INSTITUTE OF AERONAUTICAL ENGINEERING
(AUTONOMOUS)
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Regulation: R15 (JNTUH)

STEEL STRUCTURES DESIGN AND DRAWING

Prepared

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UNIT-I

MATERIALS - MAKING OF IRON AND STEEL

Iron and Steel

The transition from the Bronze Age to the Iron Age was ushered in by two developments:

- Growing scarcity of copper and/or tin
- Increased processing temperatures

- Iron and Steel Characteristics?
- What is the difference?

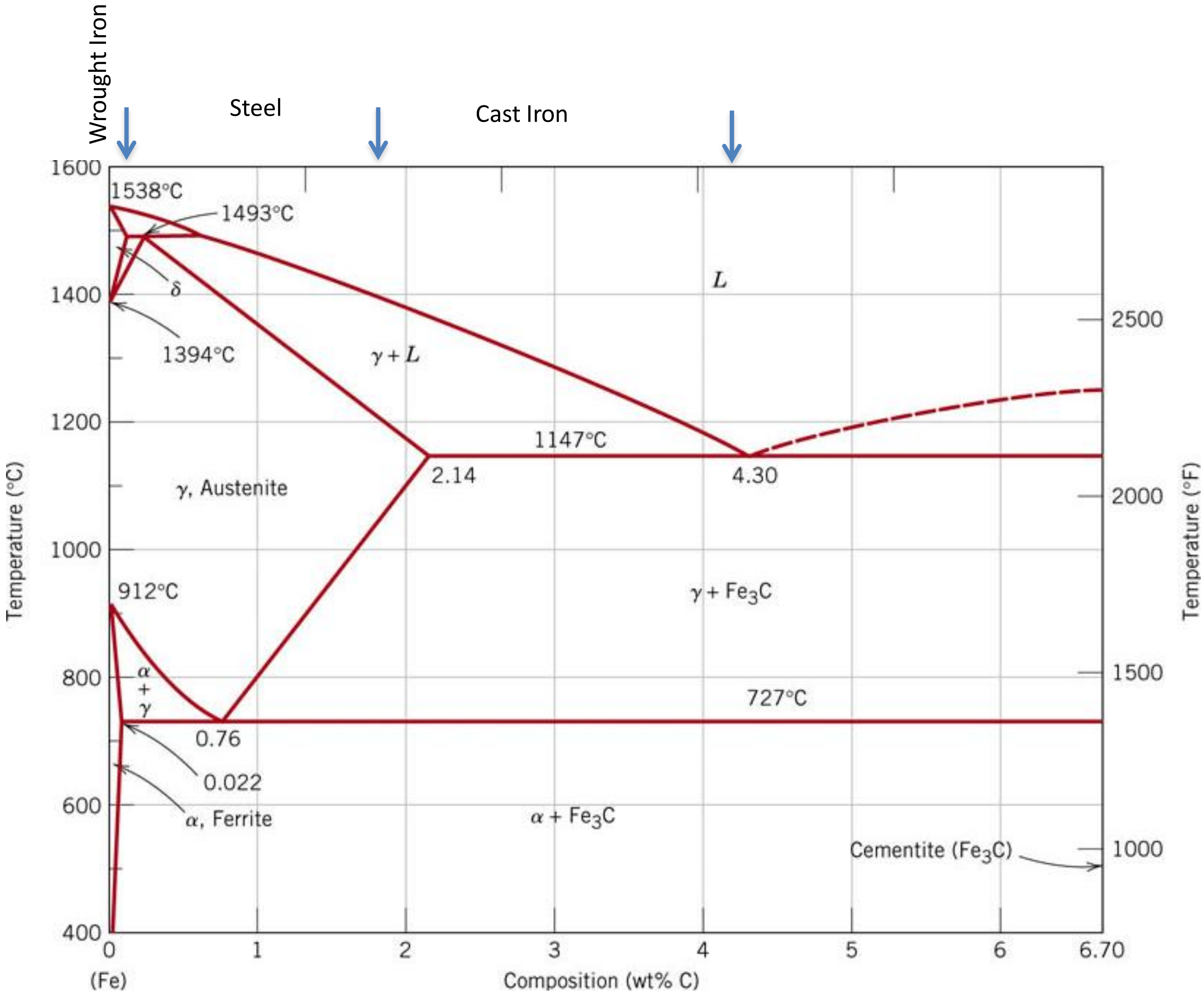
Iron vs Steel

- Bronze was an Alloy of Cu
  - Arsenic or Tin most common alloying impurity
- Steel is an Alloy of Iron
  - Carbon is the most common alloying impurity
- Iron and Carbon is probably the most useful combination in the history of Society
- Terms you should know
  - Forging – beating or hammering a material into shape
  - Casting – Pouring a liquid into a mold

Forms

- **Low Carbon 0-0.2%**
  - **<u>Wrought Iron</u>**
  - Pure, Ductile, typically about as strong as Bronze
  - High melting point (>1500°C)
- **Medium Carbon 0.2-2.1%**
  - **<u>Steel</u>**
  - Very strong, hard, forgable,
  - 1000X harder than pure Fe
  - High melting point (>1400°C)
- **High Carbon 2.3-4.3%**
  - **<u>Cast Iron or Pig Iron</u>**
  - Low Melting Point, Brittle, Not Forgable
  - Can only be Cast

The Iron-Iron Carbide Phase Diagram



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Why is Steel Strong?

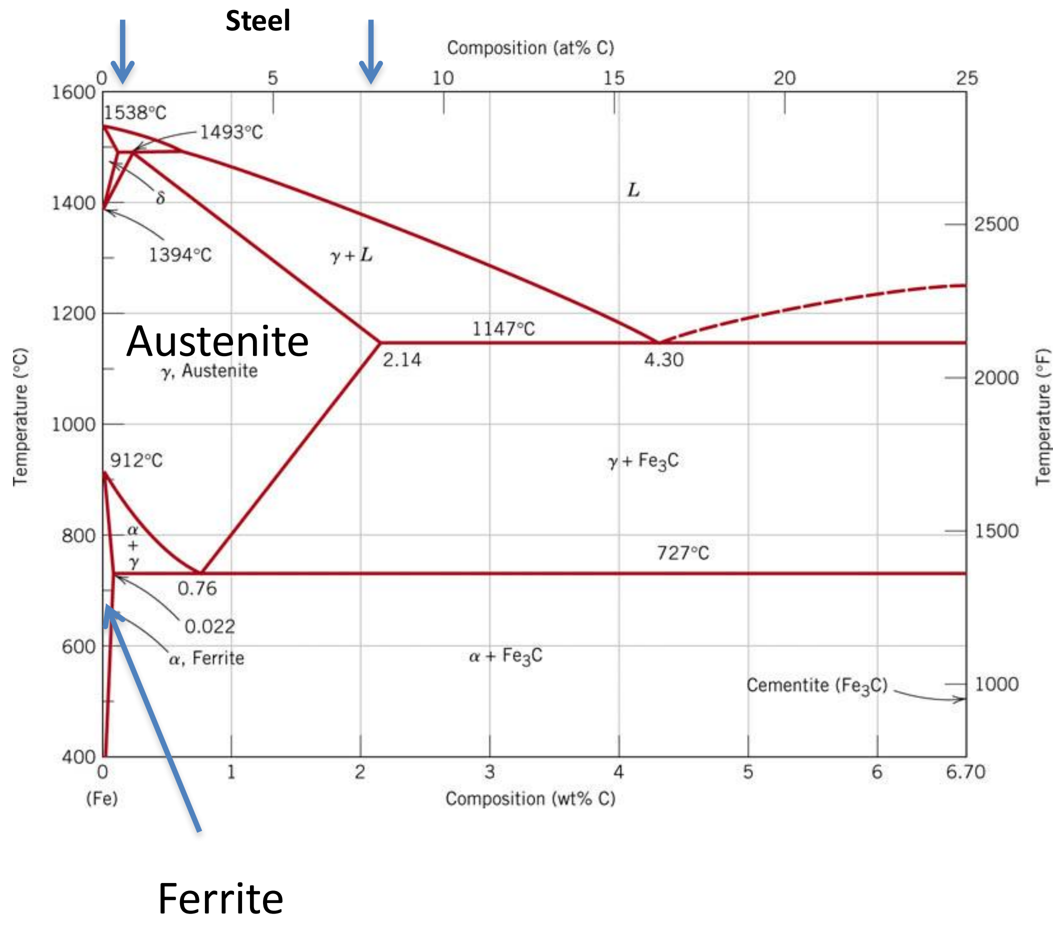
Two Important phases of iron-
α - **Ferrite** (BCC)
γ- **Austenite** (FCC)

How much carbon can you dissolve in the Iron?
Ferrite very little (<0.02%)
Austenite Lots (up to 2.1%)

Why is that important?

When you heat Steel to 1000°C
what phase do you have?
Austenite with lots of Carbon

When you cool it what does the carbon do?
It can precipitate into Carbide particles. Strains crystal makes dislocation motion difficult (hardens)



Forms of iron

- **Wrought Iron**
  - Very little carbon (<0.2%)
  - Difficult to slow the dislocations since carbon is dissolved
  - Soft, malleable, easily wrought
- **Steel**
  - Medium carbon (0.2-2.3%)
  - Can dissolve carbon in Austenite but it wants to precipitate upon cooling
  - <u>Properties depend on how fast you coo</u>l
  - Strain from Carbides greatly strengthens the steel
  - Must control both the amount of carbon and the tempering
- **Cast Iron**
  - High Carbon (2.3-4.3%)
  - Forms lots of carbide phase
  - Make the material brittle (too much of a good thing)

Why is the Heat treatment important?

- For Steel if you heat it to 1000°C make Austenite
  - **Slow Cool**
    - If you cool slowing make a mixture of Alpha iron and carbide particles
    - Give it time for the Carbon to form particles
    - Natural Composite
    - Slightly stronger than Bronze
  - **Fast Cool or Quench**
    - Not enough time to transform to Alpha Iron
    - Form Martensite (very hard phase of iron and carbon)
    - Excess Carbon strains martensite (brittle)
  - **Tempering**
    - Heat again to move carbon around and restore ductility

So how do you make Iron?

- Start with
  - Iron Oxide (either Fe_2O_3 or Fe_3O_4),
  - Limestone ($CaCO_3$, Oyster Shells) and
  - Carbon (Charcoal)
- Layer these and light on fire and use bellows to get hot
- $2C + O_2 \Rightarrow 2CO$
- $Fe_2O_3 + 3CO \Rightarrow 2\,Fe + 3CO_2$ (makes Iron)
- Now have to get rid of sand from iron oxide
- $CaCO_3 \Rightarrow CaO + CO_2$ (make lime)
- $CaO + SiO_2 \Rightarrow CaSiO_3$ (glass slag)

Get a mixture called a Bloom

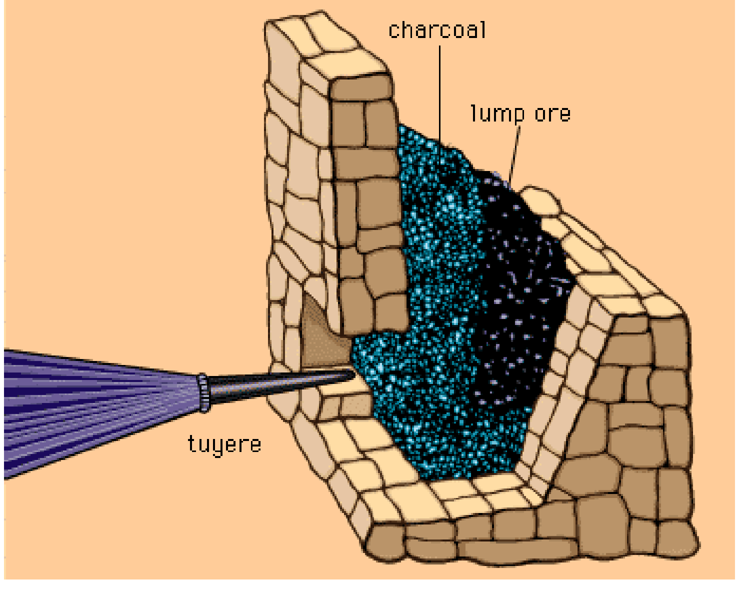
Early Wrought Iron: Bloomeries

- Bloom Iron
  - Make a Bloom by heating the mixture we just described
  - Beat on the bloom with a hammer to separate the iron from the slag
  - Left with pure wrought (low carbon) iron
- How do you add carbon to make steel?
  - Melting point too high until 1700's
  - Heat Iron in Carbon rich air – Carborization
  - Carbon diffuses into the iron making outer layer of steel
  - Skilled craftsmen

Cast Iron Making (Blast Furnace)

- Method
  - Same ingredients (Iron Ore, Charcoal and Limestone)
  - Heat hotter and collect liquid iron from bottom
  - Cast Iron is also called Pig Iron
  - Chinese skipped Bloomery stage and just made cast iron from ancient blast furnaces
  - Not seen in Europe until 1500's
- Now you have too much carbon
  - Use Finery Forge to remelt pig iron,
  - oxidize carbon and silicon
  - Make bloom and beat it, create wrought (bar) iron

Blast Furnace



Early Blast Furnace

History of Iron and Steel

- 3500BC Beads in Ancient Egypt for iron
  - From Meteor (nickel content)
- First Iron Production 3000BC Syria and Mesopotamia
- Hittites mass produced Iron 1500-1200BC
  - Bloom Iron in small batch process
  - Use water power to beat blooms
- China starts Iron age later around 700BC
  - Used the first blast furnaces to make cast iron
  - Also used Finery forges to make wrought iron 300BC

History Continued

- By 1400’s needs increased
  - Church bells, cannons etc
  - Blast furnace developed (Cast Iron)
- By 1500’s Japan is making the strongest Steel in the world
  - Samuri Sword take a week just make the iron
  - Use Magnetite (black sand Fe2O3) and charcoal
  - Broad range of carbon, separate it by feel and sound
  - Make the center from low carbon ductile steel
  - Then add high carbon hard/brittle steel to outside
  - Create a composite and the best steel for several hundred years

History Continued

  - 1588 Queen Elizabeth limits use of Timber
  - By 1700’s have a timber famine (charcoal)
- Why not use Coal?
  - Too much Sulfur weakens steel
  - Coke the coal,
    - Drives off impurities (thanks to beer)
- 1709 Darby England
  - Low sulfur coke
  - Cast iron become cheap
  - Start of Industrial revolution
    - Over 200 years
    - income increases 10X
    - Population increases 6X
    - Machine based economy
    - Free market, rule of law, no trade barriers

History continued

- Puddling
  - Invented in 1784
  - Stir Molten Pig (Cast) iron in oxidizing atmosphere
  - Observe it “come to nature”
  - Gather in a puddling ball
- Replaced by the Bessemer Furnace 1855
  - Blow Air through the melt
  - Rapidly remove carbon and silicon
  - Initially didn’t work tough to remove just the right amount
  - Instead decided to remove it all and add carbon back
  - Steel very fast (in 20 minutes) and efficient
- Today we use 1950’s Process
  - Basic Oxygen Process (BOP)
  - Inject pure Oxygen

AIMS OF STRUCURAL ENGINEER

- Structural design is a scientific & creative process.
- The structural design should satisfy
  - Safety
  - Stability
  - Serviceability
  - Durability

  And result in
  - Economic (cost of construction & maintenance),
  - Aesthetically pleasing, and
  - Environment friendly structures.

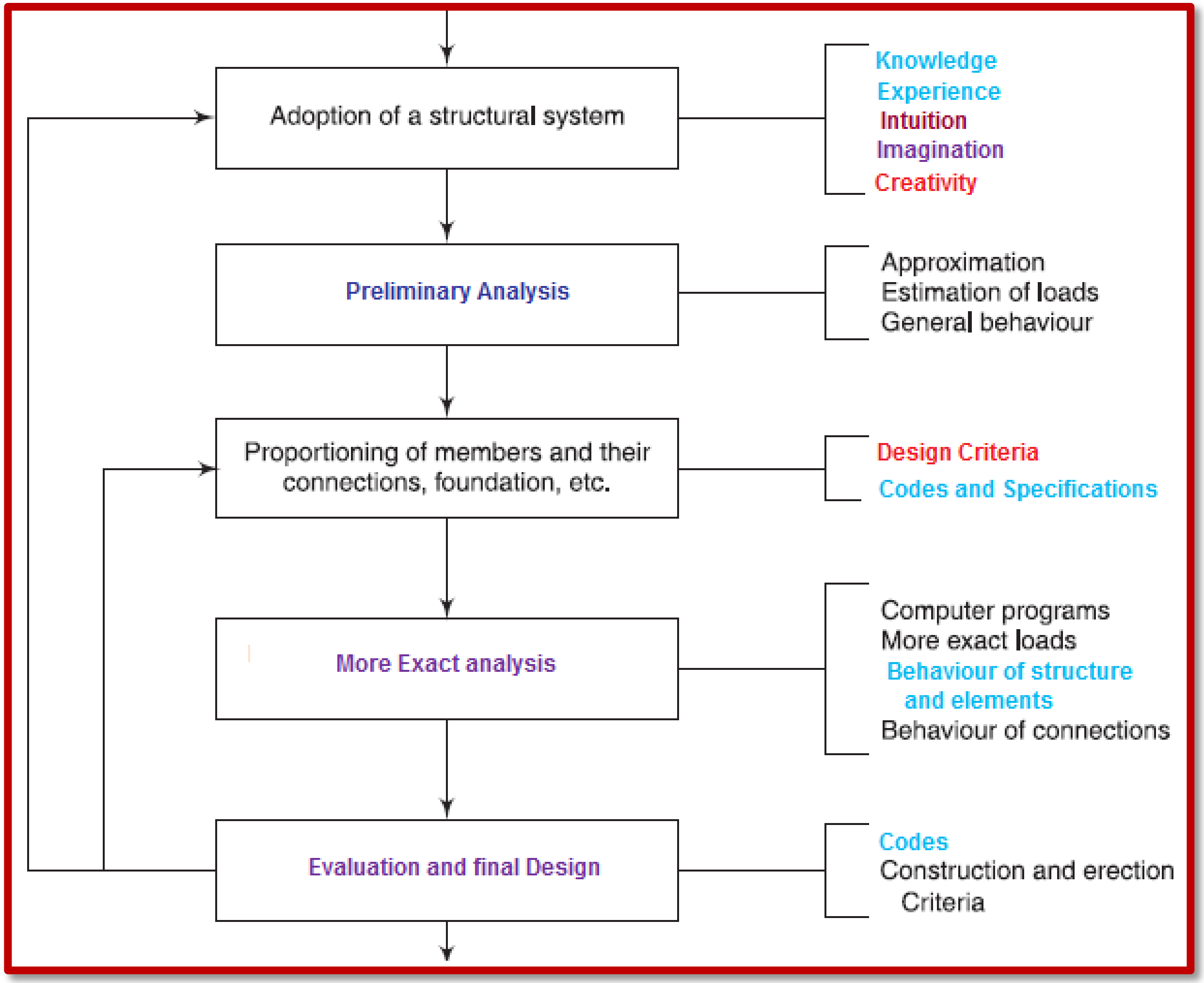
STEPS INVOLVED IN THE DESIGN PROCESS

- Estimation of probable loads, based on the plan and elevation from the architect and the soil report from the geotechnical engineer
- Arrival of the structural system
- Structural analysis(with the aid of computers)
- Design of various elements based on codal provisions
- Estimation of quantities and cost of construction

STEPS INVOLVED IN THE DESIGN PROCESS (cont.)

- Preparation of fabrication and erection drawings and bill of quantity of materials (BOQ)-should be approved by SE
- Construction by the contractor based on the specifications.
- Inspection by SE for Quality Control at regular intervals
- As built drawing
- Maintenance till intended life.

Iterative Design Process

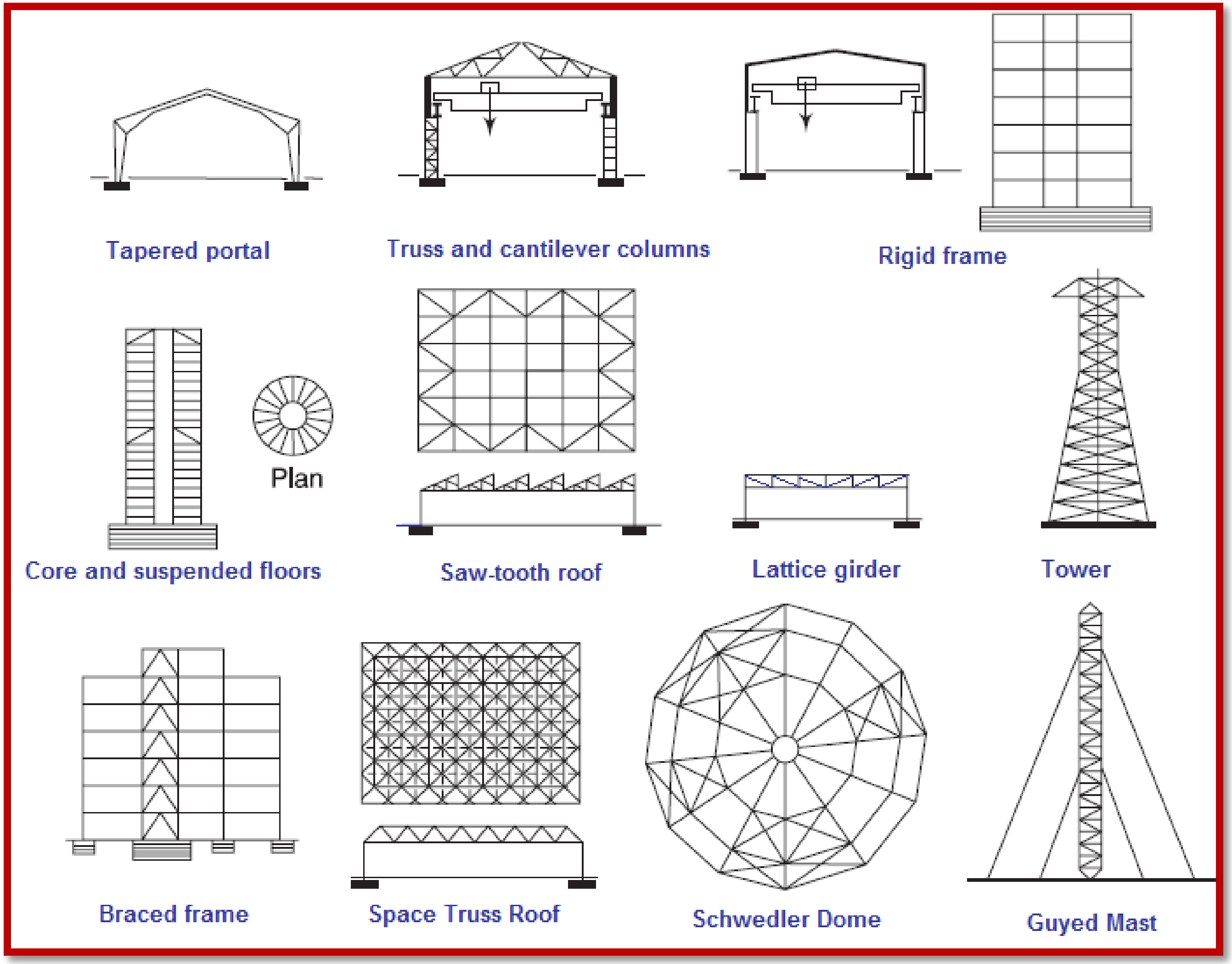


STRUCTURAL SYSTEMS

Classification of structural systems

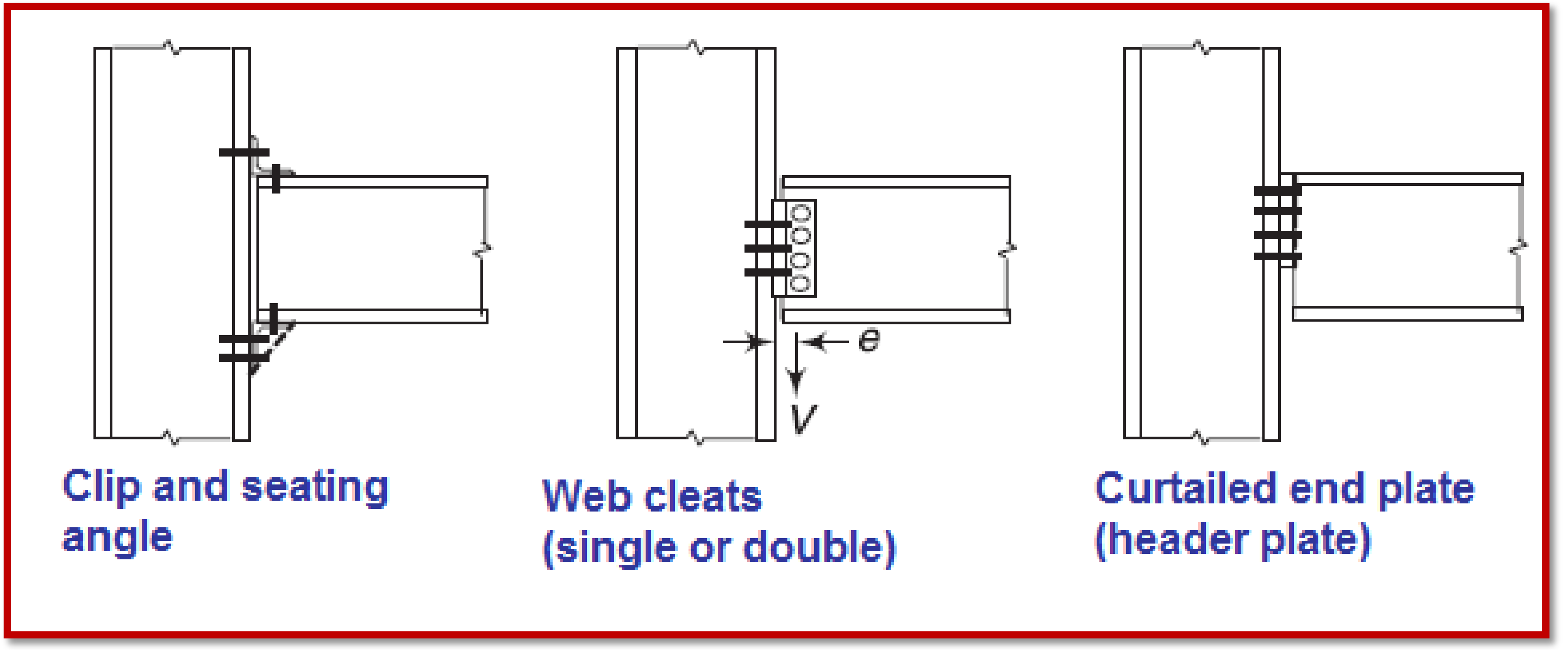
- Single-storey, single/multi-bay
- Multistorey, single or multi-bay
- Space structures: single, double or multi layer grids, steel frame folded plates, braced barrel vaults and domes
- Tension structures, tensegritic and cable-supported
- Stressed skin

STRUCTURAL SYSTEMS(cont.)

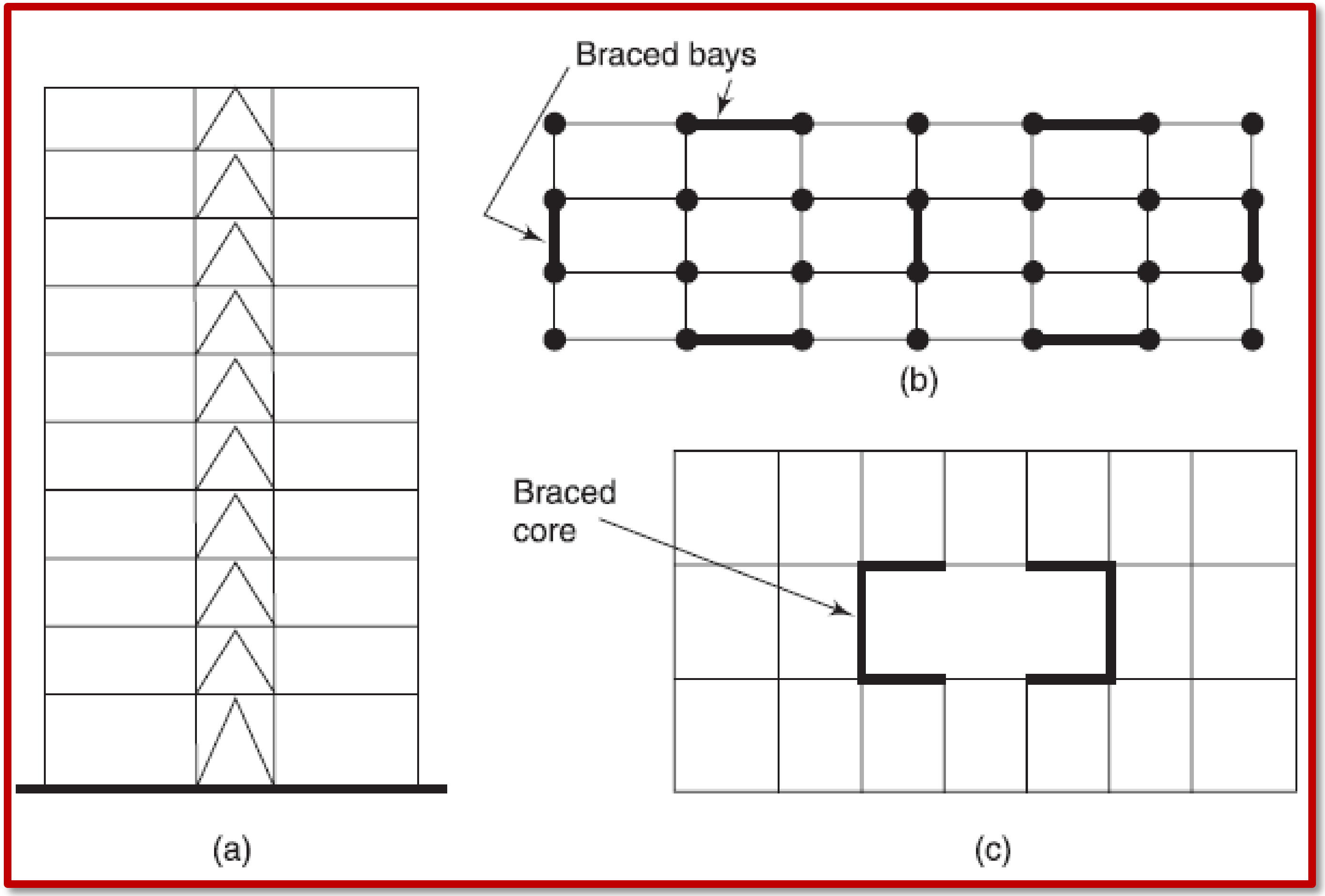


Types of Steel Framed Buildings

- **Simple constructions**



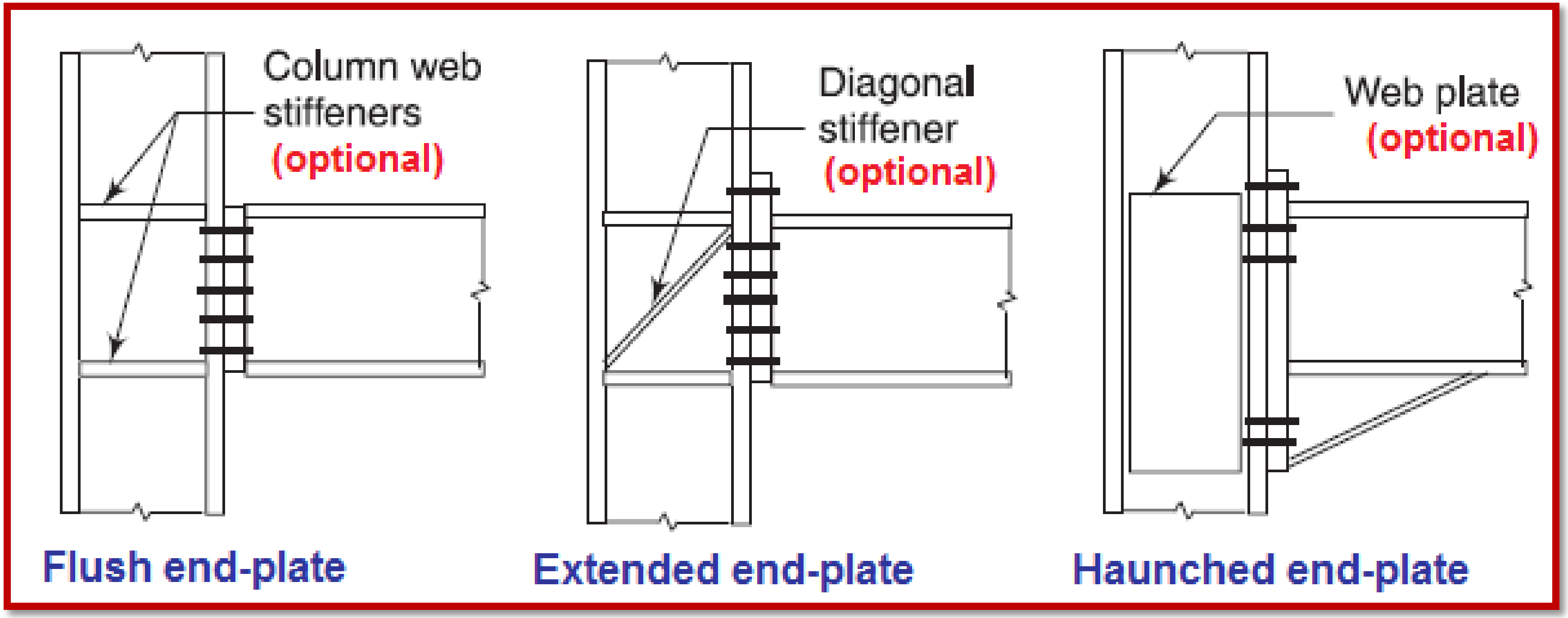
Types of Steel Framed Buildings



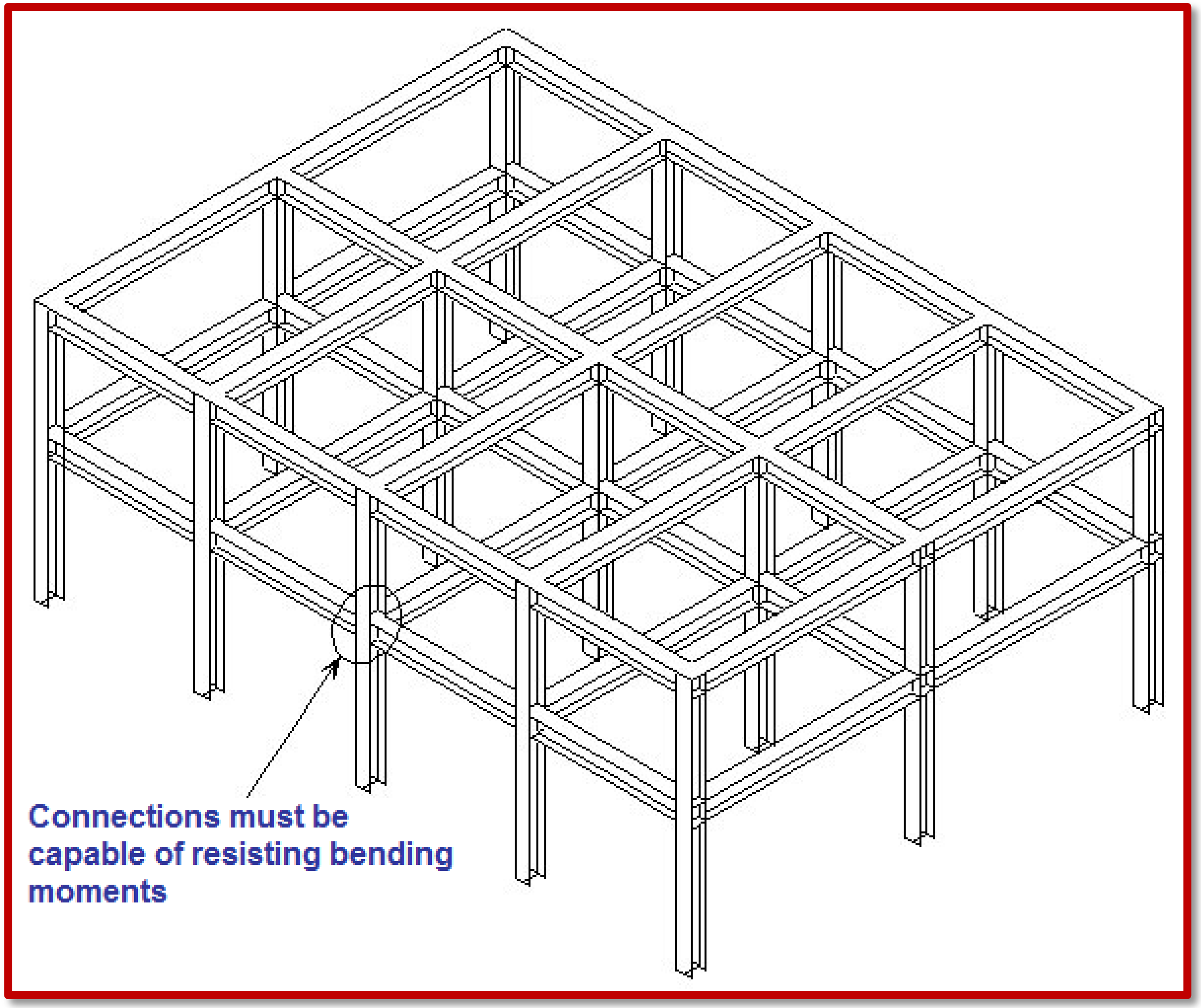
Braced frames (a) vertical (b) on perimeter/interior wall and (c) around core

Types of Steel Framed Buildings

- **Continuous or Rigid Connections**



Rigid frames

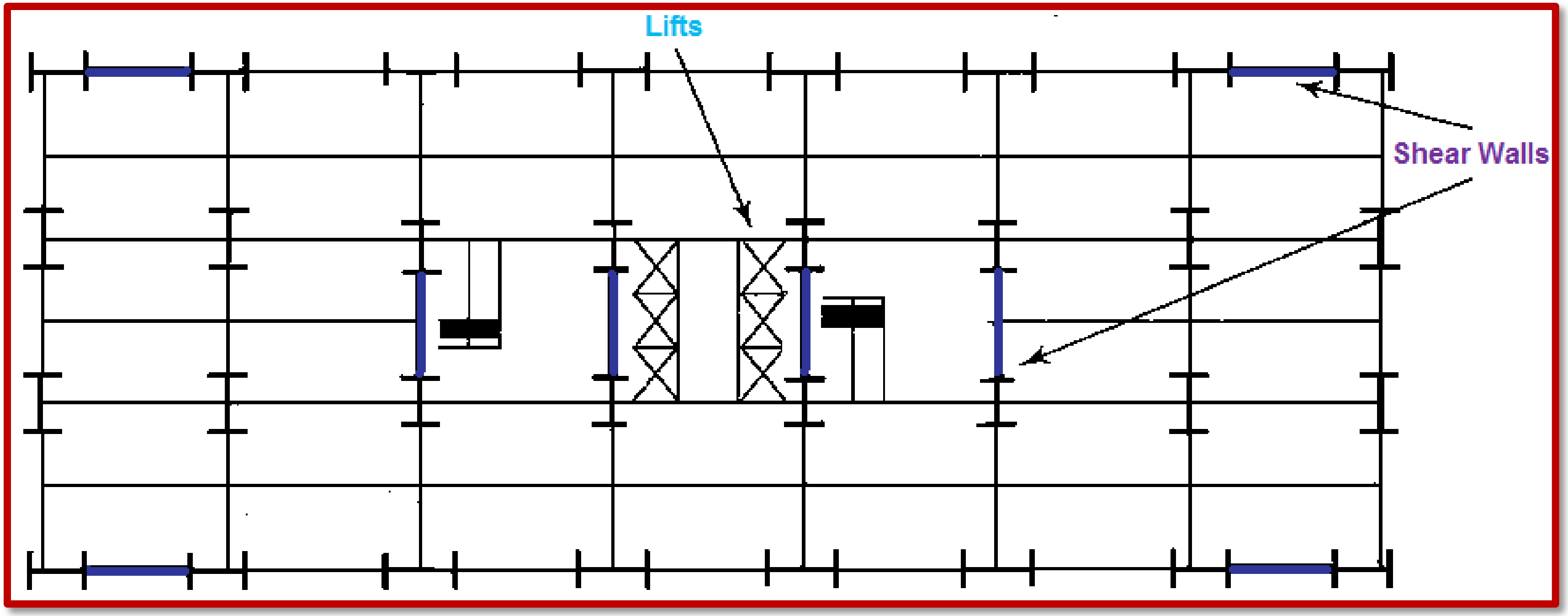


Types of Steel Framed Buildings

- **Semi-Rigid Connections**
  - They fall between the simple connections and rigid connections.
  - Most of the practical connections are semi rigid connection only.
  - However in practice they are treated at simple or rigid connections only!
  - Refer clause 4.2.1.2 or Appendix F of code for methods to deal with them.

Steel Framed Buildings (cont.)

Composite Structures

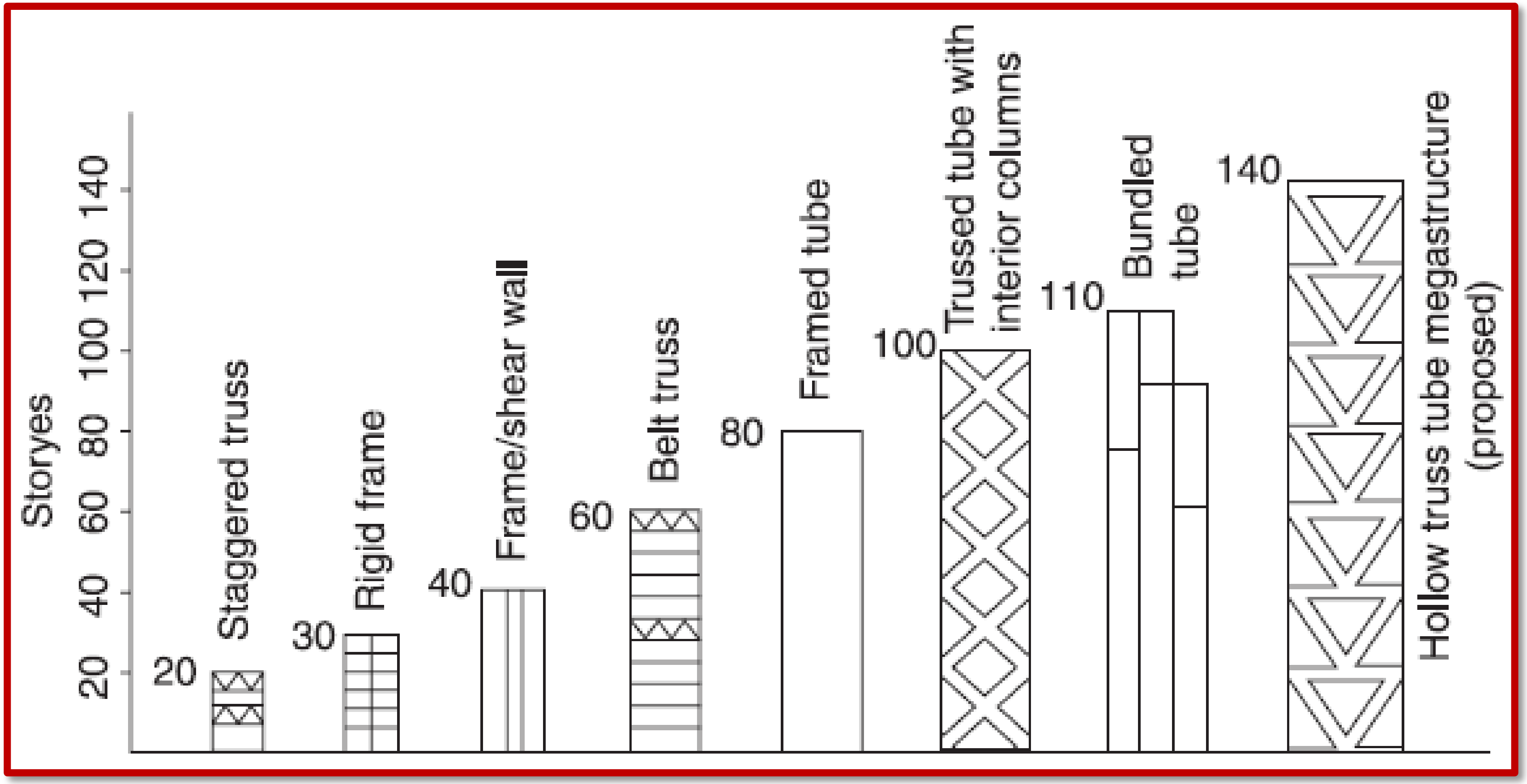


Steel frames with shear walls

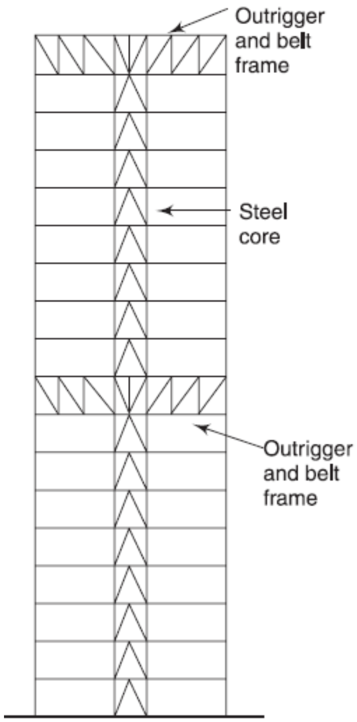
High Rise Structural Systems

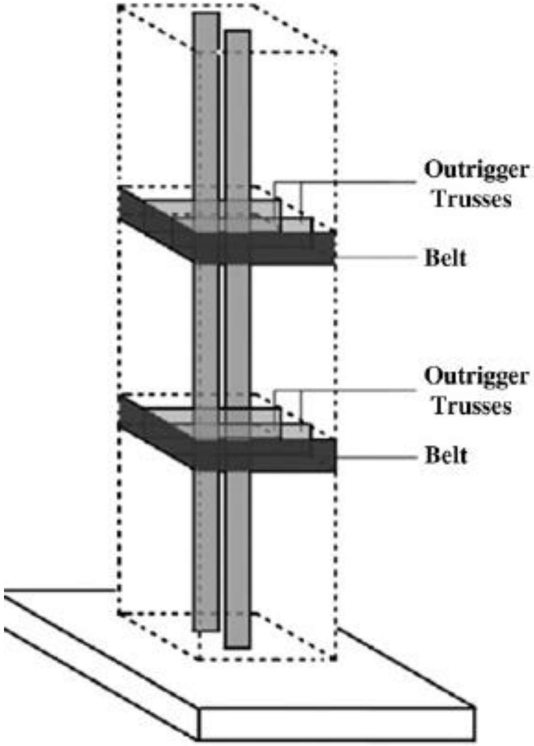
- **Several excellent systems have been invented in the past:**
  - **Outrigger and belt truss**
  - **Framed tube**
  - **braced tube**
  - **Tube in tube**
  - **Bundled tube**

High Rise Structural Systems



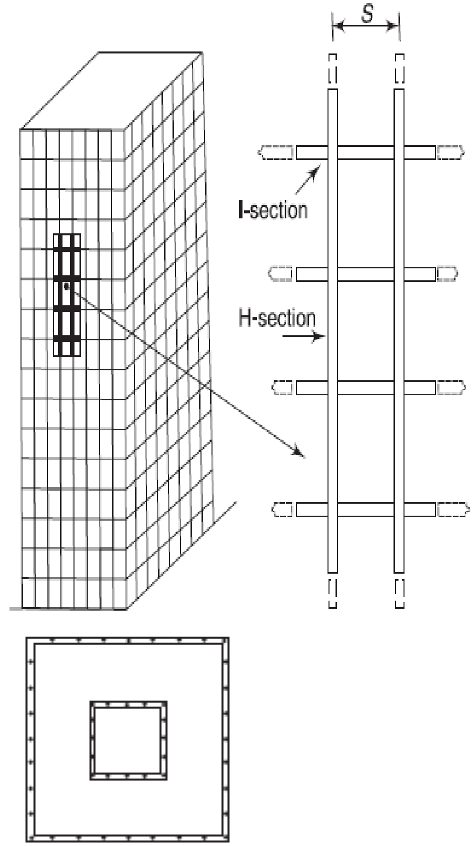
Outrigger and belt truss system





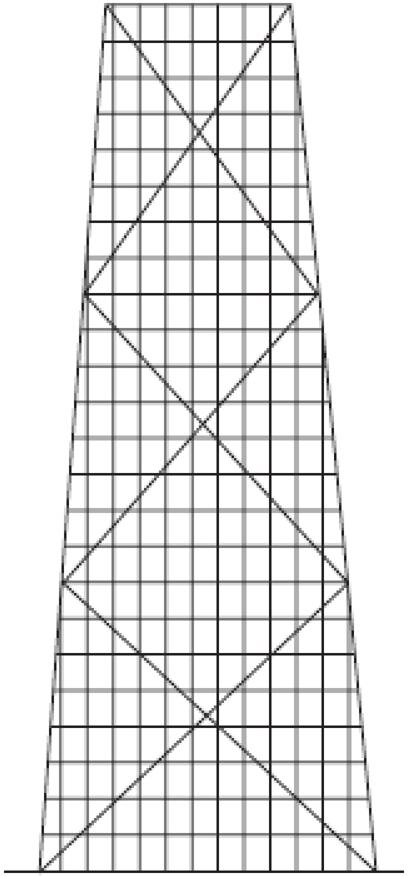
Ex: 42 Story, 183 m tall First Wisconsin Center, Milwaukee, USA., Architect & Engineers: Skidmore, Owings & Merrill.

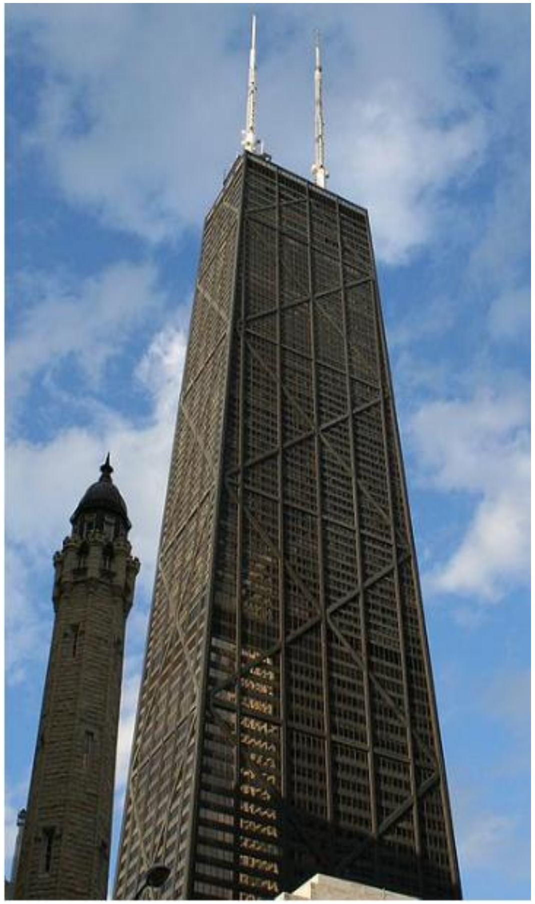
Framed Tube & Tube in Tube System



Ex: World Trade Center, NY, 110 storey; Architect: M. Yamasaki, Engineer: L.E Robertson Associates

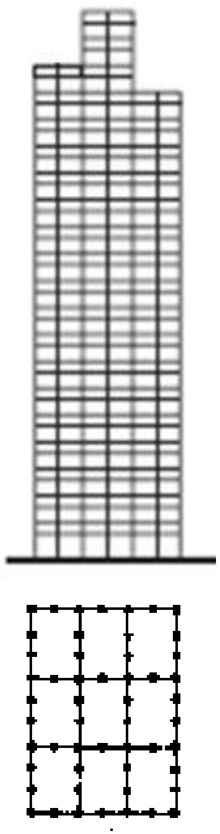
Braced Tube System





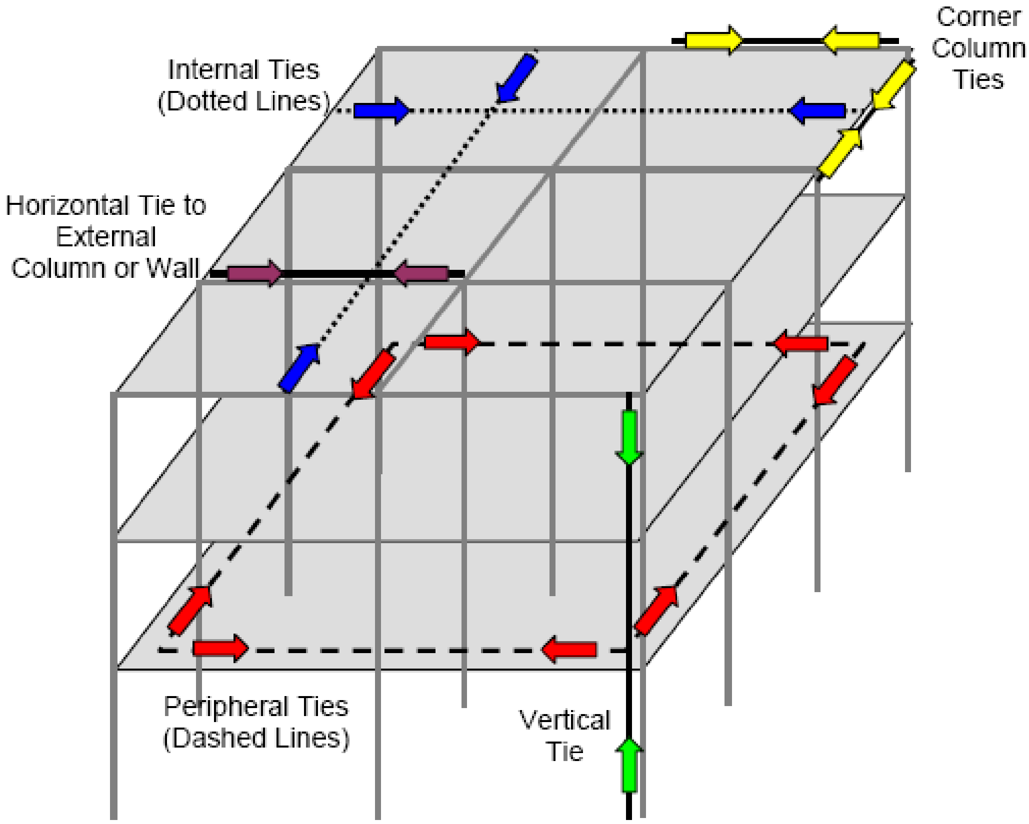
Ex: 100 story John Hancock Center of Chicago, Illinois, Engineers: Skidmore, Owings and Merrill & Fazlur Khan

Bundled Tube System



Ex: 108- Story, 442 m tall Willis (Sears)Tower has nine interconnected bundled tubes; Architect: Skidmore, Owings and Merrill; Structural Engineer: Fazlur Khan

Structural Integrity



BS 5950, 2000 , Cl 5.1.2
of IS 800:2007

Minimum tie strength :

Internal ties :0.5 $W_f S_t L_a$

External ties:0.25 $W_f S_t L_a$

- W_f is the total factored load / unit area,
- S_t is the tie spacing,
- L_a is the distance between columns in the direction of ties

Columns should be tied in all directions to achieve structural integrity

STRUCTURAL ANALYSIS

- **Analysis : Determination of the axial forces, bending moments, shears & torsional moments**
  - **Slope deflection method**
  - **Moment distribution method**
  - **Portal method**
  - **Cantilever method**
  - **Matrix methods (stiffness & Flexibility)**

Software for Structural Analysis

- SAP2000
- STAAD III & STAAD PRO
- ETABS
- MIDAS
- GT-STRUDL
- STRUDS

You should be aware of any assumptions used and limitations of these programs

Methods of Analysis

- Elastic (first-order) Analysis
- Plastic Analysis
- Advanced Analysis (second order elastic analysis or frame instability analysis)
- Dynamic Analysis

❖ Normal Steel building are analyzed using Elastic analysis.

❖ Dynamic/Advanced analysis necessary for tall/slender buildings.

CODES AND SPECIFICATIONS

- **Codes –Bureau of Standards EBCS**
  - **Ensure adequate structural safety**
  - **Aid the designer in the design process.**
  - **Ensure consistency among different engineers**
  - **Protect the structural engineer from disputes – but do not provide legal protection.**

❖ **Specifications are specific to a project - issued by Architect/Project Manager**

DESIGN PHILOSOPHIES

- **Working Stress Method of design (WSM).**
- **Ultimate Strength Design (USD) or Plastic Design**
- **Limit States Design [in USA, Load and Resistant Factor Design (LRFD)]**

Working Stress Method (WSM)

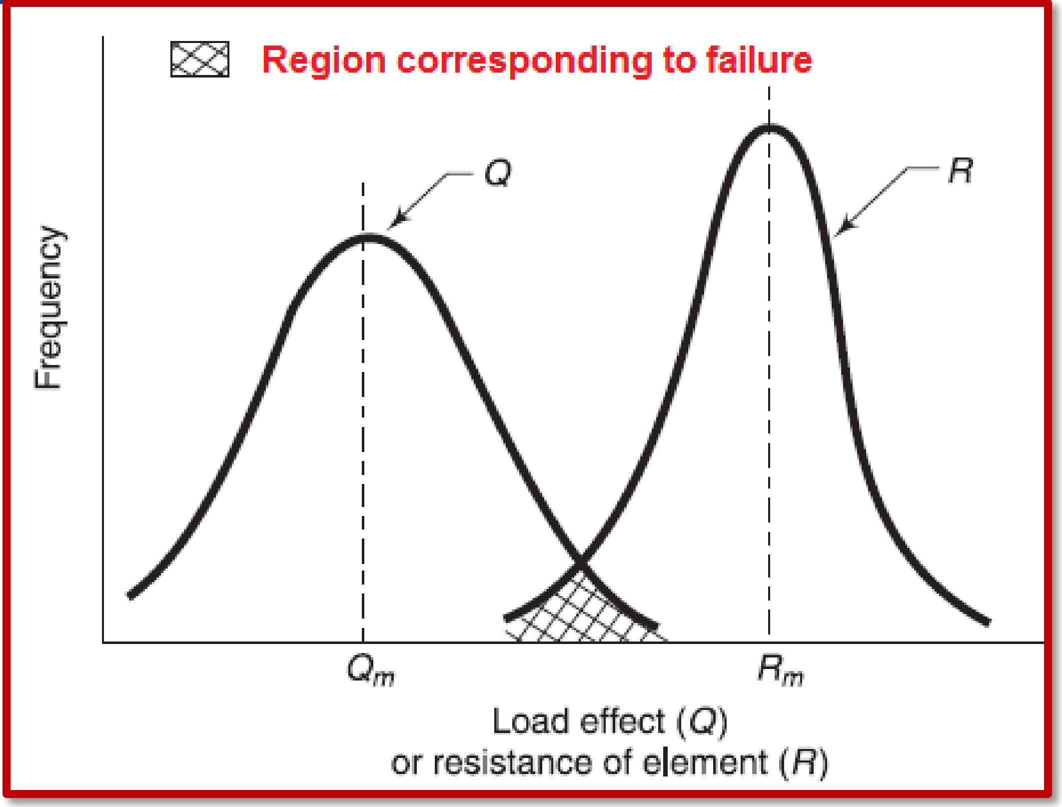
- The first attainment of yield stress of steel is taken to be the onset of failure.
- The limitations due to non-linearity (geometric as well as material) and buckling are neglected.
- Permissible (allowable) stress = Yield Stress / Factor of Safety(F.S.)
- Working Stress ≤ Permissible Stress

F.S.= 1.67 for tension members, beams and short columns; 1.92 for long columns

Limit States Design

- **Plastic design:** a special case of limit states design, wherein the limit state of strength is the attainment of plastic moment strength M_P
- **Limit state:** a state of impeding failure, beyond which a structure ceases to perform its intended function satisfactorily
- **Strength & serviceability limit states**

Design for Ultimate Limit State



The design should be such that the overlap of two curves are small-Probability of failure is within acceptable range.

Limit states considered by Code

Ultimate (safety) Limit States (ULS)	Serviceability Limit States (SLS)
1. Strength (including yielding, buckling, and transformation into a mechanism)	1. Deformation and deflection
2. Stability against overturning and sway	2. Vibration (e.g., wind-induced oscillations, floor vibration, etc.)
3. Failure due to excessive deformation or rupture	3. Repairable damage due to fatigue (cracking)
4. Fracture due to fatigue	4. Corrosion and durability
5. Brittle fracture	5. Fire

$R_d \geq \Sigma\gamma_{if} Q_{id}$

R_d - design strength computed using the reduced material strengths R_u/γ_m where R_u is the characteristic material strength
γ_{if} - Partial safety factors for Loads
Q_{id} - Characteristic Design loads

Comparison of WSM & LSD

Working stress method

- **Factor of safety for yield stress- Allowable stresses < ‘f_y’.**
- **Analysis done under working loads.**
- **Yielding or buckling never occurs at working loads**
- **Deformations are evaluated at working loads.**

Limit State Method

- **Partial material safety factors (γ_m) for yield and ultimate stress.**
- **Analysis done under factored load [Working loads x partial safety factor for loads (γ_f)]**
- **Post buckling and post yielding are considered while estimating capacity.**
- **Deformations are evaluated at working loads.**

Partial Safety factor-Material, γ_m

(a)	Resistance of Class 1, 2, or 3 cross-section:*	$\gamma_{M0} = 1.1$
(b)	Resistance of Class 4 cross-sections:*	$\gamma_{M1} = 1.1$
(c)	Resistance of member to buckling:	$\gamma_{M1} = 1.1$
(d)	Resistance of net section at bolt holes:	$\gamma_{M2} = 1.25$

Source: EBCS - 3

Partial Safety factor-Loads, γ

Table 2.2 Partial Safety Factors for Actions on Building Structures for Persistent and Transient Design Situations.

	Permanent action (γ_G)	Variable actions (γ_Q)	
		Leading variable actions	Accompanying variable actions
Favourable effect $\gamma_{F,inf}$	1.0*	-	-
Unfavourable effect $\gamma_{F,sup}$	1.30*	1.60	1.60

Source: EBCS - 3

Check for Stability

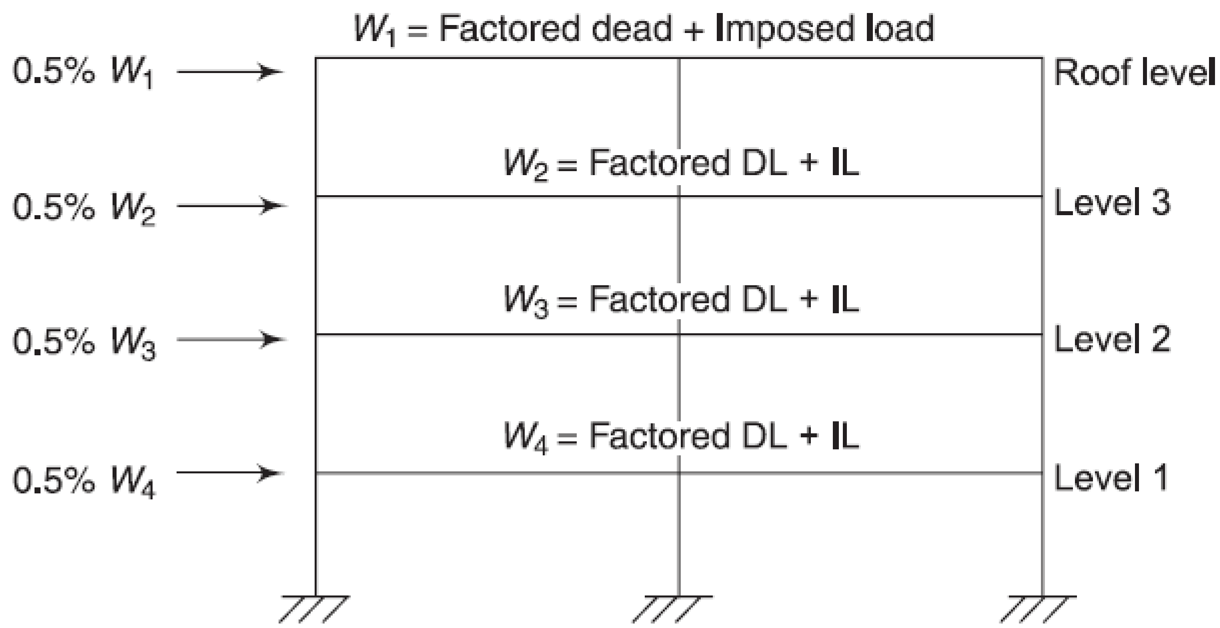
- **We should consider**
  - **General stability**
  - **Stability against overturning**
  - **Sway stability.**

Stability against overturning

- **Important for Tall buildings and Cantilevers**
  - **Components aiding instability and resisting instability identified and multiplied with appropriate partial safety factor.**
  - **Resisting components multiplied by a partial safety factor of 0.9 and added with design resistance**
  - **The resistance effect ≥ destabilizing effect**

Sway stability

- All structures to be checked for a minimum notional horizontal load (0.5% of the vertical load at each level).



Serviceability Limit States

- Deflections are to be calculated for all the combination of Working loads, and checked for the maximum values given in code.
- Vibration will have to be checked when vibrating loads (due to machinery, cranes,) or activities such as dancing, marching are involved.
- Change the natural frequency by some means to reduce vibration

Limits for Deflection

Type of building	Deflection	Design load	Member	Supporting element	Maximum deflection
Industrial building	Vertical	Live load/ Wind load	Purlin and girts	Elastic cladding	Span/150
		Live load/ Wind load	Purlin and girts	Brittle cladding	Span/180
		Live load	Simple span	Brittle cladding	Span/300
		Live load	Cantilever	Brittle cladding	Span/150
		Live load	Simple span	Elastic cladding	Span/240
		Live load	Cantilever	Elastic cladding	Span/120
		Live load/ Wind load	Rafter supporting	Profiled metal sheeting	Span/180
				Plastered sheeting	Span/240
	Lateral	No cranes	Column	Elastic cladding	Height/150
		No cranes	Column	Masonry/brittle cladding	Height/240

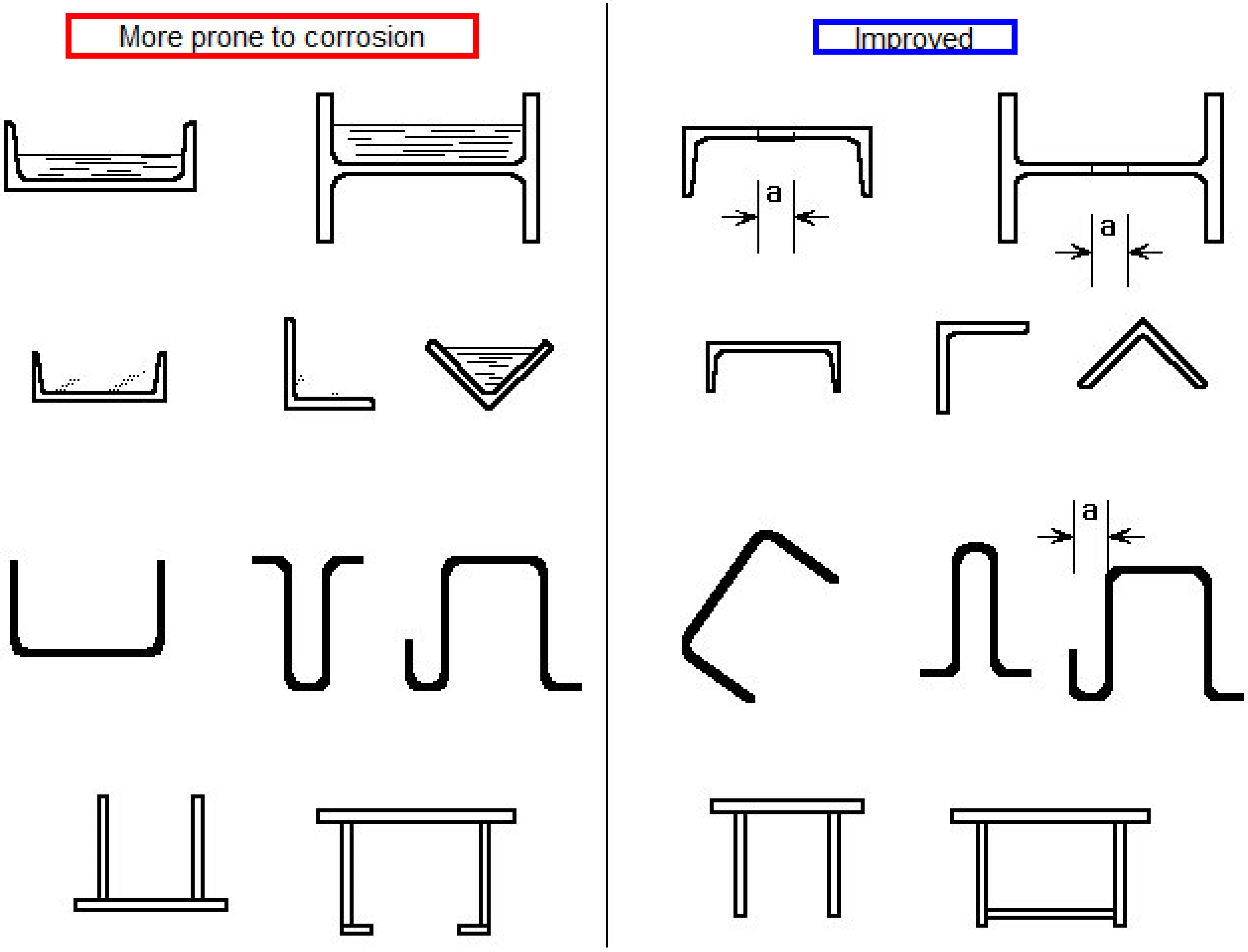
Case Study for Vibration

London Millennium Bridge experienced vibration when opened on June 2000.
37 fluid viscous and 52 Tuned Mass Dampers were added to control the vibration and the bridge opened again in Feb 2002.

Durability

- details of different coating/painting systems for durable structures
- Blast cleaning the surface enhances the life of paints
- Avoid traps for dirt and moisture
- Use of weathering / stainless steel

Details for Corrosion Prevention



Case study of Golden Gate Bridge

- During 1965 the lead paint was removed and an inorganic zinc-rich paint system applied.
- The coating system is in excellent condition, even after 44 years

Case Study: Richard J. Daley Center, Chicago

- Built in 1965 - the first building to be constructed with a type of weathering steel, (Cor-Ten)
- Even after 40 years the building and and the sculpture are in great condition!

Fatigue & Fire

- Important criteria for bridges, crane girders & platforms carrying vibrating machinery.
- Fatigue is checked at working loads
- Fire resistance is a function of mass, geometry, support conditions, type of fire, and adopted fire protection methods

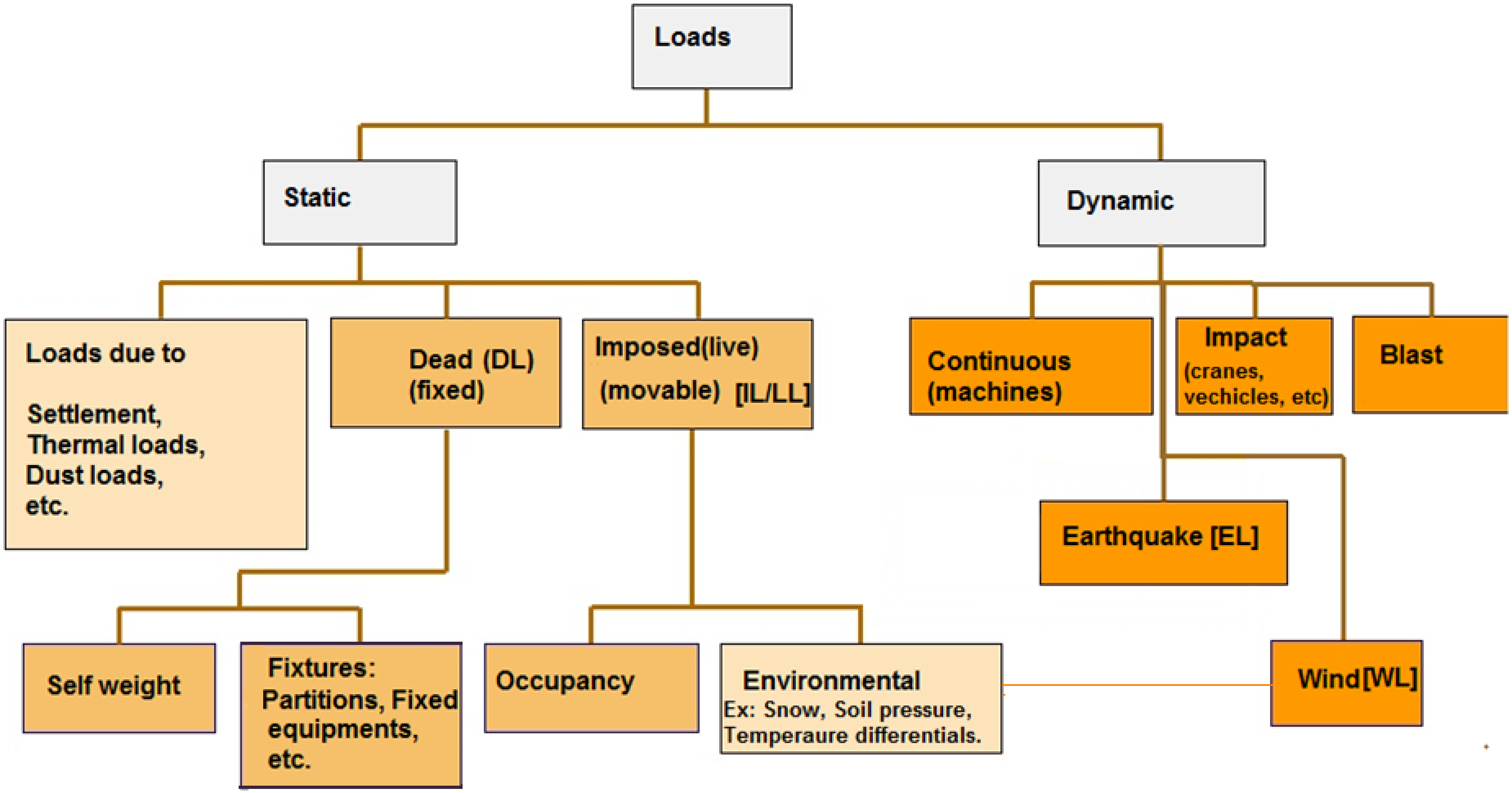
LOADING AND LOAD COMBINATIONS

Gravity

Wind

Earthquake

Different Types of Loads



Classification of Loads

Actions (Loads)

Permanent Actions:

Actions due to self weight of structural and non-structural components, fixed equipment and machinery, etc

Variable Actions :

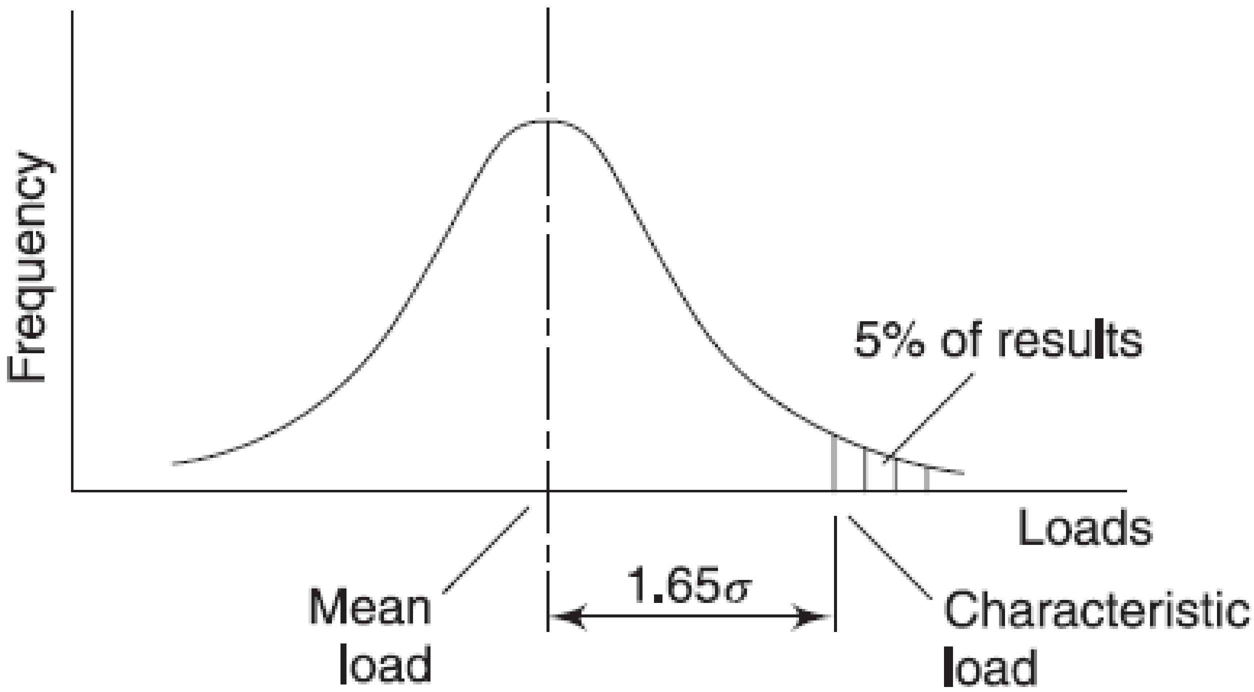
Actions due to construction and service stage loads such as imposed (live) loads (crane loads and snow loads), wind loads, earthquake loads, etc.

Accidental Actions:

Actions due to impact of vehicles, due to explosions, fire, etc.

Characteristic Loads

Definition: The load which shall not be exceeded by a certain – accepted or pre-assigned probability (usually 5%) during the life of the structure.



The specified load in the codes is normally taken as the characteristic load

Dead Loads (Fixed loads).

- **Examples:**
- **Weight of the structure**
  (walls, floors, roof, Ceiling, stairways, etc)
- **Fixed Service Equipment**
  (HVAC, Piping, Cable tray, etc.)

- **Mostly accurately estimated. Some times uncertain:**
  - **Pavement thickness, plastering thickness**
  - **Earth fill over underground structure**

Dead Loads

concrete	
lightweight	9-20
normal weight	24*
heavyweight	>28
reinforced and prestressed concrete	+ 1
unhardened concrete	+ 1
mortar	
cement mortar	23
gypsum mortar	17
lime mortar	19
masonry units	
basalt	27
limestone	25
granite	27
sandstone	23
Trachyte	26

IMPOSED LOADS(Movable)

- **Loads produced by use and occupancy**
- **Movable equipment**
- **Furniture within the buildings**
- **Stored Materials such as books, machinery**
- **Snow.**
- **They are different for different types of buildings: domestic, office, warehouse, etc.**
- **They vary often in space and in time.**
- **Generally expressed as static loads (UDL); may involve minor dynamic loads**

IMPOSED (LIVE) LOADS

Type of building	UDL (kN/m^2)	Concentrated load (kN)
Residential (dwellings, houses, hotels, hostels, hospitals and educational)		
(a) All rooms (living rooms, bedrooms, toilets and bathrooms)	2.0	1.8
(b) Kitchen	3	4.5
(c) Corridors, staircases	3–4	4.5
(d) Balconies	3.0	1.5 per m run at edge
(e) Restaurants, dining rooms, reading rooms without storage, boiler and plant	4	2.7–4.5
(f) Store rooms, boilers, projection	5	4.5
(g) Garage (vehicle up to 25 kN)	2.5	9.0 over 750 mm square
(h) Garage (vehicle up to 40 kN)	7.5	9.0 over 750 mm square
(i) Classrooms, dining halls, kitchens	3	2.7

Load due to partition: Increase the floor load by 33.3% per m subject to a minimum of 1 kN/m^2. Total weight /m < 4 kN/m.

Reduction in Live load for Columns of Multi-storey Buildings

Floor measured from top	Percentage
1 (top or roof)	0
2	10
3	20
4	30
5–10	40
11 to ground floor	50

A reduction in LL for beams equal to 5 % for each 50 m^2 floor area (for areas >50 m^2), subjected to a maximum reduction of 25% is allowed.

Imposed Loads on Roofs

Type of roof	Uniformly distributed imposed load measured on plan area	Minimum imposed load measured on plan
Flat, sloping, or curved roof with slopes up to and including 10°		3.75 kN uniformly distributed over any span of 1 m width of the roof slab and 9 kN uniformly distributed over the span of any beam or truss or wall.
(a) Access provided	1.5 kN/m^2	
(b) Access not provided (except ladder for maintenance)	0.75 kN/m^2	Half of case (a) above
Roof with slope greater than 10°	For roof membrane sheets or purlins 0.75 kN/m^2; For every degree increase in slope over 10 degrees; reduce by 0.02 kN/m^2	0.4 kN/m^2

Note:

1. All roof covering (other than glass) should be capable of carrying an incidental load of 900 N concentrated over an area of 125 mm^2.
2. Trusses, beams, columns, and girders excluding purlins can be designed for 2/3 of the live load on the roof.

Dynamic Imposed Loads

Structure	Impact allowance in percentage
Lifts and hoists	
(a) Frames	100
(b) Foundations	40
Reciprocating machinery—frames and foundations	50
Light machinery—structure and foundation	20
Electric overhead cranes	
(a) Girders	25
(b) Columns (class III and IV cranes)	25
(c) Columns (class I and II cranes)	10
(d) Foundations	0
Hand-operated cranes	
(a) Girders	10
(b) Columns and foundations	0

Note: In addition to the impact allowance in the vertical direction, additional loads in lateral and longitudinal directions must be applied on beams and columns as a percentage of the static load.

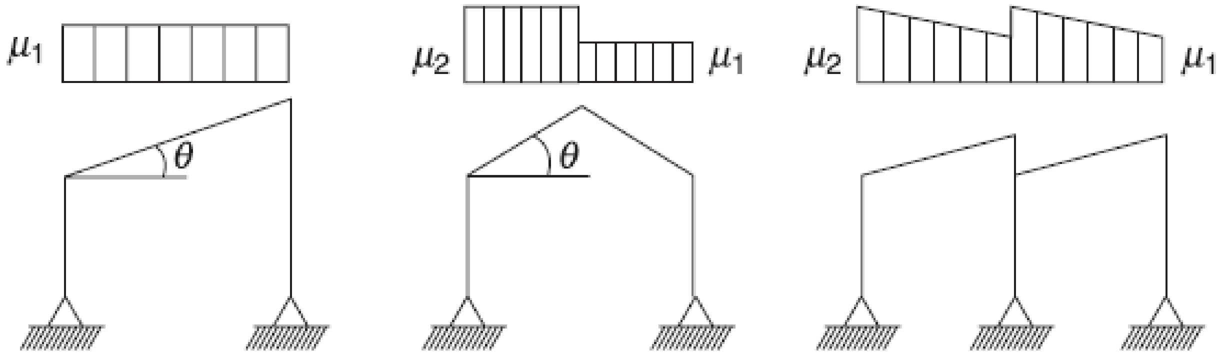
SNOW LOADS

- **Ground Snow Loads**
- **Based on Historic data (not always the maximum values)**
- **Ground Snow Load is multiplied by a shape coefficient to get design load on selected roofs.**
- **A more comprehensive formula is given for Multi-level roofs**
- **Drift of snow and Rain on-snow Surcharge near eaves of low-sloped (<15 degrees) should be given careful consideration.**

Snow Load

Roof snow load $s = \mu s_0$ Ground Snow load

d is the depth of snow in metres. $s_0 = (3 - 2e^{-1.5d})d$ kN/m^2



$\theta°$	μ_1	μ_1	μ_2	μ_1	μ_2
0–15	0.8	0.8	0.8	0.8	$0.8(1 + \theta/30)$
15–30	0.8	0.8	$0.8 + 0.4\ (\theta/15 - 1)$		
30–60	$0.8\ \alpha$	$0.8\ \alpha$	$1.2\ \alpha$	$0.8\ \alpha$	1.6
> 60	0	0	0	0	1.6

where $\alpha = (60 - \theta)/30$, A reduction of 25% is allowed in case the buildings are not sheltered by other buildings.

Temperature Effects

- Temperature effects are critical
  - **Where the difference in temperature is very large in short intervals**
  - **For structures like Microwave/Transmission line towers than protected structures like industrial bents.**
  - **Provide expansion joints at 40m intervals to minimize temperature effects or provide bracings to resist the effects.**
  - **Software such as ANSYS can tackle thermal loads**.

Erection/Loads during construction

- **Erection loads may control the design of certain members of cantilever bridges, cable supported structures, etc.**

- **Temporary bracings must be provided to take care of the stresses due to erection loads and accidental lateral loads(also due to wind). Several towers have failed due to the absence of bracings!**

- **Loads applied on structural members during construction (due to stockpiling of heavy construction materials) have lead to catastrophic failures. Several formwork failures are due to shoring of upper floors.**

- **Three beams failed at bolted connections and 10 bays destroyed in a progressive collapse in a 21-storey building under construction in Los Angeles, California , when 80 tons of structural steel was stockpiled in one bay on the 5th floor, in Dec. 1985**

Failure During Construction

The lateral-torsional buckling collapse of the 51.8m span Marcy (pedestrian) Bridge in upstate New York, during placement of concrete deck in October 2002, is due to the lack of full length lateral bracing system. The tub shaped girder is stable only when the composite concrete deck is completed to form a closed cross section.

ACCIDENTAL LOADS

- **They may be due to**
  - **Impact and collisions**
  - **Blast / Explosions**
  - **Fire**
  - **Case Study:** In May 1968, a natural gas explosion in a kitchen located in the 18th floor and in one of the four corners of a 23 – storey precast concrete building in Ronan Point, London, triggered a progressive collapse.

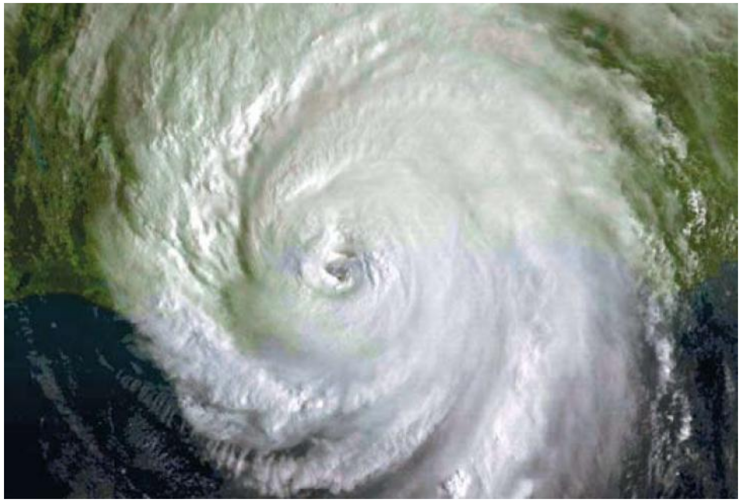
OTHER LOADS

- Differential Settlement
- Hydrostatic loads
- Wave and Current loads
- Earth pressure (basement)
- Ponding
- Lack-of-fit of members
- Resonant Loads
- Dust Loads
- Differential Shortening of columns
- Ice loads

Leaning tower of Pisa, Italy

WIND LOA

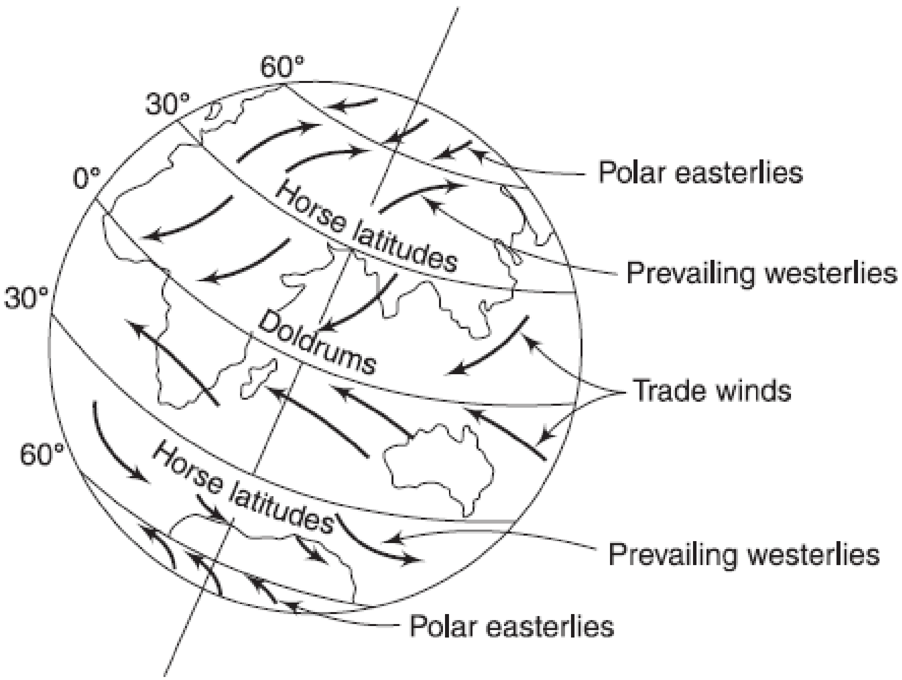
- **Wind flow manifests itself into may forms such as:**
  - **Gales**
  - **Cyclones/Hurricanes/Typhoons**
  - **Tornadoes**
  - **Thunder storms**
  - **Localized storms**

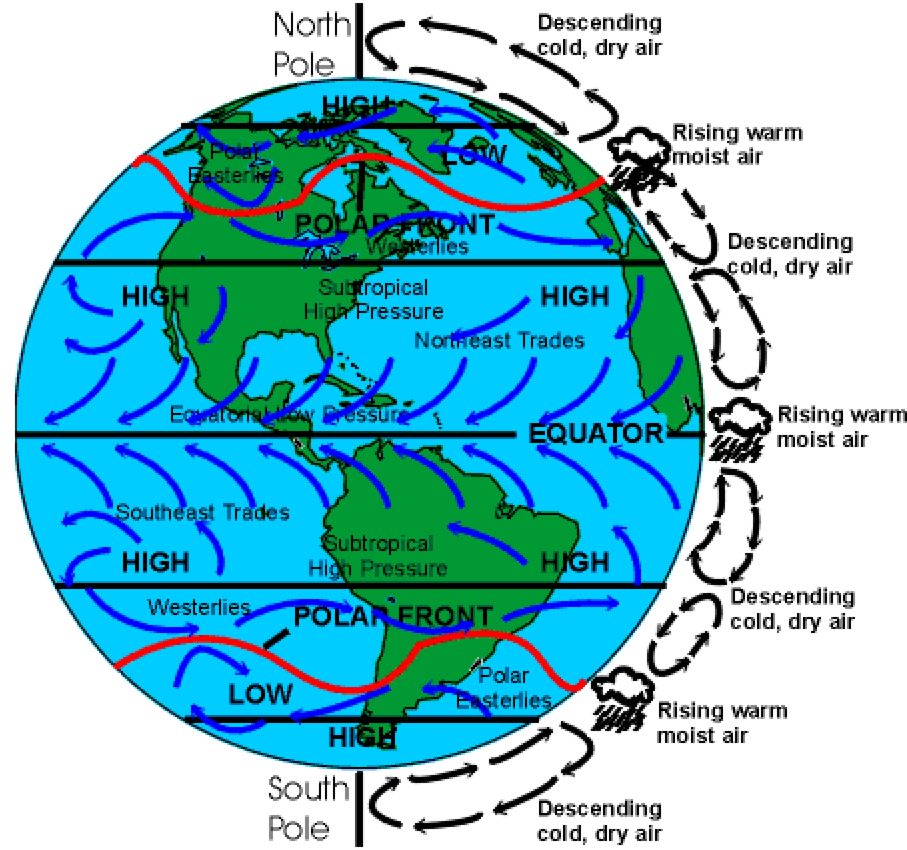


25-mile wide eye of Hurricane Katrina- from a NOAA Satellite

Spiraling tornado

Wind Loads (cont.)





Circulation of World's Winds (Taranath 1998)

Indian region experiences about 6 cyclones/year mostly on the east coast. Speed: 30-36 m/s to a max of 90m/s (324km/h)

Scales for measuring Windstorms

		Beaufort scale				Saffir–Simpson hurricane scale		
Bft	Descriptive term	Mean wind speed at 10 m above surface		Wind pressure	SS	Descriptive term	Mean wind speed	
		m/s	km/h	kg/m^2			m/s	km/h
0	Calm	0–0.2	0–1	0	1	Weak	32.7–42.6	118–153
1	Light air	0.3–1.5	1–5	0–0.1	2	Moderate	42.7–49.5	154–177
2	Light breeze	1.6–3.3	6–11	2.0–0.6	3	Strong	49.6–58.5	178–209
3	Gentle breeze	3.4–5.4	12–19	0.7–1.8	4	Very strong	58.6–69.4	210–249
4	Moderate breeze	5.5–7.9	20–28	1.9–3.9	5	Devastating	≥69.5	≥250
5	Fresh breeze	8.0–10.7	29–38	4.0–7.2			Fujita tornado scale	
6	Strong breeze	10.8–13.8	39–49	7.3–11.9	F	Descriptive term	m/s	km/h
7	Near gale	13.9–17.1	50–61	12.0–18.3	0	Weak	17.2–32.6	62–11
8	Gale	17.2–20.7	62–74	18.4–26.8	1	Moderate	32.7–50.1	118–180
9	Strong gale	20.8–24.4	75–88	26.9–37.3	2	Strong	50.2–70.2	181–253
10	Storm	24.5–28.4	89–102	37.4–50.5	3	Devastating	70.3–92.1	254–332
11	Violent storm	28.5–32.6	103–117	50.6–66.5	4	Annihilating	92.2–116.2	333–418
12	Hurricane	> 32.7	> 118	> 66.6	5	Disaster	116.3–136.9	419–493

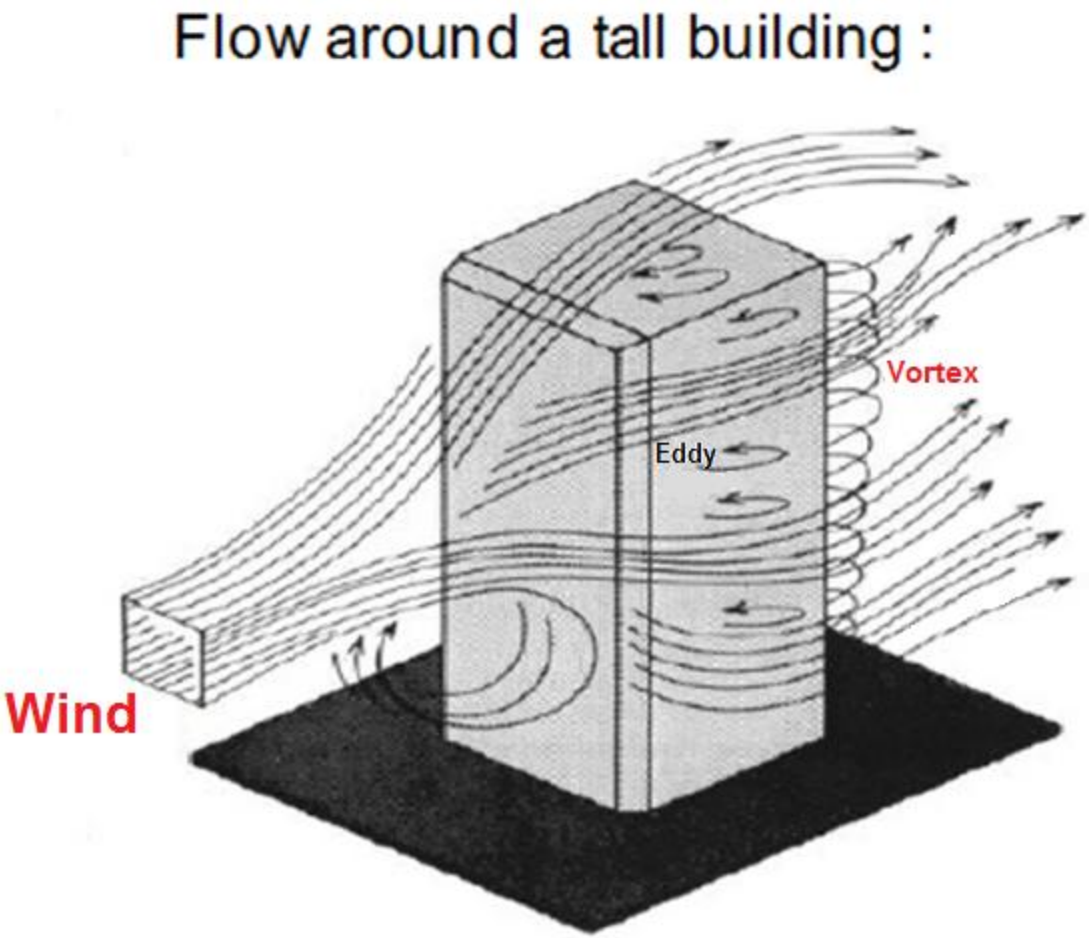
Wind Load on Buildings-IS 875(Part 3)

The wind pressure/load acting on the structural system depends on:

- **Velocity and density of air**
- **Height above ground level**
- **Shape and aspect ratio of the building**
- **Topography of the surrounding ground surface**
- **Angle of wind attack**
- **Solidity ratio or openings in the structure**
- **Dynamic effects induced by the wind load**

In determining the wind load, a probabilistic approach is often used. A mean return period of 50 years is used in the code.

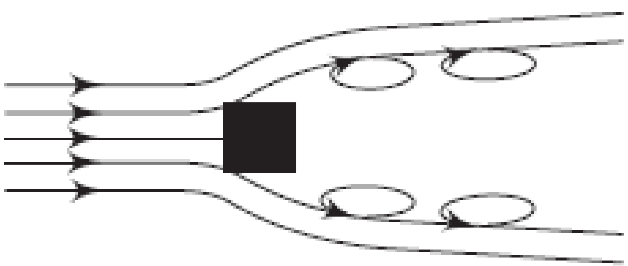
Vortex-Shedding in Tall buildings



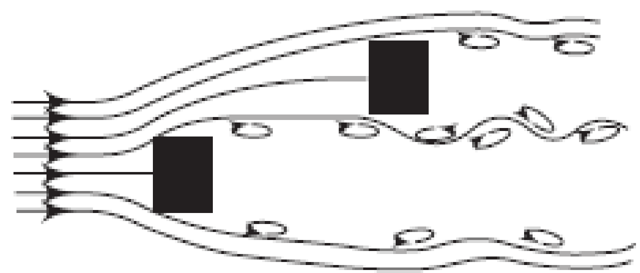
The cross wind response, that is, motion in a plane perpendicular to the direction of wind, dominates over the along-wind response for many tall buildings.

INTERFERENCE EFFECT

- **When one or more similar or dissimilar structures are placed downstream or upstream of the structure, the ‘stand-alone’ values of pressures and forces get altered. This is termed as *interference effect*.**
- **This effect is considered in the draft wind coo**



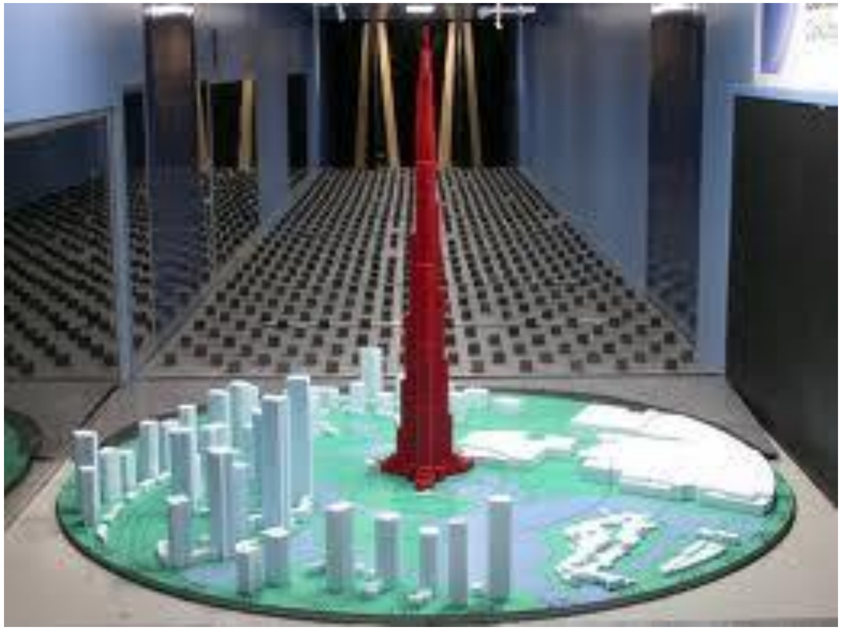
Flow around isolated building



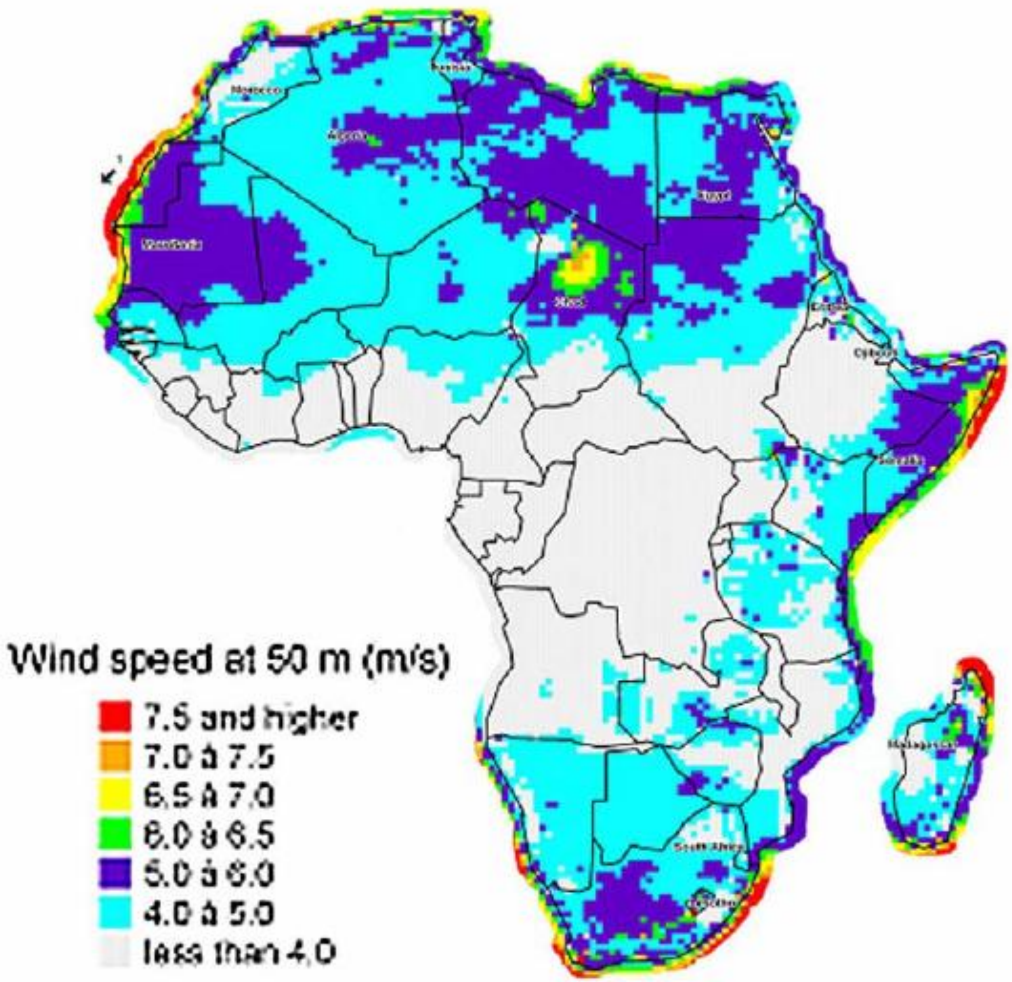
Flow around two buildings

Wind Tunnel Tests

- **Accurate determination of wind loads on complicated /tall buildings is possible only by conducting tests in Wind Tunnels.**
- **In India also several boundary-layer wind tunnel facilities exist and include NAL, Bangalore, IISc., Bangalore, SERC, Chennai, IIT Bombay, IIT Roorkee and IIT Delhi**



Basic Wind Speed Map of Africa



Design Wind Speed

$V_z = V_b \, k_1 \, k_2 \, k_3$

- **V_z = Design wind speed at any height z in m/s**
- **V_b = Basic wind speed**
- **k_1 = Probability factor or risk coefficient**
- **k_2 = Terrain, height and structure size factor**
- **k_3 = Topography factor**

The design wind pressure

$$p_d = 0.6 \, V^2_z$$

Wind force on Element/Cladding

- **Wind causes pressure or suction normal to the surface of a structure.**
- **Wind pressure acting normal to the individual element or cladding unit is given by**

  $$F = (C_{pe} - C_{pi})\, A p_d$$

Where, F = net wind force on the element

A = Surface area of element or cladding

C_{pe} = external pressure coefficient

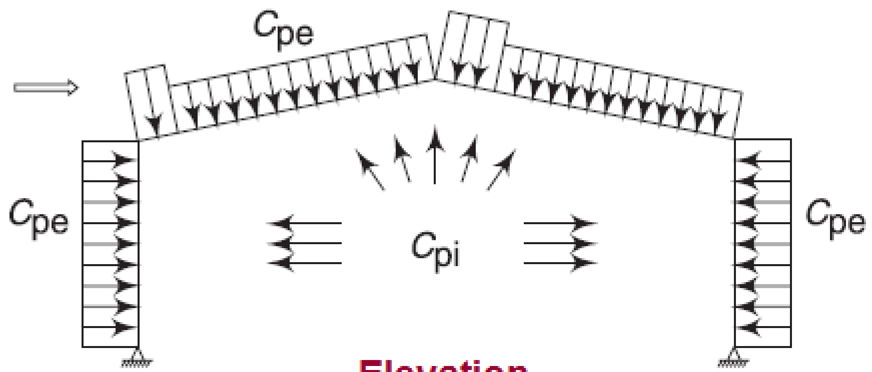
C_{pi} = internal pressure coefficient

p_d = design wind pressure

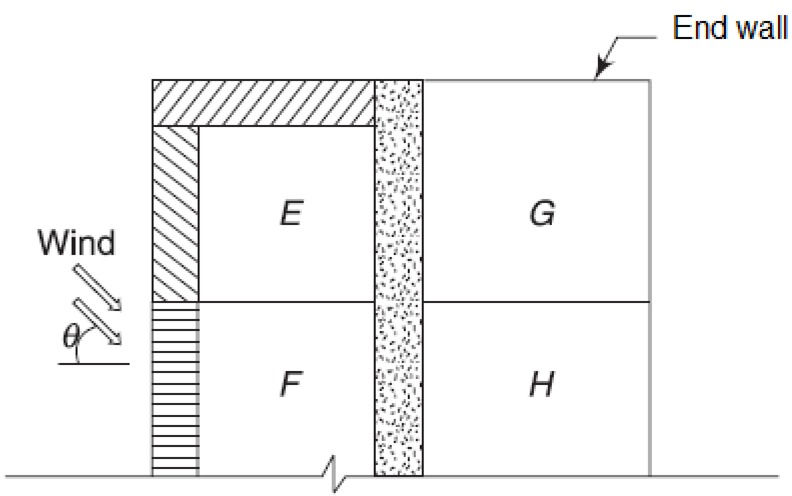
Wind Pressure Coefficients

- **The wind pressure coefficients depend on the following:**
  - **shape of the building or roof,**
  - **slope of the roof**
  - **direction of wind with respect to building**
  - **zone of the building**

The code EBCS 1 gives external coefficients for different types of structures.



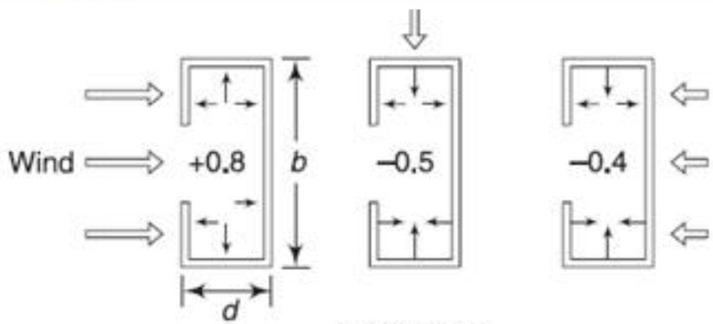
Elevation
(a)



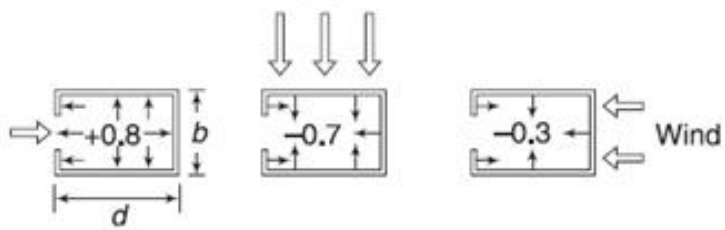
Half Plan
(b)

Internal Pressure Coefficients

Type of building	C_{pi}
Buildings with low permeability (less than 5% openings in wall area)	± 0.2
Buildings with medium permeability (5–20% openings in wall area)	± 0.5
Buildings with large permeability (openings in wall area > 20%)	± 0.7
Buildings with one side opening	See Fig.



For (b/d) > 1
(a)



For (b/d) < 1
(b)

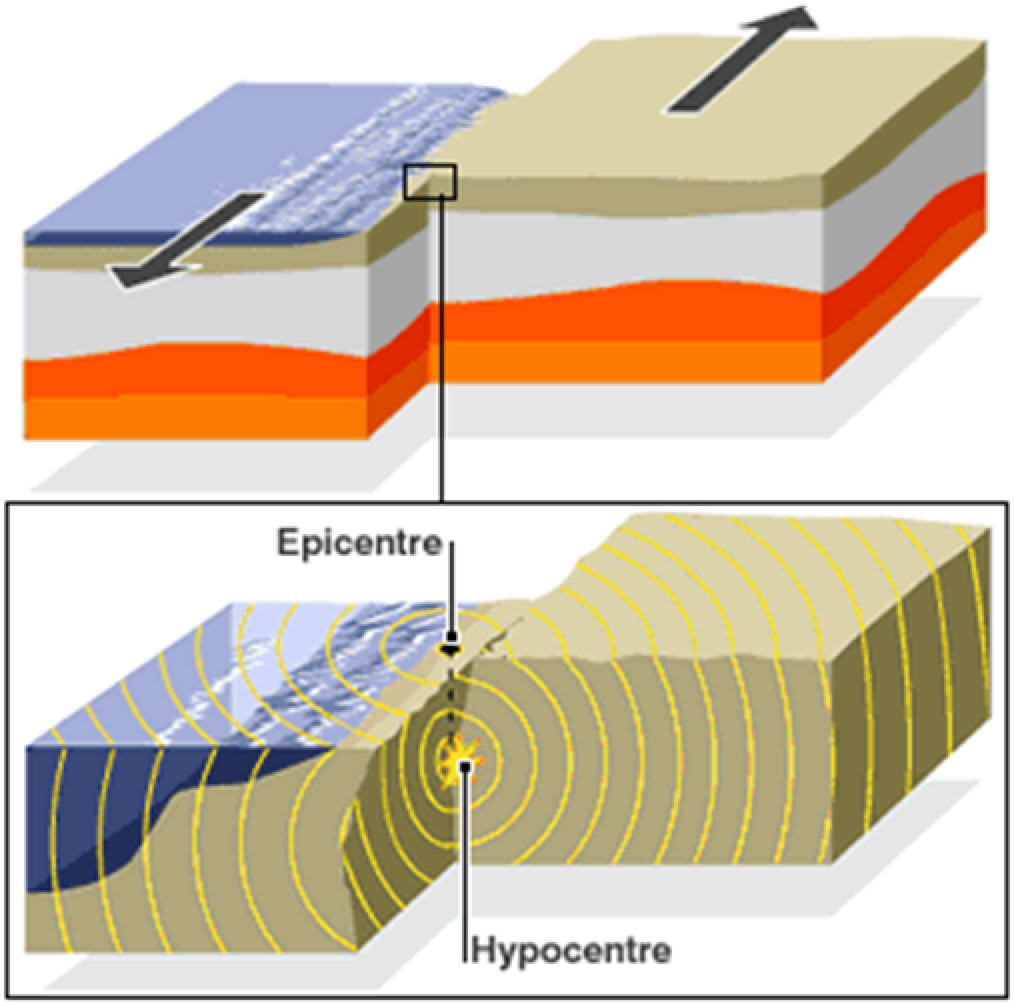
EARTHQUAKES

The crust of earth has 13 large plates with thickness-32-240 km.

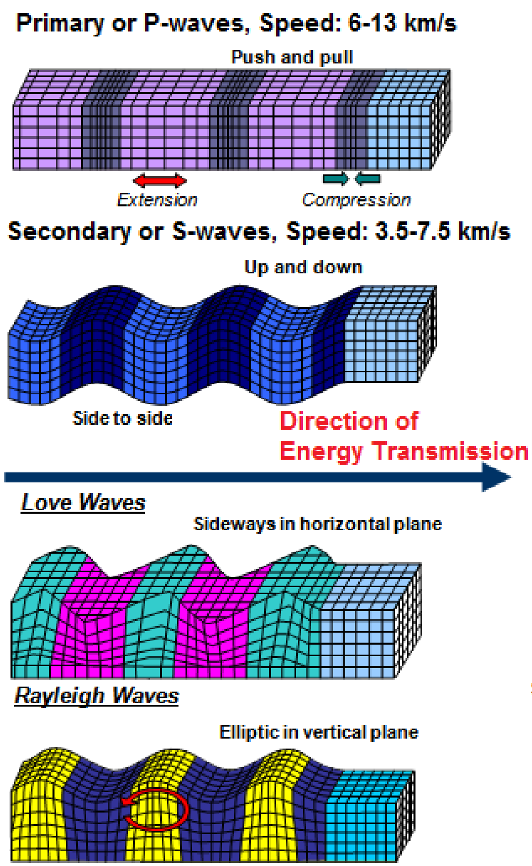
Earthquakes occur when two tectonic plates move suddenly against each other. The rocks break underground at the hypocentre and the earth starts to shake.

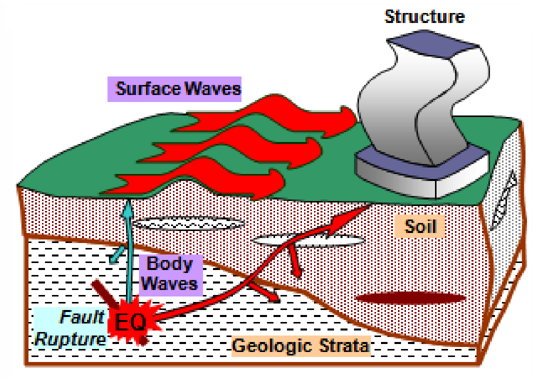
Waves spread from the epicentre, the point on the surface above the hypocentre.

If a quake occurs under the sea, it can cause a tsunami!



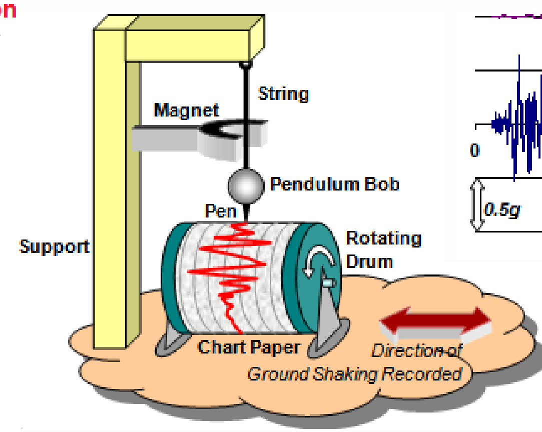
Characteristics of an Earthquake



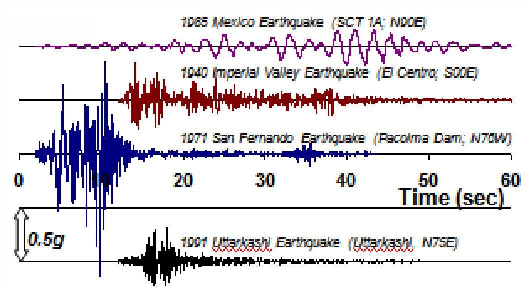


Arrival of Seismic Waves at a Site

An earthquake of magnitude 6 is 31.6 times more powerful than one measuring 5 in Richter Scale.



Schematic of Early Seismograph



Typical recorded accelerograms

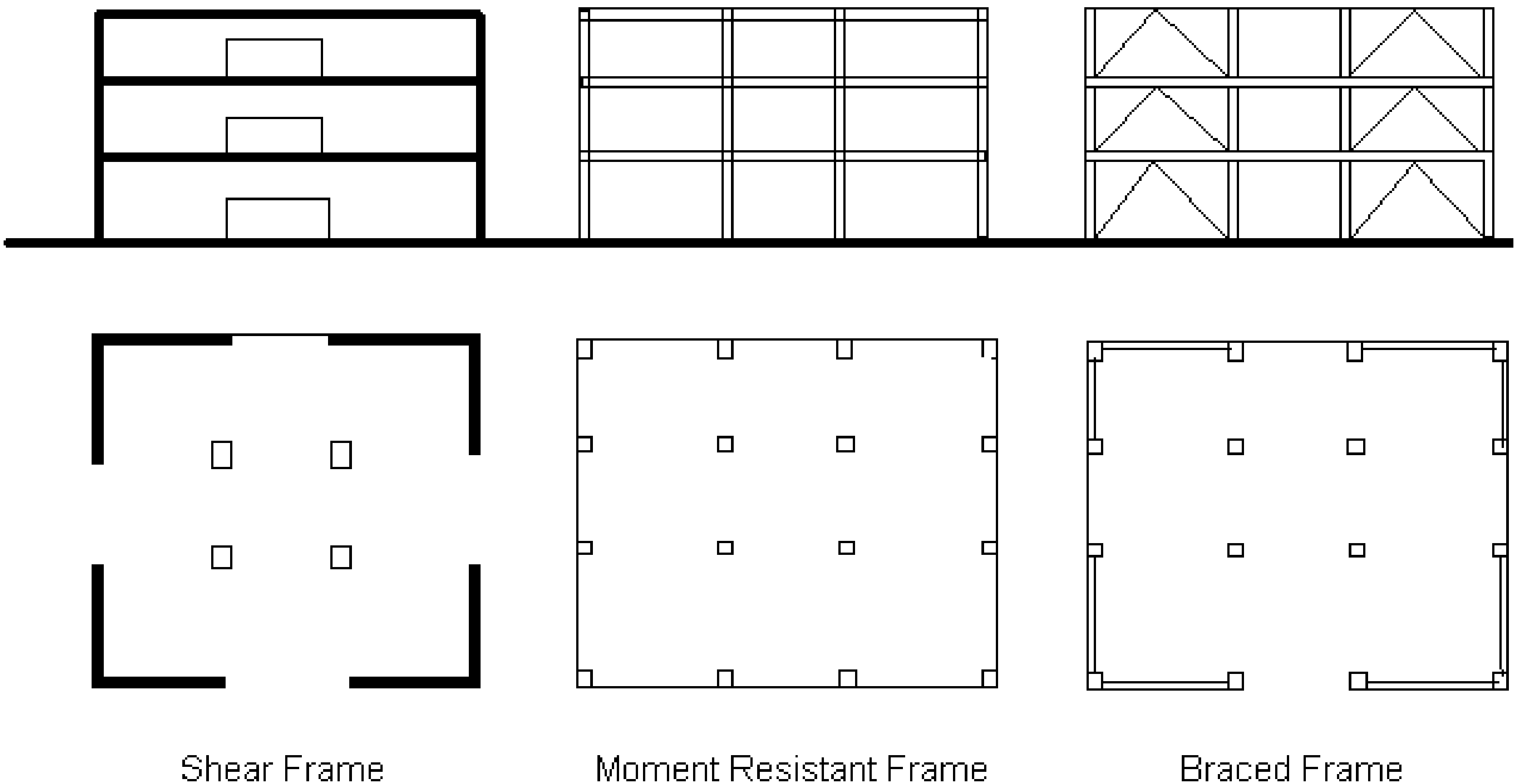
Factors Influencing Seismic Damage

The following factors influence the seismic damage:

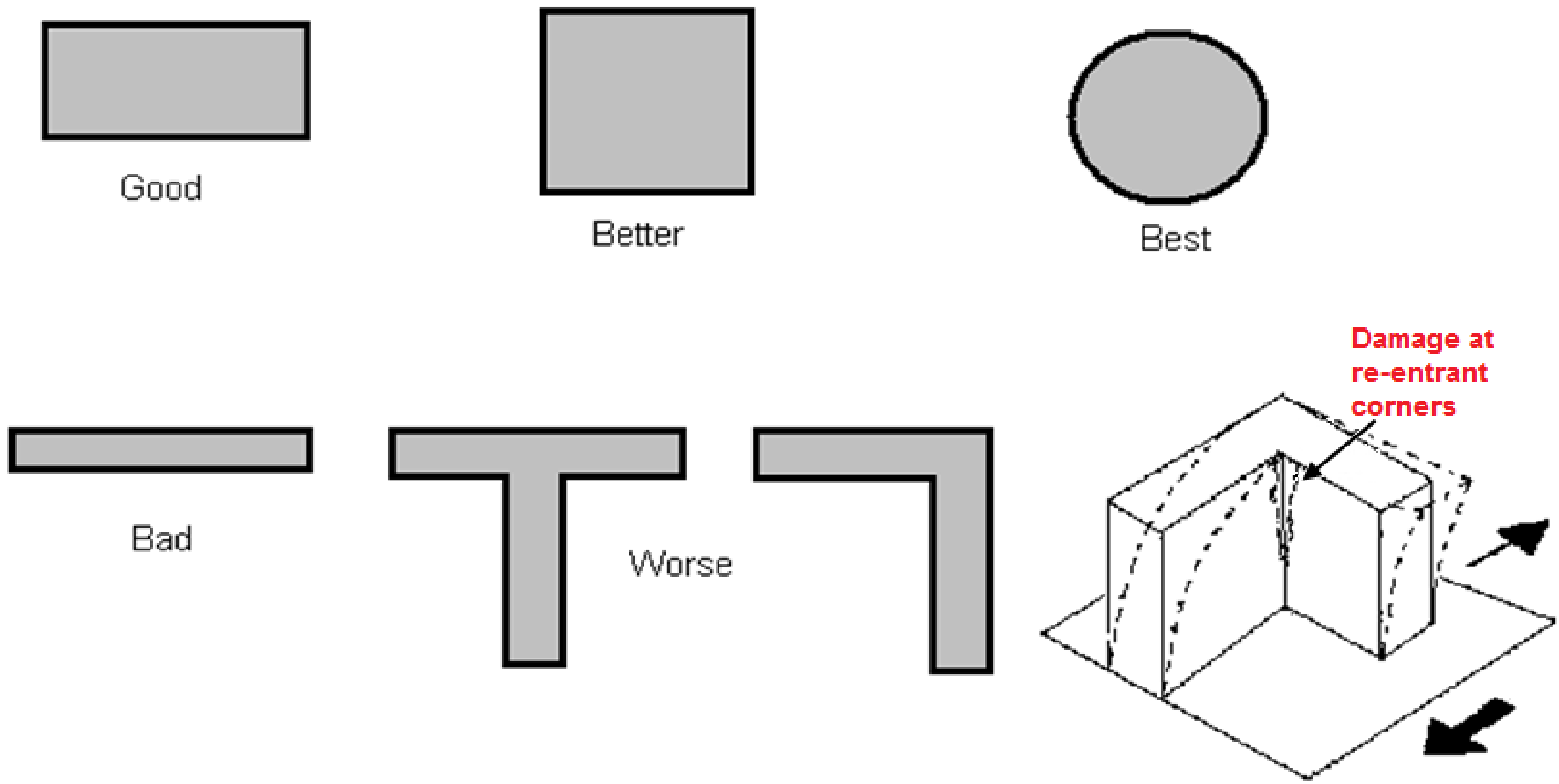
- **Peak Ground Acceleration (PGA)**
- **Amplitude,**
- **Duration and frequency of ground vibration,**
- **Magnitude,**
- **Distance from epicenter**
- **Geographical conditions between the epicenter and the site,**
- **Soil properties at the site and foundation type**
- **Building type and characteristics.**

Damage to a Steel building in Mexico City, 1985

Lateral Force Resisting Systems

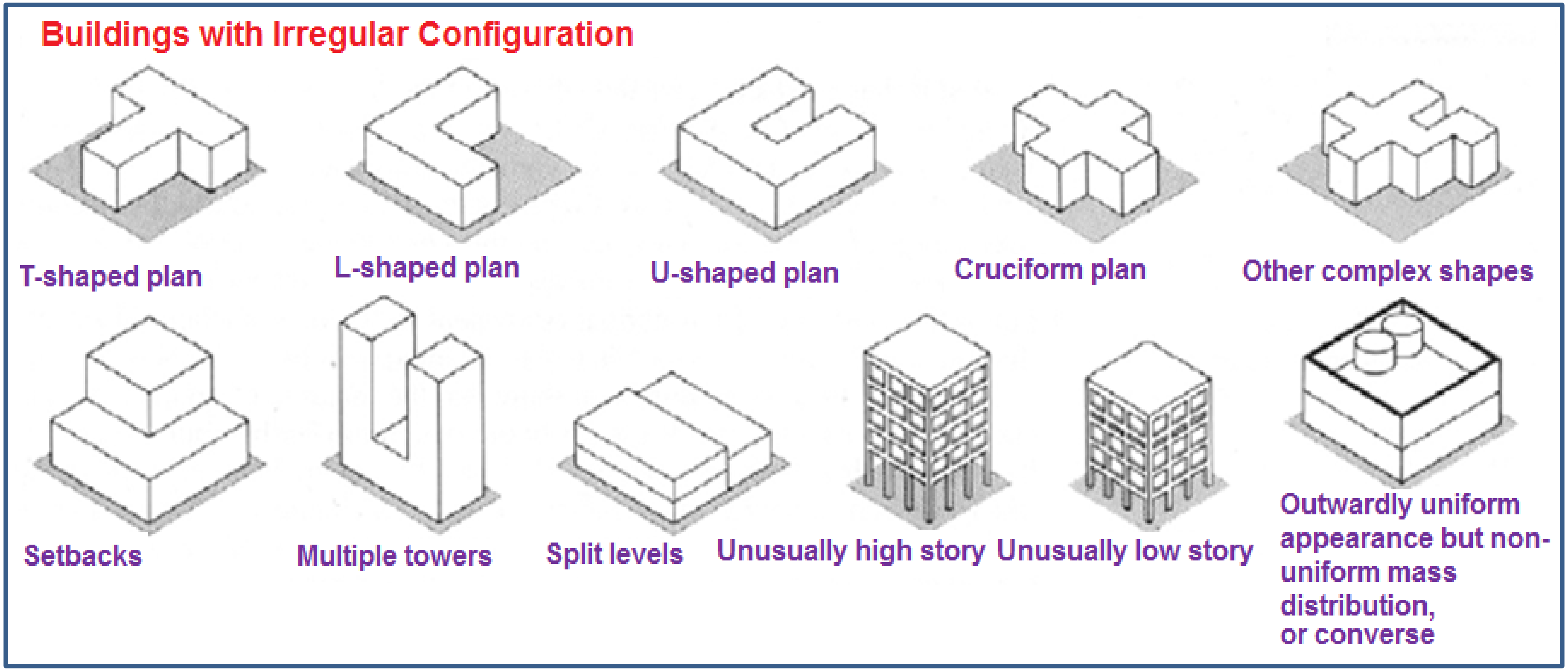


Better Performance in Earthquakes



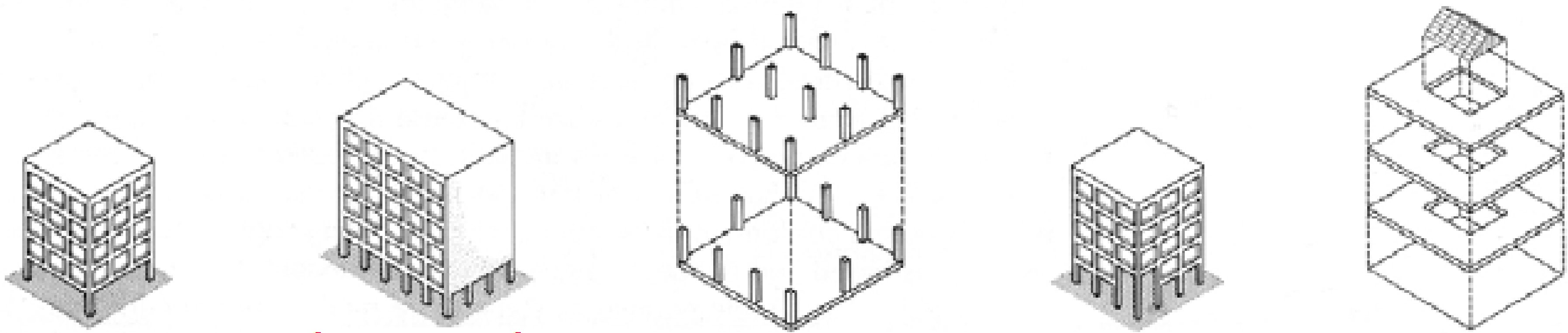
Have simple and regular Plans

Avoid Irregular Configurations



Avoid Abrupt changes in Lateral Resistance

Buildings with Abrupt Changes in Lateral Resistance



'Soft' lower levels

Large openings in shear walls

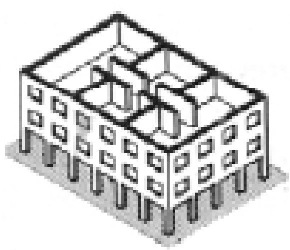
Interruption of columns

interruption of beams

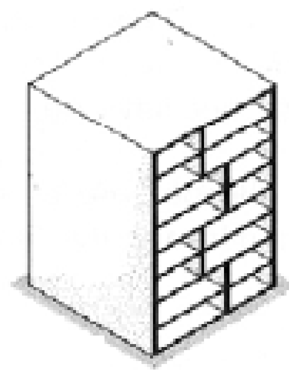
Opening in diaphragms

Avoid Abrupt Changes in Lateral Stiffness

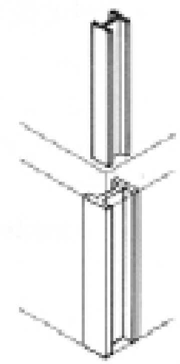
Buildings with Abrupt Changes in Lateral Stiffness



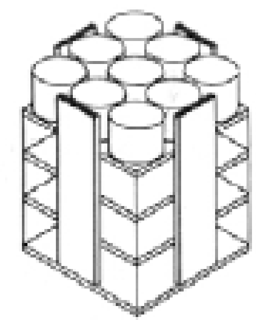
Shear walls in some stories, moment-resisting frames in others



Interruption of vertical-resisting elements

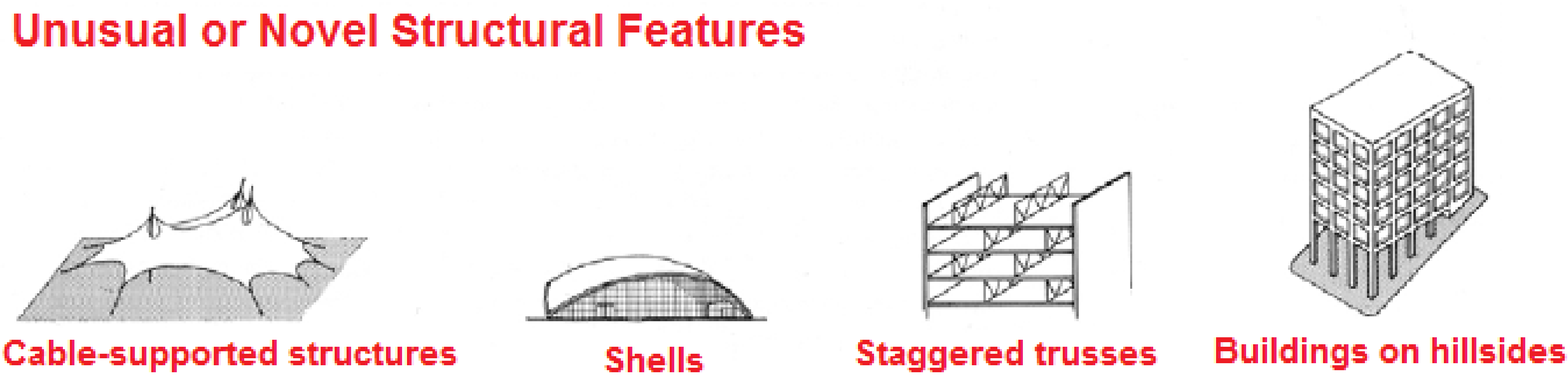


Abrupt changes in size of members

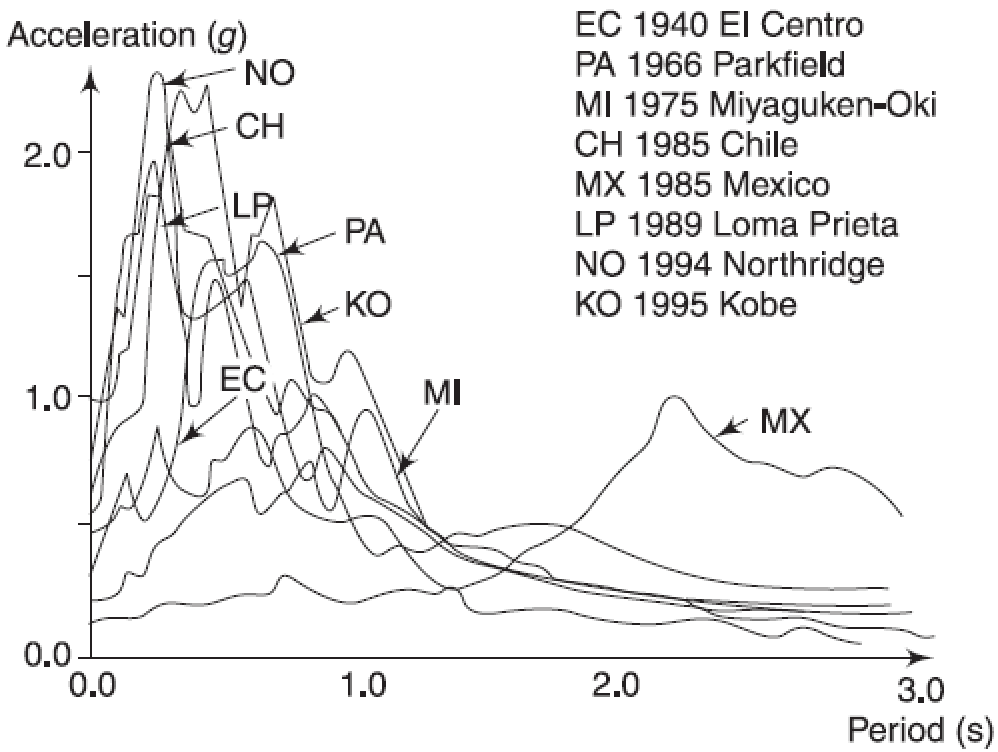


Drastic changes in mass/stiffness ratio

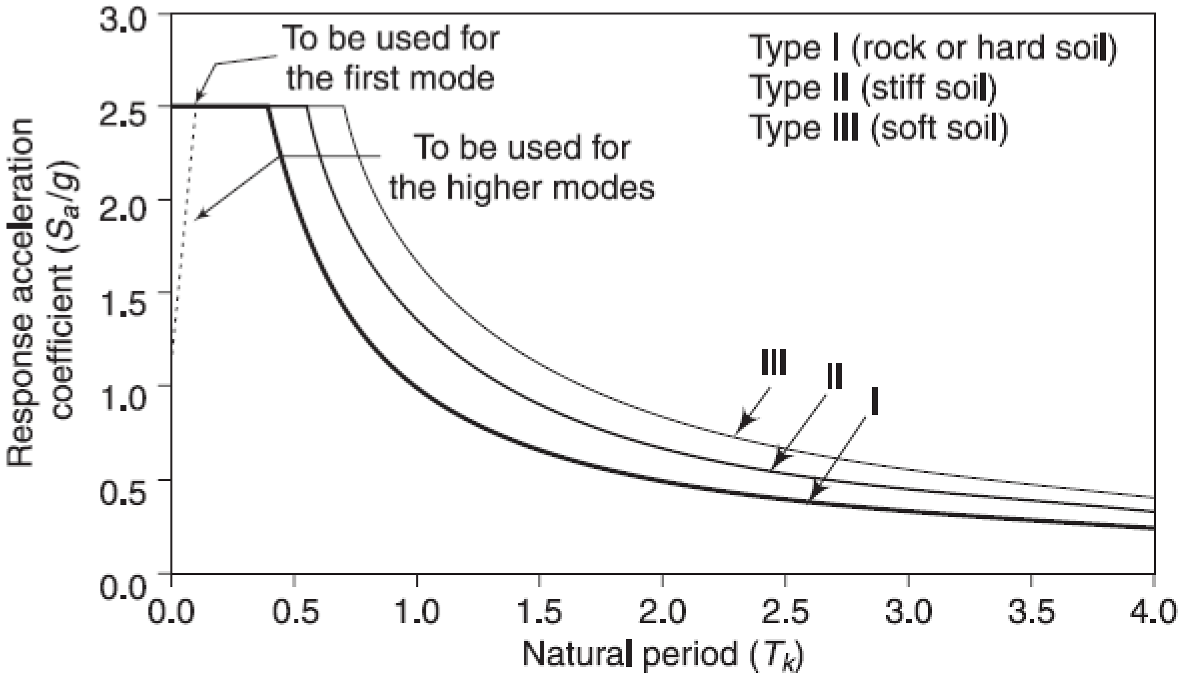
Avoid Novel Structural Features (If their EQ behavior is not known)



Response Spectra for Different Strong Earthquakes



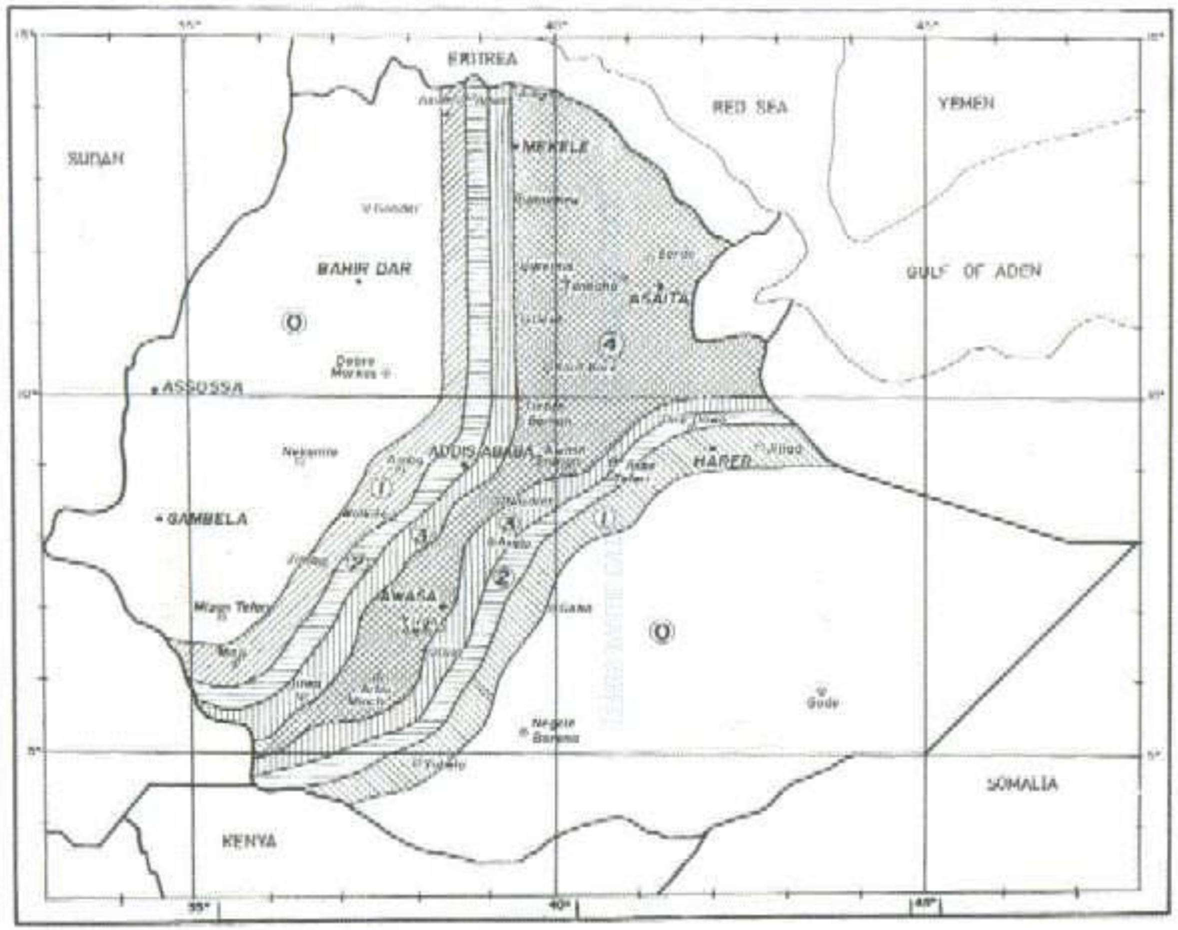
Response Acceleration coefficient as given in Code



Damping percentage	Factors
0	3.20
2	1.40
5	1.00
7	0.90
10	0.80
15	0.70
20	0.60
25	0.55
30	0.50

Smoothened Elastic Design Acceleration Response Spectrum (SEDRS) for 5% damping.
For Steel structures use 2% damping

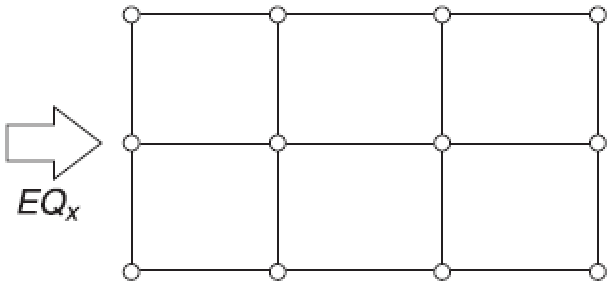
SEISMIC ZONES OF ETHIOPIA

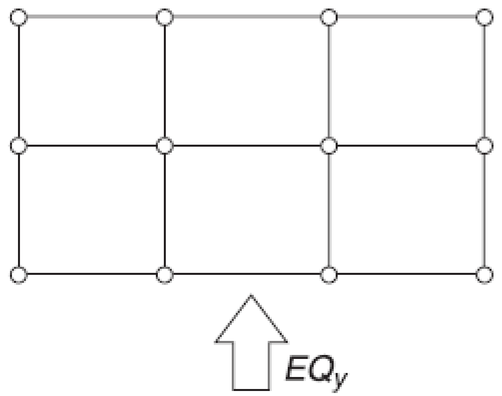


Load Combinations

In general consider the 8- load combinations:

(1) 1.5 (DL + IL) + 1.05(CL or SL)
(2) 1.2 (DL + IL) + 1.05(CL or SL) ± 0.6(WL or EL)
(3) 1.2 (DL + IL ± WL or EL) + 0.53 (CL or SL)
(4) 1.5(DL ± WL or EL)
(5) 0.9 DL ± 1.5 (WL or EL)
(6) 1.2 (DL + ER)
(7) 0.9DL + 1.2 ER
(8) DL + 0.35(IL + CL or SL) + AL





Where, DL = Dead load, IL = imposed load (live load), WL = wind load, SL = snow load, CL = crane load (vertical / horizontal), AL = accidental load, ER = erection load and EL = earthquake load.

Compression Members

- Structural elements that are subjected to axial compressive forces only are called columns.
- Columns are subjected to axial loads through the centroid.
- The stress in the column cross-section can be calculated as

$$f = P/A$$

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

UNIT-II

DESIGN OF COMPRESSION MEMBERS

Compression Members

- This ideal state is never reached. The stress-state will be non-uniform due to:
  - Accidental eccentricity of loading with respect to the centroid
  - Member out-of –straightness (crookedness), or
  - Residual stresses in the member cross-section due to fabrication processes

Compression Members

- Sometime they may carry bending moments as well about one or both axis of cross-section.
- The bending action may produce tension in part of the cross-section
- Despite of tensile stresses or forces that may produce, columns are generally referred as "Compression Members" because compression stresses normally dominate their behavior.

Compression Members

- In addition to most common type of compression members (vertical Members in structure),compression may include the
  - Arch ribs
  - Rigid frame members inclined or otherwise
  - Compression elements in trusses

Compression Members

Compression Members

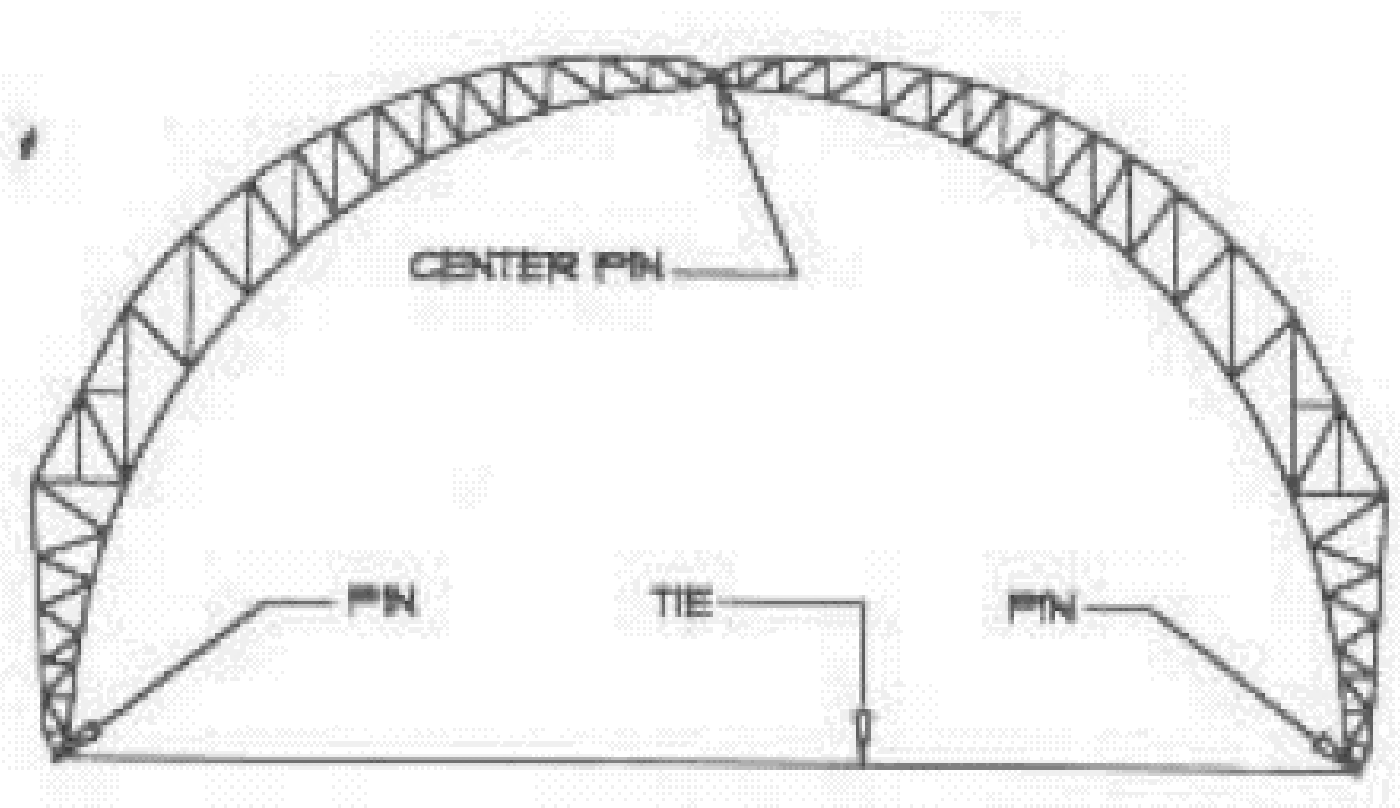


FIG. 2.8 THREE-HINGED TRUSS

Compression Members

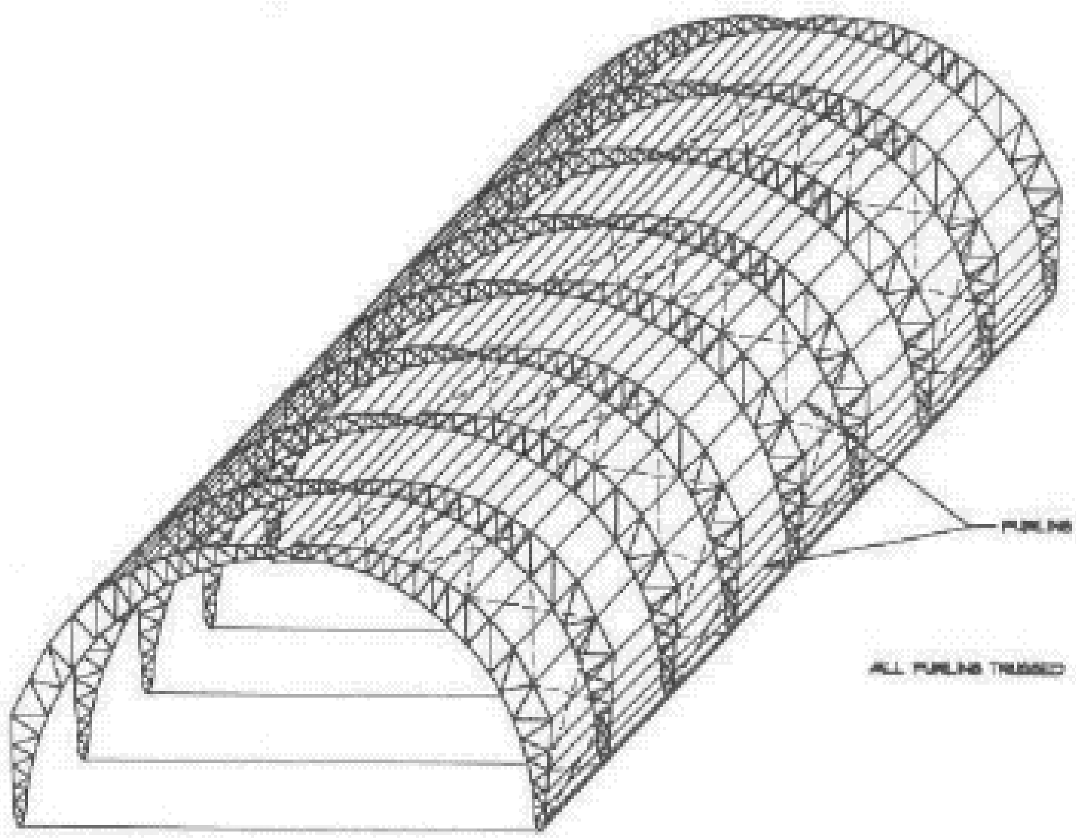


FIG. 2.9 STEEL FRAME WITH
THREE-HINGED ARCHES

Compression Members

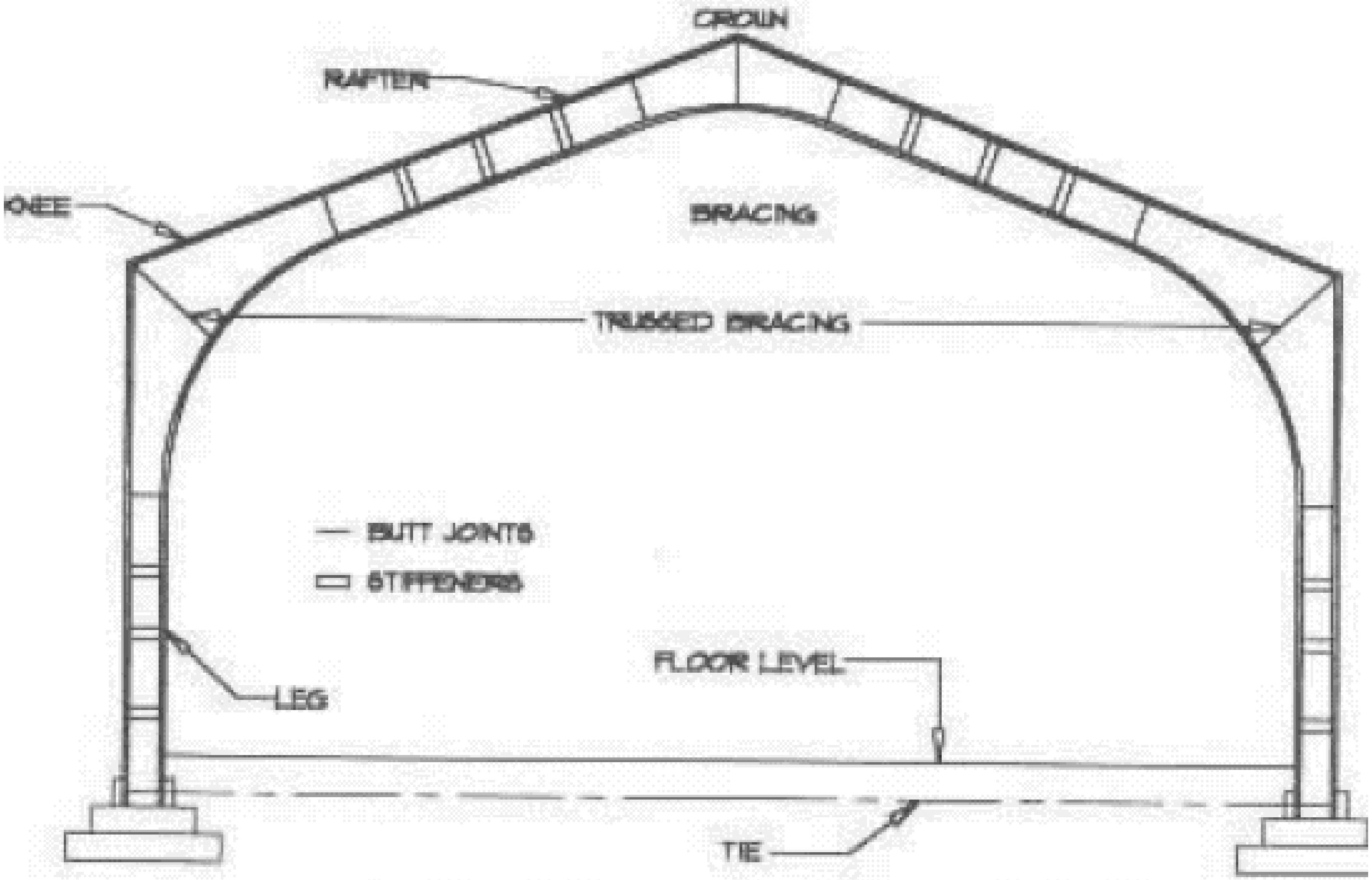


FIG. 2.7 STEEL RIGID FRAME

Compression Members

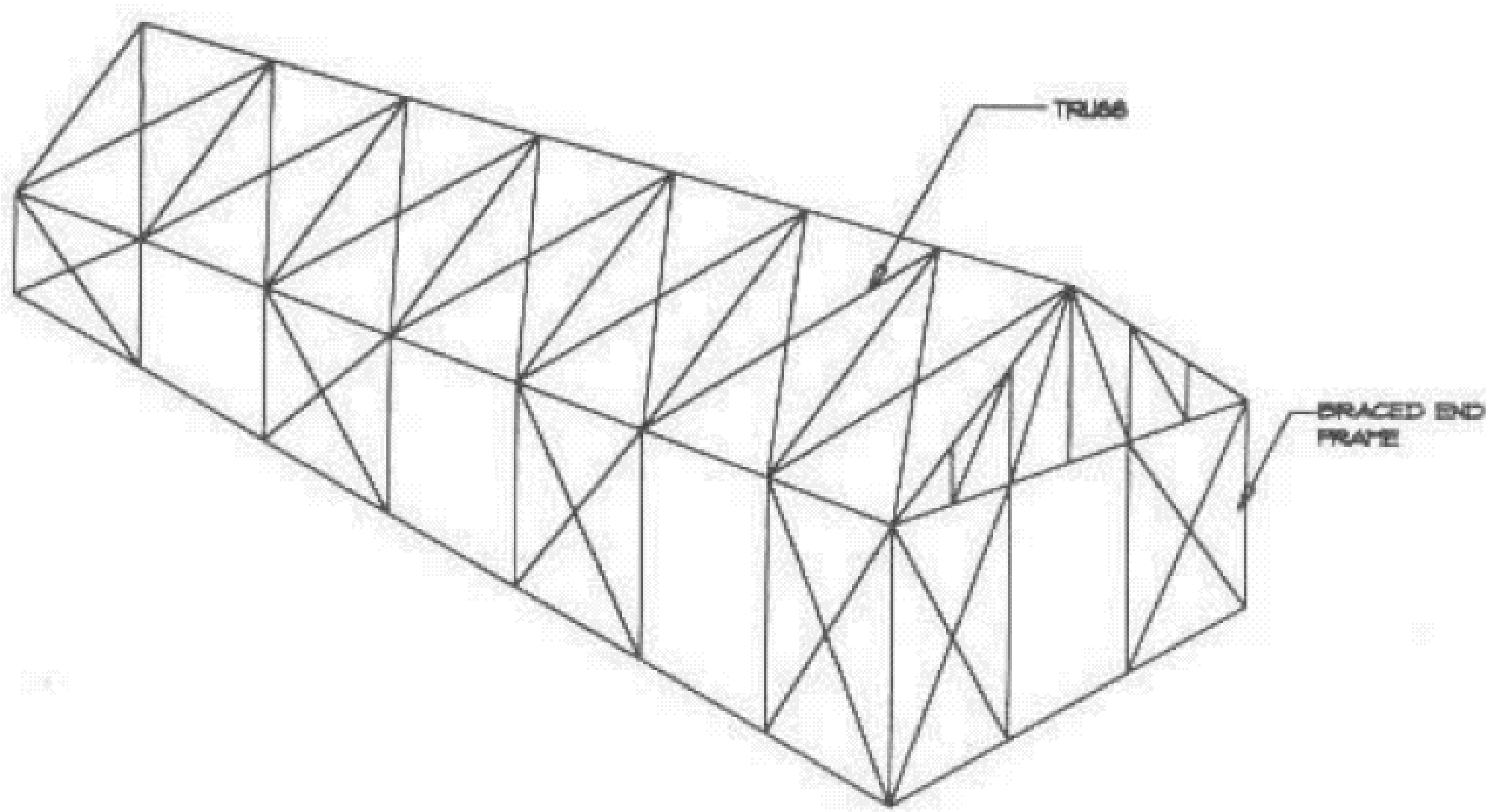


FIG. 2.6 BRACED FRAME

Compression Members

Compression Members Vs Tension Members

Slenderness Ratio

- The longer the column, for the same x-section, the greater becomes its tendency to buckle and smaller becomes its load carrying capacity.

- The tendency of column to buckle is usually measured by its slenderness ratio

$$\text{Slenderness Ratio} = \frac{L}{r}$$

$$\text{where } r = \sqrt{\frac{I}{A}} = \text{radius of gyration}$$

Compression Members Vs Tension Members

Effect of material Imperfections and Flaws

- Slight imperfections in tension members are can be safely disregarded as they are of little consequence.
- On the other hand slight defects in columns are of great significance.
- A column that is slightly bent at the time it is put in place may have significant bending resulting from the load and initial lateral deflection.

Compression Members Vs Tension Members

- Tension in members causes lengthening of members.
- Compression beside compression forces causes buckling of member.

Compression Members Vs Tension Members

- Presence of holes in bolted connection reduce Gross area in tension members.
- Presence of bolts also contribute in taking load An = Ag

WHY column more critical than tension member?

- A column is more critical than a beam or tension member because minor imperfections in materials and dimensions mean a great deal.

WHY column more critical than tension member?

- The bending of tension members probably will not be serious as the tensile loads tends to straighten those members, but bending of compression members is serious because compressive loads will tend to magnify the bending in those members.

Compression Member Failure

- There are three basic types of column failures.
- One, a compressive material failure(very short and fat).
- Two, a buckling failure,(very long and skinny).
- Three, a combination of both compressive and buckling failures.(length and width of a column is in between a short and fat and long and skinny column).

Compression Member Failure

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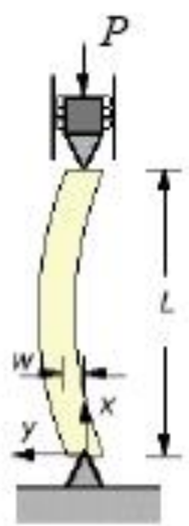
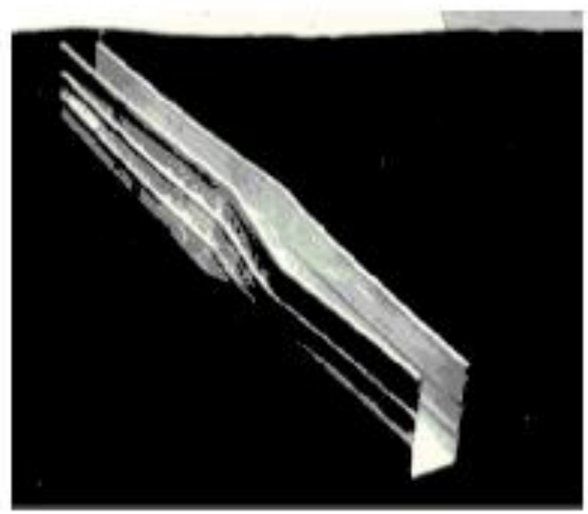
Compression Member Failure

- <u>**Flexural Buckling**</u> (also called Euler Buckling) is the primary type of buckling.members are subjected to bending or flexure when they become unstable

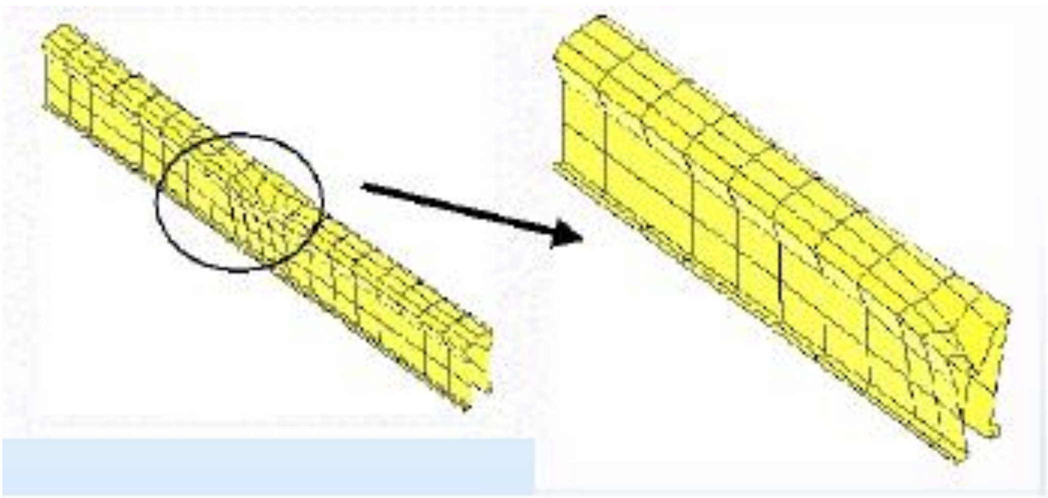


Simply supported column subjected to axial load P



Compression Member Failure

- **<u>Local Buckling</u>** This occurs when some part or parts of x-section of a column are so thin that they buckle locally in compression before other modes of buckling can occur



Compression Member Failure

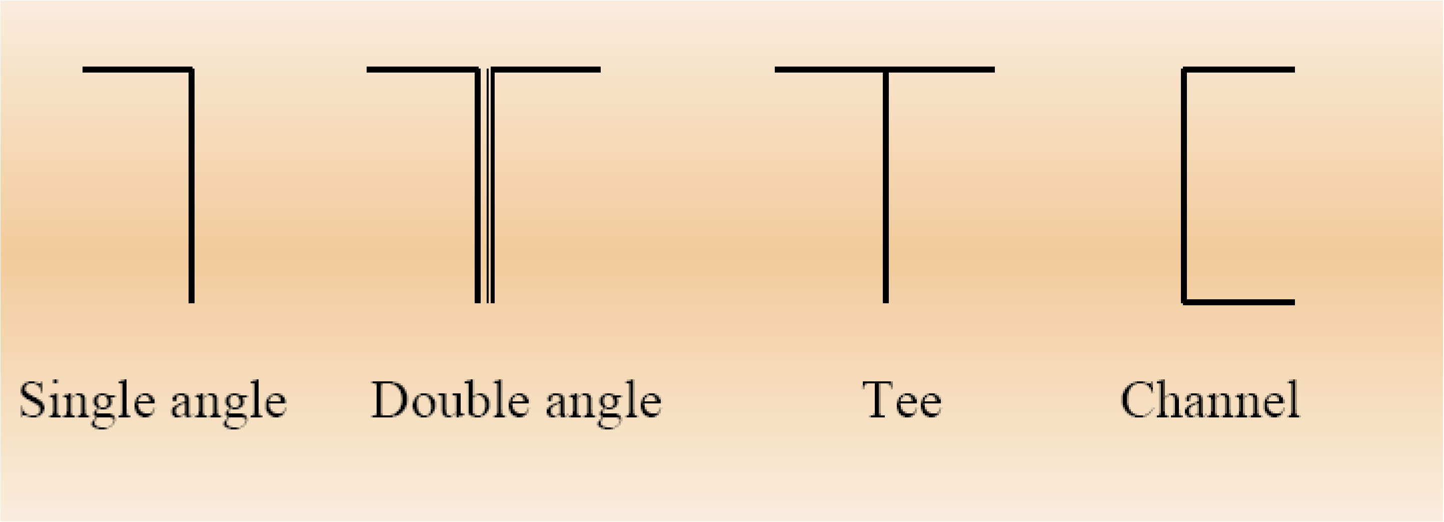
- **Torsional Buckling** These columns fail by twisting(torsion) or combined effect of torsional and flexural buckling.

Sections used for Compression Member

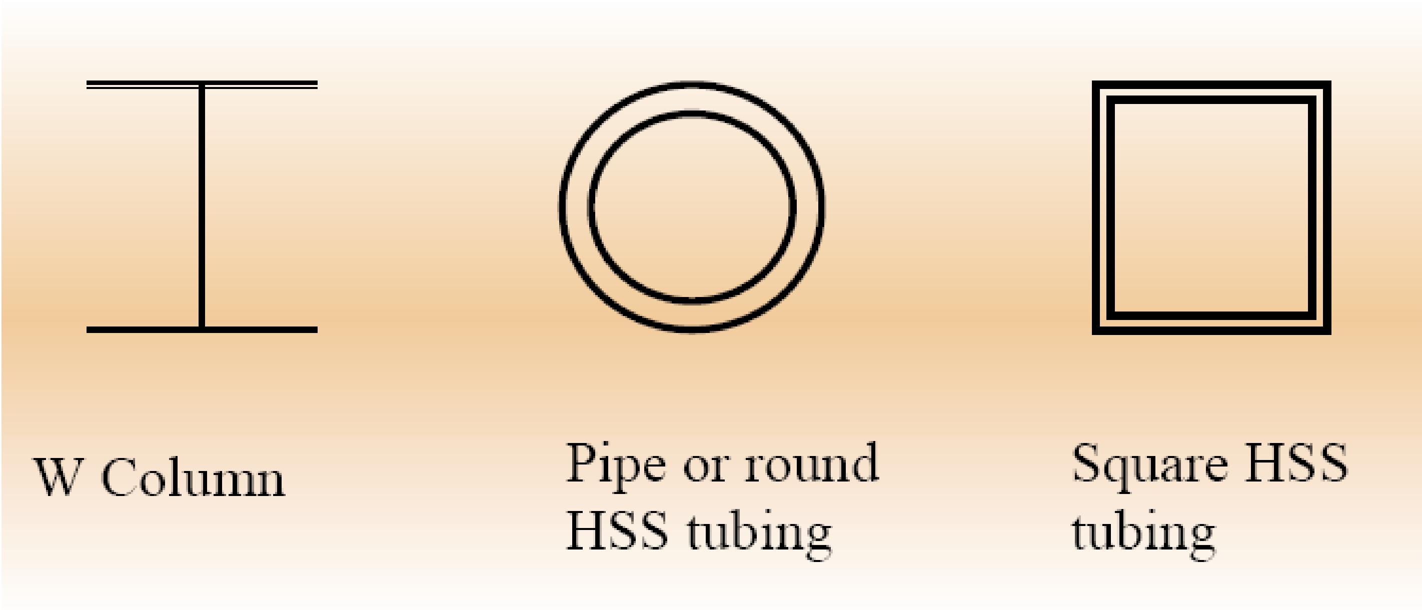
- In theory numerous shapes can be used for columns to resist given loads.
- However, from practical point of view, the number of possible solutions is severely limited by section availability, connection problems, and type of structure in which the section is to be used.

Sections used for Compression Member

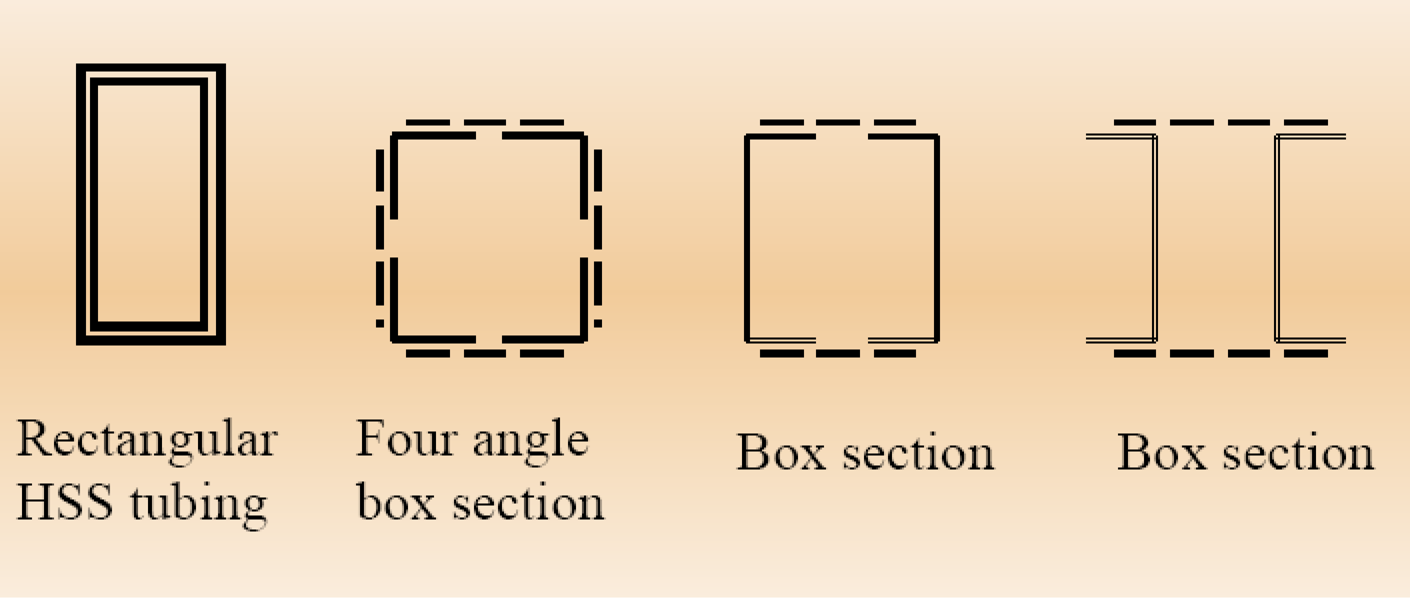
Figure 1. Types of Compression Members



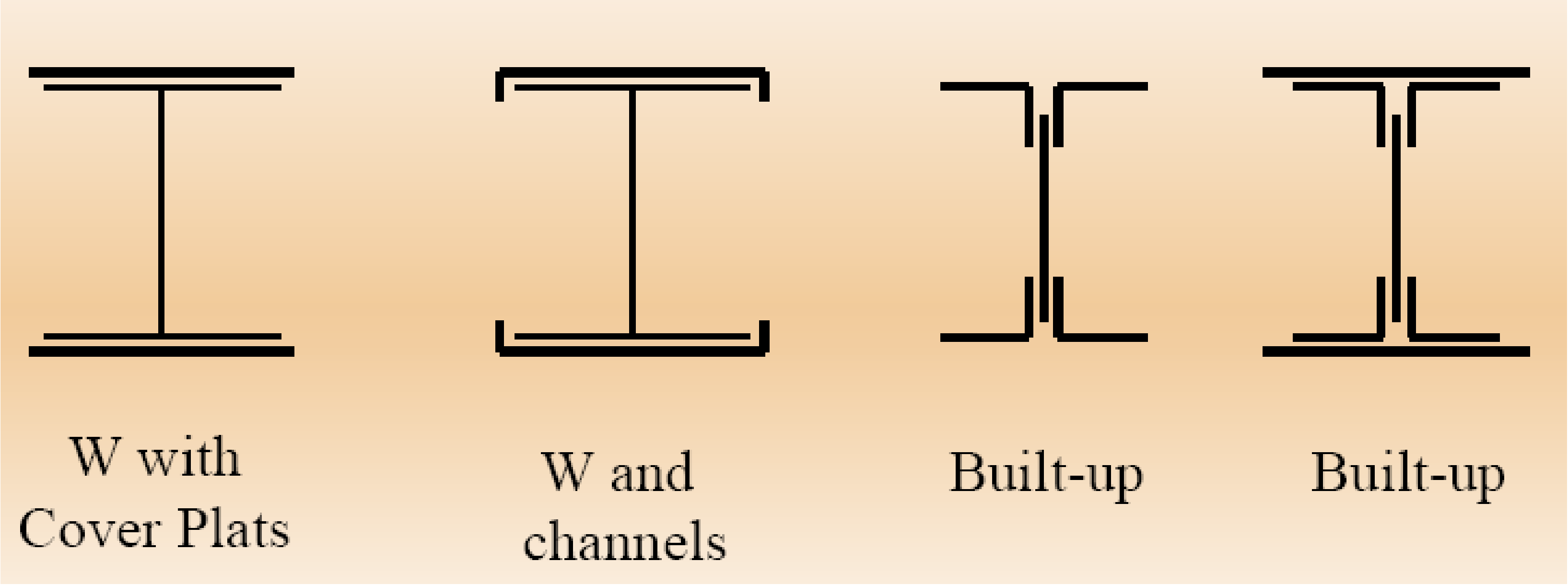
Sections used for Compression Member



Sections used for Compression Member



Sections used for Compression Member



Column Buckling

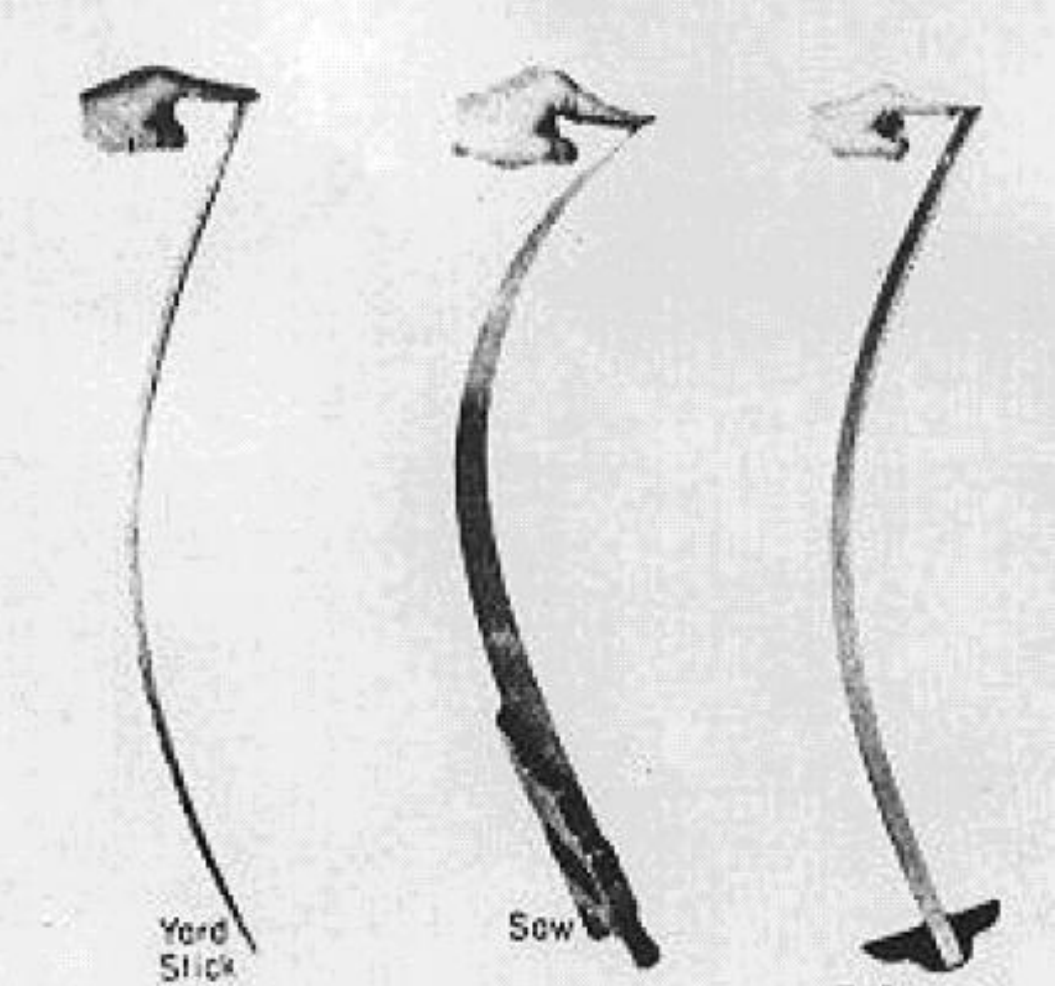
- Buckling
- Elastic Buckling
- Inelastic Buckling

Column Buckling

- Buckling is a mode of failure generally resulting from structural instability due to <u>compressive</u> action on the structural member or element involved.
- Examples of commonly seen and used tools are provided.

Buckling

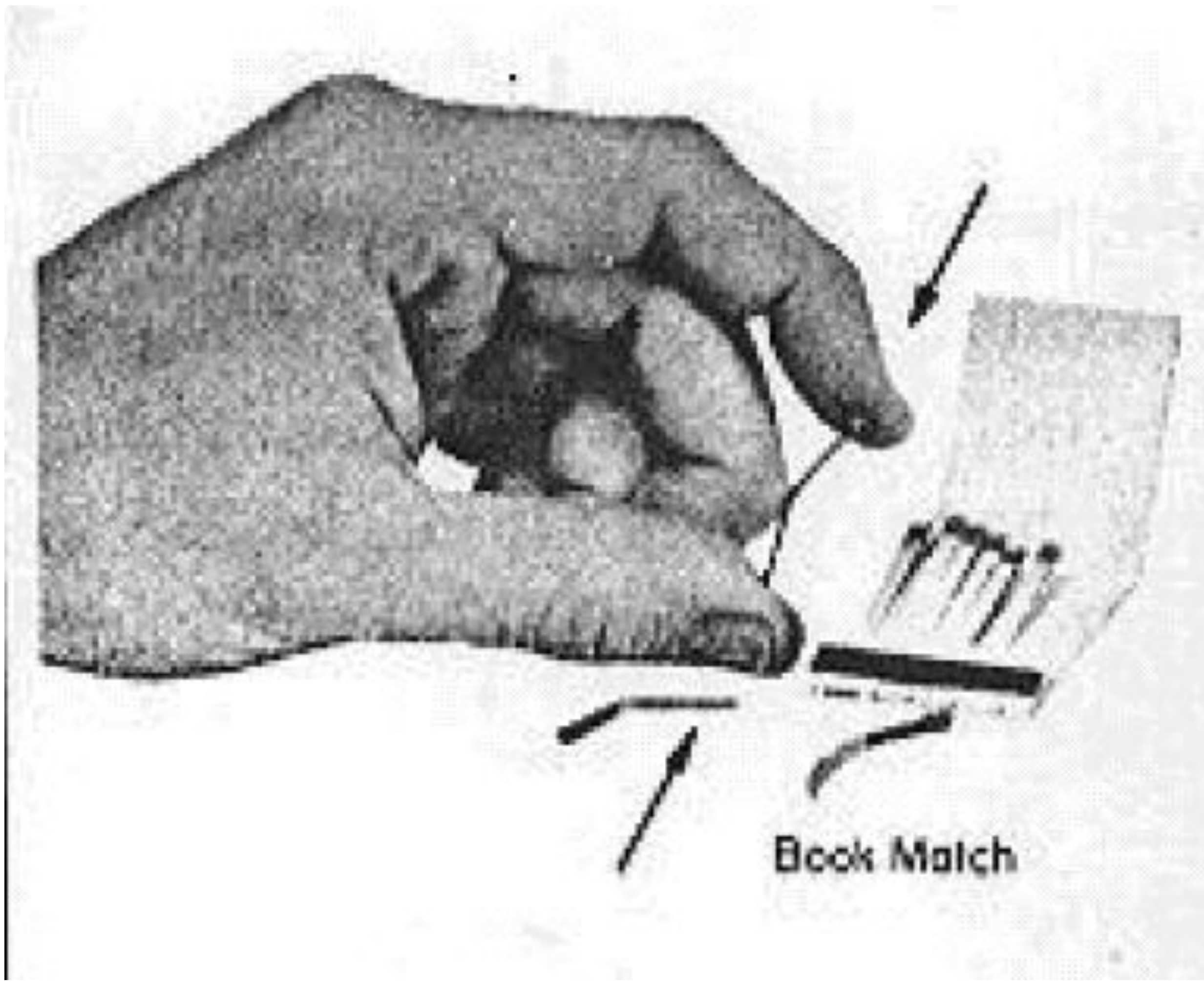
Example



(a)

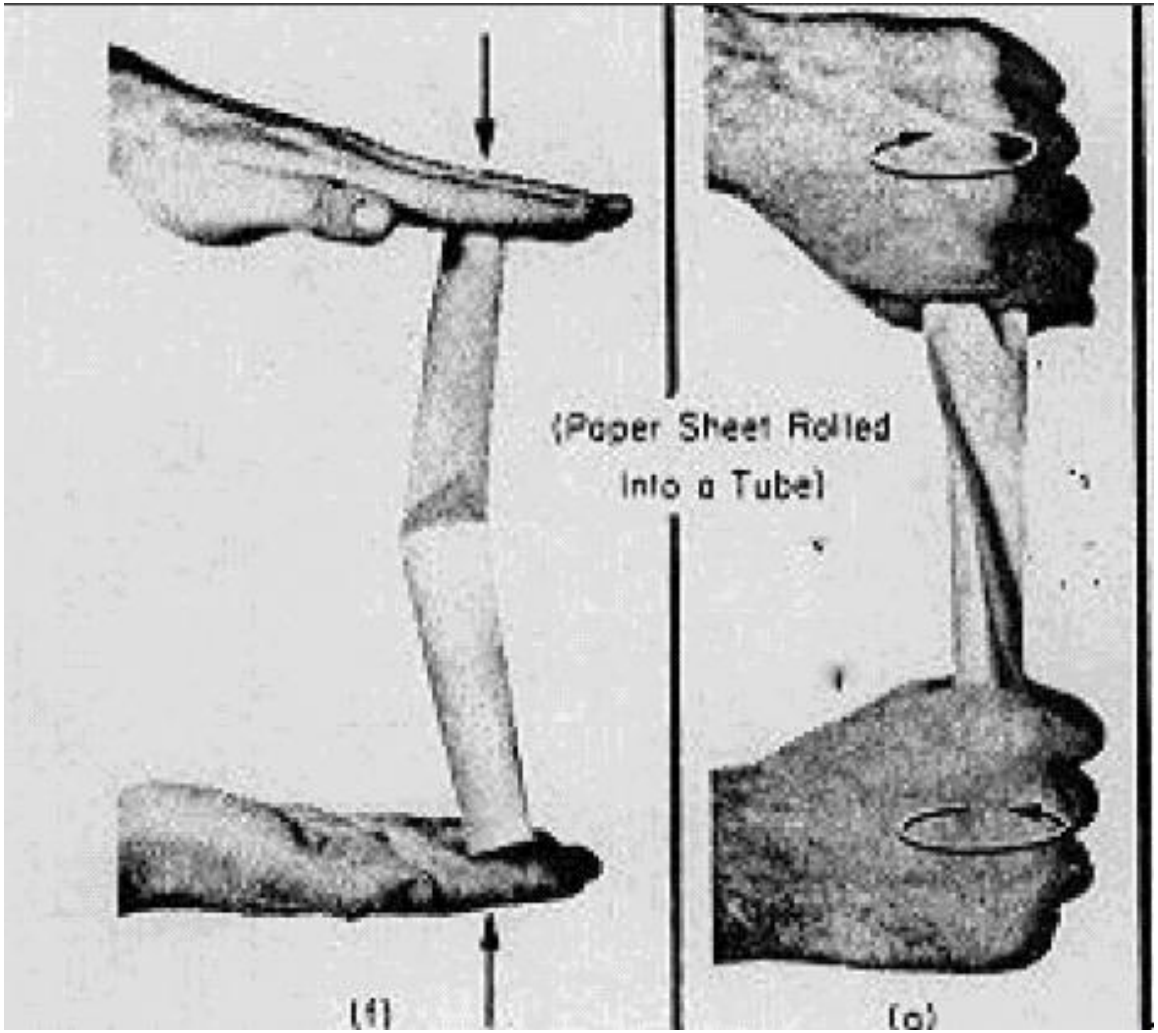
Buckling

Example



Buckling

Example



Buckling

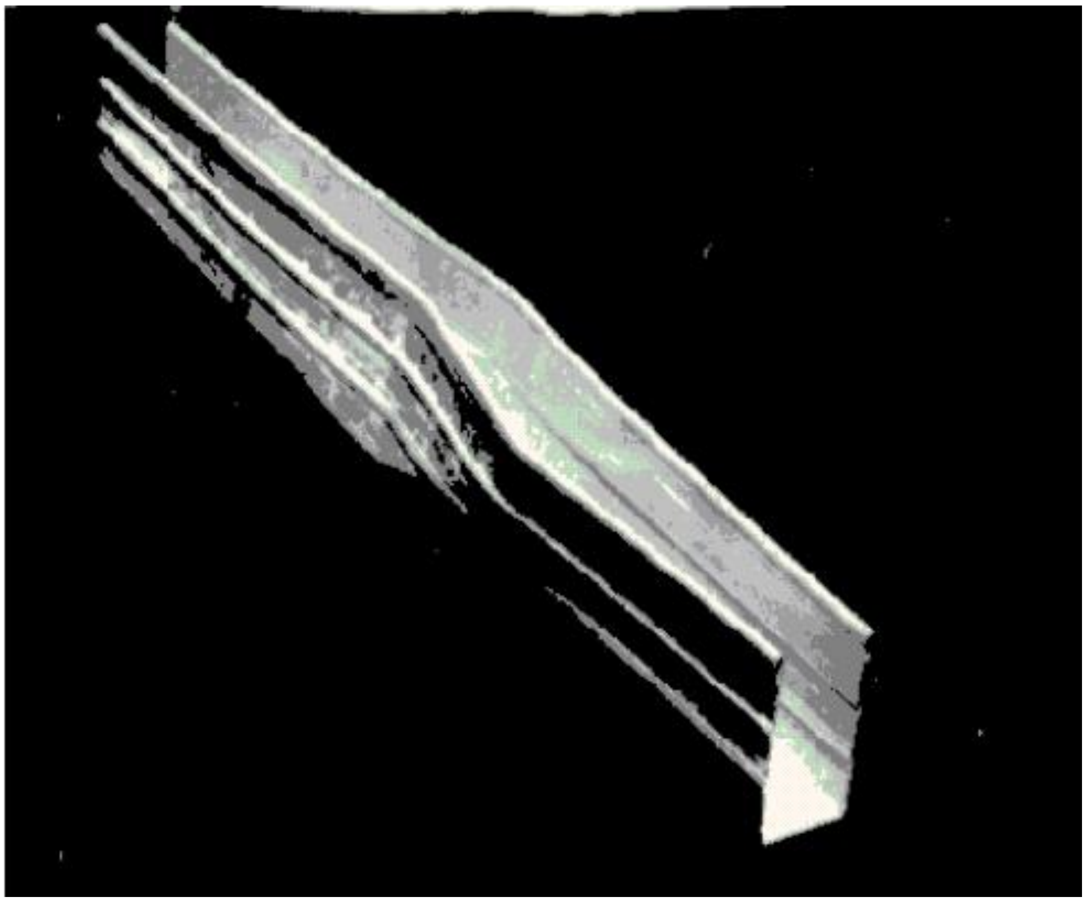
Example

Buckling

- Example (a) is temporary or elastic buckling.
- Example (b,c,d) are examples of plastic buckling.

Column Buckling

- Steel column buckling

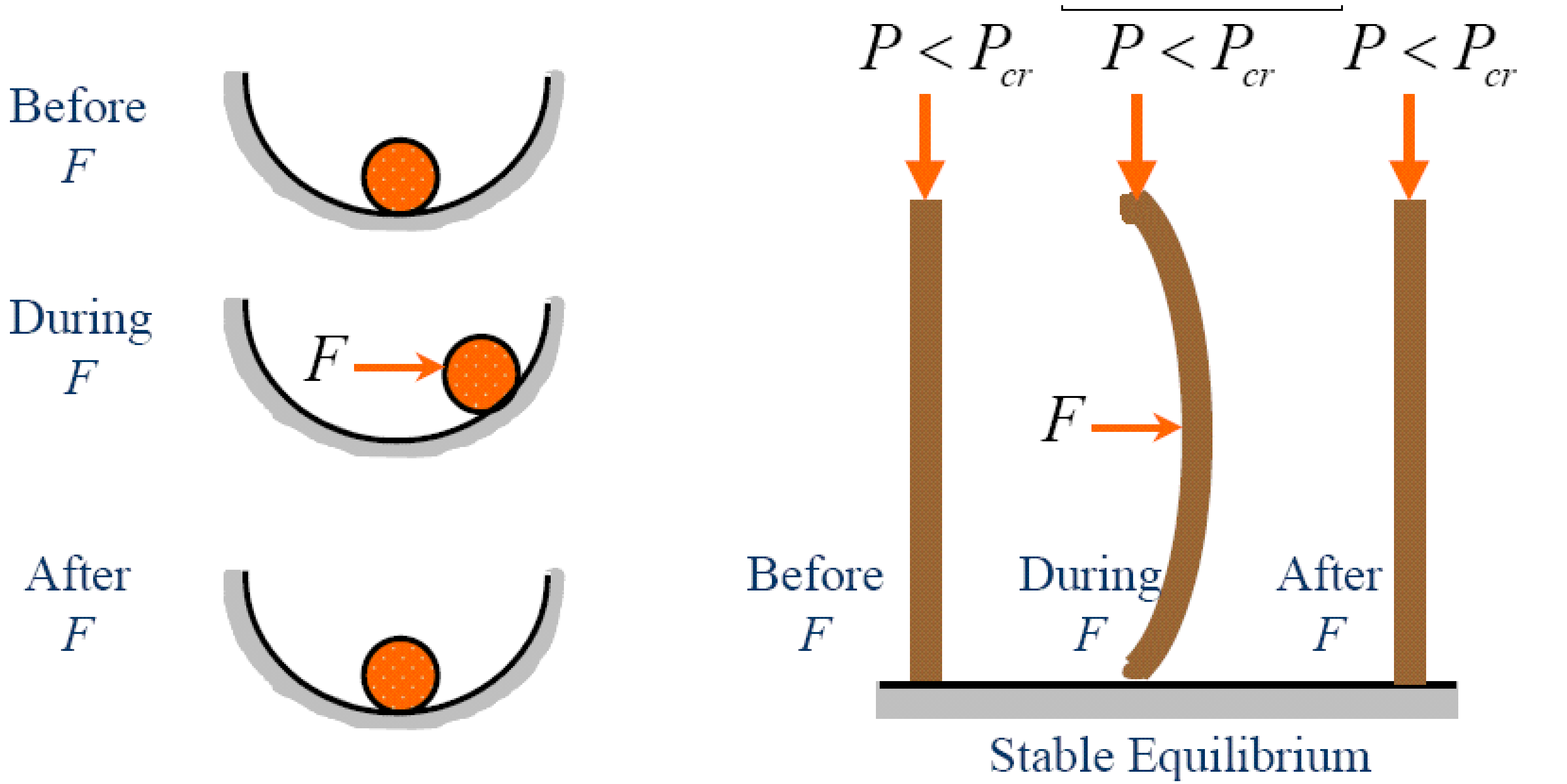


Mechanism of Buckling

- Let us consider Fig 1, 2, 3 and study them carefully.
- In fig1 some axial load P is applied to the column.
- The column is then given a small deflecion by giving a small force F.
- If the fprce P is suficiently small, when the force F is removed, the column will go back to its original straight position.

Mechanism of Buckling

Fig 1



Mechanism of Buckling

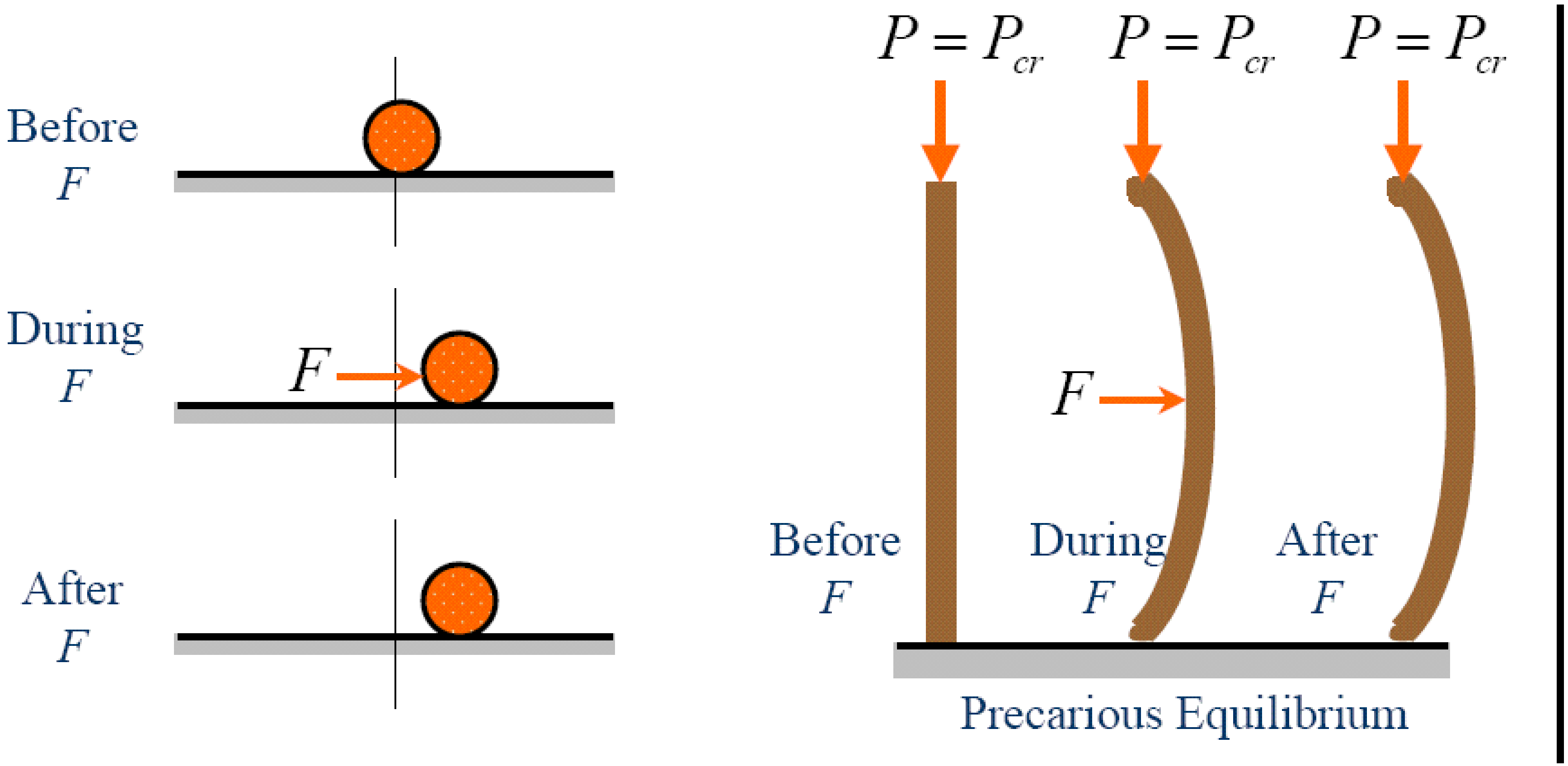
- The column will go back to its original straight position. Just as the ball returns to the bottom of the container.
- Gravity tends to restore the ball to its original position while in columns elasticity of column itself acts as a restoring force.
- This action constitutes stable equilibrium.

Mechanism of Buckling

- The same procedure can be repeated with increased load untill some critical value is reached.

Mechanism of Buckling

Fig 2

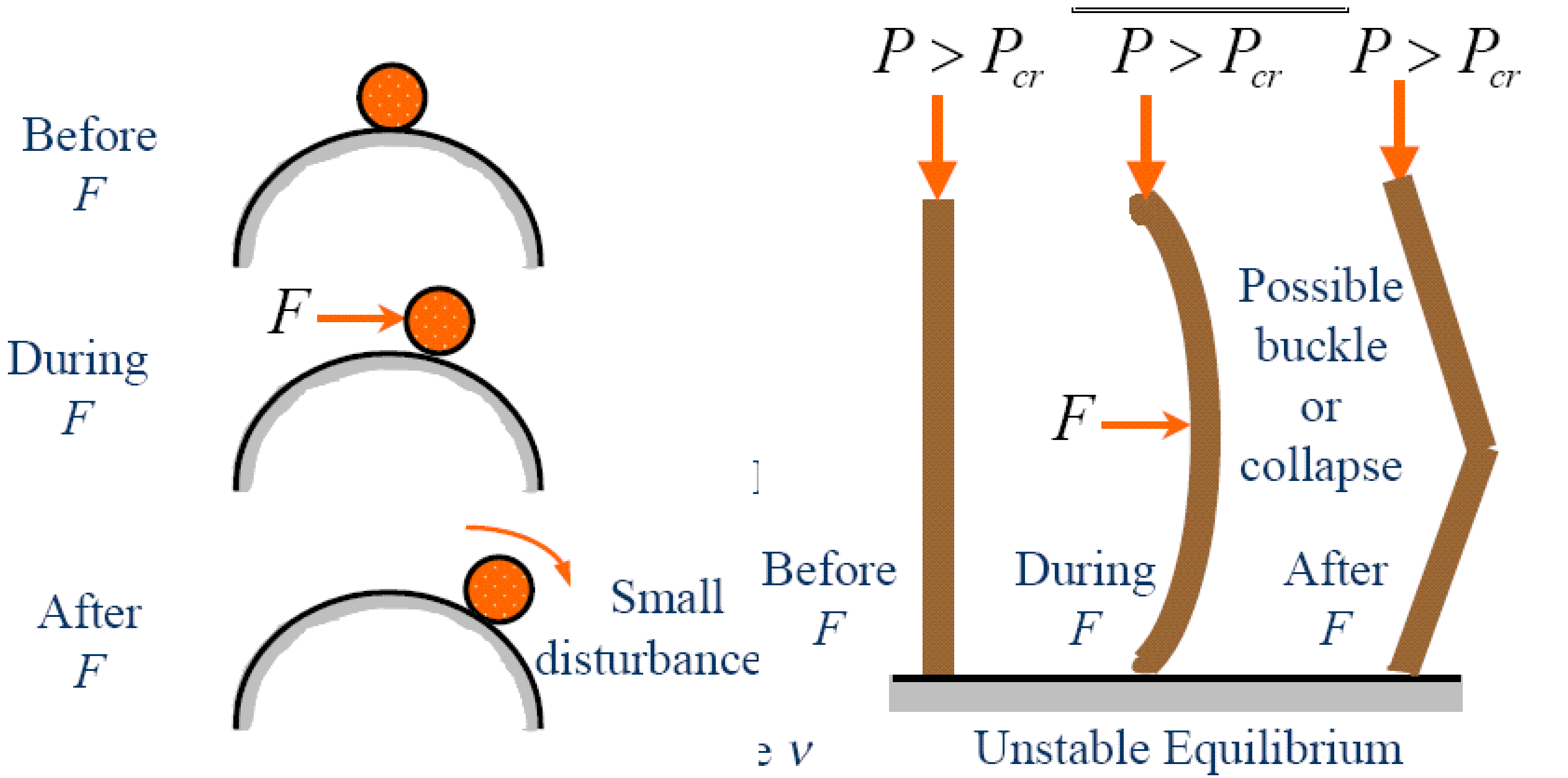


Mechanism of Buckling

- The amount of deflection depends on amount of force F.
- The column can be in equilibrium in an infinite number of bent position.

Mechanism of Buckling

Fig 3



Mechanism of Buckling

- The elastic restoring force was not enough to prevent small disturbance growing into an excessively large deflection.
- Depending on magnitude of load P, column either remain in bent position, or will completely collapse or fracture.

Mechanism of Buckling

Conclusions

- This type of behavior indicates that for axial loads greater than P_{cr} the straight position of column is one of unstable equilibrium in that a small disturbance will tend to grow into an excessive deformation.
- Buckling is unique from our other structural elements considerations in that it results from state of unstable equilibrium.

Mechanism of Buckling

Conclusions

- Buckling of long columns is not caused by failure of material of which column is composed but by determination of what was stable state of equilibrium to an unstable one.

Mechanism of Buckling

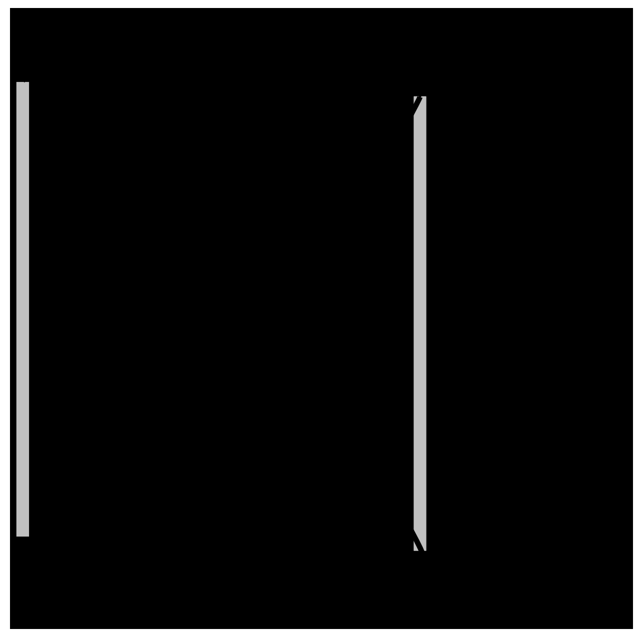
Conclusions

- Definition

"Buckling can be defined as the sudden large deformation of structure due to a slight increase of an existing load under which the structure had exhibited little, if any, deformation before the load was increased."

Compression member Buckling

- Buckling occurs when a straight, homogeneous, centrally loaded column subjected to axial compression suddenly undergoes bending.
- Buckling is identified as a failure limit-state for columns.



Compression member Buckling

- The value of P at which a straight column becomes unstable is called the Critical Load.
- When column bends at critical load, it is said to have buckled.
- Therefore, critical load is also called the buckling load.

Elastic Buckling of Columns

- The critical buckling load P_{cr} for columns is theoretically given by

$$P_E = \frac{\pi^2 E I}{L^2}$$

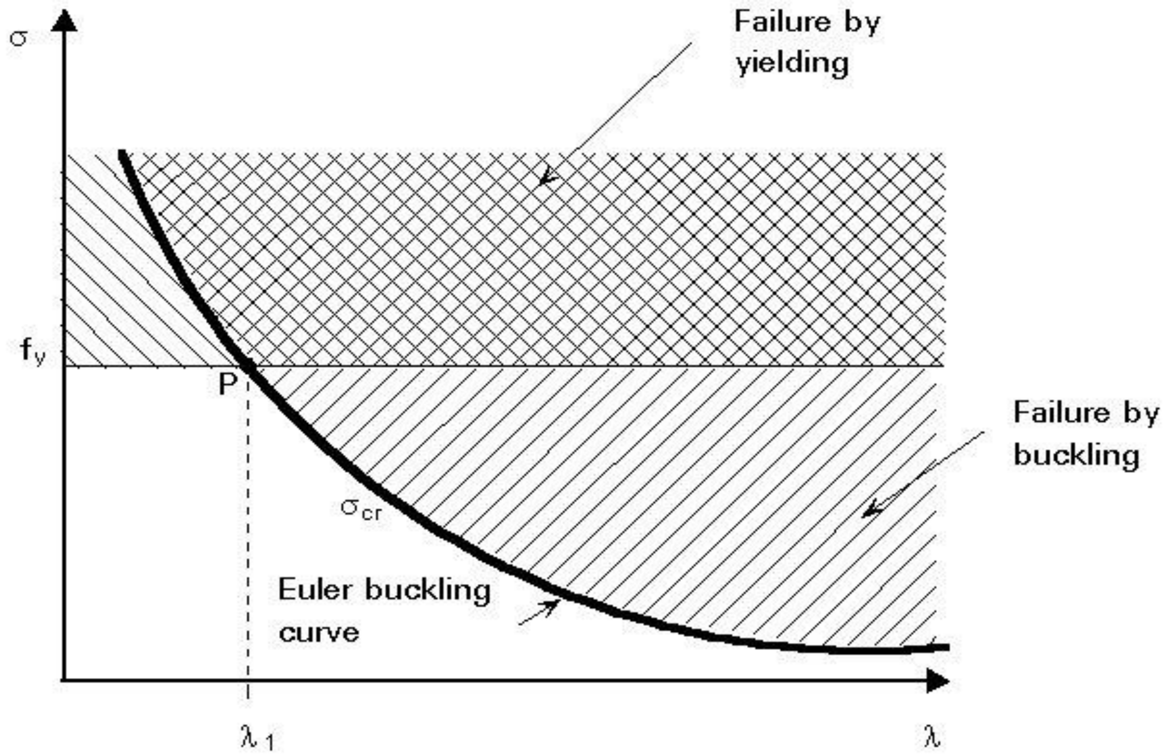
$$F_E = \frac{\pi^2 E I}{A L^2}$$

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

- Tendency of compression members to buckling is governed by L/r

Elastic Buckling of Columns

Figure 9 Local buckling curve and modes of failure



The intersection point P, of the two curves represents the maximum theoretical value of slenderness of a column compressed to the yield strength. This maximum slenderness (sometimes called Euler slenderness)

Elastic Buckling of Columns

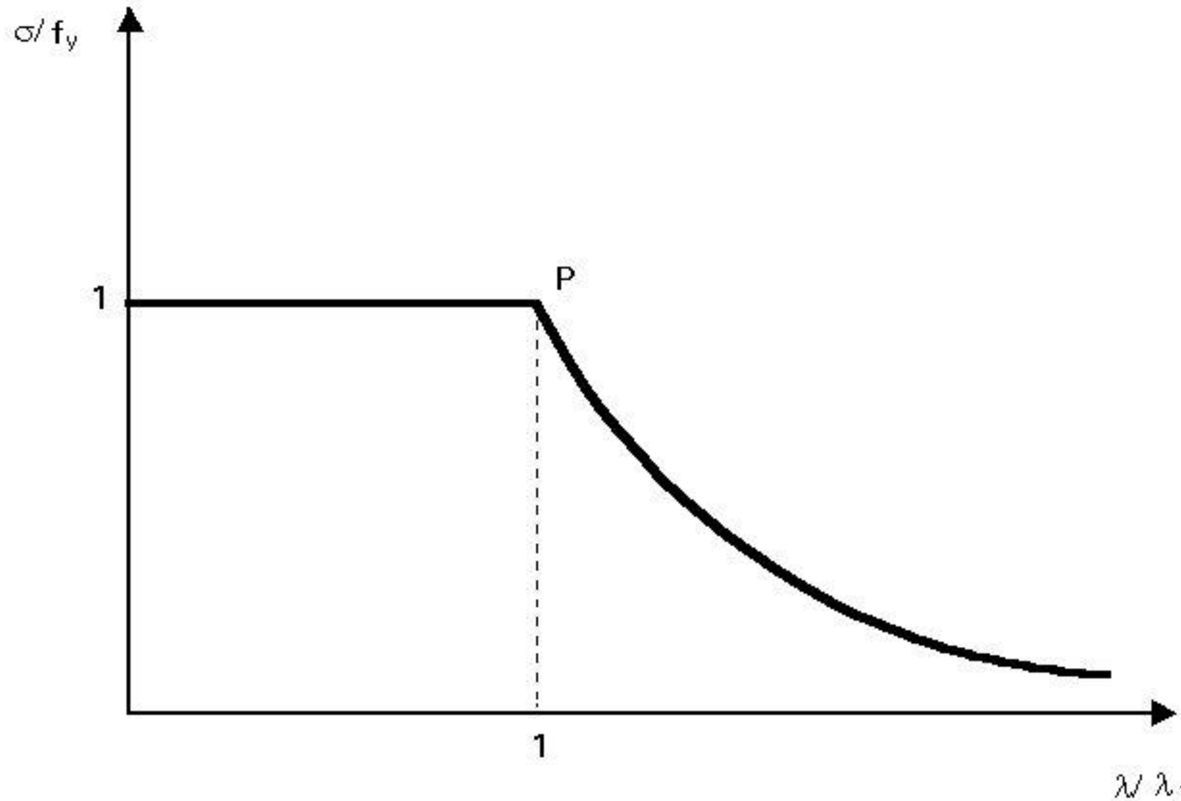


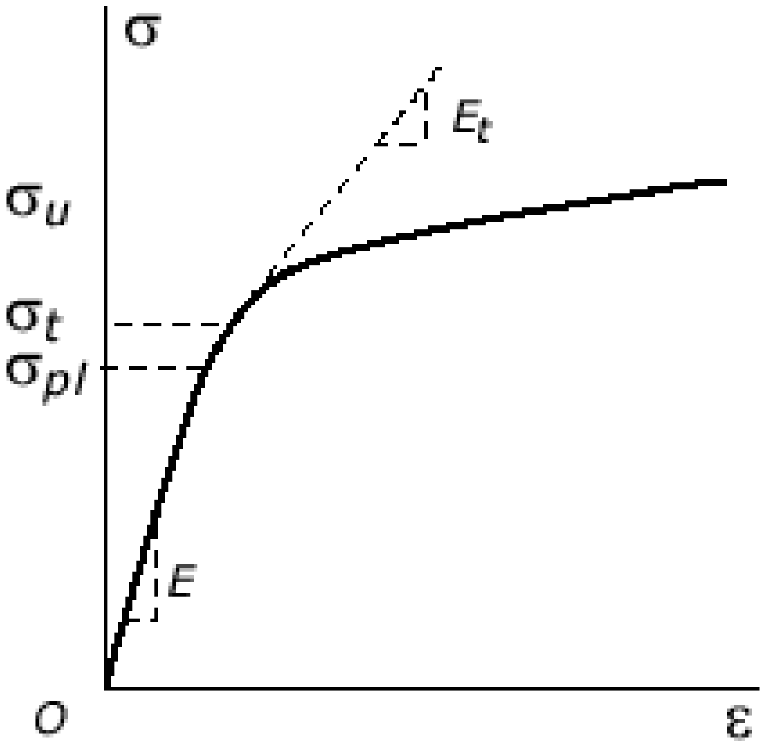
Figure 10 Non-dimensional buckling curve

Inelastic Buckling of Columns

- In elastic buckling, it was assumed that a column made of a metal whose stress-strain curve is linear until a yield plateau reached.
- For a column with intermediate length, when buckling occurs after the stress in the column exceeds the proportional limit of the column material and before the stress reaches the ultimate strength. This kind of situation is called *inelastic buckling.*

Inelastic Buckling of Columns

Tangent-Modulus Theory



$$\sigma_t = \frac{E_t \pi^2}{\left(L_{\text{eff}}/r\right)^2}$$

Inelastic Buckling of Columns

Tangent-Modulus Theory: Drawbacks

- Engesser's Conclusion was challenged with the basis that buckling begins with no increase in load.
- The tangent-modulus theory oversimplifies the inelastic buckling by using only one tangent modulus. In reality, the tangent modulus depends on the stress, which is a function of the bending moment that varies with the displacement w.

Inelastic Buckling of Columns

Tangent-Modulus Theory: Drawbacks

- The tangent-modulus theory tends to underestimate the strength of the column, since it uses the tangent modulus once the stress on the concave side exceeds the proportional limit while the convex side is still below the elastic limit.

Inelastic Buckling of Columns

Reduced Modulus Theory

- Engesser presented a second solution to the inelastic-buckling, in which the bending stiffness of the x-section is expressed in terms of double modulus E_r to compensate for the underestimation given by the tangent-modulus theory.

Inelastic Buckling of Columns

Reduced Modulus Theory

- For a column with rectangular cross section, the reduced modulus is defined by:

$$E_\gamma = \frac{4EE_t}{\left(\sqrt{E}+\sqrt{E_t}\right)^2}$$

The corresponding critical stress is,

$$\sigma_\gamma = \frac{E_\gamma \pi^2}{\left(L_{\text{eff}}/r\right)^2}$$

Inelastic Buckling of Columns

Reduced Modulus Theory: Drawbacks

- The reduced-modulus theory tends to overestimate the strength of the column, since it is based on stiffness reversal on the convex side of the column.

Inelastic Buckling of Columns

Reduced Modulus Theory: Drawbacks

- The reduced-modulus theory oversimplifies the inelastic buckling by using only one tangent modulus. In reality, the tangent modulus depends on the stress which is a function of the bending moment that varies with the displacement w.

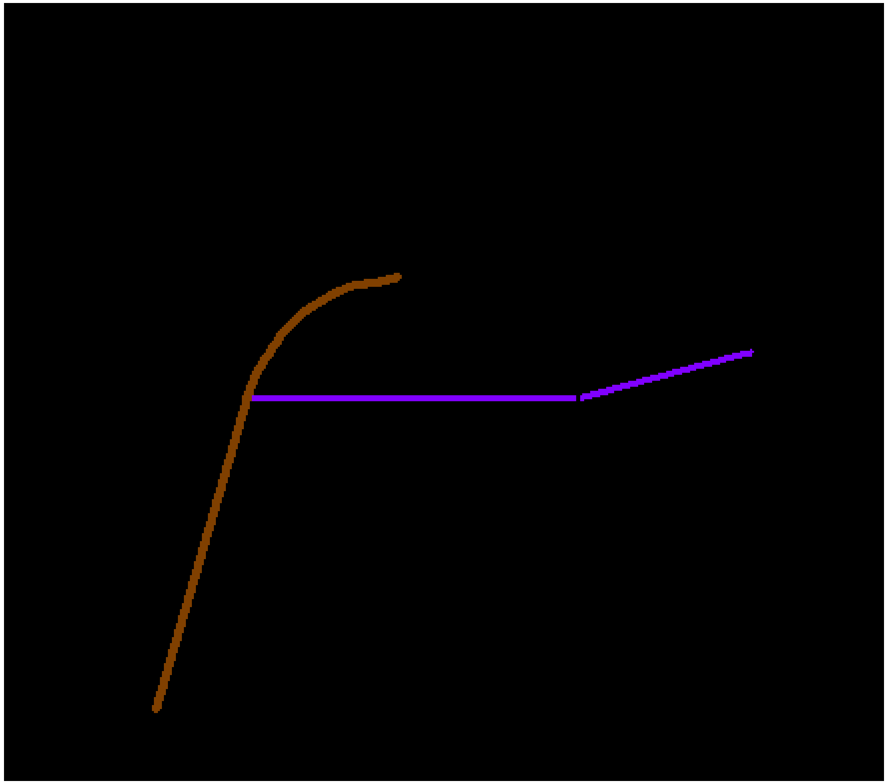
Inelastic Buckling of Columns

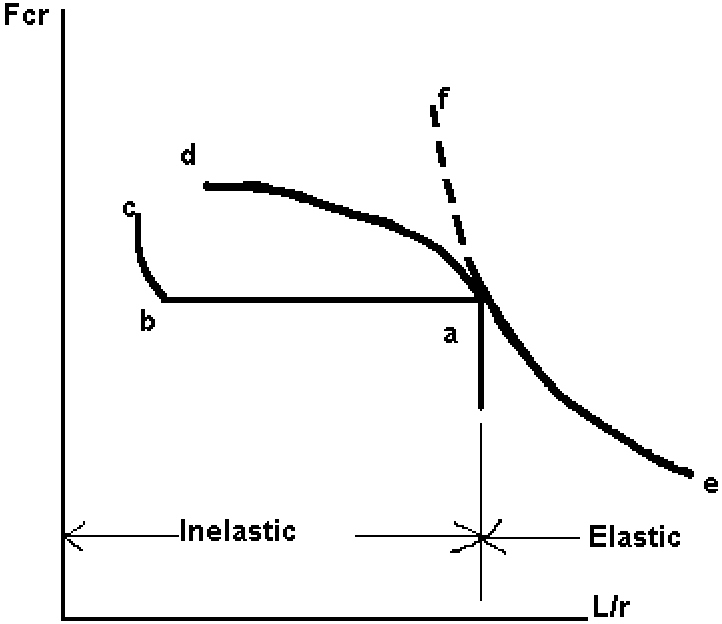
Shanley's Theory

- The critical load of inelastic buckling is in fact a function of the transverse displacement w
- Practically there are manufacturing defects in mass production and geometric inaccuracies in assembly.
- This is the reason why many design formulas are based on the overly-conservative tangent-modulus theory.

Inelastic Buckling of Columns

Shanley's Theory





Factors effecting Buckling

1. End Connections
2. Eccentricity of loads/Crookedness
3. Residual stresses

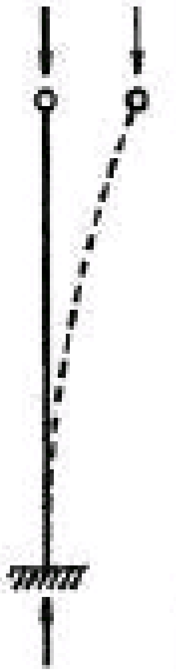
Factors effecting Buckling

1. End Connections

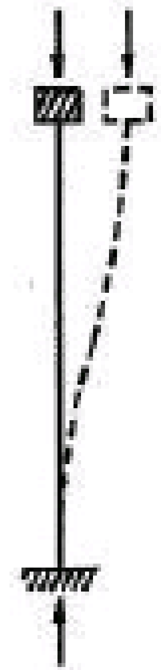
- Rotation of ends of columns in building frames is usually limited by beams connecting to them.

Factors effecting Buckling

1. End Connections: Effective length



$$P = \frac{\pi^2 E I}{(2L)^2}$$



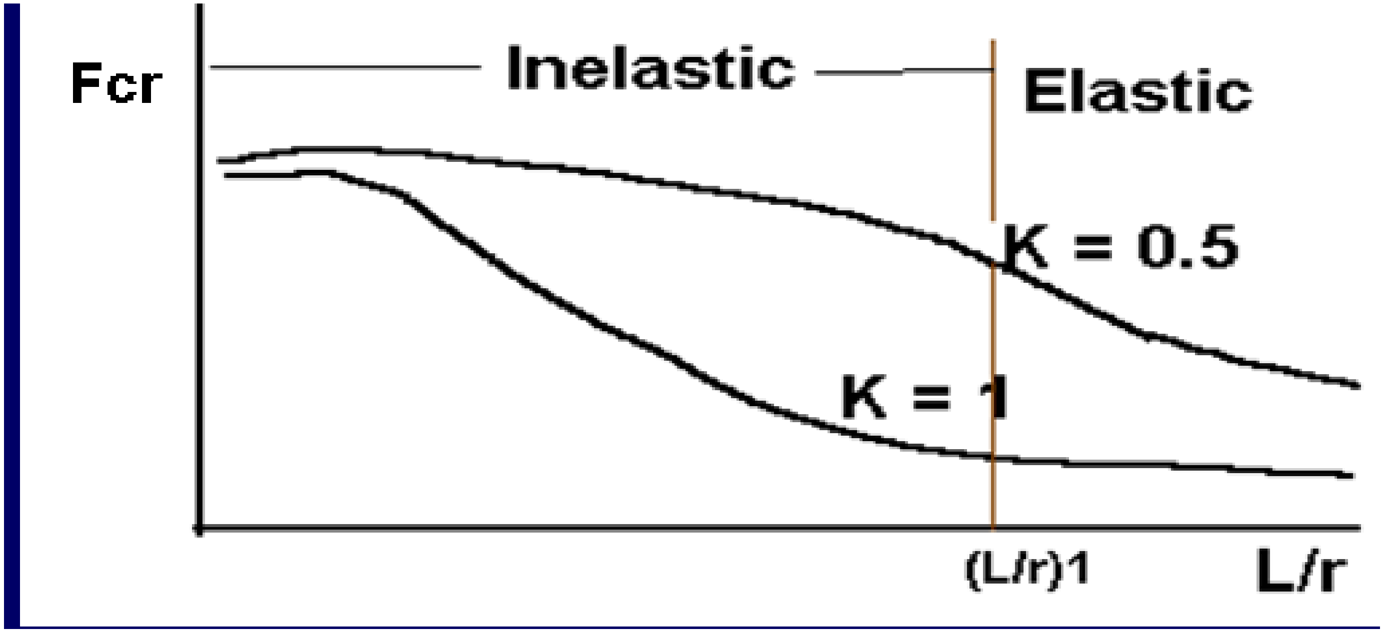
$$P_E = \frac{\pi^2 E I}{L^2}$$

$$P = \frac{\pi^2 E I}{(KL)^2}$$

- *KL* is called **effective length** of column and *K* **effective length factor.**

Factors effecting Buckling

1. End Connections: Effective length



- *A column with fixed ends can support four times as much load as a column with pinned ends*
- *This benefit decrease with decreasing L/r until Fcr finally becomes virtually independent of K*

Factors effecting Buckling

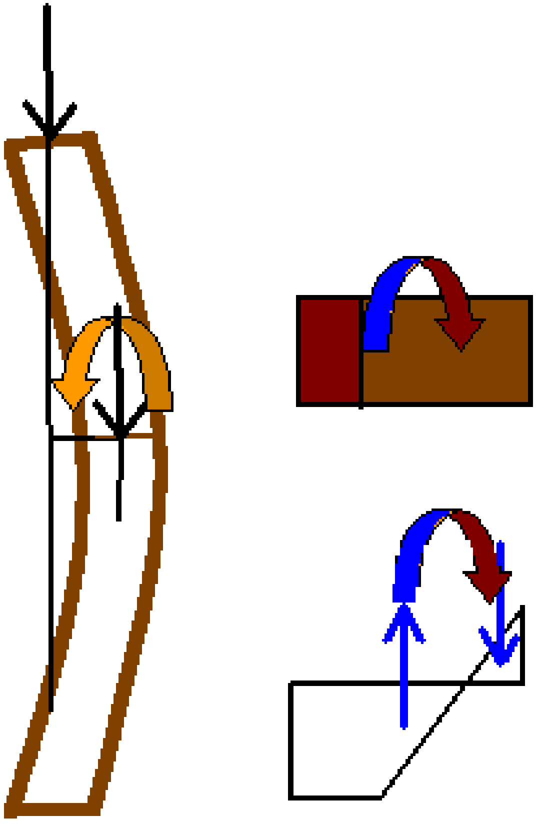
2. Effect of initial crookedness

- The initial out-of-straightness is also termed "initial crookedness" or "initial curvature".
- It causes a secondary bending moment as soon as any compression load is applied, which in turn leads to further bending deflection and a growth in the amplitude of the lever arm of the external end compression forces.

Factors effecting Buckling

2. Effect of initial crookedness

- A stable deflected shape is possible as long as the external moment, i.e. the product of the load and the lateral deflection, does not exceed the internal moment resistance of any section.



Factors effecting Buckling

2. Effect of initial crookedness

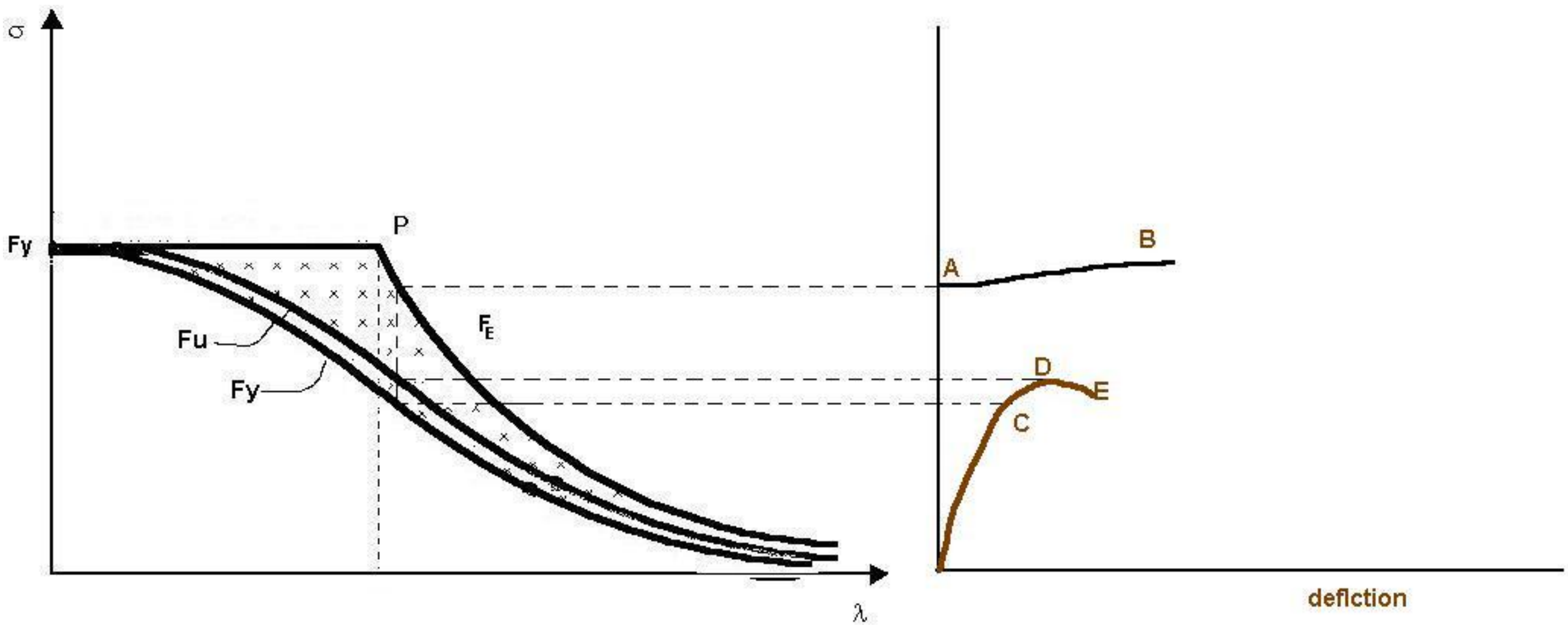


Figure 11 Real column test results
and buckling curves

Factors effecting Buckling

2. Effect of initial crookedness

- When straight column buckles, it assumes a stable, bent equilibrium, but with slightly larger load.
- In Crooked column deflection increases from beginning of loading and column is in unstable condition when it reaches to maximum load.

Factors effecting Buckling

3. Effect of Residual Stresses

- In tension members Residual stresses causes the section to yield at a stress lower than the yield point of the material.
- As a result, the elongation for a given load is greater than would be calculated form elastic properties.

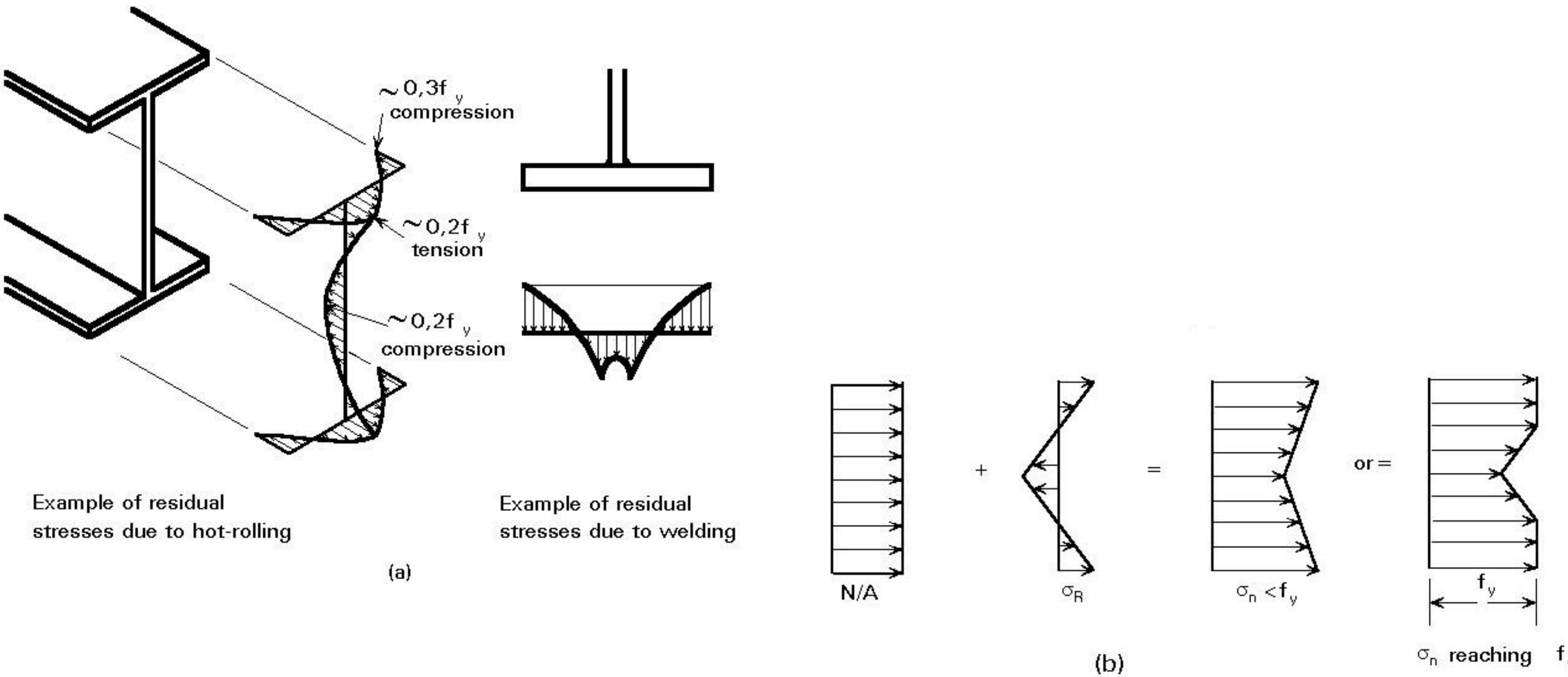
Factors effecting Buckling

3. Effect of Residual Stresses

- Complete yielding of x-section did not occur until applied strain equals the yield strain of base material.
- The residual stresses does not affect the load corresponding to full yield of x-section.

Factors effecting Buckling

3. Effect of Residual Stresses



Combination with axial stresses

Figure 12 Example and effect of residual stresses

Factors effecting Buckling

3. Effect of Residual Stresses

- If the maximum stress σ_n reaches the yield stress fy, yielding begins to occur in the cross-section. The effective area able to resist the axial load is, therefore, reduced.

$$P = \frac{\pi^2 EI_{eff}}{L^2}$$

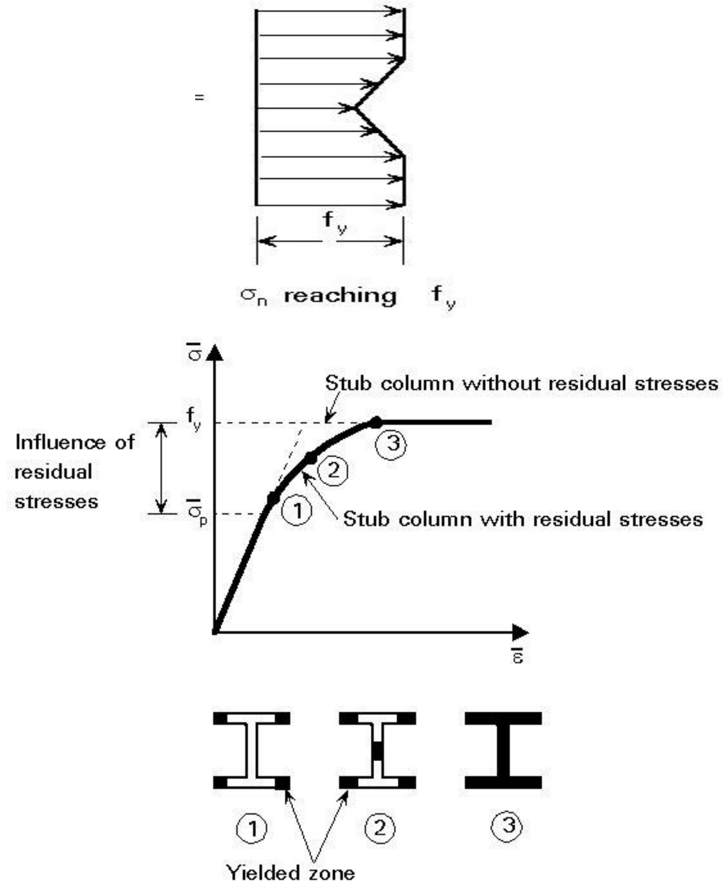


Figure 14 Average applied axial stress plotted plotted average axial strain from stub column tests

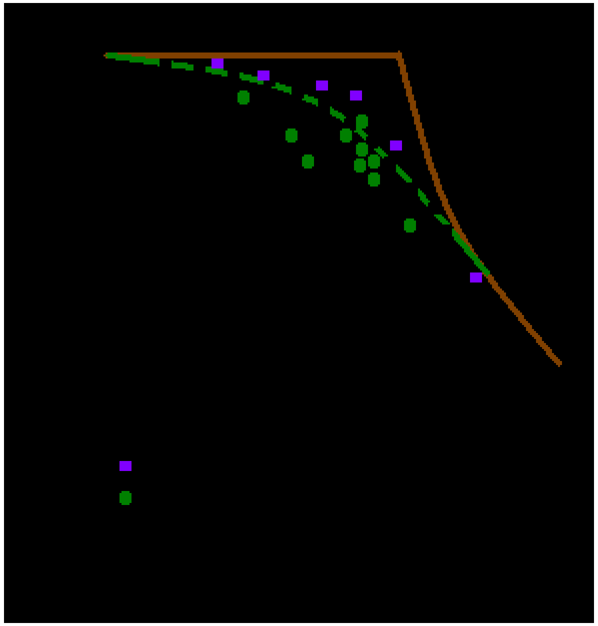
Factors effecting Buckling

3. Effect of Residual Stresses

<u>Tests carried on W shapes</u>

- Effect of residual stresses in causing weak axis buckling at loads smaller than those for strong axis buckling.
- This suggest two column formulas for the steel **W**.

Structural Stability Research Council (SSRC) proposed a single formula to simplify the deign procedure



$$\sqrt{\frac{F_y}{F_e}} = \frac{1}{\pi}\sqrt{\frac{F_y}{E}}\frac{L}{r}$$

Factors effecting Buckling

3. Effect of Residual Stresses: SSRC Formula

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL}{r}\right)^2\right]$$

$$F_{cr} = F_y\left[1 - \frac{1}{2}\left(\frac{KL/r}{C_c}\right)^2\right]$$

$$C_c = \pi\sqrt{2\,E/F_y}$$

Design procedure be simplified by using Parabola beginning with a vertex at $F_{cr}=F_y$ where L/r and terminating at $F_{cr}=F_y/2$ where it intersects and tangent to Euler Hyperbola.

Factors effecting Buckling

Combined Effect of Crookedness & Residual Stresses

- An initial out-of-straightness e_o, produces a bending moment giving a maximum bending stress s_B
- If s_{max} is greater than the yield stress the final distribution will be part plastic and part of the member will have yielded in compression.

Factors effecting Buckling

Combined Effect of Crookedness & Residual Stresses

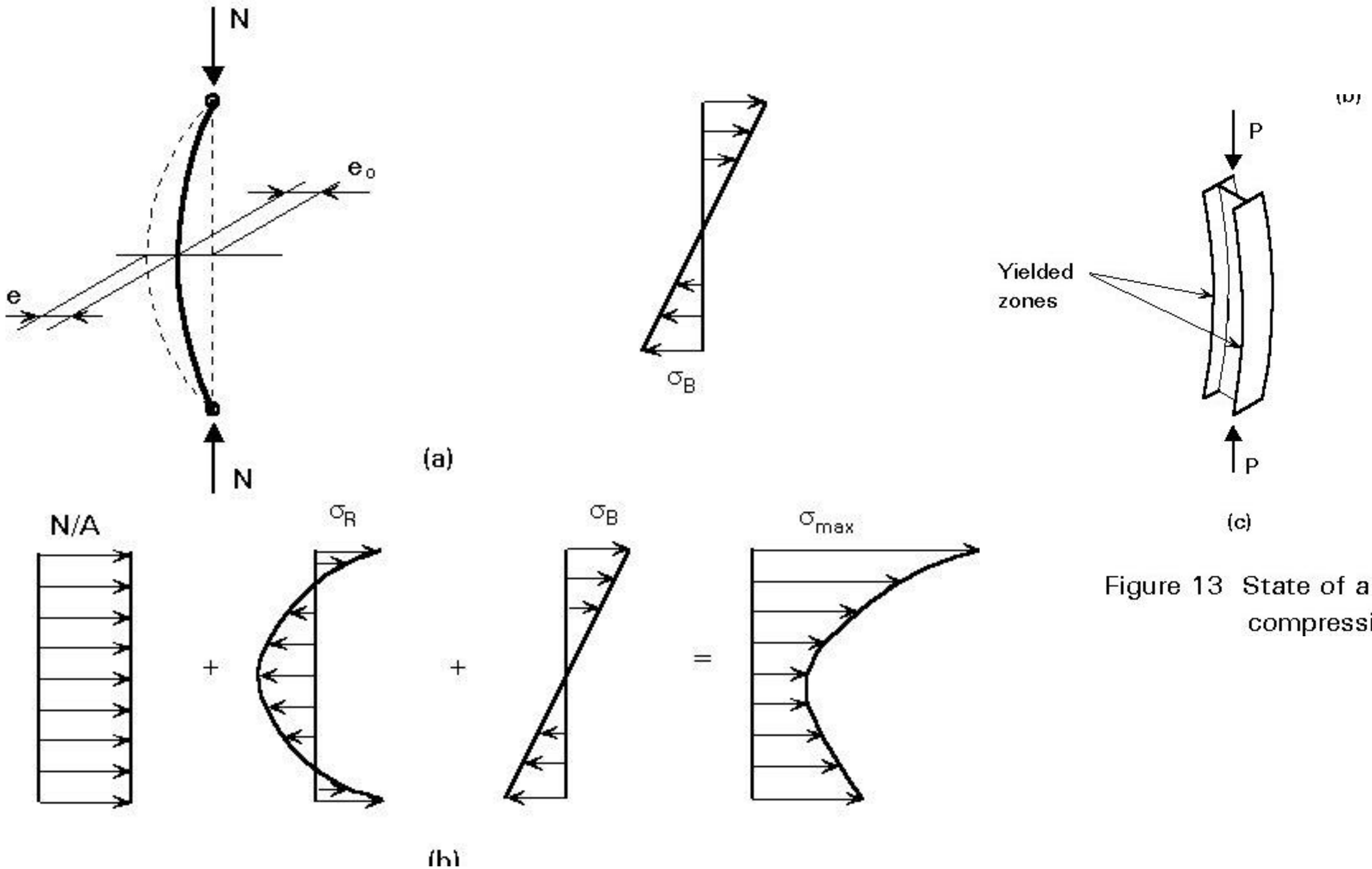


Figure 13 State of a partially yielded compression member

Code Requirements

ASD Formula

$$
Fa = \left\{
\begin{array}{ll}
\dfrac{Fy\left[1-\dfrac{1}{2}\left(\dfrac{KL/r}{Cc}\right)^2\right]}{\dfrac{5}{3}+\dfrac{3}{8}\dfrac{KL/r}{Cc}-\dfrac{1}{8}\left(\dfrac{KL/r}{Cc}\right)^3} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots & \dfrac{KL}{r} \leq Cc \\
\dfrac{12\pi^2 E}{23\left(KL/r\right)^2} = \dfrac{149000}{\left(KL/r\right)^2} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots & \dfrac{KL}{r} \geq Cc
\end{array}
\right\}
$$

Code Requirements

LRFD Specifications

- The design strength of columns for the flexural buckling limit state is equal to $\boldsymbol{\phi_c P_n}$
- Where, ϕ_c = 0.85 (Resistance factor for compression members)

$$P_n = A_g\, F_{cr}$$

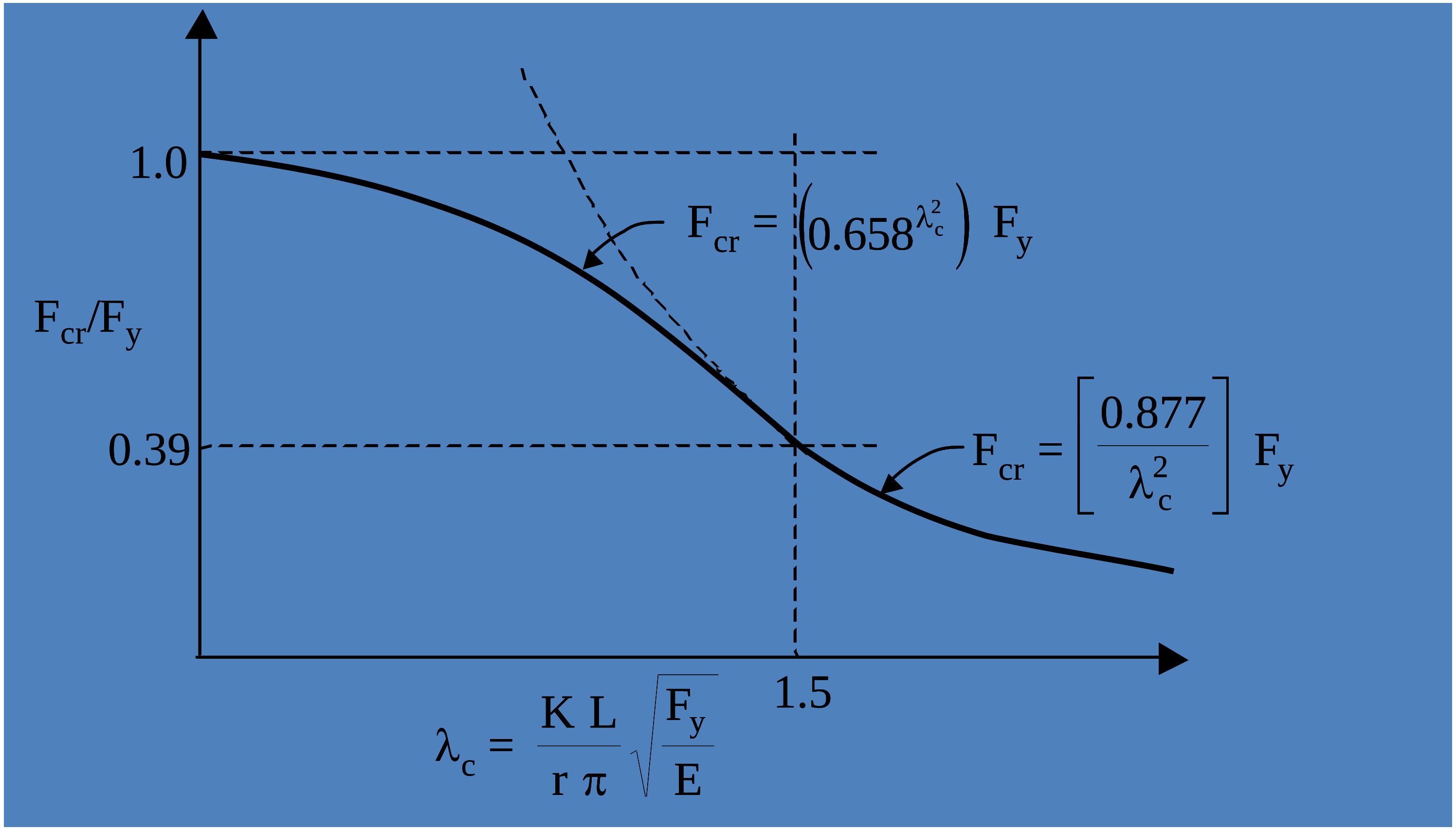
For $\lambda_c \leq 1.5$ $\quad F_{cr} =$ [redacted]

For $\lambda_c > 1.5$ $\quad F_{cr} =$ [redacted]

Where, $\lambda_c =$ [redacted]

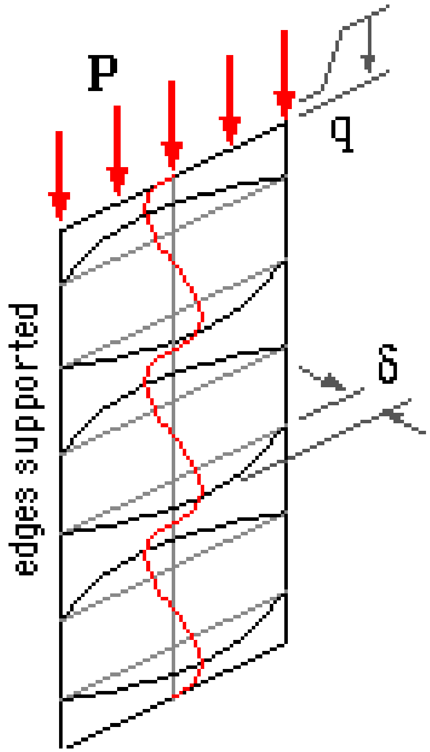
Code Requirements

LRFD Specifications



Local Buckling

- If the column section is made of thin (slender) plate elements, then failure can occur due to *local buckling* of the flanges or the webs in compression well before the calculated buckling strength of the whole member is reached.
- When thin plates are used to carry compressive stresses they are particularly susceptible to buckling about their weak axis due small moment of Inertia.



Local Buckling

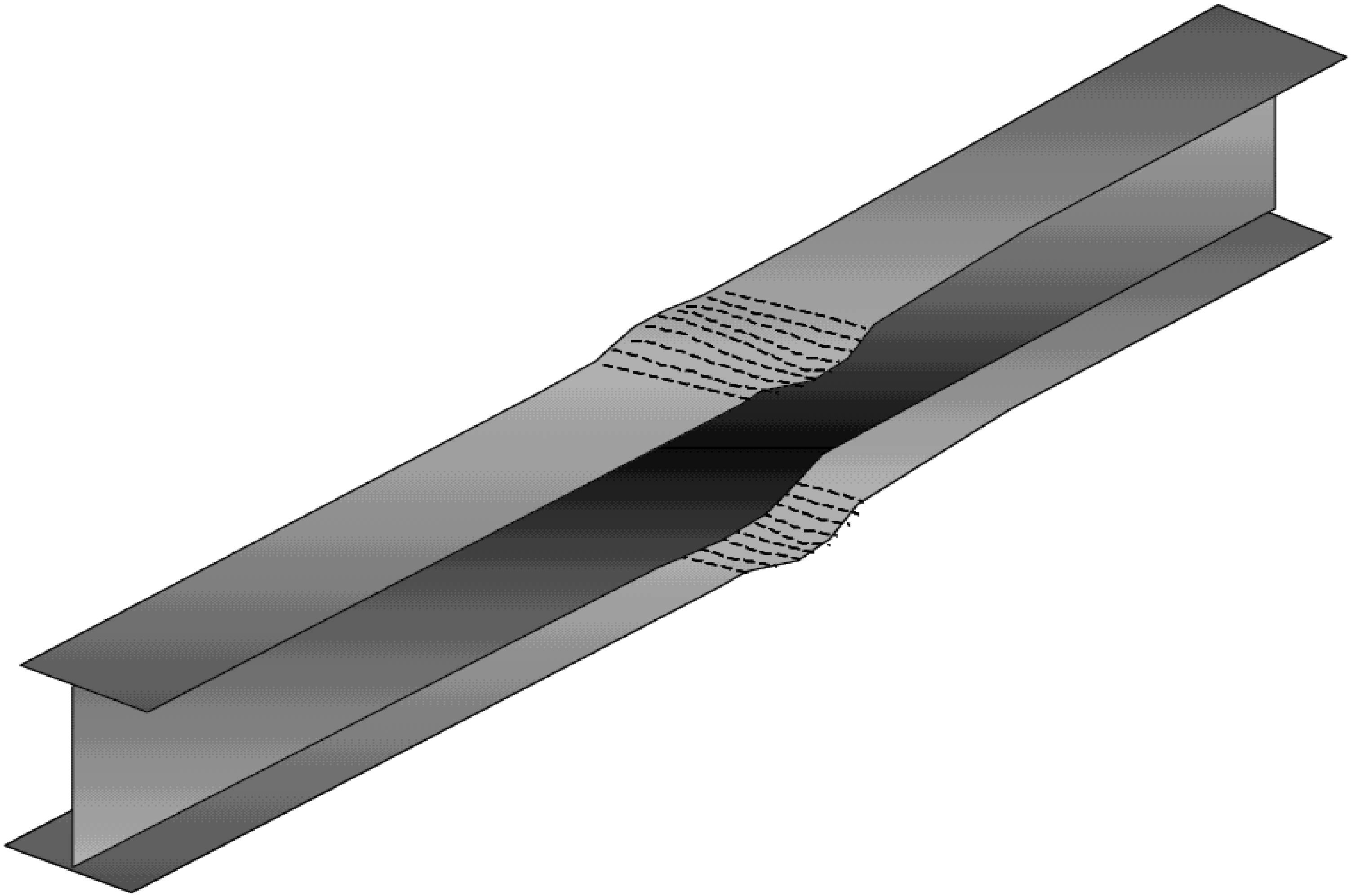
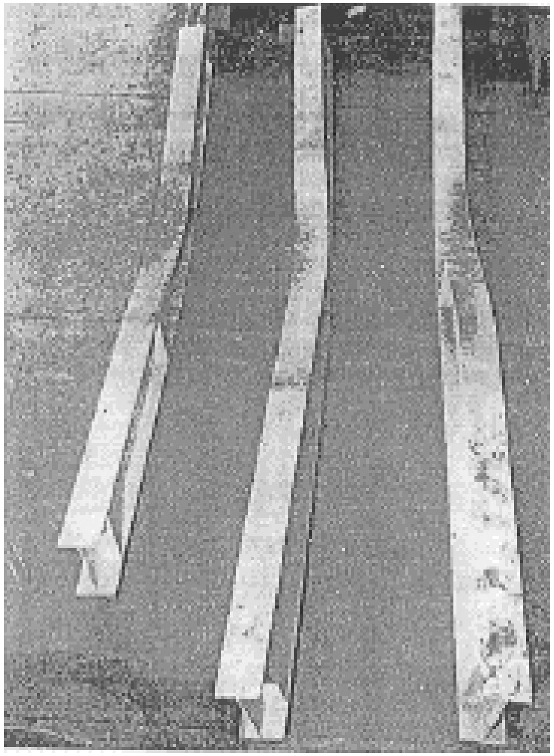
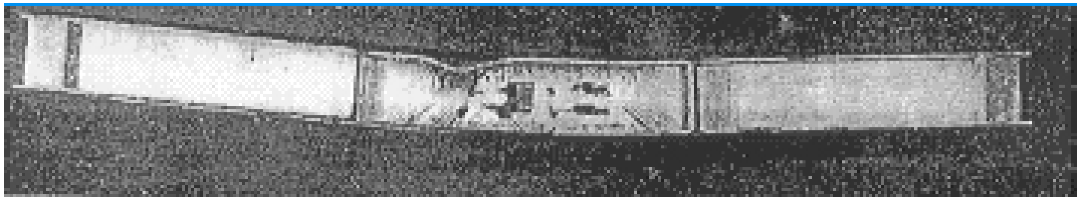


Figure 4. Local buckling of columns

Local Buckling



Laterally buckled beams



Flange Buckling

Local Buckling

Local Buckling

- If *local buckling* of the individual plate elements occurs, then the column may not be able to develop its buckling strength.
- Therefore, the local buckling limit state must be prevented from controlling the column strength.

Local Buckling

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

Local Buckling

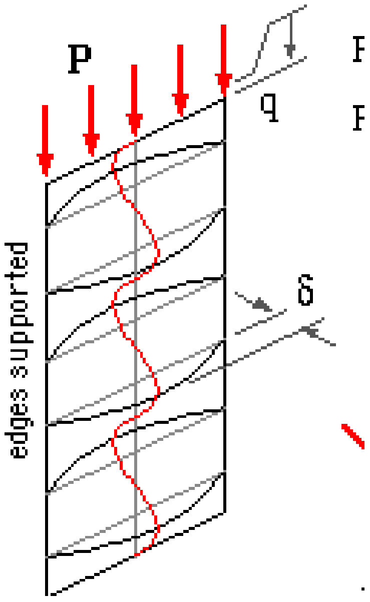
- The critical stress for rectangular plates with various types of edge supports, and with loads in the plane of the plate distributed along the edges in various ways is given by

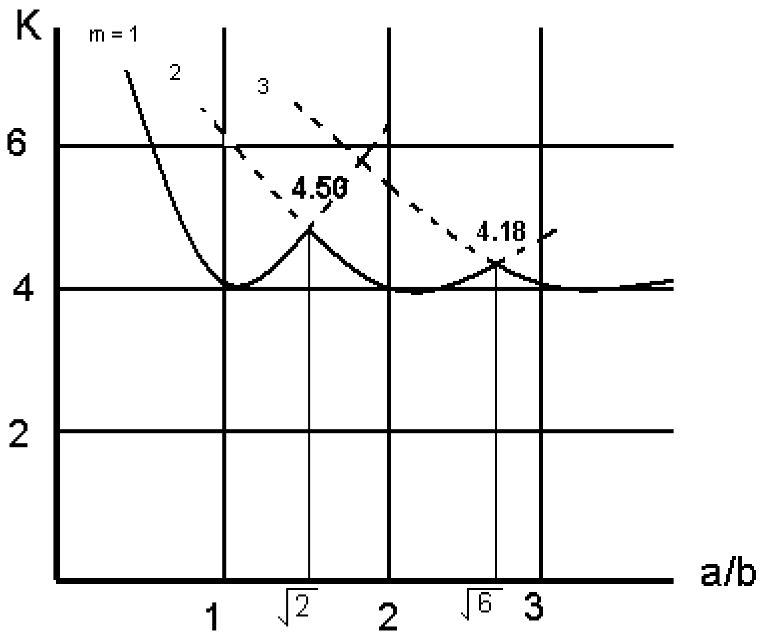
$$F_{cr} = \frac{k\,\pi^2 E}{12(1-\mu^2)(b/t)^2}$$

- **K= Constant depends on**
  - How edges are supported
  - Ratio of plate length to plate width
  - Nature of loading

Local Buckling

- **The coefficient *k* has a minimum value of 4 for a/b=1,2,3 etc.**
- **The error in using *k* =4 decreases with increasing a/b and for a/b= 10 or more, it is extremely small.**





Local Buckling

- Critical stresses for plate buckling can be evaluated by determination of equivalent slenderness ratio for which a column will buckle at same stress, using $$E = E\sqrt{\tau}$$

$$\left(\frac{L}{r}\right)_{eq} = \frac{3.3\sqrt[4]{\tau}\ b}{\sqrt{k}\ t}$$

Local Buckling

- The AISC specification provides the slenderness (b/t) limits that the individual plate elements must satisfy so that *local buckling* does not control.
- Consult table 4-4 of Gaylord

UNIT-III

DESIGN OF BEAM

Beam classification

- **Main or Primary beams / girders**
- **Secondary beams/joists**

- **Girders**
- **Joist**
- **Lintels**
- **Purlins**
- **Rafter**
- **Spandrels**
- **Stringers**

- **Laterally Stable**
- **Laterally Unstable**

PERMISSIBLE STRESS DESIGN

Stresses in Structures at working loads are not allowed to exceed a certain proportion of the yield stress of the material.

- Stress levels are limited to elastic range
- Leads to highly conservative solutions.

LIMIT STATE DESIGN OF BEAMS

- In this method, the structure has to be designed to withstand safely all loads and deformations likely to occur on it throughout its life.

- Designs should ensure that the structure does not become unfit for the use for which it is required.

- The state at which the unfitness occurs is called a limit state.

Limit States

- Ultimate Limit States

  - (**flexure, shear, bearing, compression, torsion, lateral-torsion**)

- Serviceability Limit States

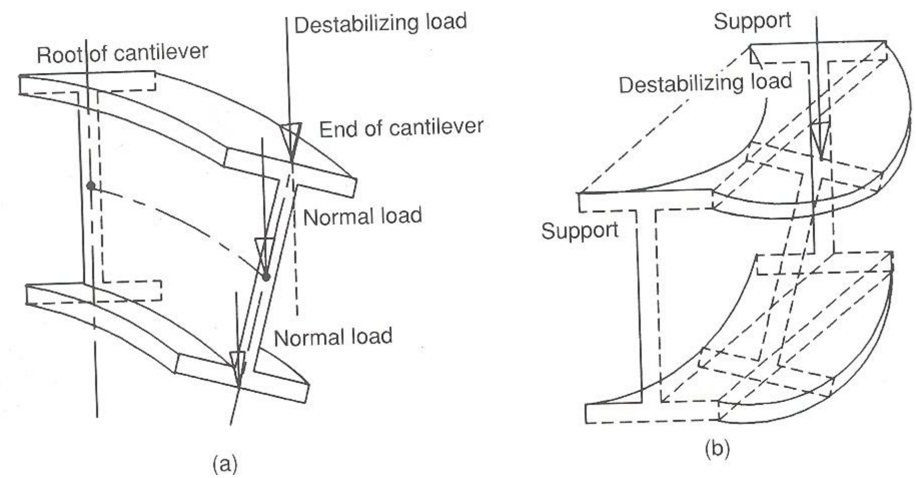
  -(deflection, vibration, fire, durability)

Types of Loads

- Dead loads
- Imposed loads (Live Load, Crane Load, Snow Load, Dust Load, Wave Load, Earth pressures)
- Wind Loads
- Earthquake Loads
- Erection Loads
- Accidental Loads (Blast, Impact of vehicles)
- Secondary Effects (temperature effects, differential settlements, eccentric connections, varied rigidity)

Stability of Beams

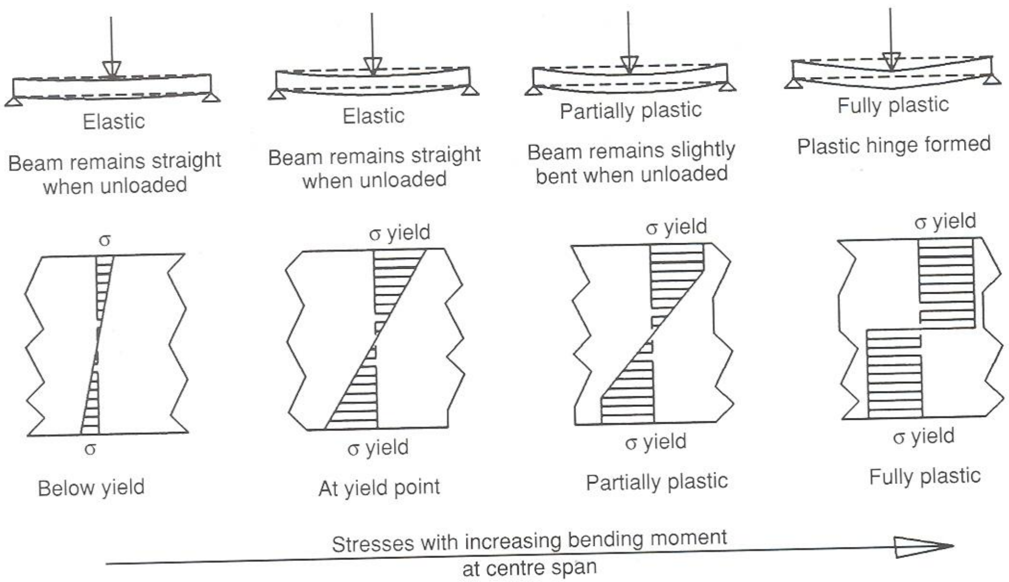
- Laterally Unrestrained Beams
- Laterally Restrained Beams



Lateral-torsional Buckling in Beams

Failure Modes in Beams

- Bending

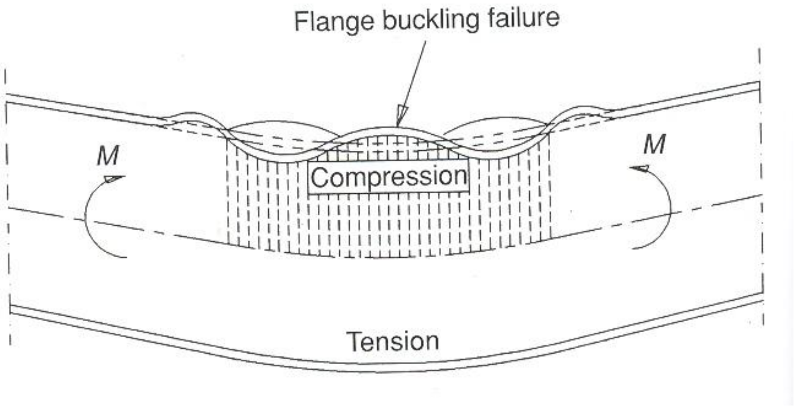


➢ **When all the beam cross-section has become plastic the beam fails by formation of a plastic hinge at the point of maximum imposed moment.**

➢ **The bending moment cannot be increased and the beam collapses as though a hinge has been inserted into the beam.**

Failure Modes in Beams…

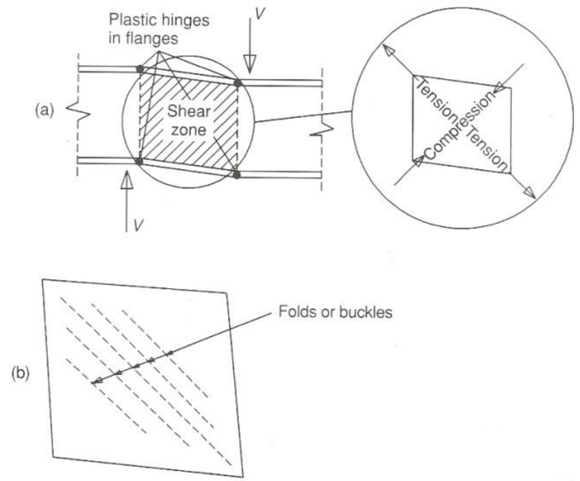
- Local buckling



Local Flange buckling failure

Failure Modes in Beams…

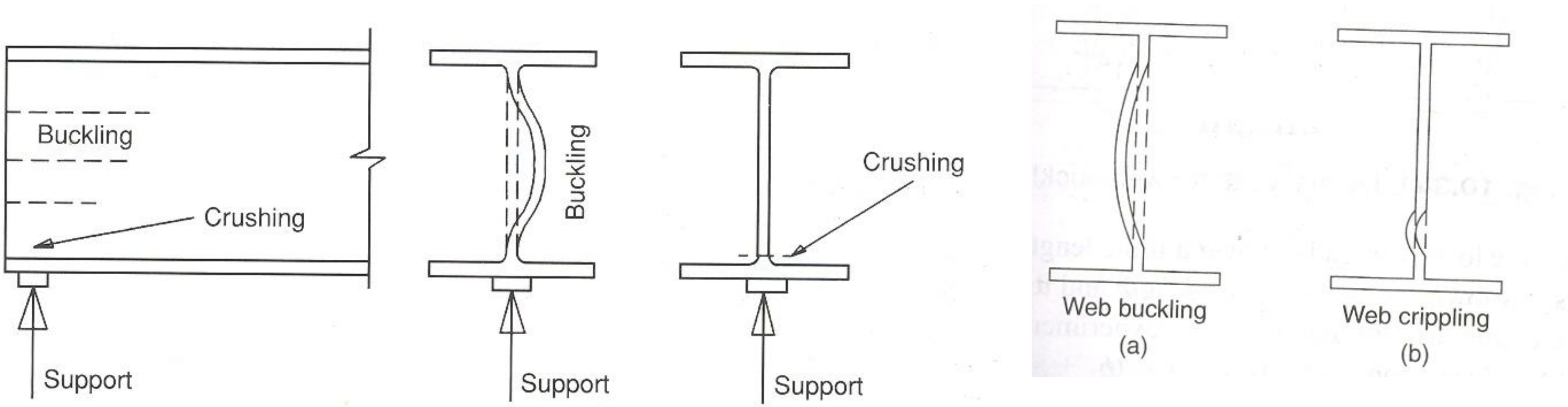
- **Shear**



During the shearing process, if the web is too thin it will fail by buckling or rippling in the shear zone as shown in fig.

Failure Modes in Beams…

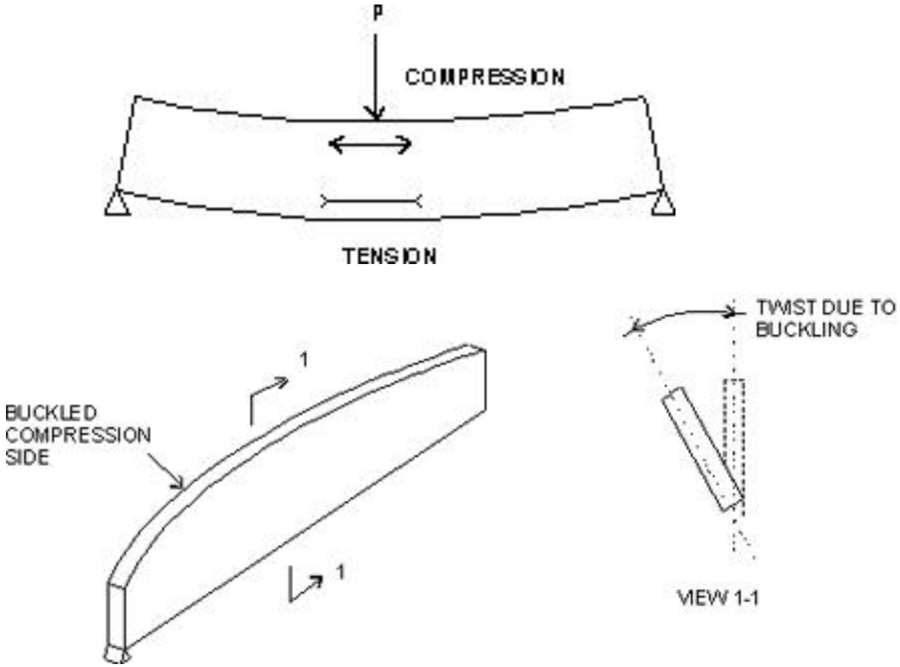
- **Web bearing and buckling**

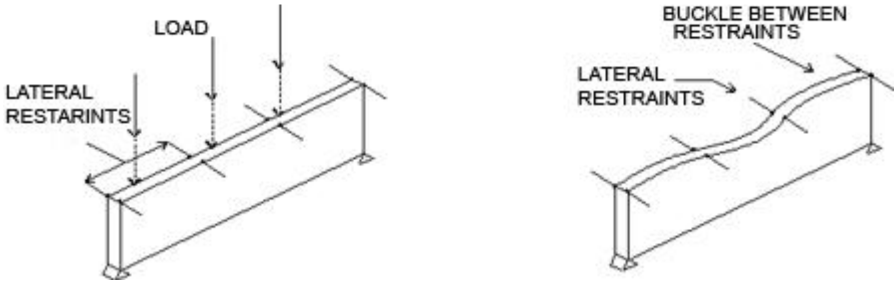


Due to high vertical stresses directly over a support or under a concentrated load, the beam web may actually crush or buckle as a result of these stresses.

Failure Modes in Beams…

- **Lateral-torsional buckling**

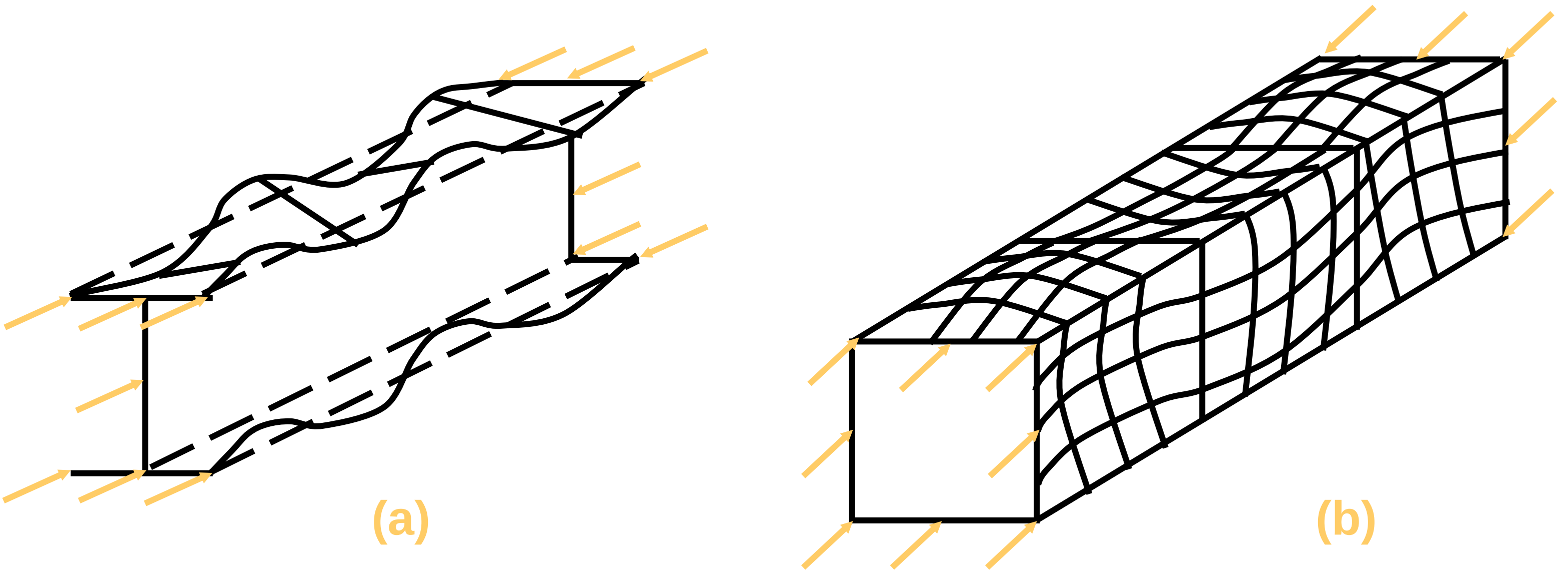




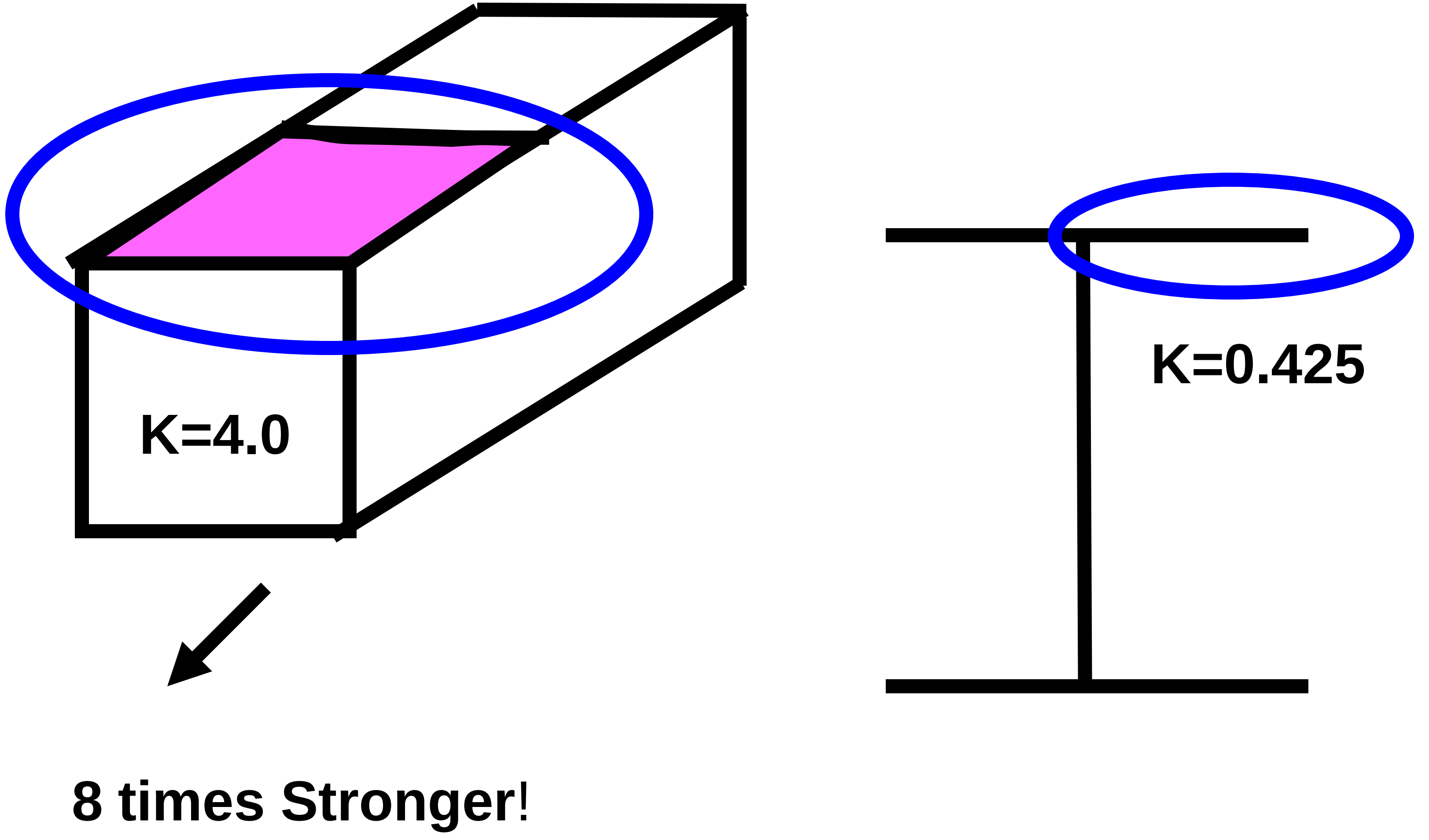
Behaviour of beam with restraints

Lateral torsional buckling of a simply supported beam

LOCAL BUCKLING



Local buckling of Compression Members

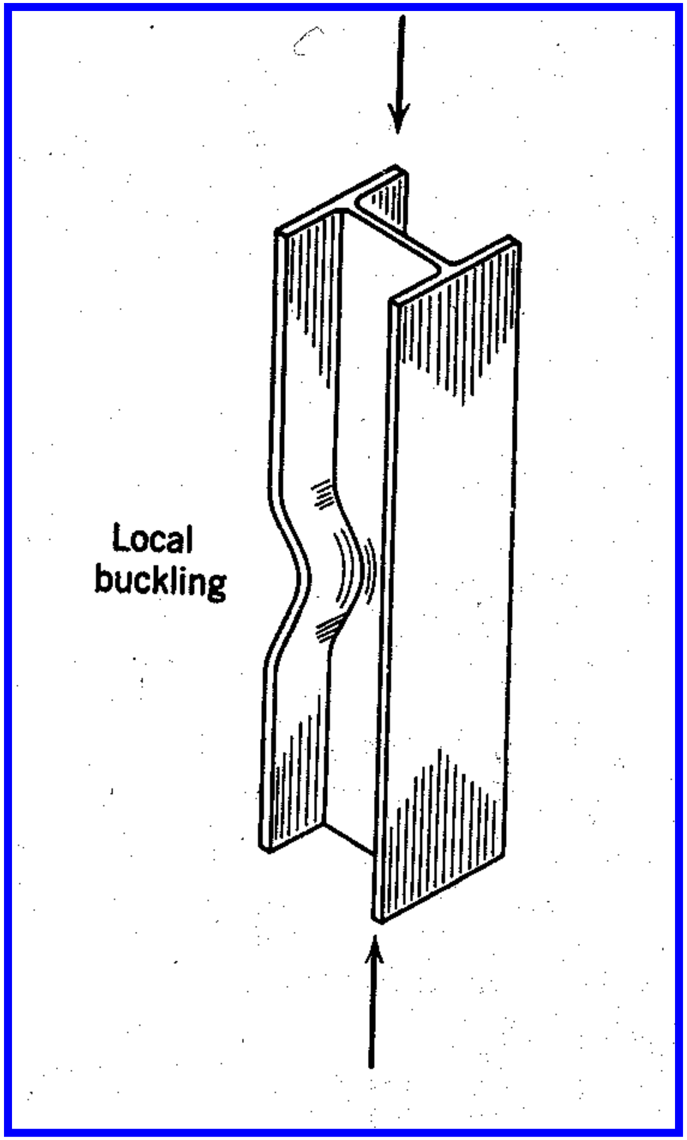

K=4.0
K=0.425
8 times Stronger!

DESIGN OF PLATE ELEMENTS

Limiting width-thickness ratio to ensure yielding before plate buckling

$$\left(\frac{b_{\lim}}{t}\right) \leq \left(\frac{k\pi^2 E}{12(1-\nu^2) f_y}\right)^{1/2}$$

$$\left(\frac{b_{\lim}}{t}\right) \leq \left(\frac{0.425\,\pi^2 E}{12(1-\nu^2) f_y}\right)^{1/2} = 16$$

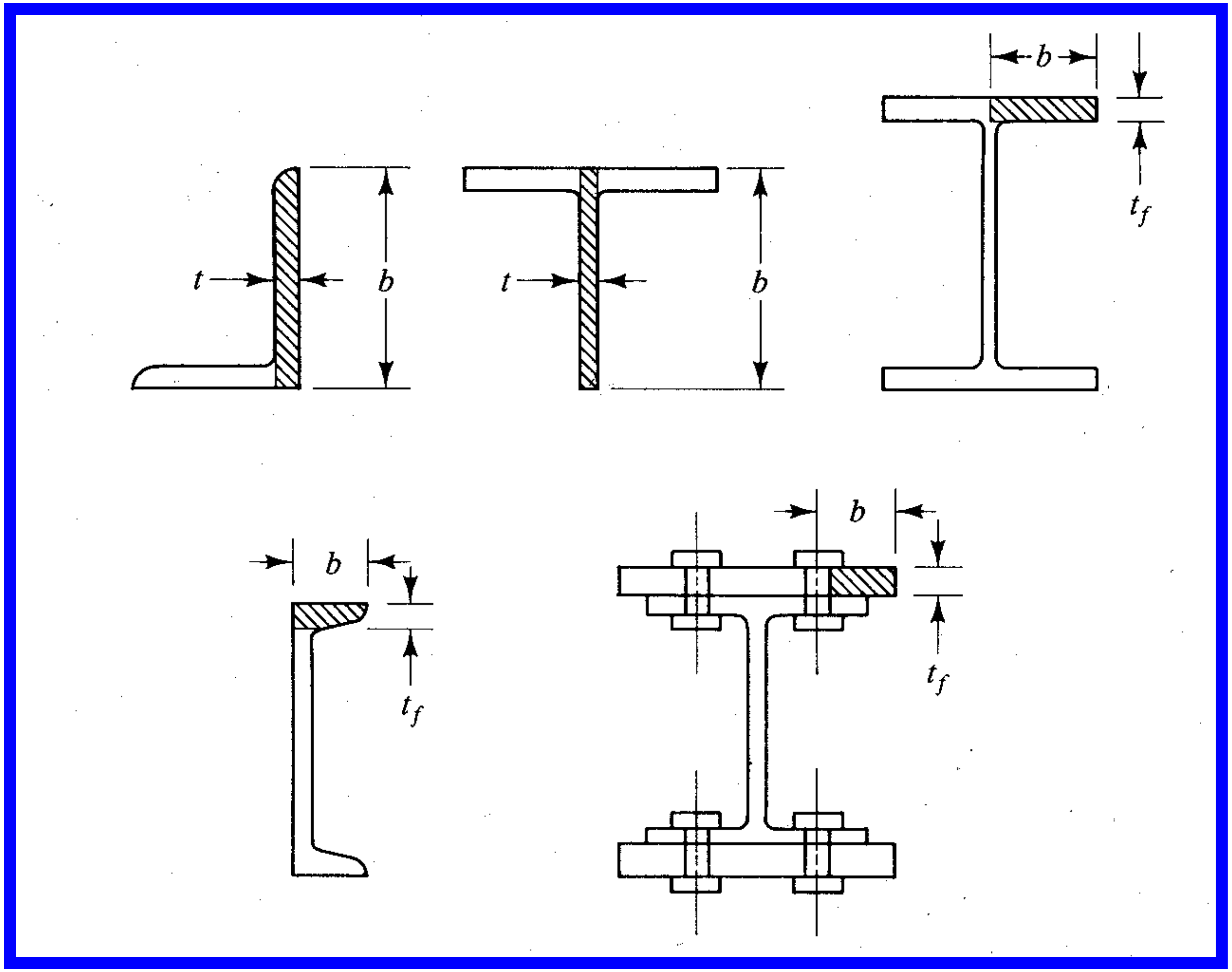


LOCAL BUCKLING

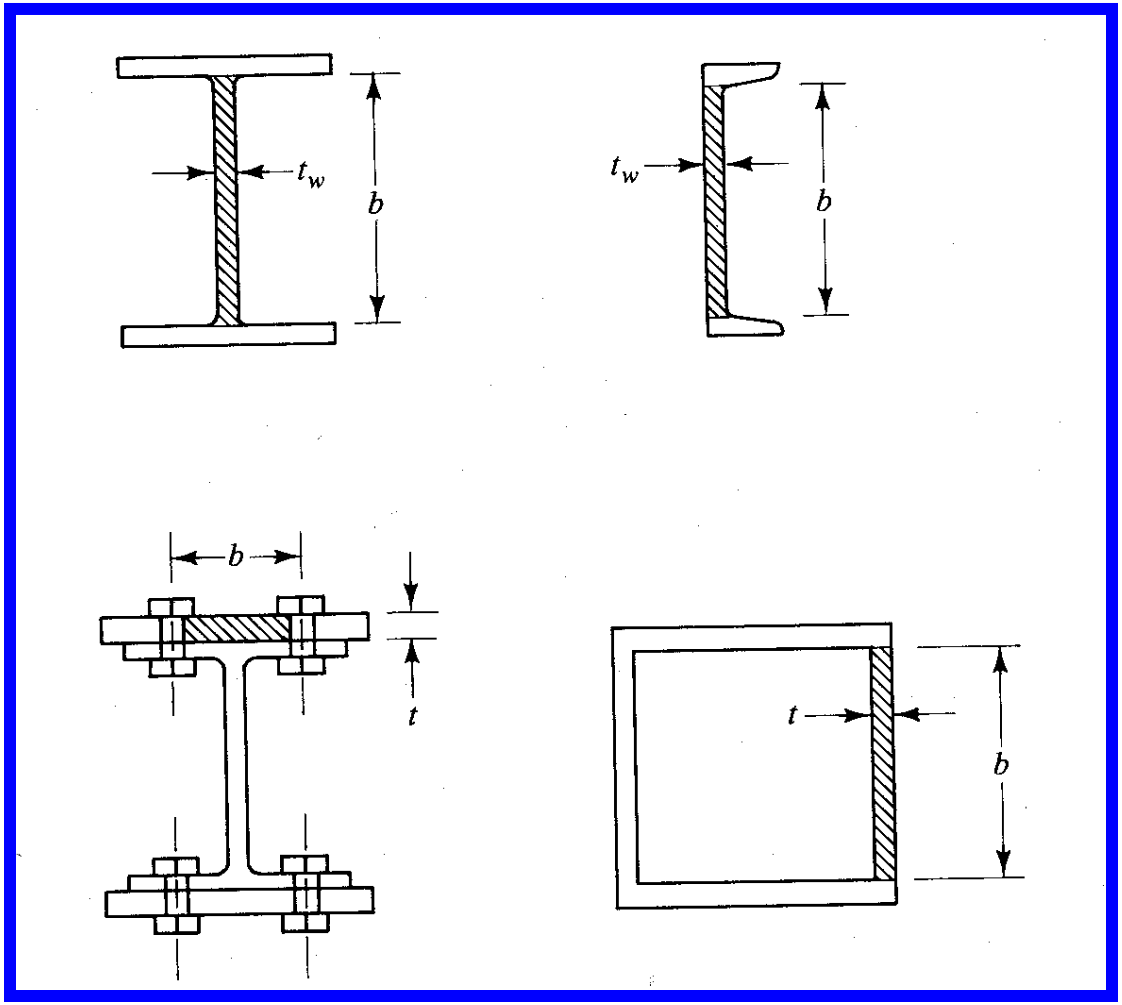
In IS:800 (1984) the local buckling is avoided by specifying b/t limits. Hence we don’t consider local buckling explicitly

However in IS:800(2007) limit state design, the local buckling would be the pivotal aspect for the design of structural components

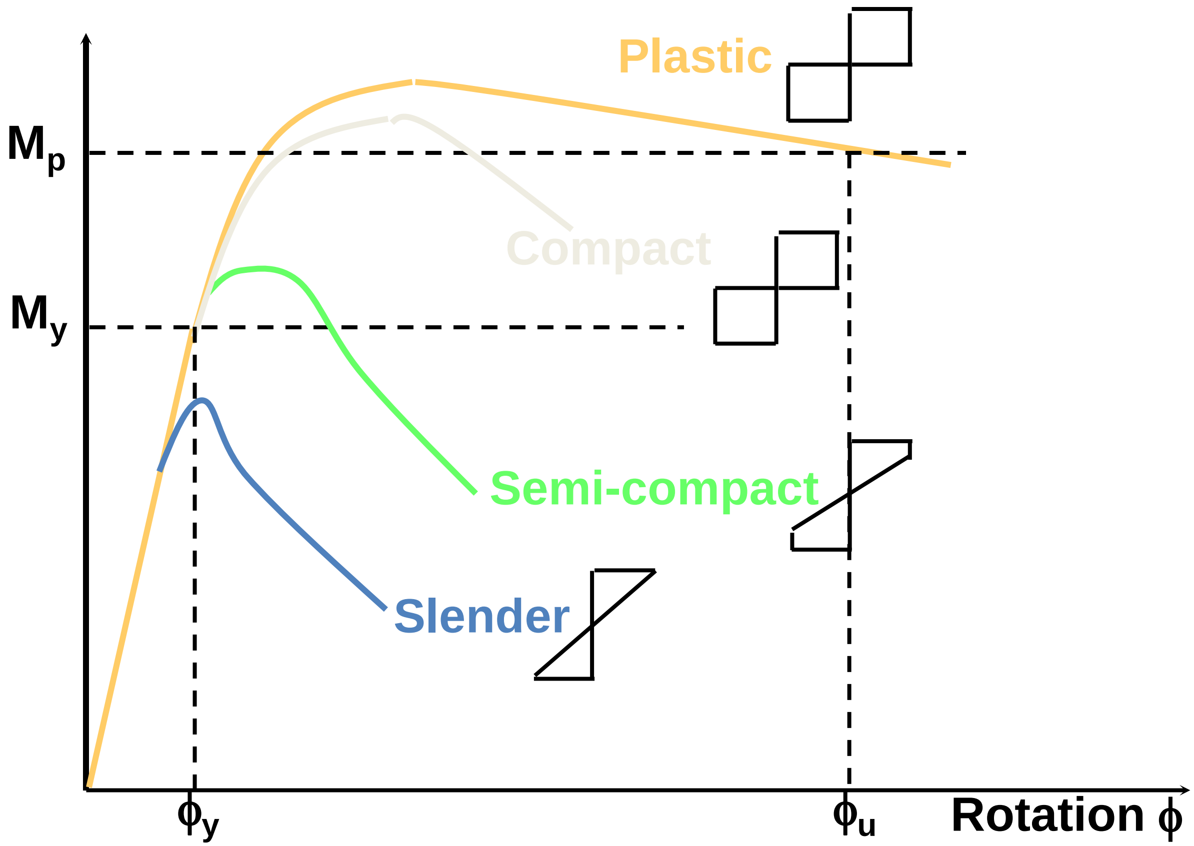
UNSTIFFENED OR OUTSTAND ELEMENTS



STIFFENED OR INTERNAL ELEMENTS



SECTION CLASSIFICATION



Section Classification based on Moment-Rotation Characteristics

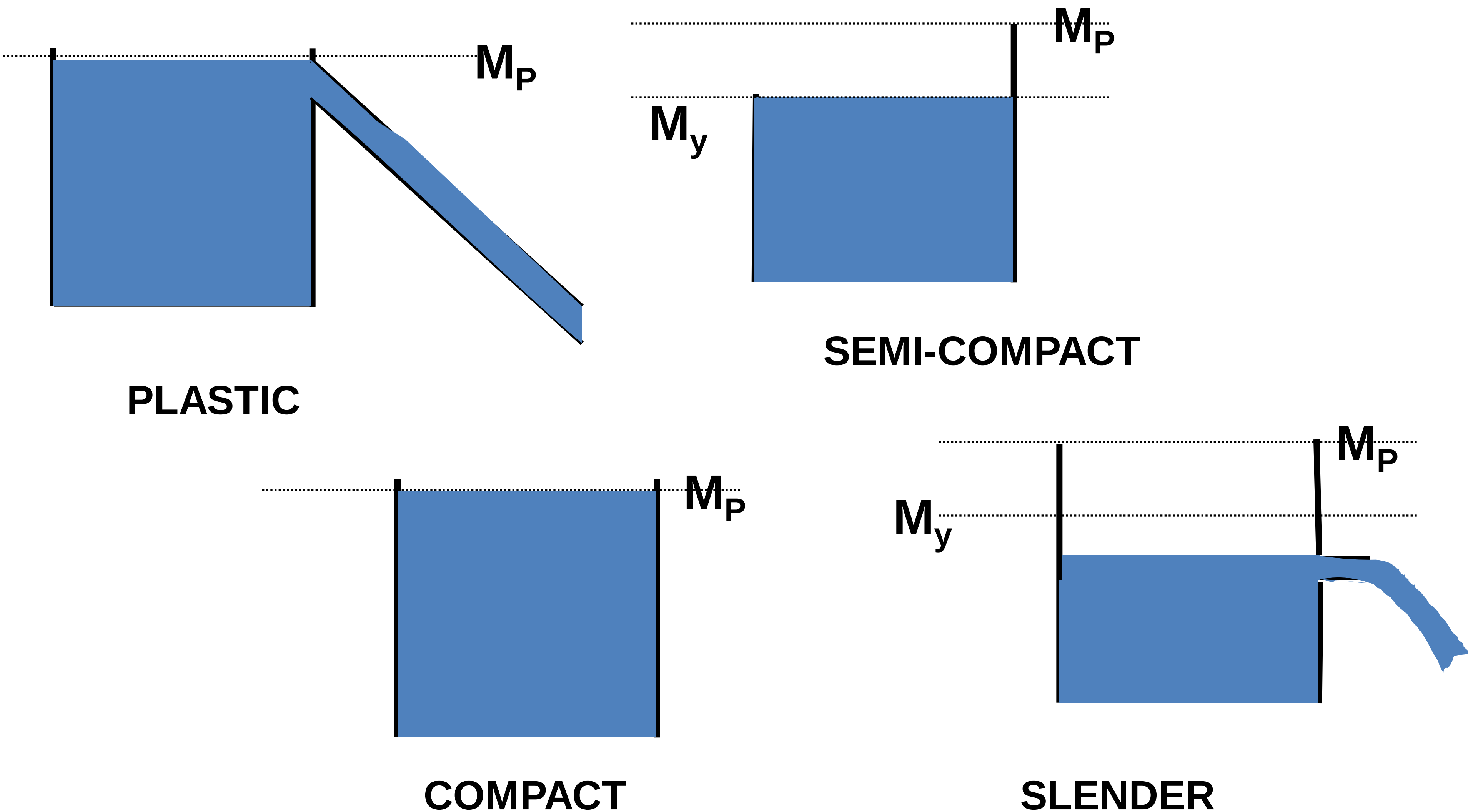
Section Classification

- a) *Plastic* – Cross sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of a plastic mechanism.

- b) *Compact* – Cross sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism.

- c) *Semi-Compact* – Cross sections, in which the extreme fibre in compression can reach, yield stress, but cannot develop the plastic moment of resistance, due to local buckling.

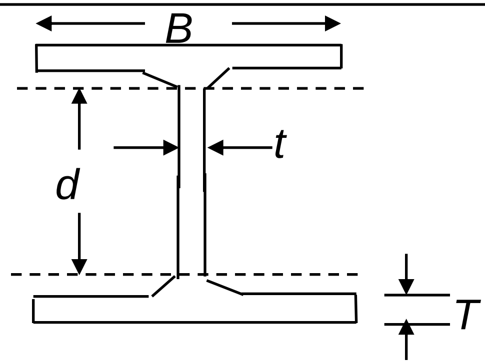
- d*) Slender* – Cross sections in which the elements buckle locally even before reaching yield stress. In such cases, the effective sections for design shall be calculated by deducting width of the compression plate element in excess of the semi-compact section limit.



MP
PLASTIC
MP
My
SEMI-COMPACT
MP
COMPACT
MP
My
SLENDER

Sectional Classification for Indian Conditions

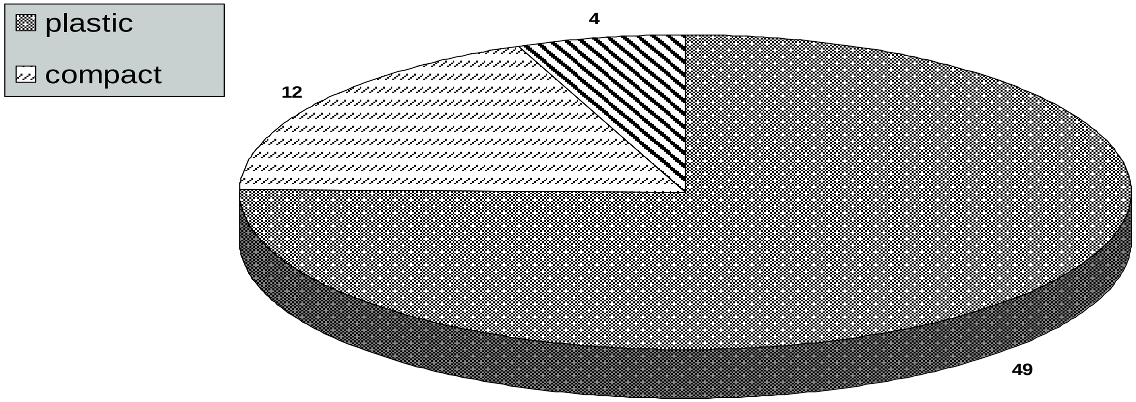
b = B/2



$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

Section type	***Flange criterion (b/T) ε***	***Web criterion (d/t) ε***
Slender	***>15.75***	***>126***
Semi-compact	***<15.75 $\geq$ 9.975***	***<126 $\geq$ 102.9***
Compact	***<9.975 $\geq$8.92***	***<102.9 $>$ 82.95***
Plastic	***<8.92***	***<82.95***

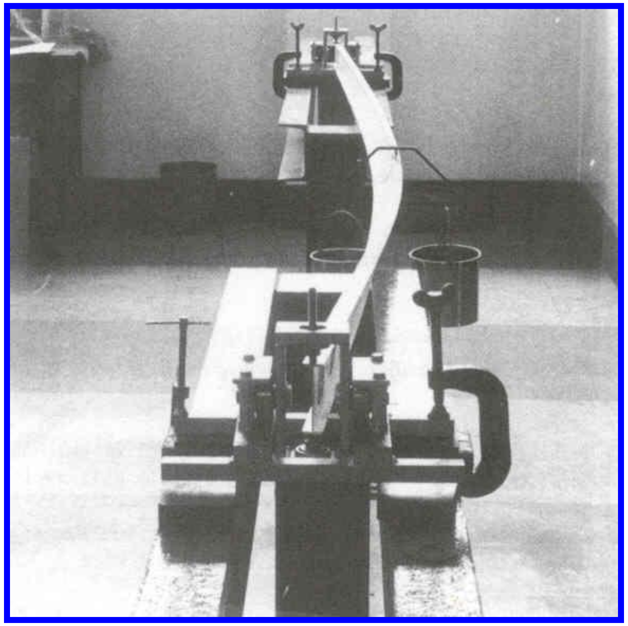
Section classification of Indian standard rolled 'I' beams

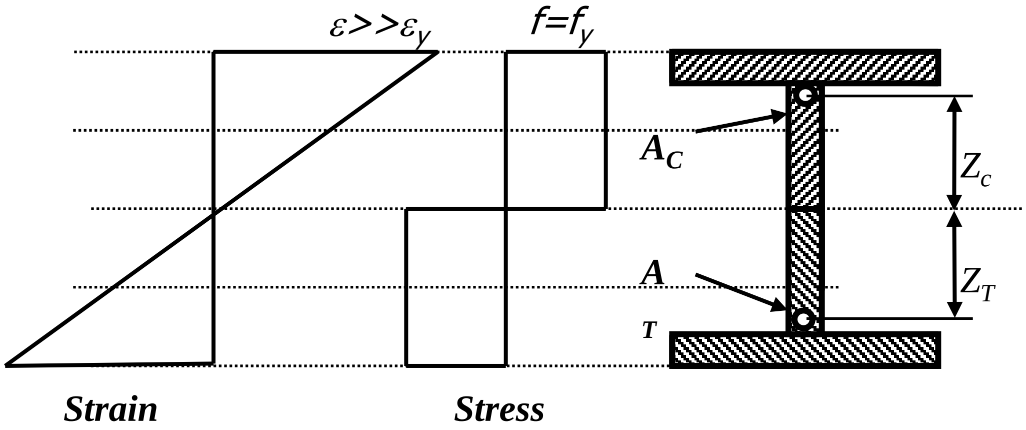


PLASTIC	49
COMPACT	12
SEMICOMPACT	4

LIMIT BEHAVIOUR OF LATERALLY RESTRAINED BEAMS AND ITS DESIGN

TYPES OF BEAM BEHAVIOUR

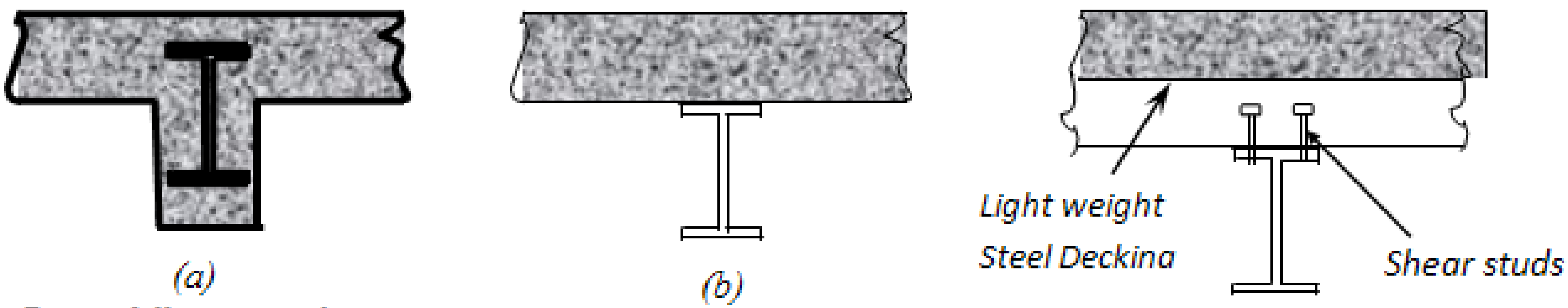




Lateral-torsional buckling

Flexural yielding

Laterally supported beams



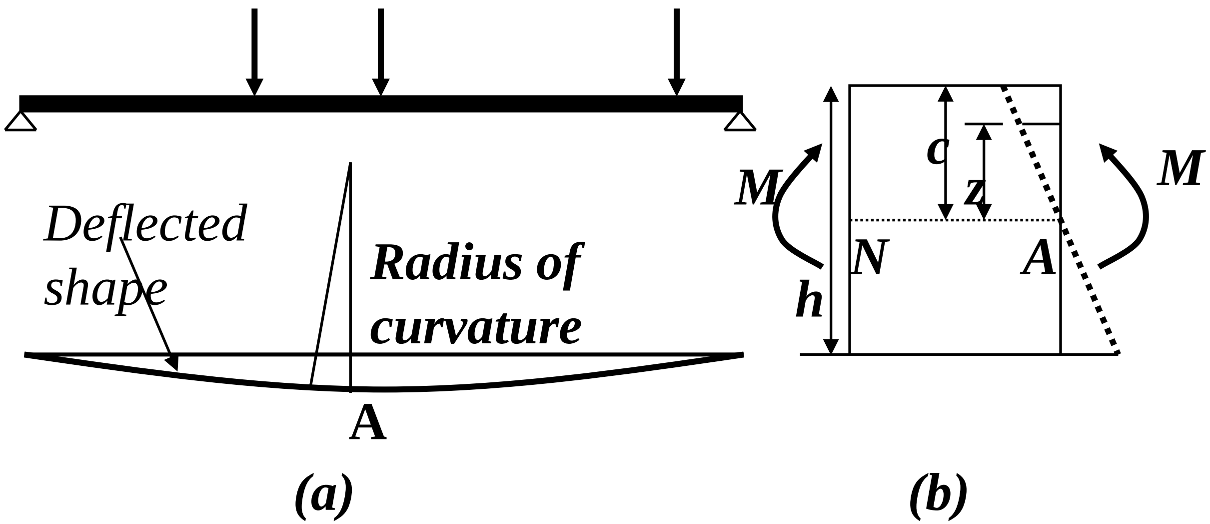
(a)
Beam fully encased in concrete

(b)
Beam in friction connection with concrete

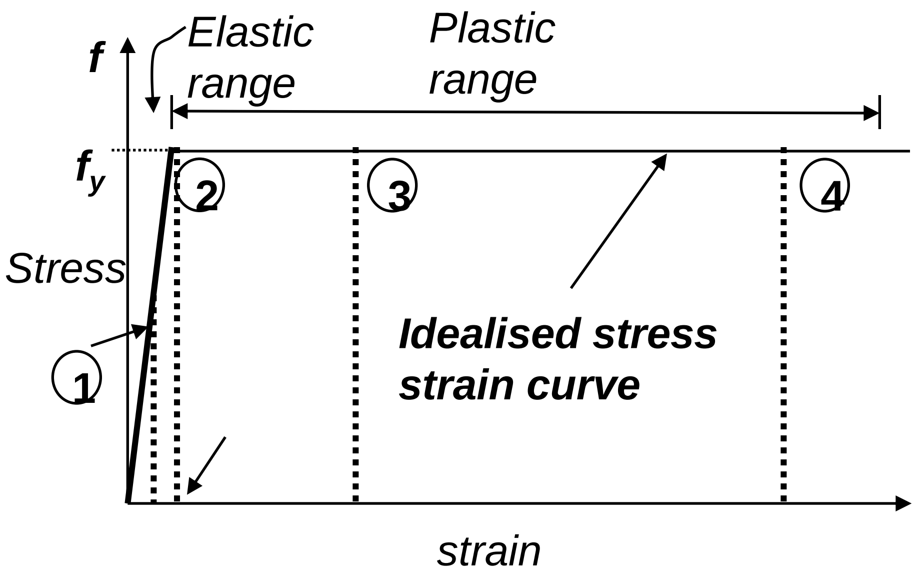
(c)
Beam with metal decking

Limit states for LR beams

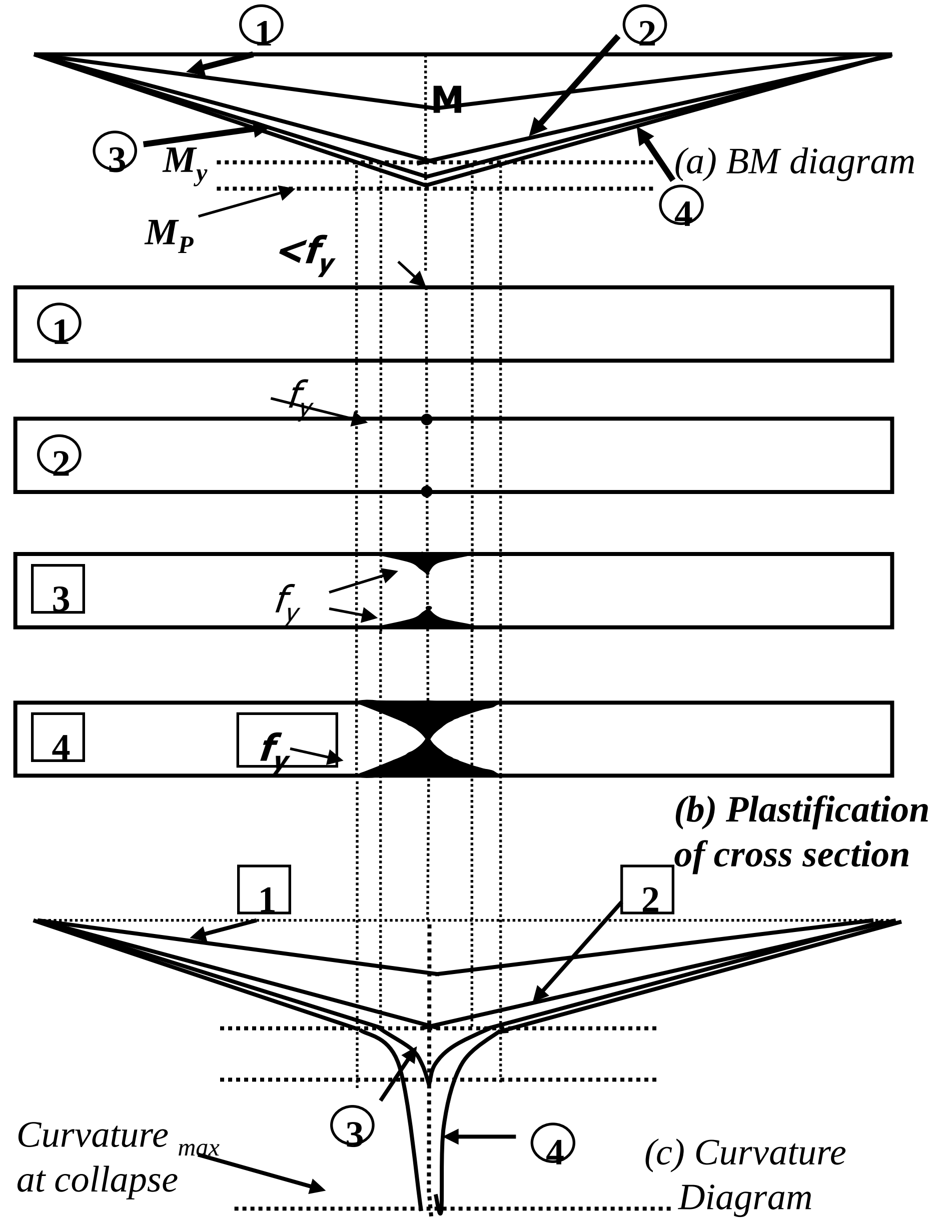
- Limit state of flexure
- Limit state of shear
- Limit state of bearing
- Limit state of serviceability

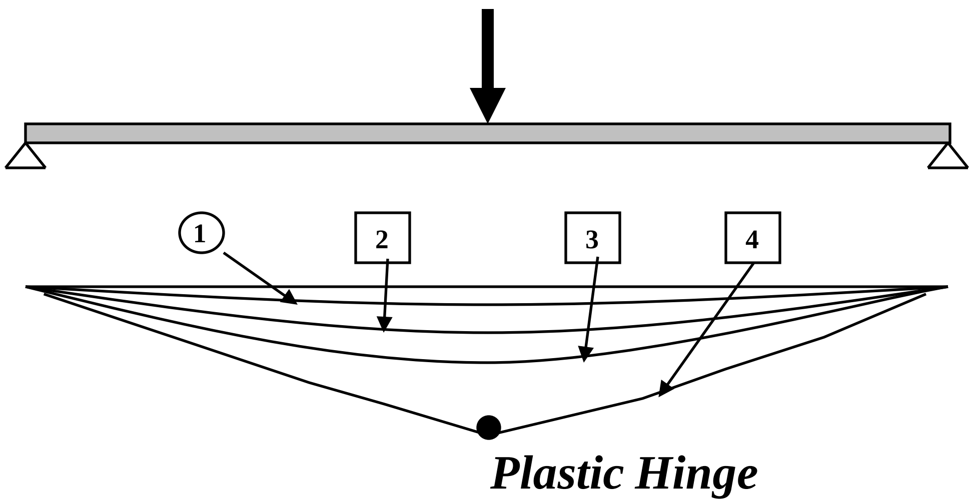


Curvature of bending

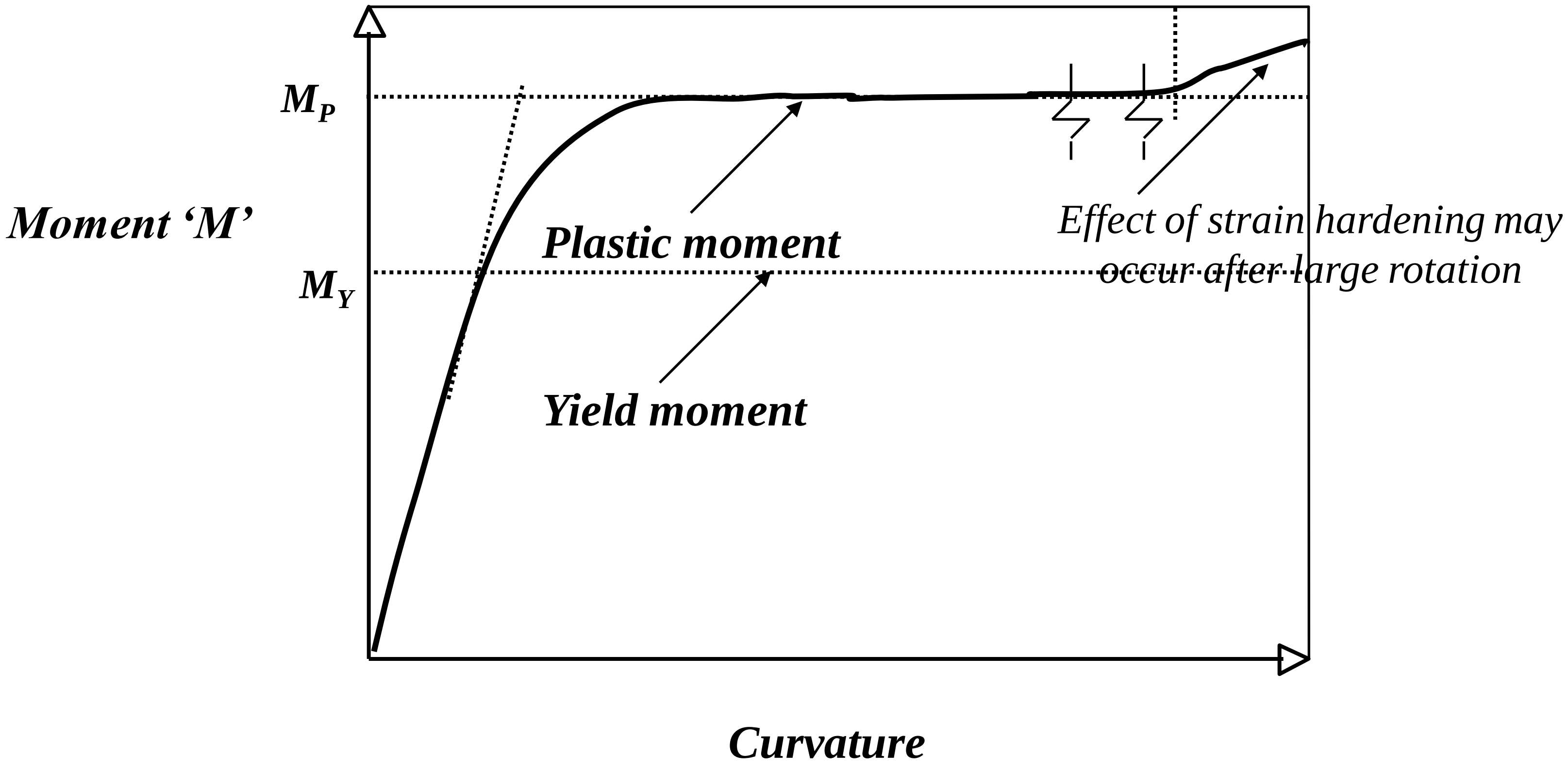


Idealised elasto- plastic stress stain curve for the purpose of design

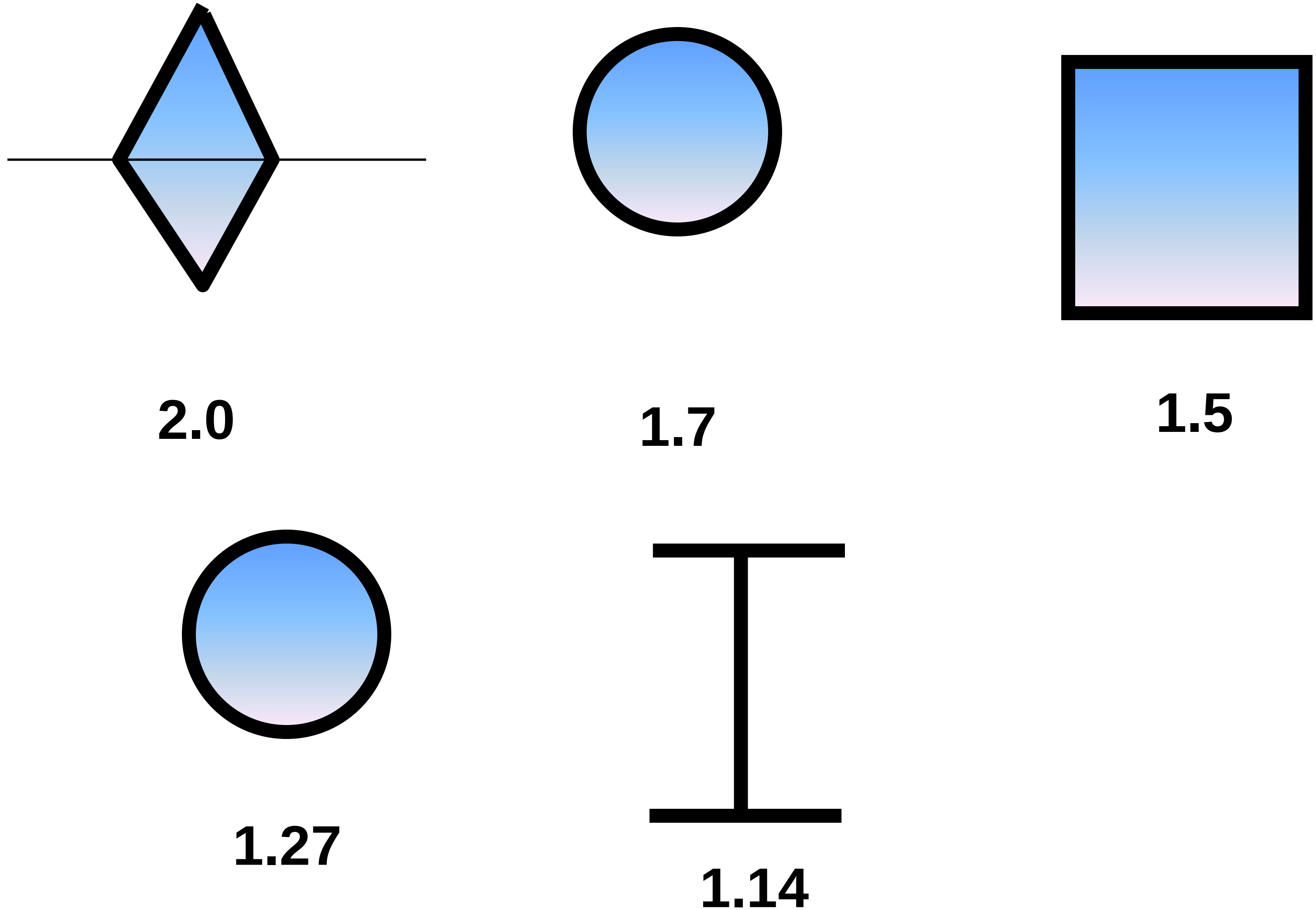

1
2
M
3
M_y
(a) BM diagram
4
M_P
$<f_y$
1
f_y
2
3
f_y
4
f_y
(b) Plastification of cross section
1
2
3
4
Curvature max at collapse
(c) Curvature Diagram



Simply supported beam and its deflection at various stages

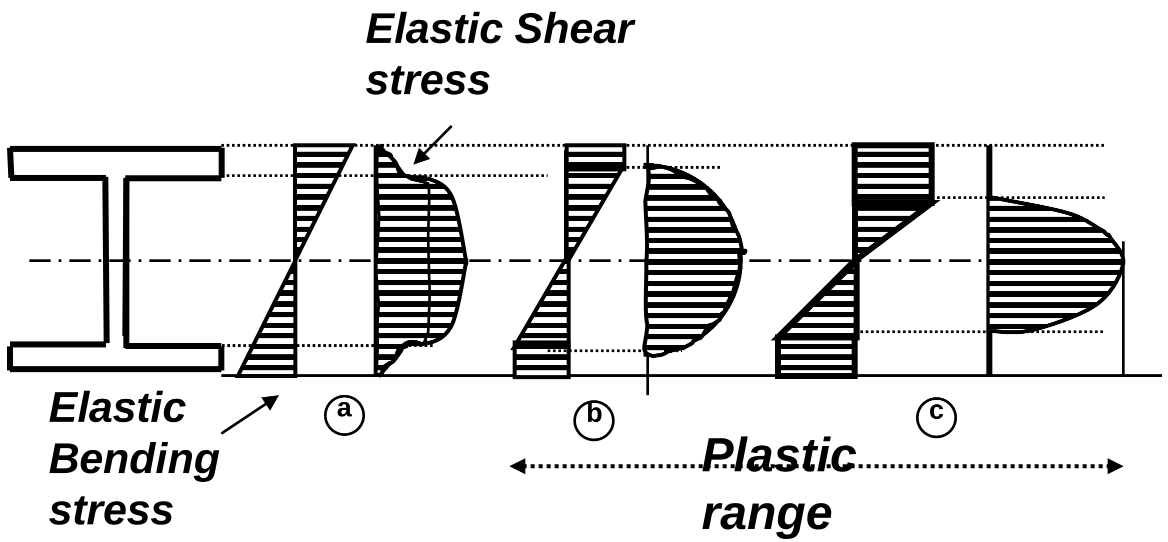


Moment curvature characteristics of the simply supported beam

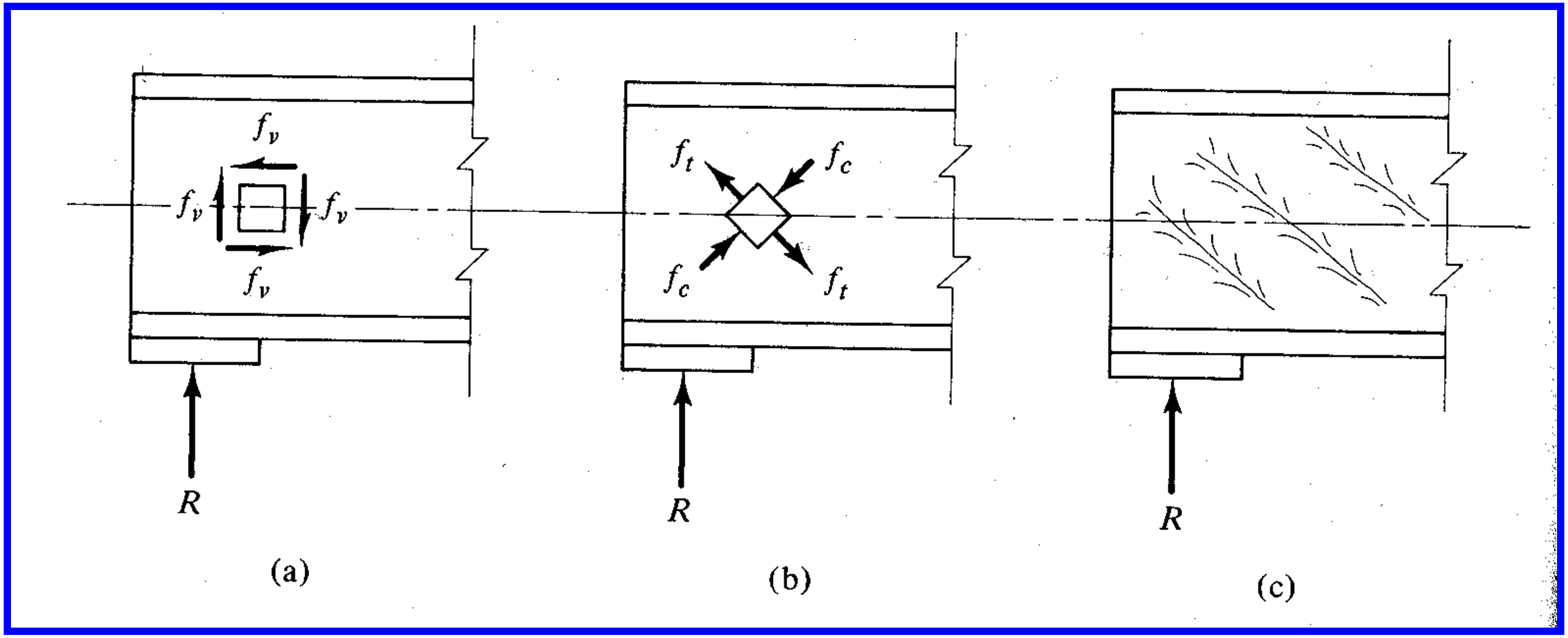


<u>Some typical shape factor</u>

LIMIT STATE OF SHEAR

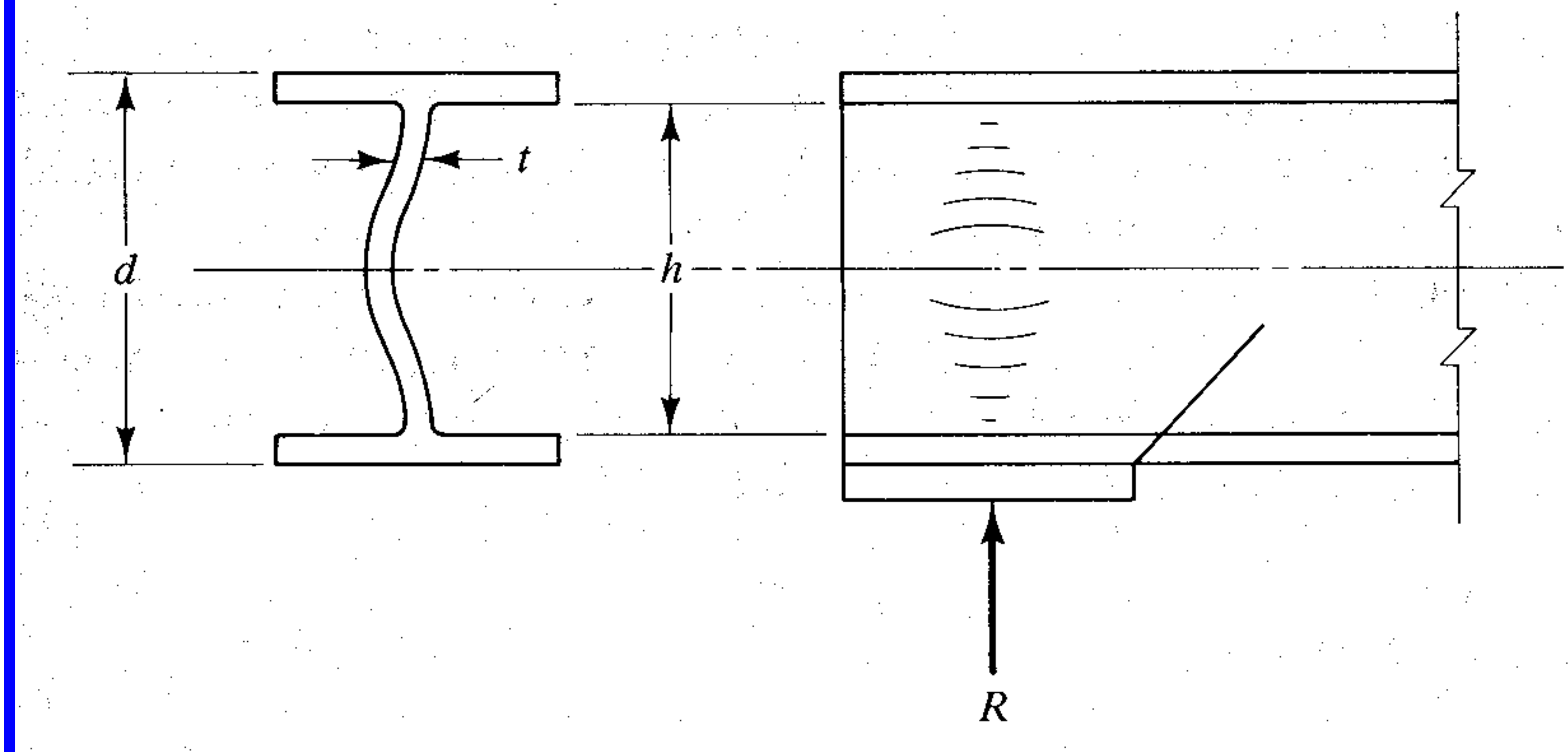


Combined bending and shear in beams



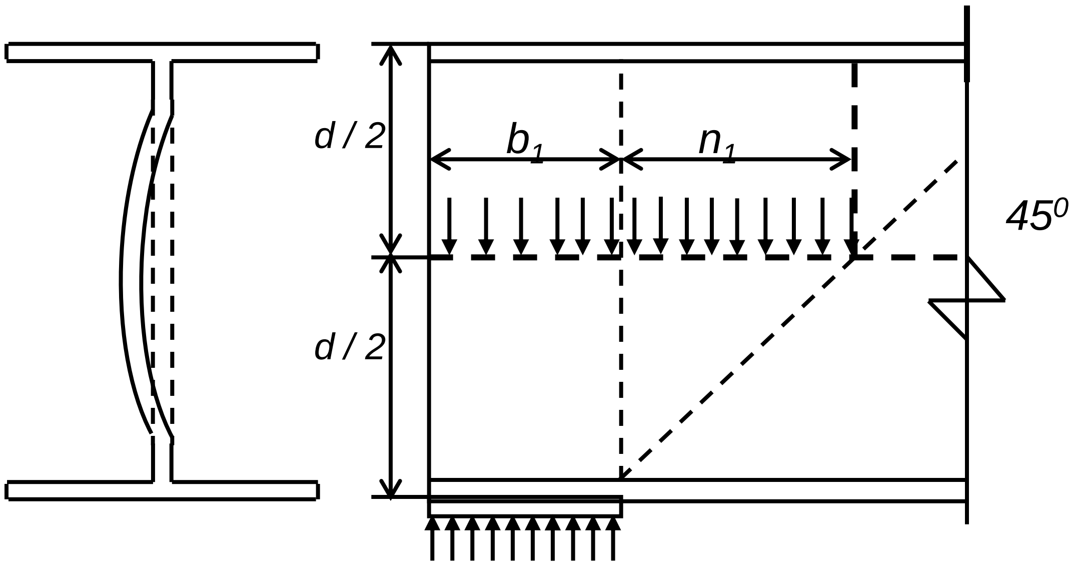
CHECK FOR BUCKLING OF WEBS

$$\frac{h}{t_w} =< 67\ \varepsilon$$



(a) (b)

WEB BUCKLING



Effective width for web buckling

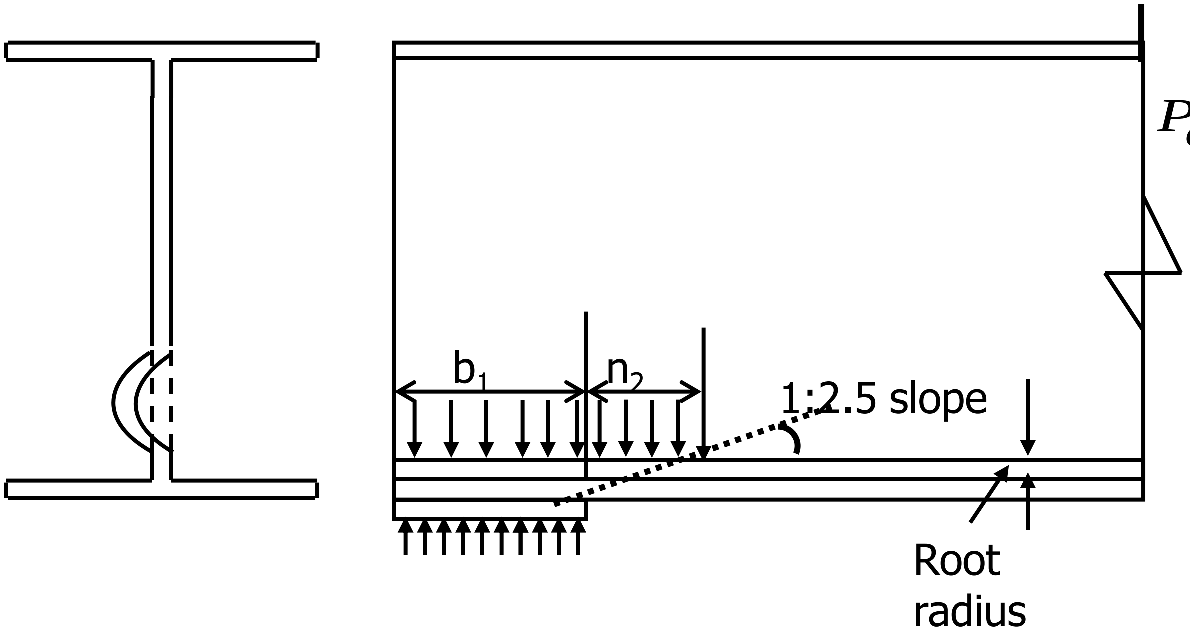
$$P_{wb} = (b_1 + n_1)\ t\ f_c$$

$$\lambda = \frac{L_E}{r_y} = \frac{0.7\,d}{r_y}$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{t^3}{12\,t}} = \frac{t}{2\sqrt{3}}$$

$$\frac{L_E}{r_y} = 0.7\,d\ \frac{2\sqrt{3}}{t} \approx 2.5\ \frac{d}{t}$$

WEB CRIPPLING



$$P_{crip} = (b_1 + n_2)\,t\, f_{yw}$$

Effective width of web bearing

Web Crippling in beams

TABLE 3.1 LIMITING WIDTH TO THICKNESS RATIOS

Compression element			Ratio	Class of Section		
				Class 1 Plastic	Class 2 Compact	Class 3 Semi-Compact
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I-, H-or box section [c]	Neutral axis at mid-depth		d/t_w	83.9ε	104.8ε	125.9ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\leq 40\varepsilon$	$\frac{104.8\varepsilon}{1+r_1}$	$\frac{125.9\varepsilon}{1+2r_2}$ but $\leq 40\varepsilon$
		If r_2 is negative:	d/t_w		$\frac{104.8\varepsilon}{1+1.5r_1}$ but $\leq 40\varepsilon$	
	Axial compression		d/t_w	Not applicable		
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component						
Circular tube subjected to moment or axial compression	CHS or built by welding		D/t	$44\varepsilon^2$	$62.7\varepsilon^2$	$88\varepsilon^2$
Stem of a T-section, rolled or cut from a rolled I-or H-section			D/t_f	8.4ε	9.4ε	18.9ε

APPENDIX F ELASTIC LATERAL TORSIONAL BUCKLING

F.1 Elastic Critical Moment

F.1.1 Basic

F.1.2 Elastic Critical Moment of a Section Symmetrical about Minor Axis

8.2 Design Strength in Bending (Flexure)

The factored design moment, *M* at any section, in a beam due to external actions shall satisfy

$$M \leq M_d$$

8.2.1 Laterally Supported Beam

The design bending strength as governed by plastic strength, M_d, shall be taken as

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2\, Z_e f_y / \gamma_{m0}$$

8.2.1.4 Holes in the tension zone

$$(A_{nf} / A_{gf}) \geq (f_y / f_u)(\gamma_{m1} / \gamma_{m0}) / 0.9$$

Cont...

8.4 Shear

The factored design shear force, V, in a beam due to external actions shall satisfy

$$V \le V_d$$

V_d = design strength calculated as , $V_d = V_n / \gamma_{m0}$

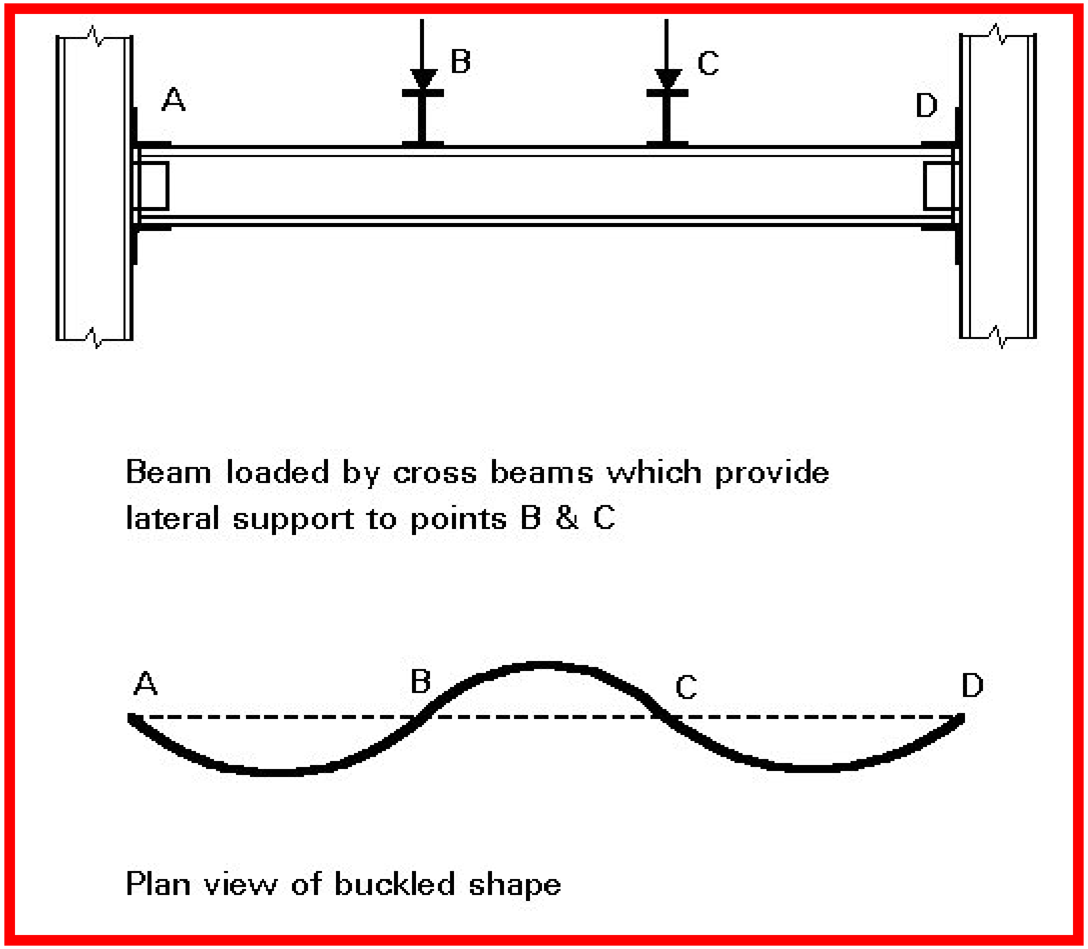
8.4.1 The nominal plastic shear resistance under pure shear is given by: $V_n = V_p$

A_v = shear area

$$V_p = \frac{A_v \, f_{yw}}{\sqrt{3}}$$

Cont…

INTERMEDIATE BEAMS OFFERING LATERAL RESTRAINT



DESIGN OF STEEL BEAMS

- **STEP 1:**

  Determination of design shear forces V and bending moments M at critical points on the element

  Table 4 (page 29) gives the factors for different load combinations

DESIGN OF STEEL BEAMS…

- **STEP 2:**

Section Modulus Required

$$Z_p \text{ (required)} = M \times \gamma_{mo} / f_y$$

– **γ_{mo} is the partial Safety Factor for materials given in Table 5 (page 30)**

Table 5 Partial Safety Factor for Materials, γ_m

(*Clause* 5.4.1)

Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}	1.25	1.25
	b) Bolts-Bearing Type, γ_{mb}	1.25	1.25
	c) Rivets, γ_{mr}	1.25	1.25
	d) Welds, γ_{mw}	1.25	1.50

DESIGN OF STEEL BEAMS…

- **STEP 3:**

Selection of Suitable Section

Shape Factor (v) -

The ratio M_p/M_y is a property of the cross-section shape and is independent of the material properties.

$$\upsilon = M_p/M_y = Z_p/Z_e$$

Hence, $Z_p = Z_e \times \upsilon$

DESIGN OF STEEL BEAMS…

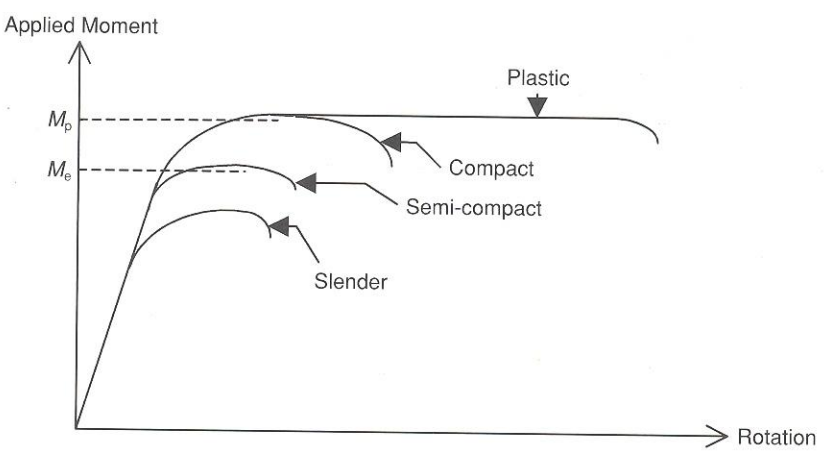
Shape factor of different cross-sections

Cross-section	**Shape Factor v**		
	Max.	**Min.**	**Avg.**
Hollow Circular	1.47	1.30	1.35
Hollow Rectangular	1.33	1.19	1.25
Wide flange I-section (major axis)	1.18	1.09	1.14
Wide flange I-section (minor axis)	1.67	-	-
Unequal angles	1.83	1.75	1.8
Equal angle	1.84	1.81	1.82
Channel (major axis)	1.22	1.16	1.18
Channel (minor axis)	1.8	-	-

DESIGN OF STEEL BEAMS…

- **STEP 4:**

Classification of Section **(Table 2, page 18)**



Check adequacy of the section including self-weight

DESIGN OF STEEL BEAMS…

- **STEP 5:**

Check shear Strength

Design shear Strength, $V_d = A_v \times f_{yw}/\sqrt{3}$ (cl. 8.4, page 59)

$(V_d > V)$

$$V_p = \frac{A_v f_{yw}}{\sqrt{3}}$$

If $V > 0.6\, V_d$, design for combined shear and bendin … **9)**

A_v = shear area, and

f_{yw} = yield strength of the web.

Where A_v = shear area

f_{yw} = yield strength of web

DESIGN OF STEEL BEAMS…

- **STEP 6:**

Check Bending Capacity

- If laterally supported beam **(cl. 8.2.1, page 52)**

- If laterally unsupported beam **(cl. 8.2.2, page 54)**

Get M_d and check if $M < M_d$

DESIGN OF STEEL BEAMS…

- **STEP 7:**

Check for deflection

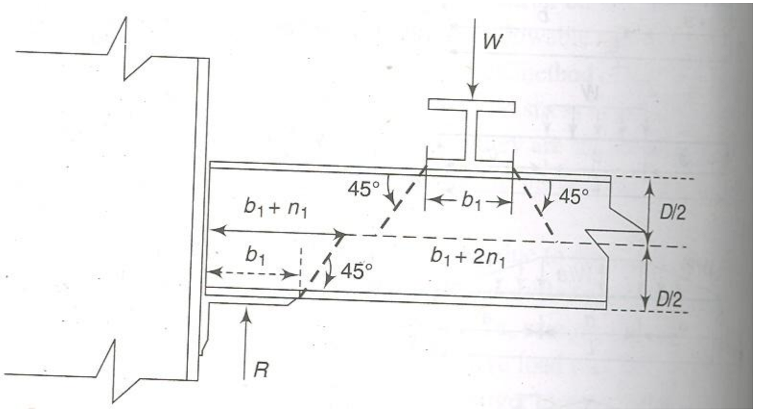
This is a serviceability limit state and hence must be calculated on the basis of unfactored imposed loads

Allowable max. deflection **–(Table 6, page 31)**

DESIGN OF STEEL BEAMS…

- **STEP 8**

Check for Web Buckling **(cl. 8.7.3.1, page 67)**



Dispersion of concentrated loads and reactions for evaluating web buckling

DESIGN OF STEEL BEAMS…

- **STEP 9**

Check for Web Bearing (cl. 8.7.4, page 67)

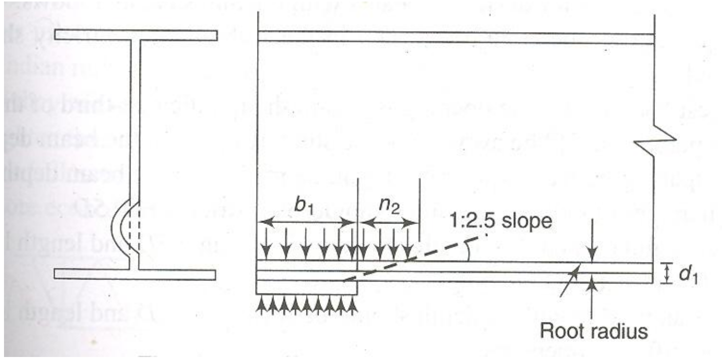


Table 6 Deflection Limits

Type of Building	Deflection	Design Load	Member	Supporting	Maximum Deflection
(1)	(2)	(3)	(4)	(5)	(6)
Industrial Buildings	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/ Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50 t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1 000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
		Crane+ wind	Column/frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400
Other Buildings	Vertical	Live load	Floor and Roof	Elements not susceptible to cracking	Span/300
				Elements susceptible to cracking	Span/360
		Live load	Cantilever	Elements not susceptible to cracking	Span/150
				Elements susceptible to cracking	Span/180

UNIT-IV

DESIGN OF ECCENTRIC CONNECTIONS WITH BRACKETS

INTRODUCTION

- **In case of overloading, failure in member is preferred to failure in connection**

- **Connections account for more than half the cost of structural steel work**

- **Connection design has influence over member design**

- **Similar to members, connections are also classified as idealised types**

  Effected through rivets, bolts or weld

- **Codal Provisions**

Why Connection Failure Should be Avoided?

- A connection failure may be lead to a catastrophic failure of the whole structure
- Normally, a connection failure is not as ductile as that of a steel member failure
- For achieving an economical design, it is important that connectors develop full or a little extra strength of the members, it is joining.
- Connection failure may be avoided by adopting a higher safety factor for the joints than the members

Classification of Connections

- Method of fastening: rivets, bolts and welding.
- Connection rigidity: simple, rigid or semi-rigid.
- Joint resistance: Bearing connections and friction connections
- Fabrication location: Shop or field connections.
- Joint location: Beam-column, beam-to beam, column to foundation

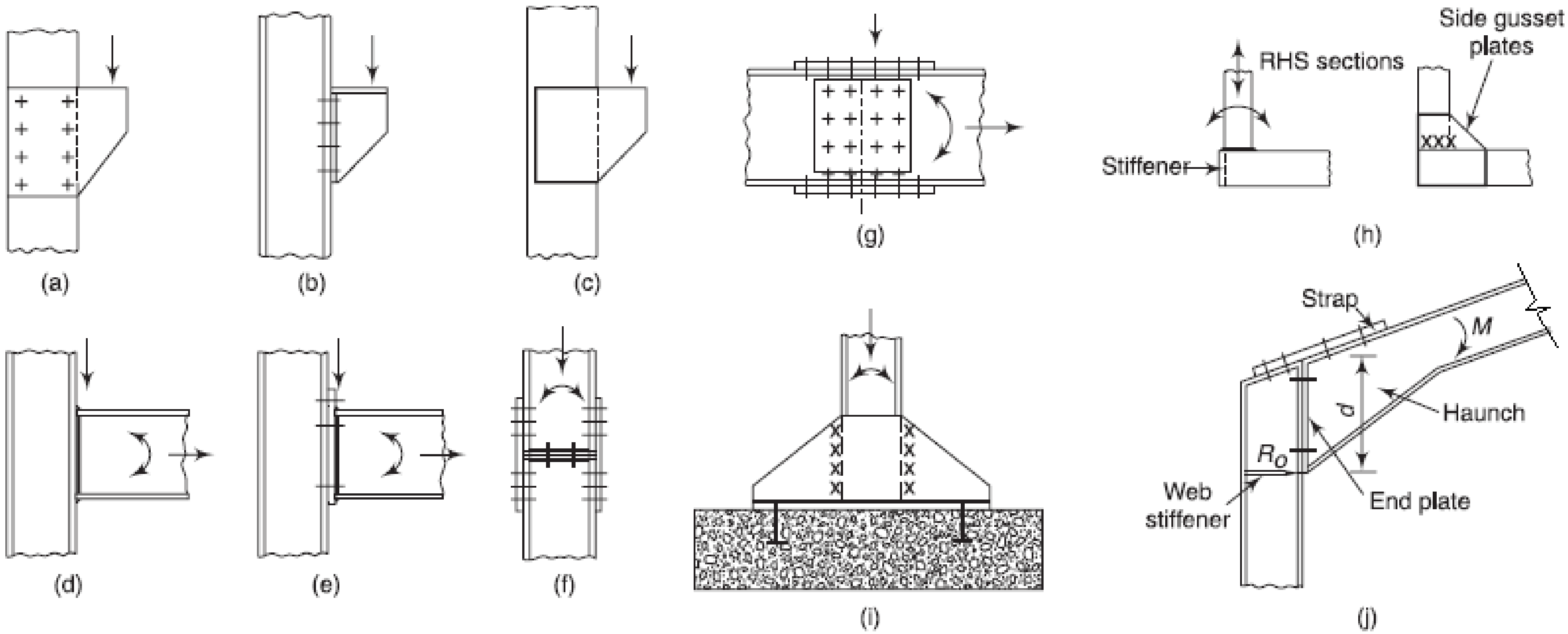
Classification of Connections

- Connection geometry: Single web angle, single plate, double web angle, top and seat angles (with and without stiffeners), end plates, or header plate, welded connections using plates and angles, etc.
- Type of force transferred across the structural connection: Shear connections, shear and moment connection or simply moment connection, tension or compression, tension or compression with shear.

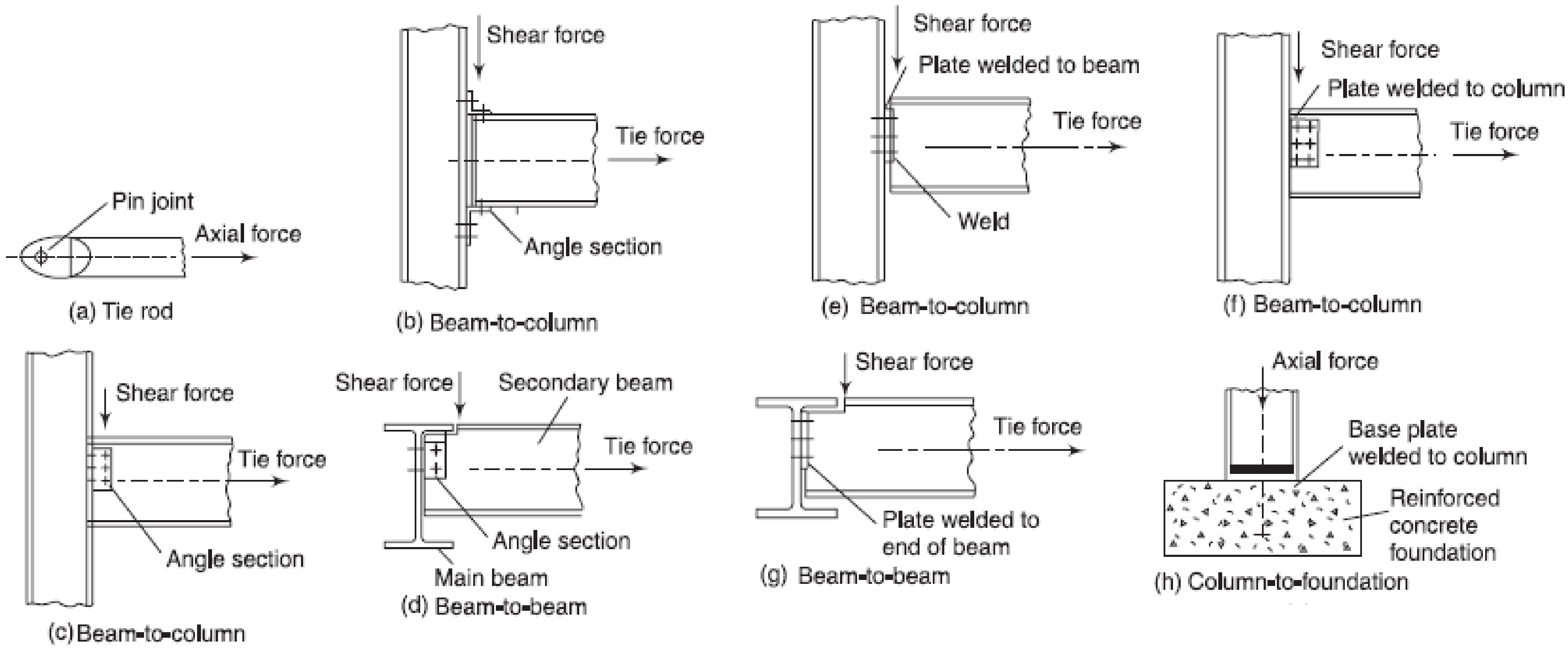
Classification Based on Joint Rigidity

- Rigid: That develop the full moment capacity of connecting members and retain the original angle between the members under any joint rotation. Rotational movement of the joint will be very small
- Simple: No moment transfer is assumed between the connected parts and hence assumed as hinged (pinned). Rotational movement of the joint will be large.
- Semi-Rigid: May not have sufficient rigidity to hold the original angles between the members and develop less than the full moment capacity of the connected members. In reality all the connections will be semi-rigid only.

Examples of Rigid Connections



Examples of Pinned Connections



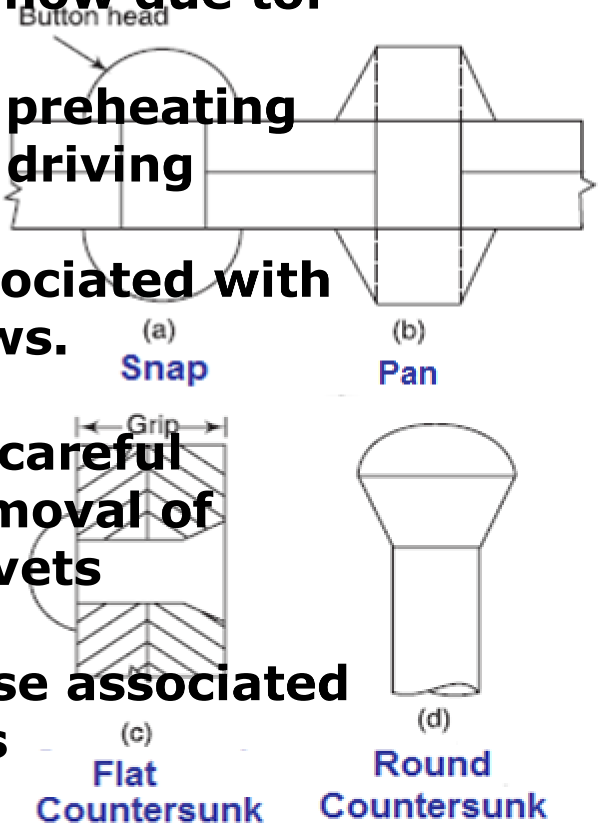
Bolted 'pin' connections

Welded 'pin' connections

Rivets and Riveted Connections

Riveting not used now due to:

- **The necessity of preheating the rivets prior to driving**
- **Labour costs associated with large riveting crews.**
- **Cost involved in careful inspection and removal of poorly installed rivets**
- **High level of noise associated with driving rivets**

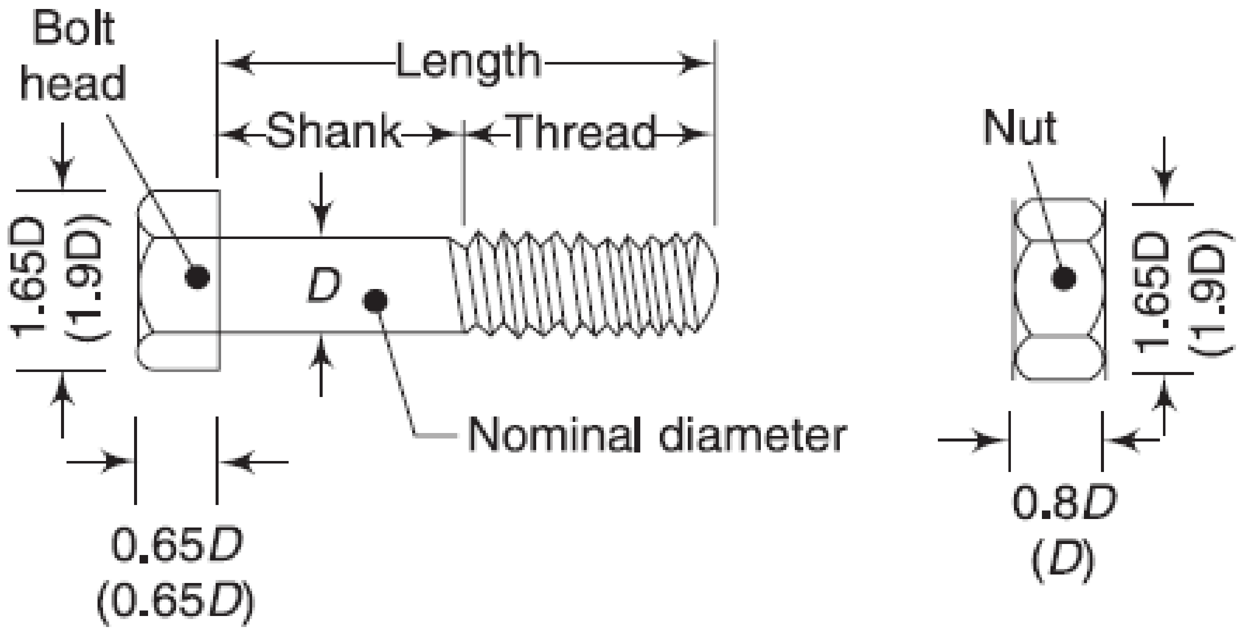


Types of Bolts

- Unfinished bolts or black bolts or C Grade bolts (IS: 1363-1992) -bearing type connections
- Turned bolts - Expensive & used in Spl. jobs
- Precision (A-Grade)& Semi-precision (B-Grade) bolts (IS: 1364-1992) -They are used when no slippage is permitted
- Ribbed bolts (Rarely used in ordinary steel structures)
- High strength bolts (IS: 3757-1985 and IS:4000 - 1992)-Friction type connections

Black or Ordinary Bolt and Nut

Hexagonal Head Black Bolt and Nut (IS 1363)



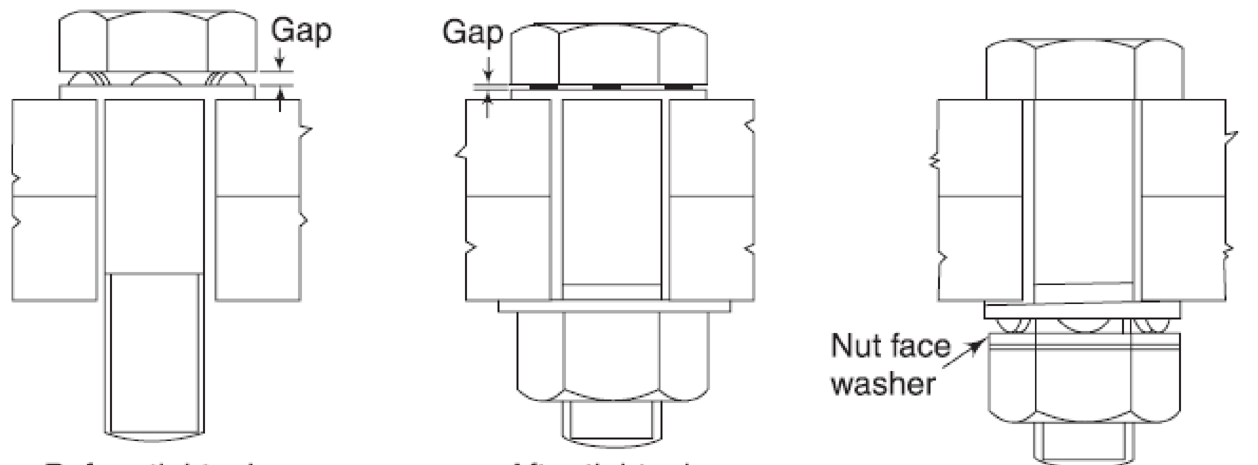
Figures in brackets are for High-strength Bolts & Nuts

Black bolts are inserted in clearance holes of about 1.5mm to 2mm more than the bolt diameter and then tightened through the nuts.

Direct Tension Indicator Tightening

- There are two types of proprietary load – indication devices.
- The first type of device indicates the load by producing a measurable change in gap between the nut and the gripped material.


Load indicating washer



Before tightening

After tightening

Arrangement if monitoring tension and tightening from same side

Monitoring tension and tightening from opposite sides

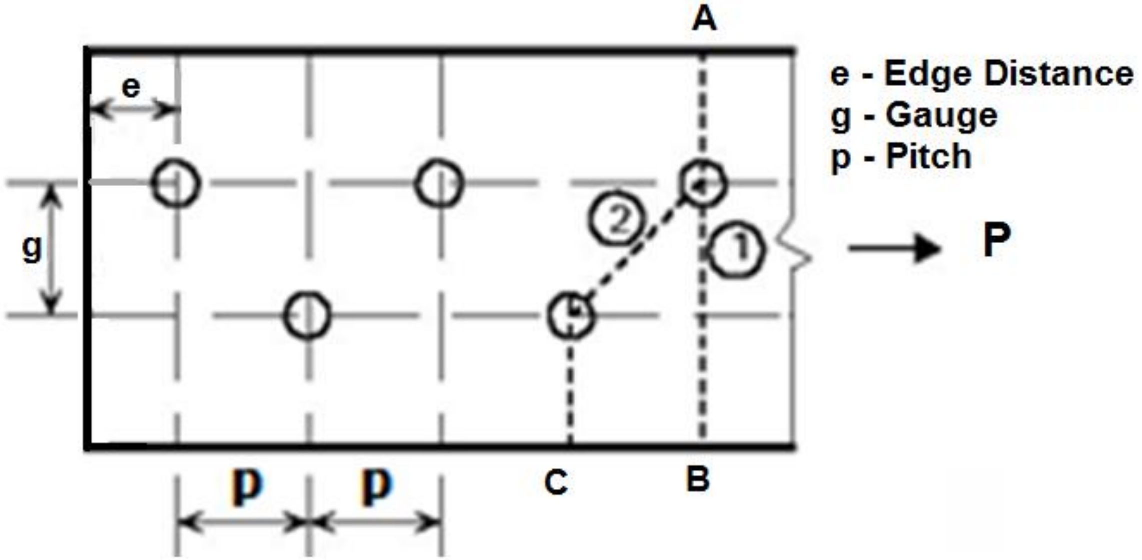
Advantages of Bolted connections

- Bolted connections offer the following advantages over riveted or welded connections:
  - **Use of unskilled labour and simple tools**
  - **Noiseless and quick fabrication**
  - **No special equipment/process needed for installation**
  - **Fast progress of work**
  - **Accommodates minor discrepancies in dimensions**
  - **The connection supports loads as soon as the bolts are tightened (in welds and rivets, cooling period is involved).**
- Main drawback of black bolt is the slip of the joint when subjected to loading

Bolt Holes

- Bolt holes are usually drilled.
- IS: 800 allows punched holes only in materials whose yield stress (f_y) does not exceed 360 MPa and where thickness does not exceed ($5600/f_y$) mm.
- Bolt holes are made larger than the bolt diameter to facilitate erection.
- Oversize holes should not exceed 1.25d or (d+8) mm in diameter, where d is the nominal bolt diameter in mm.
- Slotted hole [provided to accommodate movements) should not exceed 1.33d in length (for short slotted hole) and 2.5 d in length (for long slotted hole).

Pitch, Staggered holes & Gauge



The edge distance should be sufficient for bearing capacity and to provide space for bolt head, washer and nut.

A minimum spacing of 2.5 times the nominal diameter of the fastener is specified in the code to ensure that there is sufficient space to tighten the bolts, to prevent overlapping of the washers and to provide adequate resistance to tear-out of the bolts.

Bolt Dia, Pitch & Edge Distances as per IS 800

Nominal diameter of bolt, mm	12	14	16	18	20	22	24	27	30	Above 36
Diameter of hole, mm	13	15	18	20	22	24	26	30	33	Bolt diameter + 3 mm
Minimum edge distance,* mm										
(a) for sheared or rough edge	22	26	31	34	37	41	44	51	56	1.7 × hole diameter
(b) for rolled, sawn, or planed edge	18	23	27	30	33	36	39	45	50	1.5 × hole diameter

*The edge distances in this table, which are for standard holes, must be increased if oversize or slotted holes are used.

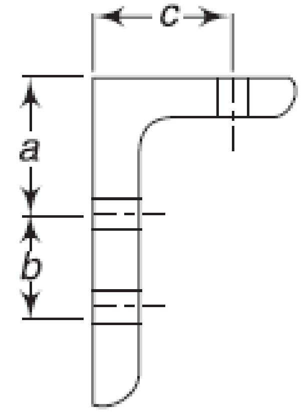
Max. edge distance = $12t\varepsilon$ where $\varepsilon = (250/f_y)^{0.5}$

Pitch (min.)	2.5 × nominal diameter of bolt
Pitch (max.)	$32t$ or 300 mm
(a) parts in tension	$16t$ or 200 mm, whichever is less
(b) parts in compression	$12t$ or 200 mm, whichever is less
(c) tacking fasteners	$32t$ or 300 mm, whichever is less $16t$ or 200 mm, whichever is less for plates exposed to weather

where t is the thickness of the thinner outside plate or angle.

Gauge Distances for bolts as per SP-1

Leg size	Double row of bolts *a*	*b*	Single row of bolts *c*	Maximum bolt size for double row of bolts
mm	mm	mm	mm	mm
200	75	85	115	27
150	55	65	90	22
130	50	55	80	20
125	45	55	75	20
115	45	50	70	12
110	45	45	65	12
100	40	40	60	12
95	—	—	55	—
90	—	—	50	—
80	—	—	45	—
75	—	—	40	—
70	—	—	40	—
65	—	—	35	—
60	—	—	35	—
55	—	—	30	—
50	—	—	28	—
45	—	—	25	—
40	—	—	21	—
35	—	—	19	—
30	—	—	17	—
25	—	—	15	—
20	—	—	12	—



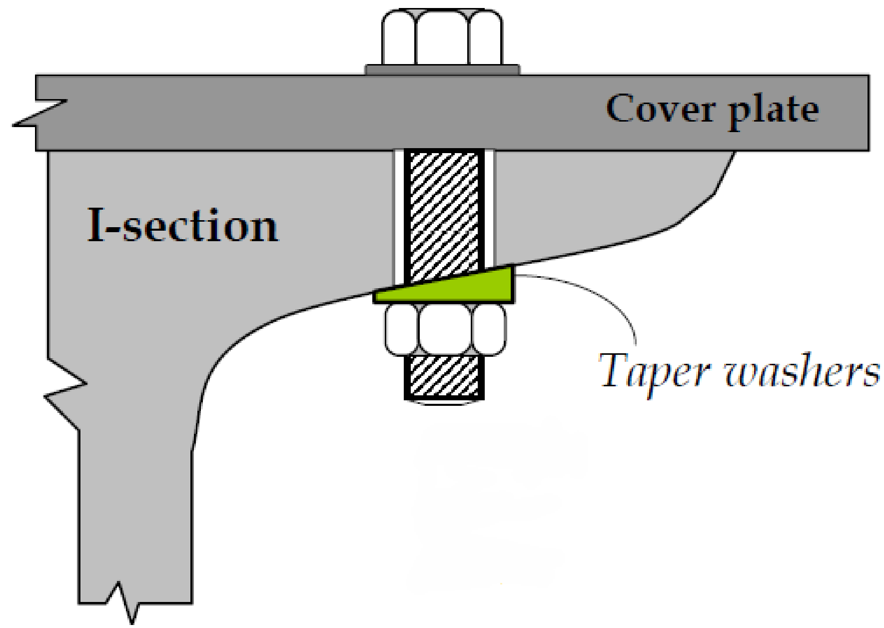
Note on IS Rolled Sections

Bolting is often poorly executed:

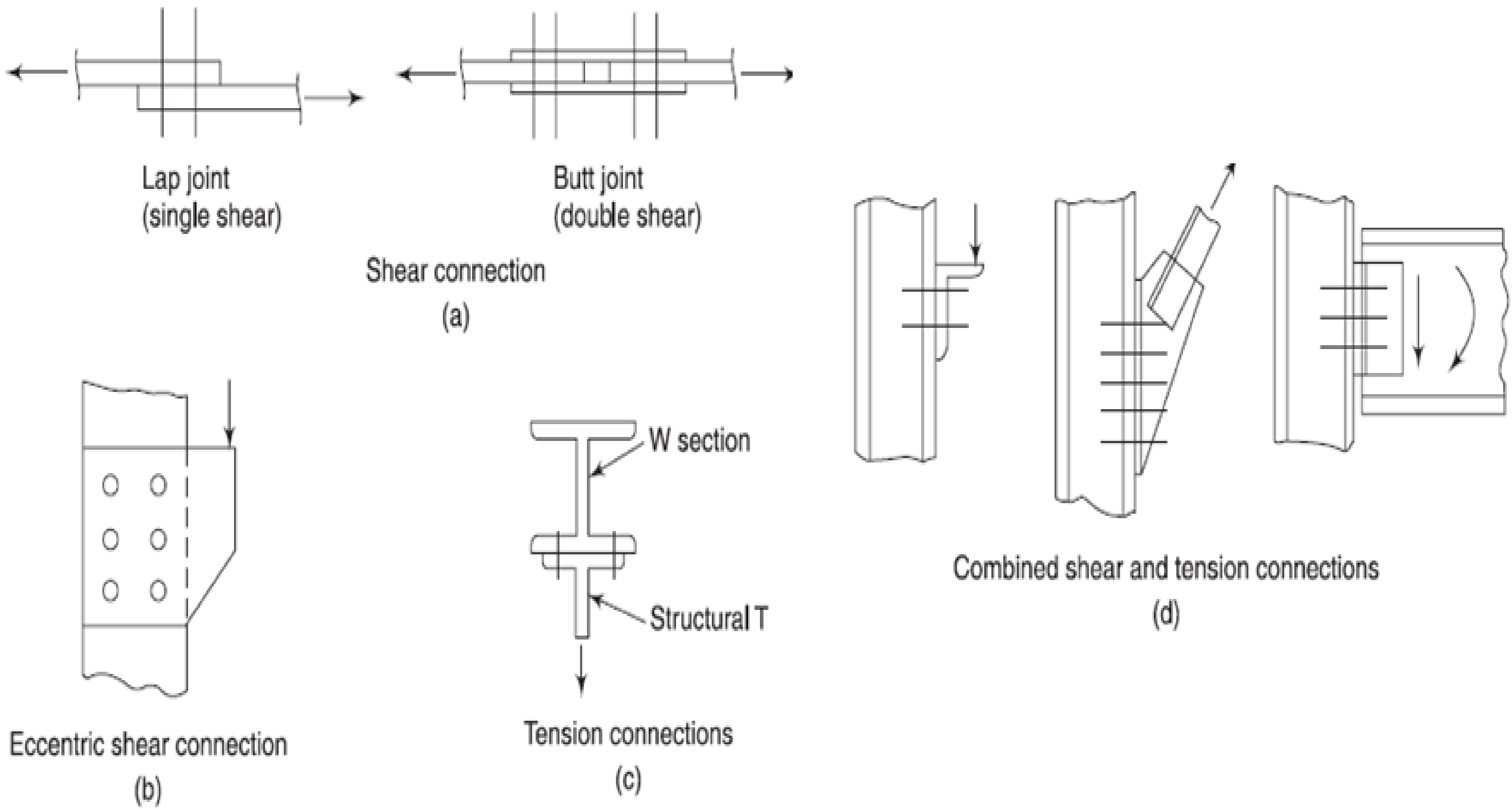
- *Shank gets bent due to tapered flange*
- *To avoid it use*

Tapered washers

(IS 5372/IS 5374)

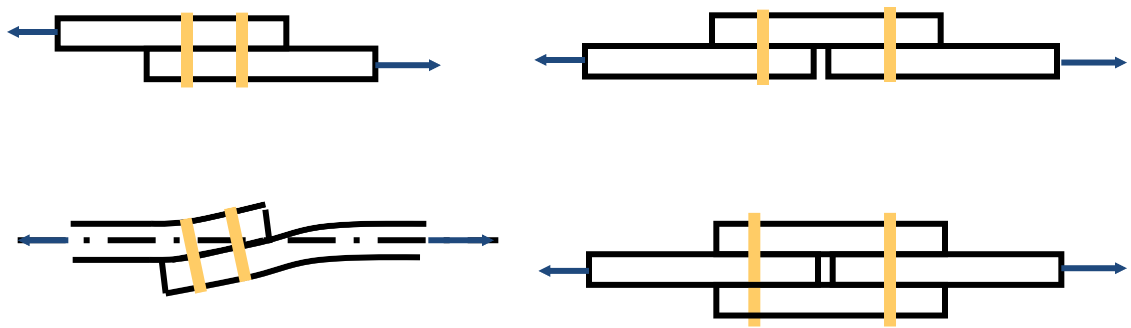


Typical Bolted Connections



TYPES OF CONNECTIONS

Classification based on type of force in the bolts



Single shear

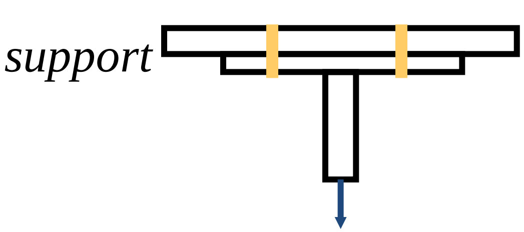
Double shear

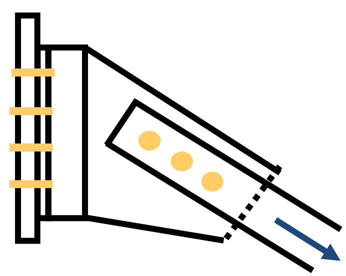
a) Lap Connection

b) Butt Connection

Shear Connections

(a)





(b)

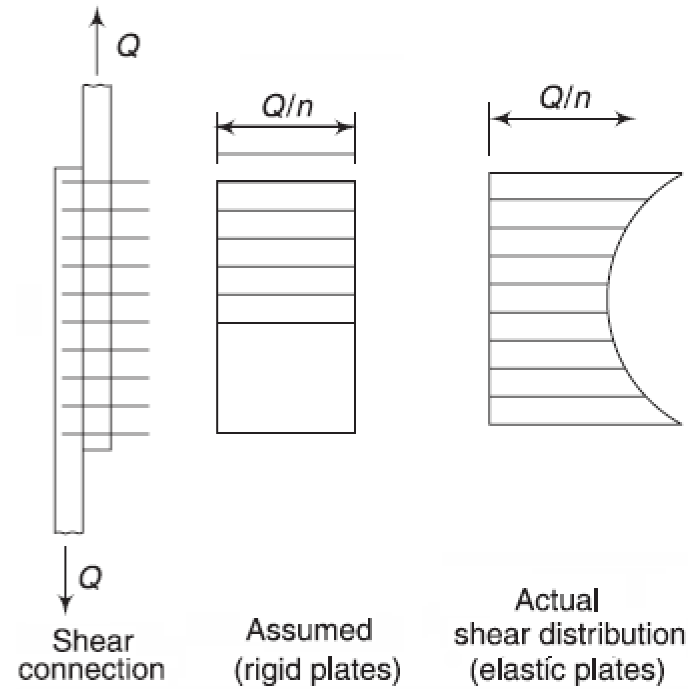
Tension Connection and Tension plus Shear Connection

Behaviour of Bolted Joints

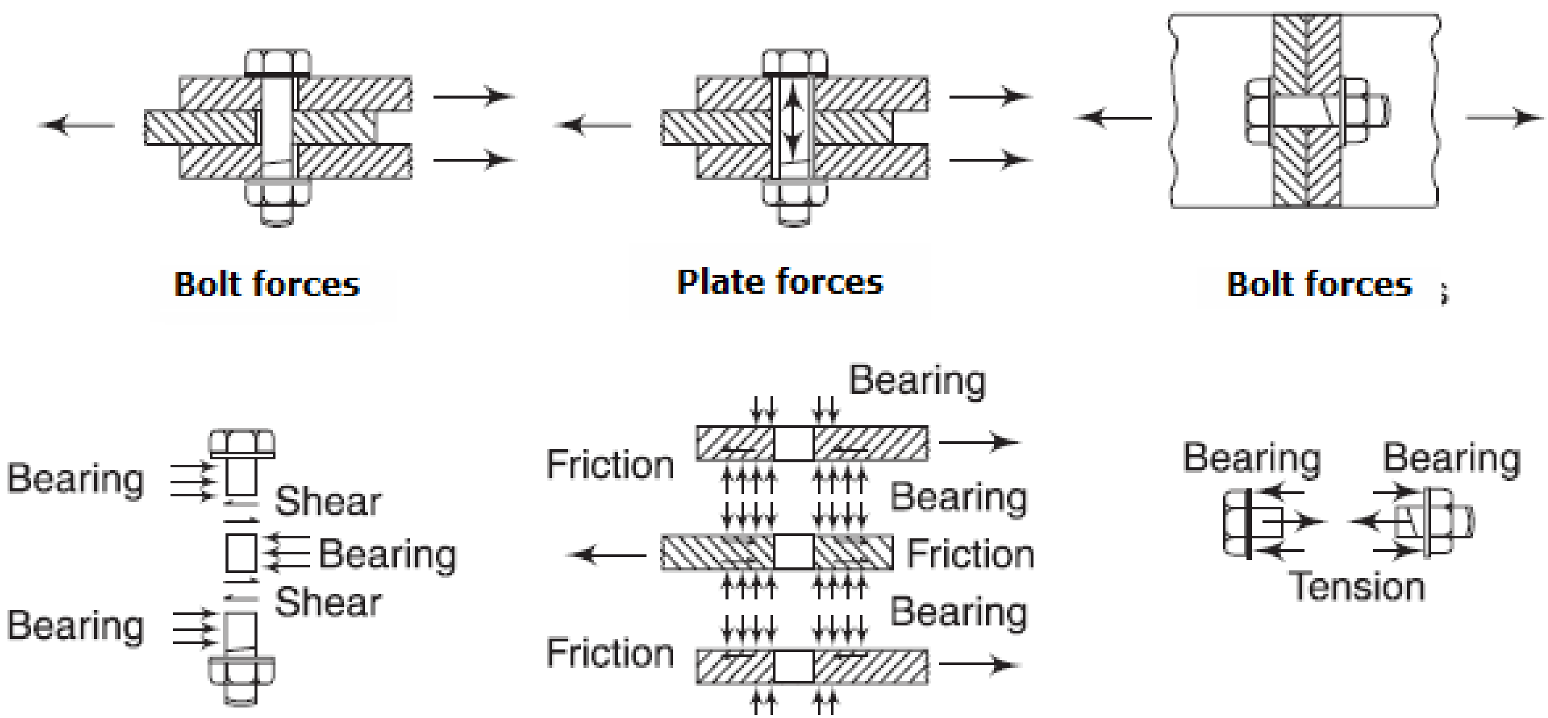
- As soon as the load is applied, there is a very small friction at the interface; slip occurs and the force is transferred from bolts to other elements through bearing of bolts.
- Once the bolts are in bearing, the connection will behave linearly, until yielding takes place at the following:
    1. At the net section of the plate(s) under combined tension and flexure.
    2. On the bolt shear plane(s)
    3. In bearing between the bolt and the side of the hole.
- The response of the connection becomes non-linear after yielding and failure takes place at one of the critical section/locations listed above.

Behaviour of Multi-Bolt Connection

- In multi-bolt connection, the behaviour is similar except that the more highly loaded bolt starts to yield first, and the connection will become less stiff.
- At a later stage, due to redistribution of forces, each bolt is loaded to its maximum capacity.
- In a long bolted connection the bolts at the end of a joint resist the highest amount of shear force.



Force Transmission Through Bolts

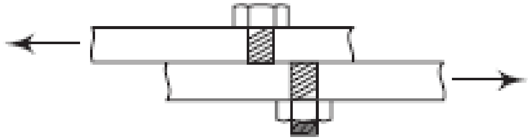


Shear and bearing joint
(a)

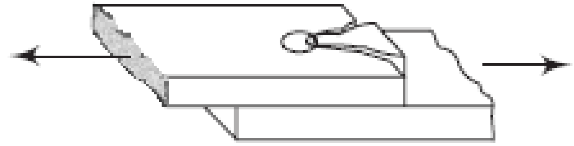
Pre-loaded friction-grip joint
(b)

Tension joint
(c)

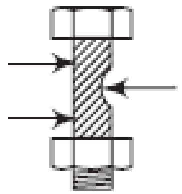
Possible Failure Modes



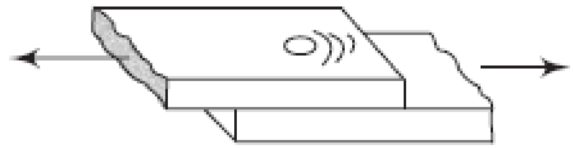
Shear failure of bolt
(a)



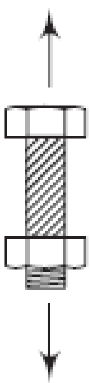
Shear failure of plate
(b)



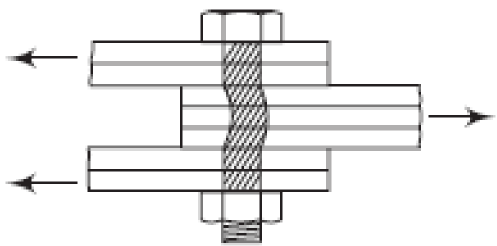
Bearing failure of bolt
(c)



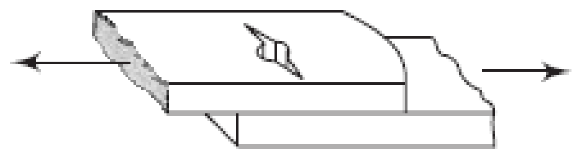
Bearing failure of plate
(d)



Tensile failure of bolts
(e)



Bending failure of bolts
(f)



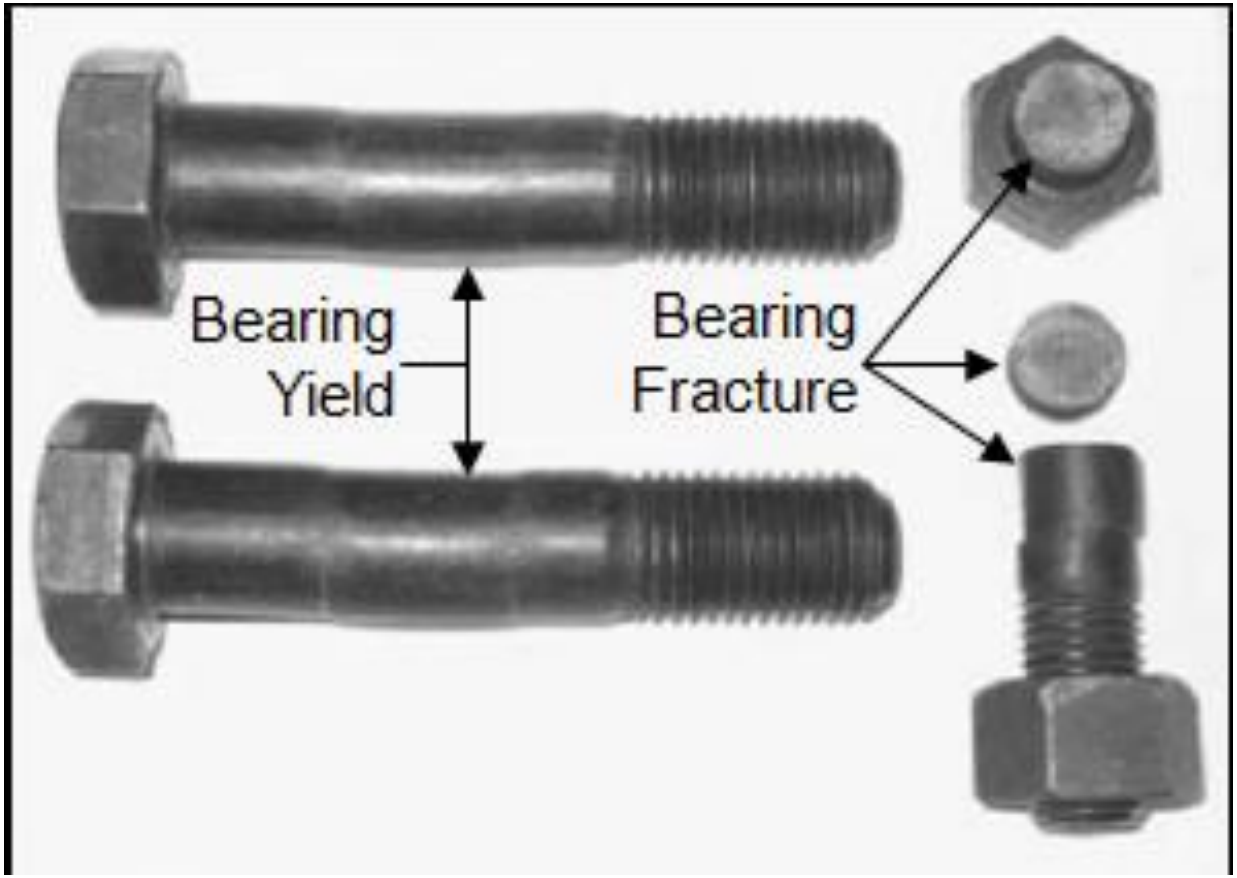
Tensile failure of plate
(g)

Possible Failure Modes

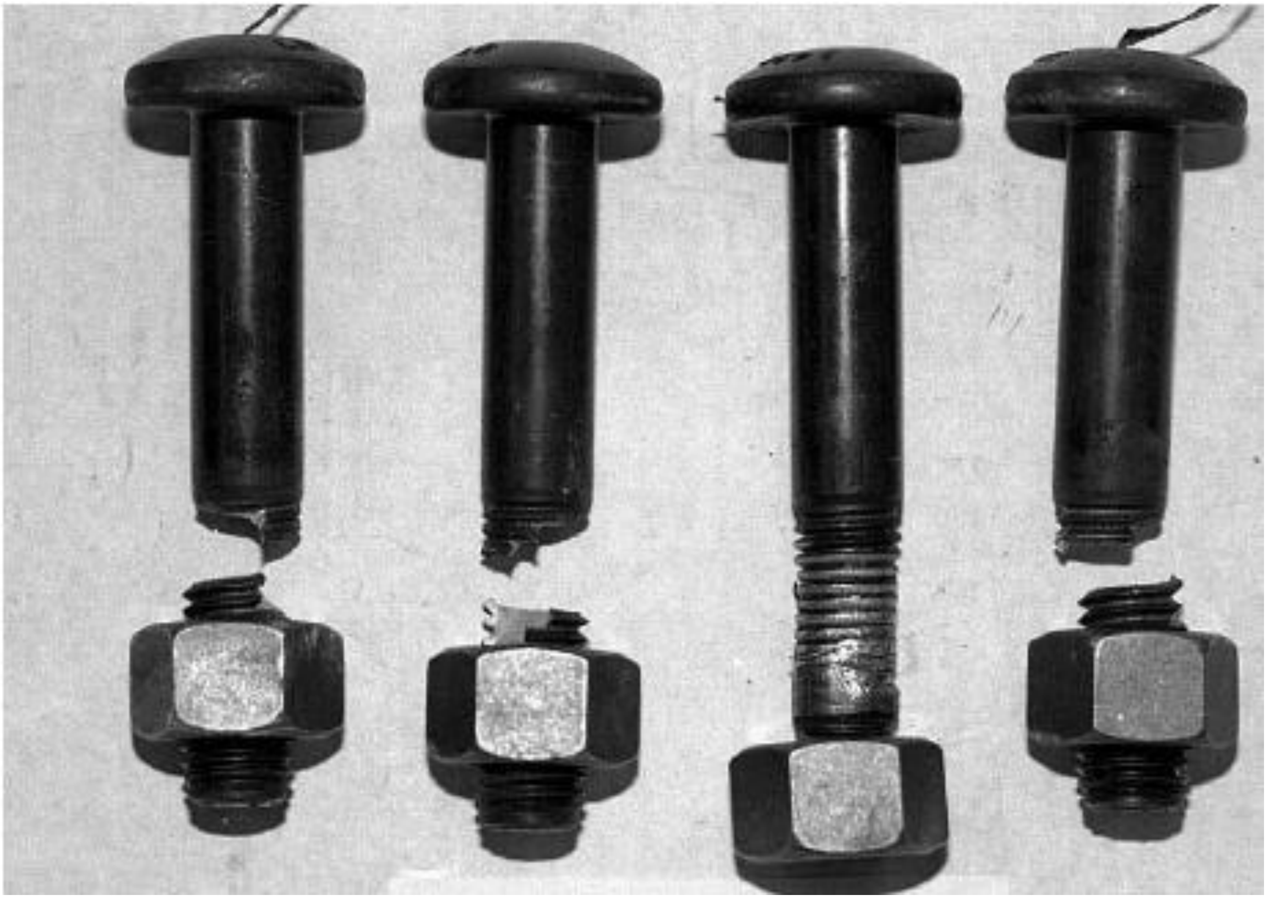
Thus any joint may fail in any one of the following modes:

- **Shear failure of bolt**
- **Shear failure of plate**
- **Bearing failure of bolt**
- **Bearing failure of plate**
- **Tensile failure of bolts**
- **Bending of bolts**
- **Tensile failure of plate**

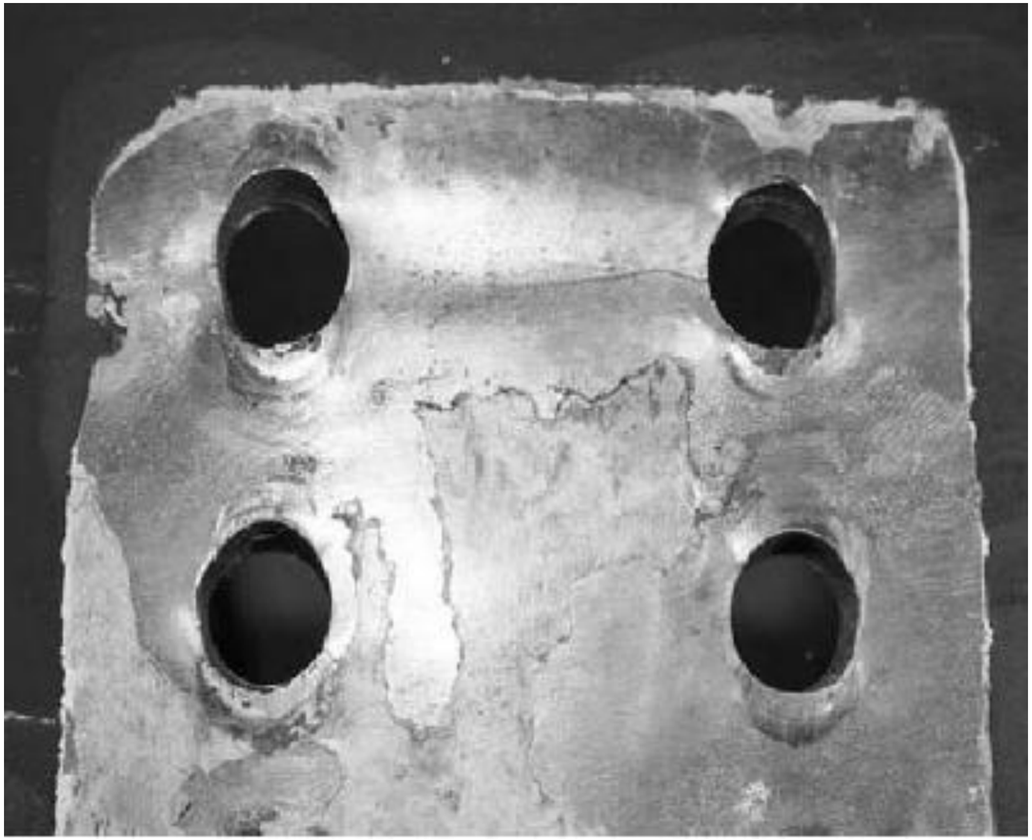
Bearing Failure of Bolt



Tension Failure of Bolts



Bearing Failure of Plates



Design Strength Of Black Bolts

- The nominal capacity, V_{nsb}, of a bolt in shear is given in the code as

$$V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

where n_n = number of shear planes with threads intercepting the shear plane,
n_s = number of shear planes without threads intercepting the shear plane,
A_{nb} =net shear are of the bolts at threads
A_{sb} =nominal plain shank area of the bolts

The factored shear force V_{dsb}

$V_{dsb} = V_{nsb} / \gamma_{mb}$ (γ_{mb} = 1.25)

Design Strength of Black Bolts

$A_{nb} = \pi / 4 (d - 0.9382p)^2 \approx 0.78 A_{sb}$

p= pitch of thread, mm

- Reduction Factor for Long Joints:

$\beta_{lj} = 1.075 - l_j (200 d)$ with $0.75 \leq \beta_{lj} \leq 1.0$

- Reduction Factor for Large Grip Length:

$\beta_{lg} = 8d / (3d + l_g); l_g \leq 8d; \beta_{lg} \leq \beta_{lj}$

- Reduction Factor for Packing plate:

$\beta_{pk} = (1-0.0125 t_{pk})$; t_{pk} is the thickness of the thicker packing plate in mm

Bolts in Tension

- The nominal capacity of a bolt in tension is:

  $T_{nb} = 0.90\ f_{ub}\ \ A_{nb} < f_{yb}\ A_{sb}\ (g_{m1}\ /\ g_{m0}\ )$

  where A_{sb} = Shank area of bolt

  A_{nb} = Net Tensile Stress area of bolt

  f_{yb} = Yield stress of the bolt

  f_{ub} = Ultimate tensile stress of the bolt

  $\gamma_{m1} = 1.25$; $\gamma_{m0} = 1.10$

- The factored tension force T_b shall satisfy

  $T_b \ \leq T_{nb}\ /\ g_{mb}$; $g_{mb} = 1.25$

If any of the connecting plates is flexible, then additional prying forces must be considered.

Bolts in Bearing

- The nominal bearing strength of the bolt is :

  $V_{npb} = 2.5\ k_b d\ t\ f_u$

f_u = Ultimate tensile stress of the plate in MPa

d = nominal diameter of the bolt in mm

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction

k_b is smaller of $e/(3d_o)$, $[p/(3d_o)-0.25]$, f_{ub}/ f_u and 1.0

where f_{ub} is the ultimate tensile stress of the bolt,

e - edge distance,

p - pitch of the fastener along bearing direction

d_o - diameter of the bolt hole.

V_{npb} should be multiplied by a factor 0.7 for over size or short slotted holes and by 0.5 for long slotted holes.

Bolts in Bearing

- The factor k_b takes into account inadequate edge distance or pitch and also prevents bearing failure of bolts.
- If we adopt a minimum edge distance of 1.5 x bolt hole diameter and a minimum pitch of 2.5 x diameter of bolt, k_b may be approximately taken as 0.50.
- The bolt bearing on any plate subjected to a factored shear force $V_{sb,}$ shall satisfy

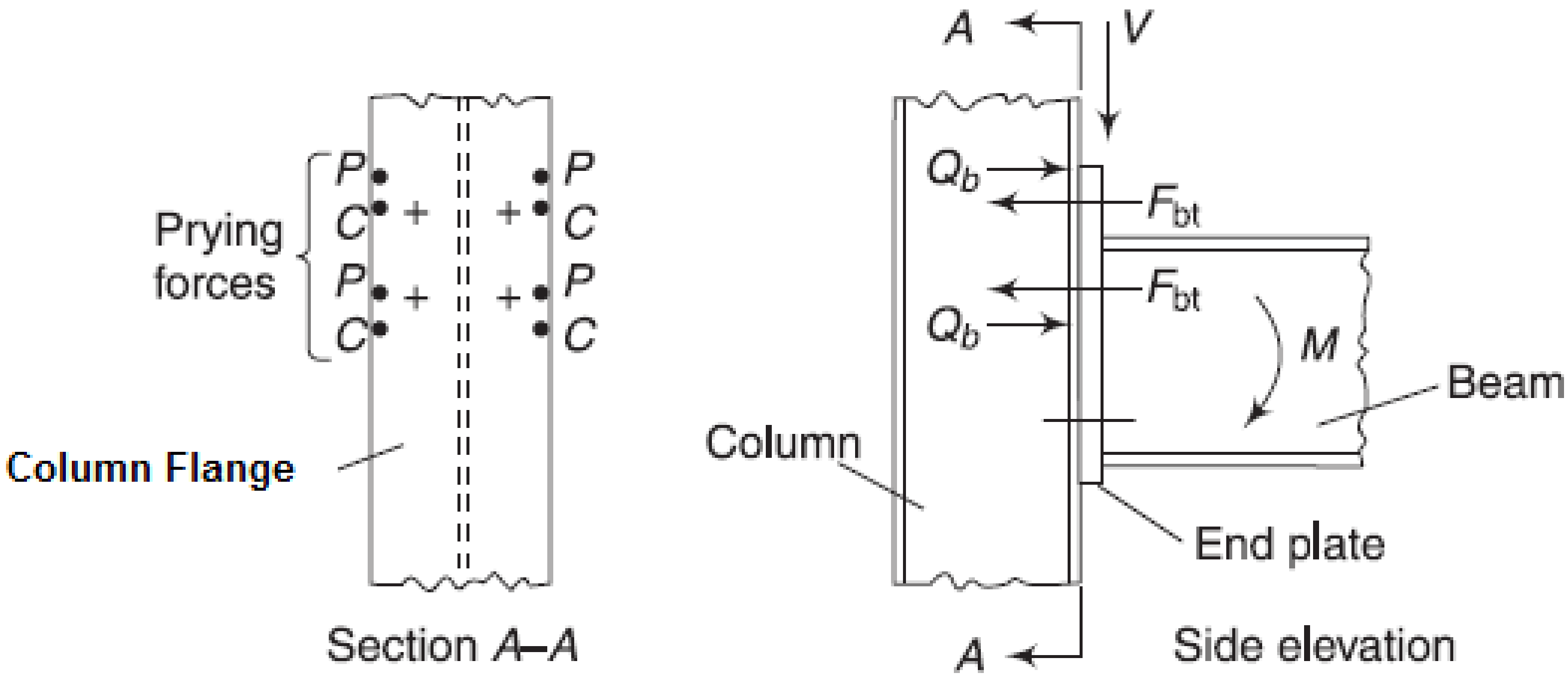
  $V_{sb} \leq V_{npb} / \gamma_{mb}$; $\gamma_{mb} = 1.25$

Capacity Of Ordinary Bolts (Grade 4.6) Based on Net Tensile Area

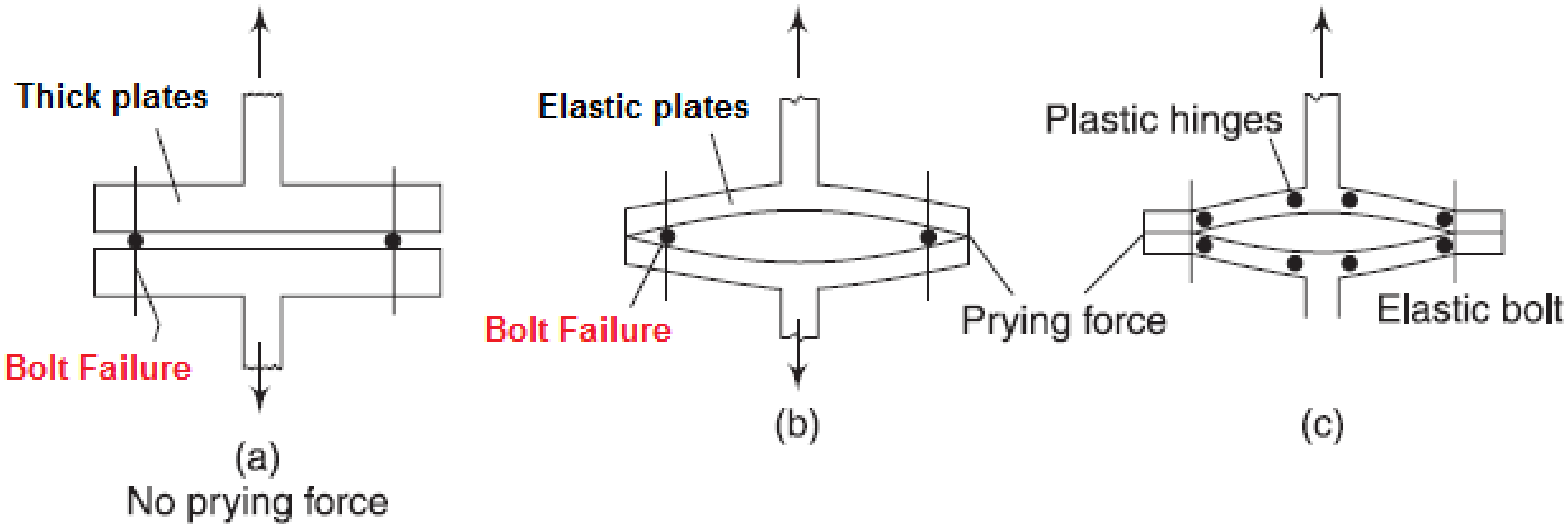
Bolt size, d(mm)	Thread stress area (mm^2)	Tension capacity, T_b(kN) $t_{nb} = 272$ MPa	Single shear capacity, V_{sb}(kN) $v_{nsb} = 185$ MPa	Minimum thickness of ply for bolt bearing $v_{npb} = 400$ MPa $t_{bb} = t_e$, mm
(12)	84.3	23.9	15.6	3.2
16	157	42.7	29.0	4.5
20	245	66.6	45.3	5.6
(22)	303	82.9	56.0	6.3
24	353	96.0	65.3	6.8
(27)	459	124.8	84.9	7.8
30	561	152.5	103.8	8.6
36	817	222.2	151.1	10.5

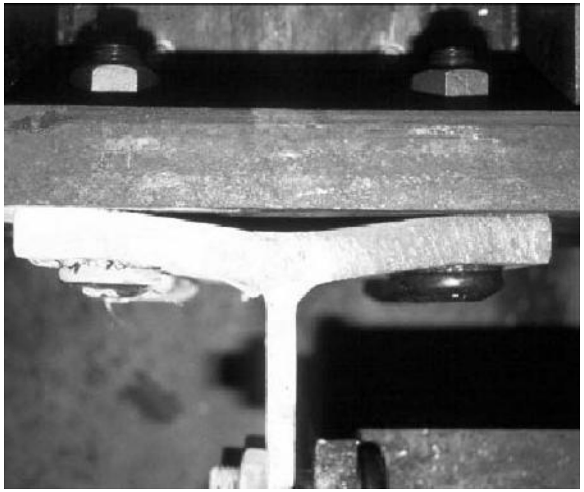
$V_{sb} = A_{nb} v_{nsb}$; $T_b = A_{nb} t_{nb}$; $t_{bb} = V_{npb}/(d v_{npb})$; Sizes in brackets are not preferred

Prying Forces in Beam-Column Connection



Failure Modes Due to Prying Forces



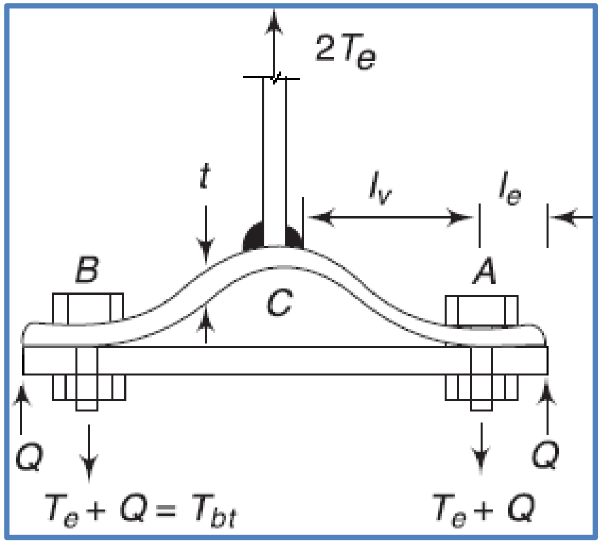


Additional Force in Bolt due to Prying

The additional force Q in the bolt due to prying action:

$$Q = \left[\frac{l_v}{2l_e}\right]\left[T_e - \frac{\beta\eta f_o b_e t^4}{27 l_e l_v^2}\right]$$

$$l_e = 1.1t\sqrt{\frac{\beta f_o}{f_y}}$$



- γ = 1.5

β= 2 for non-tensioned bolt and 1 for pre-tensioned bolt

b_e = Effective width of flange per pair of bolts, mm

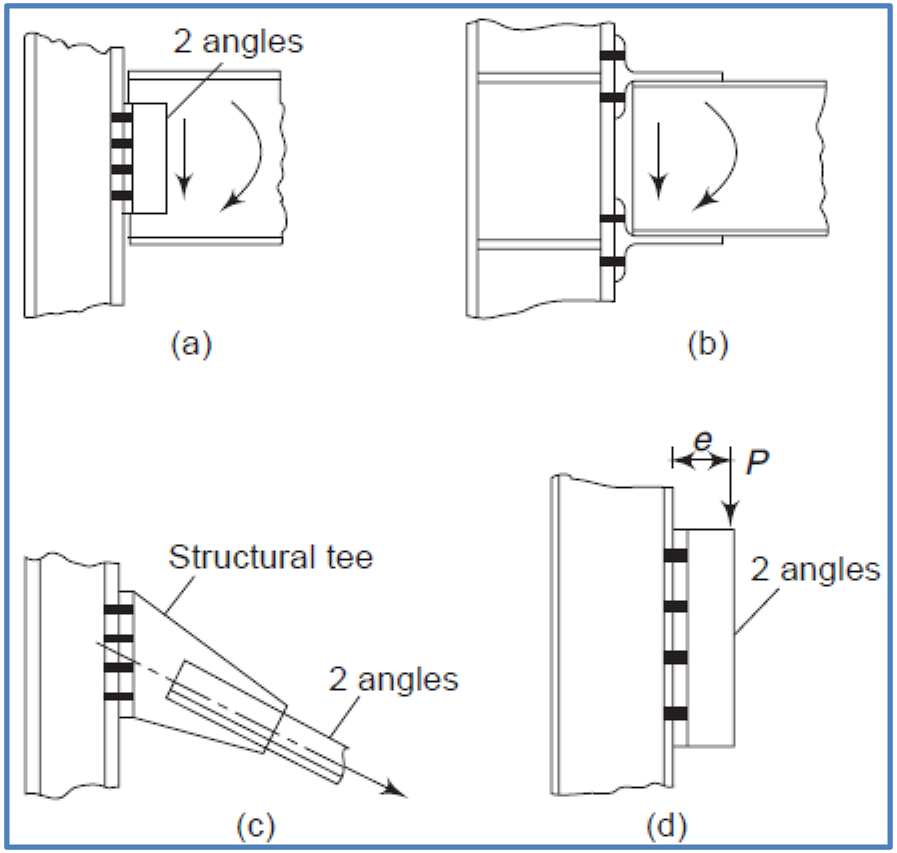
f_o = Proof stress (kN/mm^2)

Bolts With Shear and Tension

- A circular interaction curve, as per code:

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0$$

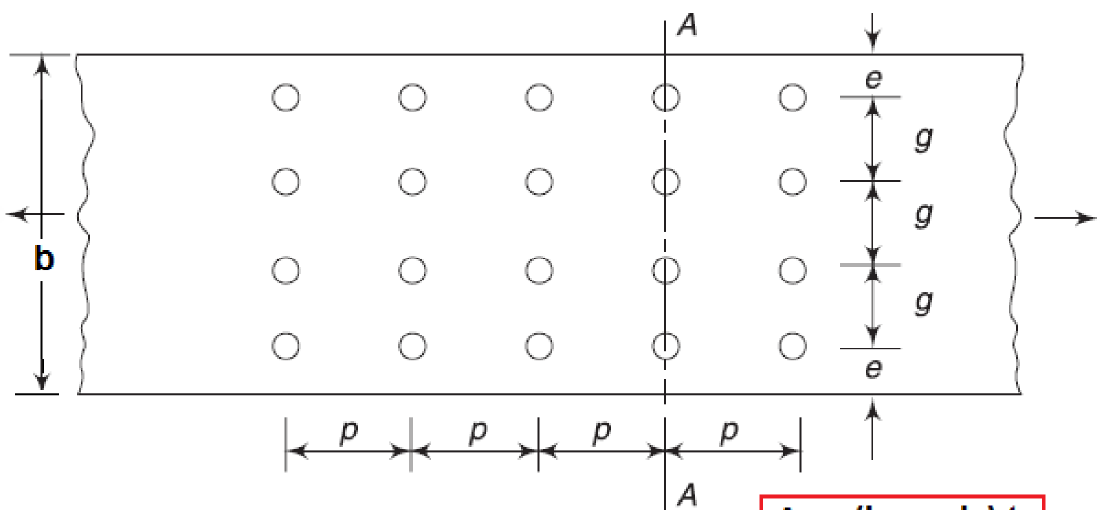
V_{sf} = Applied factored shear
V_{sd} = Design shear strength
T_f = Externally applied factored tension
T_{nd} = Design tension strength



Tension Capacity of Plate

Tension Capacity of plate:

$T_{dn} = 0.9\ f_u A_n / \gamma_{m1};\ \gamma_{m1} = 1.25$



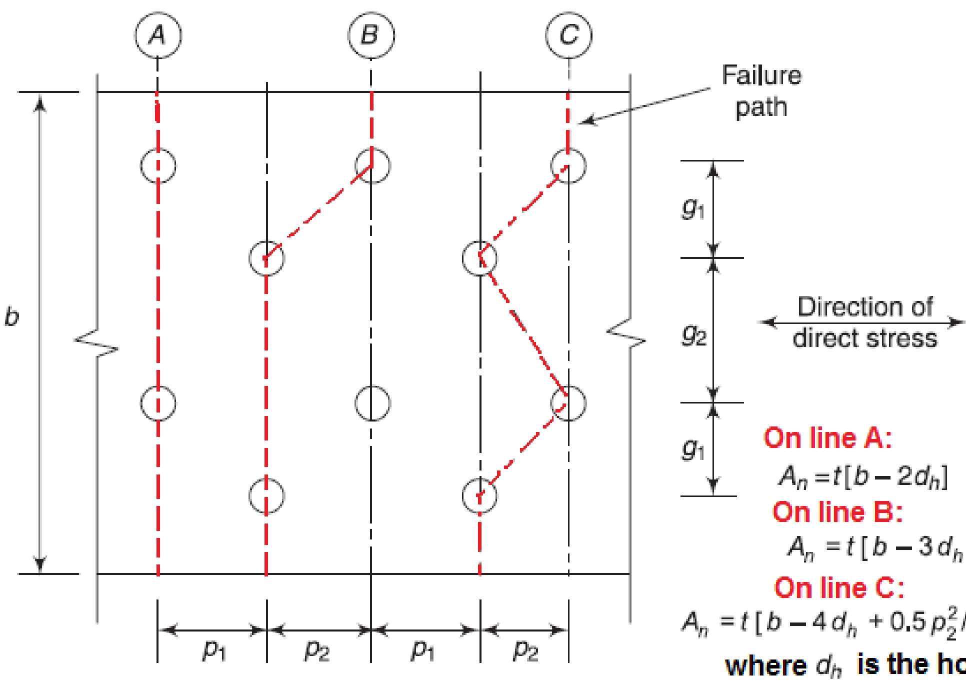
Chain of holes in rows

$\boxed{A_n = (b - n\ d_h)\ t}$

Tension Capacity of Plate- Staggered Holes

$T_{dn} = 0.9\, f_u A_n / \gamma_{m1}$

$$A_n = [b - n d_h + \sum_{i=1}^{m} p_i^2 / 4 g_i] t$$



On line A:

$A_n = t[b - 2d_h]$

On line B:

$A_n = t[b - 3d_h + 0.25 p_2^2/g_1]$

On line C:

$A_n = t[b - 4d_h + 0.5 p_2^2/g_1 + 0.25 p_2^2/g_2]$

where d_h is the hole diameter

Design Strength of HSFG Bolts

The design slip resistance or nominal shear capacity of a bolt:

$$V_{nsf} = \mu_f \, n_e \, K_h \, F_o \qquad V_{dsf} = V_{nsf} / \gamma_{mf}$$

μ = Coefficient of friction (called as slip factor) ≤0.55.

n_e = Number of effective interfaces

K_h = 1.0 for fasteners in clearance holes

= 0.85 for fasteners in oversized and short slotted holes

= 0.7 for fasteners in long slotted holes loaded parallel to the slot

F_o = Minimum bolt tension (proof load) at installation ≈ $A_{nb} f_o$

A_{sb} = Nominal shank area of bolt

f_o = Proof stress ≈ 0.7 f_{ub}

f_{ub} = Ultimate tensile stress of bolt

The factored design force V_{sf}, should satisfy:

$$V_{sf} \le V_{dsf}$$

γ_{mf} = 1.10 if slip resistance is designed at service load

γ_{mf} = 1.25 if slip resistance is designed at ultimate load.

Coefficient of Friction

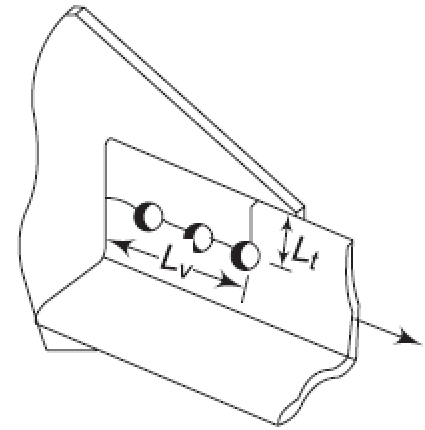
Treatment of surface	Coefficient of friction (μ_f)	Treatment of surface	Coefficient of friction (μ_f)
Surfaces not treated	0.20	Surfaces blasted with shot or grit and painted with ethylzinc silicate coat (thickness 60–80 μm)	0.30
Surfaces blasted with shot or grit with any loose rust removed, no pitting	0.50	Surfaces blasted with shot or grit and painted with alkalizinc silicate coat (thickness 60–80 μm)	0.30
Surfaces blasted with shot or grit and hot-dip galvanized or red lead painted surface	0.10	Surface blasted with shot or grit and spray-metallized with aluminium (thickness > 50 μm)	0.50
Surfaces blasted with shot or grit and spray-metallized with zinc (thickness 50–70 μm)	0.25	Clean mill scale	0.33

Block Shear Strength

It is taken as the smaller of :

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 f_u A_{tn}}{\gamma_{m1}}$$

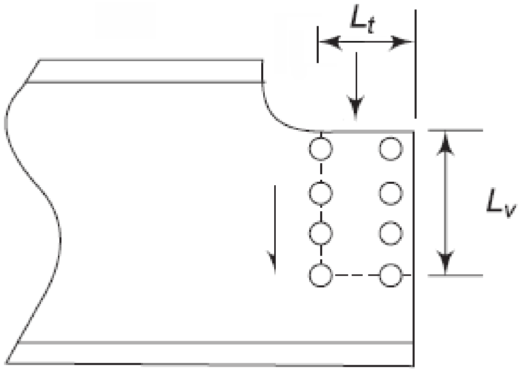
$$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{f_y A_{tg}}{\gamma_{m0}}$$



A_{vg}, A_{vn} = minimum gross and net area in shear along a line of transmitted force(along L_v)

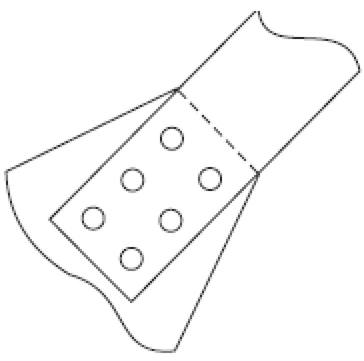
A_{tg},A_{tn} = minimum gross and net area in tension from the hole to the toe of the angle or next last row of bolt in

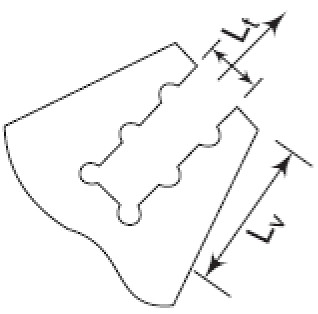
gusset plates (along L_t)



f_u,f_y = ultimate and yield stress of the material respectively

γ_{m0} = 1.10; γ_{m1} = 1.25





Typical Block Shear Failure

Capacities of HSFG Bolts

Bolt diameter (mm)	Stress area of bolt (mm^2)		Proof load of bolt (kN)		Tensile capacity (kN)		Slip resistance in single shear ($n_e = 1$, $\gamma_{mf} = 1.25$) (kN)	
	Thread	Shank	Property class					
			8.8	10.9	8.8	10.9	8.8	10.9
(12)	84.3	113	47.2	61.3	48.5	63	18.1	23.5
16	157	201	87	114	90.4	117	33.7	43.8
20	245	314	151	178	141	183	52.6	68.4
(22)	303	380	169	220	174	226	65.1	84.7
24	353	452	197	256	203	264	75.9	98.6
(27)	459	572	257	334	264	343	98.7	128
30	561	706	314	408	323	420	120	156
36	817	1017	457	595	470	611	175	228

Tension capacity = $0.9 f_u A_{nb}/1.25$

Slip resistance = $\mu_f n_e K_h F_o / \gamma_{mf}$

where $\gamma_{mf} = 1.1$ (at service load) and 1.25 (at ultimate load), F_0 is the minimum bolt tension (proof load) $= A_{nb} f_o$; $f_o = 0.7 f_{ub}$, and n_e is the number of effective interfaces offering frictional resistance to slip.

Connection with HSFG Bolts

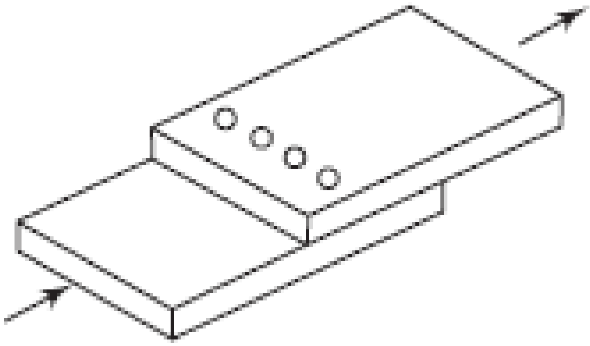
Pin Connections

Simple Connections

Connections may be classified as:

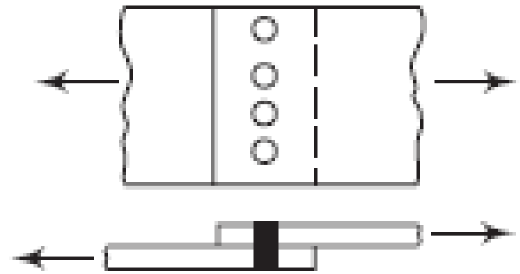
- Lap and butt joints
- Truss joint connections
- Connections at beam-column junctions
  - Seat angle connection
  - Web angle connection
  - Stiffened seat angle connection
  - Header plate connection
- Tension and flange splices

Lap Joints



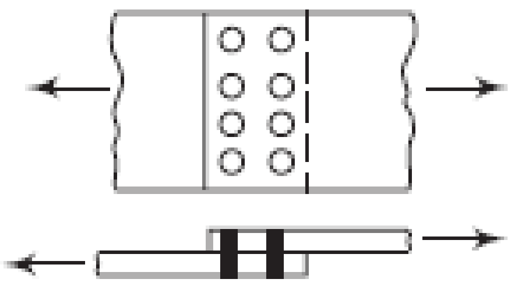
Lap joint

(a)



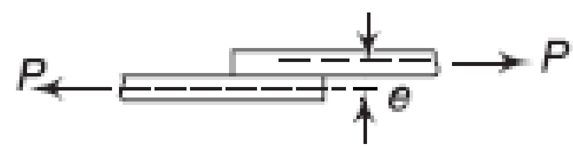
Single bolted lap joint

(b)



Double bolted lap joint

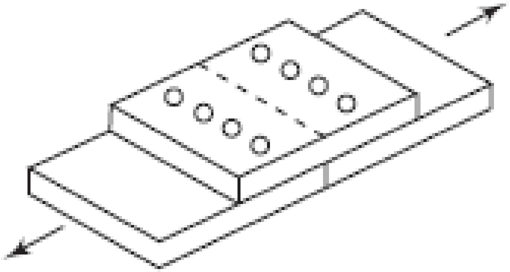
(c)



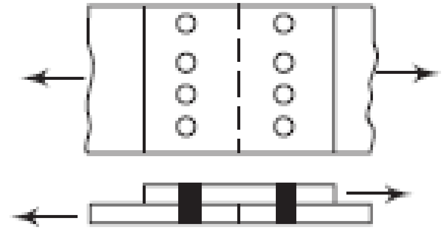
Eccentricity in lap joint

(d)

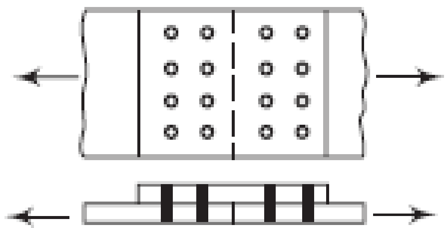
Butt Joints



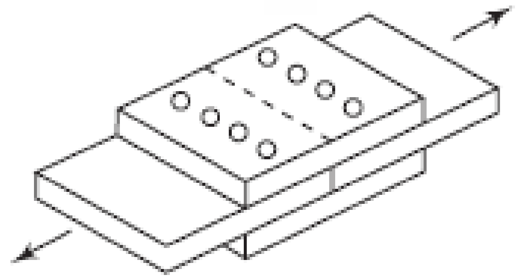
Single cover



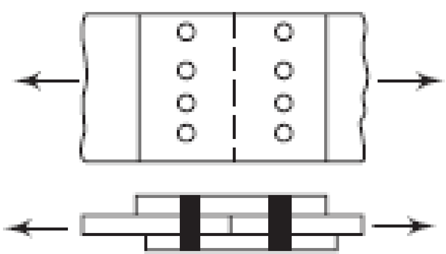
Single bolted single cover



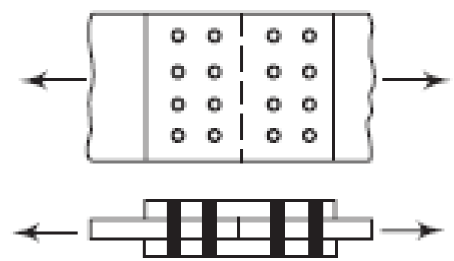
Double bolted, single cover



Double cover

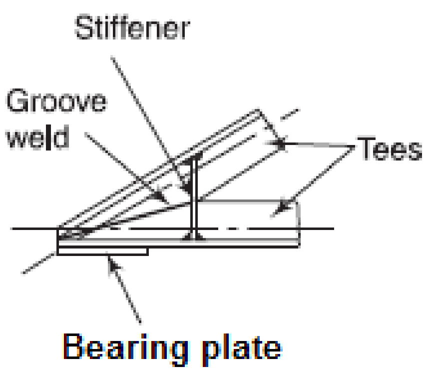


Single bolted, double cover

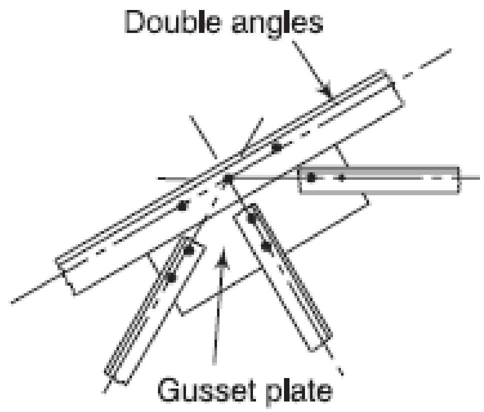


Double bolted, double cover

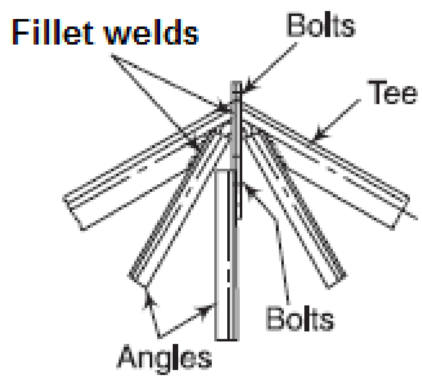
Typical Truss Connections



Groove welded joint between Ts
(a)



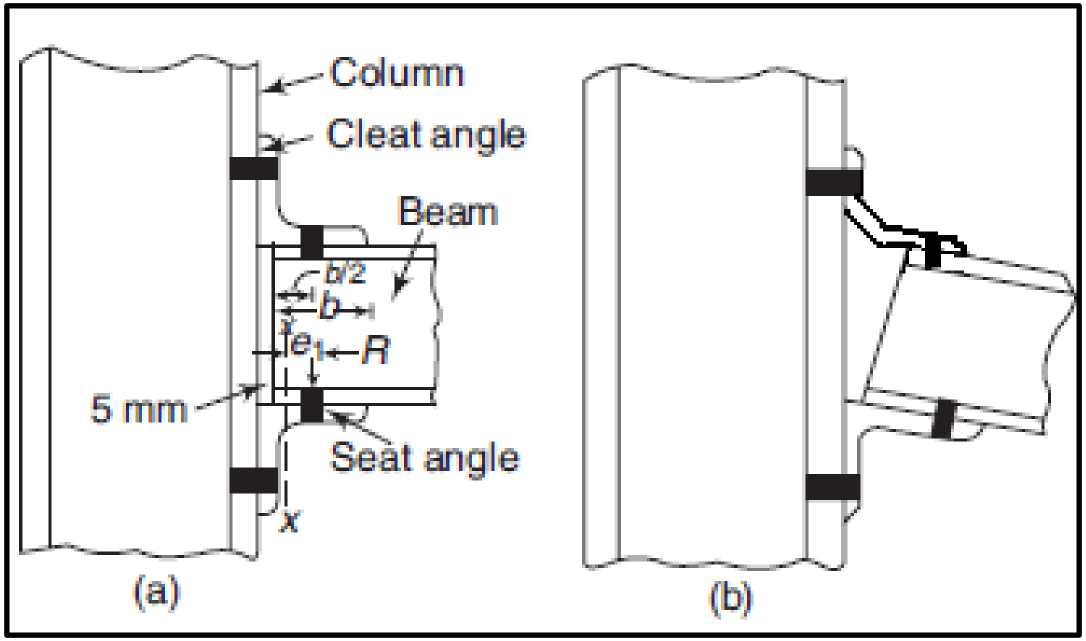
Bolted joint with double angles
(b)

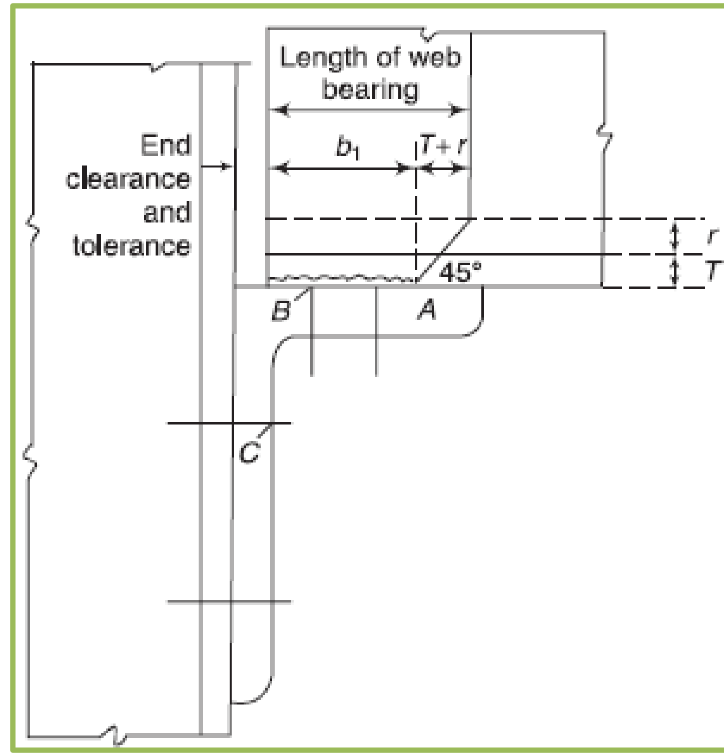


Welded and bolted joint
(c)

Block shear model may be used to predict the ultimate capacity of gusset plate connections in tension.
Local buckling **may be prevented , by restricting the unsupported edge of a gusset plate to 42ε times the thickness, where**
$\varepsilon = (250 / f_y)^{0.5}$.

Clip and Seating Angle Connections





Also called seat-angle connection.

Minimum length of bearing at edge of root radius
= Reaction / (web thickness x design strength of web)

Design of Unstiffened Seating Angle Connection

The design consist of the following steps:

- Select a seat angle having a length equal to the width of the beam.
- Keep the length of seat more than the bearing length given by

  $$b = [R / t_w (f_{yw} / \gamma_{mo})]$$

  where R = reaction from beam
- A dispersion of 45^0 is taken from the bearing on the cleat to the root line. Length of bearing on cleat,

  $$b_1 = b - (T_f + r_b)$$

  r_b, T_f = root radius and thickness of beam flange

Design of Unstiffened Seating Angle Connection (cont.)

- Distance of end bearing on cleat to root angle
  $b_2 = b_1 + g – (t_a + r_a)$

- Select an angle with connect leg > 100mm. The bending moment,
  $$M_u = R\ (b_2 / b_1) \times (b_2 / 2)$$

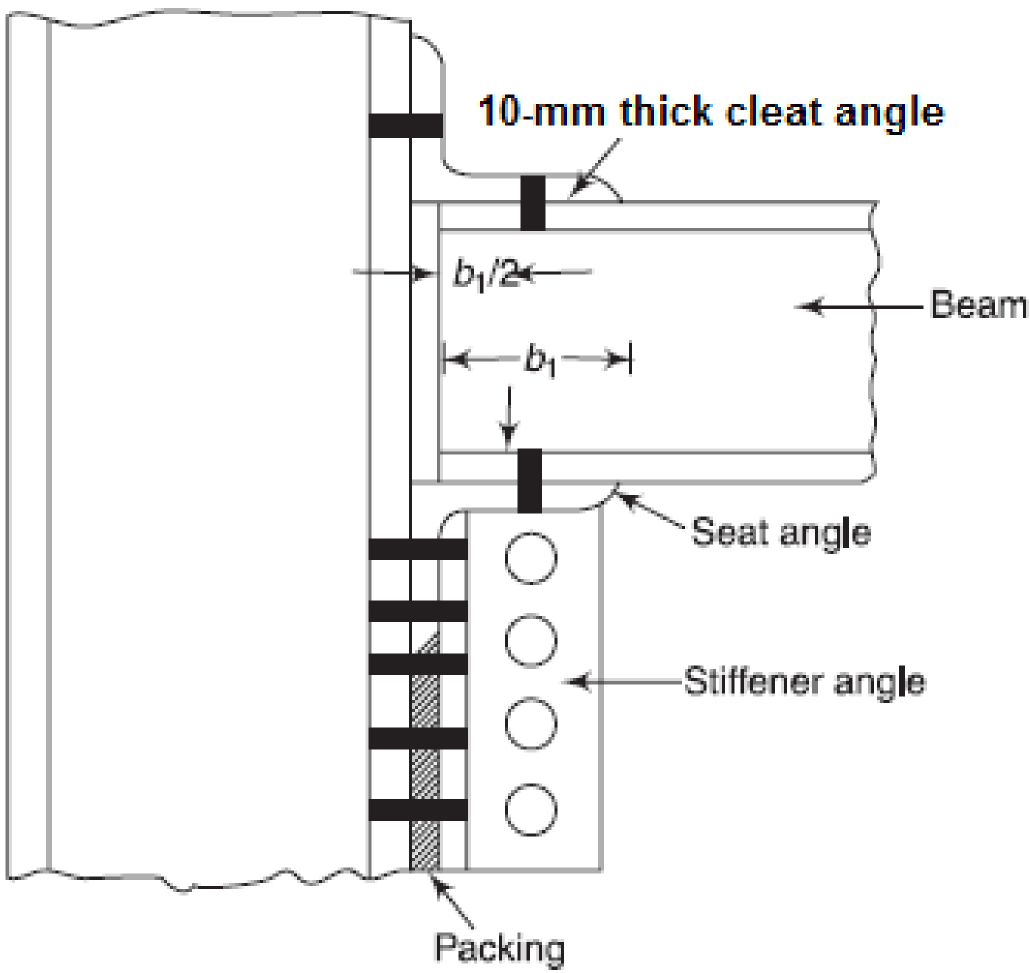
Equate it against the moment capacity
$M_d = 1.2\ Z\ (f_y / \gamma_{mo})$

When $M_d < M_u$, revise the section.

- The shear capacity of the outstanding leg of cleat is calculated as
  $V_{dp} = w\ t\ f_y / (\sqrt{3}\ \gamma_{mo})$; should be >R

- Calculate no. of bolts; Also choose a nominal top cleat angle

Stiffened Seat Angle



Design of Stiffened Seat Connection

- Assume the size of seat angle on the basis of bearing length similar to unstiffened seat connection.
- The outstanding leg must not exceed 14ε times the thickness, where $\varepsilon = (250 / f_y)^{0.5}$ to avoid local buckling). The required bearing area is calculated as

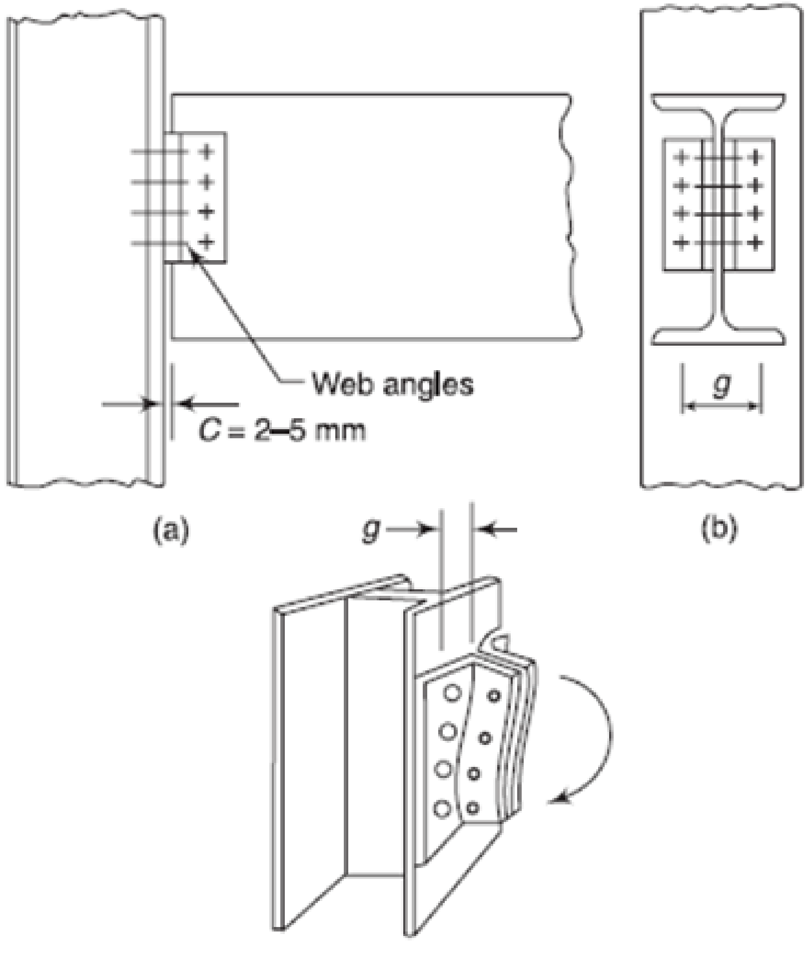
$$A_{br} = R / (f_y / \gamma_{mo})$$

- Choose the thickness of the stiffener angle >thickness of the web of the beam.

Design of Stiffened Seat Connection (cont.)

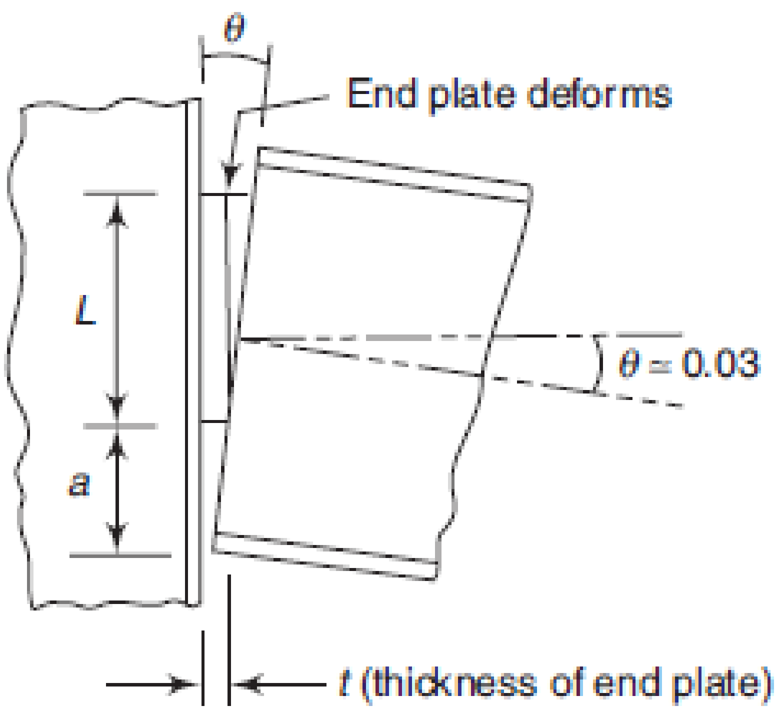
- Assume that the reaction from the beam acts at the middle of the outstanding leg of angle.
- Compute the eccentricity, B.M., and tension acting in critical bolts, similar to the bracket connection.
- Check the critical bolt using the interaction formula.
- Provide nominal angle at the top of the beam & connect with two nominal size bolts on each leg of the cleat angle.

Web Angle Connection



- **Double web cleat connection is preferred over single sided web cleat connection.**
- **The beam reaction is transferred by shear and bearing from the web of the beam to the web bolts and to the angle cleats.**
- **These are then transferred by the cleat angle to the bolts at the junction of supporting member.**
- **Then to the supporting member mainly by shear and also by tension and compression.**
- **The beam is designed as a simply supported beam**

Flexible End Plate Connection



❖**This connection behaviour is similar to the legs of web angles connected to the column flange.**

❖**Limit the thickness of the plate and position the bolts not too close to the web and flange of the beam.**

❖ **Keep the length 'a' < 30t**

❖ **Design the beam for zero end moment.**

❖ **Design the column for the eccentric beam reaction.**

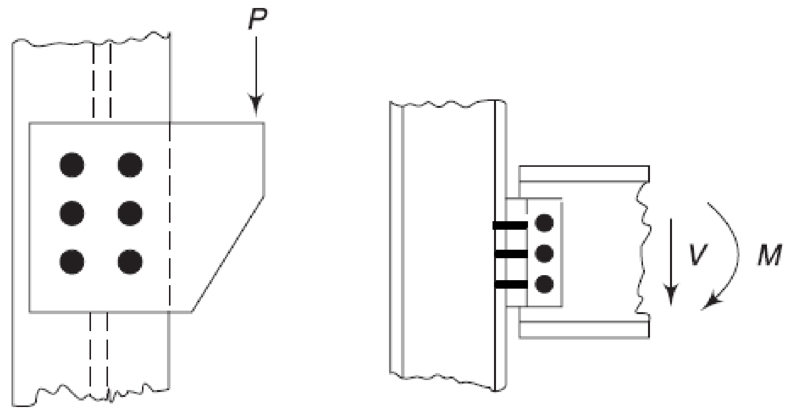
✓**The reaction is transferred**

- ✓**By weld shear to the end plate,**
- ✓ **by shear and bearing to the bolts,**
- ✓**by shear and bearing to the supporting member.**

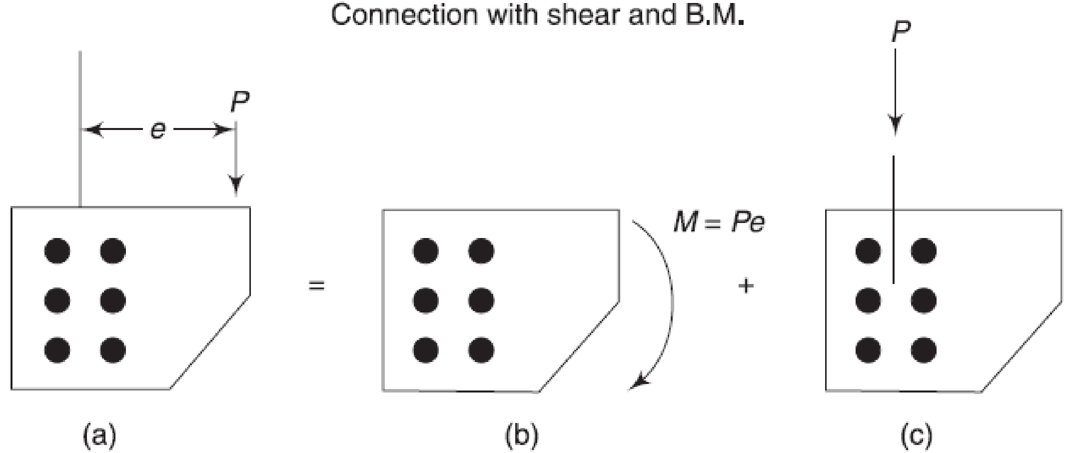
Moment Resistant Connections

- Used in framed structures, where the joints are considered rigid.
- Classified as
  - Eccentrically loaded connections
    - **Type I (Ecc. Load causing Twisting)**
    - **Type II (Ecc. Load causing BM)**
  - Tee stub connections and
  - Flange angle connections.

Eccentric Shear in Connections



Connection with shear and B.M.

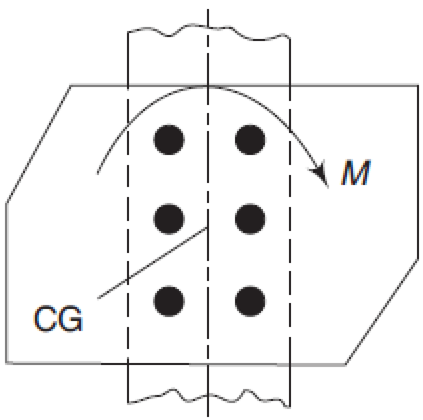


Replacement of eccentric shear with a moment and direct shear.

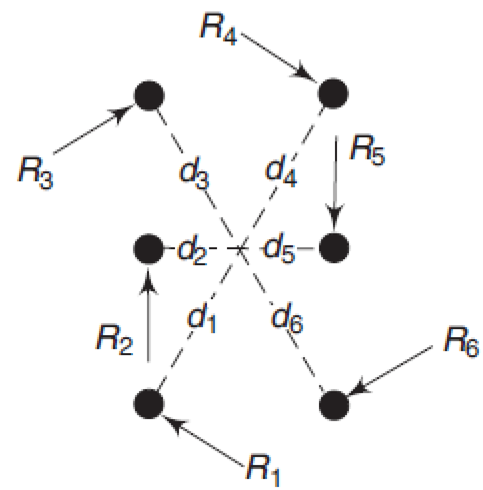
Ecc. Shear Causing Twisting

- *Elastic Analysis- assumptions*
- Deformation of the connected parts may be ignored.
- The relative movement of the connected parts are considered as the relative rigid body rotation of the two parts about some centre of rotation.
- There is friction between the ‘rigid’ plates and the elastic fasteners.
- The deformation induces reactive bolt forces- tangential to the centre of rotation.
- This elastic method yields conservative results.

Elastic Vector Analysis



Connection
(a)



Forces on connectors
(b)

Rotation Effect

$$R = \frac{M\, d_i}{\Sigma\, d_i^{\,2}}$$

Based on shaft torsion analogy

Direct Shear

$$R_v = \frac{P}{n}$$

Elastic Vector Analysis (cont.)

- In General, we have:

$$R_x = \frac{My}{\sum (x^2 + y^2)} \qquad R_y = \frac{Mx}{\sum (x^2 + y^2)} \qquad R_v = \frac{P}{n}$$

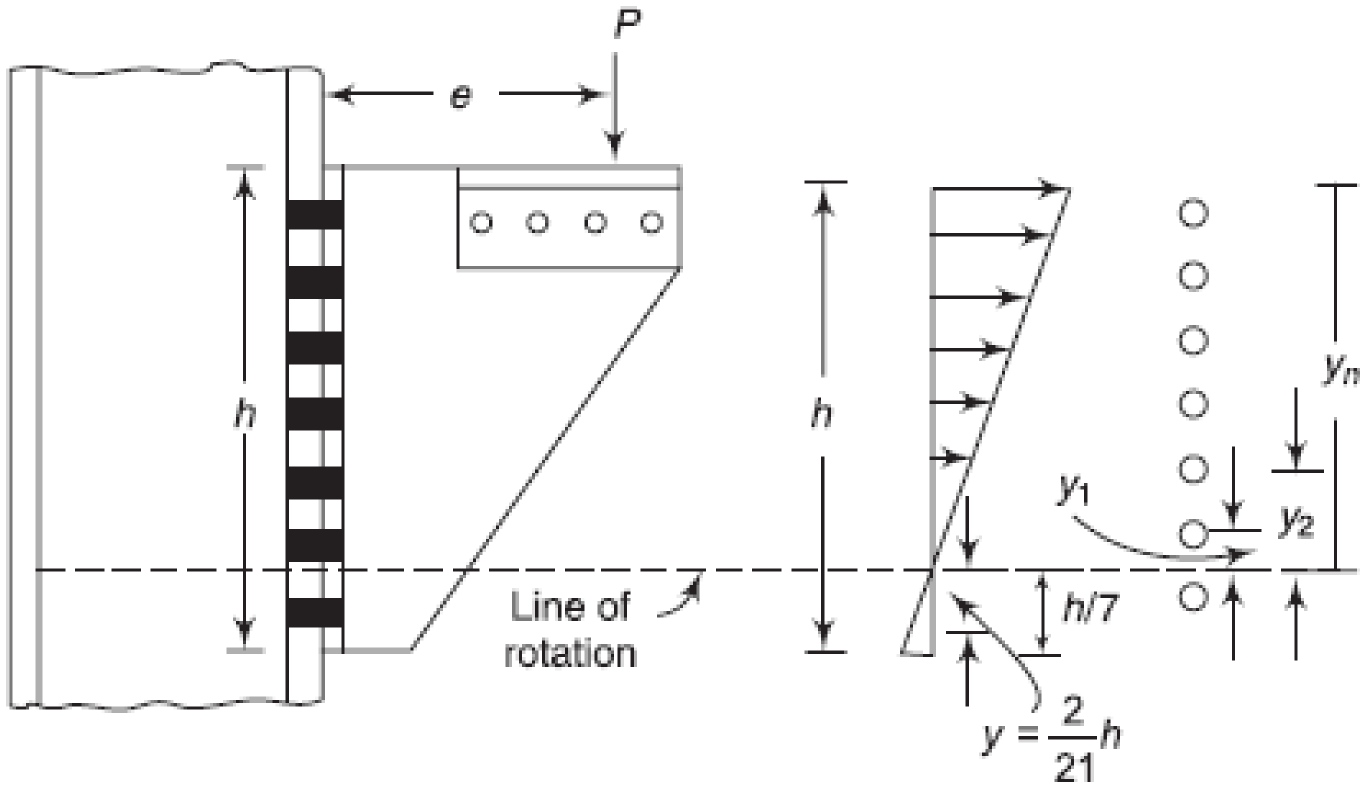
x and y are Horizontal and vertical distance of 'd'
n= number of bolts

Resultant force in Bolt:

$$R = \sqrt{(R_y + R_v)^2 + R_x^2}$$

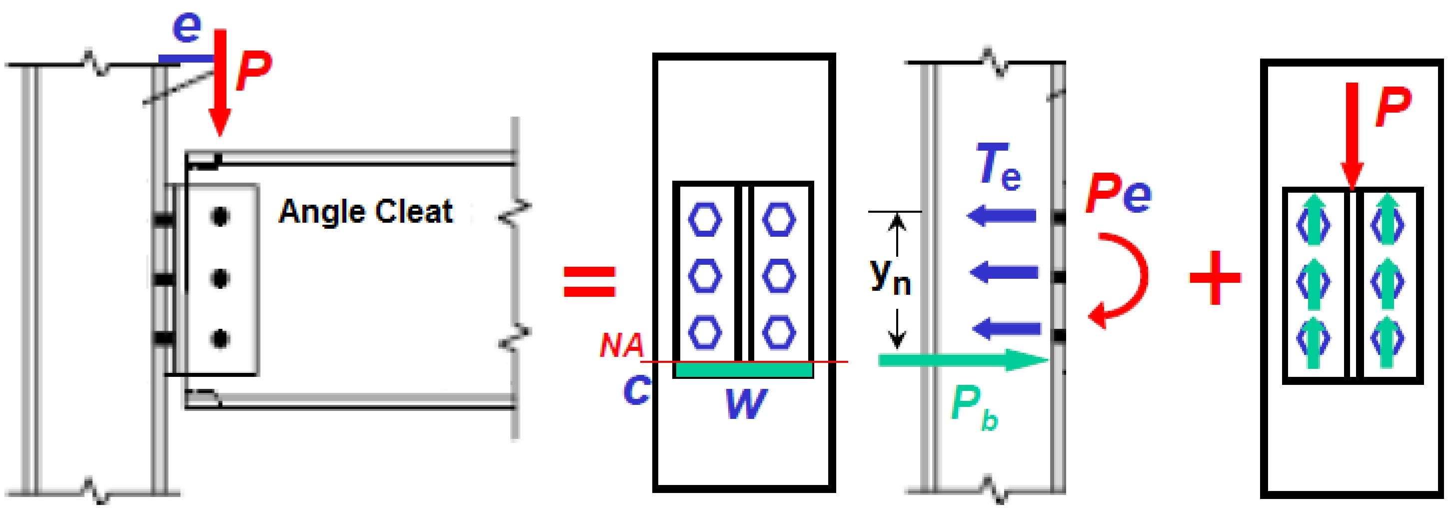
The above formula can be extended to a case having vertical and horizontal loads.

Bracket-Type II Connection



The bolts are subjected to direct shear along with tension due to the moment.

Bracket-Type II Connection (cont.)



$M^* = Pe$

Check:

$$\left(\frac{V}{V_{nd}}\right)^2 + \left(\frac{T_e}{T_{nd}}\right)^2 \leq 1.0$$

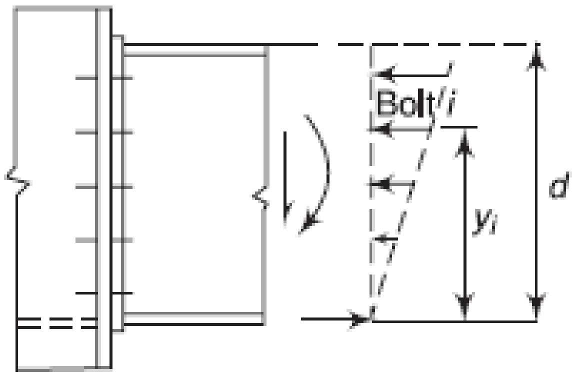
Tensile force in extreme critical bolt
Assume NA below the last bolt

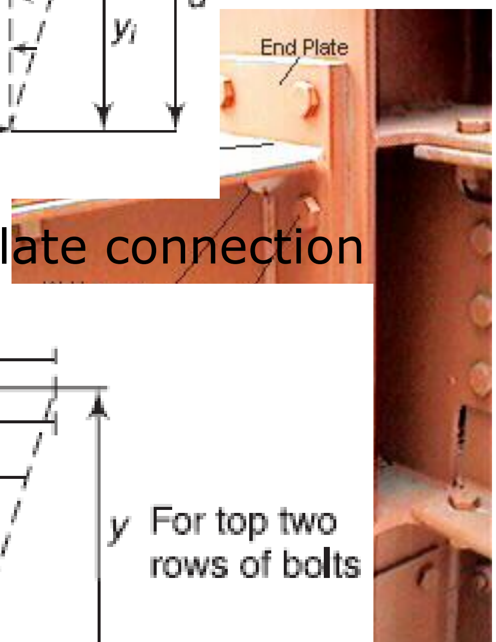
$$T_e = \frac{M^* y_n}{\sum y_i^2}$$

Direct Shear

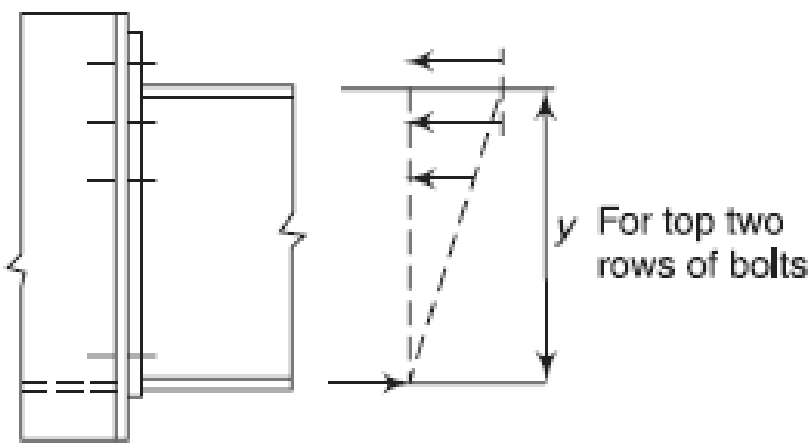
$$V = \frac{P}{n}$$

End-Plate connections



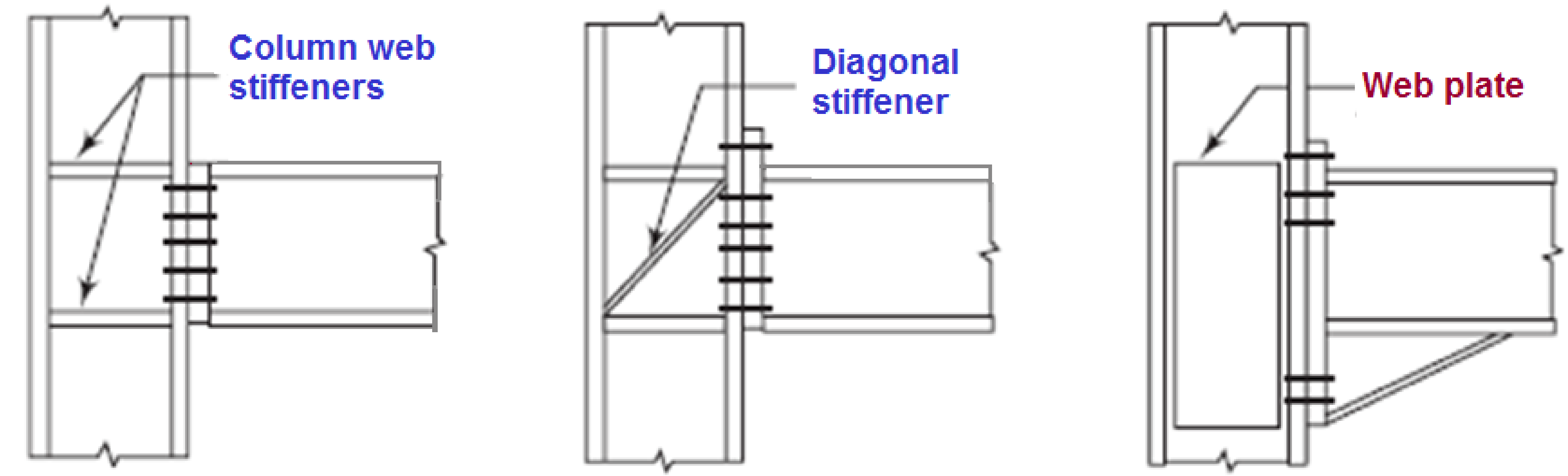


End- plate connection



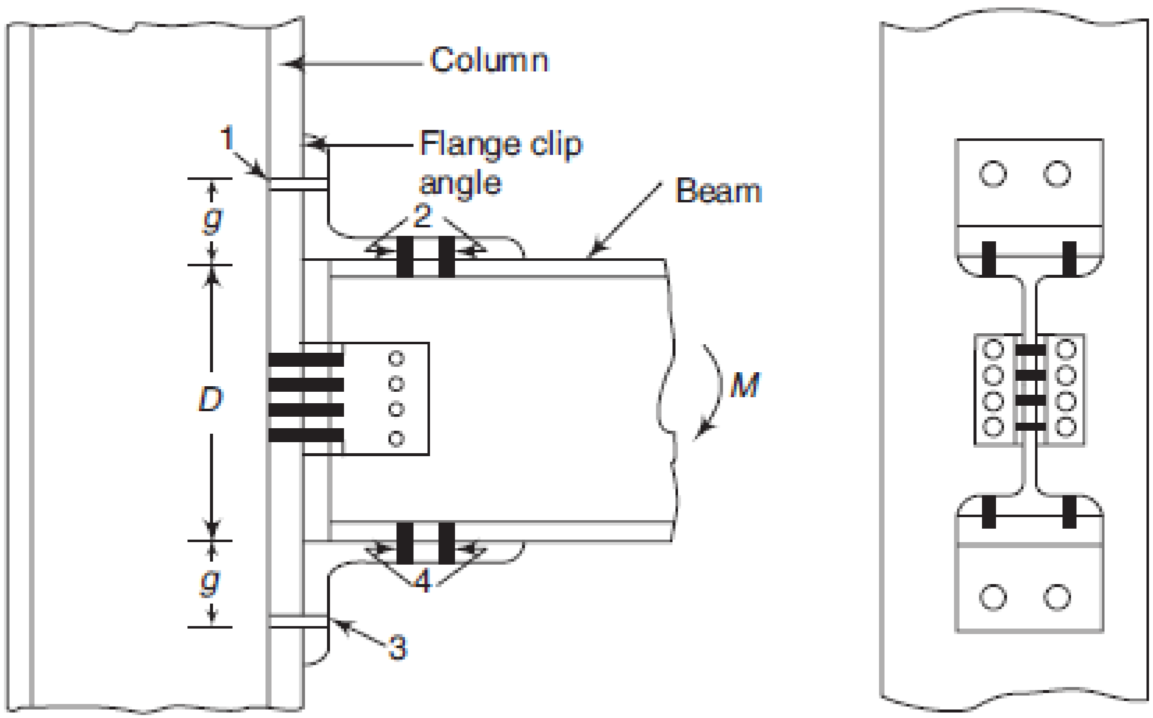
Extended end-plate connection

Rigid Beam-to-Column Connections

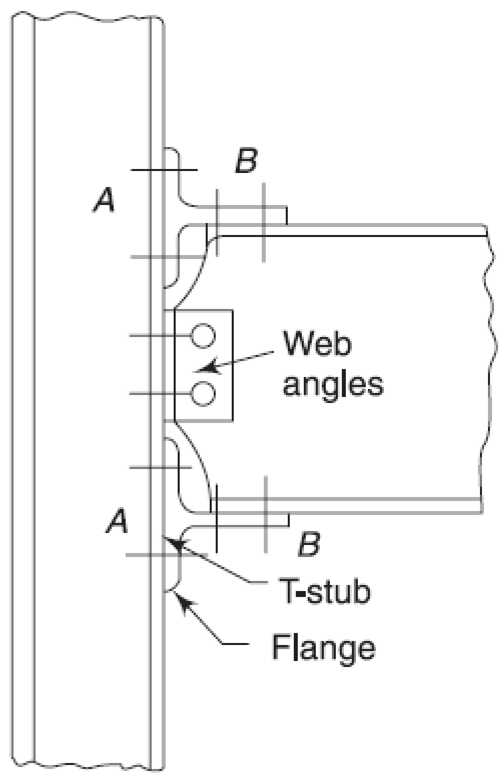


Short end-plate (a) | Extended End-plate (b) | Haunched connection (c)

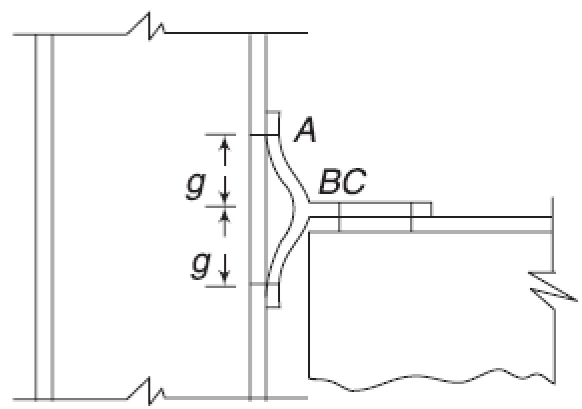
Flange-Angle Connection



T-Stub Connection

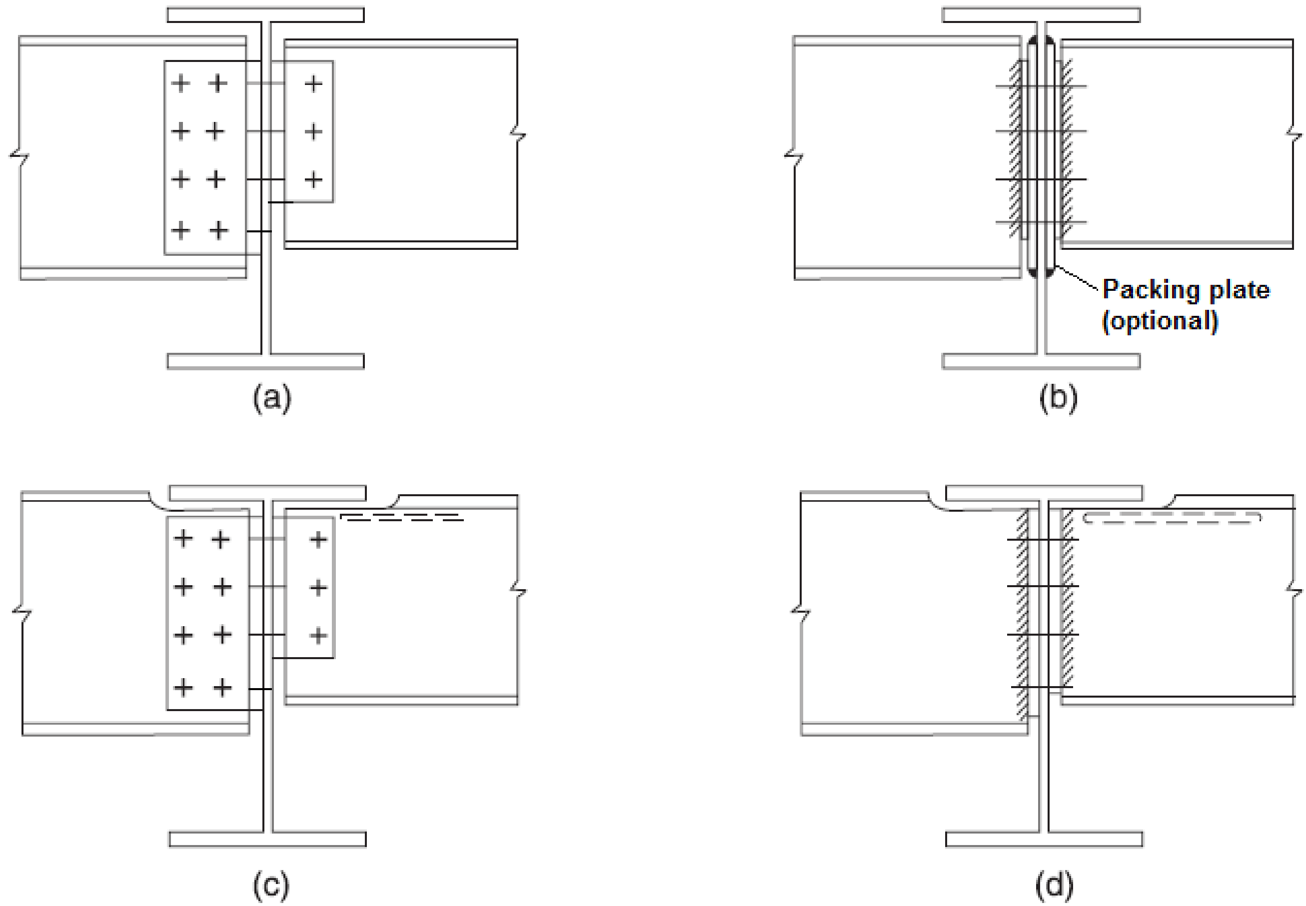


T-stub
(a)



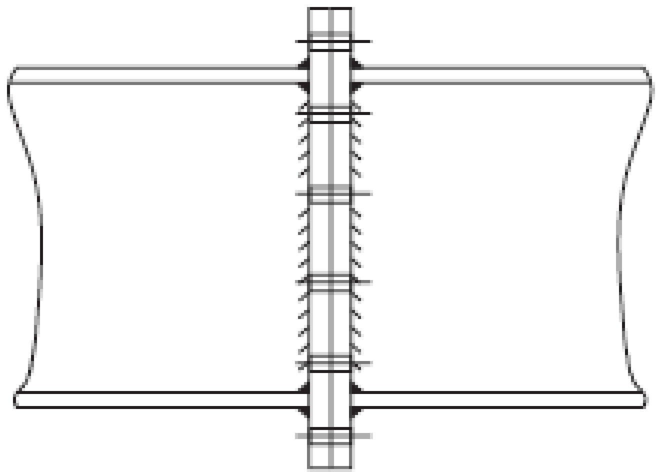
Deformation of T
(b)

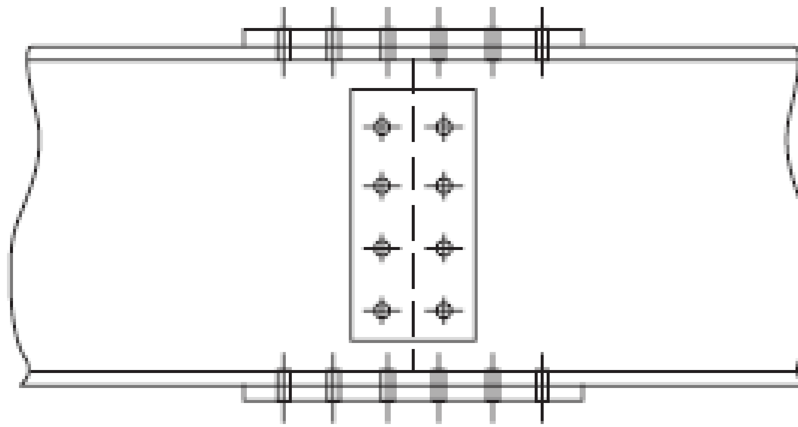
Beam-to- Beam Connections

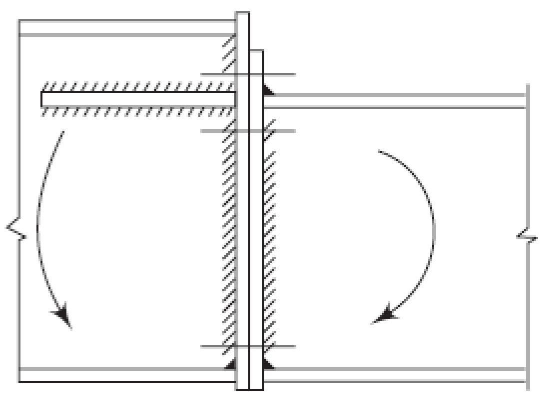


Beam-to-Beam Connections

Types of Beam-Splices

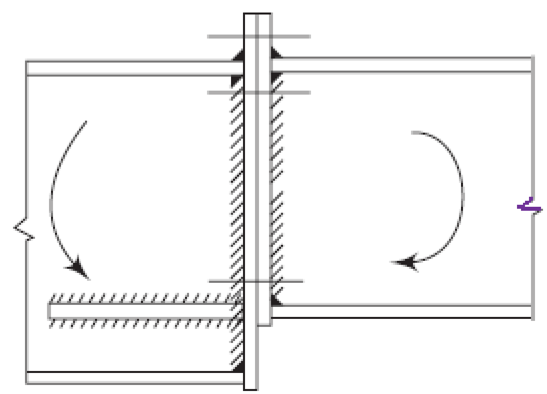






Coplanar compression flange

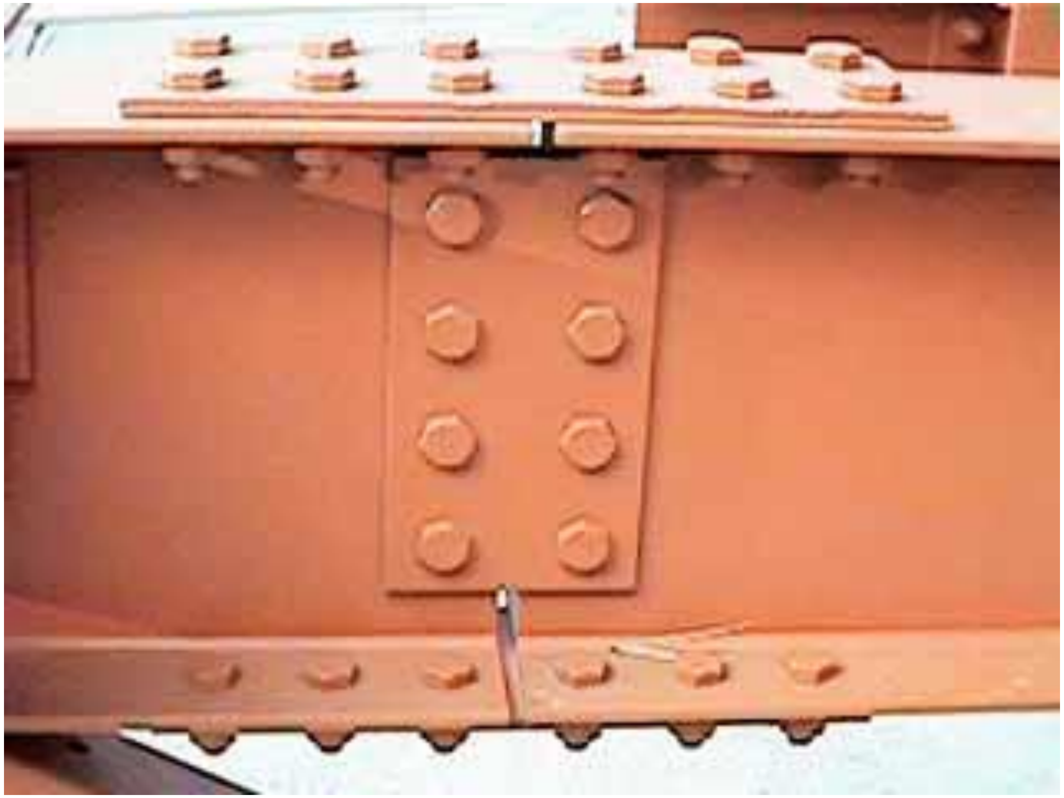
(a)



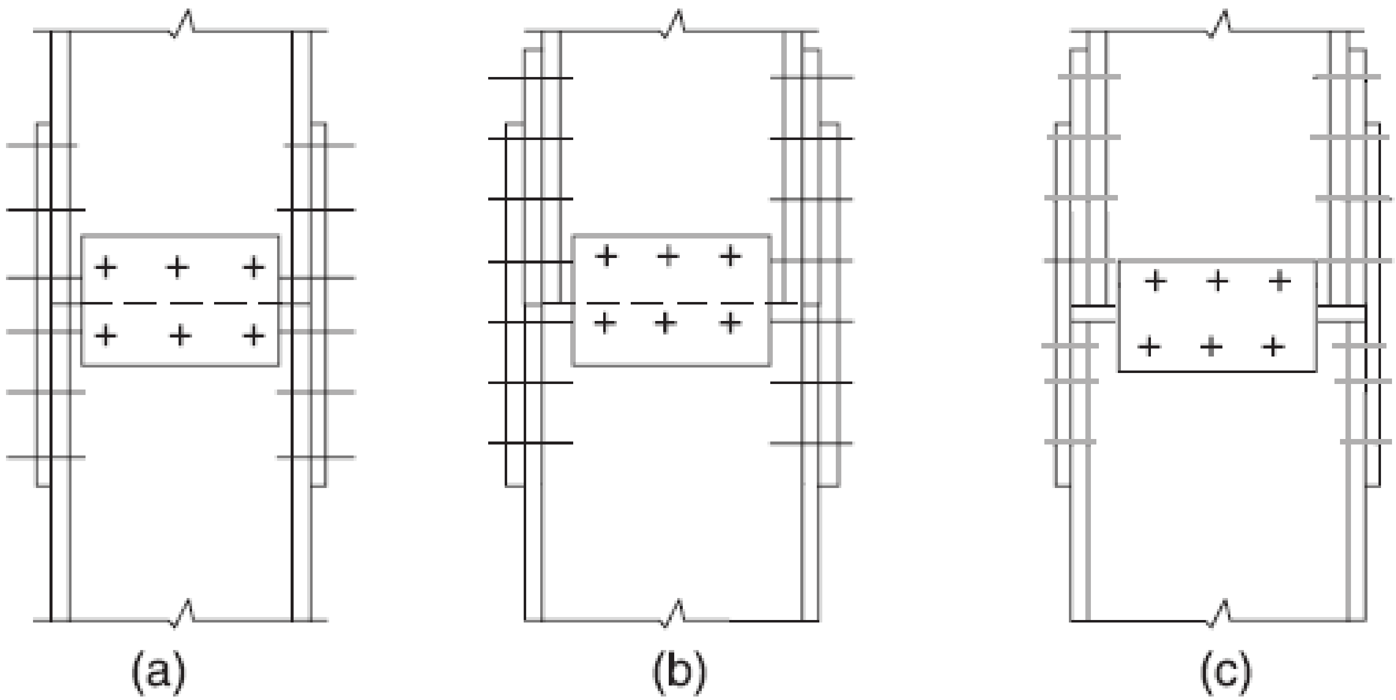
Coplanar tension flange

(b)

Bolted Beam-Splice

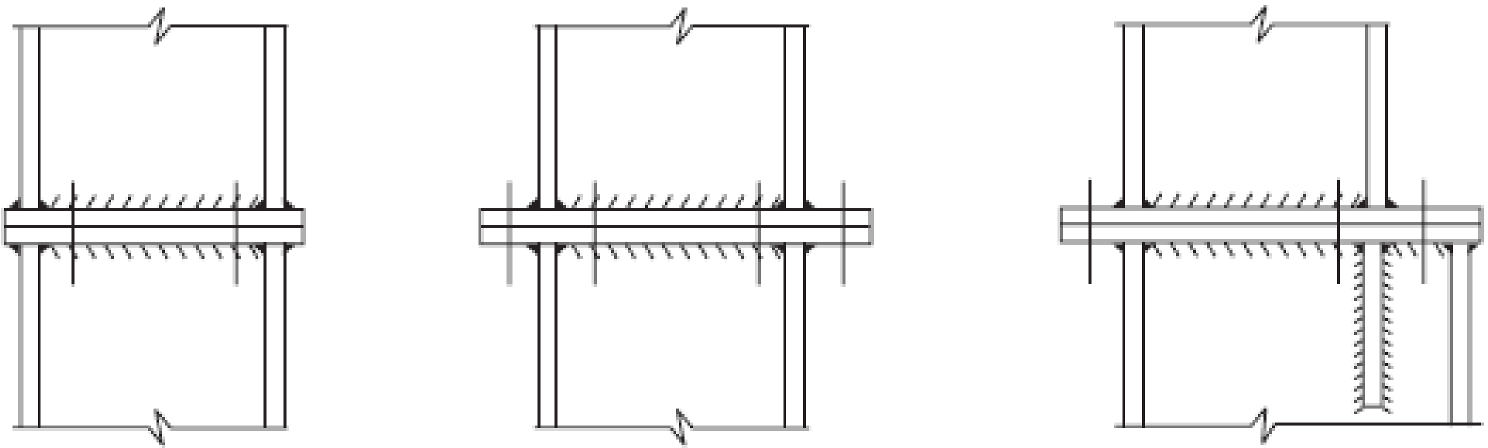


Bolted Column Splice



Bolted Column Splice

Column Splices
Using End-Plates



(a) (b) (c)

UNIT V

DESIGN OF WELDED PLATE GIRDERS

Connections and Tension Member Design

Connections

Connections must be able to transfer any axial force, shear, or moment from member to member or from beam to column.

Steel construction accomplishes this with bolt and welds. Wood construction uses nails, bolts, shear plates, and split-ring connectors.

<u>Bolted and Welded Connections</u>

The limit state for connections depends on the loads:

1. tension yielding
2. shear yielding
3. bearing yielding
4. bending yielding due to eccentric loads
5. rupture

Welds must resist tension AND shear stress. The design strengths depend on the weld materials.

Bolted Connection Design

Bolt designations signify material and type of connection where SC: slip critical
N: bearing-type connection with bolt threads *included* in shear plane
X: bearing-type connection with bolt threads *excluded* from shear plane

Bolts rarely fail in **bearing**. The material with the hole will more likely yield first. Standard bolt holes are

1/16" larger than the bolt diameter.

BOLTS AND THREADED PARTS
Bearing
Allowable loads in kips

TABLE I-E. BEARING
Slip-critical and Bearing-type Connections

Material Thickness	F_u = 58 ksi Bolt dia.			F_u = 65 ksi Bolt dia.			F_u = 70 ksi Bolt dia.			F_u = 100 ksi Bolt dia.		
	3/4	7/8	1	3/4	7/8	1	3/4	7/8	1	3/4	7/8	1
1/8	6.5	7.6	8.7	7.3	8.5	9.8	7.9	9.2	10.5	11.3	13.1	15.0
3/16	9.8	11.4	13.1	11.0	12.8	14.6	11.8	13.8	15.8	16.9	19.7	22.5
1/4	13.1	15.2	17.4	14.6	17.1	19.5	15.8	18.4	21.0	22.5	26.3	30.0
5/16	16.3	19.0	21.8	18.3	21.3	24.4	19.7	23.0	26.3	28.1	32.8	37.5
3/8	19.6	22.8	26.1	21.9	25.6	29.3	23.6	27.6	31.5	33.8	39.4	45.0
7/16	22.8	26.6	30.5	25.6	29.9	34.1	27.6	32.2	36.8		45.9	52.5
1/2	26.1	30.5	34.8	29.3	34.1	39.0	31.5	36.8	42.0			60.0
9/16	29.4	34.3	39.2	32.9	38.4	43.9		41.3	47.3			
5/8	32.6	38.1	43.5		42.7	48.8		45.9	52.5			
11/16		41.9	47.9		46.9	53.6			57.8			
3/4		45.7	52.2			58.5						
13/16			56.6									
7/8			60.9									
15/16												
1	52.2	60.9	69.6	58.5	68.3	78.0	63.0	73.5	84.0	90.0	105.0	120.0

Notes:
This table is applicable to all mechanical fasteners in both slip-critical and bearing-type connections utilizing standard holes. Standard holes shall have a diameter nominally 1/16-in. larger than the nominal bolt diameter (d + 1/16 in.).
Tabulated bearing values are based on $F_p = 1.2\ F_u$.
F_u = specified minimum tensile strength of the connected part.
In connections transmitting axial force whose length between extreme fasteners measured parallel to the line of force exceeds 50 in., tabulated values shall be reduced 20%.
Connections using high-strength bolts in slotted holes with the load applied in a direction other than approximately normal (between 80 and 100 degrees) to the axis of the hole and connections with bolts in oversize holes shall be designed for resistance against slip at working load in accordance with AISC ASD Specification Sect. J3.8.
Tabulated values apply when the distance l parallel to the line of force from the center of the bolt to the edge of the connected part is not less than 1½ d and the distance from the center of a bolt to the center of an adjacent bolt is not less than $3d$. See AISC ASD Commentary J3.8.
Under certain conditions, values greater than the tabulated values may be justified under Specification Sect. J3.7.
Values are limited to the double-shear bearing capacity of A490-X bolts.
Values for decimal thicknesses may be obtained by multiplying the decimal value of the unlisted thickness by the value given for a 1-in. thickness.

BOLTS, THREADED PARTS AND RIVETS
Shear
Allowable load in kips

TABLE I-D. SHEAR

	ASTM Designation	Connection Type[a]	Hole Type[b]	F_v ksi	Loading[c]	Nominal Diameter d, in.							
						5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
						Area (Based on Nominal Diameter) in.^{2}							
						.3068	.4418	.6013	.7854	.9940	1.227	1.485	1.767
Bolts	A307	—	STD NSL	10.0	S	3.1	4.4	6.0	7.9	9.9	12.3	14.8	17.7
					D	6.1	8.8	12.0	15.7	19.9	24.5	29.7	35.3
	A325	SC[a] Class A	STD	17.0	S	5.22	7.51	10.2	13.4	16.9	20.9	25.2	30.0
					D	10.4	15.0	20.4	26.7	33.8	41.7	50.5	60.1
			OVS, SSL	15.0	S	4.60	6.63	9.02	11.8	14.9	18.4	22.3	26.5
					D	9.20	13.3	18.0	23.6	29.8	36.8	44.6	53.0
			LSL	12.0	S	3.68	5.30	7.22	9.42	11.9	14.7	17.8	21.2
					D	7.36	10.6	14.4	18.8	23.9	29.4	35.6	42.4
		N	STD, NSL	21.0	S	6.4	9.3	12.6	16.5	20.9	25.8	31.2	37.1
					D	12.9	18.6	25.3	33.0	41.7	51.5	62.4	74.2
		X	STD, NSL	30.0	S	9.2	13.3	18.0	23.6	29.8	36.8	44.5	53.0
					D	18.4	26.5	36.1	47.1	59.6	73.6	89.1	106.0
	A490	SC[a] Class A	STD	21.0	S	6.44	9.28	12.6	16.5	20.9	25.8	31.2	37.1
					D	12.9	18.6	25.3	33.0	41.7	51.5	62.4	74.2
			OVS, SSL	18.0	S	5.52	7.95	10.8	14.1	17.9	22.1	26.7	31.8
					D	11.0	15.9	21.6	28.3	35.8	44.2	53.5	63.6
			LSL	15.0	S	4.60	6.63	9.02	11.8	14.9	18.4	22.3	26.5
					D	9.20	13.3	18.0	23.6	29.8	36.8	44.6	53.0
		N	STD, NSL	28.0	S	8.6	12.4	16.8	22.0	27.8	34.4	41.6	49.5
					D	17.2	24.7	33.7	44.0	55.7	68.7	83.2	99.0
		X	STD, NSL	40.0	S	12.3	17.7	24.1	31.4	39.8	49.1	59.4	70.7
					D	24.5	35.3	48.1	62.8	79.5	98.2	119.0	141.0
Rivets	A502-1	—	STD	17.5	S	5.4	7.7	10.5	13.7	17.4	21.5	26.0	30.9
					D	10.7	15.5	21.0	27.5	34.8	42.9	52.0	61.8
	A502-2 A502-3	—	STD	22.0	S	6.7	9.7	13.2	17.3	21.9	27.0	32.7	38.9
					D	13.5	19.4	26.5	34.6	43.7	54.0	65.3	77.7
Threaded Parts	A36 (F_u=58 ksi)	N	STD	9.9	S	3.0	4.4	6.0	7.8	9.8	12.1	14.7	17.5
					D	6.1	8.7	11.9	15.6	19.7	24.3	29.4	35.0
		X	STD	12.8	S	3.9	5.7	7.7	10.1	12.7	15.7	19.0	22.6
					D	7.9	11.3	15.4	20.1	25.4	31.4	38.0	45.2
	A572, Gr. 50 (F_u=65 ksi)	N	STD	11.1	S	3.4	4.9	6.7	8.7	11.0	13.6	16.5	19.6
					D	6.8	9.8	13.3	17.4	22.1	27.2	33.0	39.2
		X	STD	14.3	S	4.4	6.3	8.6	11.2	14.2	17.5	21.2	25.3
					D	8.8	12.6	17.2	22.5	28.4	35.1	42.5	50.5
	A588 (F_u=70 ksi)	N	STD	11.9	S	3.7	5.3	7.2	9.3	11.8	14.6	17.7	21.0
					D	7.3	10.5	14.3	18.7	23.7	29.2	35.3	42.1
		X	STD	15.4	S	4.7	6.8	9.3	12.1	15.3	18.9	22.9	27.2
					D	9.4	13.6	18.5	24.2	30.6	37.8	45.7	54.4

[a]SC = Slip critical connection.
N: Bearing-type connection with threads *included* in shear plane.
X: Bearing-type connection with threads *excluded* from shear plane.
[b]STD: Standard round holes (d + 1/16 in.)
LSL: Long-slotted holes normal to load direction
NSL: Long-or short-slotted hole normal to load direction (required in bearing-type connection).
OVS: Oversize round holes
SSL: Short-slotted holes
[c]S: Single shear D: Double shear.
For threaded parts of materials not listed, use $F_v = 0.17F_u$ when threads are included in a shear plane, and $F_v = 0.22F_u$ when threads are excluded from a shear plane.
To fully pretension bolts 1⅛-in. dia. and greater, special impact wrenches may be required.
When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in., tabulated values shall be reduced by 20%. See AISC ASD Commentary Sect. J3.4.

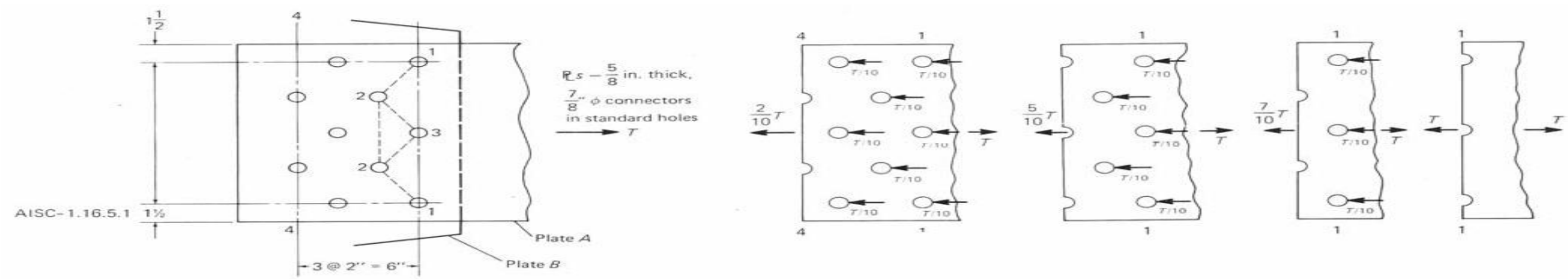
Tension Member Design

In steel tension members, there may be bolt holes that reduce the size of the cross section.

Effective Net Area:

The smallest effective are must be determined by subtracting the bolt hole areas. With staggered holes, the shortest length must be evaluated.

A series of bolts can also transfer a portion of the tensile force, and some of the effective net areas see reduced stress.



<u>ASD</u>

For other than pin connected members: $F_t = 0.60F_y$ *on gross area* $F_t = 0.50F_u$ *on net area*

For pin connected members: $F_t = 0.45F_y$ *on net area*

For threaded rods of approved steel: $F_t = 0.33F_u$ *on major diameter (static loading only)*

<u>LRFD</u>

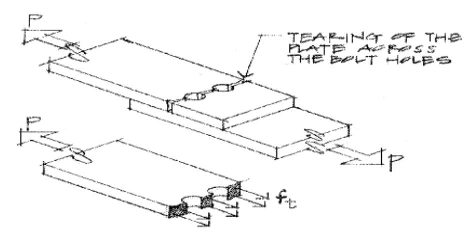
The limit state for tension members are: $P_u \leq \phi_t P_n$

1. yielding $\phi_t = 0.9 \quad P_n = F_y A_g$
2. rupture $\phi_t = 0.75 \quad P_n = F_u A_e$

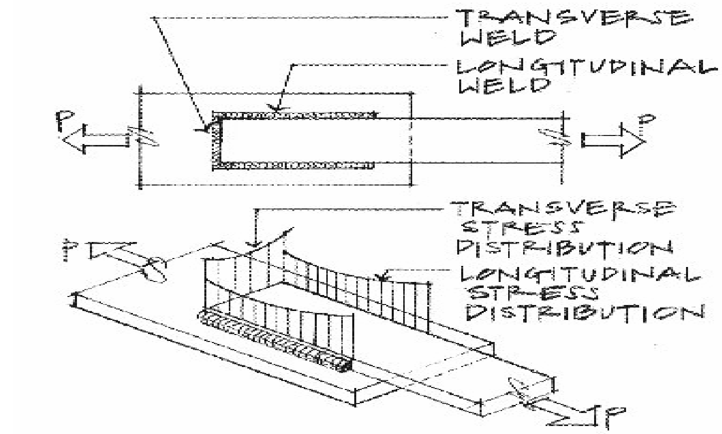
where

A_g = the gross area of the member (excluding holes)

A_e = the effective net area (with holes, etc.) F_u = the tensile strength of the steel (ultimate)



Welded Connections



Weld designations include the strength in the name, i.e. E70XX has F_y = 70 ksi.

The throat size, T, of a fillet weld is determined trigonometry by:
$T = 0.707 \times$weld size

<u>ASD</u>

Allowable shear stress of a weld is limited to 30% of the nominal strength.

F_v = 18 ksi for E60XX

F_v = 21 ksi for E70XX

Weld sizes are limited by the size of the parts being put together and are given in AISC manual table J2.4 along with the allowable strength per length of fillet weld, referred to as *S*.

Allowable Strength of Fillet Welds per inch of weld (*S*)

Weld Size (in.)	E60XX (k/in.)	E70XX (k/in.)
3/16	2.39	2.78
¼	3.18	3.71
5/16	3.98	4.64
3/8	4.77	5.57
7/16	5.57	6.94
½	6.36	7.42
5/8	7.95	9.27
¾		11.13

The *maximum* size of a fillet weld:

a) can't be greater than the material thickness if it

Intermittent fillet welds can not be less that four times the weld size, not to be less than 1 ½".

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thicker Part Joined (in.)	Minimum Size of Fillet Weld[a] (in.)
To ¼ inclusive	⅛
Over ¼ to ½	3/16
Over ½ to ¾	¼
Over ¾	5/16

[a]Leg dimension of fillet welds. Single-pass welds must be used.

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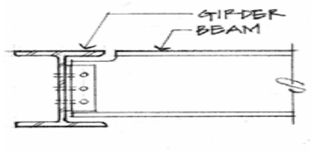
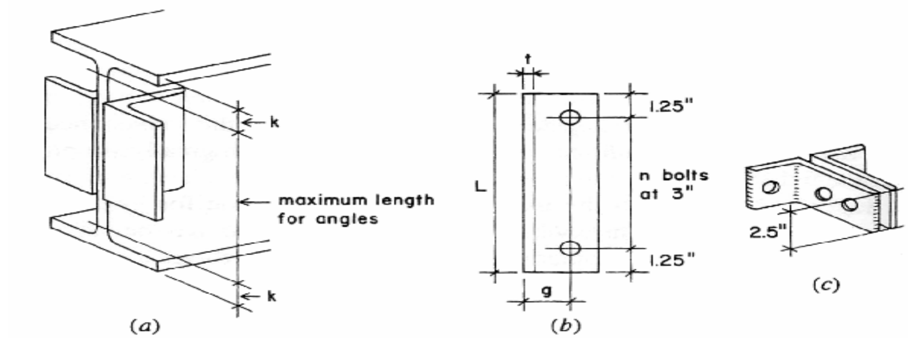
Framed Beam Connections



ing is the term for cutting away part of the flange to connect a beam to another beam using welded olted angles.

AISC provides tables that give angle sizes knowing bolt type, bolt diameter, angle leg thickness, and number of bolts (determined by *shear* capacity).

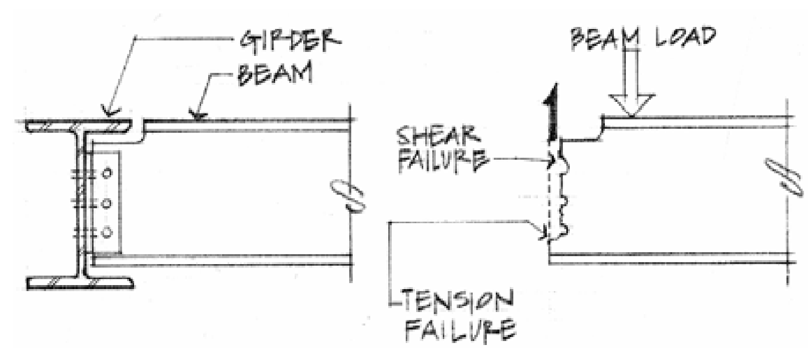


Load and Factor Resistance Design

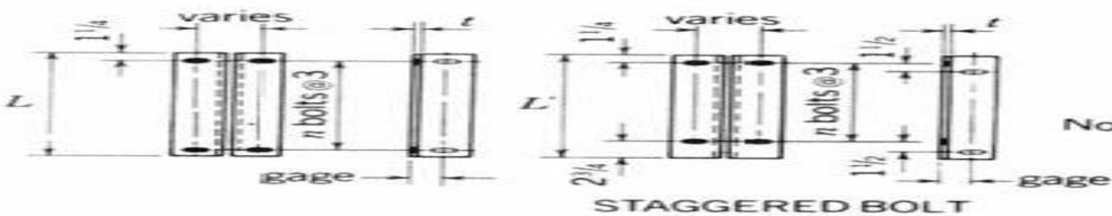
In addition to resisting shear and tension in bolts and shear in welds, the connected materials may be subjected to shear, bearing, tension, flexure and even prying action. Coping can significantly reduce design strengths and may require web reinforcement. All the following must be considered:

- shear yielding
- shear rupture
- block shear rupture -

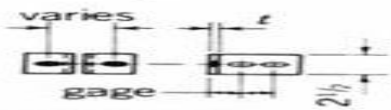
  failure of a block at a beam as a result of shear and tension
- tension yielding
- tension rupture
- local web buckling
- lateral torsional buckling



FRAMED BEAM CONNECTIONS
Bolted
TABLE II Allowable loads in kips



STAGGERED BOLT ALTERNATE



Note: For $L = 2\frac{1}{2}$ use one half the tabular load value shown for $L = 5\frac{1}{2}$, for the same bolt type, diameter, and thickness.

TABLE II-A Bolt Shear
For bolts in **bearing-type** connections with standard or slotted holes.

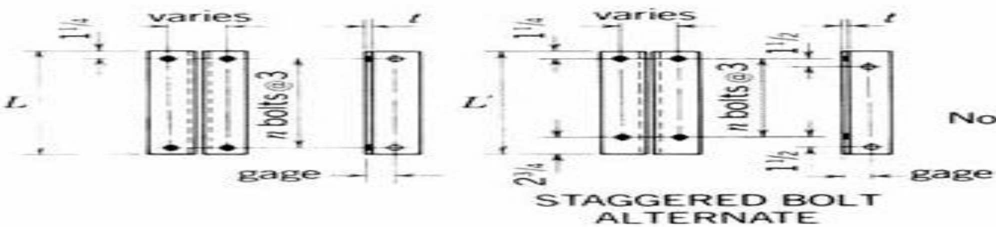
Bolt Type			A325-N			A490-N			A325-X			A490-X		
F_v, Ksi			21.0			28.0			30.0			40.0		
Bolt Dia., d In.			¾	⅞	1	¾	⅞	1	¾	⅞	1	¾	⅞	1
Angle Thickness t, In.			5/16	⅜	⅝	⅜	½	⅝	⅜	⅝	⅝	½	⅝	⅝
L In.	L' In.	n												
29½	31	10	186	253	330	247	337	440	265	361		353		
26½	28	9	167	227	297	223	303	396	239	325		318		
23½	25	8	148	202	264	198	269	352	212	289		283		
20½	22	7	130	177	231	173	236	308	186	253		247		
17½	19	6	111	152	198	148	202	264	159	216		212	289	
14½	16	5	92.8	126	165	124	168	220	133	180		177	242	
11½	13	4	74.2	101	132	99.0	135	176	106	144	188	141	192	
8½	10	3	55.7	75.8	99.0	74.2	101	132	79.5	108	141	106	144	
5½	7	2	37.1	50.5	66.0	49.5	67.3	88.0	53.0	72.2	94	70.7	96	

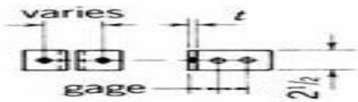
Tabulated load values are based on double shear of bolts.

Shaded values are based on double shear of the bolts; however, for length L, net shear on the angle thickness specified is critical. See Table II-C.

For shaded cells without values, shear rupture is critical for lengths L and L' on angle thickness specified. See Table II-C.

FRAMED BEAM CONNECTIONS
Bolted
TABLE II Allowable loads in kips





Note: For $L = 2\frac{1}{2}$ use one half the tabular load value shown for $L = 5\frac{1}{2}$, for the same bolt type, diameter, and thickness.

TABLE II-A Bolt Shear[a]

For A307 bolts in standard or slotted holes and for A325 and A490 bolts in **slip-critical** connections with standard holes and Class A, clean mill scale surface condition.

Bolt Type			A307			A325-SC			A490-SC			
F_v, Ksi			10.0			17.0			21.0			**Note:** For slip-critical connections with oversize or slotted holes, see Table II-B.
Bolt Dia., d In.			¾	⅞	1	¾	⅞	1	¾	⅞	1	
Angle Thickness t, In.			¼	¼	¼	¼	5/16	½	5/16	½	⅝	
L In.	L' In.	n										
29½	31	10	88.4	120	157	150	204	267	186	253	330	
26½	28	9	79.5	108	141	135	184	240	167	227	297	
23½	25	8	70.7	96.2	126	120	164	214	148	202	264	
20½	22	7	61.9	84.2	110	105	143	187	130	177	231	
17½	19	6	53.0	72.2	94.2	90.1	123	160	111	152	198	
14½	16	5	44.2	60.1	78.5	75.1	102	134	92.8	126	165	
11½	13	4	35.3	48.1	62.8	60.1	81.8	107	74.2	101	132	
8½	10	3	26.5	36.1	47.1[b]	45.1	61.3	80.1	55.7	75.8	99.0	
5½	7	2	17.7	24.1	31.4[b]	30.0	40.9	53.4	37.1	50.5	66.0	

Notes:

[a]Tabulated load values are based on double shear of bolts unless noted. See RCSC Specification for other surface conditions.

[b]Capacity shown is based on double shear of the bolts; however, for length L, net shear on the angle thickness specified is critical. See Table II-C.

Example 1

10.2 The butt splice shown in Figure 10.22 uses two $8 \times \frac{3}{8}''$ plates to "sandwich" in the $8 \times \frac{1}{2}''$ plates being joined. Four $\frac{7}{8}''\phi$ A325-SC bolts are used on both sides of the splice. Assuming A36 steel and standard round holes, determine the allowable capacity of the connection.

Solution:

Shear, bearing, and net tension will be checked to determine the critical condition that governs the capacity of the connection.

(Table I-D)

Shear: Using the AISC allowable shear in Table 10.1:

$$P_v = 20.4\,\text{k/bolt} \times 4\ \text{bolts} = 81.6\,\text{k} \quad \text{(double shear)}$$

(Table I-E)

Bearing: Using the AISC bearing in Table 10.2:

The thinner material with the largest proportional load governs, therefore, the $\frac{1}{2}''$ center plate governs. Assume the bolts are at a $3d$ spacing, center to center.

$$P_b = 30.5\,\text{k/bolt} \times 4\ \text{bolts} = 122\,\text{k}$$

Tension: The center plate is critical since its thickness is less than the combined thickness of the two outer plates.

Hole diameter = (bolt diameter) + $\frac{1}{16}'' = \frac{7}{8}'' + \frac{1}{16}'' = \frac{15}{16}''$.

$$A_{net} = (8'' - 2 \times \tfrac{15}{16}'') \times (\tfrac{1}{2}'') = 3.06\ \text{in.}^2$$

$$P_t = F_t \times A_{\text{net}}$$

where:

$$F_t = 0.5F_u = 0.5(58\,\text{ksi}) = 29\,\text{ksi}$$

$$P_t = 29\,\text{k/in.}^2 \times 3.06\ \text{in.}^2 = 88.7\,\text{k}$$

For yielding in the cross section without holes:

$$A_{gross} = (8'') \times (\tfrac{1}{2}'') = 4.0\ \text{in.}^2$$

$$P_t = F_t \times A_{gross}$$

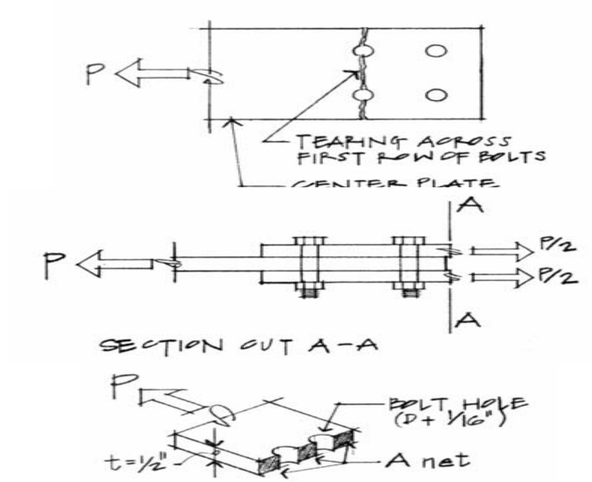
where:

$$F_t = 0.6F_y = 0.6(36\ \text{ksi}) = 21.6\ \text{ksi}$$

$$P_t = 21.6\ \text{k/in.}^2 \times 4.0\ \text{in.}^2 = 86.4\,\text{k}$$

The maximum connection capacity is governed by shear.

$$P_{\text{allow}} = 81.6\,\text{k}$$



Example 2

10.7 Determine the capacity of the connection in Figure 10.44 assuming A36 steel with E70XX electrodes.

Solution:

Capacity of weld:

For a $\frac{5}{16}''$ fillet weld, $S = 4.64$ k/in

Weld length $= 22''$

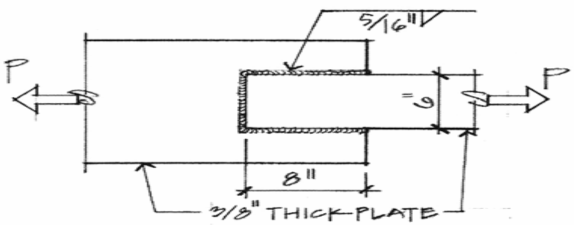
Weld capacity $= 22'' \times 4.64$ k/in $= 102.1$ k

Capacity of plate:

$$\underset{\text{allow}}{F_t} = 0.6F_y = 22\text{ ksi}$$

Plate capacity $= \frac{3}{8}'' \times 6'' \times 22\text{ k/in.}^2 = 49.5$ k

$\therefore$ Plate capacity governs, $P_{\text{allow}} = 49.5$ k



The weld size used is obviously too strong. What size, then, can the weld be reduced to so that the weld strength is more compatible to the plate capacity? To make the weld capacity $\approx$ plate capacity:

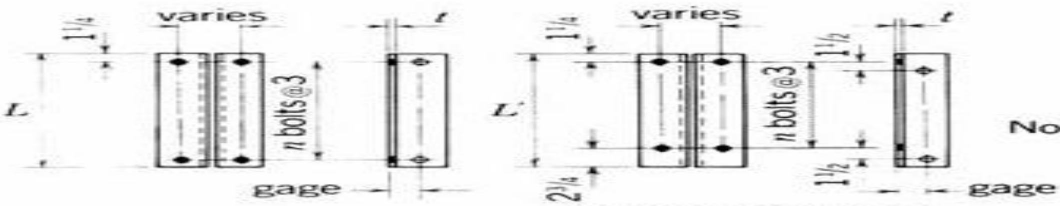
$$22'' \times (\text{weld capacity per in.}) = 49.5\text{ k}$$

$$\text{Weld capacity per inch} = \frac{49.5\text{ k}}{22\text{ in.}} = 2.25\text{ k/in.}$$

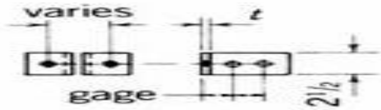
From Table 10.5, use $\frac{3}{16}''$ weld ($S = 2.78$ k/in.).

Minimum size fillet $= \frac{3}{16}''$ based on a $\frac{3}{8}''$ thick plate.

FRAMED BEAM CONNECTIONS
Bolted
TABLE II Allowable loads in kips



STAGGERED BOLT ALTERNATE



Note: For $L = 2\frac{1}{2}$ use one half the tabular load value shown for $L = 5\frac{1}{2}$, for the same bolt type, diameter, and thickness.

TABLE II-A Bolt Shear[a]

For A307 bolts in standard or slotted holes and for A325 and A490 bolts in **slip-critical** connections with standard holes and Class A, clean mill scale surface condition.

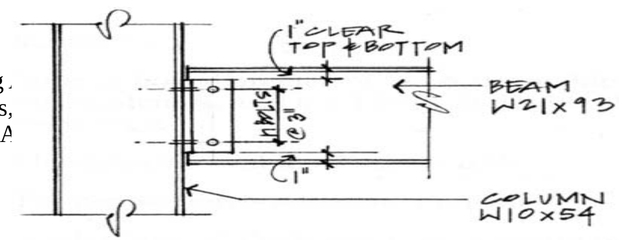
Bolt Type			A307			A325-SC			A490-SC		
F_v, Ksi			10.0			17.0			21.0		
Bolt Dia., d In.			¾	⅞	1	¾	⅞	1	¾	⅞	1
Angle Thickness t, In.			¼	¼	¼	¼	5/16	½	5/16	½	⅝
L In.	L' In.	n									
29½	31	10	88.4	120	157	150	204	267	186	253	330
26½	28	9	79.5	108	141	135	184	240	167	227	297
23½	25	8	70.7	96.2	126	120	164	214	148	202	264
20½	22	7	61.9	84.2	110	105	143	187	130	177	231
17½	19	6	53.0	72.2	94.2	90.1	123	160	111	152	198
14½	16	5	44.2	60.1	78.5	75.1	102	134	92.8	126	165
11½	13	4	35.3	48.1	62.8	60.1	81.8	107	74.2	101	132
8½	10	3	26.5	36.1	47.1[b]	45.1	61.3	80.1	55.7	75.8	99.0
5½	7	2	17.7	24.1	31.4[b]	30.0	40.9	53.4	37.1	50.5	66.0

Note: For slip-critical connections with oversize or slotted holes, see Table II-B.

Notes:

[a]Tabulated load values are based on double shear of bolts unless noted. See RCSC Specification for other surface conditions.

[b]Capacity shown is based on double shear of the bolts; however, for length L, net shear on the angle thickness specified is critical. See Table II-C.



Example 3

The steel used in the connection and beams is A992 with F_y = 50 ksi, and F_u = 65 ksi. Using bolt material, determine the maximum capacity of the connection based on shear in the bolts, bearing in all materials and pick the number of bolts and angle length (not staggered). Use for the angles.

W21x93: d = 21.62 in, t_w = 0.58 in, t_f = 0.93 in W10x54: t_f = 0.615 in

SOLUTION:

The maximum length the angles can be depends on how it fits between the top and bottom flange with some clearance allowed for the fillet to the flange, and getting an air wrench in to tighten the bolts. This example uses 1" of clearance:

Available length = beam depth – both flange thicknesses – 1" clearance at top & 1" at bottom

= 21.62 in – 2(0.93 in) – 2(1 in) = 17.76 in.

The standard lengths for non-staggered holes (L) and staggered holes (L') are shown in Table II-A. The closest size within the available length is 17 ½ in. This will fit 6 bolts (n) with a standard spacing.

We have a choice of bolt diameters of ¾", 7/8" and 1" in Table II-A. These have allowable loads for **shear** (double) of 148 kips, 202 kips, and 264 kips. But the last two values are shaded and the note says that "net shear on the angle thickness specified is critical" and to see Table II-C. The angle thickness (t) is listed below the bolt diameter.

Table II-C gives a value of 207 kips for a 7/8" bolt diameter, ½" angle thickness, and 17.5" length. It gives a value of 242 kips for a 1" bolt diameter, 5/8" angle thickness, and 17.5" length. Therefore, 242 kips is the maximum value limited by shear in the *angle.*

P_p = 264 kips for double shear of 1" bolts (Table I-D: 6 bolts·(44 k/bolt) = 264 kips)

P_v = 242 kips for net shear in angle

We also need to evaluate **bearing** of bolts on the angles, beam web, and column flange where there are bolt holes. Table I-E provides allowable bearing load for the material type, bolt diameter and some material thickness at the allowable bearing stress. It states "Values for decimal thicknesses may be obtained by multiplying the decimal value of the unlisted thickness by the value given for a 1-in. thickness". This comes from the definition for bearing stress: $f_p = \frac{P}{td} = F_p$, where P is the load and F_p is the allowable bearing stress. For a constant diameter and allowable stress, the allowable load depends only on the thickness.

a)Bearing for 5/8" thick angle: There are 12 bolt holes through two angle legs to the column, and 12 bolt holes through two angle legs either side of the beam. The material is A36 (F_u = 58 ksi), with 1" bolt diameters.

P_p = 12 bolts·(43.5 k/bolt) = 522 kips

b)Bearing for column flange: There are 12 bolt holes through two angle legs to the column. The material is A992 (F_u = 65 ksi), 0.615" thick, with 1" bolt diameters.

P_p = 12 bolts·(78 k/bolt/1")·(0.615 in) = 576 kips.

c)Bearing for beam web: There are *6* bolt holes through two angle legs either side of the beam. The material is A992 (F_u = 65 ksi), 0.58" thick, with 1" bolt diameters

P_p = 6 bolts·(78 k/bolt/1")·(0.58 in) = 271 kips.

Although, the bearing in the beam web is the smallest at 271 kips, with the shear on the bolts even smaller at 264 kips, *the maximum capacity for the simple-shear connector is 242 kips* limited by net shear in the angles.

Instructional Objectives:

At the end of this lesson, the students should be able to understand:

- Meaning of eccentricity in loading.
- Procedure for designing a screw/bolted joint in eccentric loading.
- Procedure for designing riveted joint under eccentric loading.

In many applications, a machine member is subjected to load such that a bending moment is developed in addition to direct normal or shear loading. Such type of loading is commonly known as eccentric loading.

(i) Screw joint

(ii) Riveted joint

(iii) Welded joint

1. Eccentrically loaded screwed joint:

Consider a bracket fixed to the wall by means of three rows of screws having two in each row as shown in figure 11.1.1. An eccentric load *F* is applied to the extreme end of the bracket. The horizontal component, F_h, causes direct tension

in the screws but the vertical component, F_v, is responsible for turning the bracket about the lowermost point in left (say point O), which in an indirect way introduces tension in the screws.

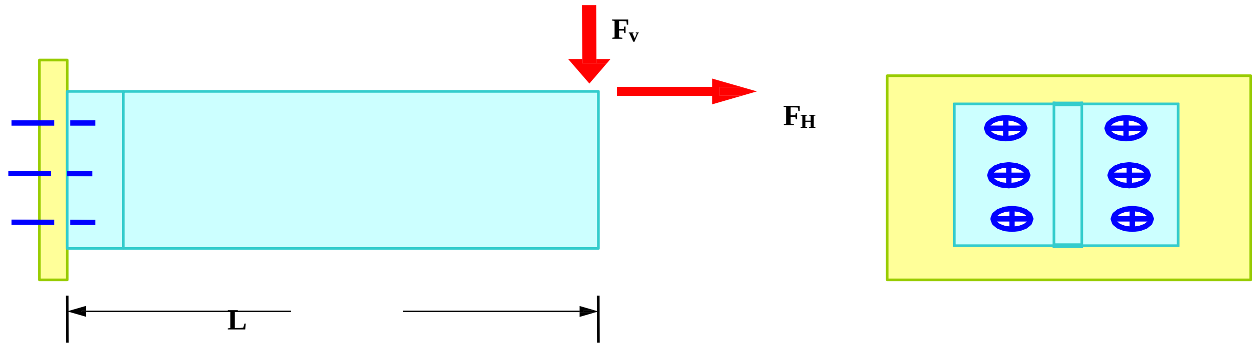


Figure 11.1.1: Eccentrically loaded bolted joint

It is easy to note that the tension in the screws cannot be obtained by equations of statics alone. Hence, additional equations must be formed to solve for the unknowns for this statically indeterminate problem. Since there is a tendency for the bracket to rotate about point O then, assuming the bracket to be rigid, the following equations are easily obtained.

$$\theta \approx \tan\theta = \frac{y_1}{l_1} = \frac{y_2}{l_2} = \frac{y_3}{l_3}$$

where y_i=elongation of the i-th bolt

l_i=distance of the axis of the i-th bolt from point O.

If the bolts are made of same material and have same dimension, then

$$f_i = k y_i$$

where f_i=force in the i-th bolt

k =stiffness of the bolts

Thus $f_i \propto l_i$ or $f_i = \alpha l_i$ (α =proportionality constant)

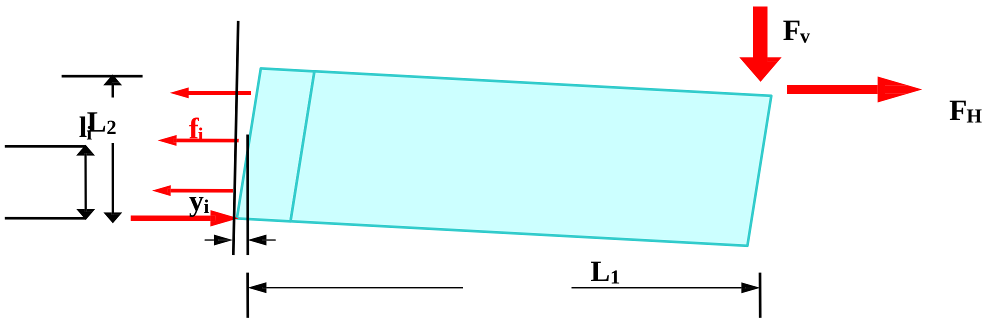


Figure 11.1.2: Determination of forces in bolts

Using the moment balance equations about O, the lowermost point in the left side, the following equation is obtained.

$$2\sum f_i l_i = F_h L_1 + F_v L_2$$

i.e., $\alpha = \dfrac{F_h L_1 + F_v L_2}{2\sum l_i^2}$. The factor 2 appears because there are two bolts in a row.

Thus the force in the i-th screw is

$$f_i = \left(\frac{F_h L_1 + F_v L_2}{2\sum l_i^2}\right) l_i + \frac{F_h}{n}$$, where n = total number of bolts.

where s_t=allowable tensile stress of the bolt.

Note that F_v causes also direct shear in the bolt. Its effect may be ignored for a preliminary design calculation.

2. Eccentrically loaded riveted joint:

Consider, now, a bracket, which carries a vertical load F. The bracket, in this case, is connected to the wall by four rivets as shown in figure 11.1.2. The force,

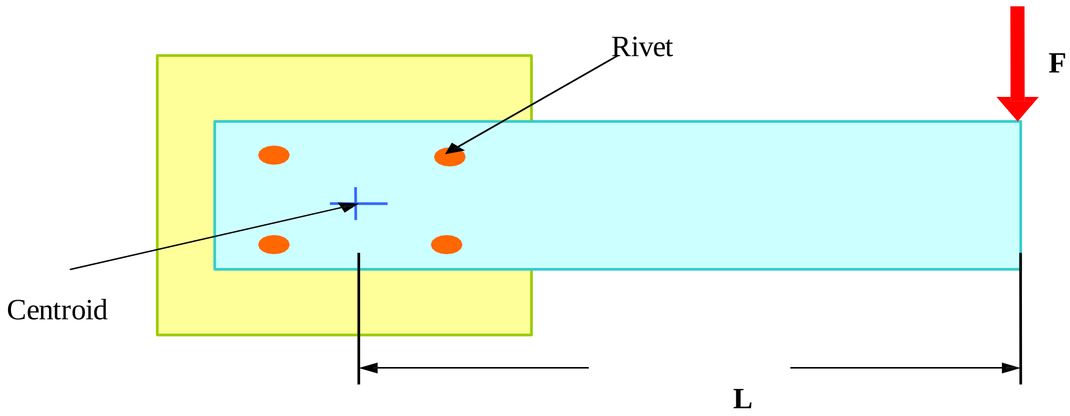


Figure 11.1.3: Eccentrically loaded rivet joint

in addition to inducing direct shear of magnitude $\frac{F}{4}$ in each rivet, causes the whole assembly to rotate. Hence additional shear forces appear in the rivets.

Once again, the problem is a statically indeterminate one and additional assumptions are required. These are as following:

(i) magnitude of additional shear force is proportional to the distance between the rivet center and the centroid of the rivet assembly, whose co- ordinates are defined as

$$\bar{x} = \frac{\sum A_i x_i}{\sum A_i}, \quad \bar{y} = \frac{\sum A_i y_i}{\sum A_i}$$

(A_i=area of the cross-section of the i-th rivet)

(ii) directions of the force is perpendicular to the line joining centroid of the rivet group and the rivet center and the sense is governed by the rotation of the bracket.

Noting that for identical rivets the centroid is the geometric center of the rectangle, the force in the i-th rivet is

$$f_i = \alpha l_i$$

where α =proportional constant

l_i=distance of the i-th rivet from centroid.

Taking moment about the centroid

$$\sum_i f_i l_i = FL \quad \text{or} \quad \alpha = \frac{FL}{\sum_i l_i^2}$$

Thus, the additional force is $f_i = \frac{FL}{\sum_i l_i^2} l_i$.

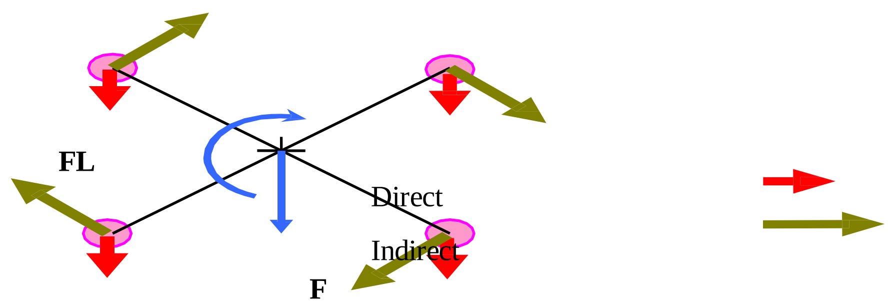


Figure 11.1.4: Forces on rivets due to

The net force in the i-th rivet is obtained by parallelogram law of vector addition as

$$f_i' = \sqrt{f_i^2 + \left(\frac{F}{4}\right)^2 + 2\cdot\frac{F}{4} f_i \cos\theta_i}$$

where θ_i=angle between the lines of action of the forces shown in the figure.

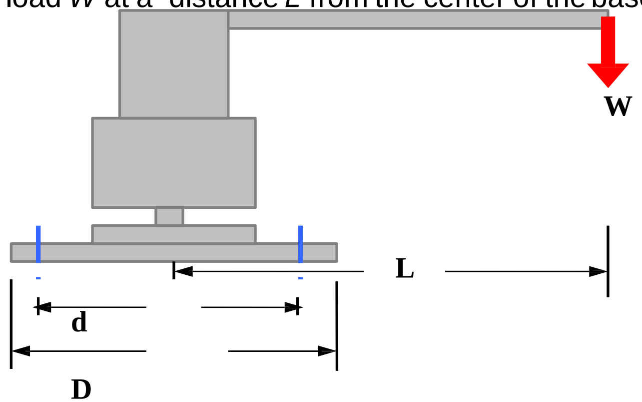
For safe designing we must have

$$\tau = \max\left(\frac{f_i'}{A}\right) \le s_s$$

where s_s=allowable shear stress of the rivet.

Model questions and answers:

Q. 1. The base of a pillar crane is fastened to the foundation by *n* bolts equally placed on a bolt circle of diameter *d*. The diameter of the pillar is *D*. Determine the maximum load carried by any bolt when the crane carries a load *W* at a distance *L* from the center of the base.



Ans. In this case the pillar have a tendency to topple about the point on the outer diameter lying closest to the point of application of the load.

Choose the line joining the center of the base and the point of application of the load as the reference line.

Q. 2. A bracket is supported by means of 4 rivets of same size as shown in figure 6. Determine the diameter of the rivet if the maximum shear stress is 140 MPa.

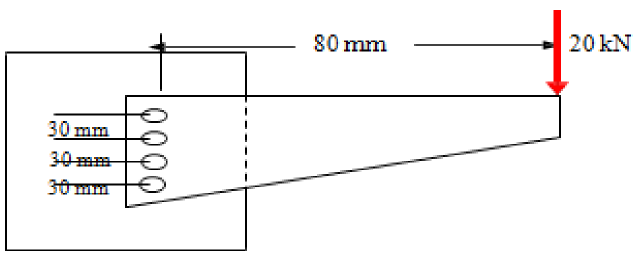
Ans. F_1= The direct shear force =5 kN per rivet. The maximum indirect shear force occurs in the topmost or bottommost rivet and its magnitude is

$$F_2 = \frac{20\times 80}{2\times 15^2+2\times 45^2}\times 45$$ kN and the direction is horizontal.

Therefore the maximum shear force on the rivet assembly is $F=\sqrt{F_1^2+F_2^2}$.

Hence

$\frac{\pi d}{4} \times s_s = F$ which yields $d \approx 16$ mm.



SIMPLE CONNECTIONS

Applied load passes through
C.G of connections

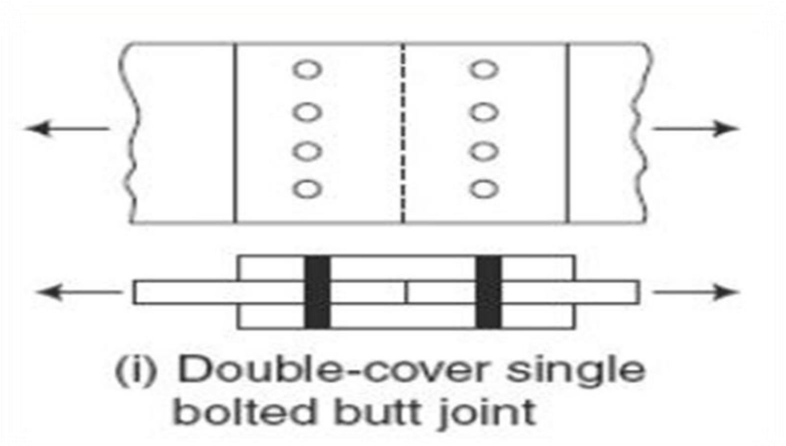


Fig -1a

Fig -1b

ECCENTRIC CONNECTIONS

Applied load does not pass through C.G of connections

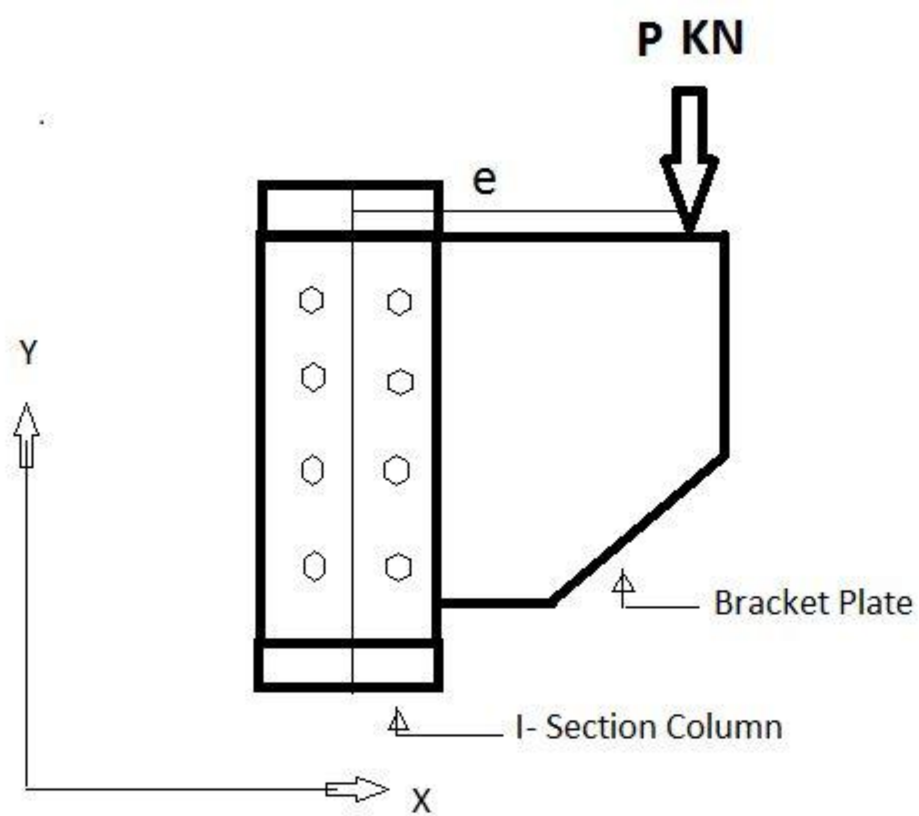


Fig 2

CONNECTIONS SUBJECTED TO ECCENTRIC SHEAR

Seat Connections
Framed Connections
Bracket Connections

SEAT CONNECTION

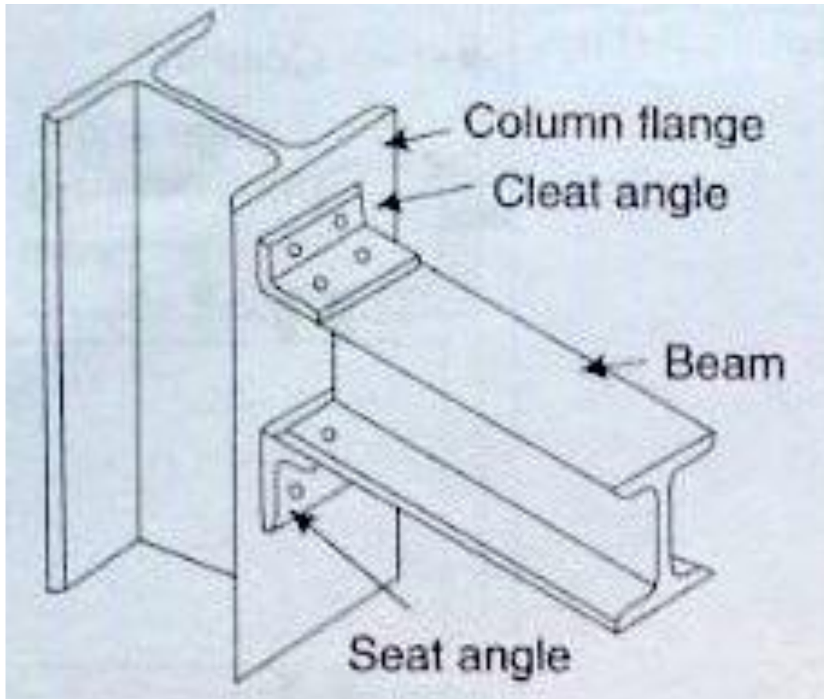


Fig 3a – Unstiffened Seat Connections

SEAT CONNECTION

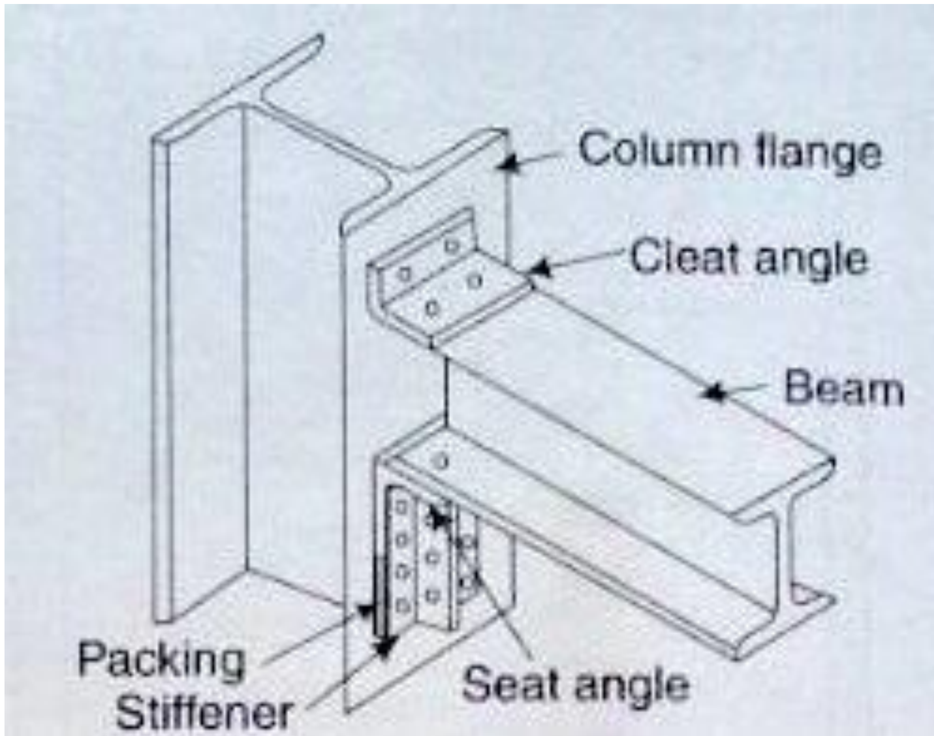


Fig 3b – Stiffened Seat Connections

FRAMED CONNECTIONS

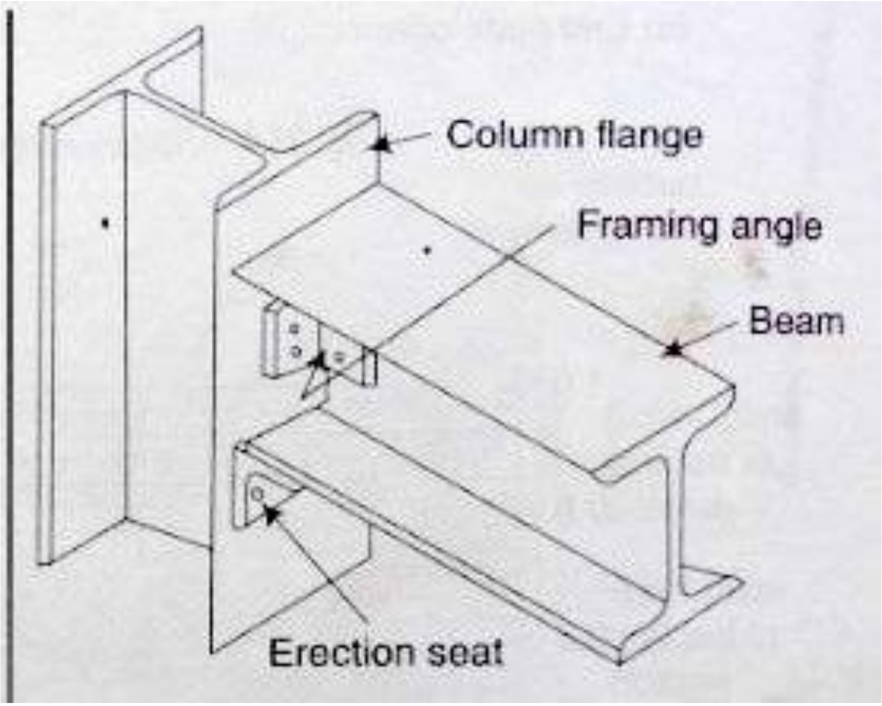


Fig - 4

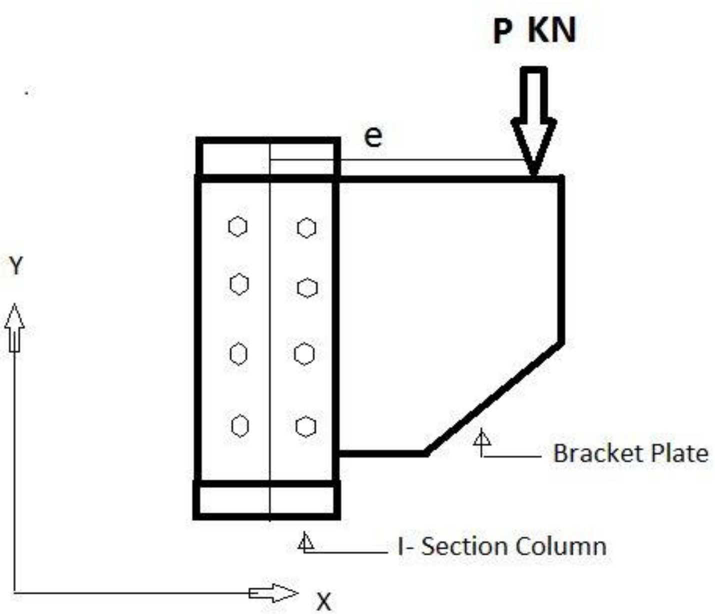
BRACKET CONNECTIONS

1) Bolted bracket – type I connections
2) Bolted bracket – type II connections
3) Welded bracket – type I connections
4) Welded bracket – type II connections

Theoretical Background

<u>BRACKET CONNECTIONS</u>

<u>BOLTED BRACKET CONNECTIONS TYPE 1</u>



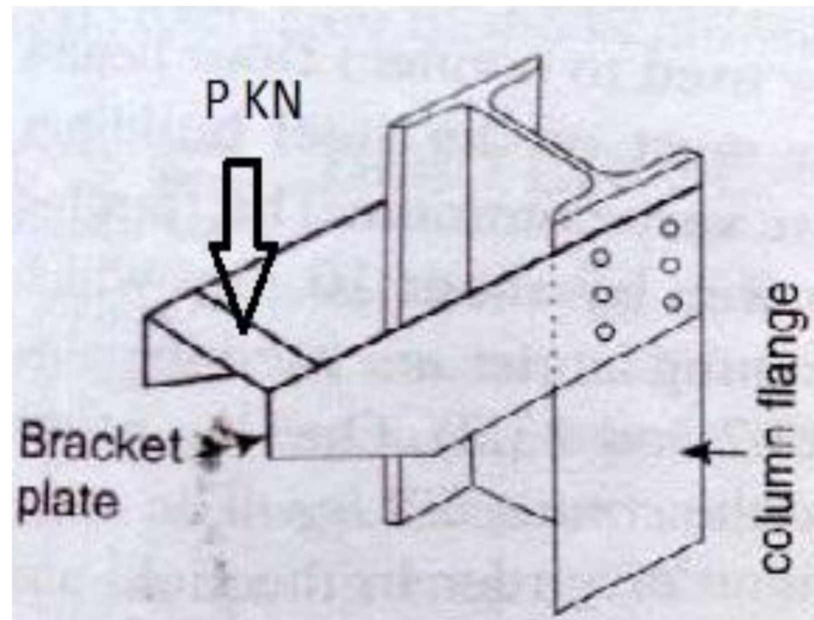


Fig 5

FORCES ON BRACKET TYPE 1

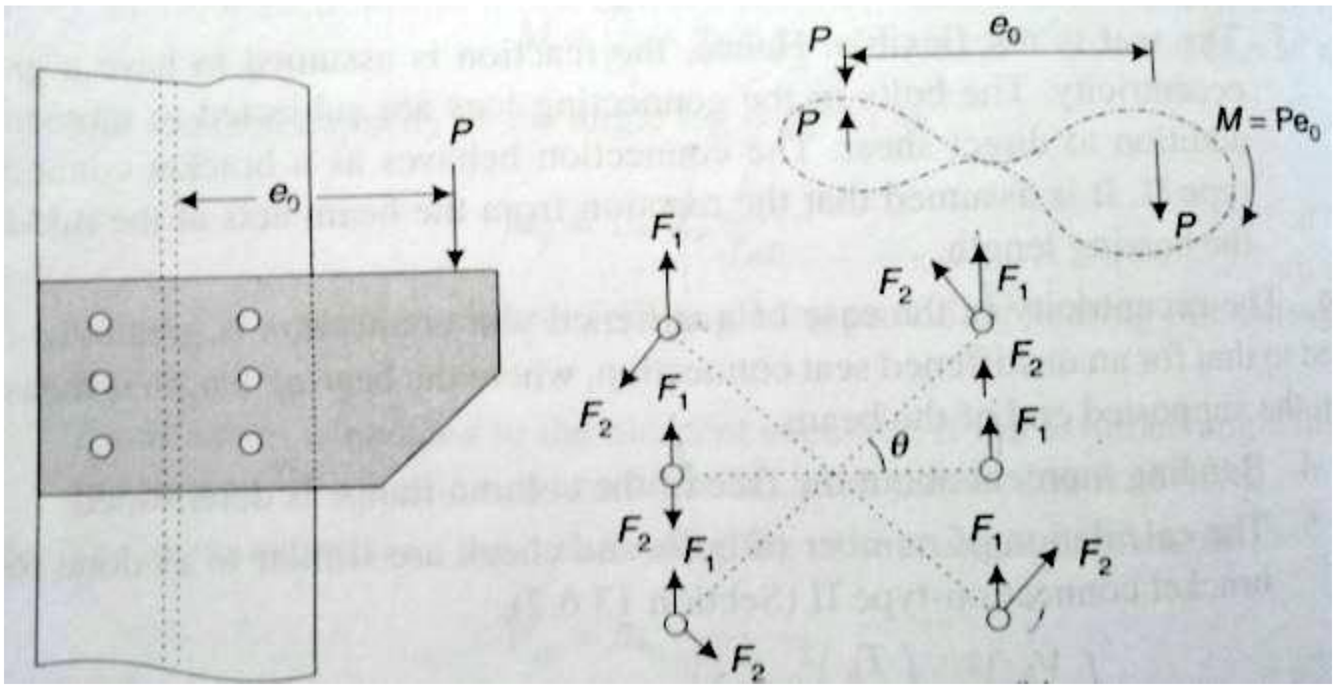


Fig 6

From the assumption made in the concentric bolted joints, ‘ the load over the joint is shared equally by all the bolts’ ,force in any bolt due to direct load is,

$$F_1 = \frac{P}{n} \tag{1}$$

We know that,

$$\frac{T}{J} = \frac{f}{r}$$

And also

Force = Stress * Area

Therefore,

$$F_2 \propto r$$

$$F_2 = k^*r$$

$$k = \frac{F_2}{r}$$

Therefore, the Torque about the center of rotation of the bolt group

$$= F_2 r = krr = kr^2$$

Total resisting toq $= \sum kr^2$

$$= k \sum r^2$$

$$= \frac{F_2}{r} \sum r^2$$

The resisting torque should be equal to torque over the connection. Hence,

$$M = \frac{F_2}{r} \sum r^2$$

$$Pe_0 = \frac{F_2}{r} \sum r^2$$

$$\frac{Pe_0 r}{\sum r^2} = F_2$$

Force F_2 is maximum when distance r is maximum. Let the distance of the extreme bolt be r_n . then,

$$F_2 = \frac{Pe_0 r_n}{\sum r^2} \qquad (2)$$

The two forces F_1 and F_2 act at some angle on various bolts in the connection. Let θ be the angle between these forces on the critical bolt. Then the resultant force F on the critical bolt will be

$$F = \sqrt{({F_1}^2 + {F_2}^2 + 2F_1F_2\cos\theta)} \qquad (3)$$

For the connection to be safe, this force must be less than the strength of the bolt.

BOLTED BRACKET CONNECTIONS TYPE 2

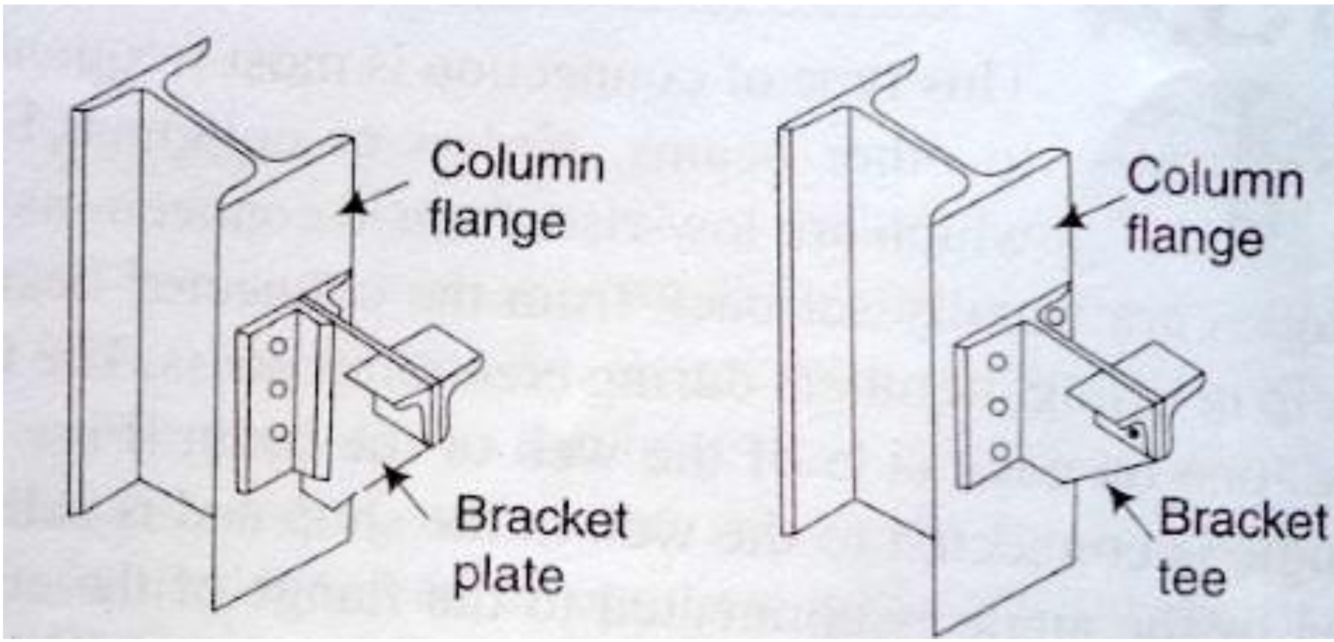
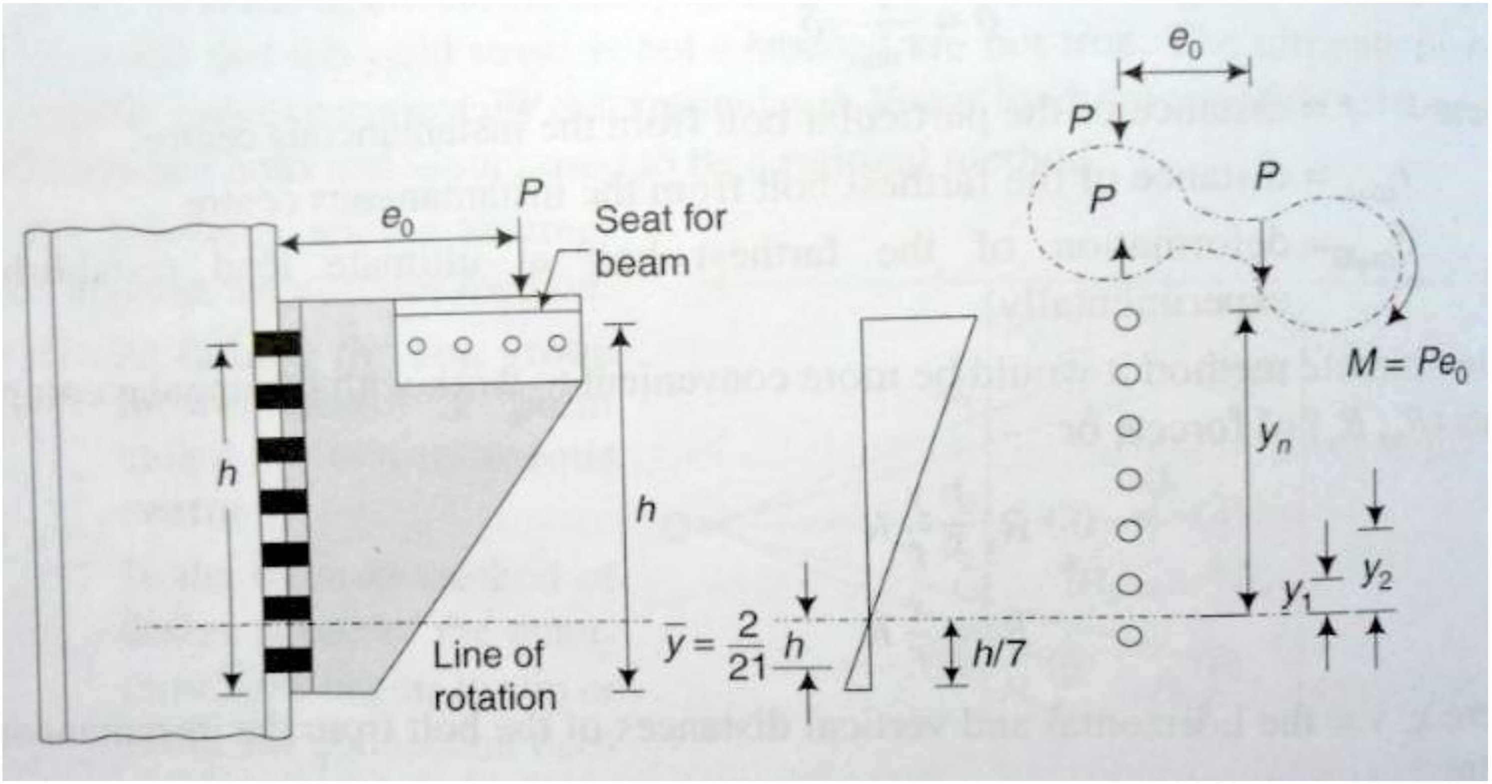


Fig 7



e_0
P
Seat for beam
h
h
Line of rotation
$\bar{y} = \frac{2}{21} h$
$h/7$
e_0
P
P
P
$M = Pe_0$
y_n
y_1
y_2

Where,

$M = Pe_0$ moment in Nmm, caused by the eccentric load

e_0 = the eccentricity of the load P from the bolt plane to the line of action of load in mm

P = the load acting over the joint in N

M' = moment of resistance provided by bolts in tension

V_b = force in a bolt due to direct shear P

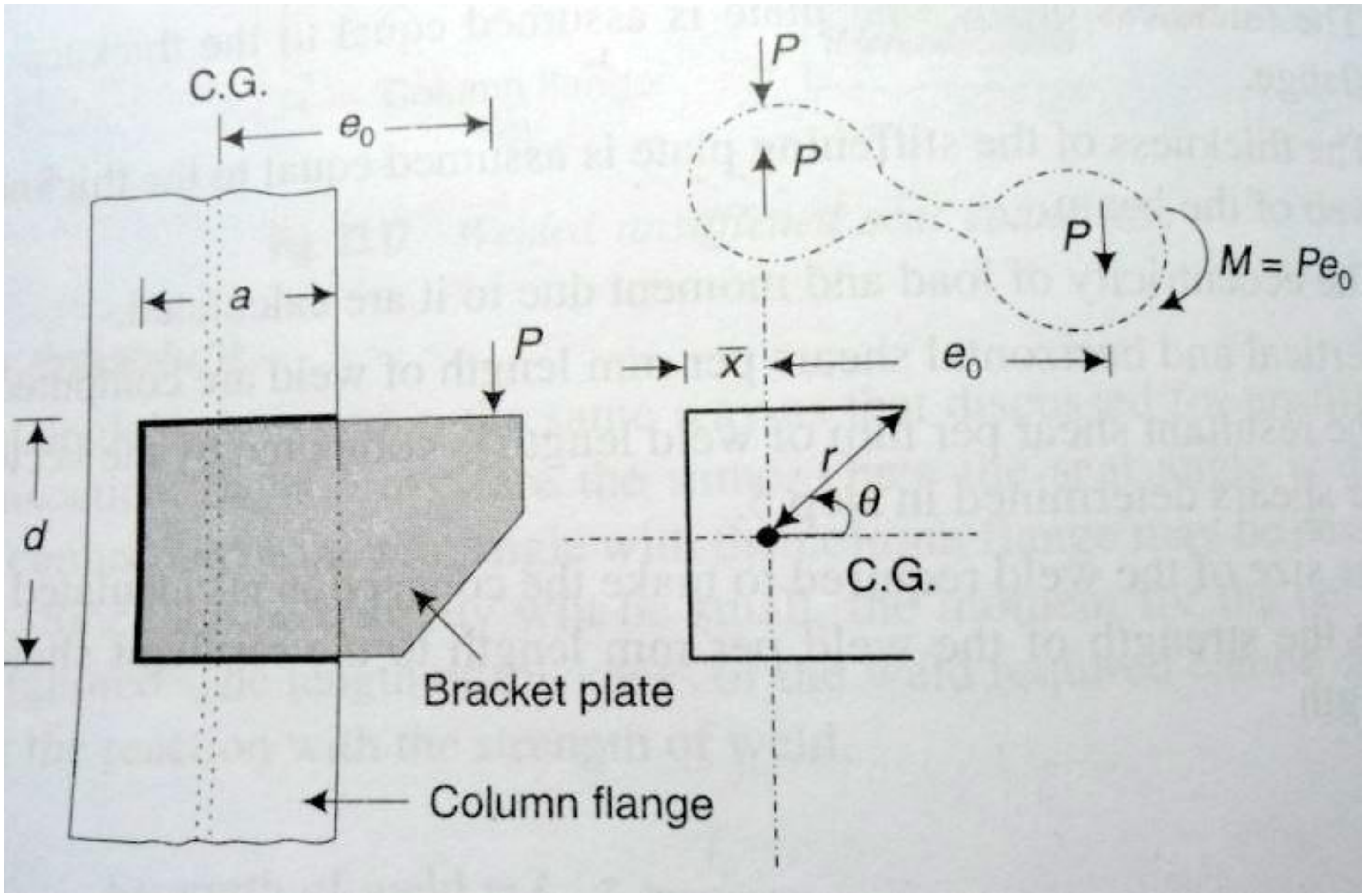
T_b = tensile force in the bolt due to bending moment (Pe_0)

n = number of bolts in the bolt group

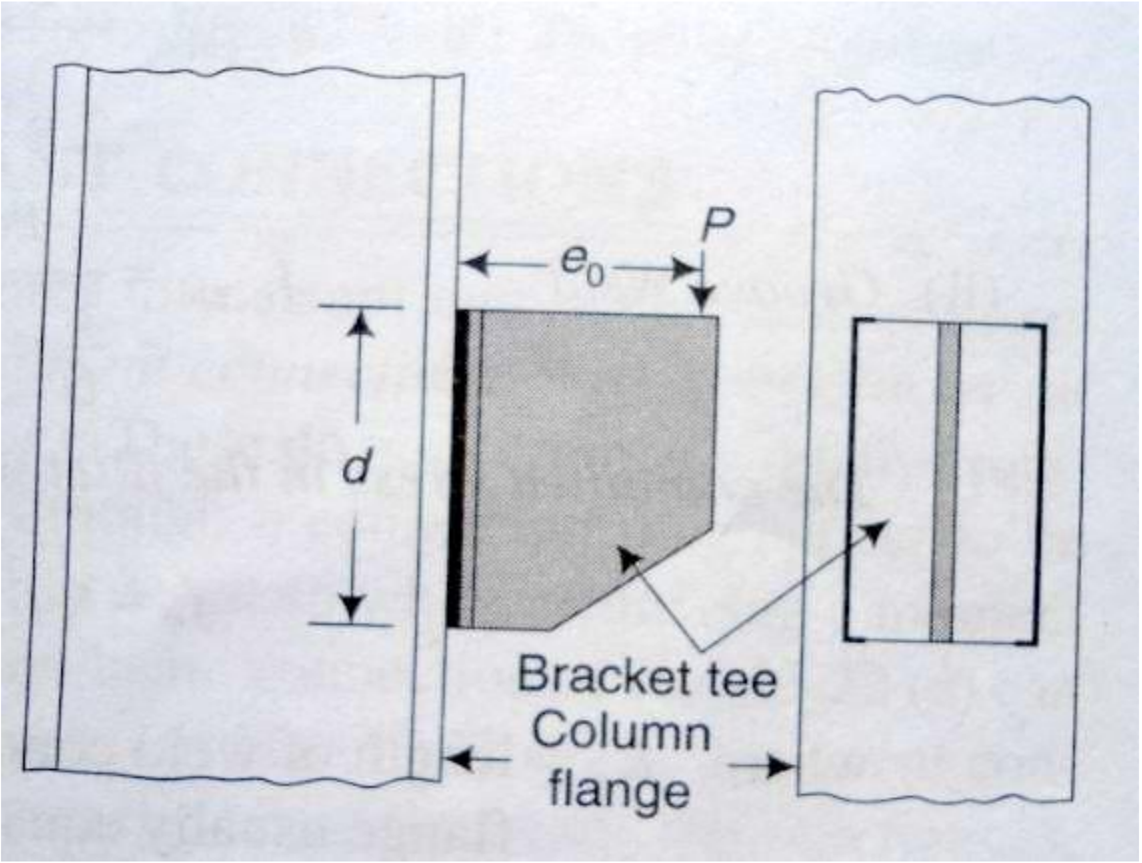
$y_1, y_{2....} y_n$ = distance of the bolts in tension from the axis of rotation

y = distance as shown in figure.

WELDED BRACKET CONNECTIONS TYPE 1



WELDED BRACKET CONNECTIONS TYPE 2



Experimental Analysis

Reference paper -The Strength of Eccentrically Loaded Shear Connections (Ultimate Method)

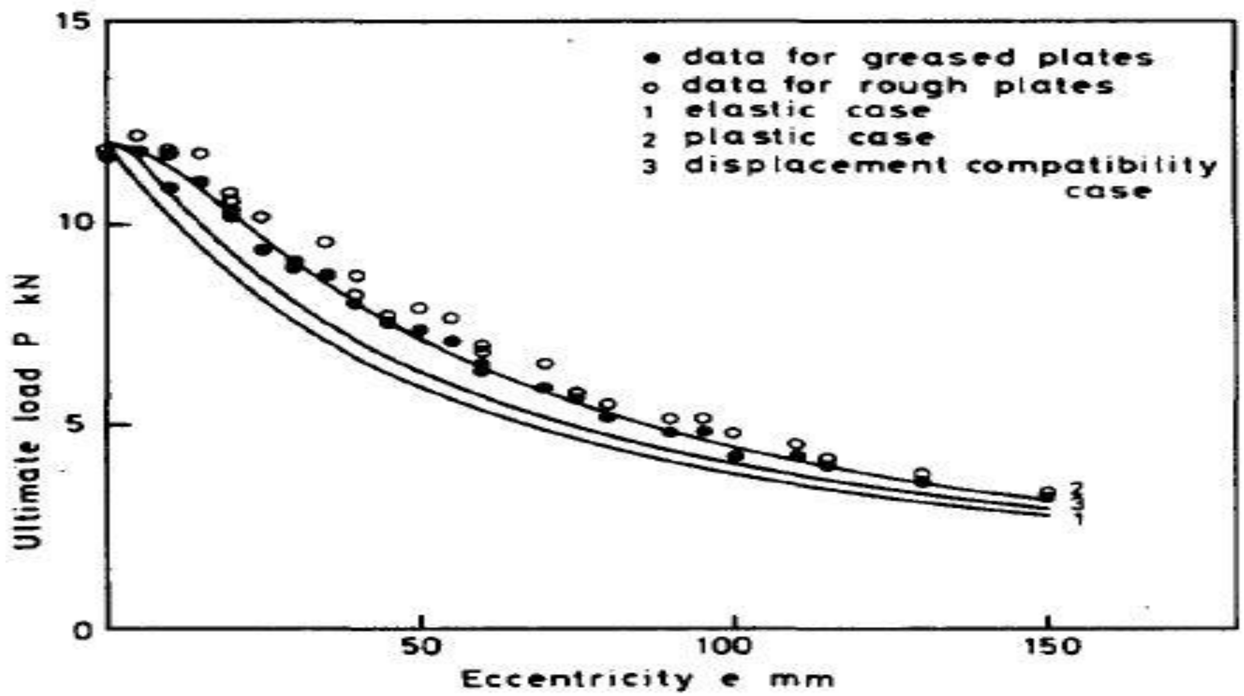


Figure 12 Ultimate load versus eccentricity for 2 by 2 groups of hollow steel rivets, pitch and gauge 60 mm

Reference paper - Eccentric Connection design- geometric approach
(Geometric Method)

TABLE 1. Comparison of Methods and Experimental Results

Mark (1)	e in. (2)	e cm (3)	$n_x \times n_y$ (4)	S_y in. (5)	S_y cm (6)	S_x in. (7)	S_x cm (8)	Test kips (9)	Test (kN) (10)	C_{test} (11)	C_g (12)	C_{ict} (13)	C_a (14)
(a) Normalized Bolt Capacities													
B1	8	20	1 × 5	2.5	6	—[a]	—[a]	225	1,001	1.52	1.62	1.69	1.70
B2	10	25	1 × 5	3	8	—[a]	—[a]	240	1,023	1.55	1.57	1.63	1.66
B3	12	30	1 × 5	3	8	—[a]	—[a]	190	485	1.28	1.35	1.39	1.39
B4	13	33	1 × 6	3	8	—[a]	—[a]	251	1,116	1.70	1.84	1.89	1.88
B5	15	38	1 × 6	3	8	—[a]	—[a]	221	983	1.49	1.63	1.66	1.65
B6	12	30	2 × 4	3	8	2.5	6	264	1,174	1.78	1.86	1.89	2.02
B7	15	38	2 × 4	3	8	2.5	6	212	943	1.43	1.52	1.53	1.61
B8	15	38	2 × 5	2	6	2.5	6	266	1,183	1.80	1.88	1.91	2.09
(b) Normalized Rivet Capacities													
TP-1	2.5	6	1 × 3	3	8	—[a]	—[a]	216	961	2.02	1.51	1.78	2.00
TP-2	3.5	9	1 × 3	3	8	—[a]	—[a]	161	716	1.51	1.28	1.43	1.64
TP-3	6.5	17	1 × 3	3	8	—[a]	—[a]	100	445	0.94	0.82	0.86	0.92
TP-4	2.5	6	1 × 6	3	8	—[a]	—[a]	550	2,446	5.15	4.36	4.55	5.22
TP-5	4.5	11	1 × 6	3	8	—[a]	—[a]	440	1,957	4.12	3.61	3.82	4.25
TP-6	6.5	17	1 × 6	3	8	—[a]	—[a]	362	1,610	3.39	3.00	3.17	3.38
TP-7	3.5	9	2 × 2	3	8	2.5	6	222	987	2.08	1.60	1.53	1.85
TP-8	6.5	17	2 × 2	3	8	2.5	6	120	534	1.12	0.89	0.88	1.15
TP-9	3.5	9	2 × 4	3	8	2.5	6	568	2,526	5.32	4.59	4.87	5.28
TP-10	6.5	17	2 × 4	3	8	2.5	6	354	1,575	3.31	3.11	3.22	3.48

[a]Not applicable.

Example Problem

An Industrial building of plan 15m×30m is to be constructed as shown in Fig.E1. Using plastic analysis, analyse and design the single span portal frame with gabled roof. The frame has a span of 15 m, the column height is 6m and the rafter rise is 3 m and the frames are spaced at 5 m centre-to-centre. Purlins are provided over the frames at

2.7 m c/c and support AC sheets. The dead load of the roof system including sheets, purlins and fixtures is 0.4 kN/m^2 and the live load is 0.52 kN/m^2. The portal frames support a gantry girder at 3.25 m height, over which an electric overhead travelling (EOT) crane is to be operated. The crane capacity is to be 300 kN and the crane girder weighs 300 kN while the crab (trolley) weight is 60 kN.

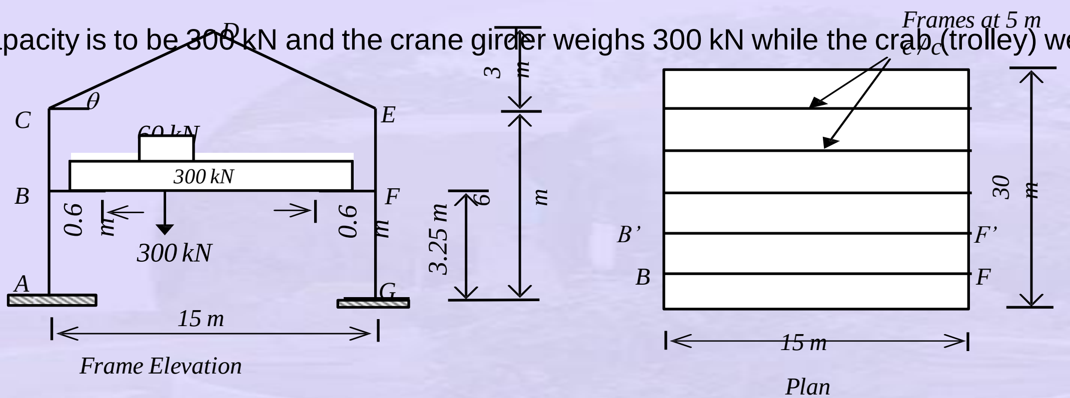


Fig. E1 Details of an Industrial Building

1.0 Load Calculations

1.1 Dead Load of roof given as 0.4 kN/m^2

1.2 Dead load/m run on rafter = 0.4 * 5 ≈ 2.0 kN/m

Live load/m run on rafter = 0.52 * 5 ≈ 2.6 kN/m

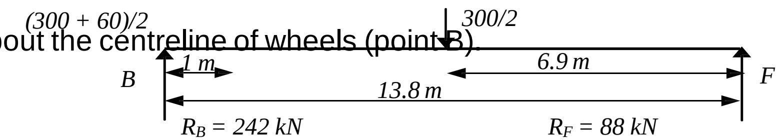
1.3 Crane Load

The extreme position of crane hook is assumed as 1 m from the centre line of rail. The span of crane is approximately taken as 13.8 m. And the wheel base along the gantry girder has been taken as 3.8 m

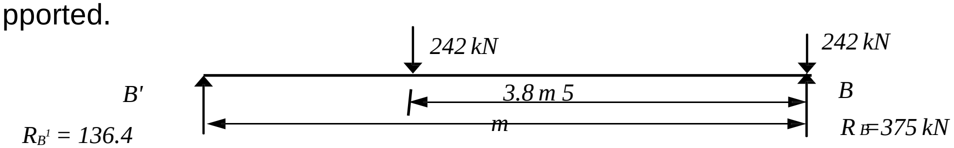
1.3.1 Vertical load on gantry

The weight of the crane is shared by two portal frames At the extreme position of

crab, the reaction on wheel due to the lifted weight and the crab can be obtained by taking moments about the centreline of wheels (point B).



To get maximum wheel load on a frame from gantry girder BB', taking the gantry girder as simply supported.



Centre to centre distance between frames is 5 m c/c. Assuming

impact factor of 25%

Maximum wheel Load @ B = 1.25 (242 (1 + (5-3.8)/5)

= 375 kN.

Minimum wheel Load @ B = (88 /242)*375

=136.4 kN

1.3.2 Transverse Load (Surge):

Lateral load per wheel = 5% (300 + 60)/2 = 9 kN

(i.e. Lateral load is assumed as 5% of the lifted load and the weight of the crab acting on each rail).

Lateral load on each column = $\frac{9}{242} * 375$ = 13.9 kN

(By proportion)

1.4 Wind Load

Design wind speed, $V_z = k_1\ k_2\ k_3\ V_b$ From Table 1; IS: 875 (part 3) – 1987

k_1 = 1.0 (risk coefficient assuming 50 years of design life)

From Table 2; IS: 875 (part 3) – 1987 k_2 = 0.8 (assuming terrain category 4) k_3 = 1.0 (topography factor)

Assuming the building is situated in Chennai, the basic wind speed is 50 m /sec

Design wind speed, $V_z = k_1\ k_2\ k_3\ V_b$ V_z =

1 * 0.8 *1 * 50

V_z = 40 m/sec

Design wind pressure, $P_d = 0.6*V_z^2$

$= 0.6 * (40)^2$

$= 0.96$ kN/m^2

1.4.1. Wind Load on individual surfaces

The wind load, W_L acting normal to the individual surfaces is given by $W_L = (C_{pe} - C_{pi})\ A^*P_d$

(a)Internal pressure coefficient

Assuming buildings with low degree of permeability

$C_{pi} = \pm 0.2$

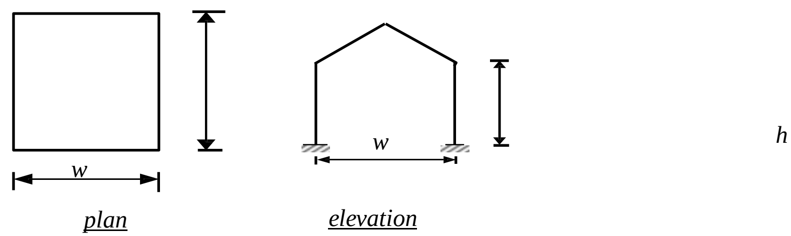
(b)External pressure coefficient

External pressure coefficient for walls and roofs are tabulated in Table 1 (a) and Table 1(b)

1.4.2 Calculation of total wind load

(a) For walls

h/w = 6/15 = 0.4 L/w = 30/15 = 2.0



Exposed area of wall per frame @ 5 m c/c is A = 5 * 6 = 30 m^2

Wind load on wall / frame, A p_d = 30 * 0.96 = 28.8 kN

Table 1 (a): Total wind load for wall

Wind Angle θ	C_{pe}		C_{pi}	$C_{pe} - C_{pi}$		Total wind(kN) $(C_{pe}-C_{pi})Ap_d$	
	Wind- ward	Lee- ward		Wind ward	Lee ward	Wind ward	Lee ward
0^0	0.7	-0.25	0.2	0.5	-0.45	14.4	-12.9
			-0.2	0.9	-0.05	25.9	-1.4
90^0	-0.5	-0.5	0.2	-0.7	-0.7	-20.2	-20.2
			-0.2	-0.3	-0.3	-8.6	-8.6

(b) For roofs

Exposed area of each slope of roof, per frame (5m length) is

$$A = 5 * \sqrt{(3.0)^2 + (7.5)^2} = 40.4\,m^2$$

For roof, Ap_d = 38.7 kN

Wind angle	Pressure Coefficient			$C_{pe} - C_{pi}$		Total Wind Load(kN) $(C_{pe} - C_{pi})\,Ap_d$	
	C_{pe}	C_{pe}	C_{pi}	Wind ward	Lee ward	Wind ward	Lee ward
	Wind	Lee				Int.	Int.
0^0	-0.328	-0.4	0.2	-0.528	-0.6	-20.4	-23.2
	-0.328	-0.4	-0.2	-0.128	-0.2	-4.8	-7.8
90^0	-0.7	-0.7	0.2	-0.9	-0.9	-34.8	-34.8
	-0.7	-0.7	-0.2	-0.5	-0.5	-19.4	-19.4

2.1 Dead Load

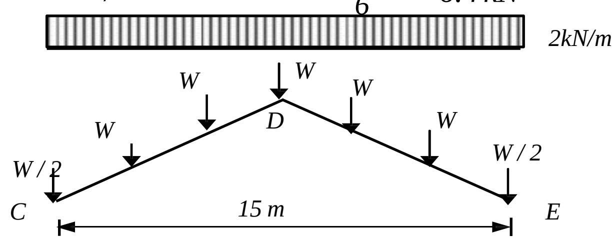
Replacing the distributed dead load of 2kN/m on rafter by equivalent concentrated loads at two intermediate points corresponding to purlin locations on each rafter,

$$W_D = \frac{2.0 * 15}{6} = 5kN$$

2.2 Superimposed Load

Superimposed Load = 2.57 kN/m

Concentrated load , $W_L = \frac{2.57 * 15}{6} = 6.4\,kN$



2.3 Crane Load

Maximum Vertical Load on columns = 375 kN (acting at an eccentricity of 600 mm from column centreline)

Moment on column = 375 *0.6 = 225 kNm.

Minimum Vertical Load on Column = 136.4 kN (acting at an eccentricity of 600 mm) Maximum moment = 136.4 * 0.6 = 82 kNm

1. Partial Safety Factors

2. Load Factors

For dead load, γ_f = 1.5

For leading live load, γ_f = 1.5

For accompanying live load, γ_f = 1.05

3. Material Safety factor

$$\gamma_m = 1.10$$

4.0 Analysis

In this example, the following load combinations is considered, as it is found to be critical. Similar steps can be followed for plastic analysis under other load combinations.

(i) 1.5D.L + 1.5 C .L + 1.05 W.L

4.1. 1.5 D.L + 1.5 C.L+ 1.05 W.L

4.1.1Dead Load and Wind Load along the ridge (wind angle = 0 º)

(a) Vertical Load

w @ intermediate points on windward side w = 1.5 * 5.0 – 1.05 *(4.8/3) cos21.8

= 6 kN.

$$w_{@\ eaves} = \frac{6}{2} = 3.0\ kN$$

$$\frac{w}{2} @ \, eaves = \frac{5.0}{2} = 2.5 \, kN$$

Total vertical load @ the ridge = 3.0 + 2.5 = 5.5 kN

b) Horizontal Load

H @ intermediate points on windward side H = 1.05 * 4.8/3 sin

21.8

= 0.62 kN

H/2 @ eaves points = 0.62/2

= 0.31 kN

H @ intermediate purlin points on leeward side

= 1.05 * 7.8 /3 sin 21.8

= 1 kN

H/2 @ eaves = 0.5 kN

Total horizontal load @ the ridge = 0.5 - 0.31 = 0.19 kN

Table 3: Loads acting on rafter points

Intermediate Points	Vertical Load (kN)		Horizontal Load (kN)	
	Windward	Leeward	Windward	Leeward
	5.2	4.2	0.62	1.0
Eaves	2.6	2.1	0.31	0.5
Ridge	4.7		0.19	

4.1.2 Crane Loading

Moment @ B = 1.5 * 225 = 337.5 kNm

Moment @ F = 1.5 * 82 = 123 kNm

Horizontal load @ B & @ F = 1.5 * 13.9 = 20.8 kN

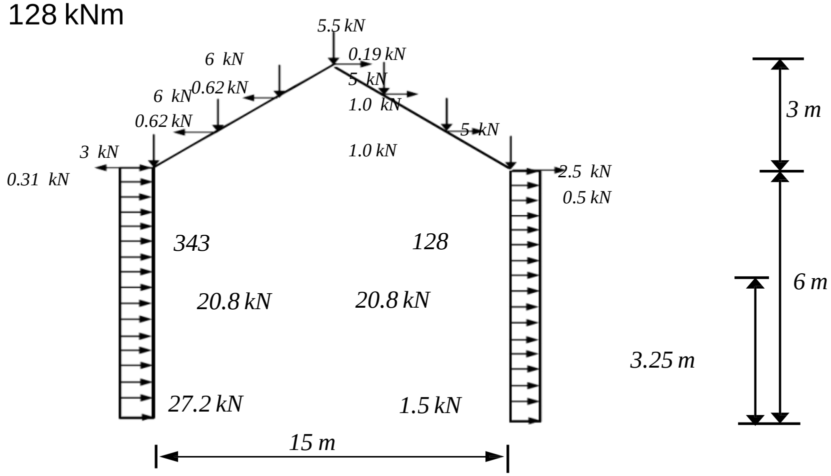
Note: To find the total moment @ B and F we have to consider the moment due to the dead load from the weight of the rail and the gantry girder. Let us assume the weight of rail as 0.3 kN/m and weight of gantry girder as 2.0 kN/m

Dead load on the column = $\left(\frac{2+0.3}{2}\right)*5 = 5.75\ kN$ acting at e=0.6m

Factored moment @ B & F = 1.5 * 5.75 * 0.6 = 5.2 kNm Total moment

@B = 337.5 + 5.2 = 342 kNm

@ F = 123 + 5.2 = 128 kNm



Factored Load (1. 5D.L+1.5 C.L +1.05 W.L)

4.2 1.5 D.L + 1.5 C.L + 1.05 L.L

4.2.1 Dead Load and Live Load

@ intermediate points on windward side = 1.5 * 5.0 + 1.05 * 6.4
= 14.2 kN

@ ridge = 14.2 kN

@ eaves = 14.2 / 2 $\approx$ 7.1 kN.

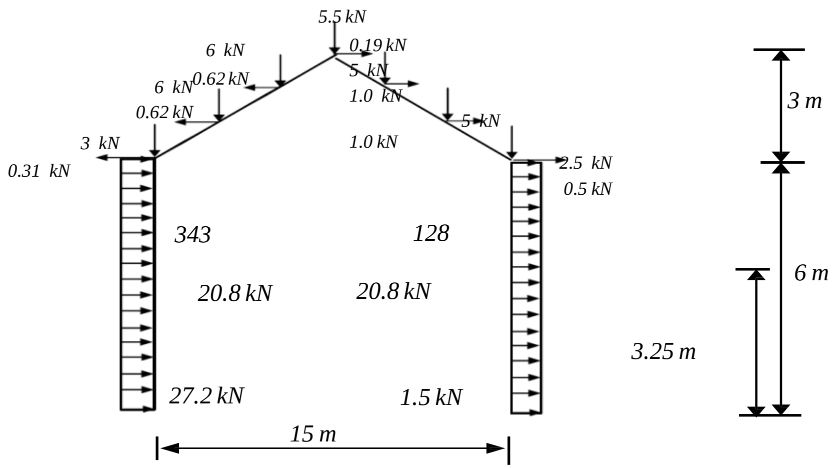
4.2.2 Crane Load

Moment @ B = 342 kNm Horizontal load

@ B = 20.8 kN

Moment @ F = 128 kNm Horizontal load

@ F = 20.8 kN



Factored Load (1. 5D.L+1.5 C.L +1.05 W.L)

4.3 Mechanisms

We will consider the following mechanisms, namely

(i) Beam mechanism

(ii) Sway mechanism

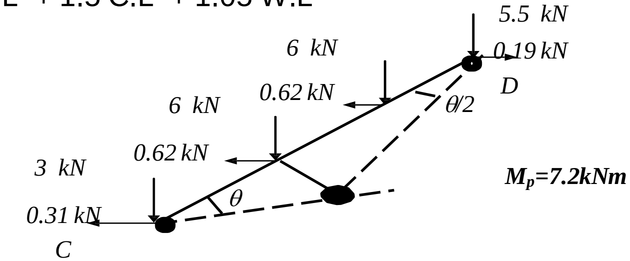
(iii) Gable mechanism and

(iv) Combined mechanism

(v) Beam Mechanism

(1) Member CD

Case 1: 1.5 D.L + 1.5 C.L + 1.05 W.L



Internal Work done, Wi = $M_p\theta$ + M_p (θ/2) + M_p (θ + θ/2)

= $M_p(3\theta)$

External Work done, W_e = 6 * 2.5θ - 0.62 * 1 * θ + 6 * 2.5 * θ/2 – 0.62 * 1 * θ/2

= 21.6θ

Equating internal work done to external work done

W_i = W_e

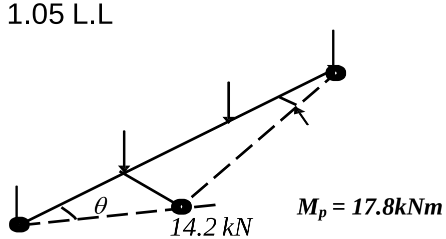
M_p (3θ) = 21.6θ

M_p = 7.2 kNm

Case 2: 1.5 D.L + 1.5 C.L + 1.05 L.L

Internal Work done,

W_i = M_p 3θ (as in case 1)



External work done, W_e = 14.2 *2.5 θ + 14.2 *2.5θ / 2

= 53.3θ

14.2 kN

14.2 kN

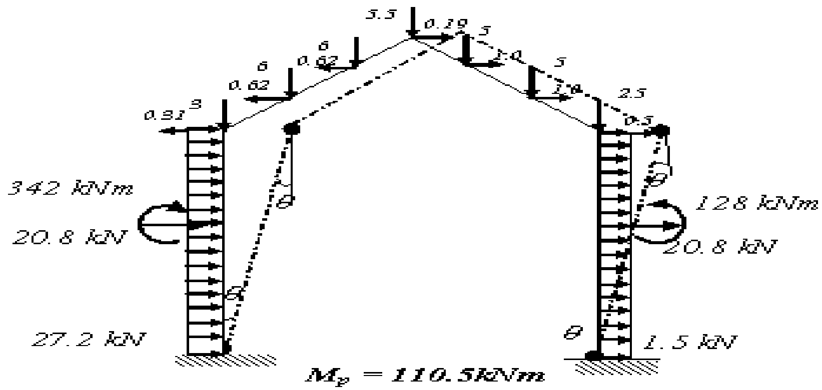
θ/2

7.1 kN

.

4.3.1 Panel Mechanism

Case 1: 1.5 D.L + 1.5 C.L + 1.05 W.L



Internal Work done, W_i = M_p (θ) + M_p (θ) + M_p (θ) + M_p (θ)

$$= 4M_p\theta$$

External Work done, W_e

W_e = 1/2 (27.2) * 6θ + 20.8 * 3.25θ + 342θ - 0.31 * 6θ - 0.62 * 6θ - 0.62 (6θ)+ 0.19 * 6θ + 1.0 *6θ + 1.0 * 6θ + 0.5 * 6θ+1/2 (1.5) * 6θ + 20.8 * 3.25θ - 128 * θ

= 442.14θ

Equating W_i = W_c, we get $4M_p\theta$ = 442.14θ

M_p = 110.5 kNm

The second load combination will not govern.

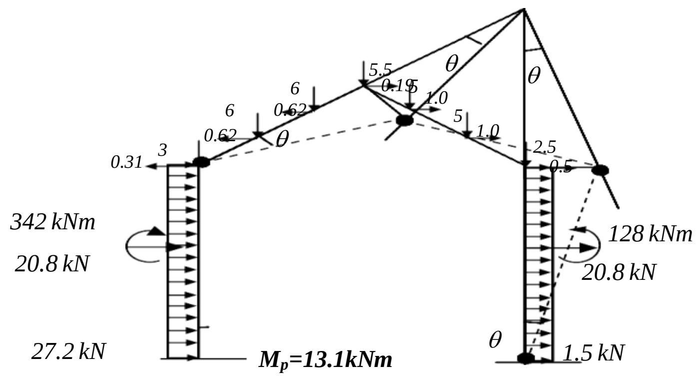
4.3.3 Gable Mechanism

Case 1: 1.5 D.L + 1.05 W.L + 1.5 C.L

Internal Work done = $M_p\theta + M_p2\theta + M_p\ (2\theta) + M_p\theta = 6M_p\theta$ External Work done, W_e =

$-0.62 * 1 * \theta - 0.62 * 2 *\theta + 0.19 * 3 * \theta + 1.0 * 4 * \theta + 1.0 * 5 * \theta + 0.5 * 6 * \theta + 6 * 2.5 * \theta + 6 * 5 * \theta + 5.5 * 7.5 * \theta + 5 * 5 * \theta + 5 * 2.5 * \theta + ½ * 1.5 * 6\theta + 20.8 * 3.25 * \theta - 128*\theta$

$W_e = 78.56\theta$

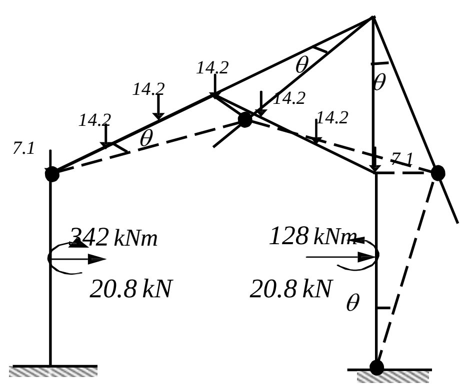


Equating $W_i = W_e$, we get $6M_p = 78.56\theta$

M_p = 13.1 kNm.

Case 2: 1.5 D.L + 1.05L.L + 1.5 C.L

M_p=37.3kNm



Internal Work done, W_i = $M_p\theta + M_p(2\theta) + M_p(2\theta) + M_p\theta = 6M_p\theta$ External Work done, W_e

$= 14.2 * 2.5*\theta + 14.2 * 5 * \theta + 14.2 * 7.5\theta + 14.2 * 5 * \theta + 14.2 * 2.5\theta - 128 * \theta + 20.8 * 3.25\theta$
$= 223.6\theta$

Equating $W_i = W_e$, we get

$6M_p\theta = 223.6\theta$

$M_p = 37.3$ kNm

4.3.4 Combined Mechanism

Case1: 1.5 D.L + 1.05 W.L + 1.5 C.L

(i)

Internal Work done, $W_i = M_p(\theta) + M_p(\theta + \theta/2) + M_p(\theta/2 + \theta/2) + M_p(\theta/2)$

$= M_p(\theta + \theta + \theta/2 + \theta/2 + \theta/2 + \theta/2 + \theta/2)$

$= 4 M_p\theta$

$$M_p = 100.7$$

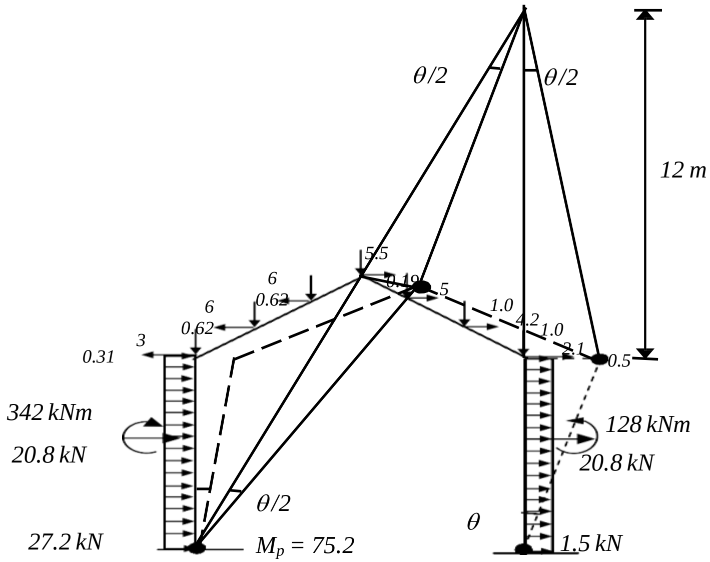
External Work done, W_e=
$1/2 * 27.2 * 6\theta + 20.8 * 3.25* \theta + 342\theta - 0.31 * 12 * \theta/2 - 0.62 * 11 * \theta/2$
$- 0.62 * 10 *\theta/2 + 0.19 * 9 * \theta/2 + 1.0 * 8 * \theta /2 + 1.0 * 7 * \theta /2 + 0.5 * 6* \theta/2 + 1/2 (1.5) * 6\theta/2 + 20.8 * 3.25 * \theta/2 - 128$
$* \theta/2 – 6 * 2.5 * \theta/2 – 6 * 5.0 * \theta/2 – 5.5 * 7.5 * \theta/2 – 5 * 5 *$
$\theta/2 – 5 * 2.5 * \theta/2$
$= 402.86\theta$

Equating $W_i = W_e$

$4M_p\theta = 402.86\theta$ $M_p = 100.7$ kNm

(ii) Internal work done, $W_i = M_p\,\theta/2 + M_p(\theta/2 + \theta/2) + M_p(\theta/2 + \theta) + M_p\theta$

$W_i = 4M_p\theta$



External Work done,

$$W_e = 20.8*3.25*\frac{\theta}{2} + 342*\frac{\theta}{2} + \frac{1}{2}*27.2*6*\frac{\theta}{2} - 0.31*6*\frac{\theta}{2} - 0.62*7*\frac{\theta}{2}$$

$$-0.62*8*\frac{\theta}{2} + 0.19*9*\frac{\theta}{2} + 6*2.5*\frac{\theta}{2} + 6*5.0*\frac{\theta}{2} + 5.5*7.5*\frac{\theta}{2} + 1.0*10*\frac{\theta}{2}$$

$$+1.0*11*\frac{\theta}{2} + 0.5*12*\frac{\theta}{2} + 5*5.0*\frac{\theta}{2} + 5*2.5*\frac{\theta}{2} + 20.8*3.25\,\theta - 128*\theta$$

$$+\frac{1}{2}*1.5*6\theta$$

$= 300.85\theta$

Equating $W_i = W_e$, we get $4M_p\theta = 300.85\theta$

$M_p = 75.2$ kNm

Similarly analysis can be performed for hinges occurring at purlin locations also but they have been found to be not critical in this example case

From all the above analysis, the largest value of M_p required was for member EG

under

1.5 DL + 1.5 CL + 1.05 WL

Therefore the Design Plastic Moment = 116.1 kNm.

1. DESIGN

For the design it is assumed that the frame is adequately laterally braced so

that it fails by forming mechanism. Both the column and rafter are analysed assuming equal plastic moment capacity. Other ratios may be adopted to arrive at an optimum design solution.

2. Selection of section

Plastic Moment capacity required= 116 kNm Required section modulus, $Z_p = M_p / f_{yd}$

$$= \frac{(116*10^6)}{250/1.10}$$

$$= 510.4*10^3 \ mm^3$$

ISMB 300 @ 0.46 kN/ m

provides

$Z_p = 683 * 10^{-3}$ mm^3

b = 140 mm

T_i = 13.1 mm

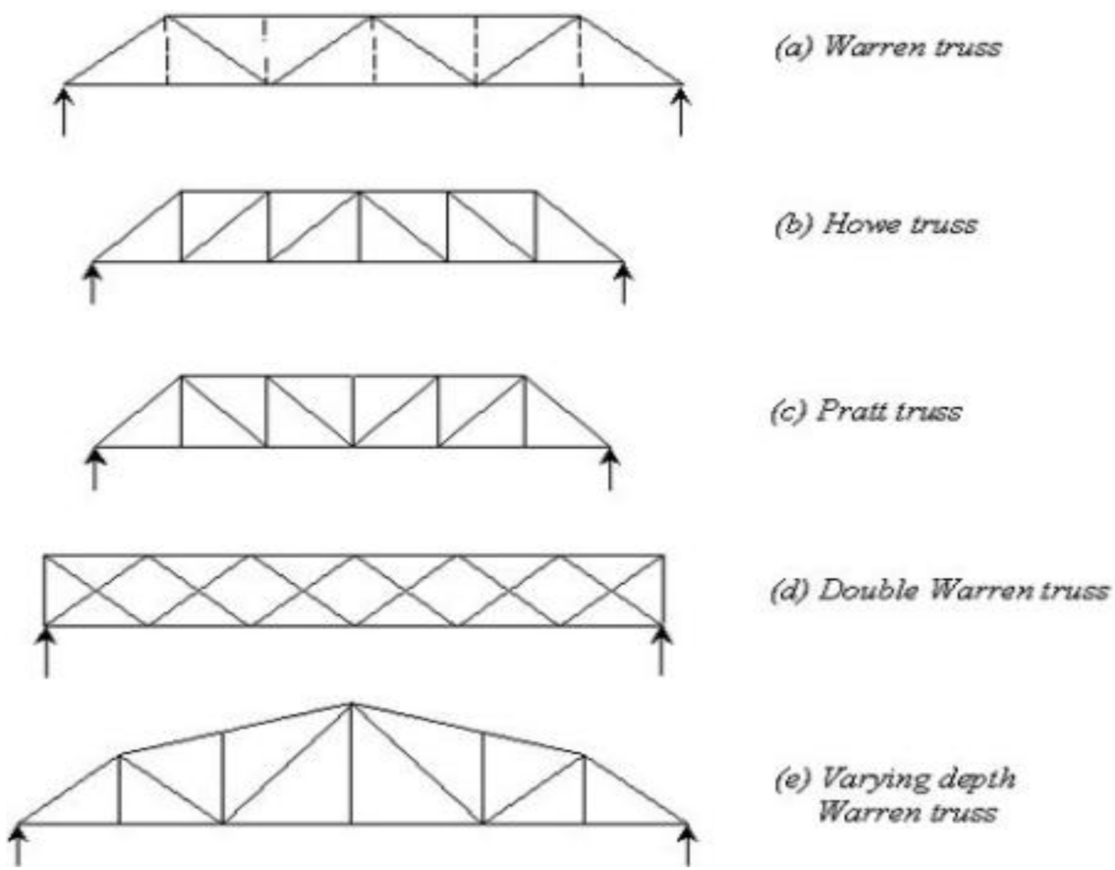
A = 5.87 * 10 ^{3} mm^2

t_w =7.7 mm

r_{xx} =124 mm

r_{yy} =28.6 mm

Truss bridges



some of the trusses that are used in steel bridges

- Truss Girders, lattice girders or open web girders are efficient and economical structural systems, since the members experience essentially axial forces and hence the material is fully utilised.

- Members of the truss girder bridges can be classified as chord members and web members.

- Generally, the chord members resist overall bending moment in the form of direct tension and compression and web members carry the shear force in the form of direct tension or compression.

- Due to their efficiency, truss bridges are built over wide range of spans.

- Truss bridges compete against plate girders for shorter spans, against box girders for medium spans and cable-stayed bridges for long spans.

General design principles

Optimum depth of truss girder

- The optimum value for span to depth ratio depends on the magnitude of the live load that has to be carried.

- The span to depth ratio of a truss girder bridge producing the greatest economy of material is that which makes the weight of chord members nearly equal to the weight of web members of truss.

- It will be in the region of 10, being greater for road traffic than for rail traffic.

Design of compression chord members

❖Generally, the effective length for the buckling of compression chord member in the plane of truss is not same as that for buckling out-of-plane of the truss i.e. the member is weak in one plane compared to the other.

❖The ideal compression chord will be one that has a section with radii of gyration such that the slenderness value is same in both planes.

❖ In other words, the member is just likely to buckle in plane or out of plane.

❖ These members should be kept as short as possible and consideration is given to additional bracing, if economical.

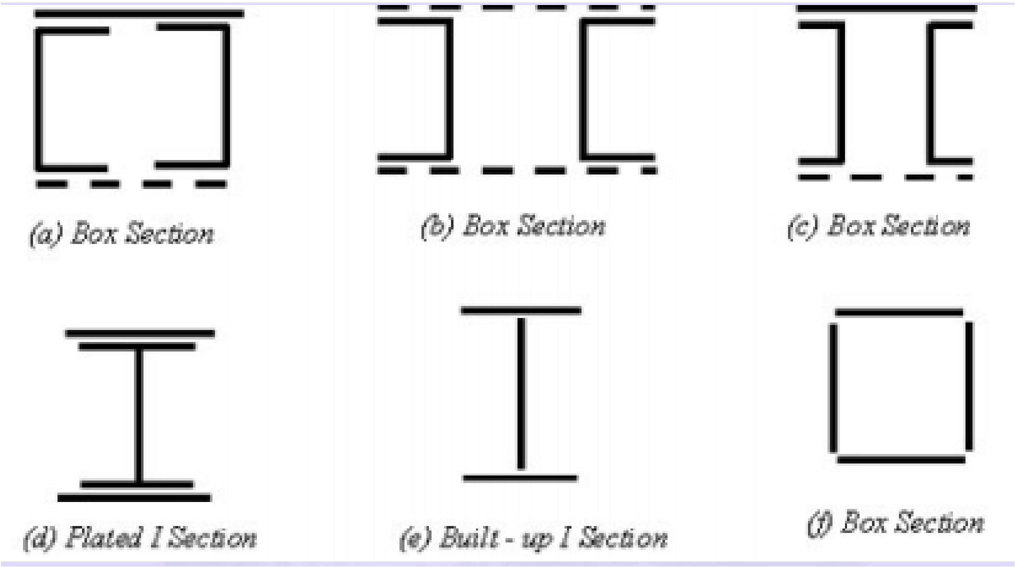
Design of tension chord members

- Tension members should be as compact as possible, but depths have to be large enough to provide adequate space for bolts at the gusset positions and easily attach cross beam.

- The width out-of-plane of the truss should be the same as that of the verticals and diagonals so that simple lapping gussets can be provided without the need for packing.

- It should be possible to achieve a net section about 85% of the gross section by careful arrangement of the bolts in the splices.

- This means that fracture at the net section will not govern for common steel grades.



Typical cross-section for truss members

Design of vertical and diagonal members

✓ Diagonal and vertical members are often rolled sections, particularly for the lightly loaded members, but packing may be required for making up the rolling margins

✓. This fact can make welded members more economical, particularly on the longer trusses where the packing operation might add significantly to the erection cost.

✓Aesthetically, it is desirable to keep all diagonals at the same angle, even if the chords are not parallel.

✓ This arrangement prevents the truss looking overcomplex when viewed from an angle.

✓ In practice, however, this is usually overruled by the economies of the deck structure where a constant panel length is to be preferred.

Lateral bracing for truss bridges

- Lateral bracing in truss bridges is provided for transmitting the longitudinal live loads and lateral loads to the bearings and also to prevent the compression chords from buckling.

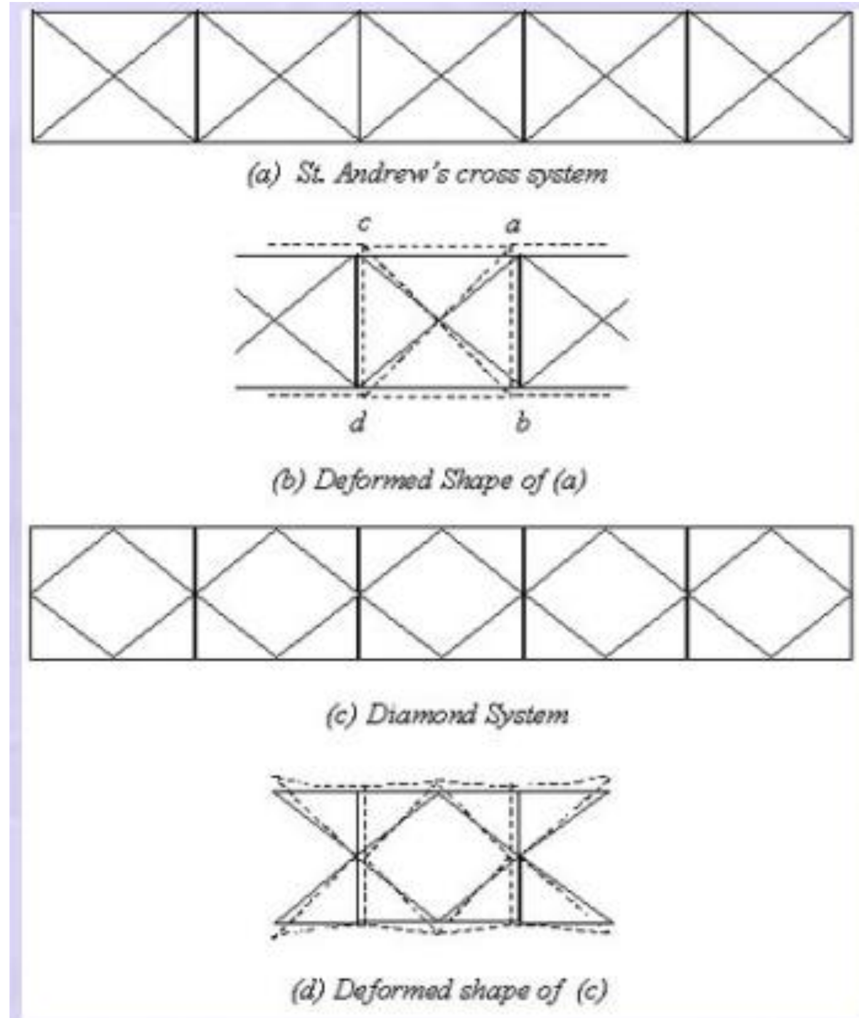
- This is done by providing stringer bracing, braking girders and chord lateral bracing.

- In case of highway truss bridges, concrete deck, if provided, also acts as lateral bracing support system.

- The nodes of the lateral system coincide with the nodes of the main trusses.

- Due to interaction between them the lateral system may cause as much as 6% of the total axial load in the chords.

- This should be taken into account.

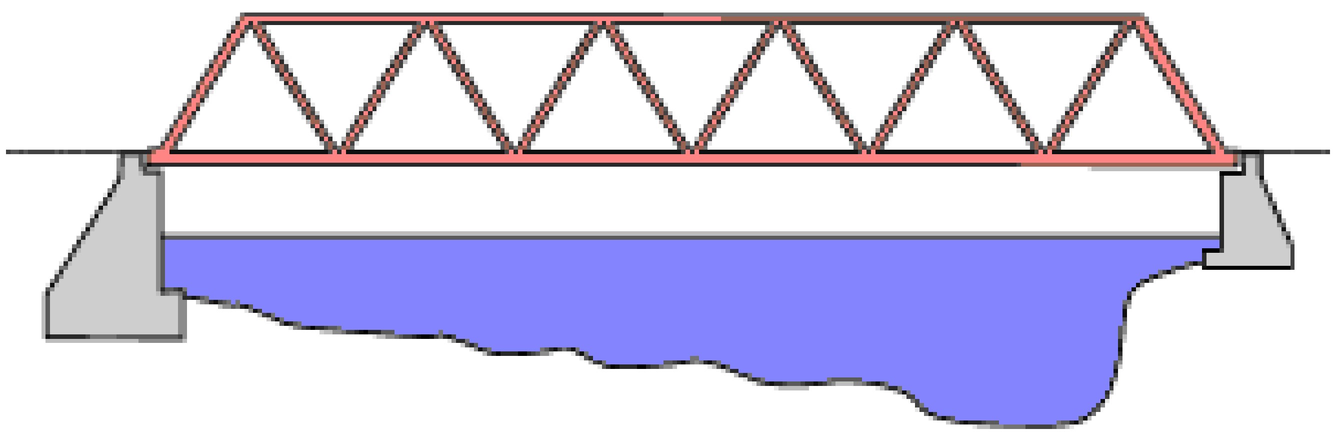


Lateral bracing systems

•It is assumed that wind loading on diagonals and verticals of the trusses is equally shared between top and bottom lateral bracing systems.

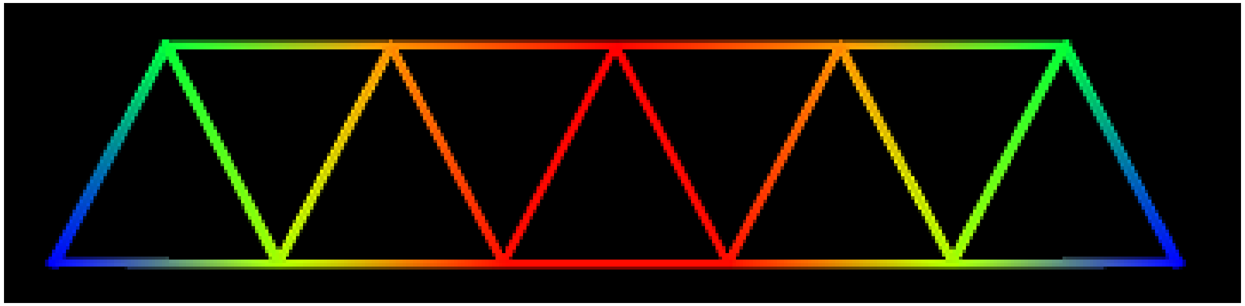
•The end portals (either diagonals or verticals) will carry the load applied to the top chord down to the bottom chord.

•In cases, where only one lateral system exists (as in Semithrough trusses), then the single bracing system must carry the entire wind load.

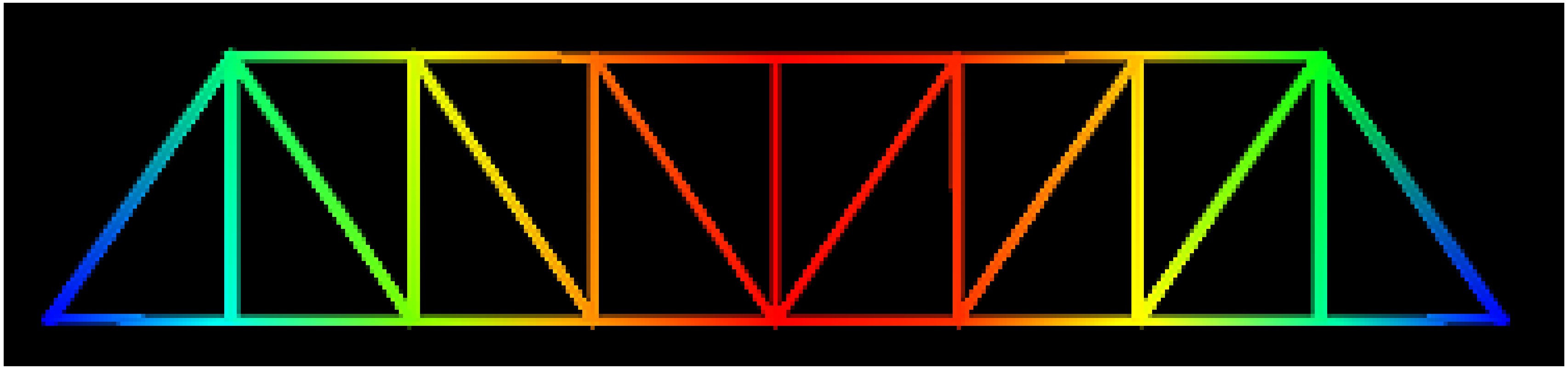


Truss bridges

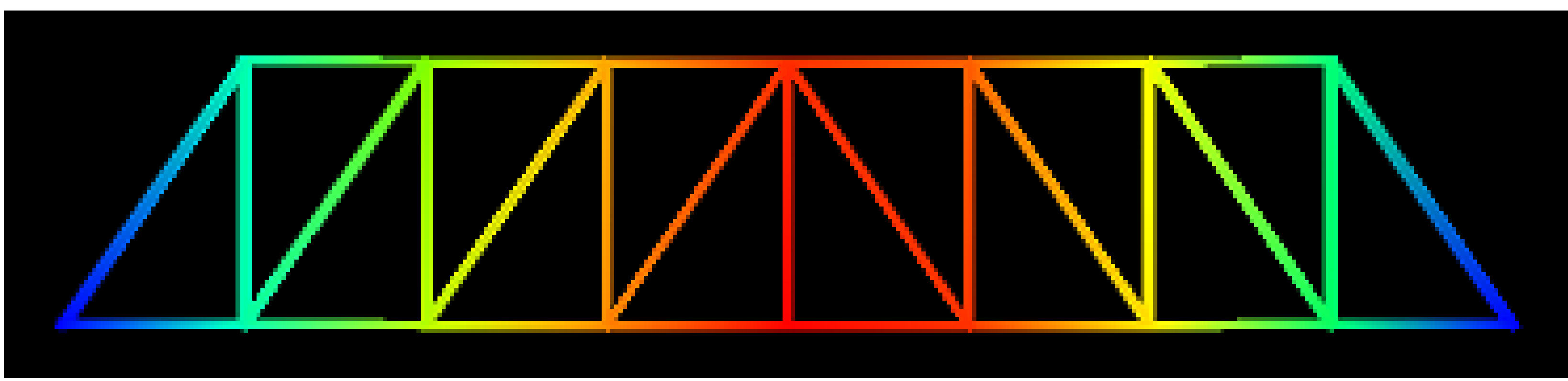
- Warren Truss
  - The Warren truss pattern features a series of isoceles or equilateral triangles. In contrast to the Pratt and Howe patterns, the diagonals alternate in direction.
  - Warren trusses are typically used in spans of between **150-300 feet**
  - The **most common truss.** For smaller spans, no vertical members are used lending the structure a simple look. For longer spans vertical members are added providing extra strength



- Pratt Truss
  - The Pratt truss design contains a downward pointing V in the center with parallel diagonals on each side.
  - Except for those diagonal members near the center, all the diagonal members are subject to tension forces only while the shorter vertical members handle the compressive forces. This allows for thinner diagonal members resulting in a **more economic** design.

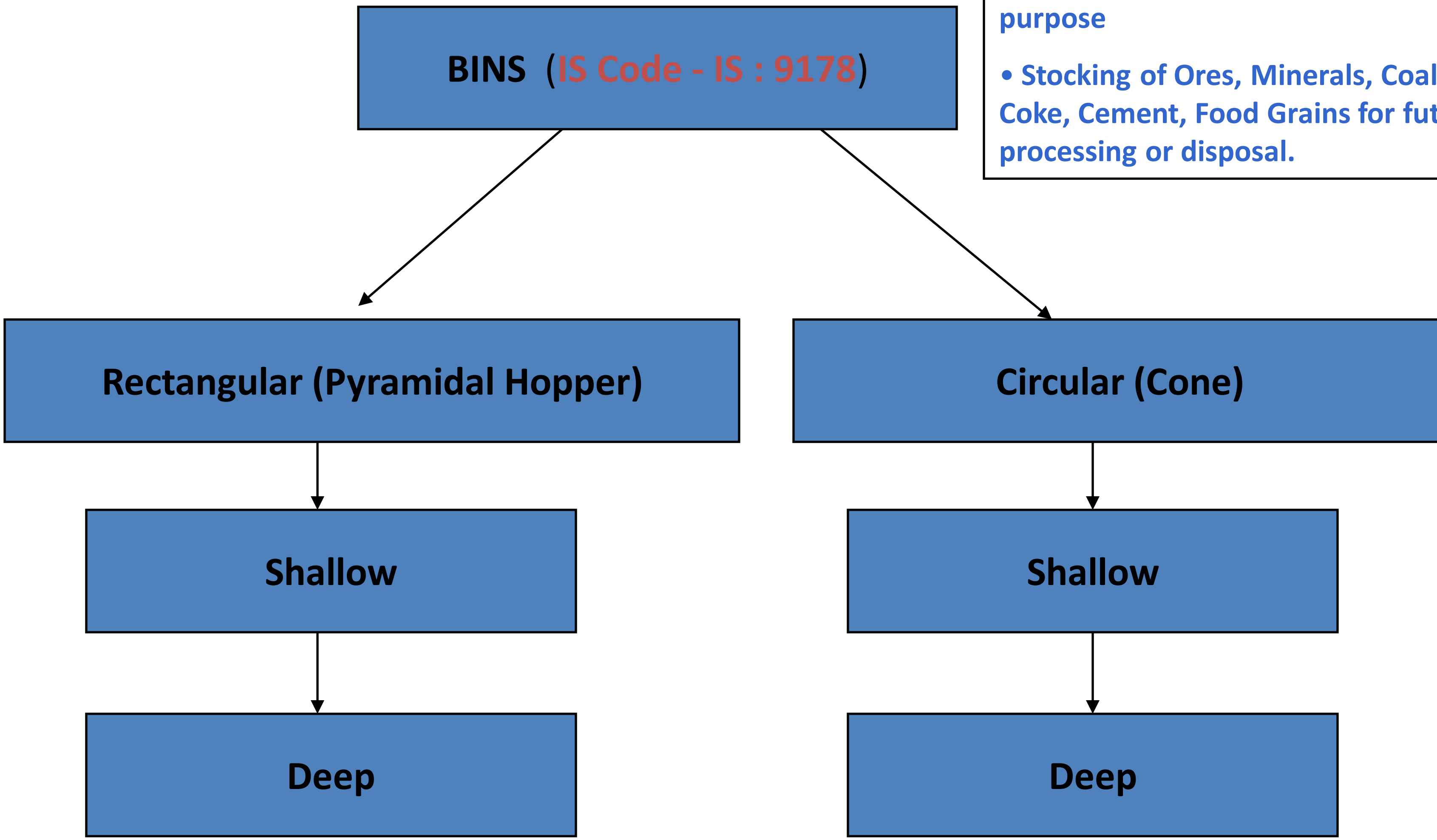


- Howe Truss
  - The Howe truss pattern features an upward pointing V formed by the central diagonals with parallel diagonals on either side. Unlike the Pratt pattern the diagonals will be in compression when loaded
  - It is the opposite of the Pratt truss. The diagonal members face in the opposite direction and handle compressive forces. This makes it very **uneconomic** design for steel bridges and is rarely used.

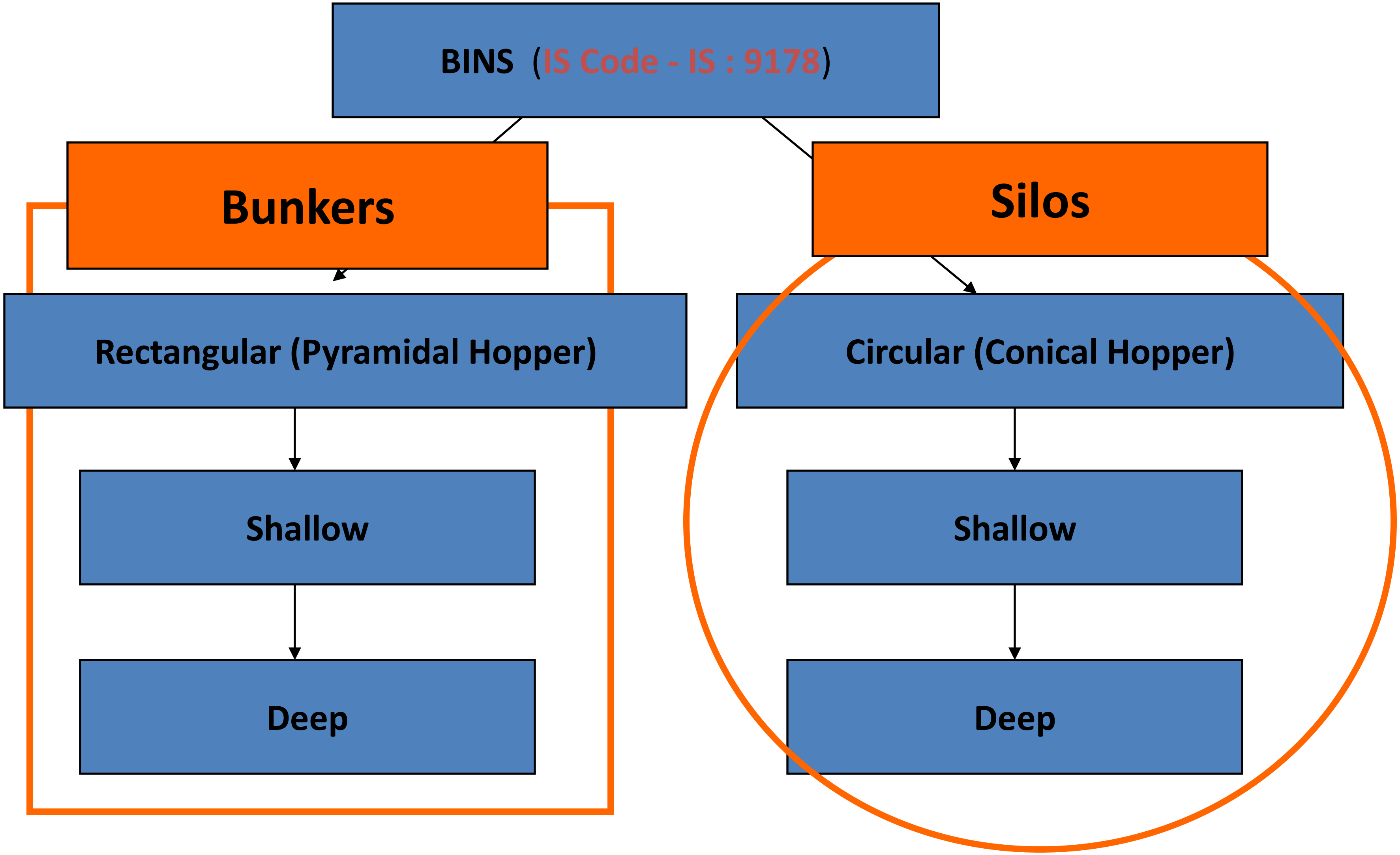


Design of Bins - Bunkers and Silos :: Introduction

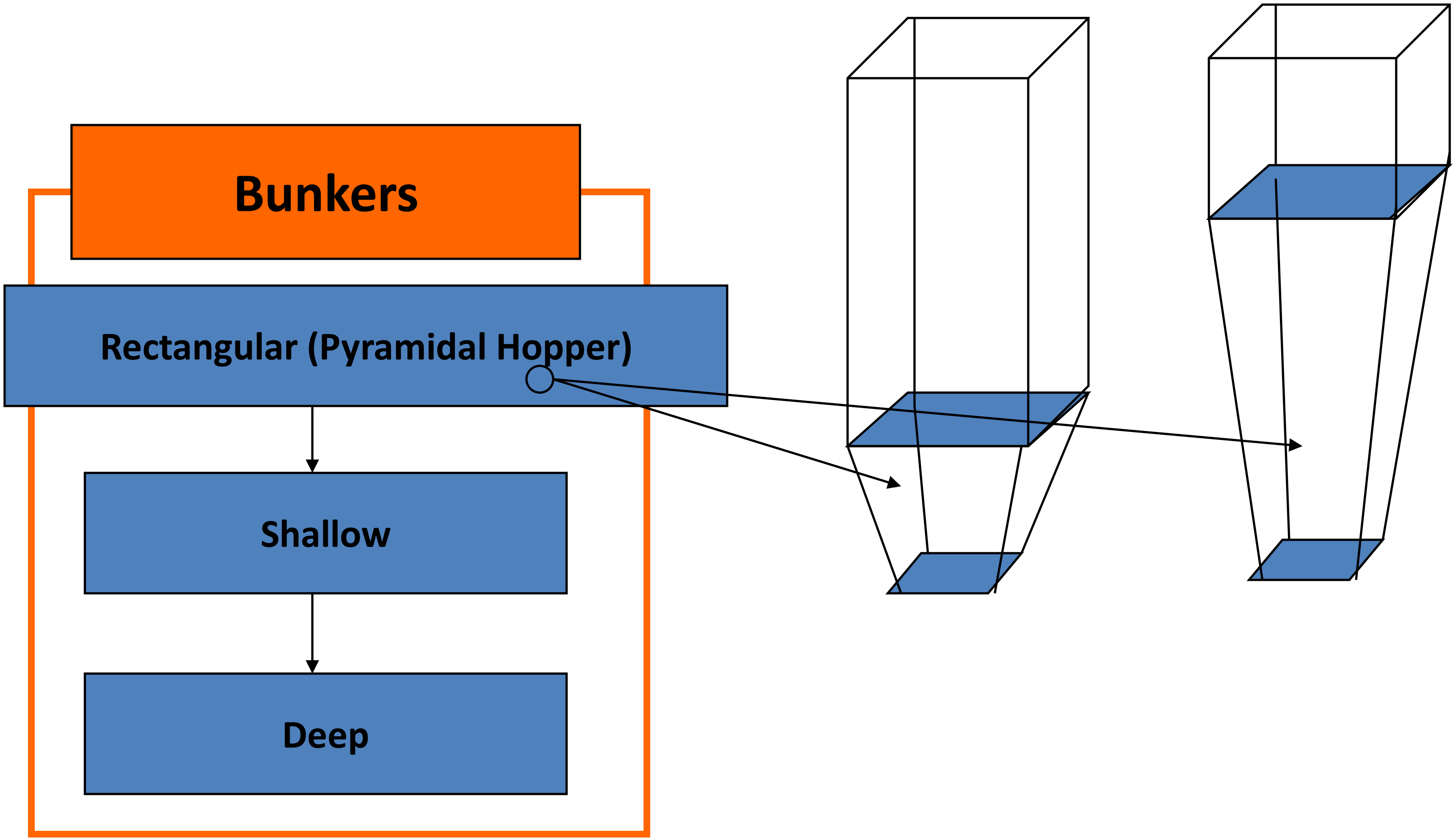
- Storage Containers for Industrial purpose
- Stocking of Ores, Minerals, Coal, Coke, Cement, Food Grains for future processing or disposal.



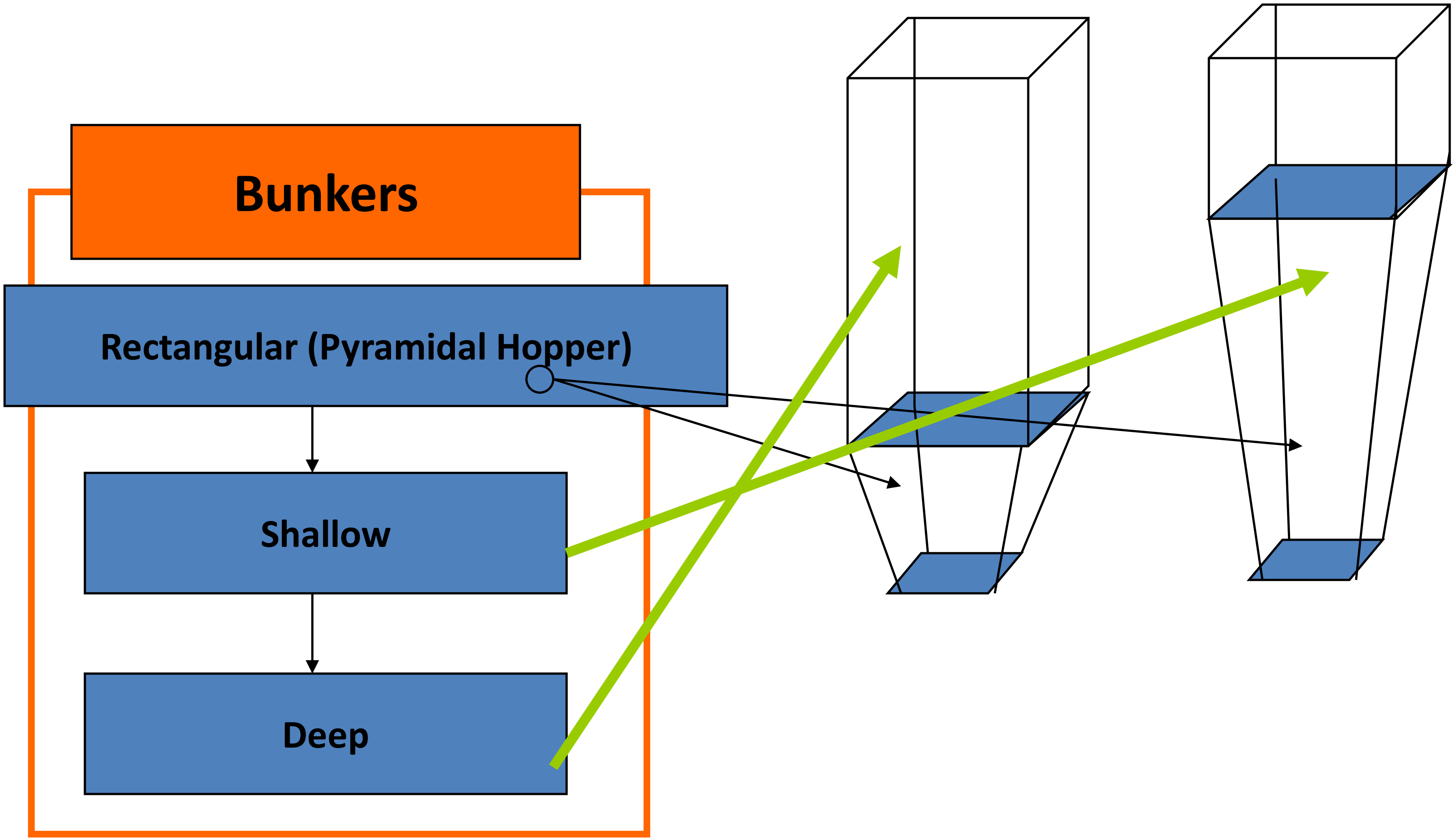
Design of Bins - Bunkers and Silos :: Introduction



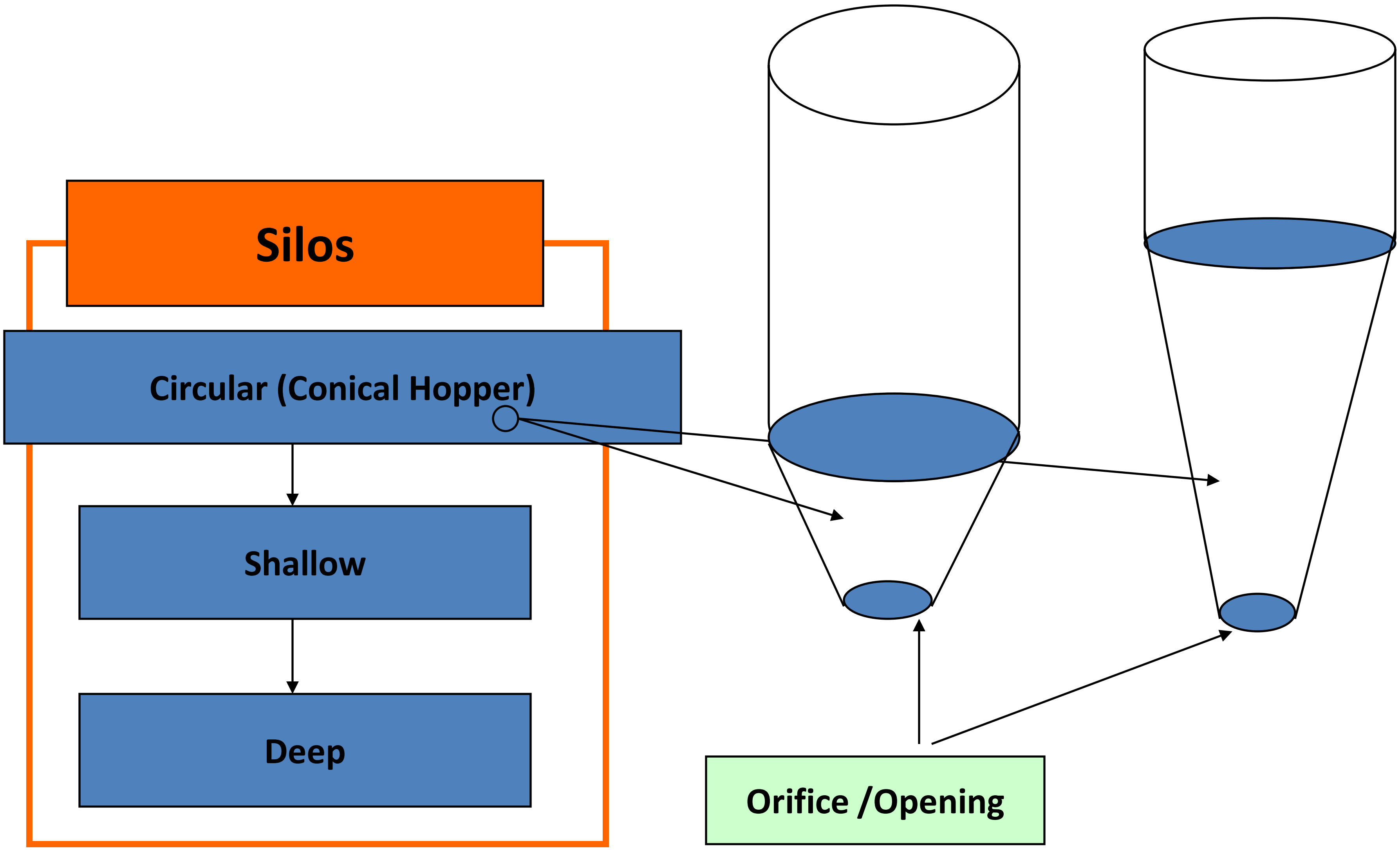
Design of Bins - Bunkers and Silos :: Introduction



Design of Bins - Bunkers and Silos :: Introduction



Design of Bins - Bunkers and Silos :: Introduction



1. **Filling the Bunker – Feed and Loading arrangement at the top.**
    - **Conveyor / Tripper Conveyor Feed**
    - **Bucket Elevator Feed**
    - **Other Mechanical Considerations.**
2. **Emptying the bunker ***
    - **free flow from Bottom opening or orifice**
    - **avoid Material Arching, Make use mechanical vibrator.**
    - **Plan orifice locations to overcome discharge problem.**
3. **Stocking of Material**
    - **Bunker Hopper + Wall system to be designed strong enough to stock the material for the required duration.**
    - **Proper application of Design Theories based on geometry of bunker + Nature of material to be stocked + type of filling and emptying.**

4. **Provision of Wearing Surface – * Use of liner plates and ease of Maintenance**
    - Rubber Liner, SS/MS Plates, Grating
    - Bricks or Tiles

5. **Minimum Slope of Trough**
    - 50 to 60 degree Wall slope.
    - Consider Corner Angle for Pyramidal Bottom.

6. **Guarding Against Over Loading**
    - Application of Load Cells at support point.

7. **Method of Support**
    - Bunker supporting Beam Arrangement.
    - Bunker Supporting Beam Connections with Portal Frames of Building.
    - Battery of Bunkers – Common Beams, Continuous, Multi Span Beams.

8. Materials of Construction & Method of Construction

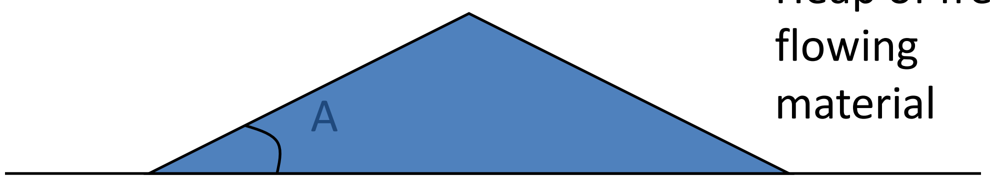
- MS Welded Construction as per IS:800-1984, IS-814,
- MS Plates/Sections IS:2062 Gr-A.
- SAIL-MA or High Yield Stress Wieldable Structural Steel may turn out to be economical for large span bunker beams spanning more than 9 m.

9. Factors of Safety and Working Stresses

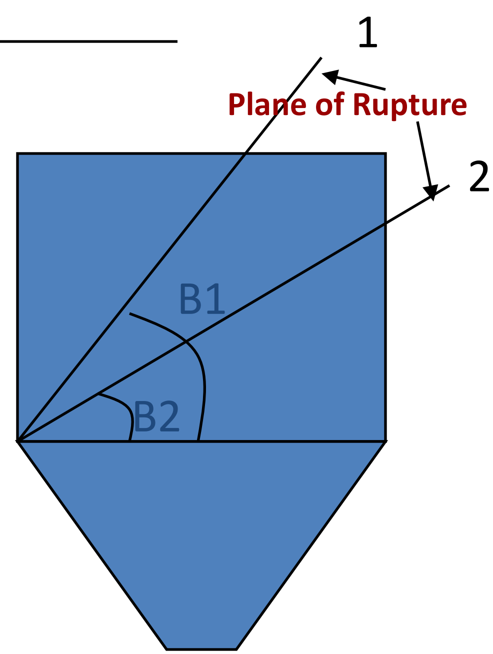
- Building Frame Loading as per IS-875 (Dead + Live + Wind), IS-1893 (Eq. Load) and IS:9178 (Material Density and Angle of repose).
- Working stresses as per IS:800-1984.

Design of Bins - Bunkers and Silos :: Theory relating to Granular Mass

Angle of Repose, A and Angle of Internal friction



Heap of free flowing material



Angle of Rupture, B and Plane of Rupture

$B = 45 + A/2$

B and A are in degrees

Plane of Rupture

Design of Bins - Bunkers and Silos :: Theory relating to Granular Mass – Pressure Calculation

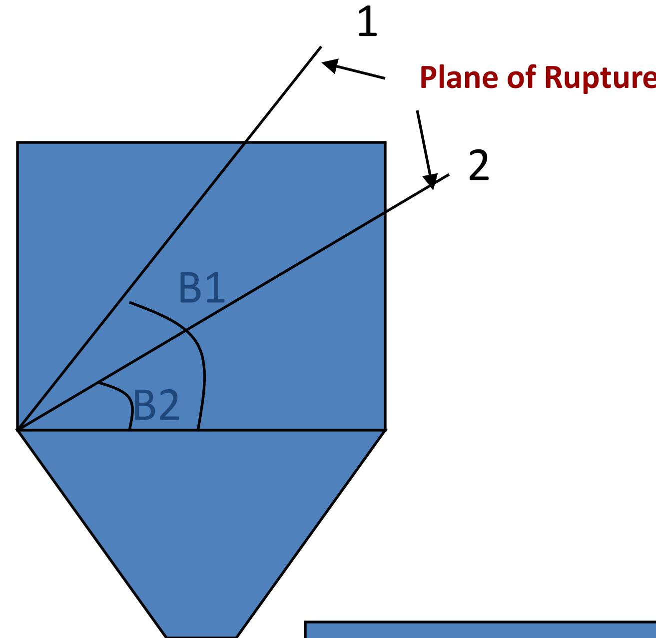
- **Rankines Theory – Case 1**
  - **Incompressible, homogeneous, granular, cohesion less, particle of mass hold together by friction on each other, indefinite extent of mass.**
  - **pv = Y. h**
  - **ph = K.Y.h**
  - **Pn = m .Y. h**

  Where,

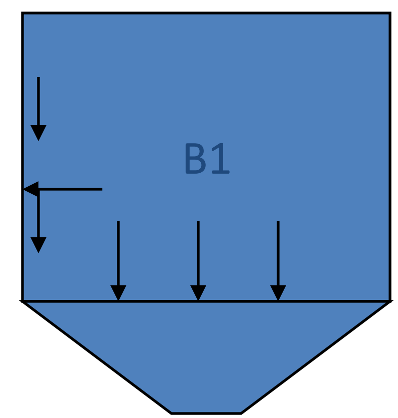
  Y = Volumetric or bulk Density.

  K = Rankine's factor = (1- sin ø) / (1+sin ø)

  $m = \cos^2 \alpha + K . \sin^2 \alpha$



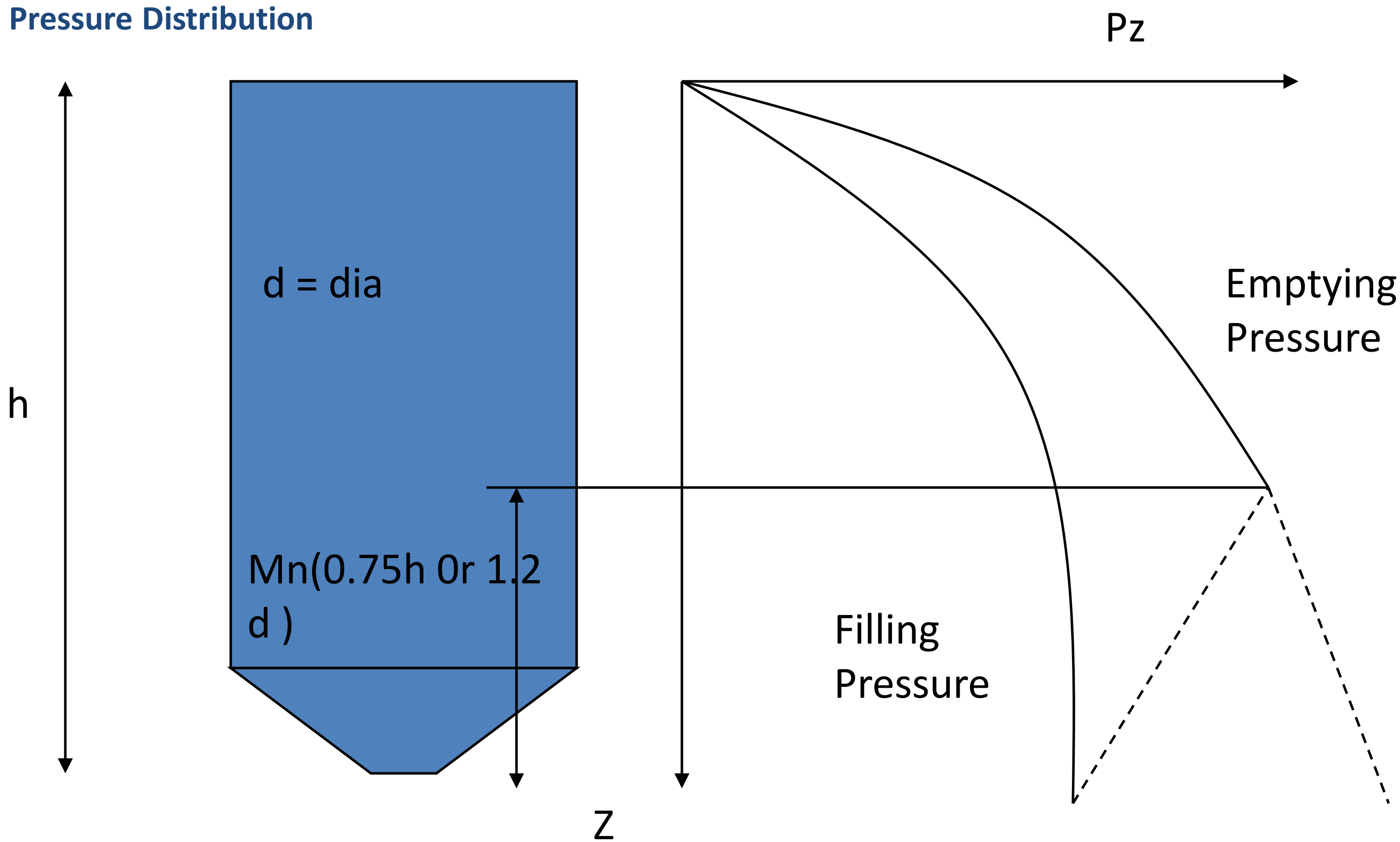
- **Jansen Theory (Recommended by IS:9178) – Case 2**
  - **Friction on the wall predominant and certain quantity of the contents will be carried on the walls due to wall friction.**
  - **Refer IS:1893 part 1 page 14 for formulae.**



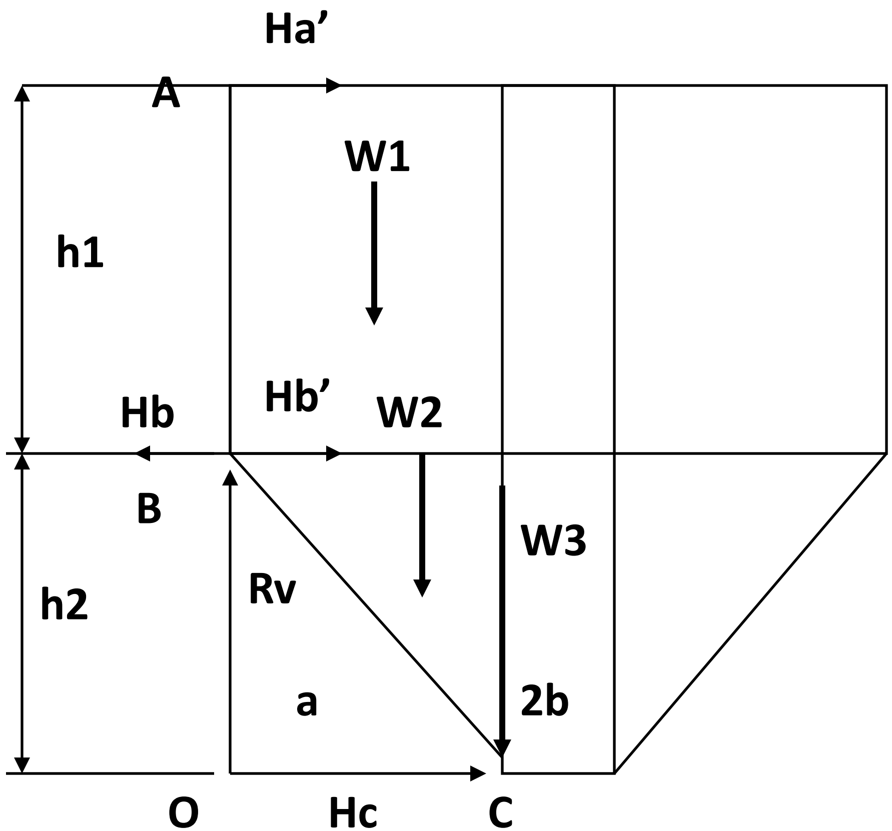
By BK Pandey, Structural Section, MECON Ranchi.

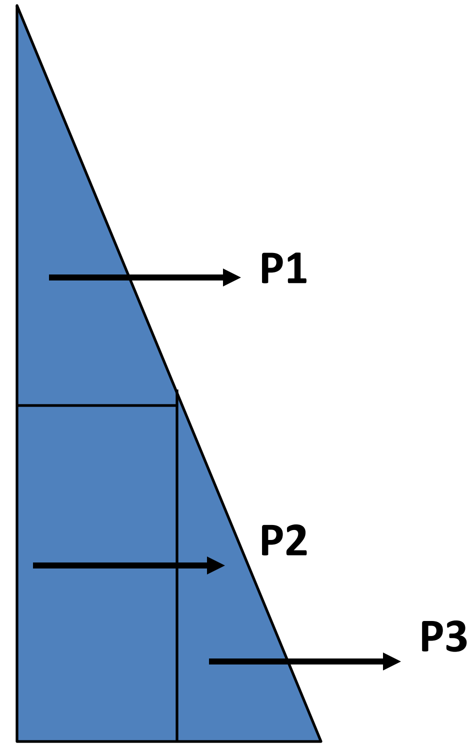
Design of Bins - Bunkers and Silos :: Theory relating to Granular Mass – Pressure Calculation

- **Jansen Theory (Recommended by IS:9178) – Case 2**
  - **Pressure Distribution**



Design of Bins - Bunkers and Silos :: Analysis of Bunker Forces : Shallow Rectangular





Refer : IS 9178 (Part II) fig 1,
Page 6

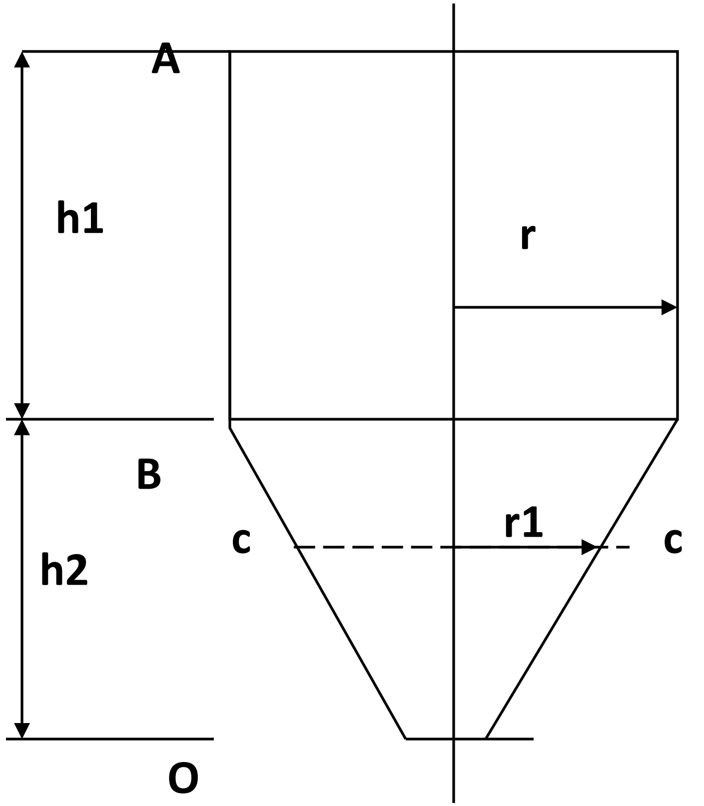
Find Hb by taking moment about O and Hc abt B.

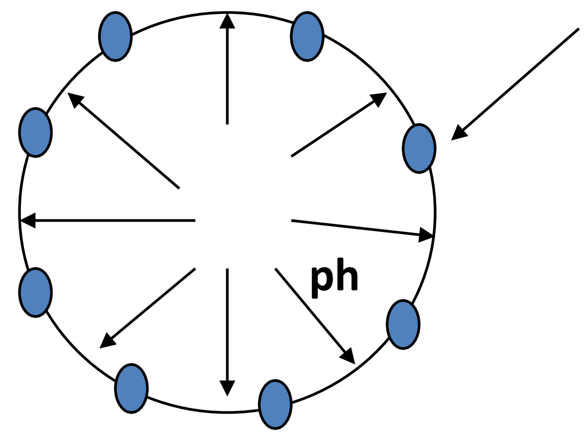
W1 = Y.a.h1 : P1 = K.Y.h1.h1/2

W2 = Y.a.h2/2 : P2 =K.Y.h1.h2

W3 = Y.b.(h1+h2) : P3 =K.Y.h2.h2/2

Ha’ = P1/3 : Hb’ = 2P1/3





Refer : IS 9178 (Part II) fig 1. Page 14

Hoop tension Wall = Ph. r

Hoop tension Hopper = Pn. R1. cosec α

TL Longitudinal tension = Wt / (2. pi . r1. cosec α)

Wt = Total weight at c-c

Design of Bins - Bunkers and Silos :: Design Methodology

- Bunker Layout – JVSL Stock House
- Bunker Structural Components
  - Bunker Beam Wall Component
    - Wall or Web plate
    - Vertical Stiffeners
    - Horizontal Stiffeners
    - Bottom Flange or Trough Beams
    - Top Flange + Floor Beams
  - Hopper Component
    - Skin Plate or Sheathing Plate
    - Ribs Beams/Angles