Arch 324
Structures II

Winter 2026 Recitation 004

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Recitation 004

Welcome to session 5!

Preliminary Report due this Sunday (Mon night OK) !

- Quick Lecture Recap
- Homework #5 Steel Beam Design
- Lab: Steel Beam Design

Feel free to ask questions anytime

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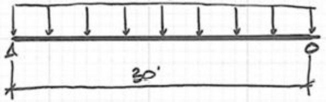
Lecture: Steel Beam Design (2/9)

Design of Steel Beam – Procedure (zone 1)

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

GIVEN: $F_y = 50$ KSI
FULLY BRACED



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

$$M_u = 247{,}500\ \#\text{-FT} = 247.5\ \text{KFT}$$

$$M_n = M_u/\phi_b = \frac{247.5\ \text{KFT}}{0.90} = 275\ \text{KFT}$$

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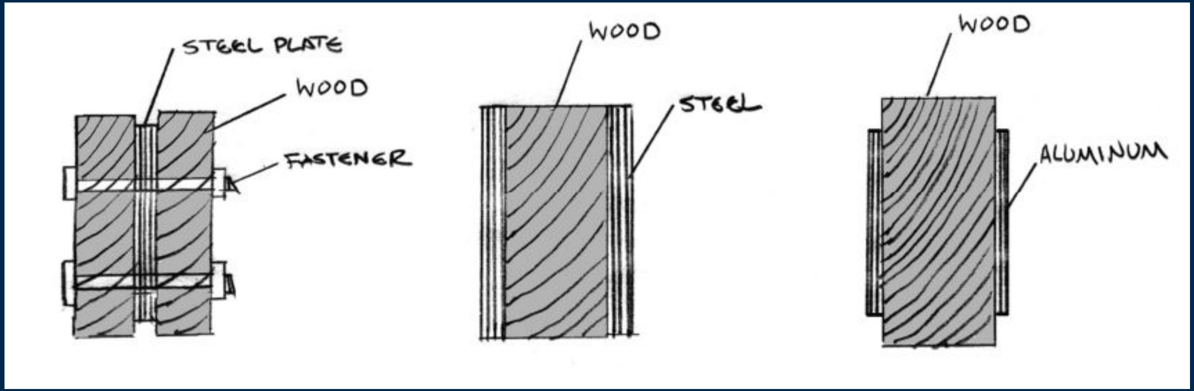
Lecture: Steel Beam Design (2/9)



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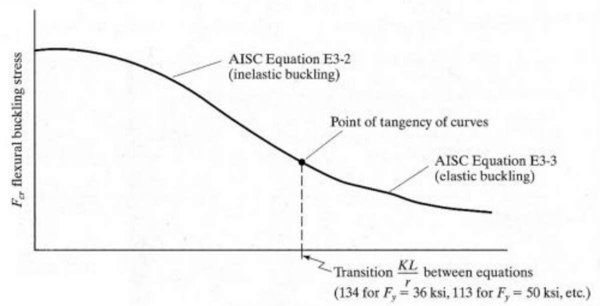
Lecture: Steel Column Analysis (2/11)

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



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

short ⟷ intermediate ⟷ long

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

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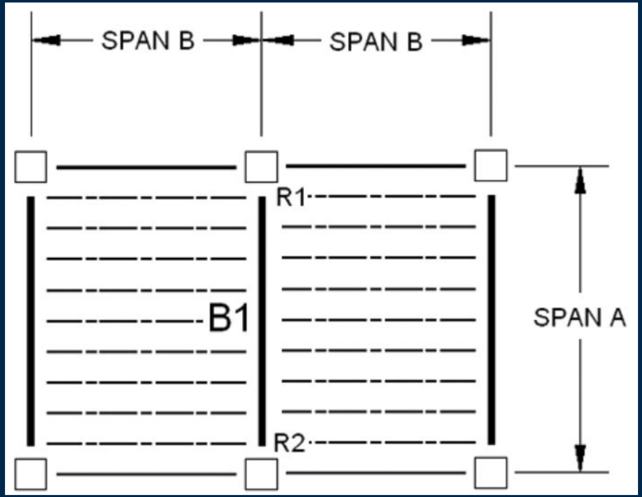
HW #5: Steel Beam Design

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	25 FT
Span B	17 FT
Floor Dead Load	14 PSF
Floor Live Load	90 PSF



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HW #5: Steel Beam Design

DATASET: 1	-2- -3-
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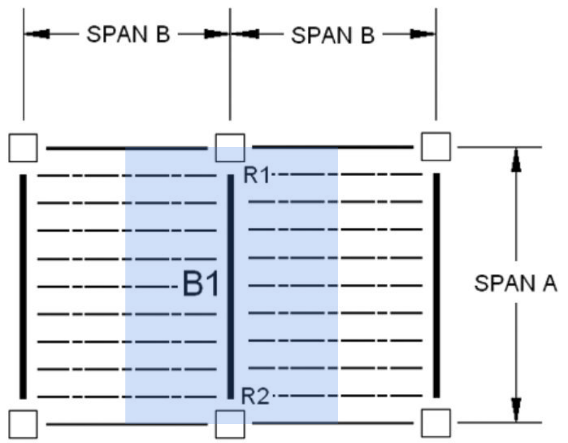


1. Unfactored floor dead load on B1

$$W_{DL} = \overset{\text{given}}{\text{Floor dead load}} \times 2\left(\overset{\text{given}}{\frac{\text{span } B}{2}}\right)$$

$$= 14\text{PSF} \times 2\left(\frac{17}{2}\right)$$

$= \boxed{238 \text{ PLF}}$ ← Answer to #1

2. Unfactored floor live load on B1

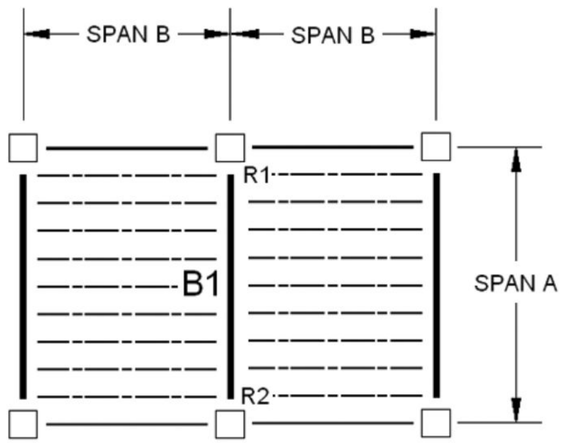
$$W_{LL} = \overset{\text{given}}{\text{Floor live load}} \times \text{tributary area}$$

$$= 90 \text{ PSF} \times 17\text{SQFT}$$

$= \boxed{1{,}530 \text{ PLF}}$ ← Answer to #2

HW #5: Steel Beam Design

DATASET: 1	-2- -3-
Fy	50 KSI
Span A	25 FT
Span B	17 FT
Floor Dead Load	14 PSF
Floor Live Load	90 PSF



3. Total factored design load on B1

$W_u = 1.2\, w_{DL} + 1.6\, w_{LL}$

$= 1.2(238)^{\#1} + 1.6(1{,}530)^{\#2}$

$= 2{,}733.6 \text{ PLF} \times \frac{1}{1000}$ ← convert!

$= \boxed{2.734 \text{ KLF}}$ ← #3

4. Factored design moment

$M_u = \frac{W_u L^2}{8}$ $\left\{ \begin{array}{l} w_u = \#3 \\ L = \text{span } A \end{array} \right.$

$= \frac{2.734(25^2)}{8}$

$= \boxed{213.59 \text{ k-ft}}$ ← #4

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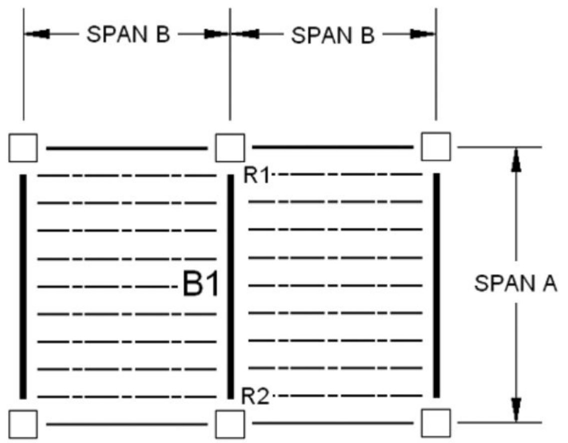
HW #5: Steel Beam Design

DATASET: 1	-2- -3-
Fy	50 KSI
Span A	25 FT
Span B	17 FT
Floor Dead Load	14 PSF
Floor Live Load	90 PSF



Nominal bending moment = $M_u < \phi M_n$

* assume $M_u = \phi M_n$

$M_n = \dfrac{M_u}{\phi}$ -#4 $\quad \phi = 0.9$

$= \dfrac{213.59}{0.9}$

convert!

$= 237.33$ k-ft $\times 12$

$= \boxed{2{,}847.92 \text{ k-in}} \leftarrow$ #5

6. Plastic modulus of section

* fully braced = Zone 1 *

$\underset{\#5}{M_n} = Z_x \underset{\text{given}}{F_y}$

$2{,}847.92 = Z_x (50)$

$Z_x = \boxed{56.96 \text{ in}^3} \leftarrow$ #6

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HW #5: Steel Beam Design

Compare Zx values in table 3-2 with required min. Zx found in #6

Find appropriate cluster of sections that will pass and choose lightest one (bold)

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

[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 $\Omega_v = 1.67$.

$F_y = 50$ ksi

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

Z_x

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
W12×16	**20.1**	**50.1**	**75.4**	**29.9**	**44.9**	**3.80**	**5.73**	**2.73**	**8.05**	**103**	**52.8**	**79.2**
W10×17	18.7	46.7	70.1	28.3	42.5	2.98	4.47	2.98	9.16	81.9	48.5	72.7
W12×14[v]	**17.4**	**43.4**	**65.3**	**26.0**	**39.1**	**3.43**	**5.17**	**2.66**	**7.73**	**88.6**	**42.8**	**64.3**
W8×18	17.0	42.4	63.8	26.5	39.9	1.74	2.61	4.34	13.5	61.9	37.4	56.2
W10×15	16.0	39.9	60.0	24.1	36.2	2.75	4.14	2.86	8.61	68.9	46.0	68.9
W8×15	13.6	33.9	51.0	20.6	31.0	1.90	2.85	3.09	10.1	48.0	39.7	59.6
W10×12[f]	**12.6**	**31.2**	**46.9**	**19.0**	**28.6**	**2.36**	**3.53**	**2.87**	**8.05**	**53.8**	**37.5**	**56.3**
W8×13	11.4	28.4	42.8	17.3	26.0	1.76	2.67	2.98	9.27	39.6	36.8	55.1
W8×10[f]	**8.87**	**21.9**	**32.9**	**13.6**	**20.5**	**1.54**	**2.30**	**3.14**	**8.52**	**30.8**	**26.8**	**40.2**

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

[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 $\Omega_v = 1.67$.

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HW #5: Steel Beam Design

Required min (#6) = 56.96in^3

Chosen section:
W<u>18</u> x <u>35</u>

Q7

Q8

Q9

$Z_x = 66.5$

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	$\phi_v V_{nx}$
	in.^{3}	kip-ft ASD	kip-ft LRFD	kip-ft ASD	kip-ft LRFD	kips ASD	kips LRFD	ft	ft	in.^{4}	kips ASD	kips 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
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HW #5: Steel Beam Design

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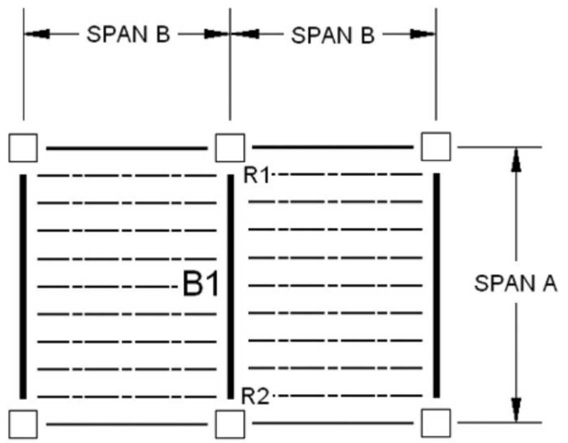

10. Revised unfactored dead load on B1

* consider self-weight now *

$$W_D = \underbrace{238\,PLF}_{\#1} + \underbrace{35\,PLF}_{\text{weight from section}} = \underbrace{\boxed{273\,PLF}}_{\#10}$$

11. Revised factored design load on B1

$$W_U = 1.2(\underbrace{273}_{\#10}) + 1.6(\underbrace{1530}_{\#2})$$

$$= 2{,}775.6\,PLF \times \frac{1}{1000} \leftarrow \text{convert!}$$

$$= \boxed{2.7756\,KLF} \leftarrow \#11$$

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HW #5: Steel Beam Design

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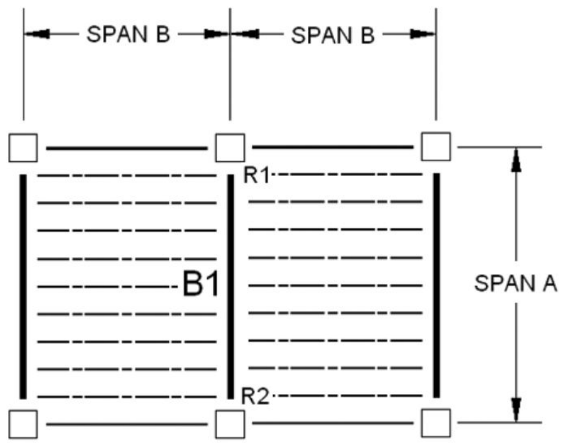

12. Factored design moment
/13

$$M_u = \frac{w_u L^2}{8}$$

$$= \frac{2.776\,(25^2)}{8}$$

#12 convert!

$$= \boxed{216.84\ k\text{-}ft} \times 12$$

$$= \boxed{2{,}602.125\ k\text{-}in} \leftarrow \#13$$

14. Nominal factored bending moment

$$\phi M_n = \phi Z_x F_y$$

$$= 0.9\,(66.5)\,(50) = \boxed{2{,}992.5\ k\text{-}in}$$

From table

$2{,}992.5 > 2{,}602.1$ ✓

:)

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HW #5: Steel Beam Design

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-

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Span A	25 FT
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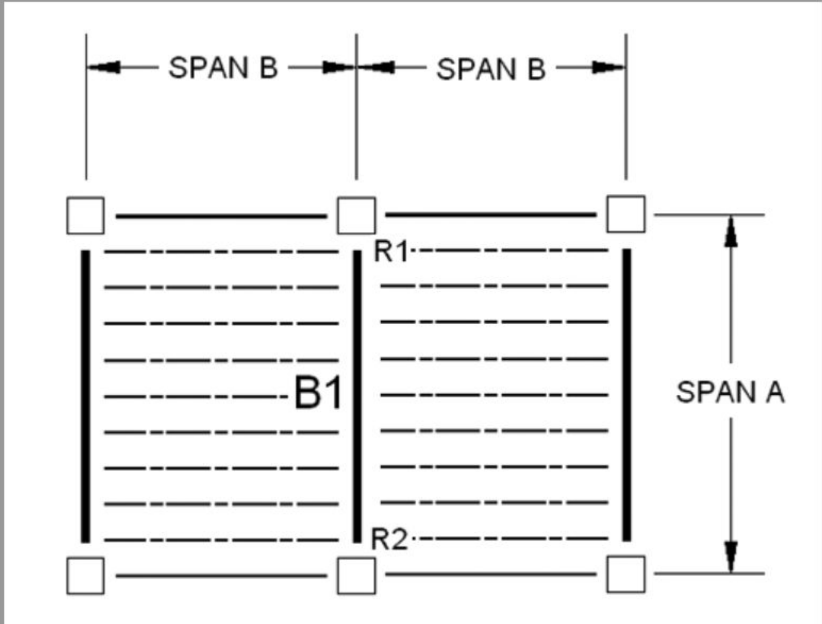

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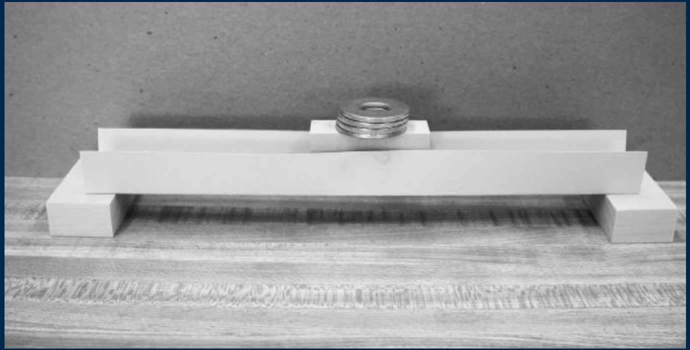
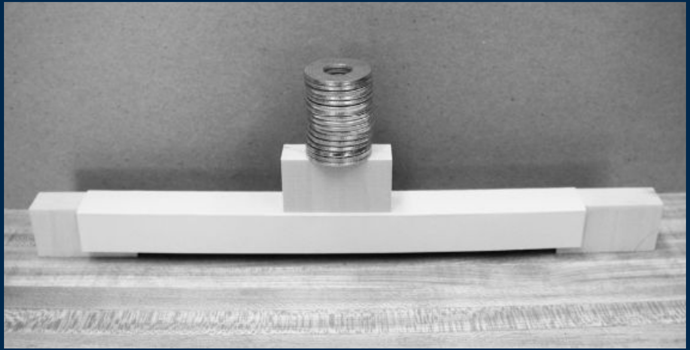
Lab: Steel Beams





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