Architecture 324

Structures II

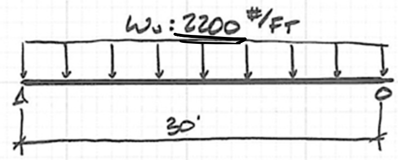
Steel Beam Design

- Design Method
- Flitched Beams

Design of Steel Beam – Procedure (zone 1)

1. Use the maximum moment equation, and solve for the ultimate moment, M_u. LOAD
2. Set $\phi M_n = M_u$ and solve for M_n STRENGTH > LOAD $F = \frac{M}{Z}$
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 SHEAR
   (case 1, most common)
6. Determine A_w:
   $A_w = d\ t_w$
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



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

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

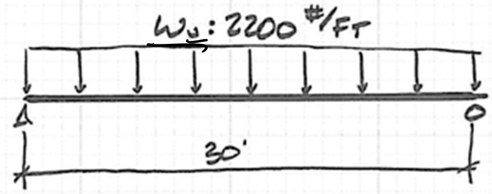
$M_n = M_u / \phi_b = \frac{247.5 \text{ KFT}}{0.90} = 275$ KFT

Design of Steel Beam

Example - Bending

Applied Load: ?
DL = 500 + beam plf   LL = 1000 plf
γ DL   γ LL
1.2(500) + 1.6(1000) = 2200 lb/ft

GIVEN: $F_y = 50$ KSI
FULLY BRACED ✓ZONE 1



1. Use the maximum moment equation, and solve for the ultimate moment, M_u.

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

2. Set ϕM_n = Mu and solve for M_n

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

$$M_N = M_u / \phi_b = \frac{247.5 \text{ KFT}}{0.90\ (\phi)} = 275 \text{ KFT}$$

Example - Design of Steel Beam

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

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

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

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

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

$$Z_{x\,REQ'D} = 66 \text{ IN}^3$$

SELECT W18x35 DL

Z_x **Table 3-2 (continued)**
W-Shapes
Selection by Z_x F_y = 50 ksi

ASD LRFD

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
W21×44	**95.4**	**238**	358	**143**	214	**11.1**	16.8	**4.45**	**13.0**	**843**	**145**	217
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	294	**119**	180	**8.94**	13.2	**4.49**	**13.1**	**612**	**113**	169
W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	425	83.5	125
W10×60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	341	85.7	129
W16×40	**73.0**	**182**	274	**113**	170	**6.67**	10.0	**5.55**	**15.9**	**518**	**97.6**	146
W12×50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	391	90.3	135
W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
W14×43	69.6	174	261	109	164	4.88	7.28	6.68	20.0	428	83.6	125
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	$\boxed{\textbf{66.5}}$	**166**	$\boxed{249}$	**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

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

[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 $\Omega_v = 1.67$.

Example - Design of Steel Beam

4. revise Dead Load to include selfweight.

Applied Load:
DL = 500+35 plf   LL = 1000 plf

1.2(535) + 1.6(1000) = 2242 lb/ft

w_u ℓ^2

M_u = (2242 x 30^2) / 8 = 252225 ft-lbs

$\phi M_n = M_u$ = 252.2 k-ft

M_n = 280.2 k-ft Z_x = 67.2 in^3

Update section and Z_x if required from AISC table.

Applied Load:
DL = 540 plf   LL = 1000 plf

1.2(540) + 1.6(1000) = 2248 lb/ft

$\phi M_n = M_u$ = 252.9 k-ft < 274 ✓ OK

use W16x40

3-26 DESIGN OF FLEXURAL MEMBERS

Z_x

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

ϕM_u

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
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.1**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
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[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, ϕ_v = 0.90 and Ω_v = 1.67.

American Institute of Steel Construction

Example - Design of Steel Beam

Check Shear

5. Determine if h/t_w < 59
   (case 1, most common)

6. Determine A_w:
   $A_w = d * t_w$ = 16.0" x 0.305"
   A_w = 4.88 in^2

FIND h/t_w FROM TABLES FOR A

W16x40

h/t_w : 46.5 < 59 ✓

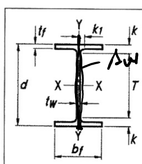
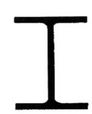


Table 1-1 (continued)
W-Shapes
Dimensions

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	Distance: 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	$\boxed{16.0}$	16	$\boxed{0.305}$	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	↓	↓

Table 1-1 (continued)
W-Shapes
Properties



W16-W14

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}
100	5.29	24.3	1490	175	7.10	198	186	35.7	2.51	54.9	2.92	16.0	7.73	11900
89	5.92	27.0	1300	155	7.05	175	163	31.4	2.49	48.1	2.88	15.9	5.45	10200
77	6.77	31.2	1110	134	7.00	150	138	26.9	2.47	41.1	2.85	15.7	3.57	8590
67	7.70	35.9	954	117	6.96	130	119	23.2	2.46	35.5	2.82	15.6	2.39	7300
57	4.98	33.0	758	92.2	6.72	105	43.1	12.1	1.60	18.9	1.92	15.7	2.22	2660
50	5.61	37.4	659	81.0	6.68	92.0	37.2	10.5	1.59	16.3	1.89	15.7	1.52	2270
45	6.23	41.1	586	72.7	6.65	82.3	32.8	9.34	1.57	14.5	1.87	15.5	1.11	1990
40	6.93	$\boxed{46.5}$	518	64.7	6.63	73.0	28.9	8.25	1.57	12.7	1.86	15.5	0.794	1730
36	8.12	48.1	448	56.5	6.51	64.0	24.5	7.00	1.52	10.8	1.83	15.5	0.545	1460

Example - Design of Steel Beam
Check Shear

5. Determine if $h/t_w < 59$
   (case 1, most common)

6. Determine Aw:
   $A_w = d * t_w = 4.88\ in^2$

7. Calculate Vn:
   $V_n = 0.6*F_y*A_w$

8. Calculate Vu for the given loading
   $Vu = w_u\ L / 2$ (unif. load)

9. Check $V_u < \phi_v\ V_n$
   $\phi_v = 1.0$

FIND h/t_w FROM TABLES FOR A

$h/t_w = 46.5 < 59$ (zone 1)

$V_N = 0.6 \cdot F_y^{ksi} \cdot A_w$ $4.88\ in^2$

$= 0.6 \cdot 50\ ksi \times 16'' \times 0.305''$

$= 146.4\ k$

$V_u = \frac{2200\ \#/FT \cdot 30'}{2} = 33{,}000\# = 33^k$

$V_u \leq \phi_v V_N$

$33^k < (1.0)\ 146.4\ k$ ✓ OK

Example - Design of Steel Beam
Check Deflection

Deflection limits by application
IBC Table 1604.3

For steel structural members, the DL can be taken as zero (note g)

DL deflection can be compensated for by beam camber

IBC

TABLE 1604.3
DEFLECTION LIMITS[a, b, c, h, i]

CONSTRUCTION	L	S or W[f]	D + L[d,g]
Roof members:[e]			
Supporting plaster ceiling	l/360	l/360	l/240
Supporting nonplaster ceiling	l/240	l/240	l/180
Not supporting ceiling	l/180	l/180	l/120
Floor members	l/360	—	l/240
Exterior walls and interior partitions:			
With brittle finishes	—	l/240	—
With flexible finishes	—	l/120	—
Farm buildings	—	—	l/180
Greenhouses	—	—	l/120

W 16 x 40

$$\Delta_{LL} = \frac{5\,w_{LL}\,l^4}{384\,EI} = \frac{5(1\ \frac{K}{FT})(30\ FT)^4\ 1728\ \frac{IN^3}{FT^3}}{384\,(29000\ \frac{K}{IN^2})(518\ IN^4)}$$

LL, in^3, STEEL

$= 1.23''$

$\frac{l}{360} = \frac{30'(12)}{360} = \boxed{1''}$ MAX < 1.21 ∴ NG!

TRY W 18x40

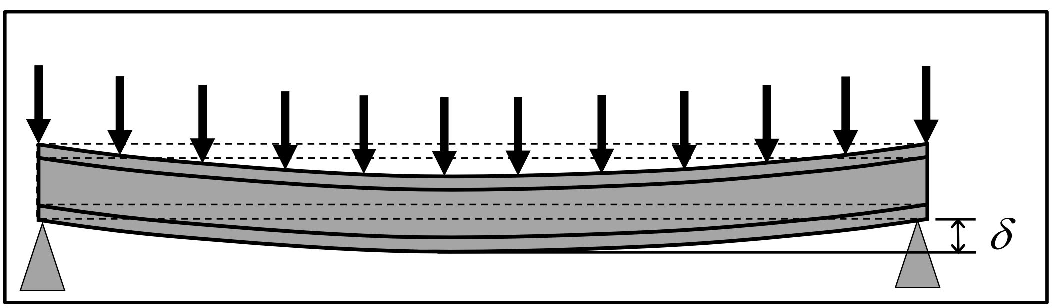
$$\Delta_{LL} = \frac{5\,w_{LL}\,l^4}{384\,EI} = \frac{5(1\ \frac{K}{FT})(30\ FT)^4\ 1728\ \frac{IN^3}{FT^3}}{384\,(29000\ \frac{K}{IN^2})(612\ IN^4)}$$

18x40

$\Delta_{LL} = 1.02'' \approx 2\%$

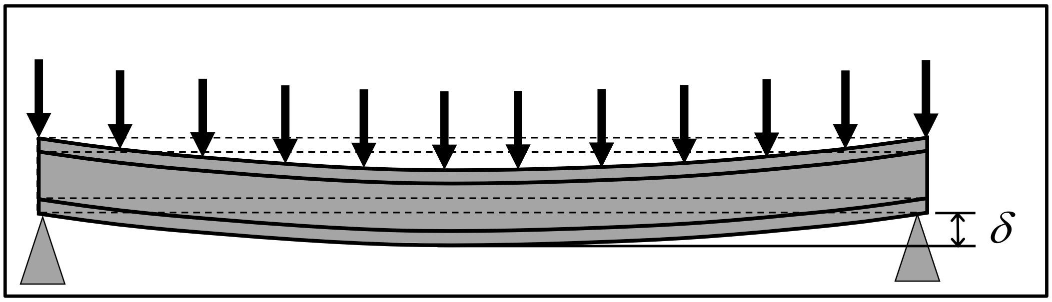
Beam without Camber

Developed by Scott Civjan
University of Massachusetts, Amherst
For AISC



Results in deflection in floor under Dead Load.
This can affect thickness of slab and fit of non-structural components.

Developed by Scott Civjan
University of Massachusetts, Amherst
For AISC



Results in deflection in floor under Dead Load.
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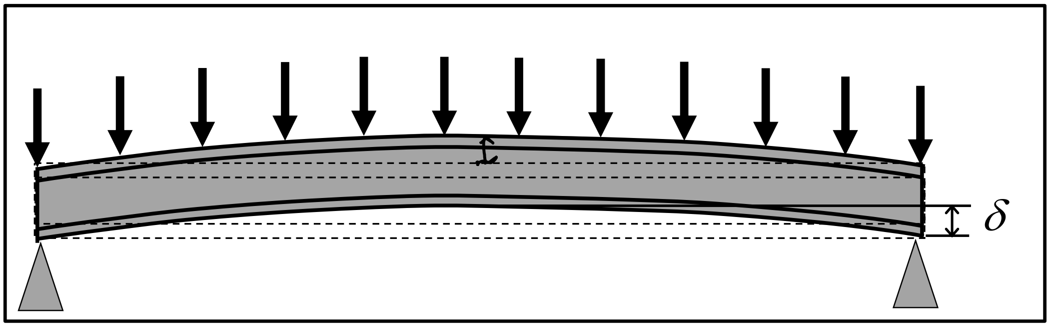
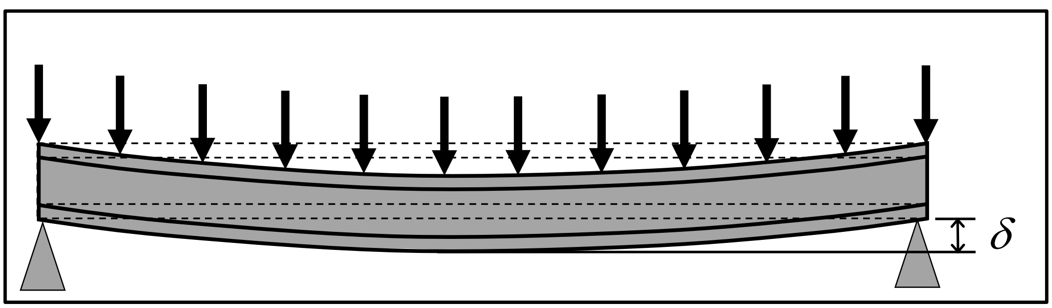
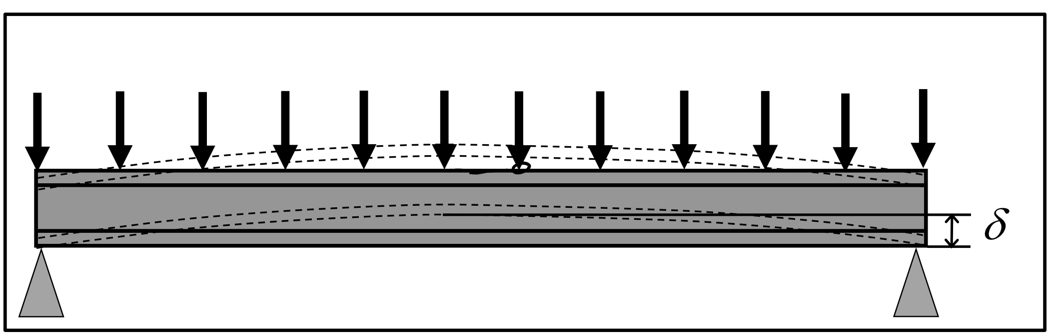


Beam with Camber

Developed by Scott Civjan
University of Massachusetts, Amherst
For AISC



Results in deflection in floor under Dead Load.
This can affect thickness of slab and fit of non-structural components.



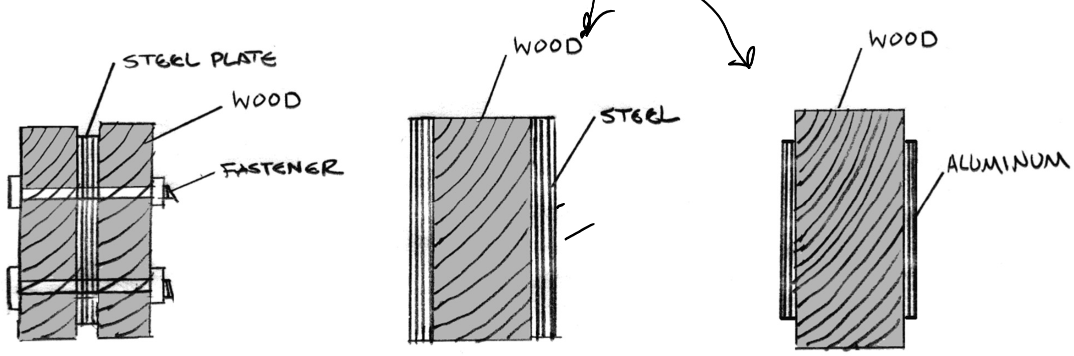
Cambered beam counteracts service dead load deflection.

Developed by Scott Civjan
University of Massachusetts, Amherst
For AISC

Flitched Beams & Scab Plates

Advantages

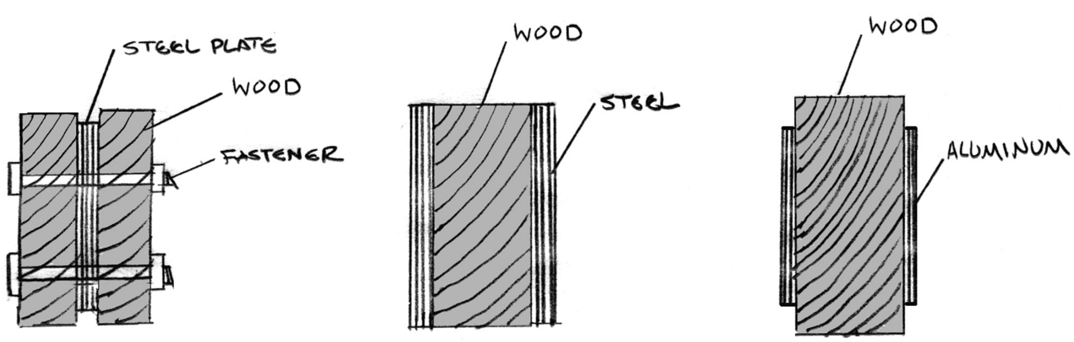
- Compatible with the wood structure,
  i.e. can be nailed
- Easy to retrofit to existing structure
- Lighter weight than a steel section
- Stronger than wood alone
  - Less deep than wood alone
  - Allow longer spans
- The section can vary over the length of the span to optimize the member (e.g. scab plates)
- The wood stabilizes the thin steel plate



Flitched Beams & Scab Plates

Disadvantages

- More labor to make – expense. Flitched beams require shop fabrication or field bolting.
- Often replaced by Composite Lumber which is simply cut to length – less labor
  - Glulam
  - LVL
  - PSL
- Flitched Beams are generally heavier than Composite Lumber



Steel Sandwiched Beams

based on strain compatibility

Applications:

Renovation in Edina, Minnesota

- Four 2x8 LVLs, with two 1/2" steel plates.
- 18 FT span
- Original house from 1949
- Renovation in 2006
- Engineer: Paul Voigt

Applications:

Renovation

Chris Withers House, Reading, UK 2007
Architect: Chris Owens, Owens Galliver
Engineer: Allan Barnes

Strain Compatibility

With two materials bonded together, both will act as one, and the deformation in each is the same.

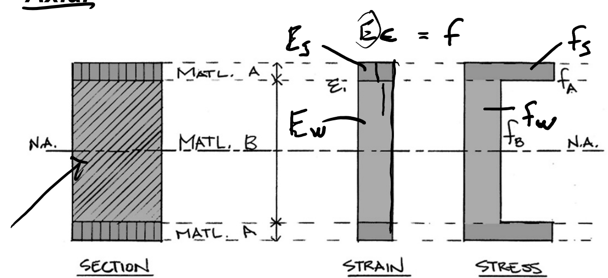
Therefore, the strains will be the same in each material under **axial load**.

In **flexure** the strains are the same as in a homogeneous section, i.e. linear.

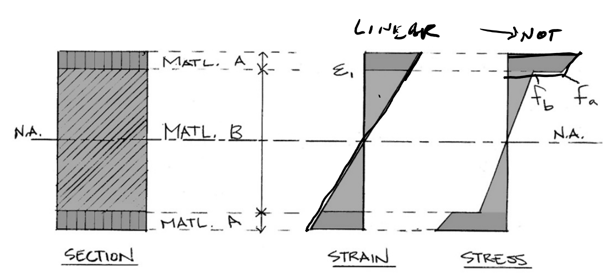
In flexure, if the two materials are at the same distance from the N.A., they will have the same strain at that point because both materials share the same strain diagram. We say the strains are "compatible".

Stress = E x Strain
So stress will be higher if E is higher.

Axial

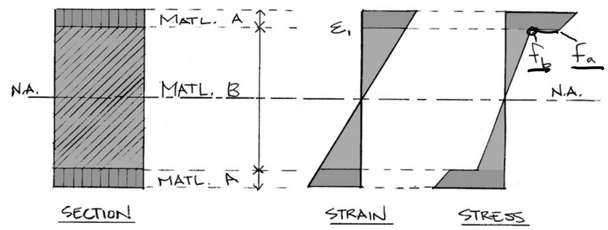


Flexure



Strain Compatibility (cont.)

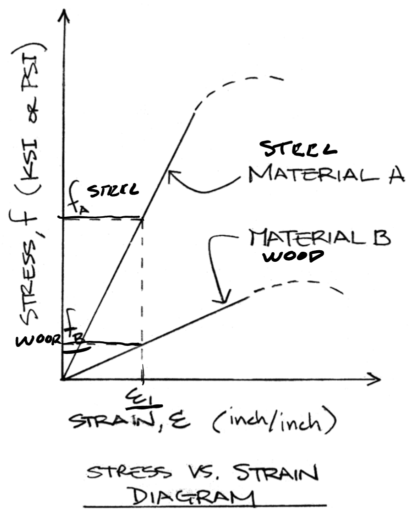
The stress in each material is determined by using Young's Modulus



flexure

$$\sigma = \mathrm{E}\varepsilon$$

Care must be taken that the elastic limit of each material is not exceeded. The elastic limit can be expressed in either stress or strain.



Capacity Analysis (ASD) Flexure

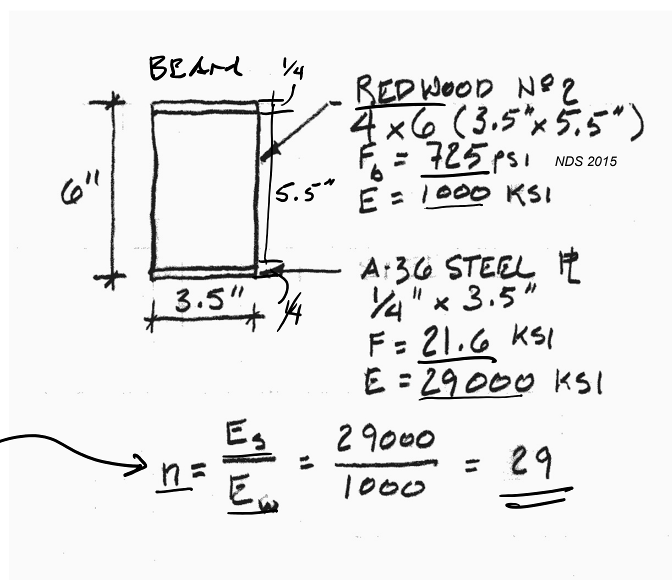
Given

- Dimensions
- Material

Required

- Load capacity ?

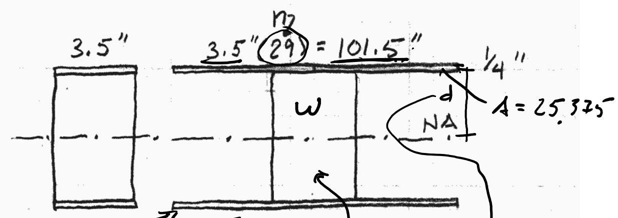
1. Determine the modular ratio. It is usually more convenient to transform the stiffer material.



Capacity Analysis (cont.)

$F = \frac{Mc}{I}$

2. Construct the transformed section. Multiply all widths of the transformed material by n. The depths remain unchanged.



3. Calculate the transformed SECTION moment of inertia, I_{tr} .

$$I_{tr} = \sum I + \sum Ad^2$$

$$I_w = \frac{3.5(5.5)^3}{12} = 48.53 \text{ in}^4$$

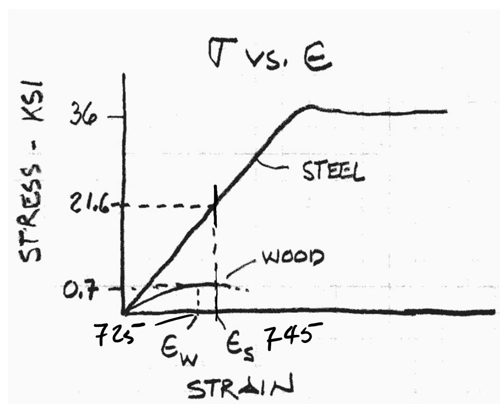
$$I_s = 2\left[\frac{101.5(0.25)^3}{12} + 25.375(2.875)^2\right]$$

$$I_s = 2\left[\underset{w}{0.132} + \underset{s}{209.74}\right] = 419.7 \text{ in}^4$$

$$I_{TR} = 48.83 + 419.7 = 468.3 \text{ in}^3$$

Capacity Analysis (cont.)

4. Calculate the allowable strain based on the allowable stress for the material.



$$\varepsilon_{allow} = \frac{F_{allow}}{E}$$

$$\epsilon = \frac{\sigma}{E}$$

$$\epsilon_W = \frac{725 \text{ PSI } W}{1000000 \, E_w} = 0.000725$$

$$\epsilon_S = \frac{21.6 \text{ KSI } S}{29000} = 0.000745$$

Capacity Analysis (cont.)

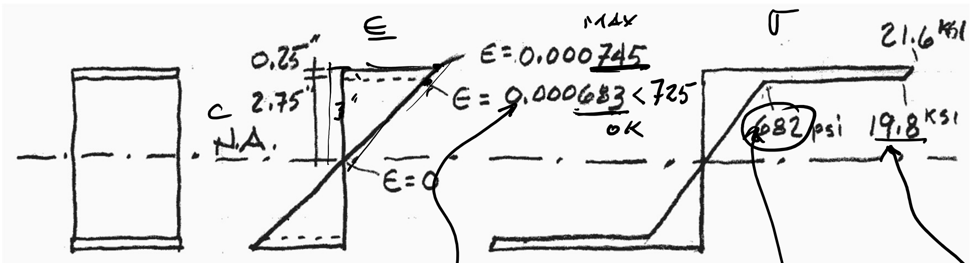
5. Construct a strain diagram to find which of the two materials will reach its limit first. The diagram should be linear, and neither material may exceed its allowable limit.

Allowable Strains:

$$\epsilon = \frac{\sigma}{E}$$

$$\epsilon_w = \frac{725}{1000000} = 0.000725$$

$$\epsilon_s = \frac{21.6}{29000} = 0.000745$$



$$\frac{\epsilon}{c} : \frac{0.000745}{3.0''} : \frac{\epsilon_w}{2.75''}$$

$$\epsilon_w = 0.000683$$

$$\sigma = E\epsilon$$

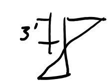
$$f_w = 1000000(0.000683)$$

$$f_w = 682 \text{ psi}$$

$$f_s = 29000(0.000683)$$

$$f_s = 19.8 \text{ KSI}$$

Capacity Analysis (cont.)



6. The allowable moments (load capacity) may now be determined based on the stress of either material. Either stress should give the same moment if the strain diagram from step 5 is compatible with the stress diagram (they align and allowables are not exceeded).

$$M_s = \frac{f_s I_{TR}}{c\,n} = \frac{21.6\,(468.3)}{3\,(29)} = 116.2^{\,K-''}$$

$$M_w = \frac{f_w I_{TR}}{c} = \frac{0.682\,(468.3)}{2.75''} = 116.1^{\,K-''}$$

7. Alternatively, the controlling moment can be found without the strain investigation by using the maximum allowable stress for each material in the moment-stress equation. The **lower moment** will be the first failure point and the controlling material.

$$M_s = \frac{F_s I_{TR}}{c\,n} = \frac{21.6\,(468.3)}{3\,(29)} = 116.2^{\,K-''}$$

$$M_w = \frac{F_w I_{TR}}{c} = \frac{.725(468.3)}{2.75''} = 123.5^{\,K-''}$$