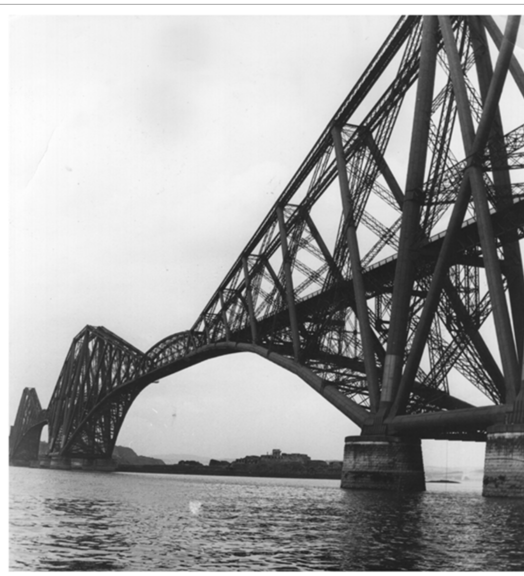
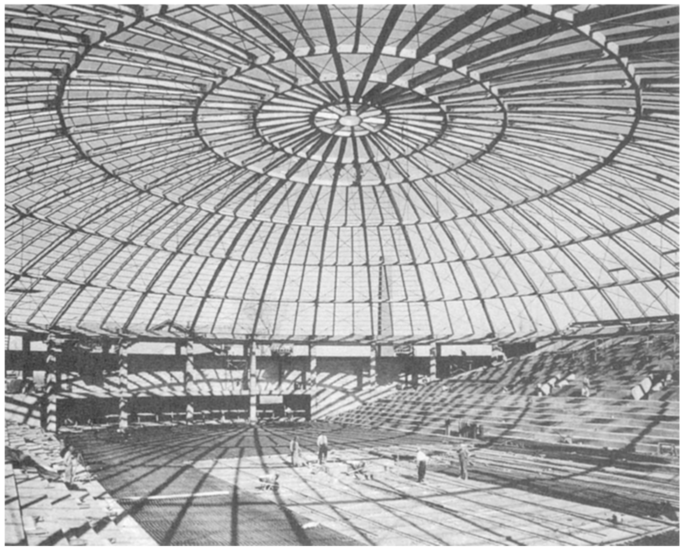
STEEL DESIGN

Chapter 1:
Introduction to Structural Steel Design

Introduction



Introduction



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Introduction

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Advantages of Steel as a Structural Material

1. *High Strength:*

- The weight of structure that is made of steel will be small.

2. *Uniformity:*

- Properties of steel do not change as oppose to concrete.
- Homogenous Material.
- Isotropic Material (E, G, v)

Advantages of Steel as a Structural Material

3. *Elasticity:*

- Steel follows Hooke's law up to fairly high stresses.
- Moment of Inertia (I) can be accurately calculated.

4. *Ductility:*

- Steel can withstand extensive deformation without failure under high tensile stresses, i.e. it gives warnings before failure takes place.

Advantages of Steel as a Structural Material

4. *Ductility:*

- Measure of Ductility:

$$e = \frac{L_f - L_0}{L_0} \times 100$$

where

e = elongation (expressed as a percent)

L_f = length of the specimen at fracture

L_0 = original length

- *Or :* $a = A_0 – A_f / A_0 \times 100$

where

a = Reduction of Area (expressed as a percent)

A_0 = Original Area

A_f = Area of specimen at fracture

Advantages of Steel as a Structural Material

5. *Permanence:*

- Steel frames that are properly maintained will last indefinitely.

6. *Toughness:*

- The ability of material to absorb energy in large amounts.

7. *Additions to existing structures:*

Disadvantages of Steel as a Structural Material

1. *Maintenance costs:*

- Steel structures are susceptible to corrosion. It must be painted periodically.

2. *Fire proofing costs:*

- The strength of the steel is reduced at high temperatures due to fires.

3. *Susceptibility to Buckling.*

4. *Susceptibility to fatigue under repeated load.*

Types of Steel

- Steel is composed almost entirely of iron, but contains small amounts of other elements such as Carbon, Manganese, Silicon, Copper, … etc.

- *Increasing the Carbon will* :

1. Increase the strength and hardness.
2. Decrease the ductility and toughness.

Types of Steel

1. Carbon Steels:

i. Low Carbon Steel (Carbon < 0.15%)
ii. Mild Carbon (Structural) (Carbon 0.15%-0.29%)
iii. Medium Carbon Steel (Carbon 0.3%-0.59%)
iv. High Carbon Steel (Carbon 0.6%-1.7%)

- The Carbon steels A36, A53, A500, A 501 and A529

Types of Steel

2. High Strength Low Alloy Steel:

- Fy = 42-70 ksi
- Corrosive resistance.
- The High Strength Low Alloy Steel are A572, A618, A913 and A992.

Types of Steel

3. Corrosion Resistant High Strength Low Alloy Steel :

- Fy = 80-110 ksi
- Corrosive resistance.
- The Alloy Steels are A242, A588 and A847.

Types of Steel

Table 1.1
Applicable ASTM Specifications for Various Structural Shapes

Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series: W	M	S	HP	C	MC	L	HSS Rect.	HSS Round	Pipe
Carbon	A36		36	58–80[b]	▒	■	■	▒	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									■	
			46	58								■		
		Gr. C	46	62									▒	
			50	62								▒		
	A501		36	58								▒	▒	
	A529[c]	Gr. 50	50	65–100	▒	▒	▒	▒	▒	▒	▒			
		Gr. 55	55	70–100	▒	▒	▒	▒	▒	▒	▒			
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50	65[d]	▒	▒	▒	■	▒	▒	▒			
		Gr. 55	55	70	▒	▒	▒	▒	▒	▒	▒			
		Gr. 60[e]	60	75	▒	▒	▒	▒	▒	▒	▒			
		Gr. 65[e]	65	80	▒	▒	▒	▒	▒	▒	▒			
	A618[f]	Gr. I & II	50[g]	70[g]								▒	▒	
		Gr. III	50	65								▒	▒	
	A913	50	50[h]	60[h]	▒	▒	▒	▒	▒	▒	▒			
		60	60	75	▒	▒	▒	▒	▒	▒	▒			
		65	65	80	▒	▒	▒	▒	▒	▒	▒			
		70	70	90	▒	▒	▒	▒	▒	▒	▒			
	A992		50–65[i]	65[i]	■	▒	▒	▒	▒	▒	▒			
Corrosion-Resistant High-Strength Low-Alloy	A242		42[j]	63[j]	▒									
			46[k]	67[k]	▒			▒			▒			
			50[l]	70[l]	▒	▒	▒	▒	▒	▒	▒			
	A588		50	70	▒	▒	▒	▒	▒	▒	▒			
	A847		50	70								▒	▒	

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

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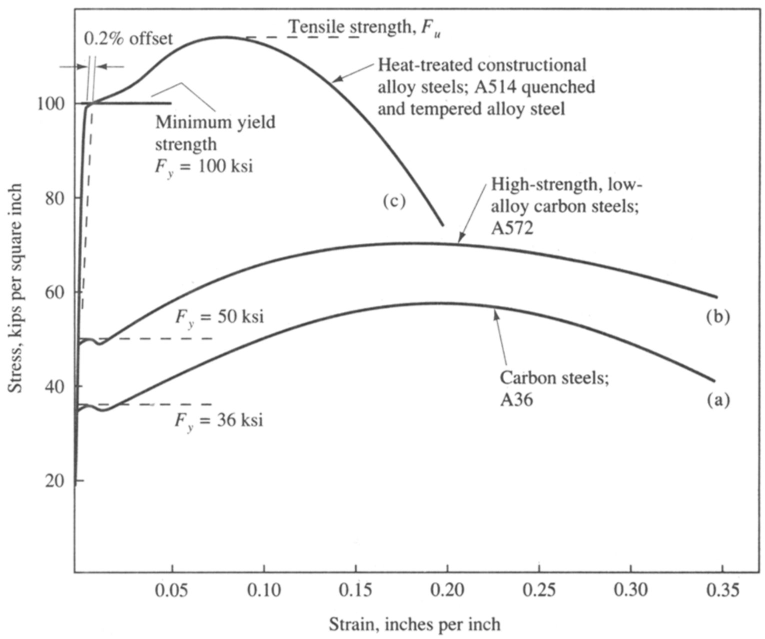

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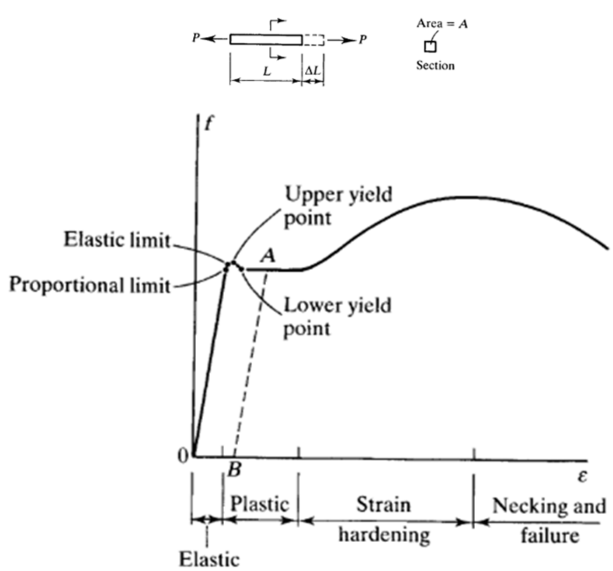
Stress-Strain Curves



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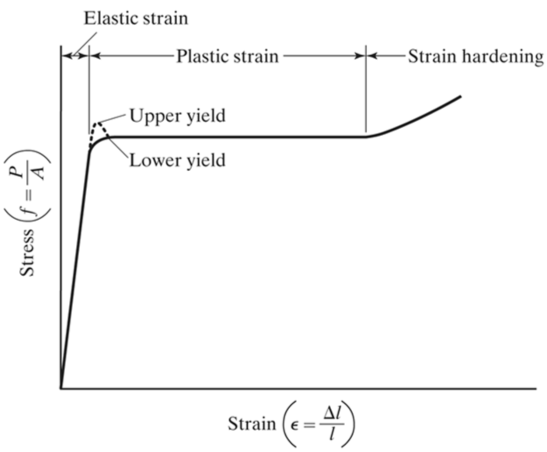
Stress-Strain Curves



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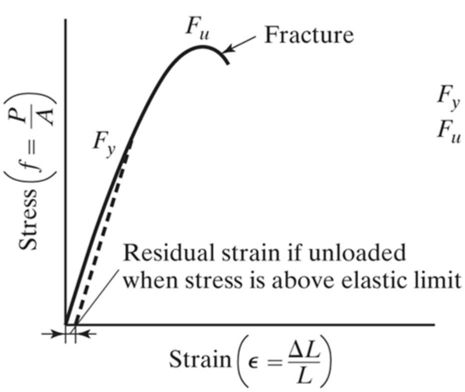
Stress-Strain Curves

- Brittle Steel:



F_y
F_u

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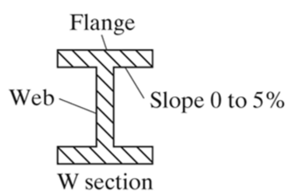
Steel Sections

I. <u>Rolled Sections:</u>

1. *Wide Flange Section (W)*:

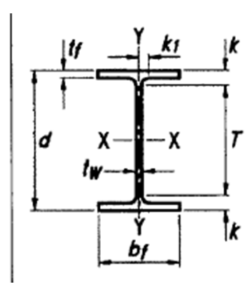
- Designation: **W dn X m**
- dn : Nominal Depth
- m : Weight per unit feet (Lb / ft)



Steel Sections

1. *Wide Flange Section (W)*:

- bf : Flange width
- tw : Thickness of web
- tf : Thickness of flange
- d : Depth



- Examples: W 21 X 201, W21 X 182 ……

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Steel Sections

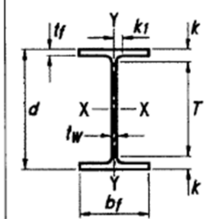


Table 1–1
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web: Thickness, t_w		Web: $\frac{t_w}{2}$	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k, k_{des}	Distance: k, k_{det}	Distance: k_1	Distance: T	Distance: Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W44×335[c]	98.5	44.0	44	1.03	1	1/2	15.9	16	1.77	1 3/4	2.56	2 5/8	1 5/16	38 3/4	5 1/2
×290[c]	85.4	43.6	43 5/8	0.865	7/8	7/16	15.8	15 7/8	1.58	1 9/16	2.36	2 7/16	1 1/4		
×262[c]	76.9	43.3	43 1/4	0.785	13/16	7/16	15.8	15 3/4	1.42	1 7/16	2.20	2 1/4	1 3/16		
×230[c,v]	67.7	42.9	42 7/8	0.710	11/16	3/8	15.8	15 3/4	1.22	1 1/4	2.01	2 1/16	1 3/16		
W40×593[h]	174	43.0	43	1.79	1 13/16	15/16	16.7	16 3/4	3.23	3 1/4	4.41	4 1/2	2 1/8	34	7 1/2
×503[h]	148	42.1	42	1.54	1 9/16	13/16	16.4	16 3/8	2.76	2 3/4	3.94	4	2		
×431[h]	127	41.3	41 1/4	1.34	1 5/16	11/16	16.2	16 1/4	2.36	2 3/8	3.54	3 5/8	1 7/8		

Steel Sections

Table 1–1 (continued)
W Shapes
Properties

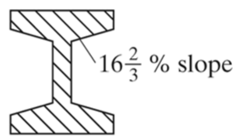
W44 – W40

Nominal Wt.	Compact Section Criteria: $\frac{b_f}{2t_f}$	Compact Section Criteria: $\frac{h}{t_w}$	Axis X-X: I	Axis X-X: S	Axis X-X: r	Axis X-X: Z	Axis Y-Y: I	Axis Y-Y: S	Axis Y-Y: r	Axis Y-Y: Z	r_{ts}	h_o	Torsional Properties: J	Torsional Properties: C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
335	4.50	38.0	31100	1410	17.8	1620	1200	150	3.49	236	4.24	42.3	74.7	535000
290	5.02	45.0	27000	1240	17.8	1410	1040	132	3.49	205	4.21	42.0	50.9	461000
262	5.57	49.6	24100	1110	17.7	1270	923	117	3.47	182	4.17	41.9	37.3	405000
230	6.45	54.8	20800	971	17.5	1100	796	101	3.43	157	4.13	41.7	24.9	346000
593	2.58	19.1	50400	2340	17.0	2760	2520	302	3.80	481	4.63	39.8	445	997000
503	2.98	22.3	41600	1980	16.8	2310	2040	249	3.72	394	4.50	39.3	277	789000
431	3.44	25.5	34800	1690	16.6	1960	1690	208	3.65	328	4.41	38.9	177	638000

Steel Sections

2. American Standard Beam (S):

- Designation: **S dn X m**
- dn : Nominal Depth
- m : Weight per unit feet (Lb / ft)

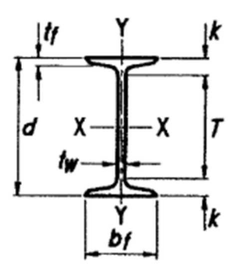


S beam

Steel Sections

2. American Standard Beam (S):

- bf : Flange width
- tw : Thickness of web
- tf : Thickness of flange
- d : Depth



- Examples: S 24 X 121 ……

Steel Sections

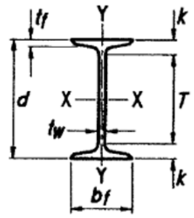


Table 1–3
S Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.
S24×121	35.5	24.5	24 1/2	0.800	13/16	7/16	8.05	8	1.09	1 1/16	2	20 1/2	4
×106	31.1	24.5	24 1/2	0.620	5/8	5/16	7.87	7 7/8	1.09	1 1/16	2	20 1/2	4
S24×100	29.3	24.0	24	0.745	3/4	3/8	7.25	7 1/4	0.870	7/8	1 3/4	20 1/2	4
×90	26.5	24.0	24	0.625	5/8	5/16	7.13	7 1/8	0.870	7/8	1 3/4	20 1/2	4
×80	23.5	24.0	24	0.500	1/2	1/4	7.00	7	0.870	7/8	1 3/4	20 1/2	4
S20×96	28.2	20.3	20 1/4	0.800	13/16	7/16	7.20	7 1/4	0.920	15/16	1 3/4	16 3/4	4
×86	25.3	20.3	20 1/4	0.660	11/16	3/8	7.06	7	0.920	15/16	1 3/4	16 3/4	4

Steel Sections

Table 1–3 (continued)
S Shapes
Properties

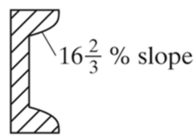
S SHAPES

Nom-inal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	Torsional Properties J	Torsional Properties C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
121	3.69	25.9	3160	258	9.43	306	83.0	20.6	1.53	36.3	1.94	23.4	12.8	11400
106	3.61	33.4	2940	240	9.71	279	76.8	19.5	1.57	33.4	1.93	23.4	10.1	10500
100	4.16	27.8	2380	199	9.01	239	47.4	13.1	1.27	24.0	1.66	23.1	7.59	6350
90	4.09	33.1	2250	187	9.21	222	44.7	12.5	1.30	22.4	1.66	23.1	6.05	5980
80	4.02	41.4	2100	175	9.47	204	42.0	12.0	1.34	20.8	1.67	23.1	4.89	5620
96	3.91	21.1	1670	165	7.71	198	49.9	13.9	1.33	24.9	1.71	19.4	8.40	4690
86	3.84	25.6	1570	155	7.89	183	46.6	13.2	1.36	23.1	1.71	19.4	6.65	4370

Steel Sections

3. American Standard Channels (C):

- Designation: **C dn X m**
- dn : Nominal Depth
- m : Weight per unit feet (Lb / ft)

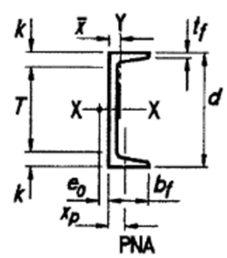


American standard channel

Steel Sections

3. American Standard Channels (C):

- bf : Flange width
- tw : Thickness of web
- tf : Thickness of flange
- d : Depth



- Examples: C 15 X 50 ……

Steel Sections

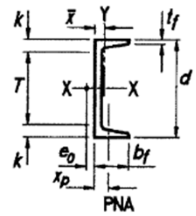


Table 1–5
C Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance T	Distance Workable Gage	r_{ts}	h_o
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
C15×50	14.7	15.0	15	0.716	11/16	3/8	3.72	3 3/4	0.650	5/8	1 7/16	12 1/8	2 1/4	1.17	14.4
×40	11.8	15.0	15	0.520	1/2	1/4	3.52	3 1/2	0.650	5/8	1 7/16	12 1/8	2	1.15	14.4
×33.9	10.0	15.0	15	0.400	3/8	3/16	3.40	3 3/8	0.650	5/8	1 7/16	12 1/8	2	1.13	14.4
C12×30	8.81	12.0	12	0.510	1/2	1/4	3.17	3 1/8	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	1.01	11.5
×25	7.34	12.0	12	0.387	3/8	3/16	3.05	3	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	1.00	11.5
×20.7	6.08	12.0	12	0.282	5/16	3/16	2.94	3	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	0.983	11.5
C10×30	8.81	10.0	10	0.673	11/16	3/8	3.03	3	0.436	7/16	1	8	1 3/4[g]	0.925	9.56

Steel Sections

Table 1–5 (continued)
C Shapes
Properties

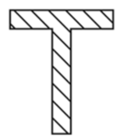
C SHAPES

Nominal Wt.	Shear Ctr, e_o	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y x̄	Axis Y-Y Z	Axis Y-Y x_p	Torsional J	Torsional C_w	Torsional $\bar{r}_o$	Torsional H
lb/ft	in.	in.⁴	in.³	in.	in.³	in.⁴	in.³	in.	in.	in.³	in.	in.⁴	in.⁶	in.	
50	0.583	404	53.8	5.24	68.5	11.0	3.77	0.865	0.799	8.14	0.490	2.65	492	5.49	0.937
40	0.767	348	46.5	5.45	57.5	9.17	3.34	0.883	0.778	6.84	0.392	1.45	410	5.73	0.927
33.9	0.896	315	42.0	5.62	50.8	8.07	3.09	0.901	0.788	6.19	0.332	1.01	358	5.94	0.920
30	0.618	162	27.0	4.29	33.8	5.12	2.05	0.762	0.674	4.32	0.367	0.861	151	4.54	0.919
25	0.746	144	24.0	4.43	29.4	4.45	1.87	0.779	0.674	3.82	0.306	0.538	130	4.72	0.909
20.7	0.870	129	21.5	4.61	25.6	3.86	1.72	0.797	0.698	3.47	0.253	0.369	112	4.93	0.899
30	0.368	103	20.7	3.42	26.7	3.93	1.65	0.668	0.649	3.78	0.441	1.22	79.5	3.63	0.922

Steel Sections

4. Structural Tees Split from W or S -Shapes (WT or ST)

- Designation: **WT dn X m or ST dn X m**
- dn : Nominal Depth
- m : Weight per unit feet (Lb / ft)



Tee

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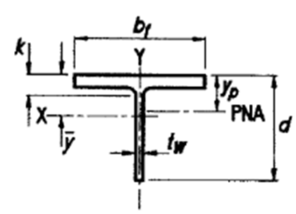
Steel Sections

4. Structural Tees Split from W or S -Shapes (WT or ST)

- bf : Flange width
- tw : Thickness of web
- tf : Thickness of flange
- d : Depth



- Examples: WT 22 X 145 ……

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Steel Sections

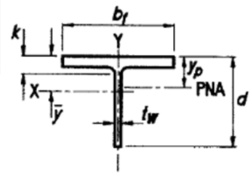


Table 1–8
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem Thickness, t_w		$\frac{t_w}{2}$	Stem Area	Flange Width, b_f		Flange Thickness, t_f		Distance k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT22×167.5[c]	49.2	22.0	22	1.03	1	1/2	22.6	15.9	16	1.77	1 3/4	2.56	2 5/8	5 1/2
×145[c]	42.7	21.8	21 3/4	0.865	7/8	7/16	18.9	15.8	15 7/8	1.58	1 9/16	2.36	2 7/16	↓
×131[c]	38.4	21.7	21 5/8	0.785	13/16	7/16	17.0	15.8	15 3/4	1.42	1 7/16	2.20	2 1/4	
×115[c,v]	33.8	21.5	21 1/2	0.710	11/16	3/8	15.2	15.8	15 3/4	1.22	1 1/4	2.01	2 1/16	
WT20×296.5[h]	87.2	21.5	21 1/2	1.79	1 13/16	15/16	38.5	16.7	16 3/4	3.23	3 1/4	4.41	4 1/2	7 1/2

Steel Sections

Table 1–8 (continued)
WT Shapes
Properties

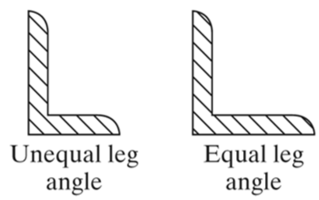
WT22-WT20

Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X $\bar{y}$	Axis X-X Z	Axis X-X y_p	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	Q_s F_y = 50 ksi	Torsional Properties J	Torsional Properties C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
167.5	4.50	21.5	2170	131	6.63	5.53	234	1.54	600	75.2	3.49	118	0.822	37.2	438
145	5.02	25.2	1830	111	6.54	5.26	196	1.35	521	65.9	3.49	102	0.629	25.4	275
131	5.57	27.6	1640	99.4	6.53	5.19	176	1.22	462	58.6	3.47	90.9	0.526	18.6	200
115	6.45	30.2	1440	88.6	6.53	5.17	157	1.07	398	50.5	3.43	78.3	0.438	12.4	139
296.5	2.58	12.0	3310	209	6.16	5.66	379	2.61	1260	151	3.80	240	1.00	221	2340

Steel Sections

5. Equal and UnEqual Leg Angles (L):

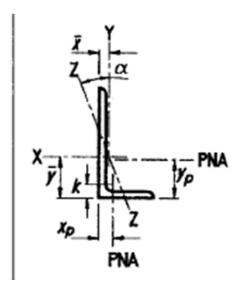
- Designation: **L L1 X L2 X t**
- L1 : Long leg
- L2 : Short leg
- t : thickness



Steel Sections

5. Equal and UnEqual Leg Angles (L):

- Examples: L 8 X 8 X 1 ……



Steel Sections

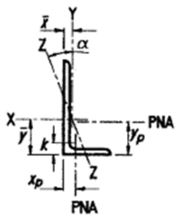


Table 1–7
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X: I	S	r	ȳ	Z	y_p	Flexural-Torsional Properties: J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L8×8×1 1/8	1 3/4	56.9	16.7	98.1	17.5	2.41	2.40	31.6	1.05	7.13	32.5	4.29
×1	1 5/8	51.0	15.0	89.1	15.8	2.43	2.36	28.5	0.943	5.08	23.4	4.32
×7/8	1 1/2	45.0	13.2	79.7	14.0	2.45	2.31	25.3	0.832	3.46	16.1	4.36
×3/4	1 3/8	38.9	11.4	69.9	12.2	2.46	2.26	22.0	0.720	2.21	10.4	4.39
×5/8	1 1/4	32.7	9.61	59.6	10.3	2.48	2.21	18.6	0.606	1.30	6.16	4.42
×9/16	1 3/16	29.6	8.68	54.2	9.33	2.49	2.19	16.8	0.548	0.961	4.55	4.43
×1/2	1 1/8	26.4	7.75	48.8	8.36	2.49	2.17	15.1	0.490	0.683	3.23	4.45
L8×6×1	1 1/2	44.2	13.0	80.9	15.1	2.49	2.65	27.3	1.47	4.34	16.3	3.88

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Steel Sections

Table 1–7 (continued)
Angles
Properties

L8-L6

Shape	Axis Y-Y: I	S	r	x̄	Z	x_p	Axis Z-Z: I	S	r	Tan α	Q_s: F_y = 36 ksi
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.		
L8×8×1 1/8	98.1	17.5	2.41	2.40	31.6	1.05	40.9	7.23	1.56	1.00	1.00
×1	89.1	15.8	2.43	2.36	28.5	0.943	36.8	6.51	1.56	1.00	1.00
×7/8	79.7	14.0	2.45	2.31	25.3	0.832	32.7	5.78	1.57	1.00	1.00
×3/4	69.9	12.2	2.46	2.26	22.0	0.720	28.5	5.04	1.57	1.00	1.00
×5/8	59.6	10.3	2.48	2.21	18.6	0.606	24.2	4.27	1.58	1.00	0.997
×9/16	54.2	9.33	2.49	2.19	16.8	0.548	22.0	3.88	1.58	1.00	0.959
×1/2	48.8	8.36	2.49	2.17	15.1	0.490	19.7	3.49	1.59	1.00	0.912
L8×6×1	38.8	8.92	1.72	1.65	16.2	0.816	21.3	4.84	1.28	0.542	1.00

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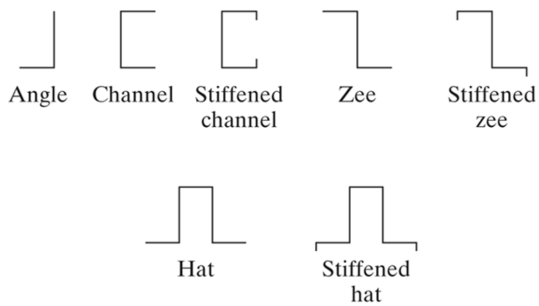
Steel Sections

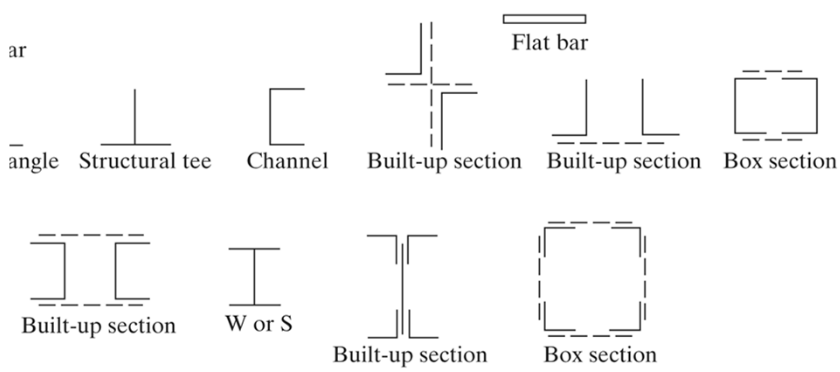
II. Cold Formed Steel Members:

- Thickness < 1 inch



Steel Sections

III. Built-up Sections:



STEEL DESIGN

Chapter 2:
Specifications, Loads and Method of Design

Specifications

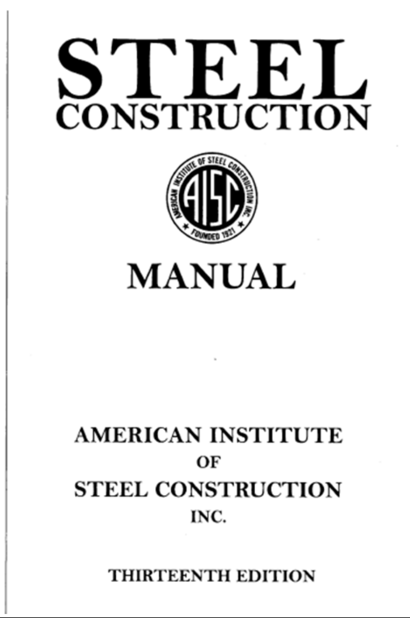
- The design of Structural Steel is controlled and governed by building codes.
- These codes specify minimum:

A. Design Loads
B. Design Stresses
C. Construction Types
D. Material Quality
E. Other Factors

Specifications

- Examples:
- American Association of State Highway and Transportation Officials (AASHTO)
- American Concrete Institute (ACI)
- American Institute of Steel Construction (AISC)

Specifications



Steel Construction Manual

- Part 1: Dimensions and Properties
- Part 2: General Design Considerations
- Part 3: Design of Flexure Members
- Part 4: Design of Compression Members
- Part 5: Design of Tension Members
- Part 6: Design of Members Subjected to Combined Loadings
- Part 7: Design Considerations for Bolts
- Part 8: Design Considerations for Welds

Steel Construction Manual

- Part 9: Design of Connecting Elements
- Part 10: Design of Simple Shear Connections
- Part 11: Design of Flexible Moment Connections
- Part 12: Design of Fully Restrained (FR) Moment Connections.
- Part 13: Design of Bracing Connections and Truss Connections
- Part 14: Design of Beam Bearing Plates, Column Base Plates, Anchor Rods, and Column Splices

Steel Construction Manual

- Part 15: Design of Hanger Connections, Bracket Plates, and Crane-Rail Connections
- Part 16: Specifications and Codes
- Part 17: Miscellaneous Data and Math Information

Loads

- The objective of a structural engineer is to design a structure that will be able to withstand all the loads to which it is subjected while serving its intended purpose throughout its intended life span.
- Loads can be classified as:

1. Dead Loads
2. Live Loads
3. Environmental Loads

Loads

- Dead Loads:
- Dead load is a fixed position gravity service load.
- Dead loads are usually known accurately.
- Dead loads can be determined from many codes such as International Building Code (IBC), ASCE, Jordanian Code for Load and Forces, … etc.

Loads

Table 17–13
Weights of Building Materials

Materials	Weight lb per sq ft	Materials	Weight lb per sq ft
CEILINGS		**PARTITIONS**	
Channel suspended system	1	Clay Tile	
Lathing and plastering	See Partitions	3 in.	17
Acoustical fiber tile	1	4 in.	18
		6 in.	28
		8 in.	34
		10 in.	40
FLOORS		Gypsum Block	
Steel Deck	See Manufacturer	2 in.	9½
		3 in.	10½
Concrete-Reinforced 1 in.		4 in.	12½
Stone	12½	5 in.	14
Slag	11½	6 in.	18½
Lightweight	6 to 10	Wood Studs 2×4	
		12–16 in. o.c.	2
Concrete-Plain 1 in.		Steel partitions	4
Stone	12	Plaster 1 inch	
Slag	11	Cement	10
Lightweight	3 to 9	Gypsum	5
		Lathing	
Fills 1 inch		Metal	½
Gypsum	6	Gypsum Board ½-in.	2
Sand	8		
Cinders	4		

Loads

Finishes			
Terrazzo 1 in.	13		
Ceramic or Quarry Tile ¾-in.	10		
Linoleum ¼-in.	1	**WALLS**	
Mastic ¾-in.	9	Brick	
Hardwood ⅞-in.	4	4 in.	40
Softwood ¾-in.	2½	8 in.	80
		12 in.	120
		Hollow Concrete Block	
		(Heavy Aggregate)	
ROOFS		4 in.	30
Copper or tin	1	6 in.	43
Corrugated steel	See Manufactuer	8 in.	55
3-ply ready roofing	1	12½-in.	80
3-ply felt and gravel	5½	Hollow Concrete Block	
5-ply felt and gravel	6	(Light Aggregate)	
		4 in.	21
Shingles		6 in.	30
Wood	2	8 in.	38
Asphalt	3	12 in.	55
Clay tile	9 to 14	Clay tile (Load Bearing)	
Slate ¼	10	4 in.	25
		6 in.	30
Sheathing		8 in.	33
Wood ¾-in.	3	12 in.	45
Gypsum 1 in.	4	Stone 4 in.	55
		Glass Block 4 in.	18
Insulation 1 in.		Window, Glass, Frame, & Sash	8
Loose	½	Curtain Walls	See Manufacturer
Poured	2	Structural Glass 1 in.	15
Rigid	1½	Corrugated Cement Asbestos ¼-in.	3

Loads

- <u>Live Loads:</u>
- Live loads are loads that may change in position and magnitude.
- Live loads are caused when a structure is occupied, used and maintained.
- Live loads can be determined from many codes such as International Building Code (IBC), ASCE, Jordanian Code for Load and Forces, … etc.

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Loads

- Examples:

i. Floor Loads:
ii. Traffic Loads for Bridges
iii. Impact Loads
iv. Longitudinal loads
v. Other loads (soil pressure, hydrostatic pressure, blast loads……)

Loads

- Environmental loads:

i. Snow Loads
ii. Rain Loads
iii. Wind Loads
iv. Earthquake Loads

Two Design Philosophies

1. <u>Allowable Stress Design (ASD)</u>

- A member is selected that has cross-sectional properties such as area and moment of inertia that are large enough to prevent the maximum applied axial force, shear, or bending moment from exceeding an allowable, or permissible value. This value is obtained by:

$$\text{required strength} \leq \text{allowable strength}$$

where

$$\text{allowable strength} = \frac{\text{nominal strength}}{\text{safety factor}}$$

Two Design Philosophies

- This approach is also called Elastic Design or Working Stress Design.

- <u>Two assumptions:</u>

1. Use service loads (working loads).

2. The allowable stress is in the elastic range of the material.

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Two Design Philosophies

2. Load and Resistance Factor Design (LRFD)

- A member is selected by using the criterion that the structure will fail at a load substantially higher than the working load.
- Two assumptions:

1. Load factors are applied to service loads.
2. The theoretical strength of the member is reduced by the applications of a resistance factor.

Two Design Philosophies

Factored load ≤ factored strength

$$\sum (\text{Loads} \times \text{load factors}) \leq \text{resistance} \times \text{resistance factor}$$

$$\Sigma \gamma_i Q_i \leq \phi R_n$$

Q_i = a load effect (a force or a moment)
γ_i = a load factor
R_n = the nominal resistance, or strength, of the component under consideration
ϕ = resistance factor

Two Design Philosophies

- The reason for amplifying the loads is to account for the uncertainty in estimating the loads
- *Load Factors (ASCE 2002) :*

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$
4. $1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + 0.5L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Two Design Philosophies

where

D = dead load
E = earthquake load
F = load due to fluids with well-defined pressures and maximum heights
H = load due to lateral earth pressure, groundwater pressure, or pressure of bulk materials
L = live load
L_r = roof live load
R = rain load*
S = snow load
T = self-straining force
W = wind load

Two Design Philosophies

$$1.4D \quad (1)$$

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \quad (2)$$

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \quad (3)$$

$$1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)^{\ddagger} \quad (4)$$

$$1.2D \pm 1.0E + 0.5L + 0.2S^{\dagger} \quad (5)$$

$$0.9D \pm (1.6W \text{ or } 1.0E) \quad (6)$$

Two Design Philosophies

- *Strength or Resistance Factor:*
- Strength factors are usually reduction factors that applied to the strength (stress , force, moment) of the member to account for the uncertainties in material strengths, dimensions and workmanship.
- These values are:

1. 0.85 for columns.
2. 0.75 or 0.90 for tension members.
3. 0.90 for bending or shear in beams.

Example

A column (compression member) in the upper story of a building is subject to the following loads:

Dead load:	109 kips compression
Floor live load:	46 kips compression
Roof live load:	19 kips compression
Snow:	20 kips compression

a. Determine the controlling load combination for LRFD and the corresponding factored load.
b. If the resistance factor ϕ is 0.90, what is the required *nominal* strength?

Example

Combination 1: $1.4D = 1.4(109) = 152.6$ kips

Combination 2: $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$. Because S is larger than L_r and $R = 0$, we need to evaluate this combination only once, using S.

$1.2D + 1.6L + 0.5S = 1.2(109) + 1.6(46) + 0.5(20) = 214.4$ kips

Combination 3: $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$. In this combination, we use S instead of L_r, and both R and W are zero.

$1.2D + 1.6S + 0.5L = 1.2(109) + 1.6(20) + 0.5(46) = 185.8$ kips

Combination 4: $1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$. This expression reduces to $1.2D + 0.5L + 0.5S$, and by inspection, we can see that it produces a smaller result than combination 3.

Combination 5: $1.2D \pm 1.0E + 0.5L + 0.2S$. As $E = 0$, this expression reduces to $1.2D + 0.5L + 0.2S$, which produces a smaller result than combination 4.

Combination 6: $0.9D \pm (1.6W \text{ or } 1.0E)$. This expression reduces to $0.9D$, which is smaller than any of the other combinations.

Example

Answer Combination 2 controls, and the factored load is 214.4 kips.

b. If the factored load obtained in part (a) is substituted into the fundamental LRFD relationship, Equation 2.6, we obtain

$$R_u \le \phi R_n$$
$$214.4 \le 0.90 R_n$$
$$R_n \ge 238 \text{ kips}$$

Answer The required nominal strength is 238 kips.

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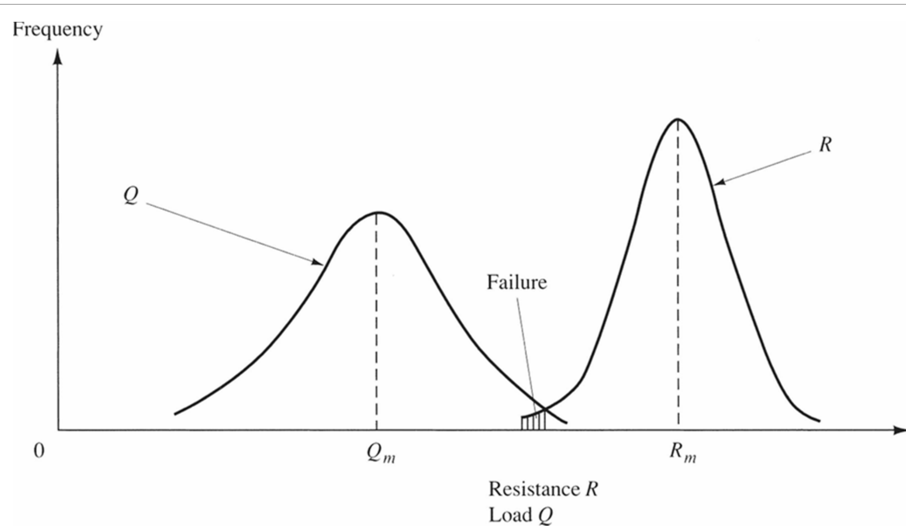
Frequency Distributions of Load Q & Resistance R



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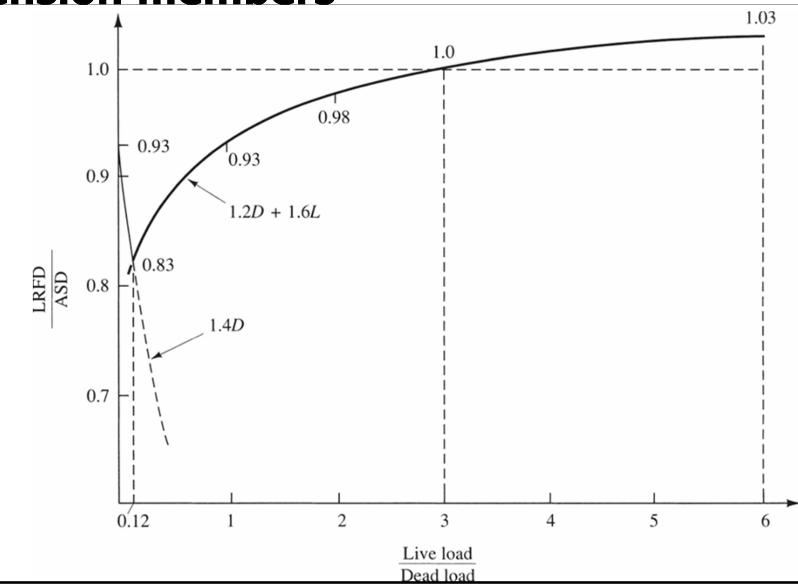
Comparison of load and resistance factor design with allowable stress design for tension members



STEEL DESIGN

Chapter 3:
Analysis and Design of Tension Members

Introduction

- Tension members are structural elements that are subjected to axial tensile forces caused by static forces acting through the centroidal axis.
- *Tension members are found in*:
- Truss members.
- Bracing for buildings and bridges.
- Cables in suspended roof systems and bridges.
- Analysis of Tension members is Chapter D in the specifications (part 16) in the Steel Manual.

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Introduction

- The stress in an axially loaded tension member is :

$$f = P/A$$

where,

- P: is the magnitude of load,
- A: is the cross-sectional area normal to the load.

- The stress in a tension member is uniform throughout the cross-section except:
  - Near the point of application of load, and
  - At the cross-section with holes for bolts or other discontinuities, etc.

Types of Sections

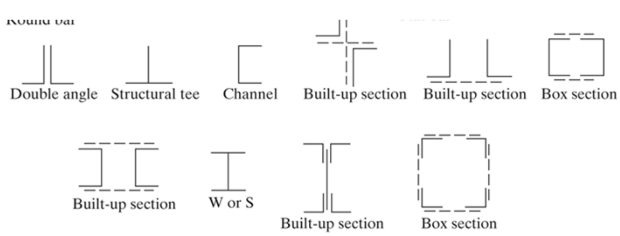
I. <u>Rolled Steel Sections:</u>

- W, S, WT, ST, C, L

II. <u>Special Sections:</u>

- Flat bar, Rods and Cables.

III. <u>Built up Sections:</u>



Tensile Strength

- A tension member can *fail* by reaching three limit states:

1. Excessive deformation initiated by yielding of the gross cross-section of the member away from the connection.
2. Fracture of the effective net area (through the holes) at the connections.
3. Block Shear fracture through the bolt holes at the connection.

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Tensile Strength

1. Yielding in the gross cross-section:

- The nominal strength in yielding is:

$$P_n = F_y A_g$$

- Where:

  P_n : Nominal Strength in the gross section.
  F_y : Yield Stress.
  A_g : Gross cross-section area

$$P_u \leq \phi_t P_n$$

Where: $\phi_t = 0.90$

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Tensile Strength

2. Fracture in the net section:

$$P_n = F_u A_e$$

- Where:

  P_n : Nominal Strength in the net section.
  F_u: Ultimate Stress.
  A_e : Effective net area

$$P_u \le \phi_t P_n$$

Where: $\phi_t = 0.75$

Tensile Strength

$$A_e = A_n U$$

- Where:

  A_e: Effective net area.
  A_n: Net area for bolted connection.
  U : Reduction Coefficient.

Tensile Strength

- Important notes:
- Note 1: Why is fracture (not yielding) the relevant limit state at the net section?
- Yielding will occur first in the net section. However, the deformations induced by yielding will be localized around the net section. These localized deformations will not cause excessive deformations in the complete tension member. Hence, yielding at the net section will not be a failure limit state.

Tensile Strength

- Note 2: Why is the resistance factor (φ_t) smaller for fracture than for yielding?
- The smaller resistance factor for fracture (φ_t = 0.75 as compared to φ_t = 0.90 for yielding) reflects the more serious nature and consequences of reaching the fracture limit state.
- Note 3: What is the design strength of the tension member?
- The design strength will be the *lesser* value of the strength for the two limit states (gross section yielding and net section fracture).

Net Area, An

- Net area is the gross sectional area of the member minus the holes or notches.
- The long used practice is to punch holes with a diameter 1/16 inch larger than that of the bolts diameter.
- Punching damages 1/16 inch more of the surrounding metal.

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Net Area, An

$$d_h = d_B + 1/16 + 1/16$$

$$d_h = d_B + 1/8 \text{ inch} \quad \text{(Based on the text book)}$$

- Where:
- d_h : diameter of the hole.
- d_B : diameter of the bolt.

$$d_h = d_B + 1/16 \text{ inch}$$

(Based on the specifications D-3)

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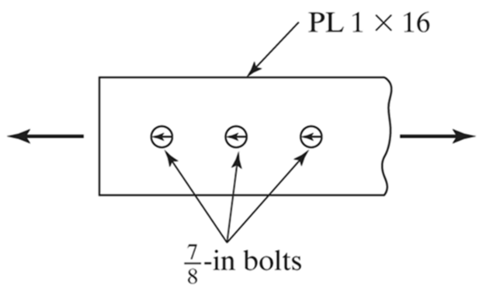
Examples (Net Area, An)

- Example 1:



Net $A = (1)(16) - (1)(\frac{7}{8} + \frac{1}{8})(1) = 15 \text{ in.}^2$

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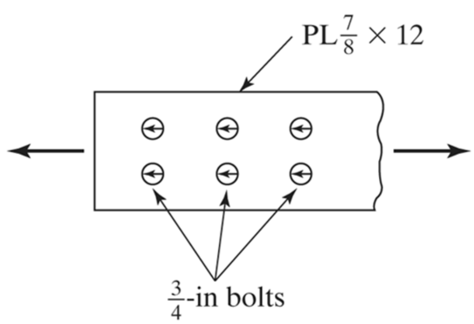
Examples (Net Area, An)

- Example 2:



Net $A = (\frac{7}{8})(12) - (2)(\frac{3}{4} + \frac{1}{8})(\frac{7}{8}) = 8.97 \text{ in.}^2$

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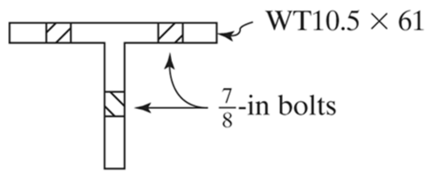
Examples (Net Area, An)

- Example 3:



$$\text{Net } A = 17.9 - (2)\left(\tfrac{7}{8}+\tfrac{1}{8}\right)(0.960) - (1)\left(\tfrac{7}{8}+\tfrac{1}{8}\right)(0.600)$$
$$= \boxed{15.38 \text{ in.}^2}$$

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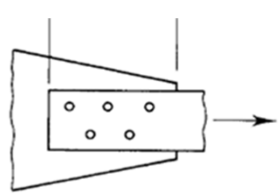
Effect of Staggered Holes

- For a bolted tension member, the connecting bolts can be staggered for several reasons:

1. To get more capacity by increasing the effective net area.
2. To achieve a smaller connection length.
3. To fit the geometry of the tension connection itself.



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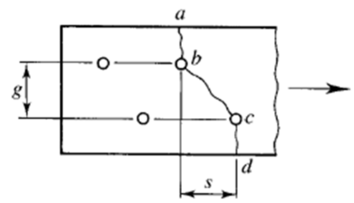
Effect of Staggered Holes



- Cochrane (1922) proposed a reduced diameter to account for the effects of staggered holes:

$$d' = d - \frac{s^2}{4g}$$

- d : is the hole diameter.
- s : pitch (spacing center to center in the direction of the load)
- g : transverse spacing (center to center)

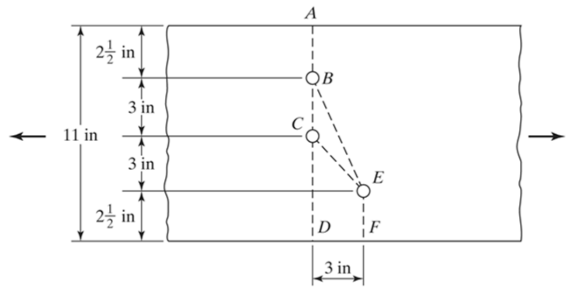
Effect of Staggered Holes

$$\begin{aligned} w_n &= w_g - \sum d' \\ &= w_g - \sum\left(d - \frac{s^2}{4g}\right) \\ &= w_g - \sum d + \sum \frac{s^2}{4g} \end{aligned}$$

- w_n : net width.
- w_g : gross width.

Examples (Staggered Holes)

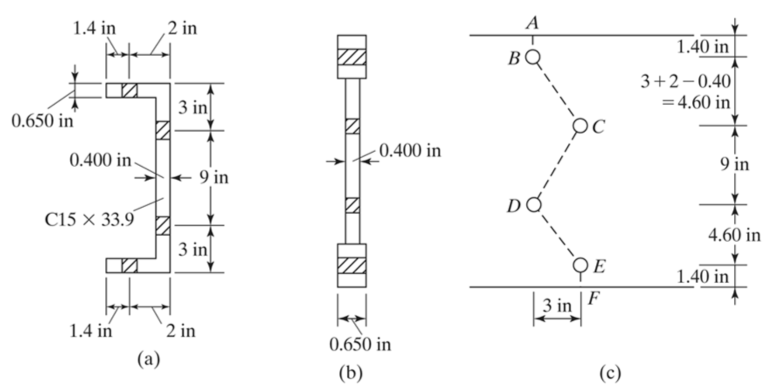
- Example 1:
- ½ inch thick plate
- d_B = ¾ inch



- Path *ABCD* :
  An = [11 – (2)(7/8)] (1/2) = 4.625 in^2
- Path *ABCEF* : An = [11 – (3)(7/8) + $(3)^2$/(4)(3)](1/2)
  = 4.56 in^2 (controls)
- Path *ABEF:* An = [11-(2)(7/8)+$(3)^2$/(4)(6)](1/2)
  = 4.8125 in^2

Examples (Staggered Holes)

- Example 2:



- C 15 X 33.9, d_B = ¾ inch

Examples (Staggered Holes)

- Example 2:

- Path *ABCDEF* :

$$A_n = 10 - (2)(7/8)(0.65) - (2)(7/8)(0.4) + (3)^2/(4)(9)\,(0.4) + (2)\,[(3)^2/(4)(4.6)]\,((0.65+0.4)/2)$$

$$= 8.778 \text{ in}^2$$

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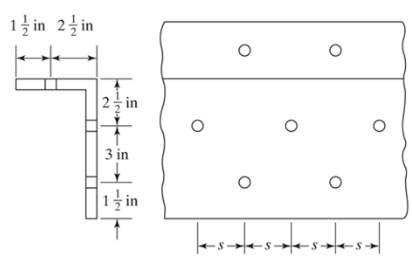
Examples (Staggered Holes)

- Example 3:
- L 7 x 4x 5/8
- d_B = 7/8 inch



- Required:

- Min S that only 2 holes need be subtracted in determining net area.

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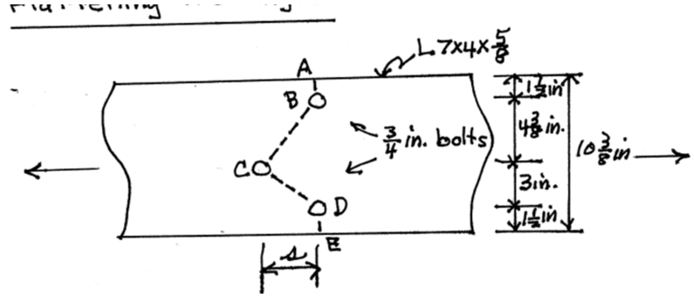
Examples (Staggered Holes)

- Example 3:



Net width ABCDE

$$= 10.375 - (3)\left(\frac{3}{4}+\frac{1}{8}\right) + \frac{s^2}{(4)(4.375)} + \frac{s^2}{(4)(3)}$$

$$= 7.75 + 0.1405\,s^2$$

Examples (Staggered Holes)

- Example 3:

Net width with 2 holes out

$$= 10.375 - (2)\left(\frac{3}{4}+\frac{1}{8}\right) = 8.625 \text{ in.}$$

Equating

$$7.75 + 0.1405\,s^2 = 8.625$$

$$\boxed{s = 2.50 \text{ in.}}$$

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Effective Area

- <u>For bolted Connections:</u>

$$A_e = A_n U$$

- <u>For welded Connections:</u>

$$A_e = A_g U$$

- An : Net area
- Ag : Gross area

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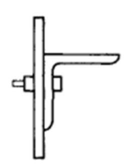
Effective Area

- <u>Shear Lag:</u>
- Occurs when some elements of the cross section are not connected.
- The connected element becomes over loaded and the unconnected part is not fully stressed, i.e. the tensile force is not uniformly distributed over the net area.



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Effective Area

- To account for the non-uniformity, the AISC specifications provide the effective area.
- Shear lag factors for connections to tension members are provided in AISC specifications in Table D3.1.
- This table is also available in the text book as table 3.2 page 75.

U Factor for Bolted Connections

- **<u>Case 1:</u>**

All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners (bolts) or welds.

U = 1.0

U Factor for Bolted Connections

- **Case 2:**

All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross sectional elements by fasteners (bolts):

$$U = 1 - \frac{\bar{x}}{\ell}$$

- Where:

$\bar{x}$ = distance from centroid of connected area to the plane of the connection
ℓ = length of the connection

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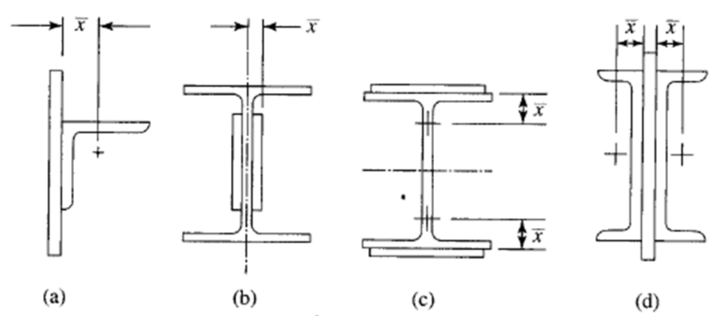
U Factor for Bolted Connections



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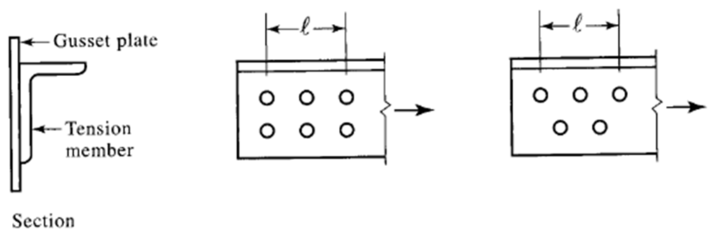
U Factor for Bolted Connections

- L: length of the line with the maximum number of bolts.
- Also, in staggered L: is the out to out dimension between the extreme bolts in a line.



U Factor for Bolted Connections

- **<u>Case 7:</u>**

  W, M, S or HP shapes or Tees cut from these shapes.

A. With flange connected with 3 or more fasteners per line in direction of loading:

$$b_f \geq 2/3\,d \ldots U = 0.90$$
$$b_f < 2/3\,d \ldots U = 0.85$$

B. With web connected with 4 or more fasteners per line in direction of loading:

U = 0.70

U Factor for Bolted Connections

- **Case 8:**

Single angles:

A. With 4 or more fasteners per line in direction of loading:

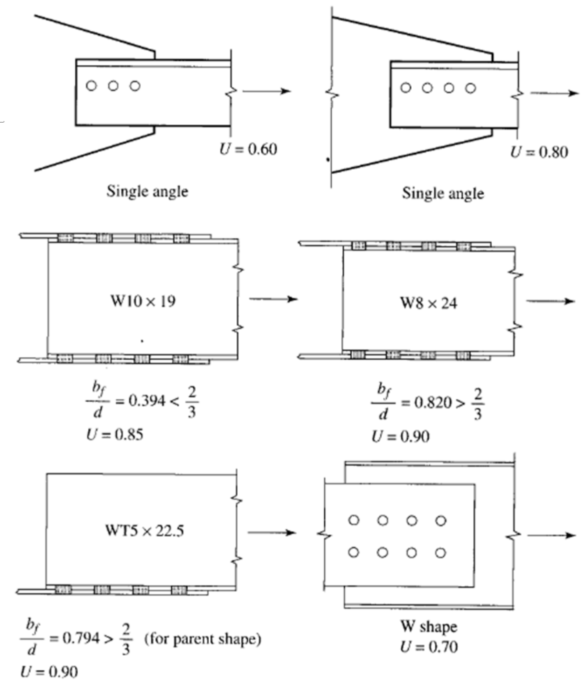
$$U = 0.80$$

B. With 2 or 3 fasteners per line in direction of loading:

$$U = 0.60$$

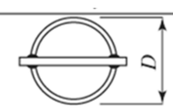
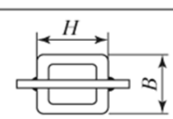
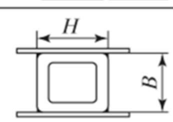
U Factor for Bolted Connections

- **Summary:**
- Use case 2 or 7 and 8.
- Larger value of above is used.



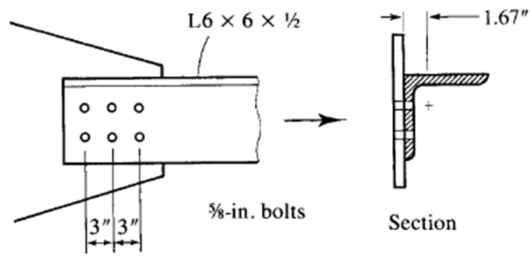
U Factor for Bolted Connections

- **Case 5 and 6:**

Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2 + 2BH}{4(B + H)}$	
	with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2}{4(B + H)}$	

Example (U Factor for Bolted Connections)

- Example 1:



- Required:
- Determine the effective net area.

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Example (U Factor for Bolted Connections)

$$A_n = A_g - A_{holes}$$

$$= 5.77 - \frac{1}{2}\left(\frac{5}{8} + \frac{1}{8}\right)(2) = 5.02 \text{ in.}^2$$

$\bar{x} = 1.67$ in.

The length of the connection is

$\ell = 3 + 3 = 6$ in.

$$\therefore U = 1 - \left(\frac{\bar{x}}{\ell}\right) = 1 - \left(\frac{1.67}{6}\right) = 0.7217$$

$$A_e = A_n U = 5.02(0.7217) = 3.623 \text{ in.}^2$$

- Case 8: $A_e = A_n U = 5.02(0.60) = 3.012 \text{ in.}^2$

Control : 3.623 in^2

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Example (U Factor for Bolted Connections)

- <u>Example 2:</u>

W10 x 45 with two lines of ¾ inch diameter bolts in each flange using A572 Grade 50. There are assumed to be at least three bolts in each line 4 inch on center, and the bolts are not staggered.

- <u>Required:</u>
- Determine the Tensile Design Strength.

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Example (U Factor for Bolted Connections)

1. <u>Yielding:</u>

$$\varphi_t\, Pn = \varphi_t\, Fy\, Ag$$
$$= (0.9)\,(50)\,(13.3) = 598.5 \text{ k}$$

2. <u>Fracture:</u>

$$\varphi_t\, Pn = \varphi_t\, Fu\, Ae$$

- An= 13.3 – (4)(3/4 + 1/8)(0.62) = 11.13 in^2
- One half of W10x45 is WT 5 x 22.5:
- $\bar{x}$ = 0.907, U = 1 – (0.907/8) = 0.89
- Case 7, U = 0.90 **(Control)**
- φ_t Pn = (0.75)(65)(0.9)(11.13) = **<u>488.5 k</u>**

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U Factor for Welded Connections

- **Case 1:**

All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners (bolts) or welds.

U = 1.0

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U Factor for Welded Connections

- **Case 2:**

All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross sectional elements by longitudinal welds:

$$U = 1 - \frac{\bar{x}}{\ell}$$

- Where:

$\bar{x}$ = distance from centroid of connected area to the plane of the connection
ℓ = length of the connection

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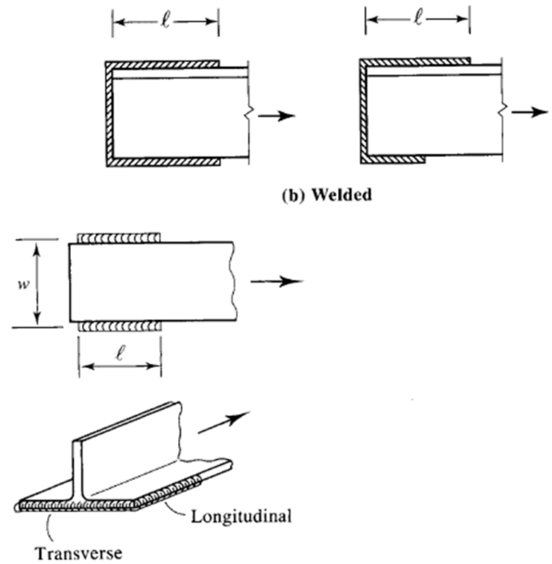
U Factor for Welded Connections



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U Factor for Welded Connections

- **<u>Case 3:</u>**

All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.

U = 1.0

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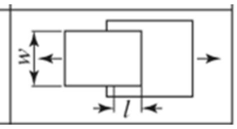
U Factor for Welded Connections

- **<u>Case 4:</u>**

Plates where the tension load is transmitted by longitudinal welds only:

$l \geq 2w \ldots U = 1.0$
$2w > l \geq 1.5w \ldots U = 0.87$
$1.5w > l \geq w \ldots U = 0.75$



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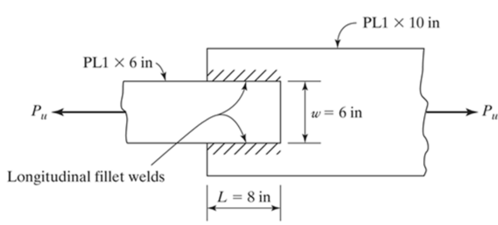
Example (U Factor for Welded Connections)

- <u>Example :</u>



- Fy = 50 Ksi, Fu = 65 Ksi
- <u>Required:</u>
- Tensile Design Strength of the member.

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Example (U Factor for Welded Connections)

1. Yielding:

$$\varphi_t Pn = \varphi_t Fy\ Ag$$
$$= (0.9)\ (50)\ (1)(6) = 270\ k$$

2. Fracture:

$$\varphi_t Pn = \varphi_t Fu\ Ae$$

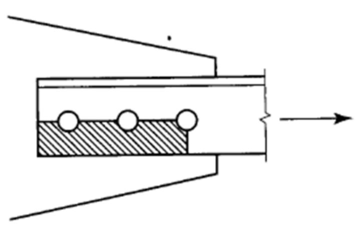
Ae = U Ag

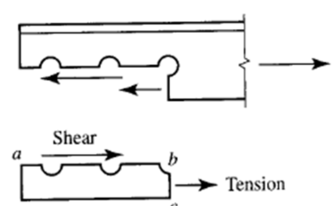
$$1.5\ w = 1.5 \times 6 = 9\ in > L = 8\ in > w = 6\ in$$
$$U = 0.75\ (Case\ 4)$$
$$Ae = (0.75)(6) = 4.5\ in^2$$

φ_t Pn =(0.75)(65)(4.5) = **219.4 k (control)**

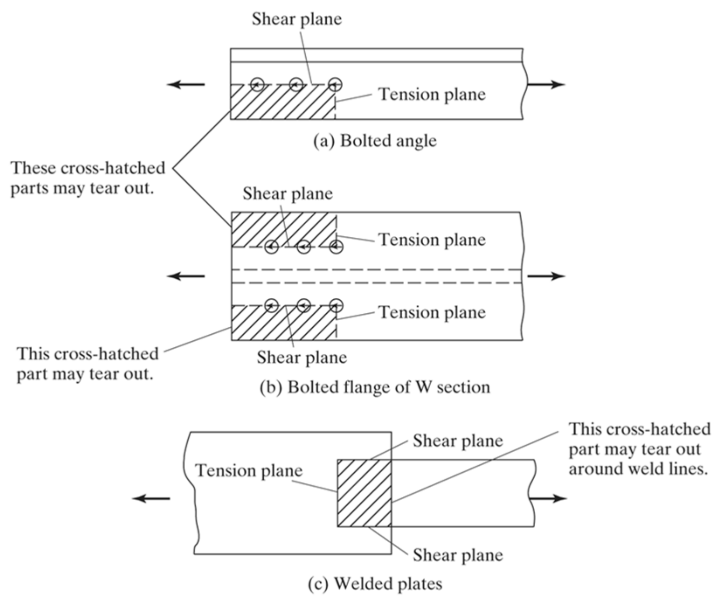
Block Shear

- For some connection configurations, the tension member can fail due to 'tear-out' of material at the connected end. This is called *block shear*.





Block Shear



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Block Shear

- The failure of a member may occur along a path involving Tension on one plane and shear on a perpendicular plane.
- When a tensile load applied to a connection is increased, the fracture strength of the weaker plane will be approached.
- That plane will not fail then, because it is restrained by the stronger plane.
- The load can be increased until the fracture strength of the stronger plane is reached.

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Block Shear

- During this time, the weaker plane is yielding.
- The total strength of the connection equals the fracture strength of the stronger plane plus the yield strength of the weaker plane.
- Block shear can be thought of as being a tearing or rupture failure and not a yielding failure at bolt holes.

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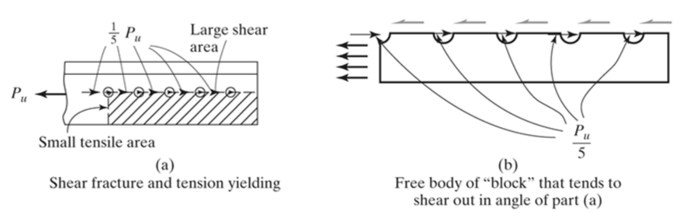
Block Shear



- The member shown above has a larger shear area and a small tensile area.
- The primary resistance to block shear failure is shearing and not tensile.
- The LRFD specifications assume shear fracture occurs on this large shear resisting area, the small tensile area has yielded.

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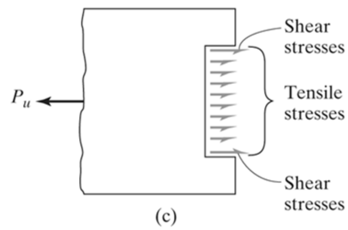
Block Shear



(c)
Tensile fracture and shear yielding

- The member shown above has a larger tensile area and a small shearing area.
- The block shear failure will be tensile and not shearing.

Block Shear

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

$0.6F_y$ = shear yield stress
A_{gv} = gross area along the shear surface or surfaces
A_{nv} = net area along the shear surface or surfaces
A_{nt} = net area along the tension surface

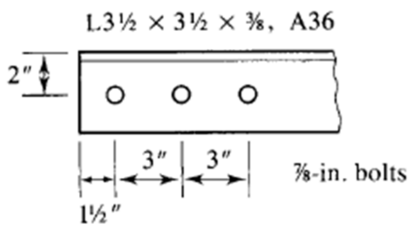
where $U_{bs} = 1.0$ when the tension stress is uniform (angles, gusset plates, and most coped beams) and $U_{bs} = 0.5$ when the tension stress is nonuniform.

For LRFD, the resistance factor ϕ is 0.75,

- The purpose of the reduction factor (U_{bs}) is to account for the fact that stress distribution may not be uniform on the tensile plane for some connections.

Example (Block Shear)

- Example :



- Required:
- Compute the block shear, A36.

Examples (Block Shear)

The shear areas are

$$A_{gv} = \frac{3}{8}(7.5) = 2.813 \text{ in.}^2$$

and, since there are 2.5 hole diameters,

$$A_{nv} = \frac{3}{8}\left[7.5 - 2.5\left(\frac{7}{8} + \frac{1}{8}\right)\right] = 1.875 \text{ in.}^2$$

The tension area is

$$A_{nt} = \frac{3}{8}\left[1.5 - 0.5\left(\frac{7}{8} + \frac{1}{8}\right)\right] = 0.3750 \text{ in.}^2$$

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$
$$= 0.6(58)(1.875) + 1.0(58)(0.3750) = 87.00 \text{ kips}$$

with an upper limit of

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(36)(2.813) + 1.0(58)(0.3750) = 82.51 \text{ kips}$$

The nominal block shear strength is therefore 82.51 kips.

The design strength for LRFD is $\phi R_n = 0.75(82.51) = 61.9$ kips.

Design of Tension Members

- Design Procedure:

1. Yielding:

$$0.90F_yA_g \geq P_u \quad \text{or} \quad A_g \geq \frac{P_u}{0.90F_y}$$

2. Fracture:

$$0.75F_uA_e \geq P_u \quad \text{or} \quad A_e \geq \frac{P_u}{0.75F_u}$$

3. Slenderness ratio:

- r : radius of gyration $\quad r \geq \frac{L}{300}$

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Examples (Design of Tension Members)

- Example 1:

A tension member with a length of 5 feet 9 inches must resist a service dead load of 18 kips and a service live load of 52 kips. Select a member with a rectangular cross section. Use A36 steel and assume a connection with one line of 7⁄8-inch-diameter bolts.

$$P_u = 1.2D + 1.6L = 1.2(18) + 1.6(52) = 104.8 \text{ kips}$$

$$\text{Required } A_g = \frac{P_u}{\phi_t F_y} = \frac{P_u}{0.90F_y} = \frac{104.8}{0.90(36)} = 3.235 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_u}{\phi_t F_u} = \frac{P_u}{0.75F_u} = \frac{104.8}{0.75(58)} = 2.409 \text{ in.}^2$$

Try $t = 1$ in.

$$\text{Required } w_g = \frac{\text{required } A_g}{t} = \frac{3.235}{1} = 3.235 \text{ in.}$$

Try a $1 \times 3\frac{1}{2}$ cross section.

$$A_e = A_n = A_g - A_{hole}$$

$$= (1 \times 3.5) - \left(\frac{7}{8} + \frac{1}{8}\right)(1) = 2.5 \text{ in.}^2 > 2.409 \text{ in.}^2 \qquad \text{(OK)}$$

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Examples (Design of Tension Members)

- <u>Example 1:</u>

Check the slenderness ratio:

$$I_{min} = \frac{3.5(1)^3}{12} = 0.2917 \text{ in.}^4$$

$$A = 1(3.5) = 3.5 \text{ in.}^2$$

From $I = Ar^2$ we obtain

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{0.2917}{3.5}} = 0.2887 \text{ in.}^2$$

$$\text{Maximum } \frac{L}{r} = \frac{5.75(12)}{0.2887} = 239 < 300 \qquad \text{(OK)}$$

Use a PL $1 \times 3\frac{1}{2}$

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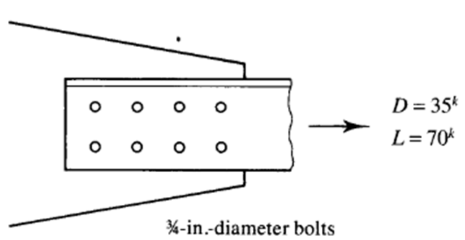
Examples (Design of Tension Members)

- <u>Example 2:</u>
- Select an unequal-leg angle tension member 15 feet long to resist a service dead load of 35 kips and a service live load of 70 kips. Use A36 steel.



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Examples (Design of Tension Members)

$$P_u = 1.2D + 1.6L = 1.2(35) + 1.6(70) = 154 \text{ kips}$$

$$\text{Required } A_g = \frac{P_u}{\phi_t F_y} = \frac{154}{0.90(36)} = 4.75 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_u}{\phi_t F_u} = \frac{154}{0.75(58)} = 3.54 \text{ in.}^2$$

The radius of gyration should be at least

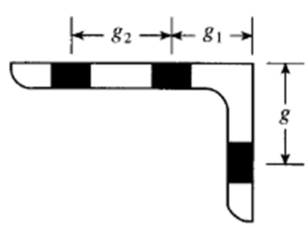
$$\frac{L}{300} = \frac{15(12)}{300} = 0.6 \text{ in.}$$

Try L6 × 4 × ½.

$$A_n = A_g - A_{holes} = 4.75 - 2\left(\frac{3}{4} + \frac{1}{8}\right)\left(\frac{1}{2}\right) = 3.875 \text{ in.}^2$$

$$A_e = A_n U = 3.875(0.80) = 3.10 \text{ in.}^2 < 3.54 \text{ in.}^2 \qquad \text{(N.G.)*}$$

Examples (Design of Tension Members)



Usual Gages for Angles (inches)

Leg	8	7	6	5	4	3½	3	2½	2	1¾	1½	1⅜	1¼	1
g	4½	4	3½	3	2½	2	1¾	1⅜	1⅛	1	⅞	⅞	¾	⅝
g_1	3	2½	2¼	2										
g_2	3	3	2½	1¾										

Examples (Design of Tension Members)

Try the next larger shape from the dimensions and properties tables.

Try L5 × 3½ × ⅝ ($A_g = 4.92$ in.^{2} and $r_{min} = 0.746$ in.)

$$A_n = A_g - A_{holes} = 4.92 - 2\left(\frac{3}{4}+\frac{1}{8}\right)\left(\frac{5}{8}\right) = 3.826 \text{ in.}^2$$

$$A_e = A_n U = 3.826(0.80) = 3.06 \text{ in.}^2 < 3.54 \text{ in.}^2 \qquad \text{(N.G.)}$$

Try L8 × 4 × ½ ($A_g = 5.75$ in.^{2} and $r_{min} = 0.863$ in.)

$$A_n = A_g - A_{holes} = 5.75 - 2\left(\frac{3}{4}+\frac{1}{8}\right)\left(\frac{1}{2}\right) = 4.875 \text{ in.}^2$$

$$A_e = A_n U = 4.875(0.80) = 3.90 \text{ in.}^2 < 3.54 \text{ in.}^2 \qquad \text{(OK)}$$

This shape satisfies all requirements, so use an L8 ×4 × ½.

Tables for the Design of Tension Members

$F_y = 50$ ksi
$F_u = 65$ ksi

Table 5–1
Available Strength in Axial Tension
W Shapes

W44–W40

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
W44×335	98.5	73.9	2950	4430	2400	3600
×290	85.4	64.1	2560	3840	2080	3120
×262	76.9	57.7	2300	3460	1880	2810
×230	67.7	50.8	2030	3050	1650	2480
W40×593[h]	174	131	5210	7830	4260	6390
×503[h]	148	111	4430	6660	3610	5410
×431[h]	127	95.3	3800	5720	3100	4650
×397[h]	117	87.8	3500	5270	2850	4280
×372[h]	109	81.8	3260	4910	2660	3990
×362[h]	107	80.3	3200	4820	2610	3910
×324	95.3	71.5	2850	4290	2320	3490
×297	87.4	65.6	2620	3930	2130	3200
×277	81.4	61.0	2440	3660	1980	2970
×249	73.3	55.0	2190	3300	1790	2680
×215	63.4	47.6	1900	2850	1550	2320
×199	58.5	43.9	1750	2630	1430	2140

Tables for the Design of Tension Members

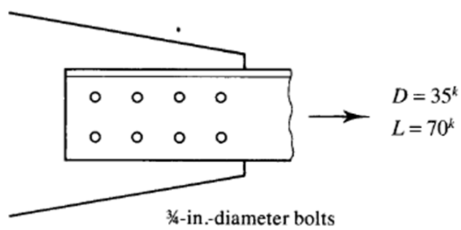
- The AISC manual tabulates the tension design strength of standard steel sections. (Table 5)
- Include: wide flange shapes, angles, tee sections, and double angle sections.
- The gross yielding design strength and the net section fracture strength of each section is tabulated.
- This provides a great starting point for selecting a section.

Tables for the Design of Tension Members

- **<u>There is one serious limitation</u>**
- The net section fracture strength is tabulated for an assumed value of U = 0.75, obviously because the precise connection details are not known.
- For all W, Tee, angle and double-angle sections, Ae is assumed to be = 0.75 Ag
- The engineer can first select the tension member based on the tabulated gross yielding and net section fracture strengths, and then check the net section fracture strength and the block shear strength using the actual connection details

Example (Tables for the Design of Tension Members)

- <u>Example :</u>
- Select an unequal-leg angle tension member 15 feet long to resist a service dead load of 35 kips and a service live load of 70 kips. Use A36 steel.



Example (Tables for the Design of Tension Members)

$$P_u = 154 \text{ kips}$$
$$r_{min} \geq 0.600 \text{ in.}$$

we find that an L6 × 4 × ½, with $\phi_t P_n = 154$ kips based on the gross section and $\phi_t P_n = 155$ kips based on the net section, is a possibility. From the dimensions and properties tables in Part 1 of the *Manual*, $r_{min} = 0.980$ in. To check this selection, we must compute the actual net area. If we assume that $U = 0.80$,

$$A_n = A_g - A_{holes} = 4.75 - 2\left(\frac{3}{4}+\frac{1}{8}\right)\left(\frac{1}{2}\right) = 3.875 \text{ in.}^2$$

$$A_e = A_n U = 3.875(0.80) = 3.10 \text{ in.}^2$$

$$\phi_t P_n = \phi_t F_u A_e = 0.75(58)(3.10) = 135 \text{ kips} < 154 \text{ kips} \qquad \text{(N.G.)}$$

$$\frac{3.10}{4.75} = 0.6526$$

Example (Tables for the Design of Tension Members)

This corresponds to a required $\phi_t P_n$ (based on rupture) of

$$\frac{0.75}{\text{actual ratio}} \times P_u = \frac{0.75}{0.6526}(154) = 177 \text{ kips}$$

Try an L8 × 4 × ½, with $\phi_t P_n = 186$ kips (based on yielding) and $\phi_t P_n = 187$ (based on rupture strength). From the dimensions and properties tables in Part 1 of the *Manual*, $r_{min} = 0.863$ in. The actual effective net area and rupture strength are computed as follows:

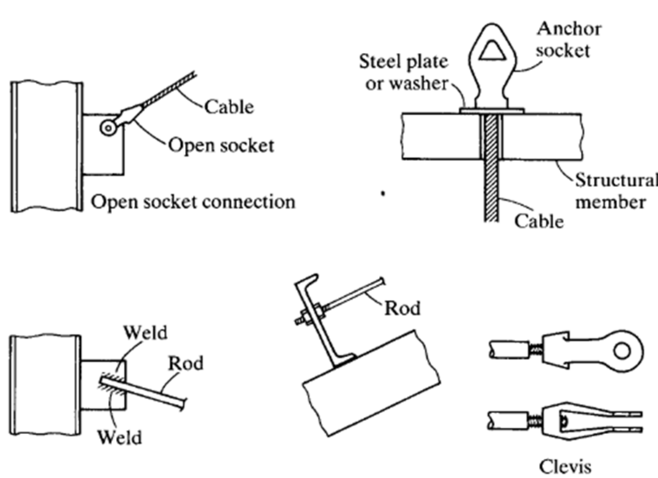
$$A_n = A_g - A_{holes} = 5.75 - 2\left(\frac{3}{4} + \frac{1}{8}\right)\left(\frac{1}{2}\right) = 4.875 \text{ in.}^2$$

$$A_e = A_n U = 4.875(0.80) = 3.90 \text{ in.}^2$$

$$\phi_t P_n = \phi_t F_u A_e = 0.75(58)(3.90) = 170 > 154 \text{ kips} \qquad \text{(OK)}$$

Use an L8 × 4 × ½, connected through the 8-inch leg.

Threaded Rods and Cables



Threaded Rods

$$P_n = A_s F_u = 0.75 A_b F_u$$

where
A_s = stress area
A_b = nominal (unthreaded) area

$$P_u \le \phi_t P_n \quad \text{or} \quad P_u \le 0.75(0.75 A_b F_u)$$

and the required area is

$$A_b = \frac{P_u}{0.75(0.75F_u)}$$

Example (Threaded Rods)

- Example:

A threaded rod is to be used as a bracing member that must resist a service tensile load of 2 kips dead load and 6 kips live load. What size rod is required if A36 steel is used?

The factored load is

$$P_u = 1.2(2) + 1.6(6) = 12 \text{ kips}$$

From Equation 3.6,

$$\text{Required Area} = A_b = \frac{P_u}{0.75(0.75F_u)} = \frac{12}{0.75(0.75)(58)} = 0.3678 \text{ in.}^2$$

From $A_b = \frac{\pi d^2}{4}$,

$$\text{Required } d = \sqrt{\frac{4(0.3678)}{\pi}} = 0.684 \text{ in.}$$

Use a ¾-inch-diameter threaded rod ($A_b = 0.442$ in.^{2}).

STEEL DESIGN

Chapter 4:

Analysis and Design of Compression Members

Introduction

- Compression Members (Columns) are Structural elements that are subjected to axial compressive forces caused by static forces acting through the centroidal axis. (Chapter E in the Specifications)
- The stress in the column cross-section can be calculated as :

$$f = P / A$$

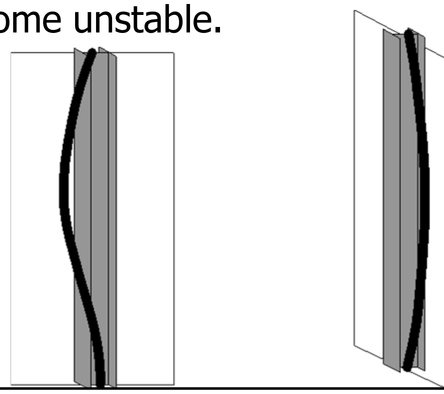
- where, f is assumed to be uniform over the entire cross-section.

Introduction

- *Mode of failure for compression members:*

1. *Flexural (Euler) Buckling:*

- Members are subjected to flexure or bending when they become unstable.



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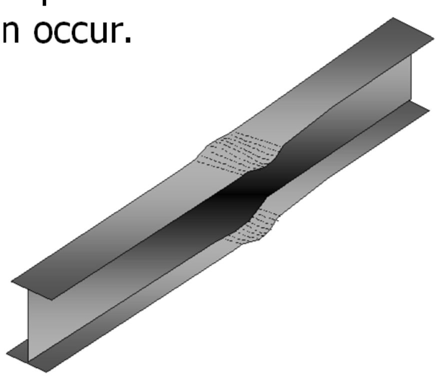
Introduction

2. *Local Buckling:*

- This type of buckling occurs when some parts of the cross-section of a column are so thin that they buckle locally in compression before the other modes of buckling can occur.



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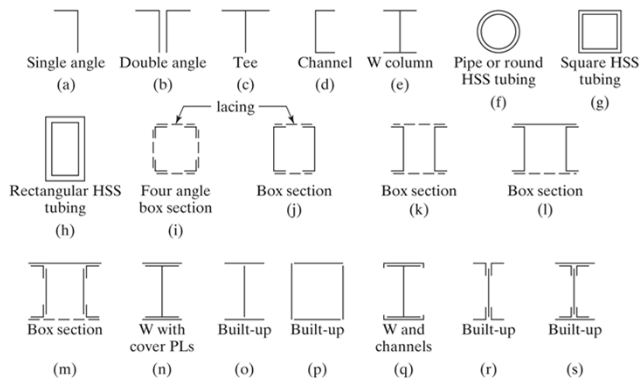
Introduction

3. Flexure torsional Buckling:

- These columns fail by twisting or by combination of torsional and flexural buckling.
- Types of sections:



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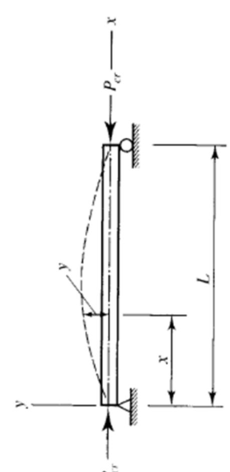
Column Theory (Euler Formula)



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Column Theory (Euler Formula)

$$\frac{d^2y}{dx^2} = -\frac{M}{EI}$$

$M = P_{cr}\ y$

$$y'' + \frac{P_{cr}}{EI}\, y = 0$$

- Second order, linear, homogeneous differential equation with constant coefficients.

$$c = \sqrt{\frac{P_{cr}}{EI}}$$

$y'' + c^2\ y = 0$

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Column Theory (Euler Formula)

$$y = A\cos(cx) + B\sin(cx)$$

At $x = 0,\ y = 0$: $\quad 0 = A\cos(0) + B\sin(0) \qquad A = 0$

At $x = L,\ y = 0$: $\quad 0 = B\sin(cL)$

B = 0 (trivial solution) , P = 0

$$cL = 0,\ \pi,\ 2\pi,\ 3\pi,\ \ldots = n\pi, \qquad n = 0, 1, 2, 3, \ldots$$

$$cL = \left(\sqrt{\frac{P_{cr}}{EI}}\right)L = n\pi, \quad \frac{P_{cr}}{EI}L^2 = n^2\pi^2 \quad \text{and} \quad P_{cr} = \frac{n^2\pi^2 EI}{L^2}$$

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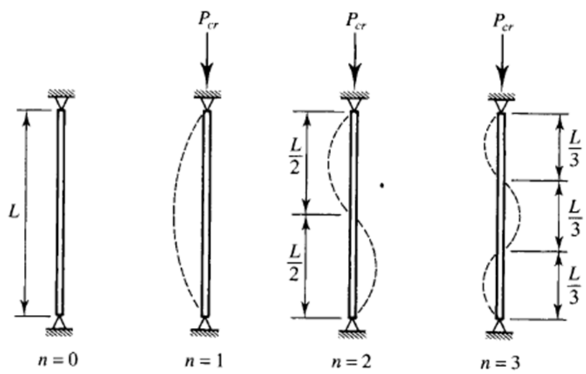
Column Theory (Euler Formula)



$$P_{cr} = \frac{\pi^2 EI}{L^2}$$

$$P_{cr} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 EAr^2}{L^2} = \frac{\pi^2 EA}{(L/r)^2}$$

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Column Theory (Euler Formula)

-

$$F_{cr} = \frac{P_{cr}}{A} = \frac{\pi^2 E}{(L/r)^2}$$

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Example

- A W12 X 50 column is used to support an axial compressive load of 145 kips. The length is 20 feet, and the ends are pinned. Investigate the stability of the column.

For a W12 × 50,

Minimum $r = r_y = 1.96$ in.

$$\text{Maximum } \frac{L}{r} = \frac{20(12)}{1.96} = 122.4$$

$$P_{cr} = \frac{\pi^2 EA}{(L/r)^2} = \frac{\pi^2 (29,000)(14.6)}{(122.4)^2} = 278.9 \text{ kips}$$

Because the applied load of 145 kips is less than P_{cr}, the column remains stable and has an overall factor of safety against buckling of 278.9/145 = 1.92.

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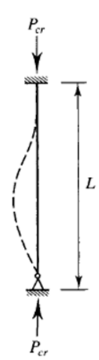
End Restraint and Effective Lengths

- *Effective length (KL):* is the distance between points of inflection (zero moments) in the buckled shape.

$$P_{cr} = \frac{\pi^2 EA}{(KL/r)^2} \quad \text{or} \quad P_{cr} = \frac{\pi^2 E_t A}{(KL/r)^2}$$

- Ex.:



k = 0.7 (Fixed – pinned)

$$P_{cr} = \frac{2.05\pi^2 EI}{L^2}$$

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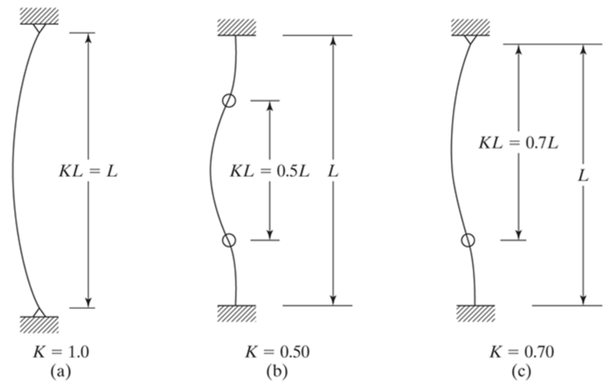
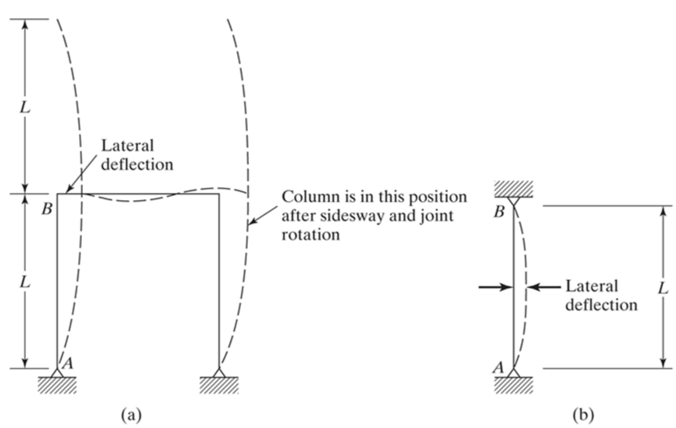
End Restraint and Effective Lengths

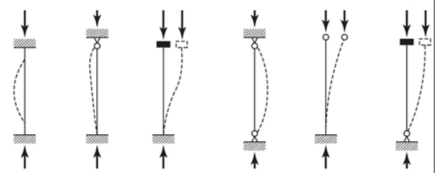


End Restraint and Effective Lengths



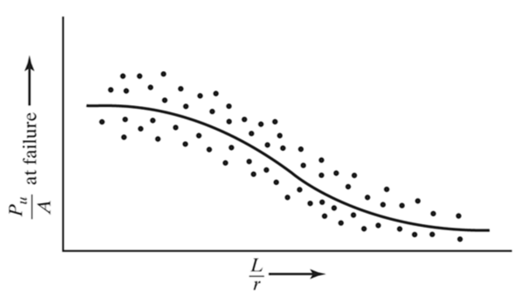
End Restraint and Effective Lengths

TABLE 5.1 Approximate Values of Effective Length Factor, K

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	*Rotation fixed and translation fixed* *Rotation free and translation fixed* *Rotation fixed and translation free* *Rotation free and translation free*					

Columns Formulas

- The testing of columns with various slenderness ratios results in scattered range of values.



Test result curve

Columns Formulas

- **<u>Column Types:</u>**

1. <u>Short columns:</u>

- No Buckling.
- Failure stress equal to yield stress.

2. <u>Intermediate columns:</u>

- Some of the fibers will reach yielding stress and some will not.
- Column will fail both by yielding and buckling (Inelastic).
- Most columns in this range.

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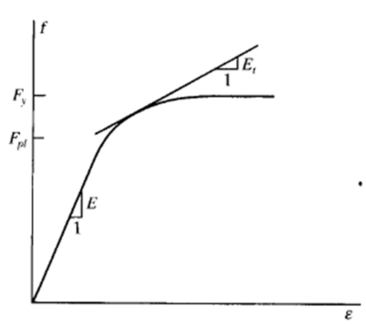
Columns Formulas

2. <u>Intermediate columns:</u>

$$P_{cr} = \frac{\pi^2 E_t I}{L^2}$$

E_t : Tangent modulus



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Columns Formulas

3. Long columns:

- Buckling will occur.
- Euler formula predicts the strength.
- Axial buckling stress below the proportional limit, i.e. Elastic.

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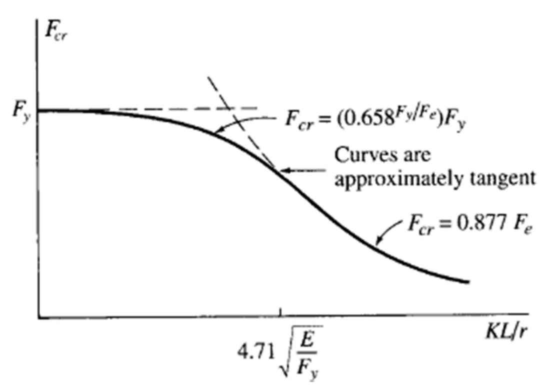
Columns Formulas



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Columns Formulas (AISC Requirements)

$$P_n = F_{cr} A_g$$

- P_n : Nominal Compressive strength.
- F_{cr}: Flexural Buckling Stress.
- A_g : Area Gross.

$$P_u \le \phi_c P_n$$

where

P_u = sum of the factored loads

ϕ_c = resistance factor for compression = 0.90

$\phi_c P_n$ = design compressive strength

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Columns Formulas (AISC Requirements)

- F_{cr} is determined as follows :
- (a) When $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ (or $F_e \ge 0.44F_y$)
-
  $$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$$

Inelastic Buckling

- (b) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ (or $F_e < 0.44F_y$)

  $$F_{cr} = 0.877 F_e$$

Elastic Buckling

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

F_e : Elastic (Euler) Buckling Stress

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Examples

- Example 1:
- A W 14 x 74 of A992 steel has a length of 20 feet and pinned ends. Compute the design compressive strength.

$(KL/r)_x = (1.0)(20)(12) / (6.04) = 39.73$

$$\text{Maximum } \frac{KL}{r} = \frac{KL}{r_y} = \frac{1.0(20 \times 12)}{2.48} = 96.77 < 200 \quad \text{(OK)}$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29,000}{50}} = 113$$

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Examples

Since 96.77 < 113, use AISC Equation E3-2.

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29,000)}{(96.77)^2} = 30.56 \text{ ksi}$$

$$F_{cr} = 0.658^{(F_y/F_e)} F_y = 0.658^{(50/30.56)}(50) = 25.21 \text{ ksi}$$

The nominal strength is

$$P_n = F_{cr}A_g = 25.21(21.8) = 549.6 \text{ kips}$$

The design strength is

$$\phi_c P_n = 0.90(549.6) = 495 \text{ kips}$$

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Examples

- <u>Solution by Table 4-22:</u>

Table 4–22
Available Critical Stress for Compression Members

	F_y = 35ksi			F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi	
$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
1	21.0	31.5	1	21.6	32.4	1	25.1	37.8	1	27.5	41.4	1	29.9	45.0
2	21.0	31.5	2	21.6	32.4	2	25.1	37.8	2	27.5	41.4	2	29.9	45.0
3	20.9	31.5	3	21.5	32.4	3	25.1	37.8	3	27.5	41.4	3	29.9	45.0
4	20.9	31.5	4	21.5	32.4	4	25.1	37.8	4	27.5	41.4	4	29.9	44.9
5	20.9	31.5	5	21.5	32.4	5	25.1	37.7	5	27.5	41.3	5	29.9	44.9
6	20.9	31.4	6	21.5	32.3	6	25.1	37.7	6	27.5	41.3	6	29.9	44.9
7	20.9	31.4	7	21.5	32.3	7	25.1	37.7	7	27.5	41.3	7	29.8	44.8
8	20.9	31.4	8	21.5	32.3	8	25.1	37.7	8	27.4	41.2	8	29.8	44.8
9	20.9	31.4	9	21.5	32.3	9	25.0	37.6	9	27.4	41.2	9	29.8	44.7

Examples

- <u>Solution by Table 4-22:</u>
- $(KL/r)y = 96.77$, $Fy = 50$ ksi
- Interpolation:

$\varphi c\ Fcr = 22.6 + (22.9 - 22.6 / 1.0)\ (0.23)$
$= 22.669$

$\varphi c\ Pn = (22.669)(21.8) = 494.18$ kips

Examples

- <u>Solution by Table 4-1:</u>

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

F_y = 50 ksi — W14

Shape		W14×											
Wt/ft		145		132		120		109		99		90	
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	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
least radius of gyration r_y	0	1280	1920	1160	1740	1060	1590	959	1440	872	1310	792	1190
	6	1250	1870	1130	1700	1030	1550	934	1400	849	1280	771	1160
	7	1240	1860	1120	1680	1020	1530	924	1390	840	1260	763	1150
	8	1220	1840	1110	1660	1010	1510	914	1370	831	1250	754	1130
	9	1210	1820	1090	1640	995	1500	902	1360	820	1230	745	1120
	10	1200	1800	1080	1620	981	1470	889	1340	808	1210	734	1100
	11	1180	1770	1060	1590	965	1450	875	1320	795	1200	722	1090
	12	1160	1740	1040	1570	949	1430	860	1290	781	1170	709	1070
	13	1140	1720	1020	1540	931	1400	844	1270	767	1150	696	1050
	14	1120	1690	1000	1510	912	1370	827	1240	751	1130	682	1020
	15	1100	1650	982	1480	893	1340	809	1220	734	1100	667	1000

Examples

- <u>Solution by Table 4-1:</u>

- KL = (1)(20) = 20 ft
- Fy = 50 ksi
- φc Pn = 494 kips

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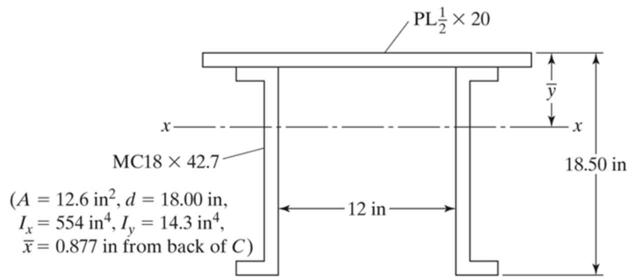
Examples

- Example 2:
- Fy = 50 ksi, Length of column = 23.75 ft, fixed-pinned. Determine the compressive design strength.



Examples

- $\bar{y}$ (from top) = [(20)(0.5)(0.25) + (2)(12.6)(9.5)] / [(20)(0.5)+(2)(12.6)]
  = 6.87 in
- Ix = (2)(554) + (2)(12.6)$(9.5 - 6.87)^2$ + [(20)$(0.5)^3$/12] + (20)(0.5)$(6.87-0.25)^2$
  = 1721 in^4
- Iy = (2)(14.3) + (2)(12.6)$(6+0.877)^2$ + [(0.5)$(20)^3$ / 12] = 1554 in^4

Examples

•

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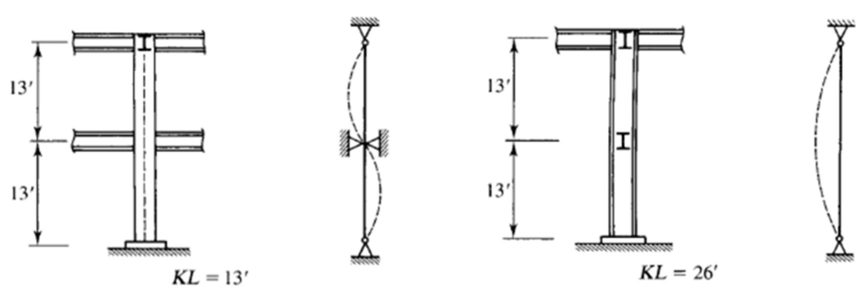
More on Effective Length



(a) Minor Axis Buckling (b) Major Axis Buckling

- Largest (KL/r) indicate the weakest direction and will be used in φc Fcr

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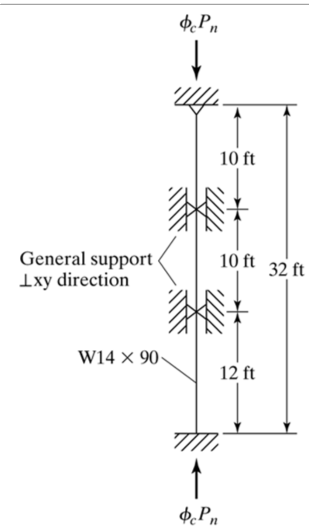
Example

- W14 x 90
- Fy = 50 ksi.
- No bracing on (x-x)
- Bracing on (y-y) as shown.

- Required:
- Design strength



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Example

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Example

- **Using Table 4-1:**
- KxLx = (0.8)(32) = 25.6 ft
- KyLy = (1.0)(10) = 10 ft
- Which one controls!!!!!
- KxLx/ rx = Equivalent KyLy/ry
- Equivalent KyLy = KxLx / (rx/ry)
- Control largest of KyLy AND KxLx/ (rx/ry)

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Example

- **Using Table 4-1:**
- Equivalent KyLy = KxLx / (rx/ry)
  = (0.8)(32)/ 1.66 =15.42 > KyLy =10 ft
- 15.42 **control**
- **Table:** Interpolation
- 1000 15 ft
  978 16 ft

φc Pn = 993 kips

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Local Buckling

- The AISC specifications for column strength assume that column buckling is the governing limit state. However, if the column section is made of thin (slender) plate elements, then failure can occur due to local buckling of the flanges or the webs.
- If local buckling of the individual plate elements occurs, then the column may not be able to develop its buckling strength.
- Therefore, the local buckling limit state must be prevented from controlling the column strength.

Local Buckling

- Local buckling depends on the slenderness (width-to-thickness b/t ratio) of the plate element and the yield stress (Fy) of the material.
- Each plate element must be stocky enough, i.e., have a b/t ratio that prevents local buckling from governing the column strength.
- Two Categories:

1. Stiffened elements: supported along both edges.
2. Unstiffened elements: unsupported along one edge parallel to the direction of the load.

TABLE 5.2 Limiting Width–Thickness Ratios for Compression Elements

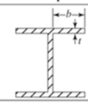
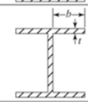
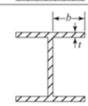
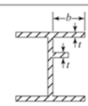
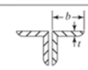
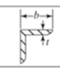
	Case	Description of Element	Width–Thickness Ratio	Limiting Width–Thickness Ratios: λ_p (compact)	λ_r (noncompact)	Example
Unstiffened Elements	1	Flexure in flanges of rolled I-shaped sections and channels	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_c E/F_L}$ [a],[b]	
	3	Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact and flanges of channels	b/t	NA	$0.56\sqrt{E/F_y}$	
	4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	NA	$0.64\sqrt{k_c E/F_y}$ [a]	
	5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45\sqrt{E/F_y}$	
	6	Flexure in legs of single angles	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	

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TABLE 5.2 (cont.) Limiting Width–Thickness Ratios for Compression Elements

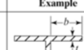
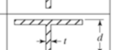
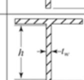
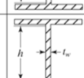
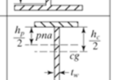
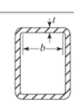
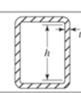
	Case	Description of Element	Width–Thickness Ratio	Limiting Width–Thickness Ratios: λ_p (compact)	λ_r (noncompact)	Example
Stiffened Elements	7	Flexure in flanges of tees	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	8	Uniform compression in stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$	
	9	Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	
	10	Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49\sqrt{E/F_y}$	
	11	Flexure in webs of single-symmetric I-shaped sections	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$	$5.70\sqrt{E/F_y}$	
	12	Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasterners or welds	b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	
	13	Flexure in webs of rectangular HSS	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	

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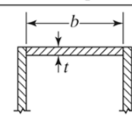
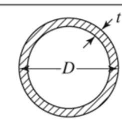
TABLE 5.2 (cont.) Limiting Width–Thickness Ratios for Compression Elements

Case	Description of Element	Width–Thickness Ratio	Limiting Width–Thickness Ratios λ_p (compact)	λ_r (noncompact)	Example
14	Uniform compression in all other stiffened elements	b/t	NA	$1.49\sqrt{E/F_y}$	
15	Circular hollow sections In uniform compression In flexure	 D/t D/t	 NA $0.07E/F_y$	 $0.11E/F_y$ $0.31E/F_y$	

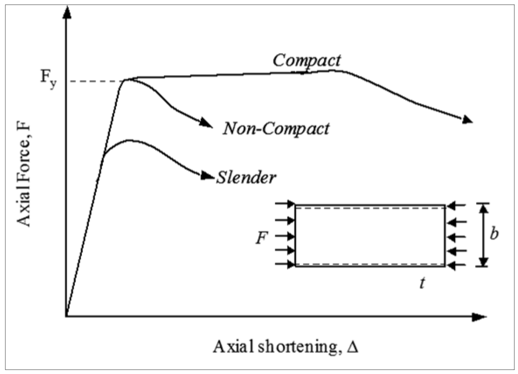
[a] $k_c = \frac{4}{\sqrt{h/t_w}}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.(See Cases 2 and 4.)

[b] $F_L = 0.7F_y$ for minor-axis bending, major-axis bending of slender-web built-up I-shaped members, and major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \geq 0.7$; $F_L = F_yS_{xt}/S_{xc} \geq 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$. (See Case 2.)

Local Buckling

- If the slenderness ratio (b/t) of the plate element is greater than λr then it is slender. It will locally buckle in the elastic range before reaching Fy.
- If the slenderness ratio (b/t) of the plate element is less than λr but greater than λp, then it is non-compact. It will locally buckle immediately after reaching Fy.
- If the slenderness ratio (b/t) of the plate element is less than λp, then the element is compact. It will locally buckle much after reaching Fy.

Local Buckling



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Local Buckling

- If all the plate elements of a cross-section are compact, then the section is compact.
- If any one plate element is non-compact, then the cross-section is non-compact.
- If any one plate element is slender, then the cross-section is slender.
- For W shape: $\lambda = bf/2 / tf$ (unstiffened)
  $\lambda = h/tw$ (stiffened)

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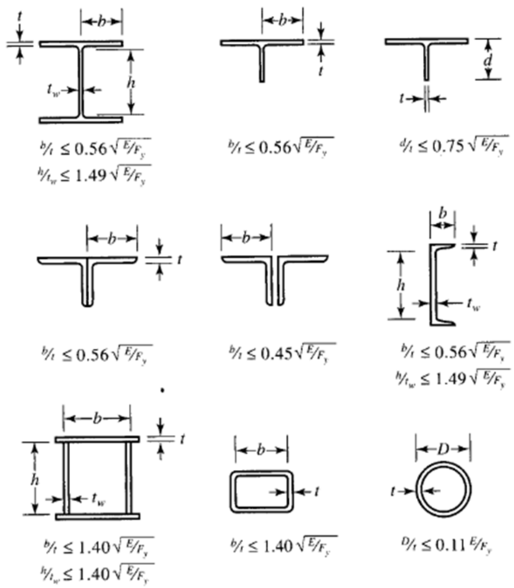

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Example

- Investigate the local buckling for W14 x 74, Fy = 50ksi.

For a W14×74, $b_f = 10.1$ in., $t_f = 0.785$ in., and

$$\frac{b_f}{2t_f} = \frac{10.1}{2(0.785)} = 6.43$$

$$0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29,000}{50}} = 13.5 > 6.43 \qquad \text{(OK)}$$

$$\frac{h}{t_w} = \frac{d - 2k_{des}}{t_w} = \frac{14.2 - 2(1.38)}{0.450} = 25.4$$

$$1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29,000}{50}} = 35.9 > 25.4 \qquad \text{(OK)}$$

Local instability is not a problem.

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Local Buckling

- The section that has local buckling, the strength of the section should be reduced.
  - If the width-thickness ratio λ is greater than λ_r, use the provisions of AISC E7 and compute a reduction factor Q.
  - Compute KL/r and F_e as usual.
  - If $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{QF_y}}$ or $F_e \geq 0.44QF_y$,

$$F_{cr} = Q\left(0.658^{\frac{QF_y}{F_e}}\right)F_y \qquad \text{(AISC Equation E7-2)}$$

  - If $\frac{KL}{r} > 4.71\sqrt{\frac{E}{QF_y}}$ or $F_e < 0.44QF_y$,

$$F_{cr} = 0.877F_e \qquad \text{(AISC Equation E7-3)}$$

  - The nominal strength is $P_n = F_{cr}A_g$ (AISC Equation E7-1)

Local Buckling

Q = Qs X Qa

- Qs : for unstiffened elements
- Qa : for stiffened elements
- If the shape has no slender unstiffened elements; Qs = 1.0.
- If the shape has no slender stiffened elements, Qa = 1.0.
- Most of the shapes used are **not slender**, and the reduction factor will not be needed. This includes most W-shapes.

Local Buckling

- **Slender unstiffened elements (Qs):**

A. For flanges, angles, and plates projecting from rolled columns or compression members:

(i) When $\frac{b}{t} \le 0.56\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0$$

(ii) When $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$

$$Q_s = 1.415 - 0.74\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}$$

(iii) When $b/t \ge 1.03\sqrt{E/F_y}$

$$Q_s = \frac{0.69E}{F_y\left(\frac{b}{t}\right)^2}$$

Local Buckling

B. For flanges, angles, and plates projecting from built up columns or compression members:

(i) When $\frac{b}{t} \le 0.64\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.0$$

(ii) When $0.64\sqrt{\frac{Ek_c}{F_y}} < b/t \le 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.415 - 0.65\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{Ek_c}}$$

(iii) When $b/t > 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = \frac{0.90Ek_c}{F_y\left(\frac{b}{t}\right)^2}$$

$$k_c = \frac{4}{\sqrt{h/t_w}},$$

$0.35 < kc < 0.76$

Local Buckling

C. For single angles:

(i) When $\frac{b}{t} \le 0.45\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0$$

(ii) When $0.45\sqrt{E/F_y} < b/t \le 0.91\sqrt{E/F_y}$

$$Q_s = 1.34 - 0.76\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}$$

(iii) When $b/t > 0.91\sqrt{E/F_y}$

$$Q_s = \frac{0.53E}{F_y\left(\frac{b}{t}\right)^2}$$

where

b = full width of longest angle leg, in. (mm)

Local Buckling

D. For stems of Tees:

b = width of unstiffened compression element, in. (mm)
d = the full nominal depth of tee, in. (mm)
t = thickness of element, in. (mm)

(i) When $\frac{d}{t} \le 0.75\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0$$

(ii) When $0.75\sqrt{\frac{E}{F_y}} < d/t \le 1.03\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.908 - 1.22\left(\frac{d}{t}\right)\sqrt{\frac{F_y}{E}}$$

(iii) When $d/t > 1.03\sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y\left(\frac{d}{t}\right)^2}$$

Local Buckling

- **Slender stiffened elements (Qa):**

$$Q_a = \frac{A_{eff}}{A}$$

A = total cross-sectional area of member, in.^{2} (mm^2)

A_{eff} = summation of the effective areas of the cross section based on the reduced *effective width*, b_e, in.^{2} (mm^2)

The reduced effective width, b_e, is determined as follows:

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Local Buckling

(a) For uniformly compressed slender elements, with $\frac{b}{t} \geq 1.49\sqrt{\frac{E}{f}}$, except flanges of square and rectangular sections of uniform thickness:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(E7-17)}$$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$.

(b) For flanges of square and rectangular *slender-element sections* of uniform thickness with $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(E7-18)}$$

where

$f = P_n/A_{eff}$

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Local Buckling

User Note: In lieu of calculating $f = P_n/A_{eff}$, which requires iteration, f may be taken equal to F_y. This will result in a slightly conservative estimate of column capacity.

(c) For axially-loaded circular sections:

When $0.11\frac{E}{F_y} < \frac{D}{t} < 0.45\frac{E}{F_y}$

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \qquad \text{(E7-19)}$$

where

D = outside diameter, in. (mm)
t = wall thickness, in. (mm)

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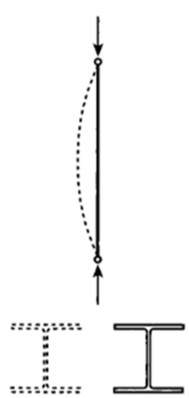
Torsional and Flexural-Torsional Buckling

- When an axially loaded compression member becomes unstable, it can buckle in one of three ways:

1. Flexural Buckling (already covered)



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Torsional and Flexural-Torsional Buckling

2. Torsional Buckling: This type of failure is caused by twisting about the longitudinal axis of the member.

- Standard hot rolled shapes are not susceptible to torsional buckling, but members built up from thin plate elements may be.

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Torsional and Flexural-Torsional Buckling

3. Flexural-Torsional Buckling: This type of failure is caused by combination of flexure and torsional buckling.

- This type of failure can occur only with unsymmetrical cross sections and one axis of symmetry.

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Torsional and Flexural-Torsional Buckling

(a) For double-angle and tee-shaped compression members:

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{\left(F_{cry} + F_{crz}\right)^2}}\right] \qquad \text{(E4-2)}$$

where F_{cry} is taken as F_{cr} from Equation E3-2 or E3-3, for *flexural buckling* about the y-axis of symmetry and $\dfrac{KL}{r} = \dfrac{KL}{r_y}$, and

$$F_{crz} = \frac{GJ}{A_g\bar{r}_o^2} \qquad \text{(E4-3)}$$

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Torsional and Flexural-Torsional Buckling

(b) For all other cases, F_{cr} shall be determined according to Equation E3-2 or E3-3, using the torsional or flexural-torsional elastic buckling *stress*, F_e, determined as follows:

(i) For doubly symmetric members:

$$F_e = \left[\frac{\pi^2 EC_w}{(K_zL)^2} + GJ\right]\frac{1}{I_x + I_y} \qquad \text{(E4-4)}$$

(ii) For singly symmetric members where y is the axis of symmetry:

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right] \qquad \text{(E4-5)}$$

(iii) For unsymmetric members, F_e is the lowest root of the cubic equation:

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \qquad \text{(E4-6)}$$

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Torsional and Flexural-Torsional Buckling

A_g = gross area of member, in.^{2} (mm^2)
C_w = warping constant, in.^{6} (mm^6)

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \quad \text{(E4-7)}$$

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \quad \text{(E4-8)}$$

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x}\right)^2} \quad \text{(E4-9)}$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2} \quad \text{(E4-10)}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \quad \text{(E4-11)}$$

G = shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)
I_x, I_y = moment of inertia about the principal axes, in.^{4} (mm^4)
J = torsional constant, in.^{4} (mm^4)
K_z = *effective length factor* for torsional buckling
x_o, y_o = coordinates of shear center with respect to the centroid, in. (mm)
$\bar{r}_o$ = polar radius of gyration about the shear center, in. (mm)
r_y = radius of gyration about y-axis, in. (mm)

Example

Example 1:

Compute the compressive strength of a WT 12 x 81 of A992 steel. The effective length with respect to x-axis is 25 ft 6 in, the effective length with respect to y-axis is 20 ft, and the effective length with respect to z-axis is 20 ft. (Use method b)

Example

Compute the flexural buckling strength for the *x*-axis:

$$\frac{K_x L}{r_x} = \frac{25.5 \times 12}{3.50} = 87.43$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29,000)}{(87.43)^2} = 37.44 \text{ ksi}$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29,000}{50}} = 113$$

Since $\frac{KL}{r} < 4.71\sqrt{\frac{E}{F_y}}$, AISC Equation E3-2 applies.

$$F_{cr} = 0.658^{(F_y/F_e)} F_y = 0.658^{(50/37.44)}(50) = 28.59 \text{ ksi}$$

The nominal strength is

$$P_n = F_{cr} A_g = 28.59(23.9) = 683.3 \text{ kips}$$

Example

Compute the flexural-torsional buckling strength about the *y*-axis (the axis of symmetry):

$$\frac{K_y L}{r_y} = \frac{20 \times 12}{3.05} = 78.69$$

$$F_{ey} = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29,000)}{(78.69)^2} = 46.22 \text{ ksi}$$

Because the shear center of a tee is located at the intersection of the centerlines of the flange and the stem,

$$x_0 = 0$$

$$y_0 = \bar{y} - \frac{t_f}{2} = 2.70 - \frac{1.22}{2} = 2.090 \text{ in.}$$

$$\bar{r}_0^2 = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} = 0 + (2.090)^2 + \frac{293 + 221}{23.9} = 25.87 \text{ in.}^2$$

$$F_{ez} = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right]\frac{1}{A_g \bar{r}_0^2}$$

$$= \left[\frac{\pi^2 (29,000)(43.8)}{(20 \times 12)^2} + 11,200(9.22)\right]\frac{1}{23.9(25.87)} = 167.4 \text{ ksi}$$

Example

$$F_{ey} + F_{ez} = 46.22 + 167.4 = 213.6 \text{ ksi}$$

$$H = 1 - \frac{x_0^2 + y_0^2}{\bar{r}_0^2} = 1 - \frac{0 + (2.090)^2}{25.87} = 0.8312$$

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right]$$

$$= \frac{213.6}{2(0.8312)}\left[1 - \sqrt{1 - \frac{4(46.22)(167.4)(0.8312)}{(213.6)^2}}\right] = 43.63 \text{ ksi}$$

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Example

$$0.44F_y = 0.44(50) = 22.0 \text{ ksi}$$

Since 43.63 ksi > 22.0 ksi, use AISC Equation E3-2.

$$F_{cr} = 0.658^{(F_y/F_e)} F_y = 0.658^{(50/37.44)}(50) = 28.59 \text{ ksi}$$

The nominal strength is

$$P_n = F_{cr}A_g = 30.95(23.9) = 739.7 \text{ kips}$$

The flexural buckling strength controls, and the nominal strength is 683.3 kips.

For LRFD, the design strength is $\phi_c P_n = 0.90(683.3) = 615$ kips.

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Design of Compression Members

- <u>Design Procedure:</u>

1. Assume a value for Fcr. (maximum = Fy)
2. Determine the required area.

$$\phi_c F_{cr} A_g \geq P_u$$

$$A_g \geq \frac{P_u}{\phi_c F_{cr}}$$

3. Select a trial shape that satisfy Ag.
4. Compute Fcr and φc Pn for the trial shape.
5. If the design strength is very close to the required value, the next tabulate size can be tried (If not, use Fcr from step 4 and Repeat)
6. Check Local and flexural-torsional buckling.

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Examples (Design of Compression Members)

- <u>Example 1:</u> Select a W18 shape of A992 steel that can resist a service dead load of 100 kips and a service live load of 300 kips. The effective length KL is 26 feet.

$P_u = 1.2D + 1.6L = 1.2(100) + 1.6(300) = 600$ kips

Try $F_{cr} = 33$ ksi (an arbitrary choice of two-thirds F_y):

$$\text{Required } A_g = \frac{P_u}{\phi_c F_{cr}} = \frac{600}{0.90(33)} = 20.2 \text{ in.}^2$$

Try a W18 × 71:

$$A_g = 20.8 \text{ in.}^2 > 20.2 \text{ in.}^2 \quad \text{(OK)}$$

$$\frac{KL}{r_{min}} = \frac{26 \times 12}{1.70} = 183.5 < 200 \quad \text{(OK)}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29{,}000)}{(183.5)^2} = 8.5 \text{ ksi}$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000}{50}} = 113$$

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Examples (Design of Compression Members)

Since $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$, AISC Equation E3-3 applies.

$$F_{cr} = 0.877F_e = 0.877(8.5) = 7.455 \text{ ksi}$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 0.90(7.455)(20.8) = 140 \text{ kips} < 600 \text{ kips} \qquad \text{(N.G.)}$$

- Try Fcr = 20 ksi.

$$\text{Required } A_g = \frac{P_u}{\phi_c F_{cr}} = \frac{600}{0.90(20)} = 33.3 \text{ in.}^2$$

Try a W18 × 119:

$$A_g = 35.1 \text{ in.}^2 > 33.3 \text{ in.}^2 \quad \text{(OK)}$$

$$\frac{KL}{r_{\min}} = \frac{26 \times 12}{2.69} = 116.0 < 200 \qquad \text{(OK)}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29{,}000)}{(116.0)^2} = 21.27 \text{ ksi}$$

Examples (Design of Compression Members)

Since $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}} = 113$, AISC Equation E3-3 applies.

$$F_{cr} = 0.877F_e = 0.877(21.27) = 18.65 \text{ ksi}$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 0.90(18.65)(35.1) = 589 \text{ kips} < 600 \text{ kips} \qquad \text{(N.G.)}$$

Try a W18 × 130:

$$A_g = 38.2 \text{ in.}^2$$

$$\frac{KL}{r_{\min}} = \frac{26 \times 12}{2.70} = 115.6 < 200 \qquad \text{(OK)}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29{,}000)}{(115.6)^2} = 21.42 \text{ ksi}$$

Since $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}} = 113$, AISC Equation E3-3 applies.

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Examples (Design of Compression Members)

$$F_{cr} = 0.877F_e = 0.877(21.42) = 18.79 \text{ ksi}$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 0.90(18.79)(38.2) = 646 \text{ kips} > 600 \text{ kips} \quad \text{(OK.)}$$

Use a W18 × 130.

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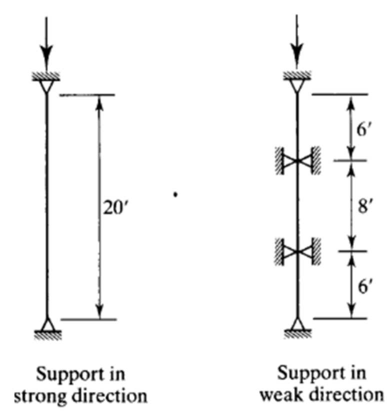
Examples (Design of Compression Members)

- Example 2: The column shown in the figure below is subjected to an ultimate load of 840 kips. Use A992 steel and select a W-shape.



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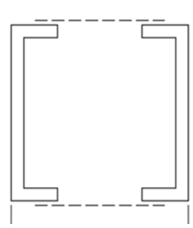
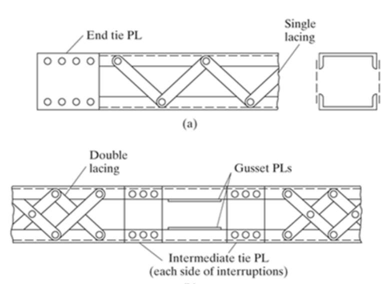
Examples (Design of Compression Members)

- KyLy = (1.0)(6.0) = 6.0

  KyLy = (1.0)(8.0) = 8.0 (<u>control in y</u>)
- Assume weak axis controls and using table 4-1 (Pu = 840kips, KyLy = 8.0 ft, Fy=50); Try W12 x 72 (φc Pn= 884>840)
- <u>Check which axis controls</u>:
- Eq. KyLy = (1.0)(20) / (1.75) =11.43 ft >8.0 ft
- KxLx controls
- Using table 4-1, KL = 11.43 , Try W12 X 79
- φc Pn >840 (Interpolation)

Built up Columns with Components not in Contact with Each Other





- <u>Purpose of Lattice work:</u>

1. To hold various parts in their position.
2. To equalize stress distribution between different parts.
3. To Prevent local Buckling.

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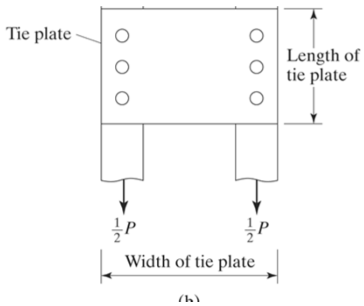
Built up Columns with Components not in Contact with Each Other

- Design of tie plates:



- Minimum thickness = B/50
- Minimum width = B + 2e
- e (edge distance) from Table.
- Minimum length = 2/3 B
- Maximum L/r = 200

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Built up Columns with Components not in Contact with Each Other

TABLE J3.4
Minimum Edge Distance,[a] in., from Center of Standard Hole[b] to Edge of Connected Part

Bolt Diameter (in.)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]
1/2	7/8	3/4
5/8	1 1/8	7/8
3/4	1 1/4	1
7/8	1 1/2 [d]	1 1/8
1	1 3/4 [d]	1 1/4
1 1/8	2	1 1/2
1 1/4	2 1/4	1 5/8
Over 1 1/4	1 3/4 × d	1 1/4 × d

[a] Lesser edge distances are permitted to be used provided provisions of Section J3.10, as appropriate, are satisfied.
[b] For oversized or slotted holes, see Table J3.5.
[c] All edge distances in this *column* are permitted to be reduced 1/8 in. when the hole is at a point where *required strength* does not exceed 25 percent of the maximum strength in the element.
[d] These are permitted to be 1 1/4 in. at the ends of *beam connection* angles and shear end plates.

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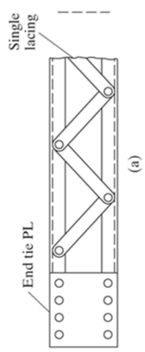
Built up Columns with Components not in Contact with Each Other

- <u>Design of Lacing:</u>

1. <u>Single Lacing:</u>

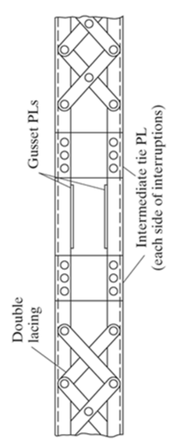


- B < 15 in
- Minimum β = 60
- Maximum KL/r = 140
- K = 1.0

Built up Columns with Components not in Contact with Each Other

- <u>Design of Lacing:</u>

2. <u>Double Lacing:</u>



- B > 15 in
- Minimum β = 45
- Maximum KL/r = 200
- K = 0.7
- Force on Lacing Bar ($V_u = 0.02 \times \varphi_c P_n$)

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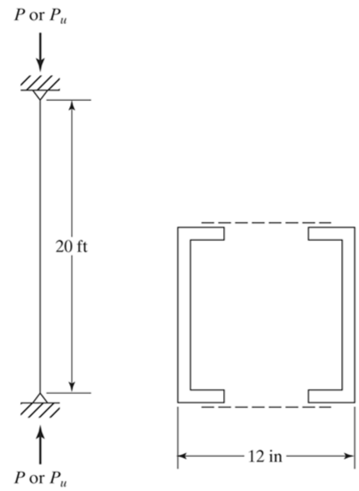
Example

- P dead = 100 kips
- P live = 300 kips
- Fy = 50 ksi
- Bolt diameter = ¾ in
- Design the lightest C12
- Design lacing and end plates.



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Example

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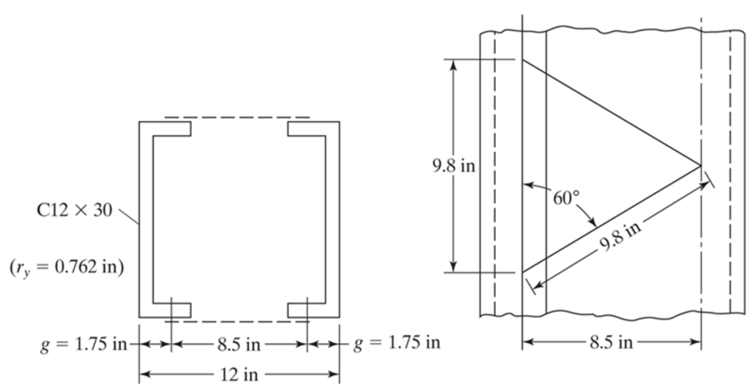
Example

- Check Local Buckling
- Use 2C 12 x 30
- <u>Design of Lacing:</u>
- B = 8.5 < 15 in (Single Lacing)
- Use β = 60
- Length = 8.5 / cos 30 = 9.8 in
- Vu = 0.02 (631) = 12.62 k
- 0.5 (12.62) = 6.31 k (shearing force on each plane of lacing)

Example



Example

•

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Example

- Req. Ag = 7.28 / 12.2 = 0.597
- Use (2.39 x 0.25)
- Min. e = 1.25 in
- Min. Length = 9.8 + (2)(1.25) = 12.3 in (use 14 in)
- Use 0.25 x 2.5 x 14

- <u>Design of End Tie Plate:</u>
- Min. Length = 8.5 in
- Min. t = 8.5/50 = 0.17 in
- Min. Width = 8.5 + (2)(1.25) = 11 in
- Use 3/16 x 8.5 x 12 in

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Effective Length Using Alignment Charts

- The resistance to rotation furnished by the beams and girders meeting at one end of a column is dependent on the rotational stiffnesses of those members.
- Rotational stiffness: The moment needed to produce a unit rotation at one end of a member if the other end is fixed. (4EI/L)
- $$G = \frac{\sum E_c I_c / L_c}{\sum E_g I_g / L_g} = \frac{\sum I_c / L_c}{\sum I_g / L_g}$$

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Effective Length Using Alignment Charts

- $\Sigma E_c I_c / L_c$ = sum of the stiffnesses of all columns at the end of the column under consideration

  $\Sigma E_g I_g / L_g$ = sum of the stiffnesses of all girders at the end of the column under consideration

  $E_c = E_g = E$, the modulus of elasticity of structural steel.

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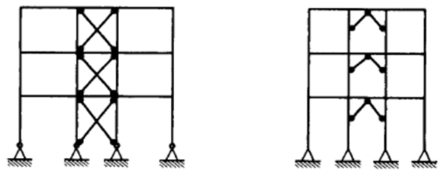
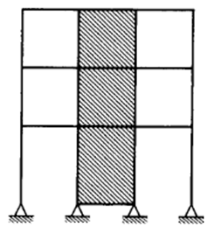
Effective Length Using Alignment Charts



(a) Diagonal Bracing



(b) Shear Walls
(masonry, reinforced concrete, or steel plate)

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Effective Length Using Alignment Charts

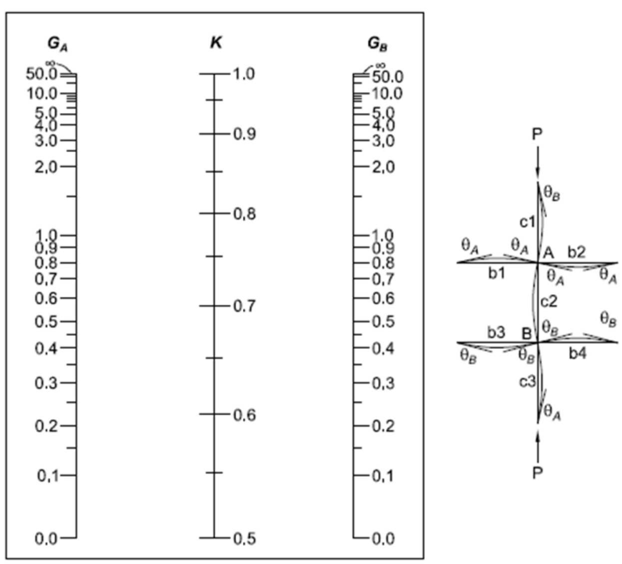

Fig. C-C2.3. Alignment chart—sidesway inhibited (braced frame).

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Effective Length Using Alignment Charts

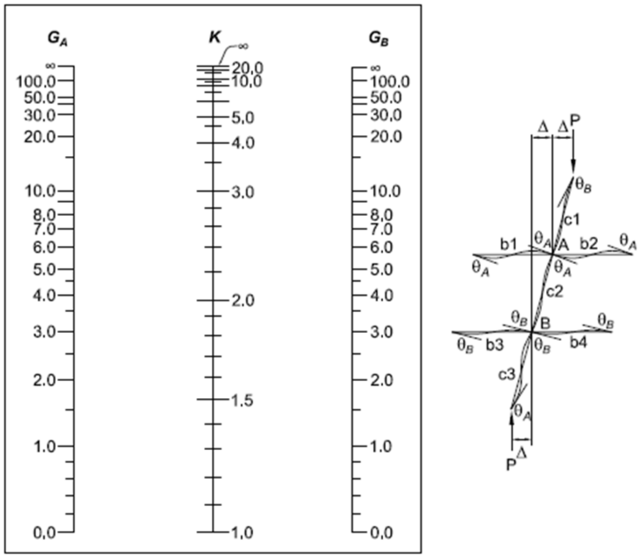

Fig. C-C2.4. Alignment chart—sidesway uninhibited (moment frame).

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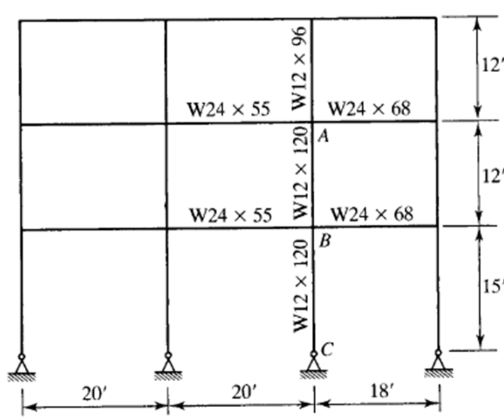
EXAMPLE

- Unbraced frame.
- Determine Kx for columns AB and BC.



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EXAMPLE

Column *AB:*

For joint *A*,

$$G = \frac{\sum I_c/L_c}{\sum I_g/L_g} = \frac{833/12 + 1070/12}{1350/20 + 1830/18} = \frac{158.6}{169.2} = 0.94$$

For joint *B*,

$$G = \frac{\sum I_c/L_c}{\sum I_g/L_g} = \frac{1070/12 + 1070/15}{169.2} = \frac{160.5}{169.2} = 0.95$$

From the alignment chart for sidesway uninhibited (AISC Figure C-C2.4), with $G_A = 0.94$ and $G_B = 0.95$, $K_x = 1.3$ for column *AB*.

For column *BC:*

For joint *B*, as before,

$G = 0.95$

From the alignment chart with $G_A = 0.95$ and $G_B = 10.0$, $K_x = 1.85$ for column *BC*.

Inelastic

$$G_{\text{inelastic}} = \frac{\sum E_t I_c/L_c}{\sum E I_g/L_g} = \frac{E_t}{E} G_{\text{elastic}}$$

- Stiffness reduction factor (ζa), Table 4-21 in the manual.

$$F_{cr} = \frac{P_u}{\phi_c A_g} \quad \text{for LRFD}$$

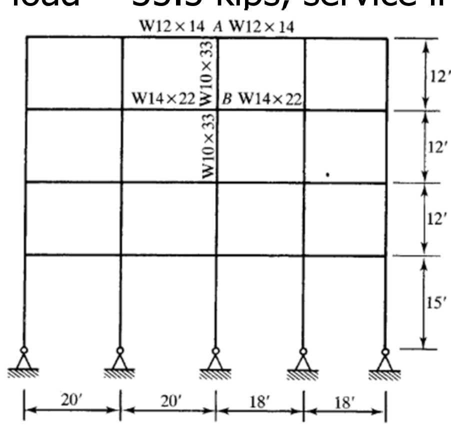
Inelastic

Table 4–21
Stiffness Reduction Factor τ_a

ASD $\frac{P_a}{A_g}$ / LRFD $\frac{P_u}{A_g}$	F_y = 35 ksi ASD	35 LRFD	36 ASD	36 LRFD	42 ASD	42 LRFD	46 ASD	46 LRFD	50 ASD	50 LRFD
45	–	–	–	–	–	–	–	–	–	–
44	–	–	–	–	–	–	–	–	–	0.0599
43	–	–	–	–	–	–	–	–	–	0.118
42	–	–	–	–	–	–	–	–	–	0.175
41	–	–	–	–	–	–	–	0.0262	–	0.231
40	–	–	–	–	–	–	–	0.0905	–	0.285
39	–	–	–	–	–	–	–	0.153	–	0.338
38	–	–	–	–	–	–	–	0.214	–	0.389
37	–	–	–	–	–	0.0570	–	0.274	–	0.438
36	–	–	–	–	–	0.127	–	0.331	–	0.486
35	–	–	–	–	–	0.194	–	0.387	–	0.532
34	–	–	–	–	–	0.260	–	0.441	–	0.577
33	–	–	–	–	–	0.323	–	0.492	–	0.620
32	–	–	–	0.0334	–	0.384	–	0.542	–	0.660
31	–	0.0429	–	0.115	–	0.443	–	0.590	–	0.699
30	–	0.127	–	0.194	–	0.500	–	0.636	–	0.736

EXAMPLE

- Unbraced Frame
- Determine the effective length factors for member AB.
- Service dead load = 35.5 kips, service live load = 142 kips.
- A992 steel



EXAMPLE

For joint A,

$$\frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} = \frac{171/12}{88.6/20 + 88.6/18} = \frac{14.25}{9.352} = 1.52$$

For joint B,

$$\frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} = \frac{2(171/12)}{199/20 + 199/18} = \frac{28.5}{21.01} = 1.36$$

- Kx = 1.45 (for elastic)

EXAMPLE

$$\frac{K_x L}{r_x} = \frac{1.45(12 \times 12)}{4.19} = 49.83$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000}{50}} = 113$$

Since

$$\frac{K_x L}{r_x} < 4.71\sqrt{\frac{E}{F_y}}$$

behavior is inelastic, and the inelastic K factor can be used.

The factored load is

$$P_u = 1.2D + 1.6L = 1.2(35.5) + 1.6(142) = 269.8 \text{ kips}$$

Enter Table 4-21 in Part 4 of the *Manual* with

$$\frac{P_u}{A_g} = \frac{269.8}{9.71} = 27.79 \text{ ksi}$$

and obtain the stiffness reduction factor $\tau_a = 0.8105$ by interpolation.

EXAMPLE

For joint A,

$$G_{\text{inelastic}} = \tau_a \times G_{\text{elastic}} = 0.8105(1.52) = 1.23$$

For joint B,

$$G_{\text{inelastic}} = 0.8105(1.36) = 1.10$$

From the alignment chart, $K_x = 1.35$.

STEEL DESIGN

Chapter 5:
Analysis and Design of Beams

Introduction

- Beams are members that are subjected to transverse loads. (Chapter F in the specifications in the manual)
- Types of beams:

1. Used in buildings.
2. Used in roofs and walls.
3. Used in bridges.

Introduction

1. Used in buildings:

A. Joists: Closely spaced beams supporting floors of buildings.

B. Lintels: On top of doors and windows.

C. Spandrel beams: To support external walls.

Introduction

2. Used in roofs and walls:

A. Purlins: on roofs.
B. Girts: on walls.

3. Used in bridges:

A. Stringers: beams running parallel to the roadway.
B. Floor beams: large beams which are perpendicular to the roadway.

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Introduction

- Types of sections:

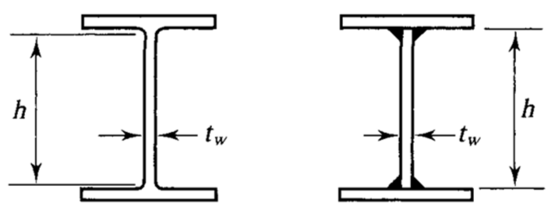
1. Rolled steel sections:

- W (most economical, used in this class), S, C, T, L.

2. Built up sections:

- Plate girder.

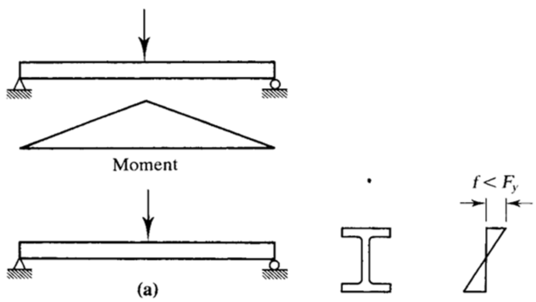
Introduction



- If h/tw < 5.70 $(E/Fy)^{0.5}$: It is considered beam.
- If h/tw > 5.70 $(E/Fy)^{0.5}$: It is considered plate girder

Plastification of Cross-section under Pure Bending

1. Elastic beam theory:

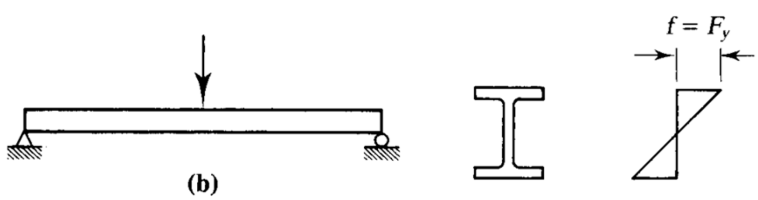


$$f_b = \frac{My}{I_x}$$

- Plane section before bending remains plane after bending.

Plastification of Cross-section under Pure Bending

2. Increasing Load (Moment), reaching yielding:



$$M_y = F_y S_x$$

- Reaches yield stress and strain.
- Still linear.
- Sx : Section modulus (I/c)

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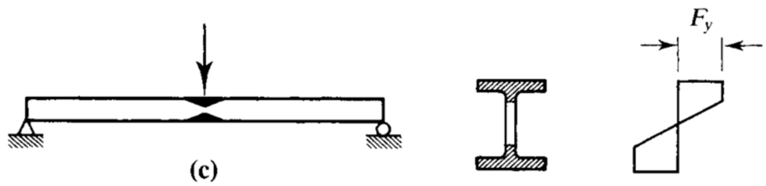
Plastification of Cross-section under Pure Bending

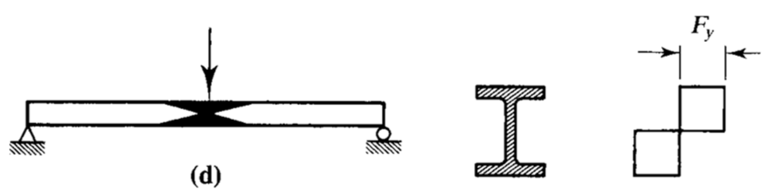
3. Increasing Load (Moment), beyond yielding:



- Elastic zone (core).
- Plastic zone.

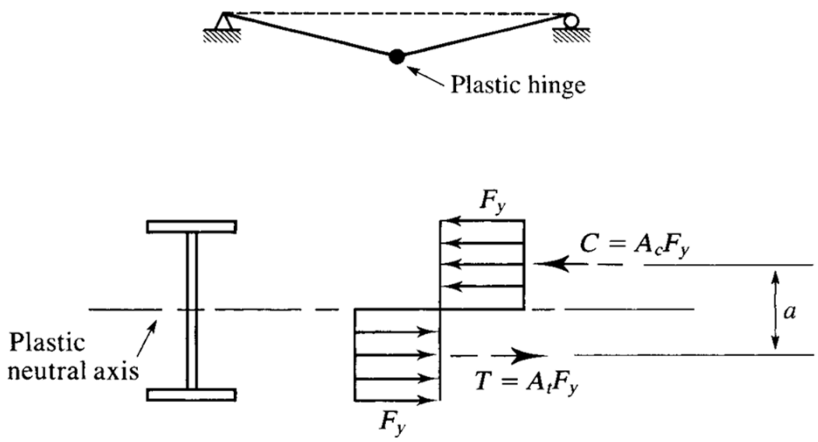
Plastification of Cross-section under Pure Bending

4. Plastic moment (Mp):

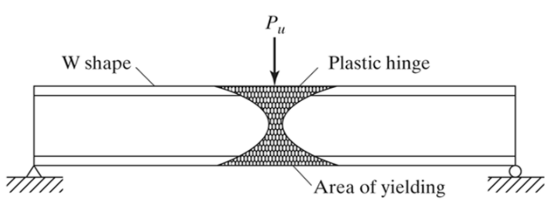


- Strain and stress reach yielding every where.
- Full Plastic zone, Pu, Mp.
- Formation of plastic hinge.

Plastification of Cross-section under Pure Bending



Plastification of Cross-section under Pure Bending



Plastification of Cross-section under Pure Bending

$$C = T$$
$$A_c F_y = A_t F_y$$
$$A_c = A_t$$

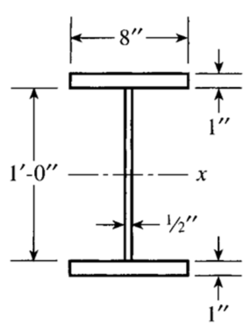
$$M_p = F_y(A_c)a = F_y(A_t)a = F_y\left(\frac{A}{2}\right)a = F_y Z$$

A = total cross-sectional area
a = distance between the centroids of the two half-areas
$Z = \left(\frac{A}{2}\right)a$ = plastic section modulus

Examples

- *Example 1:* A572, Grade 50. Determine:

1. Yielding moment and elastic section modulus.
2. Plastic moment and plastic section modulus.



Examples

1.

Component	$\bar{I}$	A	d	$\bar{I} + Ad^2$
Flange	0.6667	8	6.5	338.7
Flange	0.6667	8	6.5	338.7
Web	72	—	—	72.0
Sum				749.4

The elastic section modulus is

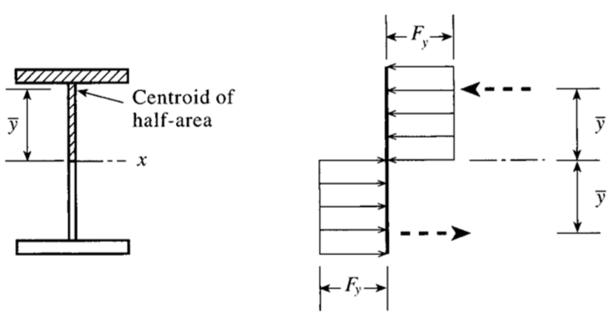
$$S = \frac{I}{c} = \frac{749.4}{1 + (12/2)} = \frac{749.4}{7} = 107 \text{ in.}^3$$

and the yield moment is

$$M_y = F_y S = 50(107) = 5350 \text{ in.-kips} = 446 \text{ ft-kips}$$

Examples

2.



Component	A	y	Ay
Flange	8	6.5	52
Web	3	3	9
Sum	11		61

$$\bar{y} = \frac{\sum Ay}{\sum A} = \frac{61}{11} = 5.545 \text{ in.}$$

Examples

2.

$$a = 2\bar{y} = 2(5.545) = 11.09 \text{ in.}$$

and that the plastic section modulus is

$$\left(\frac{A}{2}\right)a \;=\; 11(11.09) \;=\; 122 \text{ in.}^3$$

The plastic moment is

$$M_p = F_y Z = 50(122) = 6100 \text{ in.-kips} = 508 \text{ ft-kips}$$

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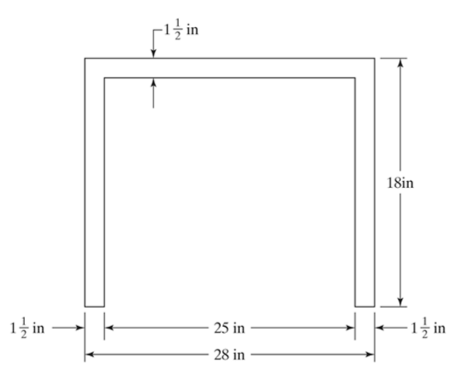
Examples

- *Example 2:*
- Determine plastic section modulus.



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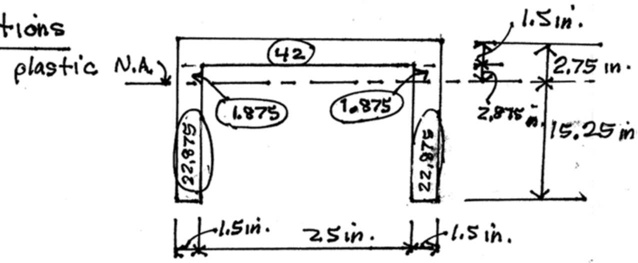
Examples

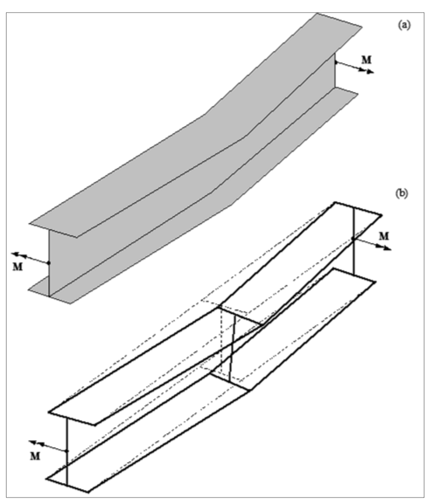
Plastic calculations



$$Z_x = (2)(22.875)\left(\frac{15.25}{2}\right) + (2)(1.875)\left(\frac{1.25}{2}\right) + (42)\left(1.25 + \frac{1.5}{2}\right) = 435.2\ in.^3$$

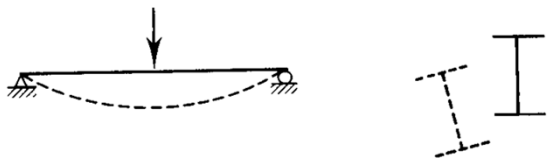
Basic Definitions

1. *Lateral Torsional Buckling (LTB):*



Basic Definitions

- Mn = Mp (if the beam remains stable up to the fully plastic condition)



(a)

(b)

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Basic Definitions

- Lateral torsional buckling will not occur if the compression flange of a member is braced or if twisting of the beam is prevented at frequent intervals.
- Two Categories of lateral support:

1. Continuous lateral support by embedment of the compression flange in a concrete slab.
2. Lateral support at short or long intervals.

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Basic Definitions

2. Classifications of Shapes :

A. Compact sections:

- Capable of developing a fully plastic stress distribution before buckling.
- If $\lambda \leq \lambda p$ and the flange is continuously connected to the web.

B. Non- Compact sections:

- Yield stress can be reached in some but not all of the elements.
- If $\lambda p < \lambda \leq \lambda r$

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Basic Definitions

C. Slender sections:

- If $\lambda > \lambda r$

Element	λ	λ_p	λ_r
Flange	$\frac{b_f}{2t_f}$	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$
Web	$\frac{h}{t_w}$	$3.76\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$

*For hot-rolled I shapes in flexure.

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Bending Strength of Compact Shapes

- Beams can fail by reaching Mp and become fully plastic , or it can fail by:

1. Lateral Torsional Buckling (LTB), elastic or inelastic.
2. Flange Local Buckling (FLB), elastic or inelastic.
3. Web Local Buckling (WLB), elastic or inelastic.

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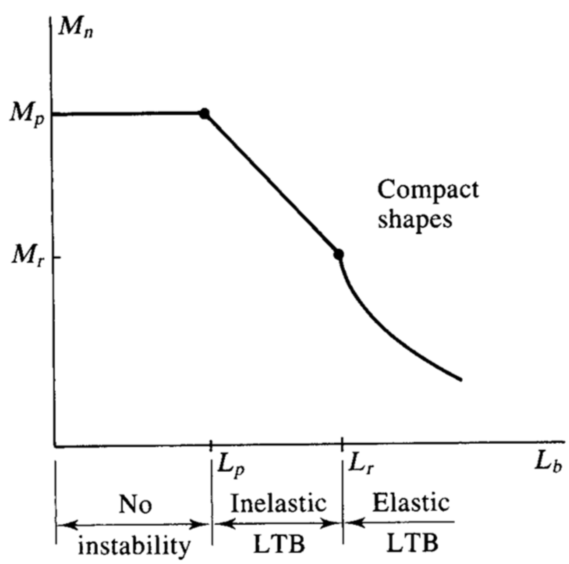
Bending Strength of Compact Shapes



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Bending Strength of Compact Shapes

1. Plastic Behavior (Zone 1):

- Compact Shape.
- Compression flange is continuously braced laterally or lateral bracing are provided at short intervals, i.e.

Lb > Lp

- Where

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

- For $L_b \le L_p$,
  $M_n = M_p$ $\quad M_n = M_p = F_y Z_x$, φb =0.9

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Bending Strength of Compact Shapes

- For bending about minor axis (y-y); there is no (LTB) in the doubly symmetrical sections.
- Always zone 1.
- Φb Mn = φb Mp
  = φb Fy Zy < φb 1.6 Fy Sy

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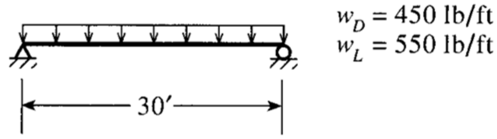
Examples (Zone 1)

- Example 1:

- W16 x 31, A992 steel.
- Continuous Lateral Support.
- Is the beam adequate in flexure?



Examples (Zone 1)

$$\frac{b_f}{2t_f} = 6.28 \qquad \text{(from Part 1 of the } \textit{Manual}\text{)}$$

$$0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29,000}{50}} = 9.15 > 6.28 \quad \therefore \text{ the flange is compact}$$

$$\frac{h}{t_w} < 3.76\sqrt{\frac{E}{F_y}}$$

(The web is compact for all shapes in the *Manual* for $F_y \leq 65$ ksi.)

$\therefore$ a W16 $\times$ 31 is compact for $F_y = 50$ ksi.

Examples (Zone 1)

$$M_n = M_p = F_y Z_x = 50(54.0) = 2700 \text{ in.-kips} = 225.0 \text{ ft kips.}$$

$$w_D = 450 + 31 = 481 \text{ lb/ft}$$

$$w_u = 1.2w_D + 1.6w_L = 1.2(0.481) + 1.6(0.550) = 1.457 \text{ kips/ft}$$

$$M_u = \frac{1}{8} w_u L^2 = \frac{1}{8}(1.457)(30)^2 = 164 \text{ ft-kips}$$

$$\phi_b M_n = 0.90(225.0) = 203 \text{ ft-kips} > 164 \text{ ft-kips} \qquad \text{(OK)}$$

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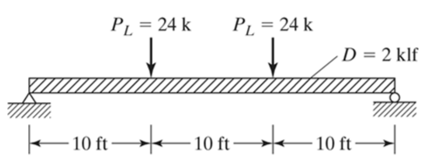
Examples (Zone 1)

- Example 2:
- Fy = 50 ksi.
- Continuous lateral support.
- Design.



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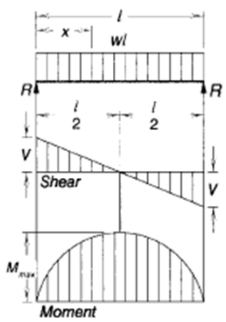
Examples (Zone 1)

- Case 1 + Case 9 Table 3-23 in the manual.

Table 3–23
Shears, Moments, and Deflections

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD

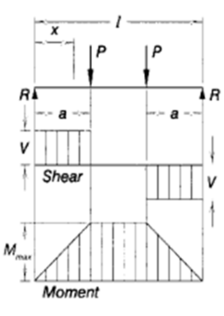


Total Equiv. Uniform Load	$= wl$
$R = V$	$= \frac{wl}{2}$
V_x	$= w\left(\frac{l}{2} - x\right)$
M_{max} (at center)	$= \frac{wl^2}{8}$
M_x	$= \frac{wx}{2}(l - x)$
Δ_{max} (at center)	$= \frac{5wl^4}{384EI}$
Δ_x	$= \frac{wx}{24EI}\left(l^3 - 2lx^2 + x^3\right)$

Examples (Zone 1)

- Case 1 + Case 9 Table 3-23 in the manual.

9. SIMPLE BEAM — TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



Total Equiv. Uniform Load	$= \frac{8Pa}{l}$
$R = V$	$= P$
M_{max} (between loads)	$= Pa$
M_x (when $x < a$)	$= Px$
Δ_{max} (at center)	$= \frac{Pa}{24EI}\left(3l^2 - 4a^2\right)$
Δ_{max} (when $a = \frac{l}{3}$)	$= \frac{Pl^3}{28EI}$
Δ_x (when $x < a$)	$= \frac{Px}{6EI}\left(3la - 3a^2 - x^2\right)$
Δ_x (when $a < x < (l - a)$)	$= \frac{Pa}{6EI}\left(3lx - 3x^2 - a^2\right)$

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Examples (Zone 1)

- Case 1 + Case 9 Table 3-23 in the manual.
- Wu = 1.2 (2) = 2.4 k /ft
- Pu = 1.6 (24) = 38.4 k
- Mu = $(2.4)(30)^2$ / 8 + (38.4)(10) = 654 k-ft
- or :

  Zreq. = (654)(12) / (0.9)(50) = 174.4 in^3
- Using Table 3-2, try W24 x 68, Zx = 177 > 174.4

Examples (Zone 1)

F_y = 50 ksi

Table 3–2
W Shapes
Selection by Z_x

Z_X

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W36×800^h	3650	9110	13700	5310	7980	47.5	71.4	14.9	94.8	64700	2030	3040
W36×652^h	2910	7260	10900	4300	6460	46.8	70.4	14.5	77.8	50600	1620	2430
W40×593^h	2760	6890	10400	4090	6140	55.5	83.5	13.4	63.8	50400	1540	2310
W36×529^h	2330	5810	8740	3480	5220	46.5	70.0	14.1	64.4	39600	1280	1920
W40×503^h	2310	5760	8660	3460	5200	54.7	82.2	13.1	55.3	41600	1290	1940
W36×487^h	2130	5310	7990	3200	4800	46.1	69.3	14.0	60.0	36000	1180	1770

Examples (Zone 1)

- $W_u = 1.2\,(2+0.068) = 2.4816$ k/ft
- $P_u = 38.4$ k
- $M_u = (2.4816)(30)^2 / 8 + (38.4)(10) = 663.18$ k-ft
- $Z_{req.} = (663.18)(12) / (0.9)(50)$
  $= 176.8 < 177$ (O.K)
- Use W24 x 68

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Bending Strength of Compact Shapes

2. Inelastic LTB (Zone 2):

- The bracing is insufficient to permit the member to reach a full plastic distribution.
- Yield strain is reached in some but not all of its compression elements.

$$L_p < L_b < L_r$$

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Bending Strength of Compact Shapes

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7 F_y S_x h_o}{EJc} \right)^2}}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x}$$

and

For a doubly symmetric I-shape: $c = 1$

For a channel: $c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}}$

where

h_o = distance between the flange centroids, in. (mm)

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Bending Strength of Compact Shapes

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
J = torsional constant, in.^{4} (mm^4)
S_x = elastic section modulus taken about the x-axis, in.^{3} (mm^3)

- Cw = Warping constant, in^4
- The moment capacity in zone (2) is:

$$M_n = C_b \left[M_p - (M_p - 0.7 F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le M_p$$

- Or : φb Mn = Cb [φb Mp – BF (Lb- Lp)] ≤ φb Mp

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Bending Strength of Compact Shapes

- Cb = Modification factor for non-uniform moment diagrams, when both ends of the beam segment are braced.

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} R_m \leq 3.0$$

M_{max} = absolute value of the maximum moment within the unbraced length (including the end points)
M_A = absolute value of the moment at the quarter point of the unbraced length
M_B = absolute value of the moment at the midpoint of the unbraced length
M_C = absolute value of the moment at the three-quarter point of the unbraced length

Bending Strength of Compact Shapes

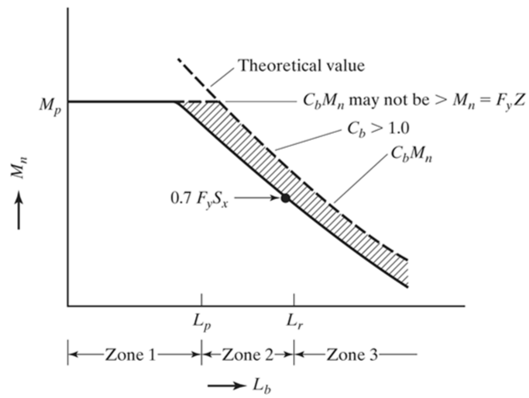
R_m = 1.0 for doubly-symmetric cross sections (such as W shapes) and singly symmetric shapes (such as channels) subject to single-curvature bending

$= 0.5 + 2\left(\frac{I_{yc}}{I_y}\right)^2$ for singly-symmetric shapes subject to reverse-curvature bending

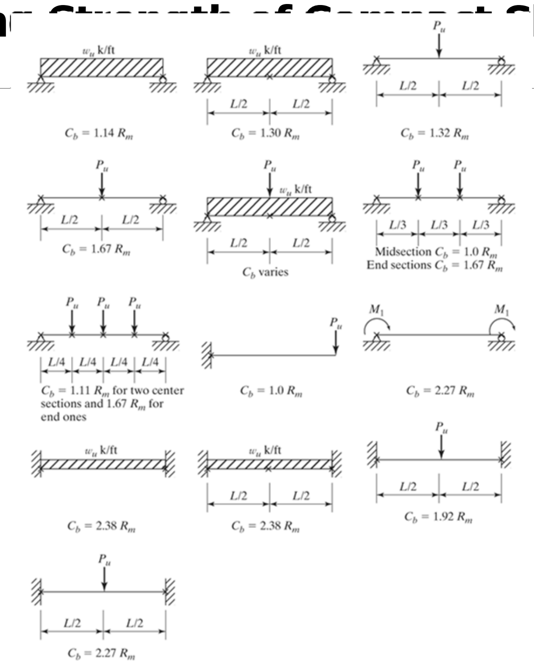
I_{yc} = moment of inertia of the compression flange about the y axis. For doubly-symmetric shapes, $I_{yc} \approx I_y/2$. For reverse-curvature bending of singly-symmetric I-shaped sections, I_{yc} is the moment of inertia of the smaller flange.

Bending Strength of Compact Shapes

- Cb is used to account for the effect of different moment gradients on LTB.
- Cb = 1.0 for Cantilever and over hangs.



Bendin… hapes



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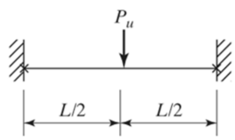
Bending Strength of Compact Shapes

- <u>Example (Cb):</u>
- Bracing at ends



- Mmax = PL/8
- MA = 0
- MB = PL/8
- MC = 0
- Cb = (12.5)(PL/8) / (2.5PL/8 + 0 + 4 PL/8 + 0)
  = 1.92

Example (Zone 2)

- <u>Example 1:</u>
- W12 x 30, A992 steel.
- Lb = 10 ft
- Cb = 1.0
- Compute the flexural design strength.

Example (Zone 2)

•

Example (Zone 2)

•

Example (Zone 2)

- Mn = 1.0 [(50)(43.1) – {(50)(43.1) – (0.7)(50)(38.6)} [(10-5.369) / (15.6-5.369)]
  = 1791.1 kip-in < Mp = 2155 kip-in
- Φb Mn = (0.9)(1791.1)
  = 1611.99 kip-in
  = 134.33 kip-ft

Example (Zone 2)

- Example 2:
- Mu = 290k-ft.
- Fy = 50 ksi
- Lb = 10 ft
- Cb = 1.0
- Design.

Example (Zone 2)

Using Table 3-2

F_y = 50 ksi

Table 3–2
W Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W36×800[h]	3650	9110	13700	5310	7980	47.5	71.4	14.9	94.8	64700	2030	3040
W36×652[h]	2910	7260	10900	4300	6460	46.8	70.4	14.5	77.8	50600	1620	2430
W40×593[h]	2760	6890	10400	4090	6140	55.5	83.5	13.4	63.8	50400	1540	2310
W36×529[h]	2330	5810	8740	3480	5220	46.5	70.0	14.1	64.4	39600	1280	1920
W40×503[h]	2310	5760	8660	3460	5200	54.7	82.2	13.1	55.3	41600	1290	1940
W36×487[h]	2130	5310	7990	3200	4800	46.1	69.3	14.0	60.0	36000	1180	1770
W40×431[h]	1960	4890	7350	2950	4440	53.6	80.6	12.9	49.0	34800	1110	1660

Example (Zone 2)

- Try W 18 x 40: (assume Zone 1)
- Φb Mp = 294 k-ft,
- Lp = 4.49 ft,
- BF = 13.3,
- Lr = 13.1 ft
- Zone 2 (4.49< 10 <13.1)
- Φb Mn = 1.0 [294 – (13.3)(10-4.49)]
  = 220.72 <290 (N.G)

Example (Zone 2)

- Try W 21 x 44:
- Φb Mp = 358 k-ft,
- Lp = 4.45 ft,
- BF = 16.8,
- Lr = 13 ft
- Zone 2 (4.45< 10 <13.0)
- Φb Mn = 1.0 [358 – (16.8)(10-4.45)]
  = 264.76 <290 (N.G)

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Example (Zone 2)

- Try W 21 x 48:
- Φb Mp = 398 k-ft,
- Lp = 6.09 ft,
- BF = 14.7,
- Lr = 16.6 ft
- Zone 2 (6.09< 10 <16.6)
- Φb Mn = 1.0 [398 – (14.7)(10-6.09)]
  = 340.52 > 290 (O.K)

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Bending Strength of Compact Shapes

3. Elastic LTB (Zone 3):

Lb > Lr

$$M_n = F_{cr} S_x \le M_p$$

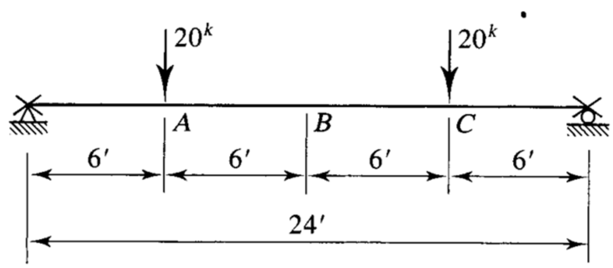
where

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$

Examples

- *Example 1: (Design by Charts)*

- A572, Grade 50.
- Bracing at ends.
- Design by charts.



Examples

$M_A = M_B = M_C = M_{max}, \quad \therefore\ C_b = 1.0$

$M_u = 6(1.6 \times 20) = 192$ ft-kips

- From charts page 3-126:

From the charts, with $L_b = 24$ ft, **try W12 × 53**:

$\phi_b M_n = 209$ ft-kips $>$ 192 ft-kips (OK)

Now, we account for the beam weight:

$$M_u = 192 + \frac{1}{8}(1.2 \times 0.053)(24)^2 = 197 \text{ ft-kips} < 209 \text{ ft-kips} \quad \text{(OK)}$$

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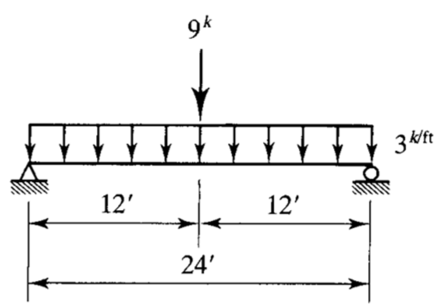
Examples

- Example 2:
- A992 steel.
- Lateral bracing at the ends and at mid span.
- 30% dead load, 70% live load.
- Design.



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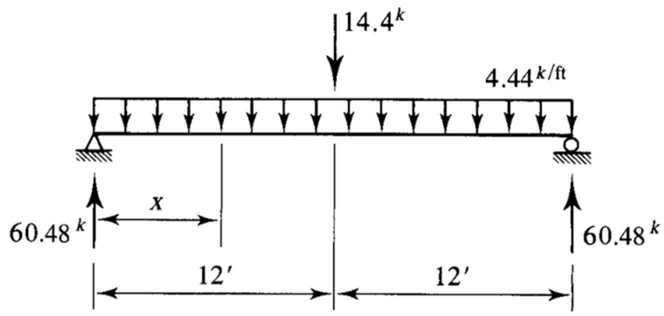
Examples

$w_u = 1.2(0.9) + 1.6(2.1) = 4.44 \text{ kips/ft}$
$P_u = 1.6(9) = 14.4 \text{ kips}$



$$M = 60.48x - 4.44x\left(\frac{x}{2}\right) = 60.48x - 2.22x^2 \quad (\text{for } x \le 12 \text{ ft})$$

Examples

For $x = 3$ ft, $M_A = 60.48(3) - 2.22(3)^2 = 161.5$ ft-kips
For $x = 6$ ft, $M_B = 60.48(6) - 2.22(6)^2 = 283.0$ ft-kips
For $x = 9$ ft, $M_C = 60.48(9) - 2.22(9)^2 = 364.5$ ft-kips
For $x = 12$ ft, $M_{max} = M_u = 60.48(12) - 2.22(12)^2 = 406.1$ ft-kips

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$
$$= \frac{12.5(406.1)}{2.5(406.1) + 3(161.5) + 4(283.0) + 3(364.5)} = 1.36$$

Enter the charts with an unbraced length $L_b = 12$ ft and a bending moment of

$$\frac{M_u}{C_b} = \frac{406.1}{1.36} = 299 \text{ ft-kips}$$

Examples

Try W21 × 48:

$$\phi_b M_n = 311 \text{ ft-kips} \qquad (\text{for } C_b = 1)$$

Since $C_b = 1.36$, the actual design strength is 1.36(311) = 423 ft-kips. But the design strength cannot exceed $\phi_b M_p$, which is only 398 ft-kips (obtained from the chart), so the actual design strength must be taken as

$$\phi_b M_n = \phi_b M_p = 398 \text{ ft-kips} < M_u = 406.1 \text{ ft-kips} \qquad \text{(N.G.)}$$

For the next trial shape, move up in the charts to the next solid curve and **try W18 × 55**. For L_b = 12 ft, the design strength from the chart is 335 ft-kips for C_b = 1. The strength for C_b = 1.36 is

$$\phi_b M_n = 1.36(335) = 456 \text{ ft-kips} > \phi_b M_p = 420 \text{ ft-kips}$$

$$\therefore \quad \phi_b M_n = \phi_b M_p = 420 \text{ ft-kips} > M_u = 406.1 \text{ ft-kips} \qquad \text{(OK)}$$

Check the beam weight.

$$M_u = 406.1 + \frac{1}{8}(1.2 \times 0.055)(24)^2 = 411 \text{ ft-kips} < 420 \text{ ft-kips} \qquad \text{(OK)}$$

Bending Strength of Non-Compact Shapes

- Flange Local Buckling (FLB):
- $\lambda_p < \lambda \le \lambda_r,$ (NON –COMPACT)

$$M_n = M_p - (M_p - 0.7F_y S_x)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)$$

$$\lambda = \frac{b_f}{2t_f}$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}}$$

$$\lambda_r = 1.0\sqrt{\frac{E}{F_y}}$$

Bending Strength of Non-Compact Shapes

- Flange Local Buckling (FLB):
- λ > λr (SLENDER)

$$M_n = \frac{0.9 E k_c S_x}{\lambda^2}$$

$k_c = \dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

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Example (Bending Strength of Non-Compact Shapes)

- Example:

A simply supported beam with a span length of 45 feet is laterally supported at its ends and is subjected to the following service loads:

Dead load = 400 lb/ft (including the weight of the beam)

Live load = 1000 lb/ft

If F_y = 50 ksi, is a W14 × 90 adequate?

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Example (Bending Strength of Non-Compact Shapes)

$$\lambda = \frac{b_f}{2t_f} = 10.2$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29,000}{50}} = 9.15$$

$$\lambda_r = 1.0\sqrt{\frac{E}{F_y}} = 1.0\sqrt{\frac{29,000}{50}} = 24.1$$

Since $\lambda_p < \lambda < \lambda_r$, this shape is noncompact. Check the capacity based on the limit state of flange local buckling:

$$M_p = F_y Z_x = 50(157) = 7850 \text{ in.-kips}$$

$$M_n = M_p - (M_p - 0.7F_y S_x)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)$$

$$= 7850 - (7850 - 0.7 \times 50 \times 143)\left(\frac{10.2 - 9.15}{24.1 - 9.15}\right) = 7650 \text{ in.-kips} = 637.5 \text{ ft-kips}$$

Example (Bending Strength of Non-Compact Shapes)

Check the capacity based on the limit state of lateral-torsional buckling. From the Z_x table,

$L_p = 15.2$ ft and $L_r = 42.6$ ft

$L_b = 45$ ft $> L_r$ $\therefore$ failure is by *elastic* LTB

From Part 1 of the *Manual,*

$I_y = 362$ in.^{4}

$r_{ts} = 4.11$ in.

$h_o = 13.3$ in.

$J = 4.06$ in.^{4}

$C_w = 16,000$ in.^{6}

For a uniformly loaded, simply supported beam with lateral support at the ends,

$C_b = 1.14$ (Fig. 5.15a)

Example (Bending Strength of Non-Compact Shapes)

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$

$$= \frac{1.14\pi^2(29,000)}{\left(\frac{45\times 12}{4.11}\right)^2}\sqrt{1 + 0.078\frac{4.06(1.0)}{143(13.3)}\left(\frac{45\times 12}{4.11}\right)^2} = 37.20 \text{ ksi}$$

From AISC Equation F2-3,

$$M_n = F_{cr}S_x = 37.20(143) = 5320 \text{ in.-kips} < M_p = 7850 \text{ in.-kips}$$

$$\phi_b M_n = 0.90(5320) = 4788 \text{ in.-kips} = 399 \text{ ft-kips}$$

The factored load and moment are

$$w_u = 1.2w_D + 1.6w_L = 1.2(0.400) + 1.6(1.000) = 2.080 \text{ kips/ft}$$

$$M_u = \frac{1}{8}w_u L^2 = \frac{1}{8}(2.080)(45)^2 = 527 \text{ ft-kips} > 399 \text{ ft-kips} \quad \text{(N.G.)}$$

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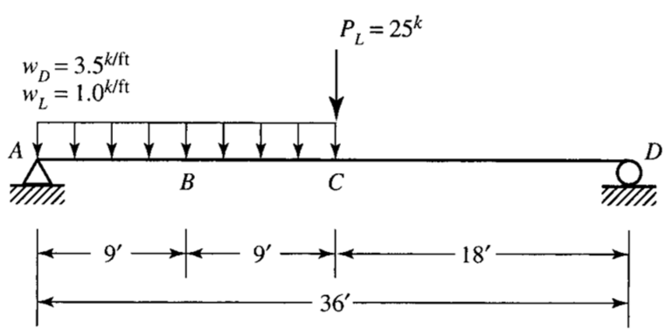
Example

- Bracing at A, B, C, and D.
- Fy = 50 Ksi.
- Is W 14 x 132 adequate in flexure?



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Example

- *Segment ABC:*
- Wu = 1.2 (3.5+0.132) + 1.6 (1.0) = 5.958 kip/ft
- *Segment CD:*
- Wu = 1.2 (0.132) = 0.1584 kip/ft
- Pu = 1.6 (25) = 40 kips.
- Mmax = 858 kip-ft at 16.97 ft
- Check Compactness (section is compact)

Example

- *W14 x 132 :*
- Lp = 13.3 ft, Lr = 56.0 ft
- *For segment BC* :
- Lb = 9 ft < Lp = 13.3 ft (Zone 1)
- Φb Mn = φb Mp = 878 kip-ft < Mu = 858 kip-ft (O.K)
- *For segment CD:*
- Lp= 13.3 ft <Lb = 18 ft < Lr = 56 ft (Zone 2)
- Cb = 12.5 (855.4) / (2.5)(855.4)+(3)(646.4)+
  (4) (434.1)+ (3)(218.7) = 1.653

Example

- Φb Mn = 1.653 [878 –(7.7)(18-13.3)]
  = 1391.512 kip-ft > φb Mp = 878 kip-ft
- Use φb Mp = 878 kip-ft > Mu = 855 kip-ft (O.K)
- *For segment AB* :
- Lb = 9 ft (Zone 1)
- φb Mp > Mu (O.K)
- Section is adequate in flexure

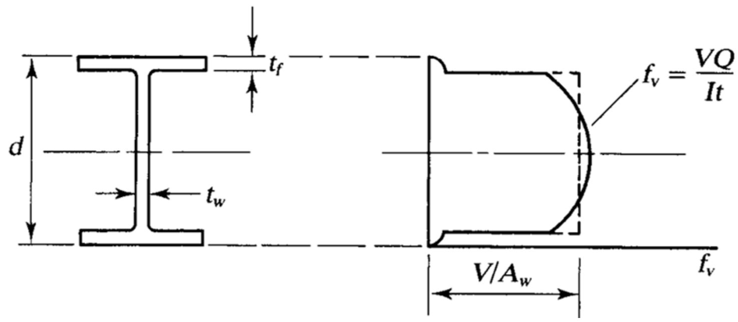
Shear

- Shear stress :

$$f_v = \frac{VQ}{Ib}$$

f_v = vertical and horizontal shearing stress at the point of interest
V = vertical shear force at the section under consideration
Q = first moment, about the neutral axis, of the area of the cross section between the point of interest and the top or bottom of the cross section
I = moment of inertia about the neutral axis
b = width of the cross section at the point of interest

Shear



$$f_v = \frac{V_n}{A_w} = 0.6F_y$$

A_w = area of the web.

$$V_n = 0.6F_yA_w$$

Shear

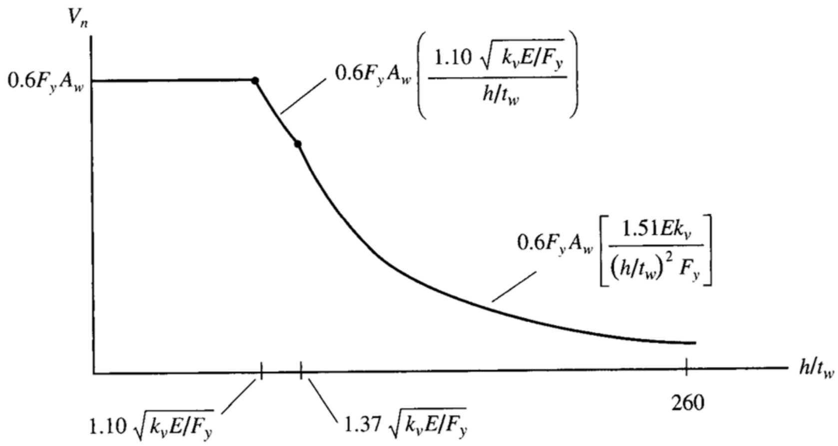
$$V_u \leq \phi_v V_n$$

V_u = maximum shear based on the controlling combination of factored loads
ϕ_v = resistance factor for shear

$$V_n = 0.6F_yA_wC_v$$

A_w = area of the web ≈ dt_w
d = overall depth of the beam
C_v = ratio of critical web stress to shear yield stress

Shear



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Shear

For the special case of hot-rolled I shapes with

$$\frac{h}{t_w} \le 2.24\sqrt{\frac{E}{F_y}}$$

The limit state is shear yielding, and

$$C_v = 1.0$$
$$\phi_v = 1.00$$
$$\Omega_v = 1.50$$

For all other doubly and singly symmetric shapes, except for round HSS

$$\phi_v = 0.90$$
$$\Omega_v = 1.67$$

and C_v is determined as follows:

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Shear

For $\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$, there is no web instability, and

$C_v = 1.0$

For $1.10\sqrt{\frac{K_v E}{F_y}} < \frac{h}{t_w} \leq 1.37\sqrt{\frac{K_v E}{F_y}}$, inelastic web buckling

$$C_v = \frac{1.10\sqrt{\frac{k_v E}{F_y}}}{h/t_w}$$

Shear

For $\frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_y}}$, the limit state is elastic web buckling:

$$C_v = \frac{1.51 E k_v}{(h/t_w)^2 F_y}$$

where

$k_v = 5$ with $h/t_w < 260$.

Example (Shear)

A simply supported beam with a span length of 45 feet is laterally supported at its ends and is subjected to the following service loads:

Dead load = 400 lb/ft (including the weight of the beam)

Live load = 1000 lb/ft

If $F_y = 50$ ksi, is a W14 × 90 adequate?

and the web area is $A_w = dt_w = 14.0(0.440) = 6.160 \text{ in.}^2$

$$\frac{h}{t_w} = 25.9$$

$$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29,000}{50}} = 54.0$$

Since

$$\frac{h}{t_w} < 2.24\sqrt{\frac{E}{F_y}}$$

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Example (Shear)

$$V_n = 0.6F_yA_wC_v = 0.6(50)(6.160)(1.0) = 184.8 \text{ kips}$$

Since $\frac{h}{t_w} < 2.24\sqrt{\frac{E}{F_y}}$,

$$\phi_v = 1.00$$

and the design shear strength is

$$\phi_v V_n = 1.00(184.8) = 185 \text{ kips}$$

$$V_u = \frac{w_u L}{2} = \frac{2.080(45)}{2} = 46.8 \text{ kips} < 185 \text{ kips} \quad \text{(OK)}$$

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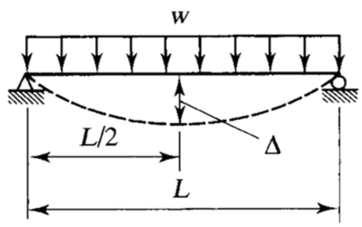
Deflection

- Example:
- W: Service live load.

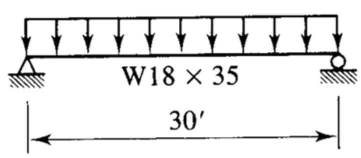


$$\Delta = \frac{5}{384}\frac{wL^4}{EI}$$

Type of member	Max. live load defl.	Max. dead + live load defl.	Max. snow or wind load defl.
Roof beam:			
Supporting plaster ceiling	$L/360$	$L/240$	$L/360$
Supporting nonplaster ceiling	$L/240$	$L/180$	$L/240$
Not supporting a ceiling	$L/180$	$L/120$	$L/180$
Floor beam	$L/360$	$L/240$	—

Example (Deflection)

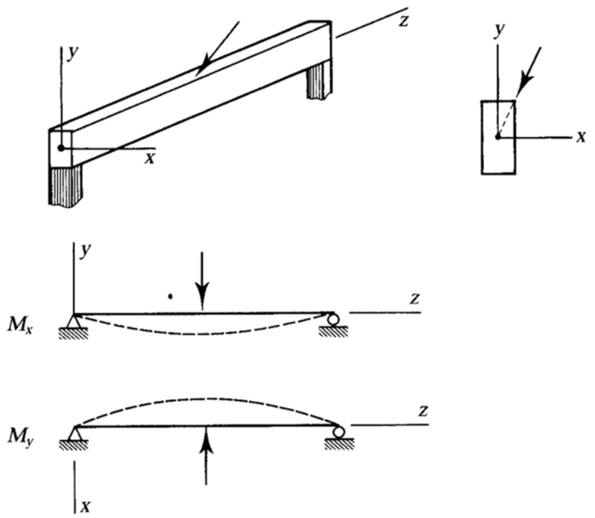
$W_D = 500$ lb/ft
$W_L = 550$ lb/ft



$$\Delta_L = \frac{5}{384}\frac{w_L L^4}{EI} = \frac{5}{384}\frac{(0.550/12)(30\times 12)^4}{29{,}000(510)} = 0.678 \text{ in.}$$

$$\frac{L}{360} = \frac{30(12)}{360} = 1.0 \text{ in.} > 0.678 \text{ in.} \quad \text{(OK)}$$

Biaxial Bending



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Biaxial Bending

$$\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0$$

where

M_{ux} = factored-load moment about the x axis
M_{nx} = nominal moment strength for x-axis bending
M_{uy} = factored-load moment about the y axis
M_{ny} = nominal moment strength for the y axis

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Example (Biaxial Bending)

A W21 × 68 is used as a simply supported beam with a span length of 12 feet. Lateral support of the compression flange is provided only at the ends. Loads act through the shear center, producing moments about the x and y axes. The service load moments about the x axis are $M_{Dx} = 48$ ft-kips and $M_{Lx} = 144$ ft-kips. Service load moments about the y axis are $M_{Dy} = 6$ ft-kips and $M_{Ly} = 18$ ft-kips. If A992 steel is used, does this beam satisfy the provisions of the AISC Specification? Assume that all moments are uniform over the length of the beam.

$L_p = 6.36$ ft, $L_r = 18.7$ ft

The unbraced length $L_b = 12$ ft, so $L_p < L_b < L_r$, and the controlling limit state is inelastic lateral-torsional buckling. Then

$$M_{nx} = C_b \left[M_{px} - (M_{px} - 0.7F_y S_x \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_{px}$$

$$M_{px} = F_y Z_x = 50(160) = 8000 \text{ in.-kips}$$

Example (Biaxial Bending)

Because the bending moment is uniform, $C_b = 1.0$.

$$M_{nx} = 1.0\left[8000 - (8000 - 0.7 \times 50 \times 140)\left(\frac{12 - 6.36}{18.7 - 6.36}\right)\right]$$

$$= 6583 \text{ in.-kips} = 548.6 \text{ ft-kips}$$

For the y axis, since the shape is compact, there is no flange local buckling and

$$M_{ny} = M_{py} = F_y Z_y = 50(24.4) = 1220 \text{ in.-kips} = 101.7 \text{ ft-kips}$$

Check the upper limit:

$$\frac{Z_y}{S_y} = \frac{24.4}{15.7} = 1.55 < 1.6 \qquad \therefore M_{ny} = M_{py} = 101.7 \text{ in.-kips}$$

Example (Biaxial Bending)

For x-axis bending,

$$M_{ux} = 1.2M_{Dx} + 1.6M_{Lx} = 1.2(48) + 1.6(144) = 288.0 \text{ ft-kips}$$

For y-axis bending,

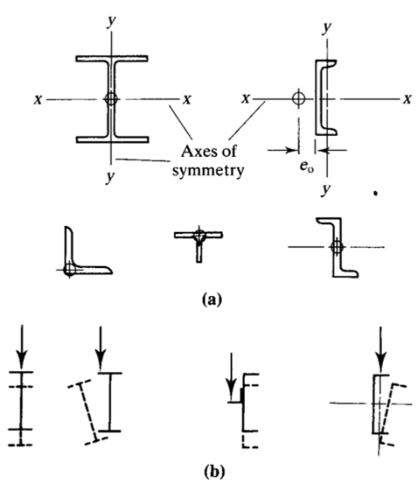
$$M_{uy} = 1.2M_{Dy} + 1.6M_{Ly} = 1.2(6) + 1.6(18) = 36.0 \text{ ft-kips}$$

$$\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} = \frac{288.0}{0.90(548.6)} + \frac{36.0}{0.90(101.7)} = 0.977 < 1.0$$

The W21 × 68 is satisfactory.

Shear Center

- Is that point through which the loads must act if there is to be no twisting or torsion of the beam.



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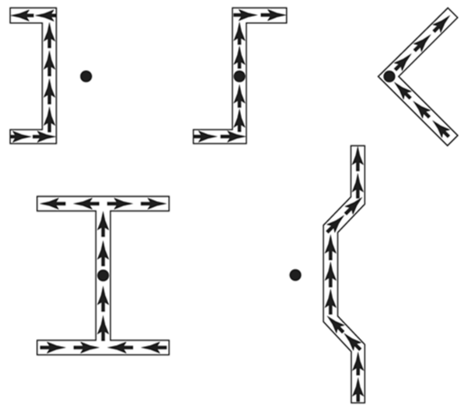
Shear Center

- Location of shear center:



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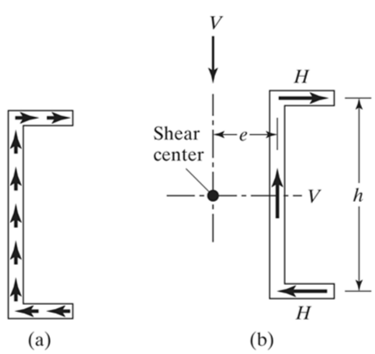
Shear Center

- Location of shear center:



qv = VQ / I (Shear flow)

- H h = V e

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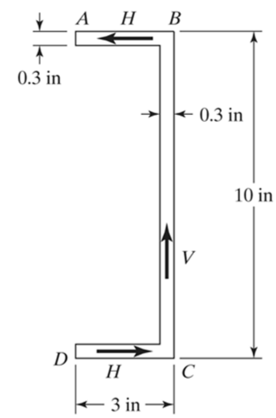
Example (Shear Center)



- qv at B = VQ / I
- $I = (0.3)(9.4)^3 / 12 +$
- $[(3)(0.3)^3 / 12 +$
- $(3)(0.3)(4.85)^2\}] (2)$
- $= 63.12 \text{ in}^4$
- Q = (3-0.15)(0.3)(5-0.15)
- = 4.146
- qv at B = V (4.146) / 63.12
- = 0.0657 V

Example (Shear Center)

- Total area H = (0.5)(0.0657 V)(2.85) = 0.0936 V

Variation of shearing stresses

V e = H h

V e = (0.0936 V) (9.7)

e = 0.91 in (from center line of web)

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STEEL DESIGN

Chapter 6:

Analysis and Design of Beam-Columns

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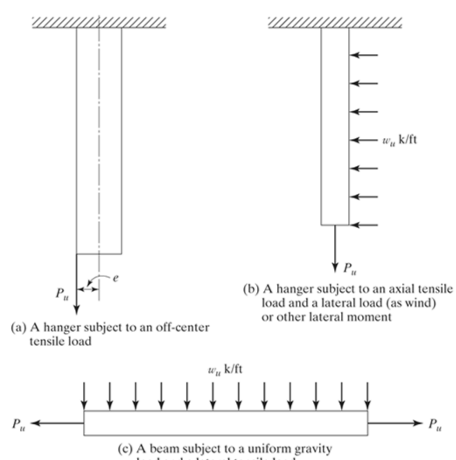
Introduction

- Beam-columns are members that are subjected to bending and axial forces. (Tension or Compression)



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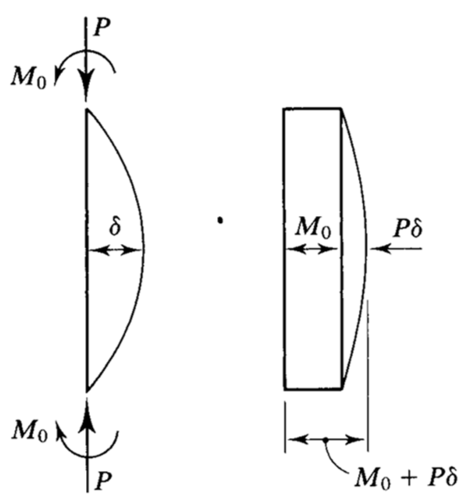
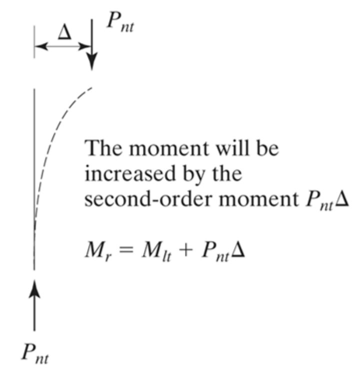
Introduction

- *First Order (F.O) Analysis:* Structural analysis using undeformed shape of the structure.
- *Second Order (S.O) Analysis:* Structural analysis considering geometric changes due to deformations in structural formulations.

Introduction



Introduction



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Introduction

- ***Second Order:***

1. Use computers and make a second order analysis to determine the maximum factored load strength.
2. Use First Order analysis and amplify the moments obtained with some amplification factors.

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Interaction Formula

- Chapter H in the specifications in the manual.

1. For $\frac{P_u}{\phi_c P_n} \geq 0.2,$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

2. For $\frac{P_u}{\phi_c P_n} < 0.2,$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

Interaction Formula

Mu = B1 Mnt + B2 Mlt

- Mnt: Maximum moment assuming that no sidesway occurs (no translation) (First order).
- Mlt :Maximum moment caused by sidesway (lateral translation) (First order)
- B1 : Amplification factor for the moments occurring in member when it is braced against sidesway.
- B2 : Amplification factor for moments resulting from sidesway.

Interaction Formula

Pu = Pnt + B2 Plt

- Pnt: Axial load assuming that no sidesway occurs (no translation).
- Plt :Axial load caused by sidesway (lateral translation.
- B2 : Amplification factor for moments resulting from sidesway.

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Interaction Formula

$$B_1 = \frac{C_m}{1-(\alpha P_r/P_{e1})} \geq 1$$

$\alpha = 1.00$ for LRFD

P_r = required axial compressive strength
$= P_u$ for LRFD

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2}$$

- K < 1.0 (BRACED FRAME)
- Cm : Modification factor due to maximum moment depends on the distribution of bending moment within the member.

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Interaction Formula

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1$$

$\alpha = 1.00$ for LRFD

ΣP_{nt} = sum of required load capacities for all columns in the story under consideration (factored for LRFD, unfactored for ASD)

ΣP_{e2} = sum of the Euler loads for all columns in the story under consideration

$$\Sigma P_{e2} = \Sigma \frac{\pi^2 EI}{(K_2 L)^2}$$

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H}$$

Interaction Formula

where

I = moment of inertia about the axis of bending

K_2 = effective length factor corresponding to the unbraced condition

L = story height

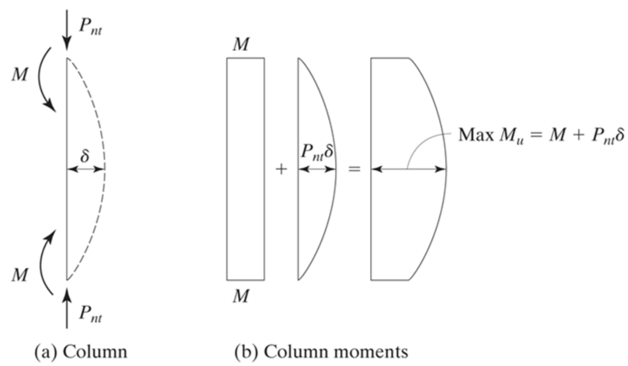
R_M = 1.0 for braced frames (although B_2 is not used for braced frames)
= 0.85 for unbraced frames and mixed systems

Δ_H = drift (sidesway displacement) of the story under consideration

ΣH = sum of all horizontal forces causing Δ_H

Interaction Formula

- *Cm:*



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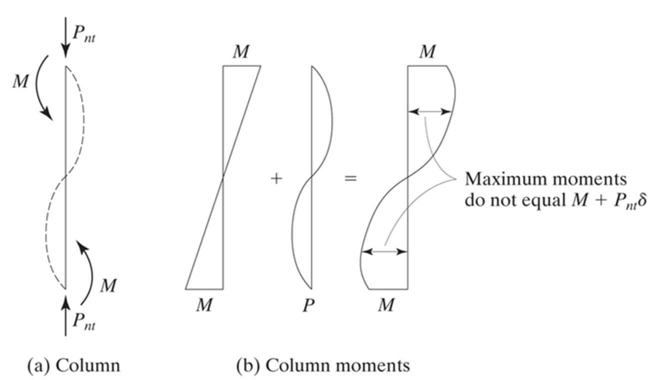
Interaction Formula

- *Cm:*



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Interaction Formula

- *Cm:*

1. If there is no transverse loads acting on the member:

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right)$$

- M1 / M2: Ratio of bending moments at the ends of the member.
- M1: Absolute smaller value.
- M2: Absolute larger value.
- Ratio (+ve) when reverse or double curvature.
  (-ve) when single curvature.

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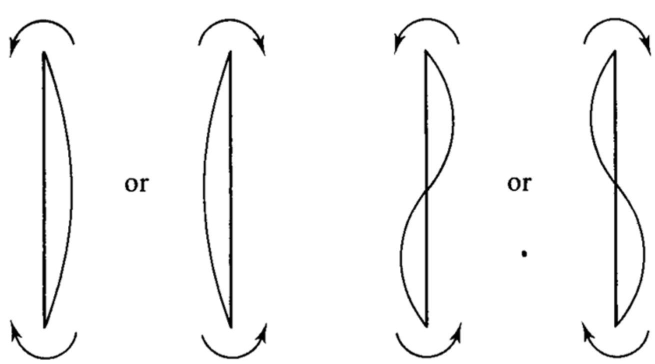
Interaction Formula

- *Cm:*



Negative M_1/M_2 Positive M_1/M_2

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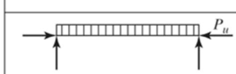
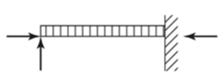
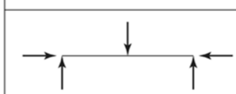
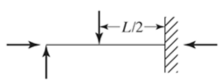
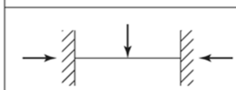
Interaction Formula

- *Cm:*

2. For transversely loaded members:

$$C_m = 1 + \Psi\left(\frac{\alpha P_r}{P_{e1}}\right)$$

Case	ψ	C_m
	0	1.0
	−0.4	$1 - 0.4\frac{P_u}{P_{e1}}$
	−0.4	$1 - 0.4\frac{P_u}{P_{e1}}$
	−0.2	$1 - 0.2\frac{P_u}{P_{e1}}$
	−0.3	$1 - 0.3\frac{P_u}{P_{e1}}$
	−0.2	$1 - 0.2\frac{P_u}{P_{e1}}$

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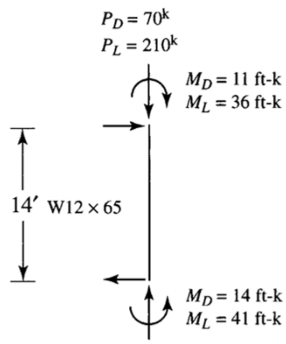
Examples (Analysis for Braced Frame)

- *Example 1:*
- Braced frame.
- Service loads and moments about strong axis are shown.
- A5732, Grade 50
- KxLx = KyLy = 14 ft

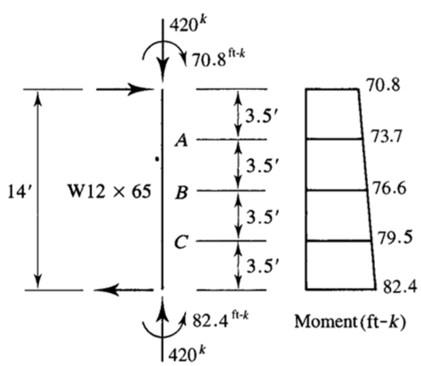


Examples (Analysis for Braced Frame)

$\phi_c P_n = 685$ kips

$$\frac{P_u}{\phi_c P_n} = \frac{420}{685} = 0.6131 > 0.2$$

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{\pi^2 EI_x}{(K_x L)^2} = \frac{\pi^2 (29{,}000)(533)}{(1.0 \times 14 \times 12)^2} = 5405 \text{ kips}$$



Examples (Analysis for Braced Frame)

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4\left(-\frac{70.8}{82.4}\right) = 0.9437$$

$$B_1 = \frac{C_m}{1-(\alpha P_r/P_{e1})} = \frac{C_m}{1-(1.00P_u/P_{e1})} = \frac{0.9437}{1-(420/5405)} = 1.023$$

From the Beam Design Charts with $C_b = 1.0$ and $L_b = 14$ feet, the moment strength is

$\phi_b M_n = 345$ ft-kips

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

$$= \frac{12.5(82.4)}{2.5(82.4) + 3(73.7) + 4(76.6) + 3(79.5)} = 1.060$$

. $\phi_b M_n = C_b(345) = 1.060(345) = 366$ ft-kips

Examples (Analysis for Braced Frame)

But $\phi_b M_p = 356$ ft-kips (from the charts) < 366 ft-kips $\quad \therefore$ use $\phi_b M_n = 356$ ft-kips (Since a W12 × 65 is noncompact for $F_y = 50$ ksi, 356 ft-kips is the design strength based on FLB rather than full yielding of the cross section.) The factored load moments are

$M_{nt} = 82.4$ ft-kips $\quad M_{\ell t} = 0$

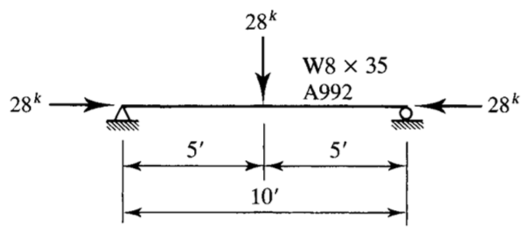
$M_r = M_u = B_1 M_{nt} + B_2 M_{\ell t} = 1.023(82.4) + 0 = 84.30$ ft-kips $= M_{ux}$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = 0.6131 + \frac{8}{9}\left(\frac{84.30}{356} + 0\right) = 0.824 < 1.0 \qquad \text{(OK)}$$

The member is satisfactory.

Examples (Analysis for Braced Frame)

- <u>Example 2:</u>
- Service live loads are shown.
- Laterally braced at ends.
- Bending about x axis.
- Fy = 50 ksi



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Examples (Analysis for Braced Frame)

The factored axial load is

$$P_u = 1.6(28) = 44.8 \text{ kips}$$

The factored transverse loads and bending moment are

$$Q_u = 1.6(28) = 44.8 \text{ kips}$$
$$w_u = 1.2(0.035) = 0.042 \text{ kips/ft}$$
$$M_u = \frac{44.8(10)}{4} + \frac{0.042(10)^2}{8} = 112.5 \text{ ft-kips}$$

This member is braced against sidesway, so $M_{\ell t} = 0$.

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Examples (Analysis for Braced Frame)

$$C_m = 1 + \Psi\left(\frac{\alpha P_r}{P_{e1}}\right)$$

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{\pi^2 EI_x}{(K_x L)^2} = \frac{\pi^2 (29,000)(127)}{(10\times 12)^2} = 2524 \text{ kips}$$

$$C_m = 1 + \Psi\left(\frac{\alpha P_r}{P_{e1}}\right) = 1 - 0.2\left(\frac{1.00 P_u}{P_{e1}}\right) = 1 - 0.2\left(\frac{44.8}{2524}\right) = 0.9965$$

$$B_1 = \frac{C_m}{1-(\alpha P_r/P_{e1})} = \frac{C_m}{1-(1.00 P_u/P_{e1})} = \frac{0.9965}{1-(44.8/2524)} = 1.015$$

$$M_u = B_1 M_{nt} + B_2 M_{\ell t} = 1.015(112.5) + 0 = 114.2 \text{ ft-kips}$$

Examples (Analysis for Braced Frame)

From the beam design charts, for $L_b = 10$ ft and $C_b = 1$,

- $\phi_b M_n = 123$ ft-kips

Cb = 1.32

- $\phi_b M_n = 1.32(123) = 162.4$ ft-kips $>$ $\phi_b M_p = 130$ ft-kips

$\phi_b M_n = 130$ ft-kips

$\phi_c P_n = 358$ kips

$$\frac{P_u}{\phi_c P_n} + \frac{44.8}{358} = 0.1251 < 0.2$$

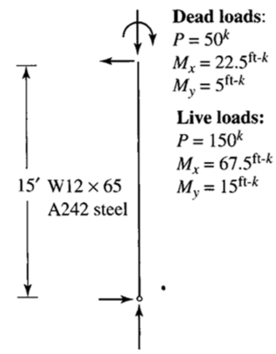
A W8 × 35 is adequate.

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = \frac{0.1251}{2} + \left(\frac{114.2}{130} + 0\right)$$

$$= 0.941 < 1.0 \text{ (OK)}$$

Examples (Analysis for Braced Frame)

- <u>Example 3:</u>
- Service Loads and Moments are shown.
- Use Kx = Ky = 1.0
- Fy = 50 ksi



Examples (Analysis for Braced Frame)

$$P_u = 1.2(50) + 1.6(150) = 300 \text{ kips}$$

$$M_{ntx} = 1.2(22.5) + 1.6(67.5) = 135.0 \text{ ft-kips}$$

$$M_{nty} = 1.2(5) + 1.6(15) = 30.0 \text{ ft-kips}$$

$$C_{mx} = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4(0) = 0.6$$

$$P_{e1x} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{\pi^2 EI_x}{(K_x L)^2} = \frac{\pi^2 (29,000)(533)}{(1.0 \times 15 \times 12)^2} = 4708 \text{ kips}$$

$$B_{1x} = \frac{C_{mx}}{1-(\alpha P_r/P_{e1x})} = \frac{C_{mx}}{1-(1.00 P_u/P_{e1x})} = \frac{0.6}{1-(300/4708)}$$

$$= 0.641 < 1.0 \quad \therefore \text{ use } B_{1x} = 1.0$$

$$M_r = M_{ux} = B_{1x} M_{ntx} + B_{2x} M_{\ell tx} = 1.0(135) + 0 = 135.0 \text{ ft-kips}$$

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Examples (Analysis for Braced Frame)

From the Beam Design Charts with $C_b = 1.0$ and $L_b = 15$ feet, the moment strength is

$\phi_b M_{nx} = 340$ ft-kips and $\phi_b M_{px} = 356$ ft-kips

$C_b \times (\phi_b M_{nx} \text{ for } C_b = 1.0) = 1.67(340) = 567.8$ ft-kips

This result is larger than $\phi_b M_{px}$; therefore use $\phi_b M_{nx} = \phi_b M_{px} = 356$ ft-kips.

$$C_{my} = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4(0) = 0.6$$

$$P_{e1y} = \frac{\pi^2 EI}{(K_1L)^2} = \frac{\pi^2 EI_y}{(K_yL)^2} = \frac{\pi^2(29{,}000)(174)}{(1.0 \times 15 \times 12)^2} = 1537 \text{ kips}$$

$$B_{1y} = \frac{C_{my}}{1-(\alpha P_r/P_{e1y})} = \frac{C_{my}}{1-(1.00P_u/P_{e1y})} = \frac{0.6}{1-(300/1537)}$$

$$= 0.746 < 1.0 \quad \therefore \text{ use } B_{1y} = 1.0$$

Examples (Analysis for Braced Frame)

$$M_r = M_{uy} = B_{1y}M_{nty} + B_{2y}M_{\ell ty} = 1.0(30) + 0 = 30 \text{ ft-kips}$$

$$\lambda = \frac{b_f}{2t_f} = 9.92$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000}{50}} = 9.152$$

$$\lambda_r = 1.0\sqrt{\frac{E}{F_y}} = 1.0\sqrt{\frac{29{,}000}{50}} = 24.08$$

$$M_n = M_p - (M_p - 0.7F_yS_y)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)$$

$$M_p = M_{py} = F_yZ_y = \frac{50(44.1)}{12} = 183.8 \text{ ft-kips}$$

Examples (Analysis for Braced Frame)

$$M_n = M_{ny} = 183.8 - (183.8 - 0.7 \times 50 \times 29.1/12)\left(\frac{9.92 - 9.152}{24.08 - 9.152}\right) = 178.7 \text{ ft-kips}$$

$$\phi_b M_{ny} = 0.90(178.7) = 160.8 \text{ ft-kips}$$

$$\phi_c P_n = 662 \text{ kips}$$

Determine which interaction formula to use:

$$\frac{P_u}{\phi_c P_n} = \frac{300}{662} = 0.4532 > 0.2 \quad \therefore \text{ use Equation 6.3 (AISC Equation H1-1a)}$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = 0.4532 + \frac{8}{9}\left(\frac{135}{356} + \frac{30}{160.8}\right)$$

$$= 0.956 < 1.0 \qquad \text{(OK)}$$

The W12 × 65 is satisfactory.

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Design of Beam- Columns

- Because of many variables in the interaction formulas, the design is essentially a trial and error process.
- Amin Mansour Method:

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0$$

$$\left(\frac{1}{P_c}\right)P_r + \left(\frac{8}{9M_{cx}}\right)M_{rx} + \left(\frac{8}{9M_{cy}}\right)M_{ry} \le 1.0$$

$$pP_r + b_x M_{rx} + b_y M_{ry} \le 1.0$$

$$P = \frac{1}{P_c} = \frac{1}{\phi_c P_n}$$

$$b_x = \frac{8}{9M_{cx}} = \frac{8}{9(\phi_b M_{nx})}$$

$$b_y = \frac{8}{9M_{cy}} = \frac{8}{9(\phi_b M_{ny})}$$

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Design of Beam- Columns

$$\frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)\leq 1.0$$

$$0.5pP_r+\frac{9}{8}(b_xM_{rx}+b_yM_{ry})\leq 1.0$$

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Design of Beam- Columns

- Procedure:

1. Select a trial shape from Table 6-1.
2. Use the effective length KL to select p, and use the unbraced length L_b to select b_x (the constant b_y determines the weak axis bending strength, so it is independent of the unbraced length). The values of the constants are based on the assumption that weak axis buckling controls the axial compressive strength and that $C_b = 1.0$.
3. Compute pP_r. If this is less than or equal to 0.2, use interaction Equation 6.8. If pP_r is greater than 0.2, use Equation 6.9.
4. Evaluate the selected interaction equation with the values of p, b_x, and b_y for the trial shape.
5. If the result is not very close to 1.0, try another shape. By examining the value of each term in Equation 6.8 or 6.9, you can gain insight into which constants need to be larger or smaller.
6. Continue the process until a shape is found that gives an interaction equation result less than 1.0 and close to 1.0 (greater than 0.9).

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Design of Beam- Columns

- If strong axis buckling controls the compressive strength, use an effective length of

$$KL = \frac{K_x}{r_x/r_y}$$

to obtain p from Table 6-1.

- If C_b is not equal to 1.0, the value of b_x must be adjusted.

Design of Beam- Columns

F_y = 50 ksi

Table 6–1
Combined Axial and Bending
W Shapes

W44

Shape		W44×											
		335[c]				290[c]				262[c]			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
st radius of gyration r_y : axis bending	0	0.345	0.230	0.220	0.146	0.417	0.278	0.253	0.168	0.476	0.317	0.281	0.187
	11	0.378	0.251	0.220	0.146	0.454	0.302	0.253	0.168	0.518	0.344	0.281	0.187
	12	0.384	0.256	0.220	0.146	0.462	0.307	0.253	0.168	0.526	0.350	0.281	0.187
	13	0.393	0.261	0.222	0.148	0.470	0.313	0.255	0.170	0.536	0.356	0.284	0.189
	14	0.402	0.267	0.225	0.150	0.480	0.319	0.259	0.173	0.546	0.363	0.289	0.192
	15	0.412	0.274	0.229	0.152	0.490	0.326	0.264	0.175	0.557	0.371	0.294	0.196
	16	0.423	0.282	0.233	0.155	0.501	0.333	0.268	0.178	0.570	0.379	0.299	0.199
	17	0.435	0.290	0.236	0.157	0.514	0.342	0.273	0.181	0.584	0.389	0.304	0.203
	18	0.449	0.299	0.240	0.160	0.527	0.351	0.277	0.184	0.599	0.399	0.310	0.206
	19	0.463	0.308	0.244	0.162	0.542	0.361	0.282	0.188	0.616	0.410	0.316	0.210

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Design of Beam- Columns

Effective												
40	1.28	0.850	0.377	0.251	1.47	0.981	0.463	0.308	1.66	1.10	0.550	0.366
42	1.41	0.937	0.405	0.269	1.62	1.08	0.498	0.331	1.83	1.21	0.593	0.394
44	1.55	1.03	0.432	0.287	1.78	1.19	0.533	0.355	2.00	1.33	0.636	0.423
46	1.69	1.12	0.459	0.306	1.95	1.30	0.569	0.378	2.19	1.46	0.679	0.452
48	1.84	1.22	0.487	0.324	2.12	1.41	0.604	0.402	2.38	1.59	0.723	0.481
50	2.00	1.33	0.514	0.342	2.30	1.53	0.640	0.426	2.59	1.72	0.767	0.510

Other Constants and Properties						
$b_y \times 10^3$ (kip-ft)$^{-1}$	1.51	1.00	1.74	1.16	1.96	1.30
$t_y \times 10^3$ (kips)$^{-1}$	0.339	0.226	0.390	0.260	0.434	0.289
$t_r \times 10^3$ (kips)$^{-1}$	0.417	0.278	0.480	0.320	0.534	0.356
r_x/r_y	5.10		5.10		5.10	

[c] Shape is slender for compression with $F_y = 50$ ksi.

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Example (Design of Beam- Columns)

- Example 1:

A structural member in a braced frame must support the following service loads and moments: an axial compressive dead load of 25 kips and a live load of 75 kips; a dead load moment of 12.5 ft-kips about the strong axis and a live load moment of 37.5 ft-kips about the strong axis; a dead load moment of 5 ft-kips about the weak axis and a live load moment of 15 ft-kips about the weak axis. The moments occur at one end; the other end is pinned. The effective length with respect to each axis is 15 feet. There are no transverse loads on the member. Use A992 steel and select a W10 shape.

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Example (Design of Beam- Columns)

The factored axial load is

$$P_u = 1.2(25) + 1.6(75) = 150 \text{ kips}$$

The factored moments are

$$M_{ntx} = 1.2(12.5) + 1.6(37.5) = 75.0 \text{ ft-kips}$$
$$M_{nty} = 1.2(5) + 1.6(15) = 30.0 \text{ ft-kips}$$

The amplification factor B_1 can be estimated as 1.0 for purposes of making a trial selection. For the two axes,

$$M_{ux} = B_{1x}M_{ntx} = 1.0(75) = 75 \text{ ft-kips}$$
$$M_{uy} = B_{1y}M_{nty} = 1.0(30) = 30 \text{ ft-kips}$$

Try a W10 × 49. From Table 6-1, $p = 2.22 \times 10^{-3}$, $b_x = 4.35 \times 10^{-3}$, $b_y = 8.38 \times 10^{-3}$

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Example (Design of Beam- Columns)

$$\frac{P_u}{\phi_c P_n} = pP_u = (2.22 \times 10^{-3})(150) = 0.333 > 0.2$$

$$pP_u + b_xM_{ux} + b_yM_{uy} = (2.22 \times 10^{-3})(150) + (4.35 \times 10^{-3})(75) + (8.38 \times 10^{-3})(30)$$
$$= 0.911 < 1.0 \qquad \text{(OK)}$$

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4\left(\frac{0}{M_2}\right) = 0.6 \quad \text{(for both axes)}$$

$$P_{e1x} = \frac{\pi^2 EI_x}{(K_xL)^2} = \frac{\pi^2(29,000)(272)}{(15 \times 12)^2} = 2403 \text{ kips}$$

$$B_{1x} = \frac{C_{mx}}{1 - \frac{P_u}{P_{e1x}}} = \frac{0.6}{1 - \frac{150}{2403}} = 0.640 < 1.0 \quad \therefore B_{1x} = 1.0 \text{ as assumed}$$

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Example (Design of Beam- Columns)

$$P_{e1y} = \frac{\pi^2 EI_y}{(K_yL)^2} = \frac{\pi^2(29,000)(93.4)}{(15\times 12)^2} = 825.1 \text{ kips}$$

$$B_{1y} = \frac{C_{my}}{1-\dfrac{P_u}{P_{e1y}}} = \frac{0.6}{1-\dfrac{150}{825.1}} = 0.733 < 1.0 \quad \therefore B_{1y} = 1.0 \text{ as assumed}$$

$C_b = 1.67$. Modify b_x to account for C_b.

$$C_b \times \phi_b M_{nx} = C_b \times \frac{8}{9} \times \frac{1}{b_x} = 1.67 \times \frac{8}{9} \times \frac{1}{4.35\times 10^{-3}} = 341.3 \text{ ft-kips}$$

Example (Design of Beam- Columns)

From the Z_x table, $\phi_b M_{px} = 227$ ft-kips < 341.3 ft-kips $\therefore \phi_b M_{nx} = 227$ ft-kips

$$b_x = \frac{8}{9(\phi_b M_{nx})} = \frac{8}{9(227)} = 3.92\times 10^{-3}$$

$p = 2.22 \times 10^{-3}$, $b_x = 3.92 \times 10^{-3}$, $b_y = 8.38 \times 10^{-3}$

$$pP_u + b_x M_{ux} + b_y M_{uy} = (2.22\times 10^{-3})(150) + (3.92\times 10^{-3})(75) + (8.38\times 10^{-3})(30)$$
$$= 0.878 < 1.0 \quad \text{(OK)}$$

Example (Design of Beam- Columns)

Try the next smaller shape. **Try a W10 × 45**, with $p = 3.01 \times 10^{-3}$, $b_x = 5.07 \times 10^{-3}$, $b_y = 11.7 \times 10^{-3}$.

$$P_{e1x} = \frac{\pi^2 EI_x}{(K_x L)^2} = \frac{\pi^2 (29,000)(248)}{(15 \times 12)^2} = 2191 \text{ kips}$$

$$B_{1x} = \frac{C_{mx}}{1 - \frac{P_u}{P_{e1x}}} = \frac{0.6}{1 - \frac{150}{2191}} = 0.644 < 1.0 \quad \therefore B_{1x} = 1.0 \text{ as assumed}$$

$$P_{e1y} = \frac{\pi^2 EI_y}{(K_y L)^2} = \frac{\pi^2 (29,000)(53.4)}{(15 \times 12)^2} = 471.7 \text{ kips}$$

$$B_{1y} = \frac{C_{my}}{1 - \frac{P_u}{P_{e1y}}} = \frac{0.6}{1 - \frac{150}{471.7}} = 0.880 < 1.0 \quad \therefore B_{1y} = 1.0 \text{ as assumed}$$

Example (Design of Beam- Columns)

$$C_b \times \phi_b M_{nx} = C_b \times \frac{8}{9} \times \frac{1}{b_x} = 1.67 \times \frac{8}{9} \times \frac{1}{5.07 \times 10^{-3}} = 292.8 \text{ ft-kips}$$

$$\phi_b M_{px} = 206 \text{ ft-kips} < 292.8 \text{ ft-kips} \quad \therefore \phi_b M_{nx} = 206 \text{ ft-kips}$$

$$b_x = \frac{8}{9(\phi_b M_{nx})} = \frac{8}{9(206)} = 4.32 \times 10^{-3}$$

$p = 3.01 \times 10^{-3}$, $b_x = 4.32 \times 10^{-3}$, $b_y = 11.7 \times 10^{-3}$.

$$pP_u + b_x M_{ux} + b_y M_{uy} = (3.01 \times 10^{-3})(150) + (4.32 \times 10^{-3})(75) + (11.7 \times 10^{-3})(30)$$
$$= 1.13 > 1.0 \quad \text{(N.G.)}$$

Use a W10 × 49.

STEEL DESIGN

Chapter 7:
Bolted Connections

Introduction

- Types of Bolts:

1. Ordinary or Common bolts:

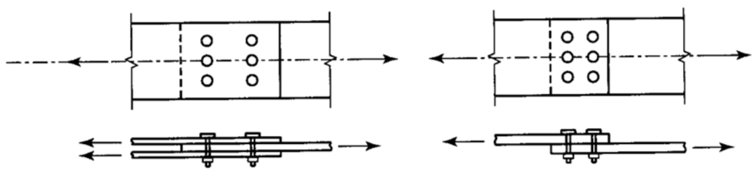
- Classified by ASTM as A307 bolts.
- Used in light structures subjected to static loading only.

2. High strength bolts:

- Classified by ASTM as A325, A490 bolts.
- Have tensile strengths two or more times those of ordinary bolts.
- Used in all types of structures (static + dynamic loads)

Introduction

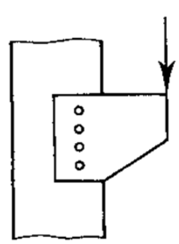
- <u>Simple Connections:</u>
- If the line of action of the resultant force to be resisted passes through the center of gravity of the connection, each part of the connection is assumed to resist an equal share of the load.



(a) (b)

Introduction

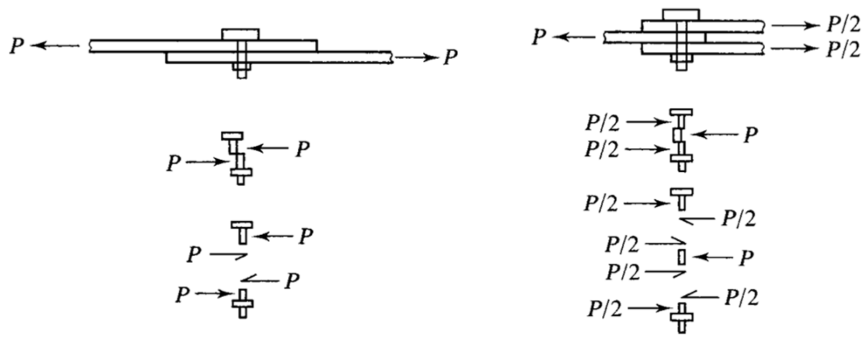
- <u>Eccentrically Loaded Connections:</u>
- If the line of action of the resultant force to be resisted does not act through the center of gravity of the connection.



Bolted Shear Connections

- <u>Failure modes:</u>

1. Shear failure of the bolts:



(a) Single Shear

(b) Double Shear

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Bolted Shear Connections

2. Tension failure in the member :

- Yielding
- Fracture
- Block shear

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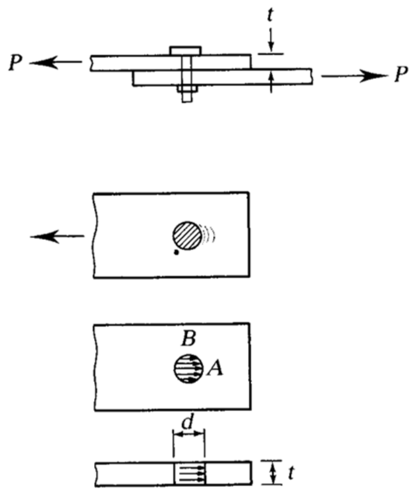
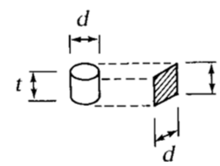
Bolted Shear Connections

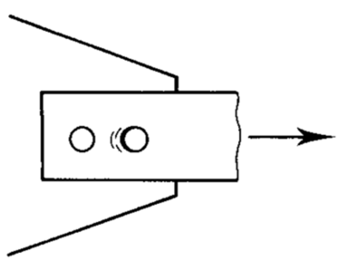
3. Bearing exerted by the bolts :

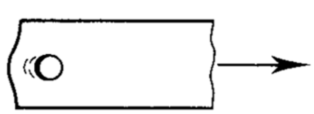




Bolted Shear Connections

3. Bearing exerted by the bolts :





Bolted Shear Connections

- Types of bolted shear connections:

1. *Bearing type connections*:

- Slip is acceptable (loose in connection)
- Load will be transferred through shear in bolts and bearing in the connected parts.

2. *Slip critical connections*:

- No slippage is permitted (shear force < friction force)
- No shear and bearing.
- Load will be transferred through friction.

Bearing Type Connections

1. Shear Strength:

$$P = f_v A_b$$

- f_v : Shearing stress on the cross-sectional area of the bolt.
- Ab : Cross-sectional area of the unthreaded part of bolt.

$$R_n = F_{nv} A_b$$

- Rn : Nominal strength.
- *Fnv* :Nominal shear stress.

Bearing Type Connections

TABLE J3.2
Nominal Stress of Fasteners and Threaded Parts, ksi (MPa)

Description of Fasteners	Nominal Tensile Stress, F_{nt}, ksi (MPa)	Nominal Shear Stress in Bearing-Type Connections, F_{nv}, ksi (MPa)
A307 bolts	45 (310) [a][b]	24 (165) [b][c][f]
A325 or A325M bolts, when threads are not excluded from shear planes	90 (620) [e]	48 (330) [f]
A325 or A325M bolts, when threads are excluded from shear planes	90 (620) [e]	60 (414) [f]
A490 or A490M bolts, when threads are not excluded from shear planes	113 (780) [e]	60 (414) [f]
A490 or A490M bolts, when threads are excluded from shear planes	113 (780) [e]	75 (520) [f]

Bearing Type Connections

Fastener	Nominal Shear Strength $R_n = F_{nv}A_b$
A307	$24A_b$
A325, threads in plane of shear	$48A_b$
A325, threads not in plane of shear	$60A_b$
A490, threads in plane of shear	$60A_b$
A490, threads not in plane of shear	$75A_b$

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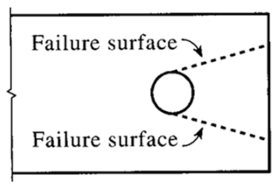
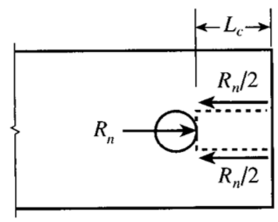
Bearing Type Connections

2. <u>Bearing Strength:</u>

- Bearing strength is independent of the type of fastener because the stress under consideration is on the part being connected rather than on the fastener.



(a)



(b)

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Bearing Type Connections

$$R_n = 1.2 L_c t F_u \le 2.4 d t F_u$$

$$\phi R_n = 0.75 R_n$$

where

L_c = clear distance, in the direction parallel to the applied load, from the edge of the bolt hole to the edge of the adjacent hole or to the edge of the material

t = thickness of the connected part

F_u = ultimate tensile stress of the connected part (*not* the bolt)

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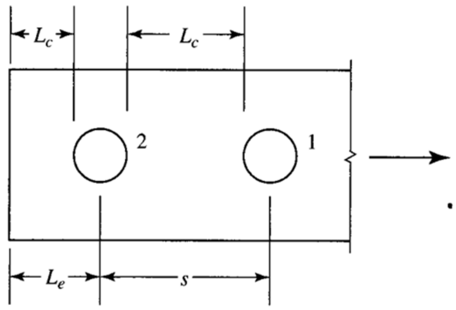
Bearing Type Connections



For the edge bolts, use $L_c = L_e - h/2$. For other bolts, use $L_c = s - h$,

where

L_e = edge-distance to center of the hole

s = center-to-center spacing of holes

h = hole diameter

$$h = d + \frac{1}{16} \text{ in.}$$

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Bearing Type Connections

- **Spacing and Edge distance requirements:**

Minimum spacing and edge distance: In any direction, both in the line of force and transverse to the line of force,

$s \geq 2\frac{2}{3}d$ (preferably $3d$)

$L_e \geq$ value from AISC Table J3.4

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Bearing Type Connections

TABLE J3.4
Minimum Edge Distance,[a] in., from Center of Standard Hole[b] to Edge of Connected Part

Bolt Diameter (in.)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]
1/2	7/8	3/4
5/8	1 1/8	7/8
3/4	1 1/4	1
7/8	1 1/2 [d]	1 1/8
1	1 3/4 [d]	1 1/4
1 1/8	2	1 1/2
1 1/4	2 1/4	1 5/8
Over 1 1/4	1 3/4 × d	1 1/4 × d

[a] Lesser edge distances are permitted to be used provided provisions of Section J3.10, as appropriate, are satisfied.
[b] For oversized or slotted holes, see Table J3.5.
[c] All edge distances in this column are permitted to be reduced 1/8 in. when the hole is at a point where *required strength* does not exceed 25 percent of the maximum strength in the element.
[d] These are permitted to be 1 1/4 in. at the ends of *beam connection* angles and shear end plates.

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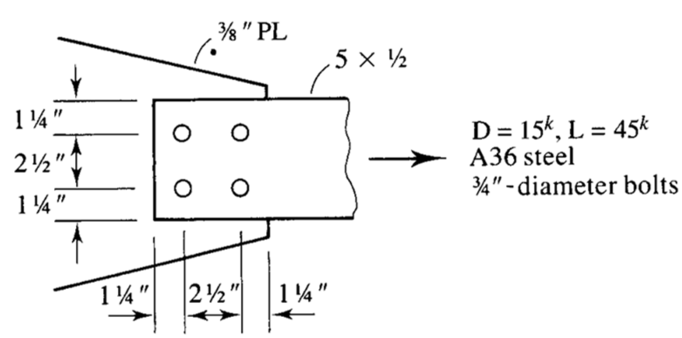
Example (Bearing Type Connections)

- Check bolt spacing, edge distances and bearing in the connection shown.
- Bolts used A325 with threads not in plane of shear.



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Example (Bearing Type Connections)

$$2\tfrac{2}{3}d = 2.667\left(\frac{3}{4}\right) = 2.00 \text{ in.}$$

Actual spacing = 2.50 in. > 2.00 in. (OK)

The minimum edge distance in any direction is obtained from AISC Table J3.4. If we assume sheared edges (the worst case), the minimum edge distance is $1\frac{1}{4}$ in., so

$$\text{Actual edge distance} = 1\frac{1}{4} \text{ in.} \quad \text{(OK)}$$

Example (Bearing Type Connections)

1. Shear strength:

$$R_n = F_{nv} A_b$$

$= (60)(\pi)(3/4)^2 / 4 = 26.46$ kips

$\varphi R_n = (0.75)(26.46) = 19.845$ kips (for each bolt)

For four bolts:

$\varphi R_n = (4)(19.845) = 79.38$ kips

Example (Bearing Type Connections)

2. Bearing Strength:

- Tension member :
- Edge holes:

$$h = d + \frac{1}{16} = \frac{3}{4} + \frac{1}{16} = \frac{13}{16} \text{ in.}$$

$$L_c = L_e - \frac{h}{2} = 1.25 - \frac{13/16}{2} = 0.8438 \text{ in.}$$

$$R_n = 1.2L_c tF_u \leq 2.4dtF_u$$

$$1.2L_c tF_u = 1.2(0.8438)\left(\frac{1}{2}\right)(58) = 29.36 \text{ kips}$$

Example (Bearing Type Connections)

Check upper limit:

$$2.4dtF_u = 2.4\left(\frac{3}{4}\right)\left(\frac{1}{2}\right)(58) = 52.20 \text{ kips}$$

$$29.36 \text{ kips} < 52.20 \text{ kips} \qquad \therefore \text{ use } R_n = 29.36 \text{ kips/bolt}$$

- Other holes:

$$L_c = s - h = 2.5 - \frac{13}{16} = 1.688 \text{ in.}$$

$$R_n = 1.2L_c tF_u \leq 2.4dtF_u$$

$$1.2L_c tF_u = 1.2(1.688)\left(\frac{1}{2}\right)(58) = 58.74 \text{ kips}$$

Example (Bearing Type Connections)

Upper limit (the upper limit is independent of L_c and is the same for all bolts):

$$2.4dtF_u = 52.20 \text{ kips} < 58.74 \text{ kips} \qquad \therefore \text{ use } R_n = 52.20 \text{ kips/bolt}$$

The bearing strength for the tension member is

$$R_n = 2(29.36) + 2(52.20) = 163.1 \text{ kips}$$

- Gusset Plate:
- Edge holes:

$$L_c = L_e - \frac{h}{2} = 1.25 - \frac{13/16}{2} = 0.8438 \text{ in.}$$

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Example (Bearing Type Connections)

$$R_n = 1.2L_c tF_u \le 2.4dtF_u$$

$$1.2L_c tF_u = 1.2(0.8438)\left(\frac{3}{8}\right)(58) = 22.02 \text{ kips}$$

$$\text{Upper limit} = 2.4dtF_u = 2.4\left(\frac{3}{4}\right)\left(\frac{3}{8}\right)(58)$$

$$= 39.15 \text{ kips} > 22.02 \text{ kips} \qquad \therefore \text{ use } R_n = 22.02 \text{ kips/bolt}$$

- Other holes:

$$L_c = s - h = 2.5 - \frac{13}{16} = 1.688 \text{ in.}$$

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Example (Bearing Type Connections)

$$R_n = 1.2L_c tF_u \le 2.4dtF_u$$

$$1.2L_c tF_u = 1.2(1.688)\left(\frac{3}{8}\right)(58) = 44.06 \text{ kips}$$

Upper limit $= 2.4dtF_u = 39.15$ kips < 44.06 kips $\quad \therefore$ use $R_n = 39.15$ kips/bolt

The bearing strength for the gusset plate is

$$R_n = 2(22.02) + 2(39.15) = 122.3 \text{ kips}$$

The gusset plate controls. The nominal bearing strength for the connection is therefore

$$R_n = 122.3 \text{ kips}$$

The design strength is $\phi R_n = 0.75(122.3) = 91.7$ kips.

Example (Bearing Type Connections)

- Tension Failure:
- Yielding:
- Φt Pn = (0.9)(36)(5)(0.5) = 81 kips
- Fracture: …………………
- Block shear: ……………………
- Control (without calculating Tension failure) = 79.38 kips
- Ru = 1.2(15) +(1.6)(45) = 90 kips
- 79.38 < 90 kips (N.G)

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Slip Critical Connections

$$R_n = \mu D_u h_{sc} T_b N_s$$

where

μ = mean slip coefficient (coefficient of static friction) = 0.35 for Class A surfaces

D_u = ratio of mean actual bolt pretension to the specified minimum pretension. This is to be taken as 1.13 unless another factor can be justified.

h_{sc} = hole factor = 1.0 for standard holes

T_b = minimum fastener tension from AISC Table J3.1

N_s = number of slip planes (shear planes)

Slip Critical Connections

TABLE J3.1
Minimum Bolt Pretension, kips*

Bolt Size, in.	A325 Bolts	A490 Bolts
1/2	12	15
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1 1/8	56	80
1 1/4	71	102
1 3/8	85	121
1 1/2	103	148

*Equal to 0.70 times the minimum *tensile strength* of bolts, rounded off to nearest kip, as specified in ASTM specifications for A325 and A490 bolts with UNC threads.

Slip Critical Connections

If slip is treated as a serviceability limit state, then

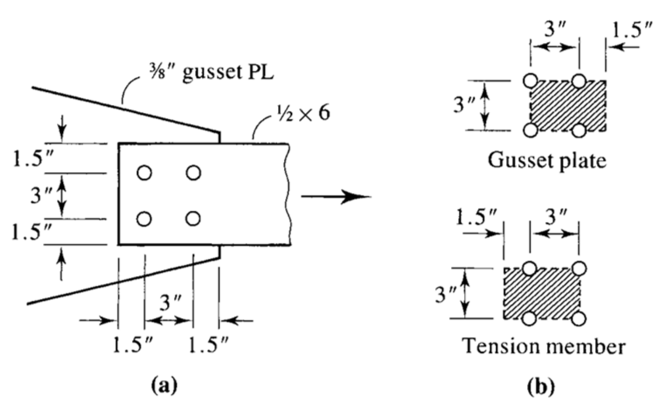
$$\phi = 1.0$$

If slip is treated as a strength limit state,

$$\phi = 0.85$$

Example (Slip Critical Connections)

- ¾ inch diameter, A325 bolts with threads in the shear plane no slip is permitted, A36.



Example (Slip Critical Connections)

Shear strength: For one bolt,

$$A_b = \frac{\pi(3/4)^2}{4} = 0.4418 \text{ in.}^2$$

$$R_n = F_{nv}A_b = 48(0.4418) = 21.21 \text{ kips/bolt}$$

For four bolts,

$$R_n = 4(21.21) = 84.84 \text{ kips}$$

Slip-critical strength: Because no slippage is permitted, this connection is classified as slip-critical (and we will treat slip as a serviceability limit state). From AISC Table J3-1, the minimum bolt tension is $T_b = 28$ kips. From AISC Equation J3-4,

$$R_n = \mu D_u h_{sc} T_b N_s = 0.35(1.13)(1.0)(28)(1.0) = 11.07 \text{ kips/bolt}$$

For four bolts,

$$R_n = 4(11.07) = 44.28 \text{ kips}$$

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Example (Slip Critical Connections)

Bearing strength: Since both edge distances are the same, and the gusset plate is thinner than the tension member, the gusset plate thickness of ⅜ inch will be used.

$$h = d + \frac{1}{16} = \frac{3}{4} + \frac{1}{16} = \frac{13}{16} \text{ in.}$$

For the holes nearest the edge of the gusset plate,

$$L_c = L_e - \frac{h}{2} = 1.5 - \frac{13/16}{2} = 1.094 \text{ in.}$$

$$R_n = 1.2L_c t F_u = 1.2(1.094)\left(\frac{3}{8}\right)(58) = 28.55 \text{ kips}$$

$$\text{Upper limit} = 2.4dtF_u = 2.4\left(\frac{3}{4}\right)\left(\frac{3}{8}\right)(58)$$

$$= 39.15 \text{ kips} > 28.55 \text{ kips} \quad \therefore \text{ use } R_n = 28.55 \text{ kips for this bolt}$$

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Example (Slip Critical Connections)

For the other holes,

$$L_c = s - h = 3 - \frac{13}{16} = 2.188 \text{ in.}$$

$$R_n = 1.2L_c tF_u = 1.2(2.188)\left(\frac{3}{8}\right)(58) = 57.11 \text{ kips}$$

Upper limit $= 2.4dtF_u$

$= 39.15 \text{ kips} < 57.11 \text{ kips} \quad \therefore$ use $R_n = 39.15$ kips for this bolt

The nominal bearing strength for the connection is

$$R_n = 2(28.55) + 2(39.15) = 135.4 \text{ kips}$$

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Example (Slip Critical Connections)

Tension on the gross area:

$$P_n = F_y A_g = 36\left(6 \times \frac{1}{2}\right) = 108.0 \text{ kips}$$

Tension on the net area: All elements of the cross section are connected, so shear lag is not a factor and $A_e = A_n$. For the hole diameter, use

$$h = d + \frac{1}{8} = \frac{3}{4} + \frac{1}{8} = \frac{7}{8} \text{ in.}$$

The nominal strength is

$$P_n = F_u A_e = F_u t(w_g - \Sigma h) = 58\left(\frac{1}{2}\right)\left[6 - 2\left(\frac{7}{8}\right)\right] = 123.3 \text{ kips}$$

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Example (Slip Critical Connections)

Block shear strength:

$$A_{gv} = 2 \times \frac{3}{8}(3+1.5) = 3.375 \text{ in.}^2$$

Since there are 1.5 hole diameters per horizontal line of bolts,

$$A_{nv} = 2 \times \frac{3}{8}\left[3+1.5-1.5\left(\frac{7}{8}\right)\right] = 2.391 \text{ in.}^2$$

For the tension area,

$$A_{nt} = \frac{3}{8}\left(3-\frac{7}{8}\right) = 0.7969 \text{ in.}^2$$

Since the block shear will occur in a gusset plate, $U_{bs} = 1.0$. From AISC Equation J4-5,

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$
$$= 0.6(58)(2.391) + 1.0(58)(0.7969) = 129.4 \text{ kips}$$

with an upper limit of

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(36)(3.375) + 1.0(58)(0.7969) = 119.1 \text{ kips}$$

The nominal block shear strength is therefore 119.1 kips.

Example (Slip Critical Connections)

Bolt shear strength:

$$\phi R_n = 0.75(84.84) = 63.6 \text{ kips}$$

Slip-critical strength: Since slip is being treated as a serviceability limit state, $\phi = 1.0$.

$$\phi R_n = 1.0(44.28) = 44.3 \text{ kips}$$

Bearing strength:

$$\phi R_n = 0.75(135.4) = 102 \text{ kips}$$

Tension on the gross area:

$$\phi_t P_n = 0.90(108.0) = 97.2 \text{ kips}$$

Tension on the net area:

$$\phi_t P_n = 0.75(123.3) = 92.5 \text{ kips}$$

Block shear strength:

$$\phi R_n = 0.75(119.1) = 89.3 \text{ kips}$$

Design strength = 44.3 kips.

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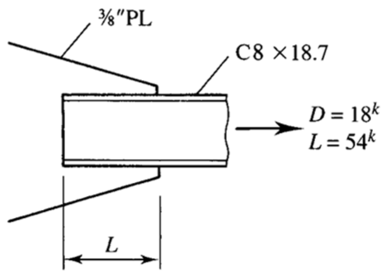
Design Example

The C8 × 18.7 shown in Figure 7.15 has been selected to resist a service dead load of 18 kips and a service live load of 54 kips. It is to be attached to a ⅜-inch gusset plate with ⅞-inch-diameter, A325 bolts. Assume that the threads are in the plane of shear and that slip of the connection is permissible. Determine the number and required layout of bolts such that the length of connection L is a minimum. A36 steel is used.



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Design Example

Shear:

$$A_b = \frac{\pi(7/8)^2}{4} = 0.6013 \text{ in.}^2$$

$$R_n = F_{nv}A_b = 48(0.6013) = 28.86 \text{ kips/bolt}$$

Bearing: The gusset plate is thinner than the web of the channel and will control. Assume that along a line parallel to the force, the length L_c is large enough so that the upper limit will control. Then

$$R_n = 2.4dtF_u = 2.4\left(\frac{7}{8}\right)\left(\frac{3}{8}\right)(58) = 45.68 \text{ kips}$$

and shear controls. The bearing strength will need to be verified once the actual bolt layout is determined.

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Design Example

The factored load is

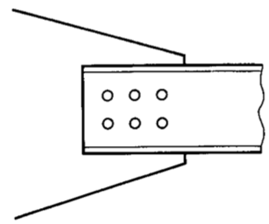
$$1.2D + 1.6L = 1.2(18) + 1.6(54) = 108.0 \text{ kips}$$

The design strength per bolt, based on shear, is

$$\phi R_n = 0.75(28.86) = 21.65 \text{ kips}$$

The number of bolts required is

$$\frac{108}{21.65} = 4.99 \text{ bolts}$$



Design Example

$$P_n = F_y A_g = 36(5.51) = 198.4 \text{ kips}$$

The design strength is

$$\phi_t P_n = 0.90(198.4) = 179 \text{ kips}$$

Tension on the effective net area:

$$A_n = 5.51 - 2\left(\frac{7}{8} + \frac{1}{8}\right)(0.487) = 4.536 \text{ in.}^2$$

$$A_e = A_n U = 4.536(0.60) = 2.722 \text{ in.}^2$$

$$P_n = F_u A_e = 58(2.722) = 157.9 \text{ kips}$$

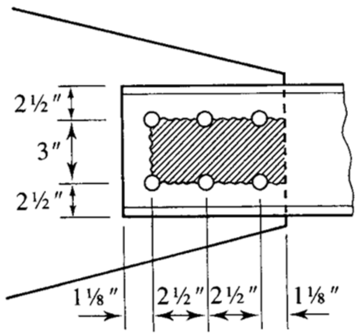
$$\phi_t P_n = 0.75(157.9) = 118 \text{ kips} \quad \text{(controls)}$$

Design Example

$$\text{Minimum spacing} = 2.667\left(\frac{7}{8}\right) = 2.33 \text{ in.}$$

From AISC Table J3.4,

Minimum edge distance = 1⅛ in.



Design Example

$$h = d + \frac{1}{16} = \frac{7}{8} + \frac{1}{16} = \frac{15}{16} \text{ in.}$$

For the holes nearest the edge of the gusset plate,

$$L_c = L_e - \frac{h}{2} = 1.125 - \frac{15/16}{2} = 0.6563 \text{ in.}$$

$$R_n = 1.2L_c t F_u = 1.2(0.6563)\left(\frac{3}{8}\right)(58) = 17.13 \text{ kips}$$

$$\text{Upper limit} = 2.4dtF_u = 2.4\left(\frac{7}{8}\right)\left(\frac{3}{8}\right)(58)$$

$$= 45.68 \text{ kips} > 17.13 \text{ kips} \qquad \therefore \text{ use } R_n = 17.13 \text{ kips for this bolt}$$

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Design Example

For the other holes,

$$L_c = s - h = 2.5 - \frac{15}{16} = 1.563 \text{ in.}$$

$$R_n = 1.2L_c tF_u = 1.2(1.563)\left(\frac{3}{8}\right)(58) = 40.79 \text{ kips}$$

Upper limit $= 2.4dtF_u$

$= 45.68$ kips > 40.79 kips $\quad \therefore$ use $R_n = 40.79$ kips for this bolt

The total nominal bearing strength for the connection is

$$R_n = 2(17.13) + 4(40.79) = 197.4 \text{ kips}$$

The design bearing strength is

$$\phi R_n = 0.75(197.4) = 148 \text{ kips} > P_u = 108 \text{ kips} \quad \text{(OK)}$$

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Design Example

Shear areas:

$$A_{gv} = 2 \times \frac{3}{8}(2.5 + 2.5 + 1.125) = 4.594 \text{ in.}^2$$

$$A_{nv} = 2 \times \frac{3}{8}[6.125 - 2.5(1.0)] = 2.719 \text{ in.}^2$$

Tension area:

$$A_{nt} = \frac{3}{8}(3 - 1.0) = 0.7500 \text{ in.}^2$$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt}$$
$$= 0.6(58)(2.719) + 1.0(58)(0.7500) = 138.1 \text{ kips}$$

with an upper limit of

$$0.6F_y A_{gv} + U_{bs} F_u A_{nt} = 0.6(36)(4.594) + 1.0(58)(0.7500) = 142.7 \text{ kips}$$

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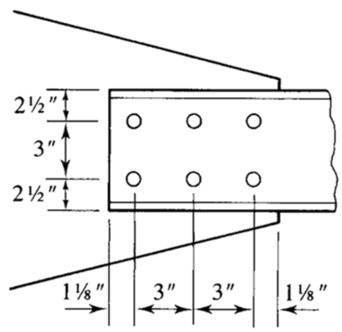
Design Example

The nominal block shear strength is therefore 138.1 kips, and the design strength is

$$\phi R_n = 0.75(138.1) = 104 \text{ kips} < 108 \text{ kips} \qquad \text{(N.G.)}$$



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Design Example

$$\phi R_n = 0.75(0.6F_u A_{nv} + U_{bs} F_u A_{nt})$$
$$= 0.75[0.6(58)A_{nv} + 1.0(58)(0.7500)] = 108 \text{ kips}$$

Required $A_{nv} = 2.888 \text{ in.}^2$

$$A_{nv} = \frac{3}{8}[s + s + 1.125 - 2.5(1.0)](2) = 2.888 \text{ in.}^2$$

Required $s = 2.61$ in. $\quad \therefore$ use $s = 3$ in.

Compute the actual block shear strength.

$$A_{gv} = 2 \times \frac{3}{8}(3 + 3 + 1.125) = 5.344 \text{ in.}^2$$

$$A_{nv} = 5.344 - \frac{3}{8}(2.5 \times 1.0)(2) = 3.469 \text{ in.}^2$$

$$\phi R_n = 0.75(0.6F_u A_{nv} + U_{bs} F_u A_{nt})$$
$$= 0.75[0.6(58)(3.469) + 1.0(58)(0.7500)] = 123 \text{ kips} > 108 \text{ kips} \quad \text{(OK)}$$

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Design Example

Check the upper limit:

$$\phi[0.6F_yA_{gv} + U_{bs}F_uA_{nt}] = 0.75[0.6(36)(5.344) + 1.0(58)(0.7500)]$$
$$= 119 \text{ kips} < 123 \text{ kips}$$

Therefore, the upper limit controls, but the strength is still adequate.

Using the spacing and edge distances selected, the minimum length is, therefore,

$L = 1\frac{1}{8}$ in. at the end of the channel
+ 2 spaces at 3 in.
+ $1\frac{1}{8}$ in. at the end of the gusset plate
= $8\frac{1}{4}$ in. total

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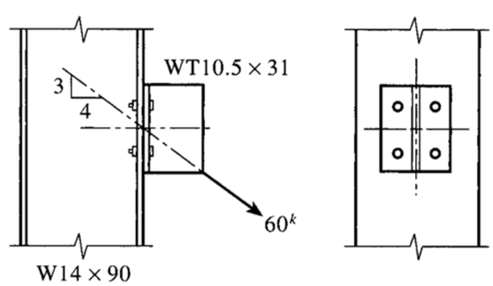
Bolts Subjected to Shear and Tension

- The vertical component f the force will put the bolts in shear, while the horizontal component will cause tension on the bolts.



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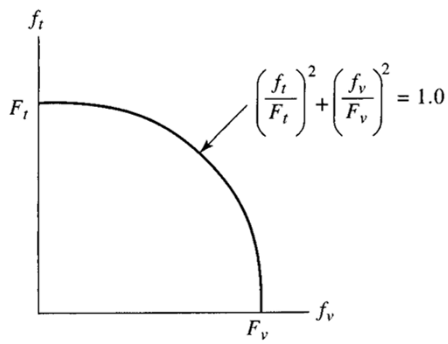
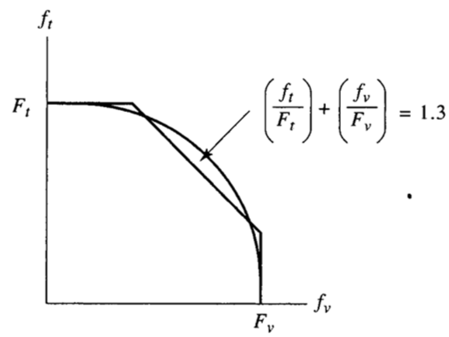
Bolts Subjected to Shear and Tension

- <u>Bearing Type connection:</u>





Bolts Subjected to Shear and Tension

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt}$$ where $\phi = 0.75$.

where

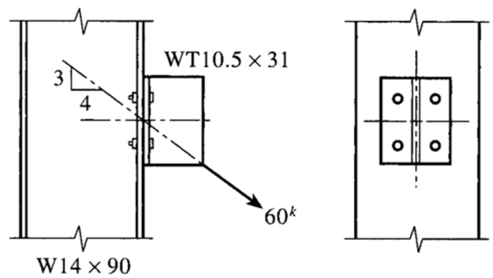
F'_{nt} = nominal tensile stress in the presence of shear
F_{nt} = nominal tensile stress in the absence of shear
F_{nv} = nominal shear stress in the absence of tension
f_v = required shear stress

Bearing-type connections:

1. Check shear and bearing against the usual strengths.
2. Check tension against the reduced tensile strength usi

Example (Bolts Subjected to Shear and Tension)

A WT10.5 × 31 is used as a bracket to transmit a 60-kip service load to a W14 × 90 column, as previously shown in Figure 7.30. The load consists of 15 kips dead load and 45 kips live load. Four ⅞-inch-diameter A325 bolts are used. The column is of A992 steel, and the bracket is A36. Assume all spacing and edge-distance requirements are satisfied, including those necessary for the use of the maximum nominal strength in bearing (i.e., $2.4dtF_u$), and determine the adequacy of the bolts for the fol-



Example (Bolts Subjected to Shear and Tension)

Compute the nominal bearing strength (flange of tee controls).

$$R_n = 2.4dtF_u = 2.4\left(\frac{7}{8}\right)(0.615)(58) = 74.91 \text{ kips}$$

Nominal shear strength:

$$A_b = \frac{\pi(7/8)^2}{4} = 0.6013 \text{ in.}^2$$

$$R_n = F_{nv}A_b = 48(0.6013) = 28.9 \text{ kips}$$

$$P_u = 1.2D + 1.6L = 1.2(15) + 1.6(45) = 90 \text{ kips}$$

Example (Bolts Subjected to Shear and Tension)

The total shear/bearing load is

$$V_u = \frac{3}{5}(90) = 54 \text{ kips}$$

The shear/bearing force per bolt is

$$V_{u\,\text{bolt}} = \frac{54}{4} = 13.5 \text{ kips}$$

The design bearing strength is

$$\phi R_n = 0.75(74.91) = 56.2 \text{ kips} > 13.5 \text{ kips} \qquad \text{(OK)}$$

The design shear strength is

$$\phi R_n = 0.75(28.9) = 21.7 \text{ kips} > 13.5 \text{ kips} \qquad \text{(OK)}$$

Example (Bolts Subjected to Shear and Tension)

The total tension load is

$$T_u = \frac{4}{5}(90) = 72 \text{ kips}$$

The tensile force per bolt is

$$T_{u\,\text{bolt}} = \frac{72}{4} = 18 \text{ kips}$$

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nt}$$

where

F_{nt} = nominal tensile stress in the absence of shear = 90 ksi

F_{nv} = nominal shear stress in the absence of tension = 48 ksi

$$f_v = \frac{V_{u\,\text{bolt}}}{A_b} = \frac{13.5}{0.6013} = 22.45 \text{ ksi}$$

Example (Bolts Subjected to Shear and Tension)

$$F'_{nt} = 1.3(90) - \frac{90}{0.75(48)}(22.45) = 60.88 \text{ ksi} < 90 \text{ ksi}$$

The nominal tensile strength is

$$R_n = F'_{nt}A_b = 60.88(0.6013) = 36.61 \text{ kips}$$

and the available tensile strength is

$$\phi R_n = 0.75(36.61) = 27.5 \text{ kips} > 18 \text{ kips} \qquad \text{(OK)}$$

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Bolts Subjected to Shear and Tension

- Slip critical connections:

$$k_s = 1 - \frac{T_u}{D_u T_b N_b}$$

where

T_u = total factored tensile load on the connection

D_u = ratio of mean bolt pretension to specified minimum pretension; default value is 1.13

T_b = prescribed initial bolt tension from AISC Table J3.1

N_b = number of bolts in the connection

Slip-critical connections:

1. Check tension, shear, and bearing against the usual strengths.
2. Check the slip-critical load against the reduced slip-critical strength.

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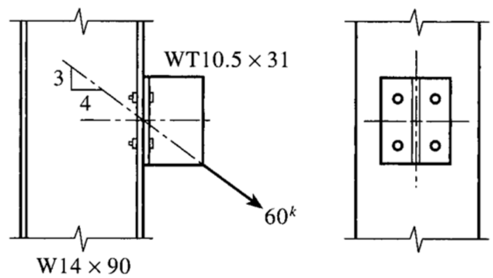
Example (Bolts Subjected to Shear and Tension- slip critical connection)

A WT10.5 × 31 is used as a bracket to transmit a 60-kip service load to a W14 × 90 column, as previously shown in Figure 7.30. The load consists of 15 kips dead load and 45 kips live load. Four ⅞-inch-diameter A325 bolts are used. The column is of A992 steel, and the bracket is A36. Assume all spacing and edge-distance requirements are satisfied, including those necessary for the use of the maximum nominal strength in bearing (i.e., $2.4dtF_u$), and determine the adequacy of the bolts for the fol-



Example (Bolts Subjected to Shear and Tension- slip critical connection)

$$R_n = \mu D_u h_{sc} T_b N_s \times 4 = 0.35(1.13)(1.0)(39)(1) \times 4 = 61.70 \text{ kips}$$

$$\phi R_n = 1.0(61.70) = 61.70 \text{ kips}$$

$$k_s = 1 - \frac{T_u}{D_u T_b N_b} = 1 - \frac{72}{1.13(39)(4)} = 0.5916$$

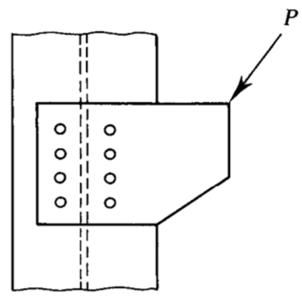
The reduced strength is therefore

$$k_s(61.70) = 0.5916(61.70) = 36.5 \text{ kips} < 54 \text{ kips} \qquad \text{(N.G.)}$$

The connection is inadequate as a slip-critical connection.

Eccentric Connections (Shear Only)

- **Elastic Analysis:**



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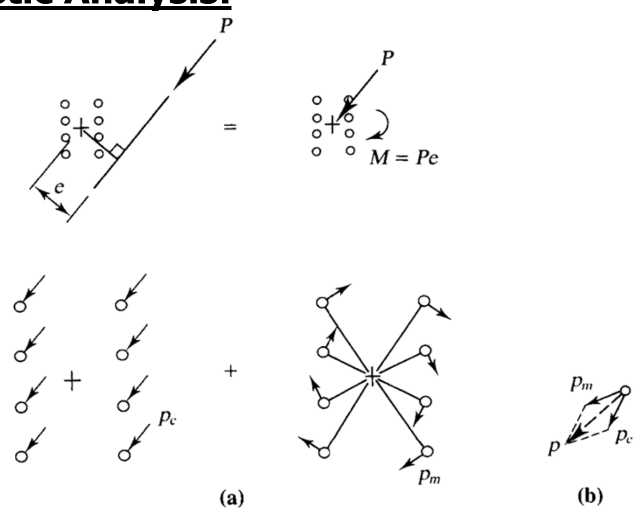
Eccentric Connections (Shear Only)

- **Elastic Analysis:**



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Eccentric Connections (Shear Only)

$$p_c = P/n,$$

$$f_v = \frac{Md}{J}$$

where

d = distance from the centroid of the area to the point where the stress is being computed

J = polar moment of inertia of the area about the centroid

- *fv* : Shearing stress in each bolt.

Eccentric Connections (Shear Only)

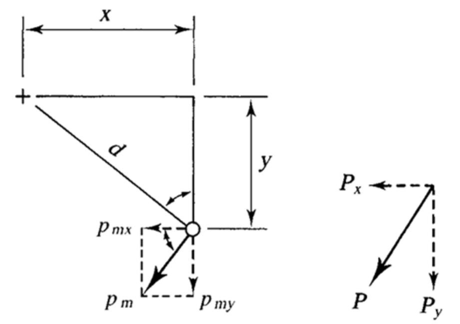
$$J = \sum Ad^2 = A \sum d^2$$

$$f_v = \frac{Md}{A \sum d^2}$$

$$p_m = Af_v = A\frac{Md}{A \sum d^2} = \frac{Md}{\sum d^2}$$

$$p_{cx} = \frac{P_x}{n} \text{ and } p_{cy} = \frac{P_y}{n}$$

Eccentric Connections (Shear Only)



$$p_{my} = \frac{Mx}{\Sigma(x^2 + y^2)}$$

and the total fastener force is

$$p = \sqrt{(\Sigma\, p_x)^2 + (\Sigma\, p_y)^2}$$

$$\Sigma\, p_x = p_{cx} + p_{mx}$$
$$\Sigma\, p_y = p_{cy} + p_{my}$$

$$\Sigma\, d^2 = \Sigma(x^2 + y^2)$$

$$p_{mx} = \frac{y}{d}\, p_m = \frac{y}{d}\,\frac{Md}{\Sigma\, d^2} = \frac{y}{d}\,\frac{Md}{\Sigma(x^2 + y^2)} = \frac{My}{\Sigma(x^2 + y^2)}$$

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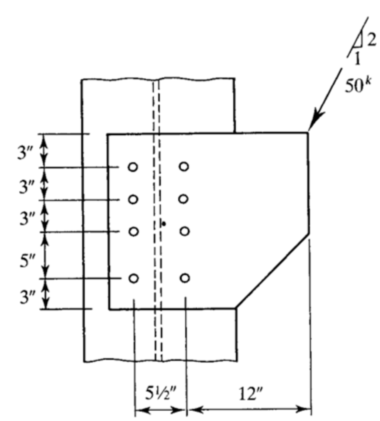
Example (Eccentric Connections (Elastic Analysis)

Determine the critical fastener force in the bracket connection



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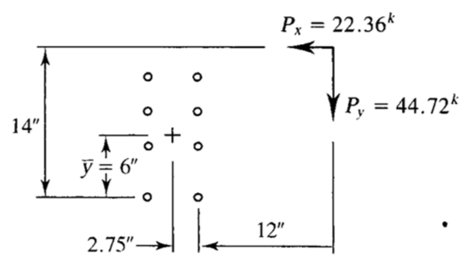
Example (Eccentric Connections (Elastic Analysis)

$$\bar{y} = \frac{2(5) + 2(8) + 2(11)}{8} = 6 \text{ in.}$$

$$P_x = \frac{1}{\sqrt{5}}(50) = 22.36 \text{ kips} \leftarrow \quad \text{and} \quad p_y = \frac{2}{\sqrt{5}}(50) = 44.72 \text{ kips} \downarrow$$

$$M = 44.72(12 + 2.75) - 22.36(14 - 6) = 480.7 \text{ in.-kips} \qquad \text{(clockwise)}$$



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Example (Eccentric Connections (Elastic Analysis)

$$p_{cx} = \frac{22.36}{8} = 2.795 \text{ kips} \leftarrow \quad \text{and} \quad p_{cy} = \frac{44.72}{8} = 5.590 \text{ kips} \downarrow$$

$$\sum(x^2 + y^2) = 8(2.75)^2 + 2[(6)^2 + (1)^2 + (2)^2 + (5)^2] = 192.5 \text{ in.}^2$$

$$p_{mx} = \frac{My}{\sum(x^2 + y^2)} = \frac{480.7(6)}{192.5} = 14.98 \text{ kips} \leftarrow$$

$$p_{my} = \frac{Mx}{\sum(x^2 + y^2)} = \frac{480.7(2.75)}{192.5} = 6.867 \text{ kips} \downarrow$$

$$\sum p_x = 2.795 + 14.98 = 17.78 \text{ kips} \leftarrow$$

$$\sum p_y = 5.590 + 6.867 = 12.46 \text{ kips} \downarrow$$

$$p = \sqrt{(17.78)^2 + (12.46)^2} = 21.7 \text{ kips}$$

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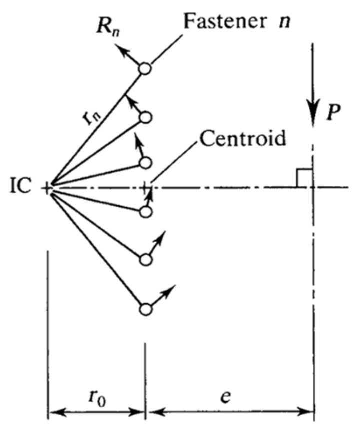
Eccentric Connections (Shear Only)

- **Ultimate Strength Analysis:**



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Eccentric Connections (Shear Only)

The bolt force R corresponding to a deformation Δ is

$$R = R_{ult}(1 - e^{-\mu\Delta})^{\lambda}$$

where

R_{ult} = bolt shear force at failure
e = base of natural logarithms
μ = a regression coefficient = 10
λ = a regression coefficient = 0.55

1. At failure, the fastener group rotates about an instantaneous center (IC).
2. The deformation of each fastener is proportional to its distance from the IC and acts perpendicularly to the radius of rotation.
3. The capacity of the connection is reached when the ultimate strength of the fastener farthest from the IC is reached.

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Eccentric Connections (Shear Only)

$$\Delta = \frac{r}{r_{max}} \Delta_{max} = \frac{r}{r_{max}} (0.34)$$

where

r = distance from the IC to the fastener

r_{max} = distance to the farthest fastener

Δ_{max} = deformation of the farthest fastener at ultimate = 0.34 in. (determined experimentally)

$$R_y = \frac{x}{r} R \text{ and } R_x = \frac{y}{r} R$$

$$\Sigma F_x = \sum_{n=1}^{m} (R_x)_n - P_x = 0$$

$$M_{IC} = P(r_0 + e) - \sum_{n=1}^{m} (r_n \times R_n) = 0$$

$$\Sigma F_y = \sum^{m} (R_y)_n - P_y = 0$$

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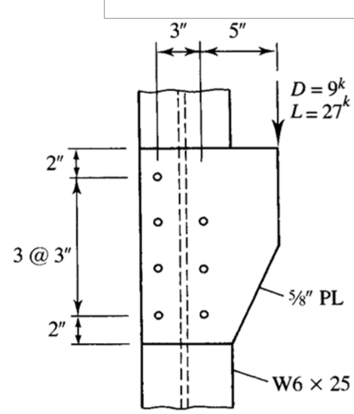
Example (Eccentric Connections (Ultimate Analysis)

The bracket connection shown must support an eccentric load consisting of 9 kips of dead load and 27 kips of live load. The connection was designed to have two vertical rows of four bolts, but one bolt was inadvertently omitted. If ⅞-inch-diameter A325 bearing-type bolts are used, is the connection adequate? Assume that the bolt threads are in the plane of shear. Use A36 steel for the bracket, A992 steel for the W6 × 25,



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Example (Eccentric Connections (Ultimate Analysis)

Compute the bolt shear strength.

$$A_b = \frac{\pi(7/8)^2}{4} = 0.6013 \text{ in.}^2$$

$$R_n = F_{nv}A_b = 48(0.6013) = 28.86 \text{ kips}$$

- For the bearing strength, use a hole diameter of

$$h = d + \frac{1}{16} = \frac{7}{8} + \frac{1}{16} = \frac{15}{16} \text{ in.}$$

For the holes nearest the edge, use

$$L_c = L_e - \frac{h}{2} = 2 - \frac{15/16}{2} = 1.531 \text{ in.}$$

Example (Eccentric Connections (Ultimate Analysis)

The strength of the W6 × 25 will control.

$$R_n = 1.2L_c tF_u = 1.2(1.531)(0.455)(65) = 54.34 \text{ kips}$$

$$\text{Upper limit} = 2.4dtF_u = 2.4\left(\frac{7}{8}\right)(0.455)(65)$$

$$= 62.11 \text{ kips} > 54.34 \text{ kips} \quad \therefore \text{ use } R_n = 54.34 \text{ kips for this bolt}$$

For the other holes, use $s = 3$ in. Then,

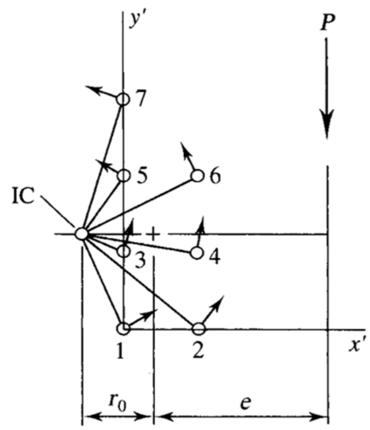
$$L_c = s - h = 3 - \frac{15}{16} = 2.063 \text{ in.}$$

$$R_n = 1.2\ L_c tF_u = 1.2(2.063)(0.455)(65) = 73.22 \text{ kips}$$

$$2.4dtF_u = 62.11 \text{ kips} < 73.22 \text{ kips} \quad \therefore \text{ use } R_n = 62.11 \text{ kips for these bolts}$$

Both bearing values are larger than the bolt shear strength, so the nominal shear strength of $R_n = 28.86$ kips controls.

Example (Eccentric Connections (Ultimate Analysis)



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Example (Eccentric Connections (Ultimate Analysis)

	Origin at Bolt 1		Origin at IC						
Fastener	x'	y'	x	y	r	Δ	R	rR	Ry
1	0.000	0.000	0.285	–3.857	3.868	0.255	70.774	273.731	5.221
2	3.000	0.000	3.285	–3.857	5.067	0.334	72.553	367.598	47.045
3	0.000	3.000	0.285	–0.857	0.903	0.060	47.649	43.046	15.050
4	3.000	3.000	3.285	–0.857	3.395	0.224	69.563	236.188	67.310
5	0.000	6.000	0.285	2.143	2.162	0.143	63.631	137.555	8.398
6	3.000	6.000	3.285	2.143	3.922	0.259	70.891	278.061	59.377
7	0.000	9.000	0.285	5.143	5.151	0.340	72.631	374.107	4.023
Sum								1710.287	206.424

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Example (Eccentric Connections (Ultimate Analysis)

$$P(r_0 + e) = \sum rR$$

$$P = \frac{\sum rR}{r_0 + e} = \frac{1710.29}{1.57104 + 6.71429} = 206.424 \text{ kips}$$

$$\sum F_y = \sum R_y - P = 206.424 - 206.424 = 0.000$$

$$P\left(\frac{R_n}{74}\right) = 206.4\left(\frac{28.86}{74}\right) = 80.50 \text{ kips}$$

The design strength of the connection is

$0.75(80.50) = 60.4 \text{ kips} > 54 \text{ kips}$ (OK)

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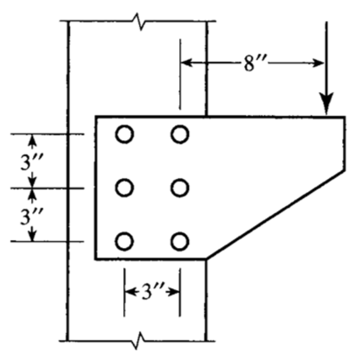
Example (Eccentric Connections , Ultimate Analysis, Tables)

- Bolts are ¾ inch
- A325 bearing with threads in plane of shear.
- Bolts are in single shear.



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Example (Eccentric Connections , Ultimate Analysis, Tables)

This connection corresponds to the connections in Table 7-8, for Angle = 0°. The eccentricity is

$e_x = 8 + 1.5 = 9.5$ in.

The number of bolts per vertical row is

$n = 3$

From Table 7-8,

$C = 1.53$ by interpolation

The nominal strength of a ¾-inch-diameter bolt in single shear is

$r_n = F_{nv}A_b = 48(0.4418) = 21.21$ kips

(Here we use r_n for the nominal strength of a single bolt and R_n for the strength of the connection.)

The nominal strength of the connection is

$R_n = Cr_n = 1.53(21.21) = 32.45$ kips $\phi R_n = 0.75(32.45) = 24.3$ kips.

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sign