AL-MUTHANNA UNIVERSITY
COLLEGE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING

DESIGN OF STEEL STRUCTURES-I

Ziyad Kubba

Assist. Prof. (Dr.)

Design of Steel Structures-I

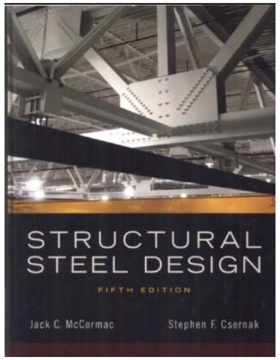
References:

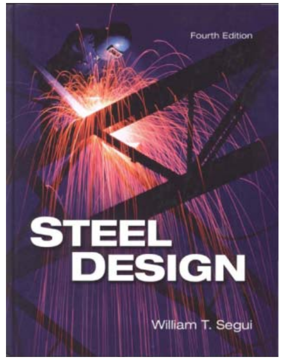
1. McCormac, Jack, C.& Csernak, Stephen, F.; Structural Steel Design; 5th Edition, 2013, Pearson.

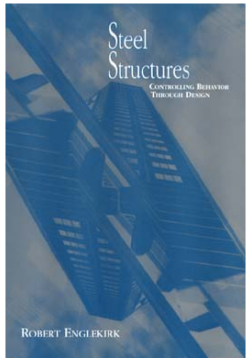
2. Segui, Wiliam, T.; Design of Steel Structures, 4th Edition; 2007, Thomson.

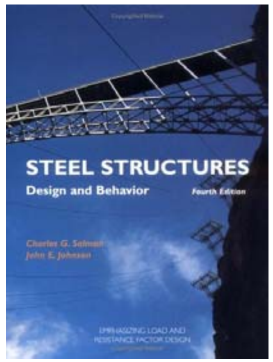
3. Salmon, Charles, G.; Johnson, John, E; Malhas, Faris, A.; Steel Structures Design and Behaviour; 5th Edition; 2009; Pearson.

4. Englekirk, Robert; Steel Structures Controlling Behavior through Design; 2003; John Wiley & Sons Incorporation.





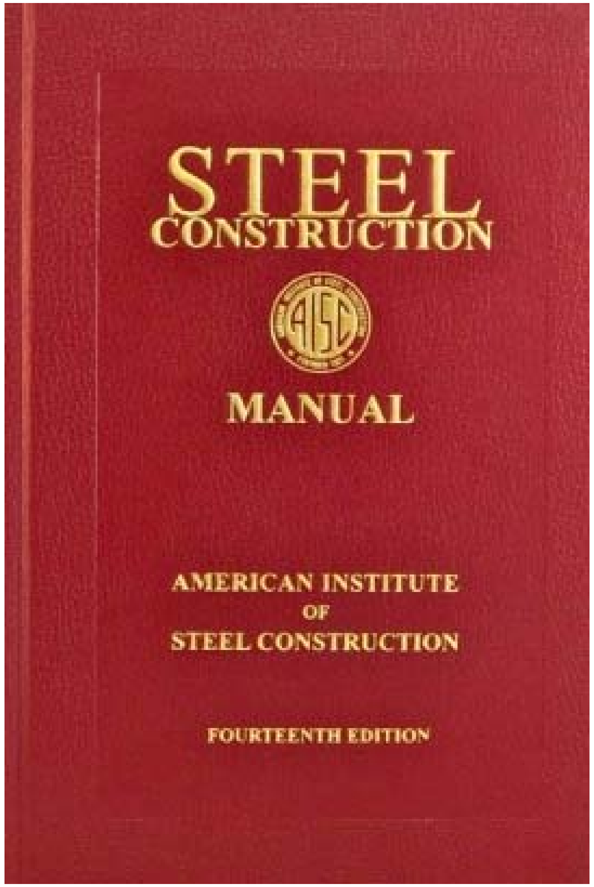




Design of Steel Structures

Manual:

Steel Construction Manual; AISC; American Institute of Steel Construction Incorporation; 13th Edition; ASD & LRFD.



Design of Steel Structures-I

Syllabus:

- **Introduction**
- **Tension Member**
- **Compression Member**
- **Design of Trusses**

Design of Steel Structures-I

NOTES

- Classes
    - 2 Hours Theoretical
    - 1 Hour Tutorial
- Homework & Assignment
    - Must be submitted while properly numbered, dated, solved and arranged.
    - Deadline for submission will be one week after assigning.

Design of Steel Structures-I

- Marks
  - ✓30 marks for First Semester
  - ✓70 marks for End-term Exam of First Semester

- **For First Semester marks:**
  - ✓20% for Homework and assignments
    (This includes submitting solutions of any in-class exam and Mid-term exam).
  - ✓60% for average of in-class exams
  - ✓20% for average of in-class quiz exams

Design of Steel Structures-I

References

- It is mandatory for any student to attend classes is to carry AISC Manual.

- No student will be allowed to attend the classes without AISC Manual.

- **AISC MANUAL IS THE ONLY ALLOWED BOOK TO BE CONSULTED DURING EXAMS.**

Design of Steel Structures-I

References

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Introduction

Advantages of Steel as a Structural Material

1. High Strength per unit of weight

Weight of the structure is small

Long Span Steel Bridge

Tall Buildings

Introduction

Advantages of Steel as a Structural Material

2. Uniformity:

Steel Properties don’t change with time

Less deformations in the structures due to sustained load

(comparing to RCC Structures)

Introduction

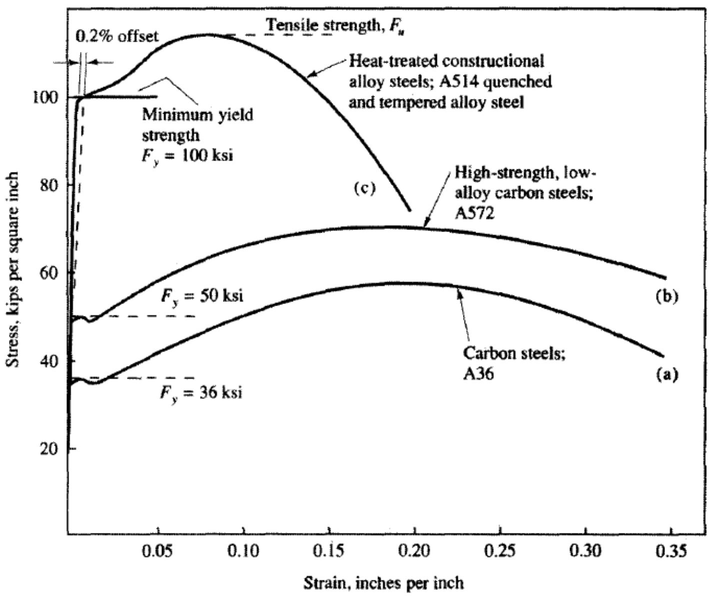
Advantages of Steel as a Structural Material

3. Elasticity:

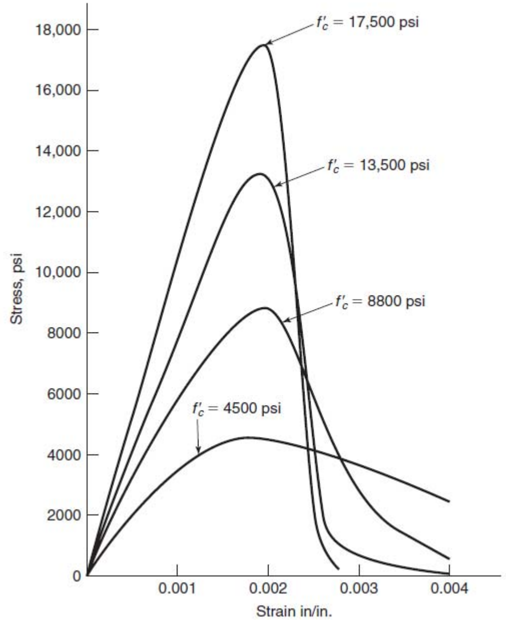
Steel follows Hooke's Law up to fairly high stress

Increase accuracy of calculations like (moment of Inertia

Hooke's Law: $\sigma = E * \varepsilon$



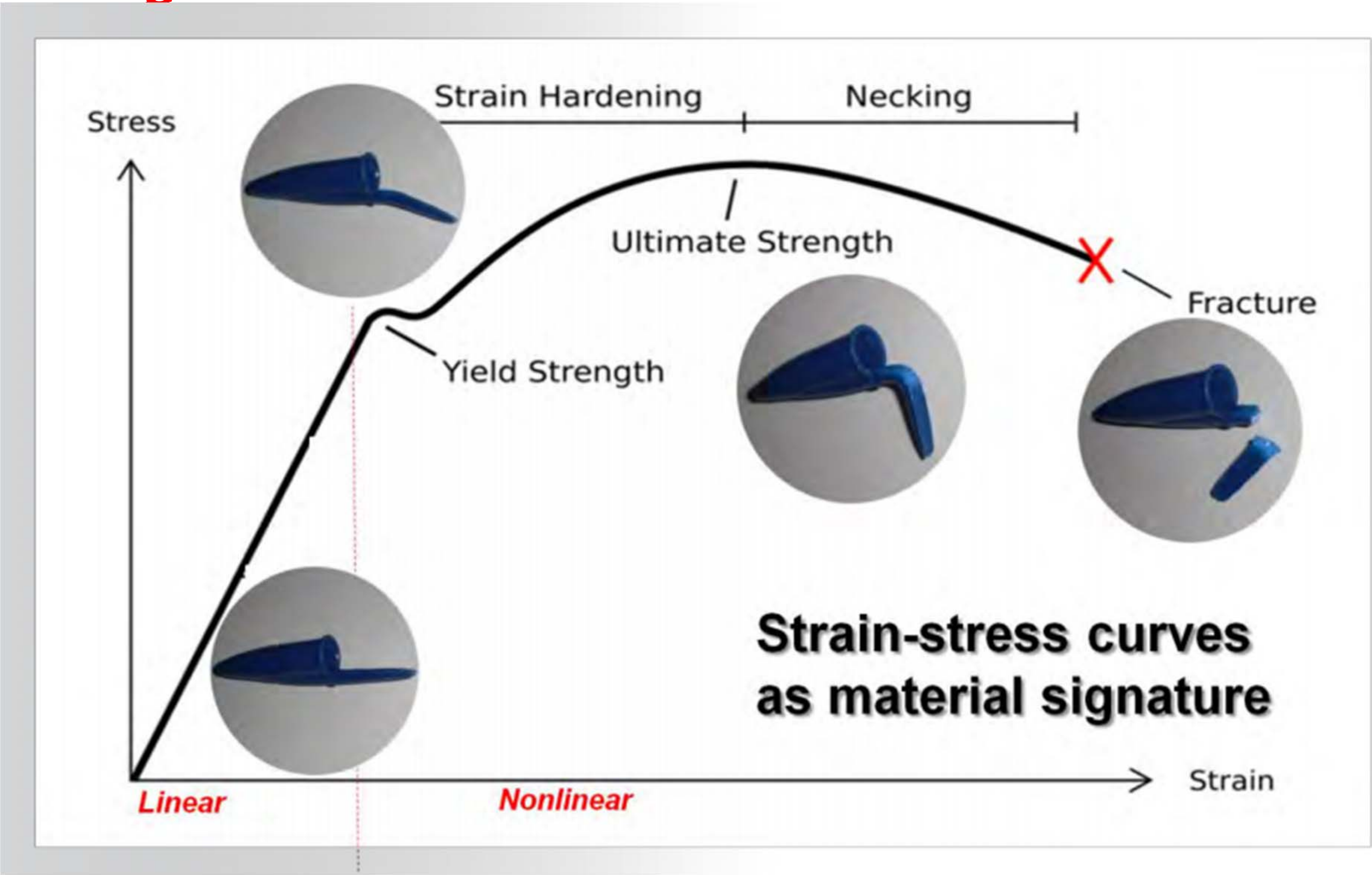
Stress-Strain Curve for steel



Stress-Strain Curve for concrete

Introduction

Advantages of Steel as a Structural Material



Introduction

Advantages of Steel as a Structural Material

4. Permanence:

Properly maintained steel frames will last indefinitely.

5. Ductility:

- *Ductility: is* property of a material by which it can withstand extensive deformation without failure under high tensile stresses.
- A material that does not have this property is generally unacceptable and is probably hard and brittle, and it might break if subjected to a sudden shock.

Introduction

Advantages of Steel as a Structural Material

When a mild or low-carbon structural steel member is being tested in tension

considerable reduction in cross section and a large amount of elongation will occur at the point of failure before the actual fracture occurs

Introduction

Advantages of Steel as a Structural Material

Advantages of Ductility

- When a member overloaded, their large deflections give visible evidence of impending failure.

  (sometimes jokingly referred to as “running time")

Impending Failure

Running Time

Failure

Introduction

Advantages of Steel as a Structural Material

6. Toughness:

- *Toughness:* is the ability of a material to absorb energy in large amounts.
- Tough structural steels have both strength and ductility.
- Toughness enables steel members to be subjected to large deformations during fabrication and erection without failure.

Introduction

Advantages of Steel as a Structural Material

- ***Advantages of Toughness***

Toughness allowing steel members to be:

Bent

Hammered

Sheared

Punched with holes in them

Without visible damage

Introduction

Advantages of Steel as a Structural Material

7.Additions to Existing Structures

Steel structures are suited to having additions made to them:

New bays or even entire new wings can be added to existing steel frame buildings and steel bridges

Introduction

Advantages of Steel as a Structural Material

8. Miscellaneous advantages

- (a) ability to be fastened together by:

✓welds

✓bolts

✓Revits

Introduction

Advantages of Steel as a Structural Material

Miscellaneous advantages of steel

(b) adaptation to prefabrication

(c) speed of erection

(d) ability to be rolled into a wide variety of sizes and shapes

(e) Possible reuse after a structure is disassembled

(f) scrap value

(g) recyclable material.

Introduction

disadvantages of Steel as a Structural Material

1. Corrosion

- Steel is susceptible to corrosion when freely exposed to air and water.

Therefore must be painted periodically

Introduction

disadvantages of Steel as a Structural Material

2. Fireproofing Costs:

Insulating steel frame buildings

sprinkler fire fighting system

Introduction

disadvantages of Steel as a Structural Material

3. Buckling Problem

Global Buckling

Local Buckling

Introduction

disadvantages of Steel as a Structural Material

4. Fatigue Failure:

Fatigue: is the reduced in tensile strength of steel due to cyclic loading (repeated applied load).

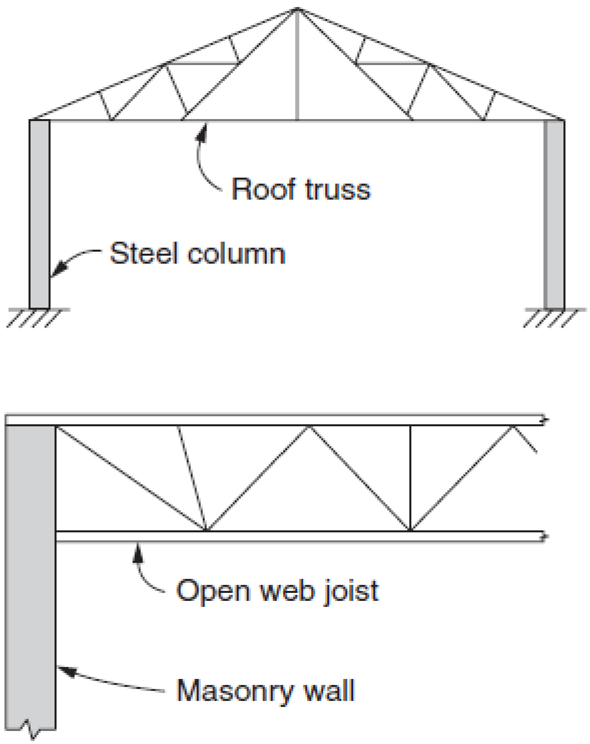
5. Brittle Failure:

- Brittle fracture may occur at places of stress concentration

Types of Steel Buildings

Types of Steel Buildings:

Steel buildings are generally framed structures and range from simple one-story buildings to multistory structures.



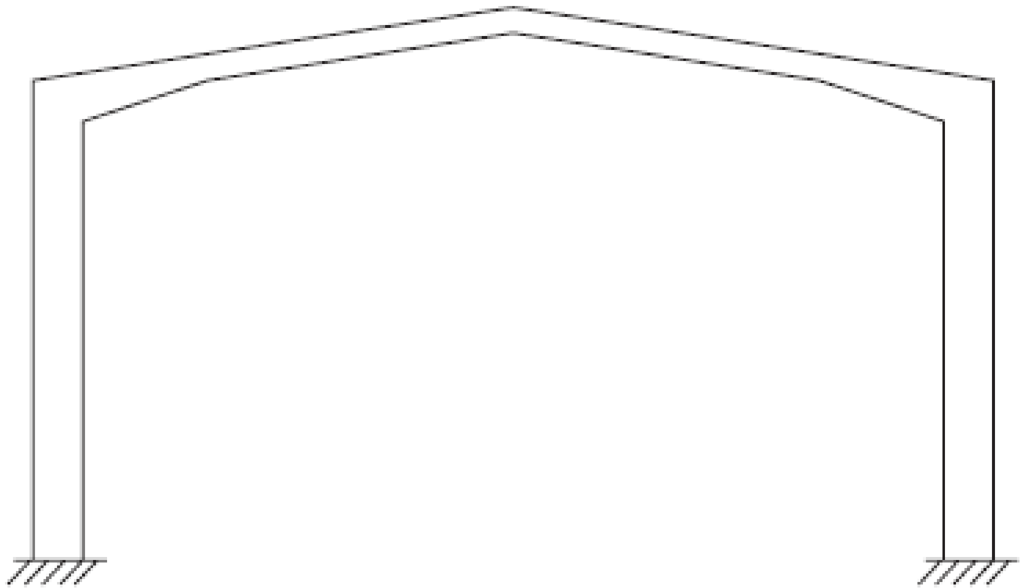
Steel roof construction

Steel roof Structures:

One of the simplest type of structures.
It is constructed with a steel roof truss or open web steel joist supported by steel columns or masonry walls

Types of Steel Buildings:

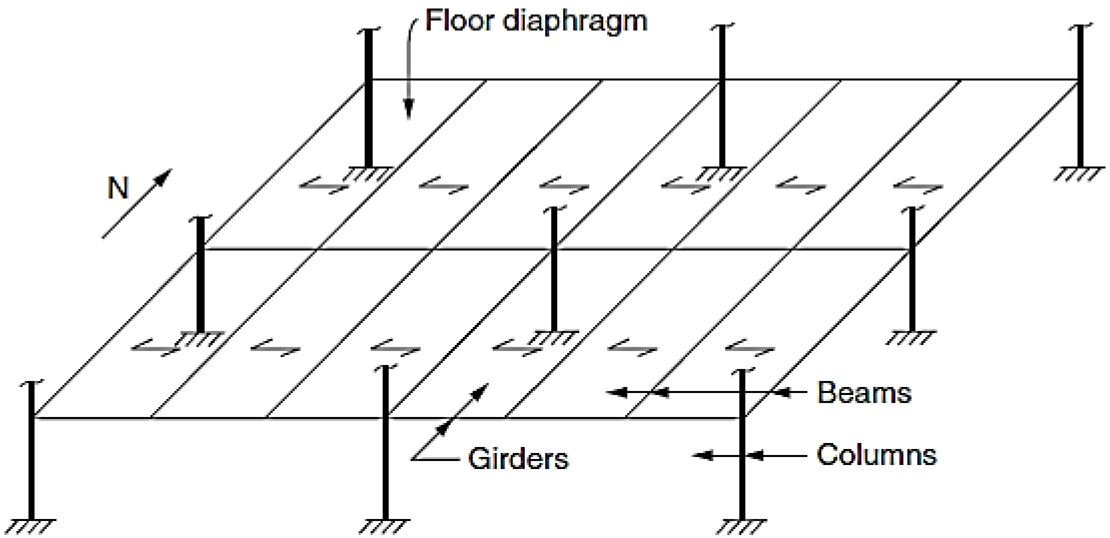
Single Bay Rigid Frame Structures



Single bay rigid frame is an alternative construction technique.

Types of Steel Buildings:

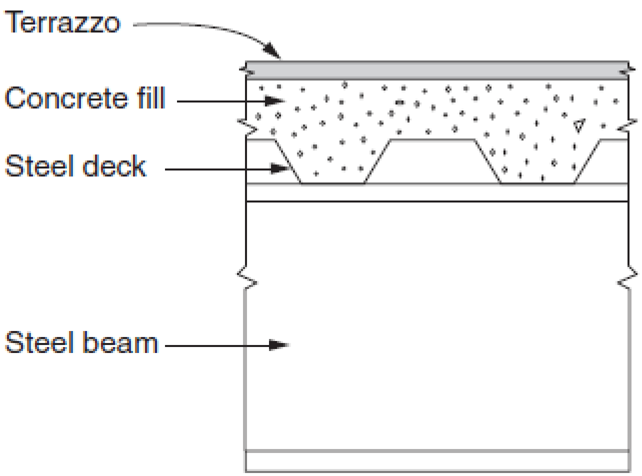
Framed Structures



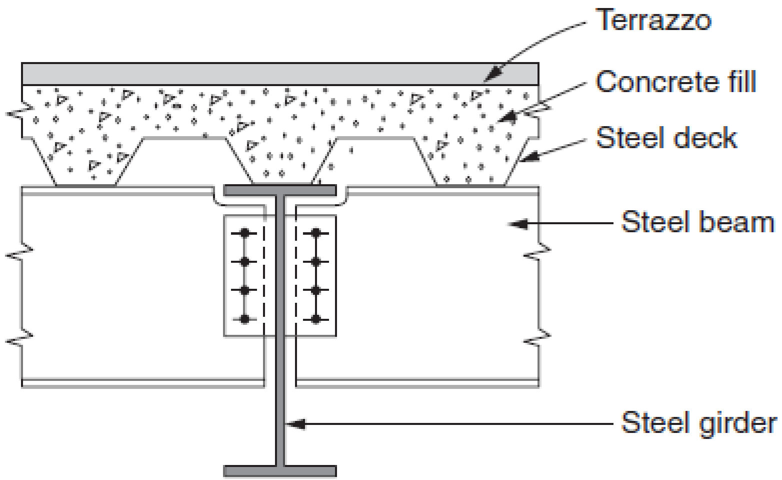
- Framed structures consist of floor and roof diaphragms, beams, girders, and columns
- The building may be one or several stories in height.

Types of Steel Buildings:

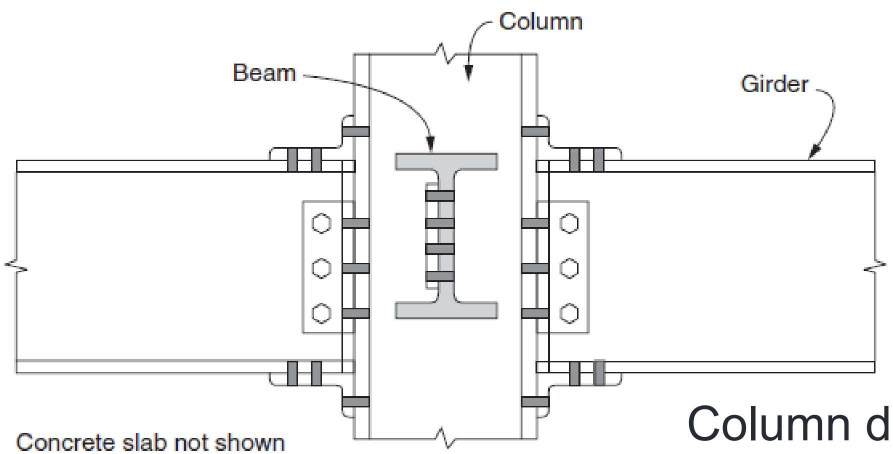
Framed Structures



Beam detail



Girder detail



Column detail

Lateral Load Resisting Systems in Steel Buildings

Lateral Load Resisting Systems in Steel Buildings

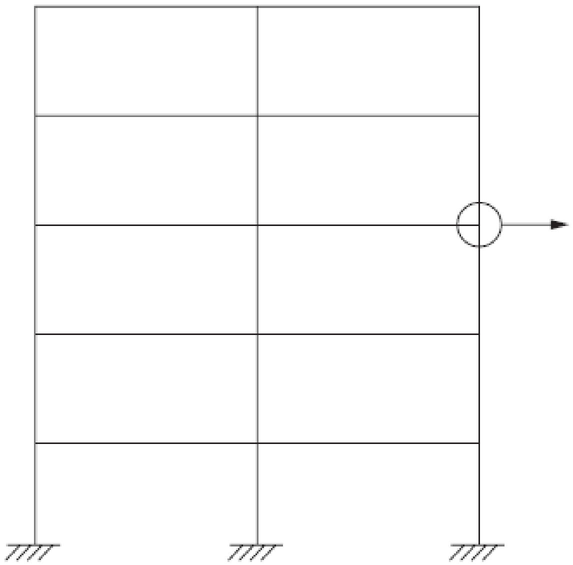
- According to previously shown design loads, framed structures must also be designed to resist lateral loads caused by wind or earthquake as well as gravity (vertical) loads.

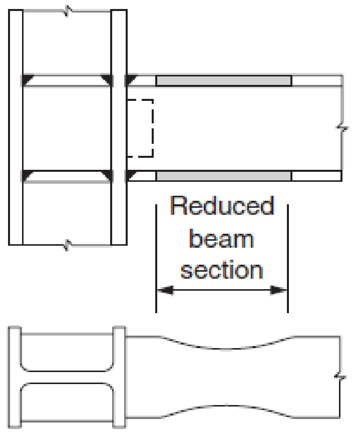
- Several techniques are used to provide lateral resistance including:
  - Moment-resisting frames
  - Braced frames
  - Shear walls

Lateral Load Resisting Systems in Steel Buildings

Moment-resisting frames

- Resist lateral loads by means of special flexural connections between the columns and beams.
- A number of different methods are used to provide the connections and these are specified in AISC, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC 358-10).
- Special detailing is required for finishes and curtain walls to accommodate, without damage, the large drifts anticipated.



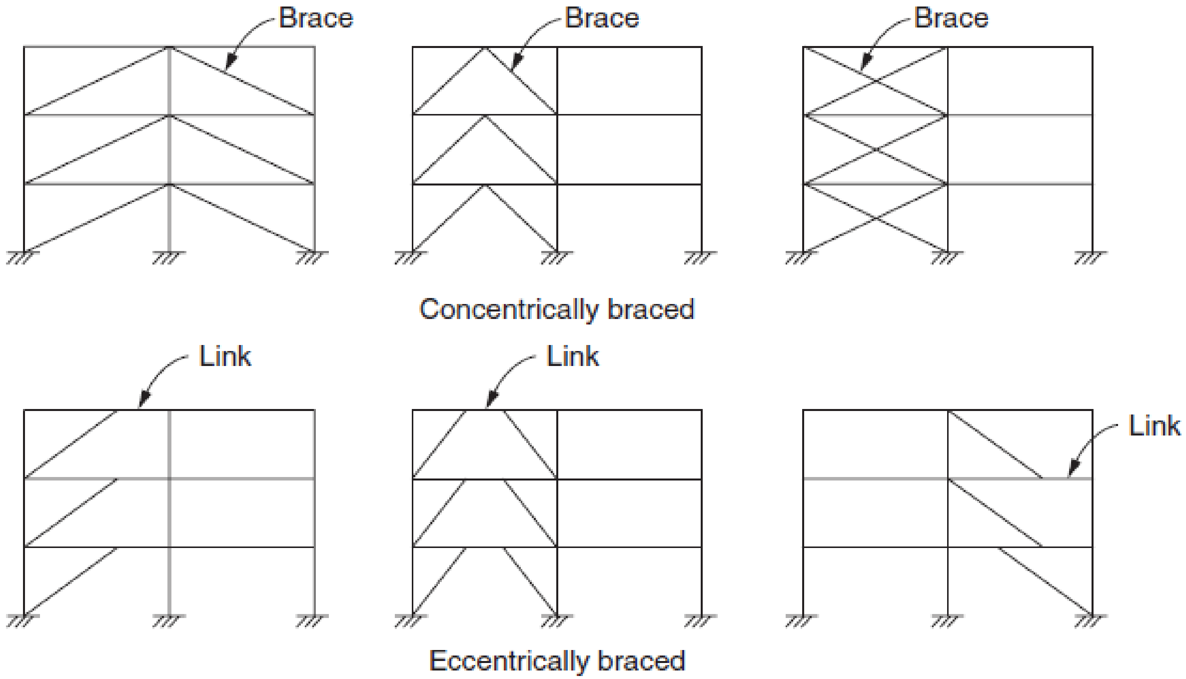


Moment-resisting frame

Lateral Load Resisting Systems in Steel Buildings

Braced frames

- These systems have the advantage over moment-resisting frames of less drift and simpler connections.
- In addition, braced frames are generally less expensive than moment-resisting frames.
- Their disadvantages are restrictions on maximum building height and architectural limitations.
- There are two commonly used braced frames; Concentrically braced frames, and eccentrically braced frames.



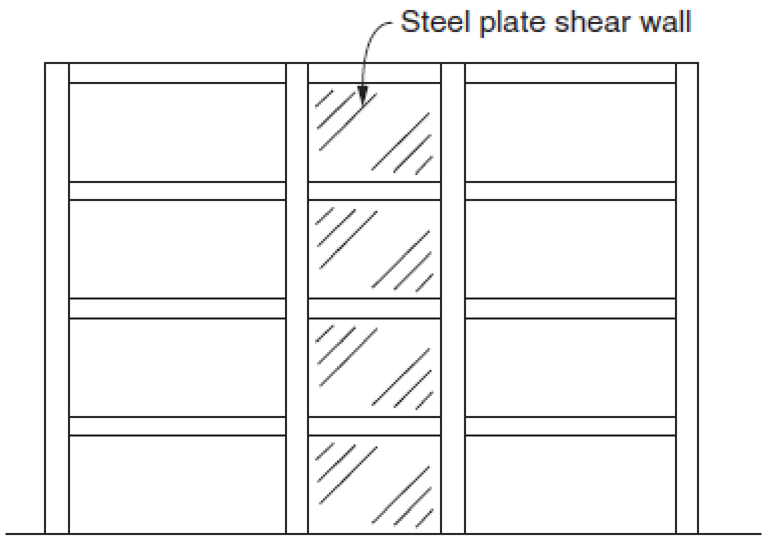
Braced Frames

Lateral Load Resisting Systems in Steel Buildings

Shear Wall Systems

- This system provides good drift control but lacks redundancy.

- Steel plate shear wall is commonly used as lateral force-resisting system.



Steel plate shear wall building

Design Loads

LOADS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Loads, Dead Loads, ASCE-7

TABLE 2.1 Typical Dead Loads for Some Common Building Materials

Reinforced concrete	150 lb/cu ft
Structural steel	490 lb/cu ft
Plain concrete	145 lb/cu ft
Movable steel partitions	4 psf
Plaster on concrete	5 psf
Suspended ceilings	2 psf
5-Ply felt and gravel	6 psf
Hardwood flooring (7/8 in)	4 psf
2 × 12 × 16 in double wood floors	7 psf
Wood studs with 1/2 in gypsum each side	8 psf
Clay brick wythes (4 in)	39 psf

Loads, Live Loads, ASCE-7

TABLE 2.2 Typical Minimum Uniform Live Loads for Design of Buildings

Type of building	LL (psf)
Apartment houses	
Apartments	40
Public rooms	100
Dining rooms and restaurants	100
Garages (passenger cars only)	40
Gymnasiums, main floors, and balconies	100
Office buildings	
Lobbies	100
Offices	50
Schools	
Classrooms	40
Corridors, first floor	100
Corridors above first floor	80
Storage warehouses	
Light	125
Heavy	250
Stores (retail)	
First floor	100
Other floors	75

Loads, Concentrated Live Loads, ASCE-7

TABLE 2.3 Typical Concentrated Live Loads for Buildings

Hospitals—operating rooms, private rooms, and wards	1000 lb
Manufacturing building (light)	2000 lb
Manufacturing building (heavy)	3000 lb
Office floors	2000 lb
Retail stores (first floors)	1000 lb
Retail stores (upper floors)	1000 lb
School classrooms	1000 lb
School corridors	1000 lb

- These loads are to be placed on floors or roofs at the positions where they will cause the most severe conditions.
- Unless otherwise specified, each of these concentrated loads is spread over an area 2.5 X 2.5 ft square (6.25 ft^2)

Loads, Wind Load

- In accordance with Bernoulli's theorem for ideal fluid striking an object, the increase in static pressure equals the decrease in dynamic pressure, or

$$q = \frac{1}{2}\rho V^2 \qquad (1)$$

- Where q is the dynamic pressure on the object, ρ is the mass density of air (specific weight w = 0.07651 pcf at sea level and 15^0 C), and V is the wind velocity.

Loads, Wind Load

- In terms of velocity V in miles per hour, the dynamic pressure q (psf) would be given by

$$q = \frac{1}{2}\rho V^2 = \frac{1}{2}\left(\frac{0.07651}{32.2}\right)\left(\frac{5280}{2600}\right) = 0.0026V^2 \qquad (2)$$

- In design of usual types of buildings, the dynamic pressure q is commonly converted into equivalent static pressure p, which may be expressed as

$$p = qC_e C_g C_p \qquad (3)$$

Loads, Wind Load

Where

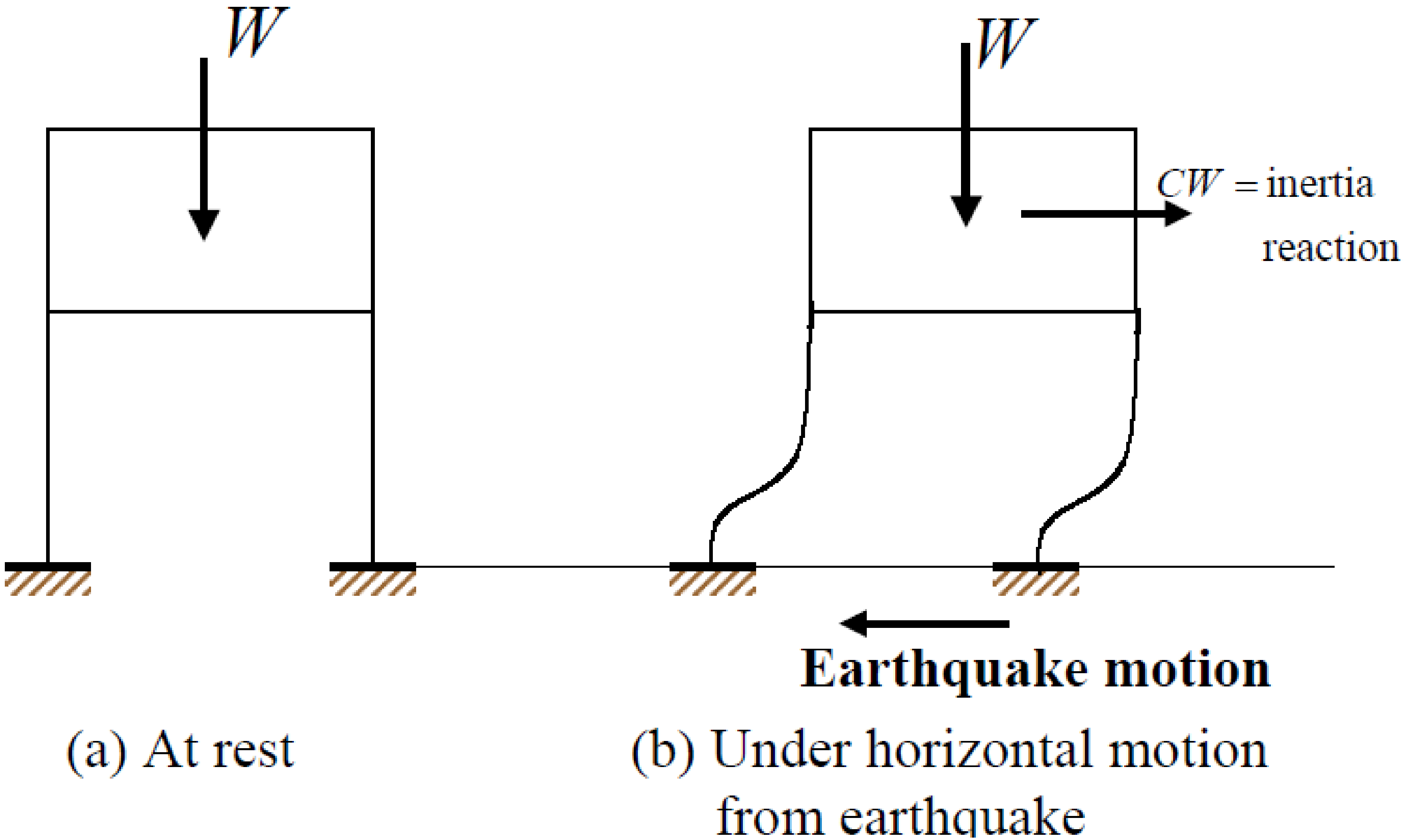
C_e = exposure factor that varies from 1.0 (for 0-40-ft height) to 2.0 (for 740-1200-ft height).

C_g = gust factor, such as 2.0 for structural members and 2.5 for small elements including cladding.

C_p = shape factor for the building as a whole.

- The commonly used wind pressure of 20 psf, as specified by many building codes, correspond to a velocity of 88 mph from Eq. 2.

Loads, Earthquake Load



Loads, Earthquake Load

- In the ANSI, the lateral seismic forces *V*, expressed as follows, are assumed to act non-concurrently in the direction of each of the main axes of the structure:

$$V = ZIKCSW \tag{4}$$

Z = seismic zone coefficient (varies from 1/8 to 1).
I = occupancy important factor (varies from 1.5 to 1.25).
K = horizontal force factor (varies from 0.67 to 2.5).
T = fundamental natural period.
S = soil profile coefficient (varies from 1.0 to 1.5).
W = total dead load of the building.

$$C = \frac{1}{15\sqrt{T}} \leq 0.12$$

Loads, Earthquake Load

- When the natural period *T* cannot be determined by rational means from technical data, it may be obtained as follows for shear walls or exterior concrete frames using deep beams or wide piers, or both:

$$T = \frac{0.05 h_n}{\sqrt{D}} \qquad (5)$$

D = dimension of the structure in the direction of the applied forces, in feet.

h_n = height of the building

PREVIEW OF AISC MANUAL, 13TH EDITION

Table 17–23
SI Conversion Factors[a]

Quantity	Multiply	by	to obtain	
Length	inch	25.400	millimeter	mm
	foot	0.305	meter	m
	yard	0.914	meter	m
	mile (U.S. Statute)	1.609	kilometer	km
	millimeter	39.370×10^{-3}	inch	in
	meter	3.281	foot	ft
	meter	1.094	yard	yd
	kilometer	0.621	mile	mi
Area	square inch	0.645×10^{3}	square millimeter	mm^2
	square foot	0.093	square meter	m^2
	square yard	0.836	square meter	m^2
	square mile (U.S. Statute)	2.590	square kilometer	km^2
	acre	4.047×10^{3}	square meter	m^2
	acre	0.405	hectare	
	square millimeter	1.550×10^{-3}	square inch	in^2
	square meter	10.764	square foot	ft^2
	square meter	1.196	square yard	yd^2
	square kilometer	0.386	square mile	mi^2
	square meter	0.247×10^{-3}	acre	
	hectare	2.471	acre	
Volume	cubic inch	16.387×10^{3}	cubic millimeter	mm^3
	cubic foot	28.317×10^{-3}	cubic meter	m^3
	cubic yard	0.765	cubic meter	m^3
	gallon (U.S. liquid)	3.785	liter	*l*
	quart (U.S. liquid)	0.946	liter	*l*
	cubic millimeter	61.024×10^{-6}	cubic inch	in^3
	cubic meter	35.315	cubic foot	ft^3
	cubic meter	1.308	cubic yard	yd^3
	liter	0.264	gallon (U.S. liquid)	gal
	liter	1.057	quart (U.S. liquid)	qt
Mass	ounce (avoirdupois)	28.350	gram	g
	pound (avoirdupois)	0.454	kilogram	kg
	short ton	0.907×10^{3}	kilogram	kg
	gram	35.274×10^{-3}	ounce (avoirdupois)	oz av
	kilogram	2.205	pound (avoirdupois)	lb av
	kilogram	1.102×10^{-3}	short ton	

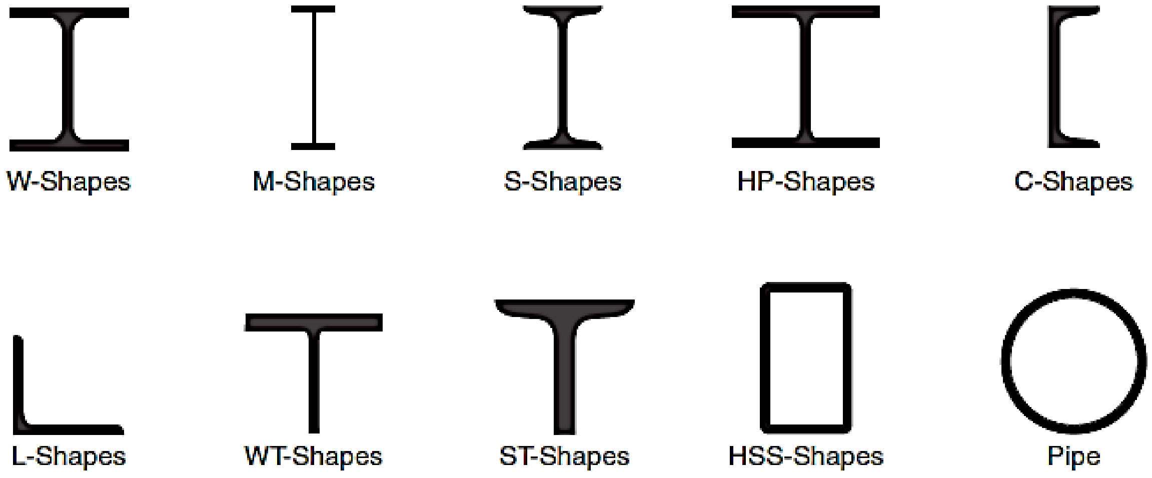
[a]Refer to ASTM E380 for more complete information on SI.
The conversion factors tabulated herein have been rounded.

Table 17–23 (continued)
SI Conversion Factors[a]

Quantity	Multiply	by	to obtain	
Force	[c]ounce-force	0.278	[c]newton	N
	[c]pound-force	4.448	[c]newton	N
	[c]newton	3.597	[c]ounce-force	
	[c]newton	0.225	[c]pound-force	lbf
Bending Moment	[c]pound-force-inch	0.113	[c]newton-meter	N-m
	[c]pound-force-foot	1.356	[c]newton-meter	N-m
	[c]newton-meter	8.851	[c]pound-force-inch	lbf-in
	[c]newton-meter	0.738	[c]pound-force-foot	lbf-ft
Pressure, Stress	[c]pound-force per square inch	6.895	[c]kilopascal	kPa
	[c]foot of water (39.2 F)	2.989	[c]kilopascal	kPa
	[c]inch of mercury (32 F)	3.386	[c]kilopascal	kPa
	[c]kilopascal	0.145	[c]pound-force per [c] square inch	lbf/in^2
	[c]kilopascal	0.335	[c]foot of water (39.2 F)	
	[c]kilopascal	0.295	[c]inch of mercury (32 F)	
Energy, Work, Heat	[c]foot-pound-force	1.356	[c]joule	J
	[b]British thermal unit	1.055×10^{3}	[c]joule	J
	[b]calorie	4.187	[c]joule	J
	[c]kilowatt hour	3.600×10^{6}	[c]joule	J
	[c]joule	0.738	[c]foot-pound-force	ft-lbf
	[c]joule	0.948×10^{-3}	[b]British thermal unit	Btu
	[c]joule	0.239	[b]calorie	
	[c]joule	0.278×10^{-6}	[c]kilowatt hour	kW-h
Power	[c]foot-pound-force/second	1.356	[c]watt	W
	[b]British thermal unit per hour	0.293	[c]watt	W
	[c]horsepower (550 ft lbf/s)	0.746	[c]kilowatt	kW
	[c]watt	0.738	[c]foot-pound-force/ [c] second	ft-lbf/s
	[c]watt	3.412	[b]British thermal unit [c] per hour	Btu/h
	kilowatt	1.341	[c]horsepower [c] (550 ft-lbf/s)	hp
Angle	[c]degree	17.453×10^{-3}	[c]radian	rad
	[c]radian	57.296	[c]degree	
Temperature	[c]degree Fahrenheit	t°C = (t°F – 32)/1.8	[c]degree Celsius	
	[c]degree Celsius	t°F = 1.8 × t°C + 32	[c]degree Fahrenheit	

[a]Refer to ASTM E380 for more complete information on SI.
[b]International Table.
[c]The conversion factors tabulated herein have been rounded.

STEEL SECTIONS



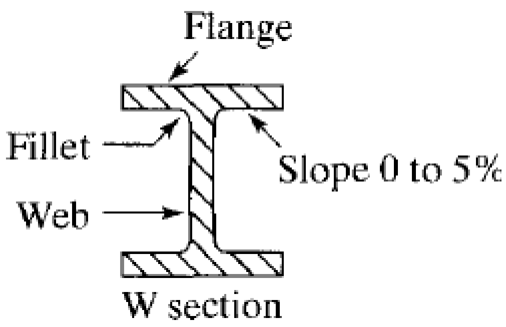
Shape	Designation
Wide flanged beams	W
Miscellaneous beams	M
Standard beams	S
Bearing piles	HP
Standard channels	C
Miscellaneous channels	MC
Angles	L
Tees cut from W-shapes	WT
Tees cut from M-shapes	MT
Tees cut from S-shapes	ST
Rectangular hollow structural sections	HSS
Square hollow structural sections	HSS
Round hollow structural sections	HSS
Pipe	Pipe

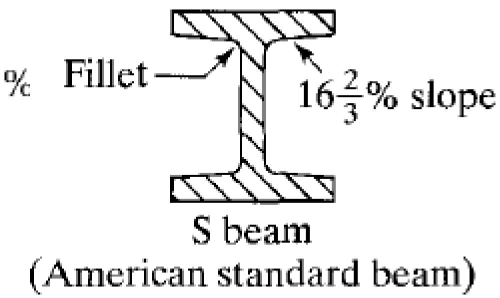
	Steel Type		
Shape	ASTM Designation	F_y, ksi	F_u, ksi
Wide flanged beams	A992	50–65	65
Miscellaneous beams	A36	36	58–80
Standard beams	A36	36	58–80
Bearing piles	A572 Gr. 50	50	65
Standard channels	A36	36	58–80
Miscellaneous channels	A36	36	58–80
Angles	A36	36	58–80
Ts cut from W-shapes	A992	50–65	65
Ts cut from M-shapes	A36	36	58–80
Ts cut from S-shapes	A36	36	58–80
Hollow structural sections, rectangular	A500 Gr. B	46	58
Hollow structural sections, square	A500 Gr. B	46	58
Hollow structural sections, round	A500 Gr. B	42	58
Pipe	A53 Gr. B	35	60

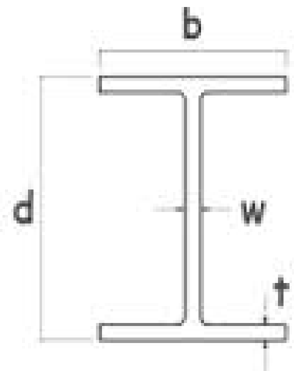
STEEL SECTIONS

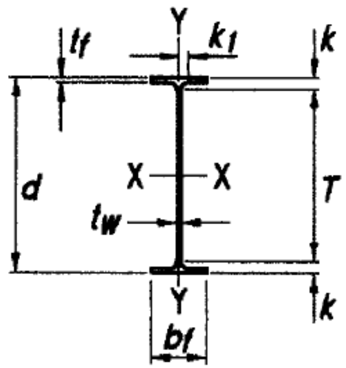
- Structural steel can be economically rolled into a wide variety of shapes and sizes without appreciably changing its physical properties.

- The desirable members are those with large moments of inertia in proportion to their areas.









W Section
(Wide Flange Section)
Straight Flange

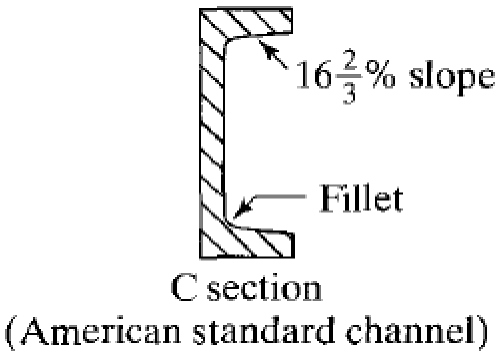
S Section
(Tapered Flange)

HP Section
(Bearing Pile Section)
Similar to W section but with thicker web

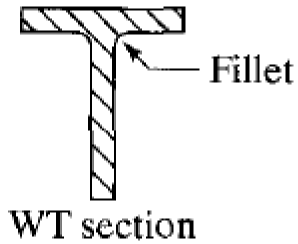
M Section
H-Shaped but not W, S or HP

I-Sections (H- Sections)

STEEL SECTIONS

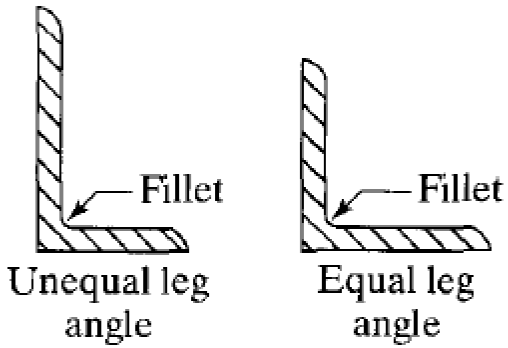


- C-Section
- MC-Section



Tee-Sections

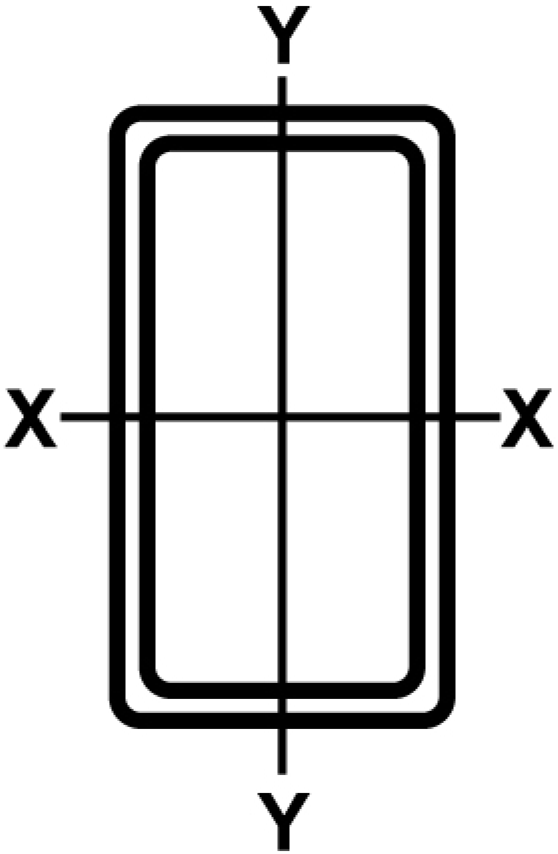
- WT-Section: Made from W-Sections
- ST-Section: Made from S-Sections
- MT-Sections: Made from M-Sections



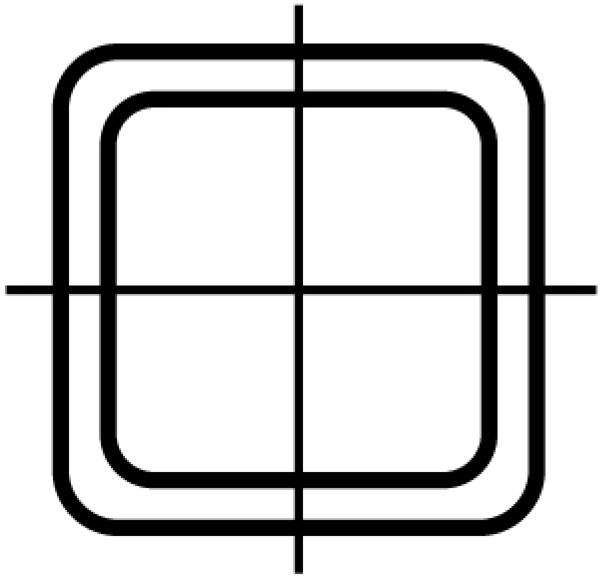
Angles

STEEL SECTIONS

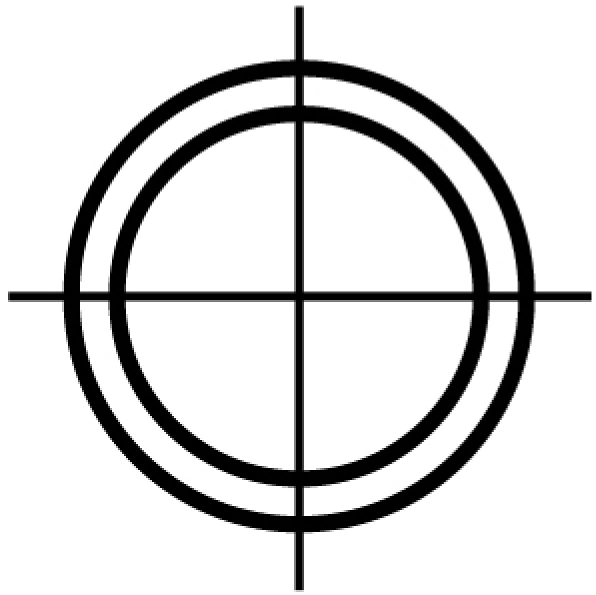
HSS-Sections (Hollow Structural Steel Sections



Rectangular HSS-Section

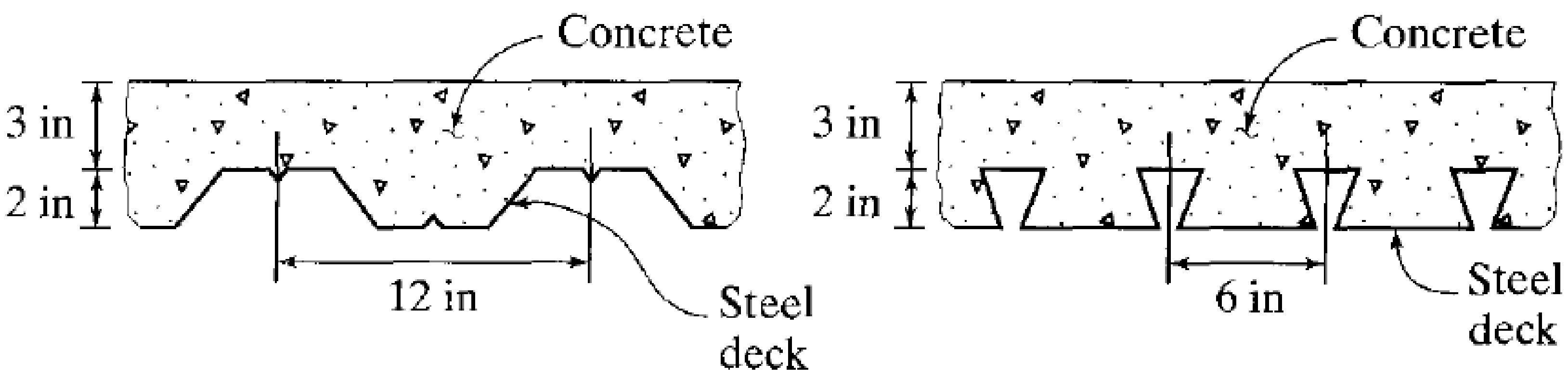


Square HSS-Section

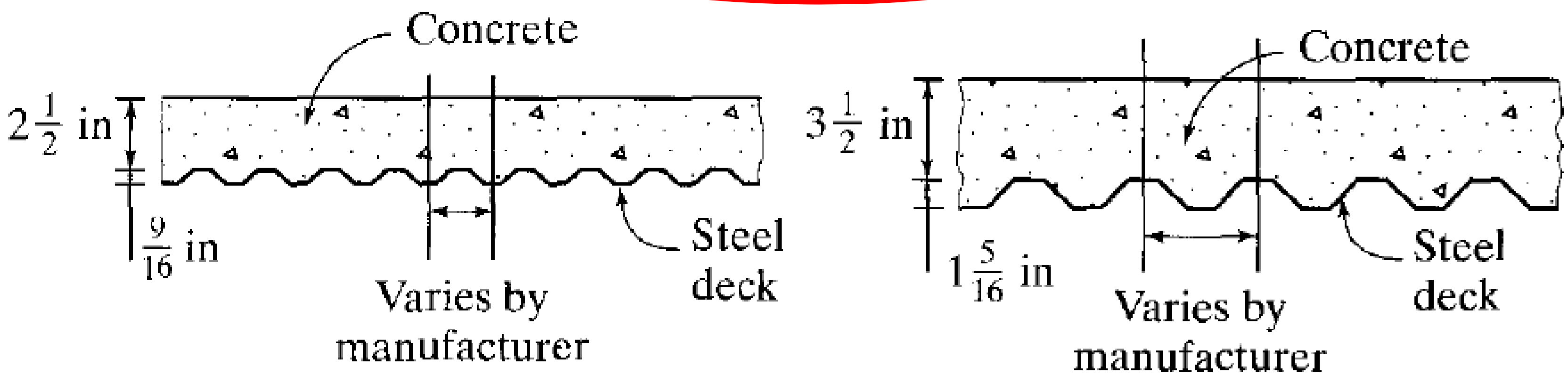


Circular (Round) HSS-Section

Composite deck floor system



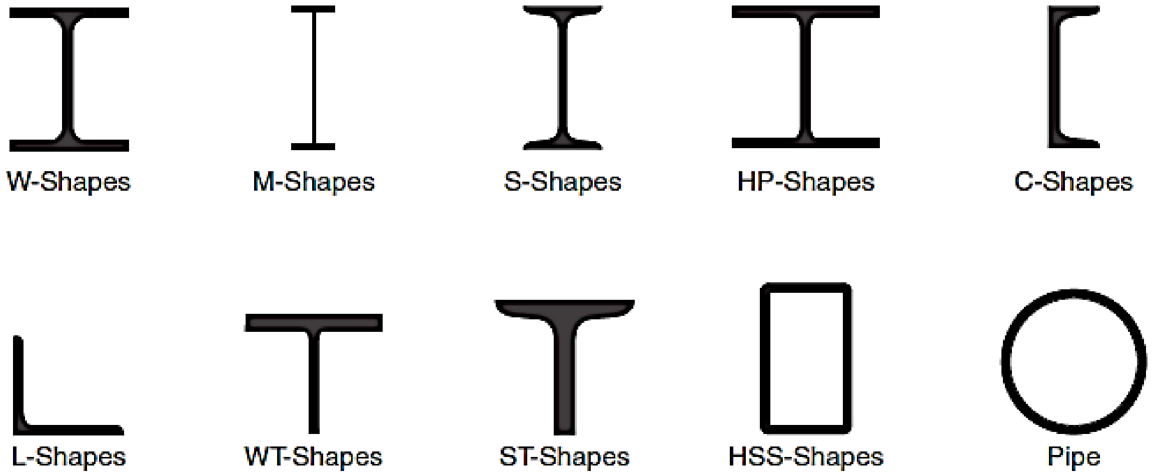
Form deck floor system



Roof deck system

- ✓ Formed steel decks serve as economical Forms.
- ✓ Sections with the deeper cells have the useful feature that electrical and mechanical conduits can be placed in them.

STEEL SECTIONS



Shape	Designation
Wide flanged beams	W
Miscellaneous beams	M
Standard beams	S
Bearing piles	HP
Standard channels	C
Miscellaneous channels	MC
Angles	L
Tees cut from W-shapes	WT
Tees cut from M-shapes	MT
Tees cut from S-shapes	ST
Rectangular hollow structural sections	HSS
Square hollow structural sections	HSS
Round hollow structural sections	HSS
Pipe	Pipe

	Steel Type		
Shape	**ASTM Designation**	**F_y, ksi**	**F_u, ksi**
Wide flanged beams	A992	50–65	65
Miscellaneous beams	A36	36	58–80
Standard beams	A36	36	58–80
Bearing piles	A572 Gr. 50	50	65
Standard channels	A36	36	58–80
Miscellaneous channels	A36	36	58–80
Angles	A36	36	58–80
Ts cut from W-shapes	A992	50–65	65
Ts cut from M-shapes	A36	36	58–80
Ts cut from S-shapes	A36	36	58–80
Hollow structural sections, rectangular	A500 Gr. B	46	58
Hollow structural sections, square	A500 Gr. B	46	58
Hollow structural sections, round	A500 Gr. B	42	58
Pipe	A53 Gr. B	35	60

SECTIONS Identification in AISC Manual

- **W27 X 114** is a W section approximately 27 in deep, weighing 114lb/ft.
- **S12 X 35** is an S section 12 in deep, weighing 35 lb/ft.
- **HP12 X 74** is a bearing pile section approximately 12 in deep, weighing 74lb/ft
- **M8 X 6.5** is a miscellaneous section 8 in deep, weighing 6.5 lb/ft.
- **C10 X 30** is a channel 10 in deep, weighing 30 lb/ft.
- **MC18 X 58** is a miscellaneous channel 18 in deep, weighing 58 lb/ft.

- ❑ **HSS14 X 10 X 5/8** is a rectangular hollow structural section 14 in deep, 10 in wide, with a 5/8-in wall thickness.
- ❑ **L6 X 6 X 1/2** is an equal leg angle, each leg being 6 in long and 1/2 in thick.
- ❑ **WT18 X 151** is a tee obtained by splitting a W36 x 302.

Structural Steel

The properties of steel can be greatly changed by:

- ❖ varying the quantities of carbon present
- ❖ by adding other elements such as:

  silicon

  nickel

  manganese

  copper

Steel that has a significant amount of these elements is referred as **alloy steel**

STRUCTURAL CARBON STEEL

Carbon Steel

The contents of carbon steel are limited to maximum percentages:

- ✓ 1.7 percent carbon
- ✓ 1.65 percent manganese
- ✓ 0.60 percent silicon
- ✓ 0.60 percent copper

Steel can be divided into four categories based on carbon content percentages:

1. **Low-carbon steel**: < 0.15 percent.
2. **Mild steel**: 0.15 to 0.29 percent. (***The structural carbon steels fall into this category.)***
3. **Medium-carbon steel**: 0.30 to 0.59 percent.
4. **High-carbon steel**: 0.60 to 1.70 percent

Grades of Structural Steel

A36 Steel → Fy= 36 ksi (Old Product)

A50 Steel → Fy= 50 ksi (in use today)

Note:
Angles are still produced with A36 steel

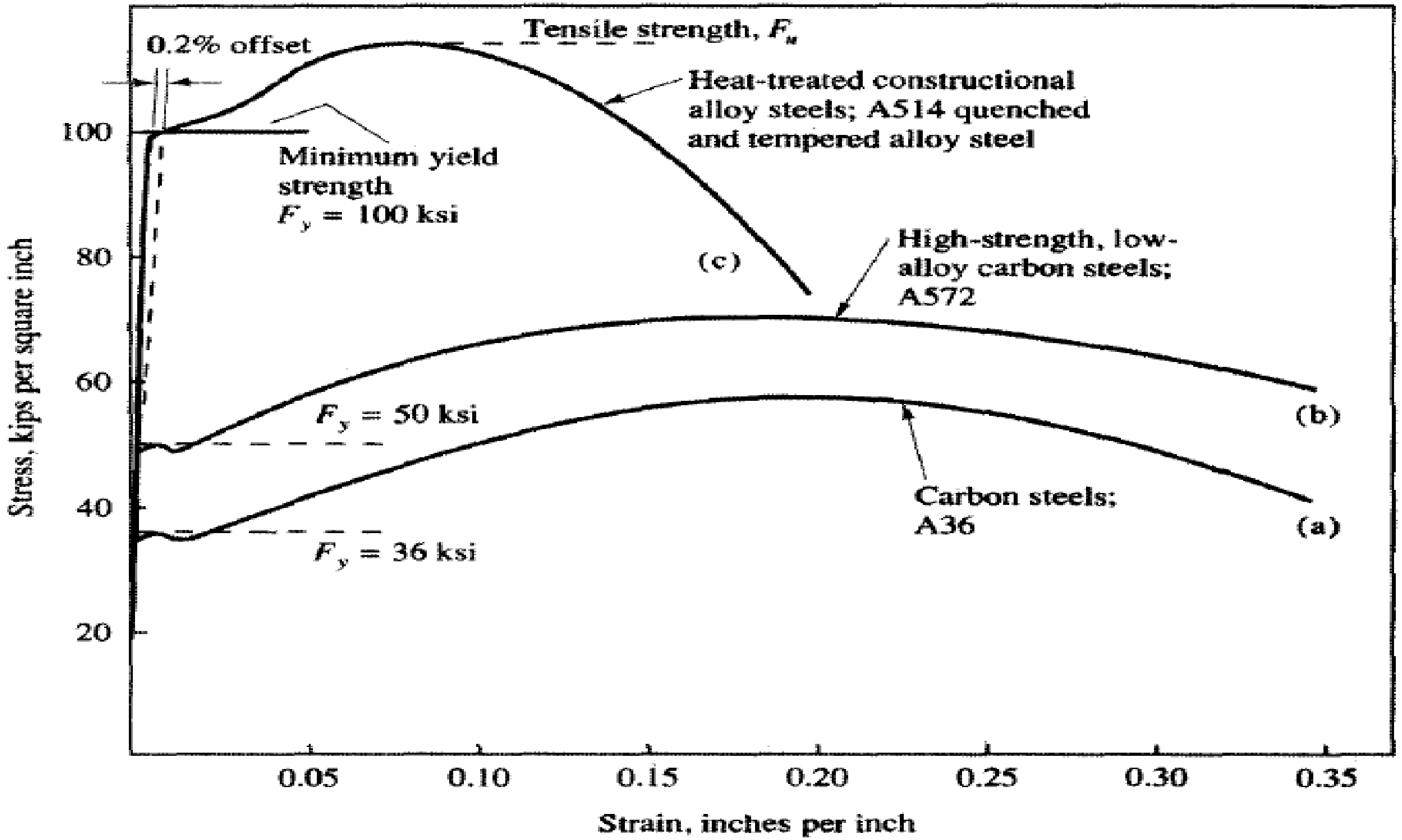


Table 2–3
Applicable ASTM Specifications for Various Structural Shapes

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.

[a] Minimum unless a range is shown.
[b] For shapes over 426 lb/ft, only the minimum of 58 ksi applies.
[c] For shapes with a flange thickness less than or equal to 1½ in. only. To improve weldability a maximum carbon equivalent can be specified (per ASTM Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM Supplementary Requirement S79).
[d] If desired, maximum tensile stress of 70 ksi can be specified (per ASTM Supplementary Requirement S91).
[e] For shapes with a flange thickness less than or equal to 2 in. only.
[f] ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.
[g] Minimum applies for walls nominally ¾-in. thick and under. For wall thicknesses over ¾ in., F_y = 46 ksi and F_u = 67 ksi.
[h] If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM Supplementary Requirement S75).
[i] A maximum yield-to-tensile strength ratio of 0.85 and carbon equivalent formula are included as mandatory in ASTM A992.
[j] For shapes with a flange thickness greater than 2 in. only.
[k] For shapes with a flange thickness greater than 1½ in. and less than or equal to 2 in. only.
[l] For shapes with a flange thickness less than or equal to 1½ in. only.

Availability of Structural Steel

Steel Type	ASTM Designation	F_y, ksi	Plate Thickness, in
Carbon	A36	36	≤ 8
		32	>8
	A529	42	$\leq \frac{1}{2}$
High-strength low-alloy	A441	50	$\leq 1\frac{1}{2}$
		46	$\frac{3}{4}-1\frac{1}{2}$
		42	$1\frac{1}{2}-4$
		40	4–8
	A572—Grade 65	65	$\leq 1\frac{1}{4}$
	—Grade 60	60	$\leq 1\frac{1}{4}$
	—Grade 50	50	≤ 4
	—Grade 42	42	≤ 6
Corrosion-resistant high-strength low-alloy	A242	50	$\leq \frac{3}{4}$
		46	$\frac{3}{4}-1\frac{1}{2}$
		42	$1\frac{1}{2}-4$
	A588	50	≤ 4
		46	4–5
		42	5–8
Quenched and tempered alloy	A514	100	$\leq 2\frac{1}{2}$
		90	$2\frac{1}{2}-6$

Methods of Design, Limit State Method

- ✓**Limit states** design principles provide the boundaries of structural usefulness.

- ✓The term **limit state** is used to describe a condition at which a structure or part of a structure ceases to perform its intended function.

Methods of Design, Limit State Method

There are two categories of limit states:

1. **<u>Limit State of Strength</u>**

Define:

- ✓ Load-carrying capacity, including excessive yielding.
- ✓ Fracture
- ✓ Buckling
- ✓ Fatigue
- ✓ Gross rigid body motion.

All limit states must be prevented

Methods of Design, Limit State Method

There are two categories of limit states:

1. Limit State of Serviceability

Define performance, including:

- ✓ Deflection
- ✓ Cracking
- ✓ Slipping
- ✓ Vibration
- ✓ Deterioration

All limit states must be prevented

Methods of Design

1. Load and Resistance Factor Design **(LRFD)**

2. Allowable Strength Design **(ASD)**

Methods of Design, LRFD vs. ASD

1. Design Loads Combinations (Factored Loads in LRFD vs. Working (service) loads in ASD).

2. Strength Reduction Factor (Φ) in LRFD vs. safety factor (Ω) in ASD.

Note:

The relationship between the safety factor (Φ) and the resistance factor (Ω) is:

$$\Omega = 1.5 / \Phi$$

Load Combinations, LRFD

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

Load Combinations, ASD

1. D
2. $D + L$
3. $D + (L_r \text{ or } S \text{ or } R)$
4. $D + 0.75L + 0.75\,(L_r \text{ or } S \text{ or } R)$
5. $D + (0.6W \text{ or } 0.7\,E)$
6. (a) $D + 0.75L + 0.75(0.6W) + 0.75\,(L_r \text{ or } S \text{ or } R)$

   (b) $D + 0.75L + 0.75(0.7E) + 0.75\,(S)$
7. $0.6D + 0.6W$
8. $0.6D + 0.7E$

Load Combinations, Notations

U = the design (ultimate) load

D = dead load

F = fluid load

T = self straining force

L = live load

L_r = roof live load

H = lateral earth pressure load, ground water pressure.

S = snow load

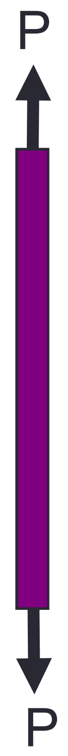
R = rain load

W = wind load

E = earthquake load

THANK YOU

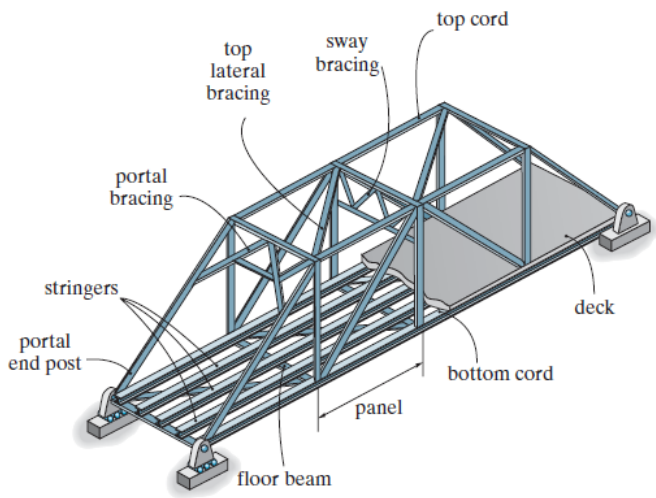
Analysis of Tension Members



Tension Member: structural member subjected to axial tensile forces.

Tension Members are found in different types of structures:

- ✓ Truss Members (especially bottom chords)
- ✓ Bracing Systems for buildings and bridges (especially with X-Configurations)
- ✓ Cables in suspended roof system
- ✓ Cables in suspension bridges and cable-stayed bridges



Steps to design tension members

In tension member there is **no danger of the member buckling**, Hence the designer needs to:

1. Determine only the load to be supported from structural analysis.

2. Then the area required to support that load is calculated based on strength of material.

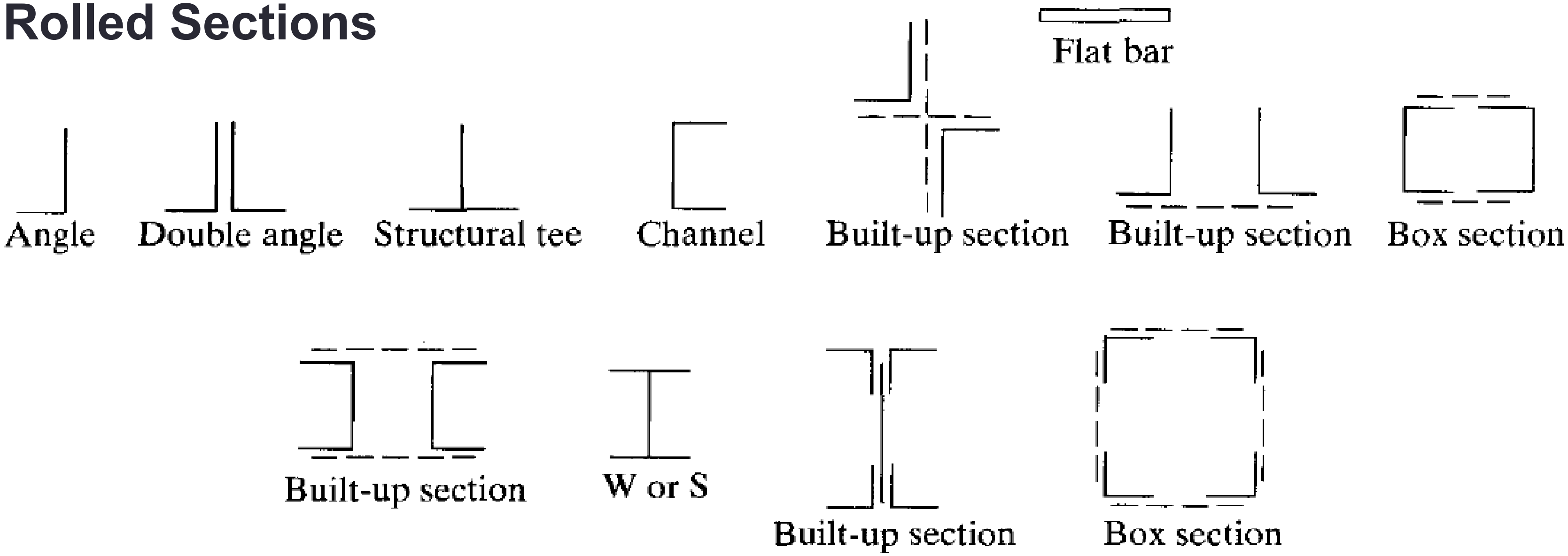
3. Finally a steel section is selected that provides the required area.

Sections used for tension members

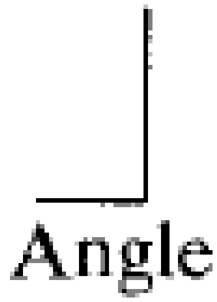
1. Circular Rod (Round Bar)

- Simplest form of tension members
- Used in past occasional use today
- Problems
  - ❖ difficulty in connecting it to many structures
  - ❖ Bad reputation (improper use in the past)
  - ❖ Little bending stiffness
  - ❖ Difficult prefabrication, installation and proper connection.

2. Rolled Sections



- ✓ Single angles and double angles are probably the most common types of tension members in use.



- ✓ A more satisfactory member is made from two angles placed back to back.

- ✓ Sufficient space between them to permit the insertion of plates (called gusset plates) for connection purposes.

- ✓ Where steel sections are used back-to-back in this manner, they should be connected to each other every 4 or 5 ft to prevent rattling, particularly in bridge trusses.

Built-Up Sections

- Built-up sections used when the designer is unable to obtain sufficient area or rigidity from single shapes.

- Members consisting of more than one section need to be tied together using tie plates or gusset plates .

- Tie plates located at various intervals or perforated cover plates serve to hold the various pieces in their correct positions.

- These plates correct any unequal distribution of loads between the various parts. They also keep the slenderness ratios of the individual parts within limitations.

- None of the intermittent tie plates may be considered to increase the effective cross-sectional areas of the sections.

- As they do not theoretically carry portions of the force in the main sections, **their sizes are usually governed by specifications** and perhaps by some judgment on the designer's part.

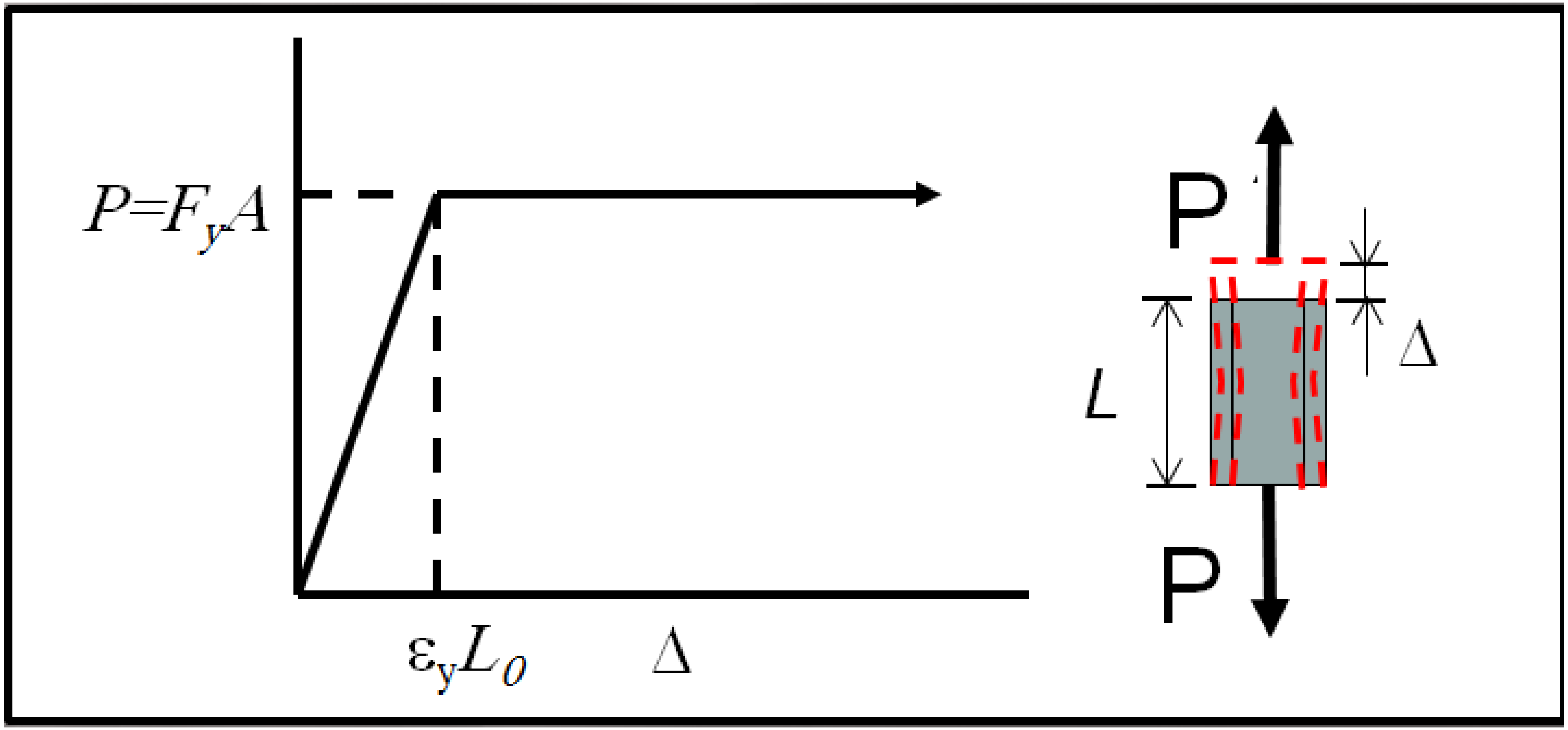
Tensile Strength

Limit States of Tensile Strength :

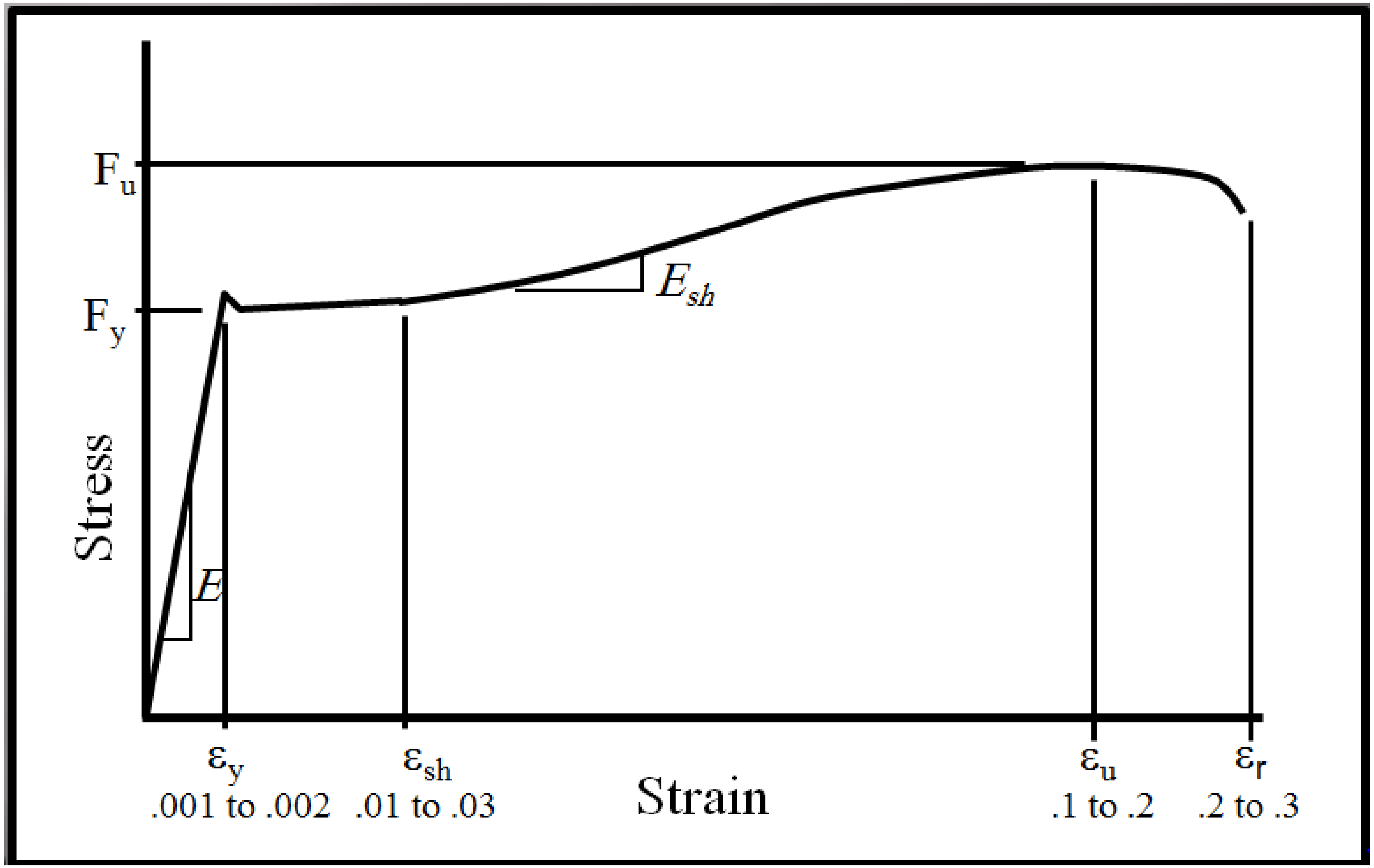
1. Yielding on Gross Area
2. Rupture on Net Area
3. Block Shear
4. Bearing or Tear-out at Bolts

Yielding on Gross Area

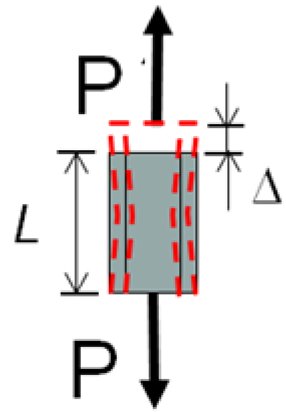
When a member is loaded the strength is limited by the yielding of the entire cross section.

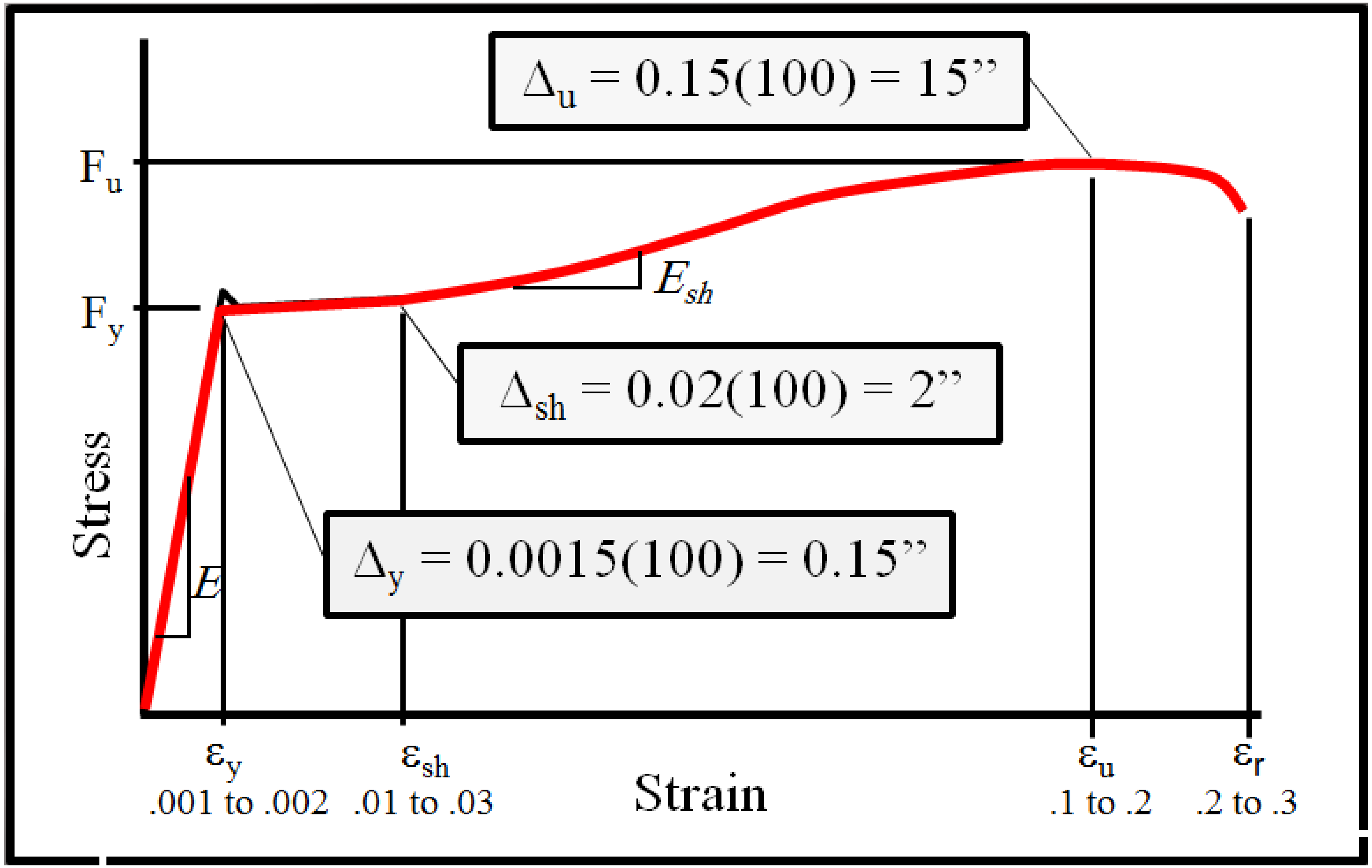


How this is affected by the stress-strain conditions?



Ans. : Consider L=100 inch long tension member.





Δ_{Yield}	= approx. 0.00172(100)	= 0.172"
$\Delta_{Onset\ of\ Strain\ Hardening}$	= approx. 0.02(100)	= 2"
$\Delta_{Peak\ Load}$	= approx. 0.15(100)	= 15"

Excessive deformations defines "Failure" for tension member yielding. Limit to $F_{y*}A_g$.

AISC Manual- Tension Members

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

D2. TENSILE STRENGTH

The *design tensile strength*, $\phi_t P_n$, and the *allowable tensile strength*, P_n/Ω_t, of tension members, shall be the lower value obtained according to the *limit states* of *tensile yielding* in the gross section and *tensile rupture* in the net section.

(a) For tensile yielding in the gross section:

$$P_n = F_y A_g \qquad \text{(D2-1)}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

A_g = Gross Area (Total cross-sectional area in the plane perpendicular to tensile stresses. (Part 1)

Tensile strength for Rupture on Effective Net Area

(b) For tensile rupture in the net section:

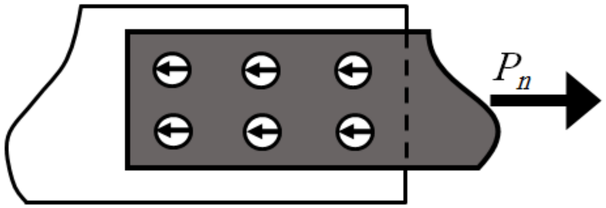
$$P_n = F_u A_e \tag{D2-2}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

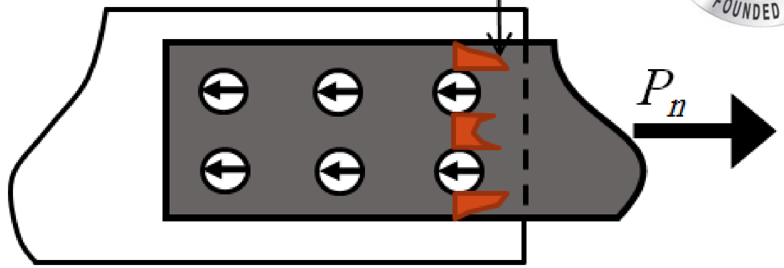
where

A_e = *effective net area*, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
F_u = *specified minimum tensile strength* of the type of steel being used, ksi (MPa)

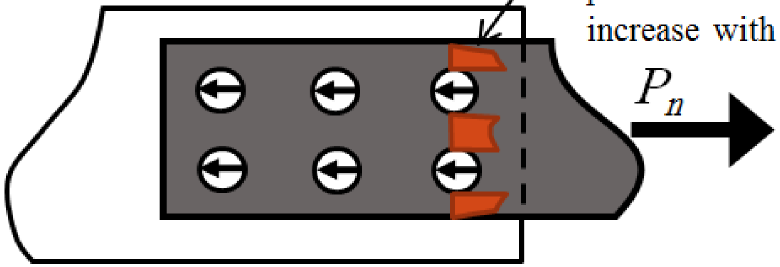
Tensile Rupture in Effective Net Area, A_e

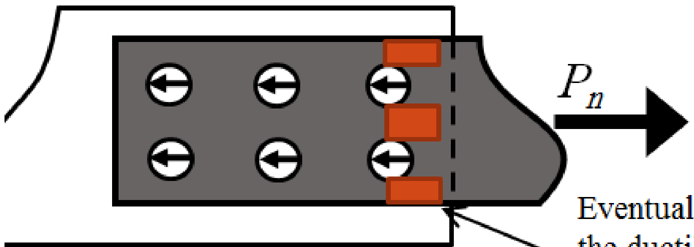


Initial stresses will typically include stress concentrations due to higher strains at these locations.

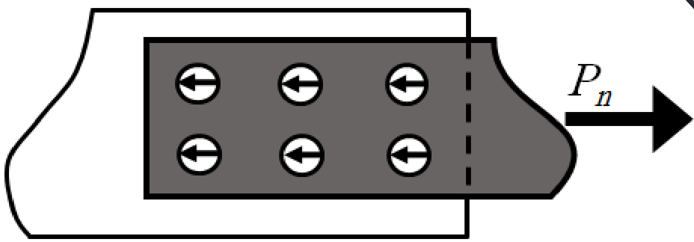


Highest strain locations yield, then elongate along plastic plateau while adjacent stresses increase with additional strain.

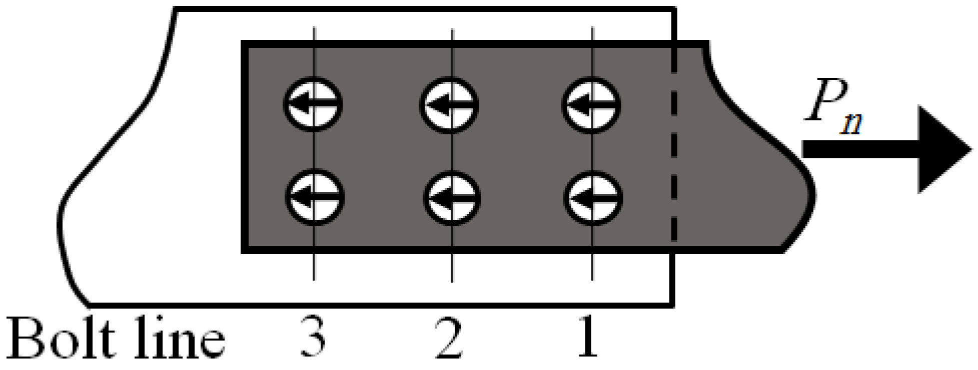




Eventually at very high strains the ductility of steel results in full yielding of the cross section.

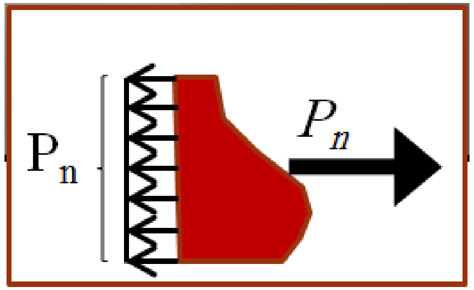


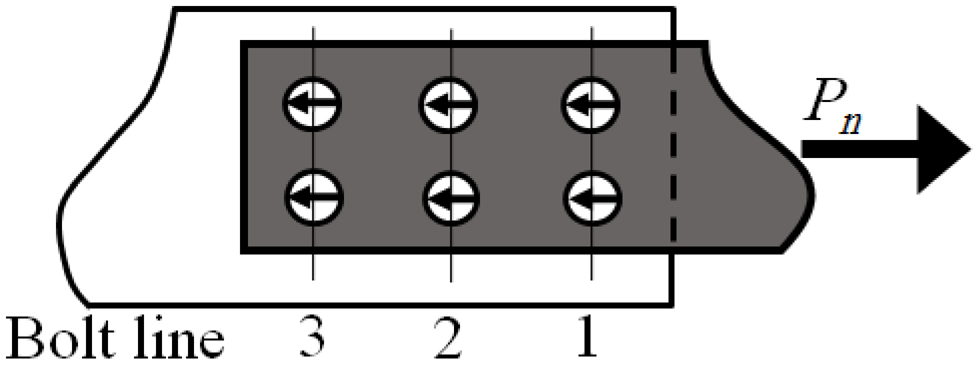
Therefore average stresses are typically used in design.

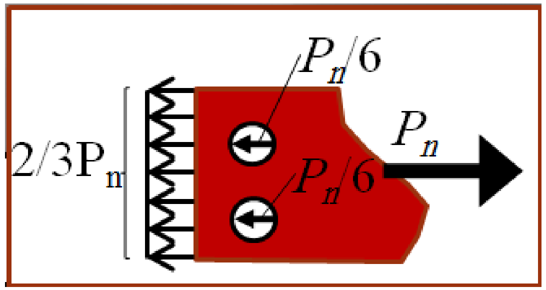


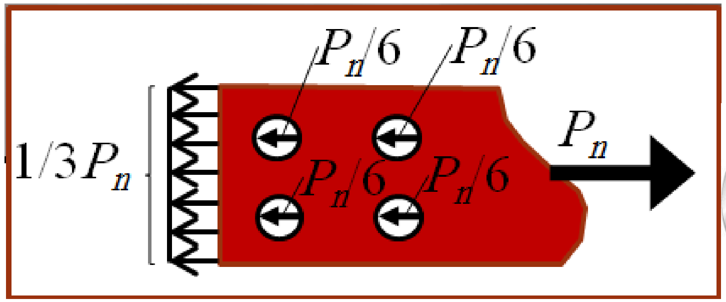
- The plate will fail in the line with the highest force (for similar number of bolts in each line).

- Each bolt line shown transfers 1/3 of the total force.









- Bolt line 1 resists P_n in the plate.
- Bolt line 2 resists $2/3P_n$ in the plate.
- Bolt line 3 resists $1/3P_n$ in the plate.

Net cross-sectional area (Net area):

Gross cross-sectional area of a member, minus any holes.

$$A_n = A_g - A_h$$

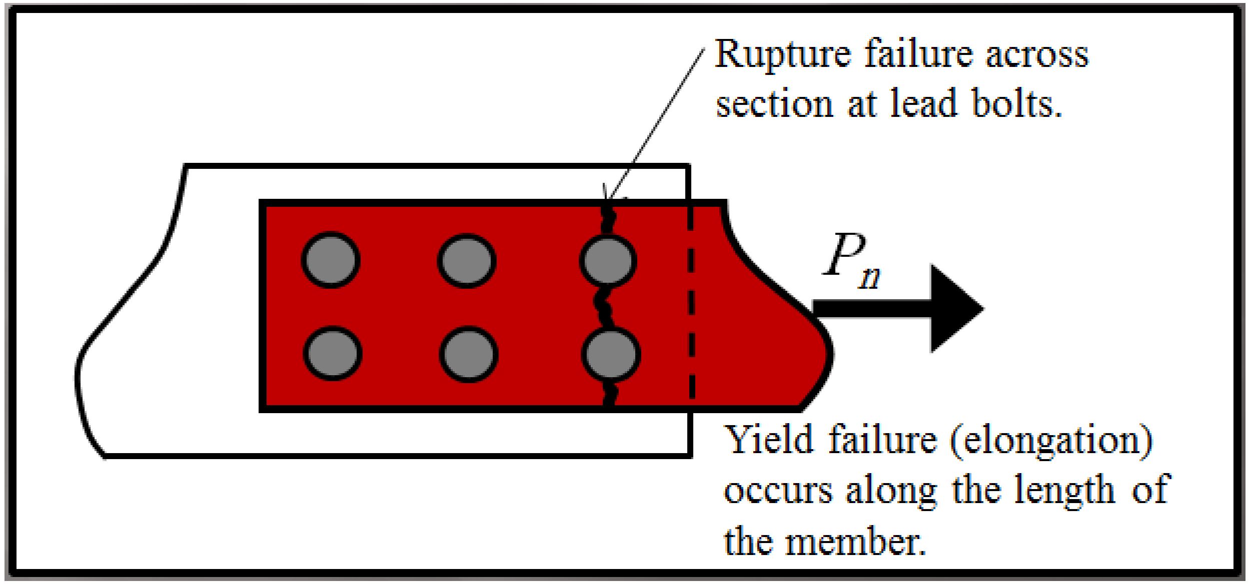
Assumptions:

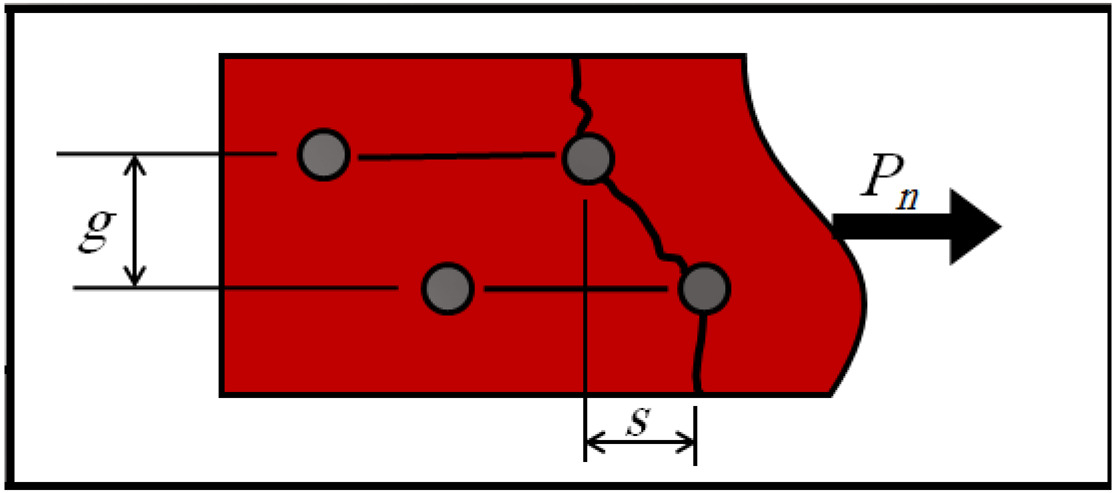
1. bolts and surrounding material will yield prior to rupture due to the inherent ductility of steel.
2. assume each bolt transfers equal force

Diameter of Hole= Bolt dia.+(1/16 inch (damage due to punching) + 1/16 inch (larger punch))

Diameter of Hole= Diameter of bolt+1/8 inch

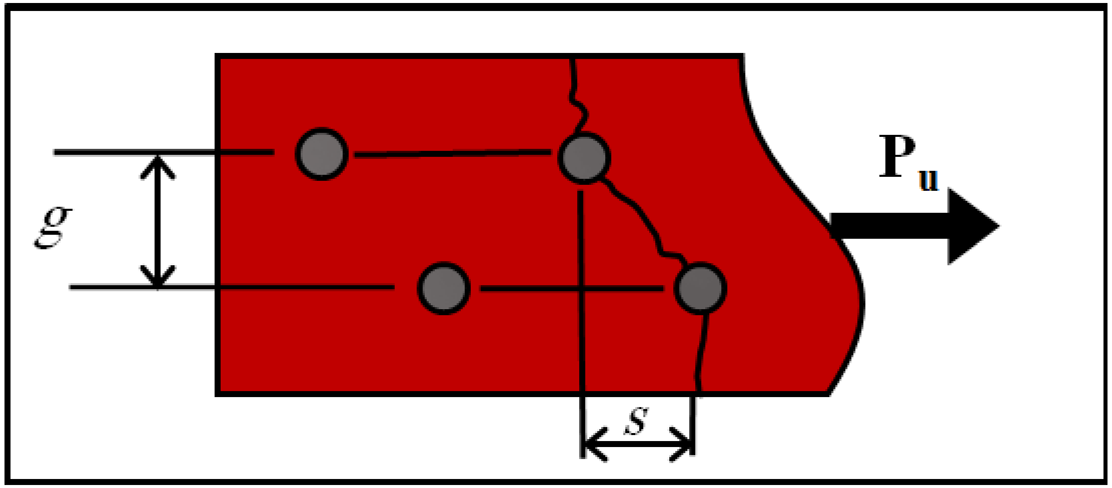
A_n = Net Area = Net Width x thickness





Need to include additional length/area of failure plane due to non-perpendicular path.

Diagonal hole pattern:



Net Width = Gross Width + $\Sigma s^2/4g$ – width of all holes

Section B4.3b and D3.2

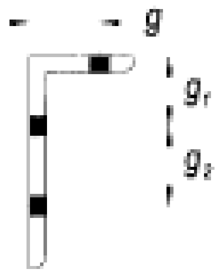
s = longitudinal center-to-center spacing of holes (pitch)
g = transverse center-to-center spacing between fastener lines (gage)

Width of holes= diameter of bolt+ 1/8”

Note:
Standard hole size used for every bolt size is given in Table J3.3.

When considering angles:

Find gage (g) on page 1-46 of Manual, “Workable Gages in Standard Angles” unless otherwise noted.



	Workable Gages in Angle Legs, in.													
Leg	**8**	**7**	**6**	**5**	**4**	**$3\frac{1}{2}$**	**3**	**$2\frac{1}{2}$**	**2**	**$1\frac{3}{4}$**	**$1\frac{1}{2}$**	**$1\frac{3}{8}$**	**$1\frac{1}{4}$**	**1**
g	$4\frac{1}{2}$	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{3}{8}$	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$
g_1	3	$2\frac{1}{2}$	$2\frac{1}{4}$	2										
g_2	3	3	$2\frac{1}{2}$	$1\frac{3}{4}$										

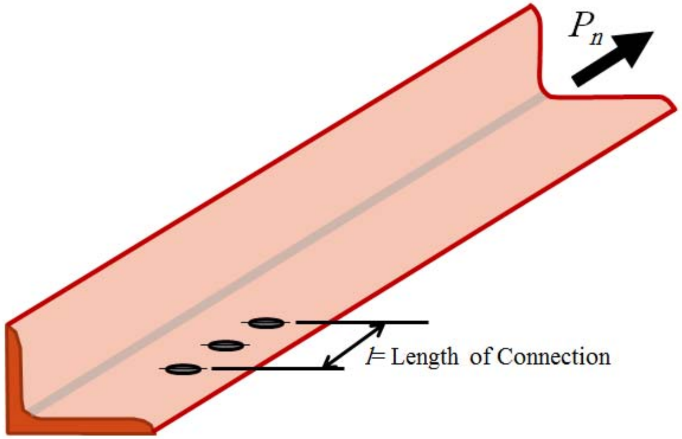
Note: Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations

$$A_n = Ag - \sum(d_b+1/8)t + \sum(s^2/(4g))t$$

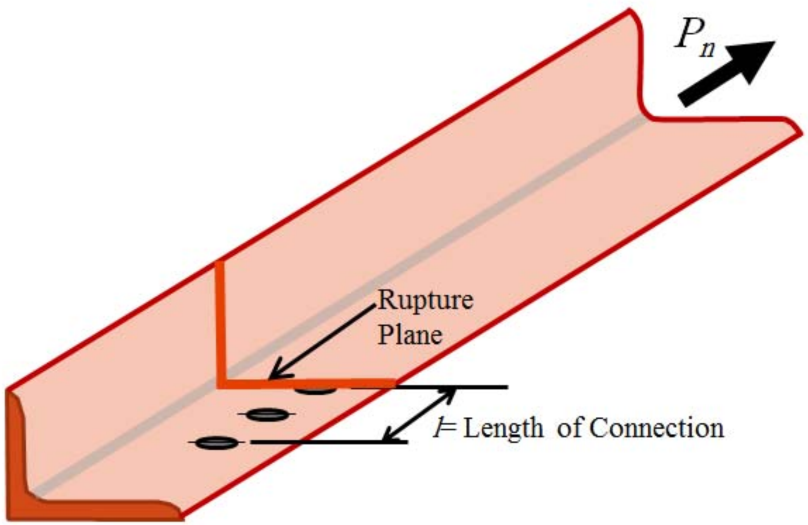
Shear Lag

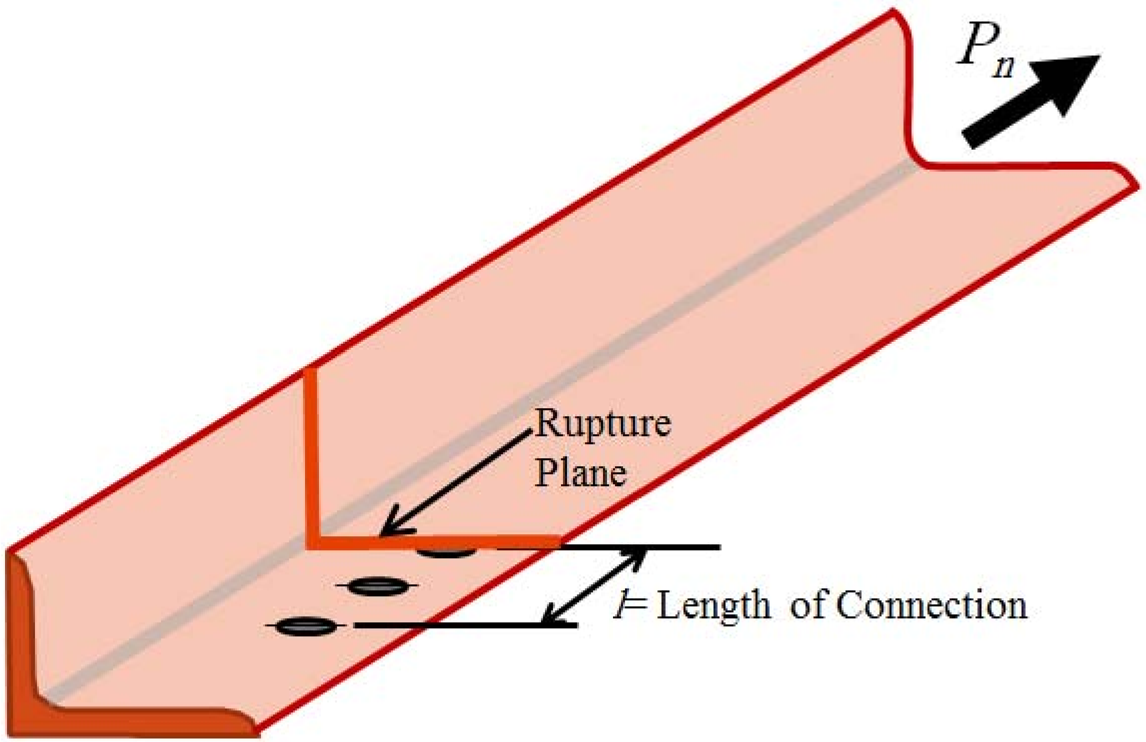
Shear Lag affects members where:

1. Only a portion of the cross section is connected
2. Connection does not have sufficient length.

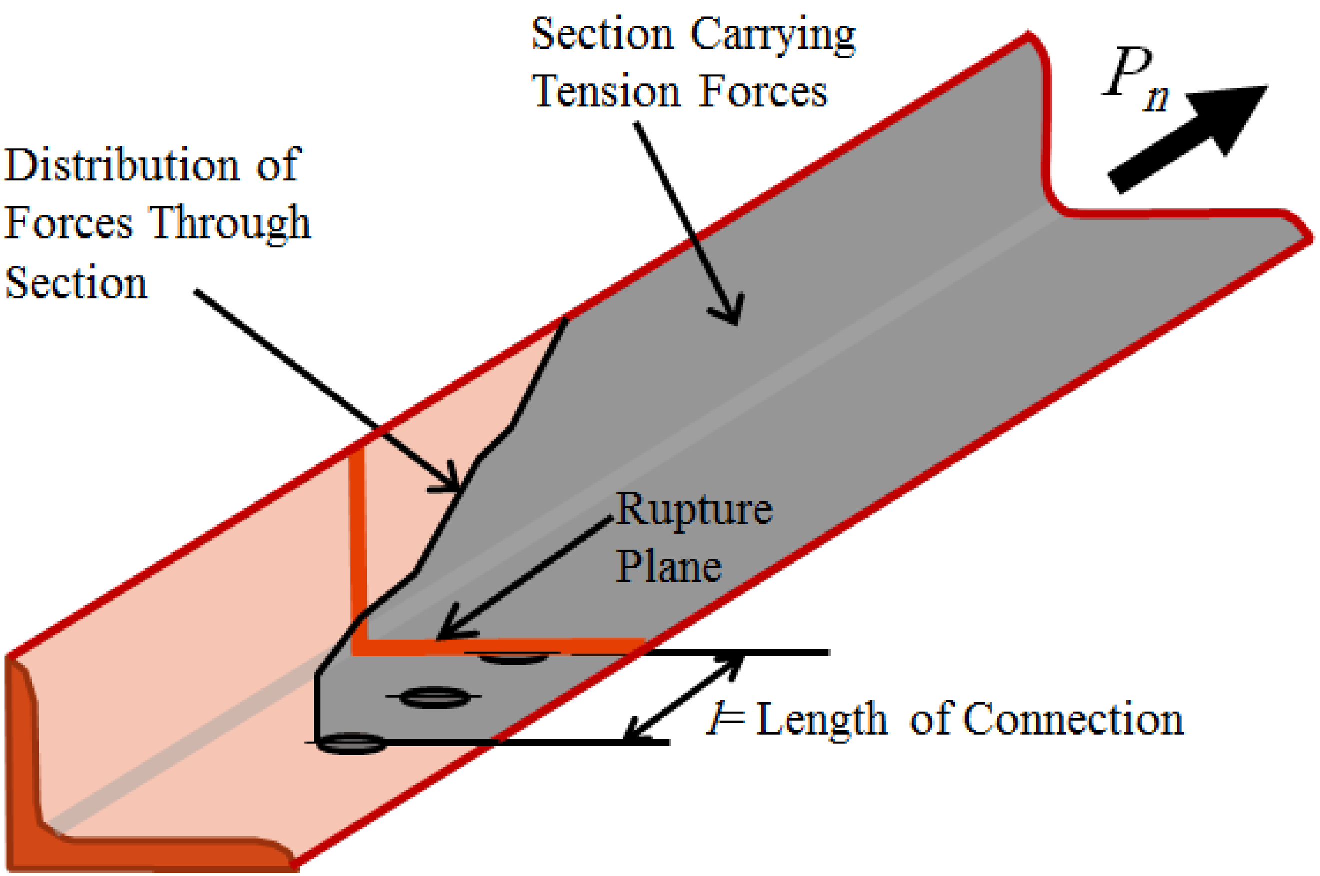


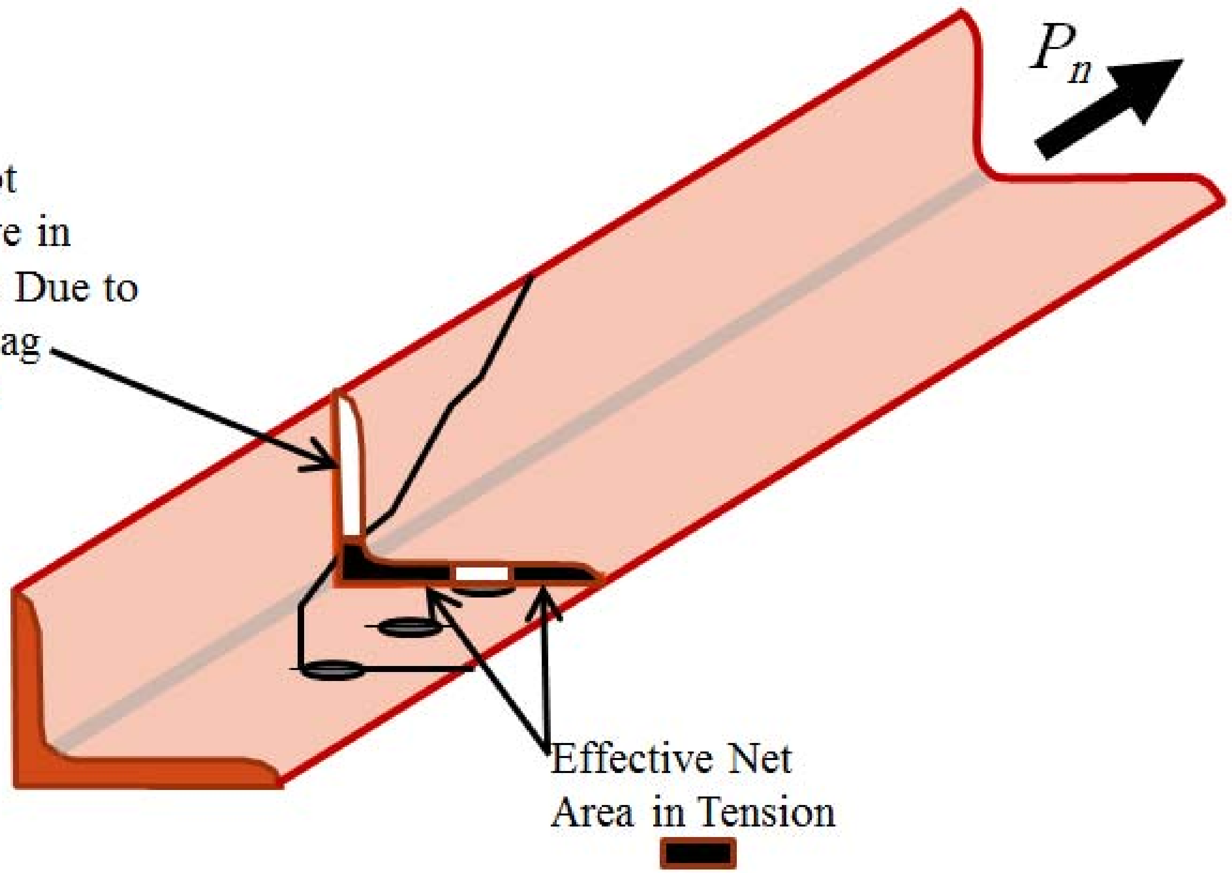
A_e= Effective Net Area
A_n= Net Area
$A_e \neq A_n$ Due to Shear Lag





P_n
Rupture
Plane
l= Length of Connection


Section Carrying
Tension Forces
P_n
Distribution of
Forces Through
Section
Rupture
Plane
l= Length of Connection



P_n
Area not Effective in Tension Due to Shear Lag
Effective Net Area in Tension

Effective Net Area, A_e

Modify net area (An) to account for shear lag.

3. Effective Net Area

The effective area of tension members shall be determined as follows:

$$A_e = A_n U \qquad \text{(D3-1)}$$

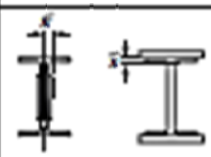
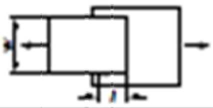
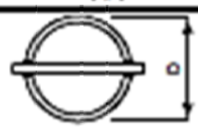
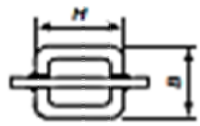
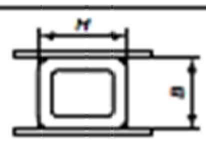
where U, the shear lag factor, is determined as shown in Table D3.1.

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

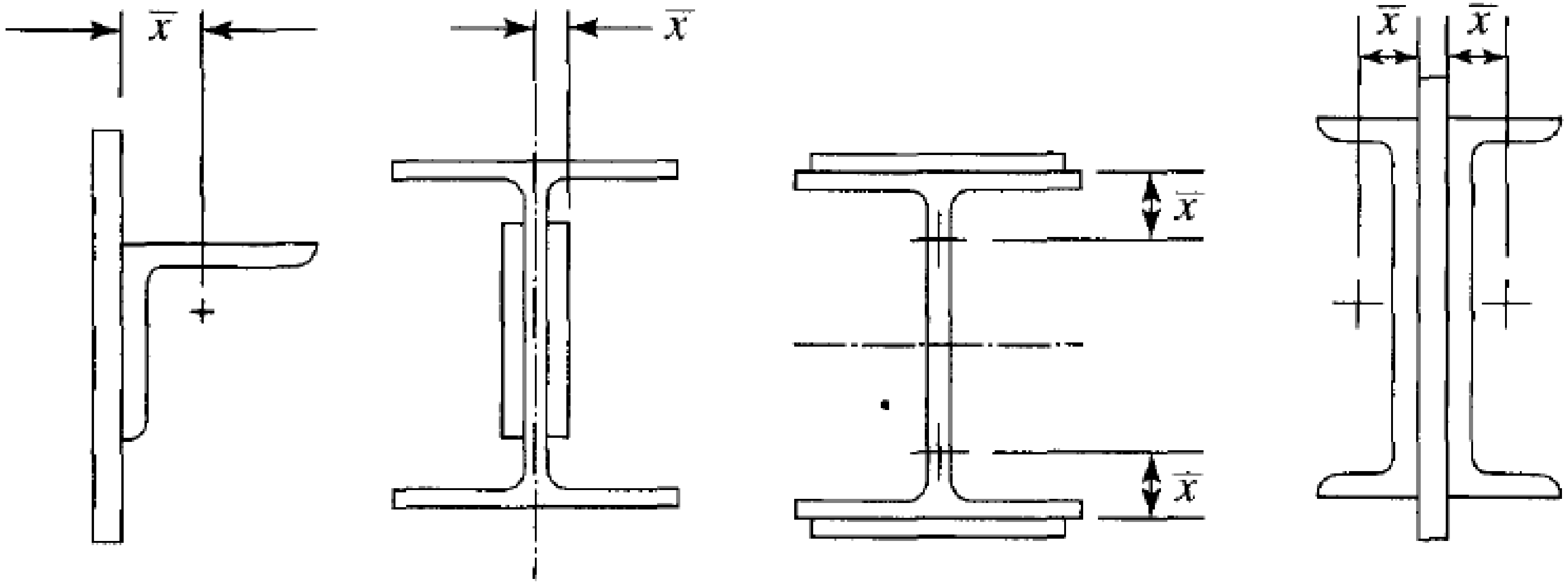
where

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

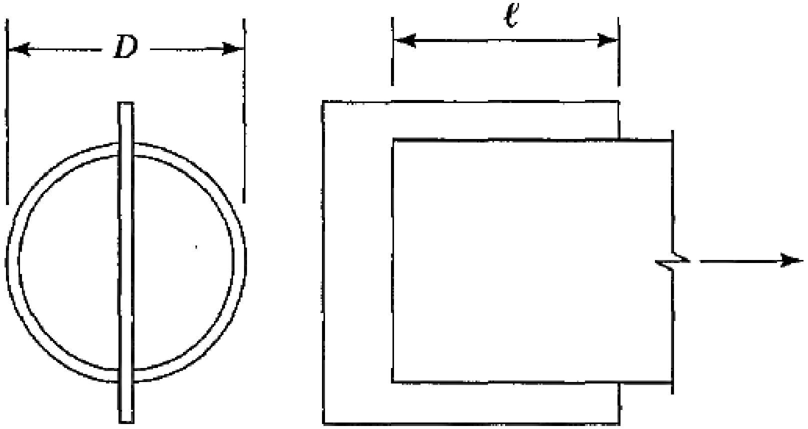
TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds. (except as in Cases 3, 4, 5 and 6)		$U = 1.0$	——
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 or longitudinal welds (Alternatively, for W, M, S and HP, Case 7 may be used.)		$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	——
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	—
		with web connected with 4 or more fasteners in the direction of loading	$U = 0.70$	—
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	—
		with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	—

l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = connection eccentricity, in. (mm); B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)



(a) (b) (c) (d)

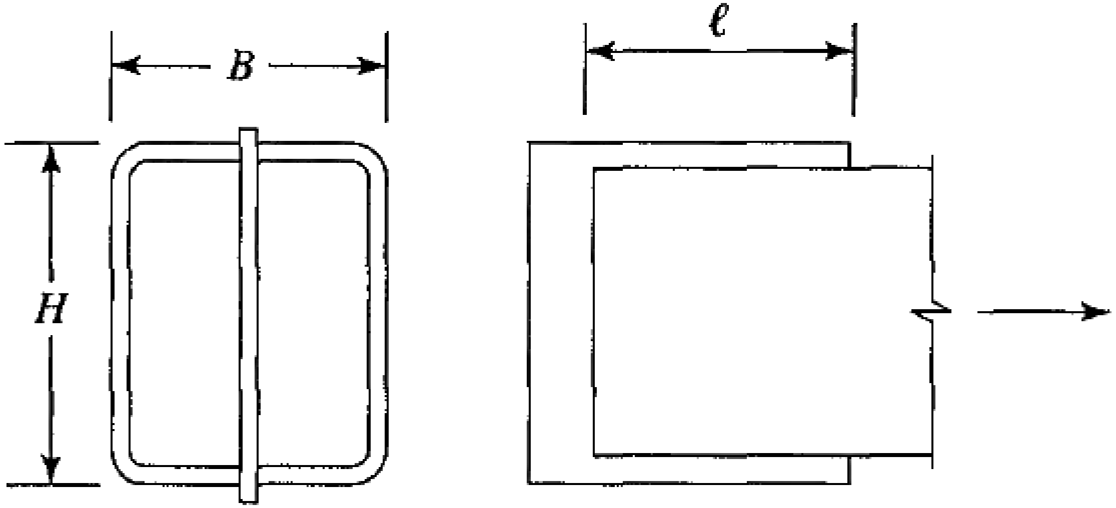


(e)

$$\ell \geq 1.3D: \quad U = 1.0$$

$$D \leq \ell < 1.3D: \quad U = 1 - \frac{\bar{x}}{\ell}$$

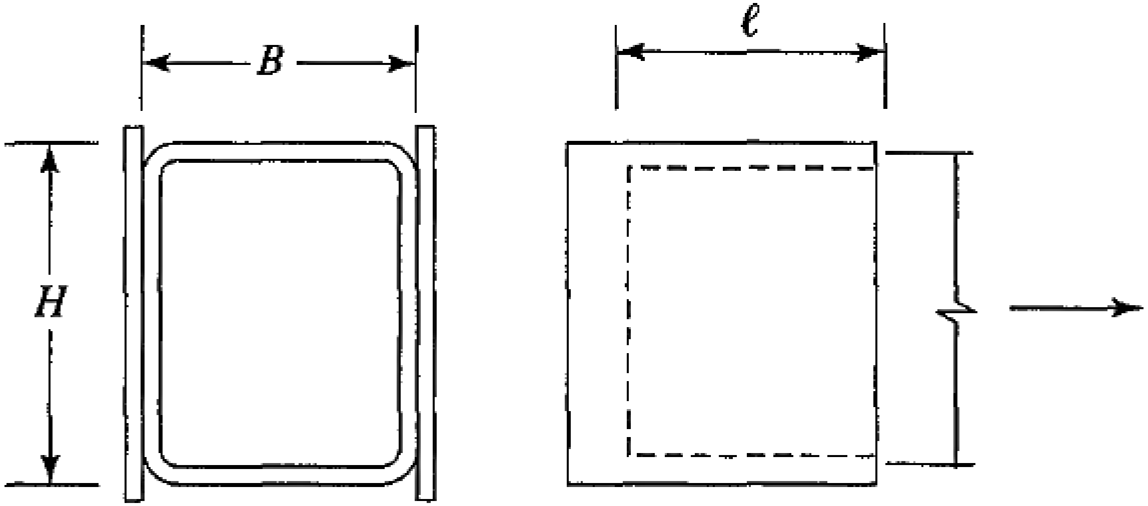
$$\bar{x} = \frac{D}{\pi}$$



$$\ell \geq H: \quad U = 1 - \frac{\bar{x}}{\ell}$$

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$$

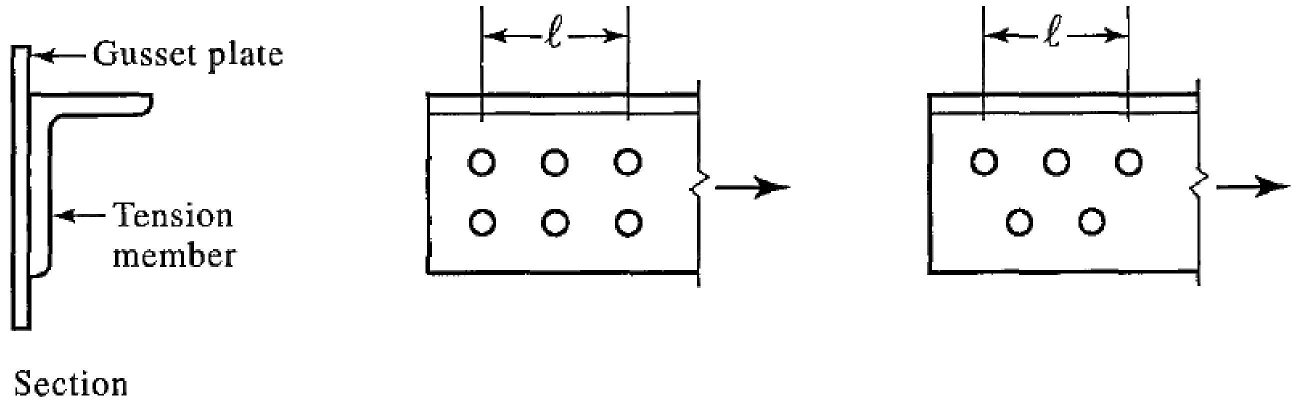
(f)



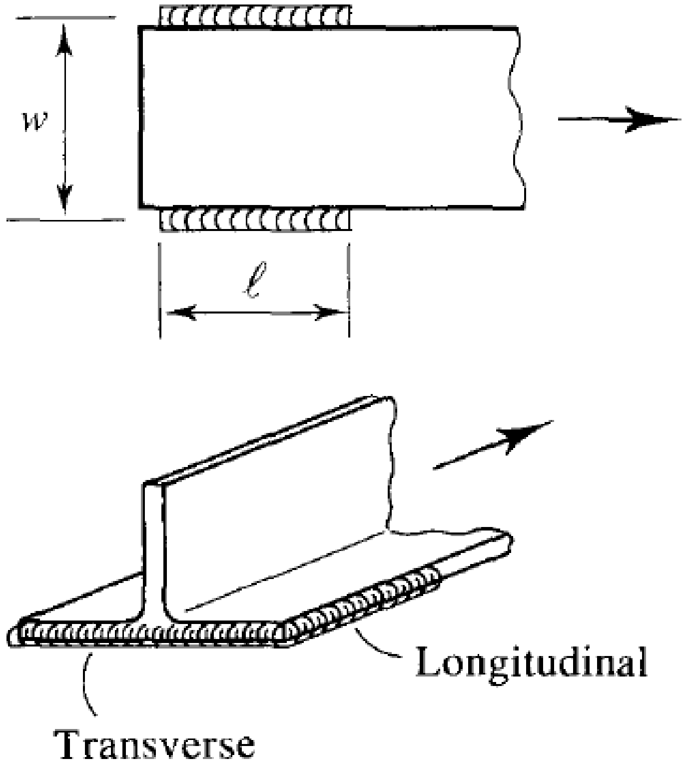
$$\ell \geq H: \quad U = 1 - \frac{\bar{x}}{\ell}$$

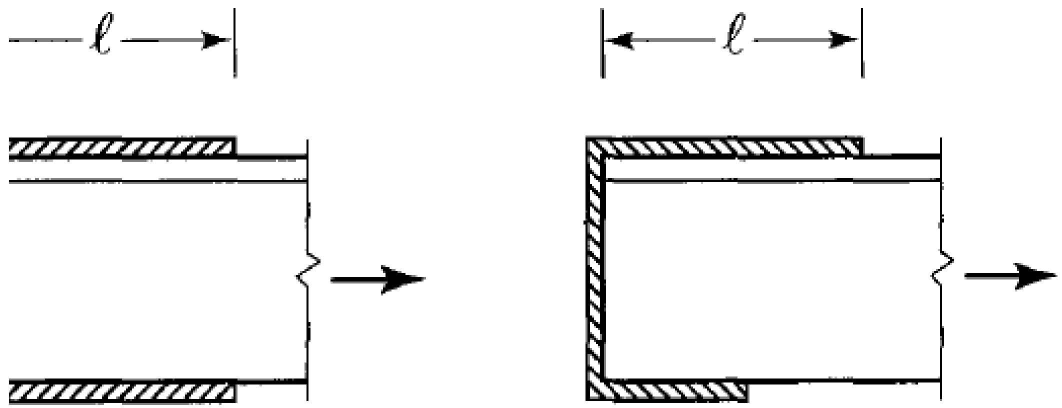
$$\bar{x} = \frac{B^2}{4(B+H)}$$

(g)



(a) Bolted





(b) Welded

Strength of plates and gusset plates used in connection subjected to tensile force, J4

When splice or gusset plates are used as statically loaded tensile connecting elements, **their** strength shall be determined as follows:

(a) For tensile yielding of connecting elements

$$R_n = F_y A_g \qquad \text{(AISC Equation J4-1)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For tensile rupture of connecting elements

$$R_n = F_u A_e \qquad \text{(AISC Equation J4-2)}$$

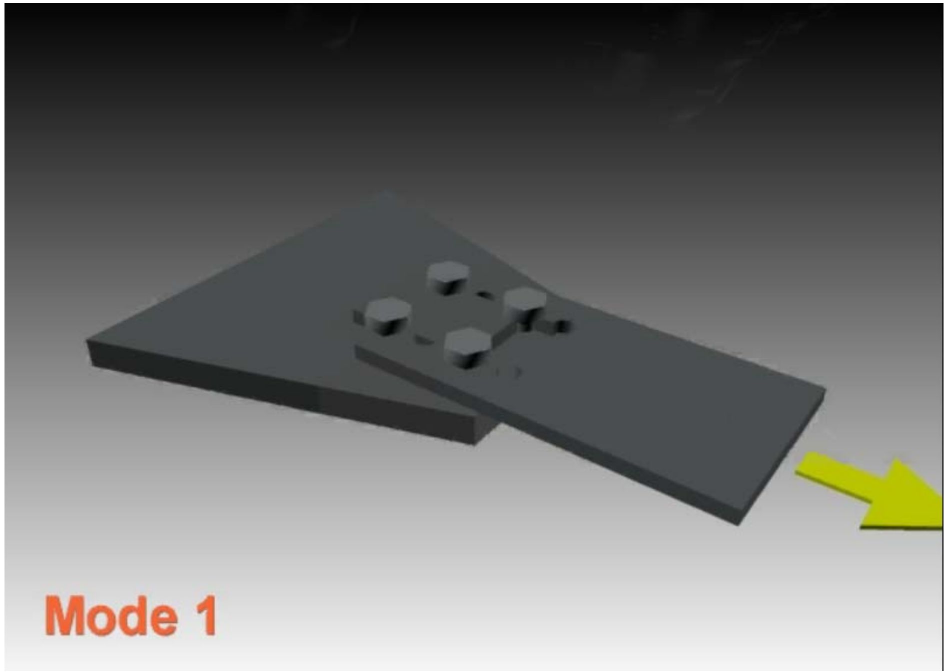
$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}.$$

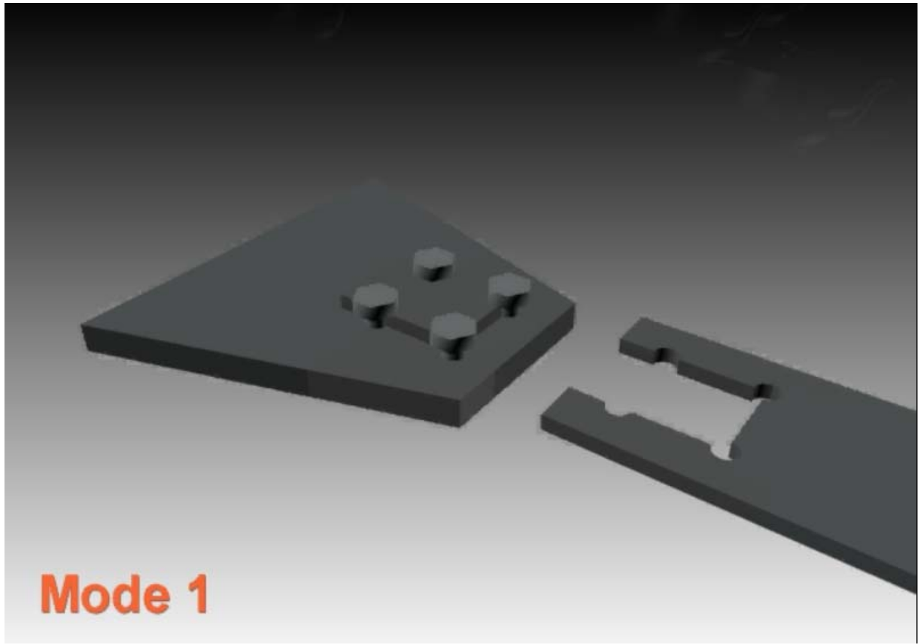
$A_e = A_n$

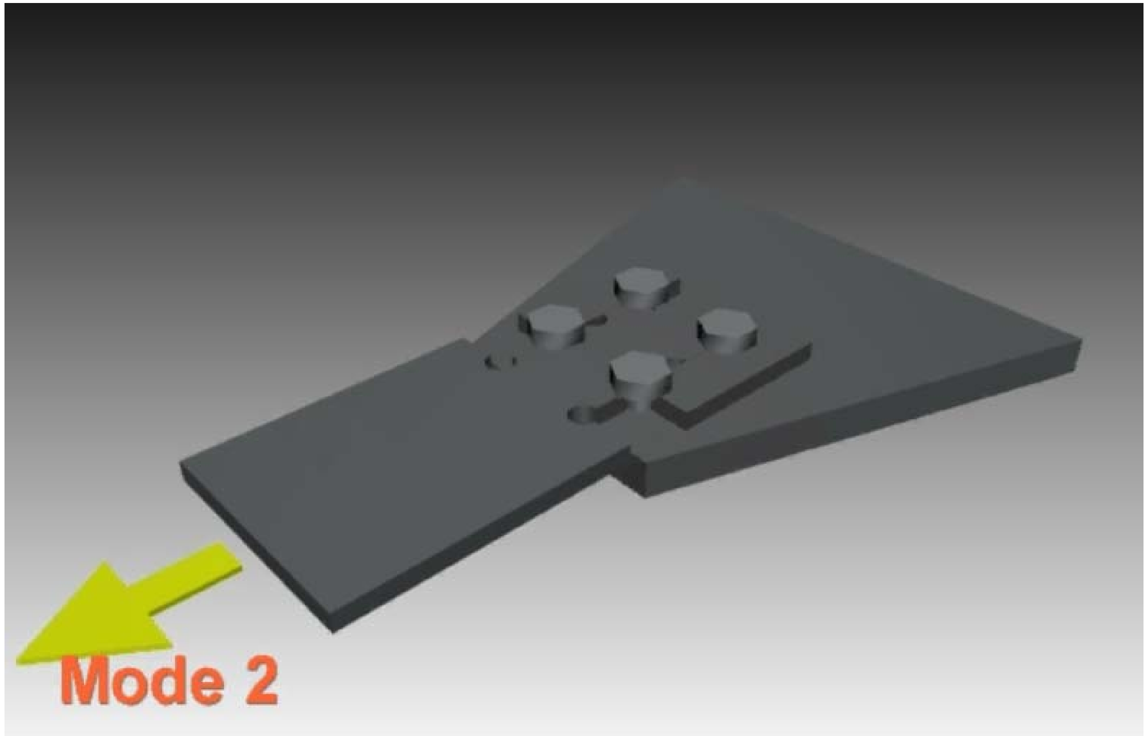
$A_n \leq 0.85\, A_g$ Often governs

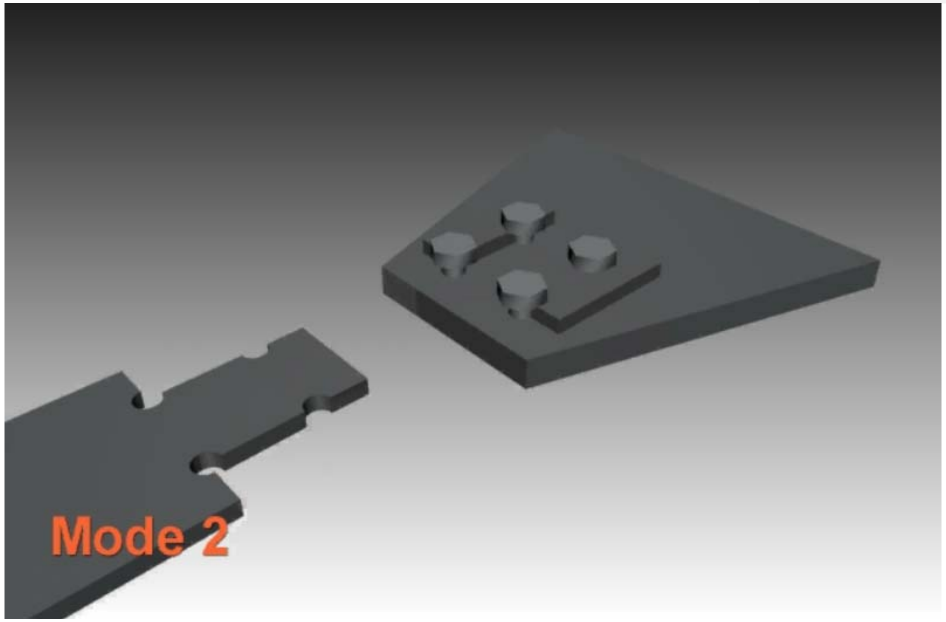
Block Shear Strength

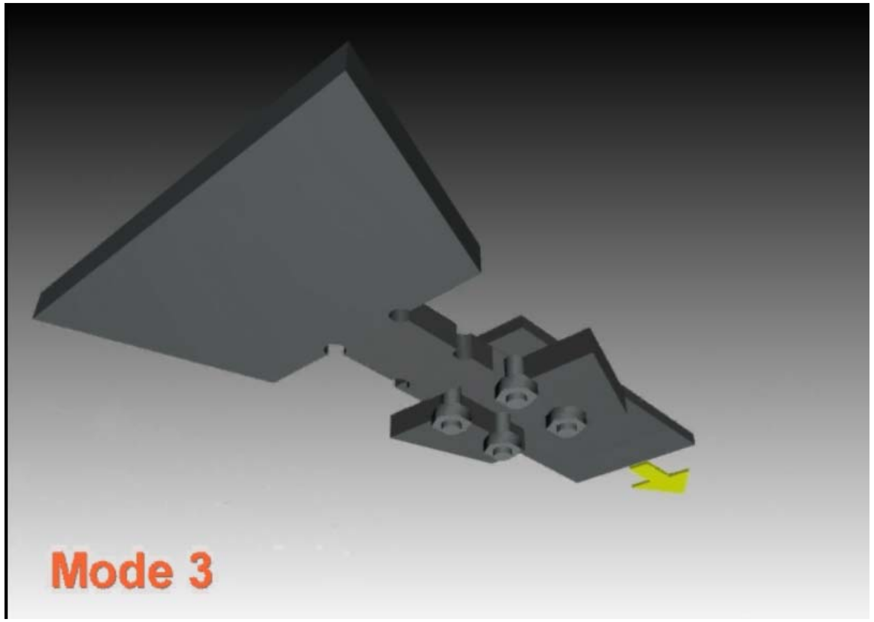
Block shear failure

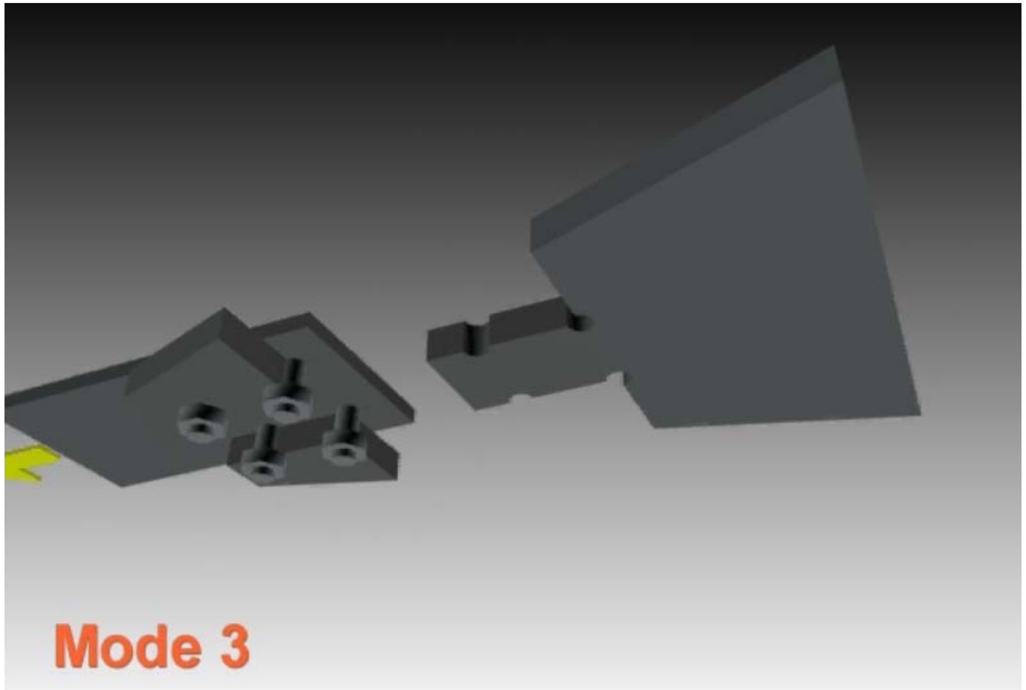

Mode 1

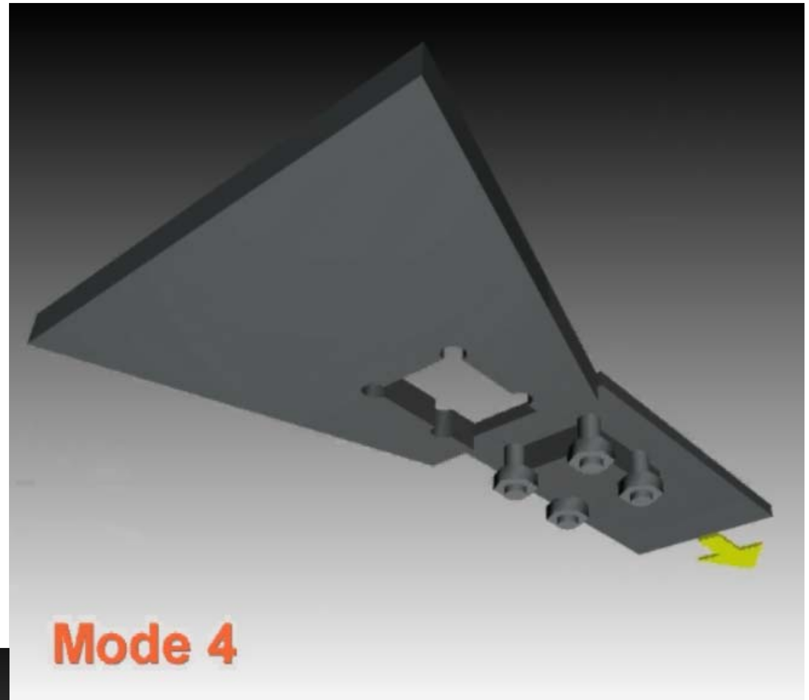

Mode 1


Mode 2


Mode 2


Mode 3


Mode 3


Mode 4


Mode 4

Block Shear

- **Failure Tears Out Block of Steel**

  Block is defined by:

  1. Center line of holes
  2. Edge of welds

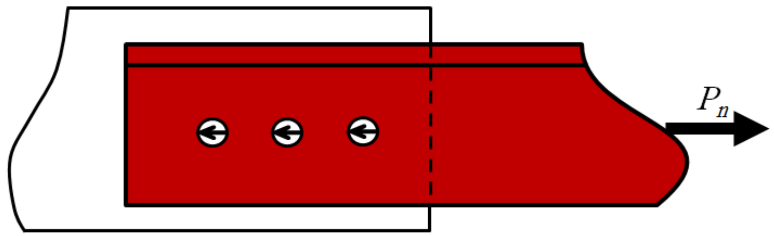
- **Block Shear is State of Combined Yielding and Rupture**

- **Failure Planes:**

  At least one each in tension and shear.

Typical Examples in Tension Members:

Angle Connected on One Leg



W-Shape Flange Connection

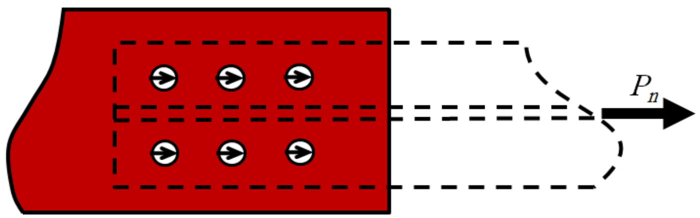
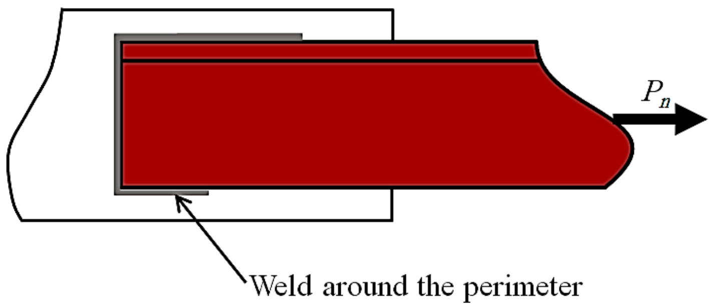


Plate Connection



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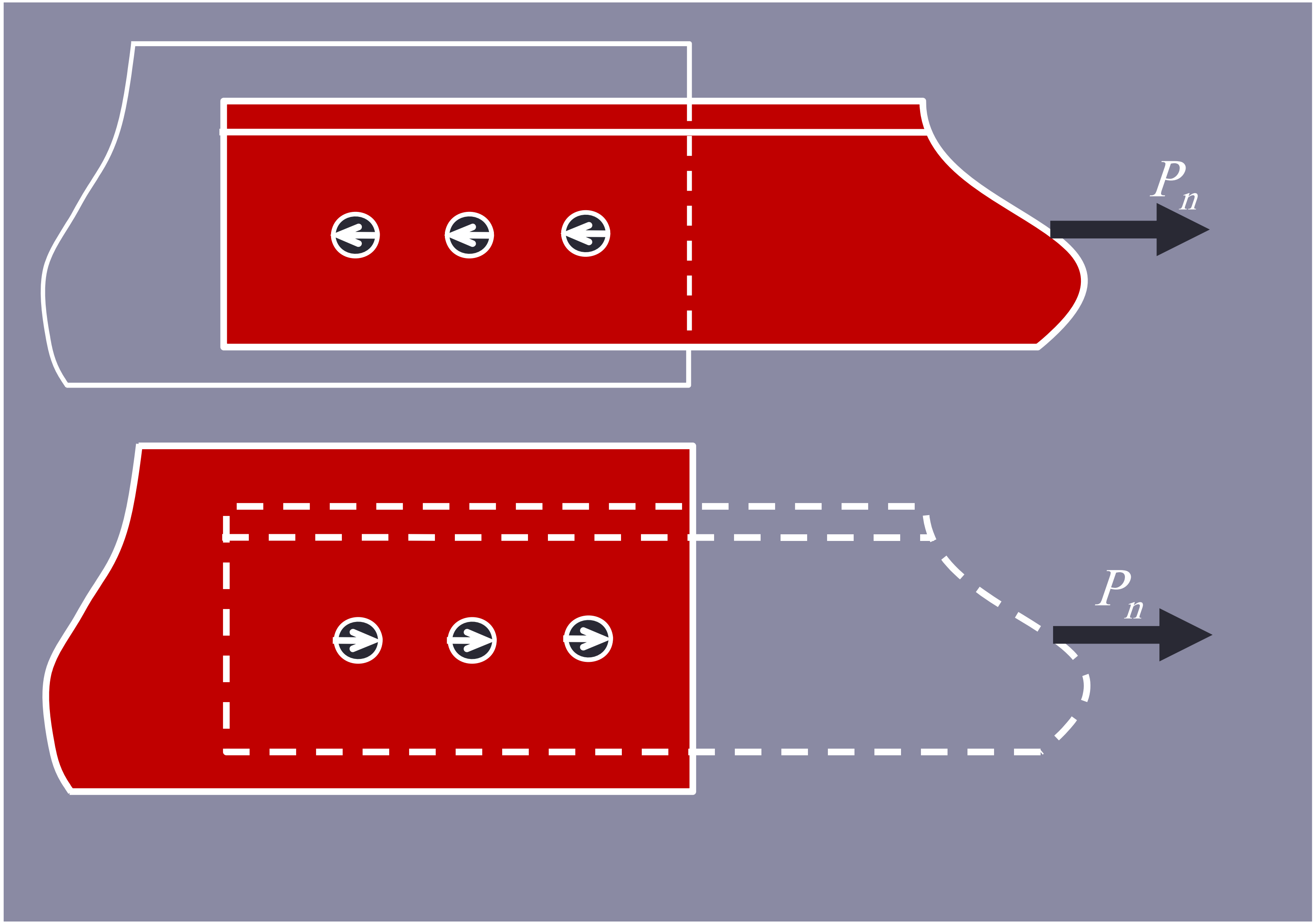
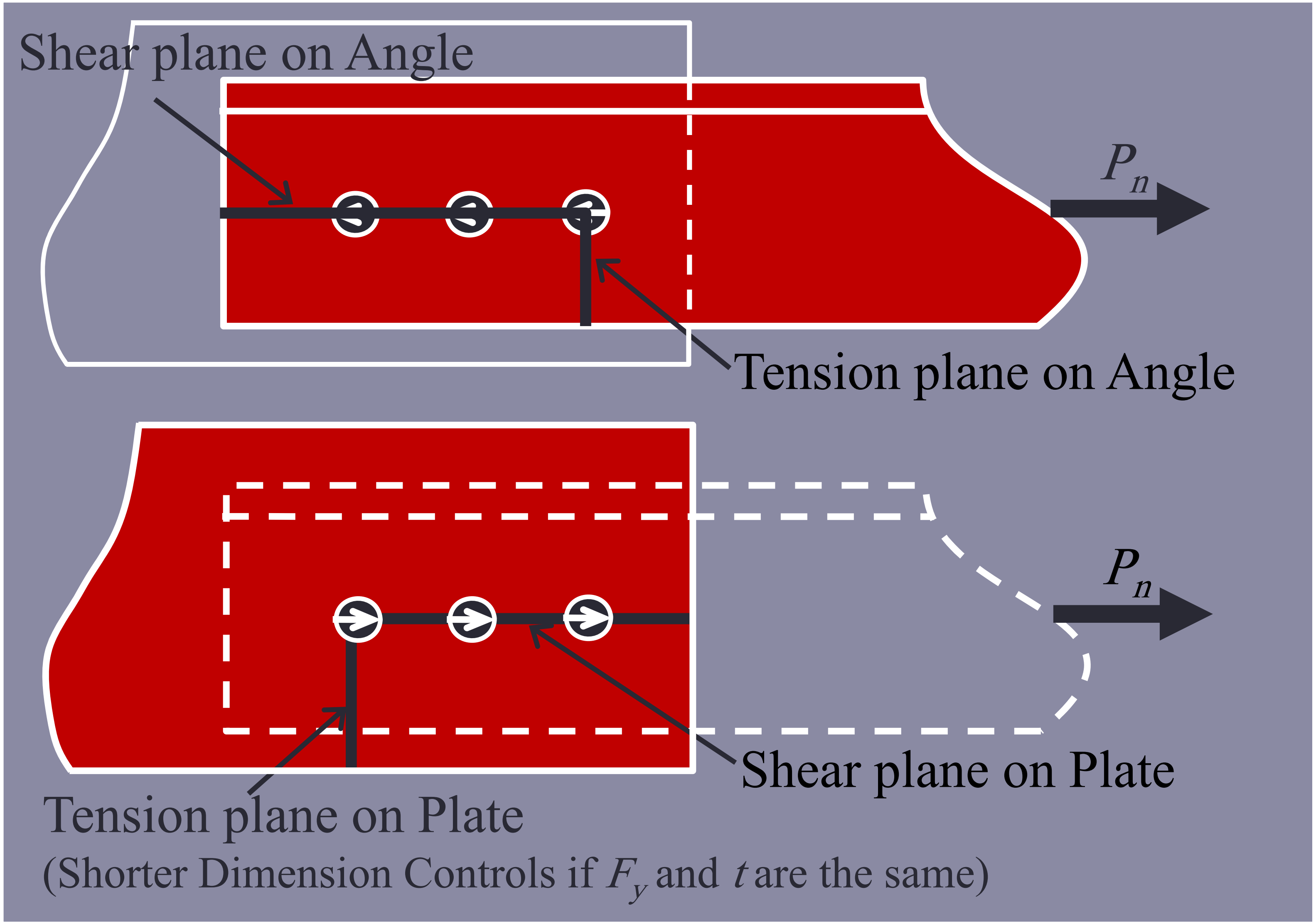
Block Shear

Angle Bolted to Plate



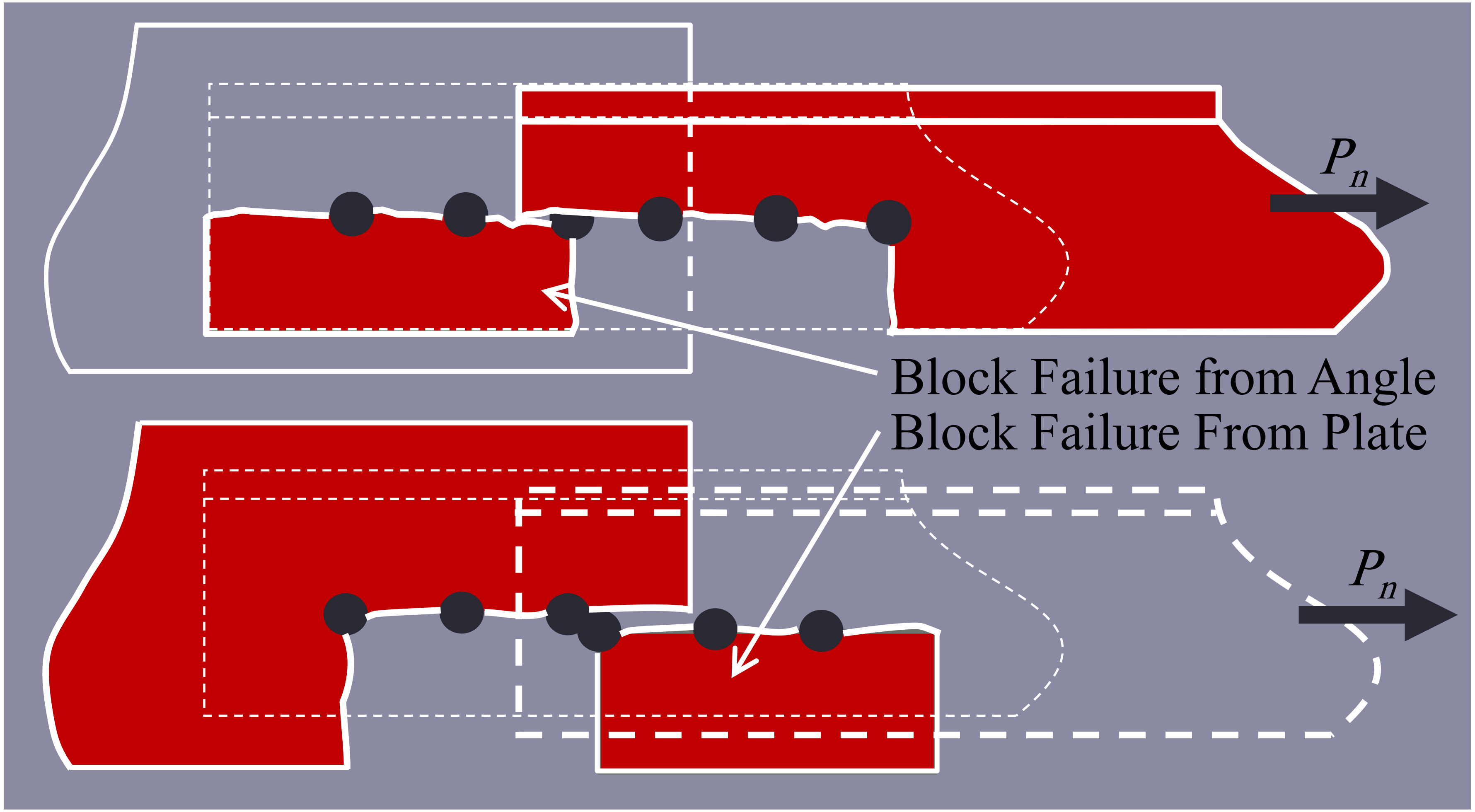
Block Shear

Angle Bolted to Plate



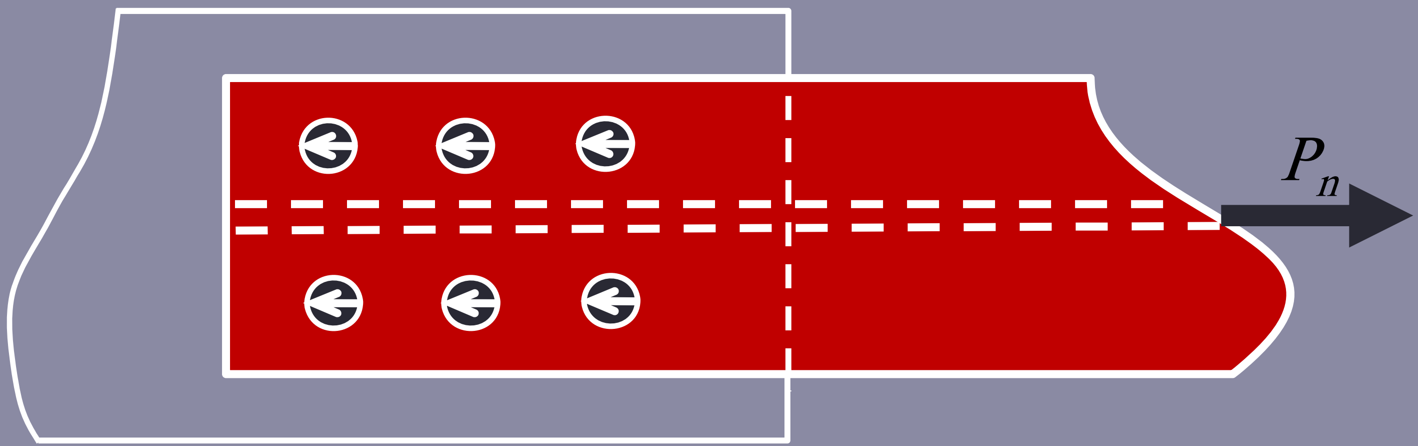
Block Shear

Angle Bolted to Plate



Block Shear

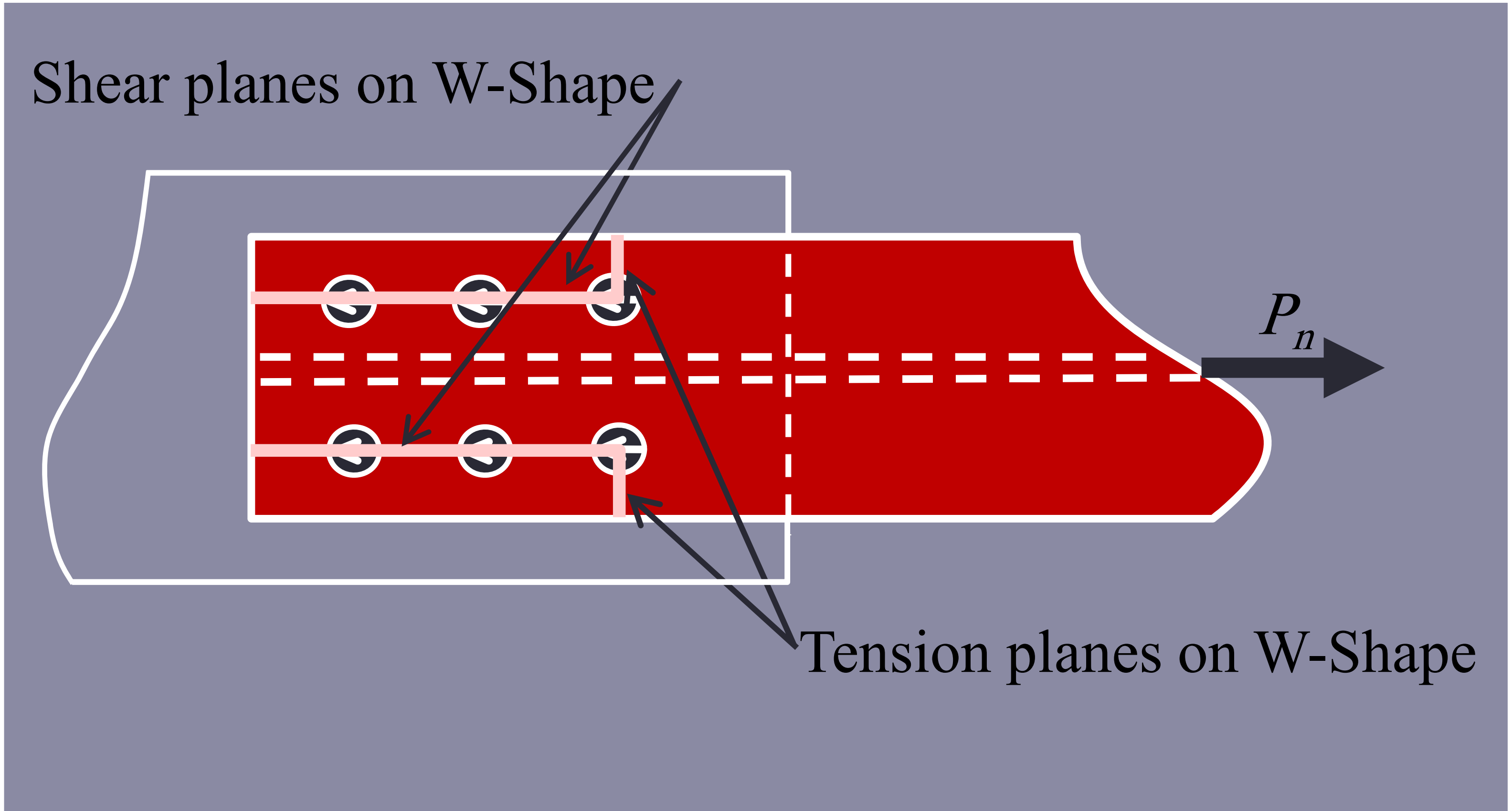
Flange of W-Shape
Bolted to Plate



First look at the W-Shape, then the plate

Block Shear

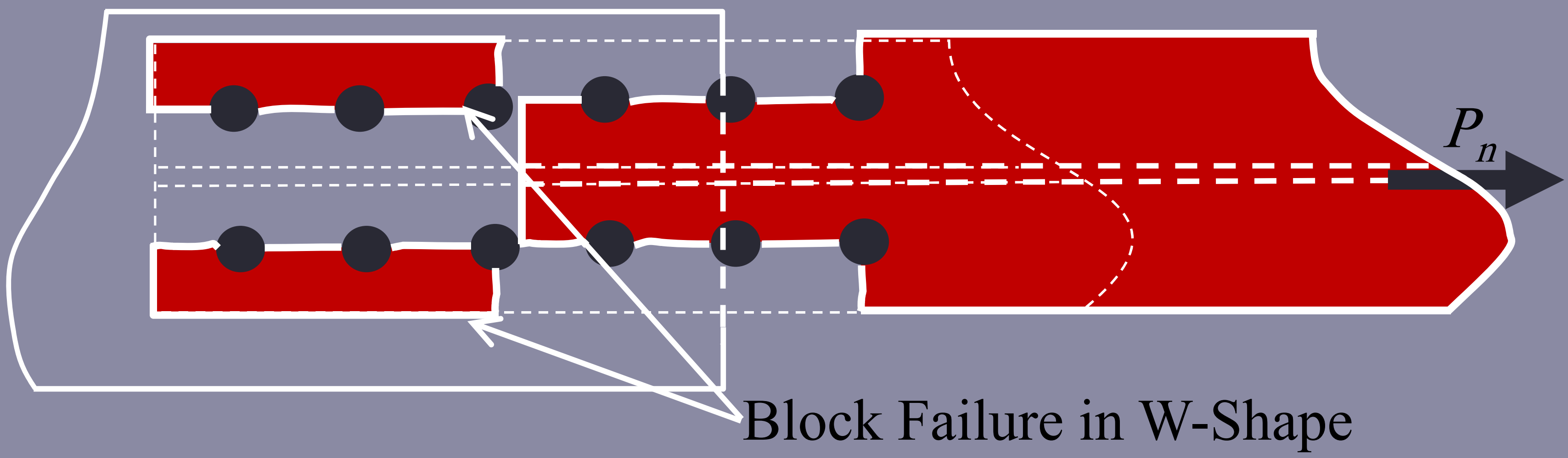
Flange of W-Shape Bolted to Plate



First look at the W-Shape, then the plate

Block Shear

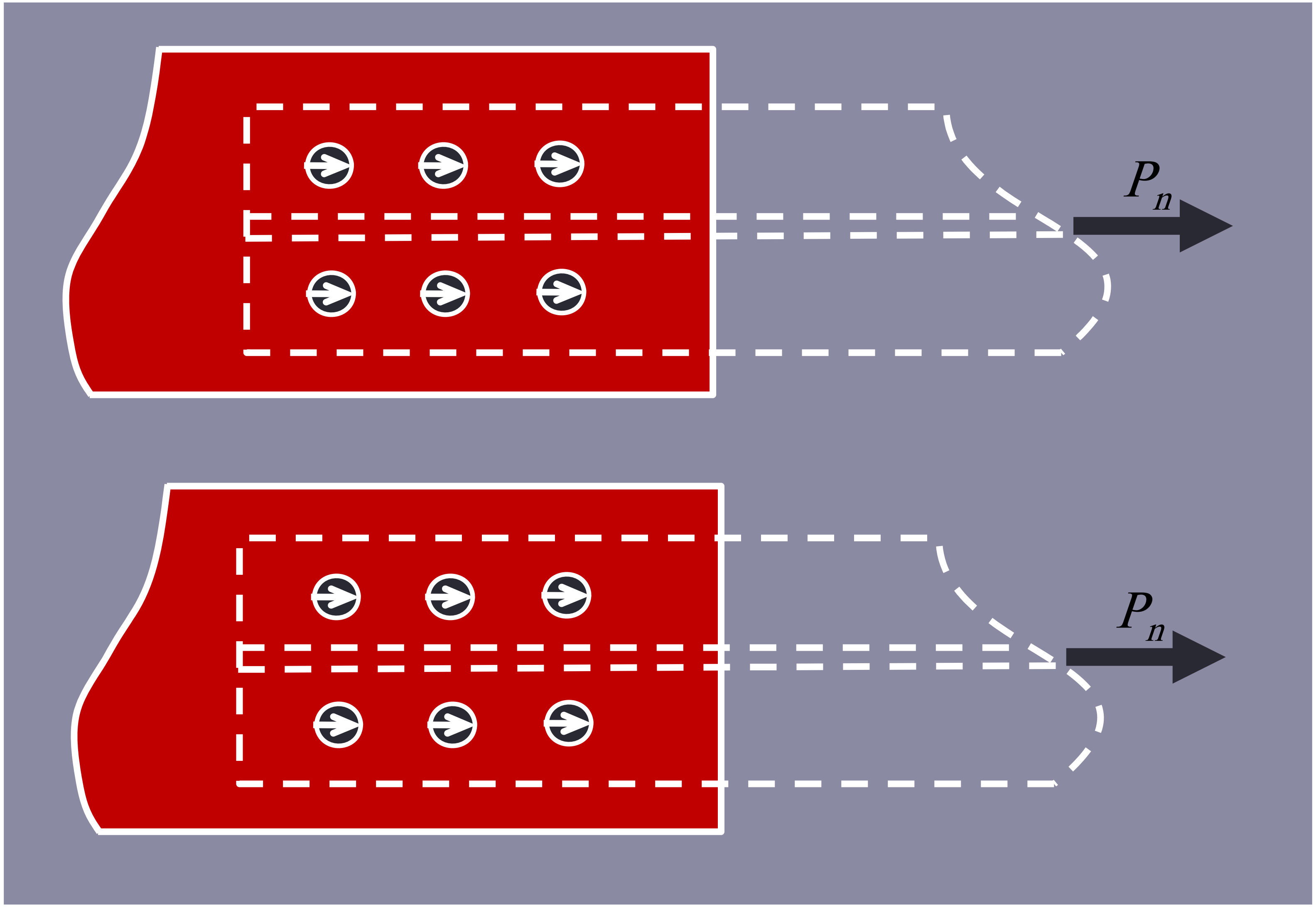
Flange of W-Shape
Bolted to Plate



First look at the W-Shape, then the plate

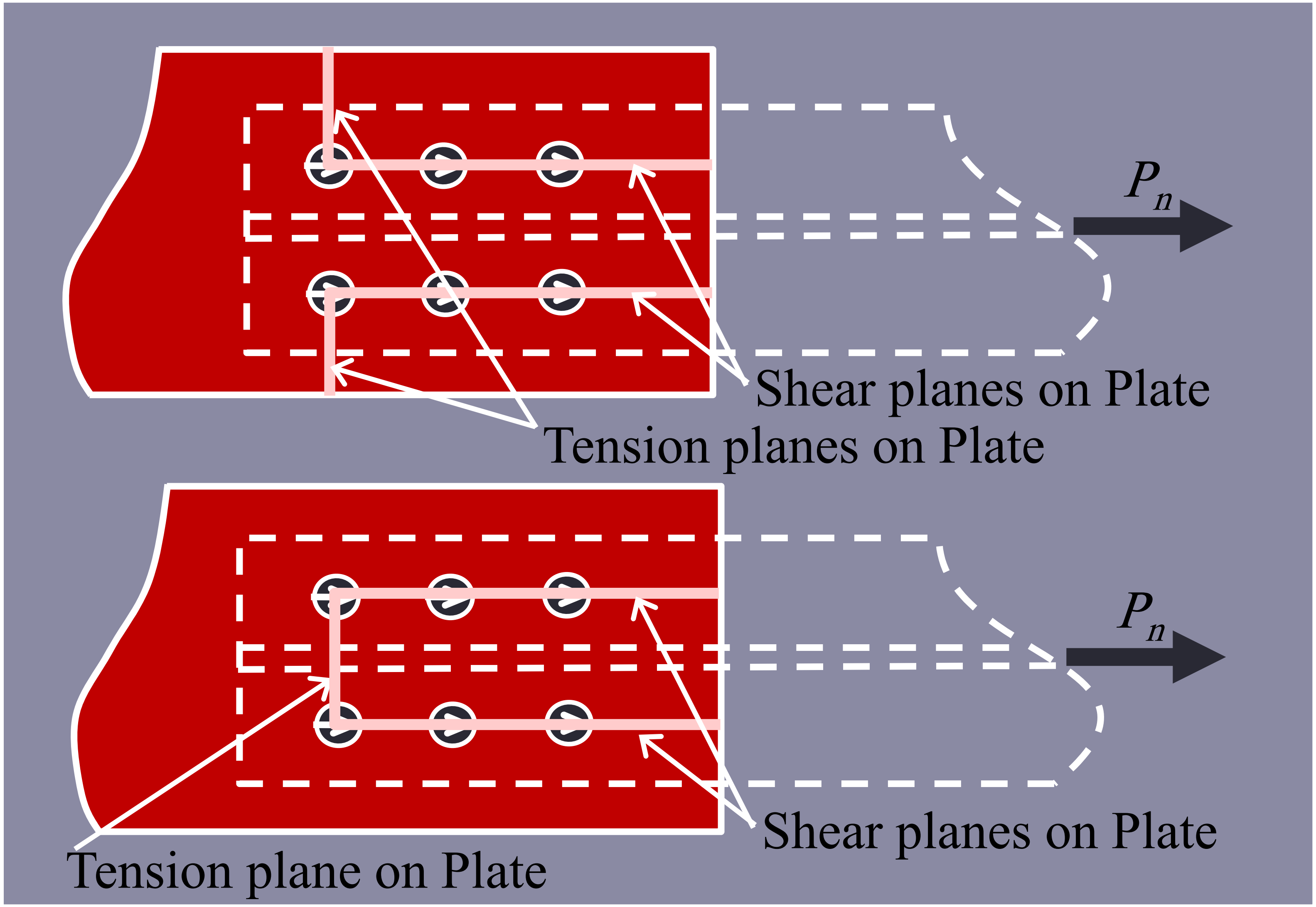
Block Shear

Flange of W-Shape Bolted to Plate



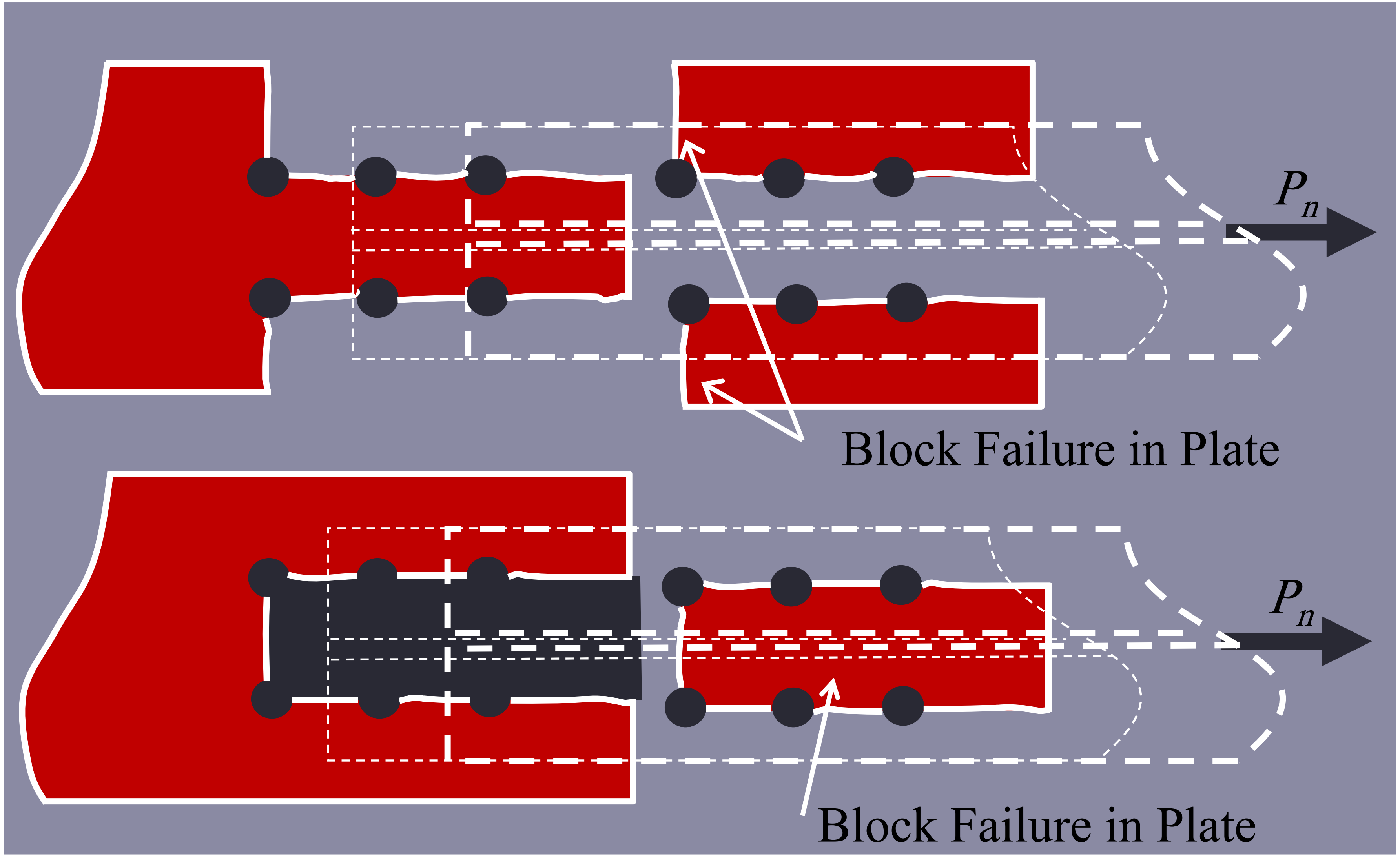
Block Shear

Flange of W-Shape Bolted to Plate



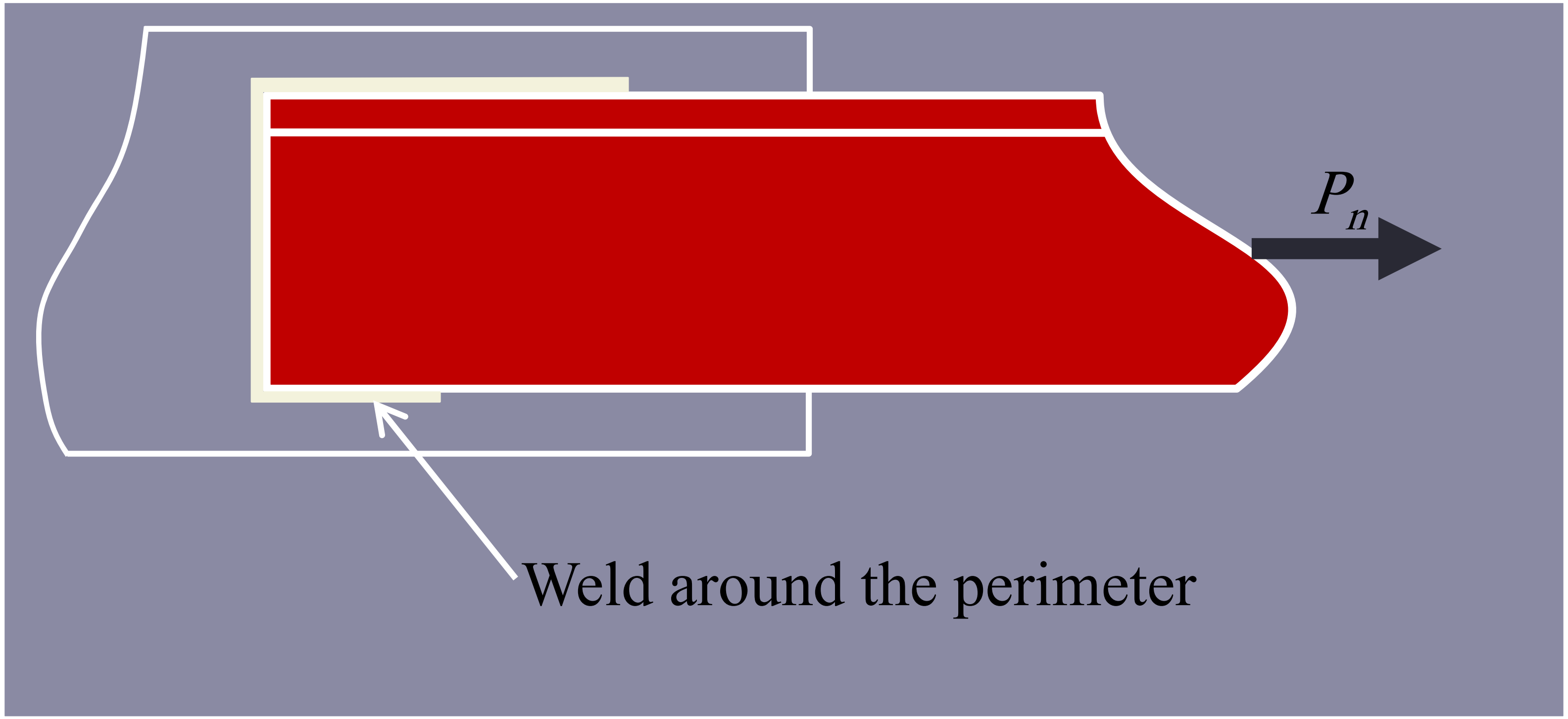
Block Shear

Flange of W-Shape Bolted to Plate



Block Shear

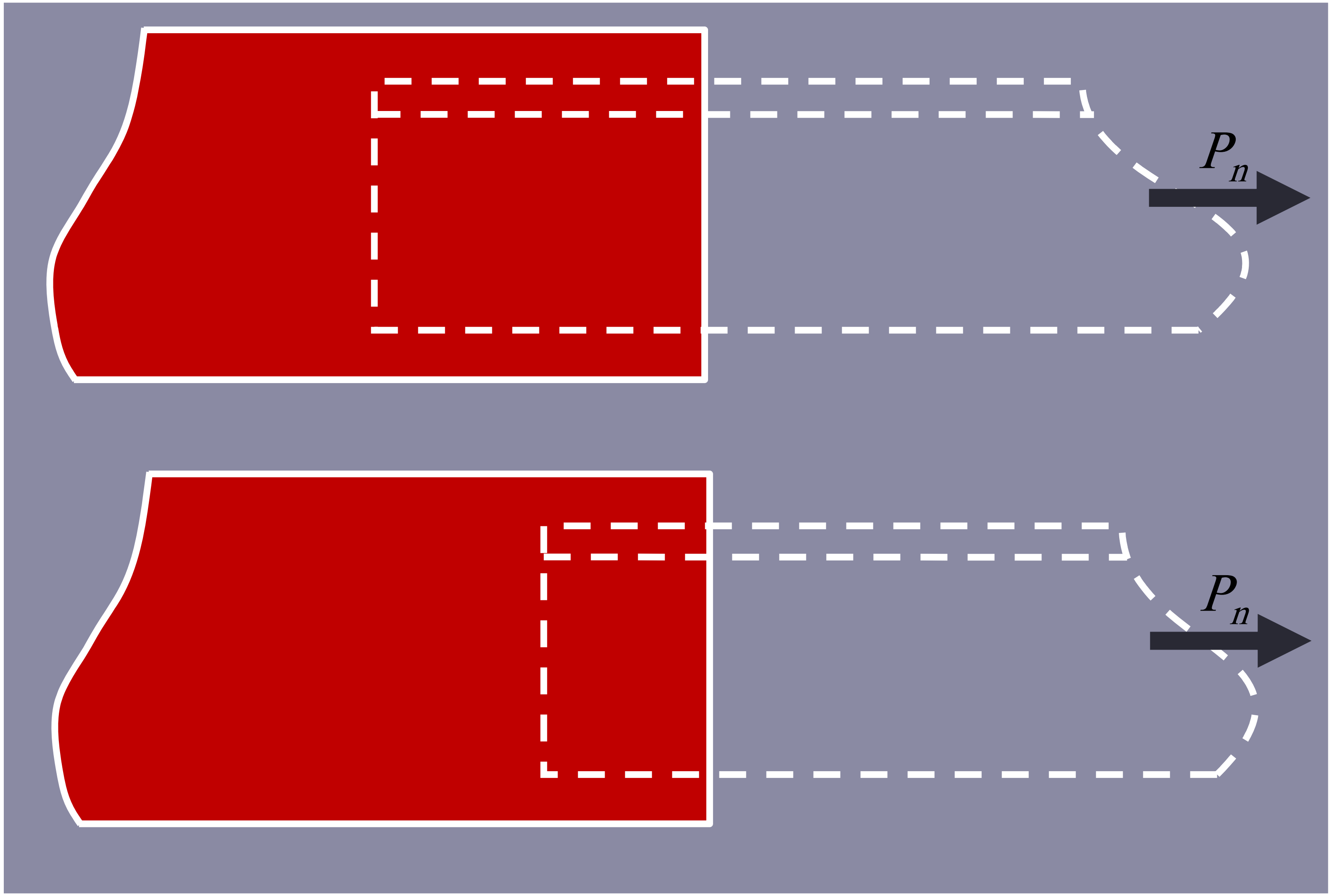
Angle or Plate Welded to Plate



Two Block Shear Failures to Check

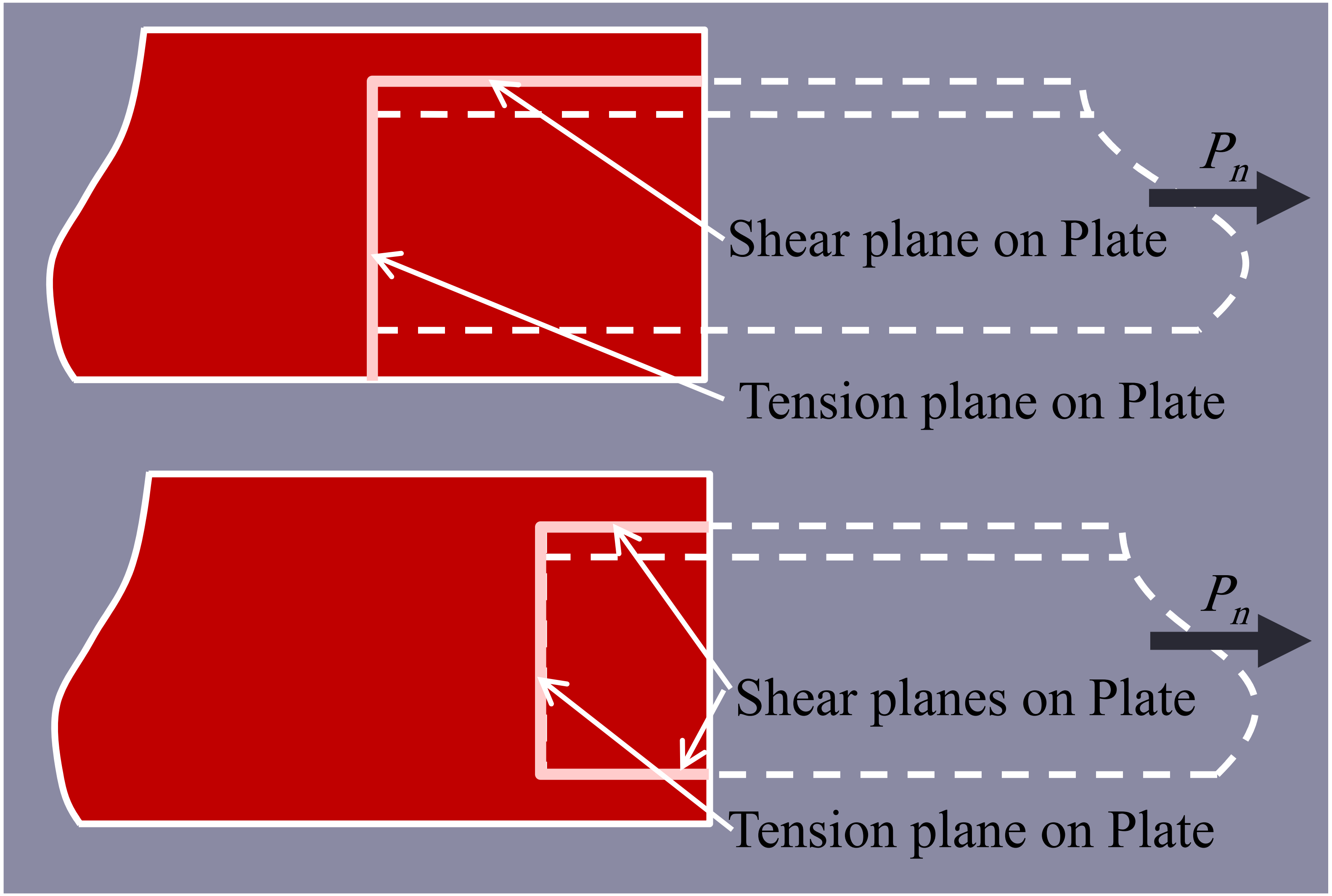
Block Shear

Angle or Plate Welded to Plate



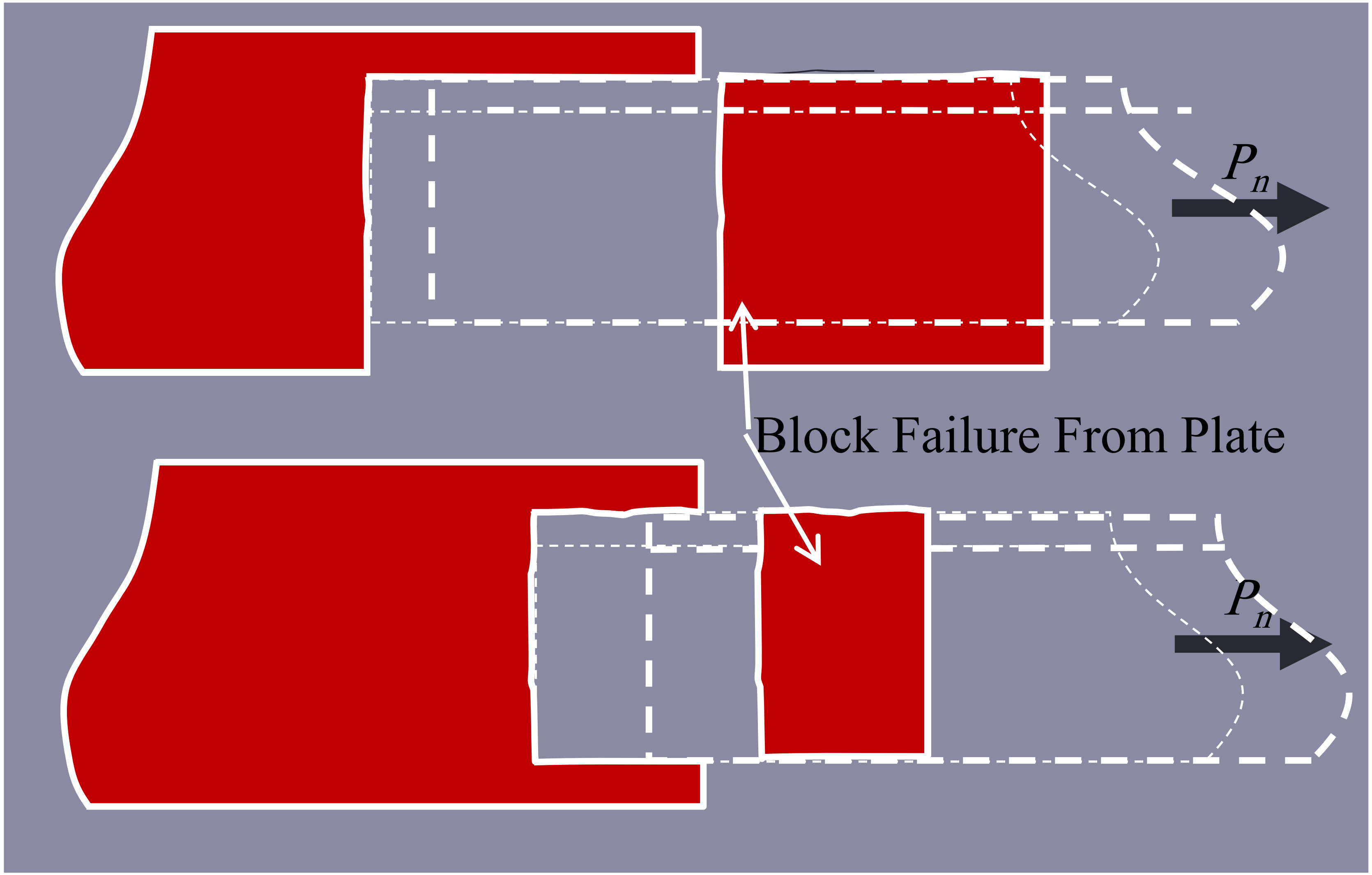
Block Shear

Angle or Plate Welded to Plate



Block Shear

Angle or Plate Welded to Plate



Block Shear- AISC Specifications, CH. J, J4

CHAPTER J

DESIGN OF CONNECTIONS

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

This section applies to elements of members at *connections* and connecting elements, such as plates, gussets, angles, and brackets.

Block Shear- AISC Specifications, CH. J, J4

1. Strength of Elements in Tension

The *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, of affected and connecting elements loaded in tension shall be the lower value obtained according to the *limit states* of *tensile yielding* and *tensile rupture*.

(a) For tensile yielding of connecting elements:

$$R_n = F_y A_g \tag{J4-1}$$

$$\phi = 0.90\,(\text{LRFD}) \qquad \Omega = 1.67\,(\text{ASD})$$

(b) For tensile rupture of connecting elements:

$$R_n = F_u A_e \tag{J4-2}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

where

A_e = *effective net area* as defined in Section D3.3, in.^{2} (mm^2); for bolted *splice* plates, $A_e = A_n \leq 0.85A_g$

Block Shear- AISC Specifications, CH. J, J4

2. Strength of Elements in Shear

The available shear yield strength of affected and connecting elements in shear shall be the lower value obtained according to the *limit states* of *shear yielding* and *shear rupture*:

(a) For shear yielding of the element:

$$R_n = 0.60 F_y A_g \tag{J4-3}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

(b) For shear rupture of the element:

$$R_n = 0.6 F_u A_{nv} \tag{J4-4}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

A_{nv} = net area subject to shear, in.^{2} (mm^2)

Block Shear- AISC Specifications, CH. J, J4

3. Block Shear Strength

The *available strength* for the *limit state* of *block shear rupture* along a shear failure path or path(s) and a perpendicular tension failure path shall be taken as

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt} \quad \text{(J4-5)}$$

$$\phi = 0.75\,\text{(LRFD)} \qquad \Omega = 2.00\,\text{(ASD)}$$

where

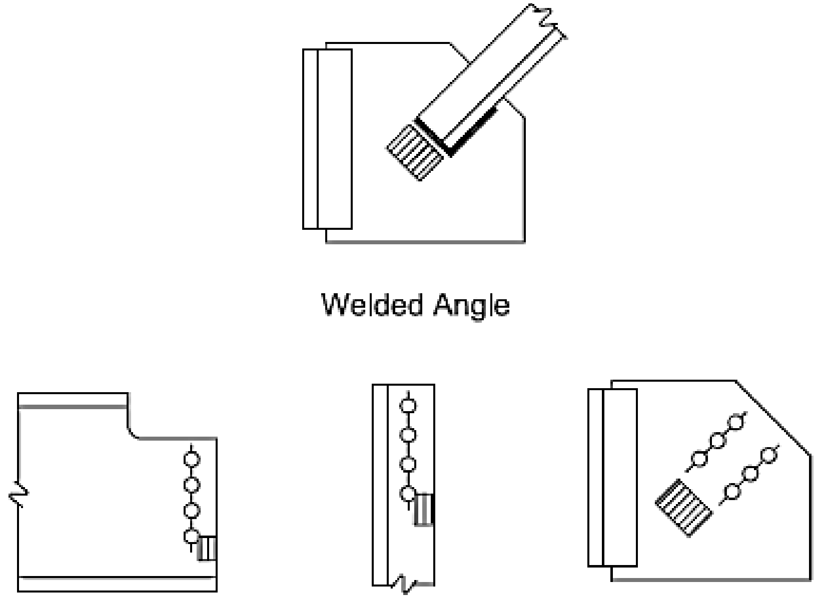
A_{gv} = gross area subject to shear, in.^{2} (mm^2)
A_{nt} = *net area* subject to tension, in.^{2} (mm^2)
A_{nv} = net area subject to shear, in.^{2} (mm^2)

Where the tension *stress* is uniform, $U_{bs} = 1$; where the tension stress is non-uniform, $U_{bs} = 0.5$.

User Note: The cases where U_{bs} must be taken equal to 0.5 are illustrated in the Commentary.

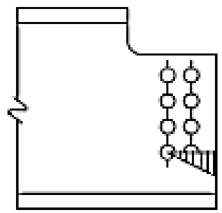
Block Shear- AISC Specifications, Commentary CH. J, Comm. J4

- A reduction factor, U_{bs} , has been included in Equation J4-5 to approximate the non-uniform stress distribution on the tensile plane.



(a) Cases for which U_{bs}= 1.0

U_{bs}= 1 for most tension members



(b) Case for which U_{bs}= 0.5

The rows of bolts nearest the beam end pick up most of the shear load

Block Shear- AISC Specifications, Commentary CH. J, Comm. J4

Block Shear Rupture Strength (Equation J4-5),

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

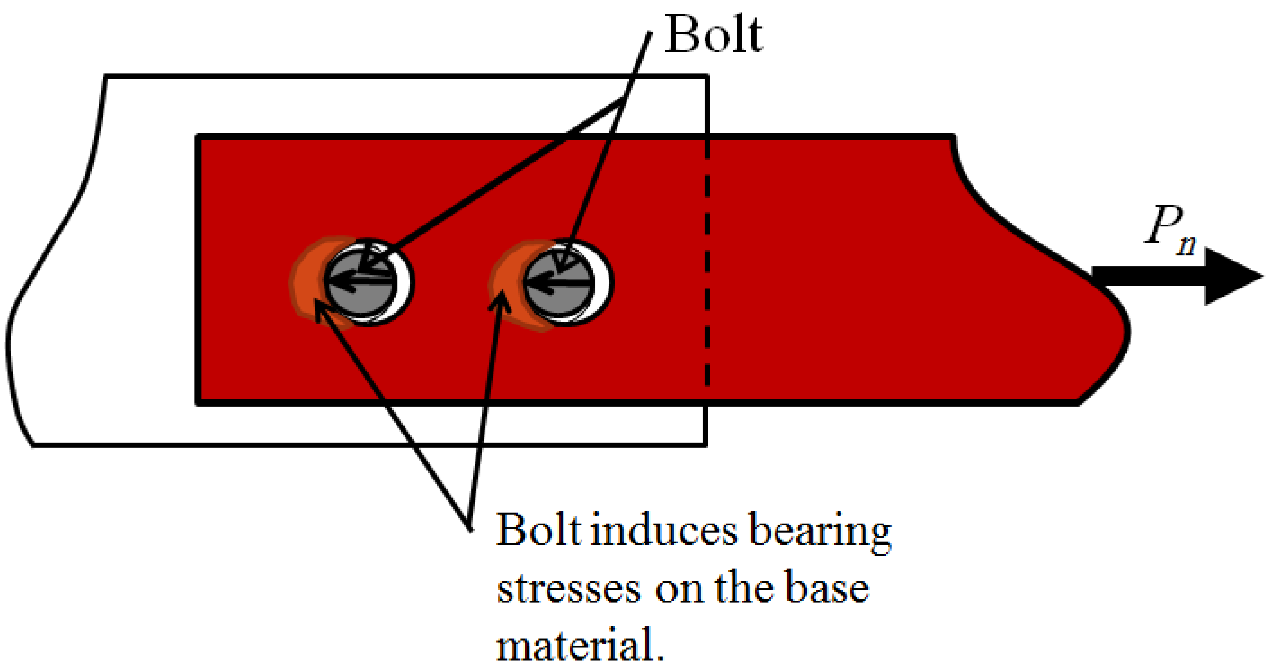
Smaller of two values will control. Why?

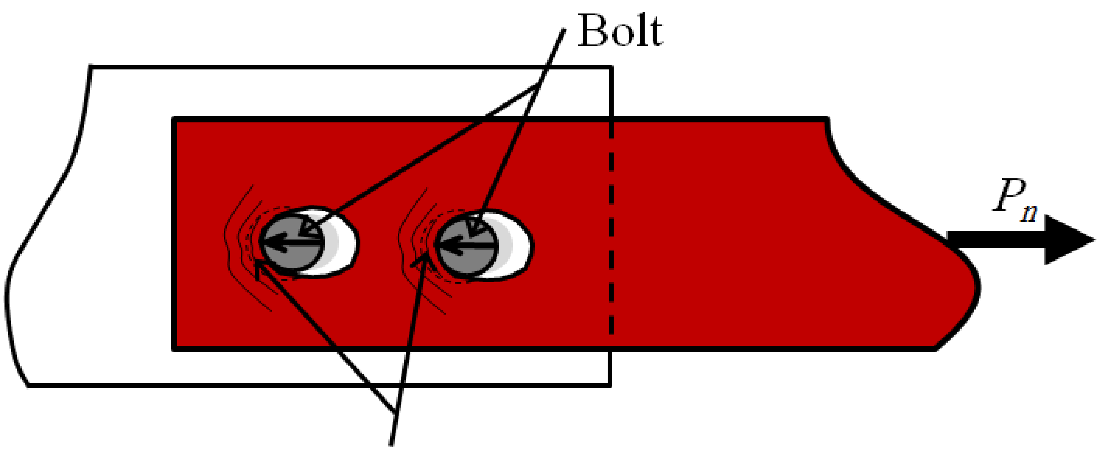
<u>Answer:</u>

- **Block shear is a rupture or tearing phenomenon, not a yielding limit state.**
- **However, gross yielding on the shear plane can occur when tearing on the tensile plane commences if 0.6Fu A_{nv} exceeds 0.6Fy A_{gv}.**
- **Hence, Equation J4-5 limits the term 0.6Fy A_{gv} to not greater than 0.6Fu A_{nv}.**

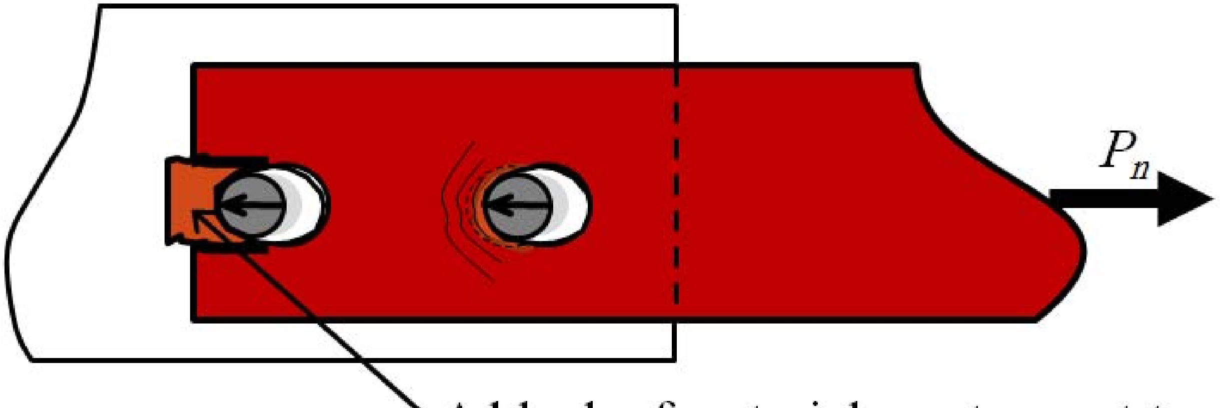
Bearing at Bolt Holes

- Bolts bear into material around hole.
- Direct bearing can deform the bolt hole an excessive amount and be limited by direct bearing capacity.
- If the clear space to adjacent hole or edge distance is small, capacity may be limited by tearing out a section of base material at the bolt.

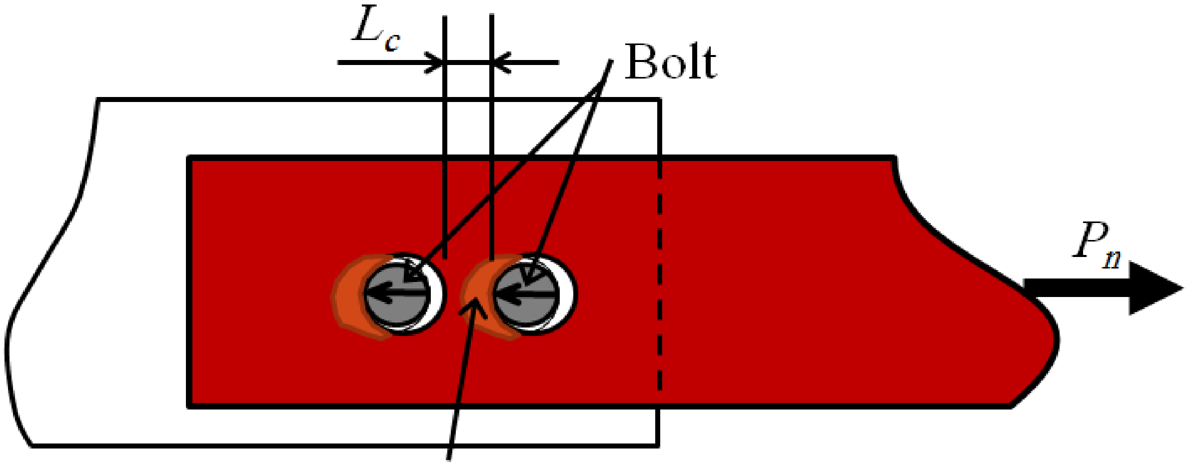




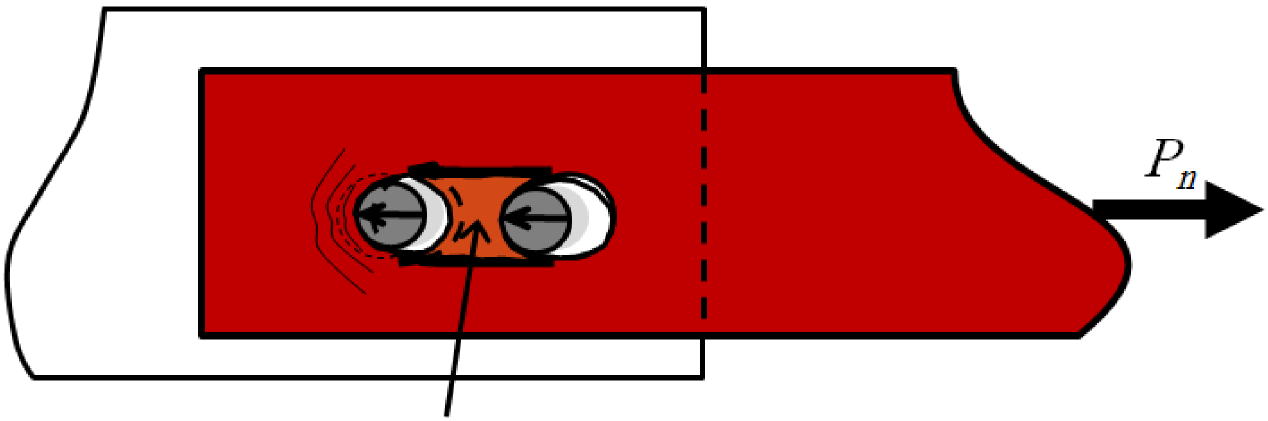
Which can result in excessive deformation of the bolt hole,



A block of material can tear out to the plate edge due to bearing.



Similarly, when bearing stresses act on bolts that are closely spaced (L_c dimension is small).



A block of material can tear out between the bolt holes due to bearing stresses.

J3. BOLTS AND THREADED PARTS

10. Bearing Strength at Bolt Holes

For standard, oversized, and short-slotted holes, or long slotted holes with slots parallel to the direction of loading:

$$R_n = 1.2 L_c t F_u \le 2.4 d t F_u \qquad \text{(Equation J3-6a)}$$

L_c = clear distance, in the direction of force, between the edge of hole and the edge of adjacent hole or edge of the material.

t = thickness of connected material

d = nominal bolt diameter

F_u = specified minimum tensile strength of the connected material

$\phi_t = 0.75$ ($\Omega_t = 2.00$)

For standard, oversized, and short-slotted holes, or long slotted holes with slots parallel to the direction of loading:

(Equation J3-6a)

$$R_n = 1.2 L_c t F_u \leq 2.4 d t F_u$$

Tearout Limit

Bearing Limit

For standard, oversized, and short-slotted holes, or long slotted holes with slots parallel to the direction of loading, but when deformation of the bolt hole is not a design consideration:

$$R_n = 1.5 L_c t F_u \leq 3.0 d t F_u$$

(Equation J3-6b)

For long-slotted holes with slot perpendicular to the direction of force:

$$R_n = 1.0 L_c t F_u \leq 2.0 d t F_u$$

(Equation J3-6c)

Design of Tension Members

Selection of members to support given loads

The selected members should have:

- ✓ **Compactness**
- ✓ **Dimensions that fit into dimension of other members**
- ✓ **Connection that reduce shear lag**
- ✓ **Slenderness ratio should, preferably, not exceed 300**

- ❑ Use L, C, W and S sections for bolted connections
- ❑ Use Plates, C and Tees Sections for welded connections

Slenderness Ratio

Slenderness ratio of a member is the ratio of its unsupported length (L) to its least radius of gyration (r).

$L/r \leq 300$

- The purpose of such limitations for tension members is to ensure the use of sections with stiffness sufficient to prevent undesirable lateral deflections or vibrations.

- Steel specifications give maximum values of slenderness ratios for both tension and compression members.

- The recommended maximum slenderness ratio of 300 is not applicable to tension rods. Maximum L/r values for rods are left to the designer's judgment

Steps to design tension members

1. Estimate the required area to support the given load

2. Select sections corresponding to the required area

3. Check section's strength.

Estimation of the required area

A. For LRFD Approach

1. To satisfy the first limit state (yielding on gross area), the minimum gross area must be at least equal to

$$\min A_g = \frac{P_u}{\phi_t F_y}$$

2. To satisfy the second limit state (rupture on net area), the minimum value of A_e must be at least equal to

$$\min A_e = \frac{P_u}{\phi_t F_u}$$

$$\min A_n = \frac{\min A_e}{U} = \frac{P_u}{\phi_t F_u U}$$

Then the minimum A_g = min A_n + estimated area of holes

$$= \frac{P_u}{\phi_t F_u U} + \text{estimated area of holes}$$

B. For ASD Approach

1. To satisfy the first limit state (yielding on gross area), the minimum gross area must be at least equal to

$$\min A_g = \frac{\Omega_t P_a}{F_y}$$

2. To satisfy the second limit state (rupture on net area), the minimum value of A_e must be at least equal to

$$\min A_g = \frac{\Omega_t P_a}{F_u U} + \text{estimated area of holes}$$

Design Tables 5-1 to Table 5-8 List available yield and rupture strength for typical sections.

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
W40×392[h]	115	86.3	3440	5180	2800	4210
×331[h]	97.5	73.1	2920	4390	2380	3560
×327[h]	96.0	72.0	2870	4320	2340	3510
×294	86.3	64.7	2580	3880	2100	3150
×278	82.0	61.5	2460	3690	2000	3000
×264	77.6	58.2	2320	3490	1890	2840
×235	69.0	51.8	2070	3110	1680	2530
×211	62.0	46.5	1860	2790	1510	2270
×183	53.3	40.0	1600	2400	1300	1950
×167	49.2	36.9	1470	2210	1200	1800
×149	43.8	32.8	1310	1970	1070	1600

Design

Tables 5-1 to Table 5-8

List available yield and rupture strength for typical sections.

L8–L6

Table 5–2
Available Strength in Axial Tension
Angles

F_y = 36 ksi
F_u = 58 ksi

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
L8×8×1$\frac{1}{8}$	16.7	12.5	360	541	363	544
×1	15.0	11.3	323	486	328	492
×$\frac{7}{8}$	13.2	9.90	285	428	287	431
×$\frac{3}{4}$	11.4	8.55	246	369	248	372
×$\frac{5}{8}$	9.61	7.21	207	311	209	314
×$\frac{9}{16}$	8.68	6.51	187	281	189	283
×$\frac{1}{2}$	7.75	5.81	167	251	168	253
L8×6×1	13.0	9.75	280	421	283	424
×$\frac{7}{8}$	11.5	8.63	248	373	250	375
×$\frac{3}{4}$	9.94	7.46	214	322	216	325
×$\frac{5}{8}$	8.36	6.27	180	271	182	273
×$\frac{9}{16}$	7.56	5.67	163	245	164	247
×$\frac{1}{2}$	6.75	5.06	146	219	147	220
×$\frac{7}{16}$	5.93	4.45	128	192	129	194
L8×4×1	11.0	8.25	237	356	239	359
×$\frac{7}{8}$	9.73	7.30	210	315	212	318
×$\frac{3}{4}$	8.44	6.33	182	273	184	275
×$\frac{5}{8}$	7.11	5.33	153	230	155	232
×$\frac{9}{16}$	6.43	4.82	139	208	140	210
×$\frac{1}{2}$	5.75	4.31	124	186	125	187
×$\frac{7}{16}$	5.06	3.80	109	164	110	165
L7×4×$\frac{3}{4}$	7.69	5.77	166	249	167	251
×$\frac{5}{8}$	6.48	4.86	140	210	141	211
×$\frac{1}{2}$	5.25	3.94	113	170	114	171
×$\frac{7}{16}$	4.62	3.47	99.6	150	101	151
×$\frac{3}{8}$	3.98	2.99	85.8	129	86.7	130
L6×6×1	11.0	8.25	237	356	238	359
×$\frac{7}{8}$	9.75	7.31	210	316	212	318
×$\frac{3}{4}$	8.46	6.34	182	274	184	276
×$\frac{5}{8}$	7.13	5.35	154	231	155	233
×$\frac{9}{16}$	6.45	4.84	139	209	140	211
×$\frac{1}{2}$	5.77	4.33	124	187	126	188
×$\frac{7}{16}$	5.08	3.81	110	165	110	166
×$\frac{3}{8}$	4.38	3.29	94.4	142	95.4	143
×$\frac{5}{16}$	3.67	2.75	79.1	119	79.8	120

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

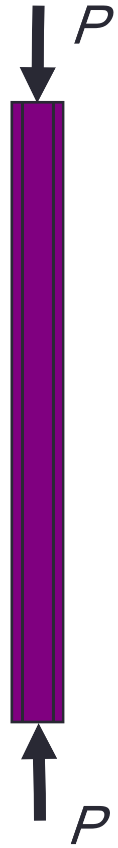
General Considerations

1. Block shear strength must be checked
2. Designer must select the largest gross area
3. Design tables assume $A_e = 0.700A_g$ to $0.952A_g$. You must check this is met in the member and connections.


THERE IS NO
ELEVATOR
TO SUCCESS.
YOU HAVE TO
TAKE THE
STAIRS

Analysis of Compression Members

COMPRESSION MEMBER/COLUMN: Structural member subjected to axial load



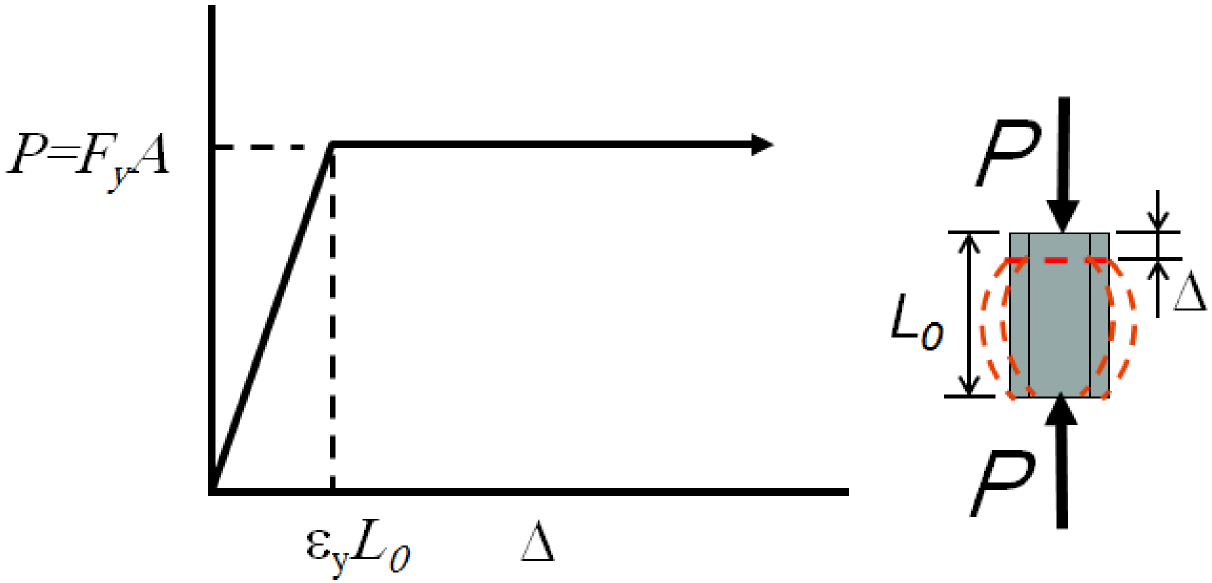
Columns are brought in with a crane and placed on over their respective base plates and fastened once they are plumbed, tie down cables are attached to keep the column from swaying during erection. Without the tie downs it would be a cantilevered column that is fully unsupported.

Axial Compressive Strength

Compressive Strength Limit States:

1. Squash Load
2. Global Buckling
3. Local Buckling

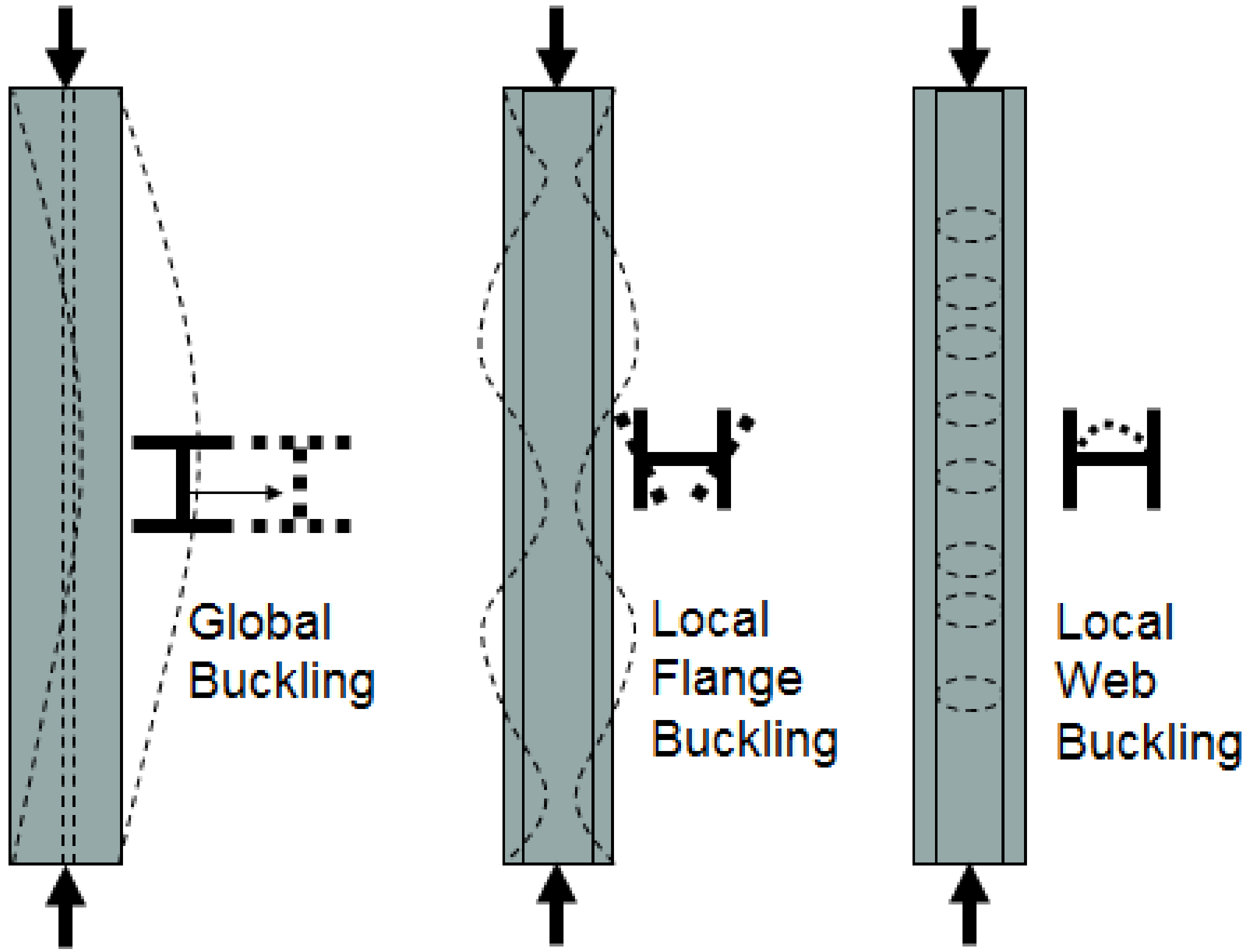
Squash Load (Fully Yielded Cross Section)



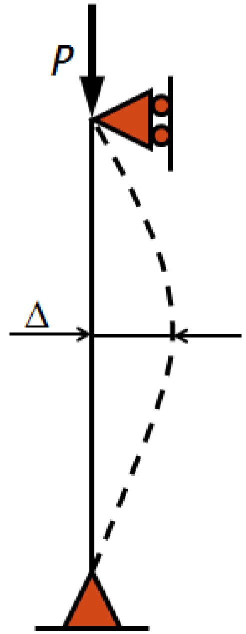
- When a short, stocky column is loaded the strength is limited by the yielding of the entire cross section.

- Absence of residual stress, all fibres of cross-section yield simultaneously at P/A=Fy.

Column Buckling



Euler Buckling

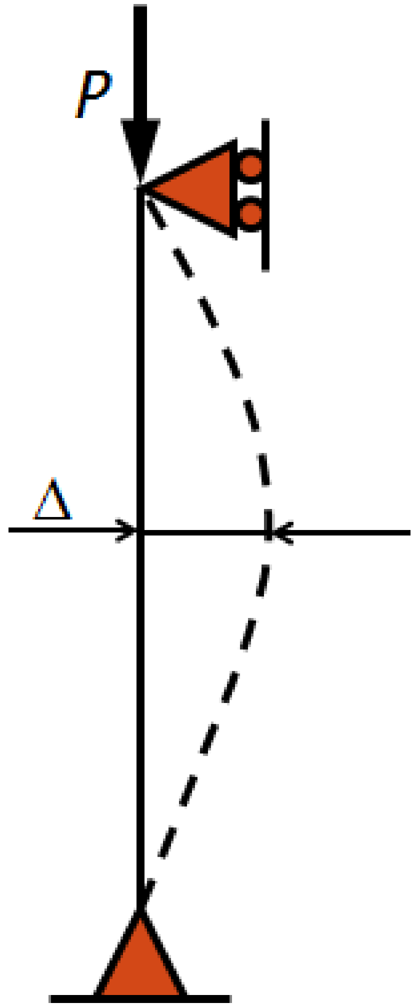


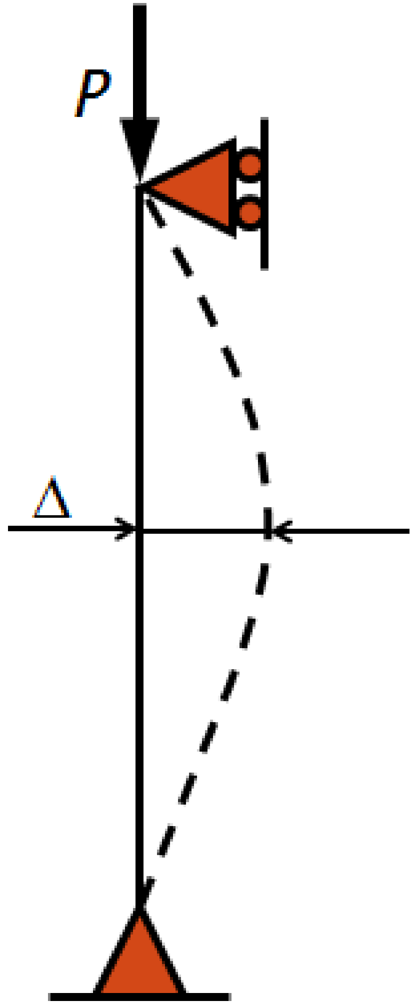
***Assumptions*:**

- Column is pin-ended.
- Column is initially perfectly straight.
- Load is at centroid.
- Material is linearly elastic (no yielding).
- Member bends about principal axis (no twisting).
- Plane sections remain Plane.
- Small Deflection Theory.

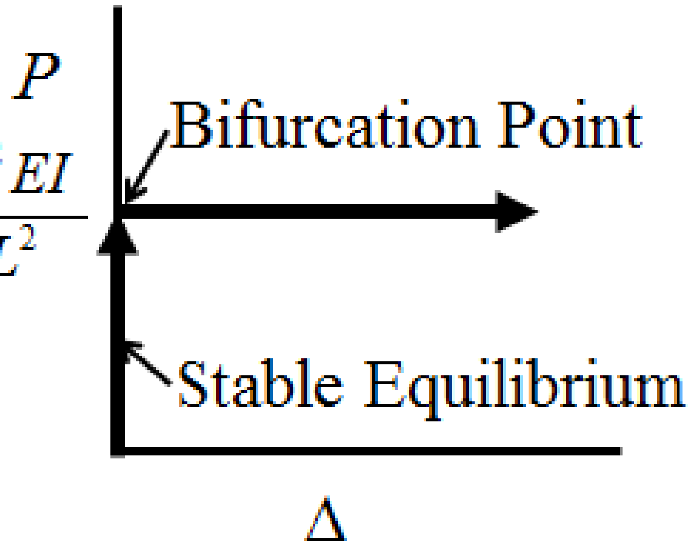
Leonhard Euler (15 April 1707 – 18 September 1783) was a Swiss mathematician, physicist, astronomer, logician and engineer who made important and influential discoveries in many branches of mathematics like infinitesimal calculus and graph theory while also making pioneering contributions to several branches such as topology and analytic number theory. He also introduced much of the modern mathematical terminology and notation, particularly for mathematical analysis, such as the notion of a mathematical function. He is also known for his work in mechanics, fluid dynamics, optics, astronomy, and music theory.

Euler Buckling

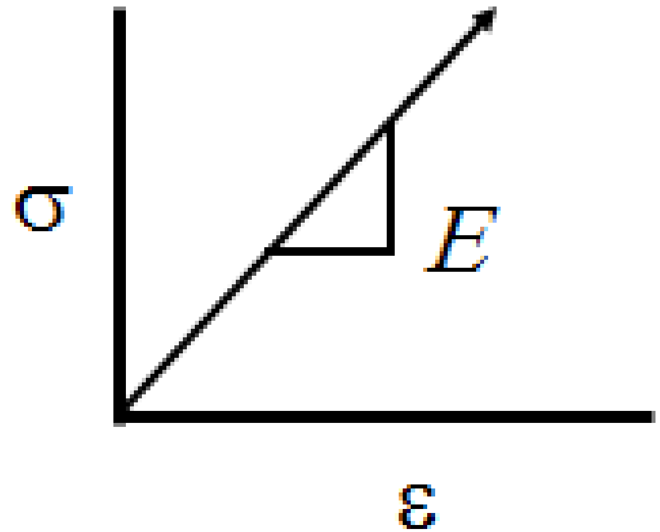




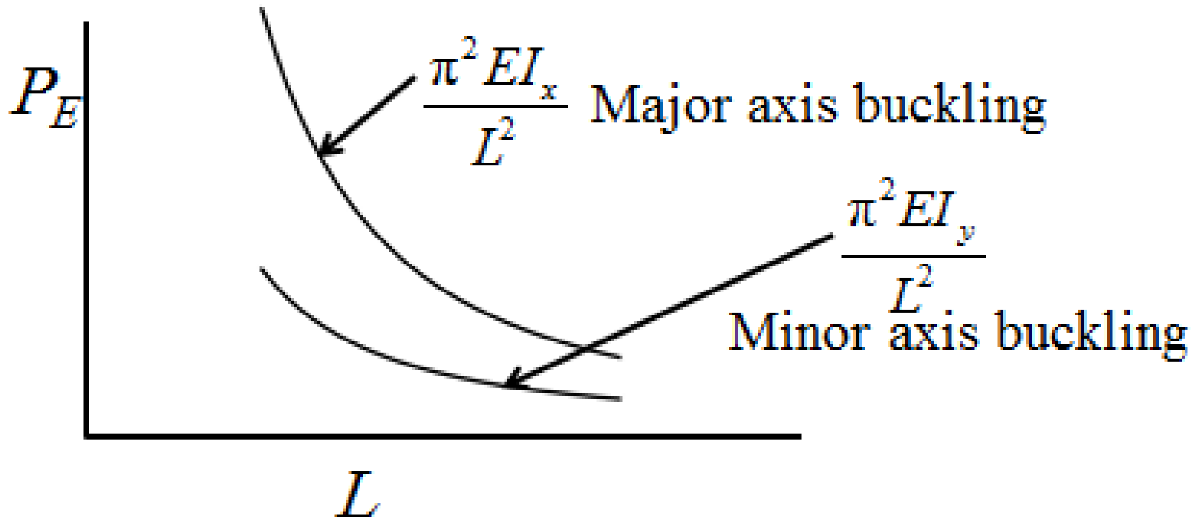
$$P_E = \frac{\pi^2 EI}{L^2}$$



This formula was derived in 1757 by Euler



Euler Buckling



- Dependent on I_{min} and L^2.
- Independent of F_y.

For similar unbraced length in each direction, "minor axis" (I_y in a W-shape) will control strength.

Euler Buckling

$$P_E = \frac{\pi^2 EI}{L^2}$$

Re-write in terms of stress:

divide by A,

$$P_E/A = \frac{\pi^2 EI}{AL^2}$$

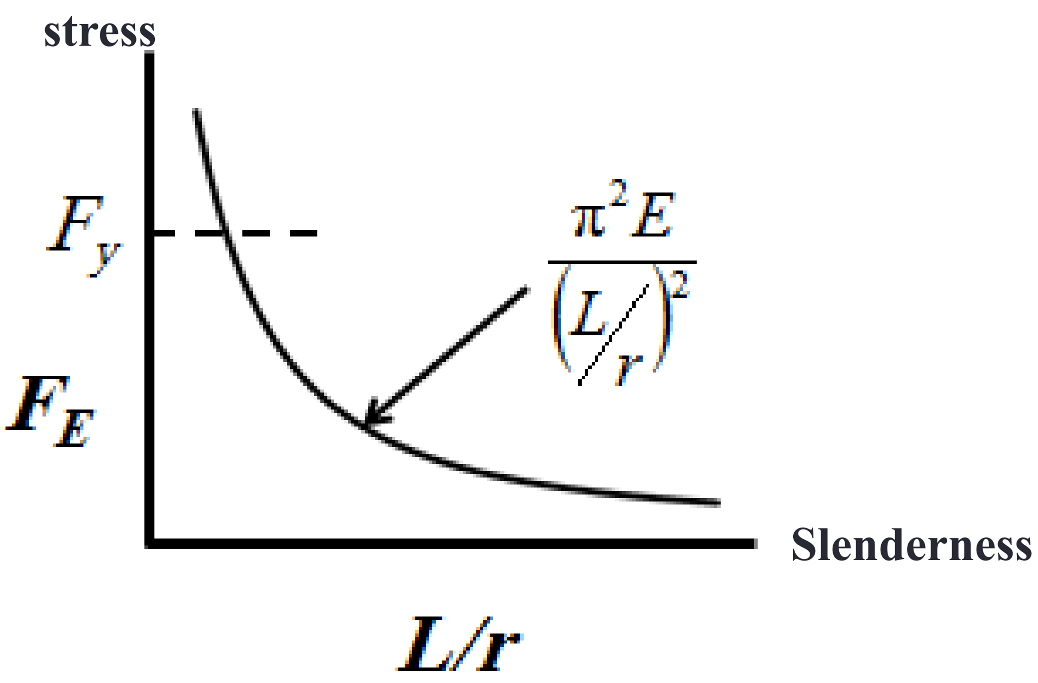
then with $r^2 = I/A$,

$$P_E/A = F_E = \frac{\pi^2 E}{\left(L/r\right)^2}$$

$F_E =$ Euler (elastic) buckling stress

$L/r =$ slenderness ratio

Euler Buckling



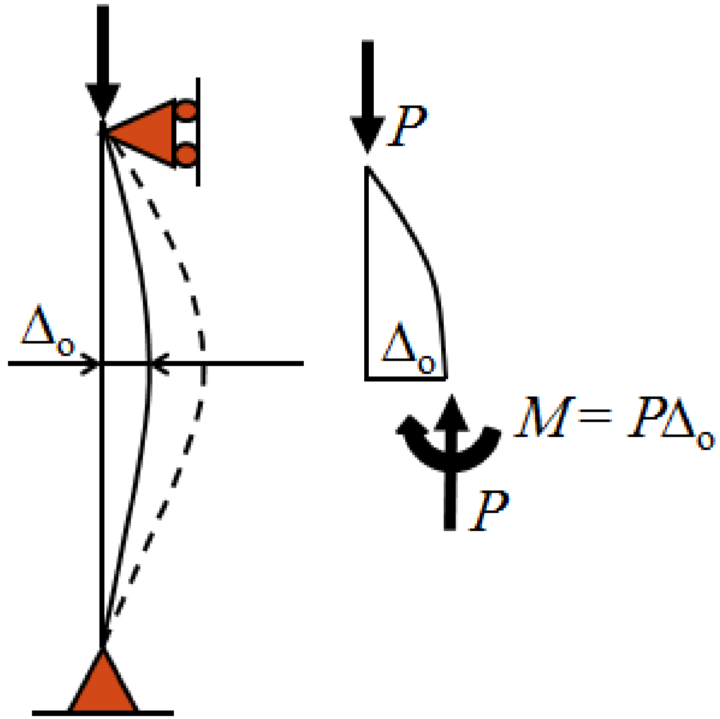
Buckling controlled by largest value of *L/r*.
Most slender section buckles first.

EULER ASSUMPTIONS (ACTUAL BEHAVIOR)

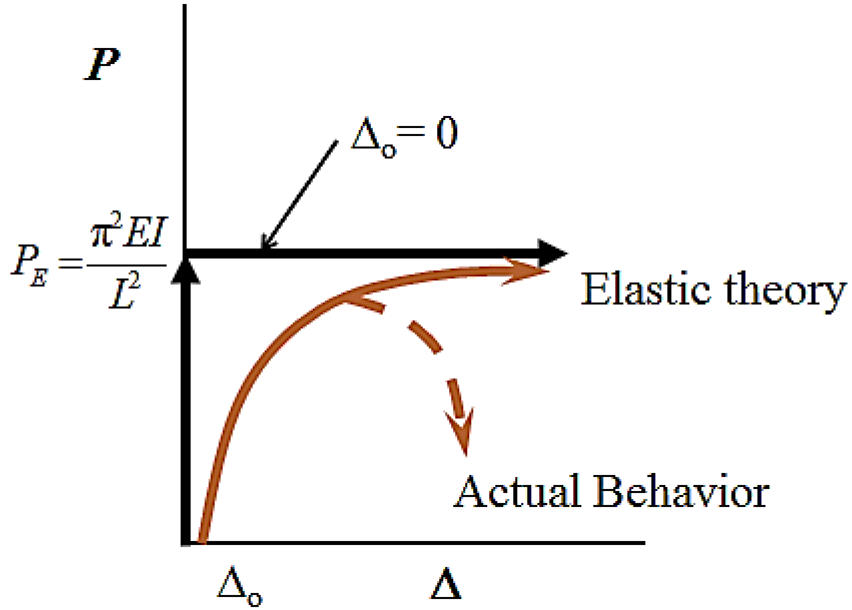
Initial Crookedness/Out of Straight

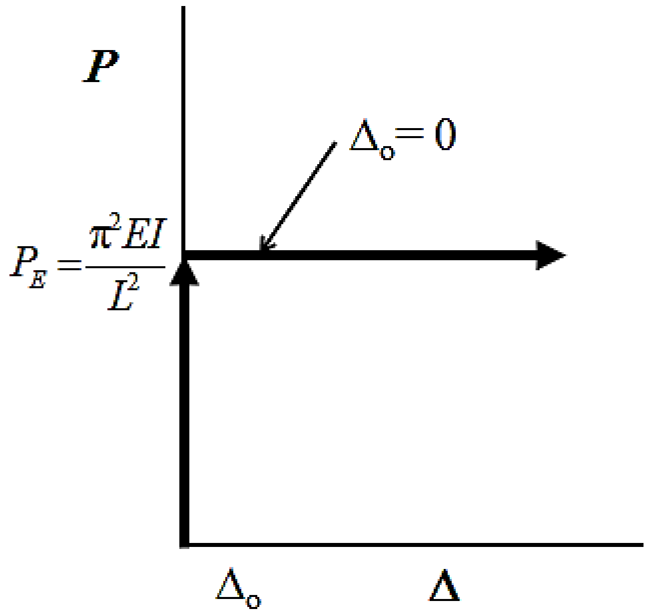
Δ_0 = initial mid-span deflection of column

- An initial displacement D_0, causes an initial moment along the length of the section.
- This is greatest at the location of maximum deflection.
- Yielding occurs from a combination of stresses due to moment and axial loads.



Initial Crookedness/Out of Straight





- Elastic theory then predicts the solid line.
- Actual behavior, is due to the additional effects of inelastic behavior.

Initial Crookedness/Out of Straight

Buckling is not instantaneous.

Additional stresses due to bending of the column,

$P/A \pm Mc/I$.

Assuming elastic material theory (never yields),

P approaches P_E.

Actually, some strength loss

small Δ_0 => small loss in strengths

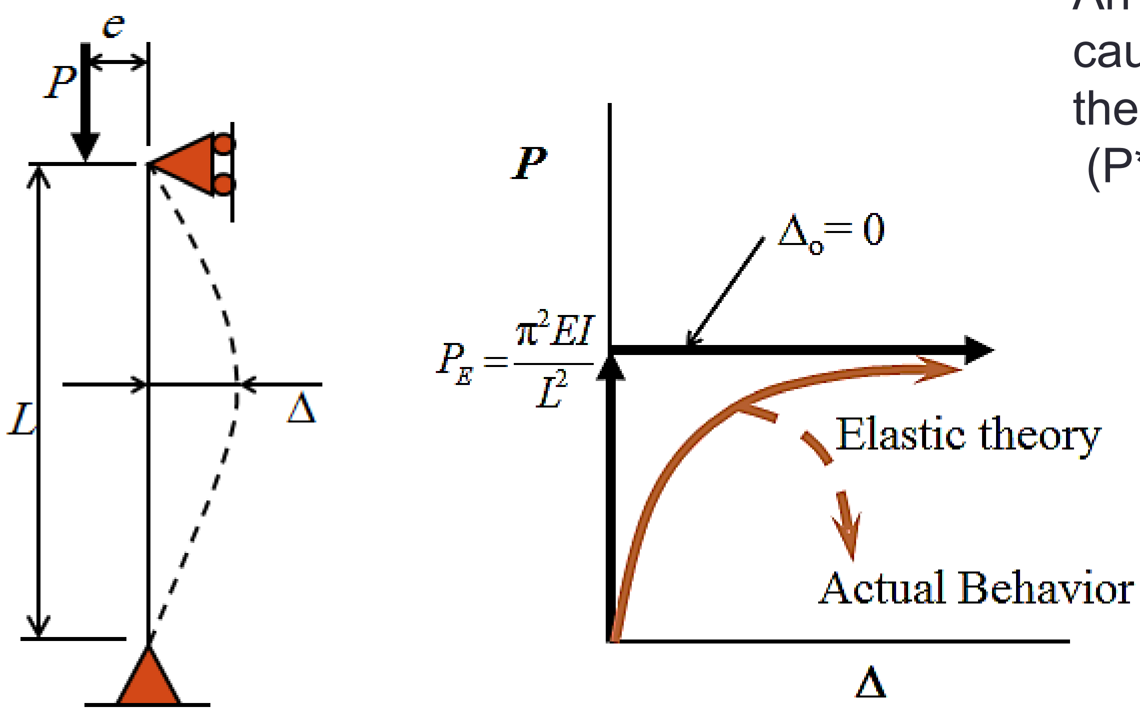
large Δ_0 => strength loss can be substantial

ASTM limits of Δ_0 = L/1000 or 0.25” in 20 feet

Typical values are Δ_0 = L/1500 or 0.15” in 20 feet

EULER ASSUMPTIONS (ACTUAL BEHAVIOR)

Load Eccentricity



An initial load eccentricity, e, causes an initial moment along the length of the section. (P*e) (at the top).

- This is a similar effect to that of an initial out-of-straightness, namely the introduction of a moment in addition to the purely axial loading.
- Yielding occurs from a combination of stresses due to moment and axial loads.

Load Eccentricity

Buckling is not instantaneous.

Additional stresses due to bending of the column,
$P/A \pm Mc/I$.

Assuming elastic material theory (never yields),
P approaches P_E.

Actually, some strength loss
small e=> small loss in strengths
large e=> strength loss can be substantial

If moment is "significant" section must be designed as a member subjected to combined loads.

EULER ASSUMPTIONS (ACTUAL BEHAVIOR)

End Restraint (Fixed)

Effective Length = KL

Length of equivalent pin ended column with similar elastic buckling load,

Distance between points of inflection in the buckled shape.

EULER ASSUMPTIONS (ACTUAL BEHAVIOR)

End Restraint (Fixed)

Set up equilibrium and solve similarly to Euler buckling derivation.

Determine a "K-factor."

$$P_e = \frac{\pi^2 EI}{(KL)^2}$$

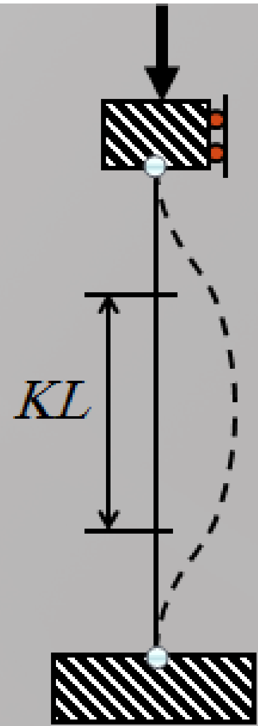
$$F_e = \frac{\pi^2 EI}{\left(\frac{KL}{r}\right)^2}$$

EXAMPLE

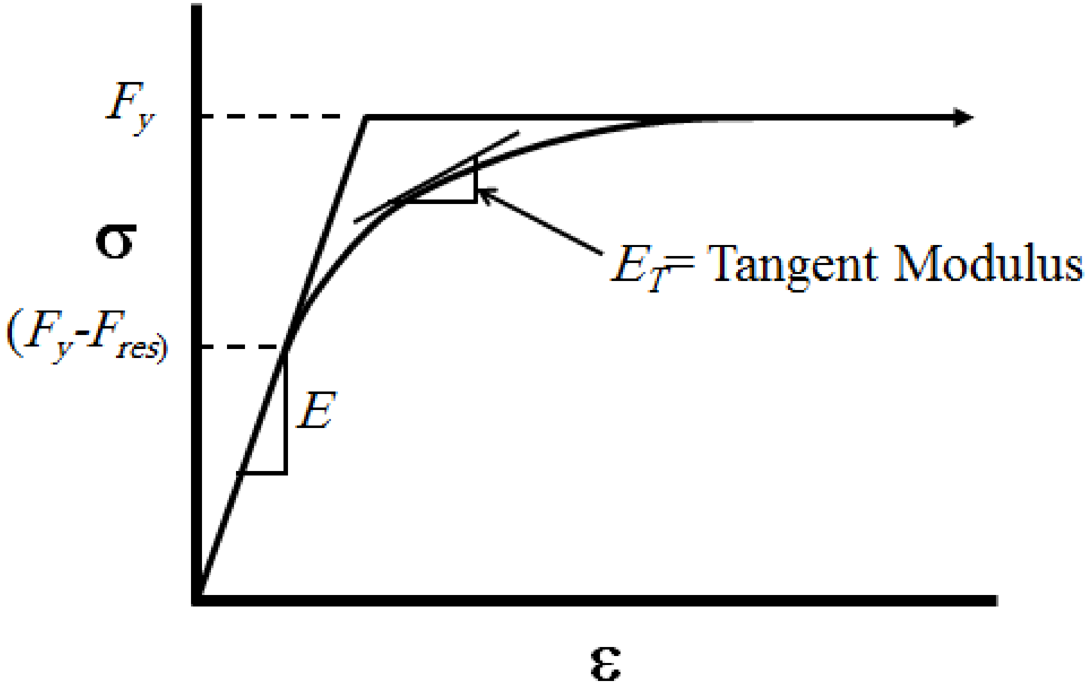
$$P_E = \frac{4\pi^2 EI}{L^2} = \frac{\pi^2 EI}{(1/2L)^2}$$

Similar to pin-pin, with $L' = L/2$.

Load Strength = 4 times as large.

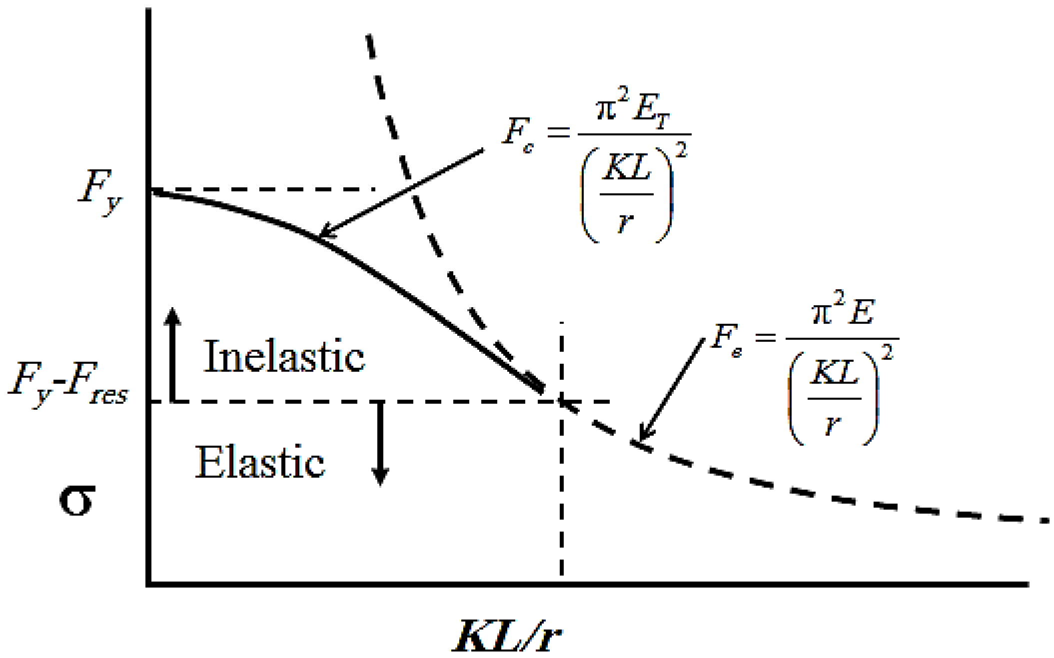


Inelastic Material Effects



- Inelastic material effects occur whenever axial stress in any portion of the cross section exceeds the first yield of the material.

- This also compounds effects of out of straightness and load eccentricity, as the bending moment term introduced also results in longitudinal stresses.



Inelastic action reduces column strength for lower values of KL/r. The maximum possible strength is the crushing limit, where all of the cross section attains F_y.

- Using tangent modulus theory, we can use a reduced modulus of elasticity, E_t in the Euler Buckling equation.

- E_t is obtained from a “Stub Column” (very short section in compression) test.

Summary

In general, the differences observed in testing of columns from Euler Buckling predicted capacities are as follows:

- ***Columns of low slenderness ratios are governed by inelastic buckling, and limited by crushing capacities.***
- ***Columns of high slenderness ratios are limited by out of straightness effects.***
- ***Columns of intermediate slenderness ratios see a combination of these effects.***

Slenderness Criteria

As Per Section E.2

Recommended to provide KL/r less than 200

Overall Column Strength

Major factors determining strength:

1) Slenderness (L/r).
2) End restraint (K factors).
3) Initial crookedness or load eccentricity.
4) Prior yielding or residual stresses.

The latter 2 items are highly variable between specimens.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

- **Local Buckling:** Criteria in Table B4.1

Local Buckling Criteria
Slenderness of the flange and web, λ, are used as criteria to determine whether local buckling might control in the elastic or inelastic range, otherwise the global buckling criteria controls.

For W-Shapes

FLB, $\lambda = b_f/2t_f$ $\qquad \lambda_{rf} = 0.56\sqrt{\dfrac{E}{F_y}}$

WLB, $\lambda = h/t_w$ $\qquad \lambda_{rw} = 1.49\sqrt{\dfrac{E}{F_y}}$

Cross-section Criteria

<u>Compact Section</u>: Stresses in the cross-section are elastic without local buckling onset in any elements.

- **Criteria of compact section:**
  - Flanges must be continuously connected to the web
  - Width-thickness ratios (λ) of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table B4.1

<u>Non- Compact Section</u>: Stresses in the cross-section are equal to yielding stresses without local buckling onset in any elements and the section does not resists any inelastic local buckling.

- **Criteria of non-compact section:**
  - If the width-thickness ratio (λ) of one or more compression elements exceeds λp, but does not exceed λr from Table B4.1

Cross-section Criteria

Slender Section: Elements are subjected to local buckling at stresses level lower than yielding stresses.

- **Criteria of slender section:**
  - If the width-thickness ratio (λ) of any element exceeds λr.

Compression Strength

Compression Members without Slender Elements E3-E7

$$P_n = F_{cr} A_g \tag{E3-1}$$

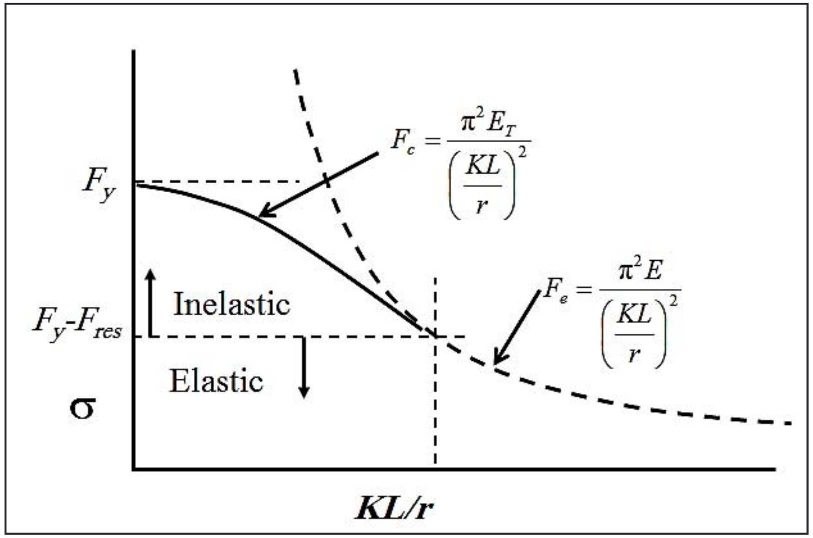
The *flexural buckling stress*, F_{cr}, is determined as follows:

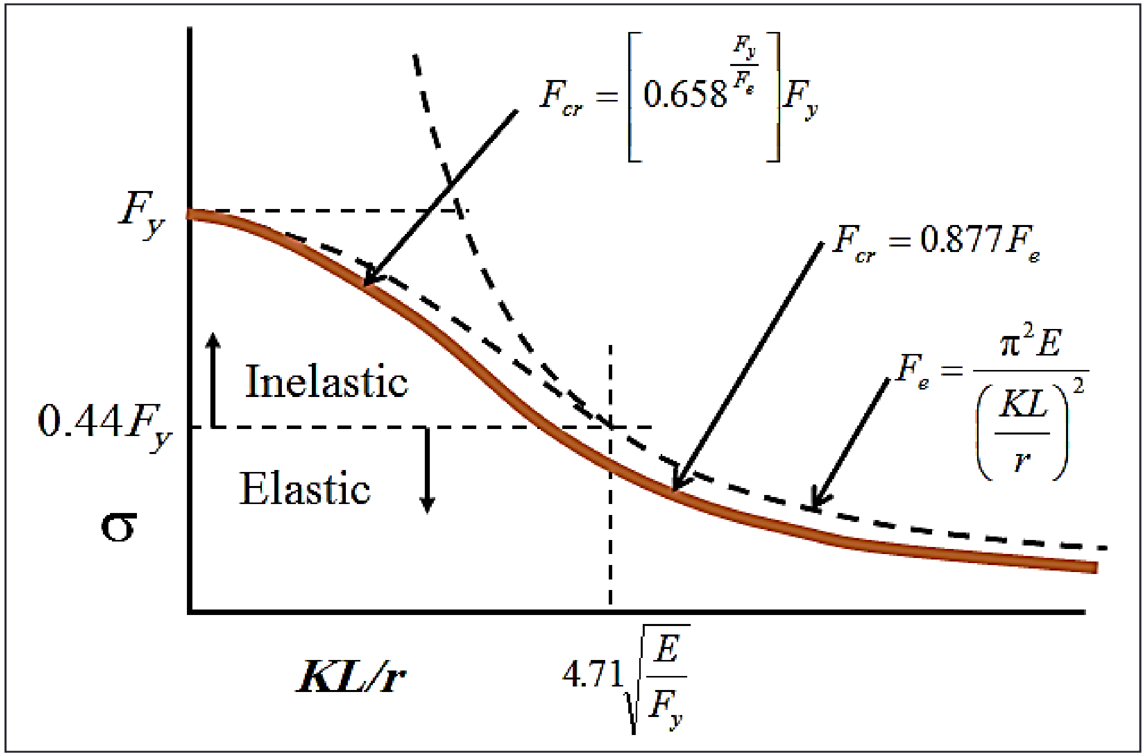
(a) When $\dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{F_y}}$ (or $F_e \geq 0.44 F_y$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \tag{E3-2}$$

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

$$F_{cr} = 0.877 F_e \tag{E3-3}$$


$F_c = \frac{\pi^2 E_T}{\left(\frac{KL}{r}\right)^2}$
F_y
Inelastic
F_y-F_{res}
$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$
Elastic
σ
KL/r


$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$
F_y
$F_{cr} = 0.877 F_e$
Inelastic
$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$
$0.44F_y$
Elastic
σ
KL/r
$4.71\sqrt{\frac{E}{F_y}}$

Compression Members with Slender Elements E7

$\lambda > \lambda_r$ "Slender Element"

Failure by local buckling occurs.
Covered in Section E7

Many rolled W-shape sections are dimensioned such that the full global criteria controls.

- **In general practically all W-shapes are non-slender as compression members, so this is covered as an advanced topic only.**
- **May control if high F_y, welded shapes, shapes not generally used for compression (such as angles, WT, etc.)**

(a) When $\dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{QF_y}}$ (or $F_e \geq 0.44QF_y$)

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

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}}$ (or $F_e < 0.44QF_y$)

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

Q = 1.0 for members with *compact* and *noncompact sections*, as defined in Section B4, for uniformly compressed elements

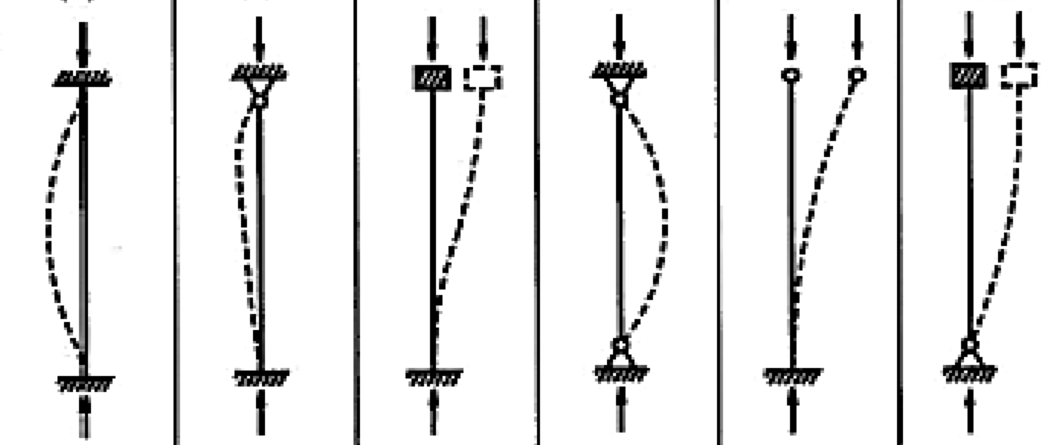
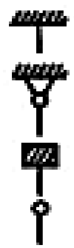
= $Q_s Q_a$ for members with *slender-element sections*, as defined in Section B4, for uniformly compressed elements.

User Note: For cross sections composed of only unstiffened slender elements, $Q = Q_s$ ($Q_a = 1.0$). For cross sections composed of only stiffened slender elements, $Q = Q_a$ ($Q_s = 1.0$). For cross sections composed of both stiffened and unstiffened slender elements, $Q = Q_s Q_a$.

Qs = **The reduction factor for slender** ***unstiffened*** ***elements, E7.1***

Qa = **The reduction factor for slender** ***stiffened*** ***elements, E7.2***

TABLE C-C2.2
Approximate Values of Effective Length Factor, *K*

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	(f)
						
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*					

Tutorial Class

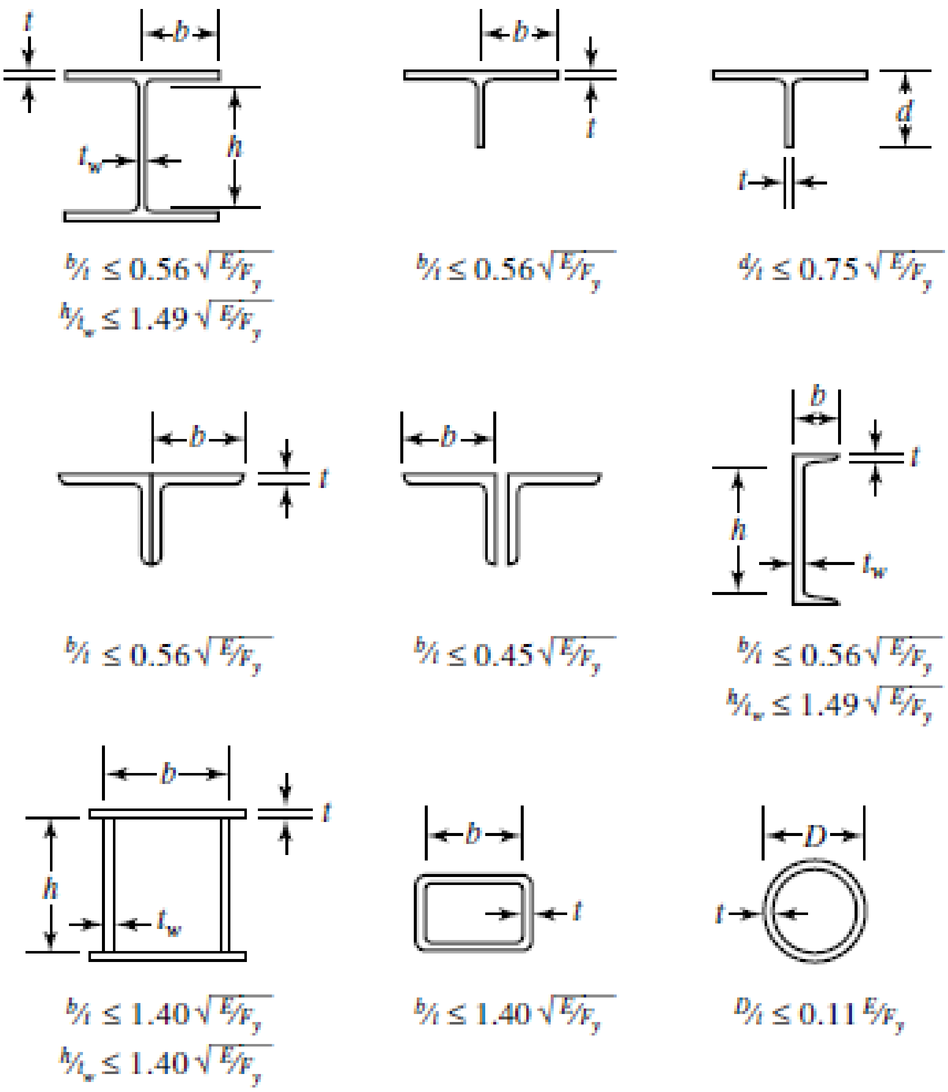
Module 3

Analysis and Design of Compression Members

Without-Slender Elements

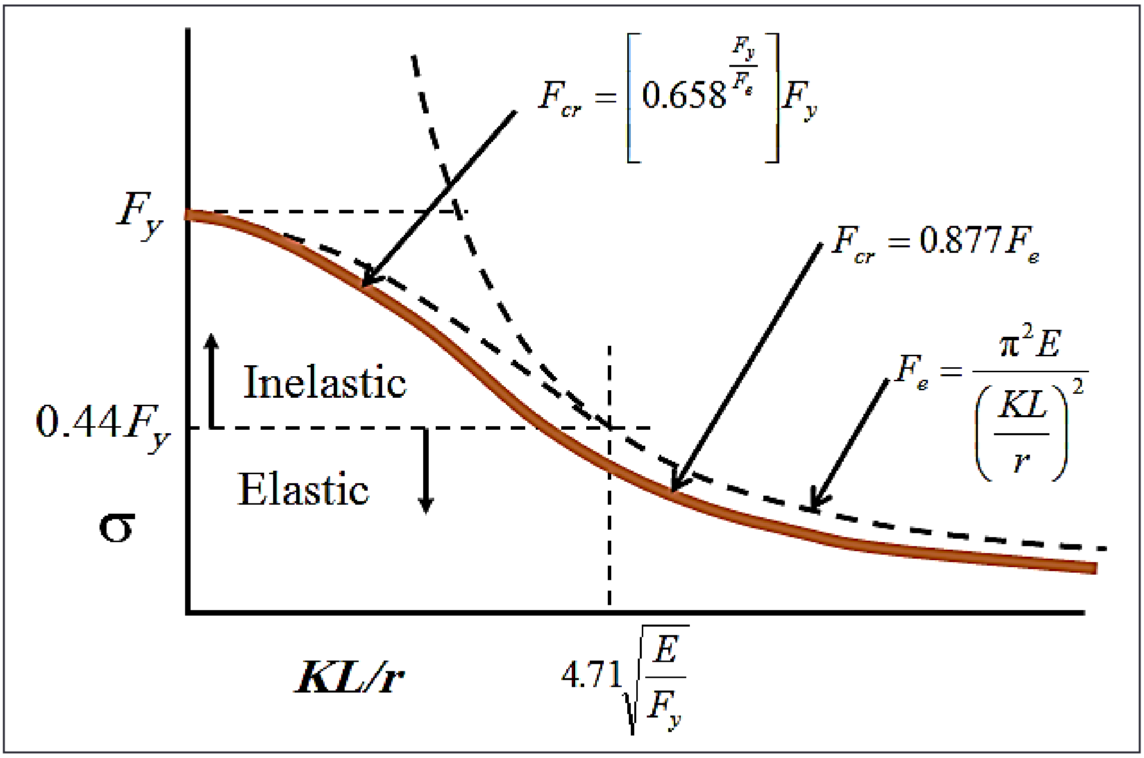
Local Stability
Cross-Sectional Stability

FIGURE 4.9



Reference, Segui, 5th Edition

Column Stability



Category: Solved Example
Topic: Analysis of Compression Members,
Reference: Example 4-2, Segui 5th Edition

A W14 × 74 of A992 steel has a length of 20 feet and pinned ends. Compute the design compressive strength for LRFD and the allowable compressive strength for ASD.

SOLUTION Slenderness ratio:

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

Check local stability

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.

Check overall stability

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

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

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

LRFD SOLUTION The design strength is

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

ASD SOLUTION From Equation 4.7, the allowable stress is

$$F_a = 0.6F_{cr} = 0.6(25.21) = 15.13 \text{ ksi}$$

The allowable strength is

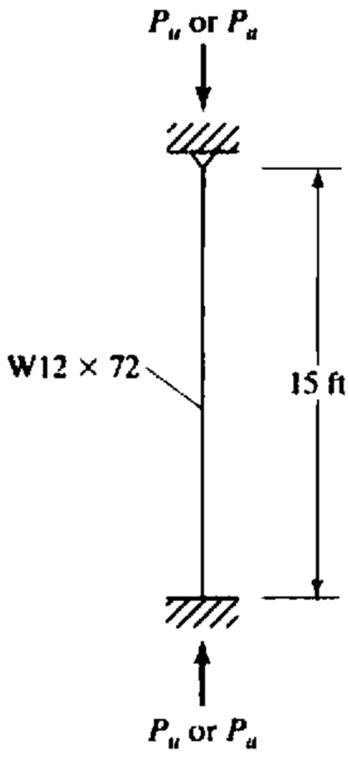
$$F_a A_g = 15.13(21.8) = 330 \text{ kips}$$

ANSWER Design compressive strength = 495 kips. Allowable compressive strength = 330 kips.

Category: Solved Example
Topic: Analysis of Compression Members,
Reference: Example 5-2, Mcormac 5th Edition

Determine LRFD design strength and the ASD allowable strength for the column shown in the Figure.

a. Using Table 4-22
b. Using Table 4-1
c. Using AISC critical stress method (Calculations Method)



(a) Using a W12 × 72 ($A = 21.1 \text{ in}^2$, $r_x = 5.31$ in, $r_y = 3.04$ in, $d = 12.3$ in, $b_f = 12.00$ in, $t_f = 0.670$ in, $k = 1.27$ in, $t_w = 0.430$ in)

$$\frac{b}{t} = \frac{12.00/2}{0.670} = 8.96 < 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000}{50}} = 13.49$$

∴ Nonslender unstiffened flange element

$$\frac{h}{t_w} = \frac{d - 2\text{k}}{t_w} = \frac{12.3 - 2(1.27)}{0.430} = 22.70 < 1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29{,}000}{50}} = 35.88$$

∴ Nonslender stiffened web element

$K = 0.80$ from Table 5.1.

Obviously, $(KL/r)_y > (KL/r)_x$ and thus controls

$$\left(\frac{KL}{r}\right)_y = \frac{(0.80)(12 \times 15) \text{ in}}{3.04 \text{ in}} = 47.37$$

By straight-line interpolation, $\phi_c F_{cr} = 38.19$ ksi, and $\frac{F_{cr}}{\Omega_c} = 25.43$ ksi from Table 4-22 in the Manual using $F_y = 50$ ksi steel.

LRFD	ASD
$\phi_c P_n = \phi_c F_{cr} A_g = (38.19)(21.1) = 805.8 \text{ k}$	$\frac{P_n}{\Omega_c} = \frac{F_{cr} A_g}{\Omega_c} = (25.43)(21.1) = 536.6 \text{ k}$

(b) Entering Table 4-1 in the Manual with KL (0.8)(15) = 12 ft

LRFD	ASD
$\phi_t P_n = 807$ k	$\dfrac{P_n}{\Omega_c} = 537$ k

(c) Elastic critical buckling stress

$$\left(\frac{KL}{r}\right)_y = 47.37 \quad \text{from part (a)}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(\pi^2)(29{,}000)}{(47.37)^2} = 127.55 \text{ ksi} \qquad \text{(AISC Equation E3-4)}$$

Flexural buckling stress F_{cr}

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 113.43 > \left(\frac{KL}{r}\right)_y = 47.37$$

$$\therefore F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{50}{127.55}}\right]50 = 42.43 \; ksi \qquad \text{(AISC Equation E3-2)}$$

LRFD $\phi_c = 0.90$	ASD $\Omega_c = 1.67$
$\phi_c F_{cr} = (0.90)(42.43) = 38.19$ ksi	$\dfrac{F_{cr}}{\Omega_c} = \dfrac{42.43}{1.67} = 25.41$ ksi
$\phi_c P_n = \phi_c F_{cr} A = (38.19)(21.1)$	$\dfrac{P_n}{\Omega_c} = \dfrac{F_{cr}}{\Omega_c} A = (25.41)(21.1)$
$= 805.8$ k	$= 536.2$ k

Tutorial Class

Module 3

Analysis and Design of Compression Members

With Slender Elements

Category: Solved Example

Topic: Analysis of Compression Members,

Reference: Example 4-4, Segui 5th Edition

Determine the axial compressive strength of an HSS 8 × 4 × 1/8 with an effective length of 15 feet with respect to each principal axis. Use $F_y = 46$ ksi.

From the dimensions and properties table in the *Manual*, the width-to-thickness ratio for the larger overall dimension is

$$\frac{h}{t} = 66.0$$

The ratio for the smaller dimension is

$$\frac{b}{t} = 31.5$$

From AISC Table B4.1a, Case 6 (and Figure 4.9 in this book), the upper limit for nonslender elements is

$$1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29,000}{46}} = 35.15$$

Since $h/t > 1.40\sqrt{E/F_y}$, the larger dimension element is slender and the local buckling strength must be computed. (Although the limiting width-to-thickness ratio is labeled b/t in the table, that is a generic notation, and it applies to h/t as well.)

Because this cross-sectional element is a stiffened element, $Q_s = 1.0$, and Q_a must be computed from AISC Section E7.2. The shape is a rectangular section of uniform thickness, with

$$\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}},$$

So AISC E7.2 (b) applies, where

$$f = \frac{P_n}{A_e}$$

and A_e is the reduced effective area. The Specification user note for square and rectangular sections permits a value of $f = F_y$ to be used in lieu of determining f by iteration. From AISC Equation E7-18, the effective width of the slender element is

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

For the 8-inch side, using $f = F_y$ and the *design* thickness* from the dimensions and properties table,

$$b_e = 1.92(0.116)\sqrt{\frac{29,000}{46}}\left[1 - \frac{0.38}{(66.0)}\sqrt{\frac{29,000}{46}}\right] = 4.784 \text{ in.}$$

From AISC B4.1(b) and the discussion in Part 1 of the *Manual*, the unreduced length of the 8-inch side between the corner radii can be taken as

$$b = 8 - 3t = 8 - 3(0.116) = 7.652 \text{ in.}$$

where the corner radius is taken as 1.5 times the design thickness.

The total loss in area is therefore

$$2(b - b_e)t = 2(7.652 - 4.784)(0.116) = 0.6654 \text{ in.}^2$$

and the reduced area is

$$A_e = 2.70 - 0.6654 = 2.035 \text{ in.}^2$$

The reduction factor is

$$Q_a = \frac{A_e}{A_g} = \frac{2.035}{2.70} = 0.7537$$

$$Q = Q_s Q_a = 1.0(0.7537) = 0.7537$$

Compute the local buckling strength:

$$4.71\sqrt{\frac{E}{QF_y}} = 4.71\sqrt{\frac{29{,}000}{0.7537(46)}} = 136.2$$

$$\frac{KL}{r} = 105.3 < 136.2 \qquad \therefore \text{ Use AISC Equation E7-2}$$

$$F_{cr} = Q\left(0.658^{\frac{QF_y}{F_e}}\right)F_y = 0.7537\left(0.658^{\frac{0.7537(46)}{25.81}}\right)46 = 19.76 \text{ ksi}$$

$$P_n = F_{cr}A_g = 19.76(2.70) = 53.35 \text{ kips}$$

Since this is less than the flexural buckling strength of 58.91 kips, local buckling controls.

Design strength $= \phi_c P_n = 0.90(53.35) = 48.0$ kips

Allowable strength $= \dfrac{P_n}{\Omega} = \dfrac{53.35}{1.67} = 32.0$ kips

(Allowable stress $= 0.6F_{cr} = 0.6(19.76) = 11.9$ ksi)

Design of Compression Members

Selection of an economical section to resist a given compressive load

Procedures for Design of Compressive Strength:

1. Design of Columns using AISC Formulas:

Involves a trial and error process. The principal is to assume or try a shape and then check the compressive strength. If the strength is too small (unsafe) or too large (uneconomical), another trial must be made.

The LRFD design stress ($\Phi c\ Fcr$) and the ASD allowable stress $Fcr/\ \Omega c$ are not known until a column size is selected, and vice versa.

A column size may be assumed, the r values for that section obtained from the Manual or calculated, and the design stress found by substituting into the appropriate column formula.

It may then be necessary to try a larger or smaller section.

Design of Columns using AISC Formulas

The LRFD design stress (Φ_c *Fcr)* and the ASD allowable stress *Fcr/ Ω*c are not known until a column size is selected, and vice versa.

Procedure:

- **Assume a value for the critical buckling stress *Fcr.***

Examination of AISC Equations E3-2 and E3-3 shows that the theoretically maximum value of Fcr is the yield stress Fy, practically 1/2 to 2/3 Fy.

- **Determine the required area**
- **Select a shape that satisfies the area requirement**
- **Compute Fcr and the strength for the trial shape**.
- **Revise if necessary.**

If the available strength is very close to the required value, the next tabulated size can be tried. Otherwise, repeat the entire procedure.

- **Check local stability**

check the width-to-thickness ratios

- **Revise if necessary.**

Design of Compression Members

2. Design of Columns using Design Tables

Design Tables

Tables 4-1 to 4-20 $\phi_c P_n$ as a function of KL_y	Can be applied to KL_x by dividing KL_y by r_x/r_y.
Table 4-22 $\phi_c F_{cr}$ as a function of KL/r	Useful for all shapes. Larger KL/r value controls.

Tutorial Class

Module 3
Analysis and Design of Compression Members

Design of Compression Members

Category: Solved Example

Topic: Design of Compression Members,

Reference: Example 4-8, Segui, 5th Edition

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.

LRFD SOLUTION

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

$$A_g = 20.9 \text{ in.}^2 > 20.2 \text{ in.}^2 \qquad \text{(OK)}$$

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

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

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

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

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

$$\phi_c P_n = \phi_c F_{cr} A_g = 0.90(7.455)(20.9) = 140 \text{ kips} < 600 \text{ kips} \qquad \text{(N.G.)}$$

Because the initial estimate of F_{cr} was so far off, assume a value about halfway between 33 and 7.455 ksi. Try $F_{cr} = 20$ 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:

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

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

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

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

$$F_{cr} = 0.877F_e = 0.877(21.27) = 18.65 \text{ ksi}$$
$$\phi_c P_n = \phi_c F_{cr} A_g = 0.90(18.65)(35.1) = 589 \text{ kips} < 600 \text{ kips} \qquad \text{(N.G.)}$$

This is very close, so try the next larger size.

Try a W18 × 130:

$$A_g = 38.3 \text{ in.}^2$$
$$\frac{KL}{r_{min}} = \frac{26 \times 12}{2.70} = 115.6 < 200 \qquad \text{(OK)}$$
$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29{,}000)}{(115.6)^2} = 21.42 \text{ ksi}$$

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

$$F_{cr} = 0.877F_e = 0.877(21.42) = 18.79 \text{ ksi}$$
$$\phi_c P_n = \phi_c F_{cr} A_g = 0.90(18.79)(38.3) = 648 \text{ kips} > 600 \text{ kips} \qquad \text{(OK.)}$$

This shape is not slender (there is no footnote in the dimensions and properties table to indicate that it is), so local buckling does not have to be investigated.

ANSWER Use a W18 × 130.

ASD SOLUTION

The ASD solution procedure is essentially the same as for LRFD, and the same trial values of F_{cr} will be used here.

$$P_a = D + L = 100 + 300 = 400 \text{ kips}$$

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

$$\text{Required } A_g = \frac{P_a}{0.6F_{cr}} = \frac{400}{0.6(33)} = 20.2 \text{ in.}^2$$

Try a W18 × 71:

$$A_g = 20.9 \text{ in.}^2 > 20.2 \text{ in.}^2 \qquad \text{(OK)}$$

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

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

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

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

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

$$\frac{P_n}{\Omega_c} = 0.6F_{cr}A_g = 0.6(7.455)(20.9) = 93.5 \text{ kips} < 400 \text{ kips} \qquad \text{(N.G.)}$$

Because the initial estimate of F_{cr} was so far off, assume a value about halfway between 33 and 7.455 ksi. Try $F_{cr} = 20$ ksi.

$$\text{Required } A_g = \frac{P_a}{0.6F_{cr}} = \frac{400}{0.6(20)} = 33.3 \text{ in.}^2$$

Try a W18 × 119:

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

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

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

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

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

$$0.6F_{cr}A_g = 0.6(18.65)(35.1) = 393 \text{ kips} < 400 \text{ kips} \quad \text{(N.G.)}$$

This is very close, so try the next larger size.

Try a W18 × 130:

$$A_g = 38.3 \text{ in.}^2$$

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

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

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

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

$$0.6F_{cr}A_g = 0.6(18.79)(38.3) = 432 \text{ kips} < 400 \text{ kips} \quad \text{(OK)}$$

This shape is not slender (there is no footnote in the dimensions and properties table to indicate that it is), so local buckling does not have to be investigated.

ANSWER Use a W18 × 130.

Category: Solved Example

Topic: Design of Compression Members,

Reference: Example 6-1, Mcormac, 5th Edition

Using $F_y = 50$ ksi, select the lightest W14 available for the service column loads $P_D = 130$ k and $P_L = 210$ k. $KL = 10$ ft.

Solution

LRFD	ASD
$P_u = (1.2)(130\text{ k}) + (1.6)(210\text{ k}) = 492\text{ k}$	$P_a = 130\text{ k} + 210\text{ k} = 340\text{ k}$
Assume $\dfrac{KL}{r} = 50$	Assume $\dfrac{KL}{r} = 50$
Using $F_y = 50$ ksi steel	Using $F_y = 50$ ksi steel
$\phi_c F_{cr}$ from AISC Table 4-22 $= 37.5$ ksi	$\dfrac{F_{cr}}{\Omega_c} = 24.9$ ksi (AISC Table 4-22)
$A\text{ Reqd} = \dfrac{P_u}{\phi_c F_{cr}} = \dfrac{492\text{ k}}{37.5\text{ ksi}} = 13.12\text{ in}^2$	$A\text{ Reqd} = \dfrac{P_a}{F_{cr}/\Omega} = \dfrac{340\text{ k}}{24.9\text{ ksi}} = 13.65\text{ in}^2$
Try W14 × 48 ($A = 14.1\text{ in}^2$, $r_x = 5.85$ in, $r_y = 1.91$ in)	Try W14 × 48 ($A = 14.1\text{ in}^2$, $r_x = 5.85$ in, $r_y = 1.91$ in)
$\left(\dfrac{KL}{r}\right)_y = \dfrac{(12\text{ in/ft})(10\text{ ft})}{1.91\text{ in}} = 62.83$	$\left(\dfrac{KL}{r}\right)_y = \dfrac{(12\text{ in/ft})(10\text{ ft})}{1.91\text{ in}} = 62.83$
$\phi_c F_{cr} = 33.75$ ksi from AISC Table 4-22	$\dfrac{F_{cr}}{\Omega_c} = 22.43$ ksi from AISC Table 4-22
$\phi_c P_n = (33.75\text{ ksi})(14.1\text{ in}^2) = 476\text{ k} < 492\text{ k}$ N.G.	$\dfrac{P_n}{\Omega_c} = (22.43\text{ ksi})(14.1\text{ in}^2) = 316\text{ k} < 340\text{ k}$ N.G.
Try next larger section W14 × 53 ($A = 15.6\text{ in}^2$, $r_y = 1.92$ in)	Try next larger section W14 × 53 ($A = 15.6\text{ in}^2$, $r_y = 1.92$ in).
$\left(\dfrac{KL}{r}\right)_y = \dfrac{(12\text{ in/ft})(10\text{ ft})}{1.92\text{ in}} = 62.5$	$\left(\dfrac{KL}{r}\right)_y = \dfrac{(12\text{ in/ft})(10\text{ ft})}{1.92\text{ in}} = 62.5$
$\phi_c F_{cr} = 33.85$ ksi	$\dfrac{F_{cr}}{\Omega_c} = 22.5$ ksi
$\phi_c P_n = (33.85\text{ ksi})(15.6\text{ in}^2) = 528\text{ k} > 492\text{ k}$ **OK**	$\dfrac{P_n}{\Omega_c} = (22.5\text{ ksi})(15.6\text{ in}^2) = 351\text{ k} > 340\text{ k}$ **OK**
Use W14 × 53.	**Use W14 × 53.**

Category: Solved Example
Topic: Design of Compression Members,
Reference: Example 6-2, Mcormac, 5th Edition

Example 6-2

Use the AISC column tables (both LRFD and ASD) for the designs to follow.

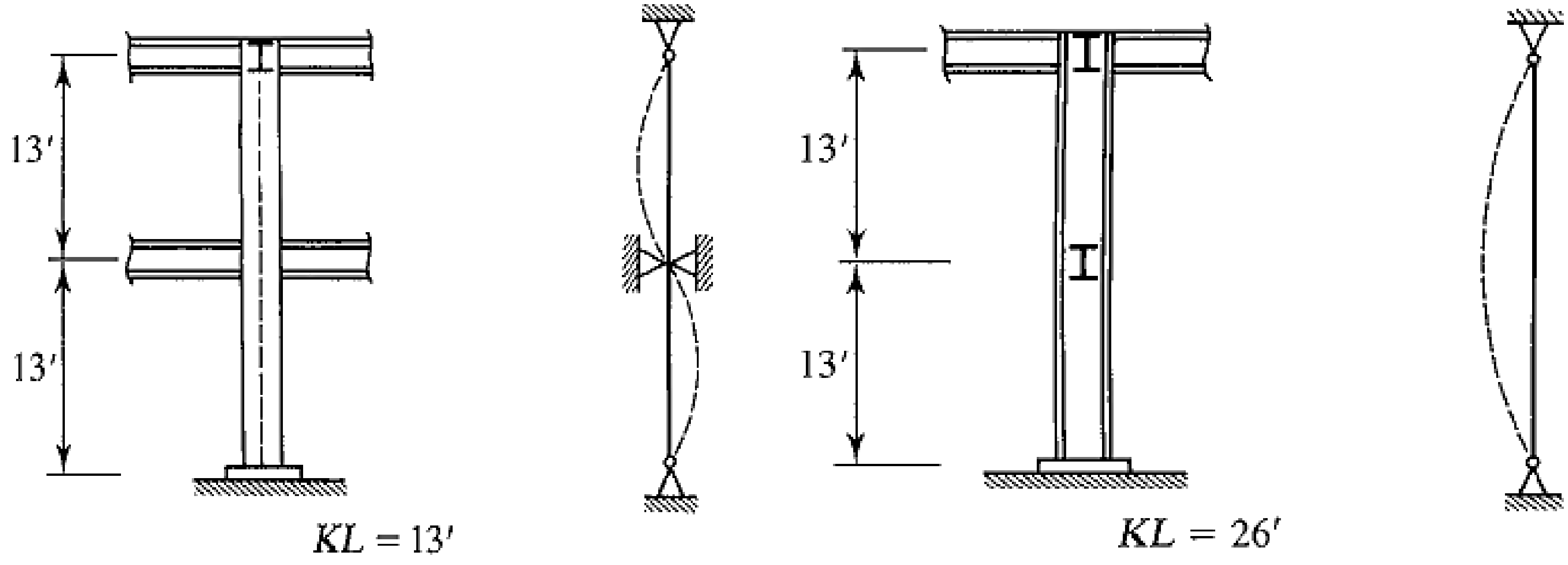
(a) Select the lightest W section available for the loads, steel, and KL of Example 6-1. $F_y = 50$ ksi.
(b) Select the lightest satisfactory rectangular or square HSS sections for the situation in part (a). $F_y = 46$ ksi.
(c) Select the lightest satisfactory round HSS section, $F_y = 42$ ksi for the situation in part (a).
(d) Select the lightest satisfactory pipe section, $F_y = 35$ ksi, for the situation in part (a).

LRFD	ASD
(a) W8 × 48 ($\phi_c P_n = 497\text{ k} > 492\text{ k}$) from Table 4-1	(a) W10 × 49 $\left(\frac{P_n}{\Omega_c} = 366\text{ k} > 340\text{ k}\right)$ from Table 4-1
(b) Rectangular HSS	(b) Rectangular HSS
HSS 12 × 8 × $\frac{3}{8}$ @ 47.8 #/ft ($\phi_c P_n = 499\text{ k} > 492\text{ k}$) from Table 4-3	HSS 12 × 10 × $\frac{3}{8}$ @ 52.9 #/ft $\left(\frac{P_n}{\Omega_c} = 379\text{ k} > 340\text{ k}\right)$ from Table 4-3
Square HSS	Square HSS
HSS 10 × 10 × $\frac{3}{8}$ @ 47.8 #/ft ($\phi_c P_n = 513\text{ k} > 492\text{ k}$) from Table 4-4	** HSS 12 × 12 × $\frac{5}{16}$ @ 48.8 #/ft $\left(\frac{P_n}{\Omega_c} = 340\text{ k} = 340\text{ k}\right)$ from Table 4-4

LRFD	ASD
(c) Round HSS 16.000 × 0.312 @ 52.3 #/ft ($\phi_c P_n = 529 \text{ k} > 492 \text{ k}$) from Table 4-5	(c) Round HSS 16.000 × 0.312 @ 52.3 #/ft $\left(\frac{P_n}{\Omega_c} = 352 \text{ k} > 340 \text{ k}\right)$ from Table 4-5
(d) XS Pipe 12 @ 65.5 #/ft ($\phi_c P_n = 530 \text{ k} > 492 \text{ k}$) from Table 4-6	(d) XS Pipe 12 @ 65.5 #/ft $\left(\frac{P_n}{\Omega_c} = 353 \text{ k} > 340 \text{ k}\right)$ from Table 4-6

Compression members having different slenderness

If a compression member is supported differently with respect to each of its principal axes, the effective length will be different for the two directions



(a) Minor Axis Buckling

(b) Major Axis Buckling

Different slenderness? Bracing?

- For some columns. particularly the long columns, bracing is supplied perpendicular to the weak axis, thus reducing the slenderness or the length free to buckle in that direction.

- Bracing may be accomplished by framing braces or beams into the sides of a column.

- These members prevent translation of the column in all directions, but the connections, permit small rotations to take place.

- **Under these conditions, the member can be treated as pin-connected**.

➢ **Because its strength decreases with increasing KL/r, a column will buckle in the direction corresponding to the largest slenderness ratio**

Bracing Members

- Bracing members must be capable of providing the necessary lateral forces. Without buckling themselves.

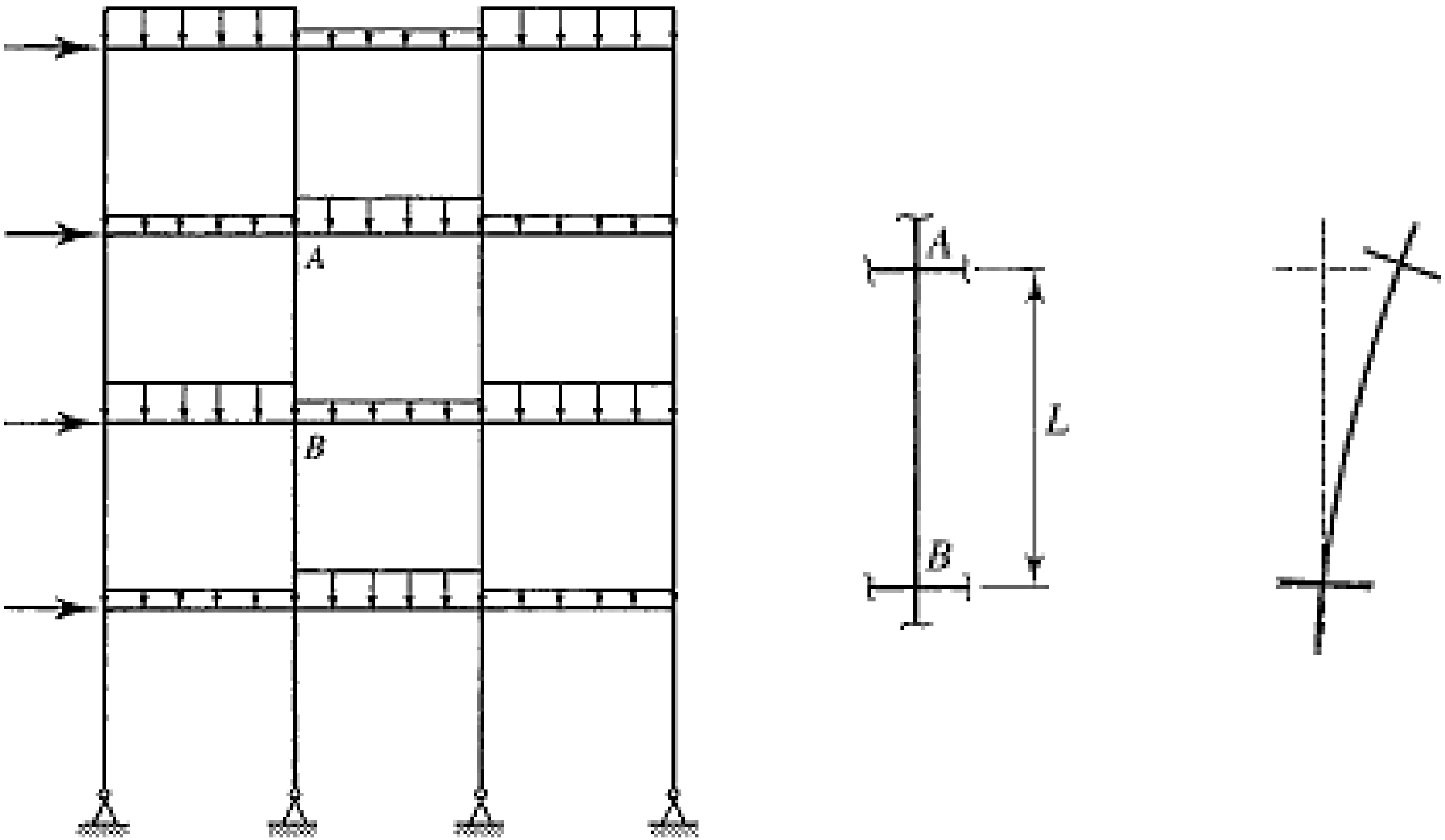
- The forces to be taken are quite small and are often conservatively estimated to equal 0.02 times the column design loads.

- These members can be selected as are other compression members.

- A bracing member must be connected to other members that can transfer the horizontal force by shear to the next restrained level.

- If this is not done, little lateral support will be provided for the original column

Columns that are part of a continuous frame



Side-sway effect on columns buckling

Side-sway is the horizontal displacements of the frame

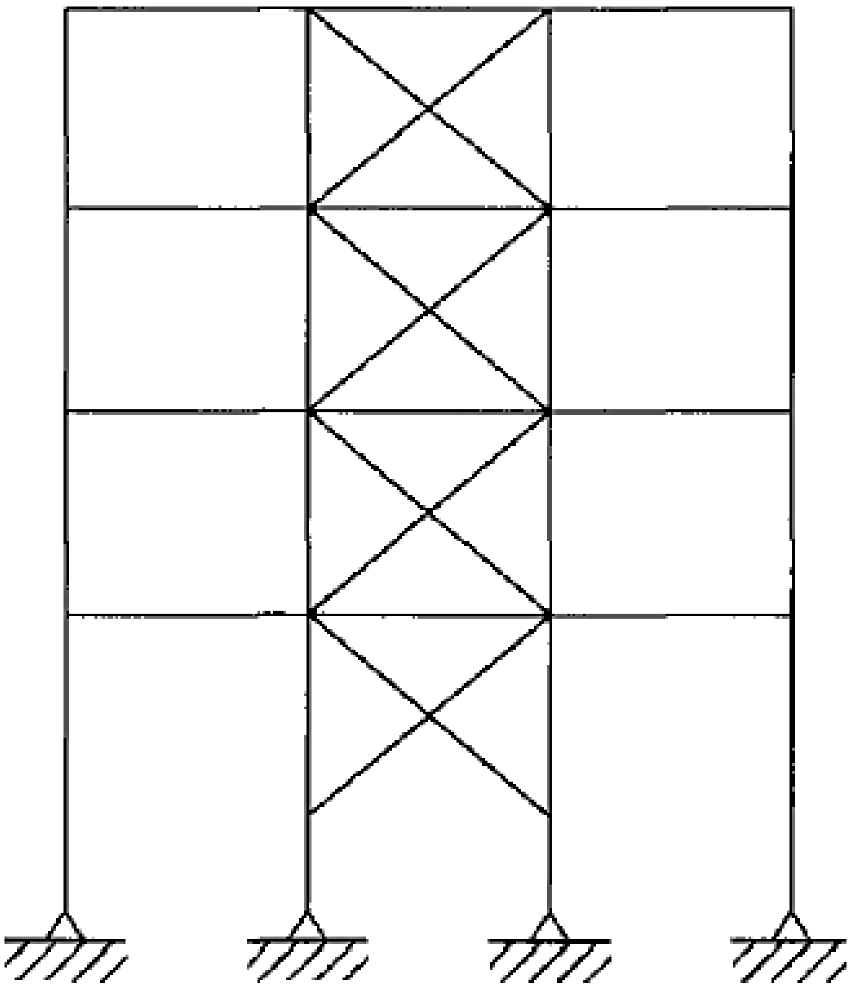
In statically indeterminate structures, sidesway occurs where:

- The frames deflect laterally due to the presence of lateral loads or unsymmetrical vertical loads;

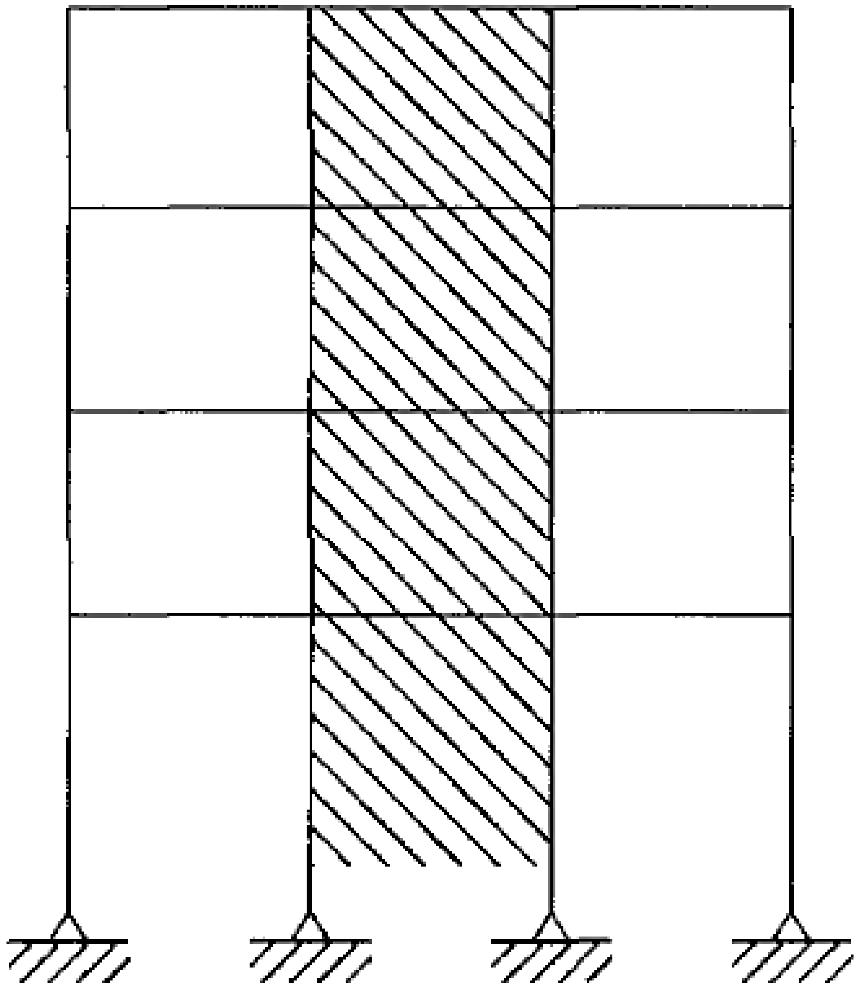
- The frames themselves are unsymmetrical.

- Sidesway also occurs in columns whose ends can move transversely when they are loaded to the point that buckling occurs.

How to prevent Side-sway?



(a) Diagonal bracing



(b) In-fill shear wall

Effective Length of continuous columns

- The rotational restraint provided by the beams, or girders, at the end of a column is a function of the rotational stiffness of the members intersecting at the joint.

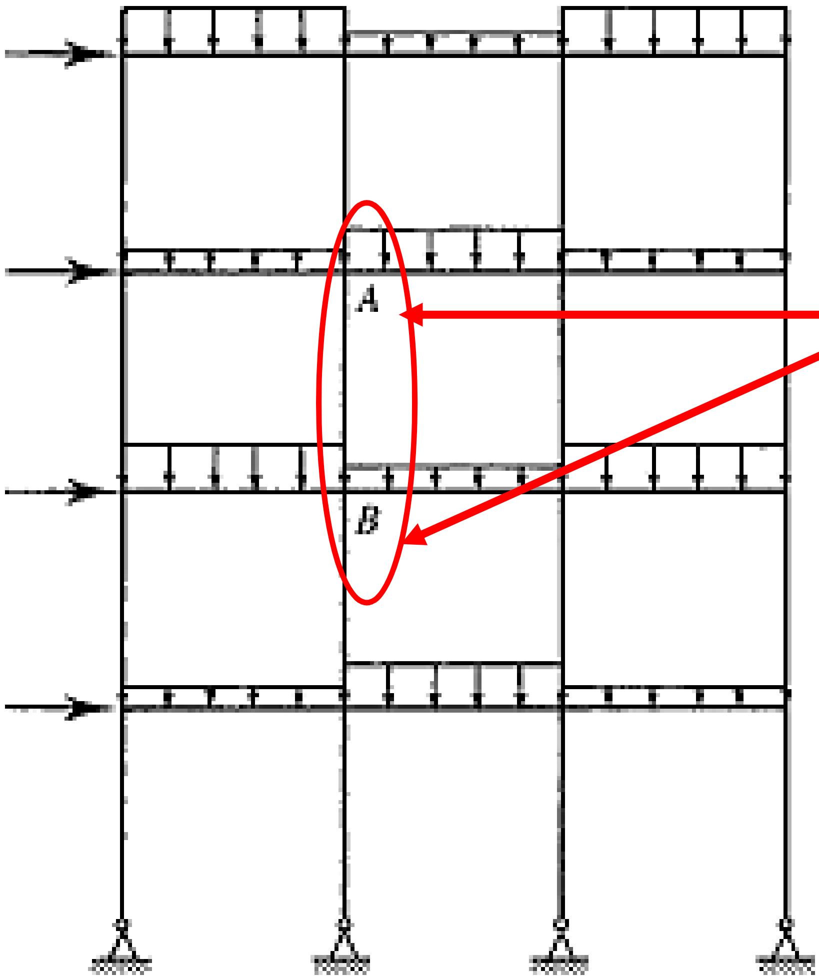
- The rotational stiffness of a member is proportional to *EI/L*,

where I is the moment of inertia of the cross section with respect to the axis of bending.

- **Gaylord, and Stallmeyer (1992)** show that the effective length factor K depends on the ratio of column stiffness to girder stiffness at each end of the member.

$$G = \frac{\sum \frac{4EI}{L} \text{ for columns}}{\sum \frac{4EI}{L} \text{ for girders}} = \frac{\sum \frac{E_c I_c}{L_c}}{\sum \frac{E_g I_g}{L_g}}.$$

- **Rotational stiffness works out to be equal to 4EI/L, for a homogeneous member of constant cross section.**



The subscripts A and B refer to the joints at the ends of the columns being considered. G is defined as:

$$G = \frac{\sum\left(\frac{E_c I_c}{L_c}\right)}{\sum\left(\frac{E_g I_g}{L_g}\right)} \quad \text{AISC Equation (C-A-7-2)}$$

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

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

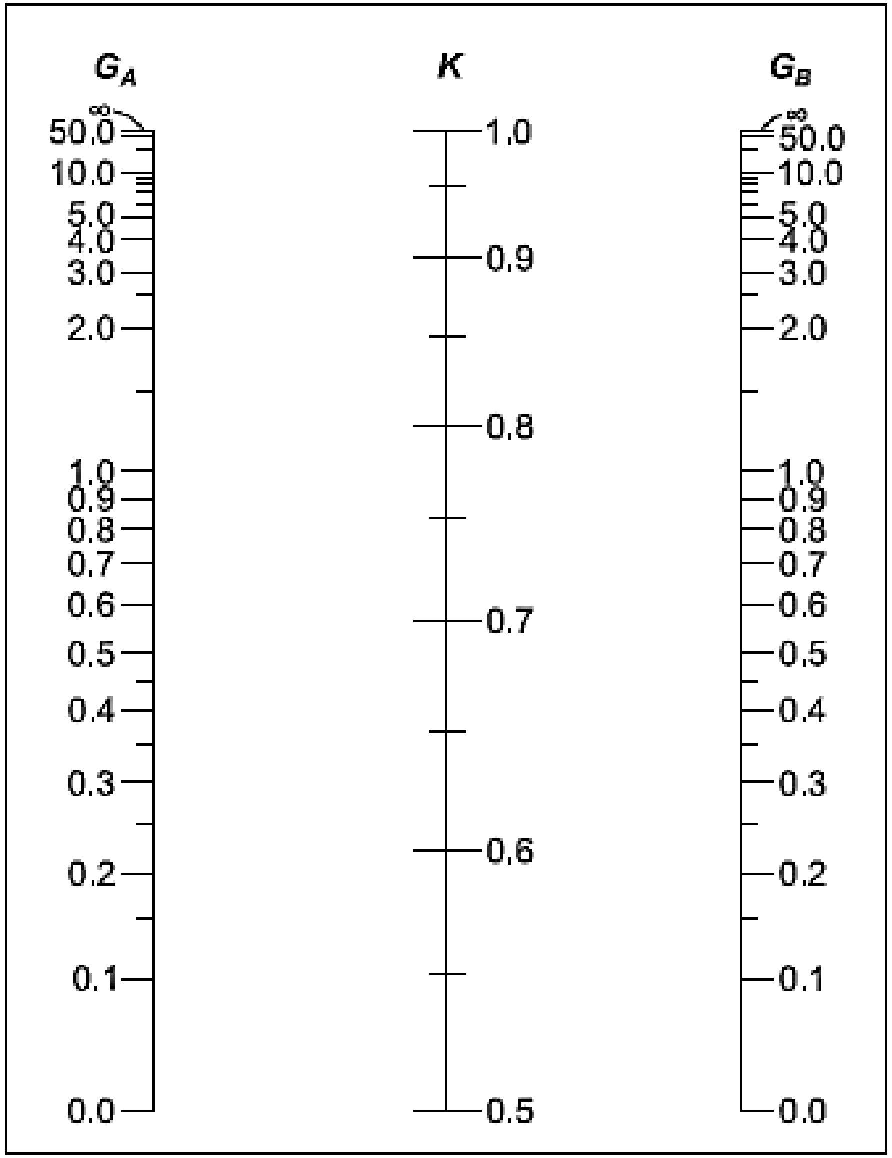
$E_c = E_g = E$, the modulus of elasticity of structural steel.

Alignment Chart,

Jackson–Mooreland Alignment Charts *(Johnston, 1976)* (nomograms)

Braced Frame

The effective length factor obtained in this manner is with respect to the axis of bending, which is the axis perpendicular to the plane of the frame.



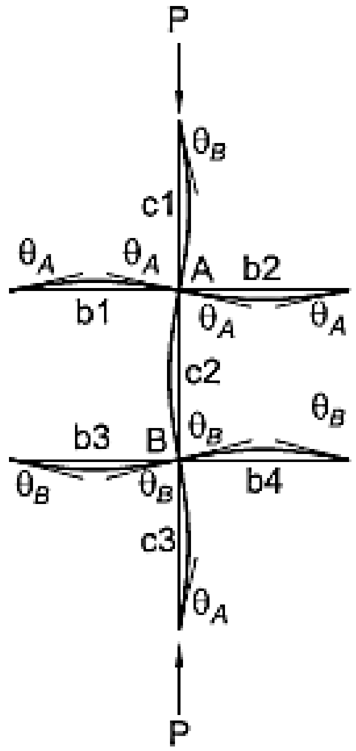


Fig. C-C2.3. Alignment chart—sidesway inhibited (braced frame).

Alignment Chart,

Jackson–Mooreland Alignment Charts *(Johnston, 1976)* (nomograms)

Unbraced Frame

The effective length factor obtained in this manner is with respect to the axis of bending, which is the axis perpendicular to the plane of the frame.

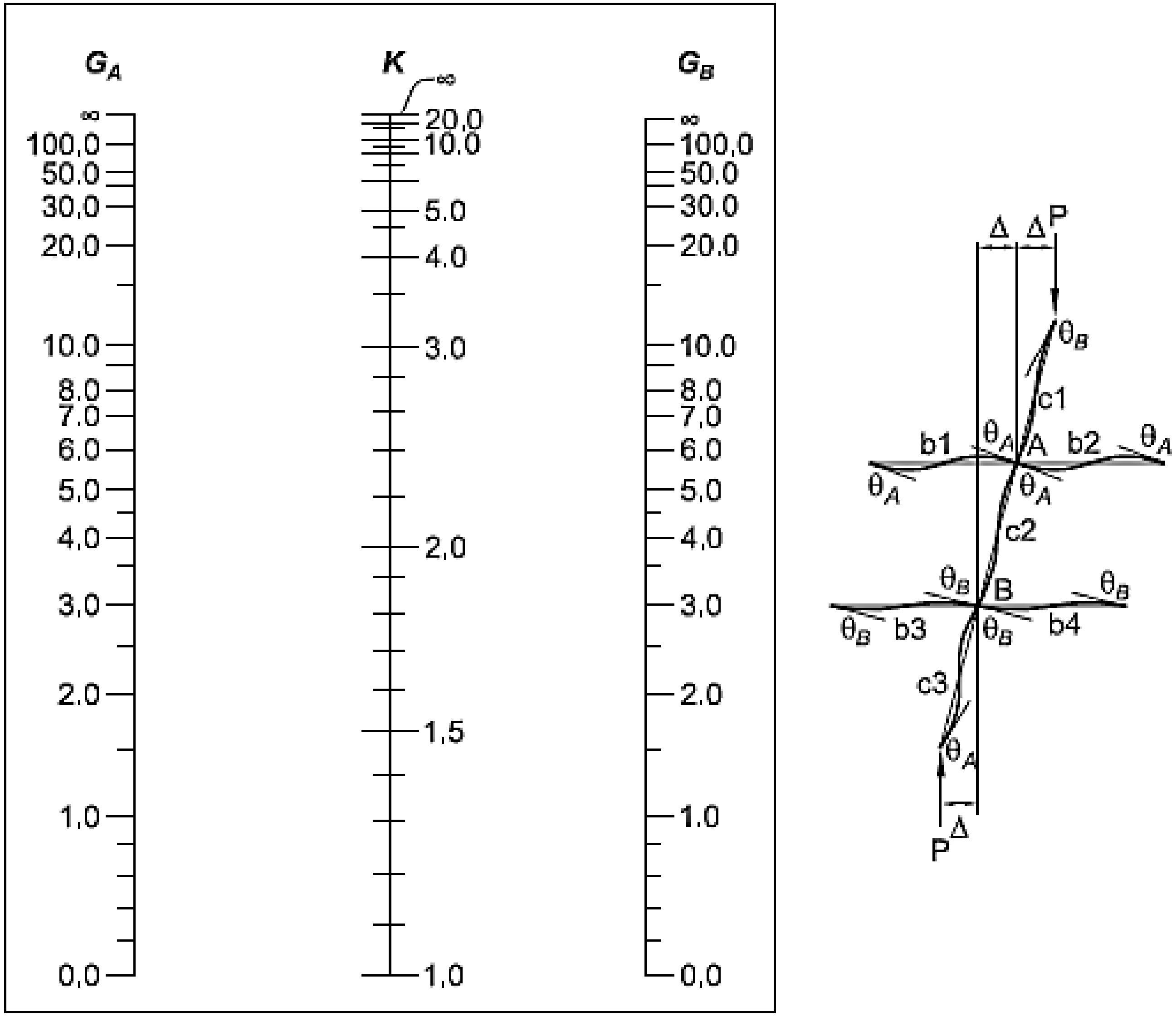


Fig. C-C2.4. Alignment chart—sidesway uninhibited (moment frame).

I. For pinned columns, G is theoretically infinite. such as when a column is connected to a footing with a frictionless hinge. Since such a connection is not frictionless. it is recommended that G be made equal to 10 where such nonrigid supports are used.

II. A For rigid connections of columns to footings. G theoretically approaches zero, but from a practical standpoint, a value of 1.0 is recommended. because no connections are perfectly rigid.