ARCHITECTURAL STRUCTURES III:

STRUCTURAL ANALYSIS AND SYSTEMS

ARCH 631

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FALL 2005

lecture

twenty

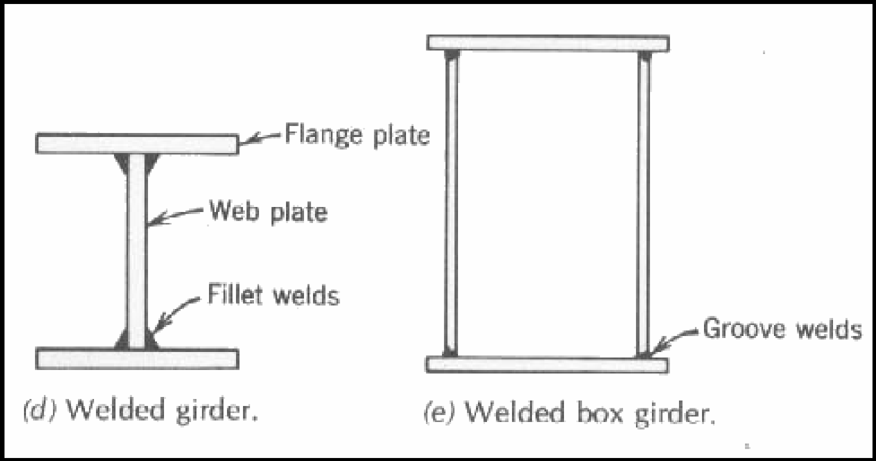
steel construction and design

Steel

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

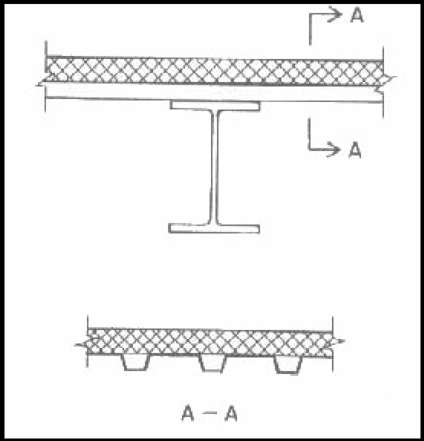
Steel Construction

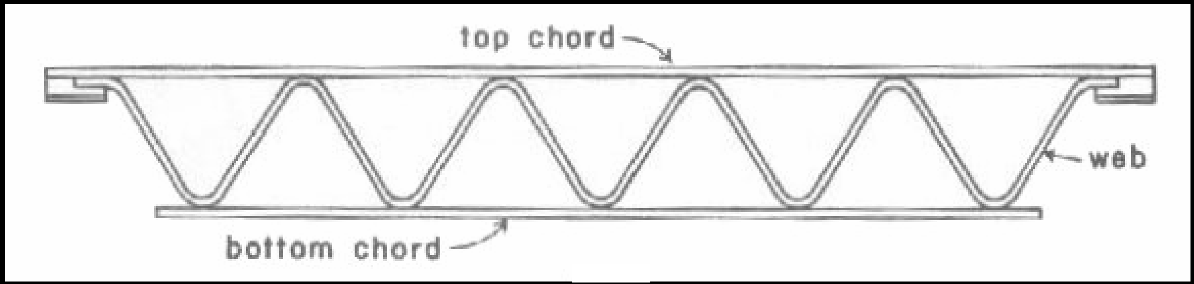
- standard rolled shapes
- open web joists
- plate girders
- decking



(d) Welded girder.

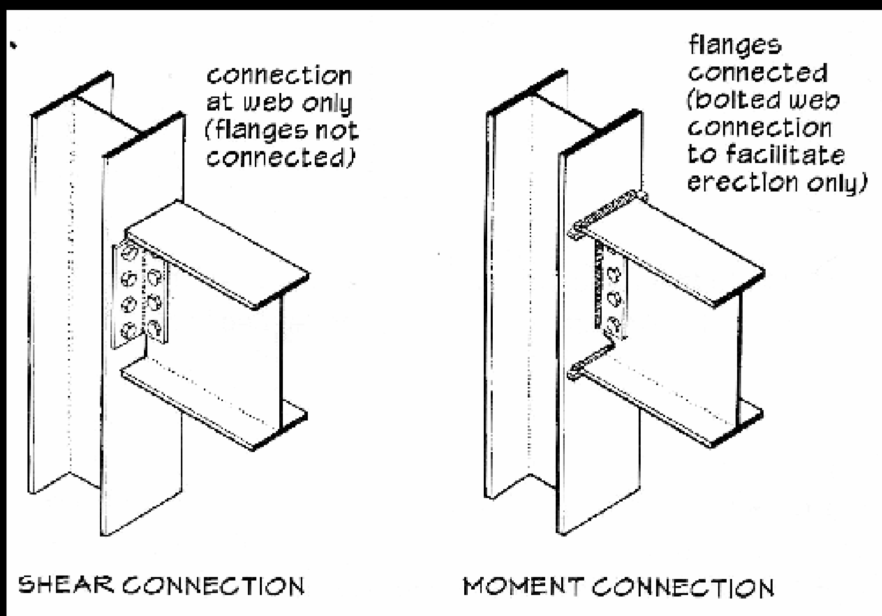
(e) Welded box girder.

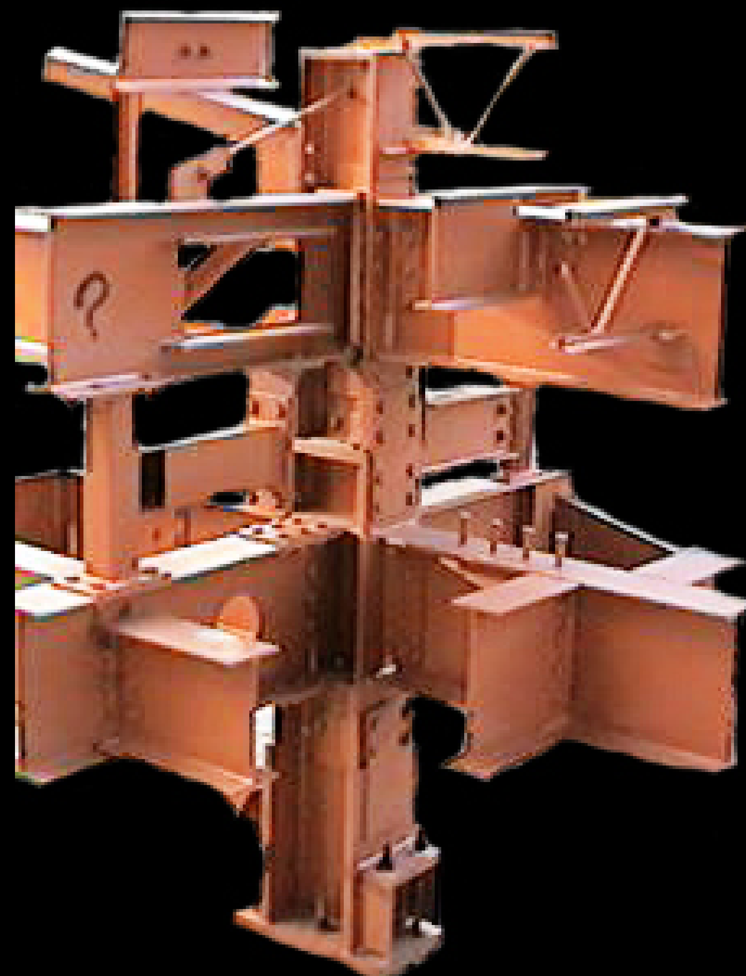




Steel Construction

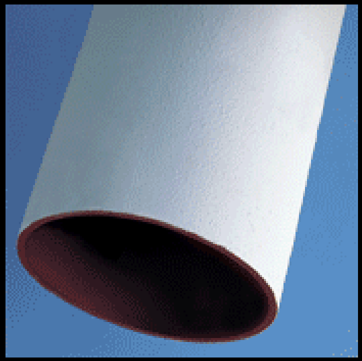
- welding
- bolts





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*

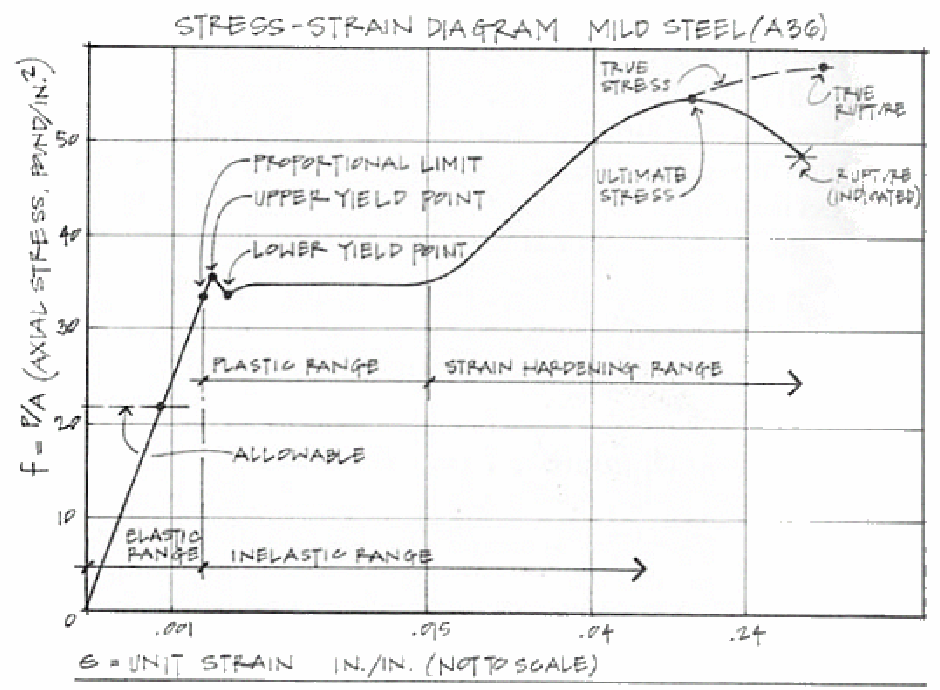
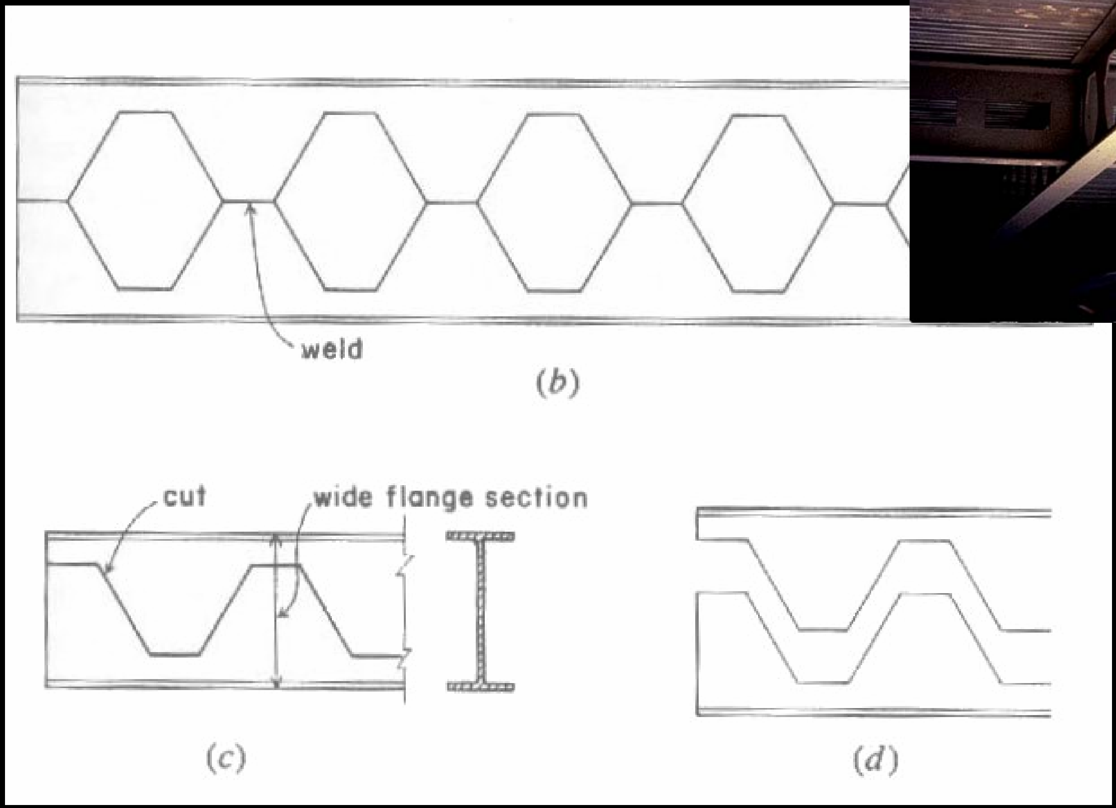


Figure 5.22 Stress-strain diagram for mild steel (A36) with key points highlighted.

Steel Beams

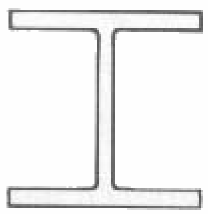
- *types*
  - *manufactured shapes*



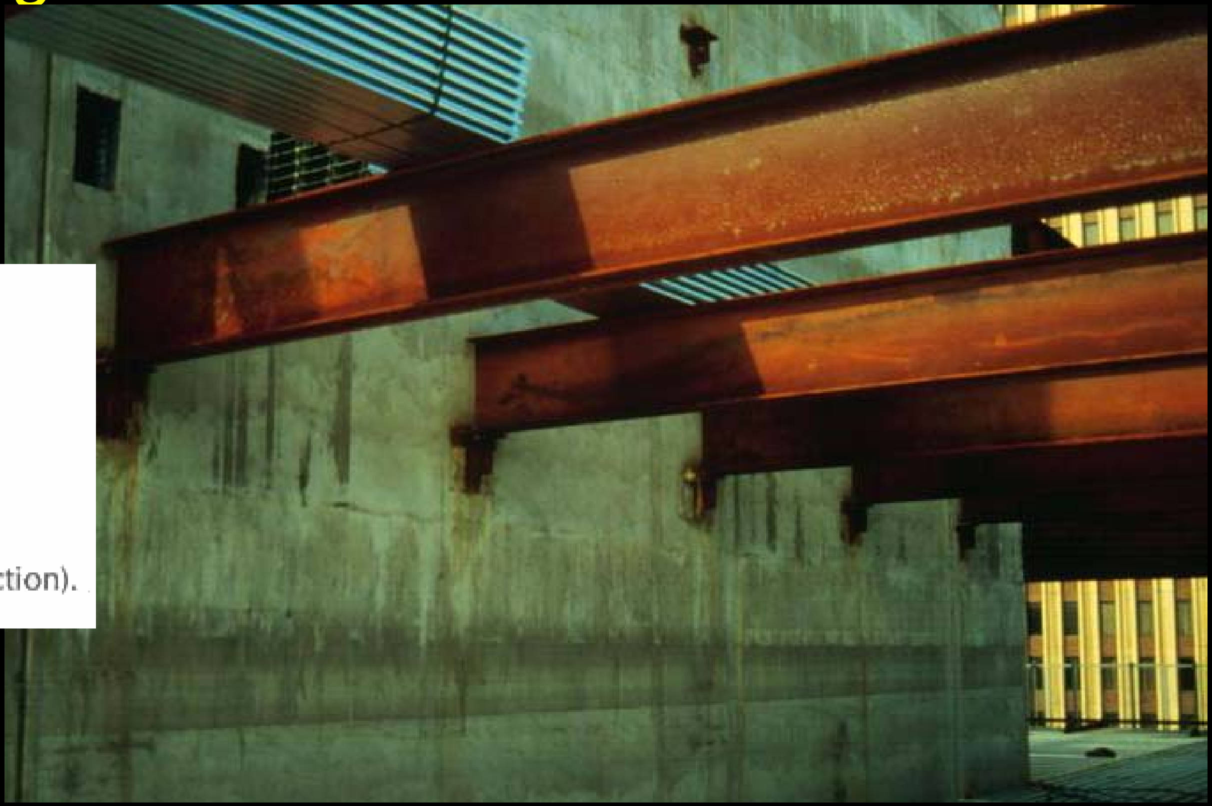
castellated

Steel Beams

- types
  - wide flange

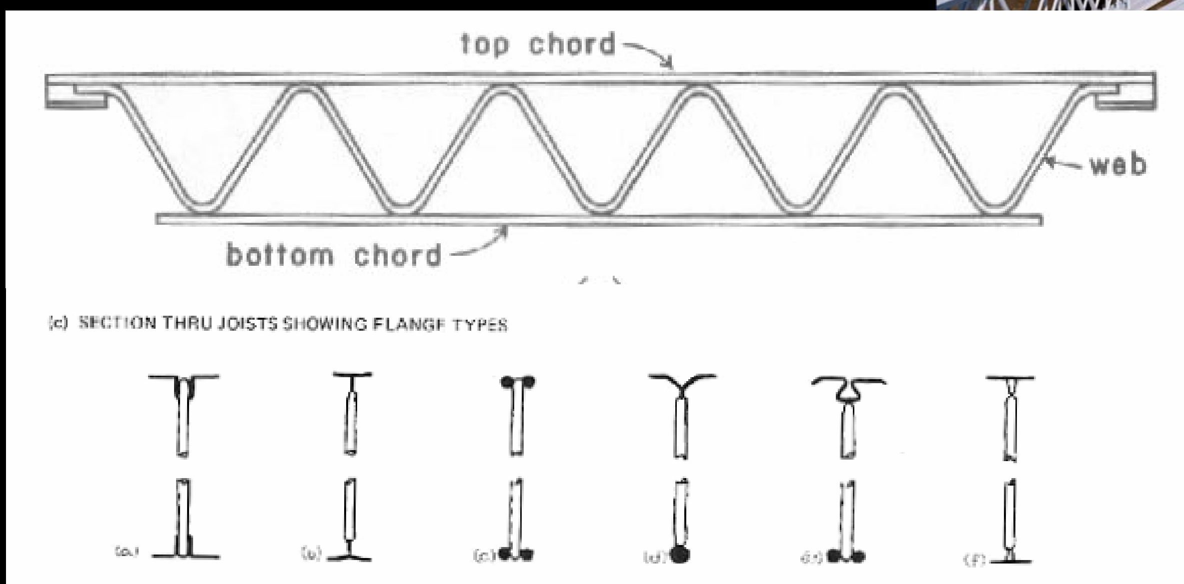


Rolled beam (W section).



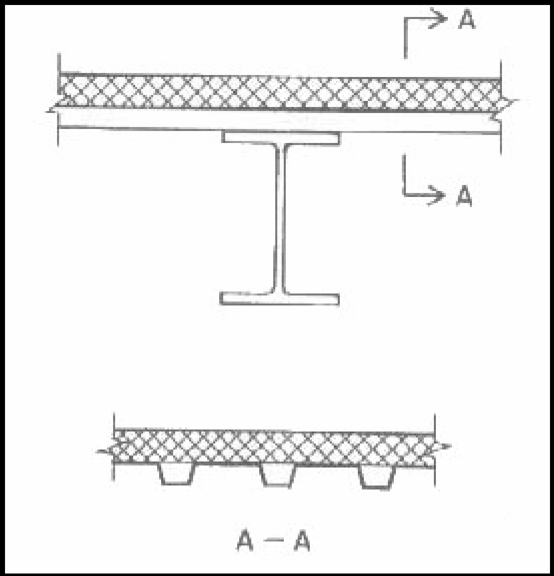
Steel Beams

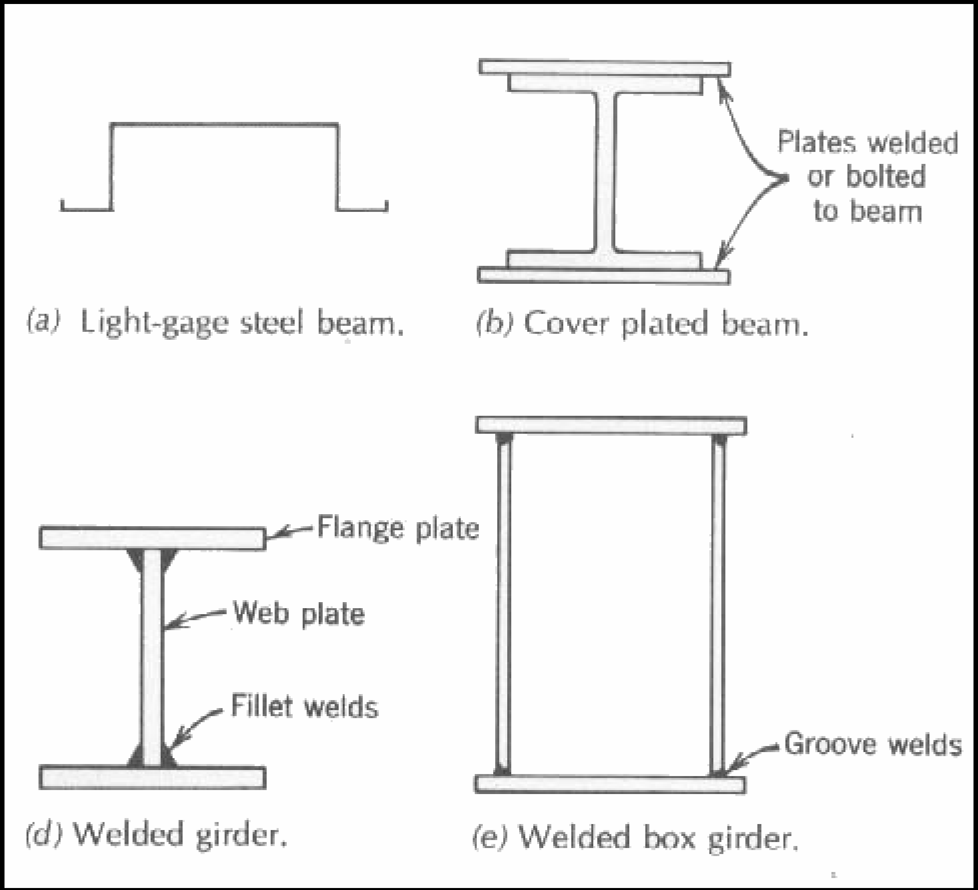
- *types*
  - *open web joists (manufactured trusses)*



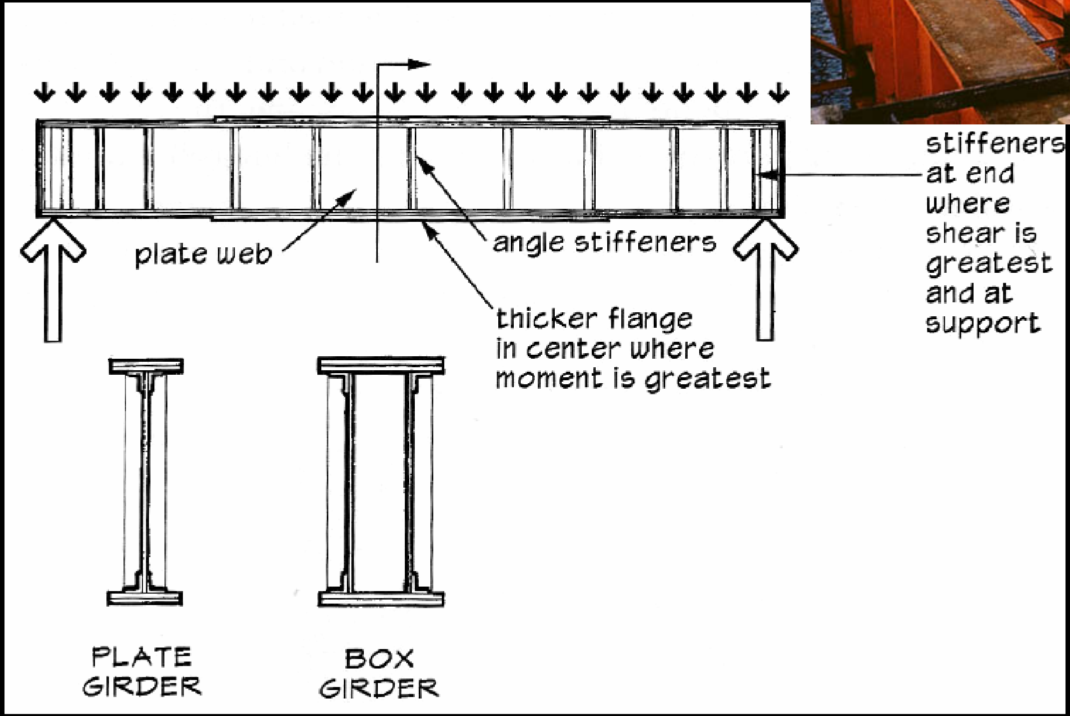
Steel Beams

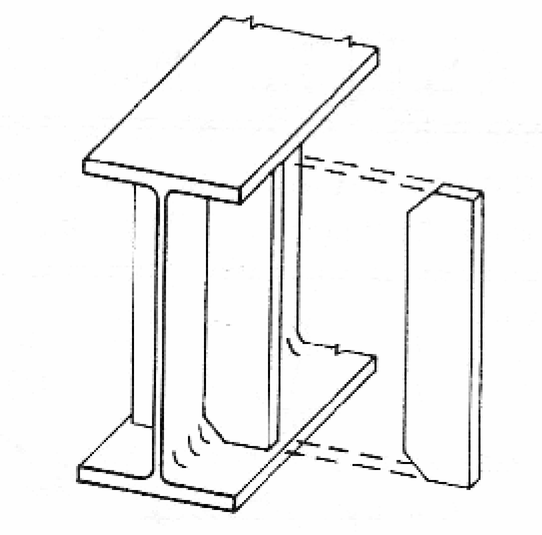
- *types (more)*
  - *plate girder*
  - *decking*





Steel Beams

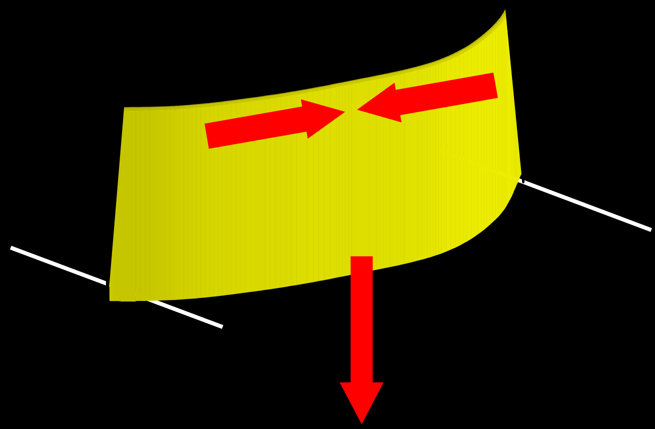


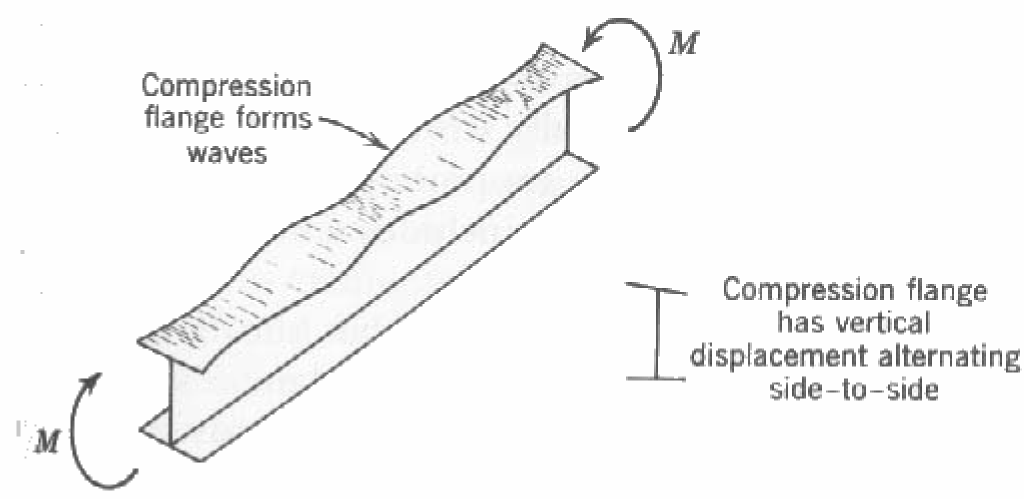


stiffeners to prevent lateral buckling

Steel Beams

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

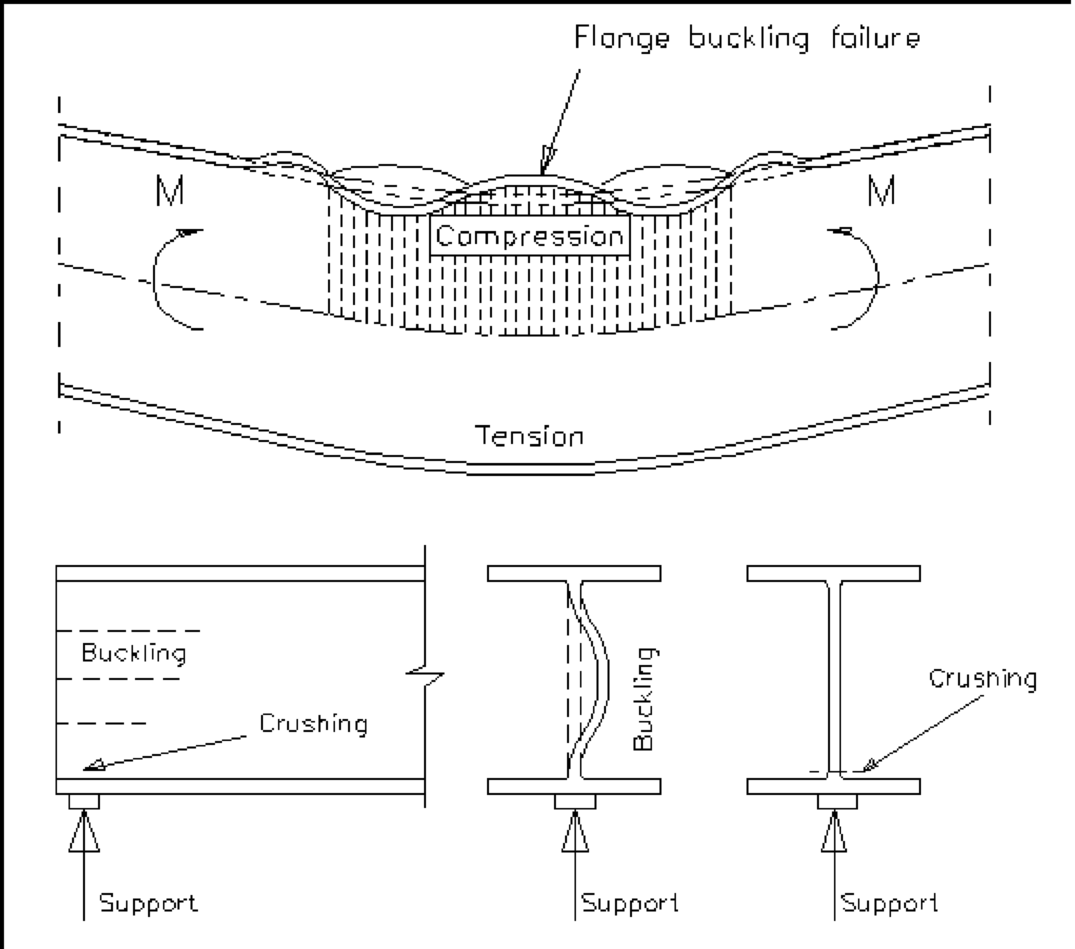




Local buckling of compression flange.

Local Buckling

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



Local Buckling

- *flange*

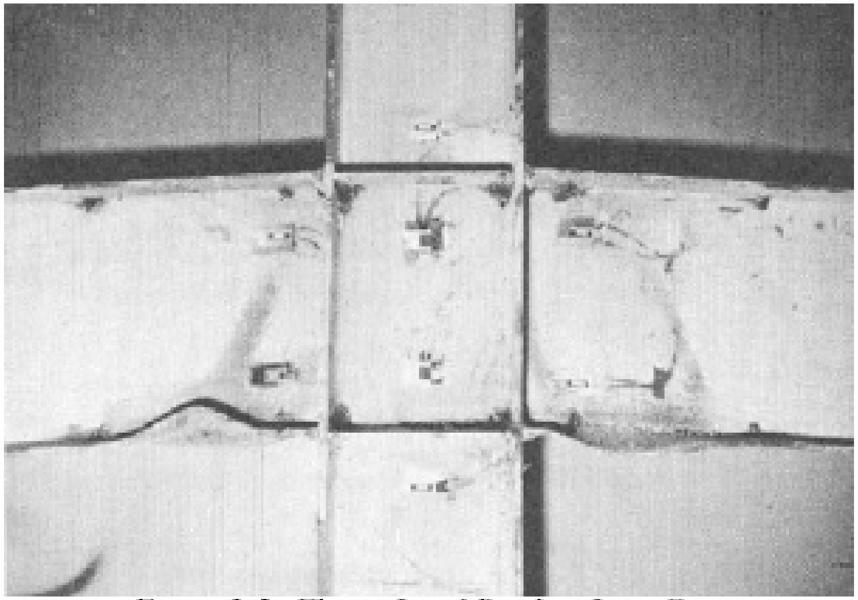


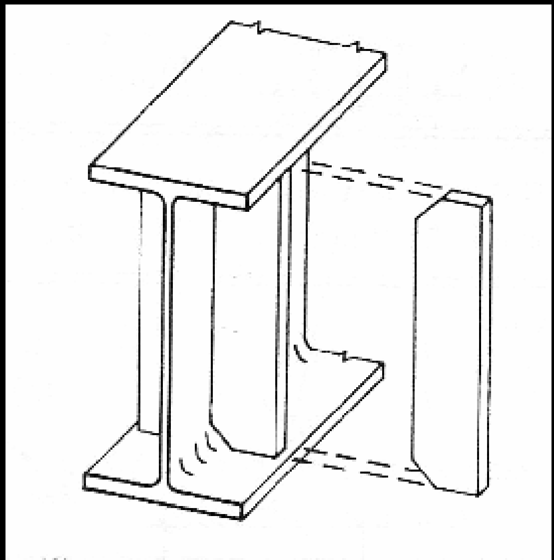
Figure 2-5. Flange Local Bending Limit State (Beedle, L.S., Christopher, R., 1964)

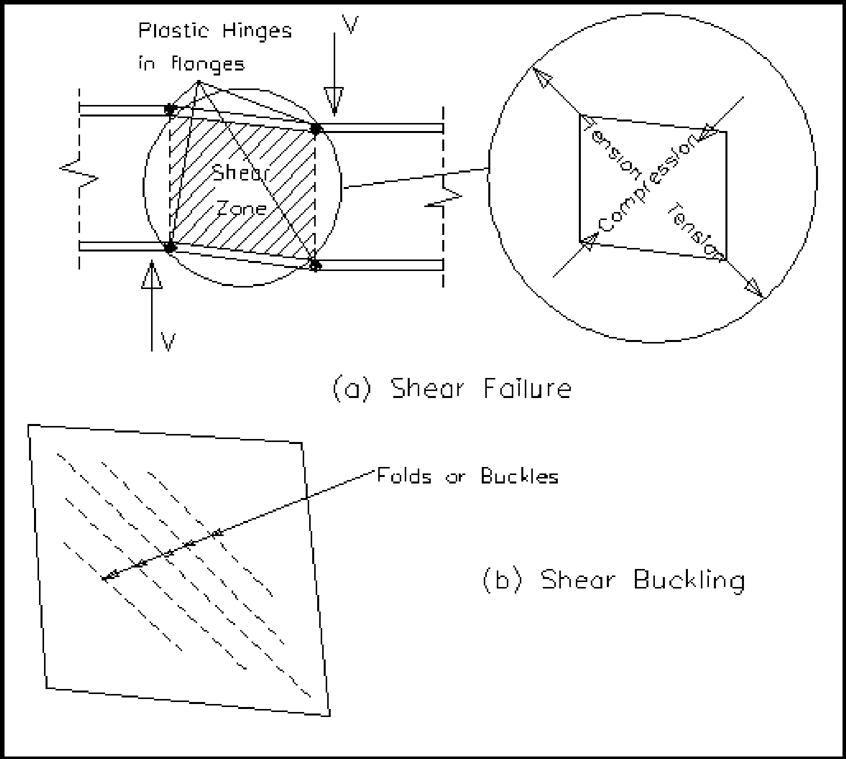
- *web*

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

Shear in Web

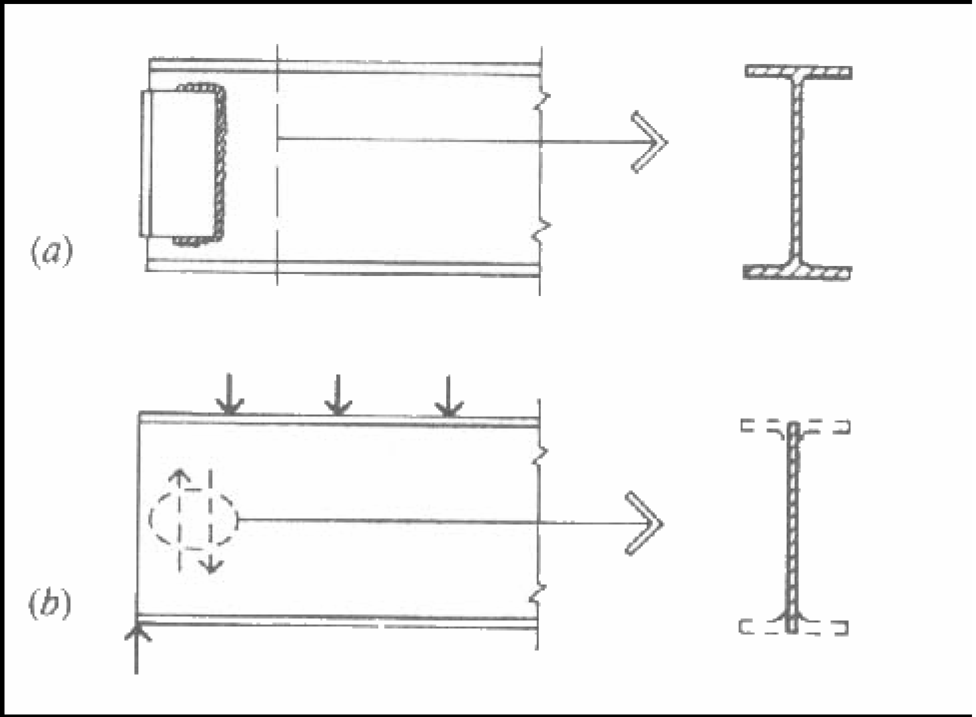
- panels in plate girders or webs with large shear
- buckling in compression direction
- add stiffeners


stiffeners to prevent lateral buckling



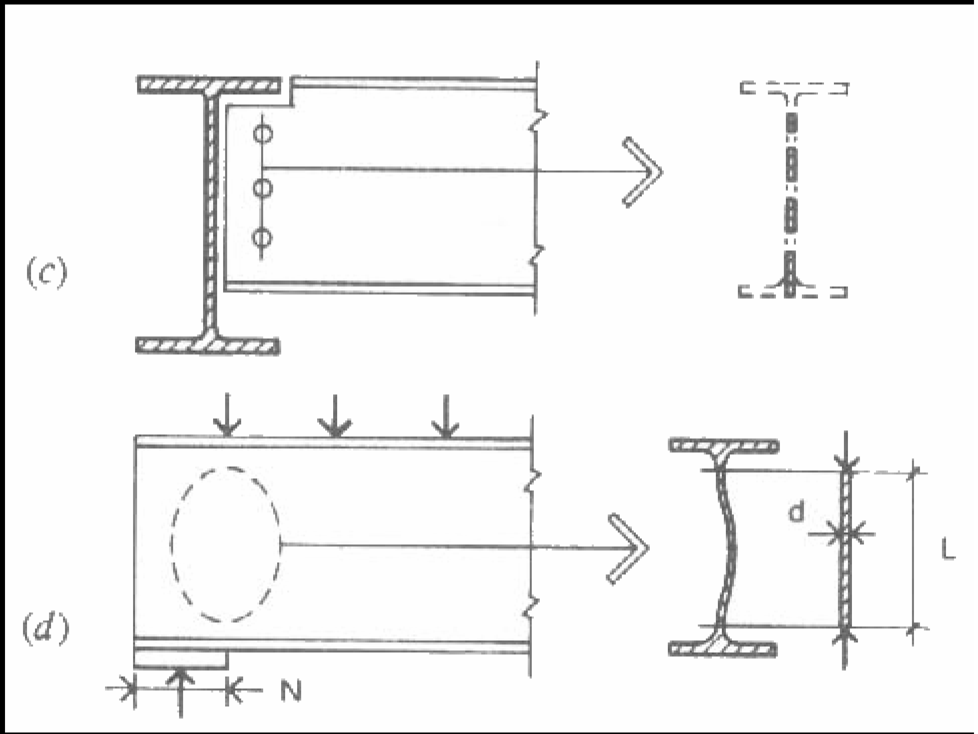
Steel Beams

- *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*

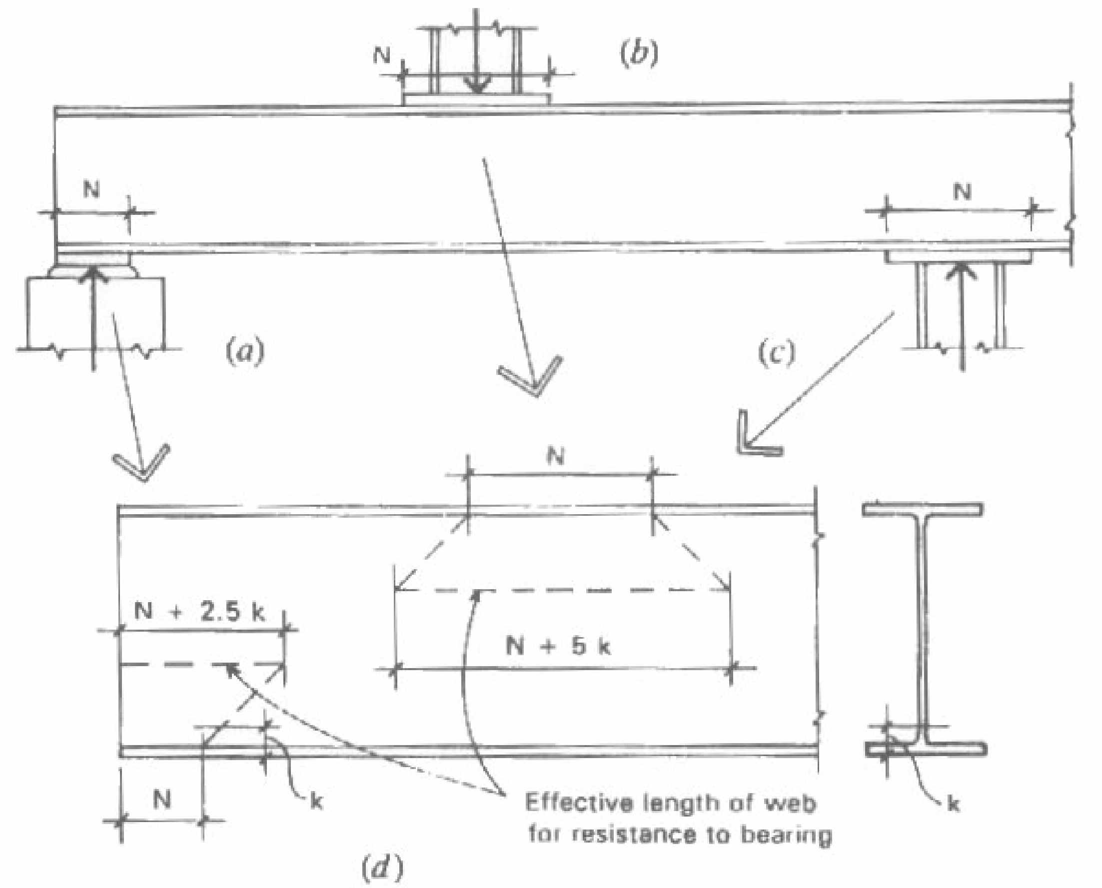
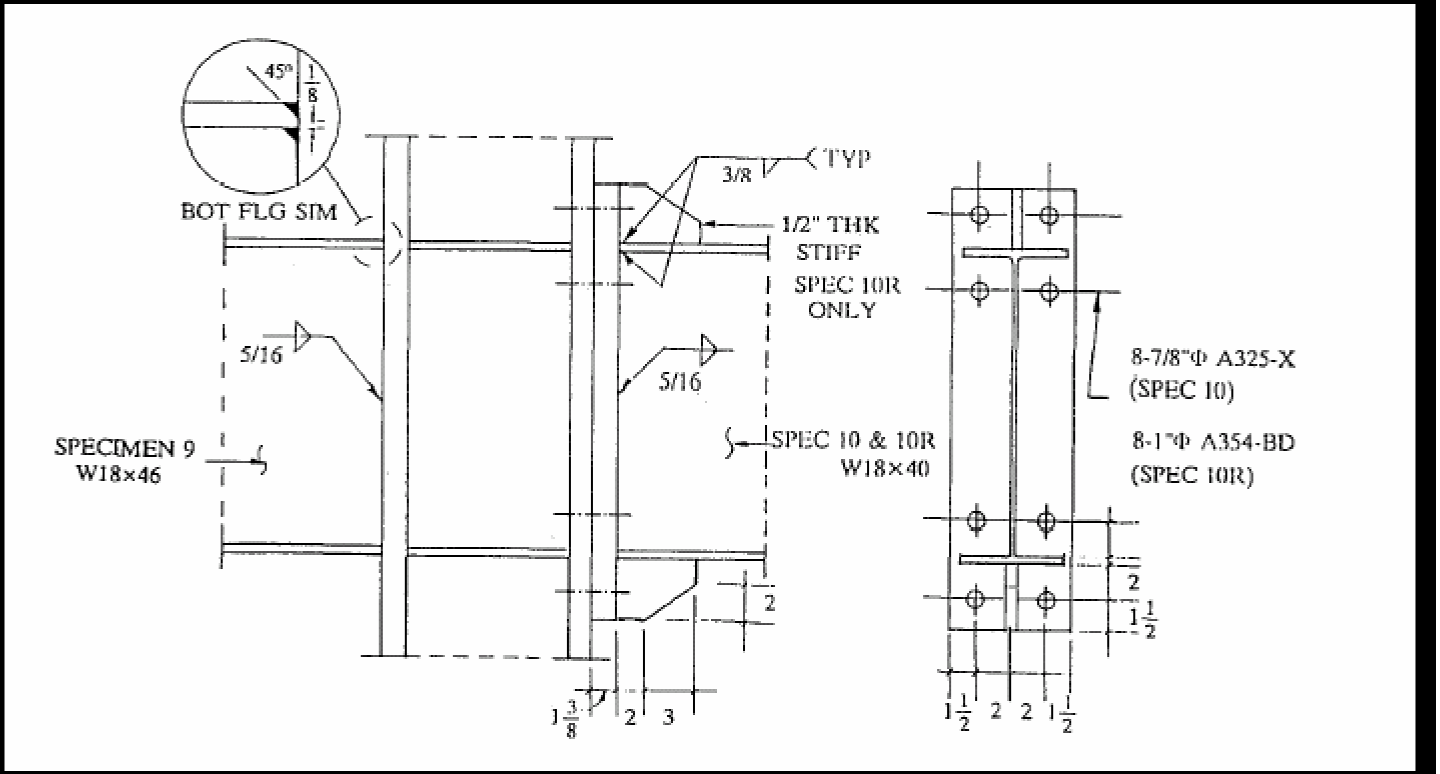


Figure 9.10 Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).

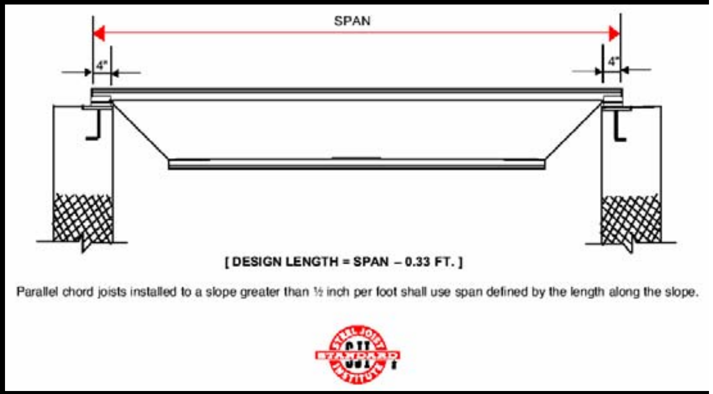
Steel Beams

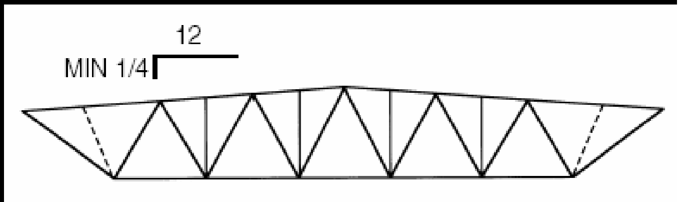
- connections
  - welds
  - bolts



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

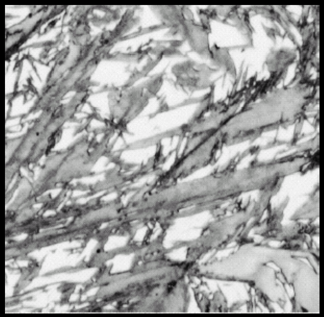
STANDARD LOAD TABLE/OPEN WEB STEEL JOISTS, K-SERIES
Based on a Maximum Allowable Tensile Stress of 30 ksi

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 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



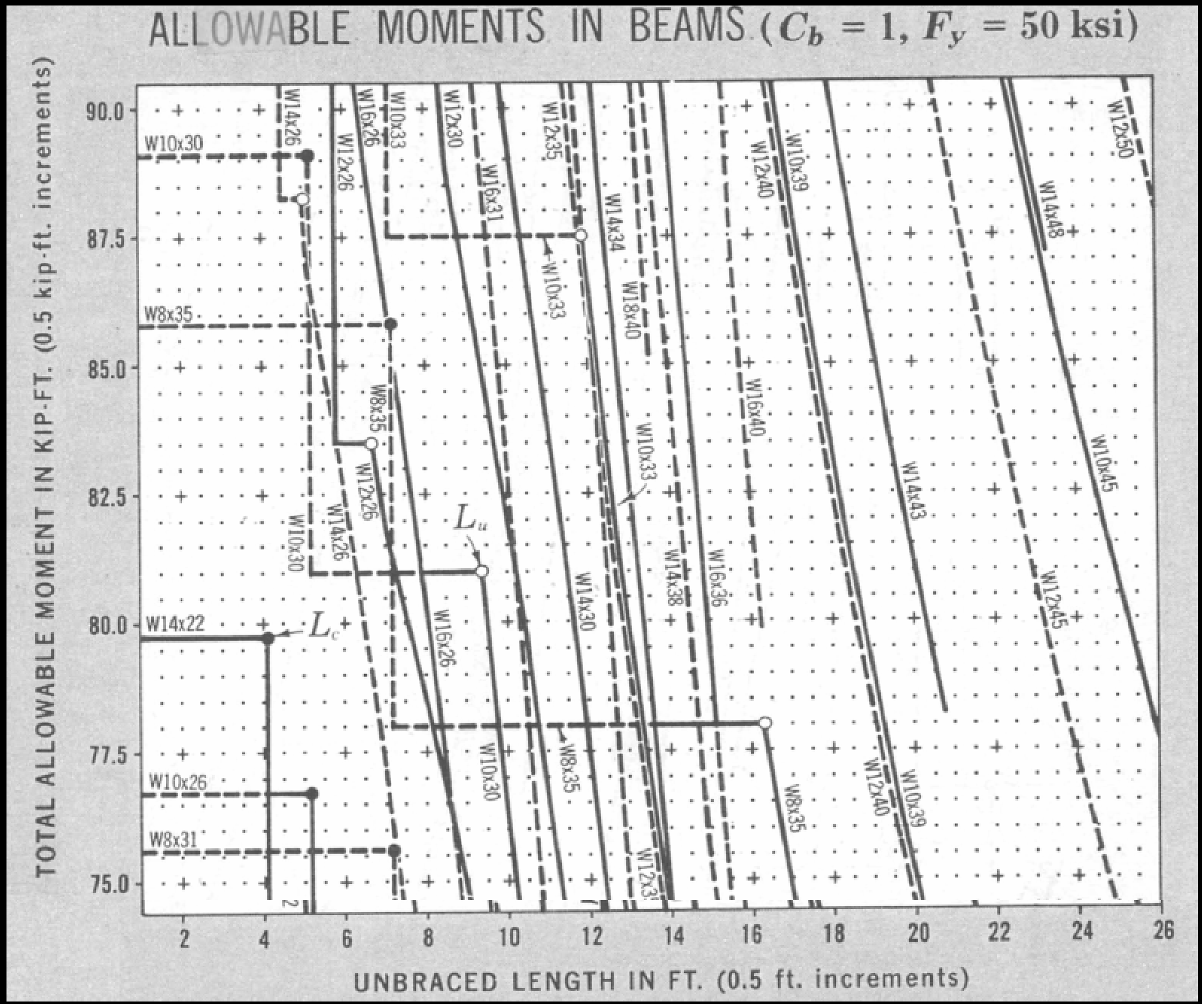
ASD Steel Beam Design

- *AISC: ASD Manual, 9th ed.*

  - *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)* $F_v = 0.30F_{weld}$

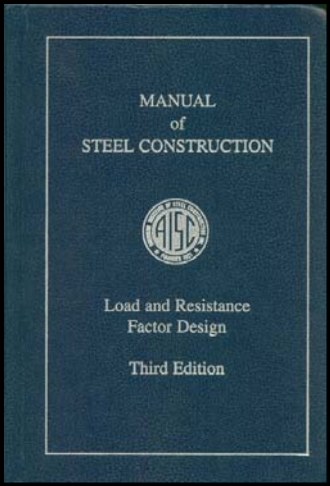
ASD Steel Beam Design

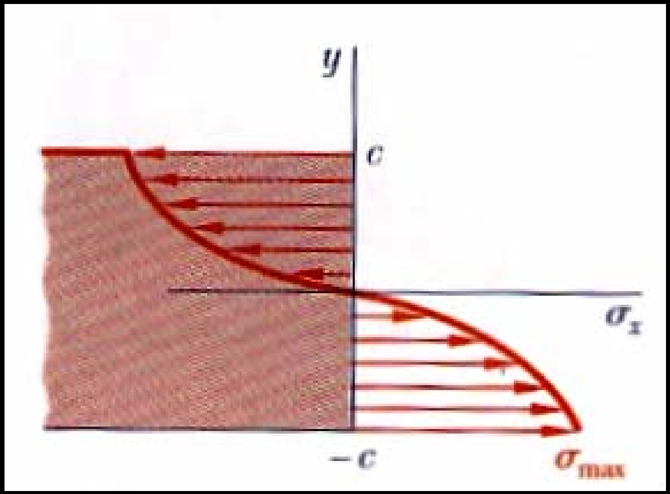
- *braced vs. unbraced*

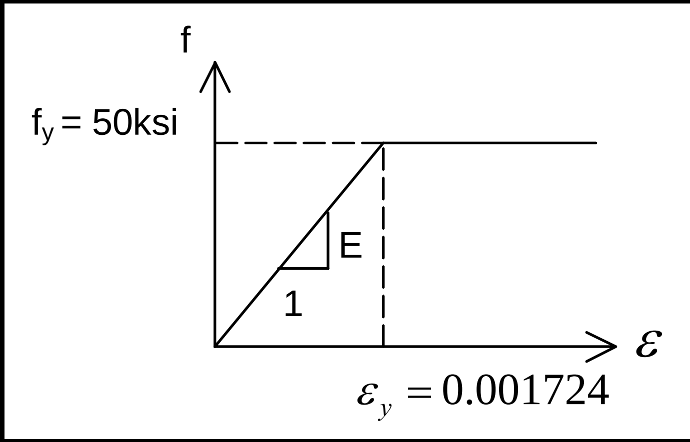


LRFD Steel Beam Design

- AISC: LRFD 3rd ed.
  - limit state is yielding <u>all across the section</u>
  - 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

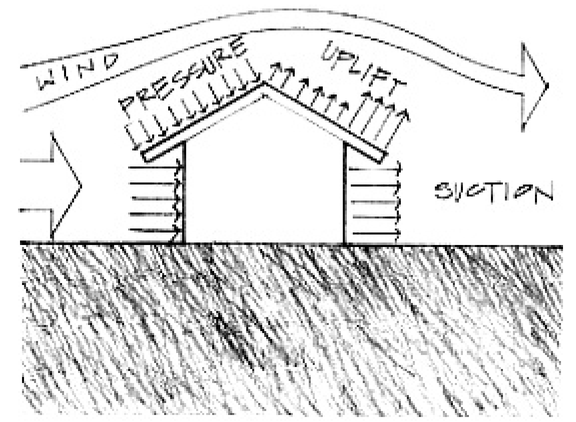


Figure 1.13 Wind loads on a structure.

LRFD Load Combinations

- $\boxed{1.4D}$
- $\boxed{1.2D + 1.6L} + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$
- $1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.5E + (0.5L \text{ or } 0.2S)$
- $0.9D \pm (1.3W \text{ or } 1.5E)$

Pure 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

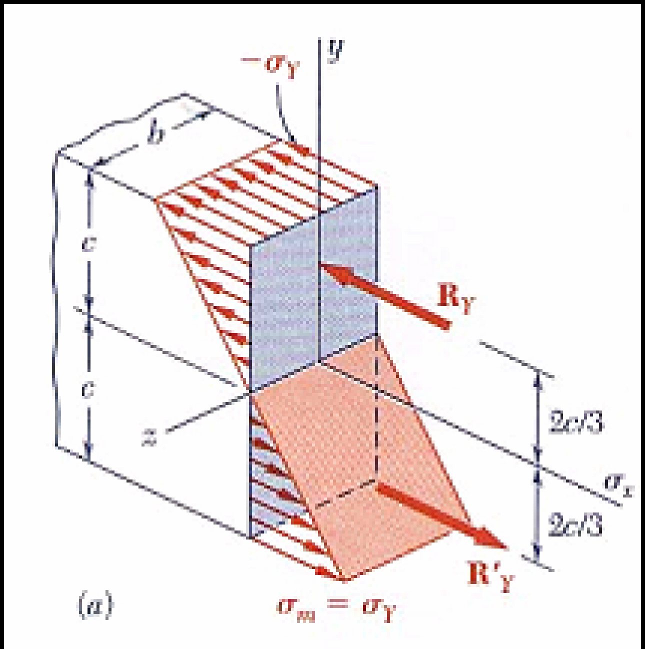
M_n - nominal moment (ultimate capacity)

F_y - yield strength of the steel

*Z - plastic section modulus**

Internal Moments - at yield

- *material hasn't failed*

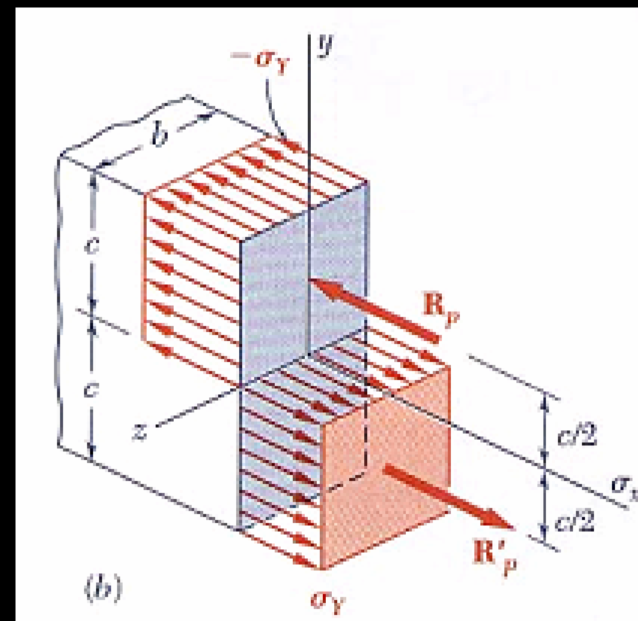


$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y$$

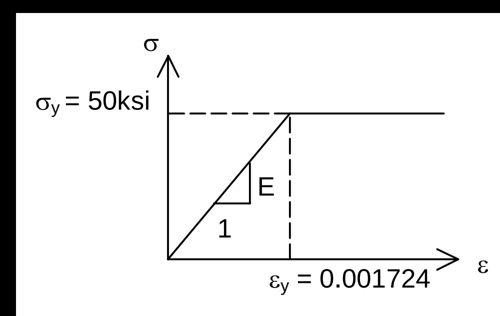
$$= \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y$$

Internal Moments - ALL at yield

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

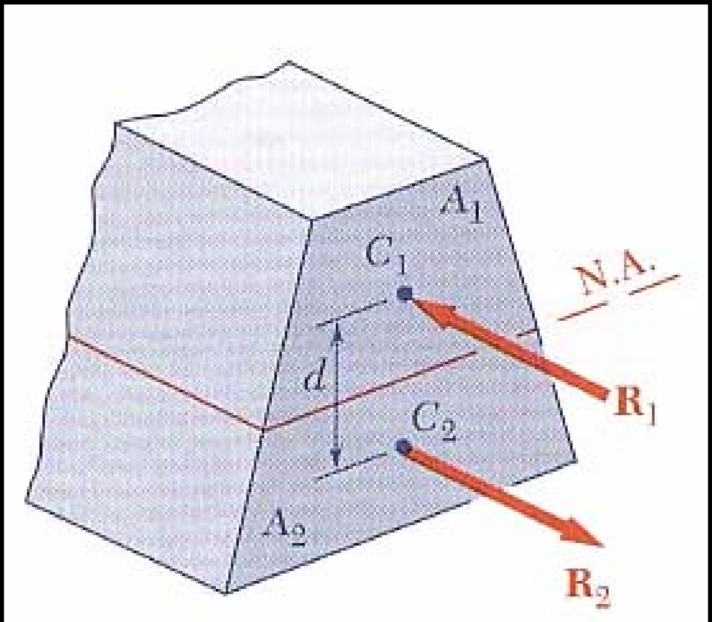


$$M_p = bc^2 f_y = \tfrac{3}{2} M_y$$



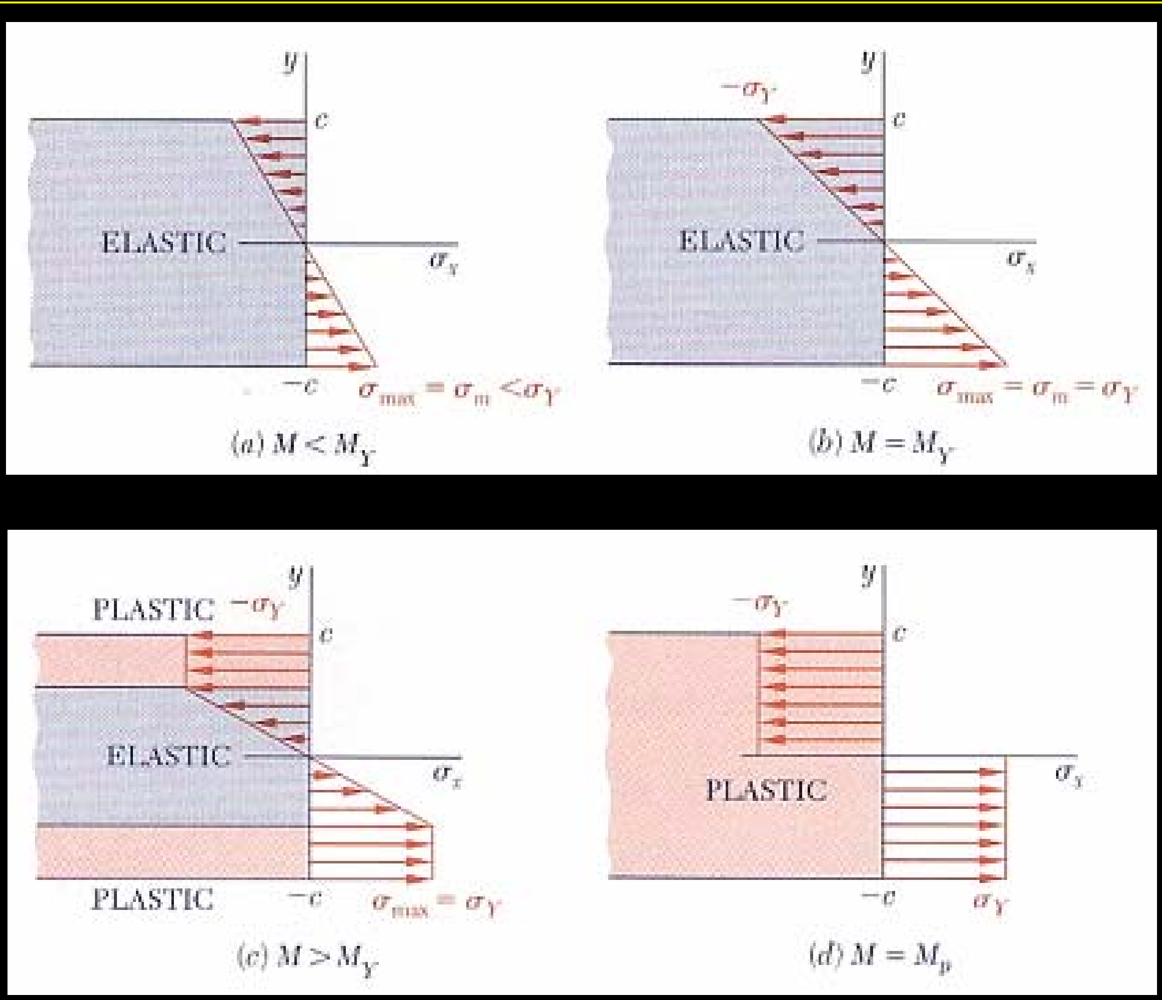
n.a. of Section at Plastic Hinge

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



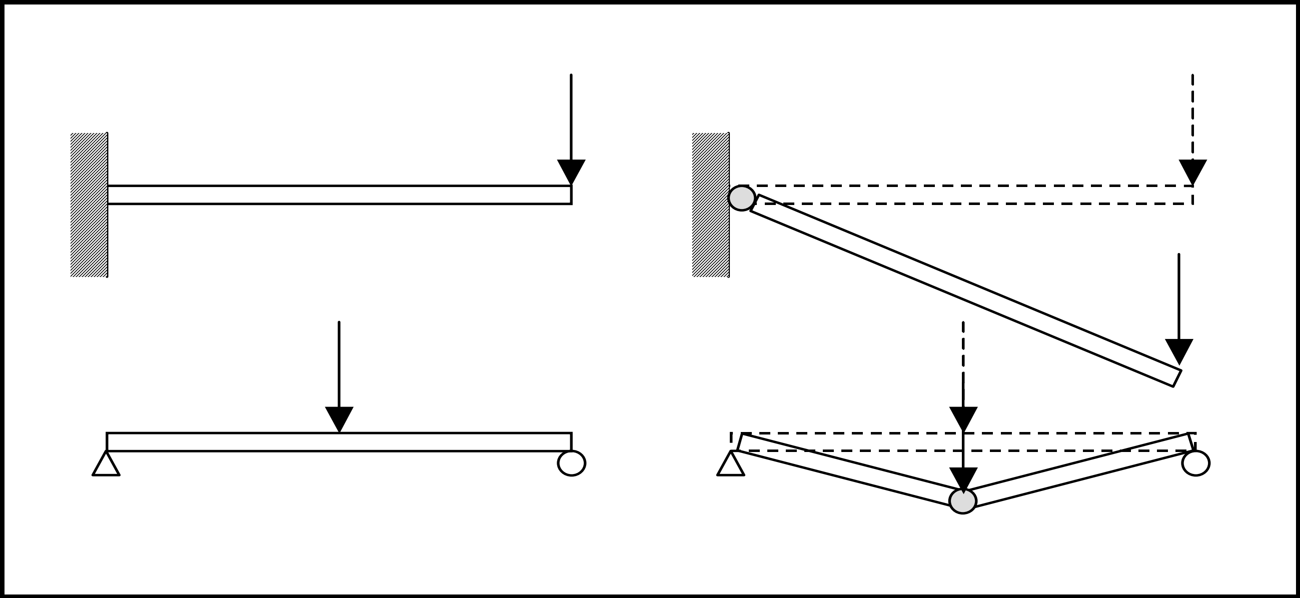
$$M_p = f_y A_1 d = f_y \sum_{n.a} A_i d_i$$

Plastic Hinge Development



Plastic Hinge Examples

- stability can be effected



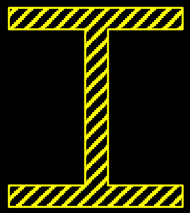
Plastic Section Modulus

- *shape factor, k*

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

= 3/2 for a rectangle

≈ 1.1 for an I



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

- *plastic modulus, Z*

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

Shear

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

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 = $t_w d$

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

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

Lateral Torsional Buckling

$$M_n = C_b\left[constant\right] \le M_p$$

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

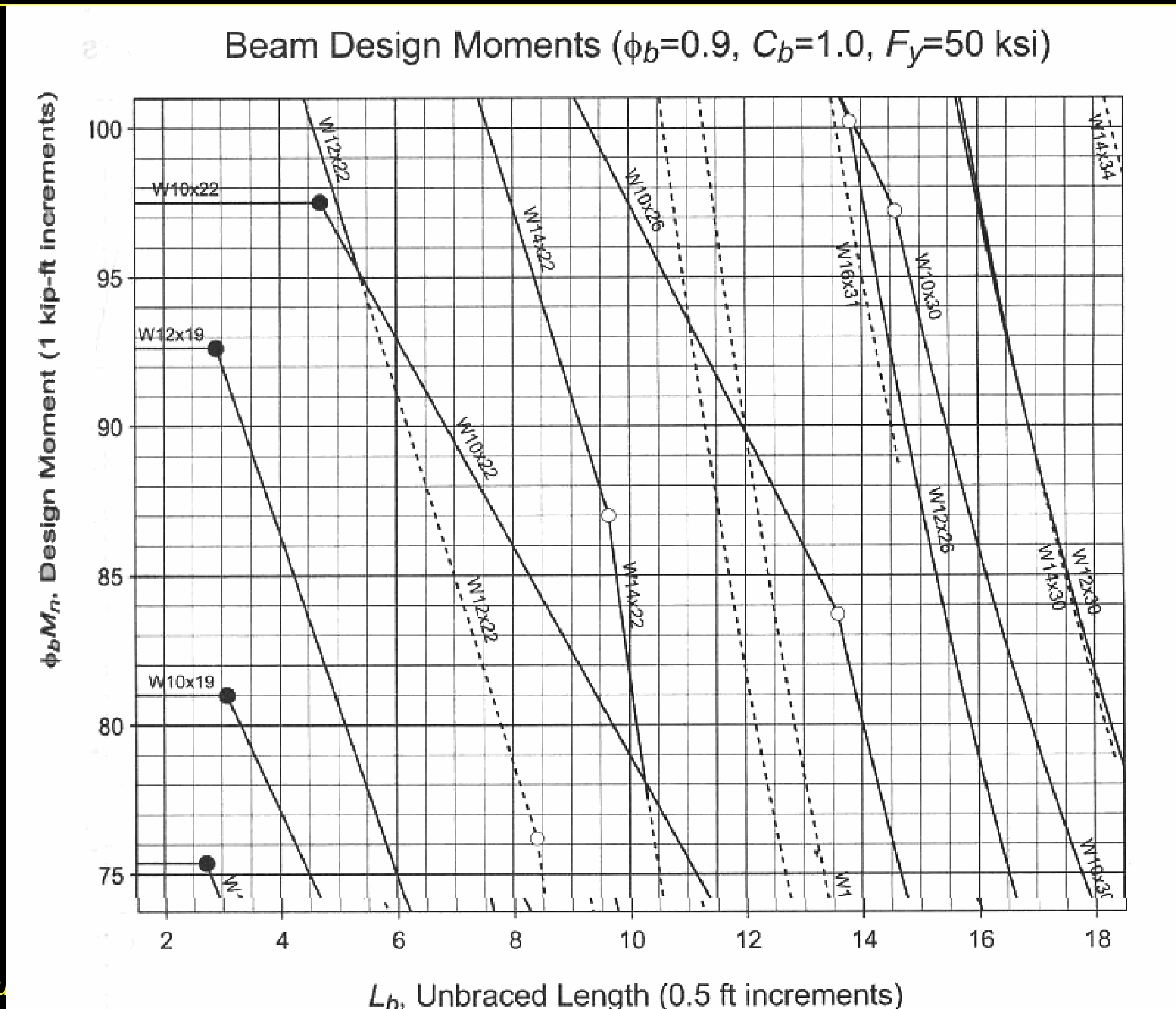
M_{max} - |max moment|, unbraced segment

M_A - |moment|, 1/4 point

M_B = |moment|, center point

M_C = |moment|, 3/4 point

Beam Design Charts



Deflection Limits

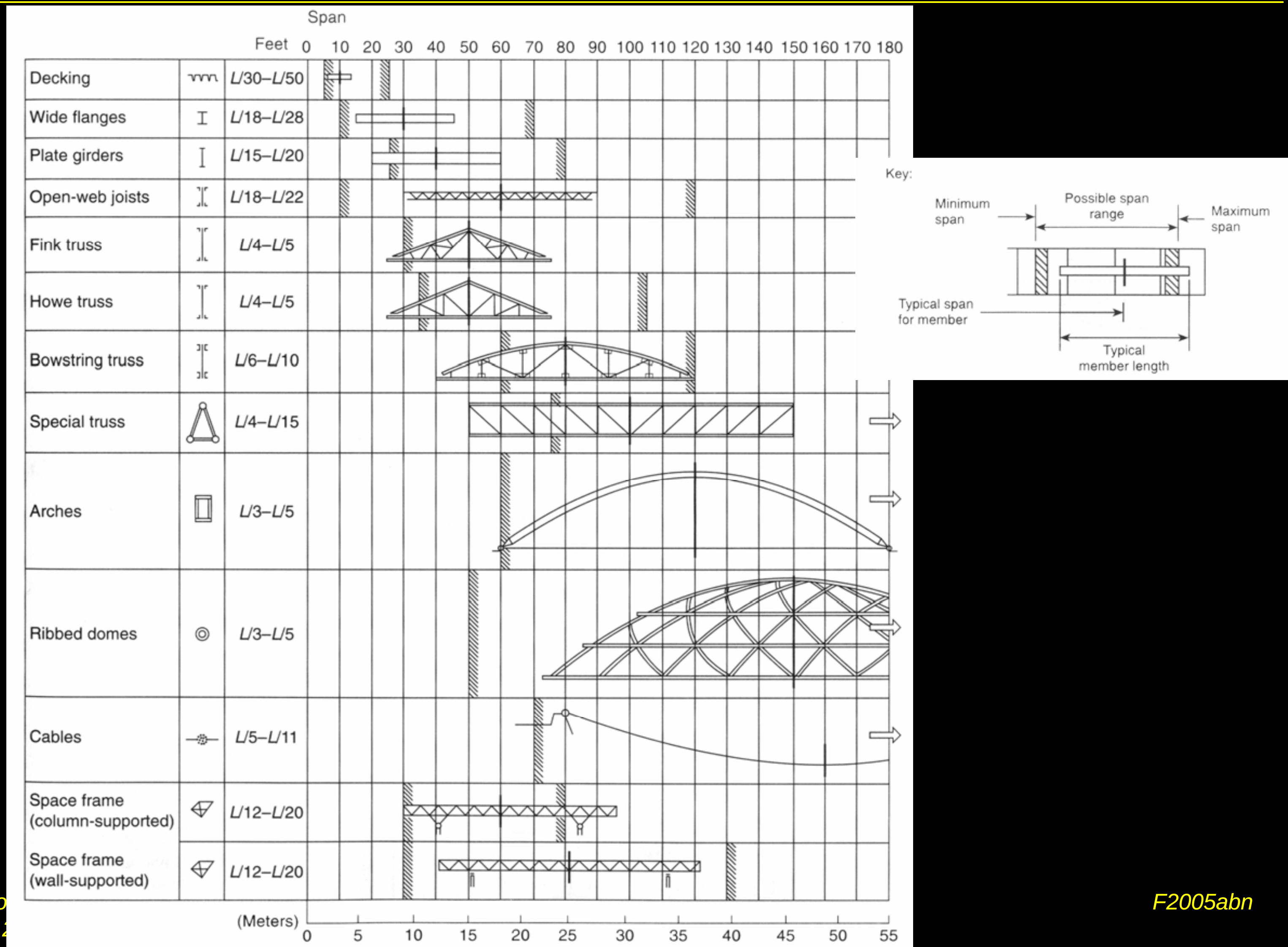
- *based on service condition*
- *no “impairment” to serviceability*
- *avoid ponding*
- *L/360 due to live load for beams & girders supporting plaster (ASD)*

Steel Arches and Frames

- solid sections
  or open web

Steel Shell and Cable Structures

Approximate Depths



ASD Column Design

- allowable stress

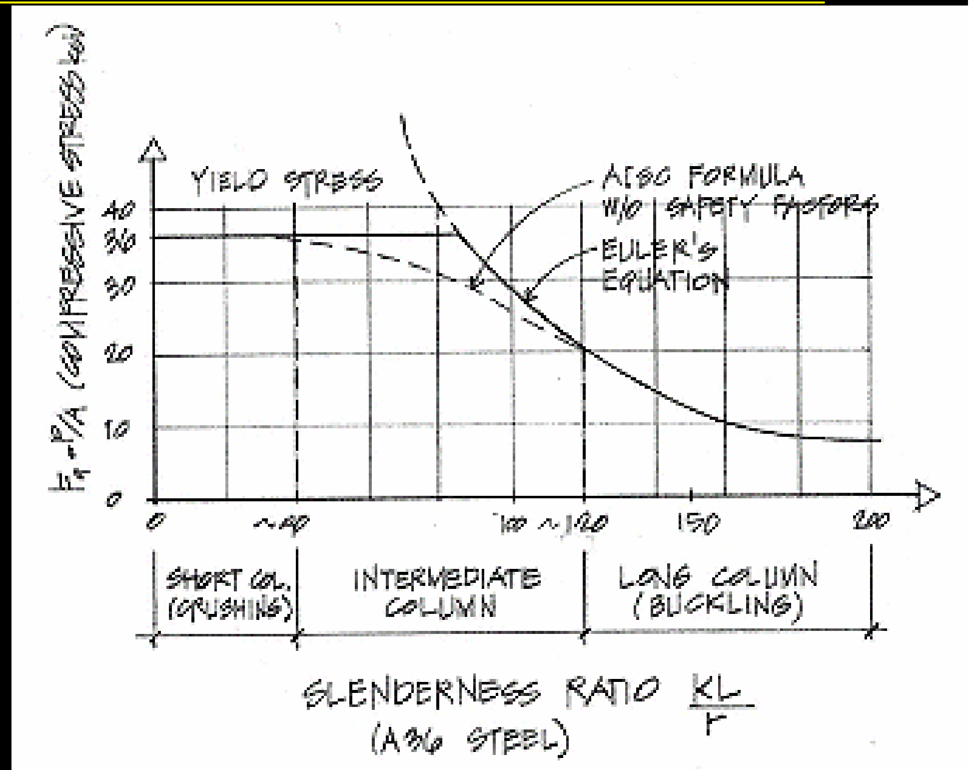
$$F_a = \frac{f_{critical}}{F.S.} = \frac{12\pi^2 E}{23\left(Kl/r\right)^2}$$

- slenderness ratio $\dfrac{Kl}{r}$

  – for kl/r $\geq C_c$ = 126.1 with F_y = 36 ksi
  = 107.0 with F_y = 50 ksi

C_c and Euler's Formula

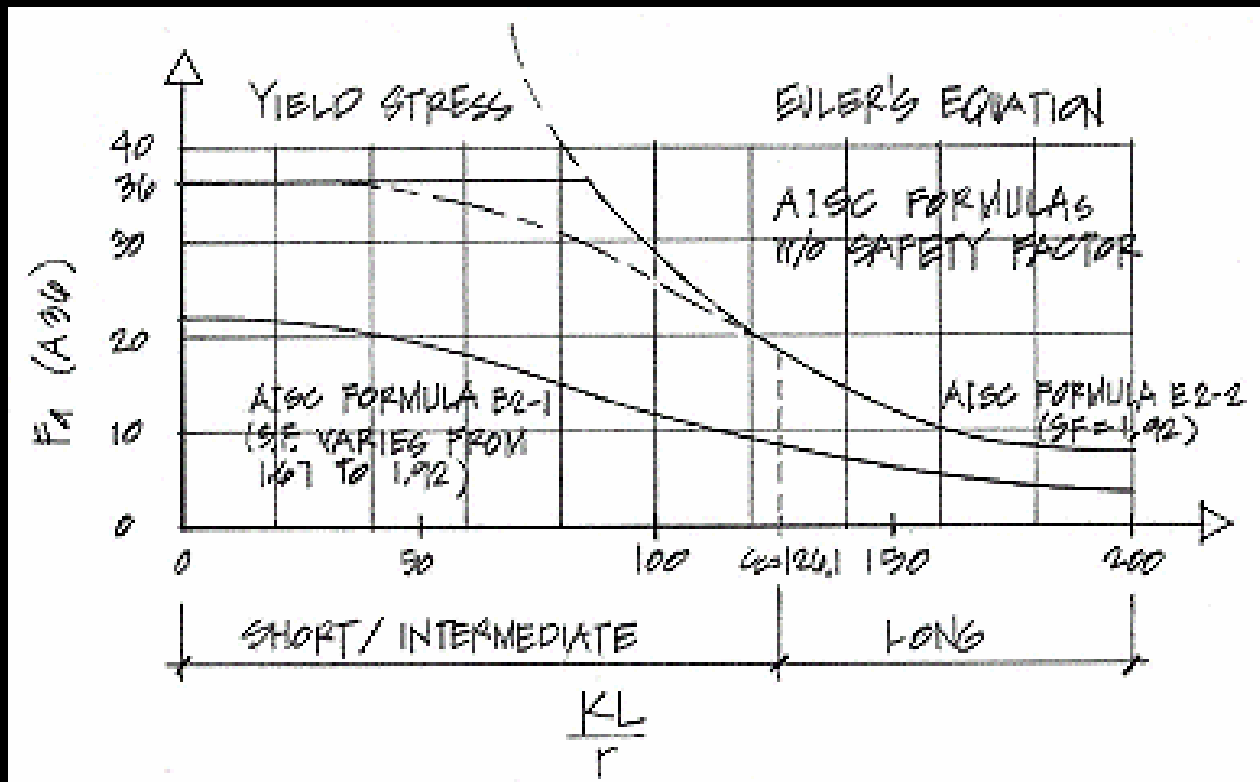
- *$Kl/r < C_c$*
  - *short and stubby*
  - *parabolic transition*



- *$Kl/r > C_c$*
  - *Euler's relationship*
  - *< 200 preferred*

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

C_c and Euler's Formula



Short / Intermediate

- $L_e/r < C_c$

$$F_a = \left[1 - \frac{\left(Kl/r\right)^2}{2C_c^2}\right]\frac{F_y}{F.S.}$$

  - where

$$F.S. = \frac{5}{3} + \frac{3\left(Kl/r\right)}{8C_c} - \frac{\left(Kl/r\right)^3}{8C_c^3}$$

Procedure

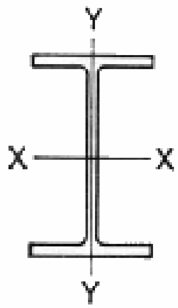
1. calculate C_c
2. determine limiting criteria
   – strength: $K\ell/r_{min} < C_c$
   – buckling: $K\ell/r_{min} \geq C_c$
3. get a section
   – if P & A known, set stress at limit
      • solve for $K\ell$, ℓ
   – if P & $K\ell$ known,
      • find r or A or I
4. continue from 3 until F_a satisfied

Column Charts

Table C-50
Allowable Stress
For Compression Members of 50-ksi Specified Yield Stress Steel[a]

$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)
1	29.94	41	25.69	81	18.81	121	10.20	161	5.76
2	29.87	42	25.55	82	18.61	122	10.03	162	5.69
3	29.80	43	25.40	83	18.41	123	9.87	163	5.62
4	29.73	44	25.26	84	18.20	124	9.71	164	5.55
5	29.66	45	25.11	85	17.99	125	9.56	165	5.49
6	29.58	46	24.96	86	17.79	126	9.41	166	5.42
7	29.50	47	24.81	87	17.58	127	9.26	167	5.35
8	29.42	48	24.66	88	17.37	128	9.11	168	5.29
9	29.34	49	24.51	89	17.15	129	8.97	169	5.23
10	29.26	50	24.35	90	16.94	130	8.84	170	5.17
11	29.17	51	24.19	91	16.72	131	8.70	171	5.11
12	29.08	52	24.04	92	16.50	132	8.57	172	5.05
13	28.99	53	23.88	93	16.29	133	8.44	173	4.99
14	28.90	54	23.72	94	16.06	134	8.32	174	4.93
15	28.80	55	23.55	95	15.84	135	8.19	175	4.88
16	28.71	56	23.39	96	15.62	136	8.07	176	4.82
17	28.61	57	23.22	97	15.39	137	7.96	177	4.77

Column Charts



COLUMNS

W shapes

Allowable axial loads in kips

F_y = 36 ksi

F_y = 50 ksi

Designation		W8											
Wt./ft.		67		58		48		40		35		31	
F_y		36	50	36	50	36	50	36	50	36	50	36	50
spect to least radius of gyration r_y	0	426	591	369	513	305	423	253	351	222	309	197	274
	6	387	525	336	455	276	375	229	310	201	272	178	241
	7	379	510	328	442	270	363	223	300	197	264	174	234
	8	370	494	320	428	263	352	218	290	191	255	170	226
	9	360	477	312	413	256	339	212	279	186	246	165	217
	10	350	459	303	397	249	326	205	268	180	236	160	208
	11	339	440	293	380	241	312	199	256	174	225	154	199
	12	328	420	283	363	233	297	192	244	168	214	149	189
	13	316	399	273	344	224	282	184	231	162	202	143	179
	14	304	378	263	325	215	266	177	217	155	190	137	168
	15	292	355	251	305	206	249	169	203	148	177	131	156
	16	279	331	240	284	196	232	160	188	141	164	124	145
	17	265	307	228	263	186	214	152	172	133	150	117	132

LRFD Column Design

- *limit states for failure* $$P_u \le \phi_c P_n$$

$$\phi_c = 0.85 \quad P_n = F_{cr} A_g$$

1. *yielding* $\lambda_c \le 1.5$
2. *buckling* $\lambda_c > 1.5$

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} \qquad L_e/r$$

λ_c – column slenderness parameter

A_g - gross area

Compact Sections

- *flanges continuously connected to the web or webs and width-thickness rations < limiting values*
  - *no local buckling of flange or web*

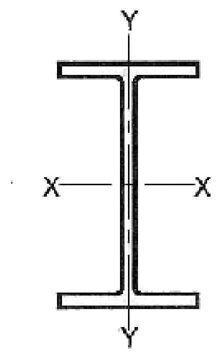
  - *for* $\lambda_c \le 1.5$ $$F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y$$
  - *for* $\lambda_c > 1.5$ $$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y$$

Column Charts

$F_y = 50$ ksi

$\phi_c P_n = 0.85 F_{cr} A_g$

Table 4-2 (cont.).
W-Shapes
Design Strength in Axial Compression, $\phi_c P_n$, kips

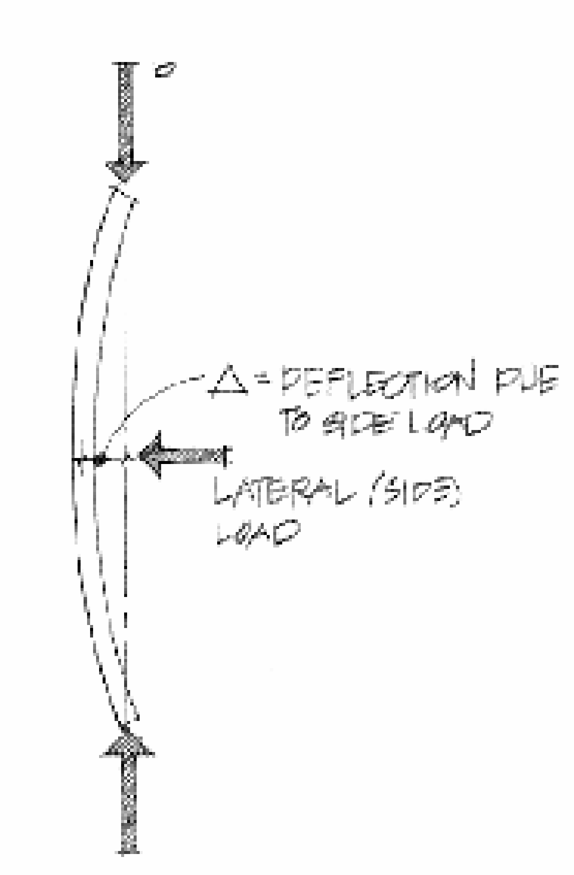


Shape		W12×										
		106	96	87	79	72	65††	58	53	50	45	40
adius of gyration r_y	0	1330	1200	1090	986	897	812	723	663	621	557	497
	6	1280	1150	1050	947	861	779	680	623	562	504	450
	7	1260	1140	1030	933	848	767	666	610	543	486	434
	8	1240	1120	1010	917	834	754	649	594	521	466	416
	9	1210	1100	994	900	818	739	631	577	497	445	396
	10	1190	1070	973	880	800	723	611	559	472	422	376
	11	1160	1050	950	860	781	706	590	539	445	398	354
	12	1130	1020	926	838	761	687	568	518	418	374	332
	13	1100	995	901	814	740	668	545	496	390	349	310
	14	1070	966	874	790	717	647	521	474	363	324	287

Eccentric Loading Stress Limit

- *in reality, as the column flexes, the moment increases*

- *P-Δ effect*



$$\frac{f_a}{F_a} + \frac{f_b \times (\text{Magnification factor})}{F_{bx}} \leq 1.0$$

Columns with Bending Design

- *ASD:*

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0$$

C_m – modification factor for end conditions

$= 0.6 - 0.4(M_1/M_2)$ or 0.85 restrained

F'_e – allowable buckling strength

() term – magnification factor for P-Δ

Columns with Bending Design

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

ϕ_c *- resistance factor for compression = 0.85*

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