Arch324

STRUCTURES II

Winter 2026
Recitation

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Recitation Guidelines

Homework Problem

No Lab Today

- Try to attend all sessions. Unexcused absences will **affect your grade** starting from the second missed class.

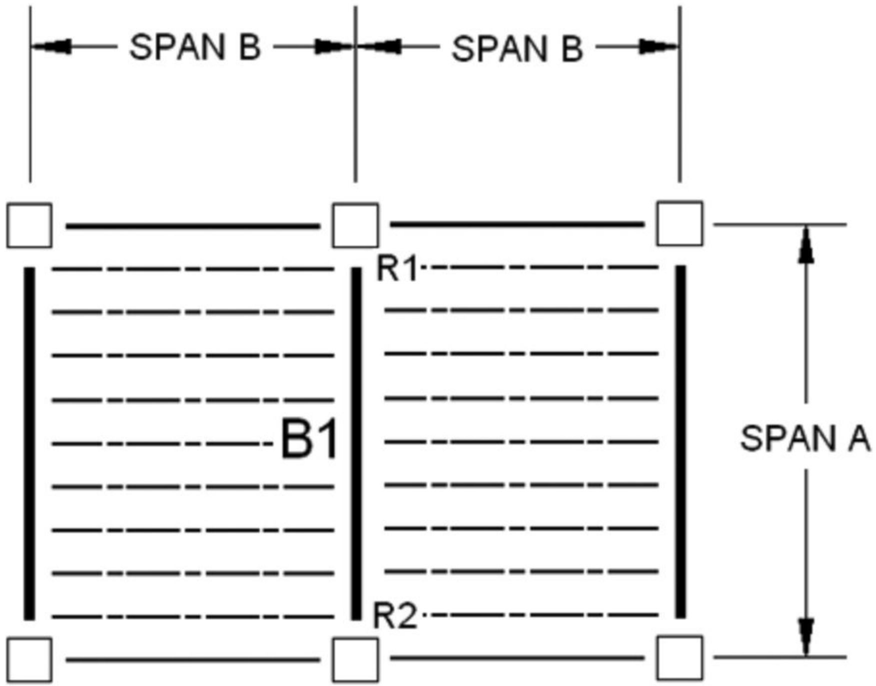
Analysis Example - HW4

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



Analysis Example - HW4

17 Questions

#	Question	Your Response	
1	The plastic modulus of the section, Zx		IN3
2	The nominal bending moment, Mn		K-IN
3	The factored bending resistance, phi Mn		K-IN
4	The factored design moment, Mu		K-FT
5	The total factored design load, wu		KLF
6	The total unfactored dead load on the beam, w_DL		KLF
7	The total factored dead load on the beam, wu_DL		KLF
8	The factored live load on the beam, wu_LL		KLF
9	The actual beam live load (capacity), w_LL		KLF
10	The actual floor live load (floor capacity), LL		PSF
11	The maximum factored design beam shear force, Vu_max		K
12	The web area, Aw		IN2
13	The factored shear resistance, phi Vn		K
14	Is the section safe for shear? (1=yes, 0=no)		
15	The actual (unfactored) deflection due to total DL + LL		IN
16	The deflection limit L/180		IN
17	Is the actual deflection less than the limit L/180? (1=yes, 0=no)		

Answer HW4

17 Questions

#	Question	Correct Answer
1	The plastic modulus of the section, Zx	102 IN3
2	The nominal bending moment, Mn	5100 K-IN
3	The factored bending resistance, phi Mn	4590 K-IN
4	The factored design moment, Mu	382.5 K-FT
5	The total factored design load, wu	3.903061224 KLF
6	The total unfactored dead load on the beam, w_DL	0.295 KLF
7	The total factored dead load on the beam, wu_DL	0.354 KLF
8	The factored live load on the beam, wu_LL	3.549061224 KLF
9	The actual beam live load (capacity), w_LL	2.218163265 KLF
10	The actual floor live load (floor capacity), LL	170.6279435 PSF
11	The maximum factored design beam shear force, Vu_max	54.64285714 K
12	The web area, Aw	5.2125 IN2
13	The factored shear resistance, phi Vn	156.375 K
14	Is the section safe for shear? (1=yes, 0=no)	1
15	The actual (unfactored) deflection due to total DL + LL	1.872653276 IN
16	The deflection limit L/180	1.866666667 IN
17	Is the actual deflection less than the limit L/180? (1=yes, 0=no)	0

Analysis Example - HW4

AISC14_Table 1-1

W-section	W14X61

Q1 Zx=102 in³

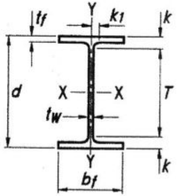


Table 1-1 (continued)
W-Shapes
Dimensions

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]

Table 1-1 (continued)
W-Shapes
Properties

W14-W12

Nom-inal 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

Analysis Example - HW4

Q2-Q4

2	The nominal bending moment, Mn		K-IN
3	The factored bending resistance, phi Mn		K-IN
4	The factored design moment, Mu		K-FT

Given: Fy=50 KSI

Assume the beam is fully braced, Lb<Lp(zone1)

Q2 Mn=Zx▪Fy=102×50=5100 K-IN

Q3 phi Mn=0.9Mn=0.9×5100=4590 K-IN

Mu≤phi Mn, assume Mu=phi Mn

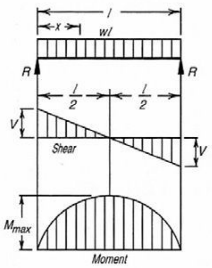
Q4 Mu=4590/12=382.5 K-FT

Analysis Example - HW4

Q5-Q11

5	The total factored design load, wu		KLF
6	The total unfactored dead load on the beam, w_DL		KLF
7	The total factored dead load on the beam, wu_DL		KLF
8	The factored live load on the beam, wu_LL		KLF
9	The actual beam live load (capacity), w_LL		KLF
10	The actual floor live load (floor capacity), LL		PSF
11	The maximum factored design beam shear force, Vu_max		K

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

Floor DL=18

Span B=13

W 14×61

For example, if your W-section is W 12×48, then use 48

Span A=28

Q5 $M_u = \frac{wl^2}{8} = \frac{W\cdot 28^2}{8} = 382.5 \longrightarrow W_u = W = \frac{382.5\times 8}{28^2} = 3.90306$ KLF

Q6 $W_{DL} = \frac{18\times 2\cdot\frac{13}{2} + 61}{1000} = 0.295$ KLF

$W_u = 1.2W_{DL} + 1.6W_{LL} = 1.2\times 0.295 + 1.6W_{LL} = 3.90306$

Q9 $\longrightarrow W_{LL} = 2.21816$ KLF

Q7 $W_{u\_DL} = 1.2W_{DL} = 1.2\times 0.295 = 0.354$ KLF

Q8 $W_{u\_LL} = 1.6W_{LL} = 1.6\times 2.21816 = 3.5491$ KLF

Q10 $LL = \frac{W_{LL}\cdot 1000}{\text{Span } B} = \frac{2.21816}{13}\times 1000 = 170.628$ PSF

Q11 $V_{u\_max} = R_1 = R_2 = \frac{W_u\cdot L}{2} = \frac{3.90306\times 28}{2} = 54.6428$ k

Span A=28

Analysis Example - HW4

Q12-Q17

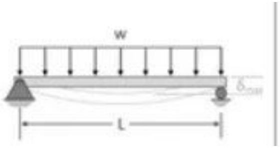
12	The web area, Aw		IN2
13	The factored shear resistance, phi Vn		K
14	Is the section safe for shear? (1=yes, 0=no)		
15	The actual (unfactored) deflection due to total DL + LL		IN
16	The deflection limit L/180		IN
17	Is the actual deflection less than the limit L/180? (1=yes, 0=no)		

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: $V_n = 0.6 F_y A_w$



$$\delta_{max} = \frac{5wL^4}{384EI}$$

Given: Fy=50 KSI

Span A=L=28 FT

From AISC14_Table1-1:

h/tw=30.4

I=640

d=13.9

tw=0.375

$$\frac{h}{t_w} = 30.4 < 2.45\sqrt{\frac{E}{F_y}} = 59$$

Q12 $A_w = d \cdot t_w = 13.9 \times 0.375 = 5.2125 \ in^2$

Q13 $V_n = 0.6 \cdot F_y \cdot A_w = 0.6 \cdot (50)(5.2125) = 156.375 \ K$

Q14 $V_{u\text{-}max} = 54.6428 < 156.375 = V_n$ ✓

$$W = DL + LL = (W_{DL} + W_{LL}) \times 1000 = (0.295 + 2.21816) \times 1000$$
$$= 2513.16$$

Transfer to inch

Q16 $\frac{L}{180} = \frac{28 \times 12}{180} = 1.8667 \ in$

Q15 Q17 $\delta_{max} = \frac{5wL^4}{384EI} = \frac{5(2513.16) \cdot 28^4 \cdot 1728}{384(29 \times 10^6) \cdot (640)} = 1.87265 > 1.8667$ ⊗

$$= \frac{5(2513.16 \times \frac{1}{12}) \cdot (28 \times 12)^4}{384(29 \times 10^6) \cdot (640)}$$

Another way to transfer into inch

Criteria & Procedure

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.

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).

Scoring

Preliminary Report	40 pts
Testing	60 pts
Final Report	150 pts

Analysis

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

Capacity

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

Design

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