CE 415
DESIGN OF STEEL STRUCTURES

LECTURE 13
COMPRESSION MEMBER

SEMESTER: SPRING 2021

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OUTLINE

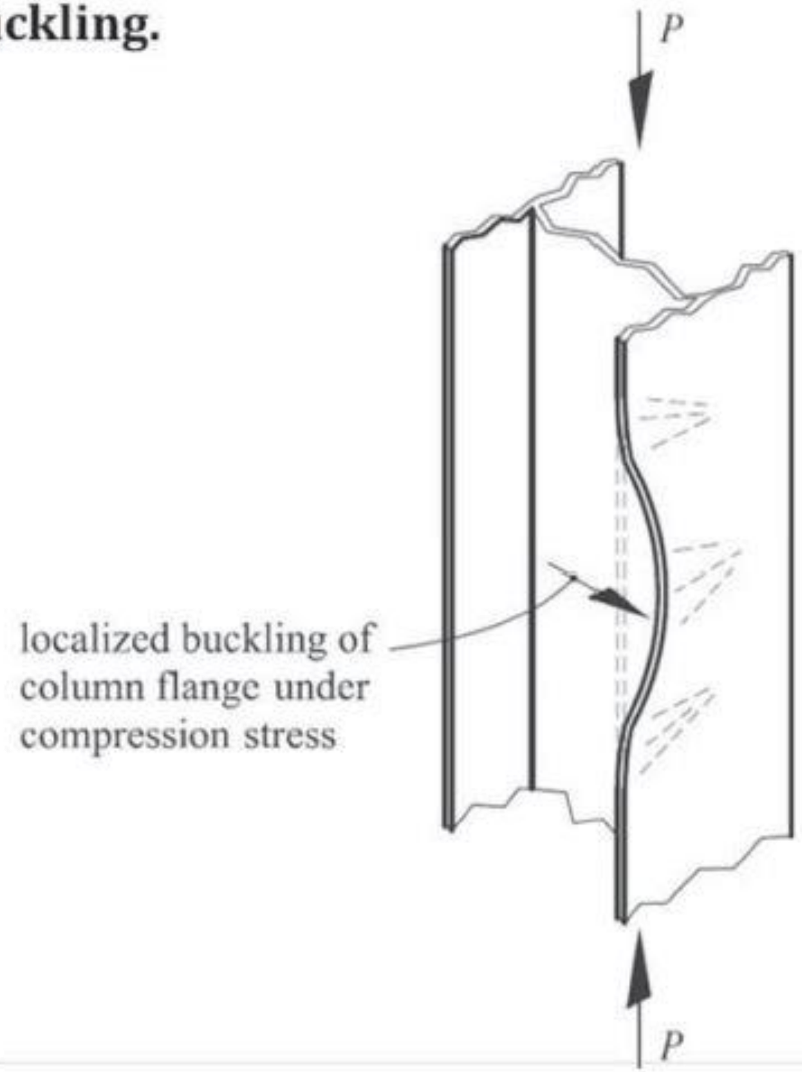
- Local buckling
- Column Biaxial bending
- Biaxial bending math problem

LOCAL BUCKLING

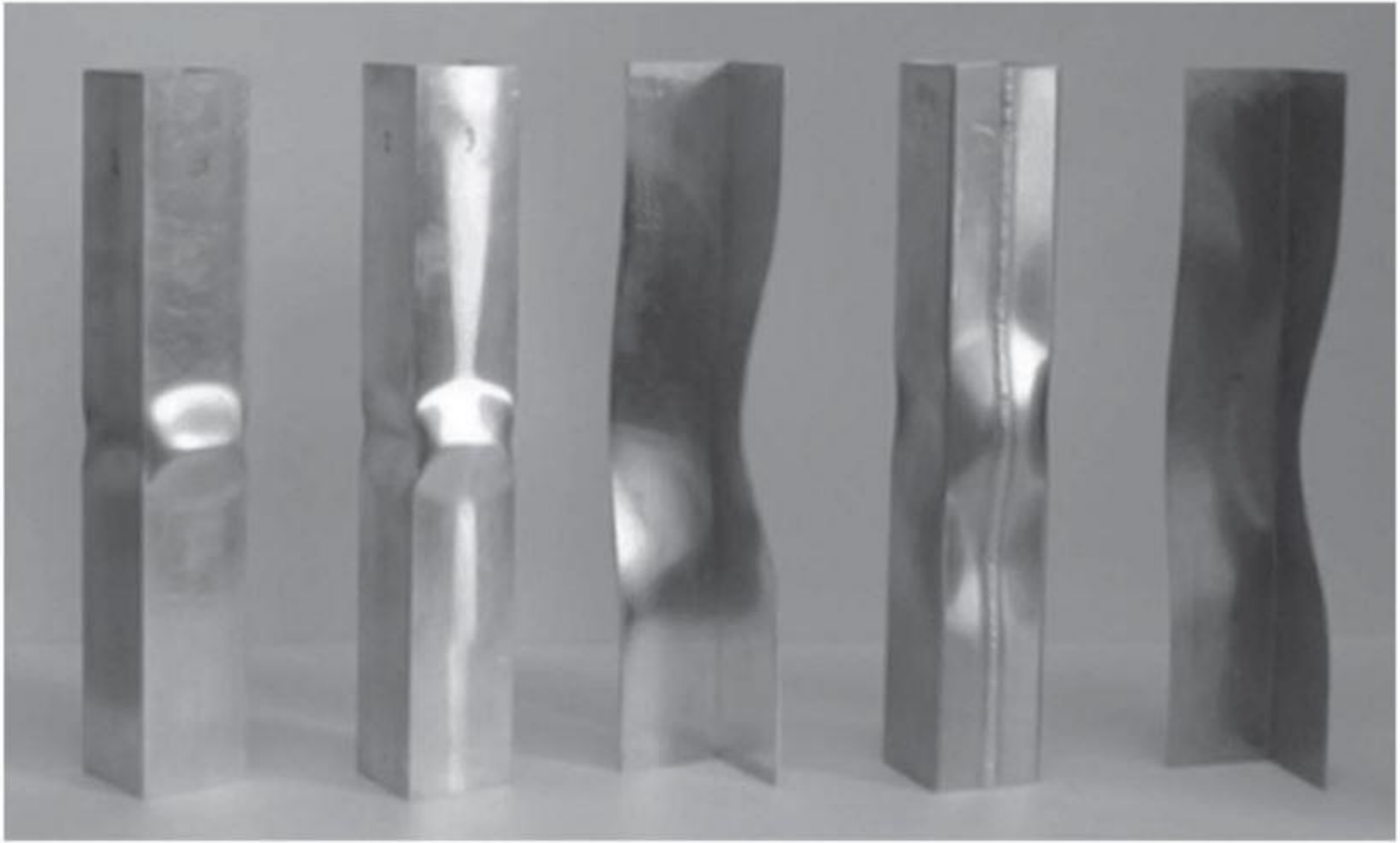
Local buckling is a phenomenon by which a portion of the section of a column or beam buckles instead of overall buckling.

Local buckling leads to a reduction in the strength of a compression member and prevents the member from reaching its overall compression capacity.

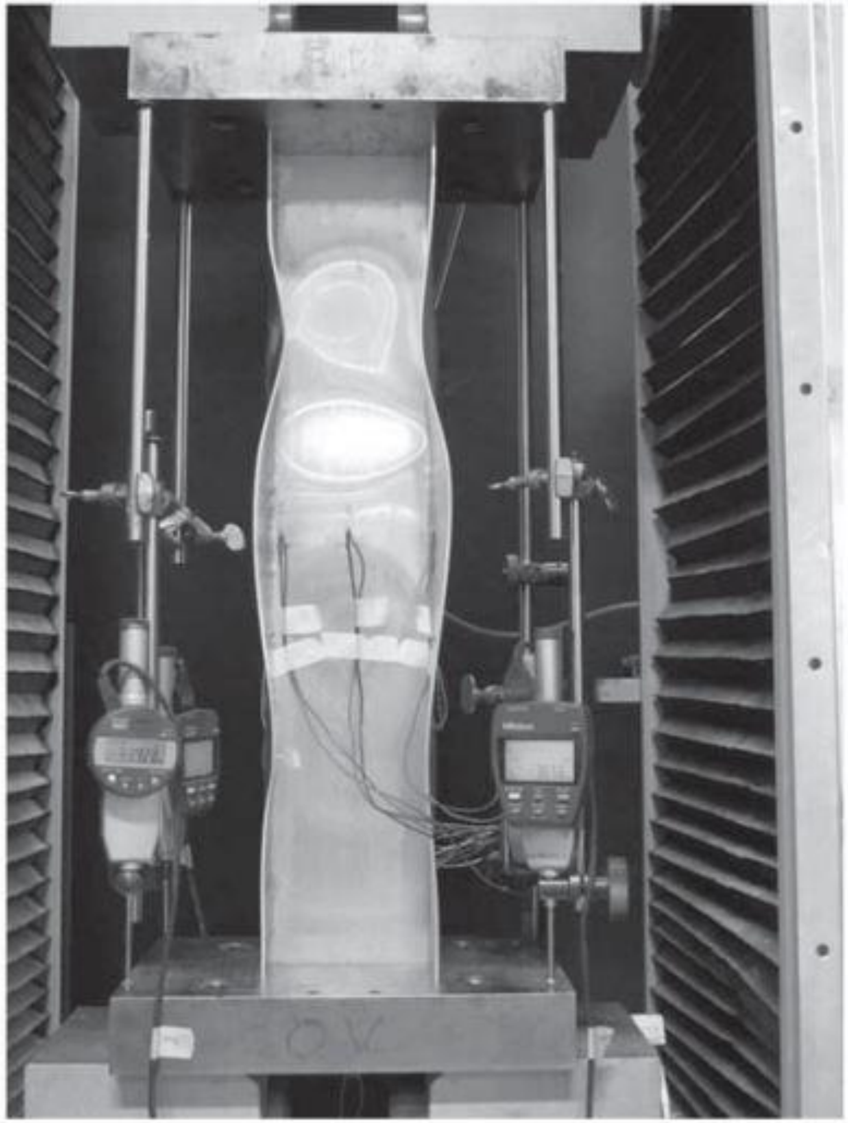
To avoid or prevent local buckling, the AISC specification prescribes limits to the width-to-thickness ratios of the plate components that make up the structural member.



LOCAL BUCKLING



LOCAL BUCKLING



Local buckling primarily depends on the ratio, *b/t*, of the width (*b*) and thickness (*t*) of the plate elements that builds up a section.

LOCAL BUCKLING

Based on the width/thickness ratio steel sections are defined as

Compact: A compact section reaches its cross-sectional material strength, or capacity, before local buckling occurs.

Non-Compact: In a non-compact section, only a portion of the cross-section reaches its yield strength before local buckling occurs.

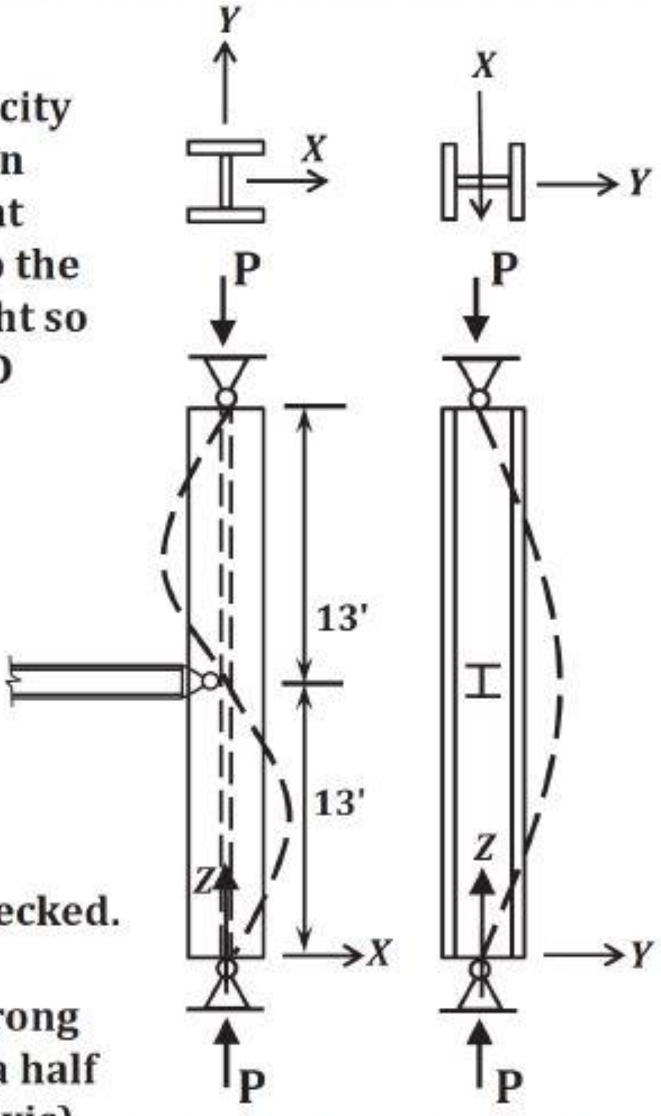
Slender: In a slender section, the cross-section does not yield and the strength of the member is governed by local buckling.

The use of slender sections as compression members is not efficient or economical; therefore, the use of slender section in design practice is not recommended.

Designation	Nom. Weight, lb/ft	Area, A, in^2	Depth, d, in	Flange		Web Thickness t_w, in	Axis X-X			Axis Y-Y		
				Width, b_f, in	Thickness, t_f, in		I, in^4	S, in^3	r, in	I, in^4	S, in^3	r, in
W10 x 68	68	20	10.4	10.13	0.77	0.47	394	75.7	4.44	134	26.4	2.59
W10 x 60	60	17.6	10.22	10.08	0.68	0.42	341	66.7	4.39	116	23	2.57
W10 x 54	54	15.8	10.09	10.03	0.615	0.37	303	60	4.37	103	20.6	2.56
W10 x 49	49	14.4	9.98	10	0.56	0.34	272	54.6	4.35	93.4	18.7	2.54
W10 x 45	45	13.3	10.1	8.02	0.62	0.35	248	49.1	4.32	53.4	13.3	2.01
W10 x 39	39	11.5	9.92	7.985	0.53	0.315	209	42.1	4.27	45	11.3	1.98
W10 x 33	33	9.71	9.73	7.96	0.435	0.29	170	35	4.19	36.6	9.2	1.94
W10 x 30	30	8.84	10.47	5.81	0.51	0.3	170	32.4	4.38	16.7	5.75	1.37
W10 x 26	26	7.61	10.33	5.77	0.44	0.26	144	27.9	4.35	14.1	4.89	1.36
W10 x 22	22	6.49	10.17	5.75	0.36	0.24	118	23.2	4.27	11.4	3.97	1.33

EXAMPLE:

Determine the allowable compressive load carrying capacity of the column shown in Fig. It consists of W10×45 section having A992 (F_y = 50 ksi) steel. There are hinge support at top and bottom that allows rotation in any direction. Also the column has weak direction support (braced) at mid-height so that lateral deflection is prevented in x direction. Use ASD approach.



SOLUTION:

For W10×45 section, from AISC Manual Chart we have

A = 13.3 in^2, r_x=4.32 in., r_y=2.01 in.

$x \rightarrow$ strong axis

$y \rightarrow$ weak axis

Column length, L = (13×2) ×12 = 312 in.

Possibility of buckling in both x and y directions to be checked.

Buckling in y direction causes bending about x axis or strong axis. For strong axis buckling, the buckling shape is like a half sine wave over full column length. Thus for strong (or x axis) axis buckling, K_x = 1.0

$\therefore K_xL/r_x = 1.0 \times 312/4.32 = 72.22$

$\therefore F_{ex} = \pi^2 E/(K_xL/r_x)^2 = 3.14^2 \times 29000/(72.2)^2 = 54.82$ ksi. ($> F_y$, note)

And $4.71\sqrt{(E/F_y)} = 4.71\sqrt{(29000/50)} = 113.43 \therefore K_xL/r_x < 4.71\sqrt{(E/F_y)}$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = [0.658^{(50/54.82)}]50 = 34.13 \text{ ksi}$$

Nominal strength for x-axis buckling $P_{nx} = F_{cr} A_g = 34.13 \times 13.3 = 454$ kip

Buckling in *x* direction causes bending about *y* axis or weak axis. For weak axis buckling, the buckling shape is like a full sine wave over full column length. Thus for weak (or *y* axis) axis buckling, $K_y = 0.5$

$\therefore K_yL/r_y = 0.5 \times 312/2.01 = 77.61$

$\therefore F_{ey} = \pi^2 E/(K_yL/r_y)^2 = 3.14^2 \times 29000/(77.61)^2 = 47.47$ ksi.

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = [0.658^{(50/47.47)}]50 = 32.17 \text{ ksi}$$

Nominal strength for y axis buckling $P_{ny} = F_{cr} A_g = 32.17 \times 13.3 = 427.9$ kip

$\therefore P_n =$ smaller of P_{nx} and $P_{ny} = 427.9$ kip

$\therefore$ Allowable strength $P = P_n/\Omega = 427.9/1.67 = 256.2$ kip