Structure II Recitation 2/9

Steel Beam Analysis

Before we start …

Today's Tasks

1. Homework Example (Steel Beam Analysis) (17 Questions)
2. Lab (Steel Beams)

Reminder

1. **Don't forget to start your report!**
2. Scale besides Peter's office if needed.

Tower Project Brief 2024

Prelim Report Guidlines 2024

Final Report Guidlines 2024

Score Sheet 2024

Study of Tower Types

Example Reports

Dr. Frame Software (download)

Dr. Frame Tutorials

STAAD example

Videos of Old Tower Tests

Go to Structure Website >> Tower 1

Architecture 324
Structures II

Prof. Peter von Buelow
Winter 2024

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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Tower Project – Preliminary Report Requirements

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

2 **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.

3 **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.

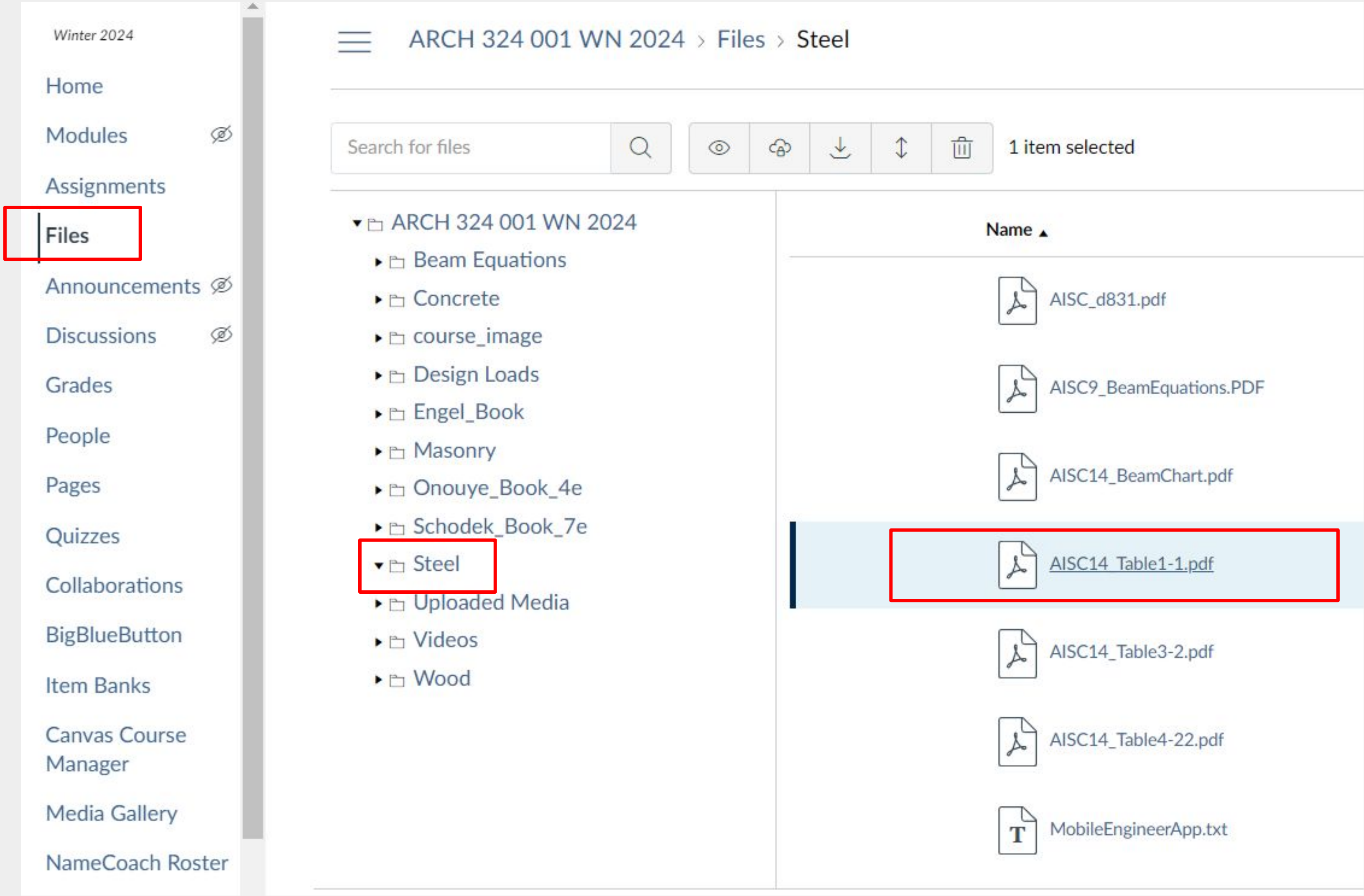
Format - Reports should be formatted for **8½ X 11** paper. 11X17 format reports will not be accepted. Once returned to you graded, **save the original copy of the preliminary report** for submission together with the Final Report.

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)	20 pcf *
E (buckling)	1,650,000 psi **
F (Compression ∥ to grain)	4745 psi *
F (Compression ⊥ to grain)	377 psi *
F (Tension ∥ to grain)	4500 psi (estimate)
F (Tension ⊥ to grain)	348 psi *
F (Shear ∥ to grain)	986 psi *
F (Flexure)	5900 psi *

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

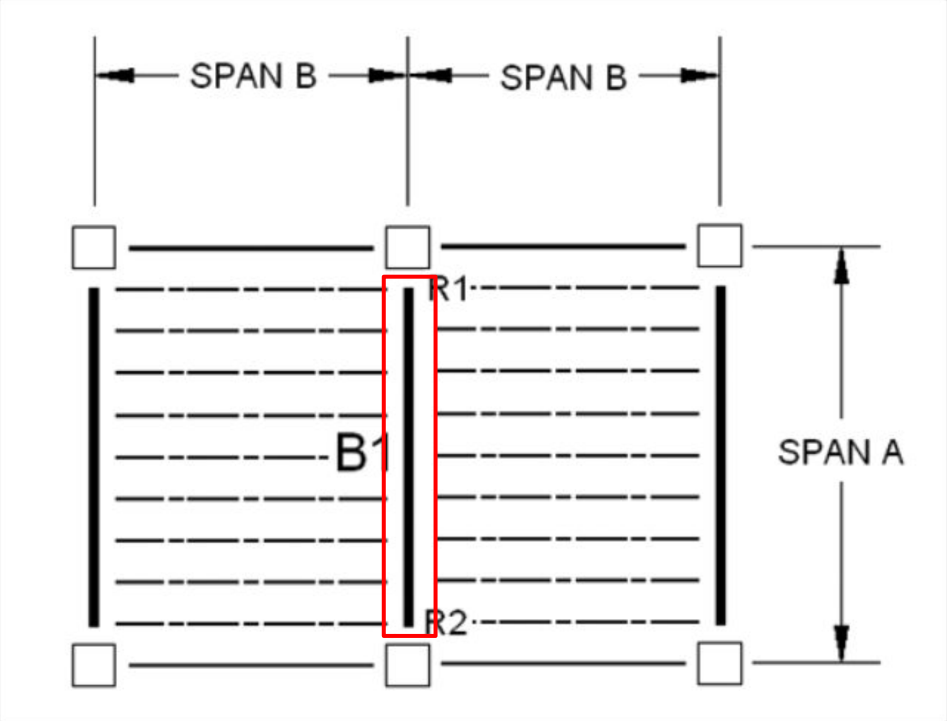

Winter 2024
Home
Modules
Assignments
Files
Announcements
Discussions
Grades
People
Pages
Quizzes
Collaborations
BigBlueButton
Item Banks
Canvas Course Manager
Media Gallery
NameCoach Roster
ARCH 324 001 WN 2024 › Files › Steel
Search for files
1 item selected
ARCH 324 001 WN 2024
Beam Equations
Concrete
course_image
Design Loads
Engel_Book
Masonry
Onouye_Book_4e
Schodek_Book_7e
Steel
Uploaded Media
Videos
Wood
Name
AISC_d831.pdf
AISC9_BeamEquations.PDF
AISC14_BeamChart.pdf
AISC14_Table1-1.pdf
AISC14_Table3-2.pdf
AISC14_Table4-22.pdf
MobileEngineerApp.txt

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	W12X45
Fy	50 KSI
Span A	25 FT
Span B	12 FT
Floor DL	18 PSF



1. Find out LL, 2. Compare Shear, 3. Compare Deflection

Example – Analysis of Steel Beam - **Capacity**

Find applied live load capacity, w_{LL} in KLF

$w_u = 1.2w_{DL} + 1.6w_{LL}$

w_{DL} = beam + floor = 44plf + 1500plf

Fy = 50 ksi, Fully Braced

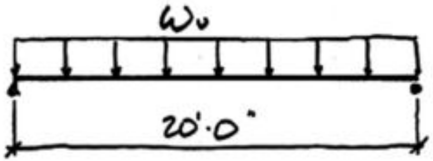
1. Find the Plastic Modulus (Z_x) for the given section from the AISC table 1-1
2. Check that $L_b < L_p$ (fully braced – ok)
3. Determine $M_n = M_p = F_y\ Z_x$
4. Set $M_u = \phi_b\ M_n$

   $\phi_b = 0.90$

GIVEN: F_y = 50 ksi

W21x44

Fully Braced



For a W21x44 from Table

Z_x = 95.4 in^3

$M_n = F_y Z_x$ = 50 ksi × 95.4 = 4,770 k-in

$M_u = \phi_b \cdot M_n$ = 0.9 × 4,770 k-in

M_u = 4,293 k-in = 357.75 kft

Example – Analysis of Steel Beam - **Capacity**

6. Using the maximum moment equation, solve for the factored distributed loading, w_u

$$M_U = \frac{w_U l^2}{8} \Rightarrow w_U = \frac{8 M_U}{l^2}$$

$$w_U = \frac{8 \times 357.75 \text{ kFT}}{20_{FT}^2}$$

$$w_U = 7.155 \text{ k/FT}$$

7. The applied (unfactored) load
$w = w_u / (\gamma \text{ factors})$
$w_u = 1.2wDL + 1.6wLL$

$$w_U = 7.155 \text{ KLF} = 1.2(\overbrace{0.044 + 1.5}^{w_{DL}}) + 1.6(w_{LL})$$

$$w_U = 1.853 + 1.6 w_{LL} = 7.155 \text{ KLF}$$

$$w_{LL} = 3.31 \text{ KLF}$$

Given from Question

W-section	W12X45
Fy	50 KSI

Q1: Plastic Modulus of the Section (Zx)

Check AISC 14, Table 1-1, for me p.8(PDF):

For W12 x 45, Zx = **<u>64.2 in^3</u>**

- Plastic Behavior (zone 1)

$\boxed{M_n = M_p = F_y Z}$

  - Braced against LTB ($L_b < L_p$)

Q2: Nominal Bending Moment (Mn)

Since its full bracing (Zone 1):

Mn = (Zx) x (Fy) = 64.2 x 50 = **<u>3210 k-in</u>**

AISC 14, Table 1-1

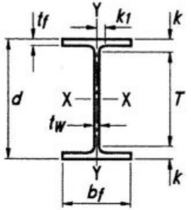


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W12×58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1 1/2	15/16	9 1/4	5 1/2
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15/16	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/8	1.14	1 1/2	15/16	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 3/8	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 3/8	7/8	↓	↓

Table 1-1 (continued)
W-Shapes
Properties

W12-W10

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}
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440

Q3: The Factored Bending Resistance (Φ Mn)

$\Phi = 0.9$

Φ Mn = Mn x 0.9 = 3210 x 0.9 = **<u>2889 k-in</u>**

Q4: The Factored Design Moment (Mu)

We assume Mu = Φ Mn in order to find our LL:

Mu = (Φ Mn) = 2889 / 12 = **<u>240.75 k-ft</u>**

Convert Unit

Q5: The Total Factored Design Load (wu)

Moment for uniformly distributed load = (w x L^2 / 8)

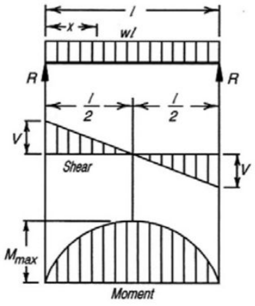
wu = Mu x 8 / L^2 = 240.75 x 8 / 25^2 = **<u>3.0816 klf</u>**

Given from Question

W-section	W12X45
Fy	50 KSI
Span A	25 FT
Span B	12 FT
Floor DL	18 PSF

$$\phi M_n = 0.90\ M_n$$

1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . . . $= wl$

$R = V$ . . . . $= \frac{wl}{2}$

V_x . . . . $= w\left(\frac{l}{2} - x\right)$

$\boxed{M_{max} \text{ (at center)} \ldots = \frac{wl^2}{8}}$

M_x . . . . $= \frac{wx}{2}(l - x)$

Δ_{max} (at center) . . . . $= \frac{5wl^4}{384EI}$

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

Q6: The Total Unfactored Dead Load (w_DL)

<u>Consider: Beam Self Weight + Dead Load</u>

For Beam Self Weight:
Check AISC 14, Table 1-1 for Nominal Weight,
For my situation, self weight = 45 lb/ft

For Dead Load:
Floor DL x Tributary Area / Beam Length (Span A)
= Floor DL x Span B = 18 x 12 = 216 lb/ft

Convert Unit (pounds to kips)

Total:
(45 + 216) / 1000 = **<u>0.261 klf</u>**

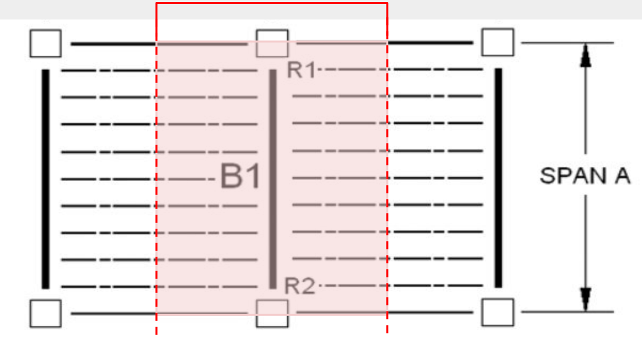
Q7: The Total Factored Dead Load (wu_DL)

wu_DL = w_DL x 1.2 = 0.261 x 1.2 = **<u>0.3132 klf</u>**

Given from Question

W-section	W12X45
Fy	50 KSI
Span A	25 FT
Span B	12 FT
Floor DL	18 PSF

Span B



$$w_u = \boxed{1.2w_{DL}} + 1.6w_{LL}$$

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.
W12×58	17.0	12.2	12¼	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1½	15/16	9¼	5½
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1⅜	15/16	9¼	5½
W12×50	14.6	12.2	12¼	0.370	3/8	3/16	8.08	8⅛	0.640	5/8	1.14	1½	15/16	9¼	5½
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1⅜	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	½	1.02	1⅜	7/8	↓	↓

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}
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440

Q8: The Factored Live Load (wu_LL)

wu_LL = wu - wu_DL = 3.0816 - 0.03132 = **2.7684 klf**

From Q5 From Q7

Q9: The Actual Beam Live Load (w_LL)

w_LL = wu_LL / 1.6 = 2.7684 / 1.6 = **1.73025 klf**

Q10: The Actual Floor Live Load (LL)

LL = w_LL / Span B

= (1.73025 / 12) x 1000 = **144.1875 psf**

Convert Unit (kips to pounds)

Given from Question

W-section	W12X45
Fy	50 KSI
Span A	25 FT
Span B	12 FT
Floor DL	18 PSF

$$w_u = \underbrace{\boxed{1.2w_{DL}}}_{\text{wu\_DL}} + \underbrace{\boxed{1.6w_{LL}}}_{\text{wu\_LL}}$$

Still Alive?

Q11: The Maximum Factored Design Beam Shear Force (Vu_max):

Vu_max = wu x L (Span A) /2 = 3.0816 x 25 /2 = <u>**38.52 k**</u>

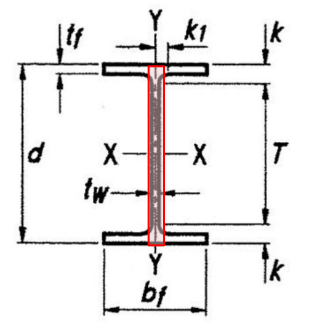
From Q5

Q12: The Web Area (Aw):

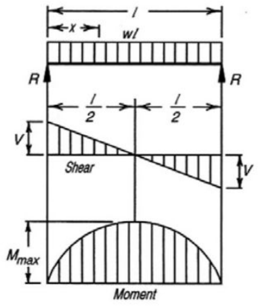
Formula: Aw = d x tw

Check AISC 14, Table 1-1 for d & tw:

Aw = 12.1 x 0.335 = <u>**4.0535 in²**</u>



1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . . . $= wl$

$\boxed{R = V \quad . . . . \quad = \frac{wl}{2}}$

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

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.
W12×58	17.0	12.2	12¼	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1½	15/16	9¼	5½
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1⅜	15/16	9¼	5½
W12×50	14.6	12.2	12¼	0.370	3/8	3/16	8.08	8⅛	0.640	5/8	1.14	1½	15/16	9¼	5½
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1⅜	15/16	↓	↓
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Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	Torsional Properties J	Torsional Properties C_w
lb/ft			in.⁴	in.³	in.	in.³	in.⁴	in.³	in.	in.³	in.	in.	in.⁴	in.⁶
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440

Q13: The Factored Shear Resistance (phi Vn)

Check my h/tw ,

Since my h/tw = 29.6 < 59,

phi Vn = 0.6 x Fy x Aw = 0.6 x 50 x 4.0535 = **121.605 k**

From Q12

Q14: Is the section safe for shear?

Check if (phi Vn) > (Vu_max),

From Q13 From Q11

If yes = Pass,

If no = Fail,

For my situation:

121.605 (phi Vn) > 38.52 (Vu_max), **It's a Pass!**

Given from Question

W-section	W12X45
Fy	50 KSI
Span A	25 FT
Span B	12 FT
Floor DL	18 PSF

AISC 14, Table 1-1

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440

Zone 1:

WEB YIELDING (Most beam sections fall into this category)

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

$\boxed{\text{then: } V_n = 0.6 F_y A_w}$

Zone 2:

INELASTIC WEB BUCKLING

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

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

Zone 3:

ELASTIC WEB BUCKLING

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

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

Course Slides p.5

Q15: The Actual (Unfactored) Deflection Due to Total DL+LL

Deflection = (5 x w x L^4) / (384 x E x I)

(E = 29000 ksi)

(I: Check AISC 14, Table 1-1, I = 348)

(L: Given from question (Span A))

(w = w_LL +w_ DL = 1.73025 + 0.261 = 1.991)

Unfactored Loads (Q9+Q6)

Actual Deflection

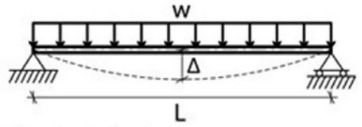
= (5 x 1.991 x 25^4) / (384 x 29000 x 348) x $(12)^3$

=**<u>1.734 in</u>**

Convert Unit

AISC 9, Beam Equations, p.2 (PDF)

E = Modulus of Elasticity of steel at 29,000 ksi.

Beam Load and Support	Actual Deflection*
 (a) Uniform load, simple span	$\boxed{\Delta_{max} = \frac{5\omega L^4}{384EI}}$ (at the centerline)

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
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45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440

AISC 14, Table 1-1

Q16: The Deflection Limit (L/180)

L/180 = 25(Span A) x 12 /180 = **<u>1.667 in</u>**

Convert Unit

Q17: Maximum Allowable Axial Load Capacity (Pmax)

Check if the actual deflection (Q15) is smaller than the deflection limit (Q16),

If yes = Pass, If no = Fail,

For my situation: 1.734 > 1.667, **It's a Fail!**

Questions?

Lab Session:

Goals:

Compare the load capacity of a free edge member in different orientations.

Steps:

1. Record the total number of washers each orientation holds when it fails.

2. Compare which one carries more load.

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.

1.

2.