ARCHITECTURAL STRUCTURES:
FORM, BEHAVIOR, AND DESIGN
ARCH 331
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SUMMER 2014

lecture
seventeen

Cor-Ten Steel Sculpture By Richard Serra
Museum of Modern Art Fort Worth, TX
(AISC - Steel Structures of the Everyday)

steel construction: columns & tension members

Structural Steel

- standard rolled shapes
  (W, C, L, T)
- tubing
- pipe
- built-up

Design Methods (revisited)

- know
  - loads or lengths
- select
  - section or load
  - adequate for strength and no buckling

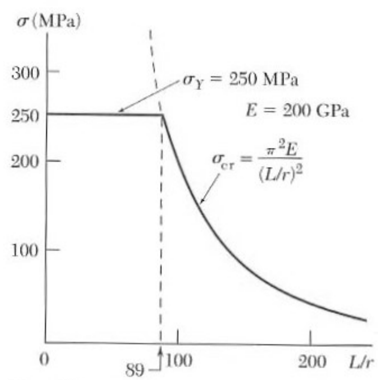


Fig. 10.9

Allowable Stress Design (ASD)

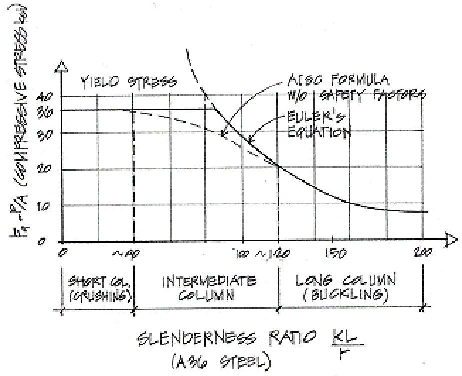
- AICS 9th ed

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

- slenderness ratio $\frac{Kl}{r}$
  - for $kl/r \geq C_c$ = 126.1 with F_y = 36 ksi
    = 107.0 with F_y = 50 ksi

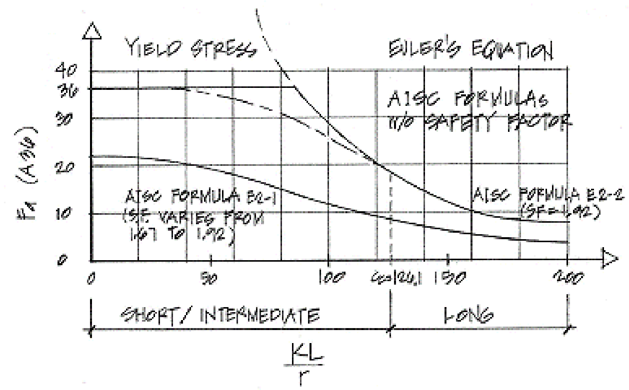
C_c and Euler's Formula

- *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*

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

  - *where*

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

Unified Design

- *limit states for failure*

$$P_a \le \frac{P_n}{\Omega}$$

$$\phi_c = 0.90 \quad P_n = F_{cr}A_g \quad P_u \le \phi_c P_n$$

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 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 $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ $\quad F_{cr} = 0.877F$
  - 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_a <u>or</u> F_{cr} from appropriate equation
   - tables are available
3. compute $P_{allowable} = F_a \cdot A$ <u>or</u> $P_n = F_{cr}A_g$
   - or find $f_{actual} = P/A$
4. is $P \le P_{allowable}$ ($P_a \le P_n/\Omega$)? <u>or</u> is $P_u \le \phi P_n$?
   - 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_a <u>or</u> F_{cr} from appropriate equations
   - or find a chart
4. compute $P_{allowable} = F_aA$ (or $P_n/\Omega = F_{cr}A$) <u>or</u> $P_n = F_{cr}A_g$
   - or find $f_{actual} = P/A$

Procedure for Design (cont'd)

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

Column Charts, F_a (pg. 361-364)

Table 10.1 Allowable stress for compression members (F_y = 36 ksi and F_y = 250 MPa).

$\frac{KL}{r}$	F_a (ksi)	F_a (MPa)	$\frac{KL}{r}$	F_a (ksi)	F_a (MPa)	$\frac{KL}{r}$	F_a (ksi)	F_a (MPa)
1	21.56	148.7	41	19.11	131.8	81	15.24	105.1
2	21.52	148.4	42	19.03	131.2	82	15.13	104.3
3	21.48	148.1	43	18.95	130.7	83	15.02	103.6
4	21.44	147.8	44	18.86	130.0	84	14.90	102.7
5	21.39	147.5	45	18.78	129.5	85	14.79	102.0
6	21.35	147.2	46	18.70	128.9	86	14.67	101.1
7	21.30	146.9	47	18.61	128.3	87	14.56	100.4
8	21.25	146.5	48	18.53	127.8	88	14.44	99.6
9	21.21	146.2	49	18.44	127.1	89	14.32	98.7
10	21.16	145.9	50	18.35	126.5	90	14.20	97.9
11	21.10	145.5	51	18.26	125.9	91	14.09	97.2
12	21.05	145.1	52	18.17	125.3	92	13.97	96.3
13	21.00	144.8	53	18.08	124.7	93	13.84	95.4
14	20.95	144.5	54	17.99	124.0	94	13.72	94.6
15	20.89	144.0	55	17.90	123.4	95	13.60	93.8
16	20.83	143.6	56	17.81	122.8	96	13.48	92.9
17	20.78	143.3	57	17.71	122.1	97	13.35	92.0
18	20.72	142.9	58	17.62	121.5	98	13.23	91.2

Column Charts, ϕF_{cr}

Available Critical Stress, $\phi_c F_{cr}$, for Compression Members, ksi (F_y = 50 ksi and ϕ_c = 0.90)

KL/r	$\phi_c F_{cr}$	KL/r	$\phi_c F_{cr}$	KL/r	$\phi_c F_{cr}$	KL/r	$\phi_c F_{cr}$	KL/r	$\phi_c F_{cr}$
1	45.0	41	39.8	81	27.9	121	15.4	161	8.72
2	45.0	42	39.6	82	27.5	122	15.2	162	8.61
3	45.0	43	39.3	83	27.2	123	14.9	163	8.50
4	44.9	44	39.1	84	26.9	124	14.7	164	8.40
5	44.9	45	38.8	85	26.5	125	14.5	165	8.30
6	44.9	46	38.5	86	26.2	126	14.2	166	8.20
7	44.8	47	38.3	87	25.9	127	14.0	167	8.10
8	44.8	48	38.0	88	25.5	128	13.8	168	8.00
9	44.7	49	37.8	89	25.2	129	13.6	169	7.91
10	44.7	50	37.5	90	24.9	130	13.4	170	7.82
11	44.6	51	37.2	91	24.6	131	13.2	171	7.73
12	44.5	52	36.9	92	24.2	132	13.0	172	7.64
13	44.4	53	36.6	93	23.9	133	12.8	173	7.55
14	44.4	54	36.4	94	23.6	134	12.6	174	7.46
15	44.3	55	36.1	95	23.3	135	12.4	175	7.38
16	44.2	56	35.8	96	22.9	136	12.2	176	7.29
17	44.1	57	35.5	97	22.6	137	12.0	177	7.21
18	43.9	58	35.2	98	22.3	138	11.9	178	7.13
19	43.8	59	34.9	99	22.0	139	11.7	179	7.05
20	43.7	60	34.6	100	21.7	140	11.5	180	6.97
21	43.6	61	34.3	101	21.3	141	11.4	181	6.90
22	43.4	62	34.0	102	21.0	142	11.2	182	6.82
23	43.3	63	33.7	103	20.7	143	11.0	183	6.75
24	43.1	64	33.4	104	20.4	144	10.9	184	6.67
25	43.0	65	33.0	105	20.1	145	10.7	185	6.60

Column Charts

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 $\quad P_{e1} = \dfrac{\pi^2 EA}{\left(\frac{Kl}{r}\right)^2}$

Beam-Column Design

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

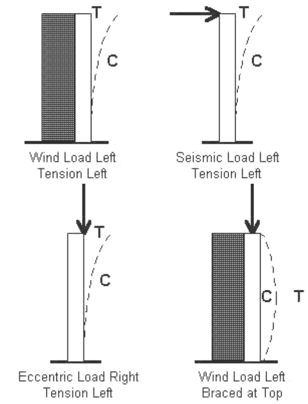
  P_r is required, P_c is capacity

  ϕ_c - resistance factor for compression = 0.9

  ϕ_b - resistance factor for bending = 0.9

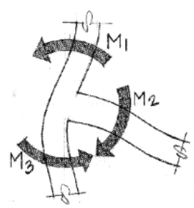
Design Steps Knowing Loads (revisited)

1. assume limiting stress
   - buckling, axial stress, combined stress
2. solve for r, A or S
3. pick trial section
4. analyze stresses
5. section ok?
6. stop when section is ok



Rigid Frame Design (revisited)

- columns in frames
  - ends can be "flexible"
  - stiffness affected by beams <u>and</u> column = EI/L

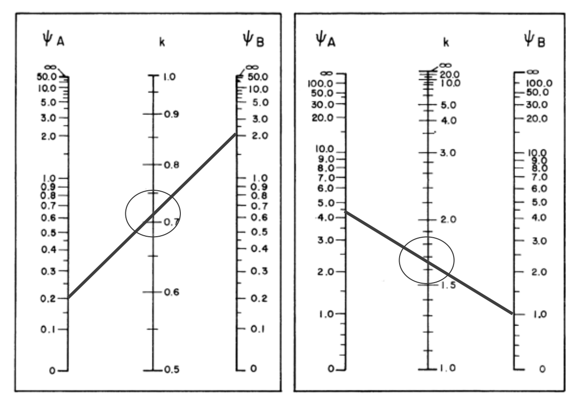


$$G = \Psi = \frac{\Sigma EI/l_c}{\Sigma EI/l_b}$$

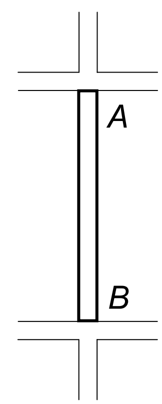
  - for the joint
    - l_c is the column length of each column
    - l_b is the beam length of each beam
    - measured center to center

Rigid Frame Design (revisited)

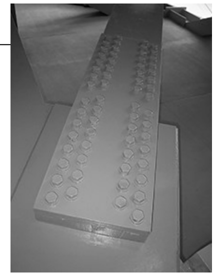
- column effective length, k



(a) Nonsway Frames (b) Sway Frames



Tension Members

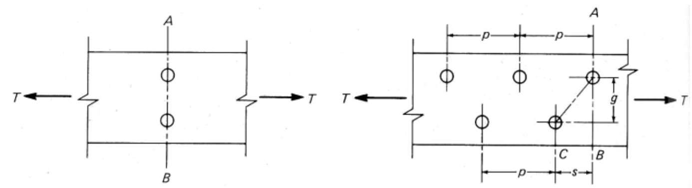


(AISC - Steel Structures of the Everyday)

- steel members can have <u>holes</u>
- reduced area

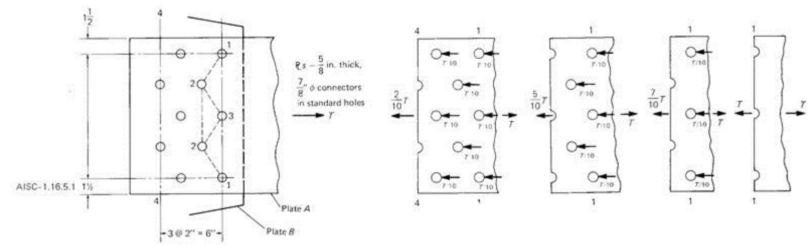
$$A_n = A_g - A_{of\ all\ holes} + t\Sigma \frac{s}{4g}$$

- increased stress



Effective Net Area

- likely path to "rip" across
- bolts divide transferred force too
- shear lag $\quad A_e \leq A_n U$



Tension Members

- limit states for failure $\quad P_a \leq P_n / \Omega \quad P_u \leq \phi_t P_n$

1. yielding $\phi_t = 0.90 \quad P_n = F_y A_g$
2. rupture* $\phi_t = 0.75 \quad P_n = F_u A_e$

A_g - gross area

A_e - effective net area (holes 1/8" + d)

F_u = the tensile strength of the steel (ultimate)