DR. ANNE NICHOLS
FALL 2013

lecture
twenty one

http://nisee.berkeley.edu/godden

steel construction and design

Steel

- cast iron – wrought iron - steel
- cables
- columns
- beams
- trusses
- frames

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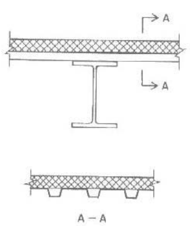
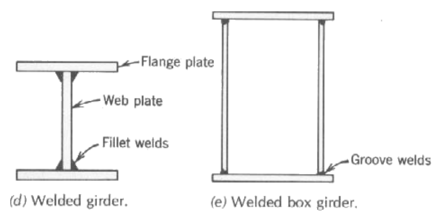
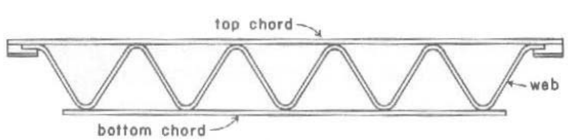
Steel Construction

- standard rolled shapes
- open web joists
- plate girders
- decking

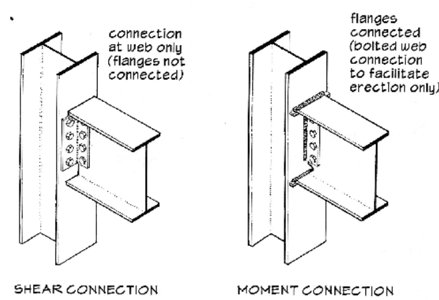






Steel Construction

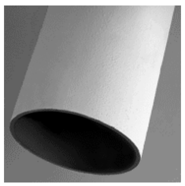
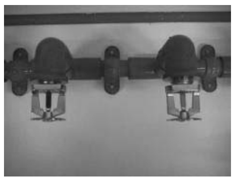
- welding
- bolts



Steel Construction

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



Steel Materials

- *high strength to weight ratio*
- *ductile*
- *beam size often limited by deflection*
- *column size limited by slenderness*

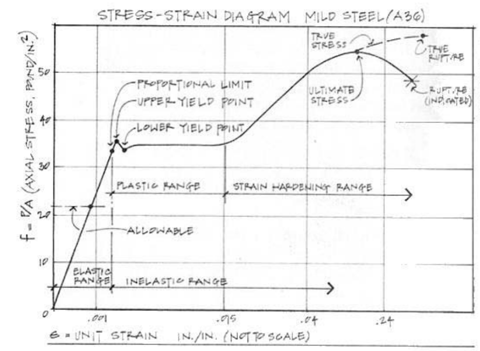
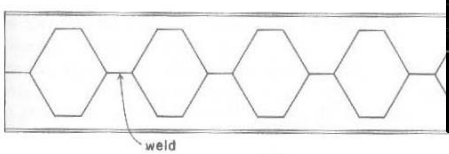


Figure 5.22 Stress-strain diagram for mild steel (A36) with key points highlighted.

Steel Beams

- *types*
  - *manufactured shapes*



(b)

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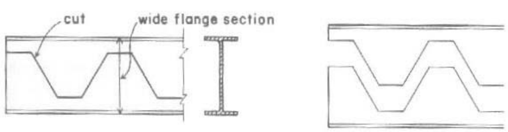
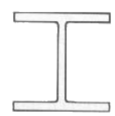
castellated



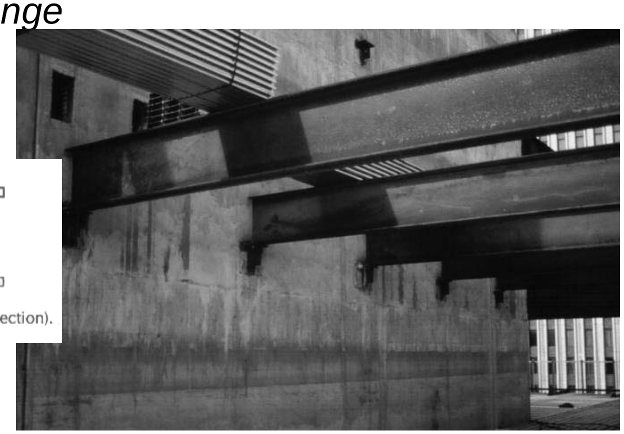
(c) (d)

Steel Beams

- *types*
  - *wide flange*

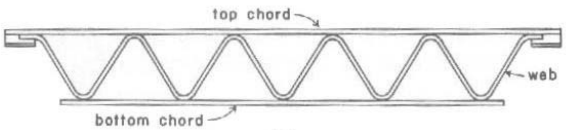


Rolled beam (W section).



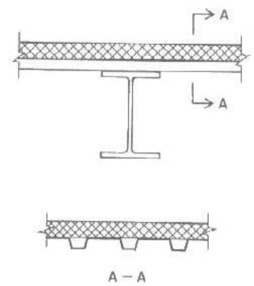
Steel Beams

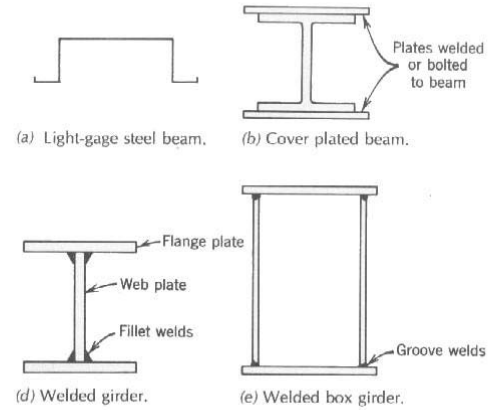
- *types*
  - *open web joists (manufactured trusses)*



Steel Beams

- *types (more)*
  - *plate girder*
  - *decking*





Steel Beams

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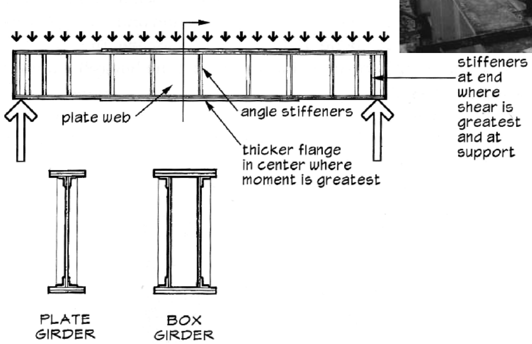
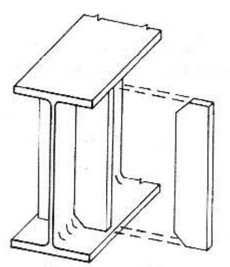
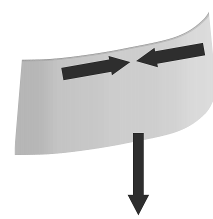


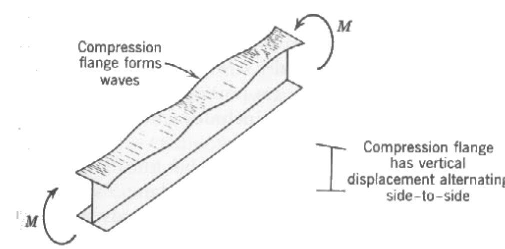


stiffeners to prevent lateral buckling

Steel Beams

- *lateral stability - bracing*
- *local buckling - stiffen*

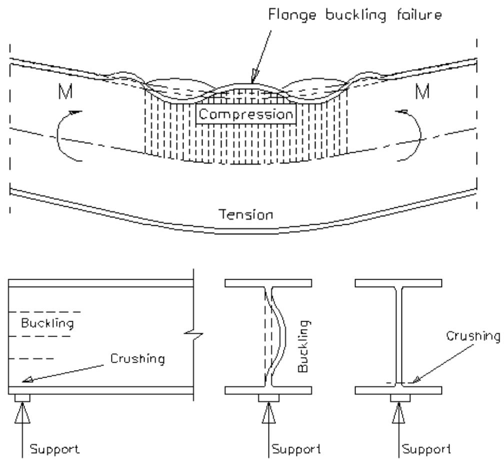




Local buckling of compression flange.

Local Buckling

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



Local Buckling

- flange

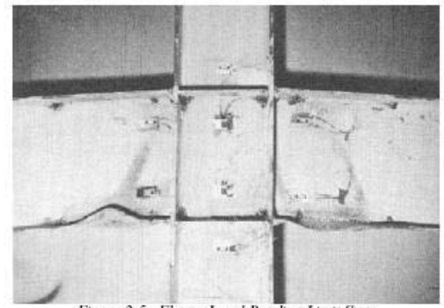


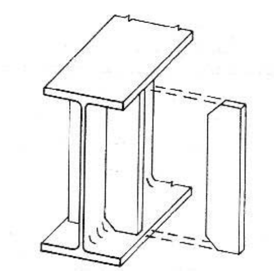
Figure 2-5. Flange Local Bending Limit State (Beedle, L.S., Christopher, R., 1964)

- web

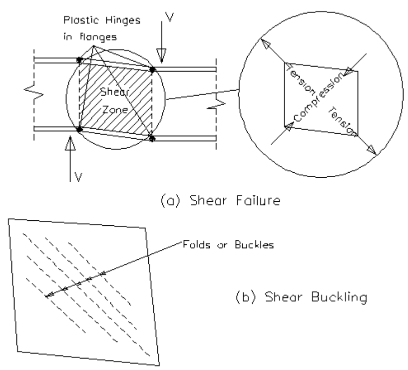
Figure 2-7. Web Local Buckling Limit State (SAC Project)

Shear in Web

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

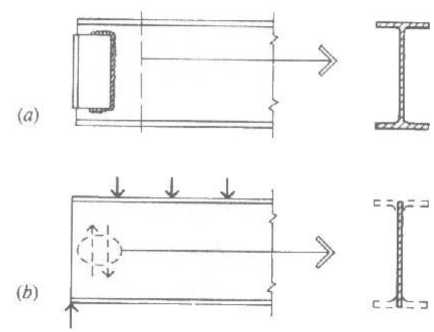


stiffeners to prevent lateral buckling



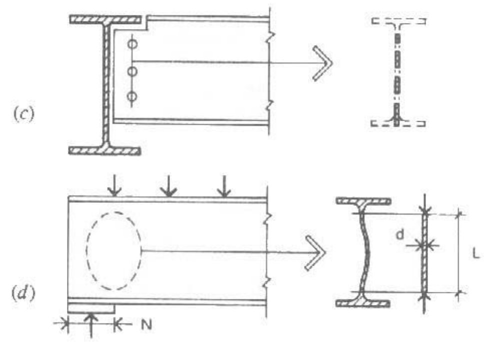
Steel Beams

- end conditions
  - a) away from connection - full section effective
  - b) high shear – only web effective



Steel Beams

- *end conditions*
  - *c) bolt holes – less material*
  - *d) local web buckling*



Steel Beams

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

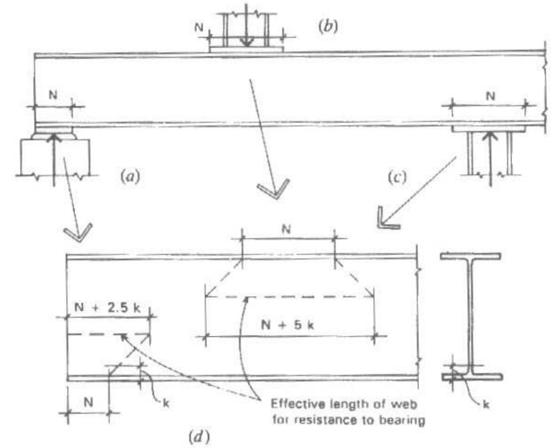
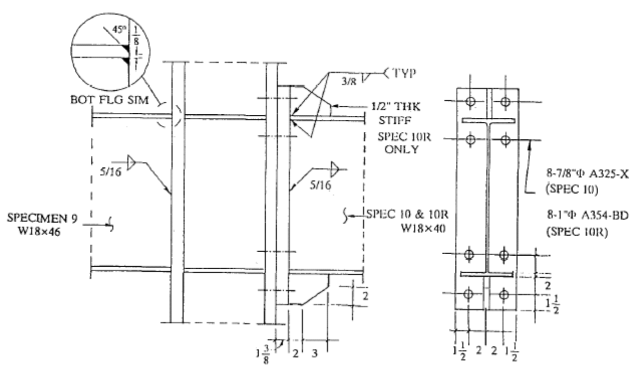


Figure 9.10 Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).

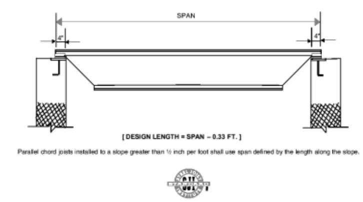
Steel Beams

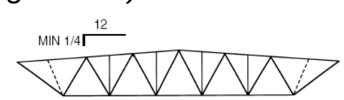
- *connections*
  - *welds*
  - *bolts*



Steel Design – Open Web Joists

- *SJI: www.steeljoist.com*
- *Vulcraft: www.vulcraft.com*
  - *K Series (Standard)*
    - *8-30" deep, spans 8-50 ft*
  - *LH Series (Long span)*
    - *18-48" deep, spans 25-96 ft*
  - *DLH (Deep Long Spans)*
    - *52-72" deep, spans 89-144 ft*
  - *SLH (Long spans with high strength steel)*
    - *pitched top chord*
    - *80-120" deep, spans 111-240 ft*





Steel Design – Open Web Joists

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

Steel Design – Open Web Joists

LRFD

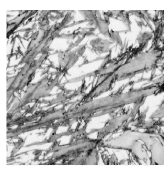
STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

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 (L/360) in RED
total in BLACK

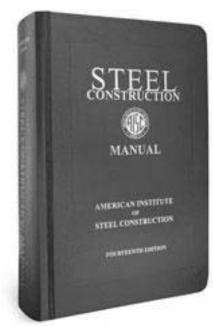
Steel Beam Design

- *American Institute of Steel Construction*
  - *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 Beam Design

- *AISC: 14th ed.*
  - *combined ASD & LRFD in one volume in 2005*



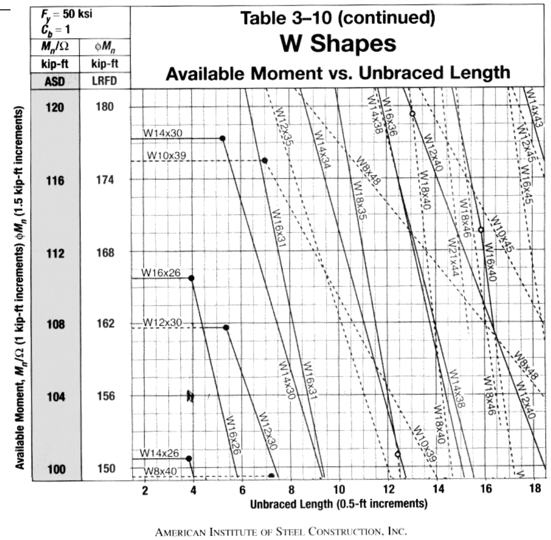
Unified Steel Design

- 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

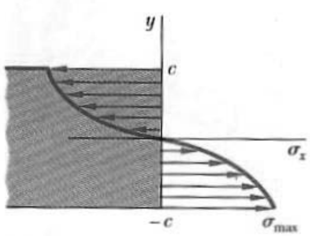
Unified Steel Design

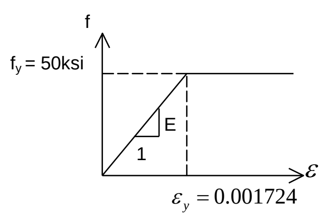
- braced vs. unbraced



LRFD Steel Beam Design

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





Load Types

- D = dead load
- L = live load
- L_r = live roof load
- W = wind load
- S = snow load
- E = earthquake load
- R = rainwater load or ice water load
- T = effect of material & temperature

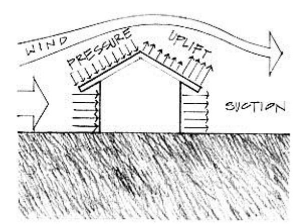


Figure 1.13 Wind loads on a structure.

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)

Pure Flexure

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

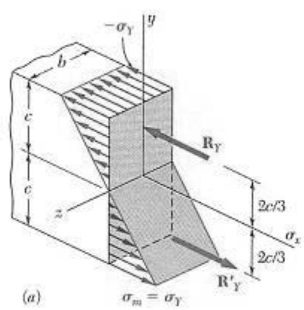
M_u - maximum moment
ϕ_b - resistance factor for bending = 0.9
M_n - nominal moment (ultimate capacity)
F_y - yield strength of the steel
Z - plastic section modulus*

Internal Moments - at yield

- material hasn't failed

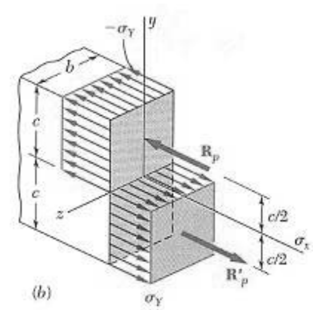
$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y$$

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

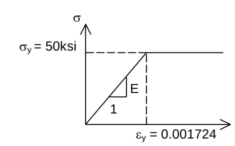


Internal Moments - ALL at yield

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

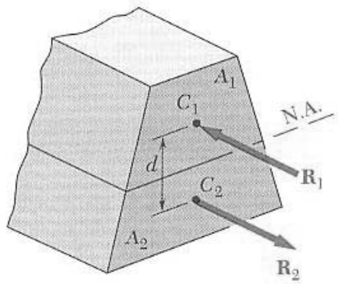


$$M_p = bc^2 f_y = \tfrac{3}{2} M_y$$



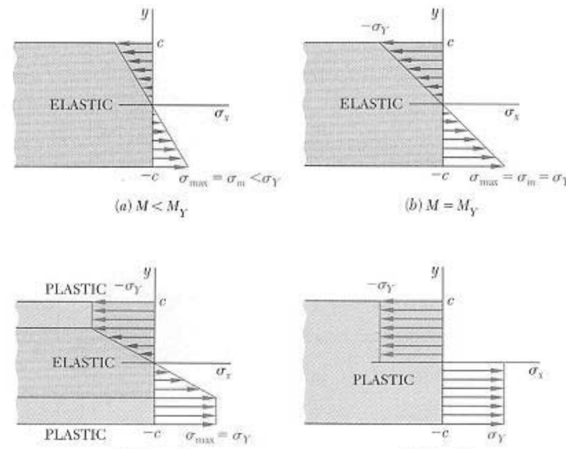
n.a. of Section at Plastic Hinge

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



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

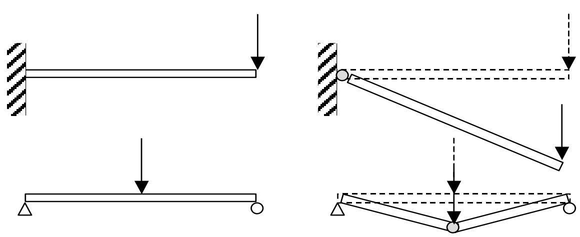
Plastic Hinge Development



(a) $M < M_Y$ (b) $M = M_Y$ (c) $M > M_Y$ (d) $M = M_p$

Plastic Hinge Examples

- stability can be effected



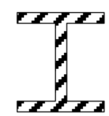
Plastic Section Modulus

- shape factor, k

$$k = M_p / M_y$$

= 3/2 for a rectangle

≈ 1.1 for an I



$$k = Z / S$$

- plastic modulus, Z

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

Shear

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

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 = $t_w d$

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*

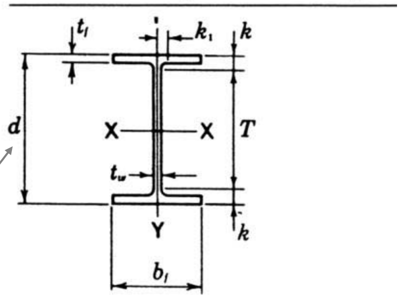
$$L_p = 1.76 r_y \sqrt{\frac{F_y}{E}}$$

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

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

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

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

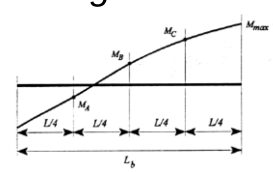
C_b = modification factor

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

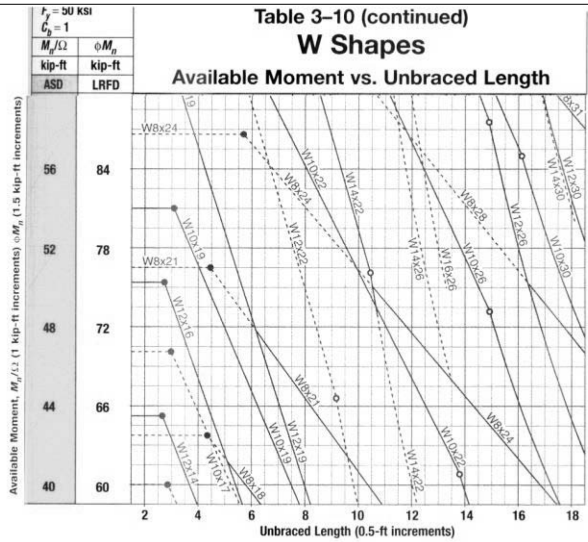
M_A - |moment|, 1/4 point

M_B = |moment|, center point

M_C = |moment|, 3/4 point



Beam Design Charts

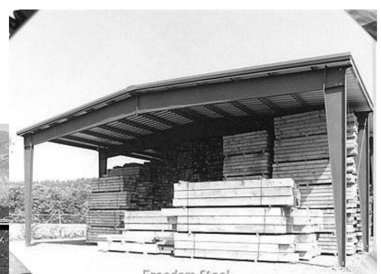


Deflection Limits

- *based on service condition*
- *no "impairment" to serviceability*
- *avoid ponding*
- *L/360 due to live load for beams & girders supporting plaster (service)*

Steel Arches and Frames

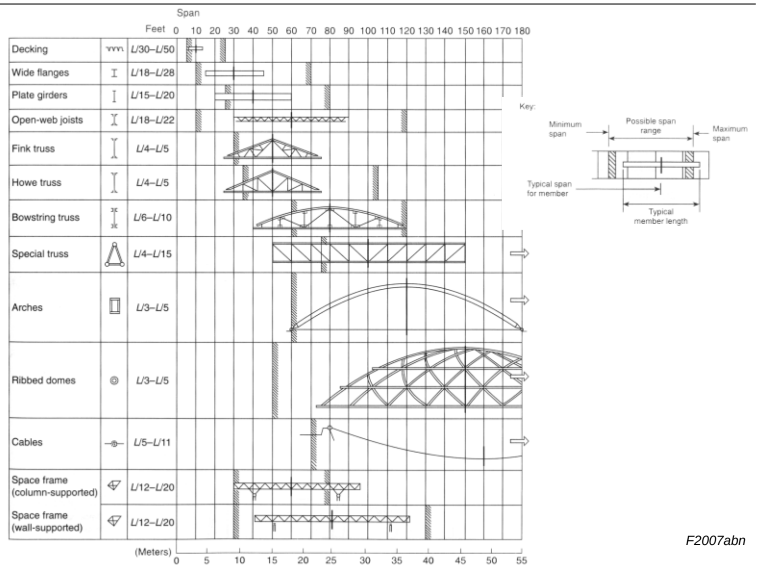
- *solid sections or open web*



http://nisee.berkeley.edu/godden

Steel Shell and Cable Structures

Approximate Depths



Unified Column Design

- limit states for failure

$$P_a \le P_n/\Omega$$

$$\phi_c = 0.90 \qquad P_n = F_{cr}A_g$$

$$P_u \le \phi_c P_n$$

1. yielding $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ or $F_e \ge 0.44F_y$
2. buckling $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ or $F_e < 0.44F_y$

F_e – elastic buckling stress (Euler)

Unified Column Design

- $P_n = F_{cr}A_g$
  - for $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ $\quad F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y$
  - for $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ $\quad F_{cr} = 0.877F_e$
  - where $\quad F_e = \frac{\pi^2 E}{\left(KL/r\right)^2}$

Procedure for Analysis

1. calculate KL/r
   - biggest of KL/r with respect to x axes and y axis
2. find F_{cr} from appropriate equation
   - tables are available
3. compute $P_n = F_{cr}A_g$
   - or find $f_c = P_a/A$ or P_u/A
4. is $P_a \le P_n/\Omega$ or $P_u \le \phi P_n$?
   - yes: ok
   - no: insufficient capacity and no good

Procedure for Design

1. *guess a size (pick a section)*
2. *calculate KL/r*
   - *biggest of KL/r with respect to x axes and y axis*
3. *find F_{cr} from appropriate equations*
   - *or find a table*
4. *compute $P_n = F_{cr}A_g$*
   - *or find $f_c = P_a/A$ or P_u/A*

Procedure for Design (cont'd)

5. *is $P_a \le P_n/\Omega$ <u>or</u> $P_u \le \phi P_n$?*
   - *yes: ok*
   - *no: pick a bigger section and **go back to step 2.***
6. *check design efficiency*
   - *percentage of stress =* $\dfrac{P_r}{P_c} \cdot 100\%$
   - *if between 90-100%: good*
   - *if < 90%: pick a smaller section and **go back to step 2.***

Column Tables

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi — W12

Shape		W12×									
Wt/ft		96		87		79		72		65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
s of gyration r_y	0	844	1270	766	1150	694	1040	633	951	571	859
	6	811	1220	735	1110	667	1000	607	913	548	824
	7	800	1200	725	1090	657	987	598	899	540	811
	8	787	1180	713	1070	646	971	588	884	531	798
	9	772	1160	699	1050	634	952	577	867	520	782
	10	756	1140	685	1030	620	932	565	849	509	765
	11	739	1110	669	1010	606	910	551	828	497	747

Beam-Column Design

- *moment magnification (P-Δ)*

$$M_u = B_1 M_{max-factored} \qquad B_1 = \frac{C_m}{1-(P_u/P_{e1})}$$

C_m – modification factor for end conditions

= 0.6 – 0.4(M_1/M_2) or

0.85 restrained, 1.00 unresrained

P_{e1} – Euler buckling strength $\quad P_{e1} = \dfrac{\pi^2 EA}{\left(Kl/r\right)^2}$

Beam-Column Design

- LRFD (Unified) Steel
  - for $\frac{P_r}{P_c} \geq 0.2:\quad \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$
  - for $\frac{P_r}{P_c} < 0.2:\quad \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$

P_r is required, P_c is capacity

ϕ_c - resistance factor for compression = 0.9

ϕ_b - resistance factor for bending = 0.9

Construction Supervision

- proper grade material
  - high strength bolts
- quality welds
- proper bolted conditions (ex. sc)
- fabrication and erection of steel frame connection details