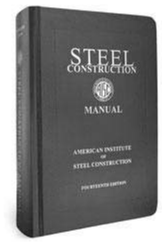
ARCHITECTURAL STRUCTURES:
FORM, BEHAVIOR, AND DESIGN
ARCH 331
DR. ANNE NICHOLS
FALL 2013

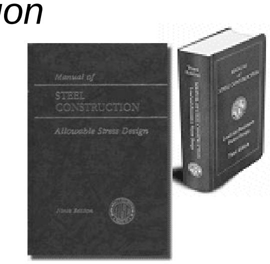
lecture
eighteen

steel construction: materials & beams

Steel Beam Design

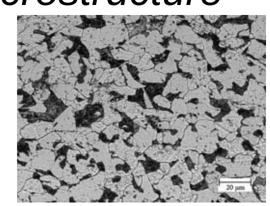
- American Institute of Steel Construction
  - Manual of Steel Construction
  - ASD & LRFD
  - combined in 2005





Steel Materials

- smelt iron ore
- add alloying elements
- heat treatments
- iron, carbon
- microstructure



A36 steel, JOM 1998

AISC

Steel Materials

- cast into billets
- hot rolled
- cold formed
- residual stress
- corrosion-resistant "weathering" steels
- stainless

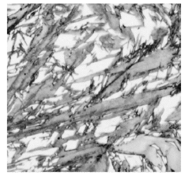
Hot Rolled

Cold Formed

AISC

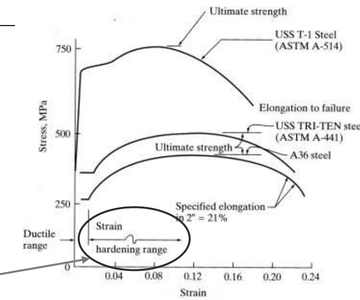
Steel Materials

- 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

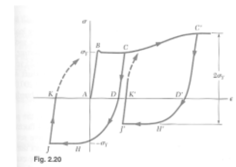


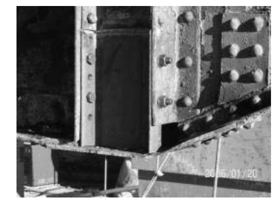
Steel Properties

- 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

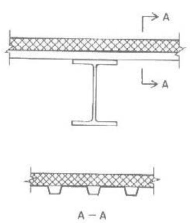


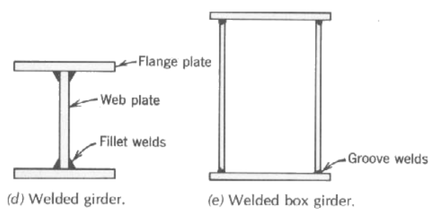


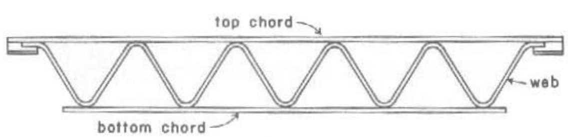
Winnepeg DOT

Structural Steel

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

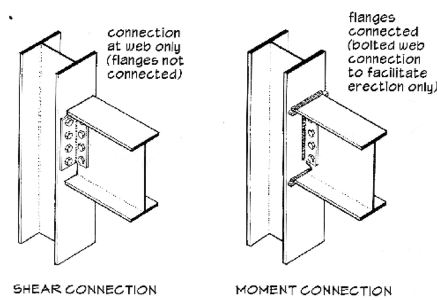


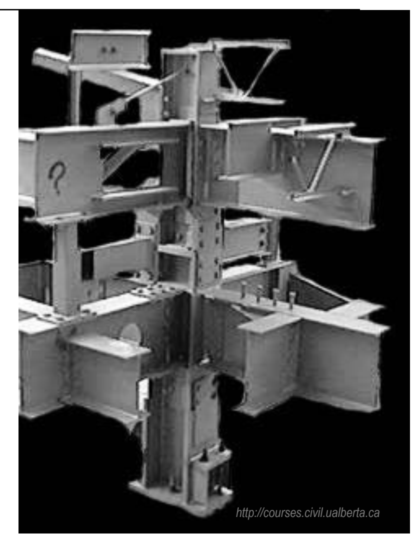




Steel Construction

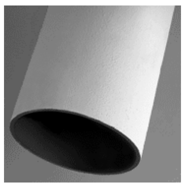
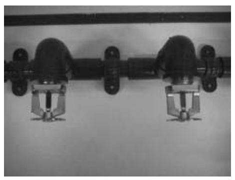
- welding
- bolts





Steel Construction

- *fire proofing*
  - *– cementicious spray*
  - *– encasement in gypsum*
  - *– intumescent – expands with heat*
  - *– sprinkler system*

Unified Steel Design

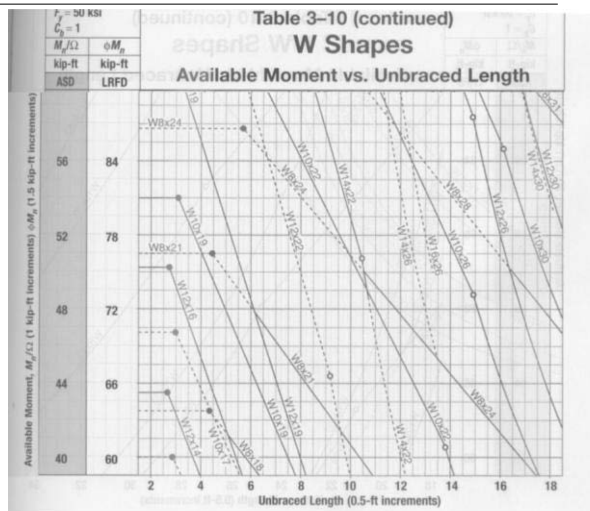
- *ASD* $$R_a \leq R_n / \Omega$$

  - *– 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*

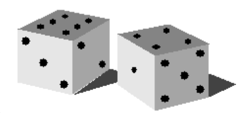
Unified Steel Design

- *braced vs. unbraced*



LRFD

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



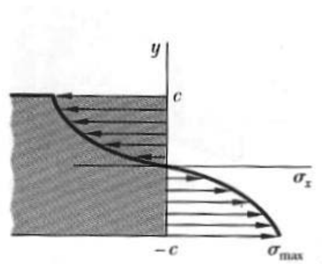
$$R_u = \gamma_D R_D + \gamma_L R_L \leq \phi R_n$$

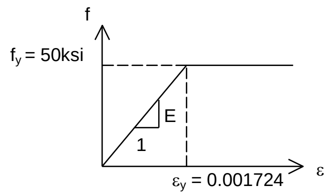
ϕ *- resistance factor*

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

LRFD Steel Beam Design

- *limit state is yielding all across section*
- *outside elastic range*
- *load factors & resistance factors*





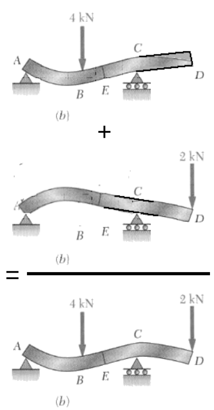
LRFD Load Combinations

ASCE-7 (2005)

- *1.4(D + F)*
- *1.2(D + F + T)+ 1.6(L + H)+ 0.5(L_r or S or R)*
- *1.2D + 1.6(L_r or S or R) + (L or 0.8W)*
- *1.2D + 1.6W + L + 0.5(L_r or S or R)*
- *1.2D + 1.0E + L + 0.2S*
- *0.9D + 1.6W + 1.6H*
- *0.9D + 1.0E + 1.6H*

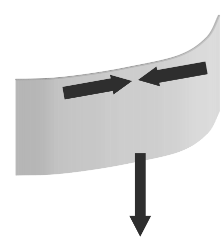
Beam Design Criteria (revisited)

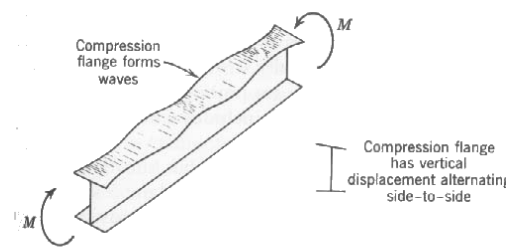
- *strength design*
  - *bending stresses predominate*
  - *shear stresses occur*
- *serviceability*
  - *limit deflection*
  - *stability*
- *superpositioning*
  - *use of beam charts*
  - *elastic range only!*
  - *"add" moment diagrams*
  - *"add" deflection CURVES (not maximums)*



Steel Beams

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

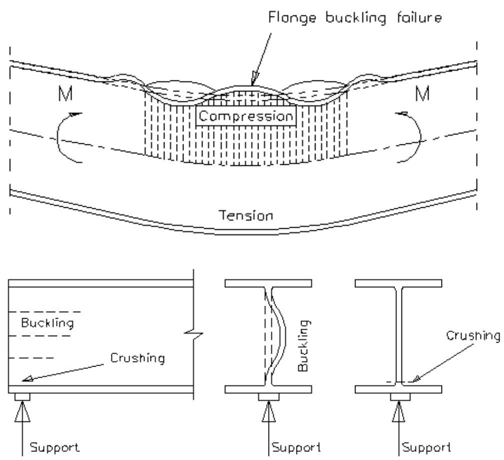




Local buckling of compression flange.

Local Buckling

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



Local Buckling

- flange

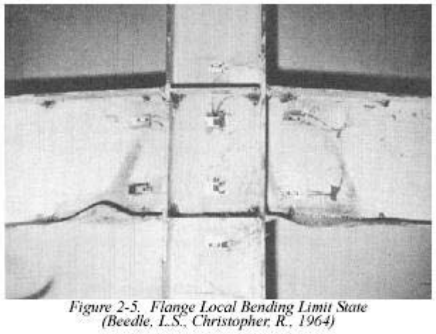


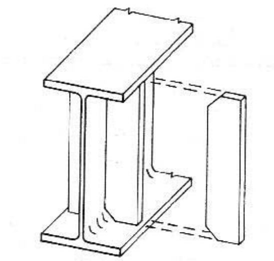
Figure 2-5. Flange Local Bending Limit State (Beedle, L.S., Christopher, R., 1964)

- web

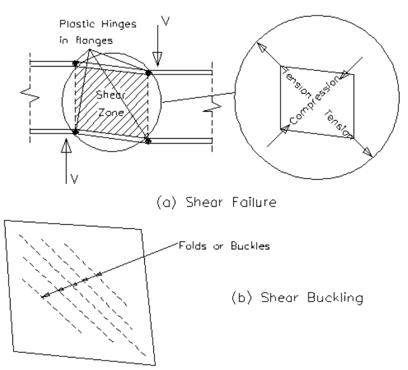
Figure 2-7. Web Local Buckling Limit State (SAC Project)

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

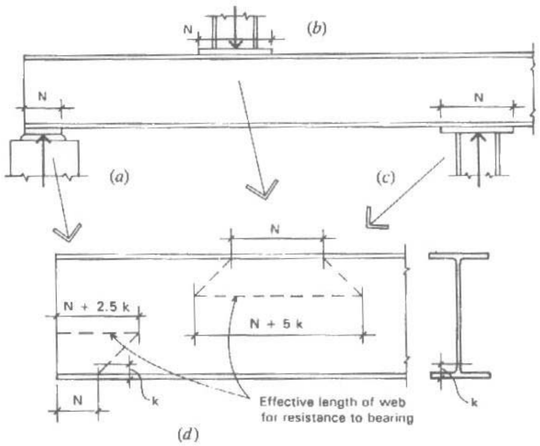


Figure 9.10 Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).

LRFD - Flexure

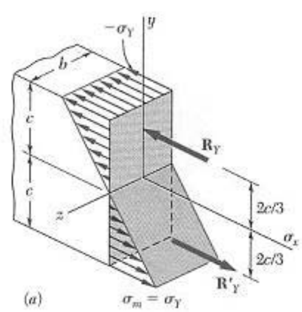
$$\Sigma\gamma_i R_i = M_u \le \phi_b M_n = 0.9F_y Z$$

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

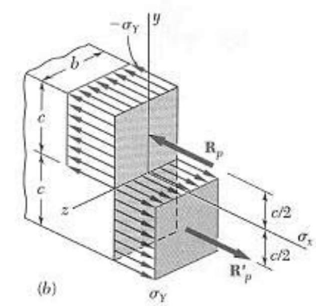
$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y$$



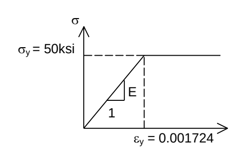
$$= \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y$$

Internal Moments - ALL at yield

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

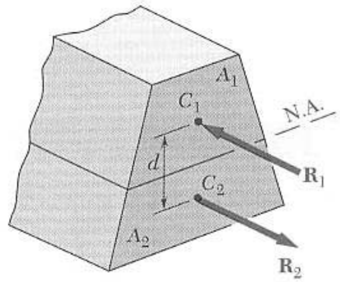


$$M_p = bc^2 f_y = \tfrac{3}{2} M_y$$



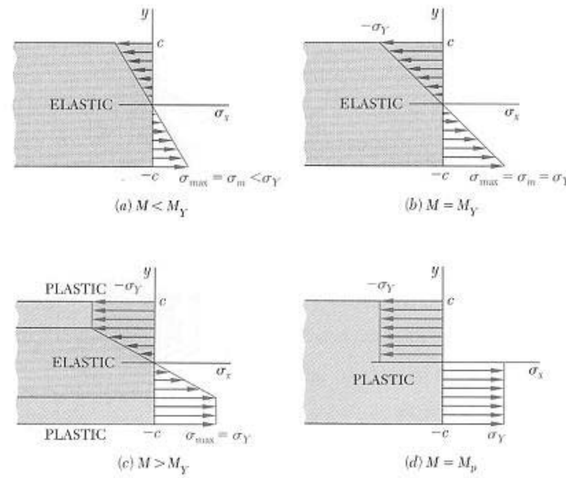
n.a. of Section at Plastic Hinge

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



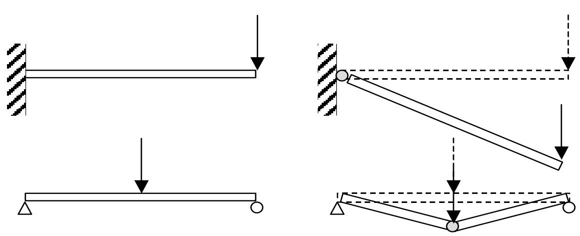
$$M_p = f_y A_1 d = f_y \sum_{n.a} A_i d_i$$

Plastic Hinge Development



Plastic Hinge Examples

- stability can be effected



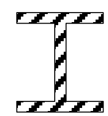
Plastic Section Modulus

- shape factor, k

$$k = M_p / M_y$$

= 3/2 for a rectangle

$\approx$ 1.1 for an I



$$k = Z / S$$

- plastic modulus, Z

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

LRFD – Shear (compact shapes)

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

V_u - maximum shear

ϕ_v - resistance factor for shear = 1.0

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

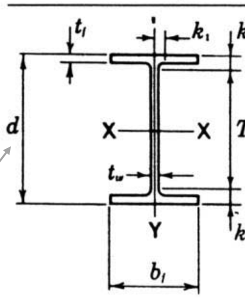
$$L_p = 1.76 r_y \sqrt{\frac{F_y}{E}}$$

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

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

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

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

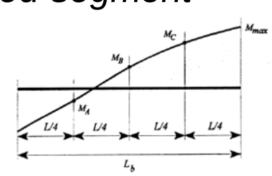
C_b = modification factor

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

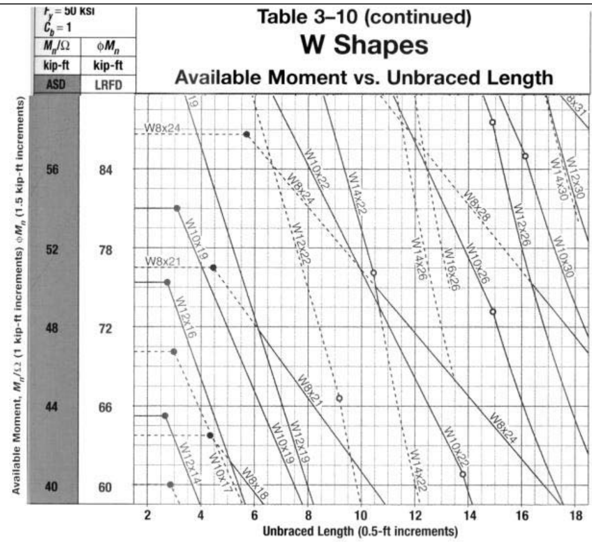
M_A - |moment|, 1/4 point

M_B = |moment|, center point

M_C = |moment|, 3/4 point



Beam Design Charts



Charts & Deflections

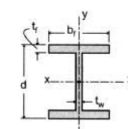
- *beam charts*
  - *solid line is most economical*
  - *dashed indicates there is another more economical section*
  - *self weight is NOT included in M_n*
- *deflections*
  - *no factors are applied to the loads*
  - *often governs the design*

Design Procedure (revisited)

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

Beam Charts by S_x (Appendix A)

Table 11 Listing of W Shapes in Descending Order of S_x for Beam Design.



S_x **Allowable Stress Design—Selected Beam Shapes** S_x

S_x—US (in.^{3})	Section	S_x—SI ($10^3 \times mm^3$)	S_x—US (in.^{3})	Section	S_x—SI ($10^3 \times mm^3$)
448	**W33 × 141**	**7350**	188	W18 × 97	3080
439	W36 × 135	7200			
411	W27 × 146	6740	**176**	**W24 × 76**	**2890**
			175	W16 × 100	2870
406	**W33 × 130**	**6660**	173	W14 × 109	2840
380	W30 × 132	6230	171	W21 × 83	2800
371	W24 × 146	6080	166	W18 × 86	2720
			157	W14 × 99	2570
359	**W33 × 118**	**5890**	155	W16 × 89	2540
355	W30 × 124	5820			
			154	**W24 × 68**	**2530**
329	**W30 × 116**	**5400**	151	W21 × 73	2480
329	W24 × 131	5400	146	W18 × 76	2390
329	W21 × 147	5400	143	W14 × 90	2350
299	**W30 × 108**	**4900**	**140**	**W21 × 68**	**2300**
299	W27 × 114	4900	134	W16 × 77	2200

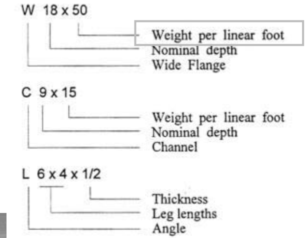
Beam Charts by Z_x

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}*
- *it's dead load*
- *and repeat 3 & 4 if necessary*



5. *Consider lateral stability*

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

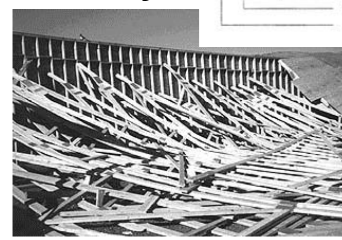


Photo: Ken Carper

Beam Design (revisited)

6. *Evaluate shear stresses - horizontal*
- $(V_a \le V_n/\Omega)$ or $(V_u \le \phi_v V_n)$
- *rectangles and W's* $f_{v-max} = \dfrac{3V}{2A} \approx \dfrac{V}{A_{web}}$

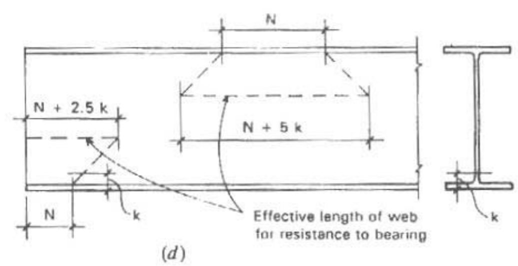
$$V_n = 0.6\, F_{yw} A_w$$

- *general* $f_{v-max} = \dfrac{VQ}{Ib}$

Beam Design (revisited)

7. *Provide adequate bearing area at supports*

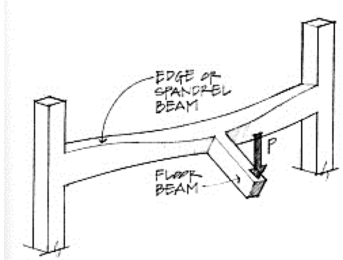
$$(P_a \le P_n/\Omega)$$
$$(P_u \le \phi P_n)$$



Beam Design (revisited)

8. Evaluate torsion

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



- circular cross section

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

- rectangular

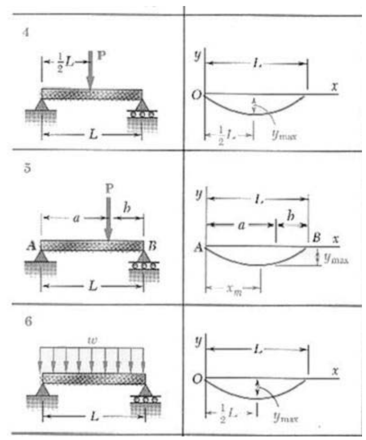
$$f_v = \frac{T}{c_1 ab^2}$$

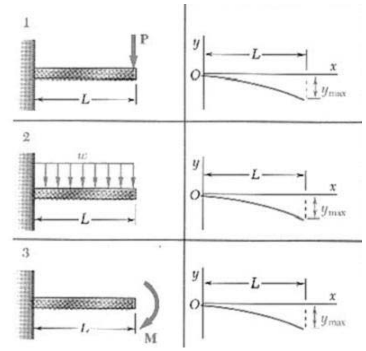
TABLE 3.1. Coefficients for Rectangular Bars in Torsion

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"

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

LRFD

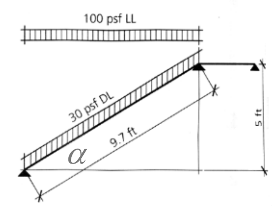
STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based On A 50 ksi Maximum Yield Strength - Loads Shown In Pounds Per Linear Foot (plf)

Joist Designation	10K1	12K1	12K3	12K5	14K1	14K3	14K4	14K6	16K2	16K3	16K4	16K5	16K6	16K7	16K9
Depth (in.)	10	12	12	12	14	14	14	14	16	16	16	16	16	16	16
Approx. Wt (lbs./ft.)	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.) ↓															
10	825 550														
11	825 542														
12	825 455	825 550	825 550	825 550											
13	718 363	825 510	825 510	825 510											
14	618 289	750 425	825 463	825 463	825 550	825 550	825 550	825 550							
15	537 234	651 344	814 428	825 434	766 475	825 507	825 507	825 507							
16	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

Sloped Beams

- stairs & roofs
- projected live load
- dead load over length



- perpendicular load to beam:

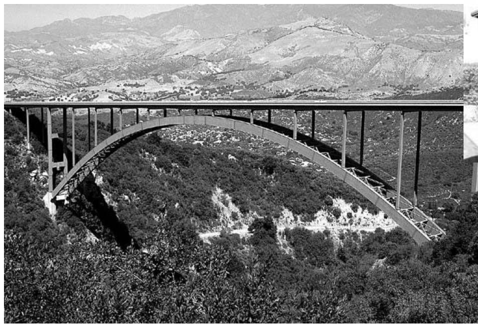
$$w_{\perp} = w \cdot \cos\alpha$$

- equivalent distributed load:

$$w_{adj.} = \frac{w}{\cos\alpha}$$

Steel Arches and Frames

- *solid sections or open web*



http:// nisee.berkeley.edu/godden

Steel Shell and Cable Structures

Approximate Depths