Introduction to Steel-Concrete Composite Building

Code: IS 11384-1985: Code of Practice for Composite Construction in Structural Steel and Concrete

Concept of Tall Building Design

From a structural engineer's point of view tall building or multi-storeyed building is one that, by virtue of its height, is affected by lateral forces to an extent that they play an important role in the structural design.

Multi-storeyed buildings provide a large floor area in a relatively small area of land in urban centres.

Advantages of Steel Tall Buildings

- Faster to erect
- Lighter
- Better quality control
- Reduced site time - Fast track Construction
- Large column free space and amenable for alteration
- Less material handling at site
- Less percentage of floor area occupied by structural elements
- Has better ductility and hence superior lateral load behavior; better earthquake resistance

Anatomy

- Beams
- Columns
- Floors
- Bracing Systems
- Connections
Beam and Column Construction

Common types of floor systems

- Concrete slabs supported by open-web joists
- One-way and two-way reinforced concrete slabs supported on steel beams
- Concrete slab and steel beam composite floors
- Composite profiled decking floors
- Precast concrete floors on steel beams

Concrete slabs supported by open-web joists
One-way and two-way reinforced concrete slabs supported on steel beams

Welded wire mesh for effectively bonding fireproofing concrete.

Steel beam encased in concrete  
(Rarely used nowadays)

Steel beam acting composite with concrete slab using shear connectors

Concrete slab and steel beam composite floors
**Profiled decking floors**

- **Popular for lighter loads**

- **Advantages:**
  - Do not need form work
  - Lightweight concrete is used resulting in reduced dead weight
  - Decking distributes shrinkage strains, thus prevents serious cracking
  - Decking stabilises the beam against lateral buckling, until the concrete hardens
  - Cells in decking are convenient for locating services

![Profiled sheet](image)

**Section A-A showing dimples**

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**Precast concrete slab floors**
Steel-concrete composite construction

A composite member is defined as consisting of a rolled or a built-up structural steel shape that is either filled with concrete, encased by reinforced concrete or structurally connected to a reinforced concrete slab. Composite members are constructed such that the structural steel shape and the concrete act together to resist axial compression and /or bending.

**Advantages:**
Advantageous properties of both steel and concrete are effectively utilized in a composite structure. For a typical three (3) to ten (10) storied structure, time of construction of the complete structure reduces by about 25 percent. The advantages can be fully utilized as summarized below:

1. Faster construction for maximum utilization of rolled and/or fabricated components (structural steel members) and hence quick return of the invested capital.
2. Advantages based on life-cycle-cost analysis instead of initial cost only.
3. Quality assurance of the steel material along with availability of proper paint system suiting to different corrosive environment.
4. Ability to cover large column free area in buildings and longer span for bridges/flyovers. This leads to more usable space.
5. Reinforced cement concrete (RCC) slab is in compression and steel joist is in tension. Hence, most effective utilization of the materials can be achieved.
6. Better seismic resistance i.e. best suited to resist repeated earthquake loadings, which require a high amount of ductility and hysteretic energy of the material/structural frame.
7. Composite sections have higher stiffness than the corresponding steel sections (in a steel structure) and thus bending stress as well as deflection are lesser.
8. Keeping span and loading unaltered, a lower structural steel section (having lesser depth and weight) can be provided in composite construction, compared to the section required for non-composite construction.
9. Reduced beam depth reduces the story height and consequently the cost of cladding in a building and lowers the cost of embankment in a flyover (due to lower height of embankment).
10. Reduced depth allows provision of lower cost for fire proofing of beam’s exposed faces.
11. Cost of formwork is lower compared to RCC construction.
12. Cost of handling and transportation is minimized for using major part of the structure fabricated in the workshop.
14. Structural steel component has considerable scrap value at the end of useful life.
15. Reductions in overall weight of structure and thereby reduction in foundation costs.
16. More use of a material i.e. steel, which is durable, fully recyclable on replacement and environment friendly.

**Composite Beams:**

Slab and beam type constructions are commonly used in bridges and buildings. Slab-beam interaction is possible through the use of shear connector welded at the top of the flanges. This behaves like a T-beam with the slab or part of it acting as a flange in compression. Further, bond between the shear connector and slab is assumed to be perfect, i.e., no slippage between the top flange of the steel beam and slab is permitted. For determining section properties, it is convenient to transform the concrete slab into an
equivalent steel section by dividing concrete area by modular ratio. The rest of the analysis is carried out as if the section were made of a homogeneous material.

In conventional composite construction, concrete slabs rest over steel beams and are supported by them. Under load these two components act independently and a relative slip occurs at the interface if there is no connection between them. With the help of a deliberate and appropriate connection provided between the beam and the concrete slab, the slip between them can be eliminated. In this case the steel beam and the slab act as a “composite beam” and their action is similar to that of a monolithic T-beam. Concrete is stronger in compression than in tension, and steel is susceptible to buckling in compression. By the composite action between the two, we can utilize their respective advantages to the fullest extent. Generally, in steel-concrete composite beams, steel beams are integrally connected to prefabricated or cast-in-situ reinforced concrete slabs. There are many advantages associated with steel concrete composite construction. Some of these are listed below.

**Advantages of Construction:**

- The most effective utilization of steel and concrete is achieved.
- Keeping the span and loading unaltered, a more economical steel section (in terms of depth and weight) is adequate in composite construction compared with conventional non-composite construction.
- As the depth of beam reduces, the construction depth reduces, resulting in enhanced headroom.
- Because of its larger stiffness, composite beams have less deflection than steel beams.
- Composite construction is amenable to “fast-tract” construction because of using rolled steel and pre-fabricated components, rather than cast-in-situ concrete.
- Encased steel beam sections have improved fire resistance and corrosion.

**Disadvantages:**

1. Additional costs for shear connectors and their installation. For lightly loaded short beams, this extra cost may exceed the cost-reduction on all accounts.

**Methods of Composite Construction of Beams**

Two methods of composite construction: **Shored and Un-shored.**
The concrete slab is usually cast after the steel beams are erected. The two methods of construction differ in the manner of supporting the formwork with fresh concrete and other construction loads.

In shored method, the weight of formwork and fresh concrete is supported by a separate system of shores and steel beams carry their own weight only. When the concrete attains at least 75% of its 28-days strength, the shores are removed and then all loads including weight of concrete and live loads are carried by the composite action of steel and concrete. Thus in shored construction, as almost the entire load is carried by composite action, it is possible to use a lighter steel beam.

In un-shored construction, no shores are used and the weight of the formwork, fresh concrete and steel beam and other construction loads are all carried by the steel beam alone. When the concrete attains at least 75% of its ultimate strength, the formwork is removed and all subsequent loads including the live loads are carried by the composite action. Thus in un-shored construction a relatively heavier steel beams is required.

In shored method though there is some saving in steel, but there is extra cost of shoring. In practice, the ultimate economy in shored method is negligible or may be even negative. So it is preferable to use heavier steel beam and do away with shoring.

**The un-shored method is preferable for the following reasons:**

- The trouble due to probable settlement of shoring is avoided in un-shored construction.
- The ultimate strength of composite beam with heavier steel section is higher. Thus with practically the same cost, higher safety margin is obtained in un-shored construction.
Codal provisions of IS 11384-1985

IS 11384-1985 i.e. Code of Practice for Composite Construction in Structural Steel and Concrete is restricted to buildings only. It stipulates that the steel-concrete composite structures may be designed by the limit state method.

As per IS 11384-1985, a composite structure or part of it, is considered, unfit for use when it exceeds the limit state, beyond which it infringes one of the criteria governing its performance or use. The limit states can be classified into the following categories:

- **Ultimate Limite State** – which corresponds to the maximum load carrying capacity.

- **Serviceability Limit State** - which are related to the criteria governing normal use and durability.

**Ultimate Limit State** to be considered in buildings and structures made of steel-concrete composite construction are:

- Collapse due to flexural failure of one or more critical sections,
- Collapse due to horizontal shear failure at the interface between the steel beam and the concrete slab,
- Collapse due to vertical separation of the concrete slab from the steel beam

The **serviceability limit states** to be considered are as follows:

- Limit state of deflection, and
- Limit state of stresses in concrete and steel

Design for the limit state of collapse in flexure is based on the following assumptions

- Plane sections normal to the axis remain plane-after bending.
- The maximum bending strain in concrete at the outermost compression fiber is taken as 0.0035.
For characteristic compressive strength of concrete $f_{ck}$, maximum permissible bending compression in the structure is assumed to be $0.67 f_{ck}$. With a value of 1.5 for the partial safety factor for the strength of concrete material, maximum design stress is $0.446 f_{ck}$.

- The tensile strength of concrete is ignored;
- The stress-strain curve for the steel section is assumed to be bilinear and partial safety factor of the material is 1.15.

**Composite Columns:**
A steel-concrete composite column is a compression member, comprising either a concrete encased hot-rolled steel section or a concrete filled hollow section of hot-rolled steel. It is generally used as a load-bearing member in a composite framed structure.

![Figure 1: Typical cross-sections of fully or partially concrete encased composite columns.](image)

![Figure 2: Typical cross-sections of fully or partially concrete encased composite columns.](image)
Typical cross-sections of composite columns with fully concrete encased steel section and two partially concrete encased steel sections are illustrated in Figure 1. Figure 2 shows three typical cross-sections of concrete filled hollow sections. Note that there is no requirement to provide additional reinforcing steel for concrete filled hollow composite sections, except for requirements for fire resistance where appropriate.

**Mechanism of Load Resistance:**
In a composite column both the steel and the concrete sections would resist the external loading by interacting together by bond and friction. Supplementary reinforcement in the concrete encasement prevents excessive spalling of concrete both under normal load and fire conditions.

**Advantages of construction:**
With the use of composite columns along with composite decking and composite beams it is possible to erect high rise structures in an extremely efficient manner. There is quite a vertical spread of construction activity carried out simultaneously at any time, with numerous trades working simultaneously. For example

- One group of workers will be erecting the steel beams and columns for one or two storeys at the top of frame.
- Two or three storeys below, another group of workers will be fixing the metal decking for the floors.
- A few storeys below, another group will be concreting the floors.
- As we go down the building, another group is tying the column reinforcing bars in cages.
- Yet another group below them are fixing the formwork, placing the concrete into the column moulds etc.

**Advantages of composite columns:**
- Increased strength for a given cross sectional dimensions.
- Increased stiffness, leading to reduced slenderness and increased buckling resistance.
- Good fire resistance
- Corrosion protection in encased columns.
- Significant economic advantages over either structural steel or reinforced concrete alternatives.
- Identical cross sections with different load and moment resistances can be produced by varying steel thickness, the concrete strength or reinforcement. This allows the
outer dimensions of a column to be held constant over a number of floors in a building, thus simplifying the construction and architectural detailing.

• Erection of high rise building in an extremely efficient manner.

• Formwork is not required for concrete filled tubular sections.
Composite construction consists of providing monolithic action between prefabricated units like steel beams or pre-cast reinforced concrete or pre-stressed concrete beams and cast-in-situ concrete, so that the two will act as one unit. Although there is bound to be a certain amount of natural bond between concrete and steel at least at the initial stages, this bond cannot be relied upon as the same is likely to be deteriorate due to use and over load. Mechanical shear connectors are therefore provided to help the steel and concrete element to act in a composite manner ignoring the contribution made by the inherent natural bond towards this effect.

Primarily shear connectors are intended to resist the horizontal movement between the concrete slab and the steel beam and to transmit the horizontal shear between the two. Shear Connectors are also called upon to prevent vertical separation of the slab from the steel girder at the contact surface.

Therefore, shear connectors are to be designed to cater for integral action of the composite structure at all load conditions on the following basis:

a) Transmission of longitudinal shear along the contact surface without slip.

b) Prevention of vertical separation of the in-situ RC slab from the pre-fabricated structural beam.

**Types of Shear Connectors**

Shear connectors are generally classified into three categories, viz.

a) Rigid type

b) Flexible type

c) Bond type

The basic characteristic of the above connectors are discussed below:

(a) **Rigid Type Connectors:**

These connectors as the name implies, are designed to be bent proof with little inherent power of deformation. These types of shear connectors could be of various shapes, but the most common types are short length of bars, angles or tees welded on
to the steel girder in manners shown in Figure 1. These connectors derive their resistance from bearing pressure of the concrete, distributed evenly over the surface because of the stiffness of the connectors. Failures in these types of connectors are generally associated with the crushing of concrete. It is customary to provide suitable anchorage device to these connectors to prevent in-situ concrete from being separated from the structural unit in the direction normal to the contact surface. The common method for this is to introduce longitudinal round bars through holes provided in the rigid connectors (Figure 1).

(b) Flexible Type Connectors:
Flexible type connectors such as studs, channels welded to the structural beams derive their resistance essentially through the bending of the connectors and normally failure occurs when the yield stress in the connector is exceeded resulting in slip between the structural beam and the concrete slab. Typical types of flexible connectors are illustrated in Figure 2.

c) Bond or Anchorage Type Connectors:
These connectors derive their resistance through bond and/or anchorage action. Typical bond type connectors have been illustrated in Figure 3. These normally consist of the following:

Inclined bars with one end welded to the flange of the steel unit and the other suitably bent.

M.S. bar welded to the flange of the steel unit in the form of helical stirrups.
M.S. bars threaded through holes in the shear connectors

Fig. 1. Typical rigid connectors with anchorage device to hold down the concrete slab against uplift

Fig. 2. Typical flexible connectors
(i). Inclined mild steel bars welded to the top flange of steel unit

(ii). Helical connector

Fig. 3: Typical bond or anchorage connectors
Design Strength of Shear Connectors:

Relevant codes:
In India, primarily two codes of practices are in use for composite construction in structural steel and concrete. These are:


While IRC: 22-1986 is based on working stress method of design and is applicable to road bridges, IS:11384-1985 is based on limit state design method, and its use has been restricted to buildings only. It is proposed to discuss in the following paragraphs the methods recommended by these codes for calculating design strength of shear connectors:

Shear Connectors as per IRC:22-1986
Shear connectors may be either mild steel or high tensile steel according to the material specification of the steel beam. As mild steel beams are commonly used as construction material in India, design methodology of mild steel shear connectors is being discussed in the following sections.

Shear capacity
Shear capacity of a shear connector may be calculated as follows:

(a) For welded channel/angle/tee connector made of mild steel with minimum ultimate strength of 420 to 500 MPa, yield point of 230 MPa and elongation 21%.

\[ Q = 3.32(h + 0.5t)Lf_{ck} \]

Where,

- \( Q \) = The safe shear resistance in Newton of one shear connector
- \( h \) = The maximum thickness of flange measured at the web in mm
- \( t \) = Thickness of the web of shear connector in mm
- \( f_{ck} \) = Characteristics compressive strength of concrete
(b) For welded stud connector of steel with minimum ultimate strength of 460 MPa, yield point of 350 MPa and elongation of 20%  

i) For a ratio of \( h/d < 4.2 \)

\[
Q = 1.49hd \sqrt{f_{ck}}
\]

ii) For a ratio of \( h/d \geq 4.2 \)

\[
Q = 6.08d^2 \sqrt{f_{ck}}
\]

Where,

- \( Q \) = The safe shear resistance in Newton of one shear connector
- \( h \) = Height of stud in mm
- \( d \) = Diameter of the stud in mm

**Longitudinal shear force**

In a composite beam, the longitudinal shear force to be transmitted by the shear connectors is given by the formula:

\[
V_1 = \frac{V A_c Y}{I}
\]

Where,

- \( V_1 \) = The longitudinal shear per unit length at the interface in the section under consideration
- \( V \) = The vertical shear due to dead load and live load including impact acting on the composite section.
- \( A_c \) = The transformed compressive area of concrete above the neutral axis of the composite Section.
- \( Y \) = The distance of the neutral axis of the composite section to the centroid of the area under consideration.
- \( I \) = The moment of inertia of the whole transformed composite section.

**The vertical shear \( V \) is to be computed as follows:**

a) When the deck slab is cast with the girder supported by adequate temporary props, the shear \( V \) is the total external shear due to dead load on the deck including the girder plus the live load with impact.
b) When the deck slab is cast with the girder un-propped, the shear $V$ will be the total external shear due to load added after the concrete has attained a strength compatible to the composite action assumed plus the live load with impact.

c) In case (b), when the slab is supported independent of the girder system, the shear $V$ will be the total external shear including the self-weight of the slab.

**Spacing of shear connectors**

i) Shear connectors are to be provided throughout the length of the beam and may be uniformly spaced between critical cross sections.

ii) Spacing of the shear connectors shall be determined from the formula

$$ p = \sum \frac{Q}{V_i} $$

Where,

- $p =$ Pitch of shear connectors
- $V_i =$ The longitudinal shear per unit length at the interface in the section under consideration
- $Q =$ Safe shear resistance of each shear connector as computed in earlier section.
- $\sum Q =$ The total shear resistance of all connectors at one transverse cross section of the girder

iii) The maximum spacing of the shear connectors in the longitudinal direction shall be limited to:

- (a) 600 mm or
- (b) three times the thickness of the slab or
- (c) four times the height of the connector

**Whichever is the least.**

iv) The spacing of the stud connectors in any direction shall not be less than 75 mm.

**Detailing**

a) *Dimensions of haunches*

The dimensions for the haunches to be provided between top of the stud and soffit of slab shall be as indicated in Figure 4, the sides of the haunches being located outside at the line drawn at 45 degrees from the outside edge of the base of the connector.
b) *Dimensions of shear connectors*

i) The diameter of the stud connectors welded to the flange plate shall not exceed twice the plate thickness.

ii) The height of the stud connectors shall not be less than four times their diameter or 100 mm.

iii) The diameter of the head of the stud shall not be less than one and a half times the diameter of the stud.

iv) The leg length of the weld joining other types of connectors to the flange plate shall not exceed half the thickness of the flange plate.

v) Channel and angle connectors shall have at least 6 mm fillet welds placed along the head and toe of the channels/angles.

vi) The clear distance between the edge of the girder flange and the edge of the shear connectors shall not be less than 25 mm.

vii) The surface of the shear connector which resists the separation between the units (i.e., the under side of the stud or the inner face of the top flange of a channel or the inside of a hoop) shall extend not less than 40 mm clear above the bottom.
transverse reinforcement, nor less than 40 mm into compression zone of the concrete flange.

viii) Where a concrete haunch is used between the steel flange and the soffit of the slab, the surface of the connector that resists separation between the two units shall be placed not less than 40 mm above the transverse reinforcement in the haunches.

ix) The overall height of the connector including any hoop which is an integral part of the connector shall be at least 100 mm with a clear cover of 25 mm.

c) Cover to the shear connectors

The clear depth of concrete cover over the top of the shear connectors shall not be less than 25 mm. The horizontal clear concrete cover to any shear connector shall not be less than 50mm.
Shear Strength

Design value:
Design values for a range of commonly used types of connectors are illustrated in Table – 1. These design values are taken as 67% of the ultimate capacity.

Number of connectors:
The number of connectors should be calculated to resist the maximum value of the total shear force to be transmitted at collapse between points of maximum and zero moment. This force is taken as the force in concrete at ultimate moment and is given by

\[ F_{ce} = 0.36 f_{ck} b X_u \]

This acts at a depth of 0.42\( X_u \) with the value restricted to maximum of \( d \).

Here,

- \( f_{ck} \) = Characteristic strength of concrete
- \( b \) = Breadth of flange in T-section
- \( X_u \) = Depth of neutral axis at ultimate limit state of flexure
- \( d \) = Thickness of concrete section

Haunch in concrete
Design values of the connectors in Table-1 are not valid where there is a concrete haunch with a slope steeper than 1 vertical to 3 horizontal between the top flange of the steel beam and the under side of the concrete slab. In such cases value of shear connector has to be based on actual shear tests of the proposed haunch and reinforcement.

Spacing of connectors
a) The number of connectors as determined above may normally be uniformly spaced between each end of beam and the section of maximum moment.

b) Spacing of connectors should not be greater than four times the slab thickness nor greater than 600 mm.
### Table 1: Design Strength of shear Connectors for Different Concrete Strengths

**As per IS:11384-1985**

<table>
<thead>
<tr>
<th>Type of Connectors</th>
<th>Connectors Material</th>
<th>Design Strength of Connectors for Concrete of Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>M-20</td>
</tr>
<tr>
<td><strong>I. Headed stud</strong></td>
<td>IS:961-1975*</td>
<td></td>
</tr>
<tr>
<td>Diameter</td>
<td>Height mm</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>100</td>
<td>86</td>
</tr>
<tr>
<td>22</td>
<td>100</td>
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</tr>
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<td>49</td>
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<tr>
<td>16</td>
<td>75</td>
<td>47</td>
</tr>
<tr>
<td>12</td>
<td>62</td>
<td>23</td>
</tr>
<tr>
<td><strong>II. Bar Connector</strong></td>
<td>IS:226-1975</td>
<td></td>
</tr>
<tr>
<td>50 mm x 38 mm x 200 mm</td>
<td></td>
<td>318</td>
</tr>
<tr>
<td><strong>III. Channel connector</strong></td>
<td>IS226-1975</td>
<td></td>
</tr>
<tr>
<td>125 mm x 65 mm x 12.7 kg x 150 mm</td>
<td></td>
<td>184</td>
</tr>
<tr>
<td>100mm x 50mm x 9.2kg x 150 mm</td>
<td></td>
<td>169</td>
</tr>
<tr>
<td>75mm x 40mm x6.8 kg x 150 mm</td>
<td></td>
<td>159</td>
</tr>
<tr>
<td><strong>IV. Tee connector</strong></td>
<td>IS:226-1975</td>
<td></td>
</tr>
<tr>
<td>100 mm x 100 mm x 10 mm</td>
<td>Tee x 50 mm</td>
<td>163</td>
</tr>
<tr>
<td><strong>V. Helical connector</strong></td>
<td>IS:226-1975</td>
<td></td>
</tr>
<tr>
<td>Bar diameter mm</td>
<td>Pitch circle diameter mm</td>
<td>131</td>
</tr>
<tr>
<td>20</td>
<td>125</td>
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<td>70</td>
</tr>
<tr>
<td>10</td>
<td>75</td>
<td>40</td>
</tr>
</tbody>
</table>
Fig 5 Typical rigid connectors with anchorage device to hold down the concrete slab against uplift

All dimensions as per IS 11384: 1985

Typical shear connectors
Detailing

a) The distance between the edge of the connector and the edge of the flange to which it is connected shall not be less than 25 mm.

b) The overall height of the connector, *i.e.*, the length of the stud, diameter of the helix, height of channel, hoop etc., should not be less than 50 mm nor less than 25 mm into the compression zone of the concrete slab.

c) The diameter of the head of the stud should not be less than 1.5 times the diameter of the stud and the thickness of the head shall not be less than 0.4 times the shank diameter.
Lecture Note – 36

Load Resisting System

Lateral Forces

- High wind pressures on the sides of tall buildings produce base shear and overturning moments.
- These forces cause horizontal deflection
- Horizontal deflection at the top of a building is called drift
- Drift is measured by drift index, $\Delta/h$, where, $\Delta$ is the horizontal deflection at top of the building and $h$ is the height of the building

Lateral Drift

\[ \text{Drift index} = \frac{\Delta}{h} \]

IS Code: Drift index $< 0.002$ of total height.

Lateral Load Resisting Systems:

- Rigid frames
- Shear walls
- Braced frames
- Advanced structural forms
Rigid frames

- Framed with moment resisting connections between beams and columns.
- Used economically to provide lateral load resistance for low-rise buildings
- Less stiff than other systems
- Moment resisting connections may be necessary in locations where loads are applied eccentrically with respect to centre line of the columns

Moment resistant connections - 1

Shop welded and field bolted connection

Moment resistant connections – 2

Field welded and field bolted connection
Shear walls

• Shear walls are specially designed RC walls parallel to the direction of load are used to resist lateral loads by acting as deep cantilever beams fixed at foundation
• Interior concrete core walls around the elevator, stair etc. may be considered as shear walls
• Advantages of shear walls:
  – Very rigid in their own plane and hence are effective in limiting deflections
  – Act as fire compartment walls
• For low and medium rise buildings, the construction of shear walls takes more time and is less precise in dimensions than steelwork
• Have lesser ductility and may not meet the energy required under severe earthquake
Braced frames

• **X-bracing system**
  
  – Works well for 20 to 60 storey height
  – Does not give room for openings such as doors and windows

• To provide more flexibility for the placing of windows and doors, $K$-bracing system or full-storey knee bracing system can be used

• Knee bracing is efficient in energy dissipation during earthquake loads by forming plastic hinge in beam at the point of their intersection with the beam
Connections

Depending upon structural behaviour, connections are classified as:

• **Simple connections**
  – Detailed to allow beam end to rotate freely and beam behaves as a simply supported beam
  – Transfers shear and axial forces between connecting members but does not transfer bending moment

• **Rigid connections**
  – Detailed to ensure a monolithic joint such that angle between beam and column before deformation remains same even after deformation
  – Transfers shear, axial force and bending moment from beam to column

• **Semi-rigid connections**
  – Designed to transmit full shear force and a fraction of the rigid joint bending moment across the joint
  – Analysis of frames with such joints is complex

Advanced Structural Forms

• **Framed tubes**
  – Lateral resistance is provided by very stiff moment resisting frames that form a tube around the perimeter of the building
  – Gravity loading is shared between tube and interior columns
  – Appropriate for buildings having 40 to 100 storeys
- **Braced tube structures**
  - Tubular system with $X$-bracing over many stories
  - Structure behaves under lateral loads more like a braced frame reducing bending in the members of the frames.
  - Spacing of the columns can be increased and depth of girders will be less, thereby allowing large size windows than in conventional framed tube structures
• Tube-in-Tube Structures
  – A type of framed tube consisting of an outer-framed tube together with an internal elevator and service core.
  – Outer and inner tubes act jointly in resisting both gravity and lateral loading in steel-framed buildings

Other advanced structural forms:

• Bundled tube
• Bundled tube with belt truss
• Tapered bundled tube

LOADING

• Gravity loads
• Wind load
• Earthquake load

Gravity Load

The self-weight of the building and the imposed loads produced by the intend occupancy or use. The loads produced by the slab and roof will be transferred to the associated beams and the beam has to be analyzed under these loads along with its own dead load.
Analysis of Buildings for Gravity Loads

- **Simple Framing**
  - Shears and moments can be determined by static analysis
- **Semi Rigid Framing**
  - Designed using techniques developed based on experiments
- **Rigid Framing**
  - Rigid frame buildings are analysed by one of approximate methods to make an estimate of member sizes before going to exact methods such as slope-deflection or moment-distribution method

Substitute frame method

- A short version of moment distribution method
- Only two cycles are carried out in analysis and also only a part of frame is considered for analysing the moments and shears in beams and columns
- Assumptions for this method:
  - Moments transferred from one floor to another floor are small. Hence, the moments for each floor are separately calculated
  - Each floor will be taken as connected to columns above and below with their far ends fixed.

Moment Distribution Method:

Most of the cases, the beams are continuous and indeterminate. The designer can analyze the continuous beam by the moment distribution method in the absence of software. However, the analysis if done by the matrix method with the help of computer, will produce reasonable accurate results. Here, an example is given to show how an indeterminate continuous beam can be analyzed manually by the moment distribution method.
Example 1: (Moment distribution method)

Calculation of fixed end moment (FEM)

Let, $F_{AB} \rightarrow$ Fixed end moment of the member AB

So, $F_{AB} = \frac{wab^2}{I^2} = \frac{8 \times 3 \times 1^2}{4^2} = 1.5 \text{kN}$

Similarly,

$F_{BA} = -\frac{wa^2b}{I^2} = \frac{8 \times 3^2 \times 1}{4^2} = 4.5 \text{kN}$

$F_{BC} = \frac{wl}{8} = \frac{10 \times 3}{8} = 3.75 \text{kN}$

$F_{CB} = -\frac{wl}{8} = -\frac{10 \times 3}{8} = -3.75 \text{kN}$

$F_{CD} = \frac{wl^2}{12} = \frac{4 \times 3^2}{12} = 3 \text{kN}$

$F_{DC} = -\frac{wl^2}{12} = -\frac{4 \times 3^2}{12} = -3 \text{kN}$

Calculation of distribution factor (DF)

Let, $I$ as moment of inertia of each section and $E$ be the modulus of elasticity of the material
### Joint Member Relative Stiffness

<table>
<thead>
<tr>
<th>Joint</th>
<th>Member</th>
<th>Relative Stiffness ( I/L )</th>
<th>Total Stiffness ( \sum (I/L) )</th>
<th>D.F. ( \frac{I}{L} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>BA</td>
<td>( \frac{I}{4} )</td>
<td>( \frac{7}{12} )</td>
<td>( \frac{3}{7} ) = 0.429</td>
</tr>
<tr>
<td></td>
<td>BC</td>
<td>( \frac{I}{3} )</td>
<td>( \frac{7}{12} )</td>
<td>( \frac{4}{7} ) = 0.571</td>
</tr>
<tr>
<td>C</td>
<td>CB</td>
<td>( \frac{I}{3} )</td>
<td>( \frac{7}{12} )</td>
<td>( \frac{4}{7} ) = 0.571</td>
</tr>
<tr>
<td></td>
<td>CD</td>
<td>( \frac{3}{4} \times \frac{I}{3} = \frac{I}{4} )</td>
<td>( \frac{7}{12} )</td>
<td>( \frac{3}{7} ) = 0.429</td>
</tr>
</tbody>
</table>

### Structural Diagram

![Structural Diagram](image)

### Moment Results

<table>
<thead>
<tr>
<th>FEM</th>
<th>1.5</th>
<th>-4.5</th>
<th>3.75</th>
<th>-3.75</th>
<th>3</th>
<th>-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>DF</td>
<td>0</td>
<td>0.429</td>
<td>0.571</td>
<td>-0.571</td>
<td>0.429</td>
<td>1</td>
</tr>
<tr>
<td>End correction</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>Total FEM</td>
<td>1.5</td>
<td>-4.5</td>
<td>3.75</td>
<td>-3.75</td>
<td>4.5</td>
<td>0</td>
</tr>
<tr>
<td>First cycle</td>
<td>0</td>
<td>0.322</td>
<td>0.428</td>
<td>-0.428</td>
<td>-0.322</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.161</td>
<td>0</td>
<td>-0.214</td>
<td>0.214</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Second cycle</td>
<td>0</td>
<td>0.092</td>
<td>0.122</td>
<td>-0.122</td>
<td>0.122</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.046</td>
<td>0</td>
<td>-0.061</td>
<td>0.061</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Third cycle</td>
<td>0</td>
<td>0.026</td>
<td>0.035</td>
<td>-0.035</td>
<td>-0.026</td>
<td>0</td>
</tr>
<tr>
<td>Total moment</td>
<td>1.707</td>
<td>-4.06</td>
<td>4.06</td>
<td>-4.06</td>
<td>4.03</td>
<td>0</td>
</tr>
</tbody>
</table>
**Wind load**

- Most important factor that determines the design of tall buildings over 10 storeys, where storey height approximately lies between 2.7 – 3.0 m
- Appropriate design wind loads are estimated based on:
  - Static approach
  - Dynamic approach

**Earthquake load**

- Seismic motion consists of horizontal and vertical ground motions
- Vertical motion is much smaller in magnitude and factor of safety provided against gravity loads will accommodate additional forces due to this motion
- Horizontal motion of ground causes most significant effect on structure by shaking foundation back and forth
- Mass of building resists this motion by setting up inertia forces throughout structure.

- **Magnitude of horizontal shear force depends on**;
  - Mass of building
  - Acceleration of ground
  - Nature of structure
- Earthquake load is estimated by;
  - Seismic co-efficient method
  - Response spectrum method
Earthquake load

Methods for lateral load analysis:

– Portal method
– Cantilever method
– Factor method

• Portal method and Cantilever method yield good results only when the height of a building is approximately more than five times its least lateral dimension

Portal Method

• Satisfactory for buildings up to 25 stories
• Simplifying assumptions made in portal method:
  – A point of contraflexure occurs at the centre of each beam
  – A point of contraflexure occurs at the centre of each column
  – Total horizontal shear at each storey is distributed between columns of that storey in such a way that each interior column carries twice the shear carried by each exterior column
• Above assumptions convert indeterminate multi-storey frame to a determinate structure

**Steps involved in analysis of frame:**

• Horizontal shears on each level are distributed between columns of that floor
• Moment in each column is equal to column shear multiplied by half the column height
• Girder moments are determined by applying moment equilibrium equation to joints
• Shear in each girder is equal to its moment divided by half the girder length
• Finally, column axial forces are determined by summing up beam shears and other axial forces at each joint

**Example 2: (Portal method)**

Analyze the frame given in the figure.

### Column shear
Let, $H_1 \rightarrow$ horizontal shear at the member AD
So,
\[ V_{AD} = V_{CF} = H_1; \ V_{BE} = 2H_1 \] [As per assumption]
So, equating the horizontal forces we get,
\[ H_1 + 2H_1 + H_1 = 20 \]
\[ \therefore H_1 = 5 \text{kN} \]
\[ \therefore V_{AD} = V_{CF} = 5 \text{kN}; \ V_{BE} = 10 \text{kN} \]
Let, $H_2 \rightarrow$ horizontal shear at the member DG
So,
\[ V_{DG} = V_{FI} = H_2; \ V_{EH} = 2H_2 \]
Again, equating the horizontal forces we get,
\[
H_2 + 2H_2 + H_2 = 20 + 40 \\
\Rightarrow H_2 = 15 \\
\therefore H_2 = 15\text{kN} \\
\therefore V_{DG} = V_{FI} = 15\text{kN}; V_{EH} = 30\text{kN}
\]

**Column moment**
Taking moment about A we get,
\[
\sum M_A = 0 \\
\Rightarrow M_{AD} = V_{AD} \times l_1 \\
\Rightarrow M_{AD} = 10\text{kNm};
\]
Similarly,
\[
M_{CF} = V_{CF} \times l_1 = 10\text{kNm} \\
M_{BE} = V_{BE} \times l_1 = 10 \times 2 = 20\text{kNm} \\
M_{DG} = V_{DG} \times l_2 = 15 \times 3 = 45\text{kNm} \\
M_{FI} = V_{FI} \times l_2 = 15 \times 3 = 45\text{kNm} \\
M_{EH} = V_{EH} \times l_2 = 30 \times 3 = 90\text{kNm}
\]

**Beam moment**
Taking moment about A
\[
\sum M_A = 0 \\
\Rightarrow M_{DA} = M_{AB} = 10\text{kNm} \\
\text{Similarly, } M_{CF} = M_{CB} = 10\text{kNm}
\]

Taking moment about B
\[
\sum M_B = 0 \\
\Rightarrow M_{AB} + M_{BC} = M_{BE} \\
\Rightarrow M_{BE} = 10 + 10 \\
\therefore M_{BE} = 20\text{kNm}
\]

Taking moment about D
\[
\sum M_D = 0 \\
\Rightarrow M_{DA} + M_{DG} = M_{DE} \\
\Rightarrow M_{DE} = 10 + 45 \\
\therefore M_{DE} = 55\text{kNm} \\
\text{Similarly for } M_{EF} = 55\text{kNm} \\
\text{Also taking moment about E we get,} \\
\sum M_E = 0 \\
\Rightarrow M_{DE} + M_{EF} = M_{EH} + M_{BE} \\
\Rightarrow M_{EH} = M_{DE} + M_{EF} - M_{BE} \\
\Rightarrow M_{EH} = 55 + 55 - 20 \\
\therefore M_{EH} = 90\text{kNm}
Beam shear
At point A

Taking moment about A
\[ \sum M_A = 0 \]
\[ \Rightarrow V_{AB} \times 3 - 5 \times 2 = 0 \]
\[ \Rightarrow V_{AB} = 3.33 \text{kN} \]

At point B

Taking moment about B we get,
\[ V_{BC} \times 4 + V_{AB} \times 3 = 10 \times 2 \]
\[ \Rightarrow V_{BC} = \frac{20 - 3.33 \times 3}{4} = 2.5 \text{kN} \]

At point D

Taking moment about D
\[ \sum M_D = 0 \]
\[ \Rightarrow 5 \times 2 + 15 \times 3 = V_{DE} \times 3 \]
\[ \Rightarrow V_{DE} = \frac{55}{3} = 18.33 \text{kN} \]

Similarly, \[ V_{EF} = \frac{2M_{EF}}{l} = \frac{2 \times 55}{8} = 13.75 \text{kN} \]

Axial force

Equating vertical forces about A
\[ V_{AB} = F_{AD} \]
\[ \Rightarrow F_{AD} = 3.33 \text{kN} \]

Equating horizontal forces about A
\[ F_{AB} + V_{AD} = 20 \]
\[ \Rightarrow F_{AB} = 20 - 5 \]
\[ \therefore F_{AB} = 15 \text{kN} \]

At point B
\[ \sum H = 0 \]
\[ \Rightarrow 15 = 10 + F_{BC} \]
\[ \therefore F_{BC} = 5 \text{kN} \]
\[ \sum V = 0 \]
\[ \Rightarrow F_{BE} + 2.5 = 3.33 \]
\[ \therefore F_{BE} = 0.83 \text{kN} \]
So, \( F_{CF} = 2.5 \text{kN} \)

Similarly,
\( F_{DE} = 30 \text{kN}; \ F_{EF} = 10 \text{kN} \)
\( F_{DG} = 21.67 \text{kN}; \ F_{EH} = 5.41 \text{kN}; \ F_{FI} = 16.25 \text{kN} \)
Cantilever Method

- Gives good results for high-narrow buildings compared to those from the Portal method and it may be used satisfactorily for buildings of 25 to 35 storeys tall

- Simplifying assumptions:
  - A point of contraflexure occurs at centre of each beam
  - A point of contraflexure occurs at centre of each column
  - Axial force in each column of a storey is proportional to the horizontal distance of the column from centre of gravity of all columns of storey under consideration

Steps involved:

- Centre of gravity of columns is located
- Axial force in one column is assumed as $F$ and axial forces of remaining columns can be expressed in terms of $F$ by taking moments about centroid of columns of the storey
- Beam shears are determined joint by joint from column axial forces
- Beam moments are determined by multiplying shear in beam by half span of beam
- Column moments are found joint by joint from beam moments
- Column shears are obtained by dividing column moments by half-column heights
Example: (Cantilever method)

Analyze the frame given in the figure

\[ \begin{array}{c c c}
\text{A} & \text{B} & \text{C} \\
\text{D} & \text{E} & \text{F} \\
\text{G} & \text{H} & \text{I}
\end{array} \]

\[ \begin{array}{c c c c}
20\text{kN} & & & \\
& 40\text{kN} & & \\
& & \text{4m} & \\
& & \text{6m} & \\
& \text{6m} & & \text{8m}
\end{array} \]

Calculation for centroidal axis
Let, \( x \) the distance of the centroidal axis from \( A \)
Assuming the forces in each member to be equal & taking moment about \( A \) we get,

\[ 6 \times F + (8 + 6) \times F = x \times 3F \]

where, \( F \) force in each member

\[ \therefore x = 6.67 \text{m} \]
Hence, the centroidal axis is 6.67m from \( A \)
Now, as pre assumption of cantilever method, force in member is proportional to centroidal distance between the member.

Therefore

\[ \frac{P_A}{0.67} = \frac{P_B}{6.67} \]

\[ \Rightarrow P_B = \frac{P_A}{6.67} \times 0.67 \]
\[ \Rightarrow P_B = 0.1P_A \]

Similarly,

\[ P_C = \frac{P_A}{6.67} \times 7.33 \]

\[ \Rightarrow P_C = 1.1P_A \]

Now, taking the moment about the point of contraflexure of the member \( AD \) we get

\[ 20 \times 2 - P_A \times 14 - P_B \times 8 = 0 \]

\[ \Rightarrow 20 \times 2 - P_A \times 14 - 1P_A \times 8 = 0 \]
⇒ \( P_A = 2.7 \)
Hence, \( P_A = 2.7 \text{kN (t)} \)
So, \( P_B = 0.1P_A = 0.1 \times 2.7 = 0.27 \text{kN (t)} \)

\( P_C = 1.1P_A = 1.1 \times 2.7 = 2.97 \text{kN (c)} \)

For bottom storey
Now, taking the moment about the point of contraflexure of the member \( DG \) we get

\[ 20 \times 7 + 40 \times 3 - P_A' \times 14 - P_B' \times 8 = 0 \]

Here also, force in member is proportional to centroidal distance between the member. Hence

\[ P_B' = 0.1P_A' ; \quad P_C' = 1.1P_A' \]

So putting the values in the above equation we get

\( P_A' = 17.57 \text{kN} \)

Similarly,

\( P_B' = 1.757 \text{kN} ; \quad P_C' = 19.32 \text{kN} \)

Beam shear
Now,
Shear in the beam \( AB = \) Axial force in the column \( AD \)
Hence, \( V_{AB} = P_A = 2.7 \text{kN} \)
Similarly,

\( M_{BC} = V_{BC} \times 4 = 0.27 \times 4 = 1.08 \text{kNm} \)
\( M_{DE} = V_{DE} \times 3 = 17.57 \times 3 = 52.71 \text{kNm} \)
\( M_{EF} = V_{EF} \times 4 = 19.32 \times 4 = 77.28 \text{kNm} \)

Column moment
Taking moment about \( A \)
\[ \sum M_A = 0 \]
⇒ \( M_{DA} = M_{AB} = 8.1 \text{kNm} \)
Similarly, \( M_{CF} = M_{CB} = 1.08 \text{kNm} \)

Taking moment about \( B \)
\[ \sum M_B = 0 \]
⇒ \( M_{AB} + M_{BC} = M_{BE} \)
⇒ \( M_{BE} = 8.1 + 1.08 \)
∴ \( M_{BE} = 9.18 \text{kNm} \)

Taking moment about \( D \)
\[ \sum M_D = 0 \]
⇒ \( M_{DA} + M_{DG} = M_{DE} \)
⇒ \( M_{DG} = M_{DA} + M_{DE} \)
\[ \Rightarrow M_{DG} = 8.1 + 52.71 \]
\[ \therefore M_{DG} = 60.81 \text{kNm} \]

Also taking moment about F we get,
\[ \sum M_F = 0 \]
\[ \Rightarrow M_{FC} + M_{FI} = M_{FE} \]
\[ \Rightarrow M_{FI} = 77.28 - 1.08 \]
\[ M_{FI} = 76.2 \text{kN} \]

Also taking moment about E we get,
\[ \sum M_E = 0 \]
\[ \Rightarrow M_{DE} + M_{EF} = M_{EH} + M_{BE} \]
\[ \Rightarrow M_{EH} = M_{DE} + M_{EF} - M_{BE} \]
\[ \Rightarrow M_{EH} = 52.71 + 77.28 - 9.18 \]
\[ \therefore M_{EH} = 120.81 \text{kNm} \]

Column shear

We have column shear = \( \frac{\text{column moment}}{\text{half of span}} \)

So, \[ V_{AD} = \frac{M_{AD}}{2} = 4.05 \text{kN} \]

\[ V_{BE} = \frac{M_{BE}}{2} = 4.59 \text{kN} \]

\[ V_{CF} = \frac{M_{CF}}{2} = 0.54 \text{kN} \]

\[ V_{DG} = \frac{M_{DG}}{3} = 20.27 \text{kN} \]

\[ V_{EH} = \frac{M_{EH}}{3} = 40.27 \text{kN} \]

\[ V_{FI} = \frac{M_{FI}}{3} = 25.4 \text{kN} \]

Axial force

Equating vertical forces about A

\[ V_{AB} = F_{AD} \]
\[ \Rightarrow F_{AD} = 2.7 \text{kN} \]

Equating horizontal forces about A

\[ F_{AB} + V_{AD} = 20 \]
\[ \Rightarrow F_{AB} = 20 - 2.7 \]
∴ $F_{AB} = 17.3\, \text{kN}$

At point B

$$\sum H = 0$$

$\Rightarrow 17.3 = 3.24 + F_{BC}$

∴ $F_{BC} = 14.06\, \text{kN}$

$$\sum V = 0$$

$\Rightarrow F_{BE} = 2.7 - 0.27$

∴ $F_{BF} = 2.43\, \text{kN}$
**Factor Method**

- More accurate than either portal method or cantilever method
- Based on assumptions regarding the elastic action of the structure
- For application of Factor method, relative stiffness \( k = \frac{l}{l} \), for each beam and column should be known or assumed

**Application of Factor method involves following steps:**

- The girder factor \( g \), is determined for each joint from the following expression
  \[
  g = \frac{\sum k_c}{\sum k}
  \]
  \( \Sigma k_c \) - Sum of relative stiffness of column members meeting at that joint
  \( \Sigma k \) - Sum of relative stiffness of all members meeting at that joint

- Column factor \( c \), is found for each joint from following expression
  \( c = 1 - g \)
- At each end of every member, there will be factors from step 1 or step 2. To these factors, half the values of those at the other end of the same member are added
- Sum obtained as per above step is multiplied by relative stiffness of respective members. This product is termed as column moment factor \( C \), for columns and girder moment factor \( G \), for girders
- Calculate column end moments
- Calculate beam end moments

**Drift in Rigid Frames**

- Lateral displacement of rigid frames subjected to horizontal loads is due to following three modes:
  - Girder Flexure
Column Flexure
Axial deformation of columns

- Sum of storey drifts from the base upward gives drift at any level and the storey drifts can be calculated from summing up contributions of all three modes in that particular storey

**Computer Analysis of Rigid Frames**
- A typical model of rigid frame consists of an assembly of beam-type elements to represent both beams and columns of frame.
- Columns are assigned their principal inertia and sectional areas.
- Beams are assigned with their horizontal axis inertia and sectional areas.