6000. STEEL

- 6100 & 6200
  - 6130 - Design Data, Principles and Tools
  - 6140 - Codes and Standards
  - 6200 - Material
- 6300
  - 6310 - Members and Components
  - 6320 - Connections, Joints and Details
  - 6330 - Frames and Assembles
- 6400
  - 6410 - AISC Specifications for Structural Joints
  - 6420 - AISC 303 Code of Standard Practice
  - 6430 - AWS D1.1 Structural Welding Code
- 6500
  - 6510 - Nondestructive Testing Methods
  - 6520 - AWS D1.1 Structural Welding Code Tests
- 6600
  - 6610 - Steel Construction
  - 6620/6630 - NUREG-0800 / RG 1.94

6300. Design -

6320. Structural Steel Connections, Joints and Details

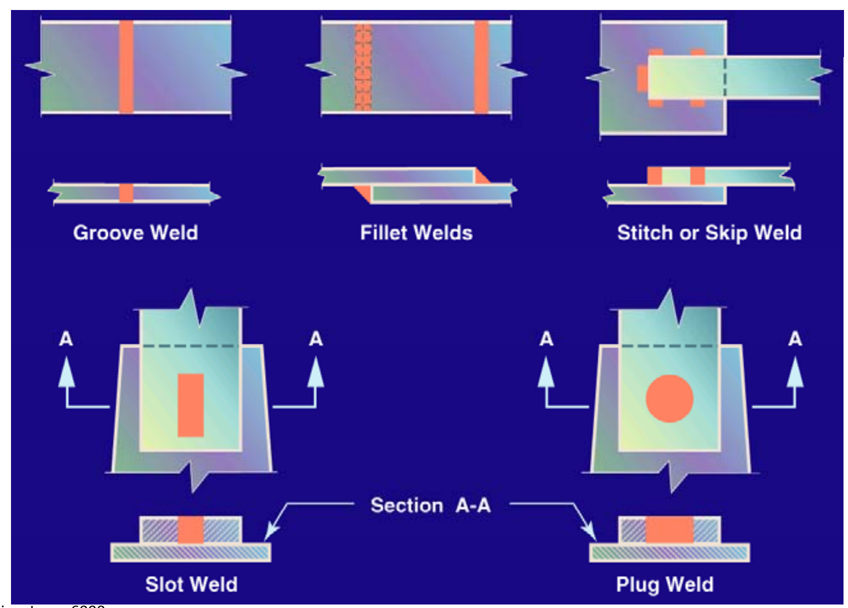
- General Provisions (Section NJ1)
- Types of Structural Welds and Their Applications (Section NJ2 and AISC Manual Part 8)
- Types of Structural Bolts and Bolted Connections (Section NJ3 and AISC Manual Part 7)
- AISC Connections (Section NJ and AISC Manual Part 9)
- HSS and Box Member Connections (Section NK)
- Selecting Standard Connections from the AISC Manual (AISC Manual Parts 9 & 10)
- Seismic Connection

6320. Structural Steel Connections, Joints and Details – Module 1: Welds

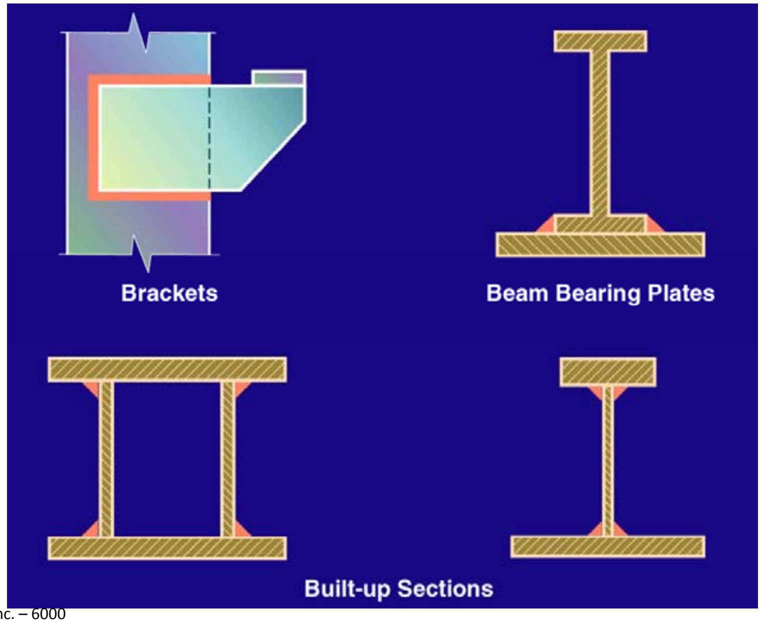
This section of the module covers:

- Introduction
- Basics of welding
- Fillet weld
- LRFD of welded connections
- Eccentric shear in welds
- Welding problems
- Prequalified welds

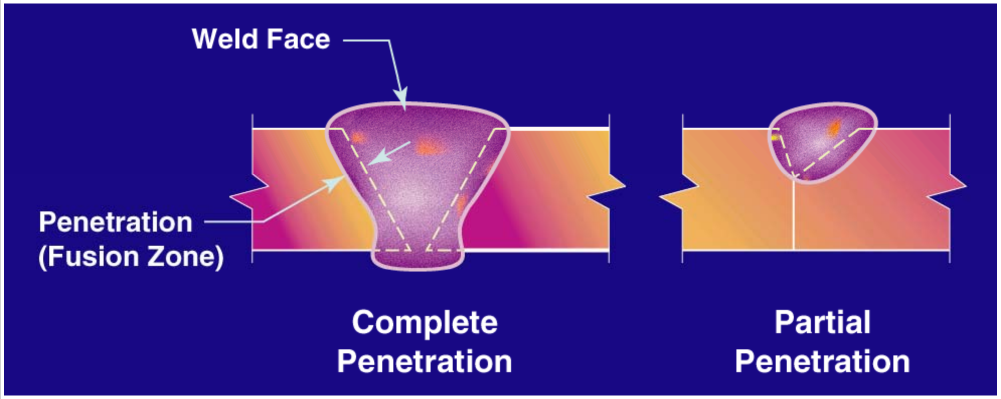
Types of Welds



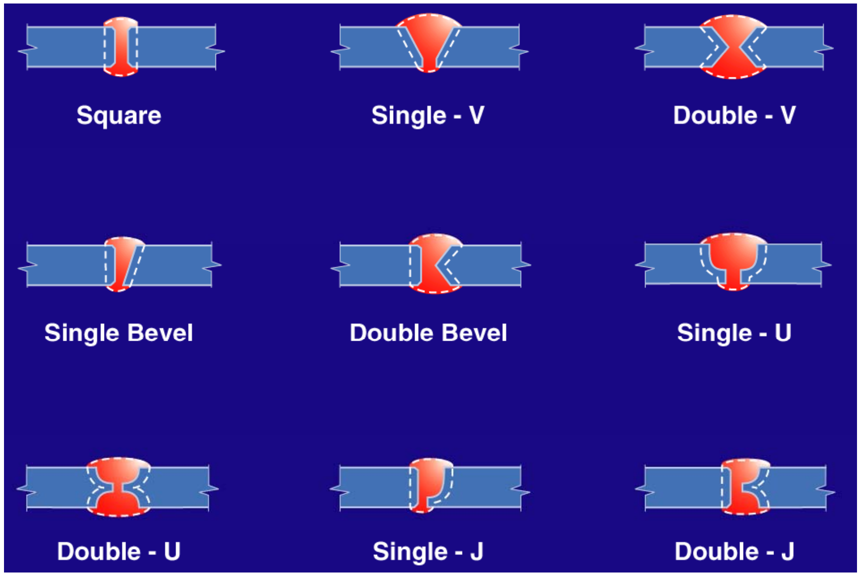
Uses of Fillet Welds



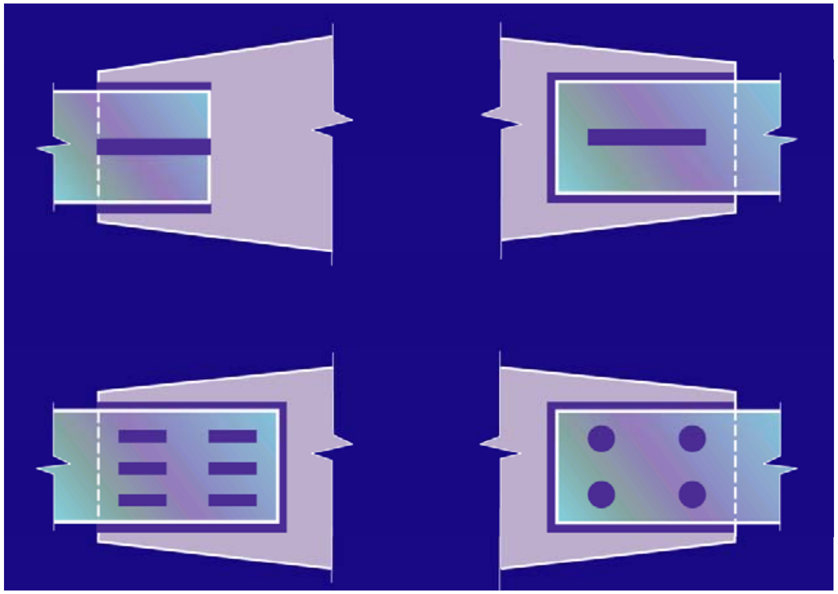
Complete and Partial Penetration Groove Welds



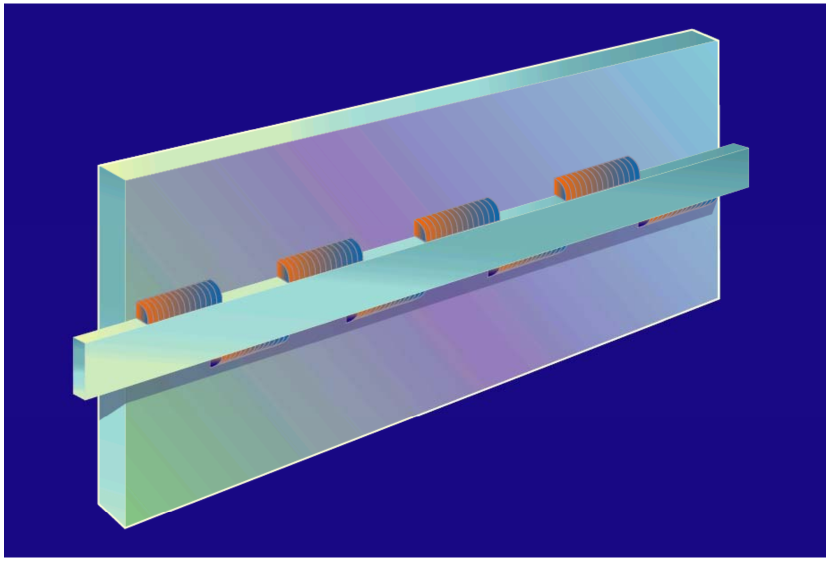
Types of Groove Welds



Plug or Slot Weld



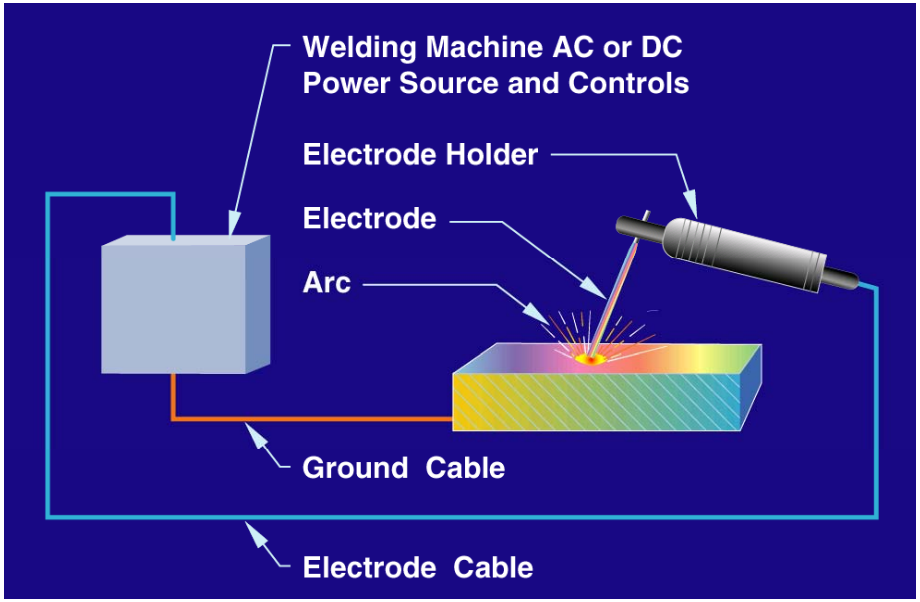
Stitch or Skip Weld



Basic of Welding

- Structural welding is a process whereby the parts to be connected are heated and fused with a molten filler metal.
- Upon cooling, the structural steel (parent metal) and weld or filler metal will act as one continuous part. The filler metal is deposited from a special electrode. A number of welding processes are used, depending on the application
  - Field welds
  - Shop welds

Welding Process and Metallurgy



Welding Electrodes

The American Welding Society (AWS) has developed specifications for the filler metals to cover arc welding of the following steels:

- Carbon
- Alloy
- Stainless and corrosion-resisting

AWS Electrode Classification

AWS ELECTRODE CLASSIFICATION SYSTEM		
Digit	**Significance**	**Example**
1st two or 1st three	Minimum tensile strength (stress relieved)	E-60xx = 60,000 psi (min) E-110xx = 110,000 psi (min)
2nd last	Welding position	E-xx1x = all positions E-xx2x = horizontal and flat E-xx3x = flat
Last	Power supply, type of slag, type of arc, amount of penetration, presence of iron powder in coating	
NOTE: Prefix "E" (to left of 4 or 5-digit number) signifies arc welding electrode		

Basic of welding

Minimum weld size, maximum weld size, and minimum length:

- The minimum size of a fillet weld is a function of the thickness of the thicker connected part. See AISC Table J2.4 for details.
- The maximum size of a fillet weld is as follows:
  - Along the edge of a connected part less than ¼-inch thick, the maximum fillet weld size (w) equals the plate thickness
  - For other values of plate thickness, t, the maximum weld size is t -1/16 in.

TABLE J2.4
Minimum Size of Fillet Welds

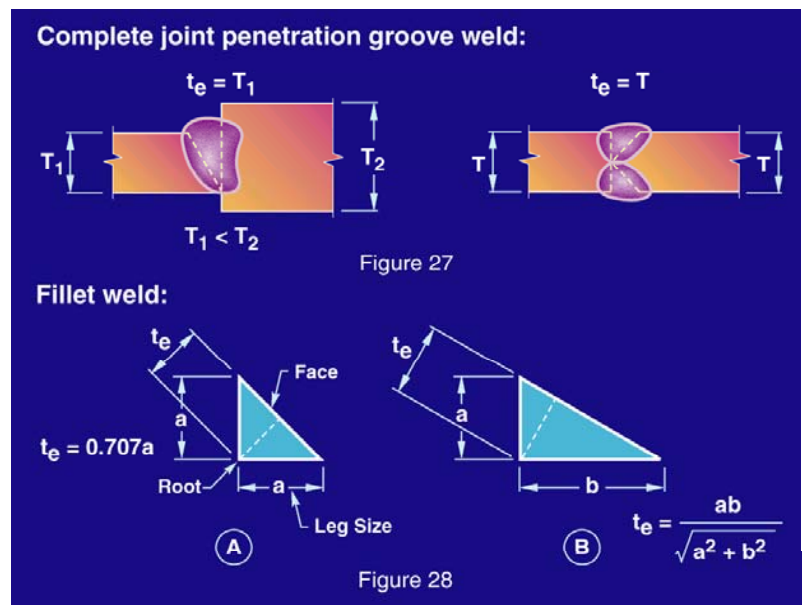
Material Thickness of Thicker Part Joined, in. (mm)	Minimum Size of Fillet Weld[a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
[b] See Section J2.2b for maximum size of fillet welds.

Basic of welding

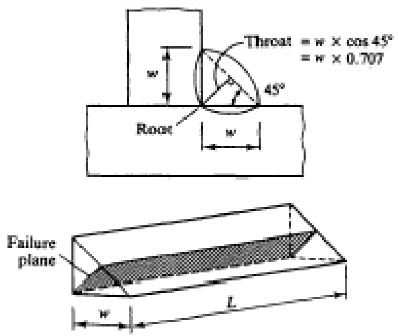
- The minimum permissible length of a fillet weld is 4 times its size. If only a shorter length is available, w = L/4. For the welds in the connection shown below, $L \geq W$ to address shear lag in such connections.
- When a weld extends to the corner of a member, it must be continued around the corner (an end return)
  - Prevent stress concentrations at the corner of the weld
  - Minimum length of return is 2w

Effective Area of Welds



Fillet Weld

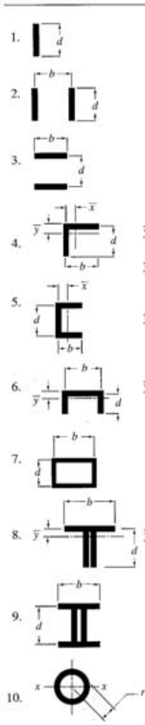
- The design and analysis of fillet welds is based on the assumption that the geometry of the weld is a 45-degree right triangle
- Standard weld sizes are expressed in sixteenths of an inch.
- Failure of fillet welds is assumed to occur in shear on the throat.



Fillet Weld

- The critical shearing stress on a weld of length *L* is given by
  $f = P/(0.707wL)$
- If the ultimate shearing stress in the weld is termed F_W, the nominal design strength of the weld can be written as
  $\varphi R_n = 0.707wL(\varphi F_w) = 0.707wL(0.75[0.6F_{EXX}]) = 0.32wLF_{EXX}$
- For E70XX and E80XX electrodes, the design stresses are φF_w, or 31.5 ksi and 36 ksi, respectively.
- In addition, the factored load shear on the base metal shall not produce a stress in excess of φF_{BM}, where F_{BM} is the nominal shear strength of the connected material. The factored load on the connection is thus subjected to the limit of
  $\varphi R_n = \varphi F_{BM} A_g = 0.90(0.6F_y)A_g = 0.54F_y A_g$

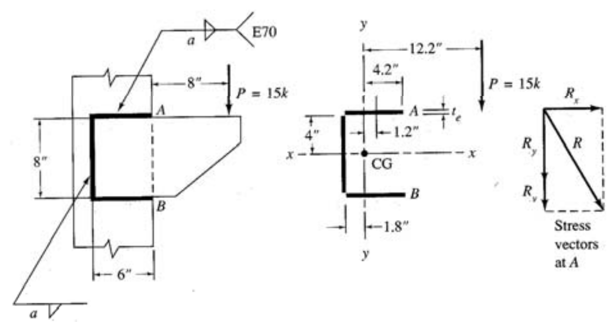
Eccentric Shear in Welds



Section b = width; d = depth		Section modulus $I_x/\bar{y}$	Polar moment of inertia, I_p about center of gravity
1.		$S = \frac{d^2}{6}$	$I_p = \frac{d^3}{12}$
2.		$S = \frac{d^2}{3}$	$I_p = \frac{d(3b^2 + d^2)}{6}$
3.		$S = bd$	$I_p = \frac{b(3d^2 + b^2)}{6}$
4.	$\bar{y} = \frac{d^2}{2(b + d)}$ $\bar{x} = \frac{b^2}{2(b + d)}$	$S = \frac{4bd + d^2}{6}$	$I_p = \frac{(b + d)^4 - 6b^2d^2}{12(b + d)}$
5.	$\bar{x} = \frac{b^2}{2b + d}$	$S = bd + \frac{d^2}{6}$	$I_p = \frac{8b^3 + 6bd^2 + d^3}{12} - \frac{b^4}{2b + d}$
6.	$\bar{y} = \frac{d^2}{b + 2b}$	$S = \frac{2bd + d^2}{3}$	$I_p = \frac{b^3 + 6b^2d + 8d^3}{12} - \frac{d^4}{2d + b}$
7.		$S = bd + \frac{d^2}{3}$	$I_p = \frac{(b + d)^3}{6}$
8.	$\bar{y} = \frac{d^2}{b + 2d}$	$S = \frac{2bd + d^2}{3}$	$I_p = \frac{b^3 + 8d^3}{12} - \frac{d^4}{b + 2d}$
9.		$S = bd + \frac{d^2}{3}$	$I_p = \frac{b^3 + 3bd^2 + d^3}{6}$
10.		$S = \pi r^2$	$I_p = 2\pi r^3$

Eccentric Shear in Welds

- Eccentricity in the plane of the faying surface
  - Instantaneous center of rotation method
  - Elastic method
- Eccentricity normal to the plane of the faying surface



Welding Problems

- Lamellar tears
- Weld shrinkage and structural distortion
- Residual stresses
- Fatigue sensitivity

Lamellar Tears

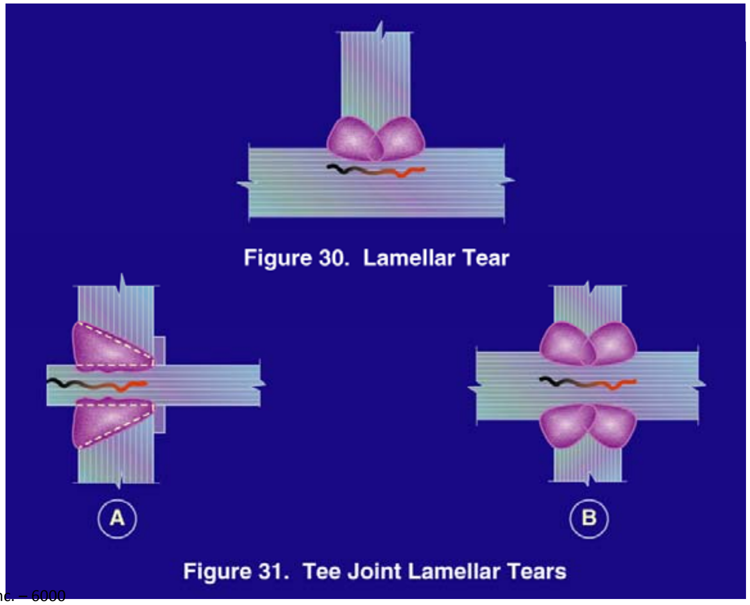
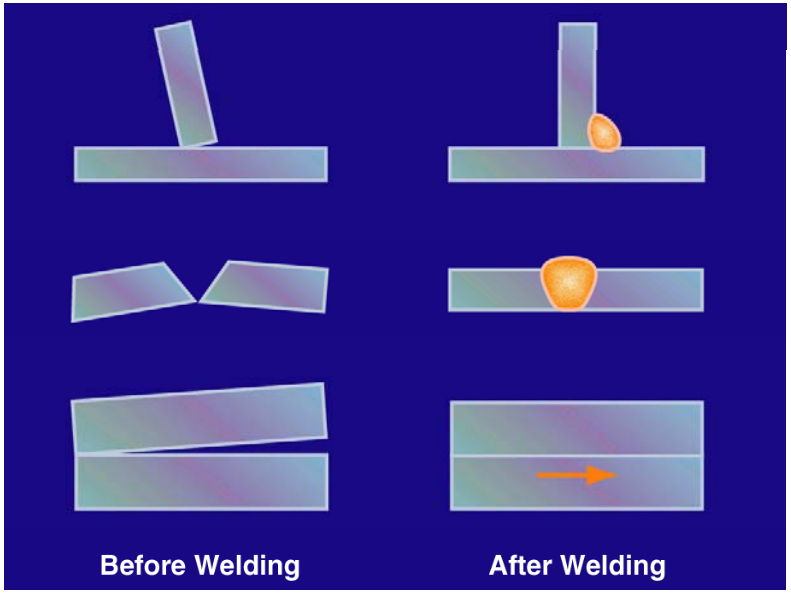


Figure 30. Lamellar Tear

Figure 31. Tee Joint Lamellar Tears

Pre-bending for Weld Shrinkage

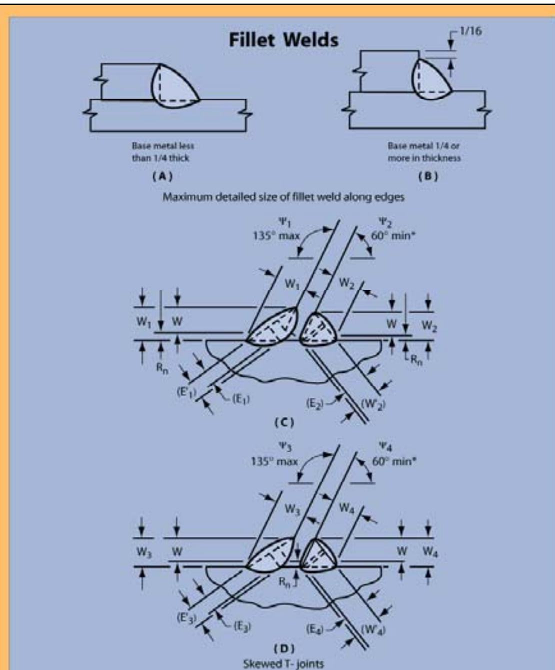


AISC Standard Connections and Suggested Details

This last section of the module covers the following:

- Prequalified welds
- Suggested details

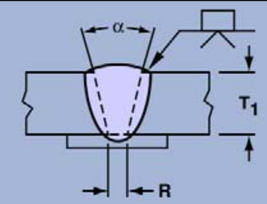
Prequalified Welds



Note: $(E)_{(n)}$, $(E')_{(n)}$ = effective throats dependent on magnitude of root opening (R_n). See AWS 3.3.1 Subscript (n) represents 1, 2, 3 or 4.
* Angles smaller tha 60 degrees are permitted; however, in such cases, the weld is considered be a partial joint penetration groove weld.
For additional requirement for skewed T-joints, see 2.3.4. of AWS D1.1-88.

Prequalified Complete Penetration Groove Welds

Single V-groove weld (2)
Butt joint (B)

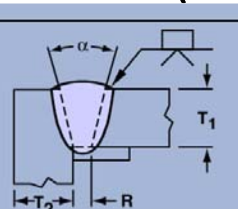


Tolerances	
As detailed	As fit up
R = + 1/16, − 0	+ 1/4, − 1/16
α = + 10°, − 0°	+ 10°, − 5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T_1	T_2	Groove Preparation Root Opening	Groove Angle	Permitted Welding Positions	Gas Shielding for (FCAW)	Notes
SMAW	B-U2a	U	—	R = 1/4	α = 45°	All	—	N
				R = 3/8	α = 30°	F,V,OH	—	N
				R = 1/2	α = 20°	F,V,OH	—	N
GMAW FCAW	B-U2a-GF	U	—	R = 3/16	α = 30°	F,V,OH	Required	A,N
				R = 3/8	α = 30°	F,V,OH	Not Req.	A,N
				R = 1/4	α = 45°	F,V,OH	Not Req.	A,N
SAW	B-L2a-S	2 max	—	R = 1/4	α = 30°	F	—	N
SAW	B-U2-S	U	—	R = 5/8	α = 20°	F	—	N

Prequalified Complete Penetration Groove Welds (Cont'd.)

Single V-groove weld (2)
Corner joint (C)



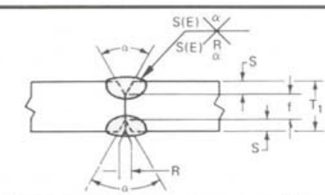
Tolerances	
As detailed	As fit up
R = + 1/16, − 0	+ 1/4, − 1/16
α = + 10°, − 0°	+ 10°, − 5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T_1	T_2	Groove Preparation Root Opening	Groove Angle	Permitted Welding Positions	Gas Shielding for (FCAW)	Notes
SMAW	C-U2a	U	U	R = 1/4	α = 45°	All	—	Q
				R = 3/8	α = 30°	F,V,OH	—	Q
				R = 1/2	α = 20°	F,V,OH	—	Q
GMAW FCAW	C-U2a-GF	U	U	R = 3/16	α = 30°	F,V,OH	Required	A
				R = 3/8	α = 30°	F,V,OH	Not Req.	A,Q
				R = 1/4	α = 45°	F,V,OH	Not Req.	A,Q
SAW	C-L2a-S	2 max	U	R = 1/4	α = 30°	F	—	Q
SAW	C-U2-S	U	U	R = 5/8	α = 20°	F	—	Q

See notes on page preceding Prequalifed Weld Joint Tables.

Prequalified Partial Penetration Groove Welds

Double V-groove weld (3)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T_1	T_2	Groove Preparation: Root Opening Root face Groove angle	Tolerances: As Detailed	As Fit Up	Permitted Welding Positions	Effective Throat (E)	Notes
SMAW	B-P3	½ min	—	R = 0 f = ⅛ min α = 60°	+1/16, −0 unlimited +10°, −0°	+⅛, −1/16 ±1/16 +10°, −5°	All	S	E,Mp, Q2
GMAW FCAW	B-P3-GF	½ min	—	R = 0 f = ⅛ min α = 60°	+1/16, −0 unlimited +10°, −0°	+⅛, −1/16 ±1/16 +10°, −5°	All	S	A,E, Mp, Q2
SAW	B-P3-S	¾ min	—	R = 0 f = ¼ min α = 60°	±0 unlimited +10°, −0°	+1/16, −0‡ ±1/16 +10°, −5°	F	S	E,Mp, Q2

‡Fit-up tolerance, SAW: See AWS 3.3.2; for rolled shapes R may be 5/16 inches in thick plates if backing is provided.

See notes on page preceding Prequalifed Weld Joint Tables.

6300. Design -

6320. Structural Steel Connections, Joints and Details

Objective and Scope Met

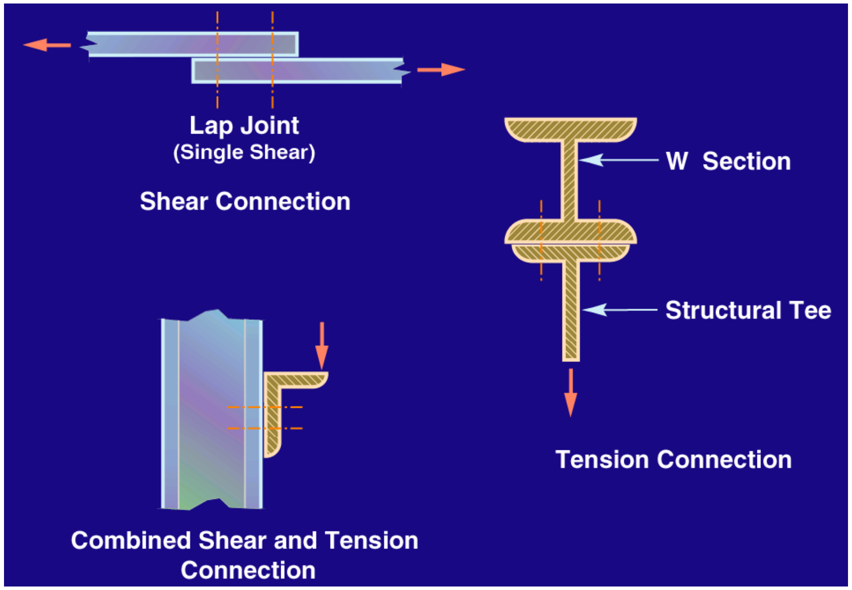
- Module 1: Welds
  - Introduction
  - Basics of welding
  - Fillet weld
  - LRFD of welded connections
  - Eccentric shear in welds

6320. Structural Steel Connections, Joints and Details – Module 2: Bolts

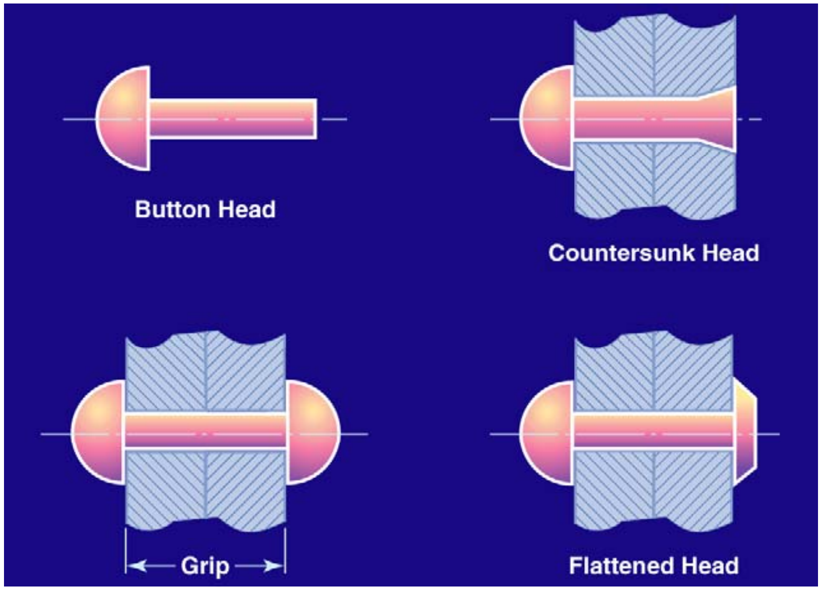
This section of the module covers:

- Introduction of Fasteners
- Failure modes of bolted shear connections
- LRFD - Fasteners
- LRFD of slip-critical connections
- Eccentric shear in bolts
- Fasteners in combined shear and tension
- Design and Erection Concerns
- Prequalified bolts

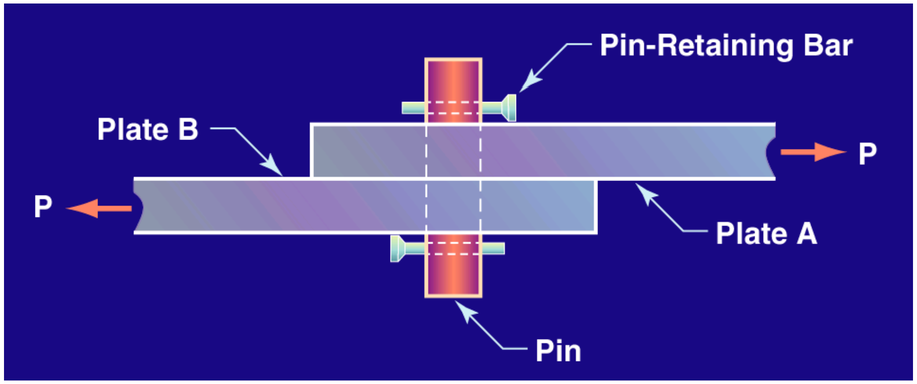
Bolted Connections



Riveted Connections



Pinned Connections



Properties of Structural Bolts

ASTM DESIGNATION	BOLT DIAMETER, in.	MINIMUM TENSILE STRENGTH, ksi	MINIMUM YIELD STRENGTH, ksi, 0.2% OFFSET
A307, low-carbon steel	1/4 to 4	60	_
High-strength Structural bolts:			
A325, medium-carbon steel	1/2 to 1 1- 1/8 to 1- 1/2	120 105	92 81
A490, alloy steel	1/2 to 1- 1/2	150	130

Unfinished Bolts (A307)

- Made from low-carbon steel
- Minimum tensile strength of 60 ksi
- Least expensive
- More are required in a particular connection
- Used in light structures
- Manufactured in grades A and B
- Induced tension is relatively small and unpredictable

High-Strength Bolts (A325)

- Most commonly used high-strength bolt
- Made of heat-treated medium-carbon steel
- Tensile strength decreases as the diameter increases
- Available in Types 1, 2, and 3

High-Strength Bolts (A490)

- Made of heat-treated alloy steel in one tensile-strength grade
- Available in Types 1, 2, and 3

High-Strength Bolts (A325) and (A490)

- 3/4 in. and 7/8 in. Most common diameters in building construction
- Used for anchor bolts and threaded rods
- Tightened to develop large tensions
- Sufficient pre-tension force required
- Installed with initial tension 70% of specified minimum tensile strength

Connection Types

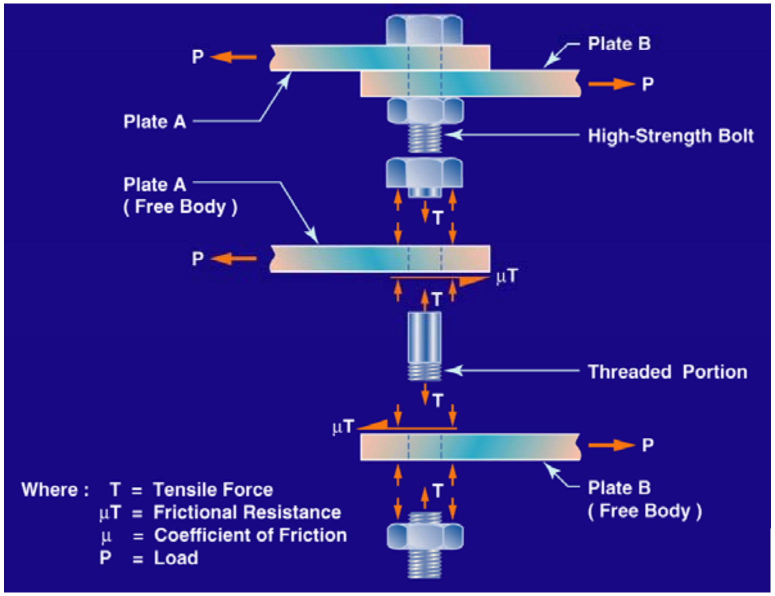
- Friction type: where high-slip resistance is desired
- Bearing type: where high-slip resistance is unnecessary

Introduction of Fasteners

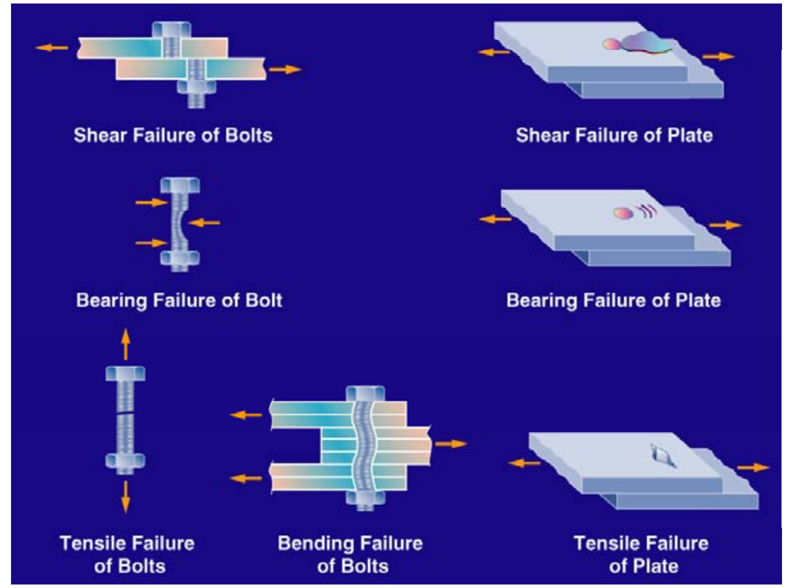
- Two conditions of bolt installation are used with high-strength bolts
  - Snug-tight (producing a bearing connection)
    - Few impacts of an impact wrench
    - Full effort of a worker with an ordinary spud wrench
  - Tensioned (producing a slip-critical connection)
    - Turn-of-nut method: specified number of rotations of the nut from snug tight (nut rotations correlated to bolt elongation)
    - Calibrated wrench tightening
    - Alternate design bolts: specially design bolts whose tops twist off when the proper tension has been achieved
    - Direct tension indicators: compress washer (under bolt head or nut) with protrusions to a gap that is correlated to bolt tension

Ref: AISC LRFD p.16.4-46 thru -52

Overview of Theory for Design



Possible Failure Modes



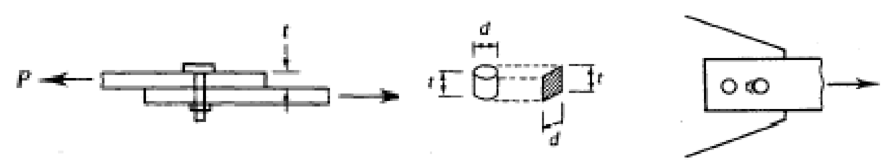
Failure Mode of Bolted Shear Connections

Failure of the connected parts , separated into two categories.

1. Failure resulting from excessive tension, shear, or bending in the parts being connected
   - For a tension member must consider tension on the net area, tension on the gross area, and block shear
   - For beam-beam or beam-column connections, must consider block shear
   - Gusset plates and framing angles must be checked for P, M, and V

Failure Mode of Bolted Shear Connections

2. Failure of the connected part because of bearing exerted by the fastener (average bearing stress is $f_p = P/dt$)
   - If the hole is slightly larger than the fastener and the fastener is assumed to be placed loosely in the hole (rarely the case), contact between the fastener and the connected part will exist over approximately 50% of the circumference of the fastener.
   - The bearing problem is affected by the edge distance and bolt spacing



LRFD - Fasteners

- $\Phi R_n \geq \Sigma \gamma_i Q_i$ general
  - where Φ = resistance factor (strength reduction factor)
  - R_n = nominal resistance (strength)
  - γ_i = overload factors (LRFD-A4.1)
  - Q_i = loads (such as dead load, live load, wind load, earthquake load) of load effects (such as bending moment, shear, axial force, and torsional moment resulting from the various loads)
- $\Phi R_n \geq P_u$ fasteners
  - where Φ = resistance factor, 0.75 for fracture in tension, shear on high-strength bolts, and bearing of bolt against side of hole
  - R_n = nominal strength of one fastener
  - P_u = factored load on one fastener

LRFD – Fasteners

Design shear strength – no threads in shear planes (X)

- $\Phi R_n = 0.75(0.50 F_u^b) m A_b$
  - where Φ = 0.75, the standard value for shear
  - F_u^b = tensile strength of the bolt material (120 ksi for A325X bolts; 150 ksi for A490X bolts)
  - m = the number of shear planes participating [usually one (*single shear*) or two (*double shear*)]
  - A_b = gross cross-sectional area across the unthreaded shank of the bolt

Design shear strength – threads in shear planes (N)

- $\Phi R_n = 0.75(0.40 F_u^b) m A_b$

LRFD – Fasteners

Design tensile strength

- $\Phi R_n = 0.75(0.75 F_u^b) A_b$
  - where Φ = 0.75, a value for the tensile fracture mode
  - F_u^b = tensile strength of the bolt material (120 ksi for A325 bolts; 150 si for A490 bolts)
  - A_b = gross cross-sectional area across the unthreaded shank of the bolt

LRFD – Fasteners

Design bearing strength

1. *Usual conditions* based on the deformation limit state, according to LRFD-Formula (J3-1a). This applies for all holes except long-slotted holes perpendicular to the line of force, where end distance is at least 1.5d, the center-to-center spacing s is at least 3d, and there are two or more bolts in the line of force.

$$\Phi R_n = \Phi(2.4 d t F_u) \quad (4.7.9)$$

- where Φ = 0.75
- d = nominal diameter of bolt at unthreaded area
- t = thickness of part against which bolt bears
- F_u = tensile strength of connected part against which bolt bears
- L_e = distance along line of force from the edge of the connected part to the center of a standard hole or the center of a short- and long-slotted hole perpendicular to the line of force.

LRFD – Fasteners

Design bearing strength (cont)

2. Deformation limit state for long-slotted holes perpendicular to the line of force, where end distance L_e is at least 1.5 *d*, the center-to-center spacing *s* is at least 3*d*, and there are two or more bolts in the line of force, according to LRFD-Formula (J3-1d).

$$\Phi R_n = \Phi(2.0dtF_u)$$

where Φ = 0.75

3. Strength limit state for the bolt nearest the edge, according to LRFD-Formulas (J3-1b), (J3-2a), and (J3-2c)

$$\Phi R_n = \Phi L_e t F_u$$

4. Strength limit state when hole elongation exceeding 0.25 in. and hole "ovalization" can be tolerated, LRFD-Formulas (J3-1b) and (J3-1c) give,

$$\Phi R_n = \Phi(3.0dtF_u)$$

LRFD – Fasteners

Minimum spacing and end distance (L_e) in line of transmitted force

$$\text{Spacing} \geq \frac{P}{\Phi F_u t} + \frac{d}{2}$$

where Φ = 0.75

P = factored load acting on one bolt

F_u = tensile strength of *plate* material

t = thickness of plate material

d = diameter of the bolt

$$L_e \geq \frac{P}{\Phi F_u t}$$

LRFD – Fasteners

Maximum edge distance – ≤12 t ≤ 6", where t is the thickness of the connected part.

Maximum spacing of connectors

(a) For painted members or unpainted members not subject to corrosion, ≤ 24t ≤ 12"

(b) For unpainted members of weathering steel subject to atmospheric corrosion, ≤ 14t ≤ 7"

LRFD Slip-critical Connections

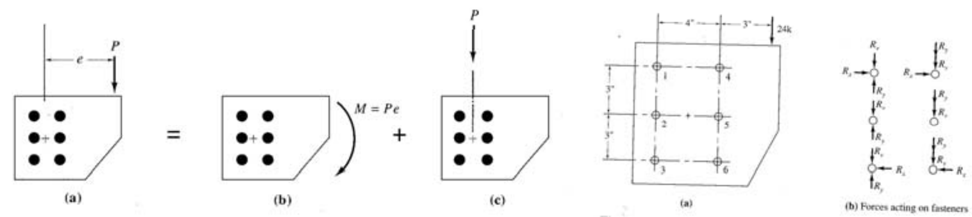
- A connection with high-strength bolts is classified as either a bearing or slip-critical connection.
- Bearing connections - the bolt is brought to a snug-tight condition so that the surfaces of the connected parts are in firm contact.
  - Slippage is acceptable
  - Shear and bearing on the connector
- Slip-critical connections - no slippage is permitted and the friction force described earlier must not be exceeded.
  - Slippage is not acceptable (Proper installation and tensioning is key)
  - Must have sufficient shear an d bearing strength in the event of overload that causes slip. AISC J3.8 for details.

LRFD Slip-critical Connections

- $\Phi R_{str} = \Phi 1.13 \mu T_i m$
  - Where R_{str} = nominal slip resistance per bolt at factored loads
  - m = number of slip (shear) planes
  - T_i = minimum fastener initial tension given in LRFD-Table J3.1
  - = mean slip coefficient, as applicable, or as established by tests
  - μ = 0.35 for Class A surface condition
  - = 0.50 for Class B surface condition
  -
  - Φ = 1.0 for standard holes (S&J Example 4.9.2)
  - = 0.85 for oversize and short-slotted holes
  - = 0.70 for long-slotted holes

Eccentric Shear

1. Instantaneous center of rotation method – more accurate but requires the use of tabulated values of an iterative solution.
2. Classic method – simplified but may be excessively conservative because it neglects the ductility of the bolt group and the potential for load redistribution.



Combined Shear and Tension

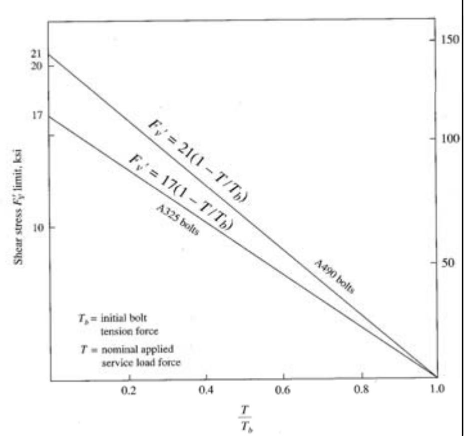
Bearing-type connections

Fastener	$\phi F'_{ut}$ (ksi)	$\phi F'_{ut}$ (MPa)
A307 bolts	$\phi(59 - 1.9 f_{uv}) \le \phi(45)$	$\phi(407 - 1.9 f_{uv}) \le \phi(310)$
A325-N bolts (threads *not* excluded)	$\phi(117 - 1.9 f_{uv}) \le \phi(90)$	$\phi(807 - 1.9 f_{uv}) \le \phi(621)$
A325-X bolts (threads excluded)	$\phi(117 - 1.5 f_{uv}) \le \phi(90)$	$\phi(807 - 1.5 f_{uv}) \le \phi(621)$
A490-N bolts (threads *not* excluded)	$\phi(147 - 1.9 f_{uv}) \le \phi(113)$	$\phi(1010 - 1.9 f_{uv}) \le \phi(779)$
A490-X bolts (threads excluded)	$\phi(147 - 1.5 f_{uv}) \le \phi(113)$	$\phi(1010 - 1.5 f_{uv}) \le \phi(779)$

* Note that $\phi = 0.75$

‡ Nominal stress due to factored load acting on gross bolt cross-sectional area, $f_{ut} = R_{ut}/A_b$

Slip-critical connections



Design and Erection Concerns

- Corrosion: reduces strength of bolts
- Misuse of bolts: engineers must adhere to AISC specifications and design requirements
- Improper torque: if torque is too small, slippage occurs; if torque is too large, the bolt fractures
- Bolt fatigue due to vibration: loosen bolts, resulting in prying action

Joint Type Specification (2)

Slip-Critical Joints

Slip-Critical Joints are only required in the following applications involving shear or combined shear and tension (i.e., not applicable for applications involving tension only):

- Joints that are subject to fatigue load with reversal of the loading direction (i.e., cycled load that does involve a change in the sign of the load);
- Joints that utilize oversized holes;
- Joints that utilize slotted holes, except those with applied load approximately normal (within 80 to 100 degrees) to the direction of the long dimension of the slot; and,
- Joints in which slip at the faying surfaces would be detrimental to the performance of the structure.

Estimating Bolting Costs

Consider a 59 kip factored load using ASTM A325 high-strength bolts. The cost estimates include one fabricator's estimate of the associated labor costs:

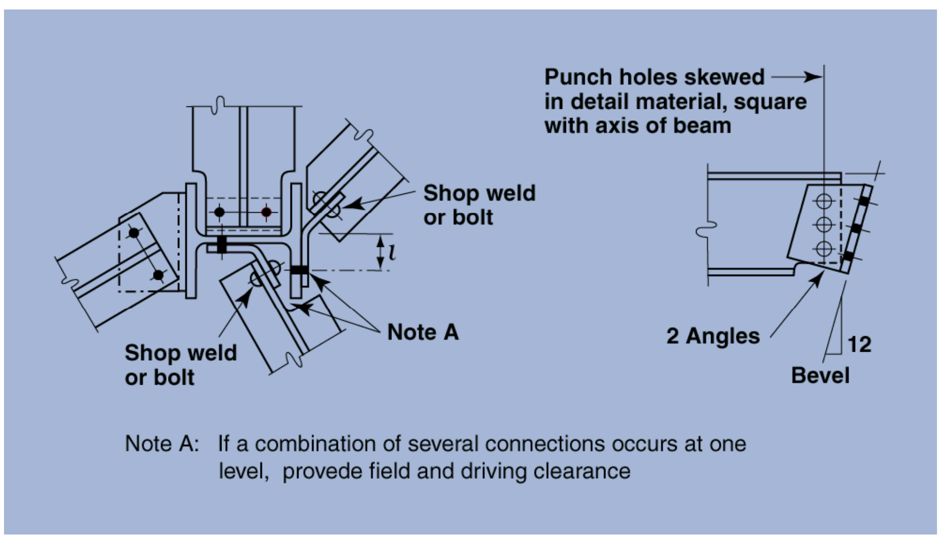
			Cost	Cost Factor
Slip Critical (N or X)	6	bolts @ 10.4 kips/bolt = 62.4 kips	$66.00	3.1
Pretensioned (N)	4	bolts @ 15.9 kips/bolt = 63.6 kips	$34.00	1.6
Pretensioned (X)	3	bolts @ 19.9 kips/bolt = 59.7 kips	$25.50	1.2
Snug-tightened (N)	4	bolts @ 15.9 kips/bolt = 63.6 kips	$28.00	1.3
Snug-tightened (X)	3	bolts @ 19.9 kips/bolt = 59.7 kips	$21.00	1.0

Suggested Details

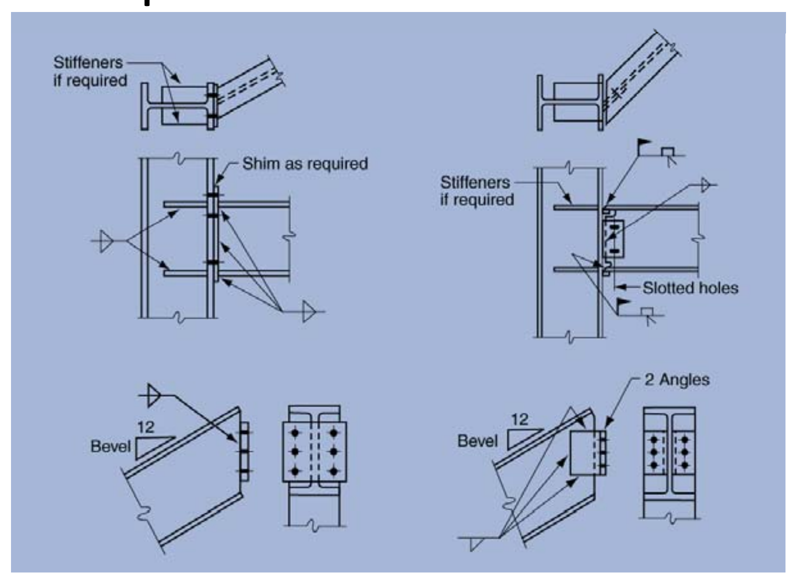
AISC Manual of Steel Construction provides a number of suggested connection details, covering the following:

- Beam framing
- Column base plates
- Column splices
- Miscellaneous

Suggested Details for Skewed and Sloped Beam Connections



Suggested Details for Skewed and Sloped Beam Connections



6300. Design -

6320. Structural Steel Connections, Joints and Details

Objective and Scope Met

- Module 2: Bolts
  - Introduction of Fasteners
  - Failure modes of bolted shear connections
  - LRFD - Fasteners
  - LRFD of slip-critical connections
  - Eccentric shear in bolts
  - Fasteners in combined shear and tension
  - Design and Erection Concerns
  - Prequalified bolts

6320. Structural Steel Connections, Joints and Details – Module 2: Connections

This section of the module covers:

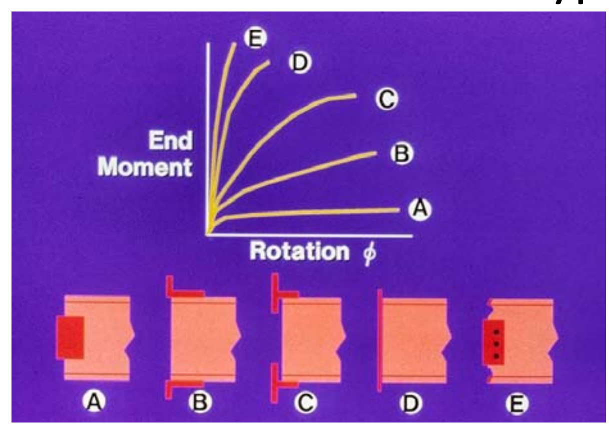
- Types of Structural Bolts and Bolted Connections (Section NJ3 and AISC Manual Part 7)
- AISC Connections (Section NJ and AISC Manual Part 9)
- HSS and Box Member Connections (Section NK)
- Selecting Standard Connections from the AISC Manual (AISC Manual Parts 9 & 10)

Steel Frame Connection Types

The Specification for Structural Steel Buildings (AISC 2005) defines two types of connections:

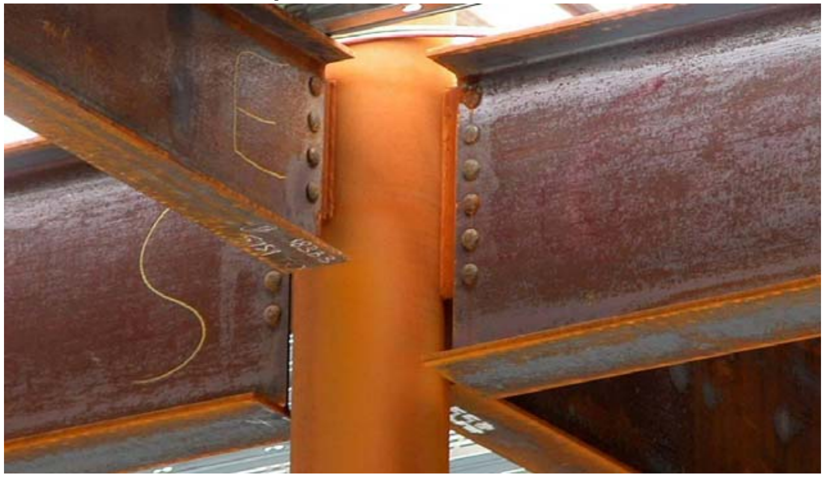
- Simple Connections (above left)
- Moment Connections (above right)
  - Fully-Restrained and Partially-Restrained

Steel Frame Connection Types



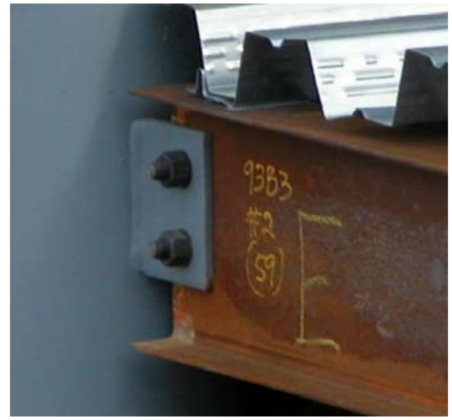
- All connections have a certain amount of rigidity
- Simple connections (A above) have some rigidity, but are assumed to be free to rotate
- Partially-Restrained moment connections (B and C above) are designed to be semi-rigid
- Fully-Restrained moment connections (D and E above) are designed to be fully rigid

Simple Connections



- Designed as flexible connections
- Connections are assumed to be free to rotate
- Vertical shear forces are the primary forces transferred by the connection
- Require a separate bracing system for lateral stability
- The following few slides show some common simple framing connections

Common Simple Connections



Single Plate Connection (Shear Tab)

A plate is welded to the supporting member and bolted to the web of the supported beam

Double Angle Connection

The in-plane pair of legs are attached to the web of the supported beam and the out-of-plane pair of legs to the flange or web of the supporting member

(Green, Sputo, and Veltri)

Common Simple Connections

Shear End Plate Connection

A plate is welded perpendicular to the end of the supported web and attached to the supporting member

Single Angle Connection

One leg is attached to the web of the supported beam and the other leg to the flange or web of the supporting member

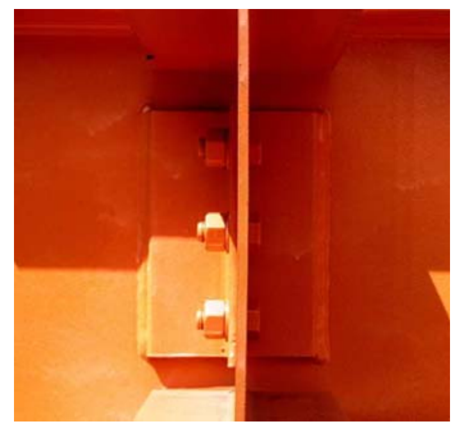
(Green, Sputo, and Veltri)

Common Simple Connections

Seated Connection

An angle is mounted with one leg vertical against the supporting column, and the other leg provides a “seat” upon which the beam is mounted

A stabilizer connection is also provided at the top of the web



Tee Connection

The stem of a WT section is connected to the supported member and the flange attached to the supporting member

(Green, Sputo, and Veltri)

Moment Connections

- Designed as rigid connections which allow little or no rotation
  - Used in rigid frames
- Moment and vertical shear forces are transferred through the connection
- Two types of moment connections are permitted:
  - Fully-Restrained
  - Partially-Restrained

Moment Connections

Fully-Restrained (FR) Connections

- Have sufficient strength to transfer moments with negligible rotation between connected members
- The angle between connected members is maintained

Partially-Restrained (PR) Connections

- Have sufficient strength to transfer moments, but the rotation between connected members is not negligible
- The angle between connected members may change

(AISC 2005)

Common FR Connections

Welded Flange Plate Connection

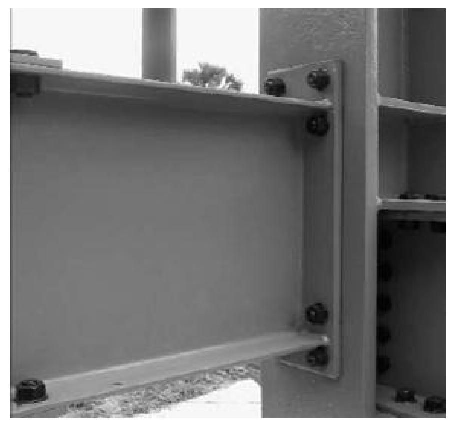
Bolted Flange Plate Connection

Top and bottom flange-plates connect the flanges of the supported member to the supporting column

A single plate connection is used to transfer vertical shear forces

(Green, Sputo, and Veltri)

Common FR Connection



Bolted Extended End-Plate Connection

A plate is welded to the flanges and web of the supported member and bolted with high-strength bolts to the supporting column

Welded Flange Connection

Complete-joint-penetration groove welds directly connect the top and bottom flanges of the supported member to the supporting column

A shear connection on the web is used to transfer vertical shear forces

(Green, Sputo, and Veltri)

Common PR Connections

PR Moment Connection – Wind Only

A double angle simple connection transfers vertical shear forces while top and bottom flange plates resist moment forces produced by wind

Note that the size of the flange plate is relatively small in comparison to the beam flange

Top and Bottom Angle with Shear End Plate Connection

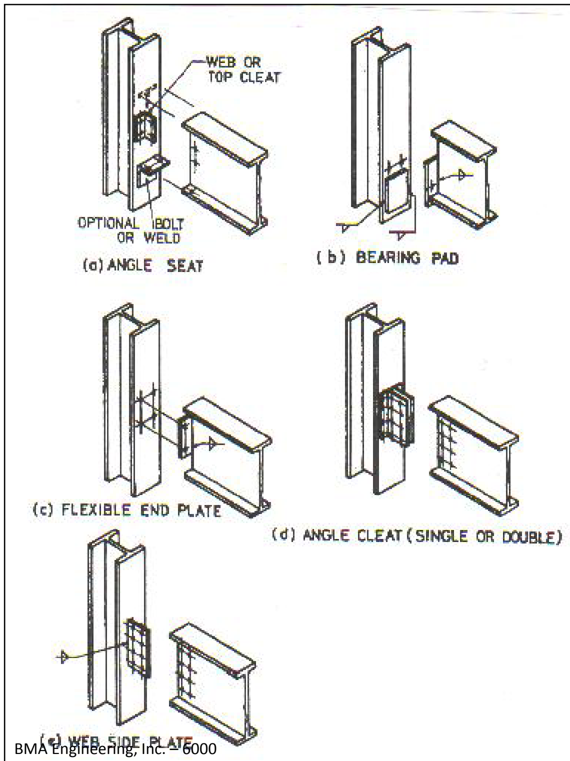
Angles are bolted or welded to the top and bottom flanges of the supported member and to the supporting column

A shear end plate on the web is used to transfer vertical shear forces

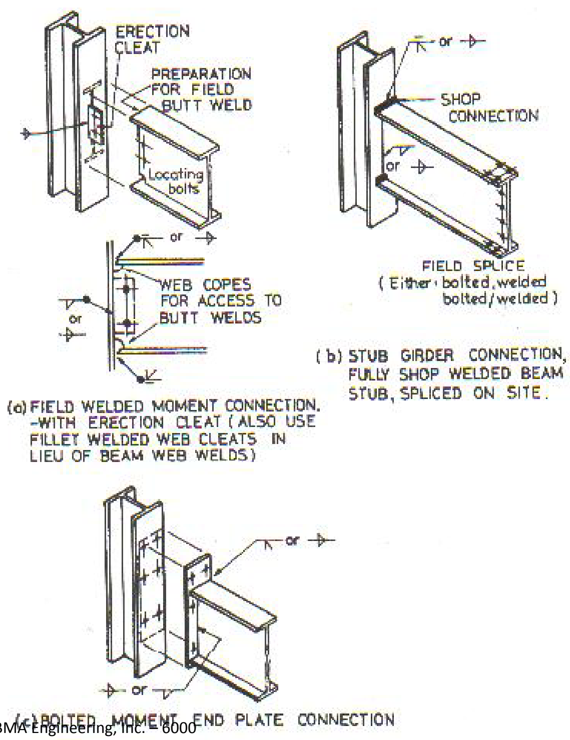
CONNECTION TYPES

(AISC 'Economical steelwork', 4th Edition)

- Flexible (AISC Fig. 3.1) (Pinned), and
- Rigid Connections (AISC Fig. 3.2)



Flexible (Pinned) Connections



Rigid Connections

Flexible Connections

- Assumed to behave as a simple support
- Simple to fabricate
- Simple to erect
- Less costly of the two connection types

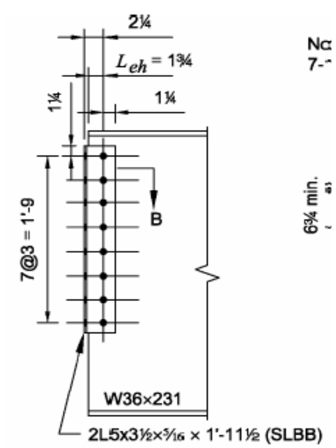
Rigid Connections

- More complex to fabricate
- More difficult to erect when tight tolerances are involved
- More costly of the two connection types
- The above connections can be used in the three basic framing systems available:
  - Two-way rigid framework (AISC Fig. 3.3)
  - One-way rigid/ one-way braced framework (AISC Fig. 3.4)
  - Two-way braced framework (AISC Fig. 3.5)

Double-Angle Connection

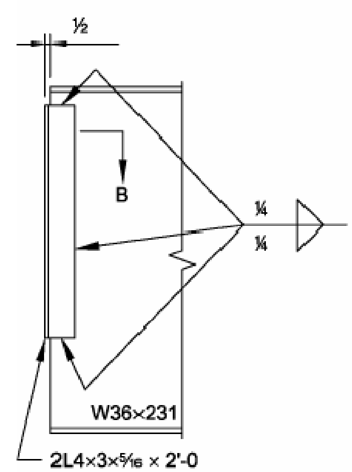
Example II.A-1 **All-Bolted Double-Angle Connection**
Example II.A-2 **Bolted/Welded Double-Angle Connection**

For bolted connection (AISC Tables 10-1 or -2)



Double-Angle Connection

Example II.A-3 **All-Welded Double-Angle Connection**

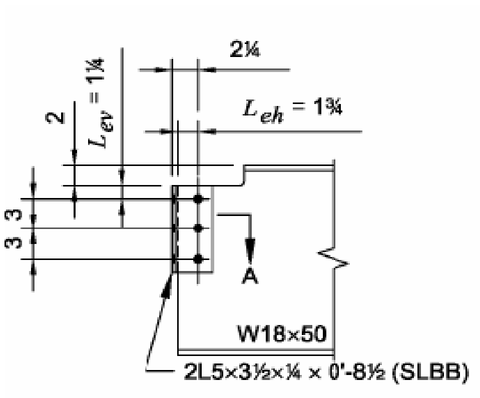


For welded connection (AISC Table 10-3)

Double-Angle Connection (coped)

Example II.A-4 All-Bolted Double-Angle Connection

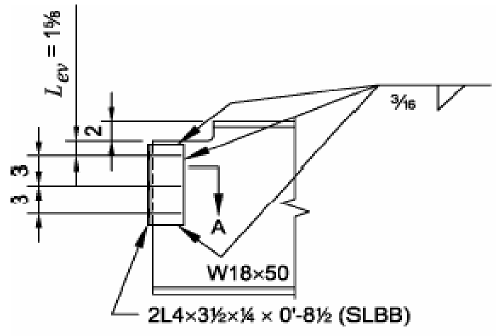
For all bolted connection (AISC Tables 10-1 or -2 w/ Tables 9-2, -3, & -4)



Double-Angle Connection (coped)

Example II.A-5 Bolted/Welded Double-Angle Connection (beam-to-girder web)

olted/welded connection (AISC Tables 10-3)

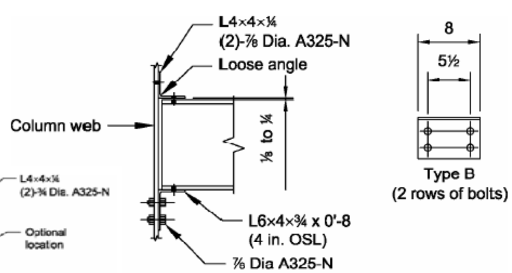


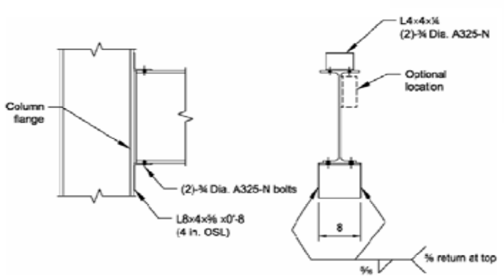
Check shear rupture on beam web

Unstiffened Seated Connection

Example II.A-12 All-Bolted Unstiffened Seated Connection (beam-to-column web).

For all bolted connection (AISC Table 10-5)



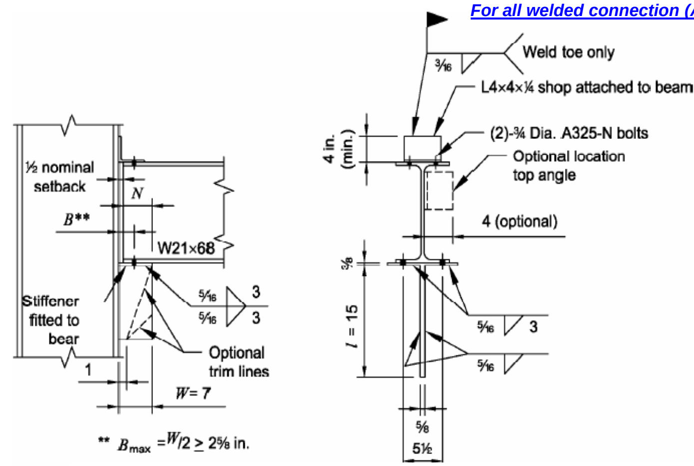


For bolted/welded connection (AISC Tables 10-5 & -6)

Stiffened Seated Connection

Example II.A-14 Stiffened Seated Connection (beam-to-column flange)

For all welded connection (AISC Tables 10-7 or 8)

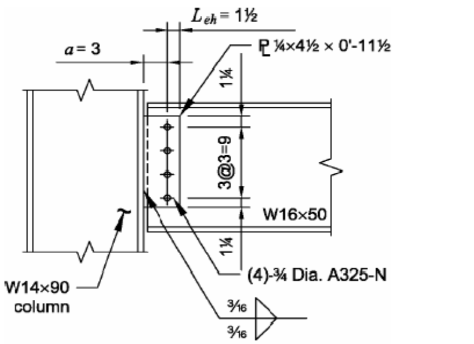


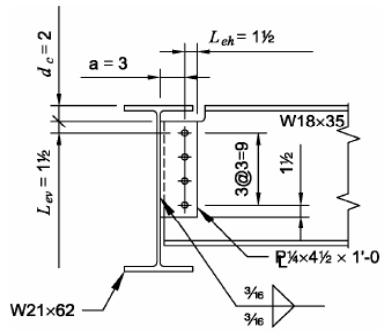
Example II.A-15 Stiffened Seated Connection (beam-to-column web)

Single-Plate Connection

Example II.A-17 Single-Plate Connection (conventional – beam-to-column flange)

For single-plate connection (AISC Table 10-9)



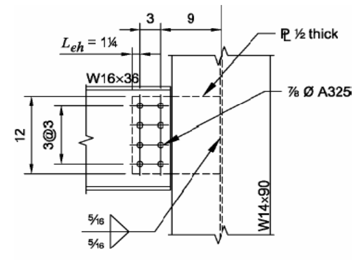


Example II.A-18 Single-Plate Connection (beam-to-girder web)

Single-Plate Connection

Example II.A-19 Extended Single-Plate Connection (beam-to-column web)

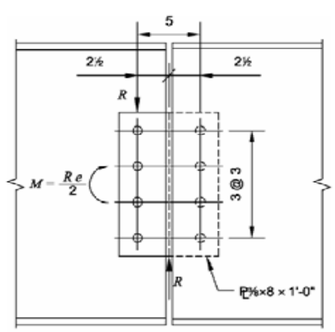
For extended single-plate connection



Single-Plate Shear Splice

Example II.A-20 All-Bolted Single-Plate Shear Splice

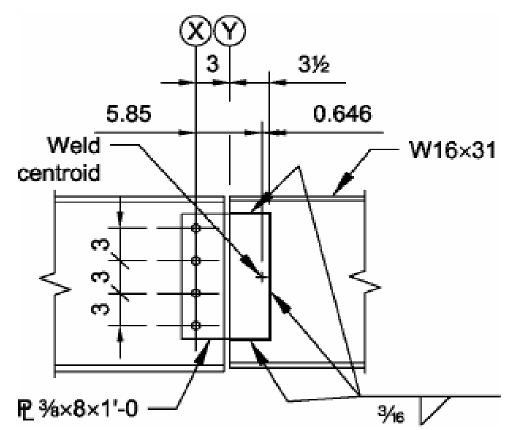
For all bolted shear splce



Single-Plate Shear Splice

Example II.A-21 Bolted/Welded Single-Plate Shear Splice

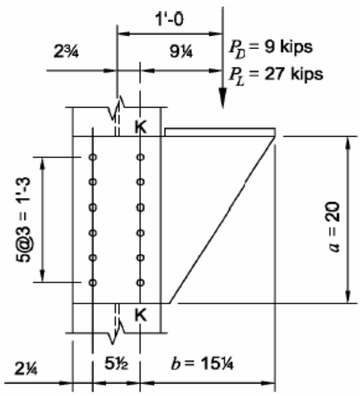
For welded shear splice



Bracket Plate Design

Example II.A-22 Bolted Bracket Plate Design

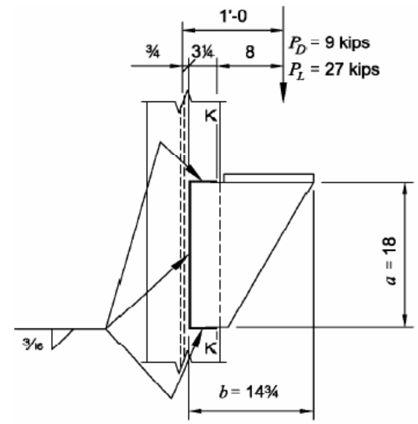
For bolt bracket plate



Bracket Plate Design

Example II.A-23 Welded Bracket Plate Design

For welded bracket plate



Eccentrically-Loaded Group

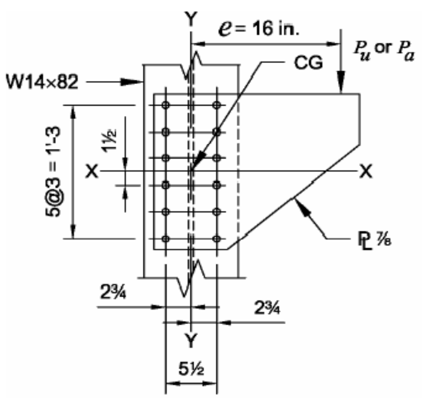
Example II.A-24 Eccentrically-Loaded Bolt Group (IC method)

Example II.A-25 Eccentrically Loaded Bolt Group (elastic method)

Elastic Method

(AISC Tables 7-7~14 for IC Method)

For bolt group



Eccentrically-Loaded Group

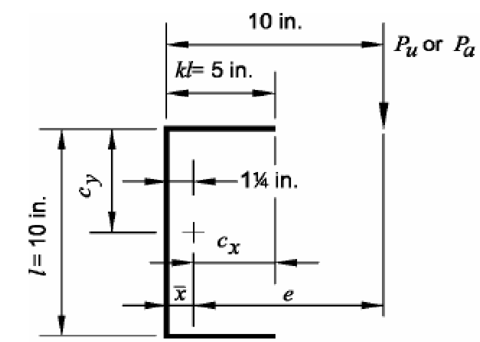
Example II.A-26 Eccentrically-Loaded Weld Group (IC method)

Example II.A-27 Eccentrically-Loaded Weld Group (elastic method)

Elastic Method

(AISC Tables 8-4~11 for IC Method)

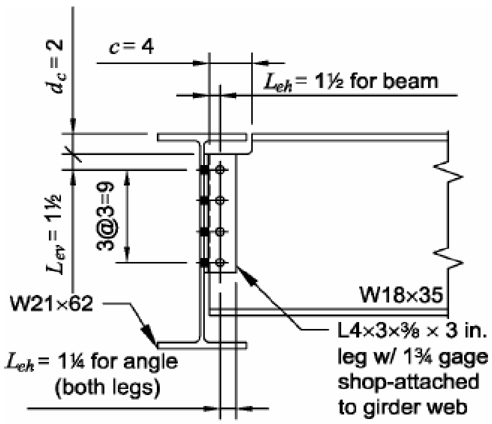
For welded group



Single-Angle Connection

Example II.A-28 All-Bolted Single-Angle Connection (beam-to-girder web)

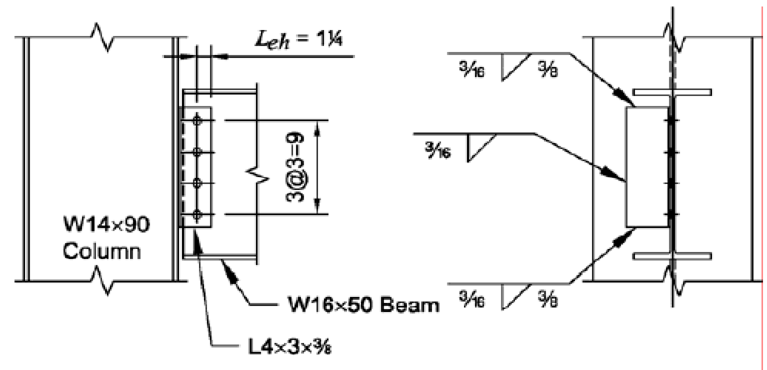
[For all bolted single-angle connection (AISC Table 10-10)]



Single-Angle Connection

Example II.A-29 Bolted/Welded Single-Angle Connection (beam-to-column flange).

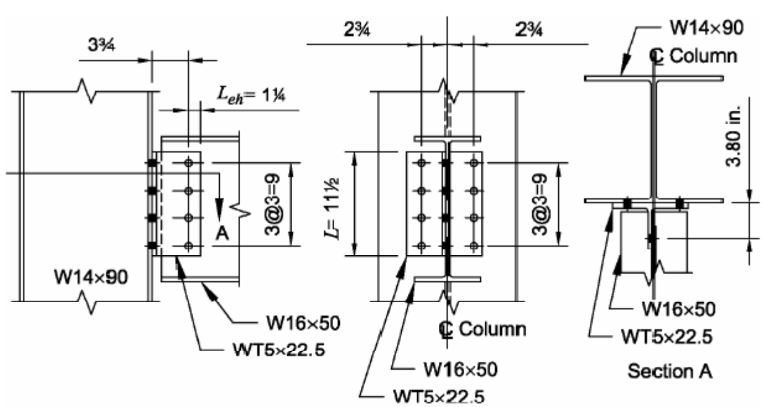
[For bolted/welded single-angle connection (AISC Tables 10-10 or -11)]



Tee Connection

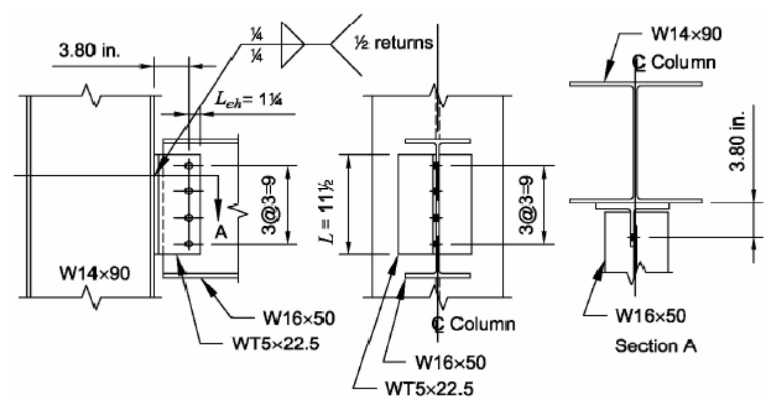
Example II.A-30 All-Bolted Tee Connection (beam-to-column flange)

[For all bolted tee connection]



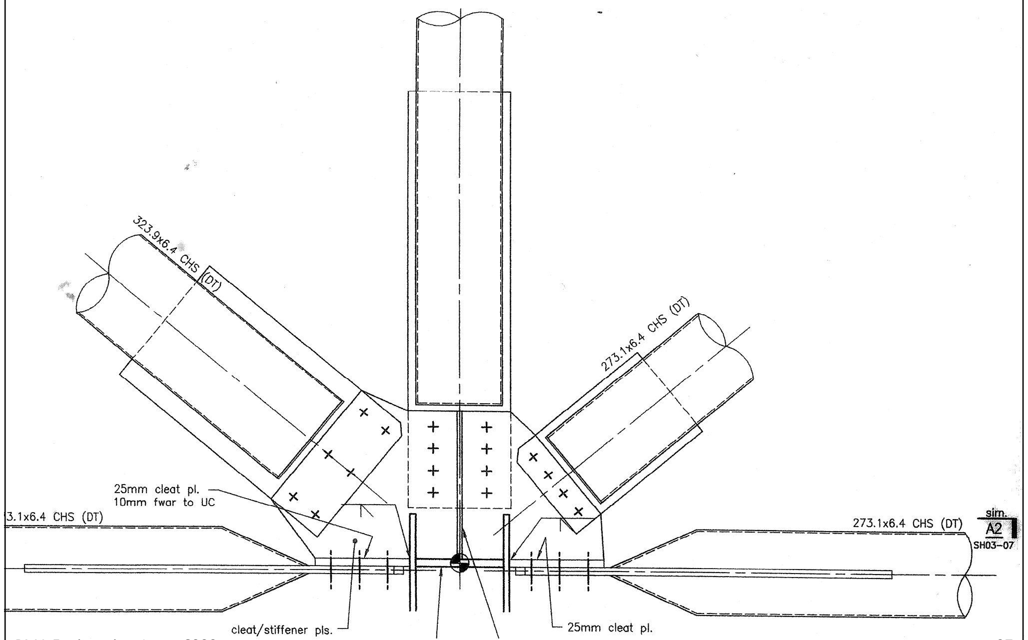
Tee Connection

[For bolted/welded tee connection]

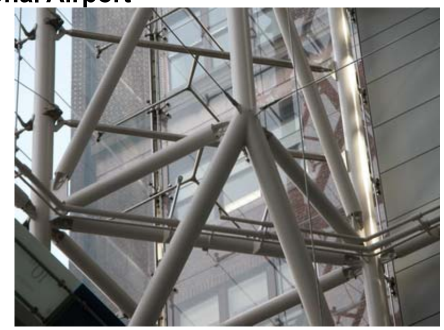


Example II.A-31 Bolted/Welded Tee Connection (beam-to-column flange)

CONNECTION JOINING TUBULAR MEMBERS (CHS) CIRCULAR HOLLOW SECTIONS




United Airlines Terminal
O'Hare International Airport

The Rose Center for Earth and Space, NY

6300. Design -

6320. Structural Steel Connections, Joints and Details

Objective and Scope Met

- Module 2: Connections
  - Types of Structural Bolts and Bolted Connections (Section NJ3 and AISC Manual Part 7)
  - AISC Connections (Section NJ and AISC Manual Part 9)
  - HSS and Box Member Connections (Section NK)
  - Selecting Standard Connections from the AISC Manual (AISC Manual Parts 9 & 10)

6320. Structural Steel Connections, Joints and Details – Module 4: Seismic Connections

This section of the module covers:

- Seismic Load Resisting Systems for Steel Buildings
  - Moment Resisting Frames
  - Concentrically Braced Frames
  - Eccentrically Braced Frames
  - Buckling Restrained Braced Frames
  - Special Plate Shear Walls
- Steel MRF Seismic Connection
  - Past
  - Present

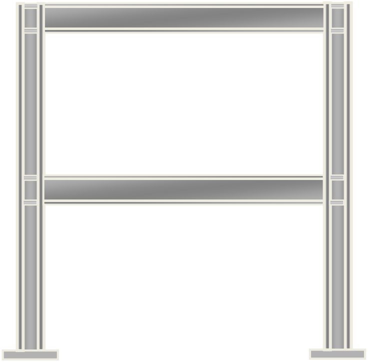
Seismic Load Resisting Systems for Steel Buildings

- Moment Resisting Frames
- Concentrically Braced Frames
- Eccentrically Braced Frames
- Buckling Restrained Braced Frames
- Special Plate Shear Walls

MOMENT RESISTING FRAME (MRF)

Beams and columns with moment resisting connections; resist lateral forces by flexure and shear in beams and columns - i.e. by frame action.

Develop ductility primarily by flexural yielding of the beams:



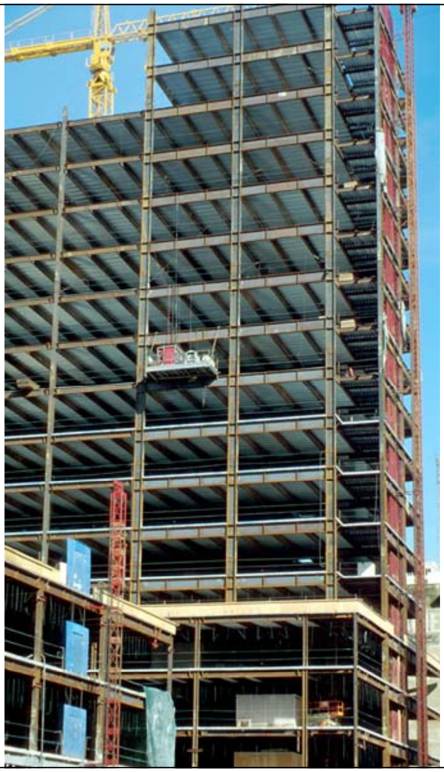
Advantages

- Architectural Versatility
- High Ductility and Safety

Disadvantages

- Low Elastic Stiffness

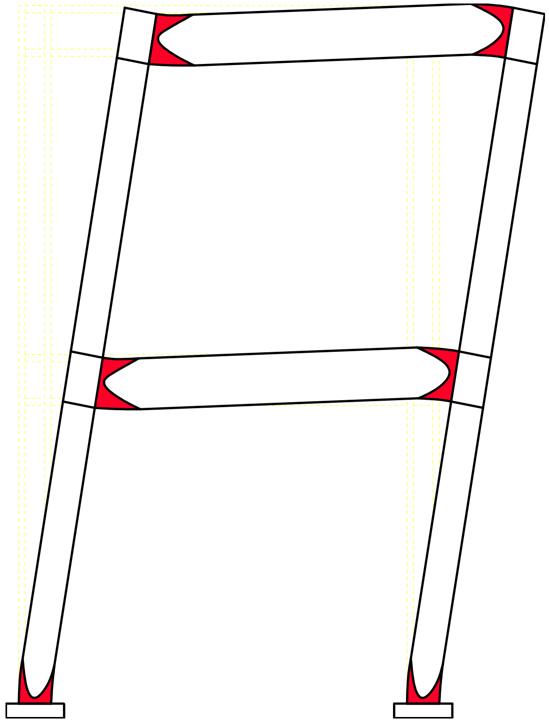
MOMENT RESISTING FRAME (MRF)



MOMENT RESISTING FRAME (MRF)

MOMENT RESISTING FRAME (MRF)

Inelastic Response of a Steel Moment Resisting Frame



Concentrically Braced Frames (CBFs)

Beams, columns and braces arranged to form a vertical truss.
Resist lateral earthquake forces by truss action.

Develop ductility through inelastic action in braces.

- braces yield in tension
- braces buckle in compression

<u>Advantages</u>

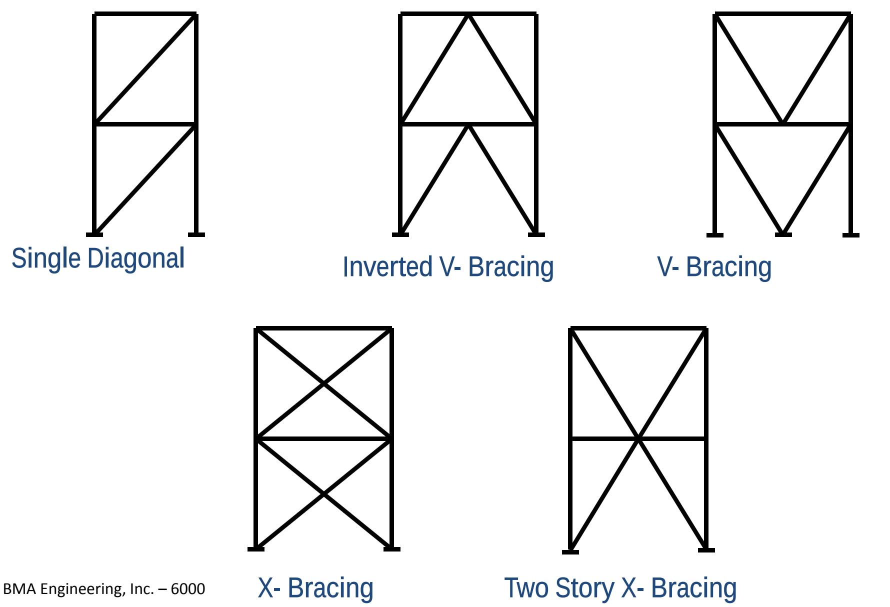
- high elastic stiffness

<u>Disadvantages</u>

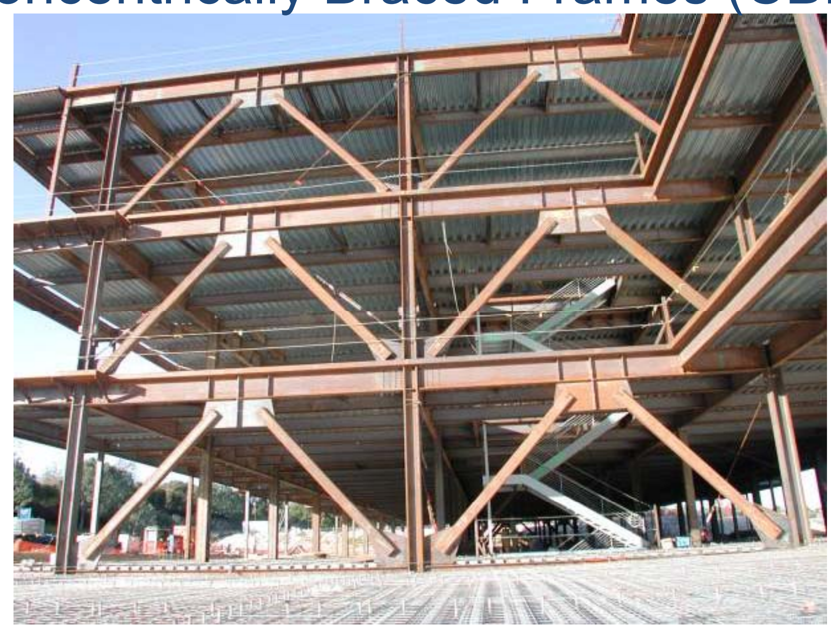
- less ductile than other systems (SMFs, EBFs, BRBFs)
- reduced architectural versatility

Concentrically Braced Frames (CBFs)

<u>Types of CBFs</u>



Concentrically Braced Frames (CBFs)

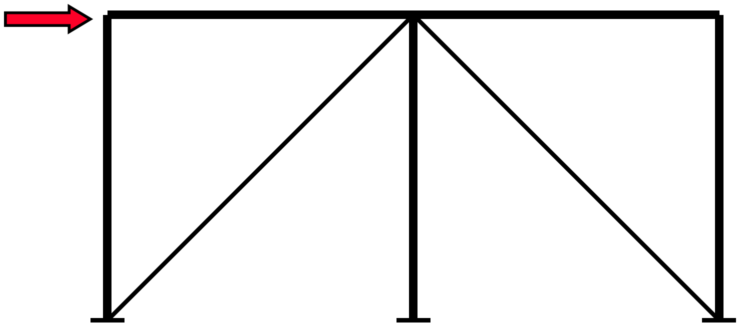


Concentrically Braced Frames (CBFs)

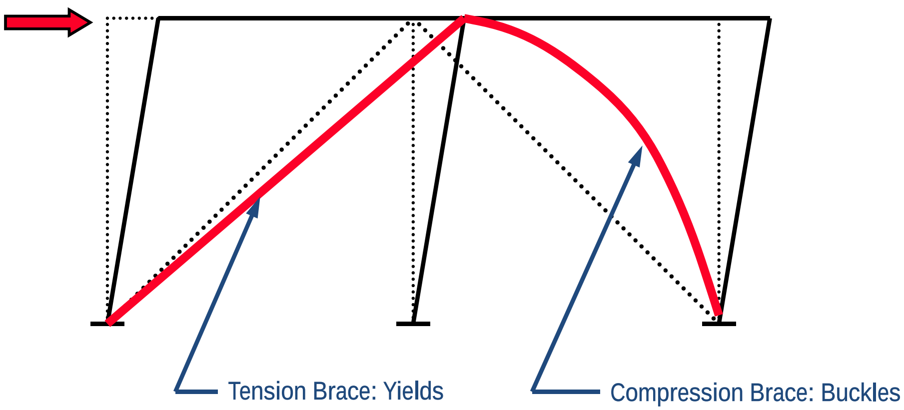
Concentrically Braced Frames (CBFs)

Concentrically Braced Frames (CBFs)

Inelastic Response of CBFs under Earthquake Loading

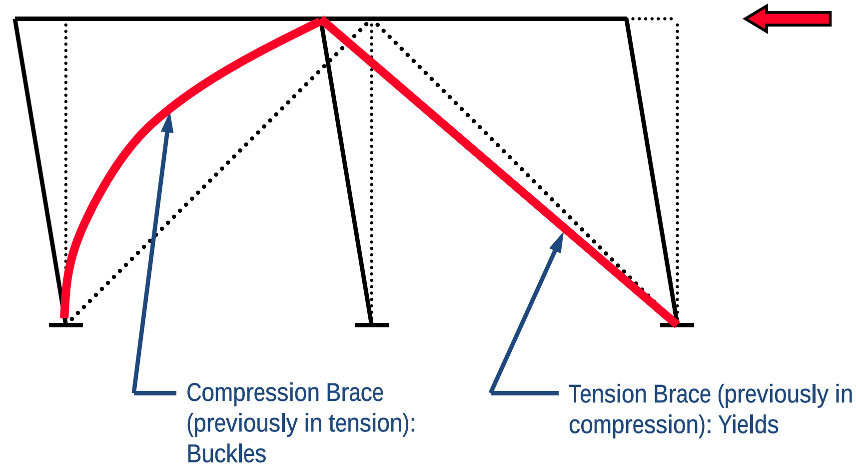


Inelastic Response of CBFs under Earthquake Loading



Columns and beams: remain essentially elastic

Inelastic Response of CBFs under Earthquake Loading

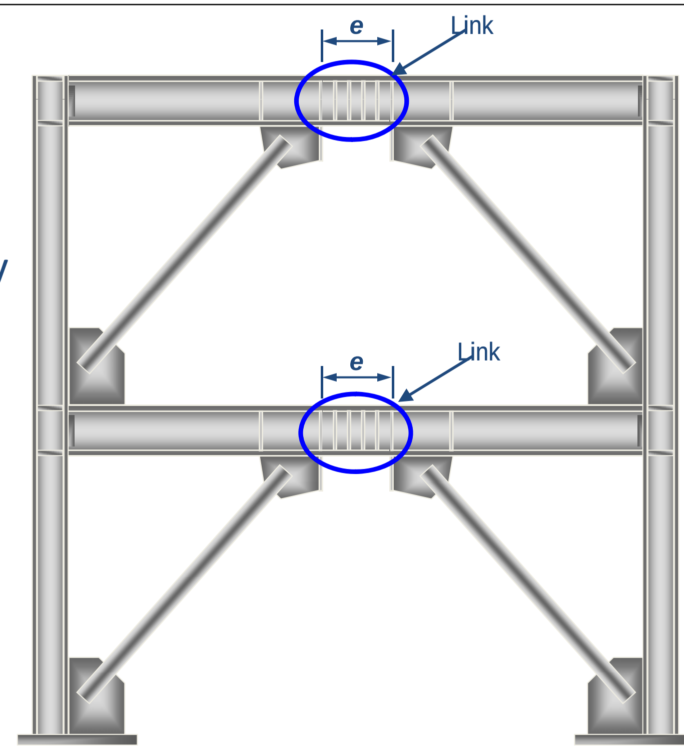


Columns and beams: remain essentially elastic

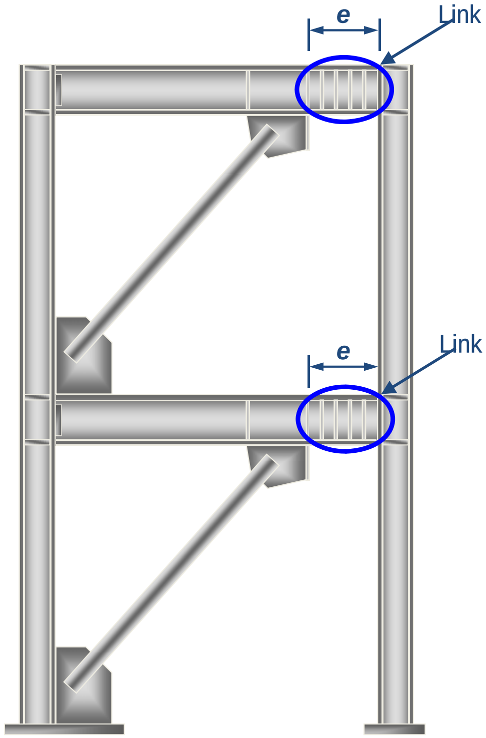
Eccentrically Braced Frames (EBFs)

- Framing system with beam, columns and braces. At least one end of every brace is connected to isolate a segment of the beam called a *link.*
- Resist lateral load through a combination of frame action and truss action. EBFs can be viewed as a hybrid system between moment frames and concentrically braced frames.
- Develop ductility through inelastic action in the links.
- EBFs can supply high levels of ductility (similar to MRFs), but can also provide high levels of elastic stiffness (similar to CBFs)

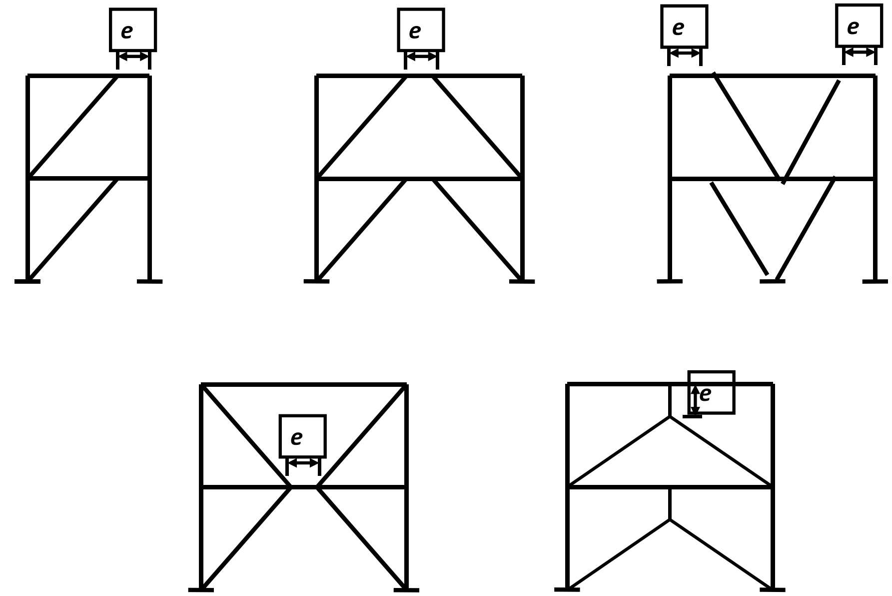
Eccentrically Braced Frames (EBFs)



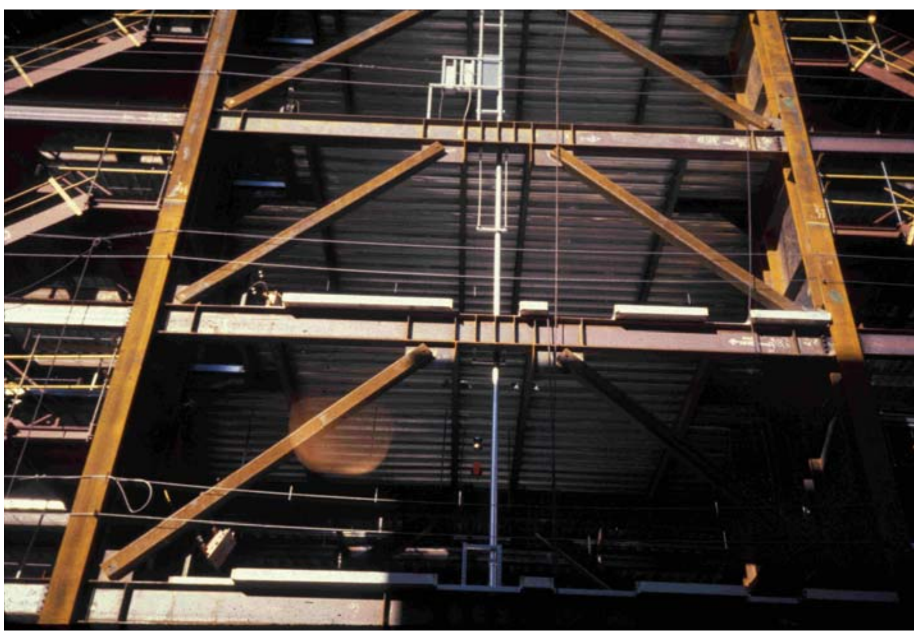
Eccentrically Braced Frames (EBFs)



Some possible bracing arrangement for EBFS



Eccentrically Braced Frames (EBFs)



Eccentrically Braced Frames (EBFs)

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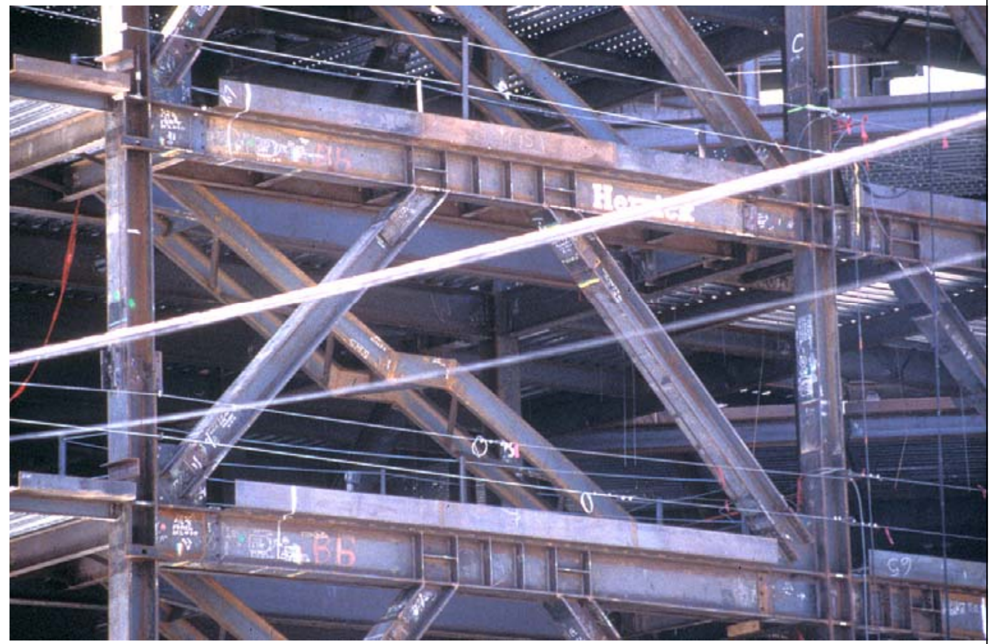
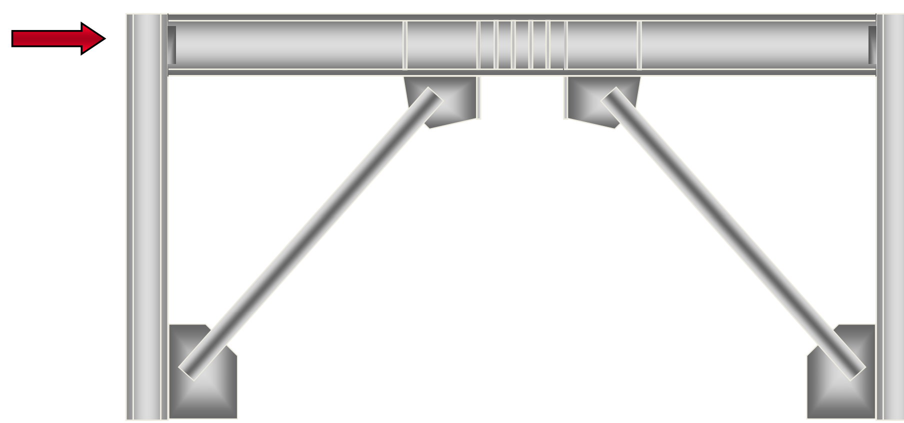
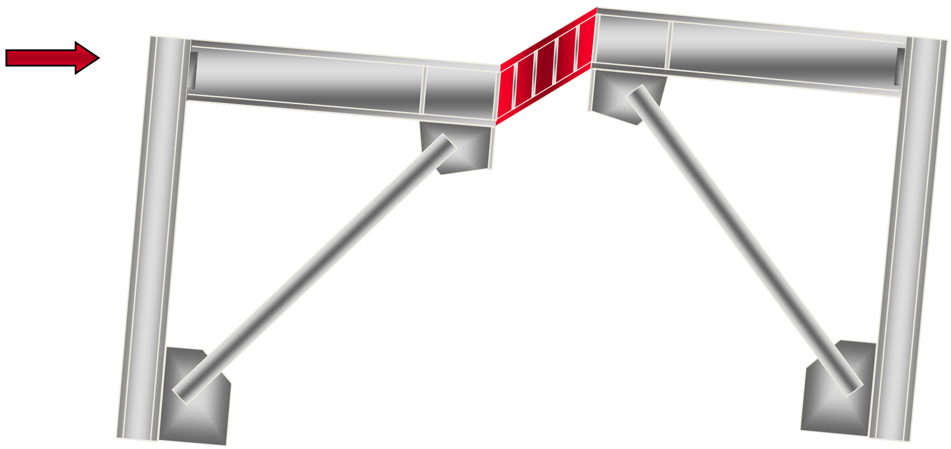


Eccentrically Braced Frames (EBFs)

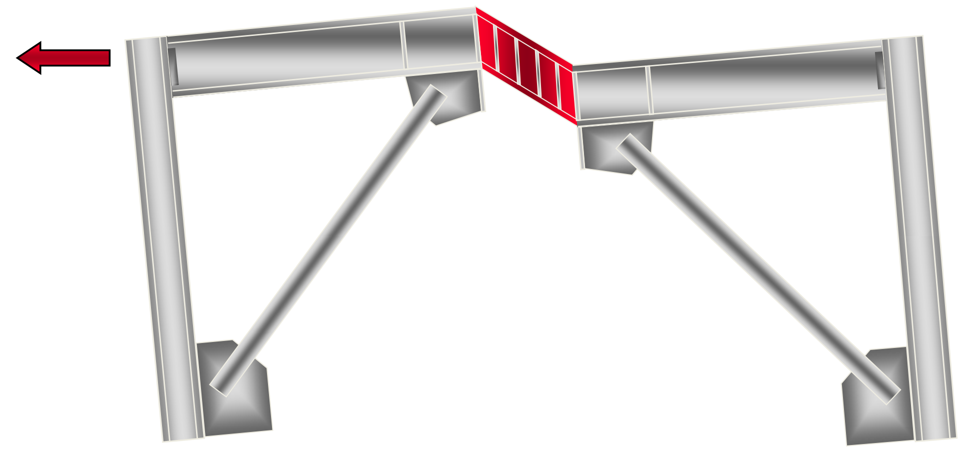
Inelastic Response of EBFs



Inelastic Response of EBFs



Inelastic Response of EBFs



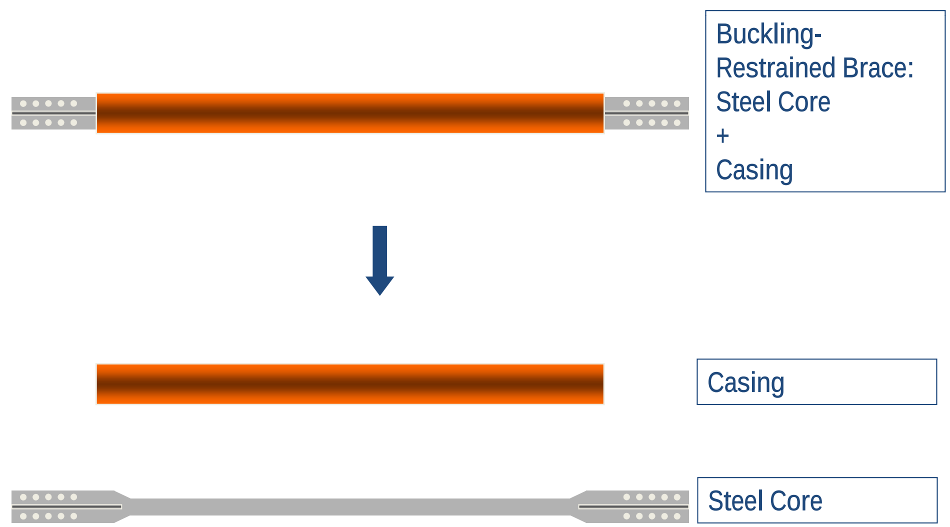
Buckling-Restrained Braced Frames (BRBFs)

- Type of concentrically braced frame.
- Beams, columns and braces arranged to form a vertical truss. Resist lateral earthquake forces by truss action.
- Special type of brace members used: *Buckling-Restrained Braces (BRBs)*. BRBS yield both in tension and compression - *no buckling !!*

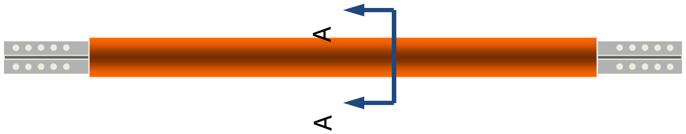
- Develop ductility through inelastic action (cyclic tension and compression yielding) in BRBs.

- System combines high stiffness with high ductility.

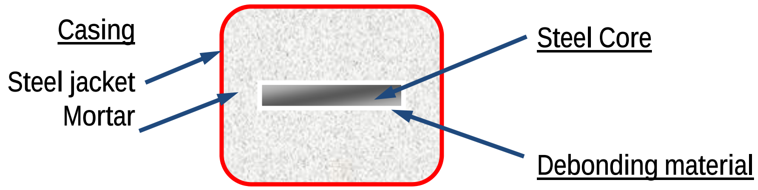
Buckling-Restrained Brace



Buckling-Restrained Brace



Buckling-Restrained Brace:
Steel Core
+
Casing



Section A-A

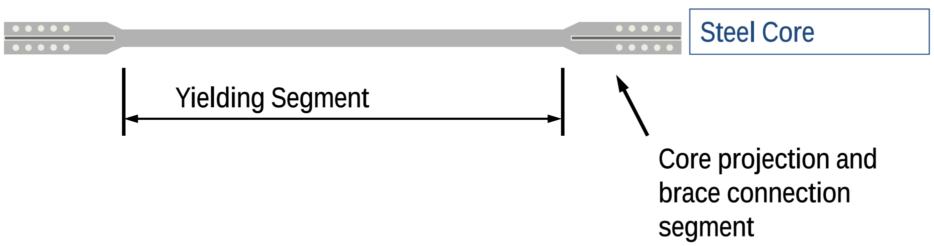
Buckling-Restrained Brace

Steel core resists entire axial force P

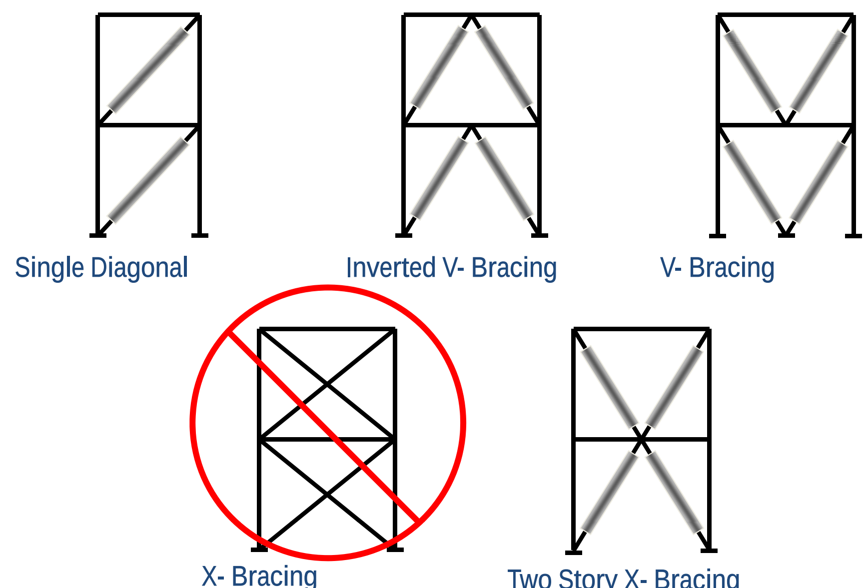
Casing is debonded from steel core
- casing does not resist axial force P
- flexural stiffness of casing restrains buckling of core

Buckling-Restrained Brace

Buckling-Restrained Brace:
Steel Core
+
Casing



Bracing Configurations for BRBFs



Buckling-Restrained Braced Frames (BRBFs)

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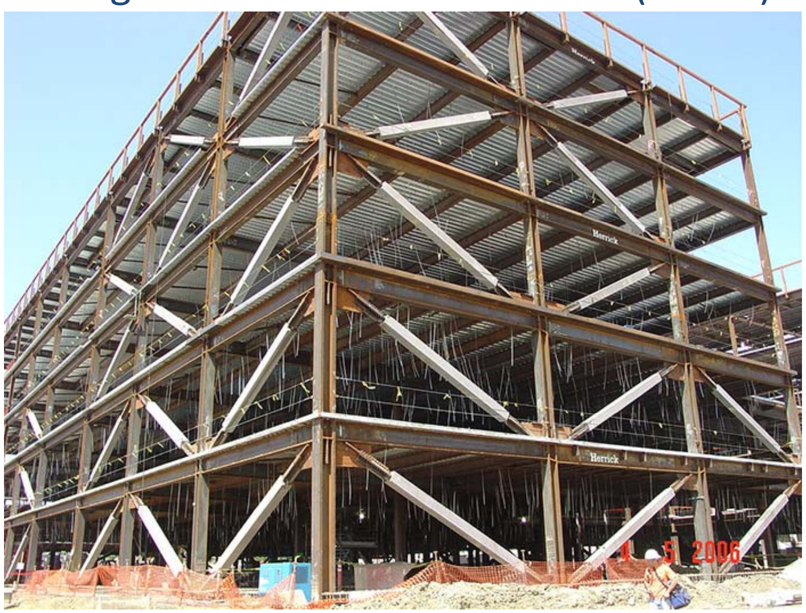
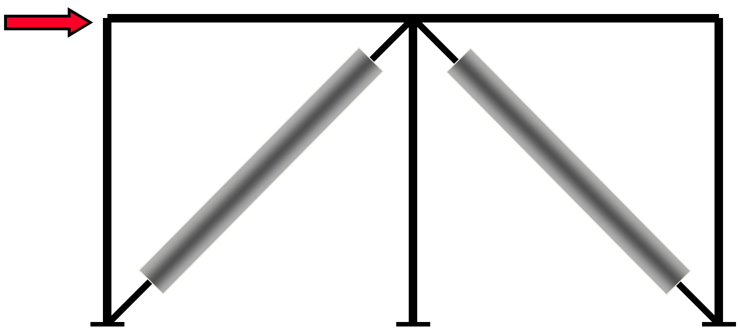
Buckling-Restrained Braced Frames (BRBFs)

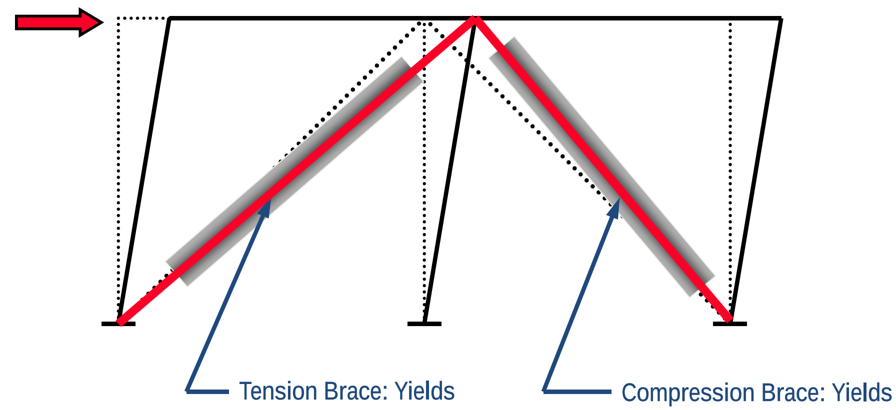


Buckling-Restrained Braced Frames (BRBFs)

Inelastic Response of BRBFs under Earthquake Loading

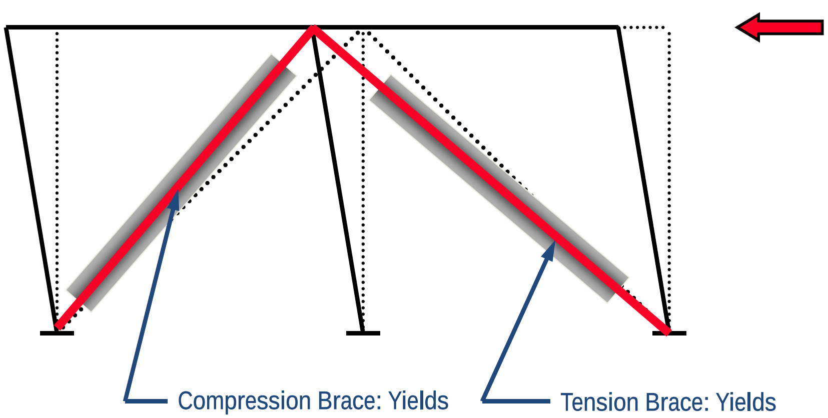


Inelastic Response of BRBFs under Earthquake Loading



Columns and beams: remain essentially elastic

Inelastic Response of BRBFs under Earthquake Loading

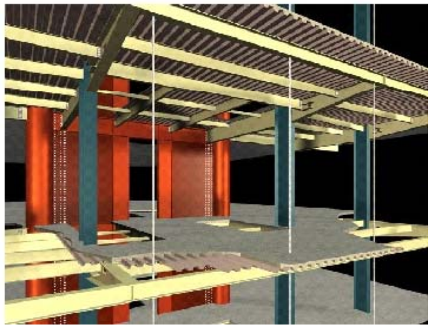


Columns and beams: remain essentially elastic

Special Plate Shear Walls (SPSW)

- Assemblage of consisting of rigid frame, infilled with thin steel plates.
- Under lateral load, system behaves similar to a plate girder. Wall plate buckles under diagonal compression and forms tension field.
- Develop ductility through tension yielding of wall plate along diagonal tension field.
- System combines high stiffness with high ductility.

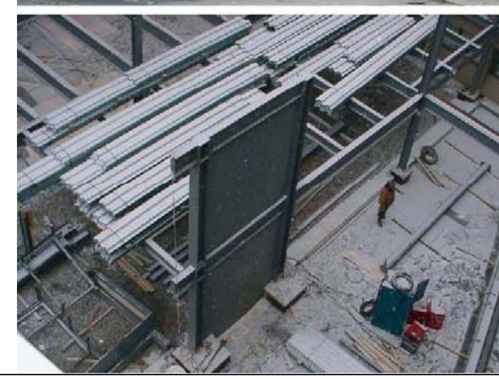
Special Plate Shear Walls (SPSW)



Special Plate Shear Walls (SPSW)

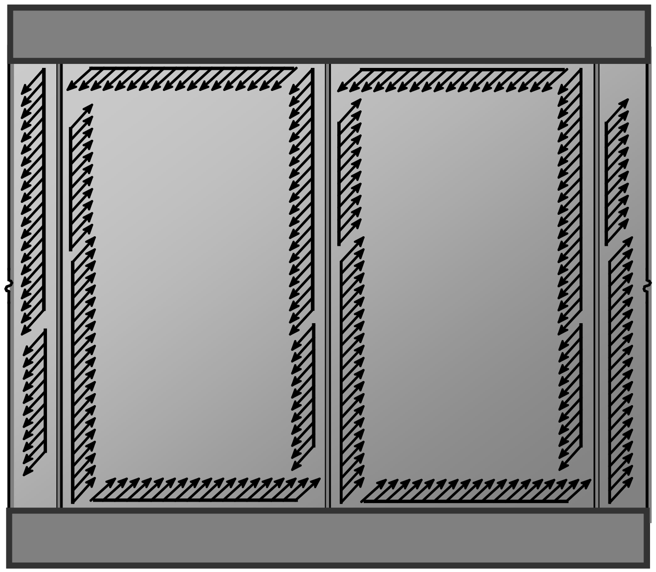
Special Plate Shear Walls (SPSW)

Special Plate Shear Walls (SPSW)

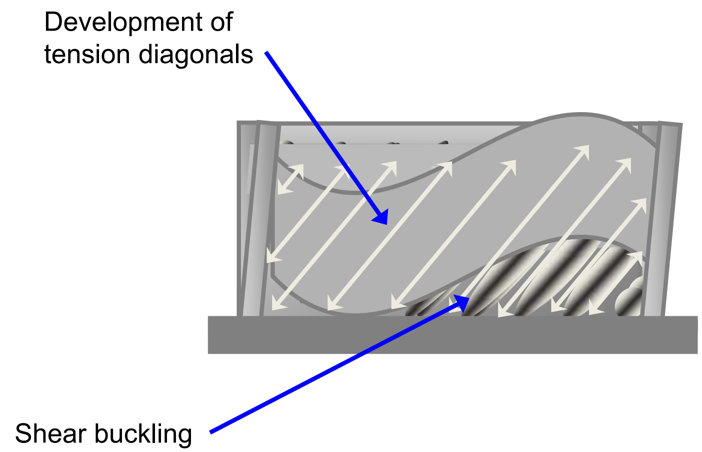


Special Plate Shear Walls (SPSW)

Plate-Girder Analogy



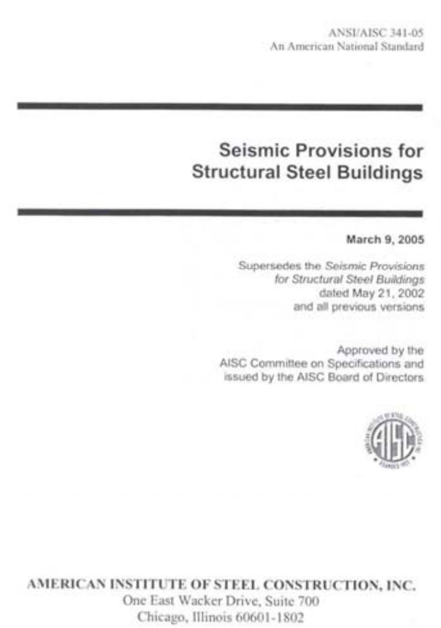
Inelastic Response of a SPSW



Design of Seismic-Resistant Steel Building Structures: A Brief Overview

- Earthquake Effects on Structures
- Performance of Steel Buildings in Past Earthquakes
- Building Code Philosophy for Earthquake-Resistant Design and Importance of Ductility
- Design Earthquake Forces: ASCE-7
- Steel Seismic Load Resisting Systems
- AISC Seismic Provisions

2005 AISC Seismic Provisions



ANSI/AISC 341-05
An American National Standard

**Seismic Provisions for
Structural Steel Buildings**

March 9, 2005

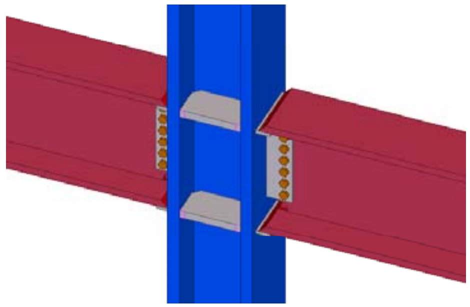
Supersedes the *Seismic Provisions for Structural Steel Buildings* dated May 21, 2002 and all previous versions

Approved by the AISC Committee on Specifications and issued by the AISC Board of Directors

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

STEEL MRF SEISMIC CONNECTION INTRO AND PRESENTATION OVERVIEW

- **Early development of steel moment connections**
- **Evolution to prequalified standard seismic steel moment connections**
- **Recent prescriptive seismic moment connection failures**
- **New AISC Seismic Provisions and prequalified connections**

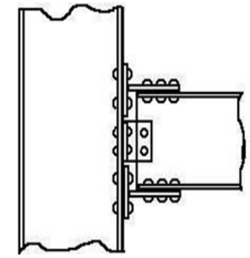


Ref: the AISC Seismic Provisions free at http://www.aisc.org/

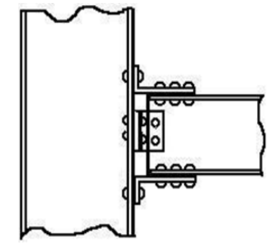
Ref: FEMA 350 free at http://www.fema.gov

EARLY DESIGN INFORMATION

- Early built up shapes gave way to rolled shapes and riveted connections in the 1920s
- Riveted steel connections: 1920s through the 1950s
  - –Angle and tee flange connections
- 1960s and 1970s earthquake resistant design philosophies began to be developed
- Buildings with these riveted connections performed satisfactorily when subjected to seismic loads
- No documented failures of these connections during the recent large-scale earthquake at Northridge in the United States

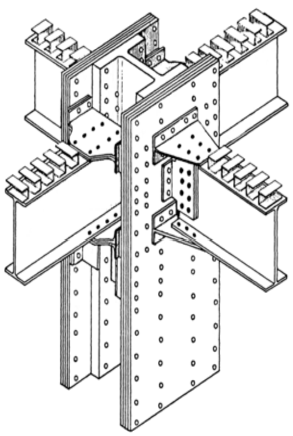


T-Stub Connection



Clip Angle Connection

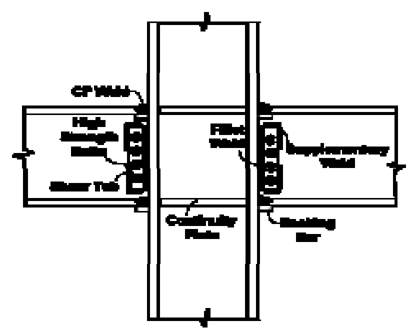
RIVETED MOMENT CONNECTION PERFORMANCE

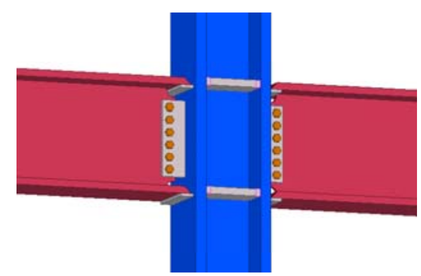


- **Results of later cyclic testing performed on the tee stub and clip angle riveted connections include the following:**
  - **Performed as partially restrained connections with the T-stub connector being stiffer**
  - **Good rotational capacity**
  - **The failure mode or yield mechanism had a direct correlation to the connection ductility**
  - **The fireproofing concrete encasement of the steel sections increased connection strength through composite action**
- **Good connection performance attributed to:**
  - **Utilization at all beam to column interfaces**
  - **Steel frames infilled with masonry partitions**
  - **Steel generally encased in concrete for fire resistance**

PREQUALIFIED BOLTED/WELDED CONNECTIONS (1960s THROUGH NORTHRIDGE)

- **Prescriptive Moment Connection**
  - **Welded flange and bolted web**
  - **Adopted by UBC in 1970s**
  - **Expected to have good ductile behavior**
  - **Develop full plastic moment of beam**





- **Monotonic and cyclic loading tests predominantly showed the connection as ductile with more than adequate rotation**
- **These tests formed the basis for the prequalified welded flange-bolted web fully restrained moment connection and further defined the design requirements**
- **Prequalified for all seismic demands**

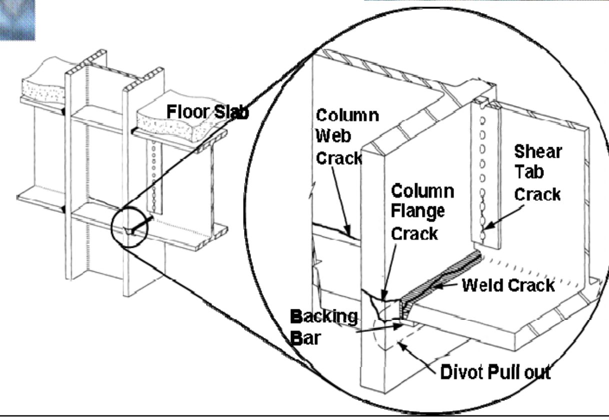
SMF CONNECTION EVOLUTION

The prequalified welded flange-bolted web moment resisting connection remained the standard despite changes within the steel industry standard design practice. Notably the following changes took place [Stojadinovic et al, 2000]:

- The moment connections were reduced from every connection to very few due to the labor costs involved in producing the connections;
- The number of moment resisting frames present in buildings were reduced to a minimum of one in each orthogonal direction with the remaining only shear connections compared to the past which had all frames resisting lateral forces;
- The moment resisting frames were moved toward the outside of the structure;
- Greater loading, longer spans and fewer moment resisting frames required much larger columns and deeper beams than tested in the past;
- The yield and ultimate strength of steel increased;
- Bolting the shear tab to the beam web without supplemental welds became the norm due to economic considerations;
- The welding process was changed from shielded metal arc welding (SMAW) to self-shielded flux core metal arc welding (FCAW) during the 1970s.

These changes led to under designed connections that were not tested in their exact condition

NORTHRIDGE FAILURES



NORTHRIDGE FAILURES

- **The Northridge, California earthquake of January 1994 and later the Kobe, Japan earthquake of January 1995 caused brittle fractures in many cases within the prequalified connections at very low levels of plastic demand**
- **Led to later investigation of structures subjected to previous earthquakes**
- **The experimental results from the 1970s through the present were evaluated**
- **There were also numerous factors observed in the field that contributed to the failure of these connections**
- **Inspection of the structures after the Northridge earthquake indicated that brittle fractures initiated within the connections at very low levels of plastic demand and in some cases while the structure remained elastic**
- **Commonly initiated at the complete joint penetration (CJP) weld**

SAC JOINT VENTURE

Structural Engineers Association of California (SEAOC)
Applied Technology Council (ATC)
California Universities for Research in Earthquake engineering (CUREe)

Before Northridge

- **Steel buildings considered to be "invulnerable"**
- **Best earthquake resisting system**

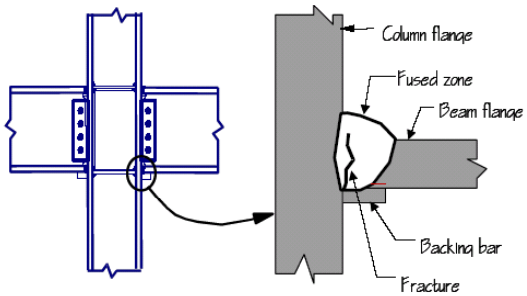
After Northridge

- **"Pre-qualified" connections withdrawn**
- **Interim Guidelines, workshops/conferences**
- **New connections to be validated by testing**

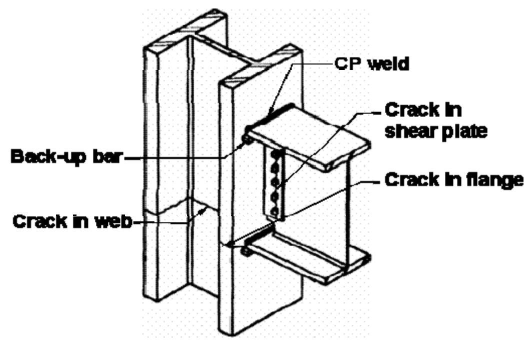
After 2000

- **Improved prescriptive connections**
- **FEMA 350: Recommendations**
- **2002 AISC Seismic Provisions**

SAC I: STUDY OF OLD/NORTHRIDGE FAILURES



Typical Fracture initiated at the CJP at the bottom flange [FEMA350]



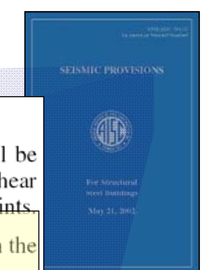
SAC I: STUDY OF OLD/NORTHRIDGE FAILURES

- Greatest stresses at the column to beam interface
- Bottom flange weld is a down hand weld performed by welder sitting on top of beam
- Difficult visual as well as ultrasonic inspection.
- Excessively weak panel zones result in local kinking of the column flanges and significant demand on the CJP weld between the beam and column flanges
- Severe strain concentrations can occur at the weld access holes for the beam flanges
- Change in the welding method produced welds with low toughness and welders were able to deposit more weld in one pass, which led to large weld defects
- Lateral force resisting systems evolved to utilize less moment frames than in the past requiring the use of deeper beams and heavier columns
- Use of recycled scrap metal resulted in steel with much greater yield strength than required which led to under designing the connections

SAC PROJECT II: NEW PROVISIONS AFTER NORTHRIDGE

- Part II of the SAC project: develop guidelines for future steel moment connection detailing and design to improve their performance
  - Provide a controlled yield mechanism and failure mode for each recommended and prequalified connection
  - The connections shall allow the building to sustain large inelastic deformations without collapse or loss of life during major earthquakes
- SAC finding published by FEMA (350) and utilized by AISC to produce the Seismic Provisions

7.2 BOLTED JOINTS



7.2. Bolted Joints

All bolts shall be pretensioned high-strength bolts. All faying surfaces shall be prepared as required for Class A or better Slip-Critical Joints. The design shear strength of bolted joints is permitted to be calculated as that for bearing-type joints.

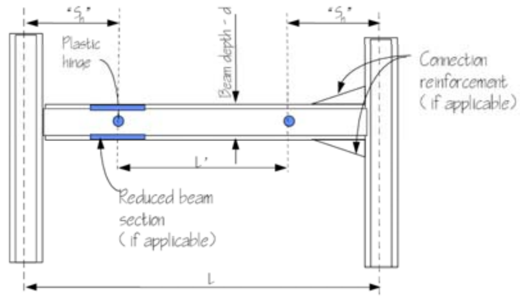
Bolted joints shall not be designed to share load in combination with welds on the same faying surface.

The bearing strength of bolted joints shall be provided using either standard holes or short-slotted holes with the slot perpendicular to the line of force, unless an alternative hole type is justified as part of a tested assembly; see Appendix S.

The Design Strength of bolted joints in shear and/or combined tension and shear shall be determined in accordance with LRFD Specification Sections J3.7 and J3.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.

Bolted connections for members that are a part of the Seismic Load Resisting System shall be configured such that a ductile limit-state either in the connection or in the member controls the design.

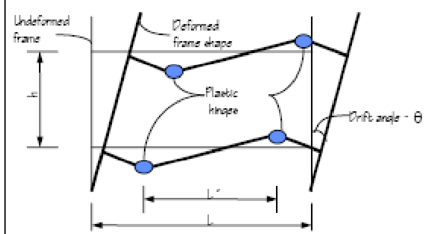
ZONE OF PLASTIC DEFORMATION



Location of plastic hinge formation (S_h)

- **S_h value Identified within each prequalified connections**
- **Welded, bolted, screwed or shot-in attachments, exterior facades, partitions, ductwork, piping or other construction openings shall not be placed within the expected zone of plastic deformation due to the regions sensitivity to discontinuities**

INTERSTORY DRIFT/DESIGN



Inelastic Behavior of Frames with Hinges in Beam Span [FEMA350]

– Achieved through combination of elastic deformation and development of plastic hinges

– Shall be capable of sustaining a drift angle of at least 0.04 radians

Strong-Column-Weak-Beam

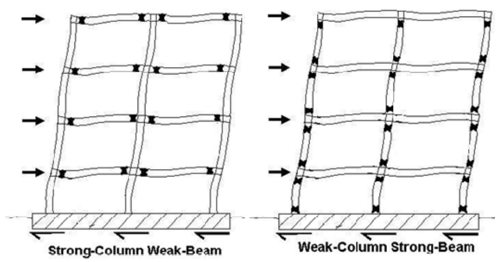
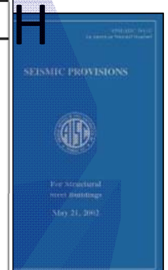


TABLE I-6-1
R_y Values for Different Member Types

Application	R_y
Hot-rolled structural shapes and bars	
ASTM A36/A36M	1.5
ASTM A572/A572M Grade 42 (290)	1.3
ASTM A992/A992M	1.1
All other grades	1.1
Hollow Structural Sections	
ASTM A500, A501, A618 and A847	1.3
Steel Pipe	
ASTM A53/A53M	1.4
Plates	1.1
All other products	1.1

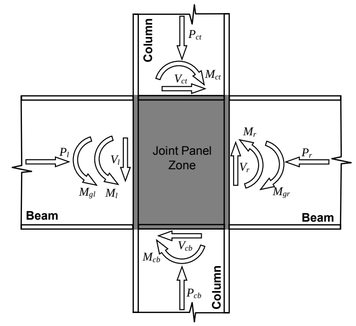


6.2. Material Properties for Determination of Required Strength

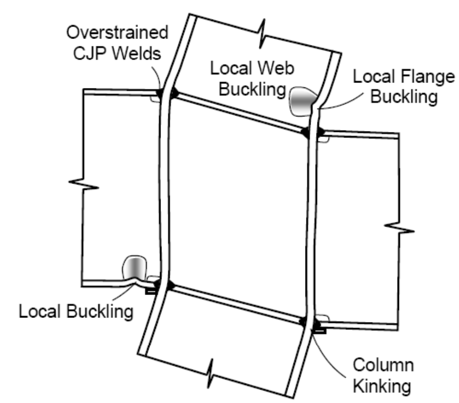
When required in these Provisions, the Required Strength of a connection or member shall be determined from the Expected Yield Strength $R_y F_y$, of the connected member, where F_y is the specified minimum yield strength of the grade of steel to be used. For rolled shapes and bars, R_y shall be as given in Table I-6-1. Other values of R_y are permitted to be used if the value of the Expected Yield Strength is determined by testing that is conducted in accordance with the requirements for the specified grade of steel.

When both the Required Strength and the Design Strength calculations are made for the same member or connecting element, it is permitted to apply R_y to F_y in the determination of the Design Strength.

BEAM-TO-COLUMN PANEL ZONE

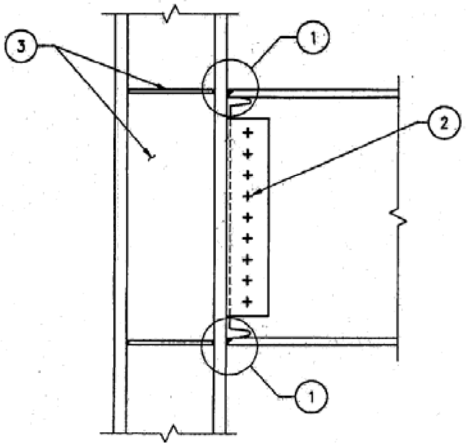


Internal forces on JPZ (axial, shearing, flexure)



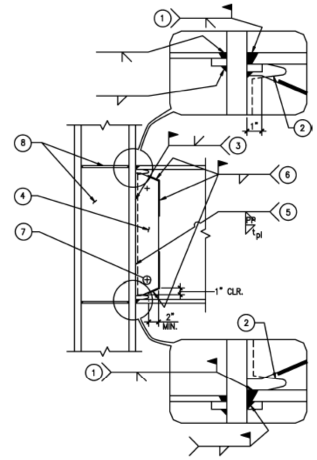
- Effects of JPZ shear distortion: Local buckling in the beam and column flanges due to excessive distortion of the JPZ. This can lead to fracture of the CJP groove welds due to the high strains and increased story drift leading to more damage, greater susceptibility to P-Δ effects and large permanent offsets of building frames.
- Shear yielding of the JPZ shall initiate at the same time as flexural yielding of the beam elements or proportioned so that all yielding occurs in the beam.

WELDED UNREINFORCED FLANGE BOLTED WEB (WUF-B) CONNECTION



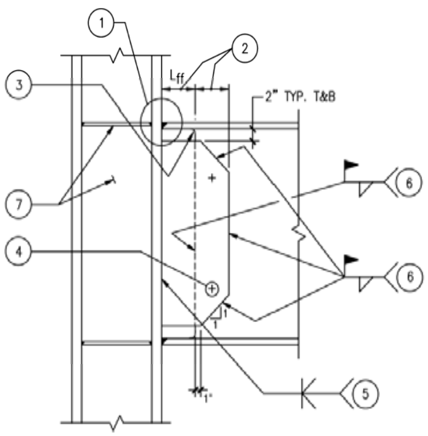
Geometric Limits of FEMA 350 prequalified connection [FEMA 350]					
Type	Frame	Maximum Beam Size	Min. Span (l)to Depth (d_b) Ration (l/d_b)	Max. Beam Flange Thickness (t_{bf}) in	Max. Column Size
WUF-B	OMF	W36	7	1	W8,W10,W12,W14

WELDED UNREINFORCED FLANGE WELDED WEB (WUF-W) CONNECTION



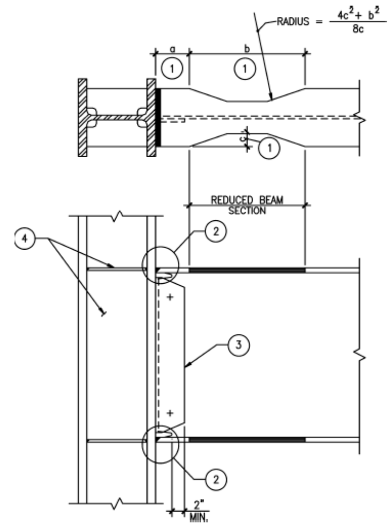
Geometric Limits of FEMA 350 prequalified connection [FEMA 350]					
Type	Frame	Maximum Beam Size	Min. Span (l)to Depth (d_b) Ration (l/d_b)	Max. Beam Flange Thickness (t_{bf}) in	Max. Column Size
WUF-W	OMF	W36	5	1.5	No Limit
	SMF	W36	7	1	W12, W14

WELDED FREE FLANGE (FF) CONNECTION



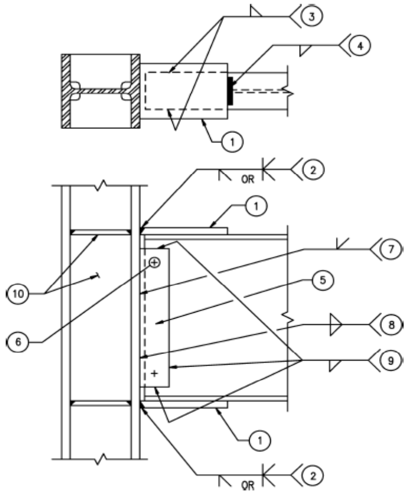
Geometric Limits of FEMA 350 prequalified connection [FEMA 350]					
Type	Frame	Maximum Beam Size	Min. Span (l)to Depth (d_b) Ration (l/d_b)	Max. Beam Flange Thickness (t_{bf}) in	Max. Column Size
WFF	OMF	W36	5	1.25	No Limit
	SMF	W30	7	0.75	W12, W14

REDUCED BEAM SECTION (RBS) CONNECTION



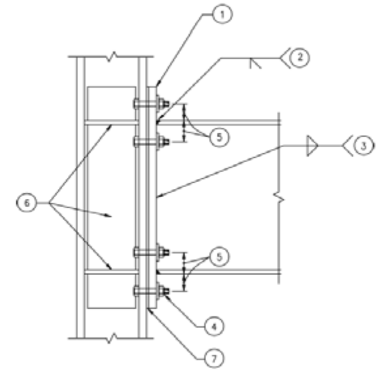
Geometric Limits of FEMA 350 prequalified connection [FEMA 350]					
Type	Frame	Maximum Beam Size	Min. Span (l)to Depth (d_b) Ration (l/d_b)	Max. Beam Flange Thickness (t_{bf}) in	Max. Column Size
RBS	OMF	W36	5	1.75	No Limit
	SMF	W36	7	1.75	W12, W14

WELDED FLANGE PLATE (WFP) CONNECTION



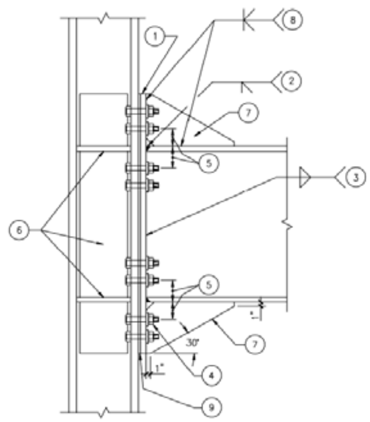
Geometric Limits of FEMA 350 prequalified connection [FEMA 350]					
Type	*Frame*	*Maximum Beam Size*	*Min. Span (l)to Depth (d_b) Ration (l/d_b)*	*Max. Beam Flange Thickness (t_{bf}) in*	*Max. Column Size*
WFP	OMF	W36	5	1.5	No Limit
	SMF	W36	7	1	W12, W14

BOLTED UNSTIFFENED END PLATE (BUEP) CONNECTION



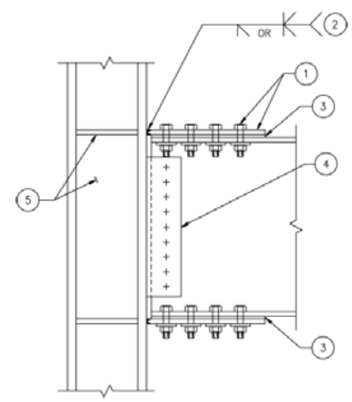
Geometric Limits of FEMA 350 prequalified connection [FEMA 350]					
Type	*Frame*	*Maximum Beam Size*	*Min. Span (l)to Depth (d_b) Ration (l/d_b)*	*Max. Beam Flange Thickness (t_{bf}) in*	*Max. Column Size*
BUEP	OMF	W30	5	0.75	No Limit
	SMF	W24	7	0.75	W8,W10,W12,W14

BOLTED STIFFENED END PLATE CONNECTION (BSEP)



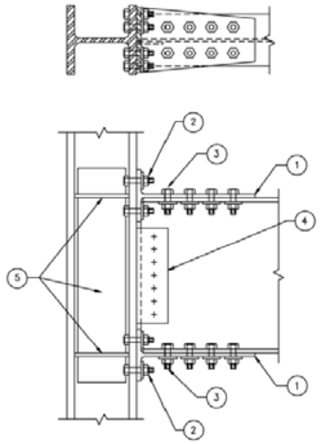
Geometric Limits of FEMA 350 prequalified connection [FEMA 350]					
Type	*Frame*	*Maximum Beam Size*	*Min. Span (l)to Depth (d_b) Ration (l/d_b)*	*Max. Beam Flange Thickness (t_{bf}) in*	*Max. Column Size*
BSEP	OMF	W36	5	1	No Limit
	SMF	W36	7	1	W12, W14

BOLTED FLANGE PLATE (BFP) CONNECTION



Geometric Limits of FEMA 350 prequalified connection [FEMA 350]					
Type	*Frame*	*Maximum Beam Size*	*Min. Span (l)to Depth (d_b) Ration (l/d_b)*	*Max. Beam Flange Thickness (t_{bf}) in*	*Max. Column Size*
BFP	OMF	W36	5	1.25	No Limit
	SMF	W30	8	0.75	W12, W14

DOUBLE SPLIT TEE (DST) CONNECTION



Type	Frame	Maximum Beam Size	Min. Span (l) to Depth (d_b) Ration (l/d_b)	Max. Beam Flange Thickness (t_{bf}) in	Max. Column Size
DST	OMF	W36	5	---	No Limit
	SMF	W24	8	---	W12, W14

Geometric Limits of FEMA 350 prequalified connection [FEMA 350]

6300. Design -

6320. Structural Steel Connections, Joints and Details

Objective and Scope Met

- Module 4: Seismic Connections
  - Seismic Load Resisting Systems for Steel Buildings
    - Moment Resisting Frames
    - Concentrically Braced Frames
    - Eccentrically Braced Frames
    - Buckling Restrained Braced Frames
    - Special Plate Shear Walls
  - Steel MRF Seismic Connection
    - Past
    - Present

6000. STEEL

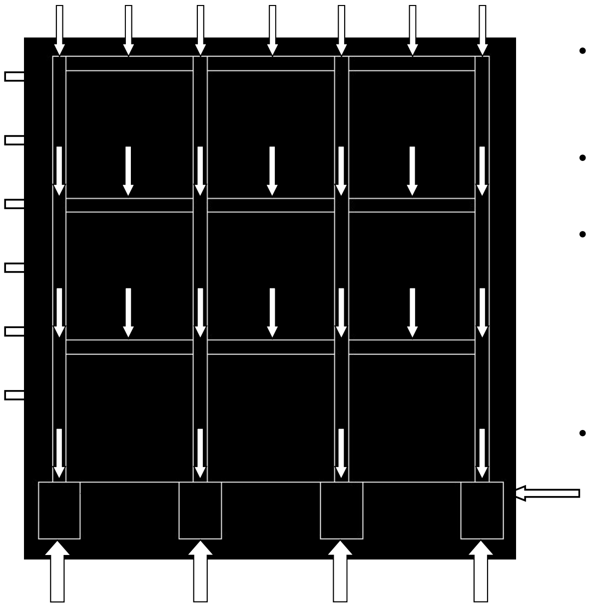
- 6100 & 6200
  - 6130 - Design Data, Principles and Tools
  - 6140 - Codes and Standards
  - 6200 - Material
- 6300
  - 6310 - Members and Components
  - 6320 - Connections, Joints and Details
  - 6330 - Frames and Assembles
- 6400
  - 6410 - AISC Specifications for Structural Joints
  - 6420 - AISC 303 Code of Standard Practice
  - 6430 - AWS D1.1 Structural Welding Code
- 6500
  - 6510 - Nondestructive Testing Methods
  - 6520 - AWS D1.1 Structural Welding Code Tests
- 6600
  - 6610 - Steel Construction
  - 6620/6630 - NUREG-0800 / RG 1.94

6300. Design -

6330. Structural Steel Frames and Assembles

- Types of Steel Frames
- Stability Bracing (Section NC and Appendix N6)
- Elastic and Inelastic Behavior
- Seismic Analysis

Forces On Structures

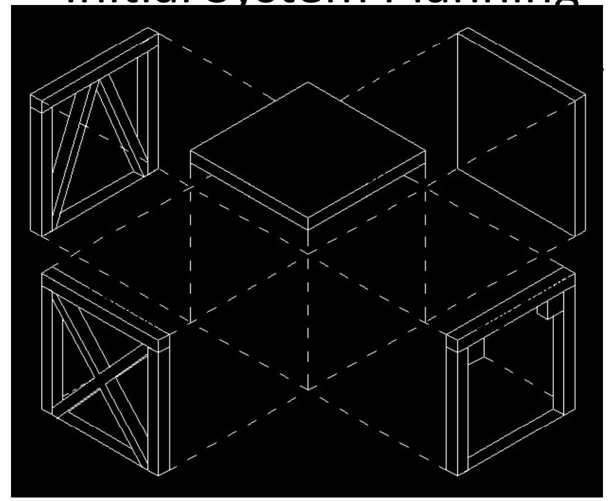


Structural Steel Frame Elevation

- Forces from gravity, wind, and seismic events are imposed on all structures
- Forces that act vertically are gravity loads
- Forces that act horizontally, such as stability, wind and seismic events (the focus of this discussion) require lateral load resisting systems to be built into structures
- As lateral loads are applied to a structure, horizontal diaphragms (floors and roofs) transfer the load to the lateral load resisting system

(AISC 2002)

Initial System Planning



- The type of lateral load resisting system to be used in a structure should be considered early in the planning stage
- Lateral stability as well as architectural needs must be met
- The three common lateral load resisting systems are:
  1. Braced Frames 2. Rigid Frames 3. Shear Walls

Braced Frames and Rigid Frames

This presentation focuses on braced frames (left) and rigid frames (right)

Types of Steel Frames and Assemblies

The three classes of construction based on the type of structural connections are as follows:

- Type 1: Rigid frame
- Type 2: Simple (flexible) frame
- Type 3: Semi-rigid frame

Type 1: Rigid Frame Construction

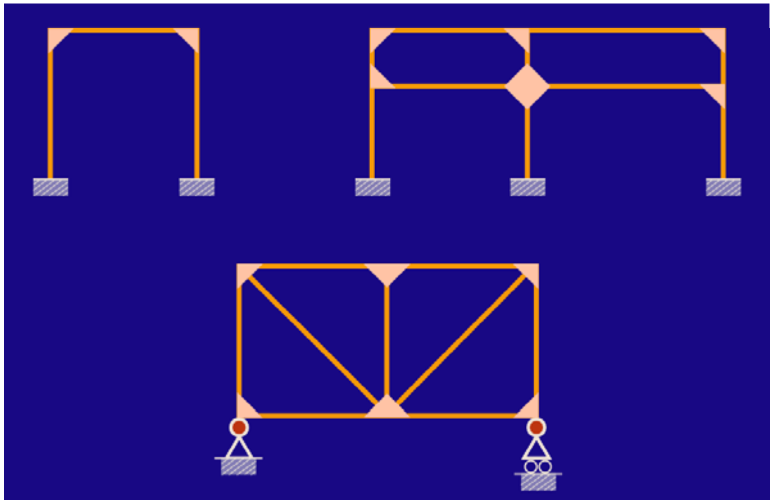


Figure 1

Type 2: Simple (Flexible) Frame

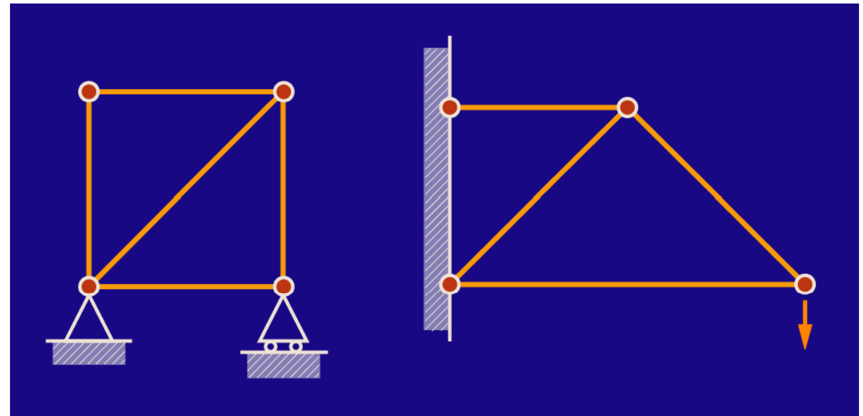


Figure 2

Type 3: Semi-Rigid Frame

Semi-rigid connections have some moment carrying capacity, but it is insufficient to develop full continuity

Braced and Unbraced Frames

– The effective length of column members (KL) for a braced frame is equal to or less than the actual length

– The effective length of column members for an unbraced frame is always greater than the actual length

Braced Frame

A braced frame is defined as a frame in which sidesway buckling is prevented by bracing elements of the structure other than the structural frame itself

Rigid Frames

- Rigid frames, utilizing moment connections, are well suited for specific types of buildings where diagonal bracing is not feasible or does not fit the architectural design
- Rigid frames generally cost more than braced frames (AISC 2002)

Braced Frames

- Diagonal bracing creates stable triangular configurations within the steel building frame (AISC 2002)
- "Braced frames are often the most economical method of resisting wind loads in multi-story buildings (AISC 1991)."
- Some structures, like the one pictured above, are designed with a combination braced and rigid frame to take advantage of the benefits of both

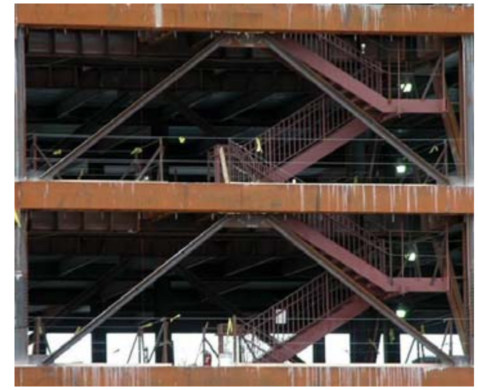
Temporary Bracing

- Structural steel frames require temporary bracing during construction
- Temporary bracing is placed before plumbing up the structural frame
- This gives the structure temporary lateral stability
- Temporary bracing is removed by the erector

Temporary Bracing

- In a braced frame, temporary bracing is removed after final bolt-up is complete and the permanent bracing system is in place
- In a rigid frame, temporary bracing is removed after final bolt-up is complete

Concentric Braced Frames

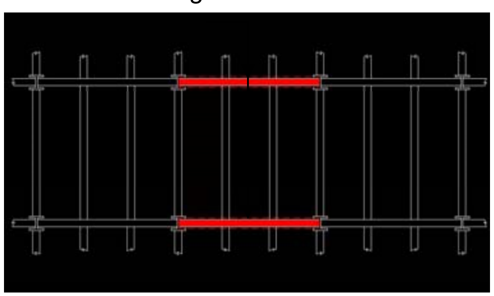


- X brace (above left)
- Chevron (above right)
- Two story X's
- Single diagonals

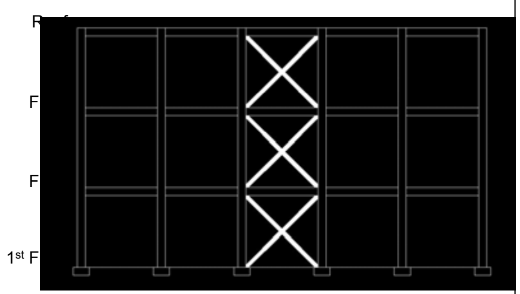
- X bracing is possibly the most common type of bracing
- Bracing can allow a building to have access through the brace line depending on configuration (AISC 2002)

X Bracing

X Bracing



Typical floor plan with X bracing



X-braced building elevation

- The diagonal members of X bracing go into tension and compression similar to a truss
- The multi-floor building frame elevation shown above has just one braced bay, but it may be necessary to brace many bays along a column line
- With this in mind it is important to determine the locations of the braced bays in a structure early in a project

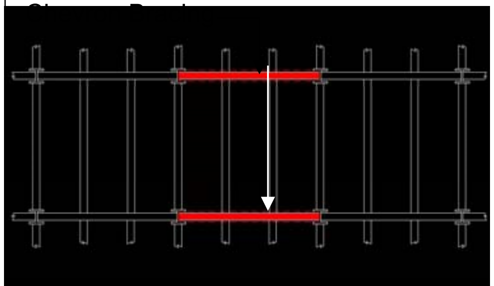
(AISC 2002)

X Bracing

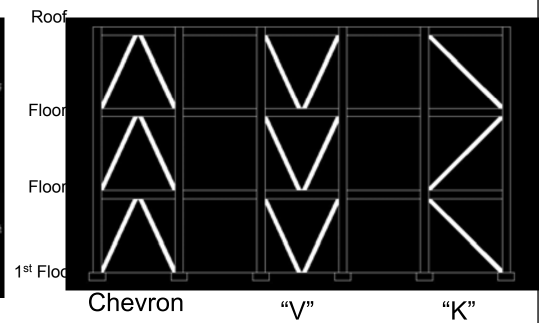
- Connections for X bracing are located at beam to column joints
- Bracing connections may require relatively large gusset plates at the beam to column joint
- The restriction of space in these areas may have an impact on the mechanical and plumbing systems as well as some architectural features

(AISC 2002)

Chevron Bracing



Typical floor plan with Chevron bracing

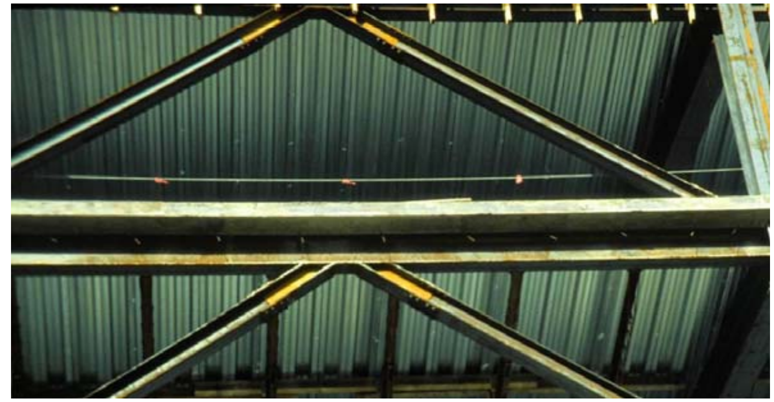


Elevation with several bracing configurations

- The members used in Chevron bracing are designed for both tension and compression forces
- Chevron bracing allows for doorways or corridors through the bracing lines in a structure
- A multi-floor frame elevation using Chevron bracing is shown above

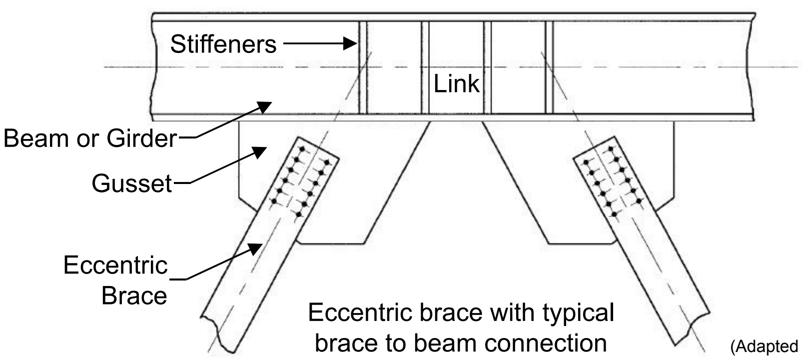
(AISC 2002)

Chevron Bracing



- Chevron bracing members use two types of connections
- The floor level connection may use a gusset plate much like the connection on X braced frames
- The bracing members are connected to the beam/girder at the top and converge to a common point
- If gusset plates are used, it is important to consider their size when laying-out

Eccentrically Braced Frames



Eccentric brace with typical brace to beam connection

(Adapted from AISC 2002)

- Eccentric bracing is commonly used in seismic regions and allows for doorways and corridors in the braced bays
- The difference between Chevron bracing and eccentric bracing is the space between the bracing members at the top gusset connection
  - In an eccentrically braced frame bracing members connect to separate points on the beam/girder
- The beam/girder segment or "link" between the bracing members absorbs energy from seismic activity through plastic deformation (AISC 2002)

Eccentrically Braced Frames

- Eccentrically braced frames look similar to frames with Chevron bracing
- A similar V shaped bracing configuration is used

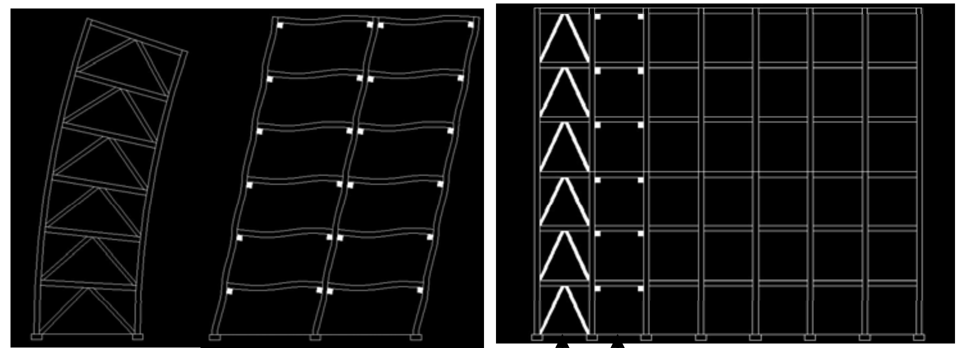
(AISC 2002)

Eccentrically Braced Frames

(EERC 1997)

Eccentric single diagonals may also be used to brace a frame

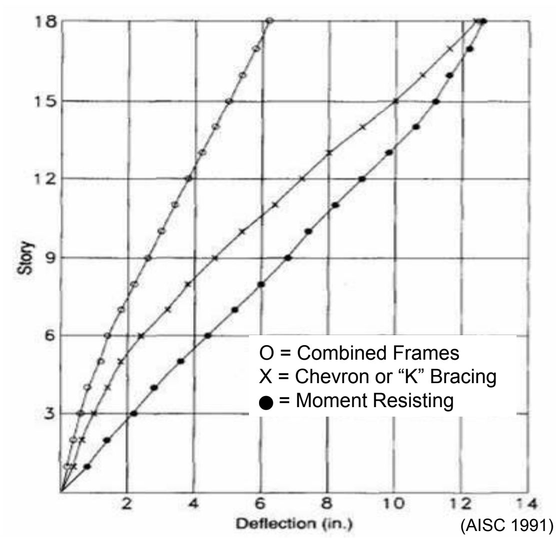
Combination Frames



- As shown above (left) a braced frame deflects like a cantilever beam while a moment resisting frame deflects more or less consistently from top to bottom
- By combining the two systems, reduced deflections can be realized
- The combination frame is shown above right

(AISC 1991)

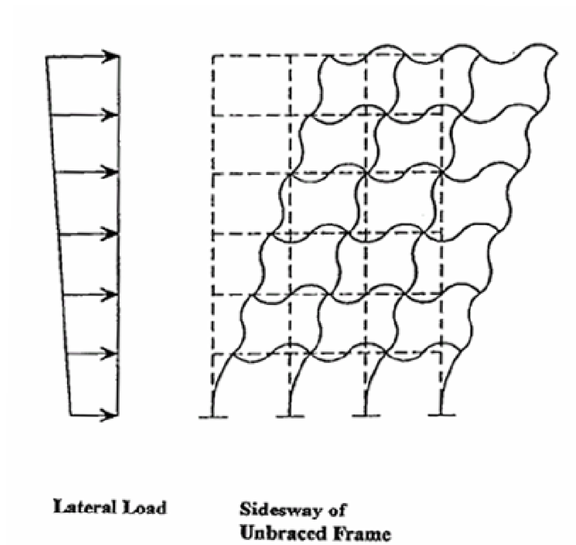
Combination Frames



- The plot shows the moment resisting frame alone, the braced frame alone, and the combined frame
- The same wind load was used for each frame model (AISC 1991)

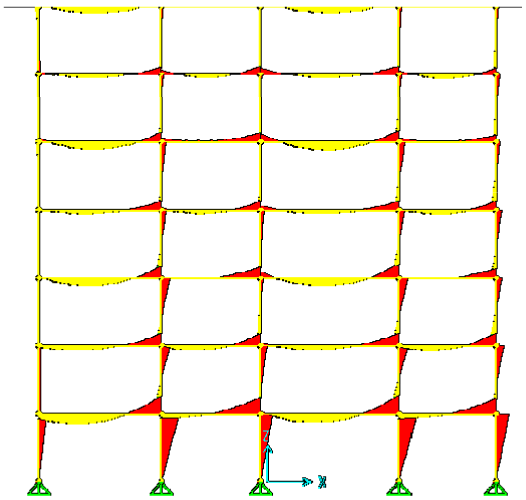
Unbraced (Rigid-Jointed) Frame

Resists loads mainly by flexure



Lateral Load Analysis

- Lateral loads
  - Seismic
  - Wind
- Frame Analysis
  - Portal method
  - FEA package (e.g.,SAP 2000)

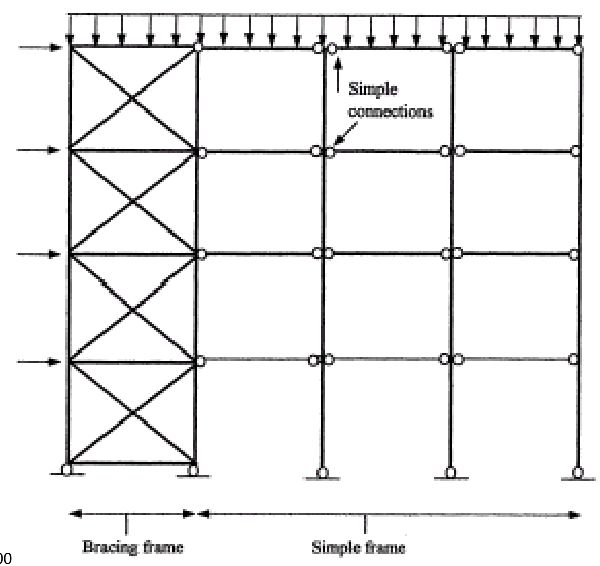


RIGID FRAME

- Derives its lateral stiffness mainly from the bending rigidity of frame members interconnected by rigid joints.
- The joints shall have adequate strength and stiffness and negligible deformations.
- A rigid unbraced frame should be capable of resisting lateral loads without relying on any additional bracing system for stability.
- The frame has to resist gravity as well as lateral forces.
- It should have adequate lateral stiffness against sidesway when it is subjected to horizontal wind or earthquake forces.

Simple Braced Frame

(Would collapse without braced bay, very easy analysis, simple connections)



Simple Frame (Pin-Connected)/1

- Beams and columns are pin-connected and the system is incapable of resisting any lateral loads, unless it is attached to a bracing system.
- Lateral loads are resisted by the bracing systems while the gravity loads are resisted by both the simple frame and the bracing system.
- Bracing system can consist of triangulated frames, shear wall/cores or rigid jointed frames.
- Pin-jointed connections are easier to fabricate and erect. For steel structures, it is more convenient to join the webs of the members without connecting the flanges.

Simple Frame (Pin-Connected)/2

- Bolted connections are preferred over welded connections which normally require weld inspection, weather protection and surface preparation.
- It is easier to design and analyze a building structure that can be separated into a system resisting vertical loads and a system resisting horizontal loads.
- It is more convenient to reduce the horizontal drift by means of bracing systems added to the simple framing than to use unbraced frame systems with rigid connections.

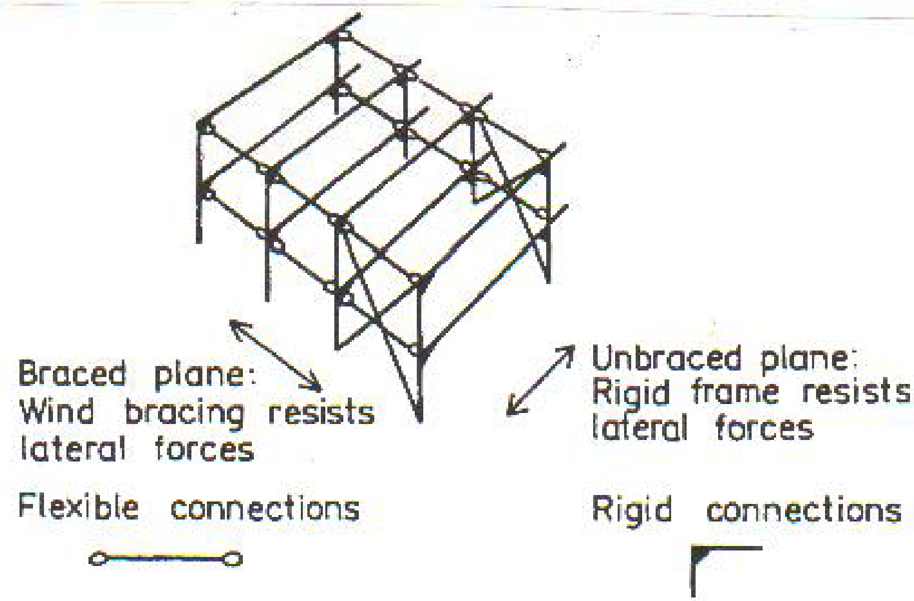


Fig 3.4 One-way braced, one-way rigid framework

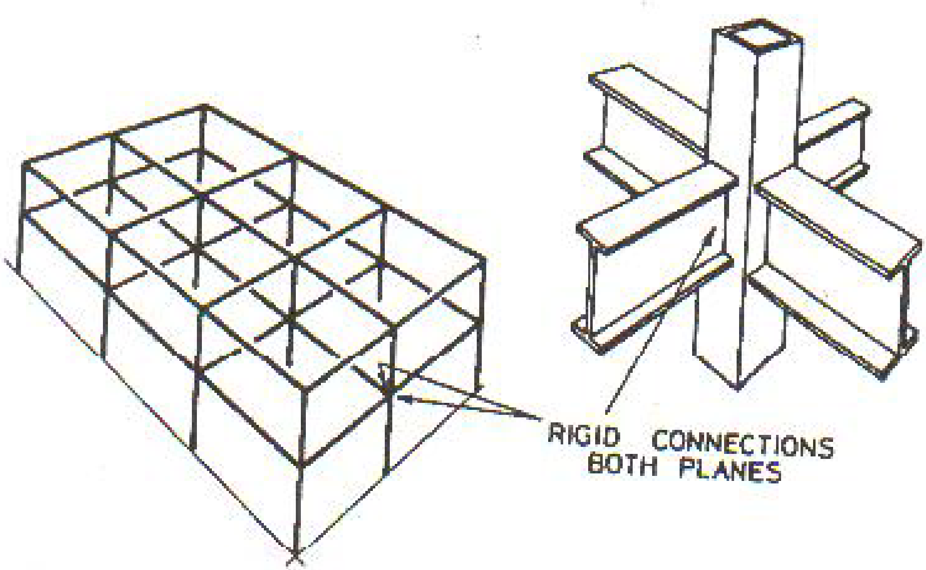


Fig 3.3 Two-way rigid framework

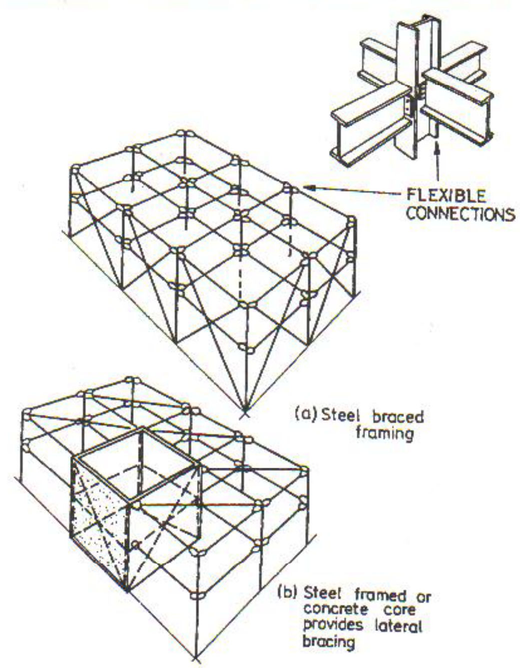


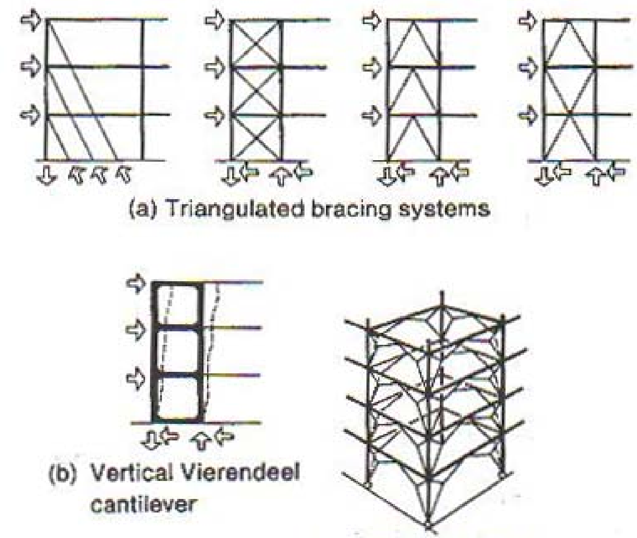
Fig 3.5 Two-way braced framework

Stabilizing Elements

To stabilize the framework in either one or two planes:

- Triangulated steel bracing panels
- Vertical Vierendeel cantilevers in steel
- Triangulated steel core
- Reinforced concrete or masonry cores or shear tubes
- Brick in-fill panels
- Light metal cladding

STABILIZING ELEMENTS IN STEEL

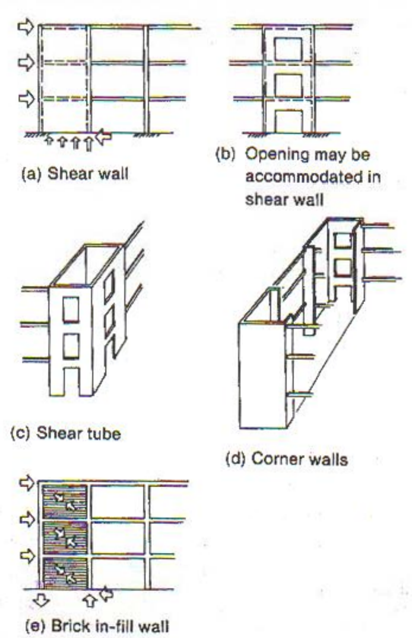


(a) Triangulated bracing systems

(b) Vertical Vierendeel cantilever

(c) Triangulated core

STABILIZING ELEMENTS IN CONCRETE



(a) Shear wall

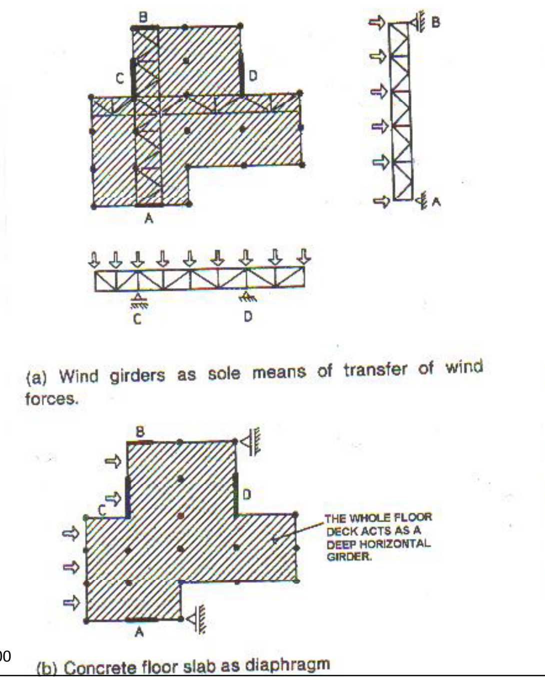
(b) Opening may be accommodated in shear wall

(c) Shear tube

(d) Corner walls

(e) Brick in-fill wall

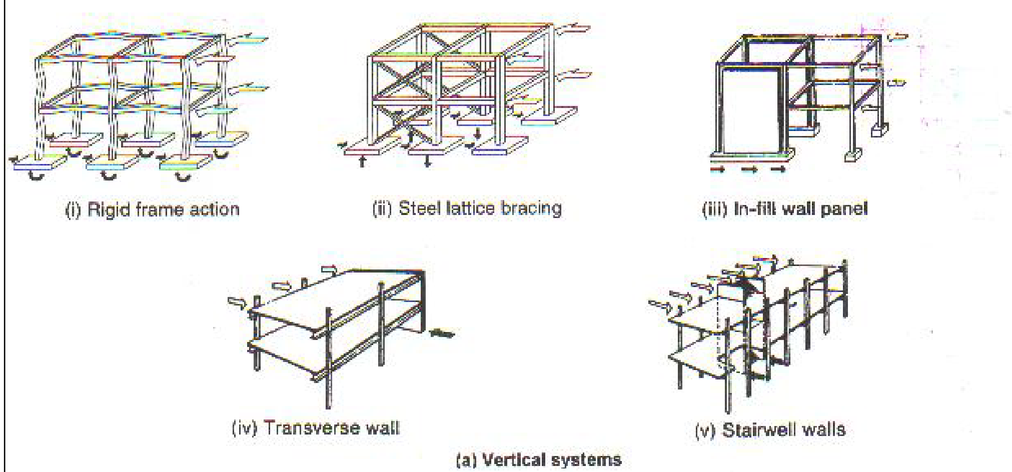
FLOOR DECK BRACING SYSTEMS



(a) Wind girders as sole means of transfer of wind forces.

(b) Concrete floor slab as diaphragm

ACTION OF LATERAL FORCE RESISTING SYSTEMS



ACTION OF LATERAL FORCE RESISTING SYSTEMS

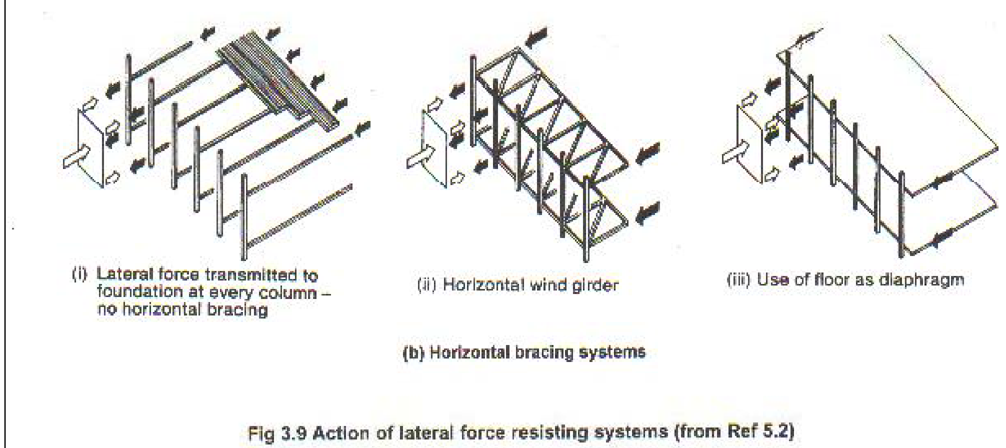
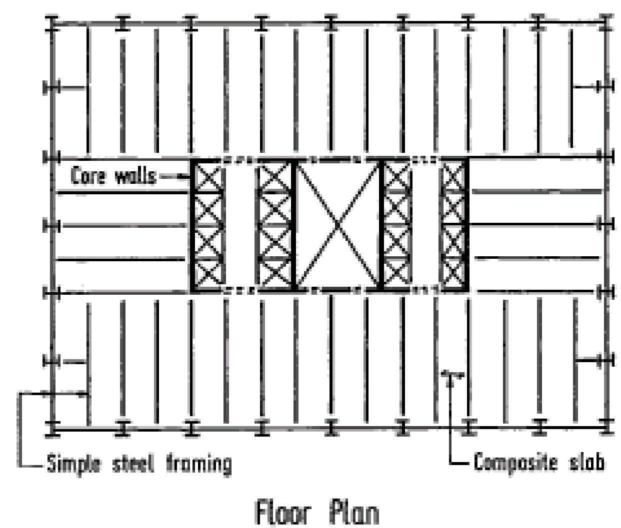


Fig 3.9 Action of lateral force resisting systems (from Ref 5.2)

TALL BUILDING FRAMING SYSTEMS

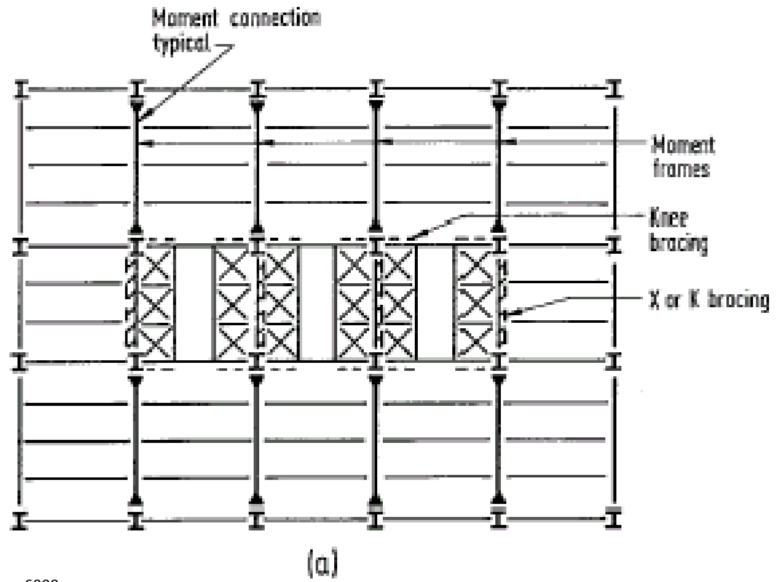
- Core braced
- Moment truss
- Outrigger and belt
- Tube

CORE BRACED SYSTEM
internal shear walls resists all lateral forces; Steel resists gravity loads

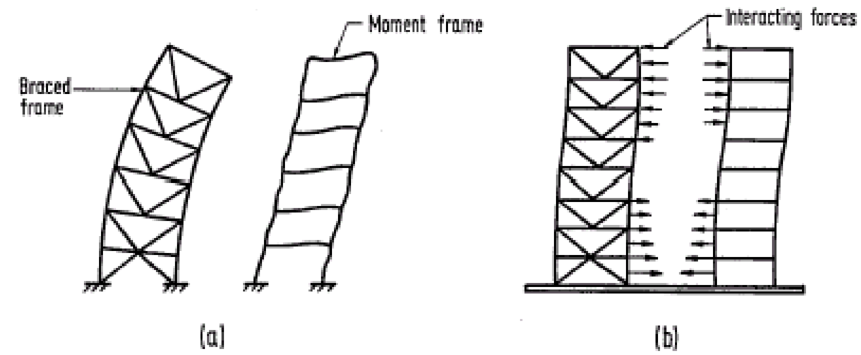


MOMENT-TRUSS SYSTEM

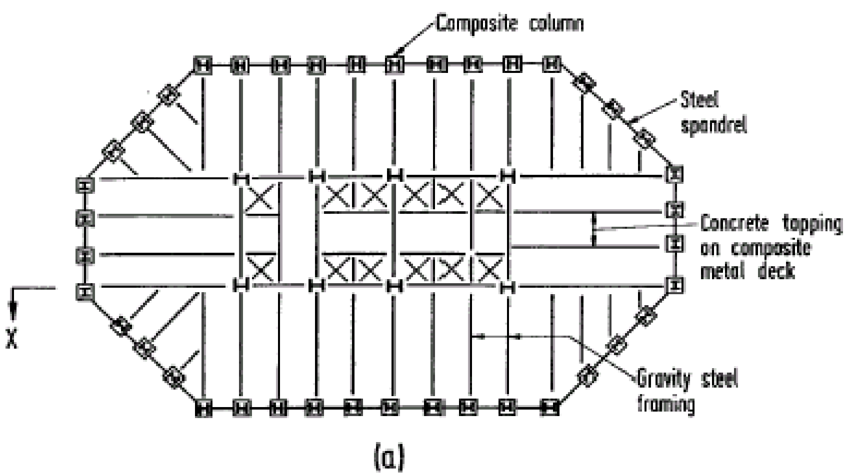
vertical shear truss and moment resisting frames; Truss minimizing sway in lower levels, rigid frame in upper levels.



DEFORMATIONS OF MOMENT-TRUSS SYSTEM

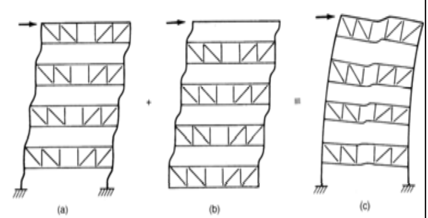


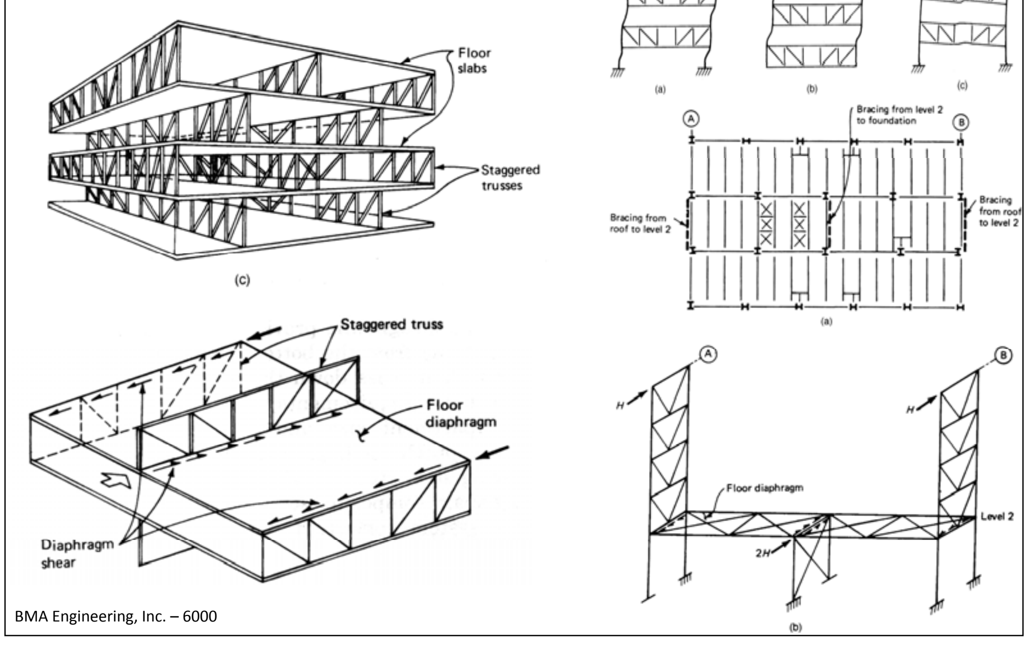
FRAMED TUBE SYSTEM - Hollow perforated tube



- Wide columns at close centers connected by deep beams.
- Tube resists all lateral forces of wind and earthquake.
- Interior its share of gravity loads.

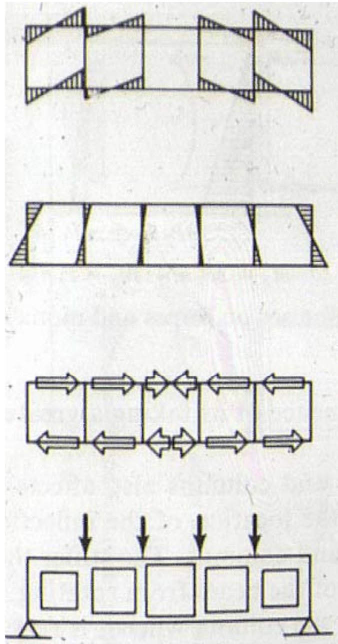
STAGGERED TRUSS





VIERENDEEL TURSS

A Vierendeel truss has rigid, welded connections so does not require the diagonals usually seen in trusses. If used, the reason might have been to provide more space for ducts or openings within the truss by eliminating the diagonals.



Conventional Building Code Philosophy for Earthquake-Resistant Design

Objective: Prevent collapse in the extreme earthquake likely to occur at a building site.

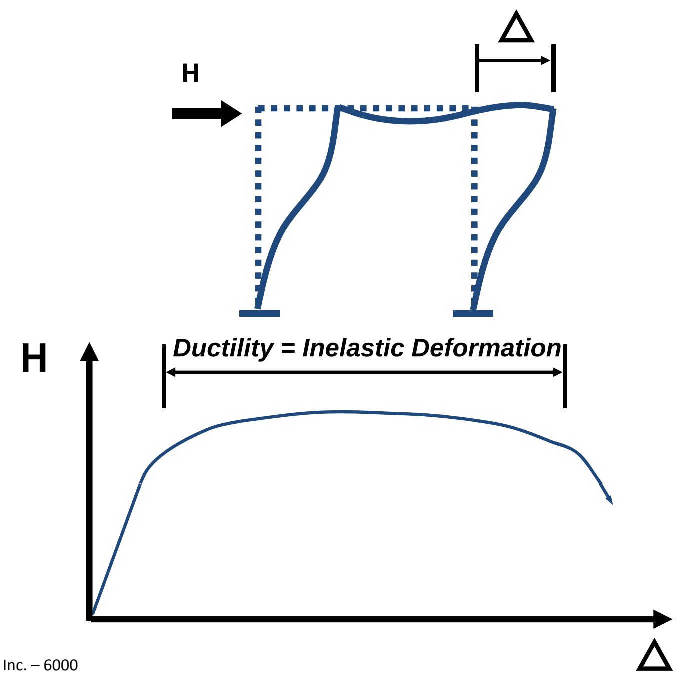
Objectives are not to:

- limit damage
- maintain function
- provide for easy repair

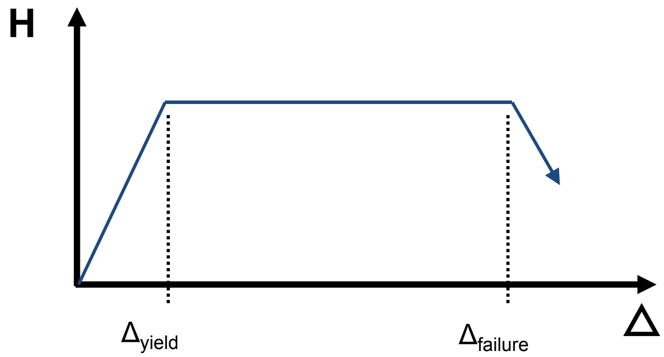
To Survive Strong Earthquake without Collapse:

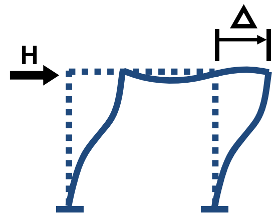
Design for Ductile Behavior

Design for Ductile Behavior



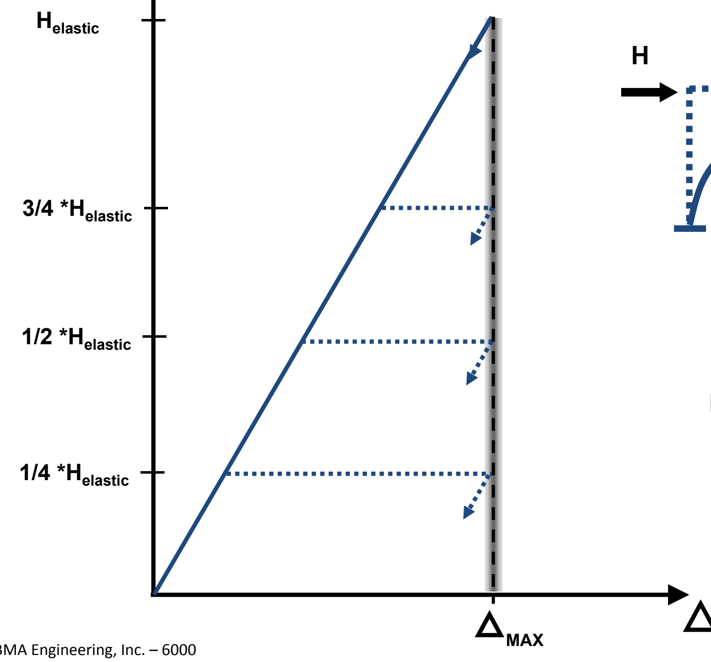
Design for Ductile Behavior

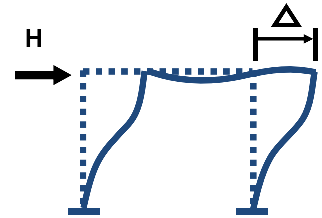


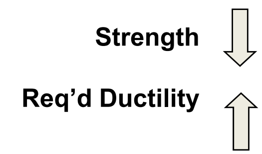


Ductility Factor $\mu = \dfrac{\Delta_{failure}}{\Delta_{yield}}$

Design for Ductile Behavior



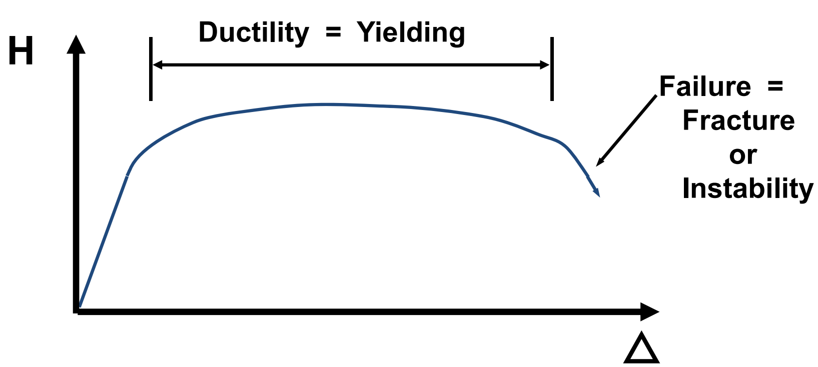




Design for Ductile Behavior

Ductility in Steel Structures: ***Yielding***

Nonductile Failure Modes: ***Fracture or Instability***



Developing Ductile Behavior:

- Choose frame elements ("fuses") that will yield in an earthquake.
- Detail "fuses" to sustain large inelastic deformations prior to the onset of fracture or instability (i.e. , detail fuses for ductility).
- Design all other frame elements to be stronger than the fuses, i.e., design all other frame elements to develop the plastic capacity of the fuses.

Key Elements of Seismic-Resistant Design

<u>Required Lateral Strength</u>

ASCE-7:

Minimum Design Loads for Buildings and Other Structures

<u>Detailing for Ductility</u>

AISC:

Seismic Provisions for Structural Steel Buildings

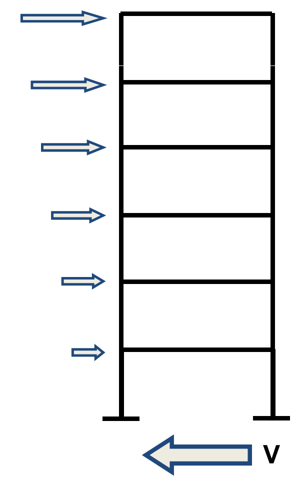
Design EQ Loads – Total Lateral Force per ASCE 7-05:

$$\boxed{V = C_s W}$$

V = total design lateral force or shear at base of structure

W = effective seismic weight of building

C_S = seismic response coefficient



Design EQ Loads – Total Lateral Force per ASCE 7-05:

$$V = C_S W$$

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \le \begin{cases} \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)} & \text{for } T \le T_L \\[2ex] \dfrac{S_{D1} T_L}{T^2\left(\frac{R}{I}\right)} & \text{for } T > T_L \end{cases}$$

S_{DS} = design spectral acceleration at short periods

S_{D1} = design spectral acceleration at 1-second period

I = importance factor

T = fundamental period of building

T_L = long period transition period

R = response modification coefficient

Design EQ Loads – Total Lateral Force per ASCE 7-05:

<u>R factors for Selected Steel Systems (ASCE 7):</u>

SMF	**(*Special Moment Resisting Frames*):**	**R = 8**
IMF	**(*Intermediate Moment Resisting Frames*):**	**R = 4.5**
OMF	**(*Ordinary Moment Resisting Frames*):**	**R = 3.5**
EBF	**(*Eccentrically Braced Frames*):**	**R = 8 or 7**
SCBF	**(*Special Concentrically Braced Frames*):**	**R = 6**
OCBF	**(*Ordinary Concentrically Braced Frames*):**	**R = 3.25**
BRBF	**(*Buckling Restrained Braced Frame*):**	**R = 8 or 7**
SPSW	**(*Special Plate Shear Walls*):**	**R = 7**

Undetailed Steel Systems in Seismic Design Categories A, B or C (AISC Seismic Provisions not needed) **R = 3**

6300. Design -

6330. Structural Steel Frames and Assembles

Objective and Scope Met

- Types of Steel Frames
- Stability Bracing (Section NC and Appendix N6)
- Elastic and Inelastic Behavior
- Seismic Analysis