ARCHITECTURE 324
Structures II

Recitation 06

Sections 04&05

Instructor
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GSI
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Feb 21, 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

→ Summary

→ Steel column analysis

→ Steel column design

→ Problem Set

→ Problem set 05 (Steel beam design)

→ Lab

→ Steel Columns

→ Tower project

→ Prelim tower report

(We will send out the reviews next Friday, February 28th)

Steel Column Analysis

Leonhard Euler (1707 – 1783)

Euler Buckling (elastic buckling)

$$P_{cr} = \frac{\pi^2 AE}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 IE}{(KL)^2}$$

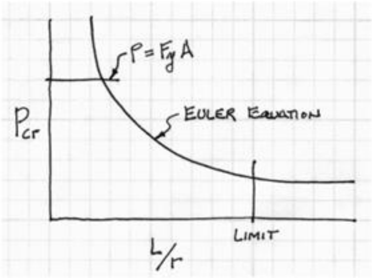
$$r = \sqrt{\frac{I}{A}}$$

$$I = Ar^2$$

portrait by Emanuel Handmann,1753

- A = Cross sectional area (in^2)
- E = Modulus of elasticity of the material (lb/in^2)
- K = Stiffness (curvature mode) factor
- L = Column length between pinned ends (in.)
- r = radius of gyration (in.)

$$f_{cr} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \le F_{cr}$$

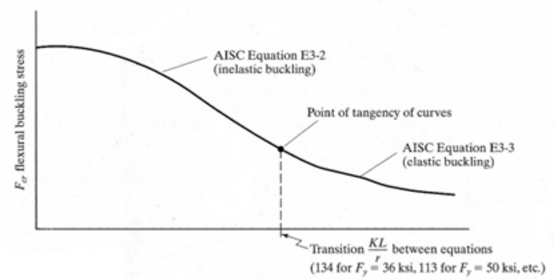


Analysis of Steel Columns

Short columns
Fail by material crushing
Plastic behavior

Intermediate columns
Crush partially and then buckle
Inelastic behavior
Local buckling – flange or web
Flexural torsional buckling - twisting

Long columns
Fail in Euler buckling
Elastic behavior



$$slenderness = \frac{KL}{r}$$

short intermediate long

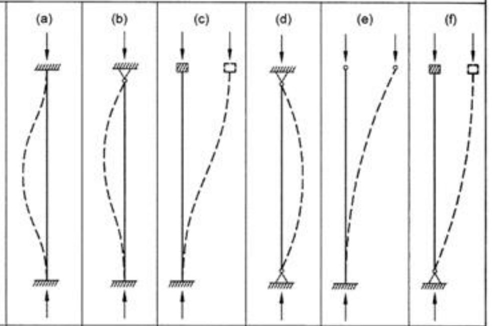
Transition Slenderness $4.71\sqrt{\frac{E}{F_y}}$

Steel Column Analysis

Analysis of Steel Columns

Estimate of K:

TABLE C-A-7.1
Approximate Values of Effective Length Factor, *K*

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.1	2.0
End condition code	Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

Analysis of Steel Columns

pass / fail by LRFD

Data:
- Column – <u>size</u>, length
- Support conditions
- Material properties – F_y
- Factored load – P_u

Required:
- $P_u \leqq \phi P_n$ (pass)

1. Calculate slenderness ratios: L_c/r_x and L_c/r_y (L_c = KL)
   The largest ratio governs.
2. Check slenderness ratio against upper limit of 200 (recommended)
3. Calculate transition slenderness $4.71\sqrt{E/Fy}$ and determine column type (short or long)
4. Calculate F_{cr} based on slenderness
5. Determine ϕP_n and compare to P_u
   $P_n = F_{cr}\ A_g$ $\quad \phi = 0.9$
6. If $P_u \leqq \phi P_n$, then OK

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \quad \text{Short}$$

$$F_{cr} = 0.877 F_e \quad \text{Long}$$

Steel Column Design

Design of Steel Columns with AISC Strength Tables

Data:
- Column – length
- Support conditions
- Material properties – F_y
- Applied design load - P_u

Required:
- Column Size

1. Enter table with height, KL = Lc
2. Read allowable load for each section to find the smallest adequate size.
3. **Tables assume weak axis buckling. If the strong axis controls the length must be divided by the ratio r_x/r_y**
4. Values stop in table (black line) at slenderness limit, KL/r = 200

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes
W8
F_y = 50 ksi

Shape	W8×											
lb/ft	67		58		48		40		35		31	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	590	886	512	769	422	634	350	526	308	463	273	411
6	542	815	470	706	387	581	320	481	281	423	249	374
7	526	790	455	685	375	563	309	465	272	409	241	362
8	508	763	439	660	361	543	298	448	262	394	232	348
9	488	733	422	634	347	521	285	429	251	377	222	333
10	467	701	403	606	331	497	272	409	239	359	211	317
11	444	668	384	576	314	473	258	388	226	340	200	301
12	421	633	363	546	297	447	243	366	213	321	189	283
13	397	597	342	514	280	421	228	343	200	301	177	266
14	373	560	321	482	262	394	213	321	187	281	165	248
15	348	523	299	450	244	367	198	298	174	261	153	230
16	324	487	278	418	226	340	183	275	160	241	141	212
17	300	450	257	386	209	314	169	253	147	221	130	195
18	276	415	236	355	192	288	154	232	135	203	118	178
19	253	381	216	325	175	264	141	211	123	184	108	162
20	231	347	197	296	159	239	127	191	111	166	97.2	146
22	191	287	163	244	132	198	105	158	91.5	138	80.3	121
24	160	241	137	205	111	166	88.2	133	76.9	116	67.5	101
26	137	205	116	175	94.2	142	75.2	113	65.5	98.5	57.5	86.5
28	118	177	100	151	81.2	122	64.8	97.4	56.5	84.9	49.6	74.5
30	103	154	87.5	131	70.7	106	56.5	84.9	49.2	74.0	43.2	64.9
32	90.3	136	76.9	116	62.2	93.5	49.6	74.6	43.3	65.0	38.0	57.1
34	79.9	120	68.1	102	55.1	82.8	44.0	66.1				
Properties												
P_{wo}, kips	126	190	102	153	72.0	108	57.2	85.9	45.9	68.9	39.4	59.1
P_{wi}, kip/in.	19.0	28.5	17.0	25.5	13.3	20.0	12.0	18.0	10.3	15.5	9.50	14.3
P_{wb}, kips	507	761	363	546	174	262	127	192	81.1	122	63.0	94.7
P_{fb}, kips	164	246	123	185	87.8	132	58.7	88.2	45.9	68.9	35.4	53.2

Properties	W8×67	W8×58	W8×48	W8×40	W8×35	W8×31
L_p, ft	7.49	7.42	7.35	7.21	7.17	7.18
L_r, ft	47.6	41.6	35.2	29.9	27.0	24.8
A_g, in.^{2}	19.7	17.1	14.1	11.7	10.3	9.13
I_x, in.^{4}	272	228	184	146	127	110
I_y, in.^{4}	88.6	75.1	60.9	49.1	42.6	37.1
r_y, in.	2.12	2.10	2.08	2.04	2.03	2.02
r_x/r_y	1.75	1.74	1.74	1.73	1.78	1.72
$P_{ex}L_c^2/10^4$, k-in.^{2}	7790	6530	5270	4180	3630	3150
$P_{ey}L_c^2/10^4$, k-in.^{2}	2540	2150	1740	1410	1220	1060

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Steel Connections
Shop vs. Field Connections

Shop Connections:

- Welding preferably performed in the shop as opposed to the field due to controlled environment
- Members can be positioned for more economical welding (welding upside down is difficult)
- Welding may have an equipment advantage in the shop
- Shops use both welding and bolting

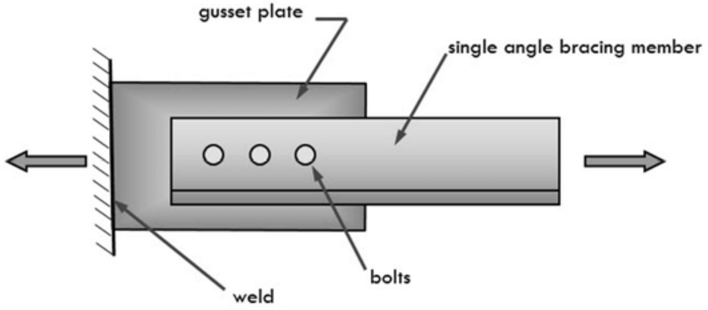
Field Connections:

- Bolting easily performed in the field and generally preferred when possible
- Bolting provides a method to erect the members and release the crane hook quickly

Connections

Tension Connection – Angle Failure example

1. Tensile Yielding
2. Tensile Rupture
3. Block Shear
4. Bearing and Tearout at Bolt Holes
5. Bolt Shear
6. Bearing and Tearout at Bolt Holes
7. Block Shear
8. Tensile Rupture
9. Tensile Yielding
10. Tension Rupture in Weld



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Steel Frame Construction

University of Michigan – North Quad

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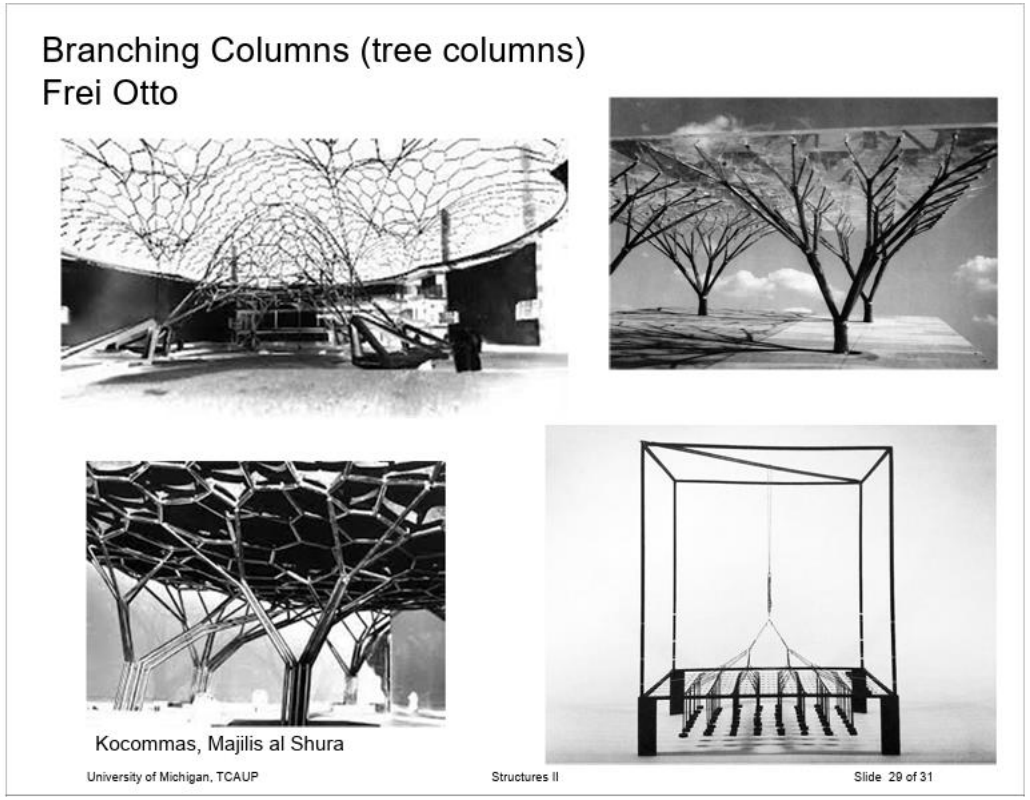
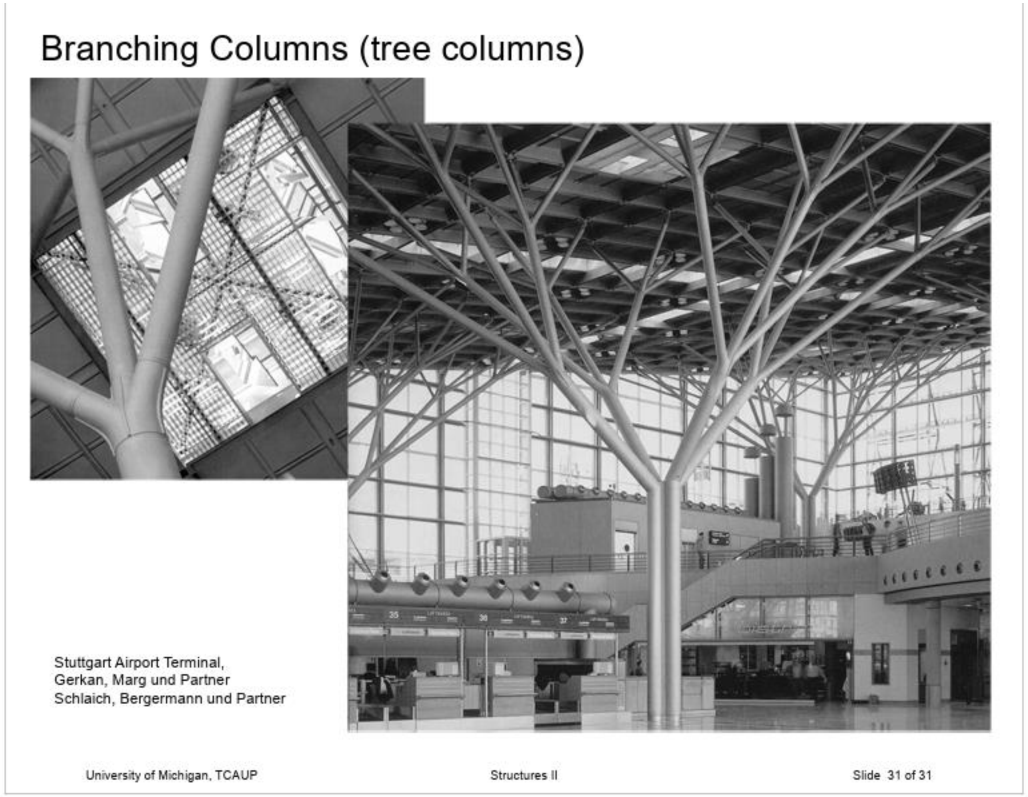
Steel Frame Construction





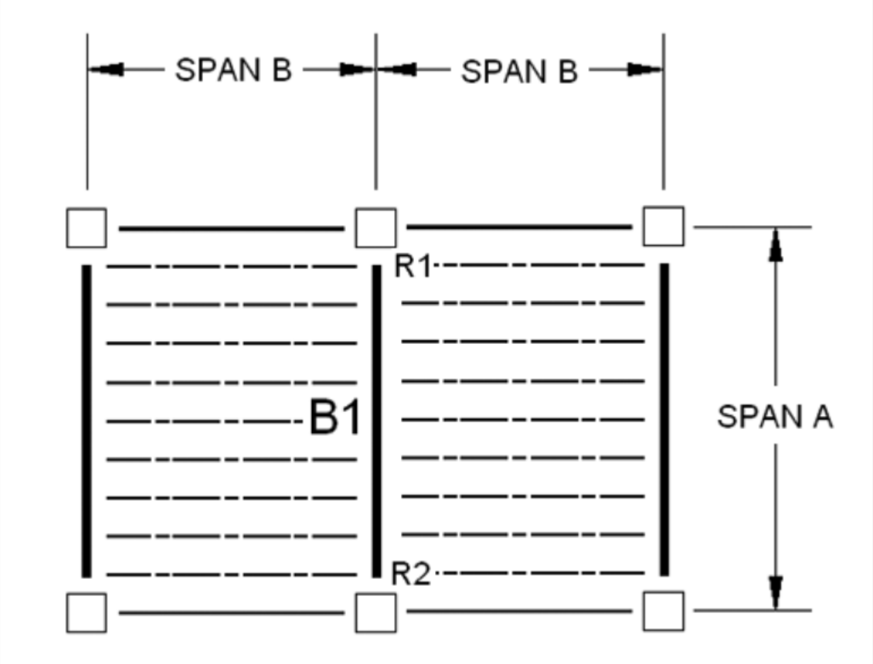
Problem Set 05

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	27 FT
Span B	14 FT
Floor Dead Load	19 PSF
Floor Live Load	90 PSF



Problem Set 05

#Q1: The unfactored floor dead load on beam B1 (neglecting selfweight), w_DL*

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	27 FT
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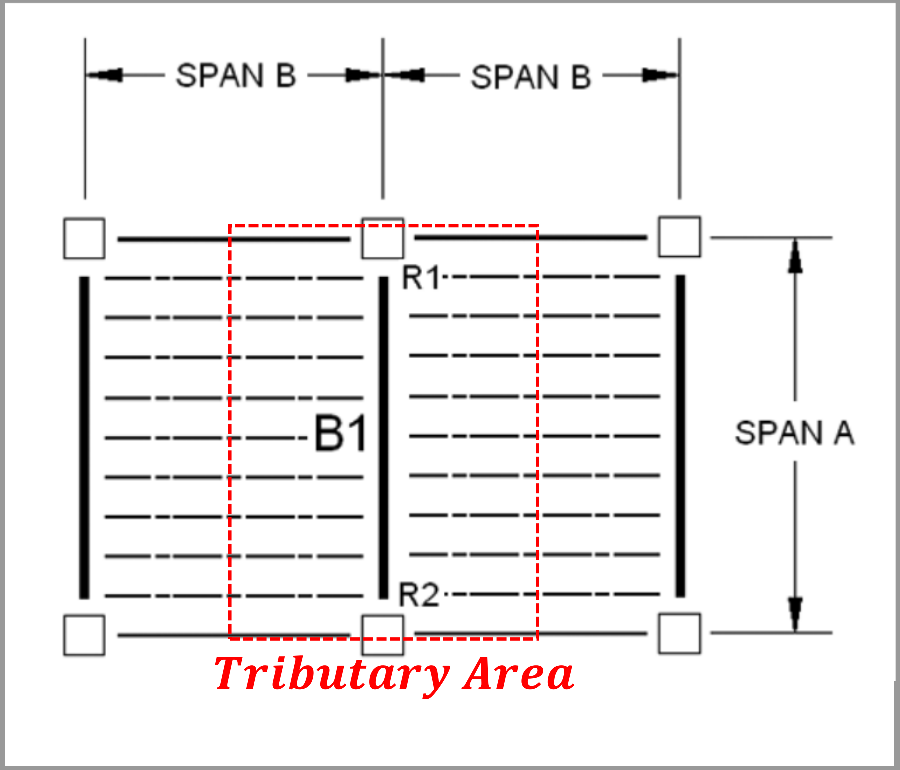
Floor Dead Load	19 PSF
Floor Live Load	90 PSF



$$Tributary\ Area = \mathbf{spanA}\left(\frac{\mathbf{spanB}}{\mathbf{2}} + \frac{\mathbf{spanB}}{\mathbf{2}}\right) = 27(14) = \mathbf{378\ FT^2}$$

$$W_{DL*} = Floor_{DL}\left(\frac{Tributary\ Area}{\mathbf{spanA}}\right) = 19\left(\frac{378}{27}\right) = \mathbf{266\ PLF}$$

Problem Set 05

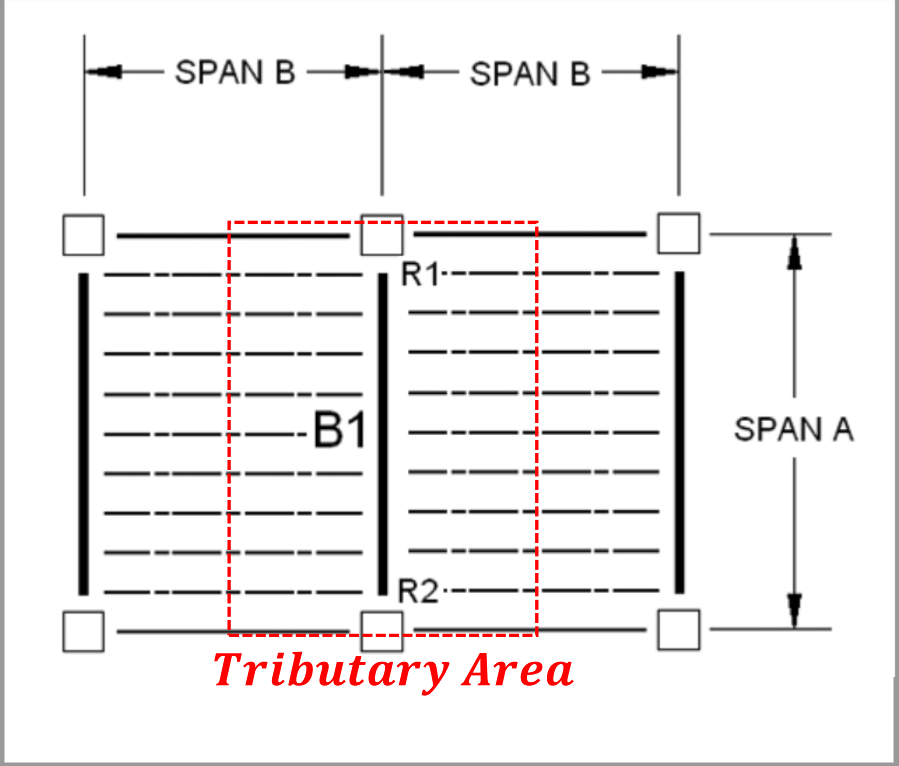
#Q2: The unfactored floor live load on the beam, w_LL

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	27 FT
Span B	14 FT
Floor Dead Load	19 PSF
Floor Live Load	90 PSF



$$W_{LL} = Floor_{LL}\left(\frac{Tributary\ Area}{spanA}\right) = 90\left(\frac{378}{27}\right) = \mathbf{1260\ PLF}$$

Problem Set 05

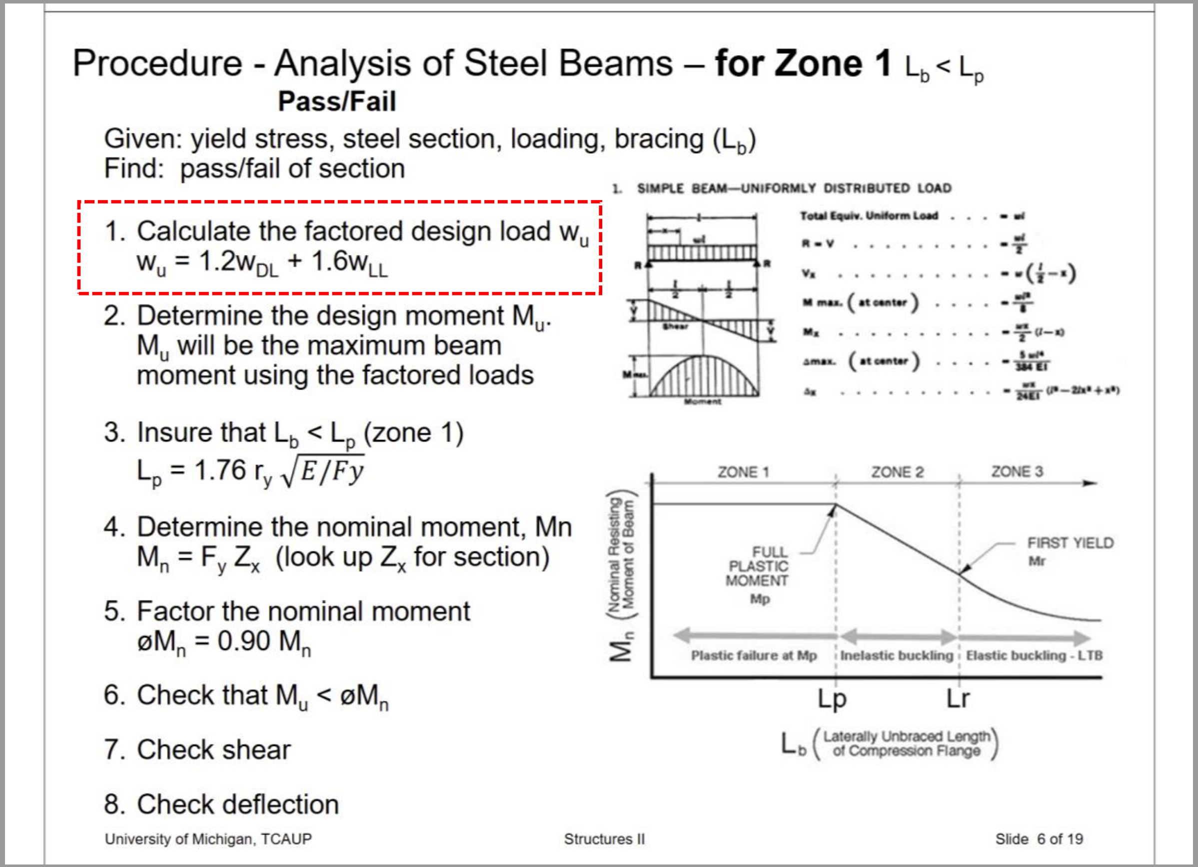
#Q3: The total factored design load on the beam (neglecting selfweight), wu*

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	27 FT
Span B	14 FT
Floor Dead Load	19 PSF
Floor Live Load	90 PSF



$$W_{u*} = \mathbf{1.2}\, W_{DL}* + \mathbf{1.6}\, W_{LL}* = 1.2(266) + 1.6\,(1260) = 2335.2\ \text{PLF} = \mathbf{2.3352\ KLF}$$

Problem Set 05

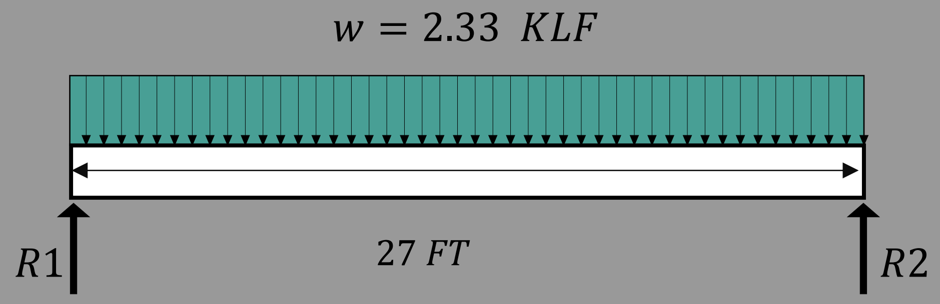
#Q4: The factored design moment (neglecting selfweight), Mu*
#Q5: The nominal bending moment (neglecting selfweight), Mn*

5. Steel Beam Design

Choose the lightest steel W-section to support the applied dead and live floor loads on Beam B1. Choose a steel W-section from AISC Table 3-2 (posted on Canvas). For the selection of the beam, neglect selfweight (for loads marked with *). After selecting the lightest section from Table 3-2, revise the DL to include the beam selfweight. Check that the final Mu including selfweight is less than the beam strength, phi Mn. Assume the beam is fully braced, Lb < Lp.

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	27 FT
Span B	14 FT
Floor Dead Load	19 PSF
Floor Live Load	90 PSF



$$\boldsymbol{M_{u*}} = \frac{\boldsymbol{W_{u*}L^2}}{\boldsymbol{8}} = \frac{2.33\,(27)^2}{8} = \boldsymbol{212.79\ K-FT}$$

$$\boldsymbol{\varphi M_{n*}} = \boldsymbol{M_{u*}} = 0.9 \times M_{n*} = 212.79 \rightarrow M_{n*} = 236.41\ K-FT \times \frac{12\ IN}{1\ FT} = \boldsymbol{2837.268\ K-IN}$$

Problem Set 05

#Q6: The plastic modulus of the section (neglecting selfweight), Zx*

#Q7: The nominal depth of the lightest passing W-section from Zx table (include selfweight)

DATASET: 1 -2- -3-

Fy	50 KSI
Span A	27 FT
Span B	14 FT
Floor Dead Load	19 PSF
Floor Live Load	90 PSF

Example - Design of Steel Beam

3. Determine Z_x required (assume zone 1)
   $M_n = F_y Z_x$

4. Select the lightest beam with a Z_x greater than the Z_x required from AISC table

M_U: 247,500 #-FT = 247.5 KFT

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

$Z_{X\,REQ'D} = \frac{M_N}{F_Y} = \frac{275\text{ KFT}\left(\frac{12''}{FT}\right)}{50\text{ KSI}}$

$Z_{X\,REQ'D}$: 66 IN^3

SELECT W18x35

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3–26 DESIGN OF FLEXURAL MEMBERS

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W18×35	66.5	166	249	101	151	8.14	12.3	4.31	12.3	510	106	159
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	54.6	136	205	84.9	128	5.01	7.55	5.40	15.6	340	79.8	120
W16×31	54.0	135	203	82.4	124	6.86	10.3	4.13	11.8	375	87.5	131
W12×35	51.2	128	192	79.6	120	4.34	6.45	5.44	16.6	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.67	2.55	7.35	35.2	184	68.0	102
W14×30	47.3	118	177	73.4	110	4.63	6.95	5.26	14.9	291	74.5	112
W10×39	46.8	117	176	73.5	111	2.53	3.78	6.99	24.2	209	62.5	93.7
W16×26[v]	44.2	110	166	67.1	101	5.93	8.98	3.96	11.2	301	70.5	106
W12×30	43.1	108	162	67.4	101	3.97	5.96	5.37	15.6	238	64.0	95.9
~~W14×26~~	40.2	100	151	61.7	92.7	5.33	8.11	3.81	11.0	245	70.9	106
W8×40	39.8	99.3	149	62.0	93.2	1.64	2.46	7.21	29.9	146	59.4	89.1
W10×33	38.8	96.8	146	61.1	91.9	2.39	3.62	6.85	21.8	171	56.4	84.7
W12×26	37.2	92.8	140	58.3	87.7	3.61	5.46	5.33	14.9	204	56.1	84.2
W10×30	36.6	91.3	137	56.6	85.1	3.08	4.61	4.84	16.1	170	63.0	94.5
W8×35	34.7	86.6	130	54.5	81.9	1.62	2.43	7.17	27.0	127	50.3	75.5
W14×22	33.2	82.8	125	50.6	76.1	4.78	7.27	3.67	10.4	199	63.0	94.5
W10×26	31.3	78.1	117	48.7	73.2	2.91	4.34	4.80	14.9	144	53.6	80.3
W8×31[f]	30.4	75.8	114	48.0	72.2	1.58	2.37	7.18	24.8	110	45.6	68.4
W12×22	29.3	73.1	110	44.4	66.7	4.68	7.06	3.00	9.13	156	64.0	95.9
W8×28	27.2	67.9	102	42.4	63.8	1.67	2.50	5.72	21.0	98.0	45.9	68.9
~~W10×22~~	26.0	64.9	97.5	40.5	60.9	2.68	4.02	4.70	13.8	118	49.0	73.4
W12×19	24.7	61.6	92.6	37.2	55.9	4.27	6.43	2.90	8.61	130	57.3	86.0
W8×24	23.1	57.6	86.6	36.5	54.9	1.60	2.40	5.69	18.9	82.7	38.9	58.3
~~W10×19~~	21.6	53.9	81.0	32.8	49.4	3.18	4.76	3.09	9.73	96.3	51.0	76.5
W8×21	20.4	50.9	76.5	31.8	47.8	1.85	2.77	4.45	14.8	75.3	41.4	62.1

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

$$Z_x = \frac{M_{n*}}{F_y} = \frac{2837.268}{50} = 56.74\ IN^3$$

$$From\ the\ table\ 3-2, the\ min\ beam\ size\ is\ W\ 18 \times 35$$

Problem Set 05

#Q8: The weight of the lightest passing W-section from Zx table

#Q9: The plastic modulus of the section for the chosen section, Zx

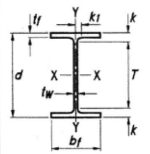


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k_{des}	Distance k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W21×93	27.3	21.6	21 5/8	0.580	9/16	5/16	8.42	8 3/8	0.930	15/16	1.43	1 5/8	15/16	18 3/8	5 1/2
×83[c]	24.4	21.4	21 3/8	0.515	1/2	1/4	8.36	8 3/8	0.835	13/16	1.34	1 1/2	7/8		
×73[c]	21.5	21.2	21 1/4	0.455	7/16	1/4	8.30	8 1/4	0.740	3/4	1.24	1 7/16	7/8		
×68[c]	20.0	21.1	21 1/8	0.430	7/16	1/4	8.27	8 1/4	0.685	11/16	1.19	1 3/8	7/8		
×62[c]	18.3	21.0	21	0.400	3/8	3/16	8.24	8 1/4	0.615	5/8	1.12	1 5/16	13/16		
×55[c]	16.2	20.8	20 3/4	0.375	3/8	3/16	8.22	8 1/4	0.522	1/2	1.02	1 3/16	13/16		
×48[c,f]	14.1	20.6	20 5/8	0.350	3/8	3/16	8.14	8 1/8	0.430	7/16	0.930	1 1/8	13/16		
W21×57[c]	16.7	21.1	21	0.405	3/8	3/16	6.56	6 1/2	0.650	5/8	1.15	1 5/16	13/16	18 3/8	3 1/2
×50[c]	14.7	20.8	20 7/8	0.380	3/8	3/16	6.53	6 1/2	0.535	9/16	1.04	1 1/4	13/16		
×44[c]	13.0	20.7	20 5/8	0.350	3/8	3/16	6.50	6 1/2	0.450	7/16	0.950	1 1/8	13/16		
W18×311[h]	91.6	22.3	22 3/8	1.52	1 1/2	3/4	12.0	12	2.74	2 3/4	3.24	3 7/16	1 3/8	15 1/2	5 1/2
×283[h]	83.3	21.9	21 7/8	1.40	1 3/8	11/16	11.9	11 7/8	2.50	2 1/2	3.00	3 3/16	1 5/16		
×258[h]	76.0	21.5	21 1/2	1.28	1 1/4	5/8	11.8	11 3/4	2.30	2 5/16	2.70	3	1 1/4		
×234[h]	68.6	21.1	21	1.16	1 3/16	5/8	11.7	11 5/8	2.11	2 1/8	2.51	2 3/4	1 3/16		
×211	62.3	20.7	20 5/8	1.06	1 1/16	9/16	11.6	11 1/2	1.91	1 15/16	2.31	2 9/16	1 3/16		
×192	56.2	20.4	20 3/8	0.960	15/16	1/2	11.5	11 1/2	1.75	1 3/4	2.15	2 7/16	1 1/8		
×175	51.4	20.0	20	0.890	7/8	7/16	11.4	11 3/8	1.59	1 9/16	1.99	2 7/16	1 1/4	15 1/8	
×158	46.3	19.7	19 3/4	0.810	13/16	7/16	11.3	11 1/4	1.44	1 7/16	1.84	2 3/8	1 1/4		
×143	42.0	19.5	19 1/2	0.730	3/4	3/8	11.2	11 1/4	1.32	1 5/16	1.72	2 3/16	1 3/16		
×130	38.3	19.3	19 1/4	0.670	11/16	3/8	11.2	11 1/8	1.20	1 3/16	1.60	2 1/16	1 3/16		
×119	35.1	19.0	19	0.655	5/8	5/16	11.3	11 1/4	1.06	1 1/16	1.46	1 15/16	1 3/16		
×106	31.1	18.7	18 3/4	0.590	9/16	5/16	11.2	11 1/4	0.940	15/16	1.34	1 13/16	1 1/8		
×97	28.5	18.6	18 5/8	0.535	9/16	5/16	11.1	11 1/8	0.870	7/8	1.27	1 3/4	1 1/8		
×86	25.3	18.4	18 3/8	0.480	1/2	1/4	11.1	11 1/8	0.770	3/4	1.17	1 5/8	1 1/16		
×76[c]	22.3	18.2	18 1/4	0.425	7/16	1/4	11.0	11	0.680	11/16	1.08	1 9/16	1 1/16		
W18×71	20.9	18.5	18 1/2	0.495	1/2	1/4	7.64	7 5/8	0.810	13/16	1.21	1 1/2	7/8	15 1/2	3 1/2[g]
×65	19.1	18.4	18 3/8	0.450	7/16	1/4	7.59	7 5/8	0.750	3/4	1.15	1 7/16	7/8		
×60[c]	17.6	18.2	18 1/4	0.415	7/16	1/4	7.56	7 1/2	0.695	11/16	1.10	1 3/8	13/16		
×55[c]	16.2	18.1	18 1/8	0.390	3/8	3/16	7.53	7 1/2	0.630	5/8	1.03	1 5/16	13/16		
×50[c]	14.7	18.0	18	0.355	3/8	3/16	7.50	7 1/2	0.570	9/16	0.972	1 1/4	13/16		
W18×46[c]	13.5	18.1	18	0.360	3/8	3/16	6.06	6	0.605	5/8	1.01	1 1/4	13/16	15 1/2	3 1/2[g]
×40[c]	11.8	17.9	17 7/8	0.315	5/16	3/16	6.02	6	0.525	1/2	0.927	1 3/16	13/16		
×35[c]	10.3	17.7	17 3/4	0.300	5/16	3/16	6.00	6	0.425	7/16	0.827	1 1/8	3/4		

Table 1-1 (continued)
W-Shapes
Properties

W21-W18

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}
93	4.53	32.3	2070	192	8.70	221	92.9	22.1	1.84	34.7	2.24	20.7	6.03	9940
83	5.00	36.4	1830	171	8.67	196	81.4	19.5	1.83	30.5	2.21	20.6	4.34	8630
73	5.60	41.2	1600	151	8.64	172	70.6	17.0	1.81	26.6	2.19	20.5	3.02	7410
68	6.04	43.6	1480	140	8.60	160	64.7	15.7	1.80	24.4	2.17	20.4	2.45	6760
62	6.70	46.9	1330	127	8.54	144	57.5	14.0	1.77	21.7	2.15	20.4	1.83	5960
55	7.87	50.0	1140	110	8.40	126	48.4	11.8	1.73	18.4	2.11	20.3	1.24	4980
48	9.47	53.6	959	93.0	8.24	107	38.7	9.52	1.66	14.9	2.05	20.2	0.803	3950
57	5.04	46.3	1170	111	8.36	129	30.6	9.35	1.35	14.8	1.68	20.5	1.77	3190
50	6.10	49.4	984	94.5	8.18	110	24.9	7.64	1.30	12.2	1.64	20.3	1.14	2570
44	7.22	53.6	843	81.6	8.06	95.4	20.7	6.37	1.26	10.2	1.60	20.3	0.770	2110
311	2.19	10.4	6970	624	8.72	754	795	132	2.95	207	3.53	19.6	176	76200
283	2.38	11.3	6170	565	8.61	676	704	118	2.91	185	3.47	19.4	134	65900
258	2.56	12.5	5510	514	8.53	611	628	107	2.88	166	3.42	19.2	103	57600
234	2.76	13.8	4900	466	8.44	549	558	95.8	2.85	149	3.37	19.0	78.7	50100
211	3.02	15.1	4330	419	8.35	490	493	85.3	2.82	132	3.32	18.8	58.6	43400
192	3.27	16.7	3870	380	8.28	442	440	76.8	2.79	119	3.28	18.7	44.7	38000
175	3.58	18.0	3450	344	8.20	398	391	68.8	2.76	106	3.24	18.4	33.8	33300
158	3.92	19.8	3060	310	8.12	356	347	61.4	2.74	94.8	3.20	18.3	25.2	29000
143	4.25	22.0	2750	282	8.09	322	311	55.5	2.72	85.4	3.17	18.2	19.2	25700
130	4.65	23.9	2460	256	8.03	290	278	49.9	2.70	76.7	3.13	18.1	14.5	22700
119	5.31	24.5	2190	231	7.90	262	253	44.9	2.69	69.1	3.13	17.9	10.6	20300
106	5.96	27.2	1910	204	7.84	230	220	39.4	2.66	60.5	3.10	17.8	7.48	17400
97	6.41	30.0	1750	188	7.82	211	201	36.1	2.65	55.3	3.08	17.7	5.86	15800
86	7.20	33.4	1530	166	7.77	186	175	31.6	2.63	48.4	3.05	17.6	4.10	13600
76	8.11	37.8	1330	146	7.73	163	152	27.6	2.61	42.2	3.02	17.5	2.83	11700
71	4.71	32.4	1170	127	7.50	146	60.3	15.8	1.70	24.7	2.05	17.7	3.49	4700
65	5.06	35.7	1070	117	7.49	133	54.8	14.4	1.69	22.5	2.03	17.7	2.73	4240
60	5.44	38.7	984	108	7.47	123	50.1	13.3	1.68	20.6	2.02	17.5	2.17	3850
55	5.98	41.1	890	98.3	7.41	112	44.9	11.9	1.67	18.5	2.00	17.5	1.66	3430
50	6.57	45.2	800	88.9	7.38	101	40.1	10.7	1.65	16.6	1.98	17.4	1.24	3040
46	5.01	44.6	712	78.8	7.25	90.7	22.5	7.43	1.29	11.7	1.58	17.5	1.22	1720
40	5.73	50.9	612	68.4	7.21	78.4	19.1	6.35	1.27	10.0	1.56	17.4	0.810	1440
35	7.06	53.5	510	57.6	7.04	66.5	15.3	5.12	1.22	8.06	1.51	17.3	0.506	1140

$$From\ the\ table\ 1-1\ ,nominal\ W_t = 35\ lb/FT$$

$$From\ the\ table\ 1-1\ ,Z_x = 66.5\ IN^3$$

Problem Set 05

#Q10: The revised unfactored dead load on the beam (including selfweight), w_DL

#Q11: The total factored design load on the beam (including selfweight), wu

DATASET: 1 -2- -3-	
Fy	50 KSI
Span A	27 FT
Span B	14 FT
Floor Dead Load	19 PSF
Floor Live Load	90 PSF

$$\boldsymbol{W_{DL}} = \boldsymbol{W_{DL*}} + \boldsymbol{W_t}(beam\ self\ weight) = 266 + 35 = 301\ PLF$$

$$\boldsymbol{W_u} = 1.2\ \mathrm{W_{DL}} + 1.6\ \mathrm{W_{LL}} = 1.2(301) + 1.6(1260) = 2377.2\ \mathrm{PLF} = \mathbf{2.3772\ KLF}$$

Problem Set 05

#Q12: The factored design moment (including selfweight), Mu in KIP-FT

#Q13: The factored design moment (including selfweight), Mu in KIP-IN

#Q14: The nominal factored bending moment for the chosen section, phi Mn

DATASET: 1	-2- -3-
Fy	50 KSI
Span A	27 FT
Span B	14 FT
Floor Dead Load	19 PSF
Floor Live Load	90 PSF

$$\boldsymbol{M_u = \frac{W_u L^2}{8}} = \frac{2.377\ (27)^2}{8} = \boldsymbol{216.62\ K - FT} \times \frac{12\ IN}{1\ FT} = \boldsymbol{2599.44\ K - IN}$$

$$\boldsymbol{\varphi M_n = 0.9(F_y \times Z_x) = 0.9(50 \times 66.5) = 2992.5\ \ K - IN}$$

$Note: For\ Q14, we\ are\ asked\ to\ find \boldsymbol{\varphi M_n}\ of\ the\ beam\ we\ have\ selected, not\ what\ we\ calculated\ in\ design$

Lab04

Structures II

Arch 324

Name 1 ____________________

Name 2 ____________________

Name 3 ____________________

Steel Columns

Description

This project gives the opportunity to identify steel sections and determine their properties and strength using the AISC tables.

Goals

To identify a steel section based on dimensions.
To determine the sectional properties using AISC table
To determine the load capacity based on AISC column table.

Procedure

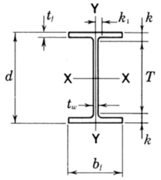
1. Measure the steel column section shown below. (your GSI will tell you which one)
2. Based on the sectional dimensions find the shape in the steel table.
3. Use the column table and the given height to find the load capacity. Both columns are A-36 steel (Fy = 36 ksi).

L= 15 ft. – 4 in.

or

L = 13 ft. 4 in.

Section: W ___ x _____ Design Strength _________ kips



W SHAPES
Dimensions

Designation	Area A	Depth d		Web Thickness t_w		$\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Distance k_1
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.
W 8x 67	19.7	9.00	9	0.570	9/16	5/16	8.280	8 1/4	0.935	15/16	6 1/8	1 7/16	11/16
x 58	17.1	8.75	8 3/4	0.510	1/2	1/4	8.220	8 1/4	0.810	13/16	6 1/8	1 5/16	11/16
x 48	14.1	8.50	8 1/2	0.400	3/8	3/16	8.110	8 1/8	0.685	11/16	6 1/8	1 3/16	5/8
x 40	11.7	8.25	8 1/4	0.360	3/8	3/16	8.070	8 1/8	0.560	9/16	6 1/8	1 1/16	5/8
x 35	10.3	8.12	8 1/8	0.310	5/16	3/16	8.020	8	0.495	1/2	6 1/8	1	9/16
x 31	9.13	8.00	8	0.285	5/16	3/16	7.995	8	0.435	7/16	6 1/8	15/16	9/16
W 8x 28	8.25	8.06	8	0.285	5/16	3/16	6.535	6 1/2	0.465	7/16	6 1/8	15/16	9/16
x 24	7.08	7.93	7 7/8	0.245	1/4	1/8	6.495	6 1/2	0.400	3/8	6 1/8	7/8	9/16
W 8x 21	6.16	8.28	8 1/4	0.250	1/4	1/8	5.270	5 1/4	0.400	3/8	6 5/8	13/16	1/2
x 18	5.26	8.14	8 1/8	0.230	1/4	1/8	5.250	5 1/4	0.330	5/16	6 5/8	3/4	7/16
W 8 x15	4.44	8.11	8 1/8	0.245	1/4	1/8	4.015	4	0.315	5/16	6 5/8	3/4	1/2
x 13	3.84	7.99	8	0.230	1/4	1/8	4.000	4	0.255	1/4	6 5/8	11/16	7/16
x 10	2.96	7.89	7 7/8	0.170	3/16	1/8	3.940	4	0.205	3/16	6 5/8	5/8	7/16
W 6x 25	7.34	6.38	6 3/8	0.320	5/16	3/16	6.080	6 1/8	0.455	7/16	4 3/4	13/16	7/16
x 20	5.87	6.20	6 1/4	0.260	1/4	1/8	6.020	6	0.365	3/8	4 3/4	3/4	7/16
x 15	4.43	5.99	6	0.230	1/4	1/8	5.990	6	0.260	1/4	4 3/4	5/8	3/8
W 6x 16	4.74	6.28	6 1/4	0.260	1/4	1/8	4.030	4	0.405	3/8	4 3/4	3/4	7/16
x 12	3.55	6.03	6	0.230	1/4	1/8	4.000	4	0.280	1/4	4 3/4	5/8	3/8
x 9	2.68	5.90	5 7/8	0.170	3/16	1/8	3.940	4	0.215	3/16	4 3/4	9/16	3/8
W 5x 19	5.54	5.15	5 1/8	0.270	1/4	1/8	5.030	5	0.430	7/16	3 1/2	13/16	7/16
x 16	4.68	5.01	5	0.240	1/4	1/8	5.000	5	0.360	3/8	3 1/2	3/4	7/16
W 4x 13	3.83	4.16	4 1/8	0.280	1/4	1/8	4.060	4	0.345	3/8	2 3/4	11/16	7/16

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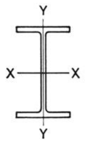

COLUMNS
W shapes
Design axial strength in kips ($\phi = 0.85$)

F_y = 36 ksi

F_y = 50 ksi

Designation	W8											
Wt./ft	67		58		48		40		35		31	
F_y	36	50	36	50	36	50	36	50	36	50	36	50
Effective length in ft KL with respect to least radius of gyration r_y: 0	603	837	523	727	431	599	358	497	315	438	279	388
6	567	770	492	667	405	549	335	454	295	399	261	354
7	555	746	481	647	396	532	327	439	288	386	255	342
8	541	721	469	624	386	513	319	423	280	372	248	329
9	526	693	455	599	374	492	309	405	272	356	240	315
10	509	662	441	572	362	470	298	386	262	339	232	300
11	492	631	425	544	349	446	287	366	252	321	223	284
12	473	598	409	515	335	422	275	345	242	303	214	268
13	453	564	391	485	321	397	263	324	231	284	204	251
14	433	529	374	455	306	372	251	303	220	265	194	234
15	412	494	355	425	291	347	238	281	208	246	184	217
16	391	460	337	394	276	321	225	260	197	228	174	200
17	370	425	318	365	260	297	212	239	185	209	163	184
18	349	392	300	335	245	272	198	219	174	191	153	168
19	328	359	281	307	229	249	186	200	162	174	143	153
20	307	328	263	279	214	226	173	180	151	157	133	138
22	267	271	228	231	185	187	148	149	129	130	114	114
24	228	228	194	194	157	157	125	125	109	109	96	96
26	194	194	165	165	134	134	107	107	93	93	82	82
28	167	167	143	143	115	115	92	92	80	80	70	70
30	146	146	124	124	100	100	80	80	70	70	61	61
32	128	128	109	109	88	88	70	70	61	61	54	54
33	120	120	103	103	83	83	66	66	58	58	51	51
34	113	113	97	97	78	78	62	62				
35	107	107	91	91								
Properties												
U	1.33	1.48	1.35	1.49	1.37	1.51	1.39	1.54	1.40	1.55	1.41	1.56
P_{wo} (kips)	147	205	120	167	86	119	69	96	56	78	48	67
P_{wi} (kips/in.)	21	29	18	26	14	20	13	18	11	16	10	14
P_{wb} (kips)	648	764	464	547	224	264	163	192	104	123	81	95
P_{fb} (kips)	177	246	133	185	95	132	64	88	50	69	38	53
L_p (ft)	8.8	7.5	8.8	7.4	8.7	7.4	8.5	7.2	8.5	7.2	8.4	7.1
L_r (ft)	64.0	41.9	56.0	36.8	46.7	31.1	39.1	26.4	35.1	24.1	32.0	22.3
A (in.^{2})	19.7		17.1		14.1		11.7		10.3		9.13	
I_x (in.^{4})	272		228		184		146		127		110	
I_y (in.^{4})	88.6		75.1		60.9		49.1		42.6		37.1	
r_y (in.)	2.12		2.10		2.08		2.04		2.03		2.02	
Ratio r_x/r_y	1.75		1.74		1.74		1.73		1.73		1.72	

†Flange is noncompact; see discussion preceding column load tables.
Note: Heavy line indicates Kl/r of 200.

Lab04

→ 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*