ARCHITECTURE 324
Structures II

Recitation 11

Sections 04&05

Instructor
Peter von Buelow

GSI
Alireza Fazel
April 11, 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)

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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

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Guidelines for Final Report

After tower testing is over and you begin to write the final reports, here are some guidelines to follow.

1. **Clarity of calculations**: Don't just show numbers but give equations and define variables. Make it legible. Either very neatly by hand or use an equation editor like in Microsoft Word. In Word, go to Insert->Object and select Microsoft Equation. In just a few minutes you should be able to get a hang of producing equations. It's pretty simple to use. If you use Excel make sure you label the equations – don't just show results.

2. **Quality of graphics**. You should have clear line-drawings from programs such as Illustrator, AutoCAD, or similar to produce dimensioned drawings of your models. If using Rhino, use the Make2D function to get clear illustrations. Photographs of your final model before and/or after testing will be required in addition to your drawings.

3. **Submit reports on 8-1/2" x 11" paper only**. Reports on 11x17 paper will not be accepted.

4. **Be clean, polished, and professional.** Write clearly, legibly, and with good grammar. Proofread your report before turning it in. Use appropriate professional language in your report. The mark of a good report is one that is easy to understand by someone not familiar with the project.

5. **Turn in the ORIGINAL graded copy of your Preliminary Report with your Final Report.**

6. In the Revised/Tested Tower section of the Final Report (as listed on the Tower Project Tally Sheet - Final Report Requirements), do all the listed calculations for your tower as tested. That is, you should be analyzing the tower that you actually built and tested. This is not a reiteration of the Preliminary Report. We expect that certain changes were made from the preliminary design in your final design.

7. In calculating the overall tower bucking (buckling of whole tower as opposed to individual member buckling), you should use the Moment of Inertia (I) for the tower as a whole. I is taken from the tower cross-section ignoring any cross bracing (only primary vertical members). Using that value for I, you then apply the Euler Bucking Equation, using K = 1.0 (this assumes the mass of the load has an inertial force that holds the top in place at the moment of buckling).

8. Mechanical properties for basswood, are given on the preliminary requirements sheet. If you used materials other than basswood, show what values you used for E, F and density. Cite your sources.

9. Throughout your report, check that your numbers are reasonable. If you get, for example, a predicted load capacity of 70 kips, you probably did something wrong.

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Group______________________________
Winter 2025

Tower Project Score Sheet

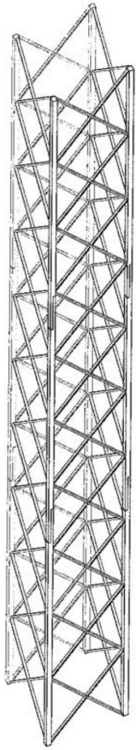
PRELIMINARY REPORT (re-submit with final report)	**40**	
TESTING	**60**	
Tower weight ≤ 4oz (15 pts); height = 48" (5 pts); holds ≥ 50 lbs (5 pts) Correct Materials (5 pts) (scaled if doesn't meet requirements)	30	
Efficiency (4/weight OZ)+(load LBS/50)+(load LBS/weight OZ)x1.5 (scaled based on class rank)	30	
FINAL REPORT REQUIREMENTS	**150**	
Preliminary Design Development	**20**	
How cross-sectional design of preliminary tower was chosen	4	
How elevation of preliminary tower was developed (e.g. bracing, taper, etc.)	4	
Why/how cross-section was or was not adjusted from preliminary report	4	
Why/how elevation of tower was or was not adjusted from preliminary report	4	
Discussion of how basic principles of columns supported these decisions	4	
Revised/Tested Tower Design Analysis [SHOW WORK AND UNITS!]	**50**	
Calculated/modeled axial forces and derivation of required member cross-sectional areas from axial forces (consider both crushing and buckling)	10	
Estimated weight calculation using actual member sizes used – include weight from members, glue, and gussets, etc.	7	
Member properties table: A, r, L, slenderness ratio (L/r), utilization ratio (actual load / allowable load)	7	
Indicate critical member (largest utilization ratio)	8	
Tower stability (as a whole) - buckling calculation	8	
Prediction of capacity of tower and mode of failure	10	
Illustration of Final/Tested Design	**20**	
Cross-section and elevations(s) of tower	5	
Perspective(s) or isometric of tower (no screenshots!)	5	
Overall dimensions labeled (height, width, etc.) with units	5	
Member sizes labeled (cross-sectional area, length of vertical members and cross-bracing) with units	5	
Testing Results	**30**	
Final weight and height of tower	6	
Tested capacity of tower	6	
Observations of testing (loading, any buckling observed, etc.)	6	
Description of mode of failure	6	
Images of failure	6	
Post-Testing Analysis	**30**	
Comparison of testing results with predicted capacity and modes of failure	10	
Discussion of discrepancies between results	10	
Suggested improvements for future designs with reasoning discussed	10	
FINAL GRADE	**250**	

(Note: re-submit your Preliminary Design Proposal with your Final Report.)

Tower project (April 18)

Tower Project: Final Report

59/60



Paul Ligeti & Yinying Chen
"Tower Group"
Structures II
03/28/2016

Testing Results/Post-Testing Analysis

Final weight of tower: 4.1 oz +2
Final height of tower: 50 in
Tested capacity of tower: 230 lbs +2

The tower continued to hold steady and stand up straight until the 200-pound mark. We had been placing weights on the tower in pairs (so 10 pounds at a time), and right before we got the tower to 230 pounds, it began to lean towards the bench, to the right (facing the bench from the camera). After placing the final 10 pounds, the tower leaned significantly more and snapped - all within a very fast timeframe of less than half a second.

+2

As shown in the picture below (figures 5 and 6), the tower buckled outwards towards the left (facing the bench from the camera), and inwards on the right side. What likely happened is that the back right column bore more than 1/4 of the weight - perhaps due to brick placement, perhaps due to craft or material deficiencies - and snapped prematurely as a consequence - it had reached its critical buckling load (*not* critical crushing, as we had expected)! Once that column was broken, the rest inevitably fell because now *they* had to split the 230 pounds evenly, as well as deal with bending and twisting.

+4

More specifically, the column snapped at the intersection of one of the notched connections of the back right column. This makes it likely that the main reason for buckling was both craft and the inherent nature of our notched connection.

Fig. 5: Outward Buckling

+4

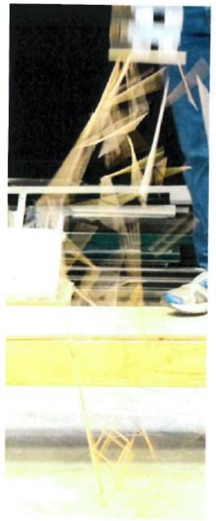


Fig. 6: Collapsing

Post-Testing Analysis

So why did we not meet our 848-lb goal? Due to the inevitable imperfections in craft, joints (both bracing and column notched connections), brick placement, material deficiencies (warping, knotting, etc.), and properties such as wood grain - which determine the integrity of the wood in certain axes - the tower did not hold the weight we expected. In fact, these properties make it incredibly likely that even under perfect environmental conditions - no humidity, a level ground, etc. - the 212 lb/column buckling capacity would have been impossible to achieve in any case.Rather, it held 230 pounds (which was still a significant amount, at 78% of the expected 296.56-lb crushing capacity)! In addition, these aforementioned factors, the tower ended up buckling, not crushing.

+2 +2

For future improvement, we could aim to make the aforementioned notched column connection stronger - either through a different methof of joining the three components of each column together, or additional support around the connection (such as a wrapping). Also the way the tower leaned suggests that there was an inbalance between the 4 columns, which caused one to bear more of the load. If we align all the columns better, it will carry more load.

+6

+20 bonus points

Complete your course and recitation evaluations for ARCH-324 to earn **20+ bonus points**! All you need to do is:

- **Finish both evaluations.**
- **Send me a quick screenshot of your completion confirmation.**

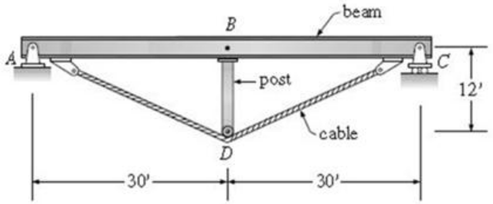
Email: arfazel@umich.edu

Pre-Post tensioning

Cable Trusses

- Reduce flexure stress
- Reduce deflection
- Produces stiffer section with less material
- Lighter weight
- Longer spans possible
- Analysis by combined stress

$$\boxed{f=-\frac{P}{A}\pm\frac{M}{S}\pm\left[\frac{Pe}{S}\right]}$$



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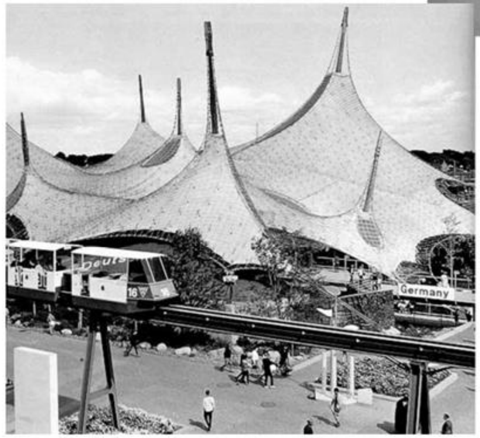
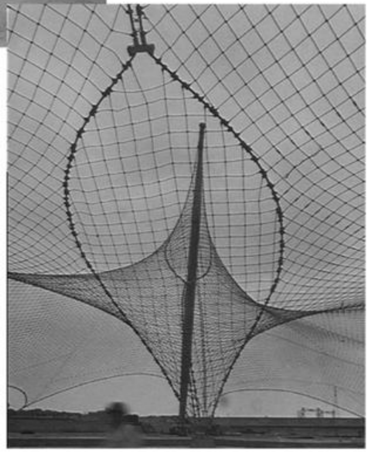
Expo '67, Montreal

Frei Otto
German Pavilion





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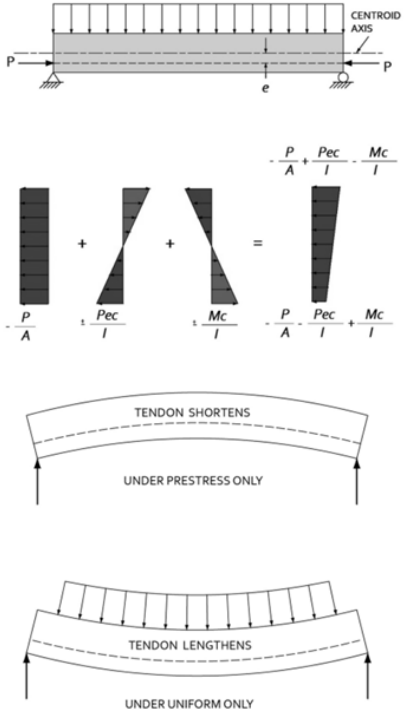
Pre-Post tensioning

Pre-stressed Concrete

- More concrete active in resisting moment
- Produces stiffer section with less material
- Lighter weight
- Longer spans possible
- Analysis by combined stress

$$f = -\frac{P}{A} \pm \frac{Pec}{I} \pm \frac{Mc}{I}$$



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Pre-stressed Concrete

Steel:
high strength wires 250 or 270 ksi
wire diameter 0.105 – 0.276
used in strands of bundled wire
most common is 7 wire strand

Concrete:
higher strength 5 – 10 ksi
to reduce creep and strain
reduced cracking
stiffer sections

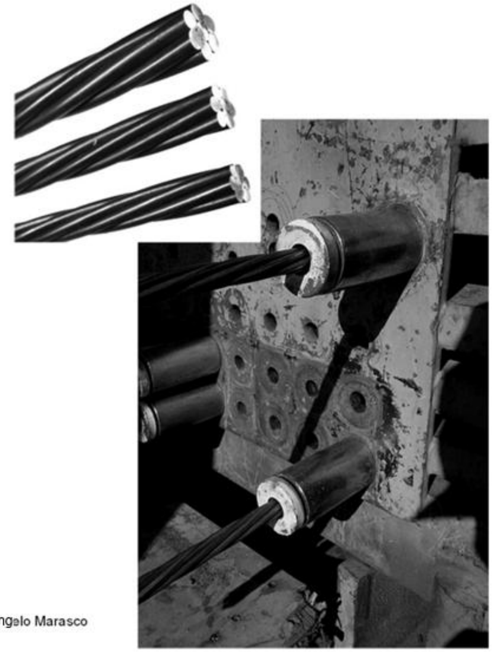

Photo by Angelo Marasco

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Composite Sections

Composite Design

Steel W section with concrete slab "attached" by shear studs.

The concrete slab acts as a wider and thicker compression flange.

Strength increase by 33% to 50%

Deflection reduced by 70% to 80%

Can attain either longer spans or smaller members – more economical in long spans

Smaller floor depth, therefore reduced overall building heights and weights

Reduced DL of system, reduction of other material vertically (façade, walls, plumbing, wiring, etc.)

Shear Studs

Also called Nelson studs after the company that originated them.

From AISC DigiLib

Can be spot welded through light gage decking onto W section

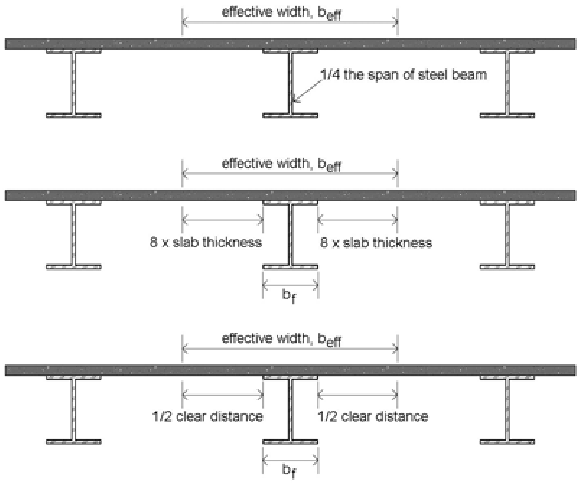
Composite Sections

Effective Flange Width, b_e

Slab on both sides:

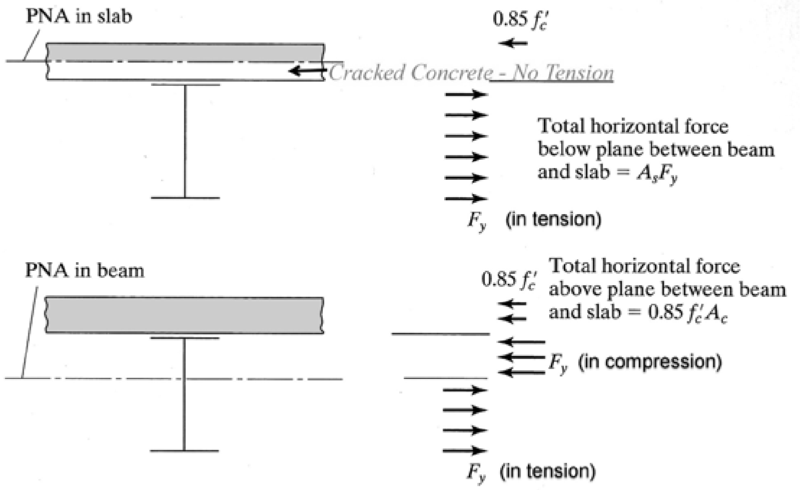
$\mathbf{b_e}$ is the **least** total width :

- Total width: ¼ of the beam span
- Overhang: 8 x slab thickness
- Overhang: ½ the clear distance to next beam (i.e. b_e is the web on center spacing)



Analysis Procedure (LRFD)

Case 1 – Plastic Neutral Axis (PNA) within slab
Case 2 – PNA within steel section



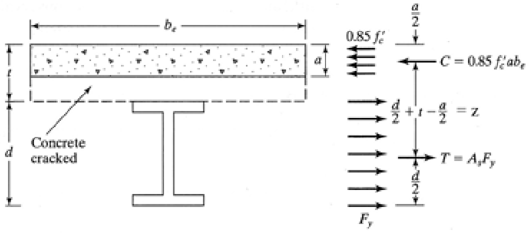
Composite Sections

Analysis Procedure (LRFD)

Case1 – PNA within slab

Given: Slab and beam geometry
W-section size and steel grade
(floor loads)

Find: pass/fail or capacities



1. Define effective flange width, b_e
2. Calculate the effective depth of the concrete stress block, a
3. If a is within concrete slab, the full steel section is in tension and:
   $Mp = T z$
   $Mn = Mp = As\ Fy\ (d/2 + t - a/2)$
4. $Mu \leq \phi\ Mn$

$$T = C$$

$$A_s f_y = 0.85 f'_c\, a\, b_e$$

$$a = \frac{A_s f_y}{0.85 f'_c\, b_e}$$

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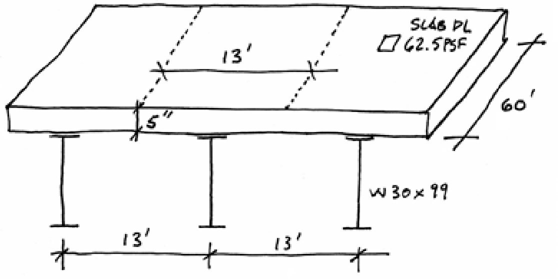
Non-composite vs. Composite Sections

Given:

- DL_{slab} = 62.5 psf = 812.5 plf
- DL_{beam} = 99 plf
- LL = ?
- W 30x99
- F_y = 50 ksi
- $f'c_{\ conc}$ = 4 ksi



Find: Load Capacity

For this example, floor capacity is found for two different floor systems:

1. Find capacity of steel section independent from slab

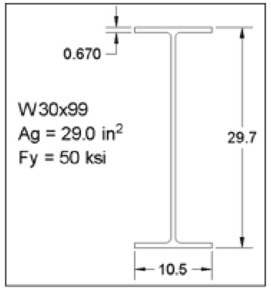
vs.

2. Find capacity of steel and slab as a composite section

WEIGHT of SLAB

$\frac{5''}{12}$ 150 PCF = 62.5 PSF

13' × 62.5 PSF = 812.5 PLF



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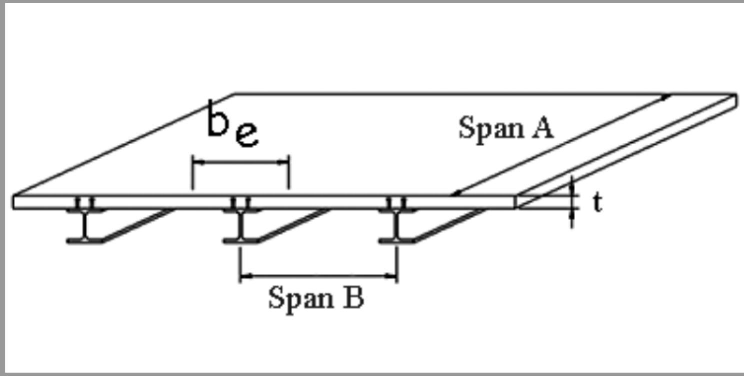
Problem Set 10

10. Composite Sections

Using the strength method, determine the required amount of flexural steel reinforcement, As, for the simple span beam (shown in section). The beam carries a dead and live floor load from a one-way slab in addition to its own self weight at 150 PCF. For the given bar size, determine the number of bars to obtain the required As. Check As,min and epsilon_t. Calculate the strength moment, Mn for the final beam design and check that phi Mn is > Mu.

DATASET: 1 -2- -3-

W-section	W16X77
span A	48 FT
span B	13 FT
slab thickness, t	5 IN
steel yield stress, Fy	50 KSI
concrete ultimate stress, f'c	6 KSI



Problem Set 10

#Q1: Effective width of the concrete flange, be

Using the strength method, determine the required amount of flexural steel reinforcement, As, for the simple span beam. The beam carries a dead and live floor load from a one-way slab in addition to its own self weight at 150 PCF. For the given bar size, determine the number of bars to obtain the required As. Check As,min and epsilon_t. Calculate the strength moment, Mn for the final beam design and check that phi Mn is > Mu.

DATASET: 1 -2- -3-

W-section	W16X77
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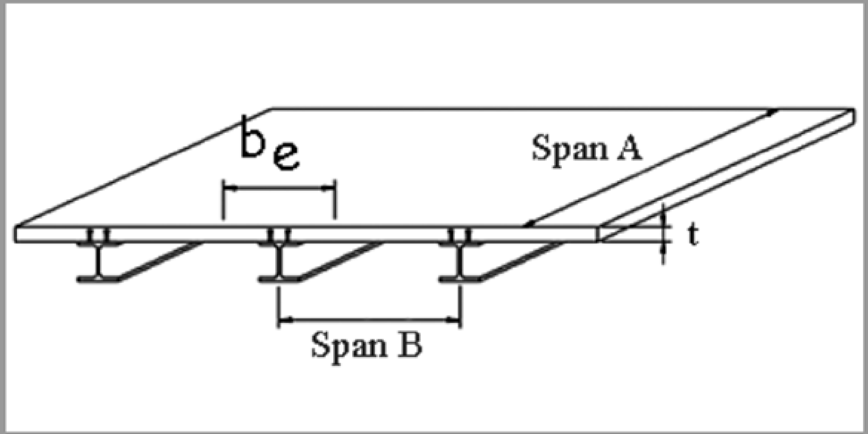
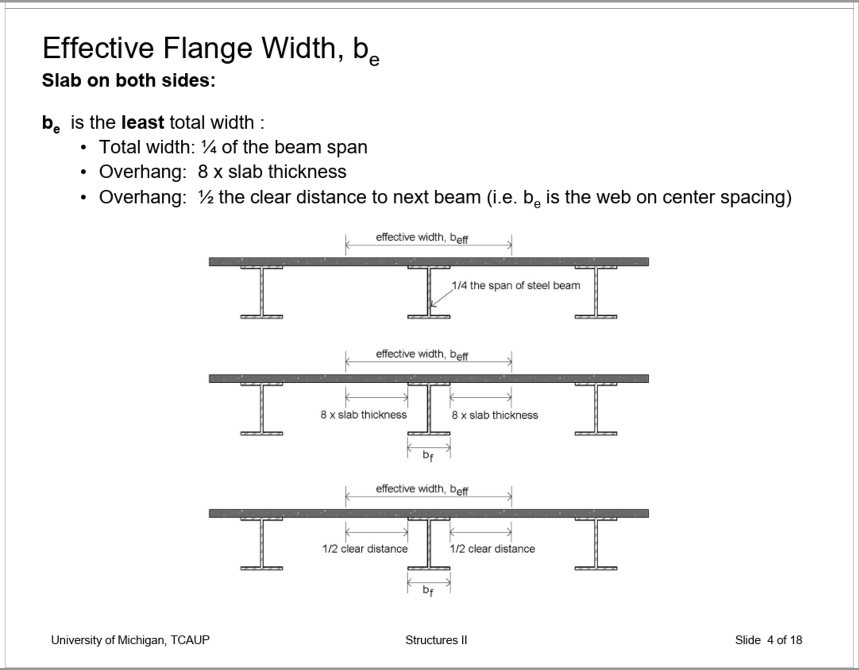




$$\boldsymbol{b_e = min \begin{cases} \frac{1}{4} BeamSpan \\ 2 \times 8 \times SlabThickness + b_f \\ 2 \times \frac{1}{2} ClearDistance \end{cases} = min \begin{cases} \frac{1}{4} \times 48 = 12\ FT = 144\ IN \\ 2(8 \times 5) + 10.3 = 90.3\ IN \surd \\ 2 \times \frac{1}{2}(13) = 13\ FT = 156\ IN \end{cases}}$$

Problem Set 10

#Q1: Effective width of the concrete flange, be

Using the strength method, determine the required amount of flexural steel reinforcement, As, for the simple span beam. The beam carries a dead and live floor load from a one-way slab in addition to its own self weight at 150 PCF. For the given bar size, determine the number of bars to obtain the required As. Check As,min and epsilon_t. Calculate the strength moment, Mn for the final beam design and check that phi Mn is > Mu.

DATASET: 1 -2- -3-

W-section	W16X77
span A	48 FT
span B	13 FT
slab thickness, t	5 IN
steel yield stress, Fy	50 KSI
concrete ultimate stress, f'c	6 KSI

$$b_e = min\begin{cases} \frac{1}{4} BeamSpan \\ 2 \times 8 \times SlabThickness + b_f \\ 2 \times \frac{1}{2} ClearDistance \end{cases} = min\begin{cases} \frac{1}{4} \times 48 = 12\ FT = 144\ IN \\ \mathbf{2(8 \times 5) + 10.3 = 90.3\ IN}\surd \\ 2 \times \frac{1}{2}(13) = 13\ FT = 156\ IN \end{cases}$$

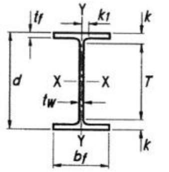


Table 1-1 (continued)
W-Shapes
Dimensions

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}	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	16.0	16	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		
W16×31[c]	9.13	15.9	15 7/8	0.275	1/4	1/8	5.53	5 1/2	0.440	7/16	0.842	1 1/8	3/4	13 5/8	3 1/2
×26[c,v]	7.68	15.7	15 3/4	0.250	1/4	1/8	5.50	5 1/2	0.345	3/8	0.747	1 1/16	3/4	13 5/8	3 1/2
W14×730[h]	215	22.4	22 3/8	3.07	3 1/16	1 9/16	17.9	17 7/8	4.91	4 15/16	5.51	6 3/16	2 3/4	10	3-7 1/2-3[g]
×665[h]	196	21.6	21 5/8	2.83	2 13/16	1 7/16	17.7	17 5/8	4.52	4 1/2	5.12	5 13/16	2 5/8		3-7 1/2-3[g]
×605[h]	178	20.9	20 7/8	2.60	2 5/8	1 5/16	17.4	17 3/8	4.16	4 3/16	4.76	5 7/16	2 1/2		3-7 1/2-3
×550[h]	162	20.2	20 1/4	2.38	2 3/8	1 3/16	17.2	17 1/4	3.82	3 13/16	4.42	5 1/8	2 3/8		
×500[h]	147	19.6	19 5/8	2.19	2 3/16	1 1/8	17.0	17	3.50	3 1/2	4.10	4 13/16	2 5/16		
×455[h]	134	19.0	19	2.02	2	1	16.8	16 7/8	3.21	3 3/16	3.81	4 1/2	2 1/4		
×426[h]	125	18.7	18 5/8	1.88	1 7/8	15/16	16.7	16 3/4	3.04	3 1/16	3.63	4 5/16	2 1/8		
×398[h]	117	18.3	18 1/4	1.77	1 3/4	7/8	16.6	16 5/8	2.85	2 7/8	3.44	4 1/8	2 1/8		
×370[h]	109	17.9	17 7/8	1.66	1 11/16	13/16	16.5	16 1/2	2.66	2 11/16	3.26	3 15/16	2 1/16		
×342[h]	101	17.5	17 1/2	1.54	1 9/16	13/16	16.4	16 3/8	2.47	2 1/2	3.07	3 3/4	2		
×311[h]	91.4	17.1	17 1/8	1.41	1 7/16	3/4	16.2	16 1/4	2.26	2 1/4	2.86	3 9/16	1 15/16		
×283[h]	83.3	16.7	16 3/4	1.29	1 5/16	11/16	16.1	16 1/8	2.07	2 1/16	2.67	3 3/8	1 7/8		
×257	75.6	16.4	16 3/8	1.18	1 3/16	5/8	16.0	16	1.89	1 7/8	2.49	3 3/16	1 13/16		
×233	68.5	16.0	16	1.07	1 1/16	9/16	15.9	15 7/8	1.72	1 3/4	2.32	3	1 3/4		
×211	62.0	15.7	15 3/4	0.980	1	1/2	15.8	15 3/4	1.56	1 9/16	2.16	2 7/8	1 11/16		
×193	56.8	15.5	15 1/2	0.890	7/8	7/16	15.7	15 3/4	1.44	1 7/16	2.04	2 3/4	1 11/16		
×176	51.8	15.2	15 1/4	0.830	13/16	7/16	15.7	15 5/8	1.31	1 5/16	1.91	2 5/8	1 5/8		
×159	46.7	15.0	15	0.745	3/4	3/8	15.6	15 5/8	1.19	1 3/16	1.79	2 1/2	1 9/16		
×145	42.7	14.8	14 3/4	0.680	11/16	3/8	15.5	15 1/2	1.09	1 1/16	1.69	2 3/8	1 9/16		

[c] Shape is slender for compression 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.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

Problem Set 10

#Q2: Depth of concrete stress block, a
#Q3: Is depth a within the slab? 1=yes, 0=no

Using the strength method, determine the required amount of flexural steel reinforcement, As, for the simple span beam. The beam carries a dead and live floor load from a one-way slab in addition to its own self weight at 150 PCF. For the given bar size, determine the number of bars to obtain the required As.

DATASET: 1 -2- -3-

W-section	W16X77
span A	48 FT
span B	13 FT
slab thickness, t	5 IN
steel yield stress, Fy	50 KSI
concrete ultimate stress, f'c	6 KSI

$$a = \frac{A_s f_y}{0.85 f'_c b_e} = \frac{22.6(50)}{0.85(6)(90.3)} = \mathbf{2.45\ IN}$$

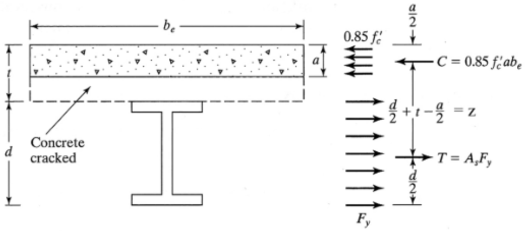
$$2.45 < 5 \Rightarrow \textbf{within the slab}$$

Analysis Procedure (LRFD)

Case1 – PNA within slab

Given: Slab and beam geometry
W-section size and steel grade
(floor loads)

Find: pass/fail or capacities



1. Define effective flange width, b_e
2. Calculate the effective depth of the concrete stress block, a
3. If a is within concrete slab, the full steel section is in tension and:
   Mp = T z
   Mn = Mp = As Fy (d/2 + t - a/2)
4. Mu ≤ ϕ Mn

$$T = C$$

$$A_s f_y = 0.85 f'_c\, a\, b_e$$

$$a = \frac{A_s f_y}{0.85 f'_c b_e}$$

Problem Set 10

#Q4: The nominal bending moment, Mn
#Q5: The factored bending resistance, phi Mn
#Q6: The factored design moment, Mu

DATASET: 1 -2- -3-

W-section	W16X77
span A	48 FT
span B	13 FT
slab thickness, t	5 IN
steel yield stress, Fy	50 KSI
concrete ultimate stress, f'c	6 KSI

$$d = 16.5\ IN$$
$$t = 5\ IN$$
$$a = 2.45\ IN$$

$$\boldsymbol{M_n = A_s f_y\left(\frac{d}{2} + t - \frac{a}{2}\right)} = 22.6(50)\left(\frac{16.5}{2} + 5 - \frac{2.45}{2}\right) = \mathbf{13{,}588.25\ K-IN}$$

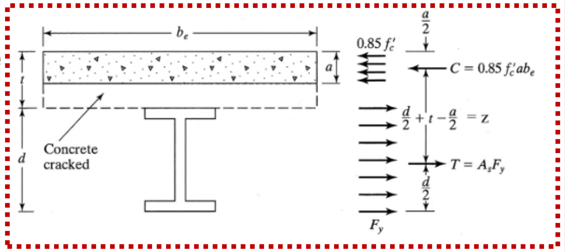
$$\boldsymbol{\varphi M_n = 0.9\ M_n} = 0.9\ (13{,}588.25) = \boldsymbol{12{,}229.425\ K-IN}$$

$$\boldsymbol{M_u} = \frac{\boldsymbol{\varphi M_n}}{12} = \frac{12{,}229.425}{12} = \boldsymbol{1019.11\ K\_FT}$$

Analysis Procedure (LRFD)
Case1 – PNA within slab

Given: Slab and beam geometry
W-section size and steel grade
(floor loads)

Find: pass/fail or capacities



1. Define effective flange width, b_e
2. Calculate the effective depth of the concrete stress block, a
3. If a is within concrete slab, the full steel section is in tension and:
   Mp = T z
   Mn = Mp = As Fy (d/2 + t - a/2)
4. Mu ≤ ϕ Mn

$T = C$

$A_s f_y = 0.85 f'_c\, a\, b_e$

$a = \frac{A_s f_y}{0.85 f'_c\, b_e}$

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Problem Set 10

#Q7: The total factored design load, wu
#Q8: The selfweight of the concrete slab

DATASET: 1 -2- -3-

W-section	W16X77
span A	48 FT
span B	13 FT
slab thickness, t	5 IN
steel yield stress, Fy	50 KSI
concrete ultimate stress, f'c	6 KSI

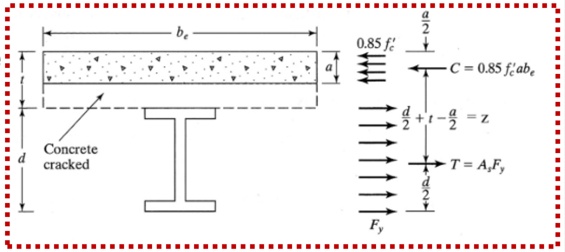
$$\boldsymbol{M_u = \frac{W_u L^2}{8} \rightarrow W_u = \frac{8M_u}{L^2}} = \frac{8(1019.11)}{48^2} = \boldsymbol{3.53\ KLF}$$

$$\boldsymbol{Selfweight_{Slab}} = t \times 1 \times 1 \times Density = \frac{5}{12}(150) = \boldsymbol{62.5\ PSF}$$

Analysis Procedure (LRFD)
Case1 – PNA within slab

Given: Slab and beam geometry
W-section size and steel grade
(floor loads)

Find: pass/fail or capacities



1. Define effective flange width, b_e
2. Calculate the effective depth of the concrete stress block, a
3. If a is within concrete slab, the full steel section is in tension and:
   Mp = T z
   Mn = Mp = As Fy (d/2 + t - a/2)
4. Mu ≤ ϕ Mn

$T = C$

$A_s f_y = 0.85 f'_c\, a\, b_e$

$a = \frac{A_s f_y}{0.85 f'_c\, b_e}$

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Problem Set 10

#Q9: The total (steel+concrete) unfactored dead load on the beam, w_DL

DATASET: 1 -2- -3-

W-section	W16X77
span A	48 FT
span B	13 FT
slab thickness, t	5 IN
steel yield stress, Fy	50 KSI
concrete ultimate stress, f'c	6 KSI

$$\boldsymbol{D_L = D_{l_{slab}} + D_{L_{Beam}}}$$

$$\boldsymbol{D_{l_{slab}}} = 62.5 \text{ PSF } (13 \text{ FT}) = 812.5 \text{ PLF} = \mathbf{0.8125\ KLF}$$

$$\boldsymbol{D_{l_{Beam}}} \ (\textbf{for W } \mathbf{16 \times 77}) = \boldsymbol{77\ PLF = 0.077\ KLF}$$

$$\Rightarrow \boldsymbol{D_L = 0.8895\ KLF}$$

Problem Set 10

#Q10: The actual, unfactored beam live load (capacity), w_LL
#Q11: The actual floor live load (floor capacity), LL

DATASET: 1 -2- -3-

W-section	W16X77
span A	48 FT
span B	13 FT
slab thickness, t	5 IN
steel yield stress, Fy	50 KSI
concrete ultimate stress, f'c	6 KSI

$$\boldsymbol{W_u = 1.2\,DL + 1.6LL} \rightarrow 3.53\text{ KLF} = 1.2\,(0.8895) + 1.6(\text{LL}) \Rightarrow \boldsymbol{LL = 1.544\,KLF}$$

$$\boldsymbol{FloorLL} = \frac{1.544\,KLF}{13FT} = 0.11880\,KSF = \boldsymbol{118.8\,PCF}$$

Lab08

Structures II

Arch 324

Name 1 ____________________

Name 2 ____________________

Name 3 ____________________

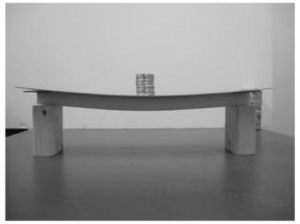
Composite Sections

Description

This project allows the students to observe the difference in stiffness between Composite and Non-Composite beam slab combinations.

Goals

To observe the bending behavior of non-connected beams and slabs
To observe the bending behavior of a composite section.
To compare the deflection of the two systems.



Procedure

1. Place the chipboard slab on the foam beam but do not attach the end clips.
2. Place the 10 washer weights in the center and measure the deflection.
3. Repeat the procedure but now with the ends of the slab and the beam clipped together.
4. Again, measure the deflection.
5. Compare the deflections of the two systems.

Due

During recitation

Lab08

→ 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*