ANALYSIS OF MULTISTORIED BUILDING BY

APPROXIMATE METHOD

In case of multistoried buildings the degree of indeterminacy is very high. Hence to solve the frame you cannot use conventional methods; slope deflection, moment distribution & constant consistent deformation. Kanič method may be employed. But it needs more computational efforts, thus for a quick solution design engineers use the approximate method.

APPROXIMATE METHOD OF ANALYSIS

The statically indeterminate structure is simplified to a statically determinate structure, the analysis is carried out using the principle of statics. The validity of results is based upon the assumptions made in the analysis.

1. Substitute Frame Method For Vertical loads (DL+LL)
2. Any one of the following methods is used for horizontal loads (wind load)
   a) Portal Framed method
   b) Cantilever Method
   c) Factor method

ANALYSIS OF VERTICAL LOADS

This method is used for determining the moment and shear force at any floor or roof levels due to vertical loads.
Here it is assumed that moment transfer from one floor to another floor are negligible & hence analysis can be made from floor to floor.

<table>
<thead>
<tr>
<th>Roof</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
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<tbody>
<tr>
<td>3rd Floor</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>9</td>
<td>10</td>
<td>11</td>
<td>12</td>
</tr>
<tr>
<td>1st Floor</td>
<td>13</td>
<td>14</td>
<td>15</td>
<td>16</td>
</tr>
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</table>

**Maximum Bending Moments in Beams**

In multistoried frames both D.L. & L.L. accounts for the vertical load. D.L. acts throughout the frame and at all times. The L.L. may act throughout the frame or on a part of it at a particular time. Hence for the analysis we consider various combination of L.L.

1) Maximum +ve BM at 'x', i.e. at middle of span CD

To get max positive bending m.m. at x there is a midpoint of span CD, the L.L. should be placed on span CD and on alternate spans.
2) Max. -ve Bending moment at 'x', it @ midpoint of span CD

3) Maximum -ve B. M @ Support C

4) Max. B.M on columns

Max. moment in columns are obtained when alternative spans are loaded with LL. The max. B.M in columns is selected by comparing the above two conditions.

In a multi-storied building the frame showed in Fig. are spaced at 8m. DL from the slab is 3 * 10^3 kN/m² and live load is 5 * 10^3 kN/m². Analyse the beam BC of the mid-span for max. -ve B.M. Self weight of 4m span may be taken as 4 kN/m, self weight of 6m span can be taken as 5 kN/m. Use two cycle moment distribution method. The stiffness of members are indicated adjacent to each member.
Ansl Substitute Frame

Substitute frame for analysing beam BC for max. neg BM at midspan

Step 2 Load Calculation

Span AB

\[ D_L \text{ slab } = \frac{3 \times 3.5 \times 10.5}{12} = 5.25 \text{ KNm} \]

\[ 4 \text{ m span } AB = 4 \text{ KNm} \]

Total \( D_L \) = 10.5 + 4 = 14.5 KNm

Span BC

\[ D_L \text{ slab } = \frac{3 \times 3.5 \times 10.5}{12} = 5.25 \text{ KNm} \]

\[ 4 \text{ m span } BC = 5 \text{ KNm} \]

Total \( D_L \) = 10.5 + 5 = 15.5 KNm

Span CD

\[ D_L \text{ slab } = \frac{5 \times 3.5 \times 14.5}{12} = 14.5 \text{ KNm} \]

4 m span AB = 4 KNm

Total \( D_L \) = 14.5 KNm

Total load = 14.5 + 14.5 = 32 KNm

Step 3 Fixed and end moments

Span AB

\[ M_{FAB} = \frac{-W L^2}{12}, \quad \frac{-32 \times 4^2}{12} = -42.67 \text{ KNm} \]

\[ M_{FBA} = \frac{W L^2}{12}, \quad \frac{32 \times 4^2}{12} = 42.67 \text{ KNm} \]

Span BC

\[ M_{FBC} = \frac{-W L^2}{12}, \quad \frac{-15.5 \times 6^2}{12} = -46.5 \text{ KNm} \]

\[ M_{FBC} = \frac{W L^2}{12}, \quad \frac{46.5}{12} = 46.5 \text{ KNm} \]

Span CD

\[ M_{FCD} = \frac{W L^2}{12}, \quad \frac{-32 \times 4^2}{12} = -42.67 \text{ KNm} \]

\[ M_{FDC} = 42.67 \text{ KNm} \]
### Step 4: Distribution Factor

<table>
<thead>
<tr>
<th>Joint</th>
<th>Members</th>
<th>Relative Stiffness</th>
<th>Total Stiffness</th>
<th>D.F</th>
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<tr>
<td>A</td>
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</tr>
<tr>
<td>B</td>
<td>BJ</td>
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<tr>
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<td>0.18</td>
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<td>BA</td>
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<tr>
<td></td>
<td>BC</td>
<td>4K</td>
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<td>0.36</td>
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<td>11K</td>
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<td>CG</td>
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<td>0.18</td>
</tr>
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<td>GB</td>
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<td>0.21</td>
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<tr>
<td>D</td>
<td>DL</td>
<td>2K</td>
<td>7K</td>
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<tr>
<td></td>
<td>DH</td>
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<td>0.29</td>
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<tr>
<td></td>
<td>DC</td>
<td>3K</td>
<td></td>
<td>0.43</td>
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### Joints

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<thead>
<tr>
<th>Member</th>
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<th>B</th>
<th>C</th>
<th>D</th>
<th>D-F</th>
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<tr>
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<td>0.43</td>
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<tr>
<td>BA</td>
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<td>46.5</td>
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</tr>
<tr>
<td>BC</td>
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<td>46.5</td>
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<td>-42.61</td>
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<td>CB</td>
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<td>0.69</td>
<td>0.69</td>
<td>0.69</td>
<td>0.52</td>
</tr>
</tbody>
</table>

### Balancing

- **Guy over:** 1.38
- **Bearing:** -2.29

### Step 5: Moment Distribution

#### Max -ve moment at midspan

Free bending moment at midspan of BC

\[
BM = \frac{wl^2}{8}
\]

\[
= \frac{15.5 \times 6^2}{8}
\]

\[
= 69.15\, \text{KNm}
\]

Net BM at centre of BC:

\[
= 69.15 - \left(48.84 + 48.84\right)
\]

\[
= 69.15 - 97.68
\]

\[
= 20.88\, \text{KNm}
\]

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Scanned by CamScanner
In a multistoried building the frame shown in Fig. below are spaced at 4m intervals. Dead load from slab is 3 kN/m². Live load from slab is 5 kN/m². Analyze the beam BC for midspan positively. Self weight of beam may be ignored.

Use a cycle moment distribution method.

**Span A-B**
- DL from slab = 3 kN/m² × 4m = 12 kN/m

**Span BC**
- DL from slab = 3 kN/m² × 4m = 12 kN/m
  - LL = 5 kN/m × 20 = 100 kN/m
  - TL = 12 kN/m = 38 kN/m

**Span CD**
- DL from slab = 3 kN/m² × 4m = 12 kN/m

**Step 2: Fixed end moments**

**Span A-B**
- \( M_{FA} = -\frac{wL^2}{12} = -\frac{12 \times 6^2}{12} = -36 \text{ kNm} \)
- \( M_{FB} = \frac{wL^2}{12} = \frac{12 \times 6^2}{12} = 36 \text{ kNm} \)

**Span B-C**
- \( M_{FB} = -\frac{wL^2}{12} = -\frac{32 \times 6^2}{12} = -96 \text{ kNm} \)
- \( M_{FC} = \frac{32 \times 6^2}{12} = 96 \text{ kNm} \)

**Span C-D**
- \( M_{FD} = -\frac{wL^2}{12} = -\frac{12 \times 6^2}{12} = -36 \text{ kNm} \)
- \( M_{FDC} = \frac{6 \times 6^2}{12} = 36 \text{ kNm} \)
**Step 4: Distribution Factor**

Assuming \( I \) to be the moment of inertia of each member,

Relative slenderness, \( K = \frac{I}{L} \)

<table>
<thead>
<tr>
<th>Joint</th>
<th>Members</th>
<th>R.S</th>
<th>T.S</th>
<th>D.F</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>AA₁</td>
<td>( \frac{\sqrt{3}}{2} )</td>
<td>( 64 \sqrt{3} )</td>
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</tr>
<tr>
<td></td>
<td>AB</td>
<td>0.167I</td>
<td>0.261I</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>AA₂</td>
<td>0.25I</td>
<td>0.37I</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>BB₁</td>
<td>0.261I</td>
<td>0.834I</td>
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</tr>
<tr>
<td></td>
<td>BA</td>
<td>0.167I</td>
<td>0.20I</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>BB₂</td>
<td>0.261I</td>
<td>0.834I</td>
<td>0.23</td>
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<td>BC</td>
<td>0.167I</td>
<td>0.20I</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>CC₁</td>
<td>0.261I</td>
<td>0.834I</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>CB</td>
<td>0.167I</td>
<td>0.20I</td>
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<td></td>
<td>CC₂</td>
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<td>0.834I</td>
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<td>0.167I</td>
<td>0.20I</td>
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<tr>
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<td>DD₁</td>
<td>0.261I</td>
<td>0.31I</td>
<td>0.31</td>
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<tr>
<td></td>
<td>DC</td>
<td>0.167I</td>
<td>0.25I</td>
<td>0.25</td>
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<td>DD₂</td>
<td>0.261I</td>
<td>0.31I</td>
<td>0.31</td>
</tr>
</tbody>
</table>

**Step 5: Moment Distribution**

**Step 6: Max. tue moment at midspan**

Free bending moment at midspan of BC

\[
= \frac{wL^2}{8} = \frac{32 \times 6^2}{8} = 144 \text{ kNm}
\]

Net BM at Centre of BC

\[
= 144 \left( \frac{89.4 - 89.1}{2} \right) = 54.3 \text{ kNm}
\]
Multistorey building consists of four storeys & 3 bay frames. Spacing at 3m cent to cent. Live load on the floor slab is 3 kN/m² & DL is 3.5 kN/m. The spacing of the beam from left to right are 6m, 4m & 4m respectively. Story height is 3.5m. Moment of inertia of beam is 15 times that of columns. Self wt of beam is 35 kN/m. Determine the max. moment in the beam at the junction of 1st span & 2nd span of an intermediate floor.

**Step-1 Substitute Frame**

![Substitute Frame Diagram]

**Step-2 Load Calculations.**

- **Span AB**
  - DL
  - Slab: $3.5 \text{kN/m}^2 \times 3 \text{m} = 10.5 \text{kN/m}$
  - Slab: $3 \text{kN/m}^2 \times 3 \text{m} = 9 \text{kN/m}$
  - Beam: $3.5 \text{kN/m}$
  - Total DL: $14 \text{kN/m}$
  - Total load: $14 + 9 = 23 \text{kN/m}$

- **Span BC**
  - DL
  - Slab: $3.5 \times 3 = 10.5 \text{kN/m}$
  - Slab: $3 \times 3 = 9 \text{kN/m}$
  - Beam: $3.5 \text{kN/m}$
  - T.DL: $14 \text{kN/m}$
  - Total load: $23 \text{kN/m}$

- **Span CD**
  - DL
  - Slab: $3.5 \times 3 = 10.5 \text{kN/m}$
  - Beam: $3.5 \text{kN/m}$
  - Total load: $14 \text{kN/m}$

**Step-3 Fixed End Moments.**

- **Span AB**
  - $M_{FBA} = \frac{wL^2}{12} = \frac{-23 \times 6^2}{12} = -69 \text{ kNm}$
  - $M_{FAB} = \frac{wL^2}{12} = 69 \text{ kNm}$

- **Span BC**
  - $M_{FBC} = \frac{-23 \times 4^2}{12} = -30.67 \text{ kNm}$
  - $M_{FBC} = \frac{4 \times 4^2}{12} = 0.83 \text{ kNm}$
**Step 4: Distribution Factors**
Assuming $M_I$ of column $1$, $M_2$ of beam $1.5I$

<table>
<thead>
<tr>
<th>Joint</th>
<th>Members</th>
<th>R</th>
<th>$S \left( \frac{I}{L} \right)$</th>
<th>T</th>
<th>S</th>
<th>DF</th>
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<tbody>
<tr>
<td>A</td>
<td>AA</td>
<td>1/8</td>
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<td>0.841</td>
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<tr>
<td></td>
<td>AB</td>
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<td>0.30</td>
<td>0.35</td>
<td></td>
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<tr>
<td></td>
<td>AA</td>
<td>0.241</td>
<td>0.241</td>
<td>0.35</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>BA</td>
<td>0.251</td>
<td>0.241</td>
<td>0.24</td>
<td>0.24</td>
<td></td>
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<tr>
<td></td>
<td>BB</td>
<td>0.24</td>
<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td></td>
<td>CB</td>
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<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
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</tr>
<tr>
<td></td>
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<td>0.31</td>
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<tr>
<td>C</td>
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<tr>
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<tr>
<td>D</td>
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**Step 5: Moment Distribution**

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<tr>
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<th>A</th>
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<th>C</th>
<th>D</th>
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</thead>
<tbody>
<tr>
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**Step 6: Max. BM @ support B**

Maximum moment at joint $B = 69.48$ kNm.

**Note:** Analyze the frame for moments in column shown in the below fig. The frames are spaced at 3.5 m, DL from slab is 3kN/m². LL is 6kN/m². Self wt of 4m span may be taken as 4kN/m and that of 6m span can be taken as 5kN/m. Use 2-cycle moment method.
### Step-2 Load Calculations & FEM

**Slab**
- D.L = 3 kN/m² x 3.6 m = 10.8 kN/m
- L.L = 5 x 3.5 = 17.5 kN/m

**Beam**
- D.L for 4 m span = 4 kN/m
- D.L for 6 m span = 5 kN/m

- Total D.L for 4 m span = 10.5 + 4 = 14.5 kN/m
- Total D.L for 6 m span = 15.5 kN/m

### Step-3 Distribution Factor

<table>
<thead>
<tr>
<th>Joint</th>
<th>Members</th>
<th>R.S</th>
<th>T.S</th>
<th>D.F</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
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<td>7K</td>
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<td>0.429</td>
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<td>AE</td>
<td>2K</td>
<td></td>
<td>0.286</td>
</tr>
<tr>
<td></td>
<td>ABA</td>
<td>3K</td>
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<td>0.182</td>
</tr>
<tr>
<td>C</td>
<td>CB</td>
<td>4K</td>
<td>11K</td>
<td>0.364</td>
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<td>CK</td>
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<td>0.182</td>
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<td>DL</td>
<td>2K</td>
<td></td>
<td>0.286</td>
</tr>
<tr>
<td></td>
<td>DH</td>
<td>2K</td>
<td></td>
<td>0.286</td>
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</table>
**Step 4: Moment Distribution**

Design moment of column: \( - \left[ \text{FEM} + \text{COM} \right] \times \text{DF of column} \)

### Case-1 (DL+LL) @ span AB & CD

<table>
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<tr>
<th>Joint</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>DF</td>
<td>0.286 (A)</td>
<td>0.182 (B)</td>
<td>0.182 (C)</td>
<td>0.286 (D)</td>
</tr>
<tr>
<td>(Top Floor)</td>
<td>0.286 (A)</td>
<td>0.182 (B)</td>
<td>0.182 (C)</td>
<td>0.286 (D)</td>
</tr>
<tr>
<td>Member</td>
<td>AB</td>
<td>BA</td>
<td>BC</td>
<td>CB</td>
</tr>
<tr>
<td>DF</td>
<td>0.429</td>
<td>0.273</td>
<td>0.364</td>
<td>0.364</td>
</tr>
<tr>
<td>FEM</td>
<td>-42.67</td>
<td>42.67</td>
<td>-46.5</td>
<td>46.5</td>
</tr>
<tr>
<td>COM</td>
<td>0.523</td>
<td>0.153</td>
<td>-0.697</td>
<td>0.697</td>
</tr>
<tr>
<td>FEM+COM</td>
<td>-42.14</td>
<td>51.823</td>
<td>-47.191</td>
<td>47.191</td>
</tr>
</tbody>
</table>

**DesignMoment of top column:**
- 18.05
- 0.432
- 8.58
- 8.58
- 4.32
- 12.05

**Design Moment of Bottom Column:**
- 12.05
- 0.842
- 12.05
- 12.05

**Design Moment of Bottom Column:**
- 12.05
- 0.842
- 12.05
- 12.05

**Case-2 (DL+LL) @ span BC**

<table>
<thead>
<tr>
<th>Joint</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>DF</td>
<td>0.286 (A)</td>
<td>0.182 (B)</td>
<td>0.182 (C)</td>
<td>0.286 (D)</td>
</tr>
<tr>
<td>(Top Floor)</td>
<td>0.286 (A)</td>
<td>0.182 (B)</td>
<td>0.182 (C)</td>
<td>0.286 (D)</td>
</tr>
<tr>
<td>Member</td>
<td>AB</td>
<td>BA</td>
<td>BC</td>
<td>CB</td>
</tr>
<tr>
<td>DF</td>
<td>0.429</td>
<td>0.273</td>
<td>0.364</td>
<td>0.364</td>
</tr>
<tr>
<td>DesignMoment of top column</td>
<td>2.418</td>
<td>16.384</td>
<td>-16.384</td>
<td>-2.418</td>
</tr>
<tr>
<td>DesignMoment of bottom column</td>
<td>2.418</td>
<td>16.384</td>
<td>-16.384</td>
<td>-2.418</td>
</tr>
</tbody>
</table>

Figure shows an indeterminate frame of a multistoried structure. If the spacing of frame is 3.5 m, analyze the frame. Given DL & LL of the slabs are 3.5 kN/m² & 4.5 kN/m². Self wt. of beam is may be taken as following values:

- Beam of 6 m span: 4.5 kN/m
- 3 m span: 3 kN/m
- 4 m span: 4 kN/m
The relative stiffness of various members are shown in figure.

2) Horizontal shear taken by (shear resistance) by the interior column is double of that taken by exterior column.

\[ Q + H = \frac{1}{2} (Q + P) \]

Point of contraflexure

? Analyse the frame shown in figure by portal frame method.

ANALYSIS OF HORIZONTAL LOADS IN MULTISTORIED STRUCTURE

\( P \) = \( Q \) + \( H \) = \( \frac{1}{2} (Q + P) \)

Environment effects such as wind & earthquakes introduce lateral forces in frames. This horizontal forces causes axial force in columns and bending moment in all the members of frame.

This forces are assumed to act at joints and use the following methods to analyse the horizontal loads in a multi-storied structure.

3) Portal Frame Method

Assumptions

Point of contraflexure occurs at midpoint of all the members of the frame.

Step 1 Assumptions

Point of contraflexure is at midpoint of all members.

Shear resistance offered by interior columns is twice that of exterior columns.

For more study materials: WWW.KTUSTUDENTS.IN
Step 2: Horizontal shear in column.

Let $P$ be the shear resistance offered by exterior column.

$$\Sigma H = 0$$

$$20 = P + P$$

$$\therefore P = 10 \text{ KN}$$

Step 3: Column moment.

$$M_{CA} - M_{AC} = M_{DB} - M_{BD} = -\left( \frac{P \times h}{2} \right) = -\left( \frac{10 \times 6}{2} \right) = -30 \text{ KNm}$$

Step 4: Beam moments.

$$M_{CD} = -M_{CA} = -(-30) = 30 \text{ KNm}$$

$$M_{DC} = -M_{DB} = -(-30) = 30 \text{ KNm}$$

Step 5: Shear in beam.

Taking mmt about C, $30 + 30 = (R_D \times 4) = 0$
Analyze the portal frame shown in the figure by portal frame method.

Step-1 Assumptions

a) Point of contraflexure is at the mid point of all members.

b) Shear resistance offered by interior columns are twice that of exterior columns.

Let $P$ be the shear resistance offered by exterior columns of top floor & $Q$ be the shear resistance offered by exterior columns of bottom floor.

Step-2 Horizontal shear in columns

Step-3 Column moments

- \[ M_{EC} = -P \times \frac{h}{2} = -10 \times 5 \times \frac{1}{2} = -25 \text{ kNm} \]
- \[ M_{CE} = -P \times \frac{h}{2} = -10 \times 5 \times \frac{1}{2} = -25 \text{ kNm} \]
- \[ M_{FD} = -10 \times 5 \times \frac{1}{2} = -25 \text{ kNm} \]
- \[ M_{OF} = -10 \times 5 \times \frac{1}{2} = -25 \text{ kNm} \]
- \[ M_{CA} = -Q \times \frac{h}{2} = -30 \times 5 \times \frac{1}{2} = -75 \text{ kNm} \]
- \[ M_{AC} = M_{DB} = M_{BD} = 75 \text{ kNm} \]

Step-4 Beam moments

- \[ M_{EF} = -M_{EC} = -(-25) = 25 \text{ kNm} \]
- \[ M_{FE} = -M_{FD} = -(-25) = 25 \text{ kNm} \]

At joint C:

- \[ M_{CO} = 15 - 25 = 0 \]
- \[ M_{DC} = 15 + 25 = 40 \text{ kNm} \]
- \[ M_{CD} = 100 \text{ kNm} \]
Step 5: Shear in beam

a) Top floor beam
   taking mmt abt E

\[ 25 + 25 - (RF \times 4) = 0 \]
\[ RF + 12.5kN(\uparrow) \]
\[ RF + RE = 0 \]
\[ RE = -12.5kN (\downarrow) \]

b) Bottom floor beam
   taking mmt abt C

\[ 100 + 100 - (RD \times 4) = 0 \]
\[ RD = 50kN (\uparrow) \]
\[ RC + RD = 0 \]
\[ RC = -50kN (\downarrow) \]

Step 6: Axial force in columns

In top floor

Axial force in EC = shear in EF = 12.5kN (\downarrow)
Axial force in FD = shear in FE = 12.5kN (\uparrow)
Axial force in CA = shear in CD + axial force in EC
   = 12.5 + 50 = 62.5kN (\downarrow)
Axial force in DB = shear in DC + axial force in FD
   = 12.5 + 50 = 62.5kN (\uparrow)

Cantilever Method

Assumptions

1) Point of contraction is at the middle of all members
2) The direct stress (axial) in column due to space horizontal
   forces is directly proportional to their distance from the
   centroidal vertical axis of frame.

Analyze the frame shown in the figure using cantilever
   method and draw the bending moment & shear force
   diagram.
Step 1 Assumptions
a) Point of contra-flexure is at the mid point of all members.
b) Axial stress & distance from centroidal vertical axis
Centroidal vertical axis (taking exterior columns as reference) Assume $a_1 = a_2 = a$ (Area of columns are same)

$$\bar{x} = \frac{a_1 x_1 + a_2 x_2}{a_1 + a_2} = \frac{a x_0 + a x_6}{a + a} = \frac{6a}{2a} = 3m$$

$$\bar{x}_1 = \bar{x}_2 = 3m$$

Step 2 Axial Force in columns.

Stress & distance from centroidal vertical axis

$$F_a \propto \bar{x}$$

Top story

$$\frac{F_{AC}}{a} = \frac{F_{AC}}{a_1} \propto \bar{x}_1 \quad \frac{F_{AC}}{a} = \frac{F_{BD}}{a_2} \propto \bar{x}_2$$

$$\frac{F_{AC}}{a_1} - \frac{F_{BD}}{a_2} = 20KN$$

$$2M_{O2} = 0 \Rightarrow (20 \times 2) - (F_{BD} \times 6) = 0$$

$$F_{BD} = 6.67 (\uparrow) KN$$

$$F_{AC} = 6.67 (\downarrow) KN$$

Bottom story

$$F_{CE} = F_{DF}$$

$$2M_{O5} = 0 \Rightarrow (20 \times 6) + (40 \times 2) - (F_{DF} \times 6) = 0$$

$$F_{DF} = 33.33 KN (\uparrow)$$

$$F_{CE} = 33.33 KN (\downarrow)$$
Step 3: Shear Force in beam

Axial Force in AC = shear in beam AB
= shear in AB = 6.67 kN (↑)

Shear in beam BA = Axial Force in DB = 6.67 kN (↓)

Shear in beam CD = Axial Force in CE - Axial Force in AC
= 33.33 - 6.67
= 26.66 kN (↑)

Shear in beam OC = Axial Force in DF - Axial Force in BD
= 33.33 - 6.67
= 26.66 kN (↑)

Step 4: Beam moments

M_{AB} = M_{BA} - 6.67 \times 3
= 20.01 kNm

M_{CD} = M_{DC} - 6.67 \times 3
= 80.01 - 80 kNm

Step 5: Column moments

Joint A
\[ \Sigma M_A = 0 \Rightarrow M_A + M_{AC} = 0 \]
\[ M_A = -M_{AC} = -20 kNm \]

Joint B
\[ \Sigma M_B = 0 \Rightarrow M_B + M_{BD} = 0 \]
\[ M_{BD} = -M_B = -20 kNm \]

Joint D
\[ \Sigma M_D = 0 \Rightarrow M_{DB} + M_{DC} + M_{DF} = 0 \]
\[ M_{DB} = -M_{DC} - M_{DF} = -20 + 80 + M_{DF} = 0 \]
\[ M_{CE} = 60 kNm \]
\[ M_{DF} = -60 kNm \]

M_{CE} = M_{EC} = -60 kNm

Step 6: Horizontal shear in columns

Considering AC
\[ \Sigma M_A = 0 \Rightarrow F_{AC} \cdot 4 \]
\[ F_{AC} = 40 / 4 = 10 kNm (↑) \]

Considering BD
\[ \Sigma M_B = 0 \Rightarrow -20 + (F_{DB} \times 4) = 0 \]
\[ F_{DB} = 40 / 4 = 10 kNm (↑) \]

Considering CE
\[ \Sigma M_C = 0 \Rightarrow -60 - (F_{EC} \times 4) = 0 \]
\[ F_{EC} = 30 kNm (↑) \]

Consider CE
\[ \Sigma M_E = 0 \Rightarrow F_{CE} = -30 kNm (↓) \]

Consider CE
\[ \Sigma M_E = 0 \Rightarrow F_{CE} = -30 kNm (↓) \]

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Scanned by CamScanner
Step-1 Assumptions

a) Point of contraflexure is at midpoint.
b) Axial force at distance from centroidal vertical axis.

Centroidal vertical axis, assuming,

c) Area of cross-section of columns are same (a)
d) Taking exterior columns as reference

\[ \bar{x} = \frac{a_1 + a_2 + a_3 + \ldots + a_n}{a_1 + a_2 + a_3 + \ldots + a_n} \]

\[ \bar{x}_1 = 0 \]
\[ \bar{x}_2 = 4 \text{ m} \]
\[ \bar{x}_3 = 8 \text{ m} \]
\[ \bar{x}_4 = 15 \text{ m} \]

\[ \bar{x} = \frac{4a + 9a + 15a}{40a} = \frac{38a}{40a} = 0.95 \text{ m} \]

Step-2 Axial force in columns

Check according to 2nd assumption

Stress at distance from vertical centroidal axis

\[ \bar{x}_1 = 1 \text{ m} \]
\[ \bar{x}_2 = 7 - 4 = 3 \text{ m} \]
\[ \bar{x}_3 = 2 \text{ m} \]
\[ \bar{x}_4 = 8 \text{ m} \]
Top Storey

\[ F_{AE} = \frac{K_1 x A}{a} \]
\[ F_{BF} = \frac{K_2 x B}{a} \]
\[ F_{CA} = \frac{K_3 x C}{a} \]
\[ F_{DH} = \frac{K_4 x D}{a} \]

\[ F_{AE} = \frac{F_{BF}}{a} = \frac{F_{CA}}{a} = \frac{F_{DH}}{a} \]

\[ \sum M_{b} = 0 \rightarrow (12 \times 5) + \frac{24 x 2}{3} + (F_{CA} x 4) = (F_{DH} x 15) \]
\[ (12 \times 5) + (24 x 2) + (F_{CA} x 4) - (F_{DH} x 15) = 0 \]

\[ F_{CA} = \frac{8 x 15}{3} \]
\[ F_{DH} = \frac{8 x 15}{3} \]

\[ F_{AE} = \frac{F_{BF}}{3a} = \frac{F_{CA}}{2a} = \frac{F_{DH}}{8a} \]

Bottom Storey

\[ F_{EI} = \frac{F_{EI}}{3a} = \frac{F_{EI}}{2a} = \frac{F_{EI}}{8a} \]

\[ F_{EI} = 6 KN \]
\[ F_{FO} = 2.56 KN \]
\[ F_{HI} = 1.16 KN \]
\[ F_{HL} = 8888 KN \]

Step 3: Shear Forces in Beams.

Shear Force in Beam AB = Axial Force in Column BC = 1 KN
\[ SF_{B C} = 1.43 \text{ KN (4)} \]
\[ SF_{C B} = 1.14 \text{ KN (4)} \]
\[ SF_{F C D} = -1.43 \text{ KN (4)} \]
\[ SF_{D C} = 1.14 \text{ KN (4)} \]

Bottom storey

\[ SF_{E D F} \]
\[ \sum V = 0 \Rightarrow F_{E A} + F_{D F} + F_{E D} = 0 \]
\[ F_{ED} = -5 \text{ KN (4)} \]
\[ SF_{E F} = 5 \text{ KN (4)} \]
\[ SF_{F G} \]
\[ \sum V = 0 \Rightarrow F_{F E} + F_{F B} + F_{E F} + F_{F G} = 0 \]
\[ F_{F G} = -5 \text{ KN (4)} \]
\[ SF_{G} = 5.14 \text{ KN (4)} \]

\[ SF_{G A} = 5.14 \text{ KN (4)} \]
\[ SF_{G H} = 5.14 \text{ KN (4)} \]

Step 4: Beam moment

\[ M_{AB} = M_{BA} (1 \times 2) = 2 \text{ kNm} \]
\[ M_{BC} = M_{CB} = (1.43 \times 2.5) = 3.575 \text{ kNm} \]
\[ M_{CD} = M_{DC} = (1.14 \times 3) = 3.42 \text{ kNm} \]
\[ M_{EF} = M_{FE} = (6 \times 2) = 10 \text{ kNm} \]
\[ M_{DG} = M_{DG} = (7.14 \times 2.5) = 17.85 \text{ kNm} \]
\[ M_{GH} = M_{HG} = (5.14 \times 3) = 15.42 \text{ kNm} \]
Step 5: Column moments

\[ \Sigma M_{A} = 0 \Rightarrow M_{AE} + M_{AB} = 0 \Rightarrow M_{AE} = -M_{AB} \]

\[ M_{AE} = -2kN.m \]

\[ \Sigma M_{B} = 0 \Rightarrow M_{BA} + M_{EB} + M_{BC} = 0 \Rightarrow M_{EB} = -[M_{BA} + M_{BC}] \]

\[ M_{EB} = -(2 + 3.42) = 5.88kN.m \]

\[ \Sigma M_{C} = 0 \Rightarrow M_{CB} + M_{CD} = 0 \Rightarrow M_{CD} = -M_{CB} \]

\[ M_{CD} = -(3.545 + 4.32) = -7.86kN.m \]

\[ \Sigma M_{D} = 0 \Rightarrow M_{DE} = 0 \Rightarrow M_{DE} = -2kN.m \]

\[ \Sigma M_{E} = 0 \Rightarrow M_{EF} = 0 \Rightarrow M_{EF} = -2.48kN.m \]

\[ \Sigma M_{F} = 0 \Rightarrow M_{FG} + M_{FH} = 0 \Rightarrow M_{FH} = 2.48kN.m \]

\[ \Sigma M_{G} = 0 \Rightarrow M_{GH} = 0 \Rightarrow M_{GH} = -2.48kN.m \]

\[ \Sigma M_{H} = 0 \Rightarrow M_{HI} = 0 \Rightarrow M_{HI} = -2.48kN.m \]

\[ \Sigma M_{I} = 0 \Rightarrow M_{IJ} = 0 \Rightarrow M_{IJ} = -2.48kN.m \]
\[ \Sigma M_{O1} = 0 \Rightarrow (F_{KCA} \times 4) - 28 - 28 = 0 \]

\[ F_{KCA} = 14 \text{ KN} \ (\leftarrow) \]

\[ F_{CAK} = 14 \text{ KN} \ (\rightarrow) \]

\[ \Sigma M_{H} = 0 \Rightarrow (F_{HL} \times 4) - 13.72 - 13.72 = 0 \]

\[ F_{HL} = 8.86 \text{ KN} \ (\leftarrow) \]

\[ F_{HL} = 8.86 \text{ KN} \ (\rightarrow) \]