ELEMENTS OF ARCHITECTURAL STRUCTURES:
FORM, BEHAVIOR, AND DESIGN

ARCH 614
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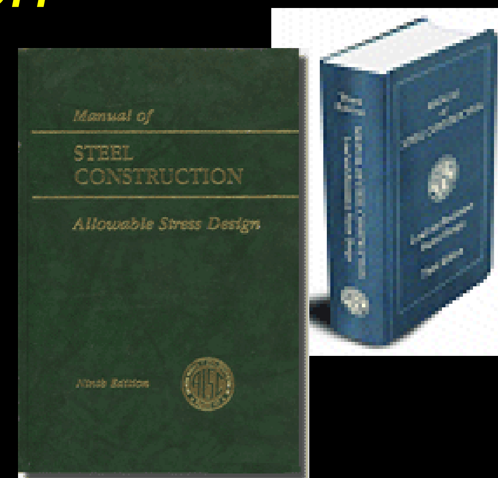
SPRING 2014

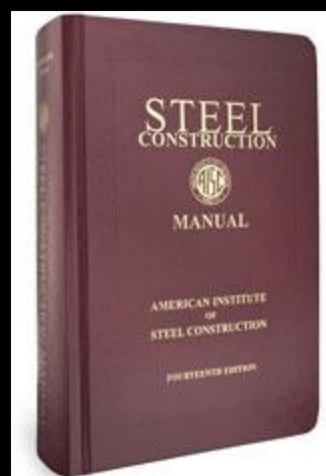
lecture
sixteen

steel construction: materials & beams

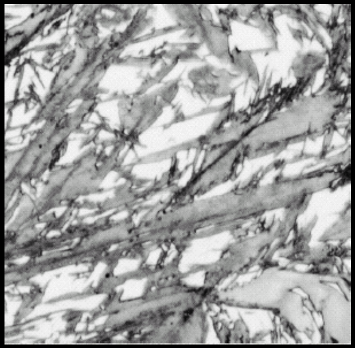
Steel Beam Design

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





Steel Materials

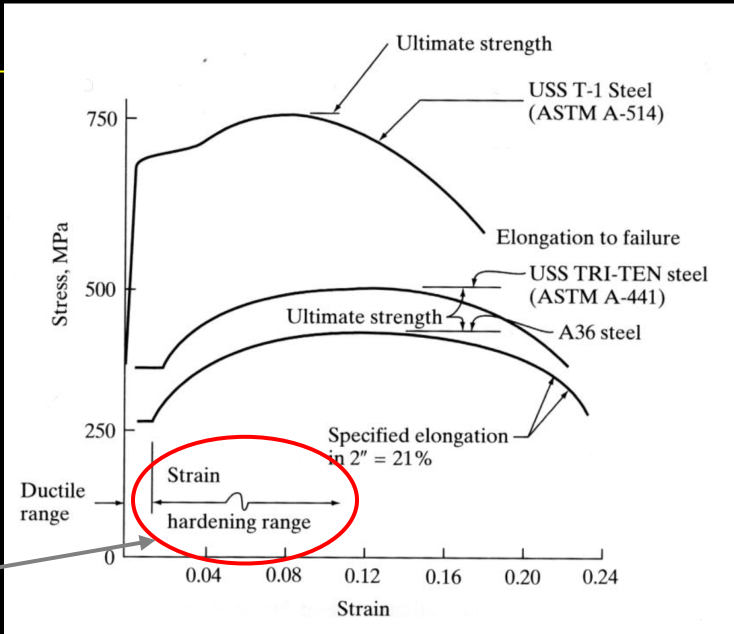


- steel grades
  - ASTM A36 – carbon
    - plates, angles
    - F_y = 36 ksi & F_u = 58 ksi
  - ASTM A572 – high strength low-alloy
    - some beams
    - F_y = 60 ksi & F_u = 75 ksi
  - ASTM A992 – for building framing
    - <u>most beams</u>
    - F_y = 50 ksi & F_u = 65 ksi

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



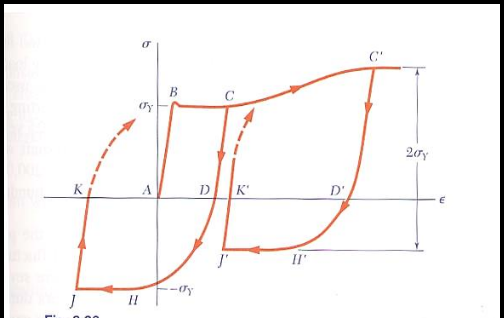
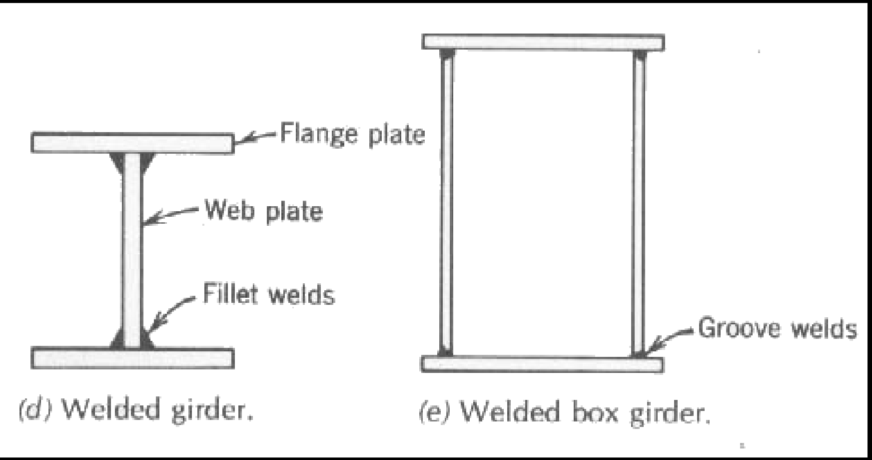


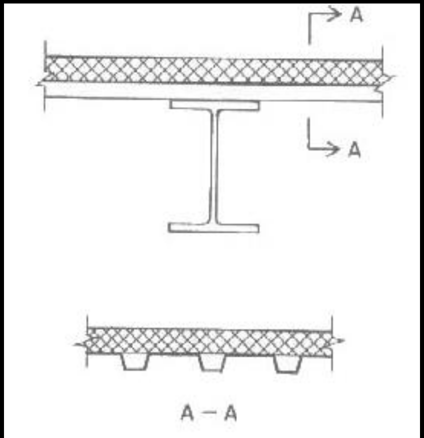
Fig. 2.20

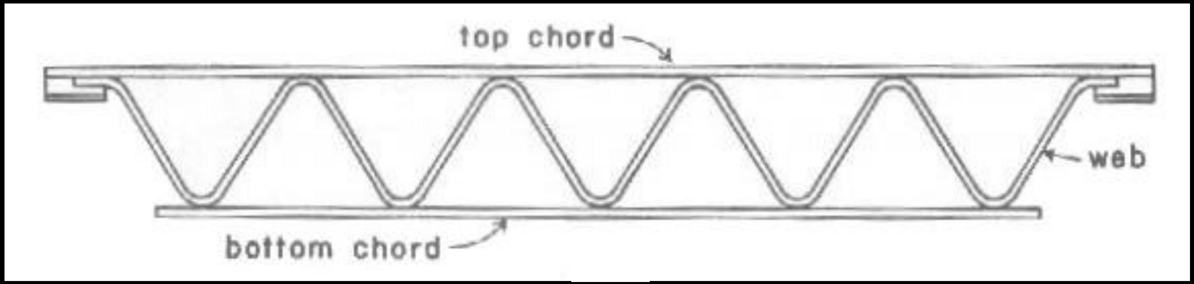
Winnepeg DOT

Structural Steel

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

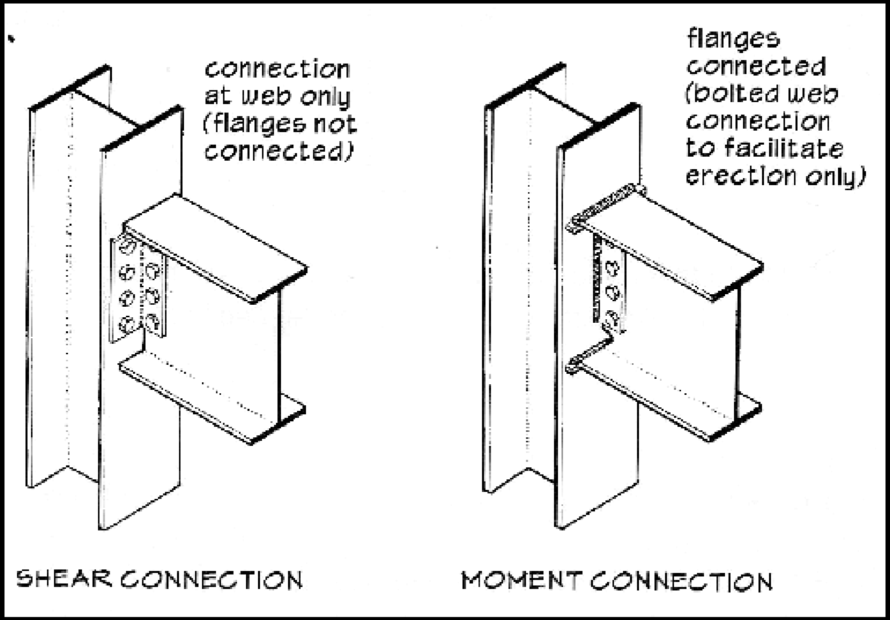


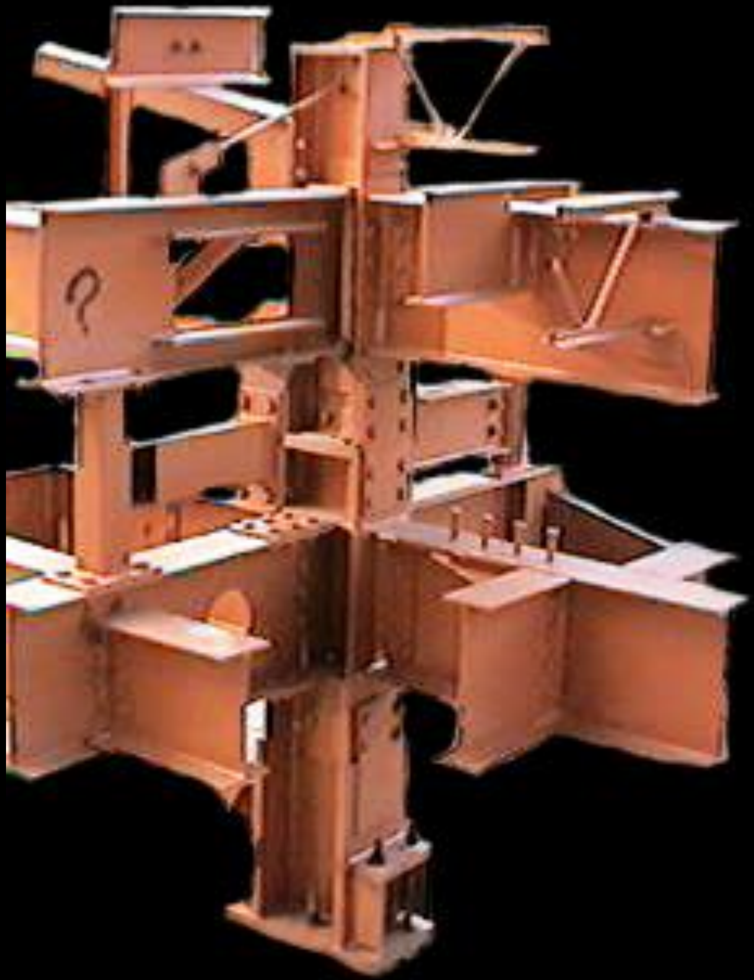




Steel Construction

- *welding*
- *bolts*





http://courses.civil.ualberta.ca

Unified Steel Design

- ASD

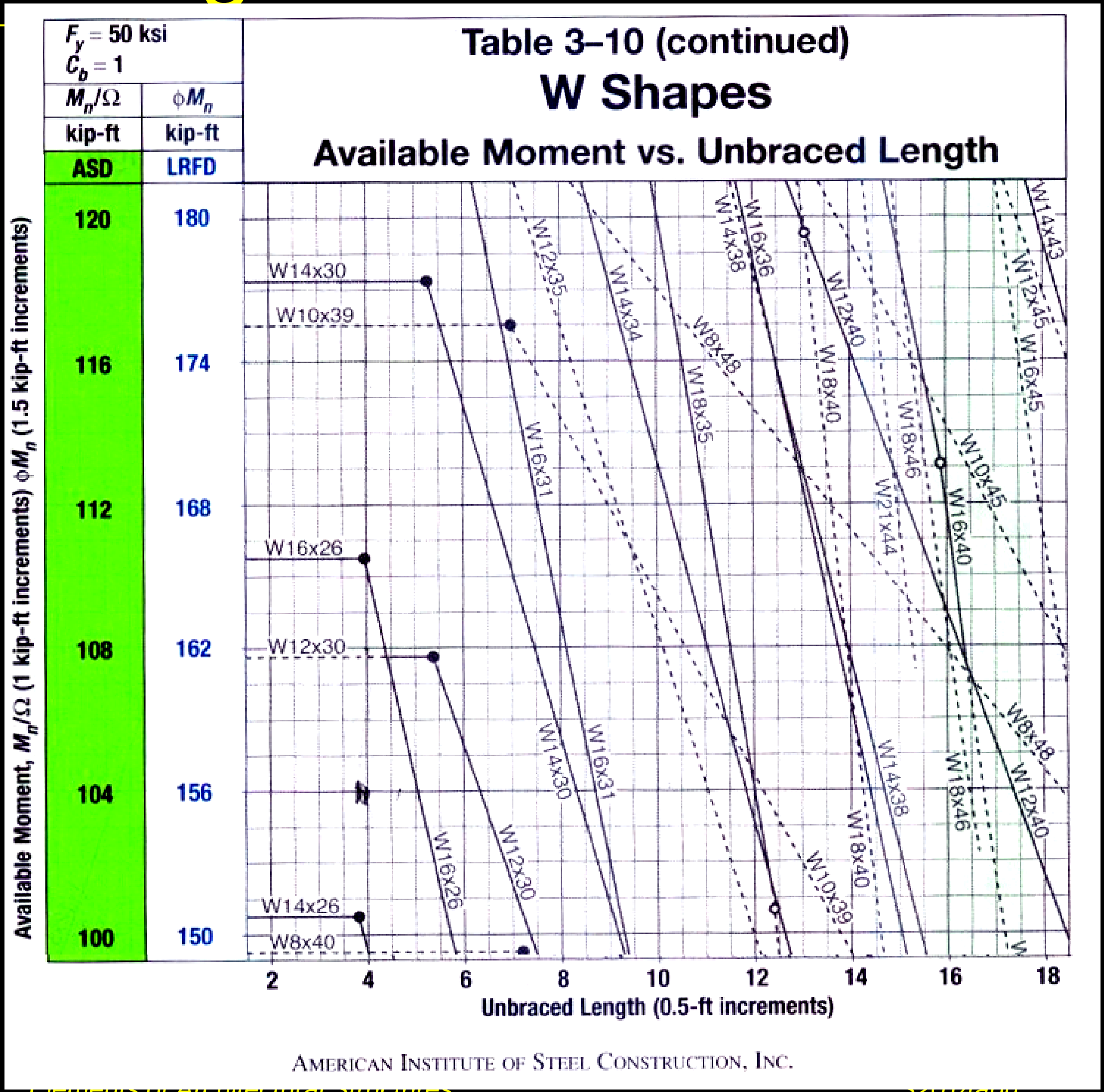
$$R_a \le \frac{R_n}{\Omega}$$

  - 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

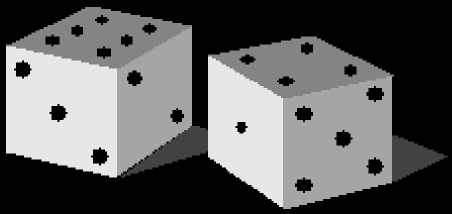
Unified Steel Design

- *braced vs. unbraced*



LRFD

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



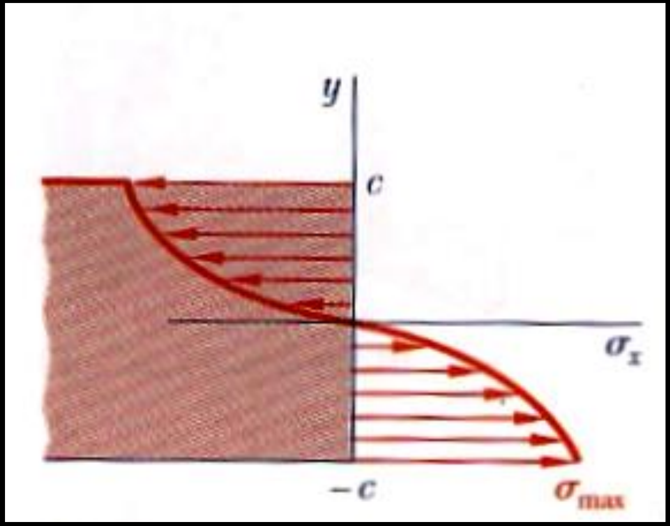
$$R_u = \gamma_D R_D + \gamma_L R_L \leq \phi R_n$$

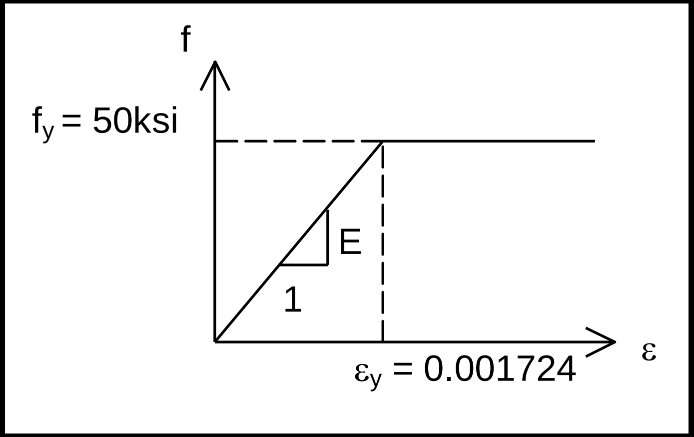
ϕ - resistance factor

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

LRFD Steel Beam Design

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





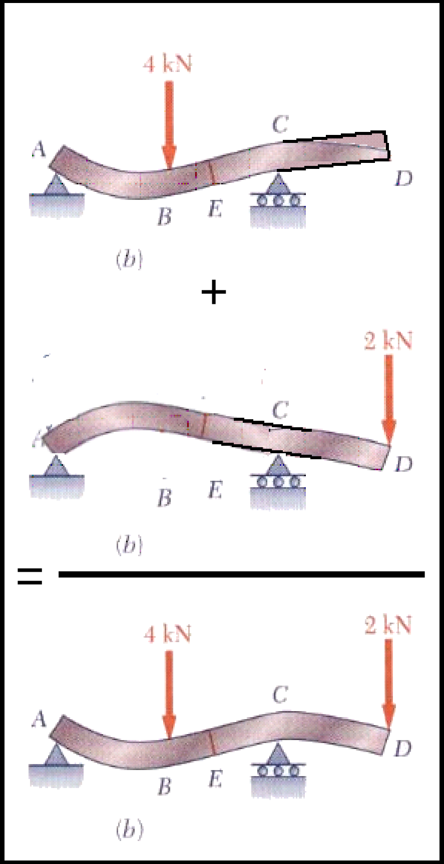
LRFD Load Combinations

ASCE-7 (2010)

- $\boxed{\begin{array}{l} 1.4D \\ 1.2D + 1.6L \end{array}}$ $+ 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
- $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.0E + L + 0.2S$
- $0.9D + 1.0W$
- $0.9D + 1.0E$
  - F has same factor as D in 1-5 and 7
  - H adds with 1.6 and resists with 0.9 (permanent)

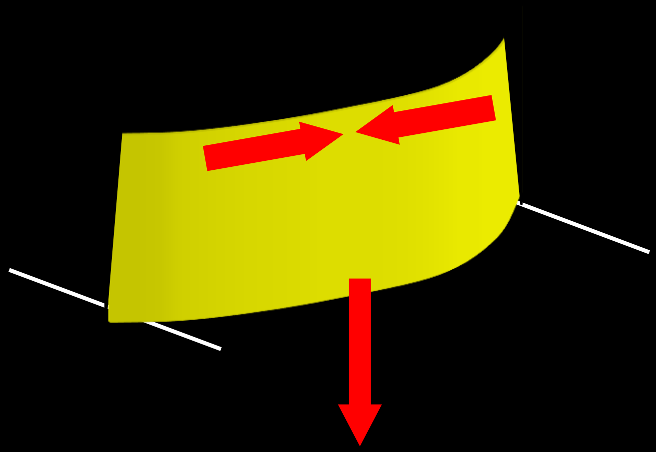
Beam Design Criteria (revisited)

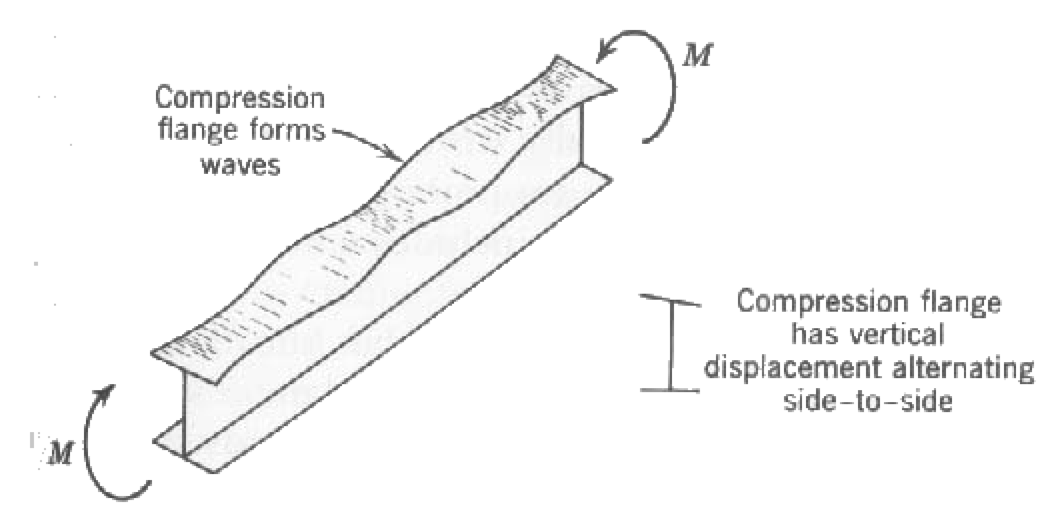
- strength design
  - bending stresses predominate
  - shear stresses occur
- serviceability
  - limit deflection
  - stability
- superpositioning
  - use of beam charts
  - elastic range only!
  - “add” moment diagrams
  - “add” deflection CURVES (not maximums)



Steel Beams

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

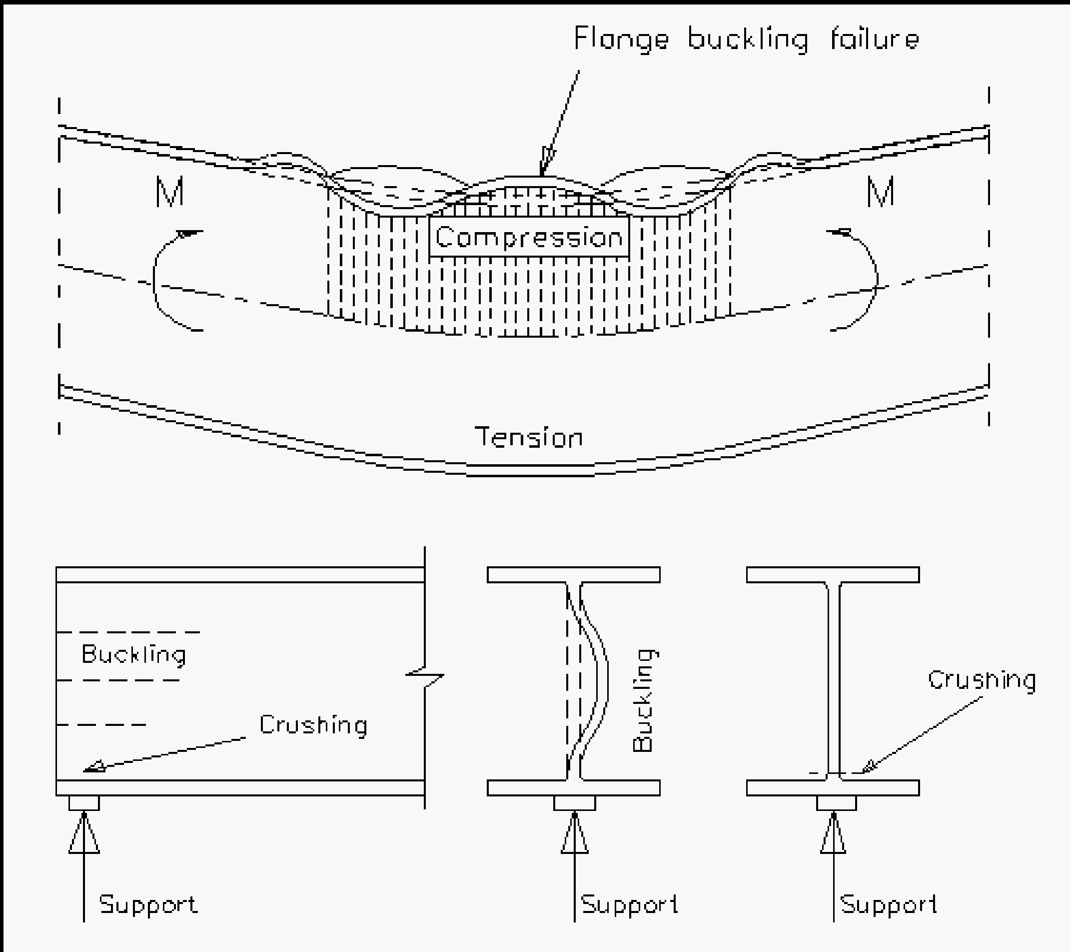




Local buckling of compression flange.

Local Buckling

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



Local Buckling

- *web*

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

- *flange*

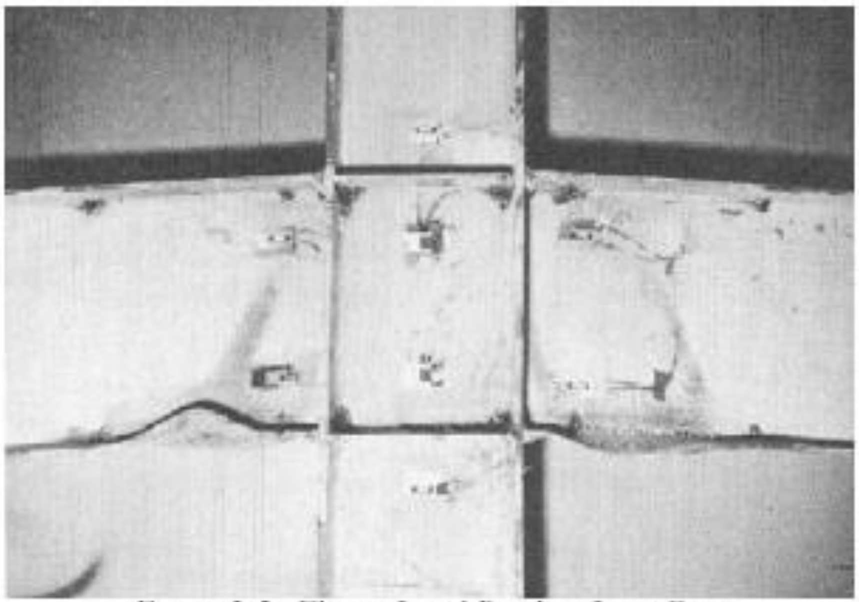
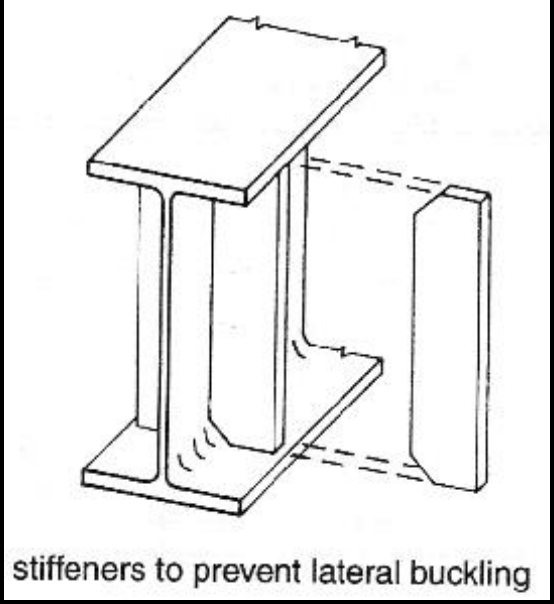
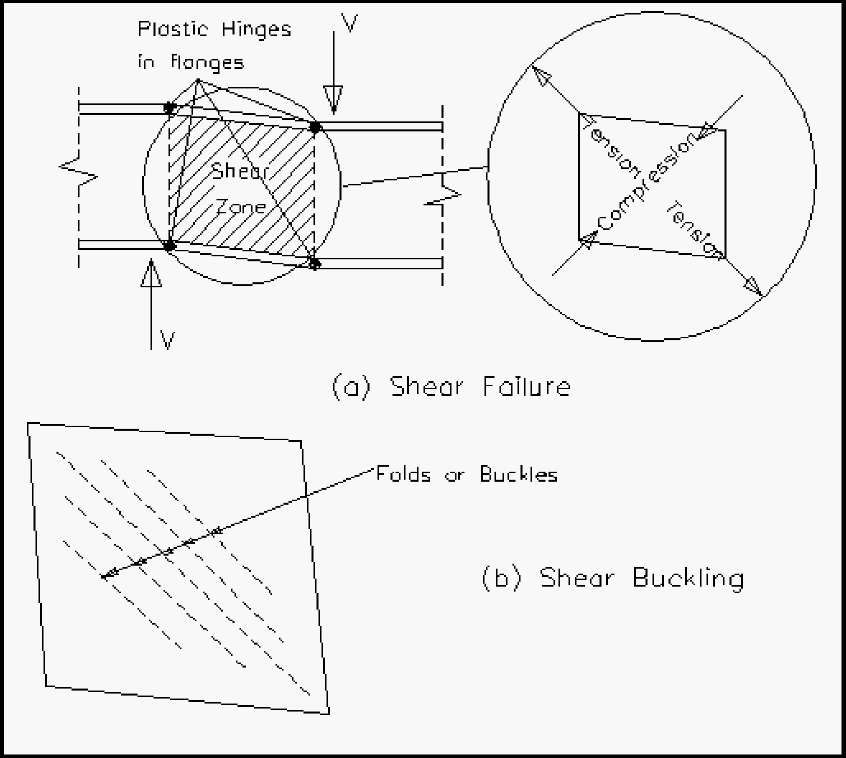


Figure 2-5. Flange Local Bending Limit State
(Beedle, L.S., Christopher, R., 1964)

Shear in Web

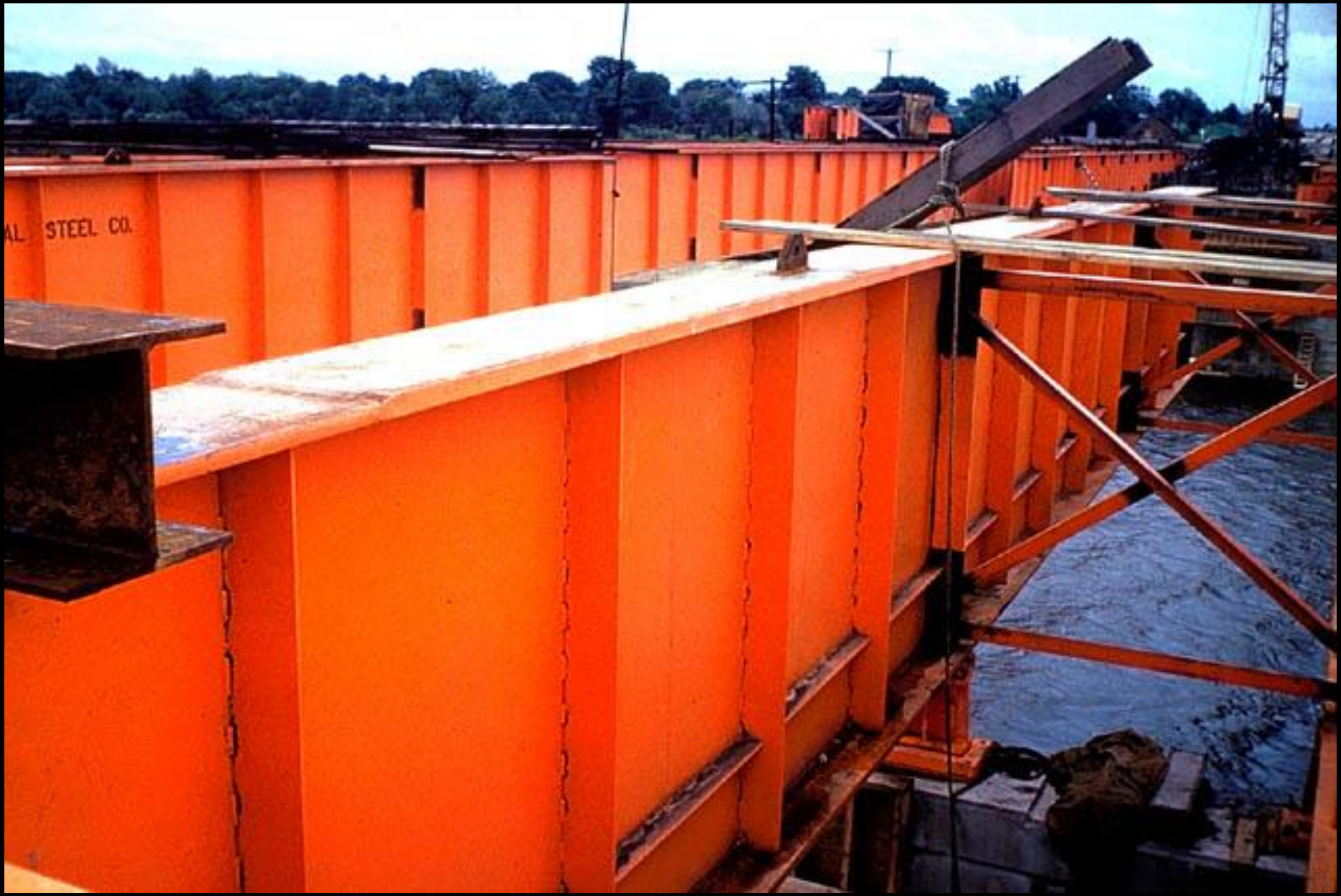
- panels in plate girders or webs with large shear
- buckling in compression direction
- add stiffeners





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*

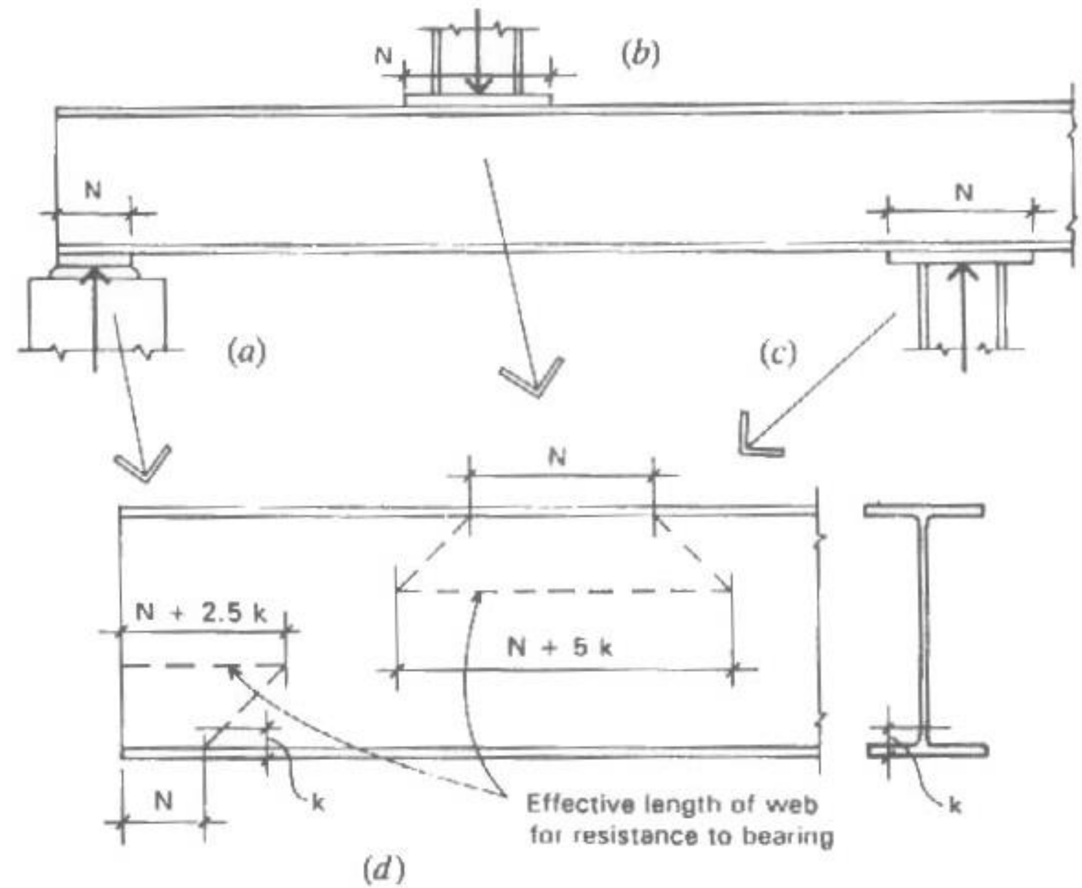


Figure 9.10 Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).

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

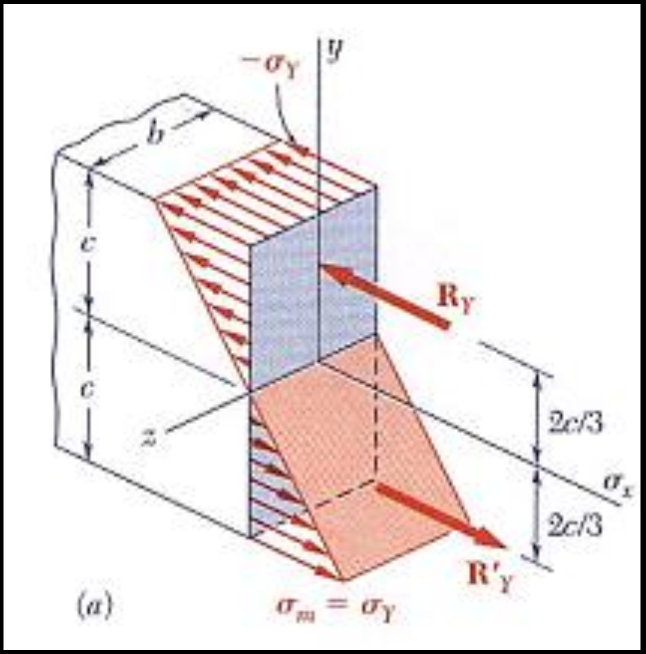
M_n - nominal moment (ultimate capacity)

F_y - yield strength of the steel

Z - plastic section modulus*

Internal Moments - at yield

- *material hasn't failed*

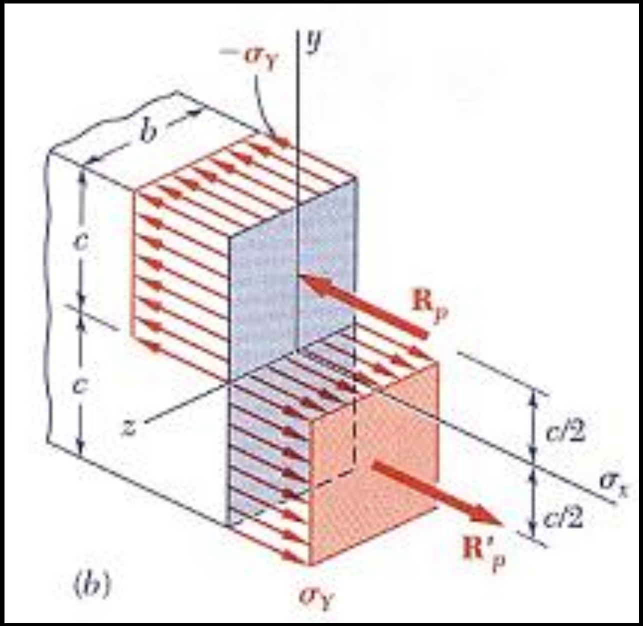


$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y$$

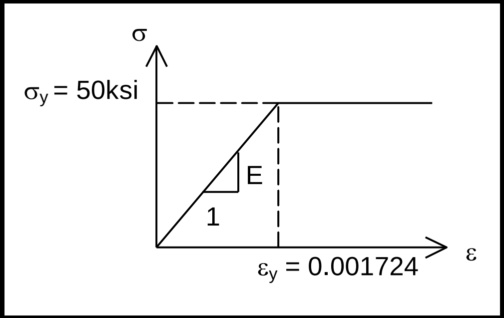
$$= \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y$$

Internal Moments - ALL at yield

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

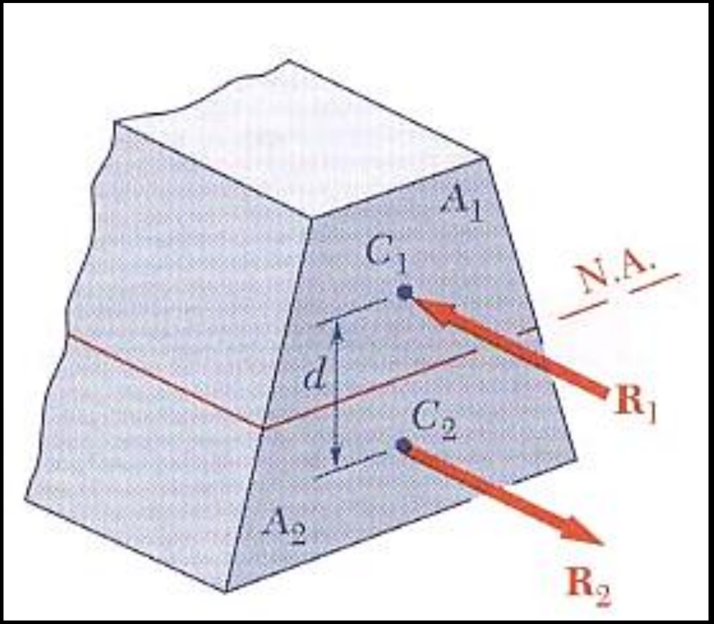


$$M_p = bc^2 f_y = \tfrac{3}{2} M_y$$



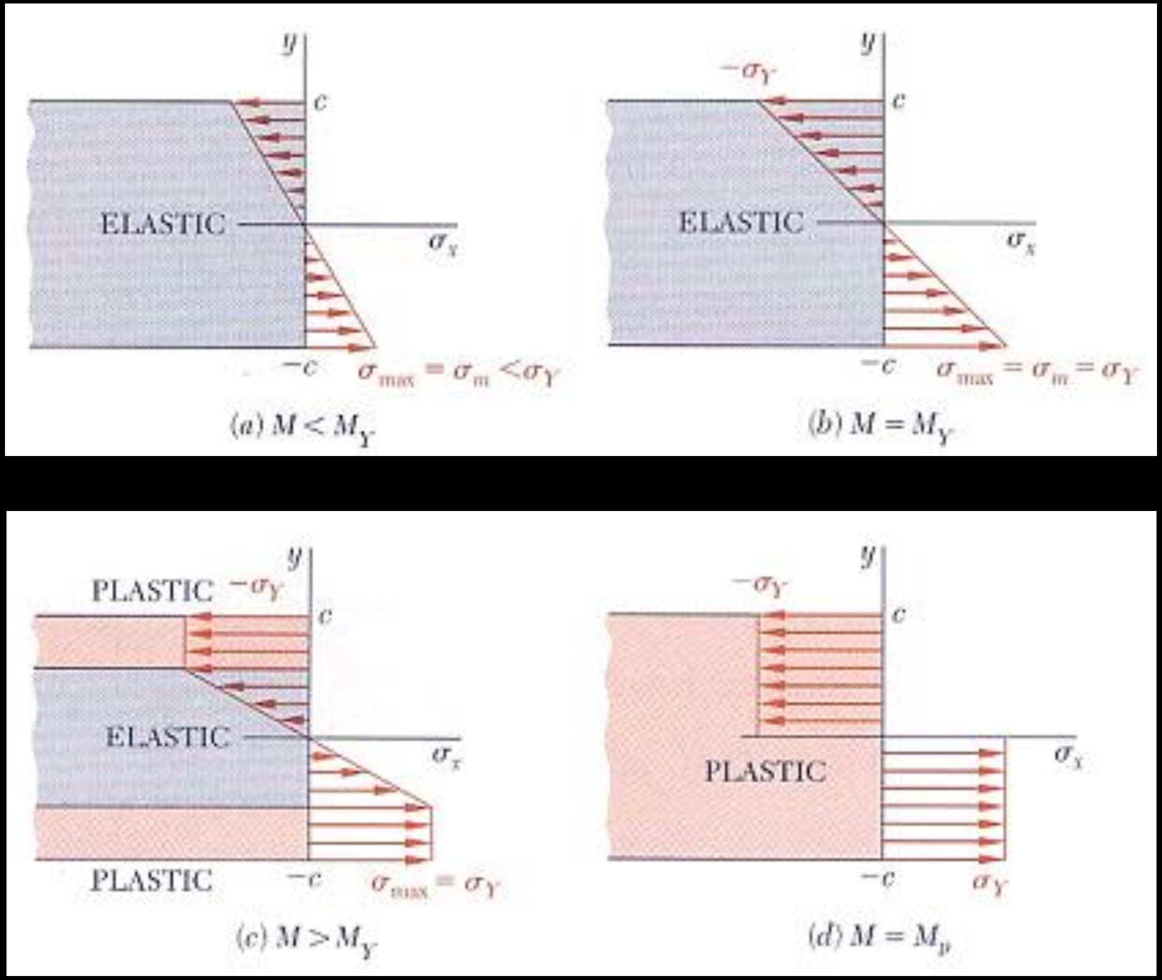
n.a. of Section at Plastic Hinge

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



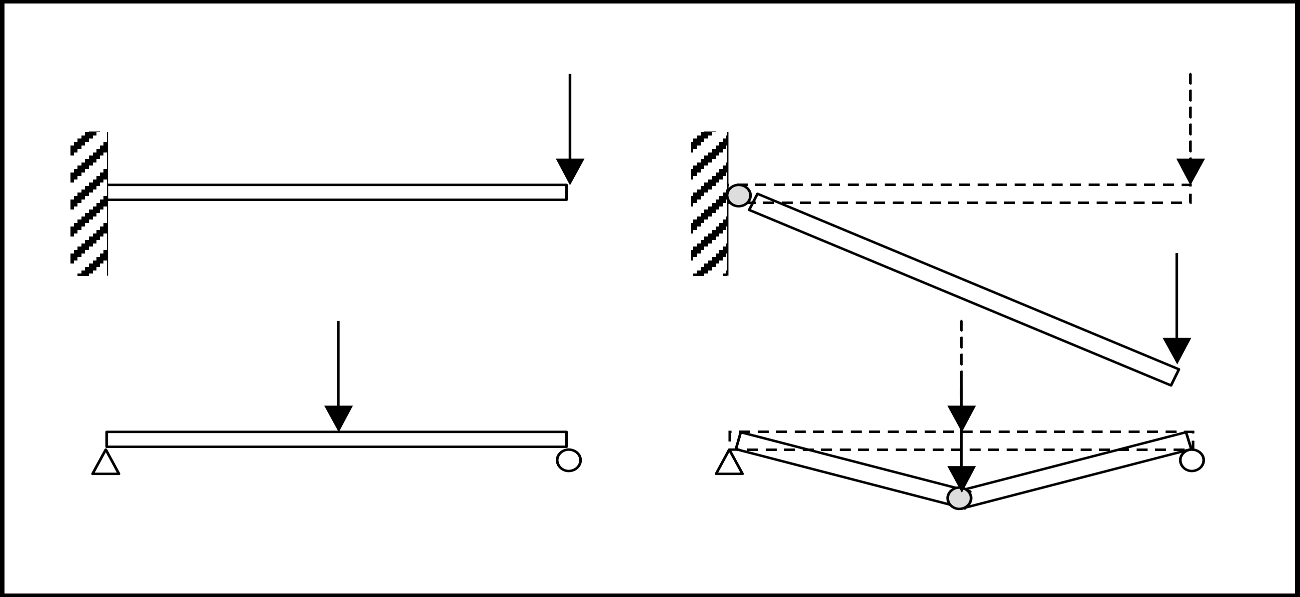
$$M_p = f_y A_1 d = f_y \sum_{n.a} A_i d_i$$

Plastic Hinge Development



Plastic Hinge Examples

- stability can be effected



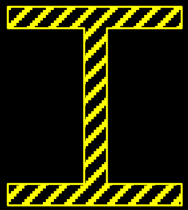
Plastic Section Modulus

- shape factor, k

$$k = {M_p}\big/{M_y}$$

= 3/2 for a rectangle

$\approx$ 1.1 for an I



$$k = {Z}\big/{S}$$

- 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 = 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

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

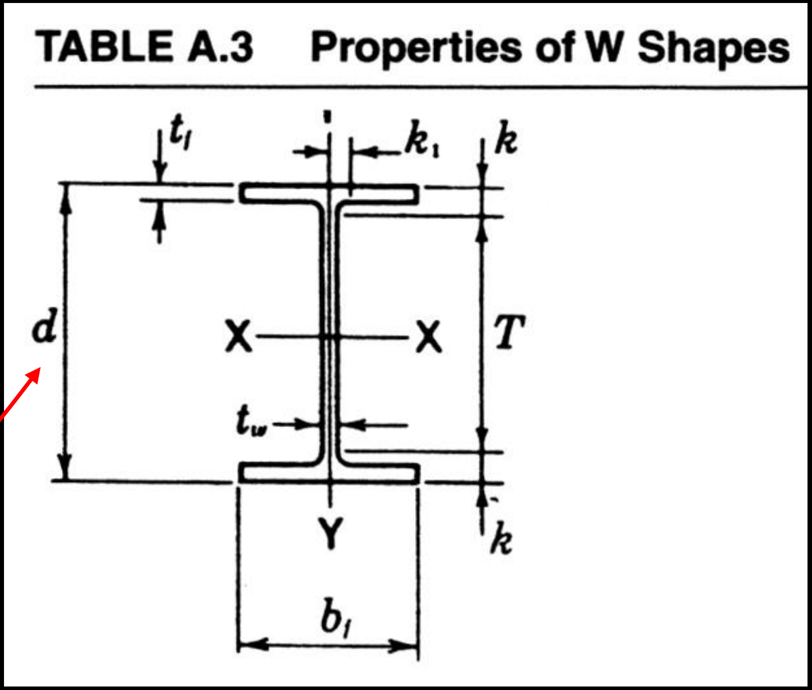
$$\Sigma \gamma_i R_i = M_u \leq \phi_b M_n$$

Compact Sections

- plastic moment can form before any buckling
- criteria

$$- \quad \frac{b_f}{2t_f} \le 0.38\sqrt{\frac{E}{F_y}}$$

$$- \text{ and } \frac{h_c}{t_w} \le 3.76\sqrt{\frac{E}{F_y}}$$



Lateral Torsional Buckling

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

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

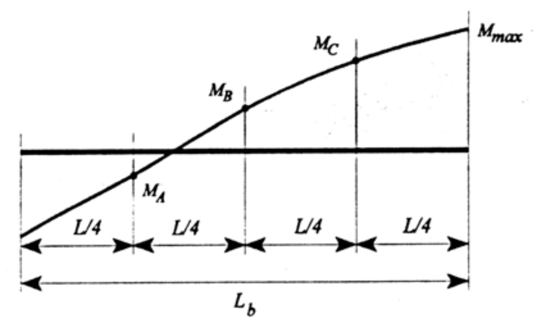
C_b = modification factor

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

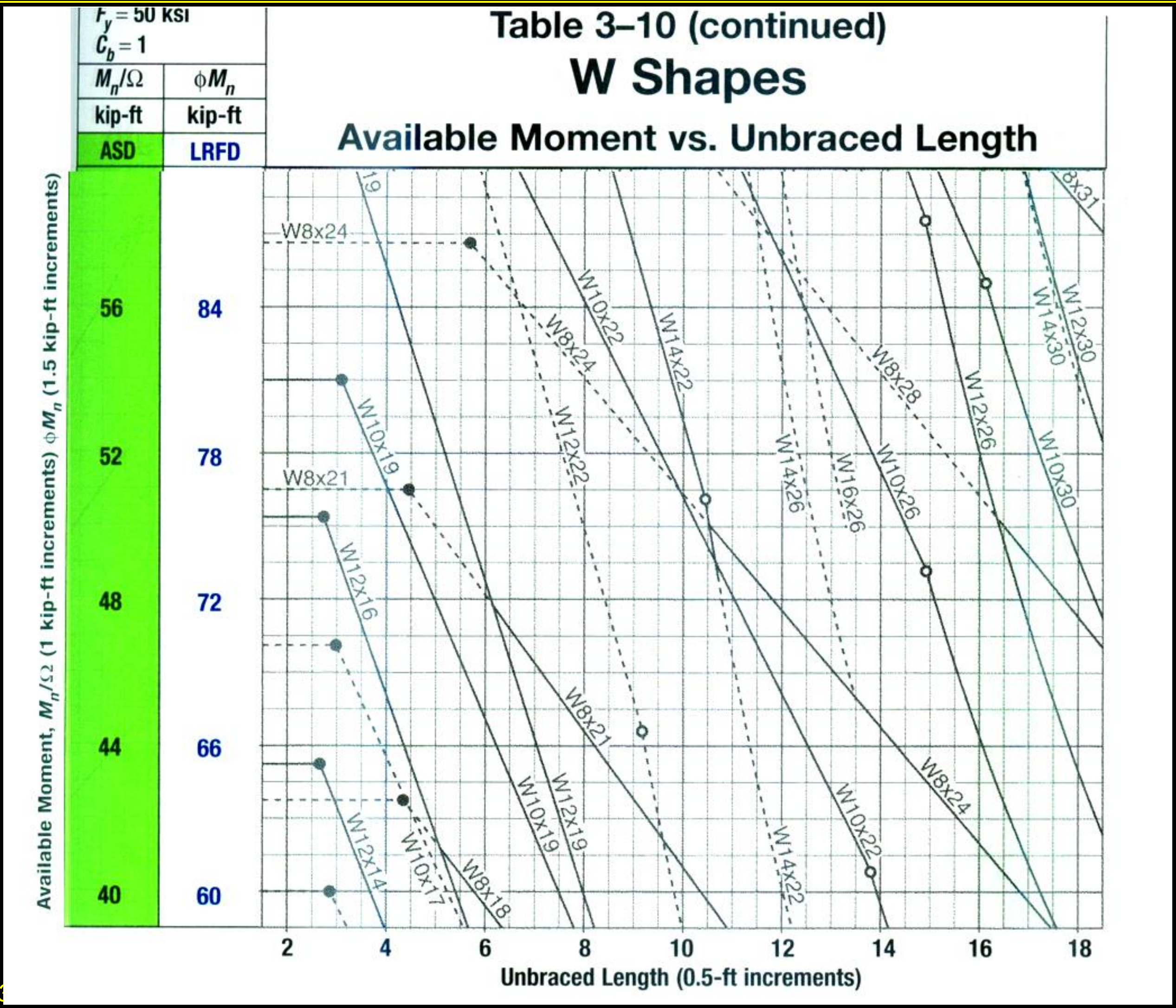
M_A - |moment|, 1/4 point

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*
  - *<u>no factors</u> 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. <u>Choose (economical) section from section or beam capacity charts</u>

Beam Charts by Z_x (pg. 250)

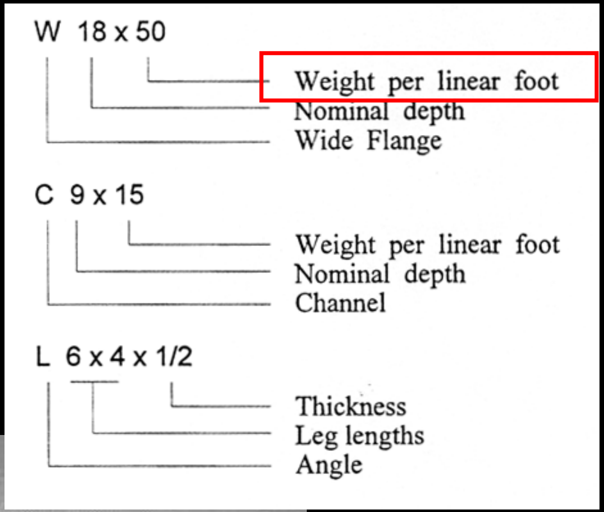
TABLE 9.1 Load Factor Resistance Design Selection for Shapes Used as Beams

		$F_y = 36$ 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*

- $\left(V_a \le V_n / \Omega\right)$ *or* $\left(V_u \le \phi_v V_n\right)$
- *W and rectangles* $f_{v-\max} = \frac{3V}{2A} \approx \frac{V}{A_{web}}$

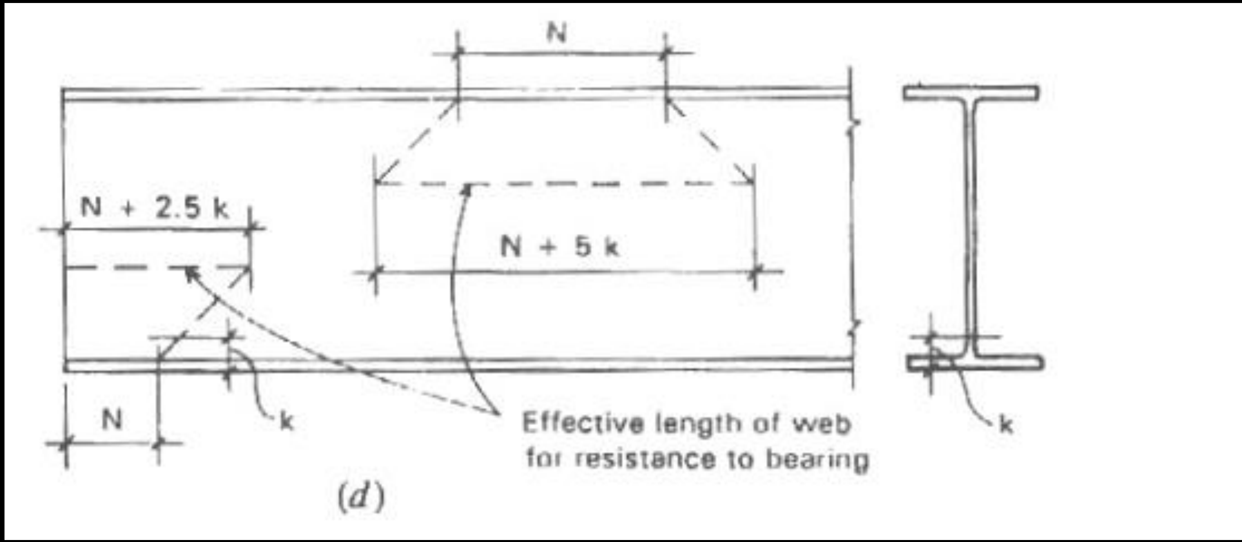
$$V_n = 0.6\, F_{yw} A_w$$

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

Beam Design (revisited)

7. Provide adequate bearing area at supports

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



Beam Design (revisited)

8. Evaluate torsion

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

- circular cross section

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

- rectangular

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

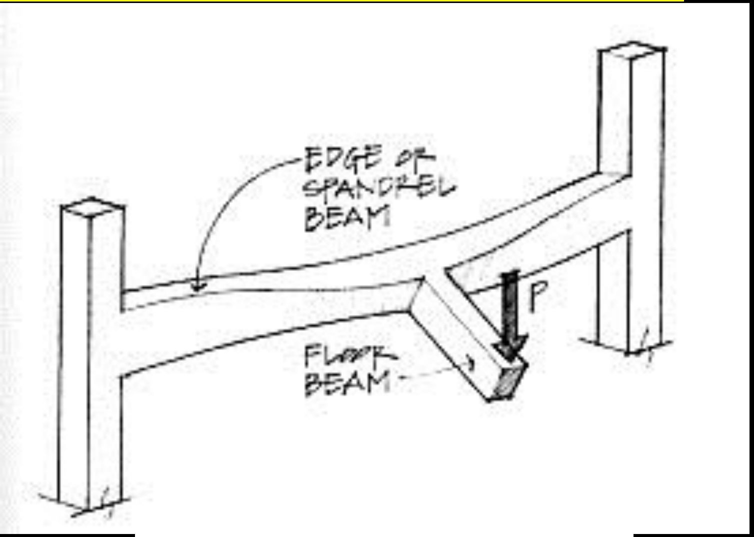
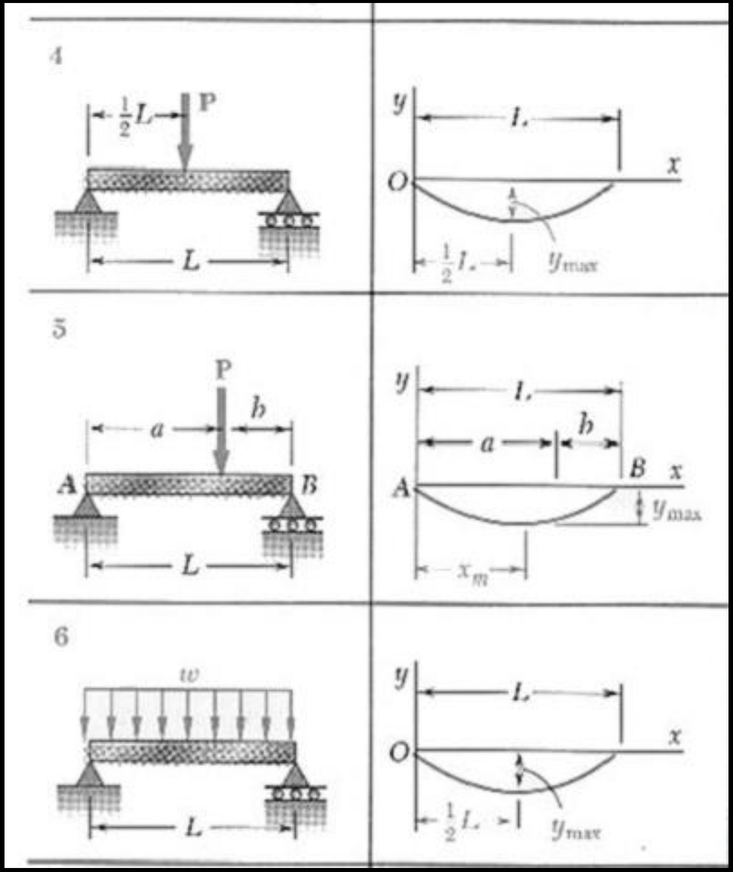


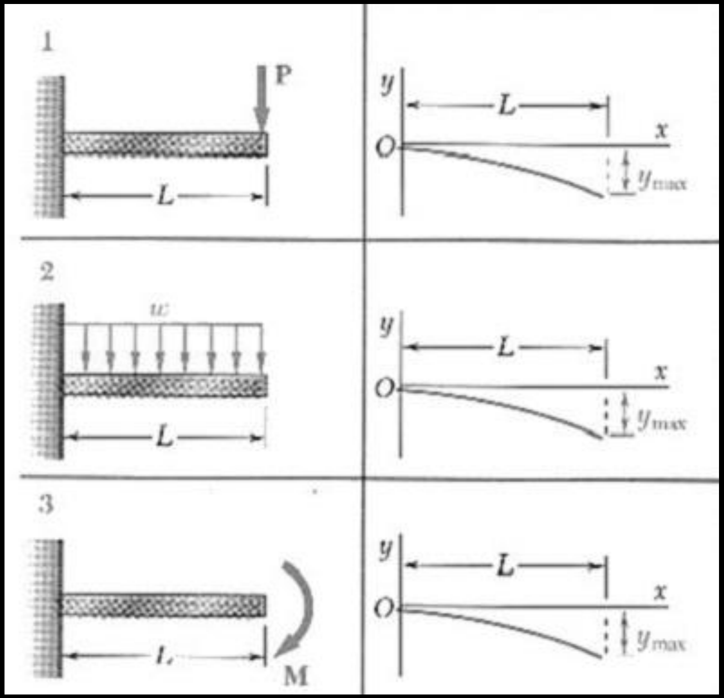
TABLE 3.1. Coefficients for Rectangular Bars in Torsion

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





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

Load Tables & Equivalent Load

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

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

LRFD

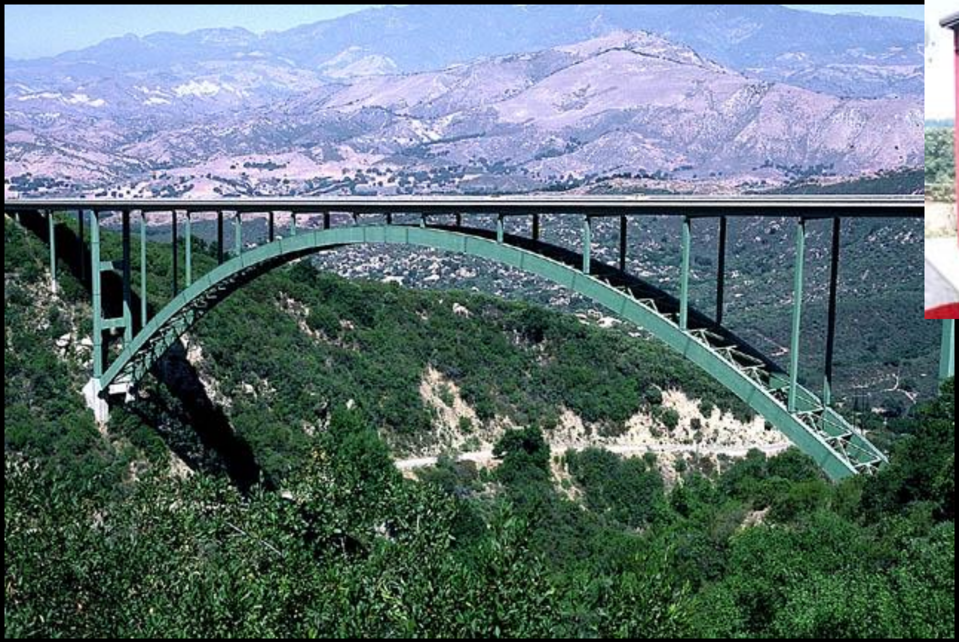
STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	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