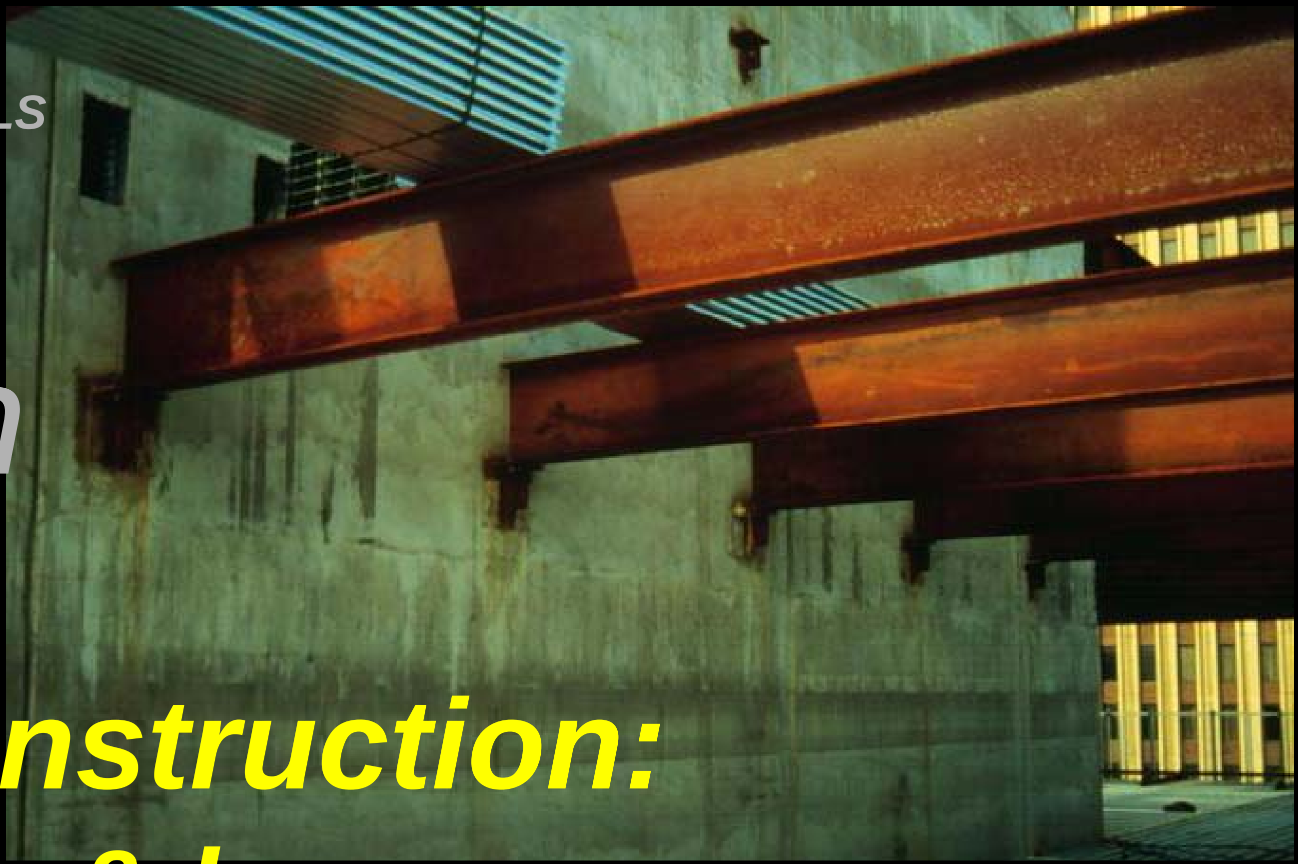
ELEMENTS OF ARCHITECTURAL STRUCTURES: FORM, BEHAVIOR, AND DESIGN

ARCH 614
DR. ANNE NICHOLS
SPRING 2007

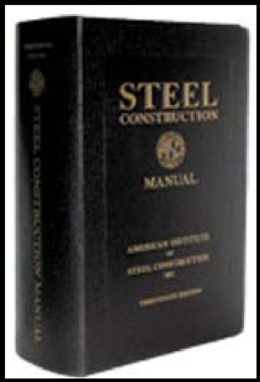
lecture
sixteen



steel construction: materials & beams

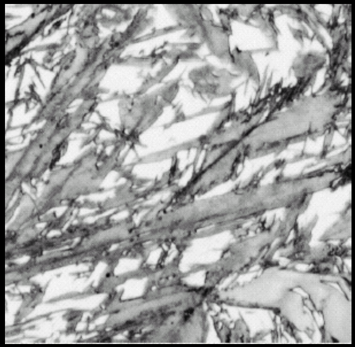
Steel Beam Design

- American Institute of Steel Construction
  - Manual of Steel Construction
  - ASD & LRFD
  - now combined in 13th ed. (2005)



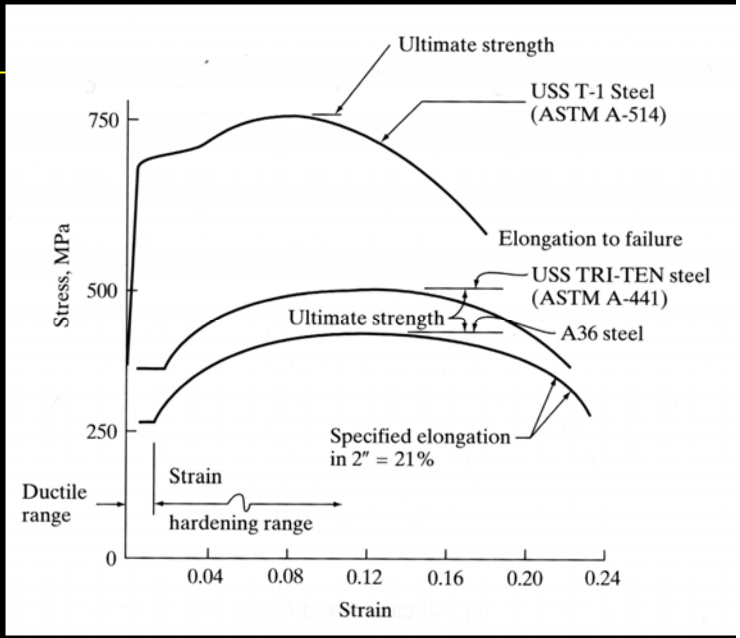
Steel Materials

- 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*



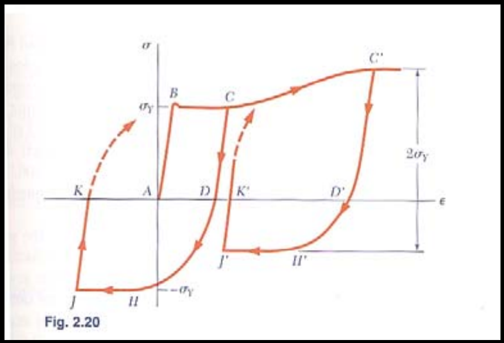
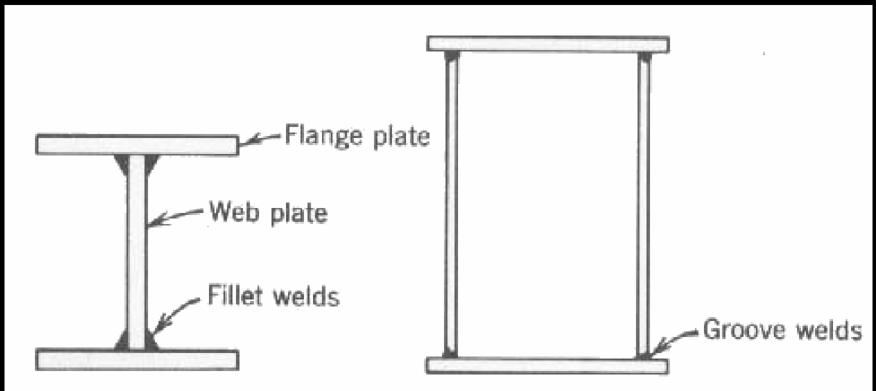


Fig. 2.20

Winnepeg DOT

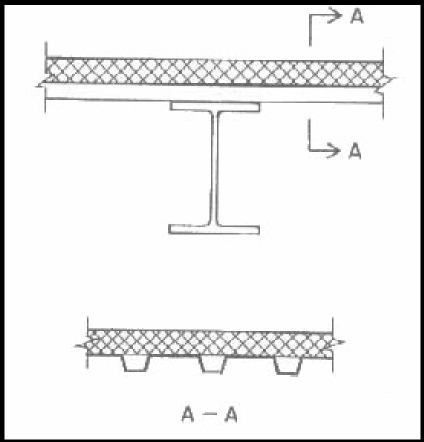
Structural Steel

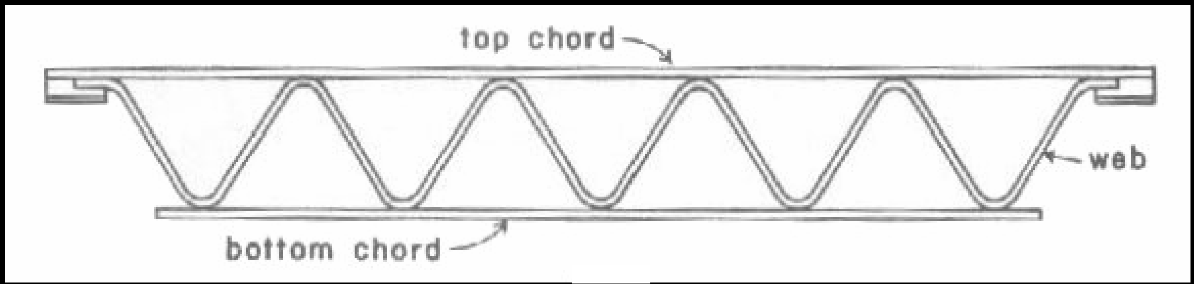
- standard rolled shapes (W, C, L, T)
- open web joists
- plate girders
- decking



(d) Welded girder.

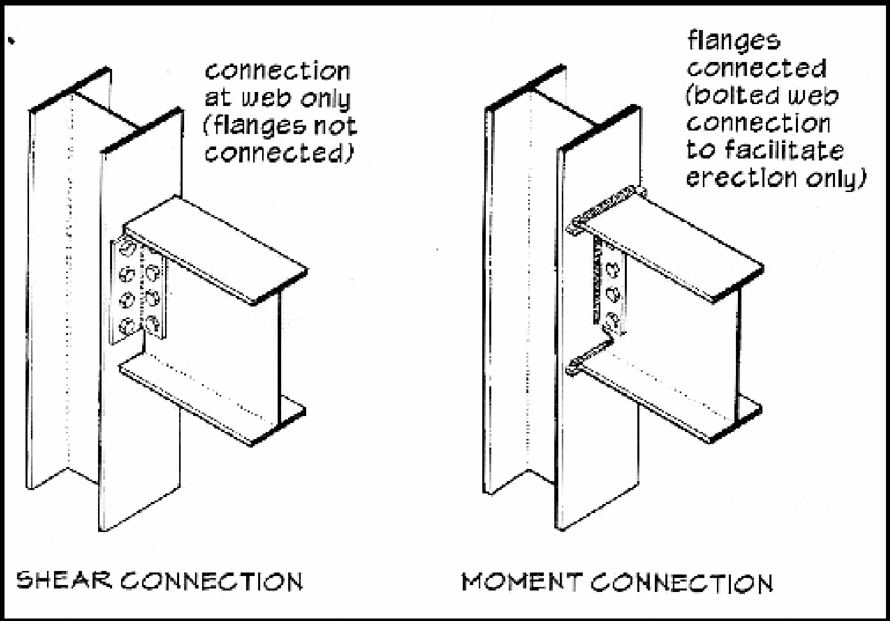
(e) Welded box girder.

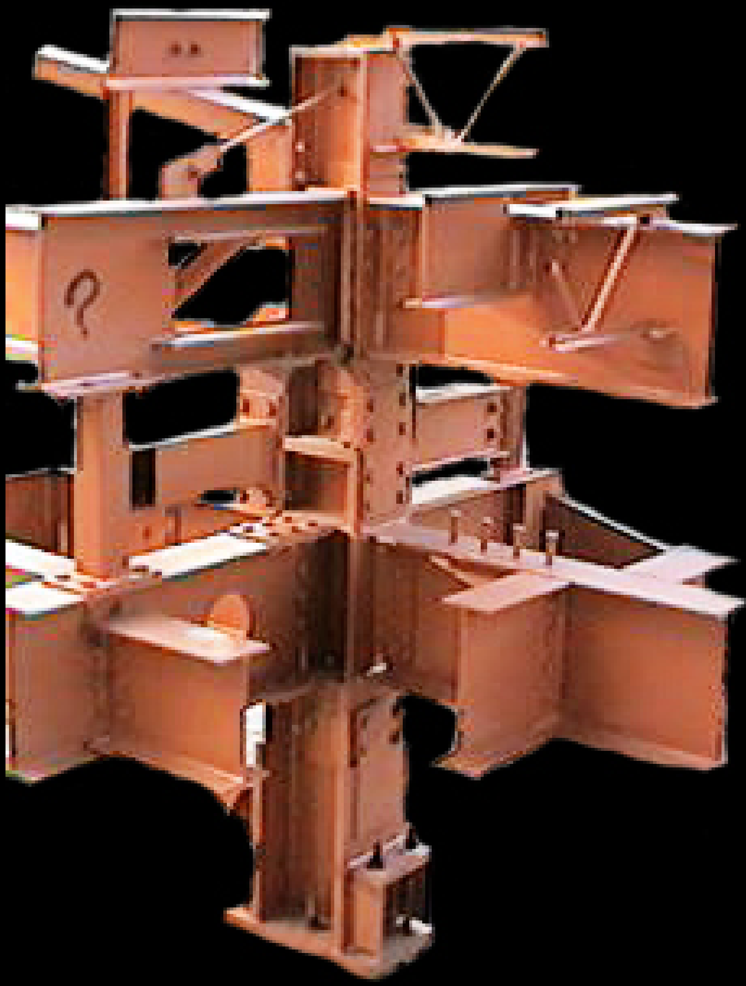




Steel Construction

- welding
- bolts





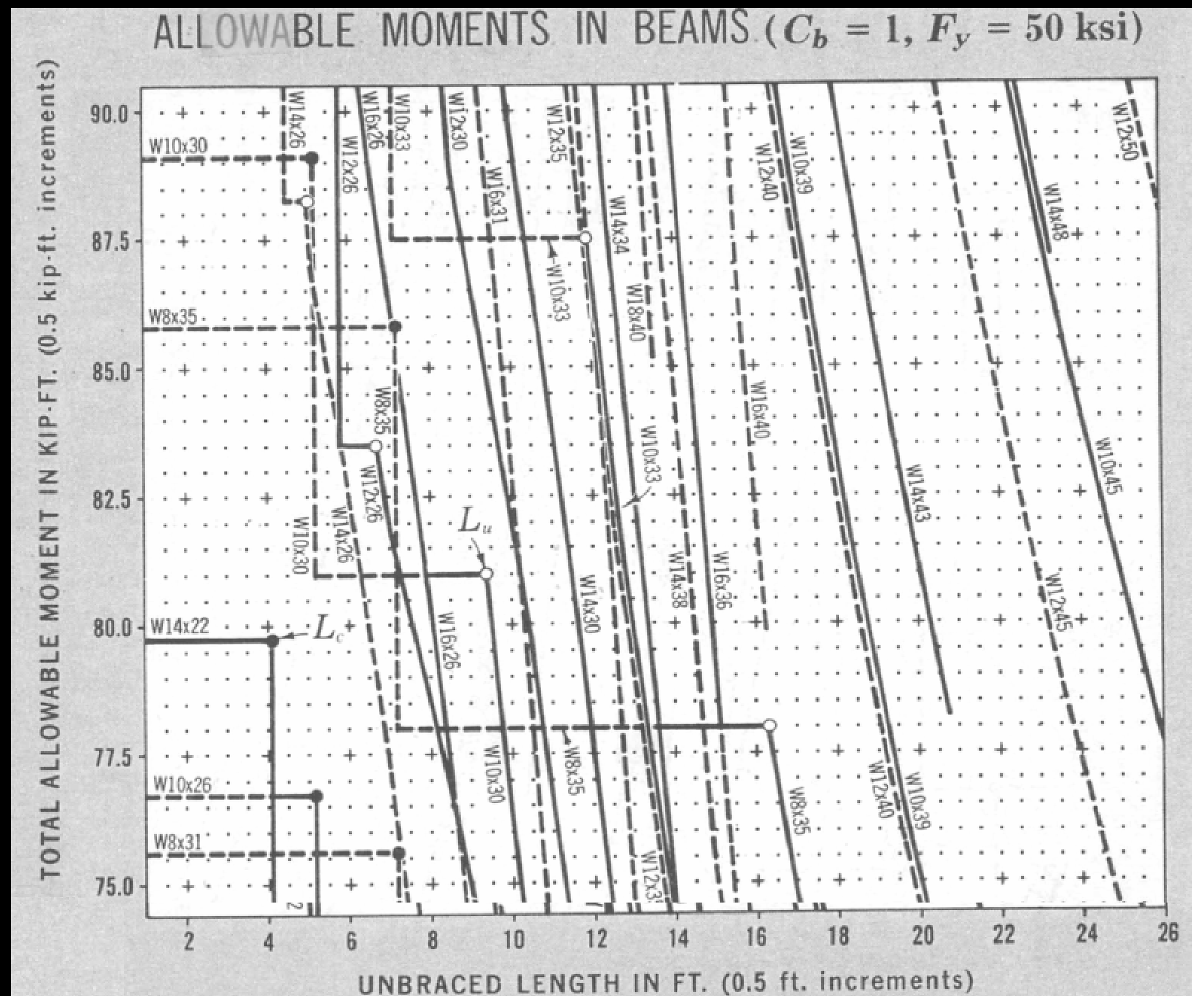
ASD Steel Design

- bending (braced) — $F_b = 0.66F_y$
- bending (unbraced*) — $F_b = 0.60F_y$
- shear — $F_v = 0.40F_y$
- shear (bolts) — tabulated
- shear (welds) — $F_v = 0.30F_{weld}$

* flanges in compression can buckle

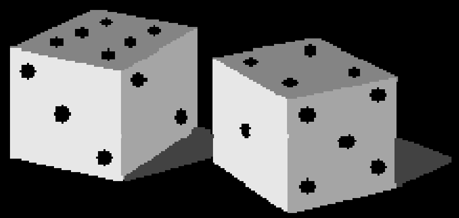
ASD Steel Design

- *braced vs. unbraced*



LRFD

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



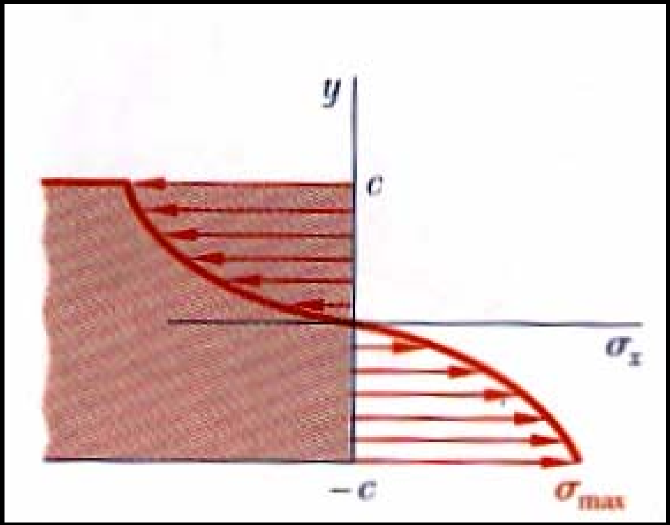
$$R_u = \gamma_D R_D + \gamma_L R_L \leq \phi R_n$$

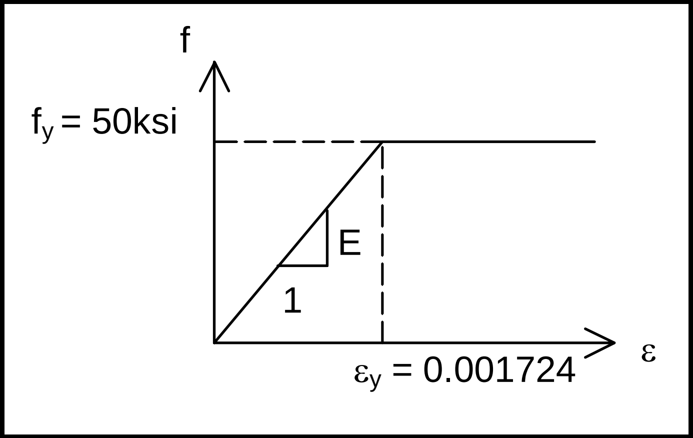
ϕ - resistance factor

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*





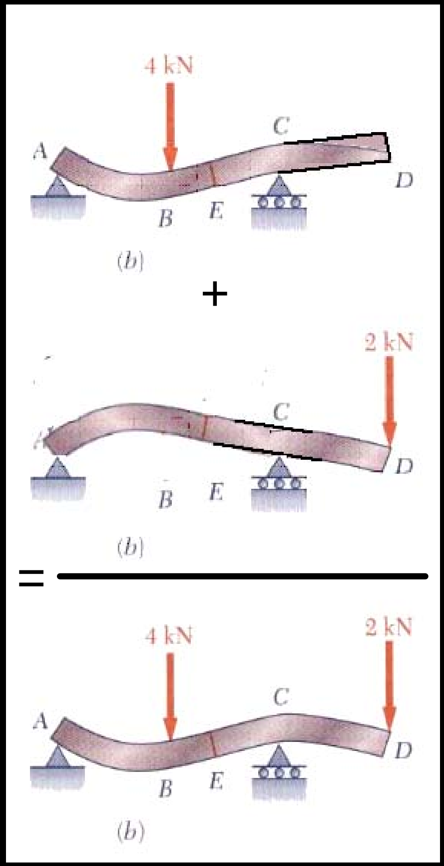
LRFD Load Combinations

ASCE-7 (2002)

- $\boxed{1.4(D + F)}$
- $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
- $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.0E + L + 0.2S)$
- $0.9D + 1.6W + 1.6H$
- $0.9D + 1.0E + 1.6H$

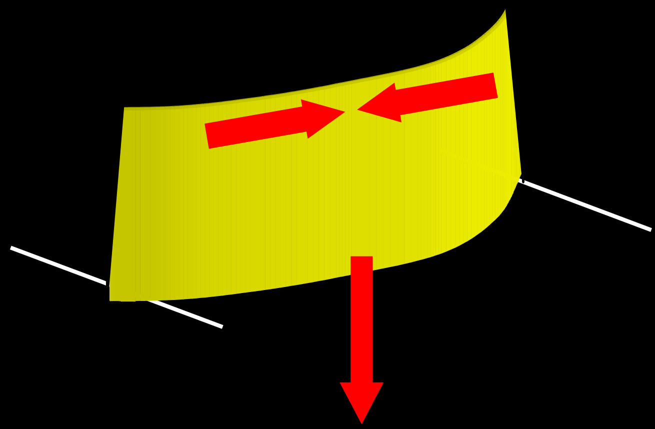
Beam Design Criteria (revisited)

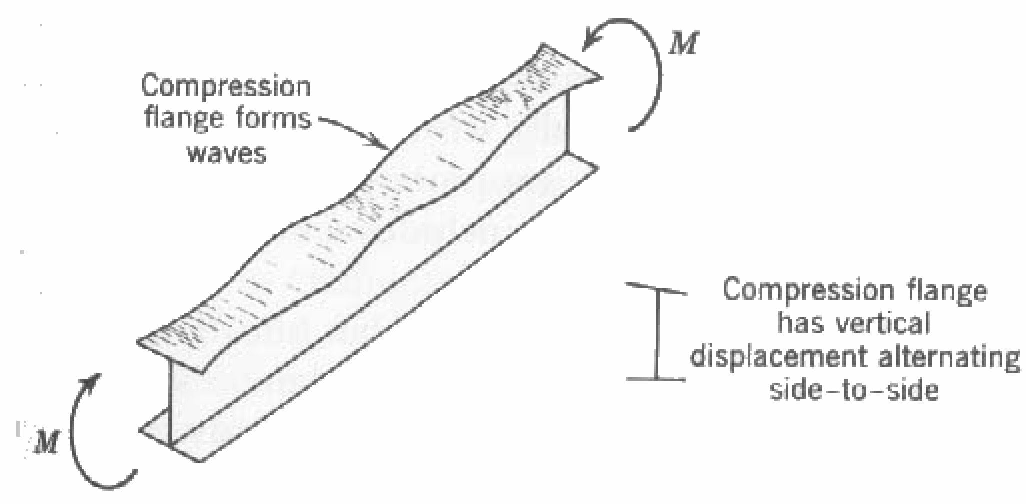
- strength design
  - bending stresses predominate
  - shear stresses occur
- serviceability
  - limit deflection
  - stability
- superpositioning
  - use of beam charts
  - elastic range only!
  - "add" moment diagrams
  - "add" deflection CURVES (not maximums)



Steel Beams

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

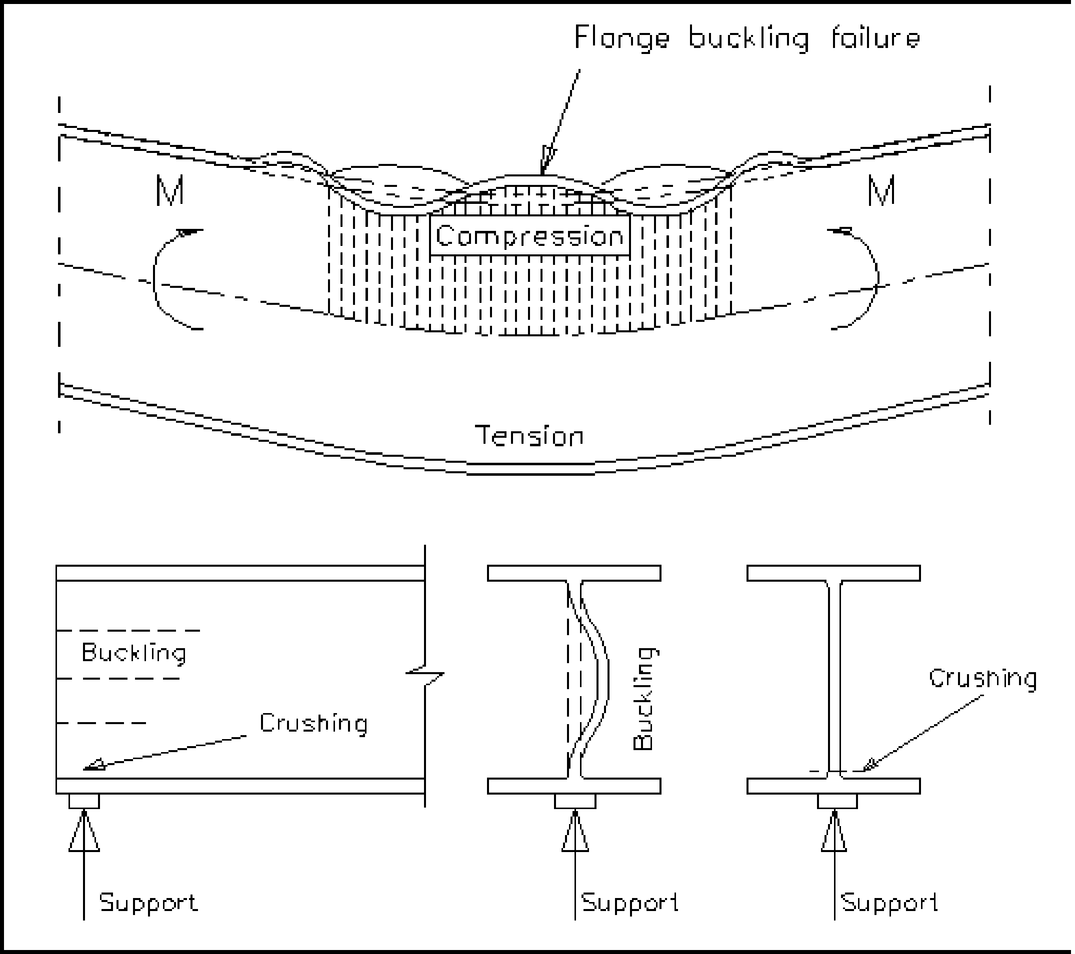




Local buckling of compression flange.

Local Buckling

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



Local Buckling

- *flange*

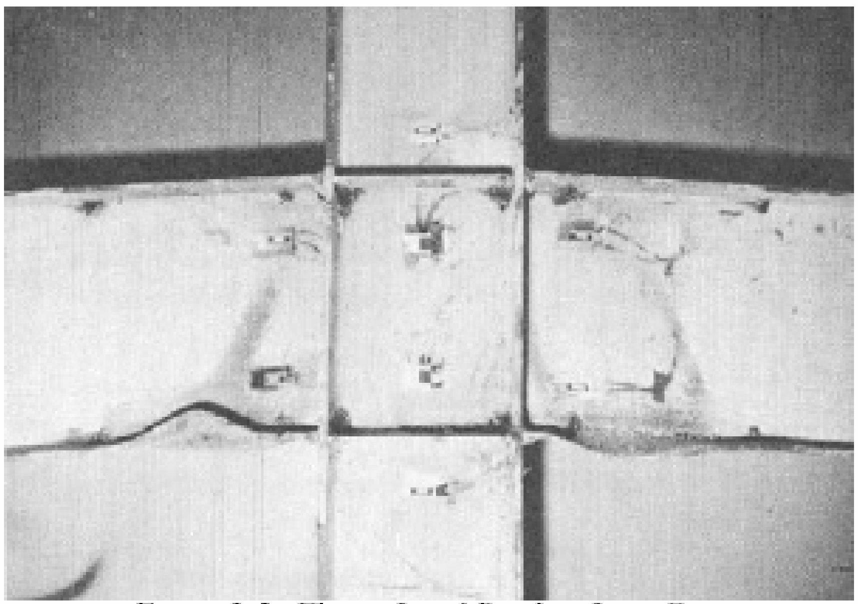


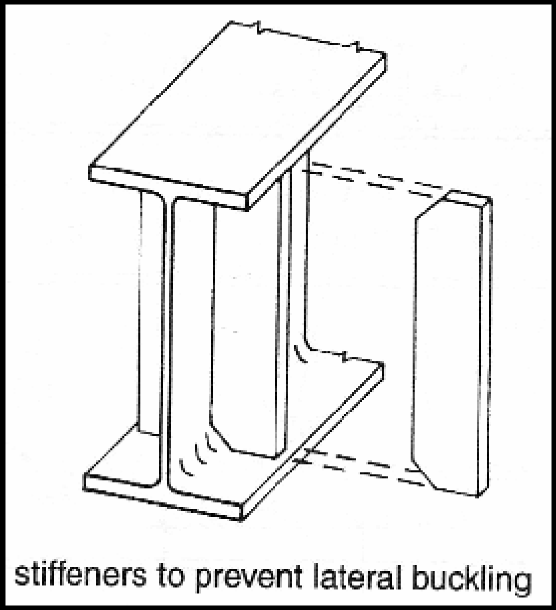
Figure 2-5. Flange Local Bending Limit State
(Beedle, L.S., Christopher, R., 1964)

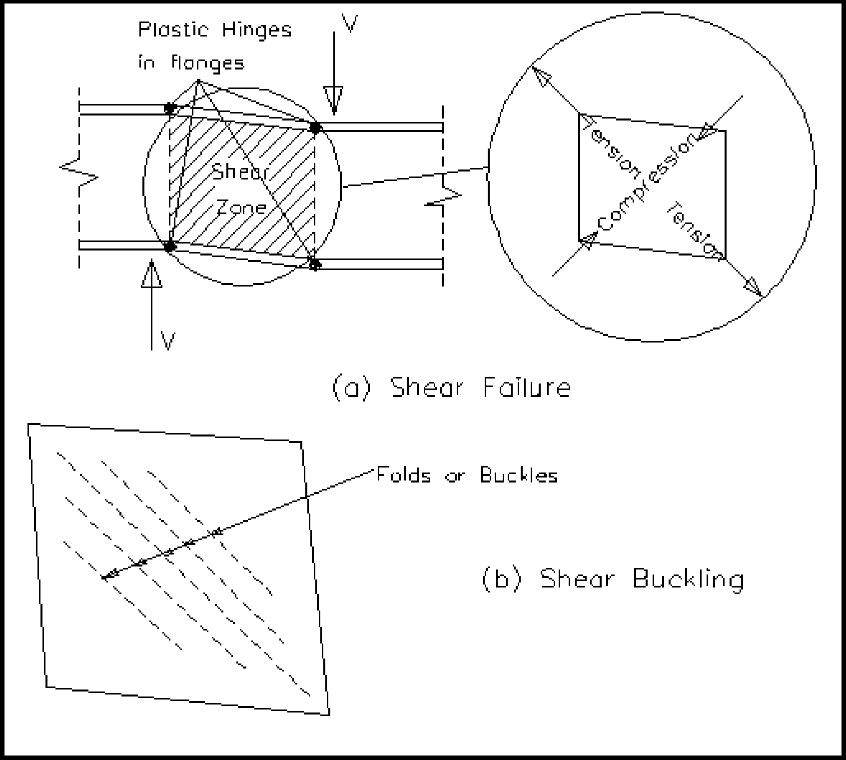
- *web*

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





Steel Beams

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

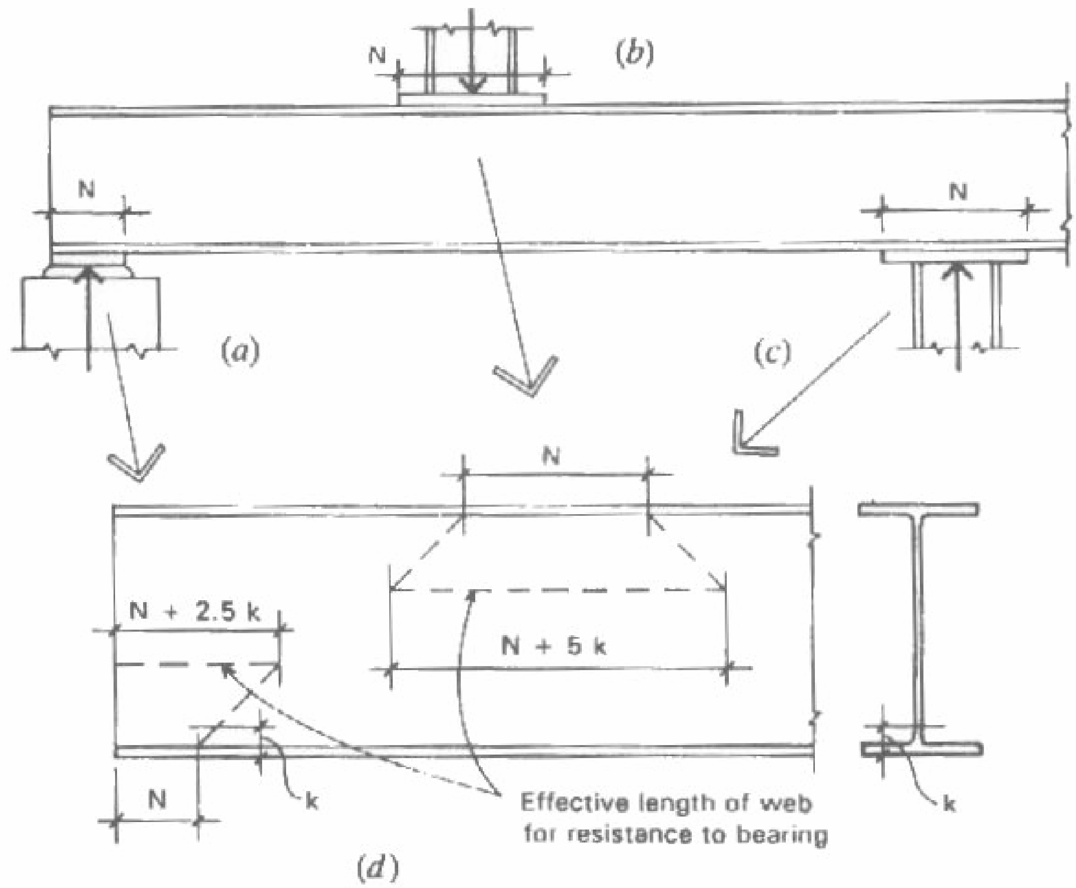


Figure 9.10 Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).

LRFD - Flexure

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

M_u - maximum moment

ϕ_b - resistance factor for bending = 0.9

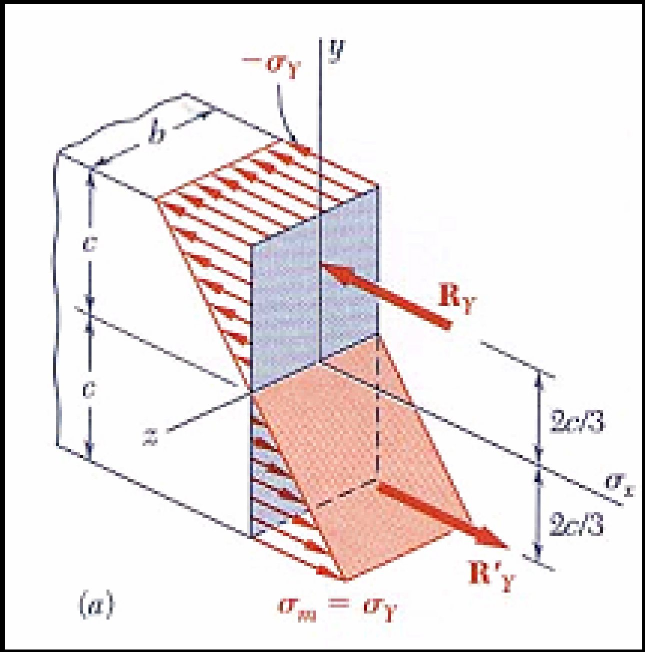
M_n - nominal moment (ultimate capacity)

F_y - yield strength of the steel

*Z - plastic section modulus**

Internal Moments - at yield

- *material hasn't failed*

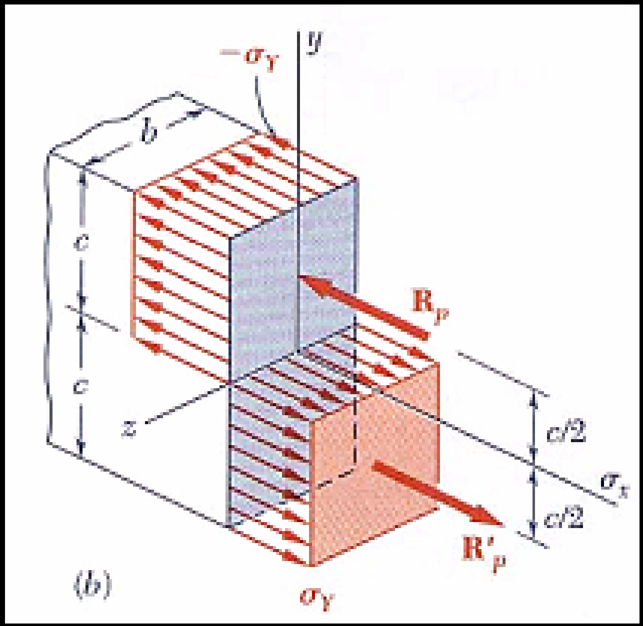


$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y$$

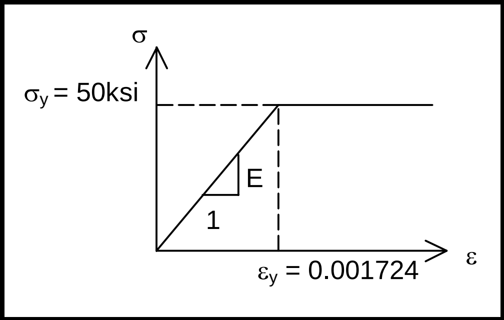
$$= \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y$$

Internal Moments - ALL at yield

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

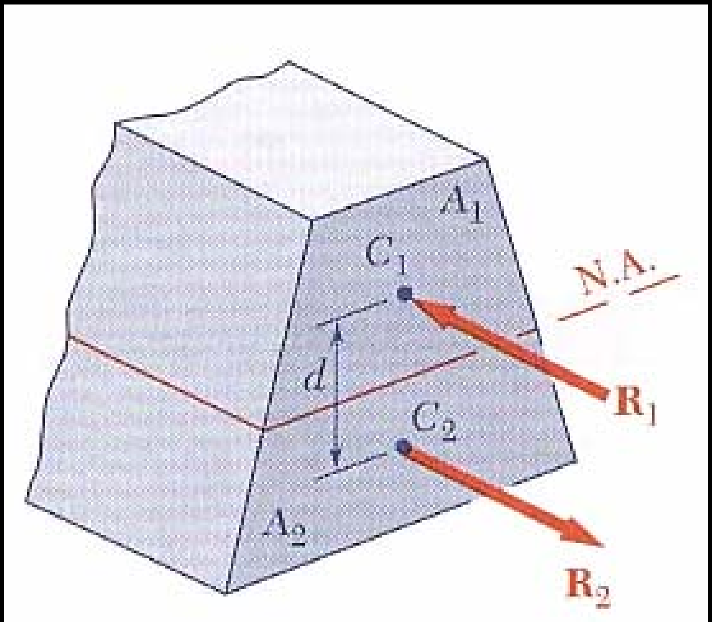


$$M_p = bc^2 f_y = \frac{3}{2} M_y$$



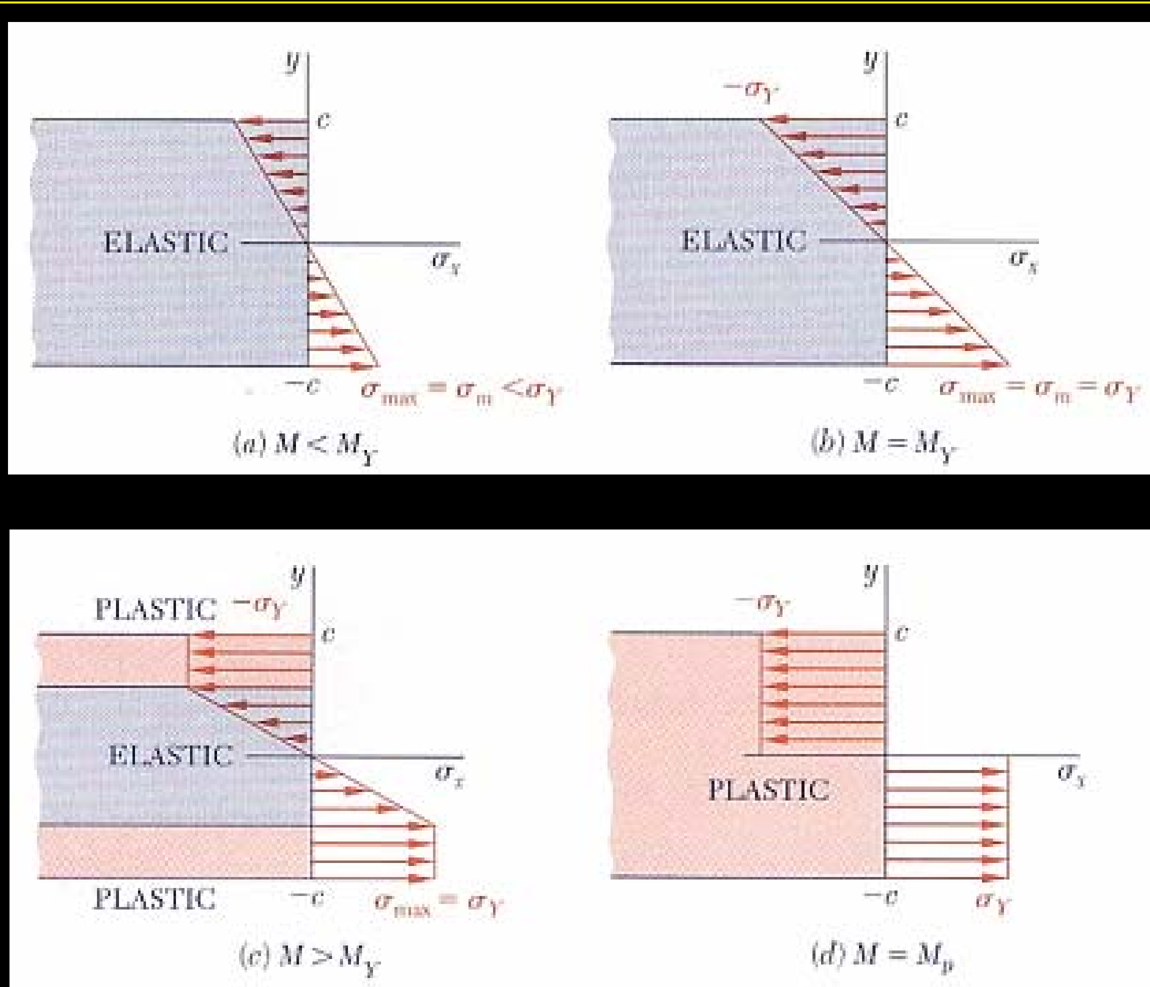
n.a. of Section at Plastic Hinge

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



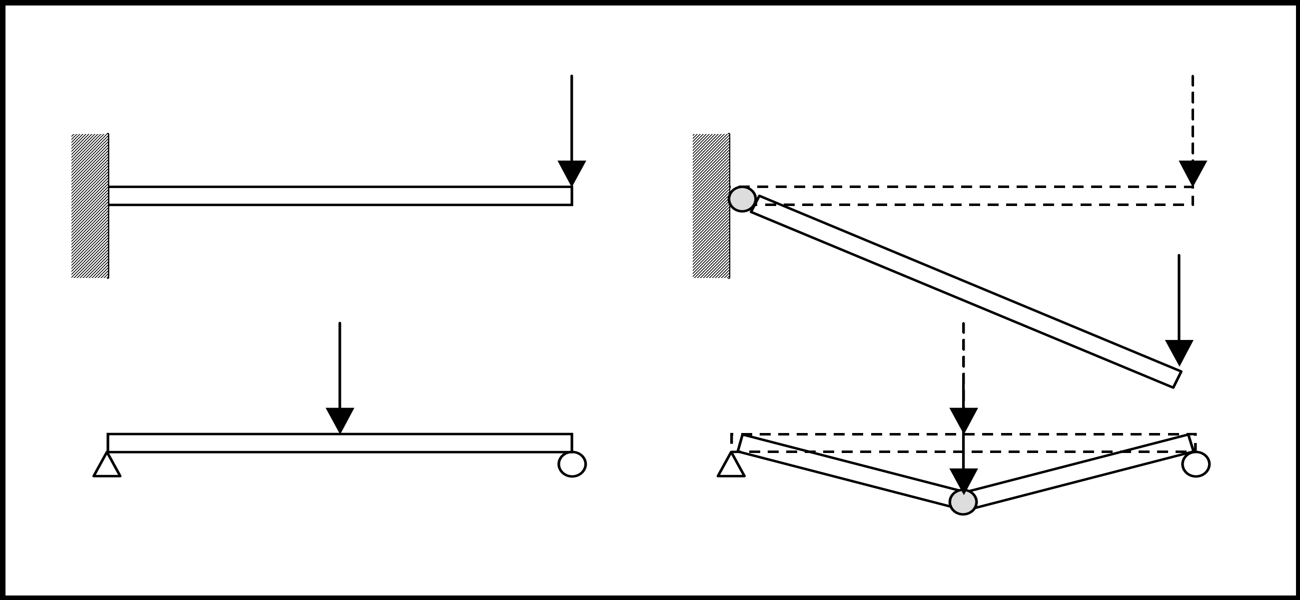
$$M_p = f_y A_1 d = f_y \sum_{n.a} A_i d_i$$

Plastic Hinge Development



Plastic Hinge Examples

- stability can be effected



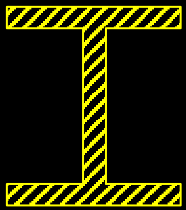
Plastic Section Modulus

- *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

$$\Sigma\gamma_i R_i = V_u \le \phi_v V_n = 0.9(0.6F_{yw}A_w)$$

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

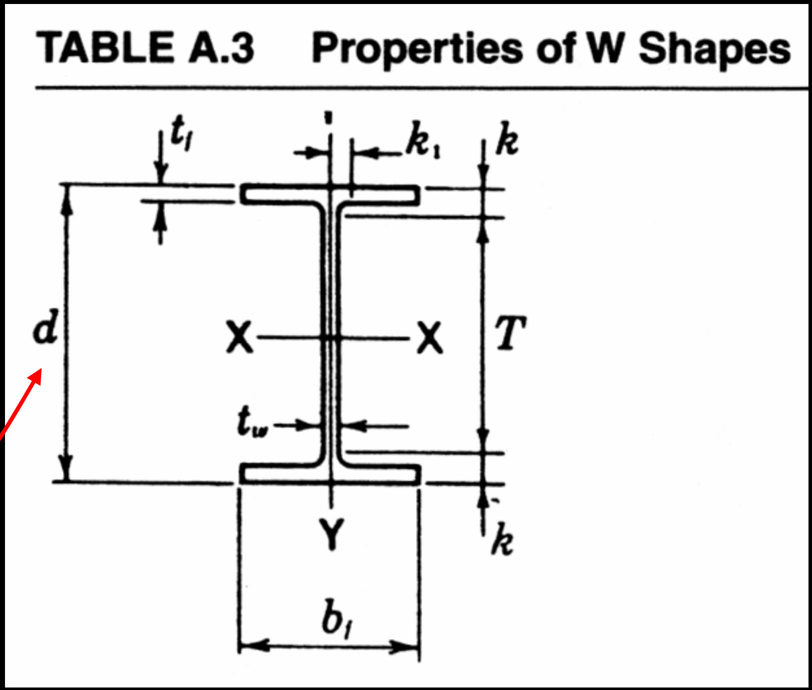
- *limit states for beam failure*
  1. *yielding*
  2. *lateral-torsional buckling**
  3. *flange local buckling*
  4. *web local buckling*
- *minimum M_n governs*

$$L_p = \frac{300 \times r_y}{\sqrt{F_y}}$$

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

Compact Sections

- *plastic moment can form before any buckling*
- *criteria*
  - $$\frac{b_f}{2t_f} \le \frac{65}{\sqrt{F_y}}$$
  - *and* $$\frac{h_c}{t_w} \le \frac{640}{\sqrt{F_y}}$$



Lateral Torsional Buckling

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

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

C_b = modification factor

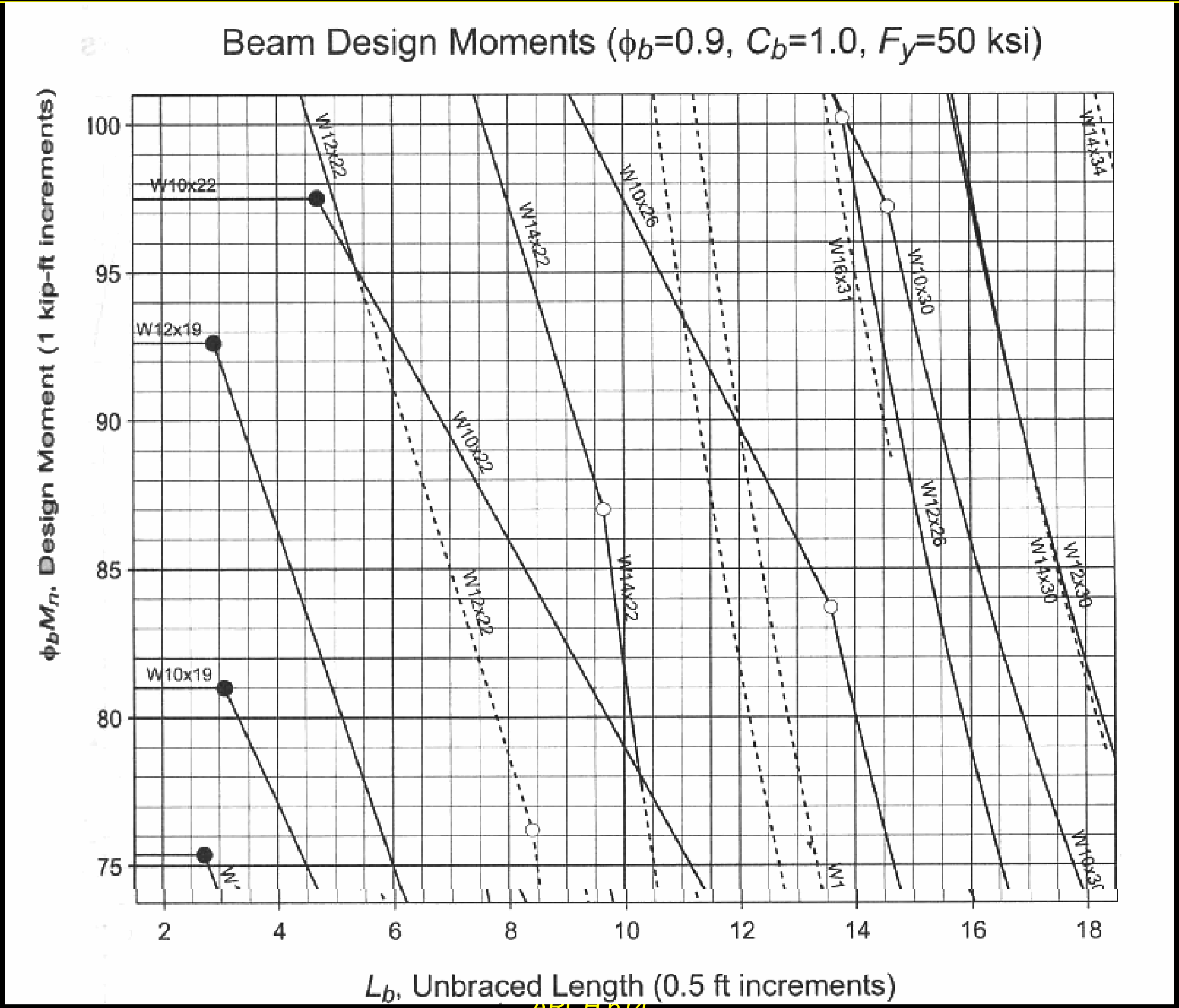
M_{max} - |max moment|, unbraced segment

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
  - <u>no factors</u> are applied to the loads
  - often governs the design

Design Procedure (revisited)

1. *Know* F_{all} *for the material or* F_U *for LRFD*

2. *Draw V & M, finding* M_{max}

3. *Calculate* $S_{req'd}$ $\left(f_b \leq F_b\right)$ *or Z*

4. *Choose (economical) section from section or beam capacity charts*

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)

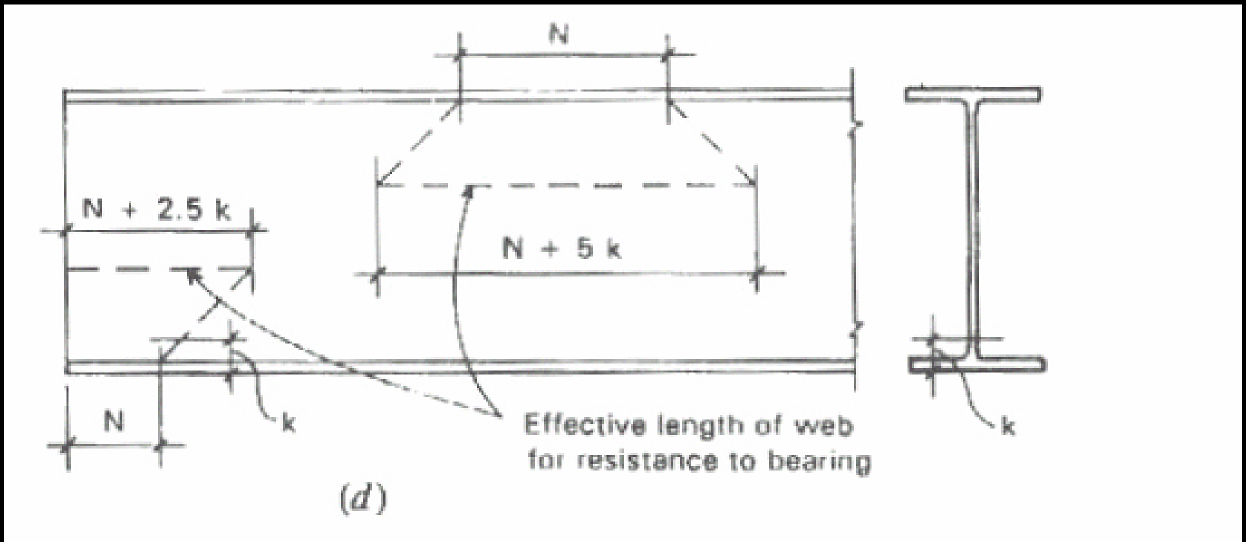
6. Evaluate shear stresses - horizontal

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

Beam Design (revisited)

7. Provide adequate bearing area at supports

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



Beam Design (revisited)

8. Evaluate torsion

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

- circular cross section

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

- rectangular

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

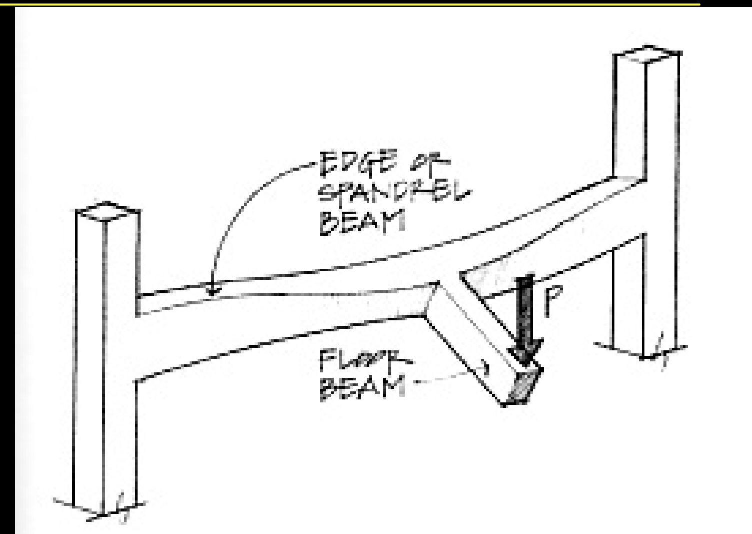
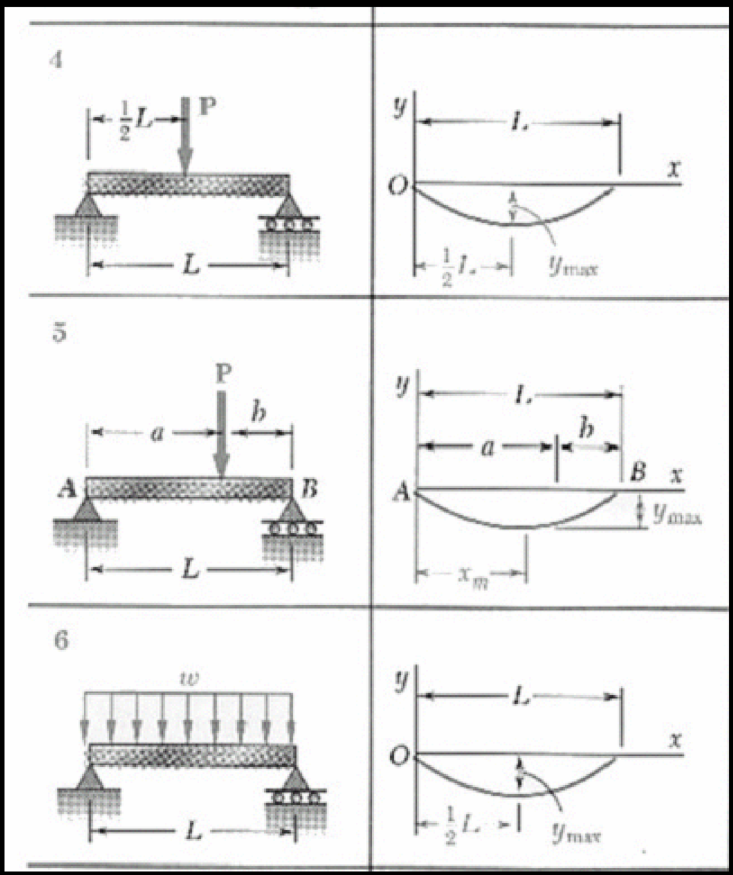


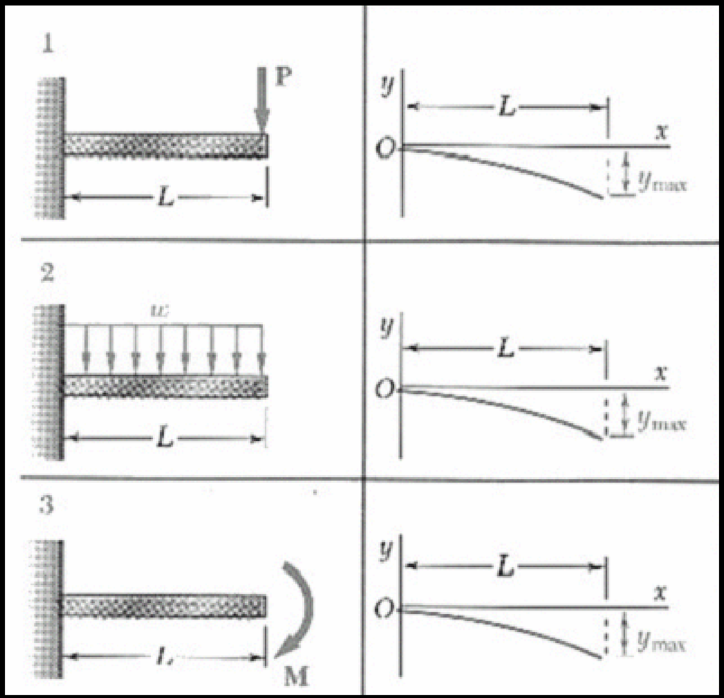
TABLE 3.1. Coefficients for Rectangular Bars in Torsion

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





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

Load Tables & Equivalent Load

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

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

STANDARD LOAD TABLE/OPEN WEB STEEL JOISTS, K-SERIES
Based on a Maximum Allowable Tensile Stress of 30 ksi

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	550 / 550															
9	550 / 550															
10	550 / 480	550 / 550														
11	532 / 377	550 / 542														
12	444 / 288	550 / 455	550 / 550	550 / 550	550 / 550											
13	377 / 225	479 / 363	550 / 510	550 / 510	550 / 510											
14	324 / 179	412 / 289	500 / 425	550 / 463	550 / 463	550 / 550	550 / 550	550 / 550	550 / 550							
15	281 / 145	358 / 234	434 / 344	543 / 428	550 / 434	511 / 475	550 / 507	550 / 507	550 / 507							
16	246 / 119	313 / 192	380 / 282	476 / 351	550 / 396	448 / 390	550 / 467	550 / 467	550 / 467	550 / 550	550 / 550	550 / 550	550 / 550	550 / 550	550 / 550	550 / 550
17		277 / 159	336 / 234	420 / 291	550 / 366	395 / 324	495 / 404	550 / 443	550 / 443	512 / 488	550 / 526	550 / 526	550 / 526	550 / 526	550 / 526	550 / 526
18		246 / 134	299 / 197	374 / 245	507 / 317	352 / 272	441 / 339	530 / 397	550 / 408	456 / 409	508 / 456	550 / 490	550 / 490	550 / 490	550 / 490	550 / 490
19		221 / 113	268 / 167	335 / 207	454 / 269	315 / 230	395 / 287	475 / 336	550 / 383	408 / 347	455 / 386	547 / 452	550 / 455	550 / 455	550 / 455	550 / 455
20		199 / 97	241 / 142	302 / 177	409 / 230	284 / 197	356 / 246	428 / 287	525 / 347	368 / 297	410 / 330	493 / 386	550 / 426	550 / 426	550 / 426	550 / 426
21			218 / 123	273 / 153	370 / 198	257 / 170	322 / 212	388 / 248	475 / 299	333 / 255	371 / 285	447 / 333	503 / 373	548 / 405	550 / 406	550 / 406
22			199 / 106	249 / 132	337 / 172	234 / 147	293 / 184	353 / 215	432 / 259	303 / 222	337 / 247	406 / 289	458 / 323	498 / 351	550 / 385	550 / 385
23			181 / 93	227 / 116	308 / 150	214 / 128	268 / 160	322 / 188	395 / 226	277 / 194	308 / 216	371 / 252	418 / 282	455 / 307	507 / 339	550 / 363

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