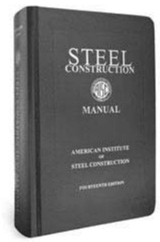
ELEMENTS OF ARCHITECTURAL STRUCTURES:
FORM, BEHAVIOR, AND DESIGN
ARCH 614
DR. ANNE NICHOLS
SPRING 2014

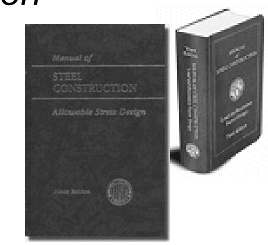
lecture
sixteen

steel construction: materials & beams

Steel Beam Design

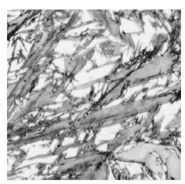
- American Institute of Steel Construction
  - Manual of Steel Construction
  - ASD & LRFD
  - combined in 2005





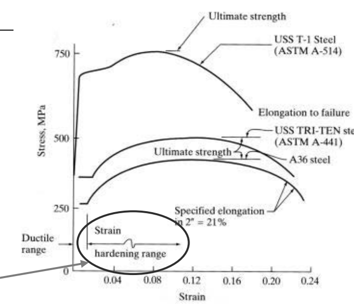
Steel Materials

- steel grades
  - ASTM A36 – carbon
    - plates, angles
    - $F_y = 36$ ksi & $F_u = 58$ ksi
  - ASTM A572 – high strength low-alloy
    - some beams
    - $F_y = 60$ ksi & $F_u = 75$ ksi
  - ASTM A992 – for building framing
    - <u>most beams</u>
    - $F_y = 50$ ksi & $F_u = 65$ ksi

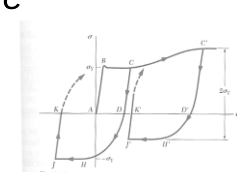


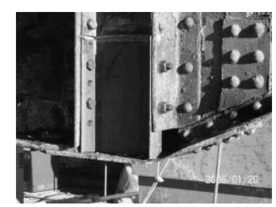
Steel Properties

- high strength to weight ratio
- elastic limit – yield (F_y)
- inelastic – plastic
- ultimate strength (F_u)
- ductile
- strength sensitive to temperature
- can corrode
- fatigue



strain hardening

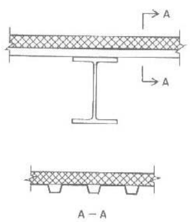


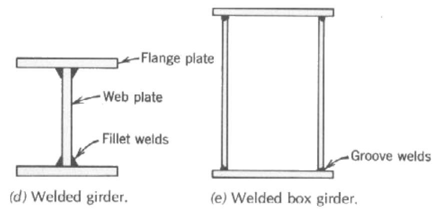


Winnepeg DOT

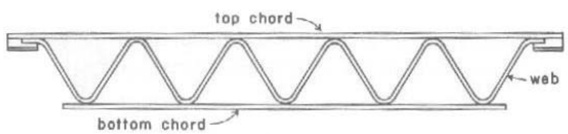
Structural Steel

- standard rolled shapes (W, C, L, T)
- open web joists
- plate girders
- decking



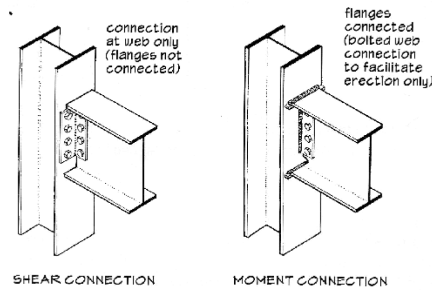


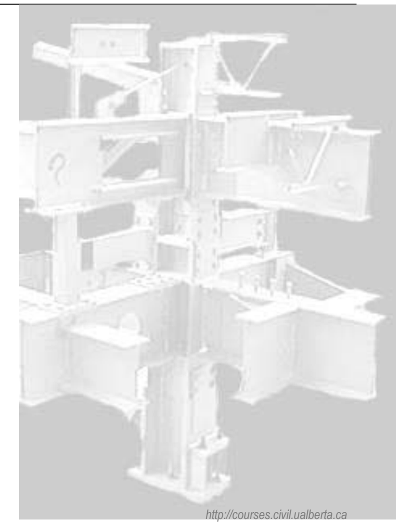
(d) Welded girder.
(e) Welded box girder.



Steel Construction

- welding
- bolts





Unified Steel Design

- ASD

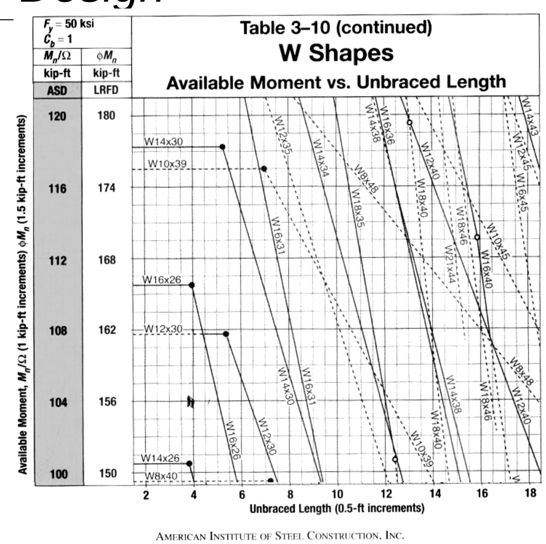
$$R_a \leq R_n/\Omega$$

- bending (braced) $\Omega = 1.67$
- bending (unbraced*) $\Omega = 1.67$
- shear $\Omega = 1.5$ or 1.67
- shear (bolts & welds) $\Omega = 2.00$
- shear (welds) $\Omega = 2.00$

* flanges in compression can buckle

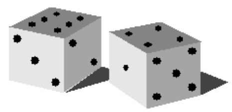
Unified Steel Design

- braced vs. unbraced



LRFD

- loads on structures are
  - not constant
  - can be more influential on failure
  - happen more or less often
  - UNCERTAINTY



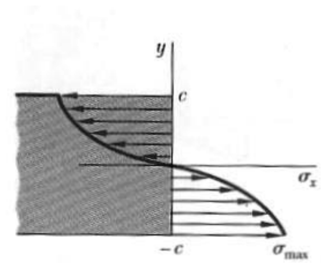
$$R_u = \gamma_D R_D + \gamma_L R_L \le \phi R_n$$

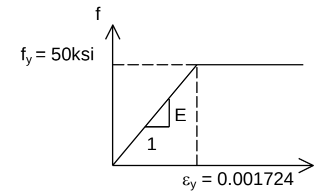
ϕ - resistance factor

γ - load factor for (D)ead & (L)ive load

LRFD Steel Beam Design

- limit state is yielding <u>all across section</u>
- outside elastic range
- load factors & resistance factors





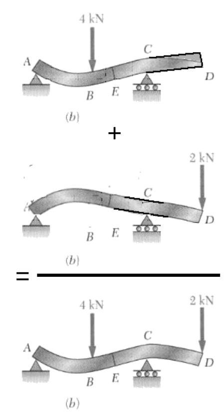
LRFD Load Combinations

ASCE-7 (2010)

- 1.4D
- 1.2D + 1.6L + 0.5(L_r or S or R)
- 1.2D + 1.6(L_r or S or R) + (L or 0.5W)
- 1.2D + 1.0W + L + 0.5(L_r or S or R)
- 1.2D + 1.0E + L + 0.2S
- 0.9D + 1.0W
- 0.9D + 1.0E
  - F has same factor as D in 1-5 and 7
  - H adds with 1.6 and resists with 0.9 (permanent)

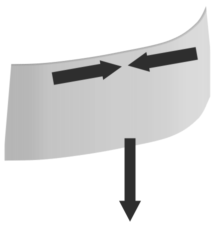
Beam Design Criteria (revisited)

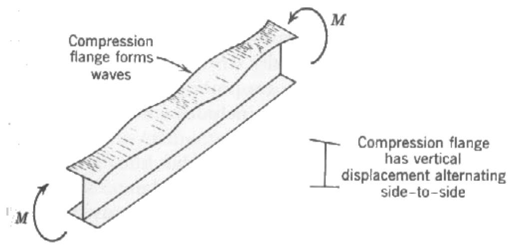
- strength design
  - bending stresses predominate
  - shear stresses occur
- serviceability
  - limit deflection
  - stability
- superpositioning
  - use of beam charts
  - elastic range only!
  - "add" moment diagrams
  - "add" deflection CURVES (not maximums)



Steel Beams

- *lateral stability - bracing*
- *local buckling – stiffen, or bigger I_y*

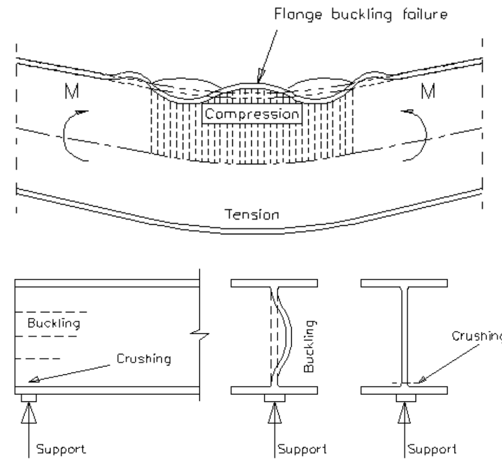




Local buckling of compression flange.

Local Buckling

- *steel I beams*
- *flange*
  - *buckle in direction of smaller radius of gyration*
- *web*
  - *force*
  - *"crippling"*



Local Buckling

- *flange*

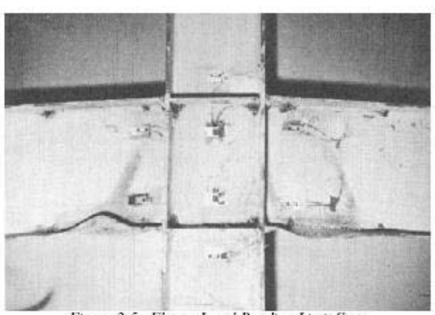


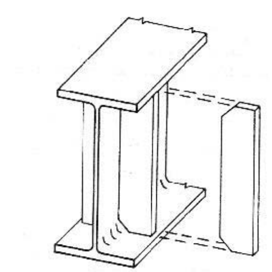
Figure 2-5. Flange Local Bending Limit State (Beedle, L.S., Christopher, R., 1964)

- *web*

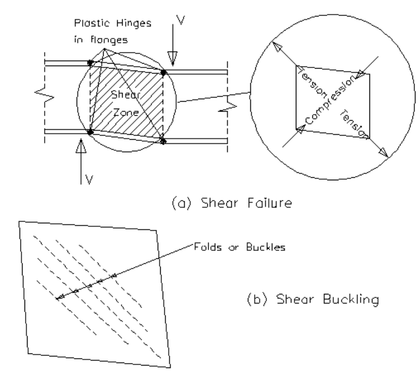
Figure 2-7. Web Local Buckling Limit State (SAC Project)

Shear in Web

- *panels in plate girders or webs with large shear*
- *buckling in compression direction*
- *add stiffeners*



stiffeners to prevent lateral buckling



Shear in Web

- plate girders and stiffeners

http:// nisee.berkeley.edu/godden

Steel Beams

- bearing
  - provide adequate area
  - prevent local yield of flange and web

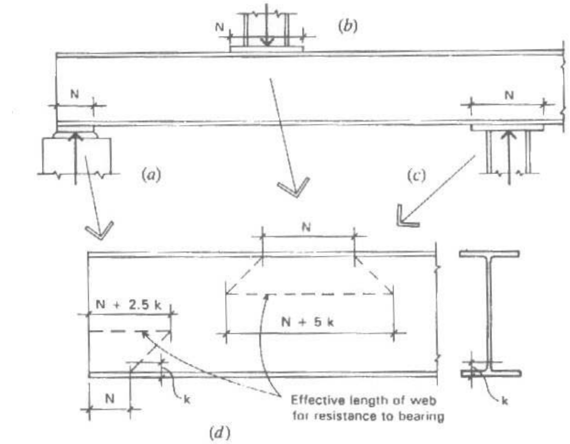


Figure 9.10 Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).

LRFD - Flexure

$$\Sigma\gamma_i R_i = M_u \leq \phi_b M_n = 0.9 F_y Z$$

M_u - maximum moment

ϕ_b - resistance factor for bending = 0.9

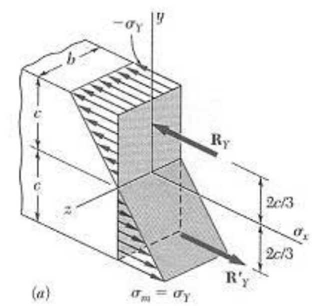
M_n - nominal moment (ultimate capacity)

F_y - yield strength of the steel

Z - plastic section modulus*

Internal Moments - at yield

- material hasn't failed

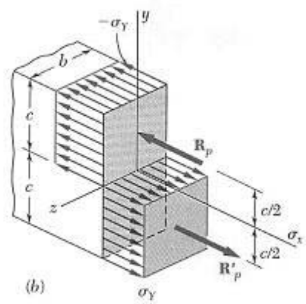


$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y$$

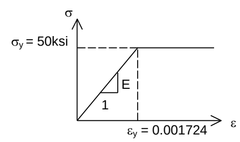
$$= \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y$$

Internal Moments - ALL at yield

- *all parts reach yield*
- *plastic hinge forms*
- *ultimate moment*
- $A_{tension} = A_{compression}$

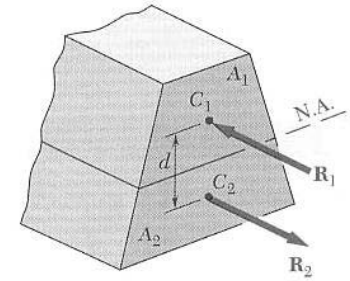


$$M_p = bc^2 f_y = \frac{3}{2} M_y$$



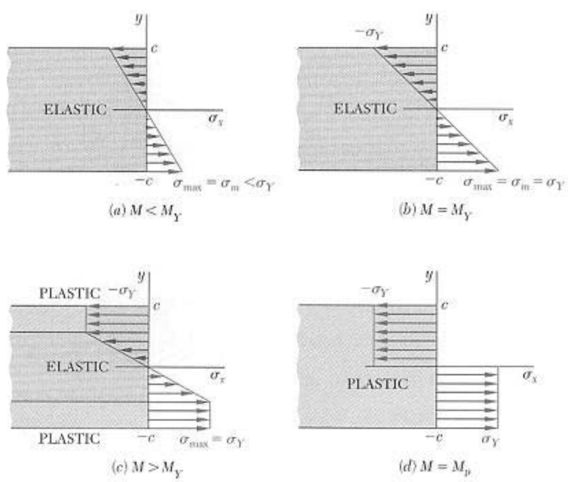
n.a. of Section at Plastic Hinge

- *cannot guarantee at centroid*
- $f_y A_1 = f_y A_2$
- *moment found from yield stress times <u>moment area</u>*



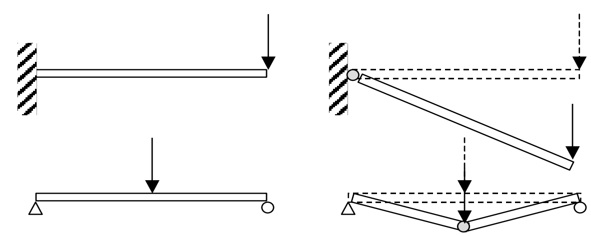
$$M_p = f_y A_1 d = f_y \sum_{n.a} A_i d_i$$

Plastic Hinge Development



Plastic Hinge Examples

- *stability can be effected*



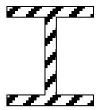
Plastic Section Modulus

- *shape factor, k*

$$k = {}^{M_p}\!/_{M_y}$$

= 3/2 for a rectangle

≈ 1.1 for an I



$$k = {}^{Z}\!/_{S}$$

- *plastic modulus, Z*

$$Z = \frac{M_p}{f_y}$$

LRFD - Shear

$$\Sigma\gamma_i R_i = V_u \le \phi_v V_n = 1.0(0.6F_{yw}A_w)$$

V_u - *maximum shear*
ϕ_v - *resistance factor for shear = 0.9*
V_n - *nominal shear*
F_{yw} - *yield strength of the steel in the web*
A_w - *area of the web =* $t_w d$

LRFD - Flexure Design

- *limit states for beam failure*
  1. *yielding*
  2. *lateral-torsional buckling*
  3. *flange local buckling*
  4. *web local buckling*
- *minimum M_n governs*

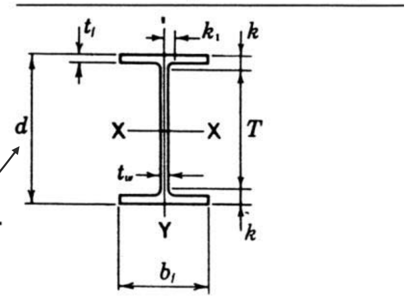
$$L_p = 1.76 r_y \sqrt{\frac{F_y}{E}}$$

$$\Sigma\gamma_i R_i = M_u \le \phi_b M_n$$

Compact Sections

- *plastic moment can form before any buckling*
- *criteria*
  – $\frac{b_f}{2t_f} \le 0.38\sqrt{\frac{E}{F_y}}$
  – *and* $\frac{h_c}{t_w} \le 3.76\sqrt{\frac{E}{F_y}}$

TABLE A.3 Properties of W Shapes



Lateral Torsional Buckling

$$M_n = C_b\left[\begin{array}{c}\text{moment based on}\\ \text{lateral buckling}\end{array}\right] \le M_p$$

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 2M_A + 4M_B + 3M_C}$$

C_b = modification factor

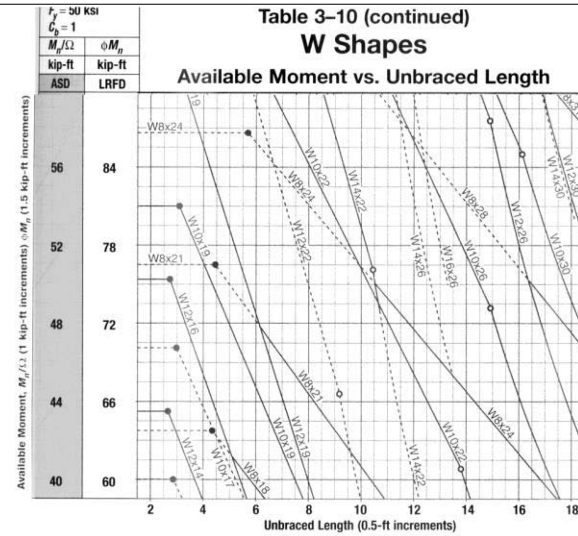
M_{max} - |max moment|, unbraced segment

M_A - |moment|, 1/4 point

M_B = |moment|, center point

M_C = |moment|, 3/4 point

Beam Design Charts



Charts & Deflections

- beam charts
  - solid line is most economical
  - dashed indicates there is another more economical section
  - self weight is NOT included in M_n
- deflections
  - no factors are applied to the loads
  - often governs the design

Design Procedure (revisited)

1. Know unbraced length, material, design method (Ω, ϕ)
2. Draw V & M, finding M_{max}
3. Calculate $Z_{req'd}$ $\left(f_b \le F_b\right)$ $(M_u \le \phi_b M_n)$
4. Choose (economical) section from section or beam capacity charts

Beam Charts by Z_x (pg. 250)

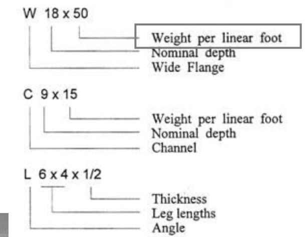
TABLE 9.1 Load Factor Resistance Design Selection for Shapes Used as Beams

		F_y = 36 ksi				F_y = 50 ksi								
Designation	Z_x in.^{3}	L_p ft	L_r ft	M_p kip-ft	M_r kip-ft	L_p ft	L_r ft	M_p kip-ft	M_r kip-ft	r_y in.	$b_f/2t_f$	h/t_w	X_1 ksi	$X_2 \times 10^6$ (1/ksi)^2
W 33 × 141	**514**	**10.1**	**30.1**	**1,542**	**971**	**8.59**	**23.1**	**2,142**	**1,493**	**2.43**	**6.01**	**49.6**	**1,800**	**17,800**
W 30 × 148	500	9.50	30.6	1,500	945	8.06	22.8	2,083	1,453	2.28	4.44	41.6	2,310	6,270
W 24 × 162	468	12.7	45.2	1,404	897	10.8	32.4	1,950	1,380	3.05	5.31	30.6	2,870	2,260
W 24 × 146	418	12.5	42.0	1,254	804	10.6	30.6	1,742	1,237	3.01	5.92	33.2	2,590	3,420
W 33 × 118	**415**	**9.67**	**27.8**	**1,245**	**778**	**8.20**	**21.7**	**1,729**	**1,197**	**2.32**	**7.76**	**54.5**	**1,510**	**37,700**
W 30 × 124	408	9.29	28.2	1,224	769	7.88	21.5	1,700	1,183	2.23	5.65	46.2	1,930	13,500
W 21 × 147	373	12.3	46.4	1,119	713	10.4	32.8	1,554	1,097	2.95	5.44	26.1	3,140	1,590
W 24 × 131	370	12.4	39.3	1,110	713	10.5	29.1	1,542	1,097	2.97	6.70	35.6	2,330	5,290
W 18 × 158	356	11.4	56.5	1,068	672	9.69	38.0	1,483	1,033	2.74	3.92	19.8	4,410	403
W 30 × 108	**346**	**8.96**	**26.3**	**1,038**	**648**	**7.60**	**20.3**	**1,442**	**997**	**2.15**	**6.89**	**49.6**	**1,680**	**24,200**
W 27 × 114	343	9.08	28.2	1,029	648	7.71	21.3	1,429	997	2.18	5.41	42.5	2,100	9,220
W 24 × 117	327	12.3	37.1	981	631	10.4	27.9	1,363	970	2.94	7.53	39.2	2,090	8,190
W 21 × 122	307	12.2	41.0	921	592	10.3	29.8	1,279	910	2.92	6.45	31.3	2,630	3,160
W 18 × 130	290	11.3	47.7	870	555	9.55	32.8	1,208	853	2.7	4.65	23.9	3,680	810
W 30 × 90	**283**	**8.71**	**24.8**	**849**	**531**	**7.39**	**19.4**	**1,179**	**817**	**2.09**	**8.52**	**57.5**	**1,410**	**49,600**
W 24 × 103	280	8.29	27.0	840	531	7.04	20.0	1,167	817	1.99	4.59	39.2	2,390	5,310
W 27 × 94	278	8.83	25.9	834	527	7.50	19.9	1,158	810	2.12	6.70	49.5	1,740	19,900
W 14 × 145	260	16.6	81.6	780	503	14.1	54.7	1,083	773	3.98	7.11	16.8	4,400	348
W 24 × 94	254	8.25	25.9	762	481	7.00	19.4	1,058	740	1.98	5.18	41.9	2,180	7,800

Beam Design (revisited)

4*. Include self weight for M_{max}

– and repeat 3 & 4 if necessary



5. Consider lateral stability

Unbraced roof trusses were blown down in 1999 at this project in Moscow, Idaho.

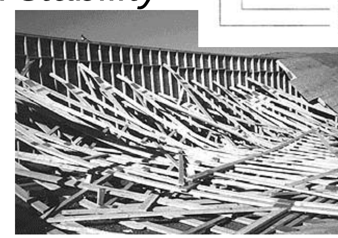


Photo: Ken Carper

Beam Design (revisited)

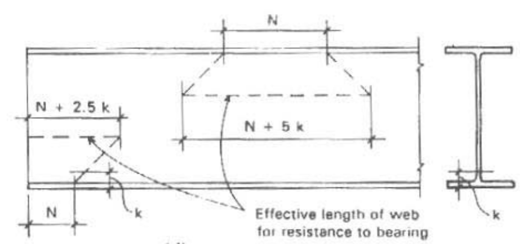
6. Evaluate shear stresses - horizontal

- $\left(f_v \le F_v\right)$ or $\left(V_u \le \phi_v V_n\right)$
- W and rectangles $\quad f_{v-\max} = \dfrac{3V}{2A} \approx \dfrac{V}{A_{web}}$
- thin walled sections $\quad f_{v-\max} = \dfrac{VQ}{Ib}$

Beam Design (revisited)

7. Provide adequate bearing area at supports

$$f_p = \frac{P}{A} \le F_p$$

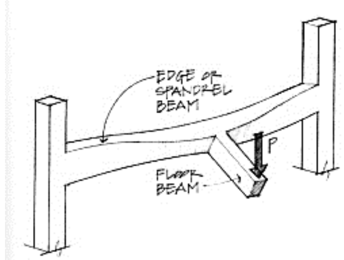


(d)

Beam Design (revisited)

8. Evaluate torsion

$$\left(f_v \le F_v\right)$$



- *circular cross section*

$$f_v = \frac{T\rho}{J}$$

- *rectangular*

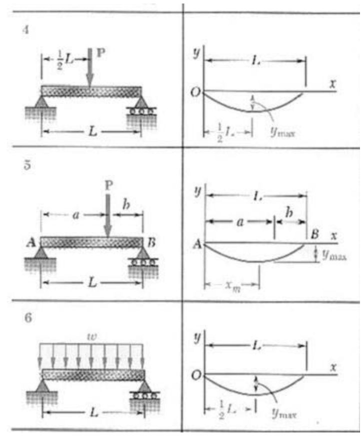
$$f_v = \frac{T}{c_1 ab^2}$$

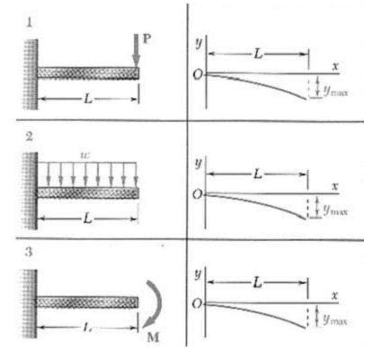
TABLE 3.1. Coefficients for Rectangular Bars in Torsion

a/b	c_1	c_2
1.0	0.208	0.1406
1.2	0.219	0.1661
1.5	0.231	0.1958
2.0	0.246	0.229
2.5	0.258	0.249
3.0	0.267	0.263
4.0	0.282	0.281
5.0	0.291	0.291
10.0	0.312	0.312
∞	0.333	0.333

Beam Design (revisited)

9. Evaluate deflections *– NO LOAD FACTORS*





$$y_{\max}(x) = \Delta_{actual} \le \Delta_{allowable}$$

Load Tables & Equivalent Load

- *uniformly distributed loads*
- *equivalent "w"*

$$M_{\max} = \frac{w_{equivalent}L^2}{8}$$

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	8K1	10K1	12K1	12K3	12K5	14K1	14K3	14K4	14K6	16K2	16K3	16K4	16K5	16K6	16K7	16K9
Depth (in.)	8	10	12	12	12	14	14	14	14	16	16	16	16	16	16	16
Approx. Wt (lbs./ft.)	5.1	5.0	5.0	5.7	7.1	5.2	6.0	6.7	7.7	5.5	6.3	7.0	7.5	8.1	8.6	10.0
Span (ft.) ↓																
8	825 / 550															
9	825 / 550															
10	825 / 480	825 / 550														
11	798 / 377	825 / 542														
12	666 / 288	825 / 455	825 / 550	825 / 550	825 / 550											
13	565 / 225	718 / 363	825 / 510	825 / 510	825 / 510											
14	486 / 179	618 / 289	750 / 425	825 / 463	825 / 463	825 / 550	825 / 550	825 / 550	825 / 550							
15	421 / 145	537 / 234	651 / 344	814 / 428	825 / 434	766 / 475	825 / 507	825 / 507	825 / 507							
16	369 / 119	469 / 192	570 / 282	714 / 351	825 / 396	672 / 390	825 / 467	825 / 467	825 / 467	825 / 550	825 / 550	825 / 550	825 / 550	825 / 550	825 / 550	825 / 550
17		415 / 159	504 / 234	630 / 291	825 / 366	592 / 324	742 / 404	825 / 443	825 / 443	768 / 488	825 / 526	825 / 526	825 / 526	825 / 526	825 / 526	825 / 526
18		369 / 134	448 / 197	561 / 245	760 / 317	528 / 272	661 / 339	795 / 397	825 / 408	684 / 409	762 / 456	825 / 490	825 / 490	825 / 490	825 / 490	825 / 490
19		331 / 113	402 / 167	502 / 207	681 / 269	472 / 230	592 / 287	712 / 336	825 / 383	612 / 347	682 / 386	820 / 452	825 / 455	825 / 455	825 / 455	825 / 455
20		298 / 97	361 / 142	453 / 177	613 / 230	426 / 197	534 / 246	642 / 287	787 / 347	552 / 297	615 / 330	739 / 386	825 / 426	825 / 426	825 / 426	825 / 426
21			327 / 123	409 / 153	555 / 198	385 / 170	483 / 212	582 / 248	712 / 299	499 / 255	556 / 285	670 / 333	754 / 373	822 / 405	825 / 406	825 / 406
22			298 / 106	373 / 132	505 / 172	351 / 147	439 / 184	529 / 215	648 / 259	454 / 222	505 / 247	609 / 289	687 / 323	747 / 351	825 / 385	825 / 385
23			271 / 93	340 / 116	462 / 150	321 / 128	402 / 160	483 / 188	592 / 226	415 / 194	462 / 216	556 / 252	627 / 282	682 / 307	760 / 339	825 / 363

load for live load deflection limit in RED, total in BLACK

Steel Arches and Frames

- *solid sections or open web*

http:// nisee.berkeley.edu/godden

Steel Shell and Cable Structures

Approximate Depths