic modulus of the section, Zx

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	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF

According to the table, for W14×61:

$$Z_X = 102\ IN^3$$

1–24 DIMENSIONS AND PROPERTIES

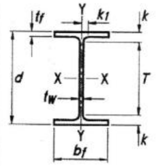


Table 1-1 (continued)
W-Shapes
Dimensions

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.
W14×132	38.8	14.7	14 5/8	0.645	5/8	5/16	14.7	14 3/4	1.03	1	1.63	2 5/16	1 9/16	10	5 1/2
×120	35.3	14.5	14 1/2	0.590	9/16	5/16	14.7	14 5/8	0.940	15/16	1.54	2 1/4	1 1/2		
×109	32.0	14.3	14 3/8	0.525	1/2	1/4	14.6	14 5/8	0.860	7/8	1.46	2 3/16	1 1/2		
×99[f]	29.1	14.2	14 1/8	0.485	1/2	1/4	14.6	14 5/8	0.780	3/4	1.38	2 1/16	1 7/16		
×90[f]	26.5	14.0	14	0.440	7/16	1/4	14.5	14 1/2	0.710	11/16	1.31	2	1 7/16		
W14×82	24.0	14.3	14 1/4	0.510	1/2	1/4	10.1	10 1/8	0.855	7/8	1.45	1 11/16	1 1/16	10 7/8	5 1/2
×74	21.8	14.2	14 1/8	0.450	7/16	1/4	10.1	10 1/8	0.785	13/16	1.38	1 5/8	1 1/16		
×68	20.0	14.0	14	0.415	7/16	1/4	10.0	10	0.720	3/4	1.31	1 9/16	1 1/16		
×61	17.9	13.9	13 7/8	0.375	3/8	3/16	10.0	10	0.645	5/8	1.24	1 1/2	1		
W14×53	15.6	13.9	13 7/8	0.370	3/8	3/16	8.06	8	0.660	11/16	1.25	1 1/2	1	10 7/8	5 1/2
×48	14.1	13.8	13 3/4	0.340	5/16	3/16	8.03	8	0.595	5/8	1.19	1 7/16	1		
×43[c]	12.6	13.7	13 5/8	0.305	5/16	3/16	8.00	8	0.530	1/2	1.12	1 3/8	1		
W14×38[c]	11.2	14.1	14 1/8	0.310	5/16	3/16	6.77	6 3/4	0.515	1/2	0.915	1 1/4	13/16	11 5/8	3 1/2[g]
×34[c]	10.0	14.0	14	0.285	5/16	3/16	6.75	6 3/4	0.455	7/16	0.855	1 3/16	3/4		3 1/2
×30[c]	8.85	13.8	13 7/8	0.270	1/4	1/8	6.73	6 3/4	0.385	3/8	0.785	1 1/8	3/4		3 1/2
W14×26[c]	7.69	13.9	13 7/8	0.255	1/4	1/8	5.03	5	0.420	7/16	0.820	1 1/8	3/4	11 5/8	2 3/4[g]
×22[c]	6.49	13.7	13 3/4	0.230	1/4	1/8	5.00	5	0.335	5/16	0.735	1 1/16	3/4	11 5/8	2 3/4[g]
W12×336[h]	98.9	16.8	16 7/8	1.78	1 3/4	7/8	13.4	13 3/8	2.96	2 15/16	3.55	3 7/8	1 11/16	9 1/8	5 1/2
×305[h]	89.5	16.3	16 3/8	1.63	1 5/8	13/16	13.2	13 1/4	2.71	2 11/16	3.30	3 5/8	1 5/8		
×279[h]	81.9	15.9	15 7/8	1.53	1 1/2	3/4	13.1	13 1/8	2.47	2 1/2	3.07	3 3/8	1 5/8		
×252[h]	74.1	15.4	15 3/8	1.40	1 3/8	11/16	13.0	13	2.25	2 1/4	2.85	3 1/8	1 1/2		
×230[h]	67.7	15.1	15	1.29	1 5/16	11/16	12.9	12 7/8	2.07	2 1/16	2.67	2 15/16	1 1/2		
×210	61.8	14.7	14 3/4	1.18	1 3/16	5/8	12.8	12 3/4	1.90	1 7/8	2.50	2 13/16	1 7/16		
×190	56.0	14.4	14 3/8	1.06	1 1/16	9/16	12.7	12 5/8	1.74	1 3/4	2.33	2 5/8	1 3/8		
×170	50.0	14.0	14	0.960	15/16	1/2	12.6	12 5/8	1.56	1 9/16	2.16	2 7/16	1 5/16		
×152	44.7	13.7	13 3/4	0.870	7/8	7/16	12.5	12 1/2	1.40	1 3/8	2.00	2 5/16	1 1/4		
×136	39.9	13.4	13 3/8	0.790	13/16	7/16	12.4	12 3/8	1.25	1 1/4	1.85	2 1/8	1 1/4		
×120	35.2	13.1	13 1/8	0.710	11/16	3/8	12.3	12 3/8	1.11	1 1/8	1.70	2	1 3/16		
×106	31.2	12.9	12 7/8	0.610	5/8	5/16	12.2	12 1/4	0.990	1	1.59	1 7/8	1 1/8		
×96	28.2	12.7	12 3/4	0.550	9/16	5/16	12.2	12 1/8	0.900	7/8	1.50	1 13/16	1 1/8		
×87	25.6	12.5	12 1/2	0.515	1/2	1/4	12.1	12 1/8	0.810	13/16	1.41	1 11/16	1 1/16		
×79	23.2	12.4	12 3/8	0.470	1/2	1/4	12.1	12 1/8	0.735	3/4	1.33	1 5/8	1 1/16		
×72	21.1	12.3	12 1/4	0.430	7/16	1/4	12.0	12	0.670	11/16	1.27	1 9/16	1 1/16		
×65[f]	19.1	12.1	12 1/8	0.390	3/8	3/16	12.0	12	0.605	5/8	1.20	1 1/2	1		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

DIMENSIONS AND PROPERTIES 1–25

Table 1-1 (continued)
W-Shapes
Properties

W14-W12

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}
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.7	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.6	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.10	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.4	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.83	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.3	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.2	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.2	1.05	1950
38	6.57	39.6	385	54.6	5.87	61.5	26.7	7.88	1.55	12.1	1.82	13.6	0.798	1230
34	7.41	43.1	340	48.6	5.83	54.6	23.3	6.91	1.53	10.6	1.80	13.5	0.569	1070
30	8.74	45.4	291	42.0	5.73	47.3	19.6	5.82	1.49	8.99	1.77	13.4	0.380	887
26	5.98	48.1	245	35.3	5.65	40.2	8.91	3.55	1.08	5.54	1.30	13.5	0.358	405
22	7.46	53.3	199	29.0	5.54	33.2	7.00	2.80	1.04	4.39	1.27	13.4	0.208	314
336	2.26	5.47	4060	483	6.41	603	1190	177	3.47	274	4.13	13.8	243	57000
305	2.45	5.98	3550	435	6.29	537	1050	159	3.42	244	4.05	13.6	185	48600
279	2.66	6.35	3110	393	6.16	481	937	143	3.38	220	4.00	13.4	143	42000
252	2.89	6.96	2720	353	6.06	428	828	127	3.34	196	3.93	13.2	108	35800
230	3.11	7.56	2420	321	5.97	386	742	115	3.31	177	3.87	13.0	83.8	31200
210	3.37	8.23	2140	292	5.89	348	664	104	3.28	159	3.81	12.8	64.7	27200
190	3.65	9.16	1890	263	5.82	311	589	93.0	3.25	143	3.77	12.7	48.8	23600
170	4.03	10.1	1650	235	5.74	275	517	82.3	3.22	126	3.70	12.4	35.6	20100
152	4.46	11.2	1430	209	5.66	243	454	72.8	3.19	111	3.66	12.3	25.8	17200
136	4.96	12.3	1240	186	5.58	214	398	64.2	3.16	98.0	3.61	12.2	18.5	14700
120	5.57	13.7	1070	163	5.51	186	345	56.0	3.13	85.4	3.56	12.0	12.9	12400
106	6.17	15.9	933	145	5.47	164	301	49.3	3.11	75.1	3.52	11.9	9.13	10700
96	6.76	17.7	833	131	5.44	147	270	44.4	3.09	67.5	3.49	11.8	6.85	9410
87	7.48	18.9	740	118	5.38	132	241	39.7	3.07	60.4	3.46	11.7	5.10	8270
79	8.22	20.7	662	107	5.34	119	216	35.8	3.05	54.3	3.43	11.7	3.84	7330
72	8.99	22.6	597	97.4	5.31	108	195	32.4	3.04	49.2	3.41	11.6	2.93	6540
65	9.92	24.9	533	87.9	5.28	96.8	174	29.1	3.02	44.1	3.38	11.5	2.18	5780

Problem Set 04

#Q2: The nominal bending moment, Mn

#Q3: The factored bending resistance, phi Mn

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	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF

$$\boldsymbol{M_n = F_y Z} = 50\ KSI \times 102\ IN^3$$
$$= \mathbf{5100\ K - IN}$$

$$\boldsymbol{\varphi M_n = 0.9\ M_n} = 0.9 \times 5100$$
$$= \mathbf{4590\ K - IN}$$

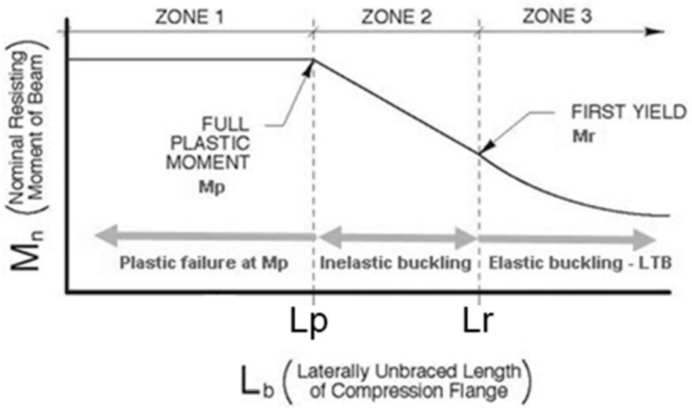
Steel Beams by LRFD

Yield Stress Values

- A36 Carbon Steel $F_y = 36$ ksi
- A992 High Strength $F_y = 50$ ksi

Elastic Analysis for Bending

- **Plastic Behavior (zone 1)**
  $M_n = M_p = F_y Z < 1.5 M_y$
  - Braced against LTB ($L_b < L_p$)



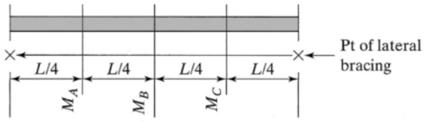
- **Inelastic Buckling "Decreased" (zone 2)**
  $M_n = C_b(M_p-(M_p-M_r)[(L_b-L_p)/(L_r-L_p)] < M_p$
  - $L_p < L_b < L_r$
- **Elastic Buckling "Decreased Further" (zone 3)**
  $M_{cr} = C_b * \pi/L_b \sqrt{(E*I_y*G*J + (\pi*E/L_b)^2 * I_y C_w)}$
  - $L_b > L_r$

$L_p = 1.76\ r_y \sqrt{E/Fy}$

$M_p = F_y Z_x$

$M_r = 0.7\ F_y S_x$

C_b is LTB modification factor



$$C_b = \frac{12.5\ Mmax}{2.5\ Mmax + 3\ MA + 4MB + 3MC}$$

University of Michigan, TCAUP Structures II Slide 2 of 19

Problem Set 04

#Q4: The factored design moment, Mu

#Q5: The total factored design load, wu

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).

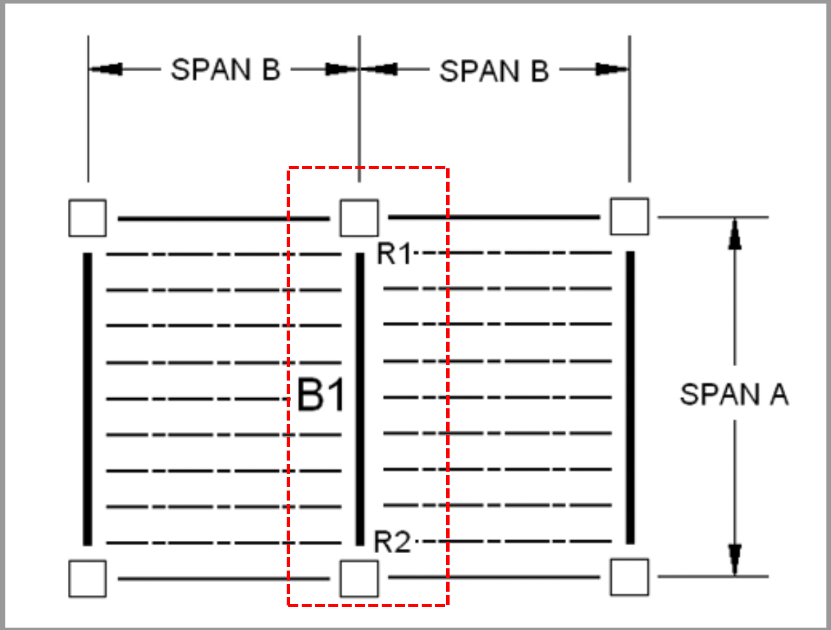
DATASET: 1 -2- -3-

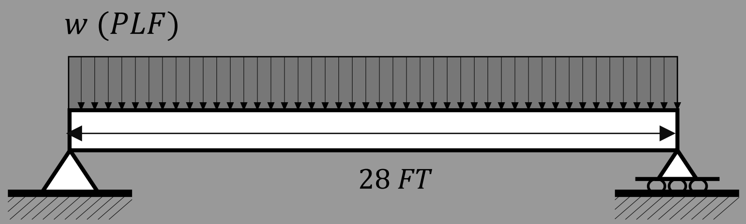
W-section	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF

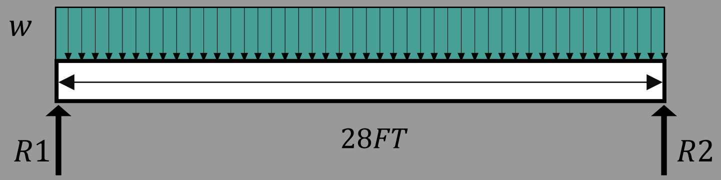
$$M_u \leq \varphi M_n = 4590\ K - IN \times \frac{1\ FT}{12\ IN}$$

$$= \mathbf{382.5\ K - FT}$$

$$M_u = \frac{W_u L^2}{8} \rightarrow W_u = \frac{8 M_u}{L^2} = \frac{8(382.5)}{28^\wedge 2} = \mathbf{3.9\ KLF}$$







Problem Set 04

#Q6: The total unfactored dead load on the beam, w_DL

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	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF

$$w_{DL} = \boldsymbol{Beam\ Self\ load + Floor\ DL}$$

$$w_{DL} = 61\ PLF\ + 18\left(\frac{13}{2} + \frac{13}{2}\right)\ PLF$$

$$= 295\ PLF = \mathbf{0.295\ KLF}$$

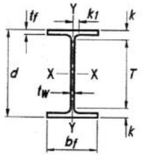


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k_{des}	k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.7	14 5/8	0.645	5/8	5/16	14.7	14 3/4	1.03	1	1.63	2 5/16	1 9/16	10	5 1/2
×120	35.3	14.5	14 1/2	0.590	9/16	5/16	14.7	14 5/8	0.940	15/16	1.54	2 1/4	1 1/2		
×109	32.0	14.3	14 3/8	0.525	1/2	1/4	14.6	14 5/8	0.860	7/8	1.46	2 3/16	1 1/2		
×99[f]	29.1	14.2	14 1/8	0.485	1/2	1/4	14.6	14 5/8	0.780	3/4	1.38	2 1/16	1 7/16		
×90[f]	26.5	14.0	14	0.440	7/16	1/4	14.5	14 1/2	0.710	11/16	1.31	2	1 7/16		
W14×82	24.0	14.3	14 1/4	0.510	1/2	1/4	10.1	10 1/8	0.855	7/8	1.45	1 11/16	1 1/16	10 7/8	5 1/2
×74	21.8	14.2	14 1/8	0.450	7/16	1/4	10.1	10 1/8	0.785	13/16	1.38	1 5/8	1 1/16		
×68	20.0	14.0	14	0.415	7/16	1/4	10.0	10	0.720	3/4	1.31	1 9/16	1 1/16		
×61	17.9	13.9	13 7/8	0.375	3/8	3/16	10.0	10	0.645	5/8	1.24	1 1/2	1		
W14×53	15.6	13.9	13 7/8	0.370	3/8	3/16	8.06	8	0.660	11/16	1.25	1 1/2	1	10 7/8	5 1/2
×48	14.1	13.8	13 3/4	0.340	5/16	3/16	8.03	8	0.595	5/8	1.19	1 7/16	1		
×43[c]	12.6	13.7	13 5/8	0.305	5/16	3/16	8.00	8	0.530	1/2	1.12	1 3/8	1		
W14×38[c]	11.2	14.1	14 1/8	0.310	5/16	3/16	6.77	6 3/4	0.515	1/2	0.915	1 1/4	13/16	11 5/8	3 1/2[g]
×34[c]	10.0	14.0	14	0.285	5/16	3/16	6.75	6 3/4	0.455	7/16	0.855	1 3/16	3/4		3 1/2
×30[c]	8.85	13.8	13 7/8	0.270	1/4	1/8	6.73	6 3/4	0.385	3/8	0.785	1 1/8	3/4		3 1/2
W14×26[c]	7.69	13.9	13 7/8	0.255	1/4	1/8	5.03	5	0.420	7/16	0.820	1 1/8	3/4	11 5/8	2 3/4[g]
×22[c]	6.49	13.7	13 3/4	0.230	1/4	1/8	5.00	5	0.335	5/16	0.735	1 1/16	3/4	11 5/8	2 3/4[g]
W12×336[h]	98.9	16.8	16 7/8	1.78	1 3/4	7/8	13.4	13 3/8	2.96	2 15/16	3.55	3 7/8	1 11/16	9 1/8	5 1/2
×305[h]	89.5	16.3	16 3/8	1.63	1 5/8	13/16	13.2	13 1/4	2.71	2 11/16	3.30	3 5/8	1 5/8		
×279[h]	81.9	15.9	15 7/8	1.53	1 1/2	3/4	13.1	13 1/8	2.47	2 1/2	3.07	3 3/8	1 5/8		
×252[h]	74.1	15.4	15 3/8	1.40	1 3/8	11/16	13.0	13	2.25	2 1/4	2.85	3 1/8	1 1/2		
×230[h]	67.7	15.1	15	1.29	1 5/16	11/16	12.9	12 7/8	2.07	2 1/16	2.67	2 15/16	1 1/2		
×210	61.8	14.7	14 3/4	1.18	1 3/16	5/8	12.8	12 3/4	1.90	1 7/8	2.50	2 13/16	1 7/16		
×190	56.0	14.4	14 3/8	1.06	1 1/16	9/16	12.7	12 5/8	1.74	1 3/4	2.33	2 5/8	1 3/8		
×170	50.0	14.0	14	0.960	15/16	1/2	12.6	12 5/8	1.56	1 9/16	2.16	2 7/16	1 5/16		
×152	44.7	13.7	13 3/4	0.870	7/8	7/16	12.5	12 1/2	1.40	1 3/8	2.00	2 5/16	1 1/4		
×136	39.9	13.4	13 3/8	0.790	13/16	7/16	12.4	12 3/8	1.25	1 1/4	1.85	2 1/8	1 1/4		
×120	35.2	13.1	13 1/8	0.710	11/16	3/8	12.3	12 3/8	1.11	1 1/8	1.70	2	1 3/16		
×106	31.2	12.9	12 7/8	0.610	5/8	5/16	12.2	12 1/4	0.990	1	1.59	1 7/8	1 1/8		
×96	28.2	12.7	12 3/4	0.550	9/16	5/16	12.2	12 1/8	0.900	7/8	1.50	1 13/16	1 1/8		
×87	25.6	12.5	12 1/2	0.515	1/2	1/4	12.1	12 1/8	0.810	13/16	1.41	1 11/16	1 1/16		
×79	23.2	12.4	12 3/8	0.470	1/2	1/4	12.1	12 1/8	0.735	3/4	1.33	1 5/8	1 1/16		
×72	21.1	12.3	12 1/4	0.430	7/16	1/4	12.0	12	0.670	11/16	1.27	1 9/16	1 1/16		
×65[f]	19.1	12.1	12 1/8	0.390	3/8	3/16	12.0	12	0.605	5/8	1.20	1 1/2	1		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 1-1 (continued)
W-Shapes
Properties

W14-W12

Nom-inal 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}
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.7	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.6	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.10	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.4	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.83	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.3	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.2	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.2	1.05	1950
38	6.57	39.6	385	54.6	5.87	61.5	26.7	7.88	1.55	12.1	1.82	13.6	0.798	1230
34	7.41	43.1	340	48.6	5.83	54.6	23.3	6.91	1.53	10.6	1.80	13.5	0.569	1070
30	8.74	45.4	291	42.0	5.73	47.3	19.6	5.82	1.49	8.99	1.77	13.4	0.380	887
26	5.98	48.1	245	35.3	5.65	40.2	8.91	3.55	1.08	5.54	1.30	13.5	0.358	405
22	7.46	53.3	199	29.0	5.54	33.2	7.00	2.80	1.04	4.39	1.27	13.4	0.208	314
336	2.26	5.47	4060	483	6.41	603	1190	177	3.47	274	4.13	13.8	243	57000
305	2.45	5.98	3550	435	6.29	537	1050	159	3.42	244	4.05	13.6	185	48600
279	2.66	6.35	3110	393	6.16	481	937	143	3.38	220	4.00	13.4	143	42000
252	2.89	6.96	2720	353	6.06	428	828	127	3.34	196	3.93	13.2	108	35800
230	3.11	7.56	2420	321	5.97	386	742	115	3.31	177	3.87	13.0	83.8	31200
210	3.37	8.23	2140	292	5.89	348	664	104	3.28	159	3.81	12.8	64.7	27200
190	3.65	9.16	1890	263	5.82	311	589	93.0	3.25	143	3.77	12.7	48.8	23600
170	4.03	10.1	1650	235	5.74	275	517	82.3	3.22	126	3.70	12.4	35.6	20100
152	4.46	11.2	1430	209	5.66	243	454	72.8	3.19	111	3.66	12.3	25.8	17200
136	4.96	12.3	1240	186	5.58	214	398	64.2	3.16	98.0	3.61	12.2	18.5	14700
120	5.57	13.7	1070	163	5.51	186	345	56.0	3.13	85.4	3.56	12.0	12.9	12400
106	6.17	15.9	933	145	5.47	164	301	49.3	3.11	75.1	3.52	11.9	9.13	10700
96	6.76	17.7	833	131	5.44	147	270	44.4	3.09	67.5	3.49	11.8	6.85	9410
87	7.48	18.9	740	118	5.38	132	241	39.7	3.07	60.4	3.46	11.7	5.10	8270
79	8.22	20.7	662	107	5.34	119	216	35.8	3.05	54.3	3.43	11.7	3.84	7330
72	8.99	22.6	597	97.4	5.31	108	195	32.4	3.04	49.2	3.41	11.6	2.93	6540
65	9.92	24.9	533	87.9	5.28	96.8	174	29.1	3.02	44.1	3.38	11.5	2.18	5780

Problem Set 04

#Q6: The total unfactored dead load on the beam, w_DL

#Q7: The total factored dead load on the beam, wu_DL

DATASET: 1 -2- -3-

W-section	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF

$$w_{DL} = \textit{Beam Self load} + \textit{Floor DL}$$

$$= 61\ PLF + 18\left(\frac{13}{2} + \frac{13}{2}\right)\ PLF$$

$$= 295\ PLF = \mathbf{0.295\ KLF}$$

$$w_{u\_DL} = \mathbf{1.2\ W_{Dl}} = 1.2\ (0.295\ )$$

$$= \mathbf{0.354\ KLF}$$

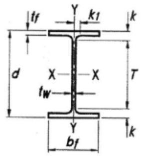


Table 1-1 (continued)
W-Shapes
Dimensions

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_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.7	14 5/8	0.645	5/8	5/16	14.7	14 3/4	1.03	1	1.63	2 5/16	1 9/16	10	5 1/2
×120	35.3	14.5	14 1/2	0.590	9/16	5/16	14.7	14 5/8	0.940	15/16	1.54	2 1/4	1 1/2		
×109	32.0	14.3	14 3/8	0.525	1/2	1/4	14.6	14 5/8	0.860	7/8	1.46	2 3/16	1 1/2		
×99[f]	29.1	14.2	14 1/8	0.485	1/2	1/4	14.6	14 5/8	0.780	3/4	1.38	2 1/16	1 7/16		
×90[f]	26.5	14.0	14	0.440	7/16	1/4	14.5	14 1/2	0.710	11/16	1.31	2	1 7/16		
W14×82	24.0	14.3	14 1/4	0.510	1/2	1/4	10.1	10 1/8	0.855	7/8	1.45	1 11/16	1 1/16	10 7/8	5 1/2
×74	21.8	14.2	14 1/8	0.450	7/16	1/4	10.1	10 1/8	0.785	13/16	1.38	1 5/8	1 1/16		
×68	20.0	14.0	14	0.415	7/16	1/4	10.0	10	0.720	3/4	1.31	1 9/16	1 1/16		
×61	17.9	13.9	13 7/8	0.375	3/8	3/16	10.0	10	0.645	5/8	1.24	1 1/2	1		
W14×53	15.6	13.9	13 7/8	0.370	3/8	3/16	8.06	8	0.660	11/16	1.25	1 1/2	1	10 7/8	5 1/2
×48	14.1	13.8	13 3/4	0.340	5/16	3/16	8.03	8	0.595	5/8	1.19	1 7/16	1		
×43[c]	12.6	13.7	13 5/8	0.305	5/16	3/16	8.00	8	0.530	1/2	1.12	1 3/8	1		

Table 1-1 (continued)
W-Shapes
Properties

Nom-inal 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}
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.7	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.6	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.10	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.4	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.83	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.3	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.2	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.2	1.05	1950

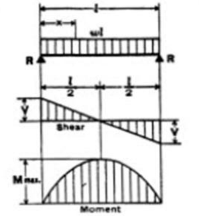
Procedure - Analysis of Steel Beams – **for Zone 1** $L_b < L_p$

Pass/Fail

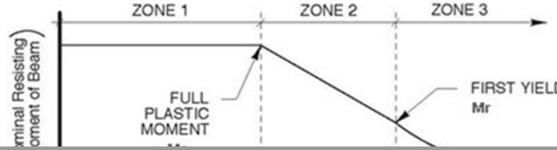
Given: yield stress, steel section, loading, bracing (L_b)
Find: pass/fail of section

1. Calculate the factored design load w_u
   $\boxed{w_u = 1.2w_{DL} + 1.6w_{LL}}$
2. Determine the design moment M_u. M_u will be the maximum beam moment using the factored loads
3. Insure that $L_b < L_p$ (zone 1)
   $L_p = 1.76\ r_y \sqrt{E/Fy}$
4. Determine the nominal moment, Mn
   $M_n = F_y Z_x$ (look up Z_x for section)

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 . . . . = $\frac{wx}{2}(l-x)$
Δmax. (at center) . . . . = $\frac{5wl^4}{384EI}$
Δx . . . . = $\frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$



Problem Set 04

#Q8: The factored live load on the beam, wu_LL

#Q9: The actual beam live load (capacity), w_LL

#Q10 : The actual floor live load (floor capacity), LL

DATASET: 1 -2- -3-

W-section	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF

$$w_{u_{LL}} = W_u - w_{u_{DL}} = 3.9 - 0.354$$
$$= 3.546\ KLF$$

$$w_{u_{LL}} = 1.6\ W_{LL}$$
$$\rightarrow W_{LL} = \frac{3.546}{1.6} = 2.216\ KLF$$

$$LL = \frac{W_{LL}}{SpanB} = \frac{2.216}{13} = 0.17048\ KSF = 170.48\ PSF$$

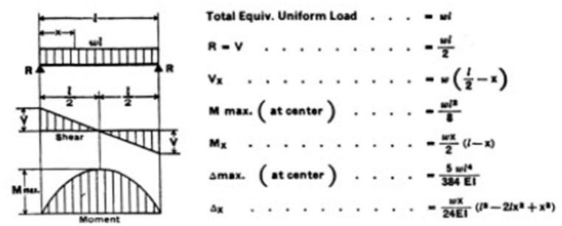
Procedure - Analysis of Steel Beams – **for Zone 1** $L_b < L_p$

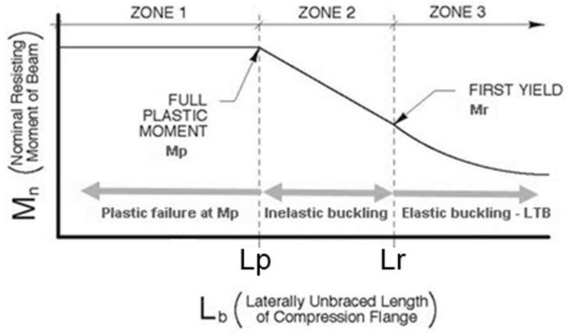
Pass/Fail

Given: yield stress, steel section, loading, bracing (L_b)
Find: pass/fail of section

1. Calculate the factored design load w_u
   $w_u = 1.2w_{DL} + 1.6w_{LL}$
2. Determine the design moment M_u. M_u will be the maximum beam moment using the factored loads
3. Insure that $L_b < L_p$ (zone 1)
   $L_p = 1.76\ r_y \sqrt{E/Fy}$
4. Determine the nominal moment, Mn
   $M_n = F_y Z_x$ (look up Z_x for section)
5. Factor the nominal moment
   $øM_n = 0.90\ M_n$
6. Check that $M_u < øM_n$
7. Check shear
8. Check deflection

1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD





University of Michigan, TCAUP Structures II Slide 6 of 19

Problem Set 04

#Q11: The maximum factored design beam shear force, Vu_max

#Q12: The web area, Aw

#Q13: The factored shear resistance, phi Vn

#Q14: Is the section safe for shear? (1=yes, 0=no)

DATASET: 1 -2- -3-

W-section	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF

$$V_{u_{max}} = \frac{W_u L}{2} = \frac{3.9\,(28)}{2} = \mathbf{54.6\ K} < \varphi V_n (pass)$$

$$A_w = d.t_w = \mathbf{13.9(0.375)}$$

$$= 13.9(0.375) = \mathbf{5.212\ IN^2}$$

$$\varphi V_n = \mathbf{0.6\ F_y A_w} = 0.6(50)(5.212)$$

$$= \mathbf{156.375\ K}$$

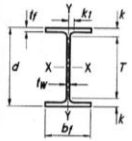


Table 1-1 (continued)
W-Shapes
Dimensions

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}	Distance k k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.7	14 5/8	0.645	5/8	5/16	14.7	14 3/4	1.03	1	1.63	2 5/16	1 9/16	10	5 1/2
×120	35.3	14.5	14 1/2	0.590	9/16	5/16	14.7	14 5/8	0.940	15/16	1.54	2 1/4	1 1/2	↓	↓
×109	32.0	14.3	14 3/8	0.525	1/2	1/4	14.6	14 5/8	0.860	7/8	1.46	2 3/16	1 1/2	↓	↓
×99[f]	29.1	14.2	14 1/8	0.485	1/2	1/4	14.6	14 5/8	0.780	3/4	1.38	2 1/16	1 7/16	↓	↓
×90[f]	26.5	14.0	14	0.440	7/16	1/4	14.5	14 1/2	0.710	11/16	1.31	2	1 7/16	↓	↓
W14×82	24.0	14.3	14 1/4	0.510	1/2	1/4	10.1	10 1/8	0.855	7/8	1.45	1 11/16	1 1/16	10 7/8	5 1/2
×74	21.8	14.2	14 1/8	0.450	7/16	1/4	10.1	10 1/8	0.785	13/16	1.38	1 5/8	1 1/16	↓	↓
×68	20.0	14.0	14	0.415	7/16	1/4	10.0	10	0.720	3/4	1.31	1 9/16	1 1/16	↓	↓
×61	17.9	13.9	13 7/8	0.375	3/8	3/16	10.0	10	0.645	5/8	1.24	1 1/2	1	↓	↓
W14×53	15.6	13.9	13 7/8	0.370	3/8	3/16	8.06	8	0.660	11/16	1.25	1 1/2	1	10 7/8	5 1/2
×48	14.1	13.8	13 3/4	0.340	5/16	3/16	8.03	8	0.595	5/8	1.19	1 7/16	1	↓	↓
×43[c]	12.6	13.7	13 5/8	0.305	5/16	3/16	8.00	8	0.530	1/2	1.12	1 3/8	1	↓	↓

Table 1-1 (continued)
W-Shapes
Properties

W14-W12

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}
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.7	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.6	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.10	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.4	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.83	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.3	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.2	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.2	1.05	1950

Design for Shear

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

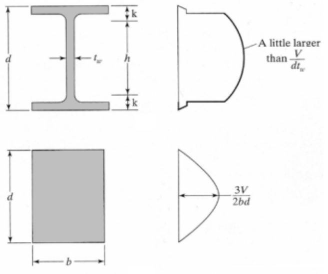
$F_v = V / A_w$

$A_w = d * t_w$

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

Problem Set 04

#Q15: The actual (unfactored) deflection due to total DL + LL

#Q16: The deflection limit L/180

#Q17 : s the actual deflection less than the limit L/180?

DATASET: 1 -2- -3-

W-section	W14X61
Fy	50 KSI
Span A	28 FT
Span B	13 FT
Floor DL	18 PSF

$$w = (w_{DL} + w_{LL}) = 0.295 + 2.216 = 2.511\ KLF$$

$$\Delta = \frac{5\,wL^4}{384\,EI} = \frac{5(2.511)(28)^4\,(1728)}{384(29000)(640)} = 1.78\ IN$$

$$\Delta_{Limit} = \frac{L}{180} = \frac{28 * 12}{180} = 1.86\ IN$$

$$\Delta < \Delta_{Limit} \rightarrow pass$$

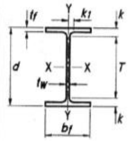


Table 1-1 (continued)
W-Shapes
Dimensions

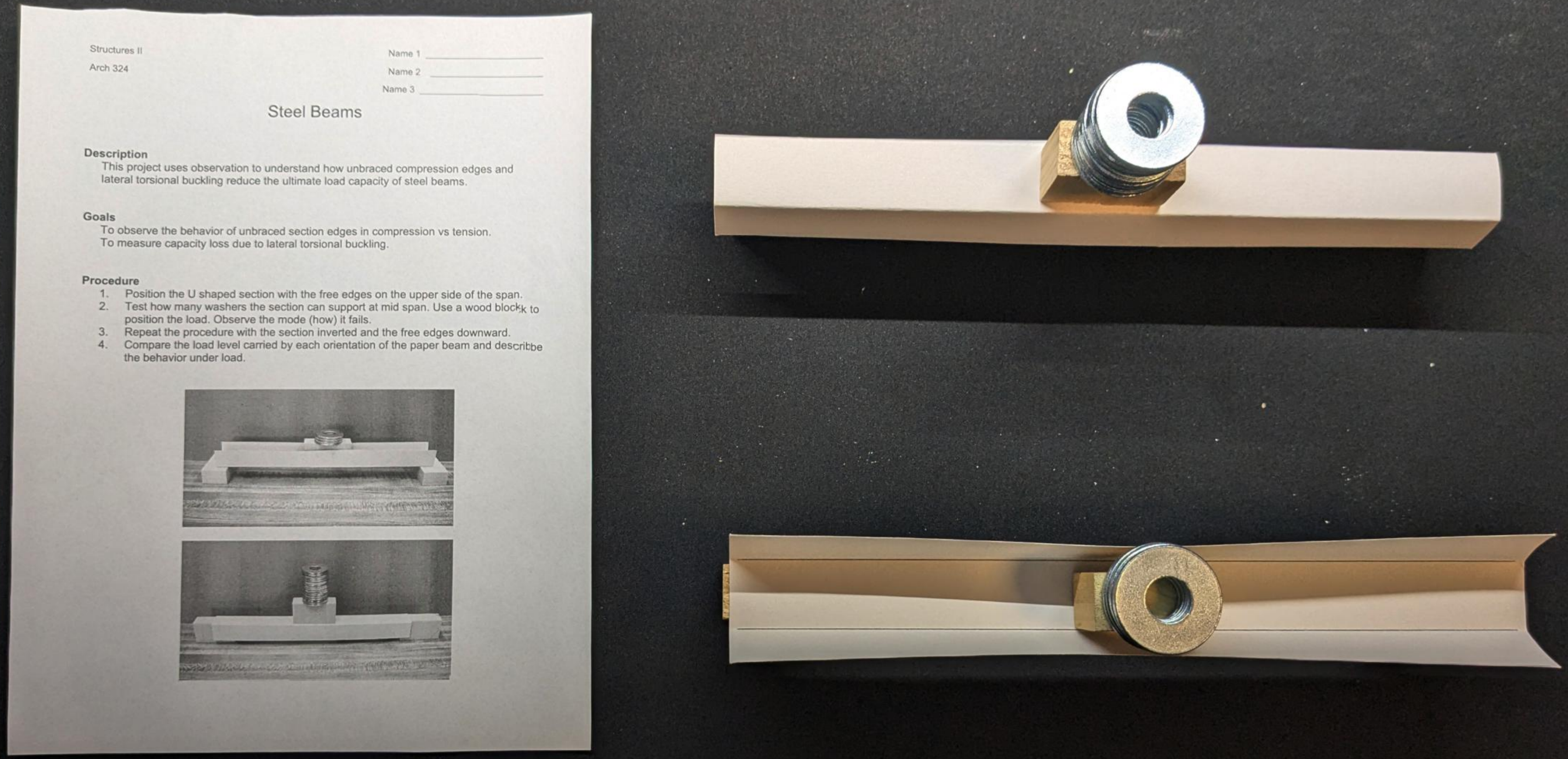
Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.7	14 5/8	0.645	5/8	5/16	14.7	14 3/4	1.03	1	1.63	2 5/16	1 9/16	10	5 1/2
×120	35.3	14.5	14 1/2	0.590	9/16	5/16	14.7	14 5/8	0.940	15/16	1.54	2 1/4	1 1/2		
×109	32.0	14.3	14 3/8	0.525	1/2	1/4	14.6	14 5/8	0.860	7/8	1.46	2 3/16	1 1/2		
×99[f]	29.1	14.2	14 1/8	0.485	1/2	1/4	14.6	14 5/8	0.780	3/4	1.38	2 1/16	1 7/16		
×90[f]	26.5	14.0	14	0.440	7/16	1/4	14.5	14 1/2	0.710	11/16	1.31	2	1 7/16	↓	↓
W14×82	24.0	14.3	14 1/4	0.510	1/2	1/4	10.1	10 1/8	0.855	7/8	1.45	1 11/16	1 1/16	10 7/8	5 1/2
×74	21.8	14.2	14 1/8	0.450	7/16	1/4	10.1	10 1/8	0.785	13/16	1.38	1 5/8	1 1/16		
×68	20.0	14.0	14	0.415	7/16	1/4	10.0	10	0.720	3/4	1.31	1 9/16	1 1/16		
×61	17.9	13.9	13 7/8	0.375	3/8	3/16	10.0	10	0.645	5/8	1.24	1 1/2	1	↓	↓
W14×53	15.6	13.9	13 7/8	0.370	3/8	3/16	8.06	8	0.660	11/16	1.25	1 1/2	1	10 7/8	5 1/2
×48	14.1	13.8	13 3/4	0.340	5/16	3/16	8.03	8	0.595	5/8	1.19	1 7/16	1		
×43[c]	12.6	13.7	13 5/8	0.305	5/16	3/16	8.00	8	0.530	1/2	1.12	1 3/8	1	↓	↓

Table 1-1 (continued)
W-Shapes
Properties

W14-W12

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}
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.7	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.6	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.10	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.4	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.83	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.3	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.2	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.2	1.05	1950

Lab 03



Structures II

Arch 324

Name 1 ____________

Name 2 ____________

Name 3 ____________

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 blockk 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 describbe the behavior under load.

Lab03

Structures II

Arch 324

Name 1 ____________________

Name 2 ____________________

Name 3 ____________________

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.

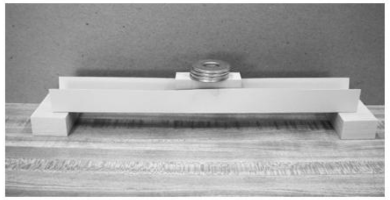
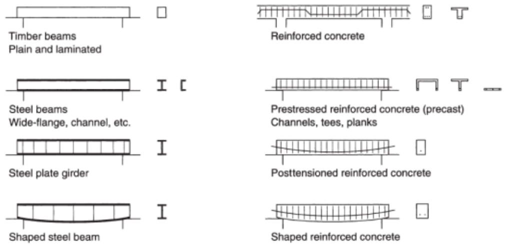
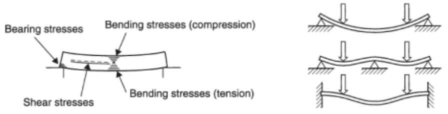


FIGURE 6.3 Typical beams and important factors in beam design.



(a) Typical beams



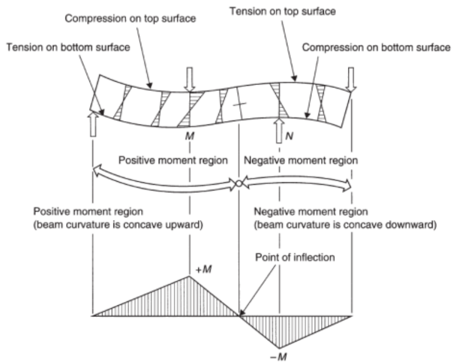
Transversely applied loads cause internal bending stresses, shear stresses, bearing stresses, and accompanying deflections to develop. Other phenomena (torsion) may also occur. Making the beam deeper rather than shallower improves its load-carrying capabilities. Either making a beam continuous over several supports or restraining its ends greatly increases the rigidity and load-carrying capacity of the member.

(b) General behavior of beams

is *continuous* over several supports also offers some advantages compared with a series of simply supported beams, although some trade-offs are involved. (See Chapter 8.)

While the term *beam* is commonly used to describe a straight horizontal member transversely loaded and in a state of bending, many different structural elements behave in a comparable manner. Vertical members that make up part of a façade are often in bending and can be considered vertical beams and analyzed and designed according to principles discussed in this chapter. The key here is *not* the orientation of a member but that it is primarily in a state of bending. In a similar vein and depending on support conditions, some members that are curved—even in a complex way—may also be in a primary state of bending; and hence analyzed and designed as *beams*. An arch-shaped member with a pin on one end and a roller on the other end, for example, does not carry loads primarily through the development of internal forces in compression (as an arch does); rather, it carries loads by bending. Compared to a true arch (e.g., with pins on both ends), the curved member would then need a much larger cross section to carry the loads safely because bending is an inefficient way to carry loads.

FIGURE 6.12 Stress variations.



Distribution of Bending Stresses. The previous examples considered the bending stresses at only one cross section of a beam (where the maximum stresses exist). In Chapter 2, the distribution of bending moments along the length of a member was discussed and depicted via moment diagrams. Because bending stresses are directly dependent on bending moments, it follows that bending stress magnitudes vary along the length of a beam as described by the moment diagram in Figure 6.12. When checking to see if a beam with no variation in depth or width along its length is adequately large, take care to use the maximum bending moment anywhere in the structure. When sizing a beam with no variation in width or depth along its length, using the maximum bending moment value to determine needed depth and width dimensions means that the beam may be oversized where moments are smaller. Beam depths can also be made to vary along the length of a beam because, stress levels are not constant throughout the beam. (See Section 6.4.1.)

6.3.2 Lateral Buckling of Beams

Consider the thin, deep beam illustrated in Figure 6.13. Applying a load may cause *lateral buckling* in the beam, and failure will occur before the strength of the section can be utilized. The phenomenon of lateral buckling in beams is similar to that found in trusses. An instability in the lateral direction occurs because of the compressive forces developed in the upper region of the beam, coupled with insufficient rigidity of the beam in that direction. In the examples discussed thus far, it was assumed that this type of failure does not occur. Depending on the proportions of the beam cross section, lateral buckling can occur at relatively low stress levels.

Lateral buckling can be prevented in two primary ways: (1) by using transverse bracing and (2) by making the beam stiff in the lateral direction. When a beam is used to support a roof deck or a secondary framing system, these elements automatically

FIGURE 6.13 Lateral buckling in beams.

Top region of beam is in compression. A beam not rigid in the lateral direction can buckle laterally because of the compressive force present.

Lab03

→ Group work instructions

Please form groups of 2 to 4 students.

Please do not forget to write all group members' names on both sheets.

Return the completed sheets to me at the end of the session.

Please ensure that you attend the recitation sessions.

If you are unable to attend a session, send me an email so that we can discuss how to proceed. *Email: arfazel@umich.edu*

Tower project

Architecture 324
Structures II

Prof. Peter von Buelow
Winter 2025

Tower Project

Description

This project gives students the chance to apply concepts learned in column analysis to the design of a structural system that carries primarily a compression load – a tower. Work is to be done in groups of up to four people. The project is divided into 3 parts: 1) initial conceptual design, 2) design development and testing, 3) final analysis and documentation.

Goals

- to explore design parameters of geometry and material under compression.
- to develop a design of a compression member to meet the criteria below.
- to make some rough hand calculation to estimate the expected performance.
- to test the compression member and record the results.
- to document the results in a well organized and clear report format.

Criteria

- The tower is to be made of wood. Either linear wood (sticks) or wood panels (sheets) can be used. Glue can be used to connect the elements. Gusset plates at the joints are allowed and can also be glued. But **no steel pins** or fasteners may be used.
- Wood: **any species**. **maximum cross-sectional dimension = 1/4".**
- **NO** paper, mylar or plastic or string or dental floss.
- If a member is made by laminating multiple pieces together, the maximum cross-sectional dimension or thickness still cannot exceed 1/4".
- The **height of the tower = 48"**.
- The tower **must hold at least 50 lbs**.
- The entire tower **can weigh no more than 4 oz**.
- The top of the tower must be loadable. The weights will be stacked on top of the tower, but you may optionally use a loose piece of MDF or plywood as a tray under the weights. (It will not be counted in either weight or load)
- Towers will be graded on their low weight, high load-carrying capacity, and the load/weight ratio. The evaluation formula is:

  (4/weight in OZ) + (load in LBS/50) + (load LBS/weight OZ)x1.5
- The score will be normalized to a range of 50 to 100. It is used together with report scores to assess your project (a detailed evaluation form is given separately).

Procedure

1. Develop a structural concept for a tower meeting the above criteria.
2. Analyze the design concept with **either** hand calculations or a computer program (e.g. Dr. Frame)
3. Determine the capacity of the major members and of the overall tower (total capacity in LBS)
4. Estimate your expected score using the formula above.
5. Write the preliminary report.
6. Construct the structural model.
7. Test the model. 5-pound steel bars will be placed on top of the model, until the model fails. (bar size: 1 ½" x 2" x 5 13/16").
8. Produce final report documenting requirements and process. See also score sheet.

Due Dates
See Course Schedule

Scoring	
Preliminary Report	40 pts
Testing	60 pts
Final Report	150 pts

Architecture 324
Structures II

Prof. Peter von Buelow
Winter 2025

Tower Project – Preliminary Report Requirements

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

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

Analysis – the report should include the following:

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

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

Format - Reports should be formatted **for 8½ X 11** paper. 11X17 format reports will not be accepted. Once returned to you graded, **save the original copy of the preliminary report** for submission together with the Final Report.

The report is a professional document. Text should be clear, grammatically correct, and language should be appropriate and professional. All calculations should be legible and clearly described – not just numbers or results, but with a clear description of what is being calculated included.

Properties of Basswood: (like in the Media Center)

Density (oven dry)	29 pcf **
E (buckling)	1,650,000 psi **
F (Compression ‖ to grain)	4745 psi *
F (Compression ⊥ to grain)	377 psi *
F (Tension ‖ to grain)	4500 psi (estimate)
F (Tension ⊥ to grain)	348 psi *
F (Shear ‖ to grain)	986 psi *
F (Flexure)	5900 psi *

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

Tower project

Prelim Tower Report

The deadline has been postponed to **Sunday, February 16th**.