



Lecture 08

Analysis of Reinforced Concrete Structures

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Lecture Contents

- General
- **Section – I : Gravity Load Analysis**
 - Point of Inflection Method
 - Equivalent Frame Method
 - Example 8.1
 - Case Study 1



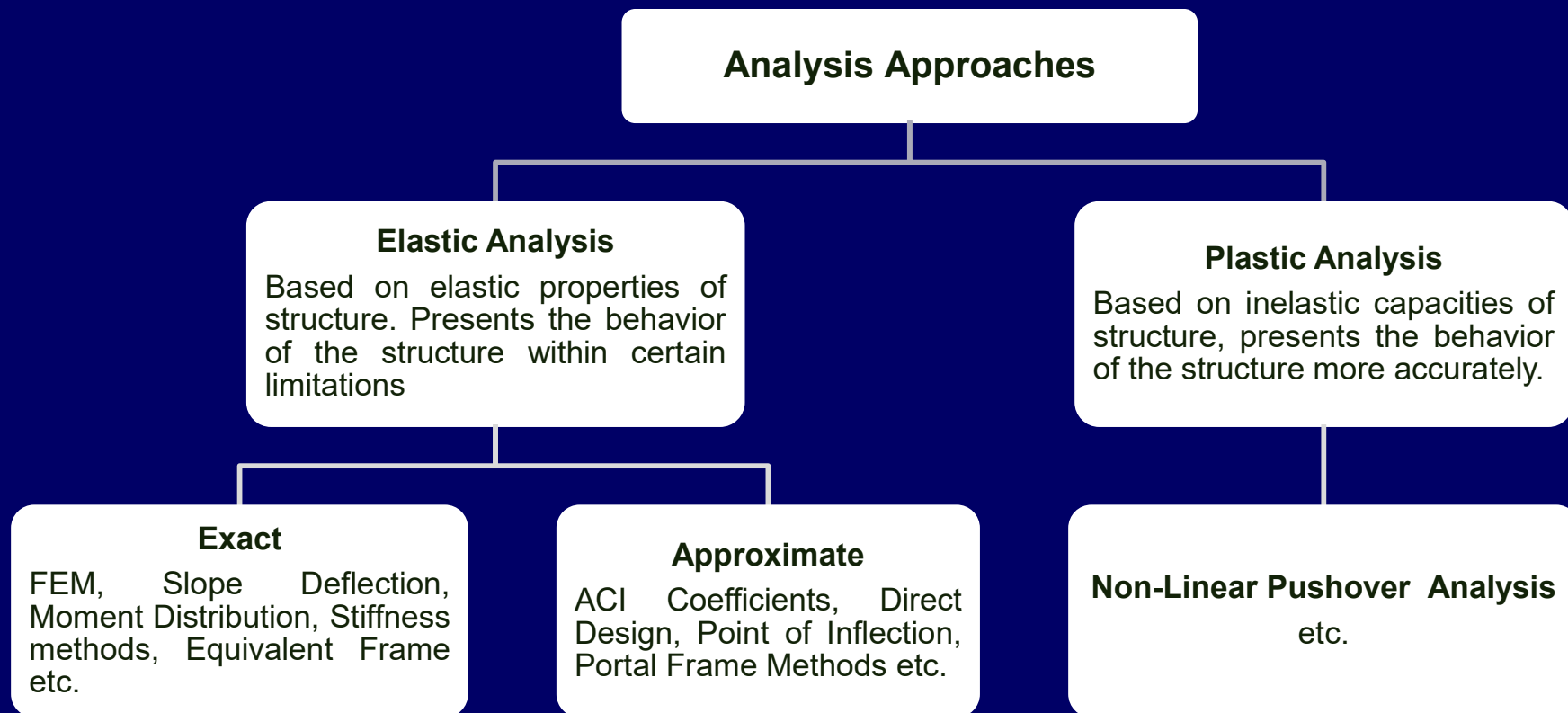
Lecture Contents

- **Section – II : Lateral Load Analysis**
 - Portal Frame Method
 - Case Study 2
- Limit Analysis
- Example 8.2
- Plastic Analysis
- Modern Analysis Tools
- References



General

□ Analysis Approaches





General

□ Analysis Approaches

- The approximate analysis methods such as ACI Coefficients and Direct Design Method have been discussed in detail in earlier lectures.
- In this lecture, another approximate method known as Point of Inflection Method will be briefly discussed.
- The exact analysis methods such as Slope Deflection, Moment Distribution and Stiffness method etc. have already been studied. The Equivalent Frame Analysis method will be discussed in detail in this lecture.



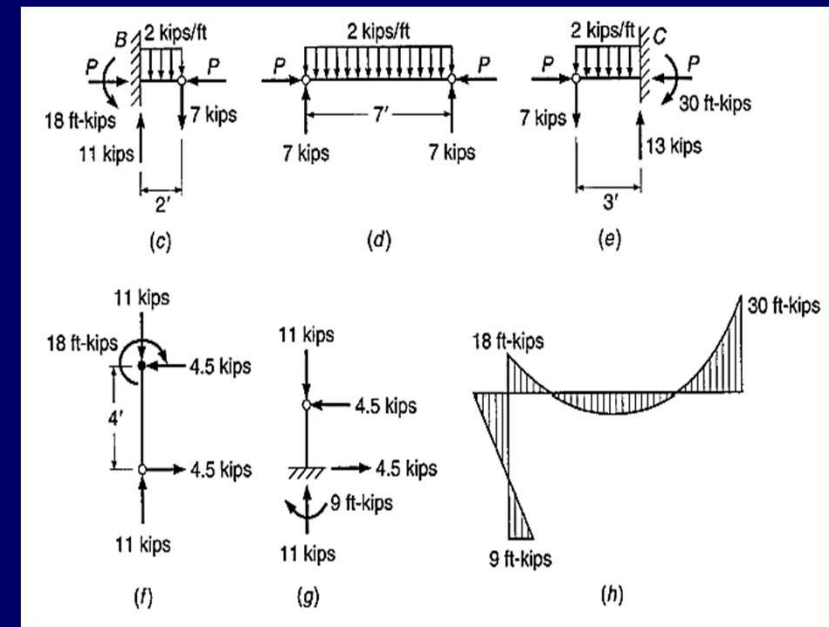
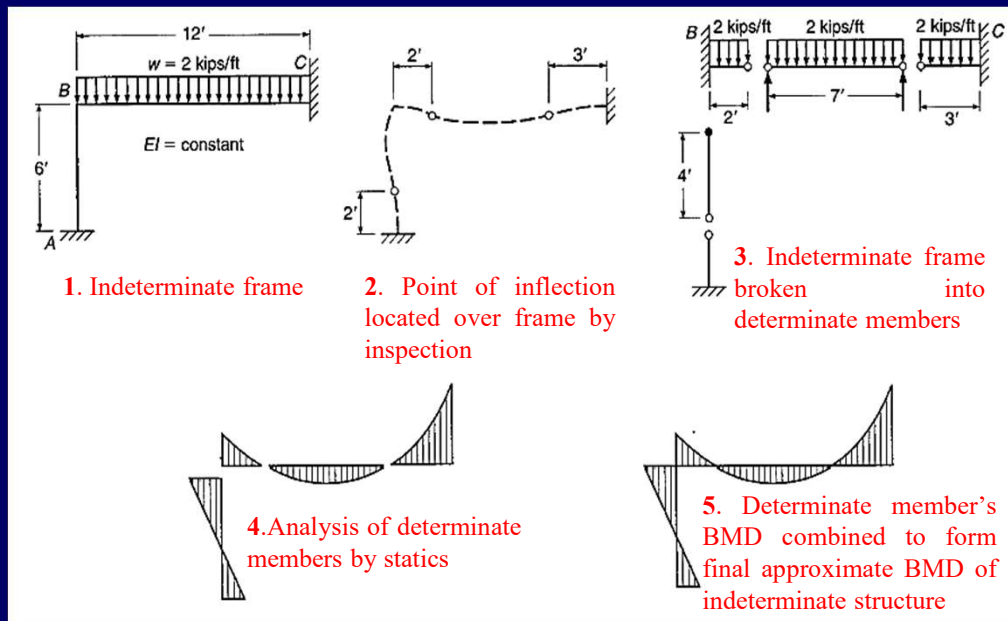
Gravity Load Analysis of RC Frames



Point of Inflection Method

Introduction

- In this method, points of inflection are located on the frame and the members are assumed separate determinate members at point of inflection.
- The individual members can be analyzed by statics as shown below.





Equivalent Frame Method

□ Introduction

- The equivalent frame method involves the representation of the three-dimensional slab system by a series of two-dimensional frames that are then analyzed for loads acting in the plane of the frames.
- The negative and positive moments so determined at the critical design sections of the frame are distributed to the slab sections.
- While no longer included in the latest editions of the ACI Code, a comprehensive explanation of this method is available in section 8.11 of the ACI 318-14.
- The stepwise procedure of this method is described next.



Equivalent Frame Method

□ Steps in Equivalent Frame Method

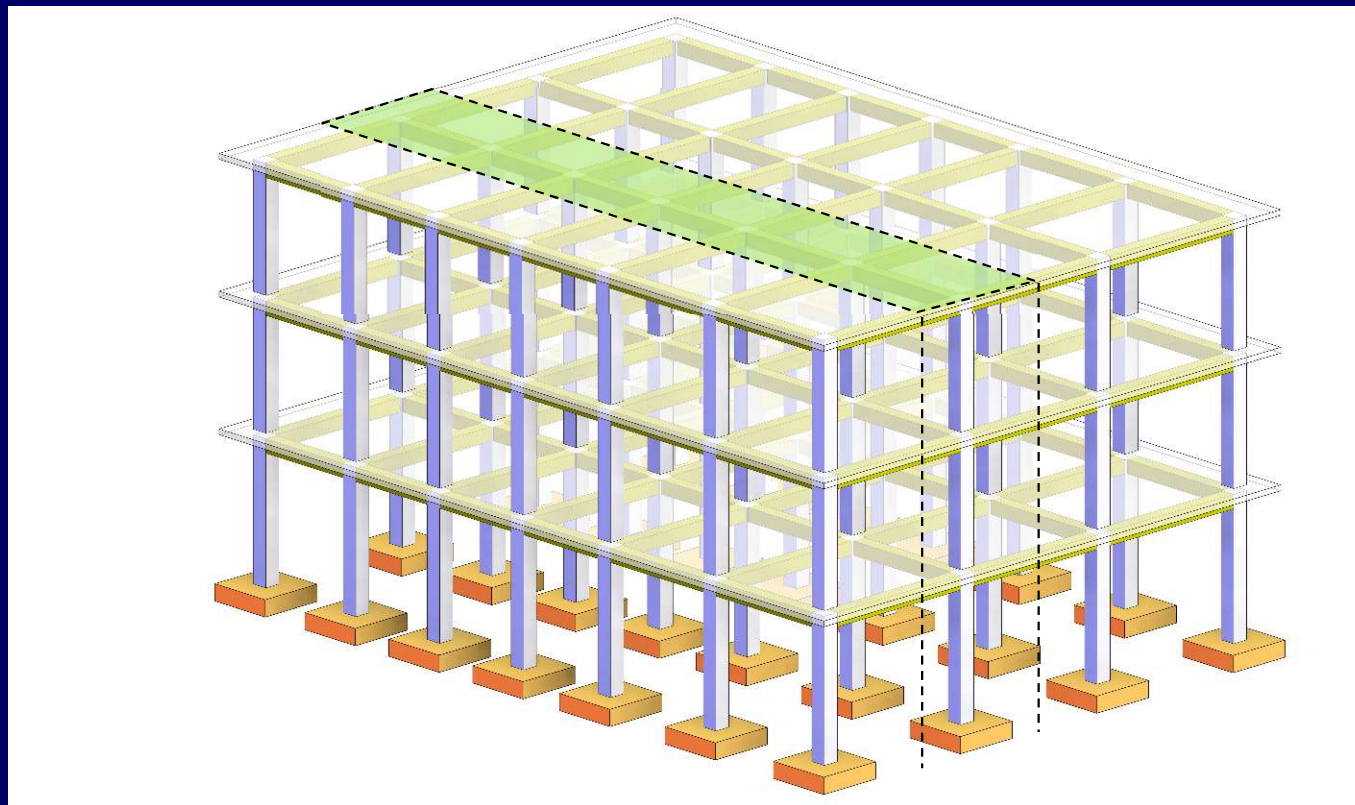
- The equivalent frame method involves the following three major steps.
 1. Extraction of Frame from 3D Model
 2. Determination of Stiffnesses
 3. Analysis of Frame using Moment Distribution Method
- Each step is comprehensively described in subsequent slides.



Equivalent Frame Method

□ Step 1: Extraction of Frame

- The initial step involves selecting or marking a 3D frame within the 3D building model.

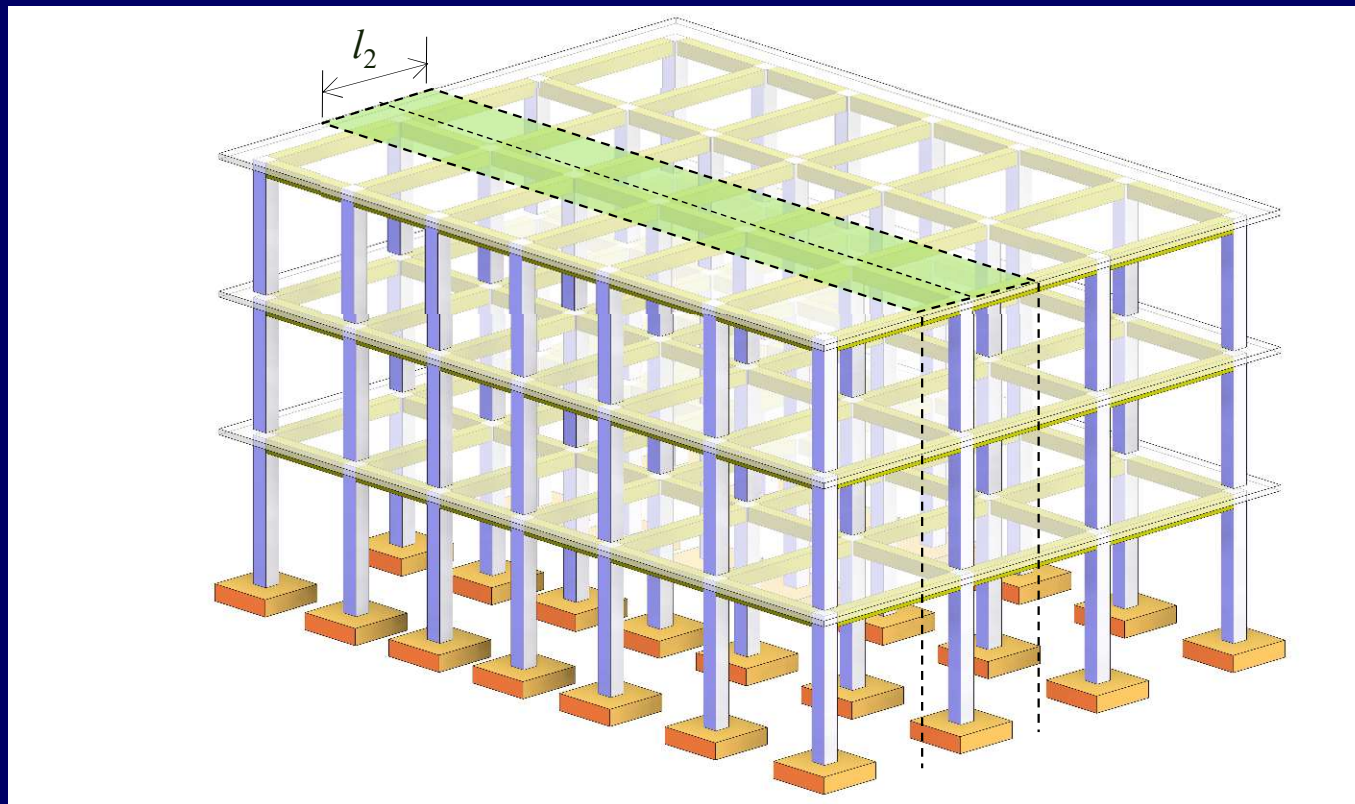




Equivalent Frame Method

□ Step 1: Extraction of Frame

- The width of the frame is same as in DDM and length of the frame extends up to full length of 3D system and the full height of the building as shown below.

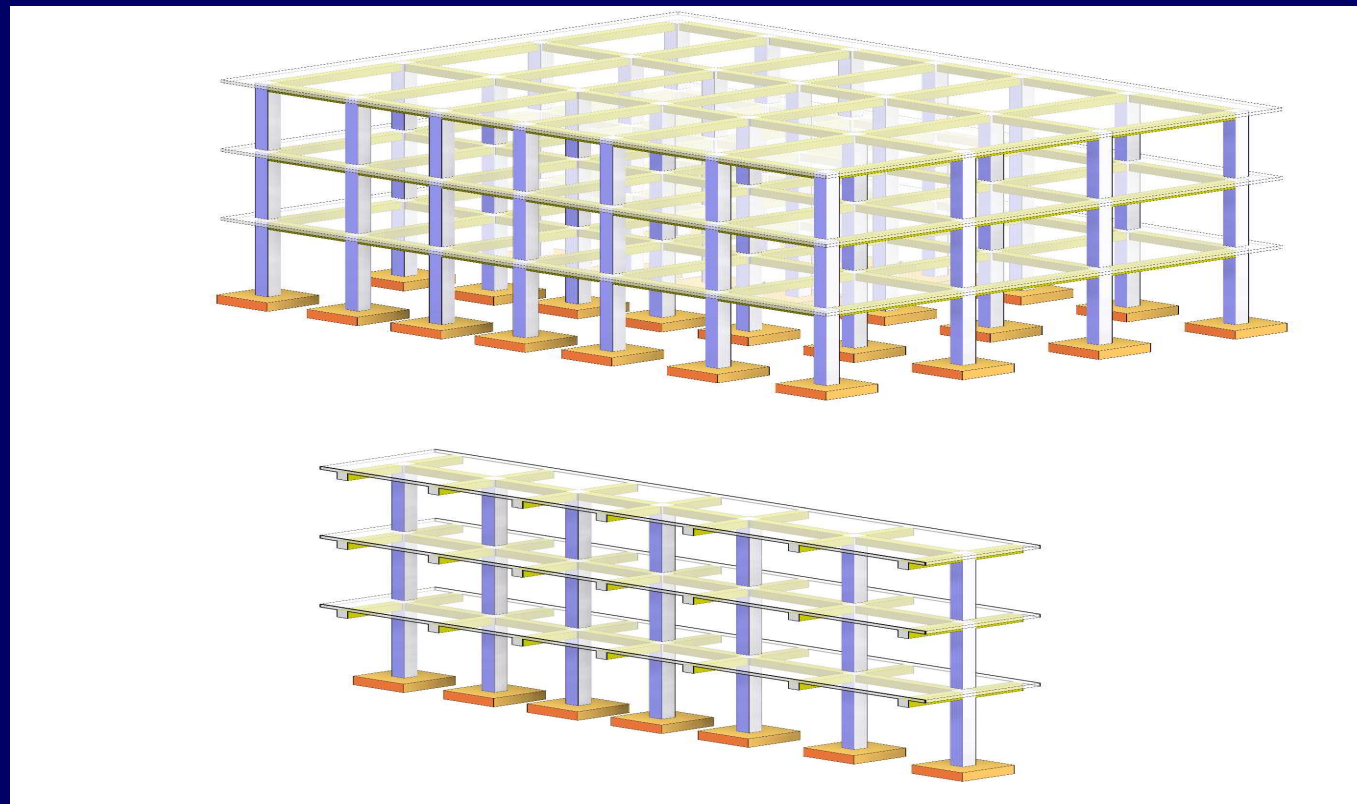




Equivalent Frame Method

□ Step 1: Extraction of Frame

- The selected 3D Frame is then extracted from the 3D Model. This is now called as **equivalent frame**.

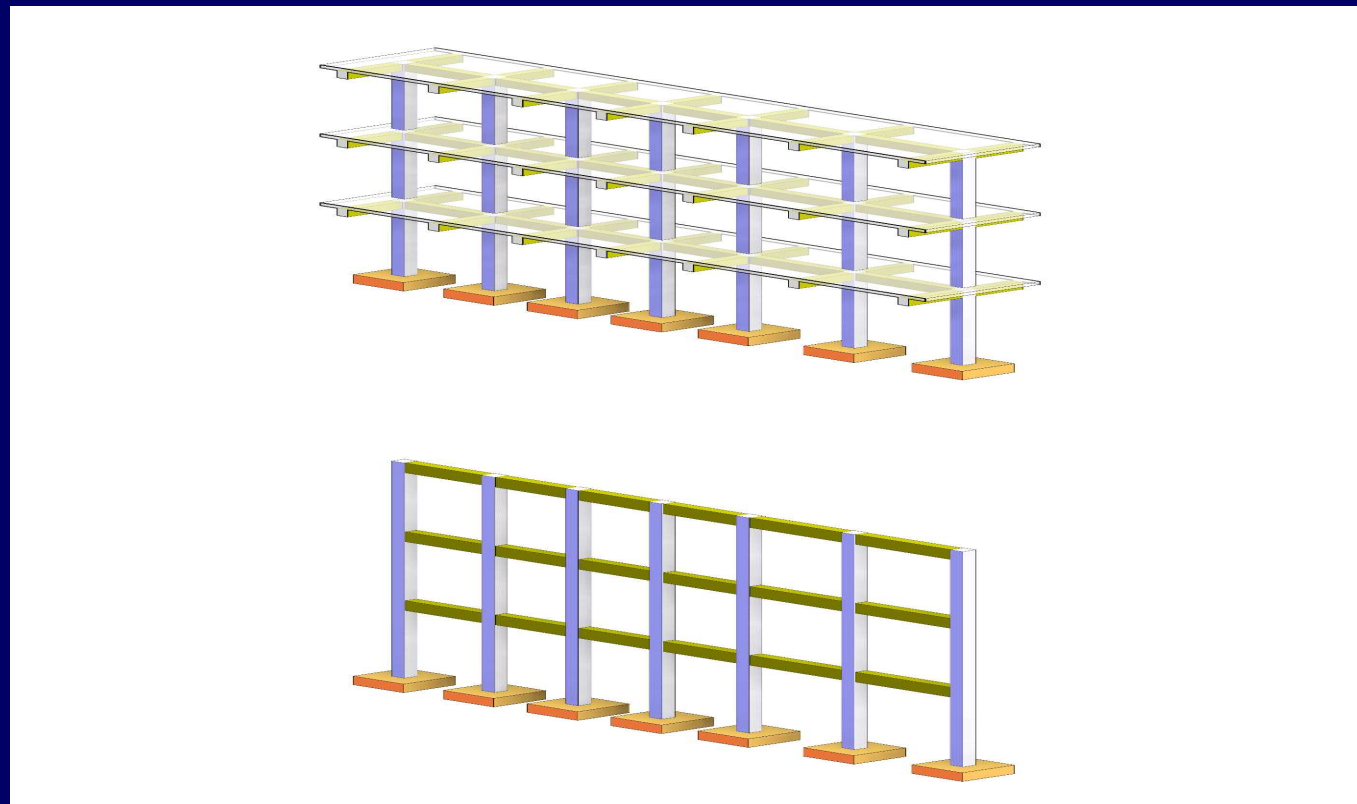




Equivalent Frame Method

□ Step 1: Extraction of Frame

- The equivalent frame is transformed into a 2D frame by taking the effect of stiffnesses of laterally present members (slabs and beams).

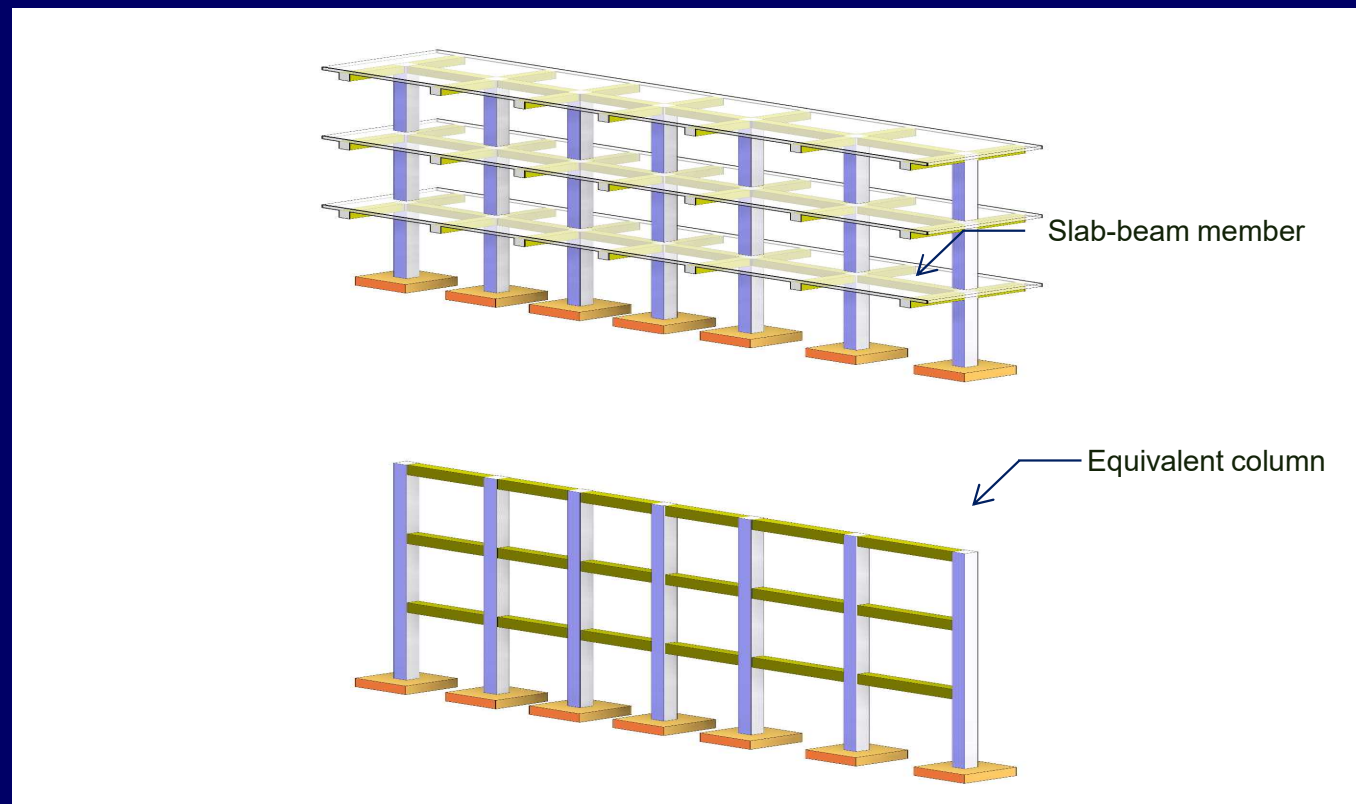




Equivalent Frame Method

□ Step 1: Extraction of Frame

- The horizontal members of the converted 2D frame are called **slab-beam members** and the vertical members are called **equivalent columns**.

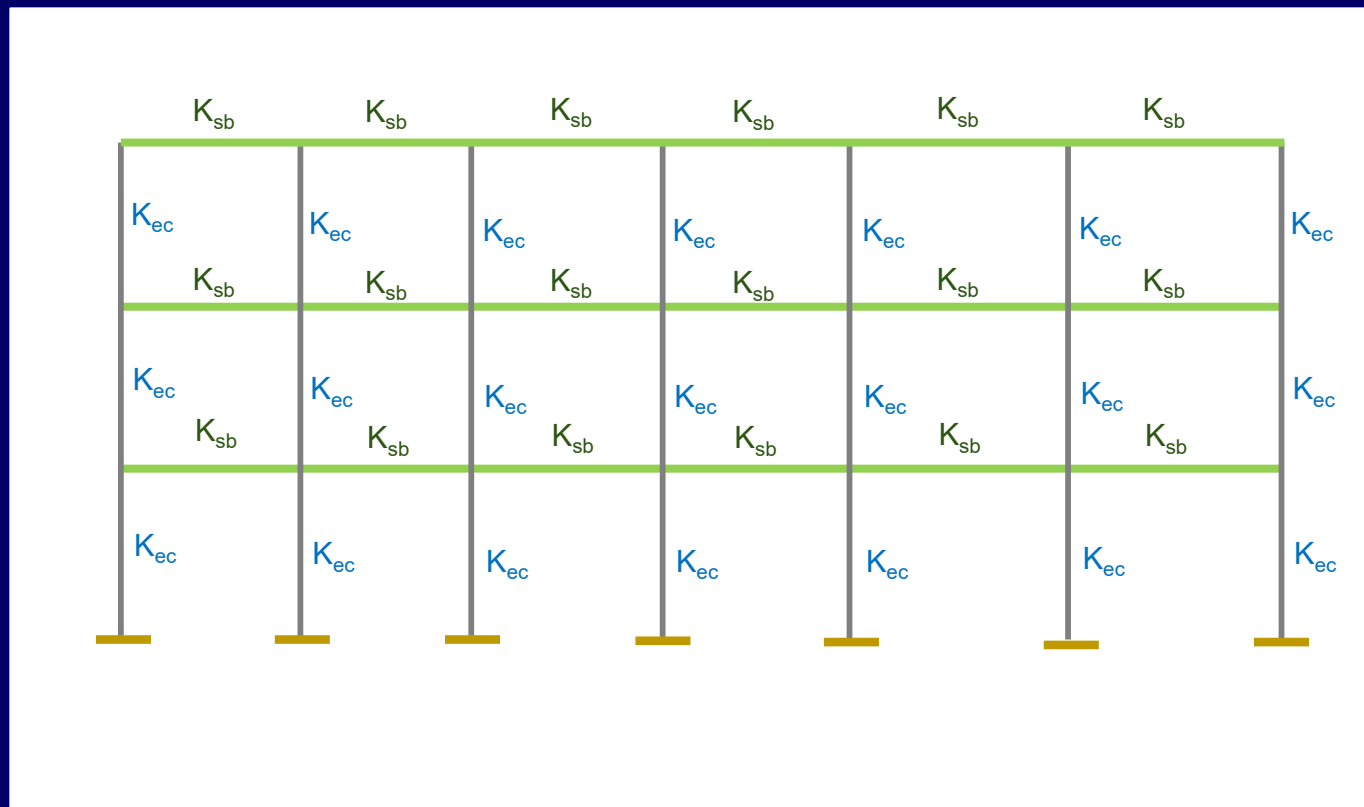




Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

- Stiffnesses are calculated and assigned to the slab-beam and equivalent columns.

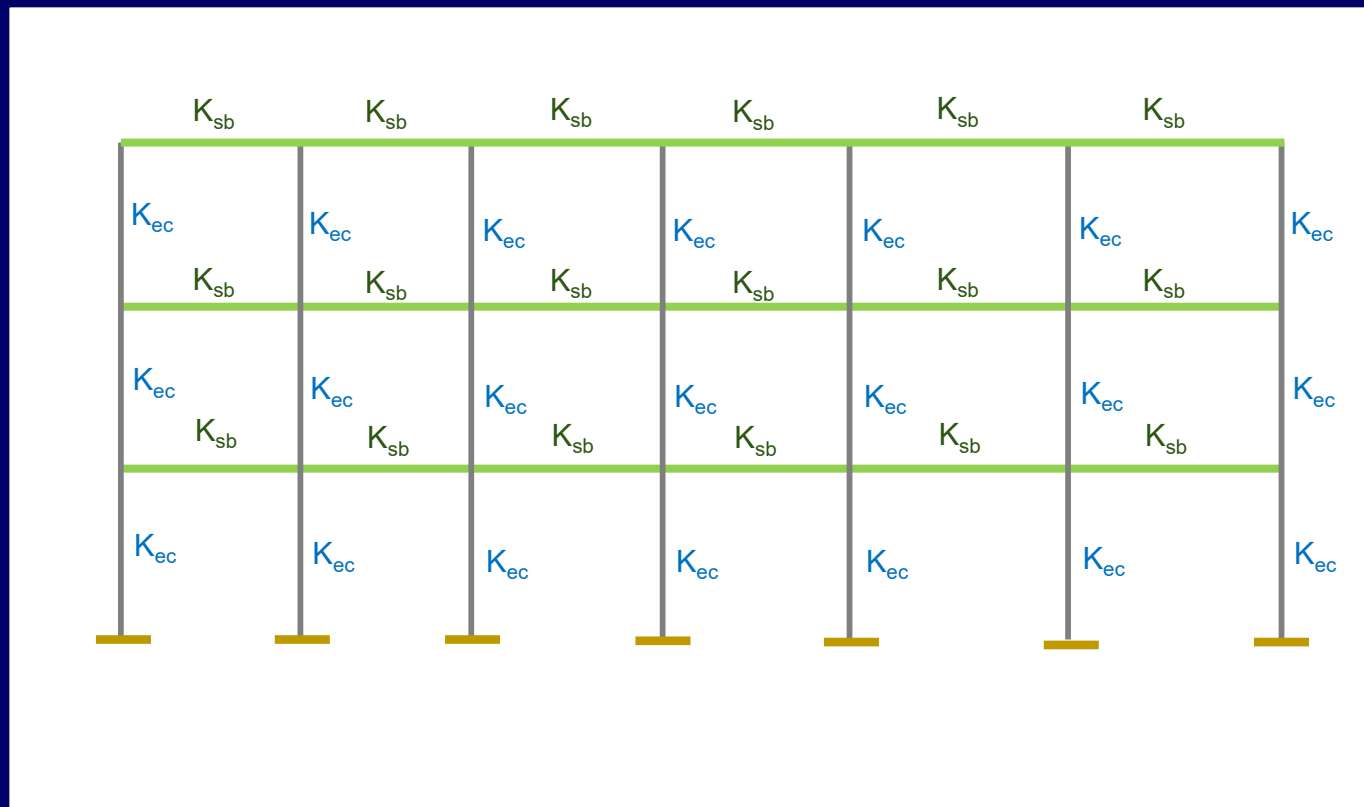




Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

- K_{sb} represents the combined stiffness of slab and longitudinal beam and K_{ec} represents the modified column stiffness.



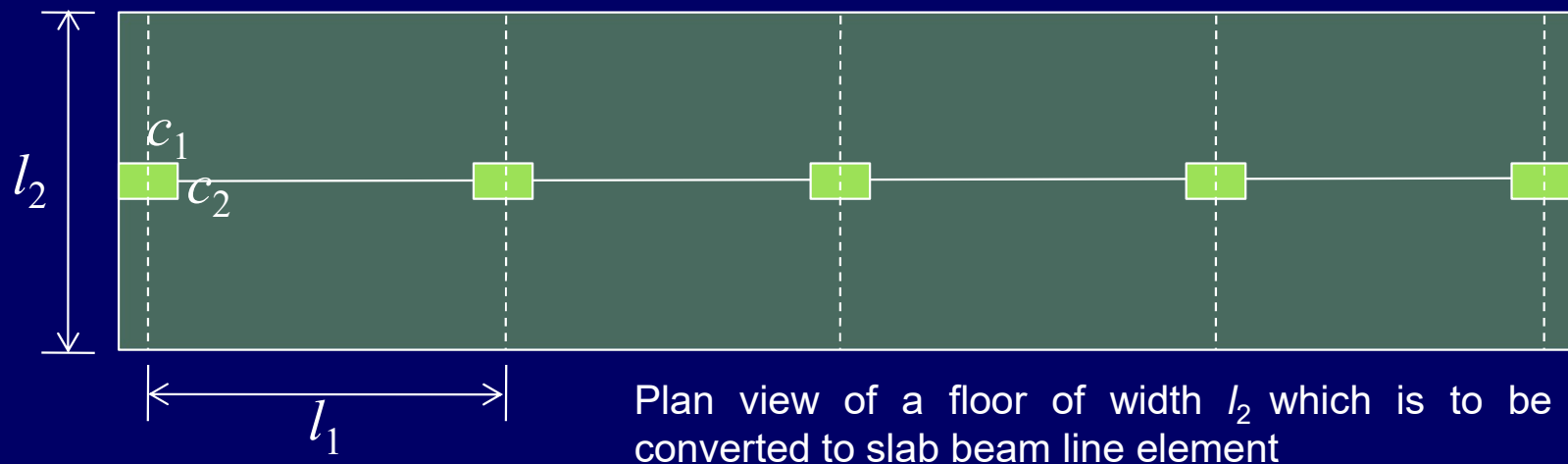


Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Slab Beam Member K_{sb}

- The stiffness of slab beam ($K_{sb} = kEI_{sb}/l$) consists of combined stiffness of slab and any longitudinal beam present within.
- For a span, the k factor is a direct function of ratios c_1/l_1 and c_2/l_2
- Tables are available for determination of k for various conditions of slab systems.





Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Slab Beam Member K_{sb} – Determination of k

| Moment-Distribution Factors for Slabs without Drop Panels ^a | | | | | | | |
|------------------------------------------------------------------------|-----|--------------|------------------------------------------|-------|-------|-------|-------|
| FEM (uniform load w) = $Mwl_2\ell_1^2$ | | | K (stiffness) = $kE\ell_2t^3/12\ell_1$ | | | | |
| Carryover factor = COF | | | | | | | |
| | | c_2/ℓ_2 | | | | | |
| c_1/ℓ_1 | | 0.00 | 0.05 | 0.10 | 0.15 | 0.20 | 0.25 |
| 0.00 | M | 0.083 | 0.083 | 0.083 | 0.083 | 0.083 | 0.083 |
| | k | 4.000 | 4.000 | 4.000 | 4.000 | 4.000 | 4.000 |
| | COF | 0.500 | 0.500 | 0.500 | 0.500 | 0.500 | 0.500 |
| 0.05 | M | 0.083 | 0.084 | 0.084 | 0.084 | 0.085 | 0.085 |
| | k | 4.000 | 4.047 | 4.093 | 4.138 | 4.181 | 4.222 |
| | COF | 0.500 | 0.503 | 0.507 | 0.510 | 0.513 | 0.516 |
| 0.10 | M | 0.083 | 0.084 | 0.085 | 0.085 | 0.086 | 0.087 |
| | k | 4.000 | 4.091 | 4.182 | 4.272 | 4.362 | 4.449 |
| | COF | 0.500 | 0.506 | 0.513 | 0.519 | 0.524 | 0.530 |
| 0.15 | M | 0.083 | 0.084 | 0.085 | 0.086 | 0.087 | 0.088 |
| | k | 4.000 | 4.132 | 4.267 | 4.403 | 4.541 | 4.680 |
| | COF | 0.500 | 0.509 | 0.517 | 0.526 | 0.534 | 0.543 |
| 0.20 | M | 0.083 | 0.085 | 0.086 | 0.087 | 0.088 | 0.089 |
| | k | 4.000 | 4.170 | 4.346 | 4.529 | 4.717 | 4.910 |
| | COF | 0.500 | 0.511 | 0.522 | 0.532 | 0.543 | 0.554 |
| 0.25 | M | 0.083 | 0.085 | 0.086 | 0.087 | 0.089 | 0.090 |
| | k | 4.000 | 4.204 | 4.420 | 4.648 | 4.887 | 5.138 |
| | COF | 0.500 | 0.512 | 0.525 | 0.538 | 0.550 | 0.563 |
| $x = (1 - c_2/\ell_2^3)$ | | 1.000 | 0.856 | 0.729 | 0.613 | 0.512 | 0.421 |

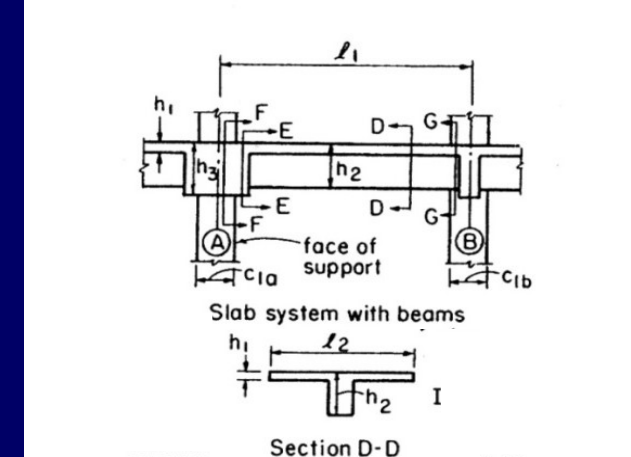
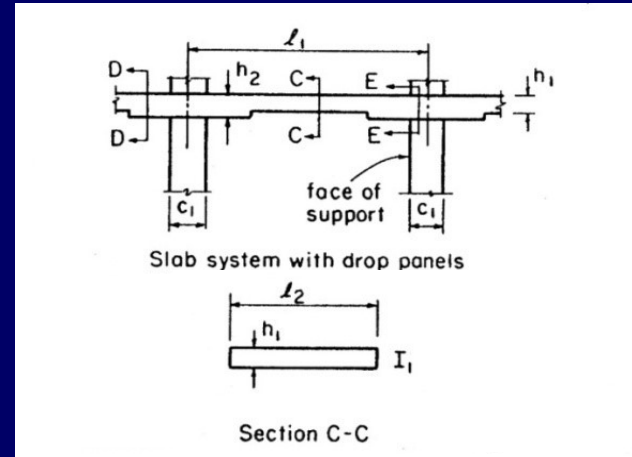
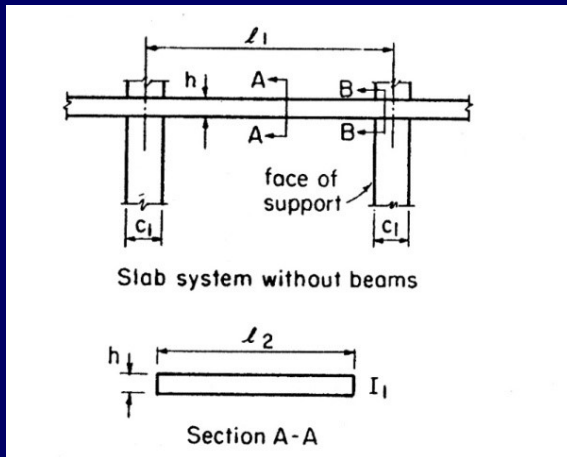
^a c_1 and c_2 are the widths of the column measured parallel to ℓ_1 and ℓ_2 .
Source: [13-16].



Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Slab Beam Member K_{sb} – Determination of I_{sb}





Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Equivalent Column K_{EC}

- Stiffness of equivalent column consists of stiffness of actual columns (above and below the slab) plus stiffness of torsional members.

- Mathematically,

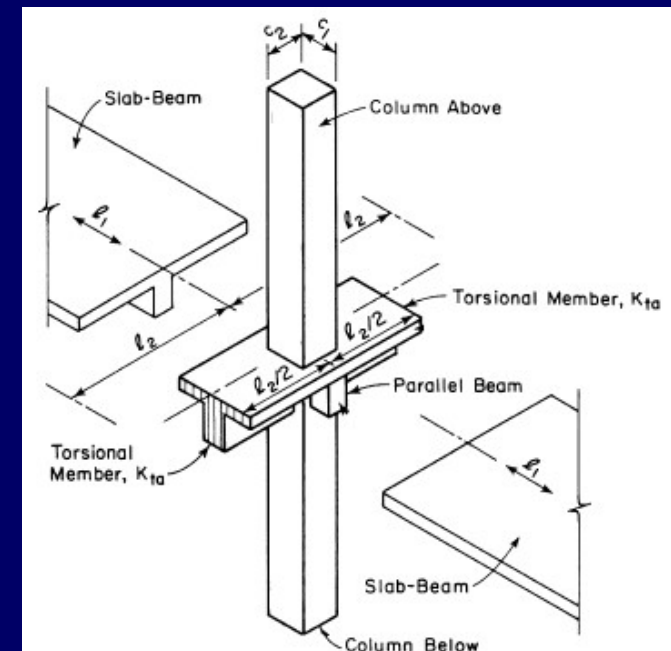
$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{\sum K_t}$$

$$K_{ec} = \frac{\sum K_c \times \sum K_t}{\sum K_c + \sum K_t}$$

where;

$\sum K_c$ = sum of flexural stiffnesses of columns above and below the slab.

$\sum K_t$ = Torsional stiffness of attached torsional members



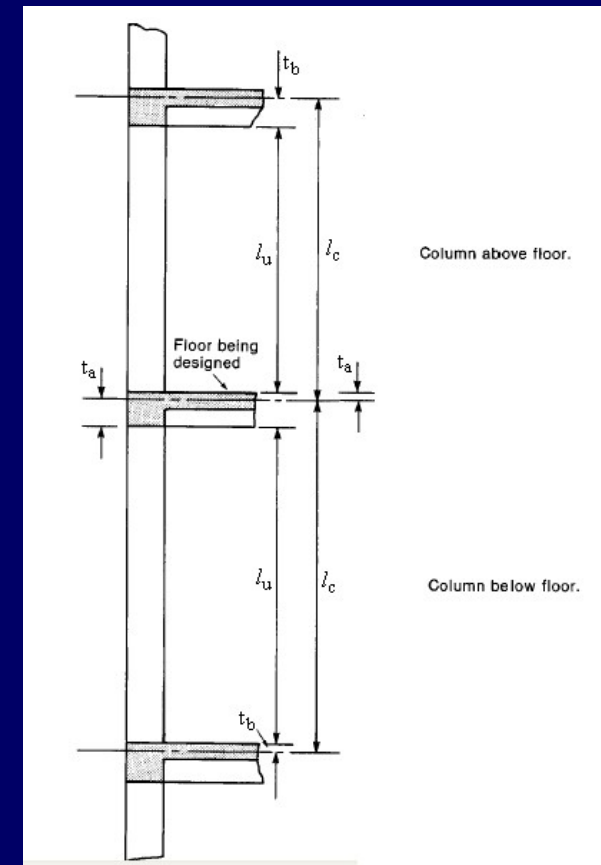


Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Equivalent Column K_{EC}

- General formula of flexural stiffness is given by $K = kEI/I$
- Design aids are available from which value of k can be readily obtained for different values of (t_a/t_b) and (l_u/l_c) .
- These design aids can be used if moment distribution method is used as method of analysis.





Equivalent Frame Method

Step 2: Determination of Stiffnesses

Stiffness of Equivalent Column K_{EC} – Determination of k

TABLE A-17 Stiffness and Carryover Factors for Columns

$$K_c = \frac{EL_c}{l_c}$$

| t_a/t_b | l_c/l_u | | | | | | | | | |
|-----------|-----------|------|------|------|------|------|------|------|-------|-------|
| | | 1.05 | 1.10 | 1.15 | 1.20 | 1.25 | 1.30 | 1.35 | 1.40 | 1.45 |
| 0.00 | k_{AB} | 4.20 | 4.40 | 4.60 | 4.80 | 5.00 | 5.20 | 5.40 | 5.60 | 5.80 |
| | C_{AB} | 0.57 | 0.65 | 0.73 | 0.80 | 0.87 | 0.95 | 1.03 | 1.10 | 1.17 |
| 0.2 | k_{AB} | 4.31 | 4.62 | 4.95 | 5.30 | 5.65 | 6.02 | 6.40 | 6.79 | 7.20 |
| | C_{AB} | 0.56 | 0.62 | 0.68 | 0.74 | 0.80 | 0.85 | 0.91 | 0.96 | 1.01 |
| 0.4 | k_{AB} | 4.38 | 4.79 | 5.22 | 5.67 | 6.15 | 6.65 | 7.18 | 7.74 | 8.32 |
| | C_{AB} | 0.55 | 0.60 | 0.65 | 0.70 | 0.74 | 0.79 | 0.83 | 0.87 | 0.91 |
| 0.6 | k_{AB} | 4.44 | 4.91 | 5.42 | 5.96 | 6.54 | 7.15 | 7.81 | 8.50 | 9.23 |
| | C_{AB} | 0.55 | 0.59 | 0.63 | 0.67 | 0.70 | 0.74 | 0.77 | 0.80 | 0.83 |
| 0.8 | k_{AB} | 4.49 | 5.01 | 5.58 | 6.19 | 6.85 | 7.56 | 8.31 | 9.12 | 9.98 |
| | C_{AB} | 0.54 | 0.58 | 0.61 | 0.64 | 0.67 | 0.70 | 0.72 | 0.75 | 0.77 |
| 1.0 | k_{AB} | 4.52 | 5.09 | 5.71 | 6.38 | 7.11 | 7.89 | 8.73 | 9.63 | 10.60 |
| | C_{AB} | 0.54 | 0.57 | 0.60 | 0.62 | 0.65 | 0.67 | 0.69 | 0.71 | 0.73 |
| 1.2 | k_{AB} | 4.55 | 5.16 | 5.82 | 6.54 | 7.32 | 8.17 | 9.08 | 10.07 | 11.12 |
| | C_{AB} | 0.53 | 0.56 | 0.59 | 0.61 | 0.63 | 0.65 | 0.66 | 0.68 | 0.69 |
| 1.4 | k_{AB} | 4.58 | 5.21 | 5.91 | 6.68 | 7.51 | 8.41 | 9.38 | 10.43 | 11.57 |
| | C_{AB} | 0.53 | 0.55 | 0.58 | 0.60 | 0.61 | 0.63 | 0.64 | 0.65 | 0.66 |
| 1.6 | k_{AB} | 4.60 | 5.26 | 5.99 | 6.79 | 7.66 | 8.61 | 9.64 | 10.75 | 11.95 |
| | C_{AB} | 0.53 | 0.55 | 0.57 | 0.59 | 0.60 | 0.61 | 0.62 | 0.63 | 0.64 |
| 1.8 | k_{AB} | 4.62 | 5.30 | 6.06 | 6.89 | 7.80 | 8.79 | 9.87 | 11.03 | 12.29 |
| | C_{AB} | 0.52 | 0.55 | 0.56 | 0.58 | 0.59 | 0.60 | 0.61 | 0.61 | 0.62 |



Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Equivalent Column K_{EC} – Determination of k

| t_a/t_b | | l_c/l_u | | | | | | | | |
|-----------|----------|-----------|------|------|------|------|-------|-------|-------|-------|
| | | 1.05 | 1.10 | 1.15 | 1.20 | 1.25 | 1.30 | 1.35 | 1.40 | 1.45 |
| 2.0 | k_{AB} | 4.63 | 5.34 | 6.12 | 6.98 | 7.92 | 8.94 | 10.06 | 11.27 | 12.59 |
| | C_{AB} | 0.52 | 0.54 | 0.56 | 0.57 | 0.58 | 0.59 | 0.59 | 0.60 | 0.60 |
| 2.2 | k_{AB} | 4.65 | 5.37 | 6.17 | 7.05 | 8.02 | 9.08 | 10.24 | 11.49 | 12.85 |
| | C_{AB} | 0.52 | 0.54 | 0.55 | 0.56 | 0.57 | 0.58 | 0.58 | 0.59 | 0.59 |
| 2.4 | k_{AB} | 4.66 | 5.40 | 6.22 | 7.12 | 8.11 | 9.20 | 10.39 | 11.68 | 13.08 |
| | C_{AB} | 0.52 | 0.53 | 0.55 | 0.56 | 0.56 | 0.57 | 0.57 | 0.58 | 0.58 |
| 2.6 | k_{AB} | 4.67 | 5.42 | 6.26 | 7.18 | 8.20 | 9.31 | 10.53 | 11.86 | 13.29 |
| | C_{AB} | 0.52 | 0.53 | 0.54 | 0.55 | 0.56 | 0.56 | 0.56 | 0.57 | 0.57 |
| 2.8 | k_{AB} | 4.68 | 5.44 | 6.29 | 7.23 | 8.27 | 9.41 | 10.66 | 12.01 | 13.48 |
| | C_{AB} | 0.52 | 0.53 | 0.54 | 0.55 | 0.55 | 0.55 | 0.56 | 0.56 | 0.56 |
| 3.0 | k_{AB} | 4.69 | 5.46 | 6.33 | 7.28 | 8.34 | 9.50 | 10.77 | 12.15 | 13.65 |
| | C_{AB} | 0.52 | 0.53 | 0.54 | 0.54 | 0.55 | 0.55 | 0.55 | 0.55 | 0.55 |
| 3.5 | k_{AB} | 4.71 | 5.50 | 6.40 | 7.39 | 8.48 | 9.69 | 11.01 | 12.46 | 14.02 |
| | C_{AB} | 0.51 | 0.52 | 0.53 | 0.53 | 0.54 | 0.54 | 0.54 | 0.53 | 0.53 |
| 4.0 | k_{AB} | 4.72 | 5.54 | 6.45 | 7.47 | 8.60 | 9.84 | 11.21 | 12.70 | 14.32 |
| | C_{AB} | 0.51 | 0.52 | 0.52 | 0.53 | 0.53 | 0.52 | 0.52 | 0.52 | 0.52 |
| 4.5 | k_{AB} | 4.73 | 5.56 | 6.50 | 7.54 | 8.69 | 9.97 | 11.37 | 12.89 | 14.57 |
| | C_{AB} | 0.51 | 0.52 | 0.52 | 0.52 | 0.52 | 0.52 | 0.51 | 0.51 | 0.51 |
| 5.0 | k_{AB} | 4.75 | 5.59 | 6.54 | 7.60 | 8.78 | 10.07 | 11.50 | 13.07 | 14.77 |
| | C_{AB} | 0.51 | 0.51 | 0.52 | 0.52 | 0.51 | 0.51 | 0.51 | 0.50 | 0.49 |
| 6.0 | k_{AB} | 4.76 | 5.63 | 6.60 | 7.69 | 8.90 | 10.24 | 11.72 | 13.33 | 15.10 |
| | C_{AB} | 0.51 | 0.51 | 0.51 | 0.51 | 0.50 | 0.50 | 0.49 | 0.49 | 0.48 |
| 7.0 | k_{AB} | 4.78 | 5.66 | 6.65 | 7.76 | 9.00 | 10.37 | 11.88 | 13.54 | 15.34 |
| | C_{AB} | 0.51 | 0.51 | 0.51 | 0.50 | 0.50 | 0.49 | 0.48 | 0.48 | 0.47 |
| 8.0 | k_{AB} | 4.78 | 5.68 | 6.69 | 7.82 | 9.07 | 10.47 | 12.01 | 13.70 | 15.54 |
| | C_{AB} | 0.51 | 0.51 | 0.50 | 0.50 | 0.49 | 0.49 | 0.48 | 0.47 | 0.46 |
| 9.0 | k_{AB} | 4.80 | 5.71 | 6.74 | 7.89 | 9.18 | 10.61 | 12.19 | 13.93 | 15.83 |
| | C_{AB} | 0.50 | 0.50 | 0.50 | 0.49 | 0.48 | 0.48 | 0.47 | 0.46 | 0.45 |

Source: Reinforced Concrete Mechanics and Design 6th Ed. Page #1100

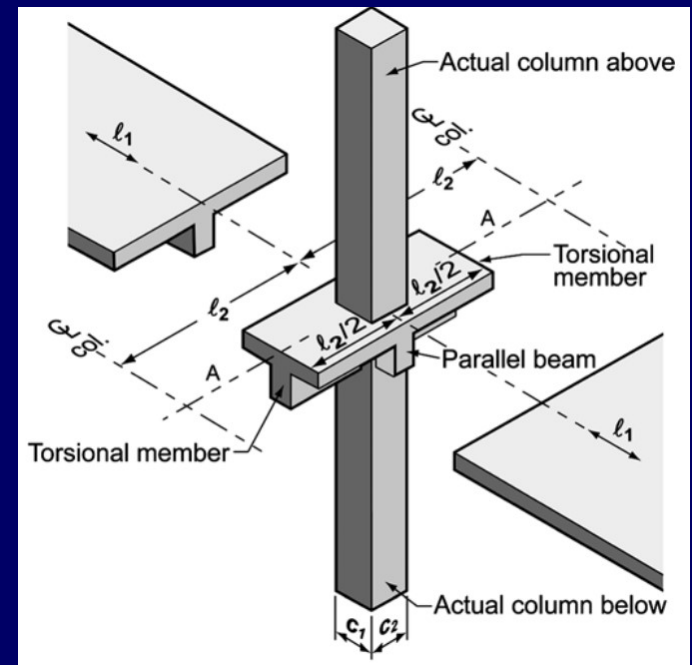


Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Torsional Member K_T

- Torsional members (transverse members) provide moment transfer between the slab-beams and the columns.
- Assumed to have constant cross-section throughout their length.





Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Torsional Member K_T

- The torsional stiffness K_t of the torsional member is given as:

$$K_t = \sum \left[\frac{9E_{cs}C}{l_2 \left(1 - \frac{c_2}{l_2}\right)^3} \right]$$

- If beams frame into the support in the direction of analysis, the torsional stiffness K_t needs to be increased.

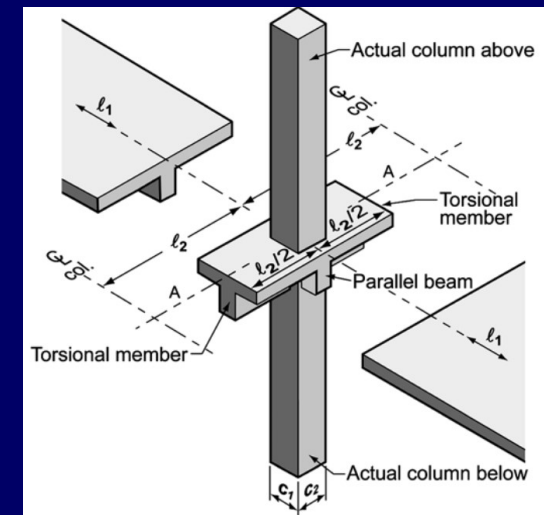
$$K_{ta} = \frac{K_t I_{sb}}{I_s}$$

where;

E_{cs} = modulus of elasticity of slab concrete;

I_{sb} = I of slab with beam;

I_s = I of slab without beam = $l_2 h^3 / 12$

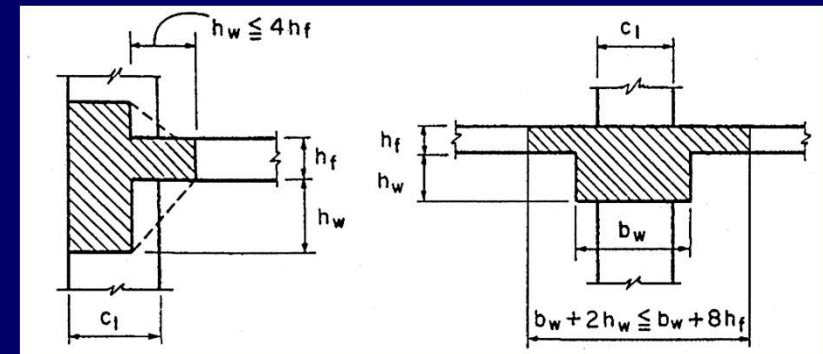
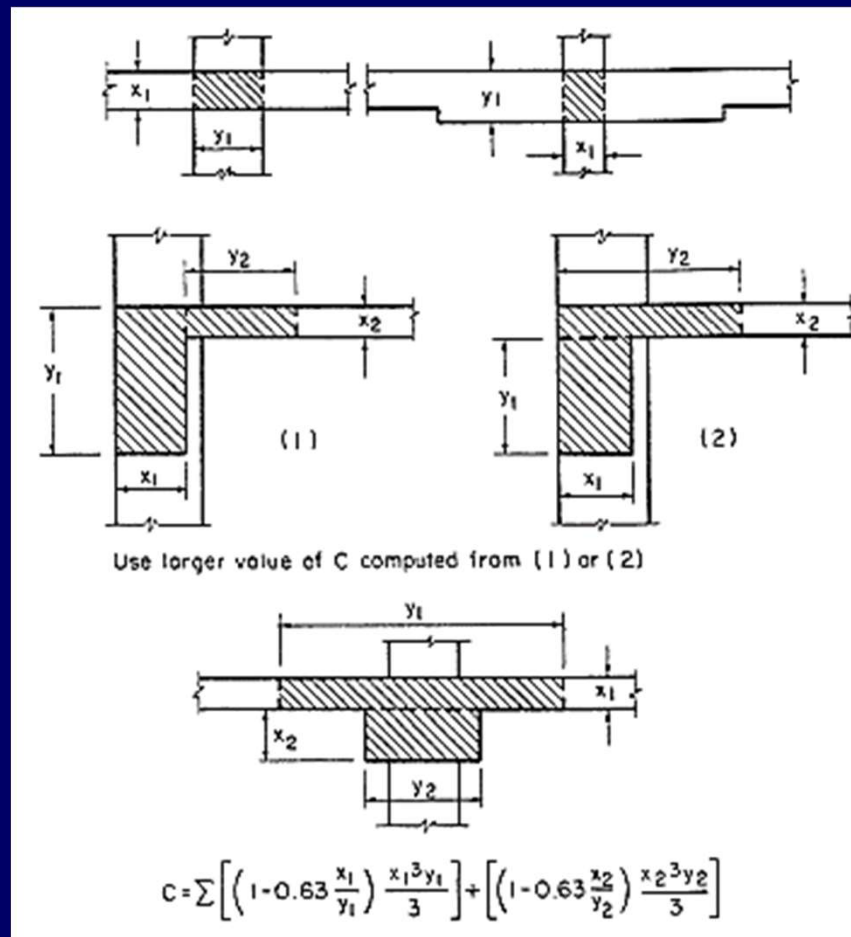




Equivalent Frame Method

□ Step 2: Determination of Stiffnesses

❖ Stiffness of Torsional Member K_T – Determination of C





Equivalent Frame Method

□ Step 3: Analysis of Frame using MDM

- The original derivation of EFM assumed that moment distribution would be the procedure used to analyze the slabs.
- In lieu of computer software, moment distribution is a convenient hand calculation method for analyzing partial frames in the Equivalent Frame Method.



Equivalent Frame Method

□ Step 3: Analysis of Frame using MDM

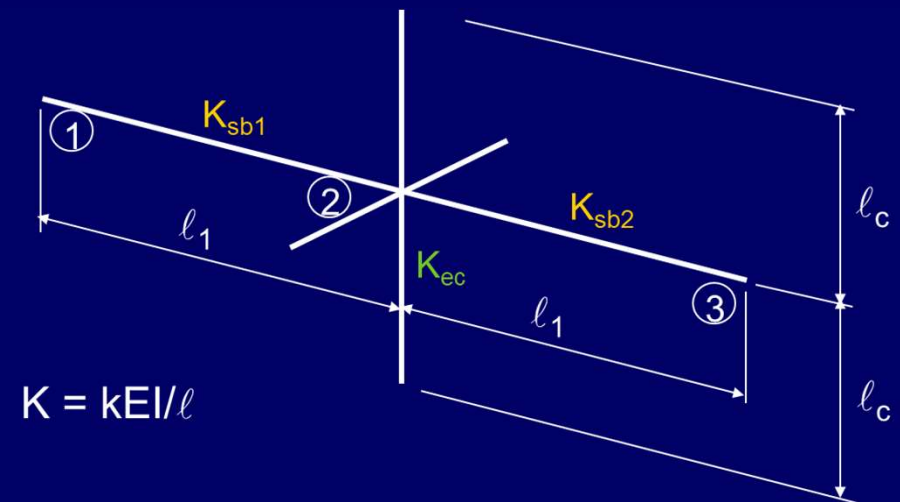
❖ Distribution Factors for Slab Beam

$$DF_{2 \rightarrow 1} = \frac{K_{sb1}}{K_{sb1} + K_{sb2} + K_{ec}}$$

$$DF_{2 \rightarrow 3} = \frac{K_{sb2}}{K_{sb1} + K_{sb2} + K_{ec}}$$

❖ Distribution Factors for Equivalent Column

$$DF = \frac{K_{ec}}{K_{sb1} + K_{sb2} + K_{ec}}$$





Equivalent Frame Method

□ Step 3: Analysis of Frame using MDM

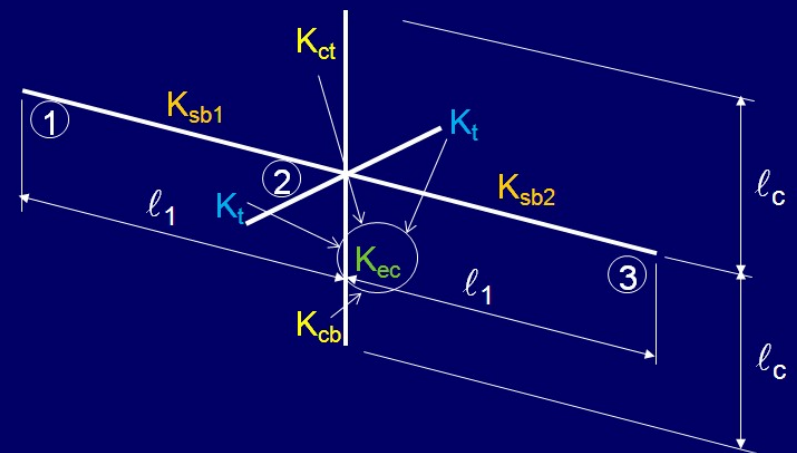
❖ Distribution of Unbalanced Moment to Columns

- Portion of Unbalanced Moment from Beam to Upper Column

$$DF_{uc} = \frac{K_{ct}}{K_{cb} + K_{ct}}$$

- Portion of Unbalanced Moment from Beam to Lower Column

$$DF_{lc} = \frac{K_{cb}}{K_{cb} + K_{ct}}$$





Equivalent Frame Method

□ Step 3: Analysis of Frame using MDM

❖ Arrangement of Live loads

- ACI 8.11.1 states that when the loading pattern is known, the equivalent frame shall be analyzed for that load.
- When $LL \leq 0.75DL$
 - Maximum factored moment when Full factored LL on all spans
- Other cases
 - Pattern live loading using 0.75 (Factored LL) to determine maximum factored moment.

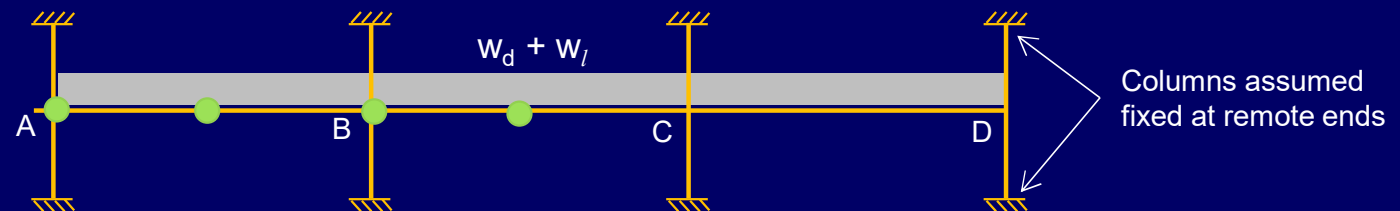


Equivalent Frame Method

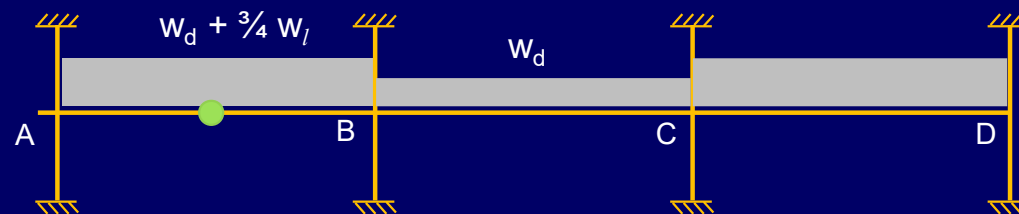
Step 3: Analysis of Frame using MDM

Arrangement of Live loads for Positive Moments

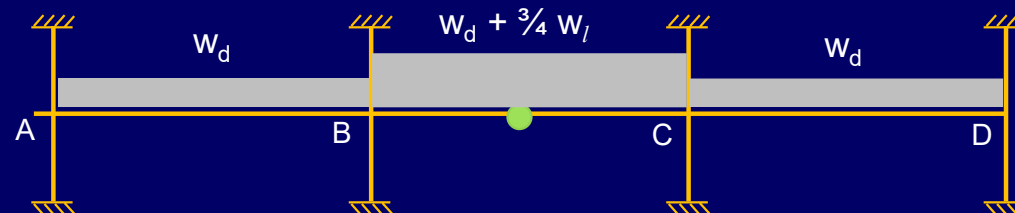
(1) For design moments in all spans with $L \leq 3/4 D$



(2) For positive design moment in span AB'



(3) For positive design moment in span BC'

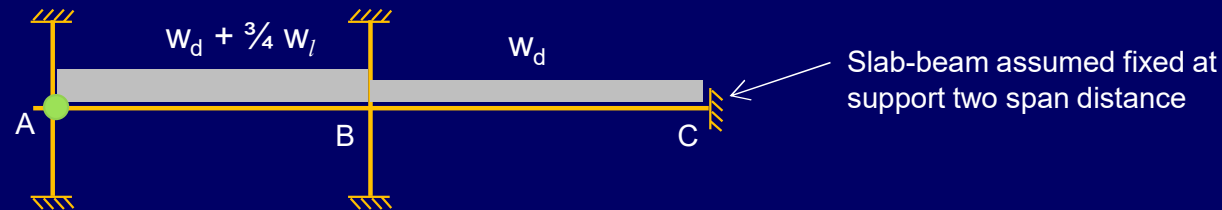




Equivalent Frame Method

□ Step 3: Analysis of Frame using MDM

❖ Arrangement of Live loads for Negative Moments



(4) Loading pattern for negative design moment at support A'



(5) Loading pattern for negative design moment at support B'



Equivalent Frame Method

□ Summary of Analysis Steps in EFM

- Extract the 3D frame from the 3D structure.
- Extract a story from 3D frame for gravity load analysis.
- Identify EF members i.e., slab beam, torsional member and columns.
- Find stiffness (kEI/I) of each EF member using tables.
- Assign stiffnesses of each EF member to its corresponding 2D frame member.



Equivalent Frame Method

□ Summary of Analysis Steps in EFM

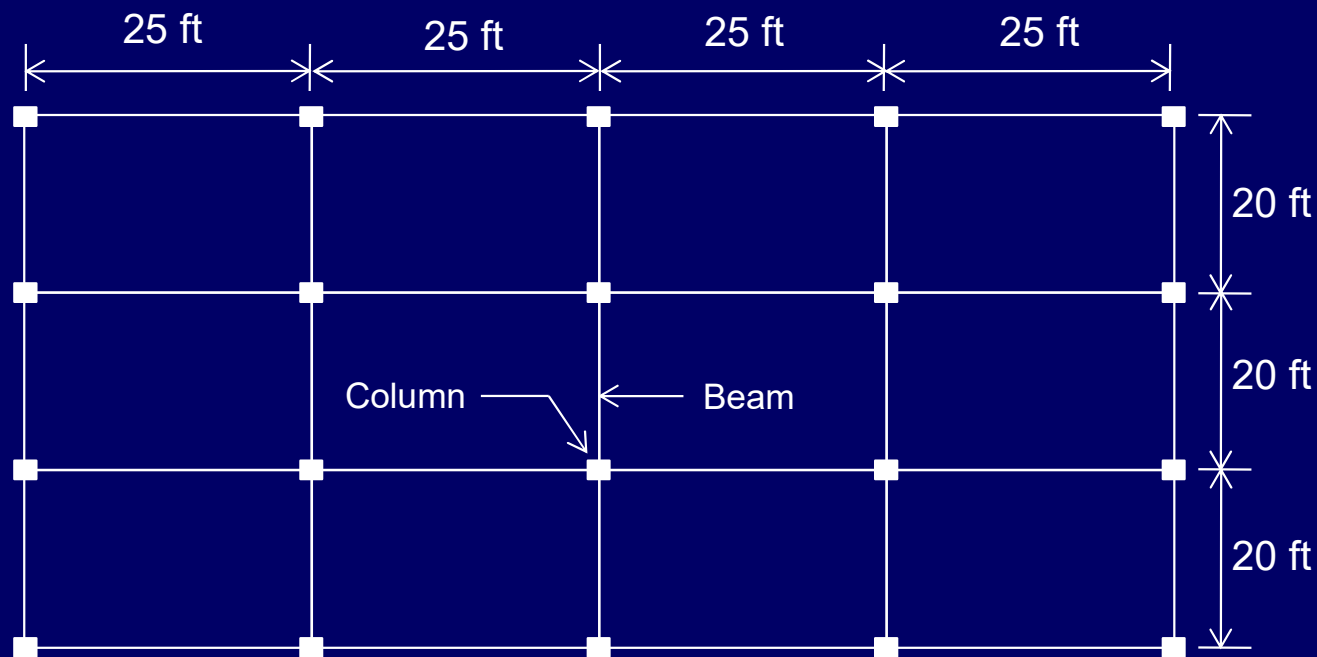
- Analyze the obtained 2D frame using Moment Distribution method of analysis to get longitudinal moments based on center-to-center span.
- Distribute slab-beam longitudinal moment laterally using lateral distribution procedures of DDM.
- Slab analysis can be done using DDM.



Example 8.1

□ Problem Statement

Analyze the three-story building whose typical floor is shown below, using Equivalent Frame Method.



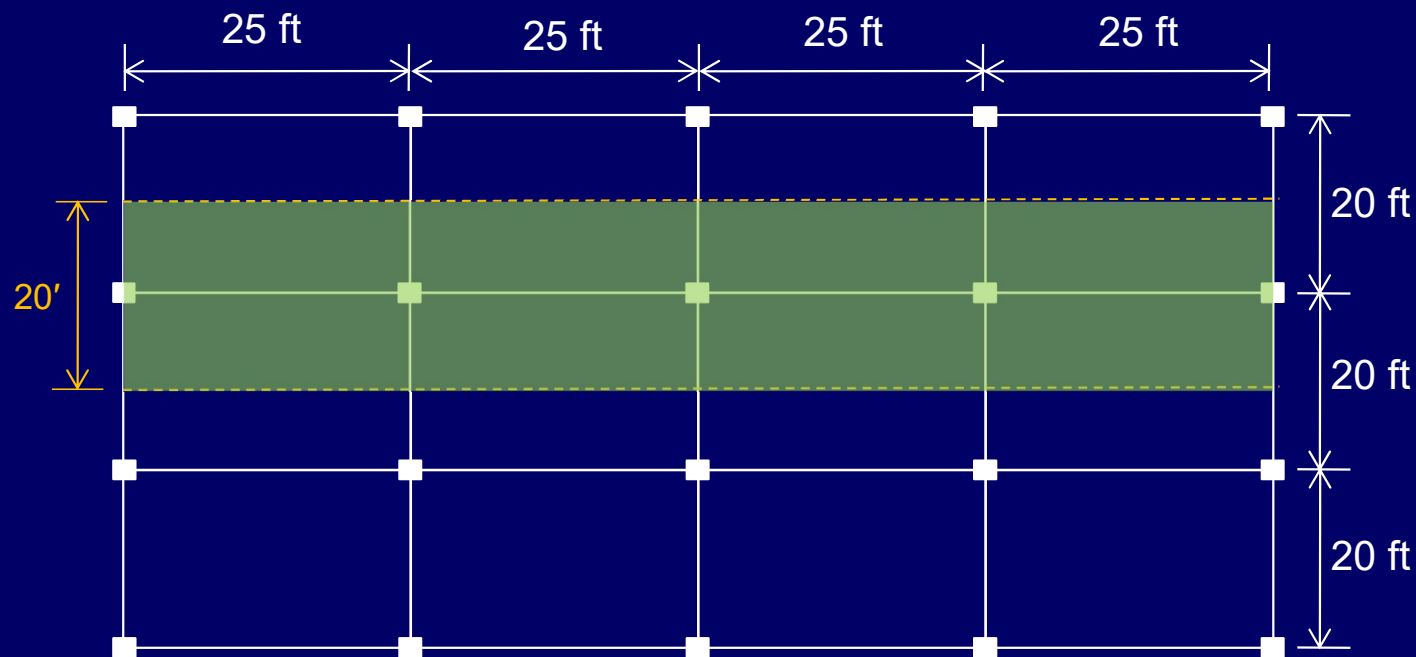
Slab thickness: 7", Columns: 14" × 14" and Beams: 14" × 20"



Example 8.1

□ Solution

➤ Step 1: Extraction of Frame



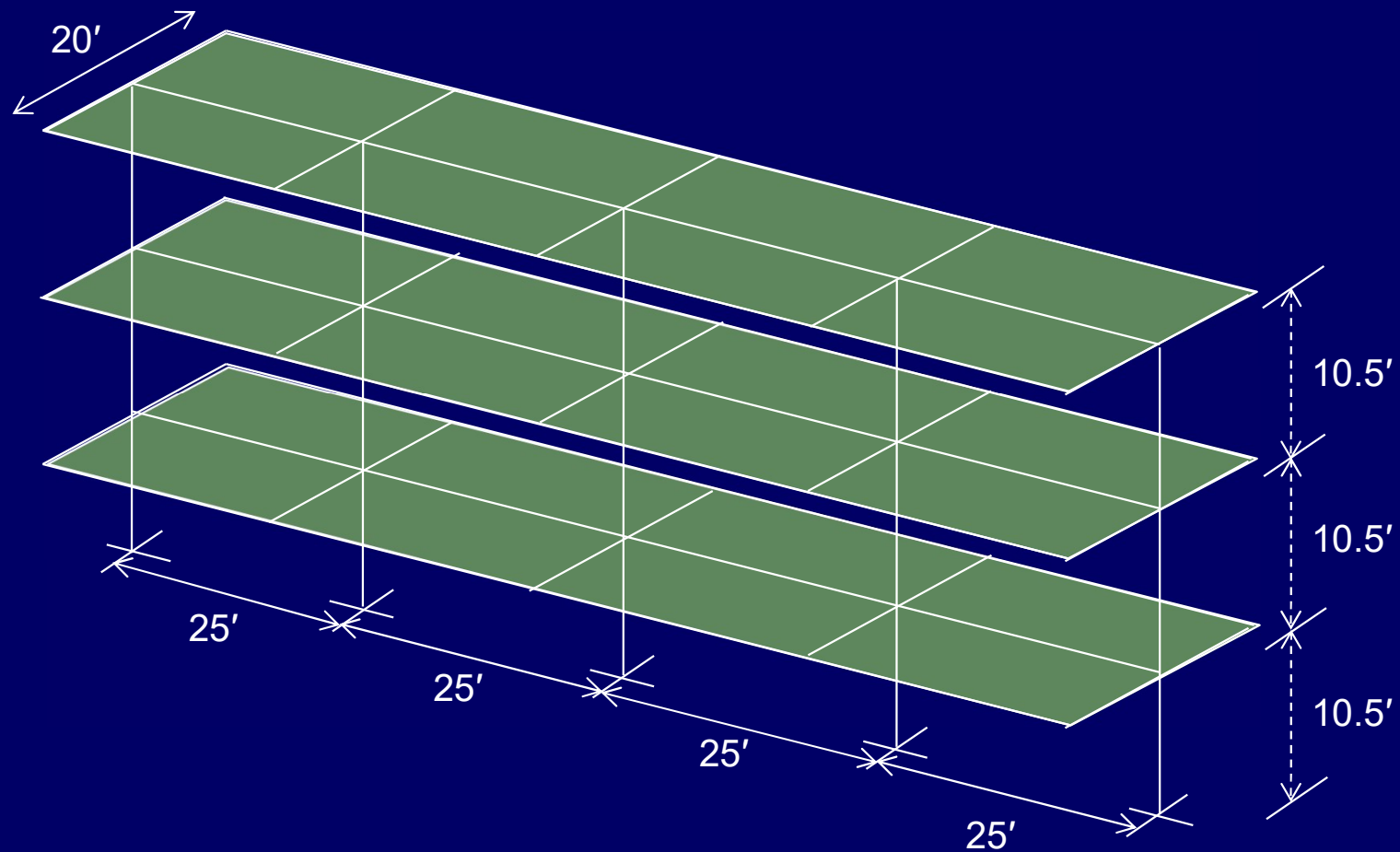
Slab thickness: 7", Columns: 14" × 14" and Beams: 14" × 20"



Example 8.1

□ Solution

➤ Step 1: Extraction of Frame



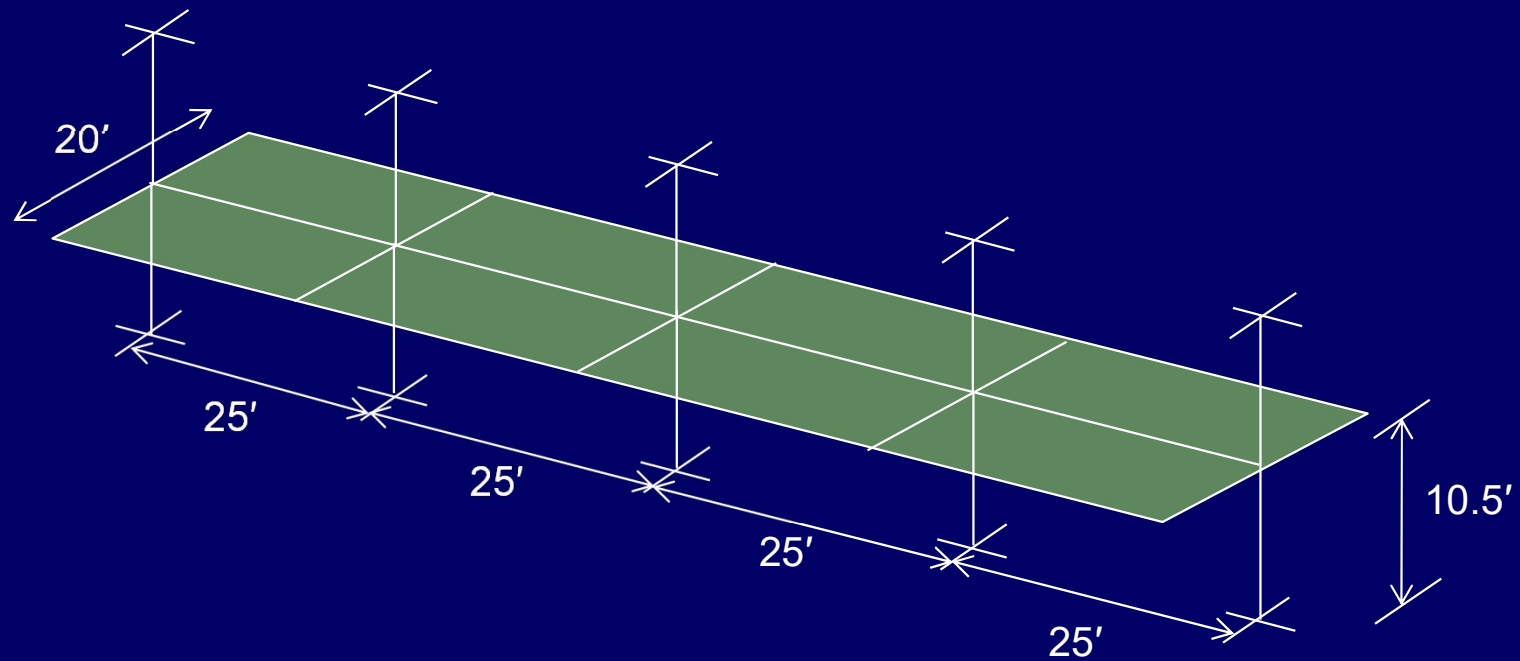


Example 8.1

□ Solution

➤ Step 1: Extraction of Frame

- According to ACI 8.11.2.5, it shall be permitted to assume that the far ends of columns built integrally with the structure are considered to be fixed for gravity load analysis.



Frame Extracted from at Intermediate Story



Example 8.1

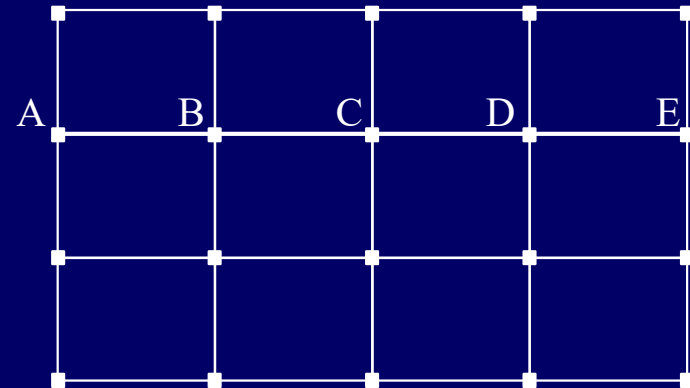
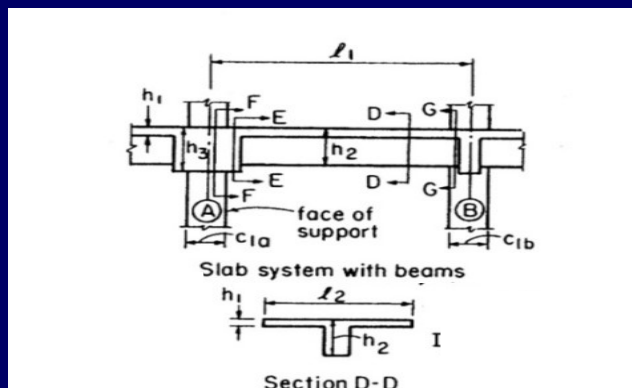
□ Solution

➤ Step 2: Determination of Stiffnesses

i. Slab-beam Stiffness Calculation

| Slab beam stiffness (K_{sb}) | | | | | | | |
|----------------------------------|-----------------|-----------------|-----------|-----------|-------|----------|-------------------------|
| Span | l_1 and c_1 | l_2 and c_2 | c_1/l_1 | c_2/l_2 | k | I_{sb} | $K_{sb} = kEI_{sb}/l_1$ |
| AB | 25' & 14" | 20' and 14" | 0.0467 | 0.058 | 4.051 | 25844 | 349E |

The remaining spans will have the same values as the geometry is same.
Table A-20 (Reinforced concrete: Mechanics and Design, 3rd Ed)





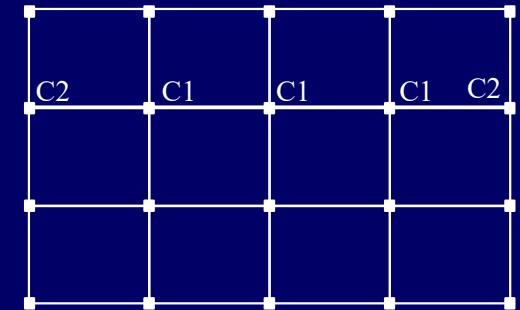
Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{\sum K_t}$$



| Calculation of K_t | | | | |
|----------------------|-------|-------|---------------------------------------------------|-------------------------------------------------|
| Column location | l_2 | c_2 | $C = \sum (1 - 0.63x/y)x^3y/3$ (in ⁴) | $K_t = \sum 9E_{cs} C / \{l_2(1 - c_2/l_2)^3\}$ |
| C2 | 20' | 14" | 11208 | 3792.63E _{cs} |
| C1 | 20' | 14" | 12190 | 4295.98E _{cs} |

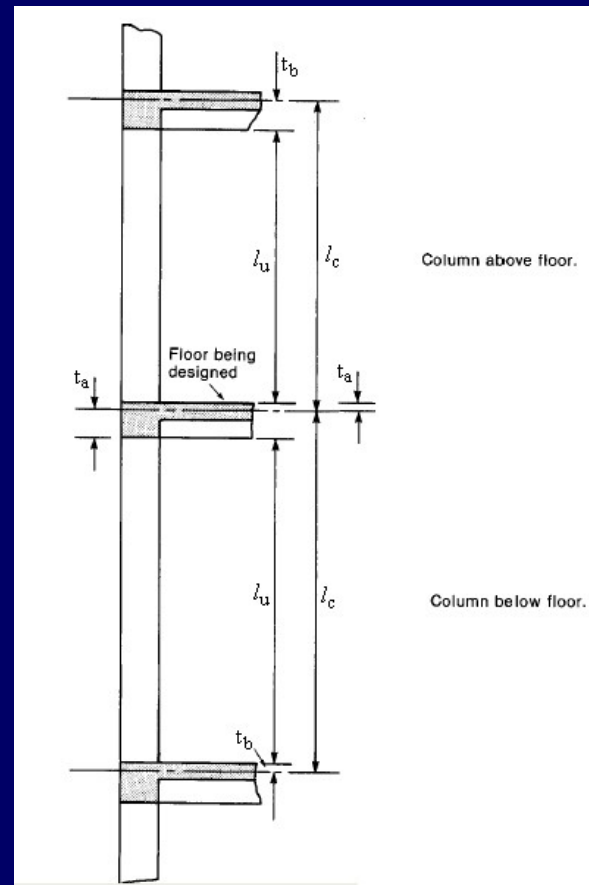


Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation





Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation

$K_c = k \frac{EL_c}{\ell_c}$

| | | ℓ_c/ℓ_u | | | | | | | | |
|-----------|----------|-----------------|------|------|------|------|-------|-------|-------|-------|
| t_a/t_b | | 1.05 | 1.10 | 1.15 | 1.20 | 1.25 | 1.30 | 1.35 | 1.40 | 1.45 |
| 2.4 | k_{AB} | 4.66 | 5.40 | 6.22 | 7.12 | 8.11 | 9.20 | 10.39 | 11.68 | 13.08 |
| | C_{AB} | 0.52 | 0.53 | 0.55 | 0.56 | 0.56 | 0.57 | 0.57 | 0.58 | 0.58 |
| 2.6 | k_{AB} | 4.67 | 5.42 | 6.26 | 7.18 | 8.20 | 9.31 | 10.53 | 11.86 | 13.29 |
| | C_{AB} | 0.52 | 0.53 | 0.54 | 0.55 | 0.56 | 0.56 | 0.56 | 0.57 | 0.57 |
| 2.8 | k_{AB} | 4.68 | 5.44 | 6.29 | 7.23 | 8.27 | 9.41 | 10.66 | 12.01 | 13.48 |
| | C_{AB} | 0.52 | 0.53 | 0.54 | 0.55 | 0.55 | 0.55 | 0.56 | 0.56 | 0.56 |
| 3.0 | k_{AB} | 4.69 | 5.46 | 6.33 | 7.28 | 8.34 | 9.50 | 10.77 | 12.15 | 13.65 |
| | C_{AB} | 0.52 | 0.53 | 0.54 | 0.54 | 0.55 | 0.55 | 0.55 | 0.55 | 0.55 |
| 3.5 | k_{AB} | 4.71 | 5.50 | 6.40 | 7.39 | 8.48 | 9.69 | 11.01 | 12.46 | 14.02 |
| | C_{AB} | 0.51 | 0.52 | 0.53 | 0.53 | 0.54 | 0.54 | 0.54 | 0.53 | 0.53 |
| 4.0 | k_{AB} | 4.72 | 5.54 | 6.45 | 7.47 | 8.60 | 9.84 | 11.21 | 12.70 | 14.32 |
| | C_{AB} | 0.51 | 0.52 | 0.52 | 0.53 | 0.53 | 0.52 | 0.52 | 0.52 | 0.52 |
| 4.5 | k_{AB} | 4.73 | 5.56 | 6.50 | 7.54 | 8.69 | 9.97 | 11.37 | 12.89 | 14.57 |
| | C_{AB} | 0.51 | 0.52 | 0.52 | 0.52 | 0.52 | 0.52 | 0.51 | 0.51 | 0.51 |
| 5.0 | k_{AB} | 4.75 | 5.59 | 6.54 | 7.60 | 8.78 | 10.07 | 11.50 | 13.07 | 14.77 |
| | C_{AB} | 0.51 | 0.51 | 0.52 | 0.52 | 0.51 | 0.51 | 0.51 | 0.50 | 0.49 |
| 6.0 | k_{AB} | 4.76 | 5.63 | 6.60 | 7.69 | 8.90 | 10.24 | 11.72 | 13.33 | 15.10 |
| | C_{AB} | 0.51 | 0.51 | 0.51 | 0.51 | 0.50 | 0.50 | 0.49 | 0.49 | 0.48 |
| 7.0 | k_{AB} | 4.78 | 5.66 | 6.65 | 7.76 | 9.00 | 10.37 | 11.88 | 13.54 | 15.34 |
| | C_{AB} | 0.51 | 0.51 | 0.51 | 0.50 | 0.50 | 0.49 | 0.48 | 0.48 | 0.47 |
| 8.0 | k_{AB} | 4.78 | 5.68 | 6.69 | 7.82 | 9.07 | 10.47 | 12.01 | 13.70 | 15.54 |
| | C_{AB} | 0.51 | 0.51 | 0.50 | 0.50 | 0.49 | 0.49 | 0.48 | 0.47 | 0.46 |
| 9.0 | k_{AB} | 4.80 | 5.71 | 6.74 | 7.89 | 9.18 | 10.61 | 12.19 | 13.93 | 15.83 |
| | C_{AB} | 0.50 | 0.50 | 0.50 | 0.49 | 0.48 | 0.48 | 0.47 | 0.46 | 0.45 |



Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{\sum K_t}$$

| Calculation of $\sum K_c$ for Column C2 | | | | | | | |
|------------------------------------------------|-----------------|-------|-----------|----------------------------------------------|-------------------|----------|---------------------|
| Column location | l_c | l_u | l_c/l_u | I_c (in ⁴) 14" × 14" column | t_a/t_b | k_{AB} | $K_c = k(EI_c/l_c)$ |
| C2 (bottom) | 10.5' (126") | 106" | 1.20 | $14 \times 14^3/12 = 3201$ | $16.5/3.5 = 4.71$ | 7.57 | $192E_{cc}$ |
| C2 (top) | 10.5' (126") | 106" | 1.20 | $14 \times 14^3/12 = 3201$ | $3.5/16.5 = 0.21$ | 5.3 | $135E_{cc}$ |
| $\sum K_c = 192E_{cc} + 135E_{cc} = 327E_{cc}$ | | | | | | | |



Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{\sum K_t}$$

| Calculation of $\sum K_c$ for Column C1 | | | | | | | |
|------------------------------------------------|--------------|-------|----------------|----------------------------------------------|-----------------|----------|---------------------|
| Column location | l_c | l_u | l_c/l_u | I_c (in ⁴) 14" × 14" column | t_a/t_b | k_{AB} | $K_c = k(EI_c/l_c)$ |
| C1 (bottom) | 10.5' (126") | 106" | 126/106 ≈ 1.20 | 14 × 14 ³ /12 = 3201 | 16.5/3.5 = 4.71 | 7.57 | 192E _{cc} |
| C1 (top) | 10.5' (126") | 106" | 126/106 ≈ 1.20 | 14 × 14 ³ /12 = 3201 | 3.5/16.5 = 0.21 | 5.3 | 135E _{cc} |
| $\sum K_c = 192E_{cc} + 135E_{cc} = 327E_{cc}$ | | | | | | | |



Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation

For column C2 (exterior column), we have

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{\sum K_t} = \frac{1}{327E_{cc}} + \frac{1}{3792.63E_{cs}}$$

Because the slab and the columns have the same strength concrete, $E_{cc} = E_{cs} = E_c$. Therefore,

$$K_{ec} = 301E_c$$

Similarly, for column C1 (interior column), we get

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{\sum K_t} = \frac{1}{327E_{cc}} + \frac{1}{4295.98E_{cs}}; \quad K_{ec} = 303E_c$$

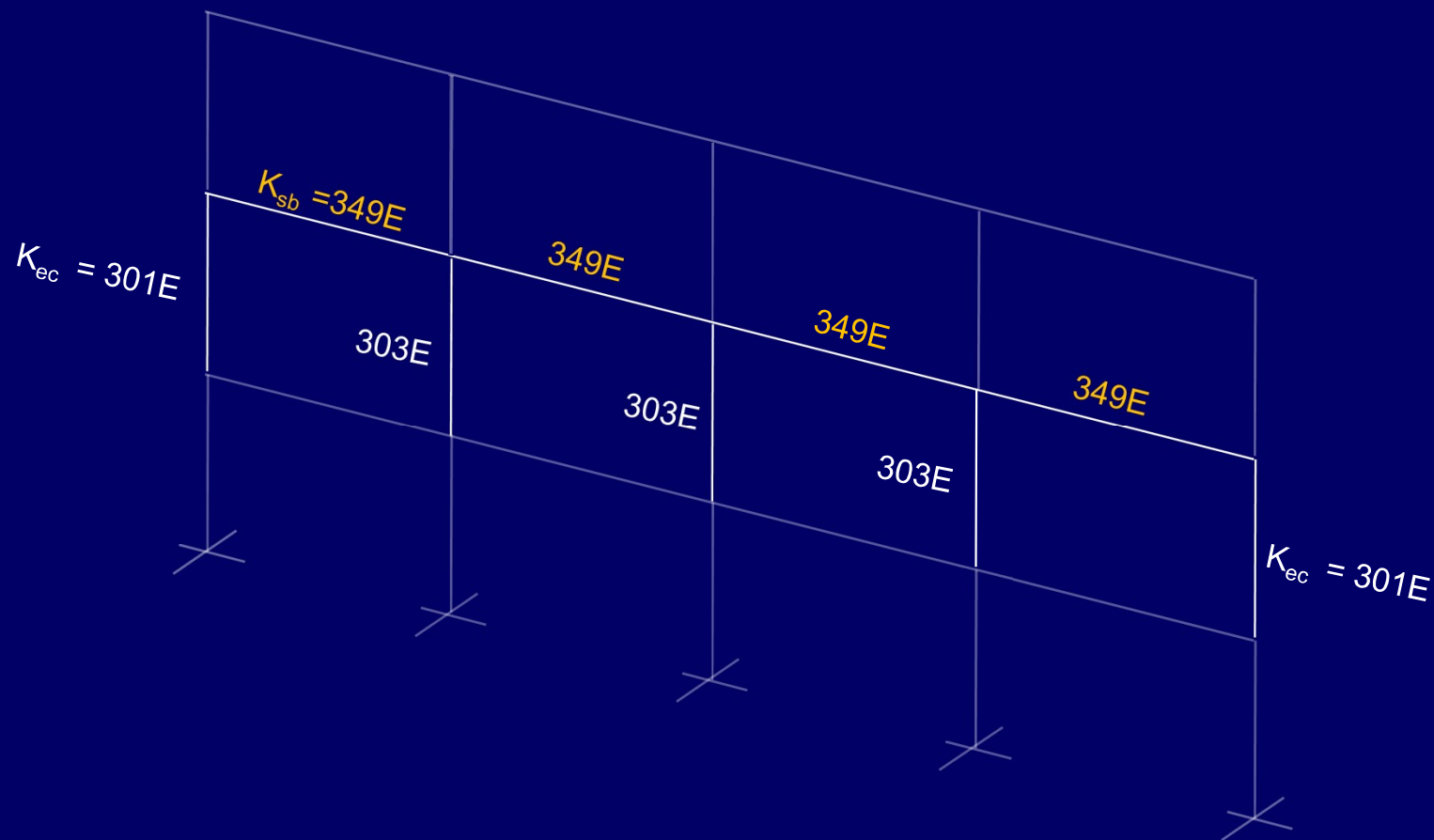


Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation





Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation

- As the ground story is same as 1st one, therefore the stiffness calculated shall also be assigned to ground story.
- For the top story, the slab beam stiffness will be same as lower stories. However, the equivalent stiffness of the top story column is computed next.



Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation

Calculation of $\sum K_c$ for Column C2 (Top Story)

| Column location | l_c | l_u | l_c / l_u | I_c (in ⁴) 14" × 14" column | t_a / t_b | k_{AB} | $K_c = k(EI_c / l_c)$ |
|-----------------|--------------|-------|------------------------|----------------------------------------------|-------------------|----------|-----------------------|
| C2 (bottom) | 10.5' (126") | 100" | $126/106 \approx 1.20$ | $14 \times 14^3 / 12 = 3201$ | $16.5/3.5 = 4.71$ | 7.57 | $192E_{cc}$ |

$$\sum K_c = 192E_{cc}$$

Similarly for interior column, $\sum K_c = 192E_{cc}$



Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

ii. Equivalent Column Stiffness Calculation

For column C2, we have

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{\sum K_t} = \frac{1}{192E_{cc}} + \frac{1}{3792.63E_{cs}}$$

Because the slab and the columns have the same strength concrete, $E_{cc} = E_{cs} = E_c$. Therefore,

$$K_{ec} = 182E_c$$

Similarly, for column C1, we get

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{\sum K_t} = \frac{1}{92E_{cc}} + \frac{1}{4295.98E_{cs}}; \quad K_{ec} = 183E_c$$

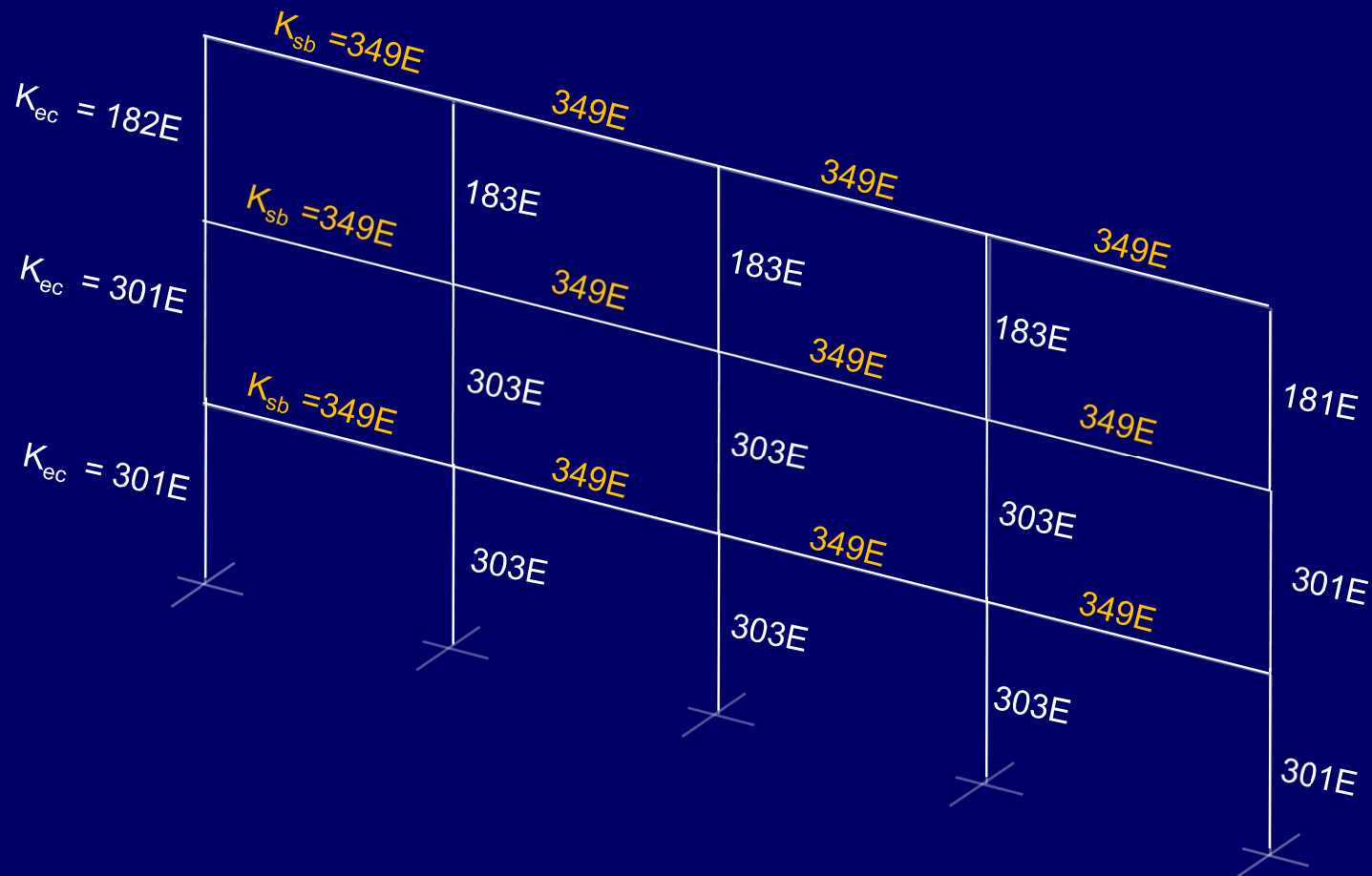


Example 8.1

□ Solution

➤ Step 2: Determination of Stiffnesses

❖ Equivalent Frame

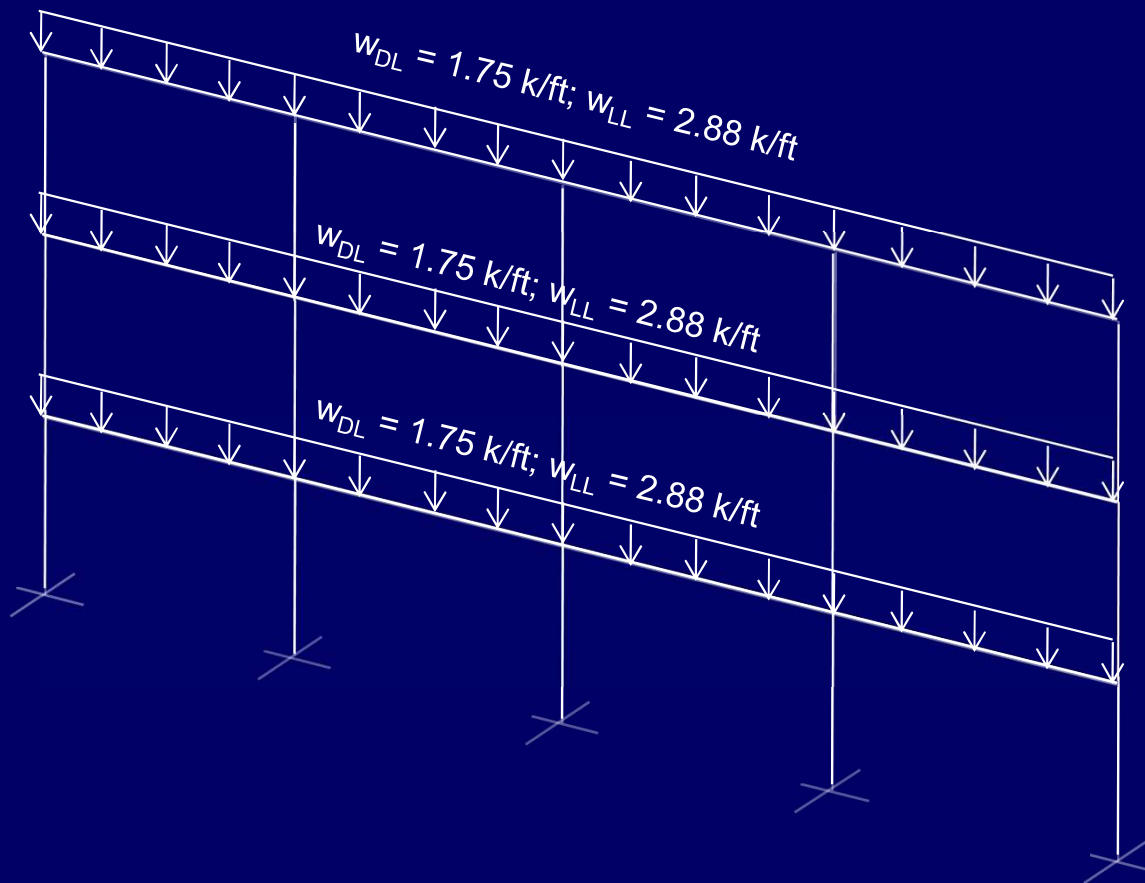




Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM



Load on frame for Bending Analysis:

As the horizontal frame element represents slab beam, load is computed by multiplying slab load with width of frame

$$w_{DL} = 0.0875 \times 20 = 1.75 \text{ kip/ft}$$

$$w_{LL} = 0.144 \times 20 = 2.88 \text{ kip/ft}$$



Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Dead Loads (top storey).

| Joint | A | | | | B | | | C | | | D | | | E | | |
|------------|-------|---------|---------|----------|--------|---------|---------|--------|--------|----------|--------|---------|---------|--------|--------|--|
| CarryOver | | | 0.5034 | | | | 0.5034 | | | | 0.5034 | | | | 0.5034 | |
| DF | 0.000 | 0.344 | 0.656 | 0.396 | 0.209 | 0.396 | 0.396 | 0.209 | 0.396 | 0.396 | 0.209 | 0.396 | 0.656 | 0.344 | 0.000 | |
| | Slab | Column | Slab | Slab | Column | Slab | Slab | Column | Slab | Slab | Column | Slab | Slab | Column | Slab | |
| FEM | 0.000 | 0.000 | 91.802 | -91.802 | 0.000 | 91.802 | -91.802 | 0.000 | 91.802 | -91.802 | 0.000 | 91.802 | -91.802 | 0.000 | 0.000 | |
| Bal | 0.000 | -31.611 | -60.191 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 60.191 | 31.611 | 0.000 | |
| Carry over | | | 0.000 | -30.301 | | 0.000 | 0.000 | | 0.000 | 0.000 | | 0.000 | 30.301 | 0.000 | | |
| Bal | 0.000 | 0.000 | 0.000 | 11.985 | 6.330 | 11.985 | 0.000 | 0.000 | 0.000 | -11.985 | -6.330 | -11.985 | 0.000 | 0.000 | 0.000 | |
| Carry over | | | 6.034 | 0.000 | | 0.000 | 6.034 | | -6.034 | 0.000 | | 0.000 | -6.034 | | | |
| Bal | 0.000 | -2.078 | -3.956 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 3.956 | 2.078 | 0.000 | |
| Carry over | | | 0.000 | -1.992 | | 0.000 | 0.000 | | 0.000 | 0.000 | | 0.000 | 1.992 | 0.000 | | |
| Bal | 0.000 | 0.000 | 0.000 | 0.788 | 0.416 | 0.788 | 0.000 | 0.000 | 0.000 | -0.788 | -0.416 | -0.788 | 0.000 | 0.000 | 0.000 | |
| Carry over | | | 0.397 | 0.000 | | 0.000 | 0.397 | | -0.397 | 0.000 | | 0.000 | -0.397 | | | |
| Bal | 0.000 | -0.137 | -0.260 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.260 | 0.137 | 0.000 | |
| Carry over | | | 0.000 | -0.131 | | 0.000 | 0.000 | | 0.000 | 0.000 | | 0.000 | 0.131 | 0.000 | | |
| Bal | 0.000 | 0.000 | 0.000 | 0.052 | 0.027 | 0.052 | 0.000 | 0.000 | 0.000 | -0.052 | -0.027 | -0.052 | 0.000 | 0.000 | 0.000 | |
| Carry over | | | 0.026 | 0.000 | | 0.000 | 0.026 | | -0.026 | 0.000 | | 0.000 | -0.026 | | | |
| Bal | 0.000 | -0.009 | -0.017 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.017 | 0.009 | 0.000 | |
| Carry over | | | 0.000 | -0.009 | | 0.000 | 0.000 | | 0.000 | 0.000 | | 0.000 | 0.009 | 0.000 | | |
| Bal | 0.000 | 0.000 | 0.000 | 0.003 | 0.002 | 0.003 | 0.000 | 0.000 | 0.000 | -0.003 | -0.002 | -0.003 | 0.000 | 0.000 | 0.000 | |
| Carry over | | | 0.002 | 0.000 | | 0.000 | 0.002 | | -0.002 | 0.000 | | 0.000 | -0.002 | | | |
| Bal | 0.000 | -0.001 | -0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.001 | 0.000 | |
| Carry over | | | 0.000 | -0.001 | | 0.000 | 0.000 | | 0.000 | 0.000 | | 0.000 | 0.001 | 0.000 | | |
| Bal | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | |
| Carry over | | | 0.000 | 0.000 | | 0.000 | 0.000 | | 0.000 | 0.000 | | 0.000 | 0.000 | | | |
| Bal | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | |
| Total | 0.000 | -33.835 | 33.835 | -111.406 | 6.776 | 104.631 | -85.344 | 0.000 | 85.344 | -104.631 | -6.776 | 111.406 | -33.835 | 33.835 | 0.000 | |

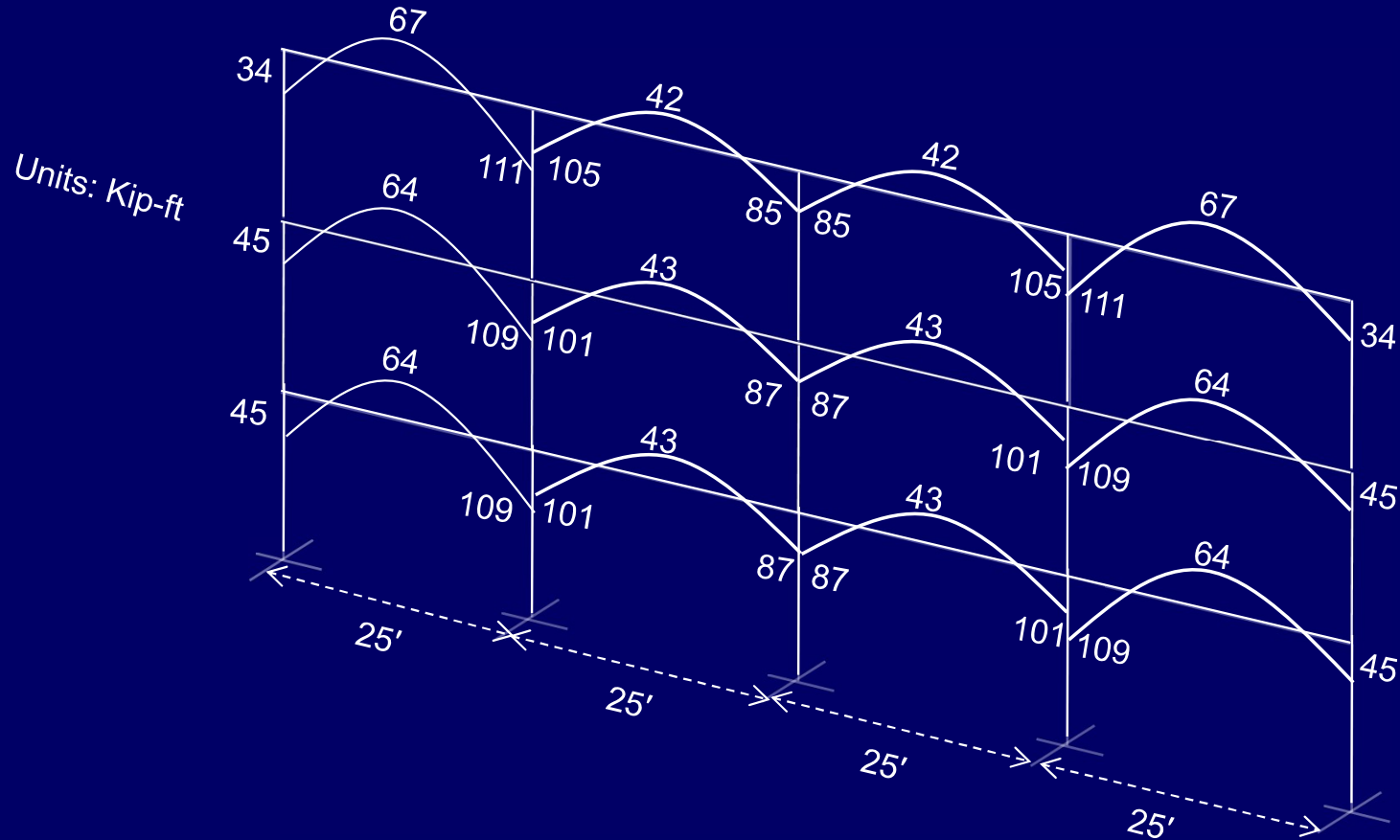


Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Dead Loads (values at centerline).





Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Dead Loads

| E-W Interior Frame Analysis (Top Story) | | | | | | |
|-----------------------------------------|-----------------------------|---------------------------|--------------------------------------------|-------------------------------------------|----------------------------------------|--------------------------|
| Length | Longitudinal moment section | Longitudinal moments (LM) | Column Strip Moment %age factor (Graph A4) | Column Strip slab Moment (CSSM) = 0.15CSM | Column strip Beam Moment (BM)= 0.85CSM | Middle Strip slab Moment |
| 25'-0" (Exterior) | Ext - | 34 | 0.93 | 5 | 27 | 2.38 |
| | + | 67 | 0.8 | 8 | 46 | 13.4 |
| | Int - | 111 | 0.8 | 13 | 75 | 22.2 |
| 25'-0" (Interior) | - | 105 | 0.8 | 13 | 71 | 21 |
| | + | 42 | 0.8 | 5 | 29 | 8.4 |
| | - | 85 | 0.8 | 10 | 58 | 17 |



Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Dead Loads

| E-W Interior Frame Analysis (Intermediate Story) | | | | | | |
|--------------------------------------------------|-----------------------------|---------------------------|--------------------------------------------|-------------------------------------------|----------------------------------------|--------------------------|
| Length | Longitudinal moment section | Longitudinal moments (LM) | Column Strip Moment %age factor (Graph A4) | Column Strip slab Moment (CSSM) = 0.15CSM | Column strip Beam Moment (BM)= 0.85CSM | Middle Strip slab Moment |
| 25'-0" (Exterior) | Ext - | 45 | 0.93 | 6 | 36 | 3.15 |
| | + | 64 | 0.8 | 8 | 44 | 12.8 |
| | Int- | 109 | 0.8 | 13 | 74 | 21.8 |
| 25'-0" (Interior) | - | 101 | 0.8 | 12 | 69 | 20.2 |
| | + | 43 | 0.8 | 5 | 29 | 8.6 |
| | - | 87 | 0.8 | 10 | 59 | 17.4 |



Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Dead Loads

- Analysis of columns for DL (factors for moment distribution)

The computed unbalanced longitudinal moments shall be transferred to columns and shall be distributed to top and bottom columns as follows:

$$DF_{uc} = \frac{K_{ct}}{K_{cb} + K_{ct}} \quad (\text{Portion of unbalanced moment to upper column})$$

$$DF_{uc} = \frac{K_{cb}}{K_{cb} + K_{ct}} \quad (\text{Portion of unbalanced moment to lower column})$$



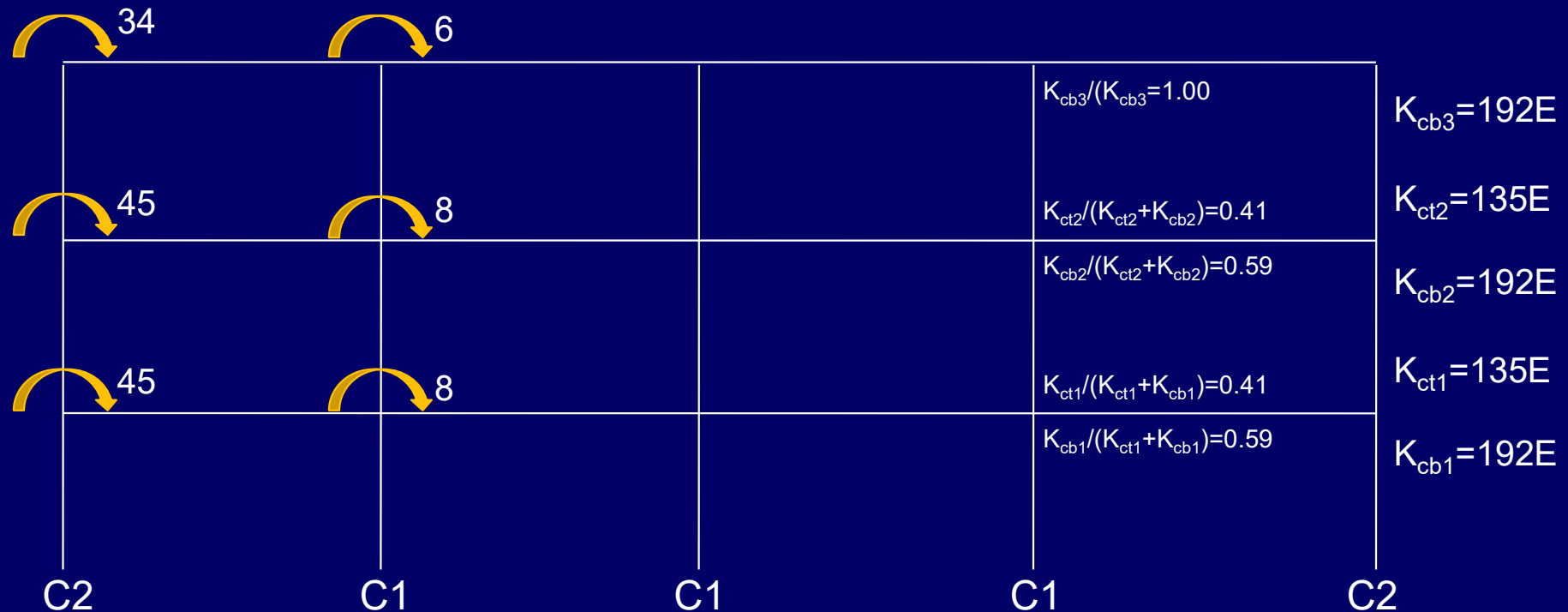
Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Dead Loads

- Analysis of columns for DL (factors for moment distribution)





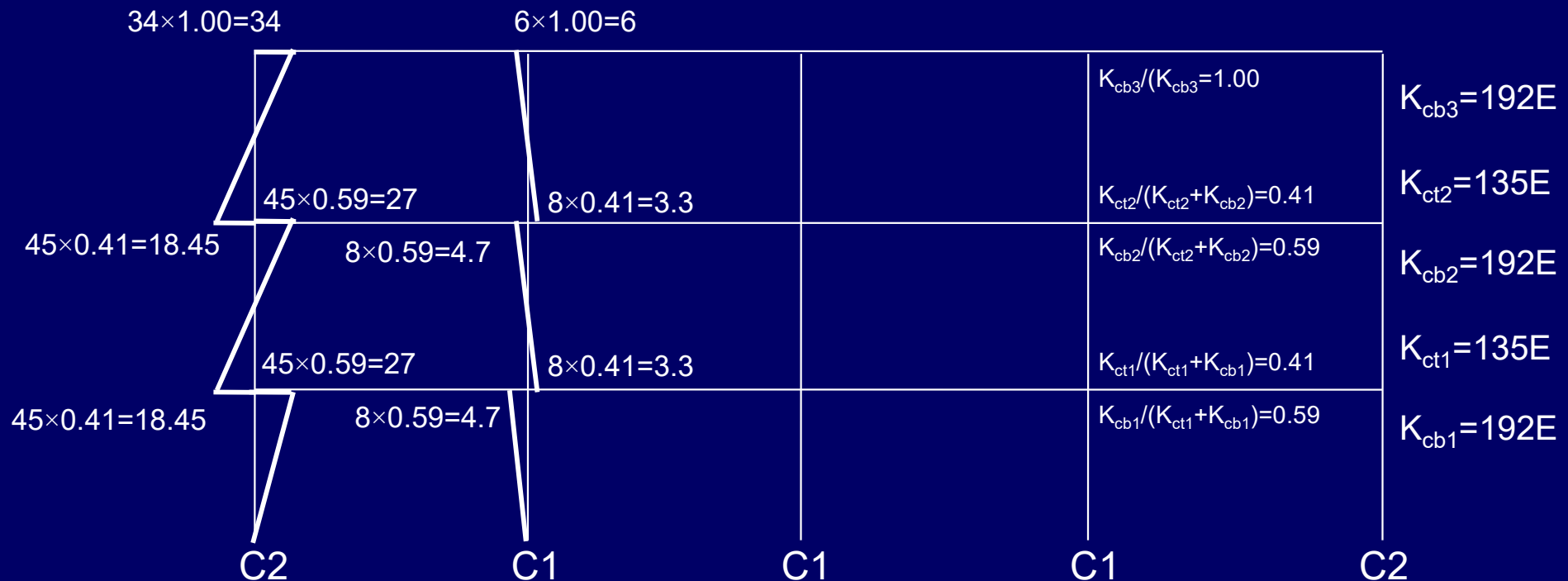
Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Dead Loads

▪ Analysis of columns for DL



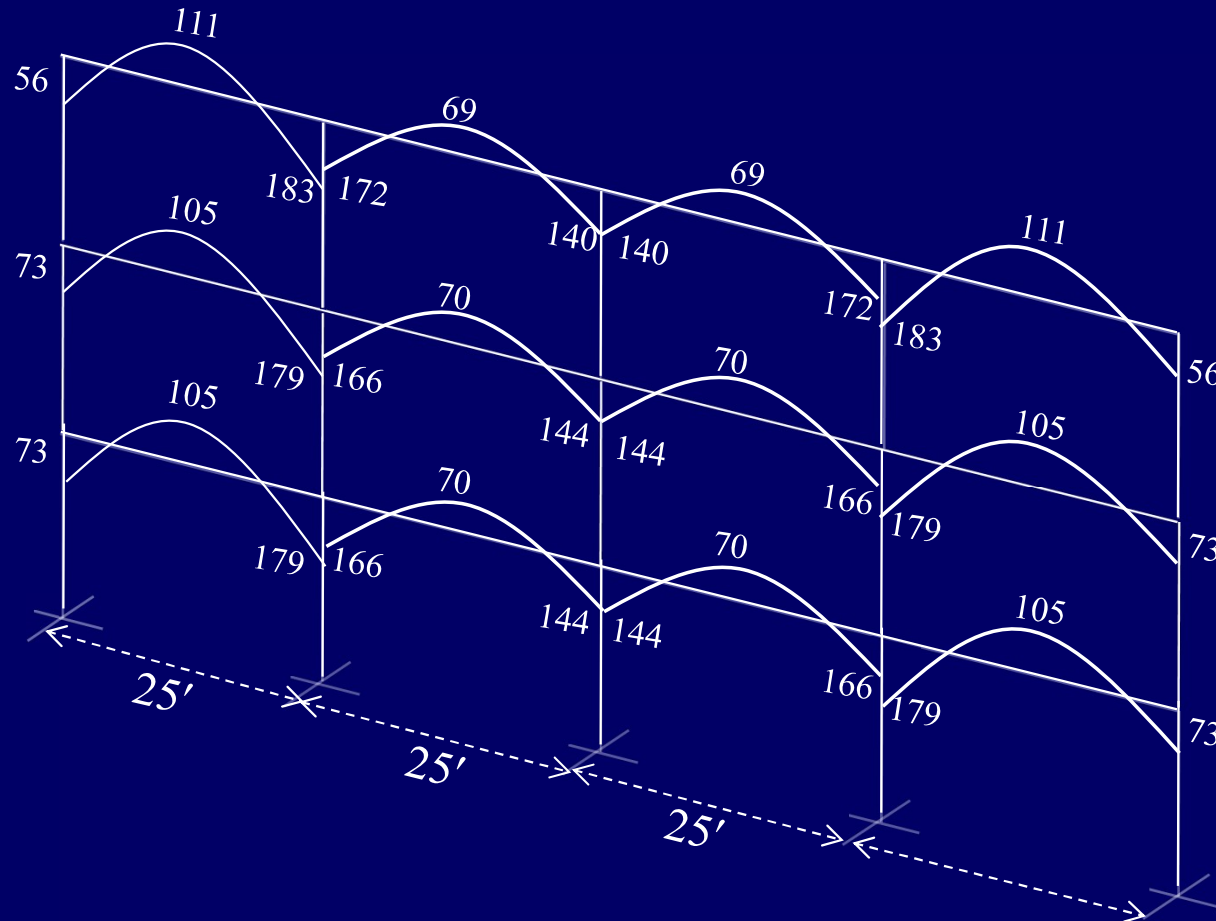


Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Live Loads





Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Live Loads

- Distribution of Moments to Slab and Beam

| E-W Interior Frame Analysis (Top Story) | | | | | | |
|-----------------------------------------|-----------------------------|---------------------------|--------------------------------------------|-------------------------------------------|----------------------------------------|--------------------------|
| Length | Longitudinal moment section | Longitudinal moments (LM) | Column Strip Moment %age factor (Graph A4) | Column Strip slab Moment (CSSM) = 0.15CSM | Column strip Beam Moment (BM)= 0.85CSM | Middle Strip slab Moment |
| 25'-0" (Exterior) | Ext - | 56 | 0.93 | 7.8 | 44 | 3.92 |
| | + | 111 | 0.8 | 13.3 | 75 | 22.2 |
| | Int- | 183 | 0.8 | 22.0 | 124 | 36.6 |
| 25'-0" (Interior) | - | 172 | 0.8 | 20.6 | 117 | 34.4 |
| | + | 69 | 0.8 | 8.3 | 47 | 13.8 |
| | - | 140 | 0.8 | 16.8 | 95 | 28 |



Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Live Loads

- Distribution of Moments to Slab and Beam

| E-W Interior Frame Analysis (Interior Story) | | | | | | |
|----------------------------------------------|-----------------------------|---------------------------|--------------------------------------------|-------------------------------------------|----------------------------------------|--------------------------|
| Length | Longitudinal moment section | Longitudinal moments (LM) | Column Strip Moment %age factor (Graph A4) | Column Strip slab Moment (CSSM) = 0.15CSM | Column strip Beam Moment (BM)= 0.85CSM | Middle Strip slab Moment |
| 25'-0" (Exterior) | Ext - | 73 | 0.93 | 10.2 | 58 | 5.11 |
| | + | 105 | 0.8 | 12.6 | 71 | 21 |
| | Int - | 179 | 0.8 | 21.5 | 122 | 35.8 |
| 25'-0" (Interior) | - | 166 | 0.8 | 19.9 | 113 | 33.2 |
| | + | 70 | 0.8 | 8.4 | 48 | 14 |
| | - | 144 | 0.8 | 17.3 | 98 | 28.8 |



Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Live Loads

- Analysis of columns for LL (factors for moment distribution)

The computed unbalanced longitudinal moments shall be transferred to columns and shall be distributed to top and bottom columns as follows:

$$DF_{uc} = \frac{K_{ct}}{K_{cb} + K_{ct}} \quad (\text{Portion of unbalanced moment to upper column})$$

$$DF_{uc} = \frac{K_{cb}}{K_{cb} + K_{ct}} \quad (\text{Portion of unbalanced moment to lower column})$$



Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Live Loads

- Analysis of columns for LL (factors for moment distribution)





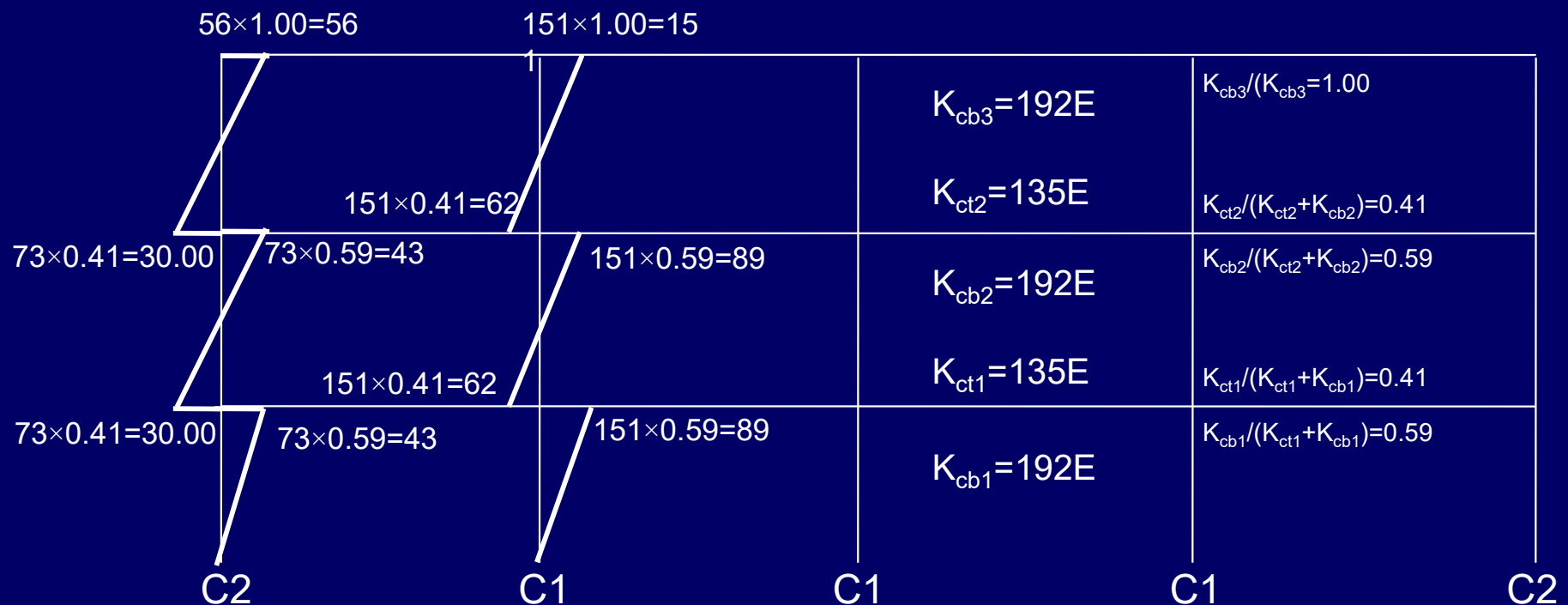
Example 8.1

□ Solution

➤ Step 3: Analysis of Frame Using MDM

❖ Analysis Results for Live Loads

- Analysis of columns for LL (factors for moment distribution)





Case Study 1

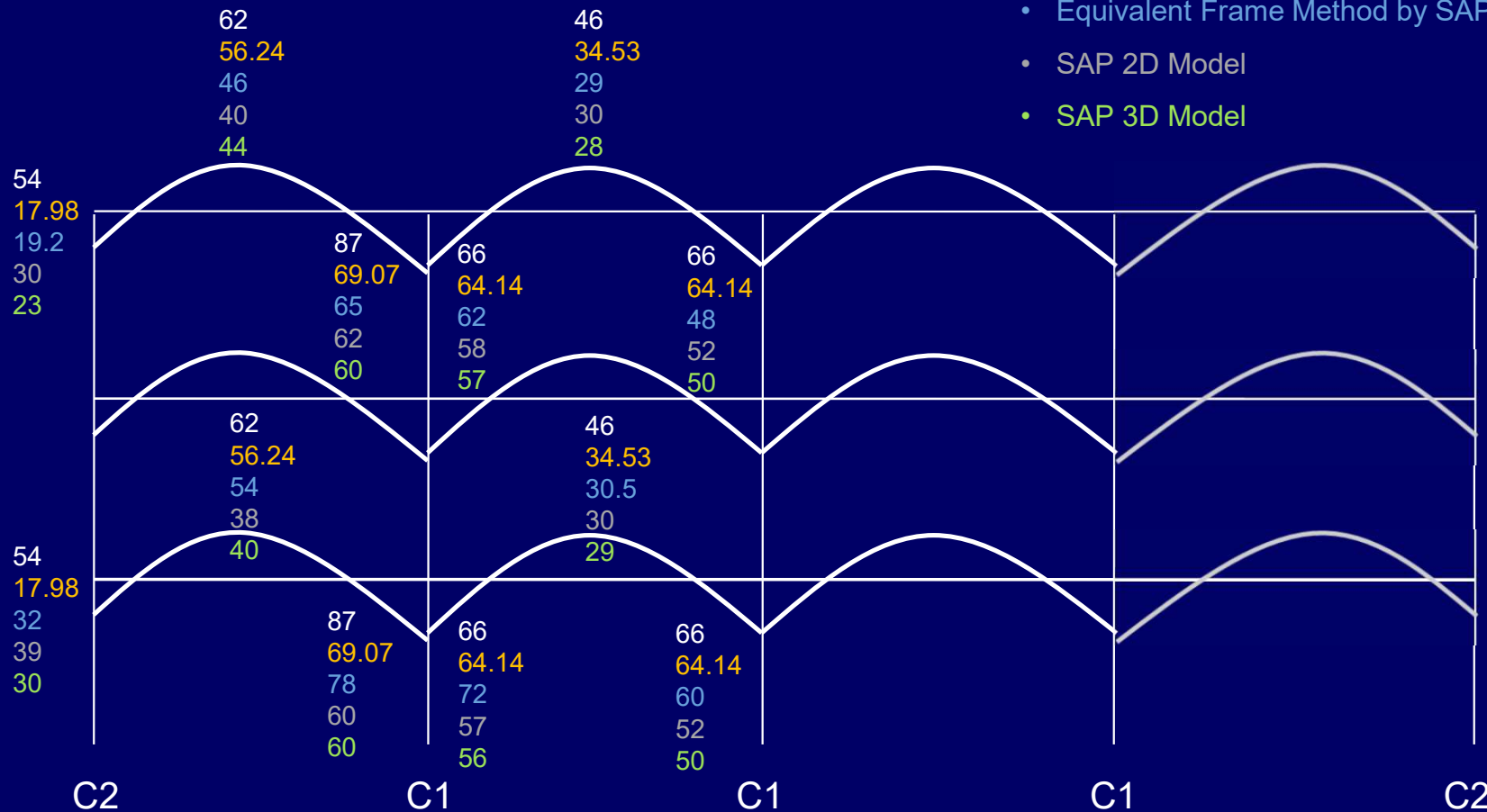
**Comparison of the Results of
EFM, ACI Coefficient Method, DDM & SAP 2D
Model with respect to SAP2000 3D Line Model**



Case Study 1

Dead Load Bending Moment in Beams

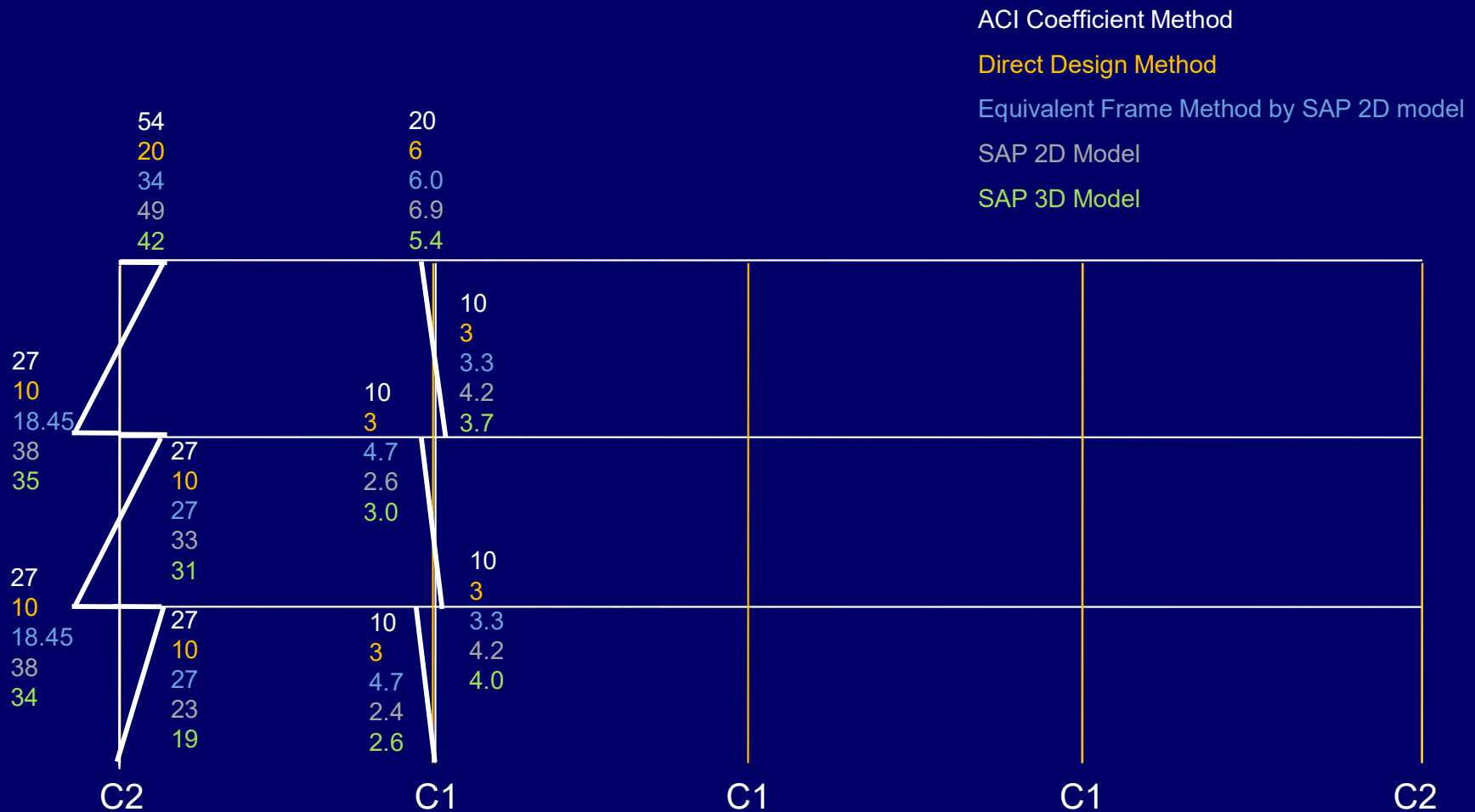
- ACI Coefficient Method
- Direct Design Method
- Equivalent Frame Method by SAP 2D model
- SAP 2D Model
- SAP 3D Model





Case Study 1

Dead Load Bending Moment in Columns

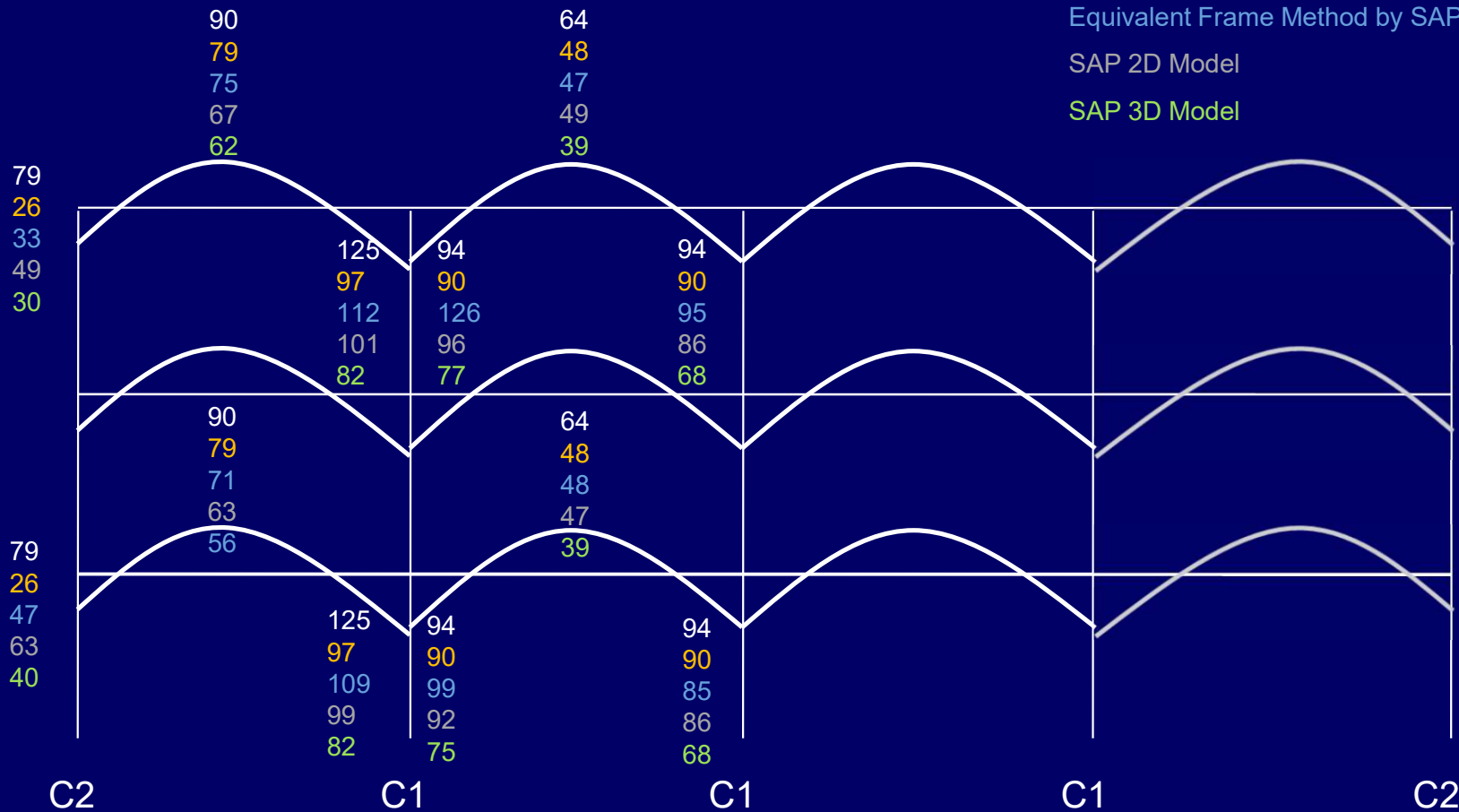




Case Study 1

Live Load Bending Moment in Beams

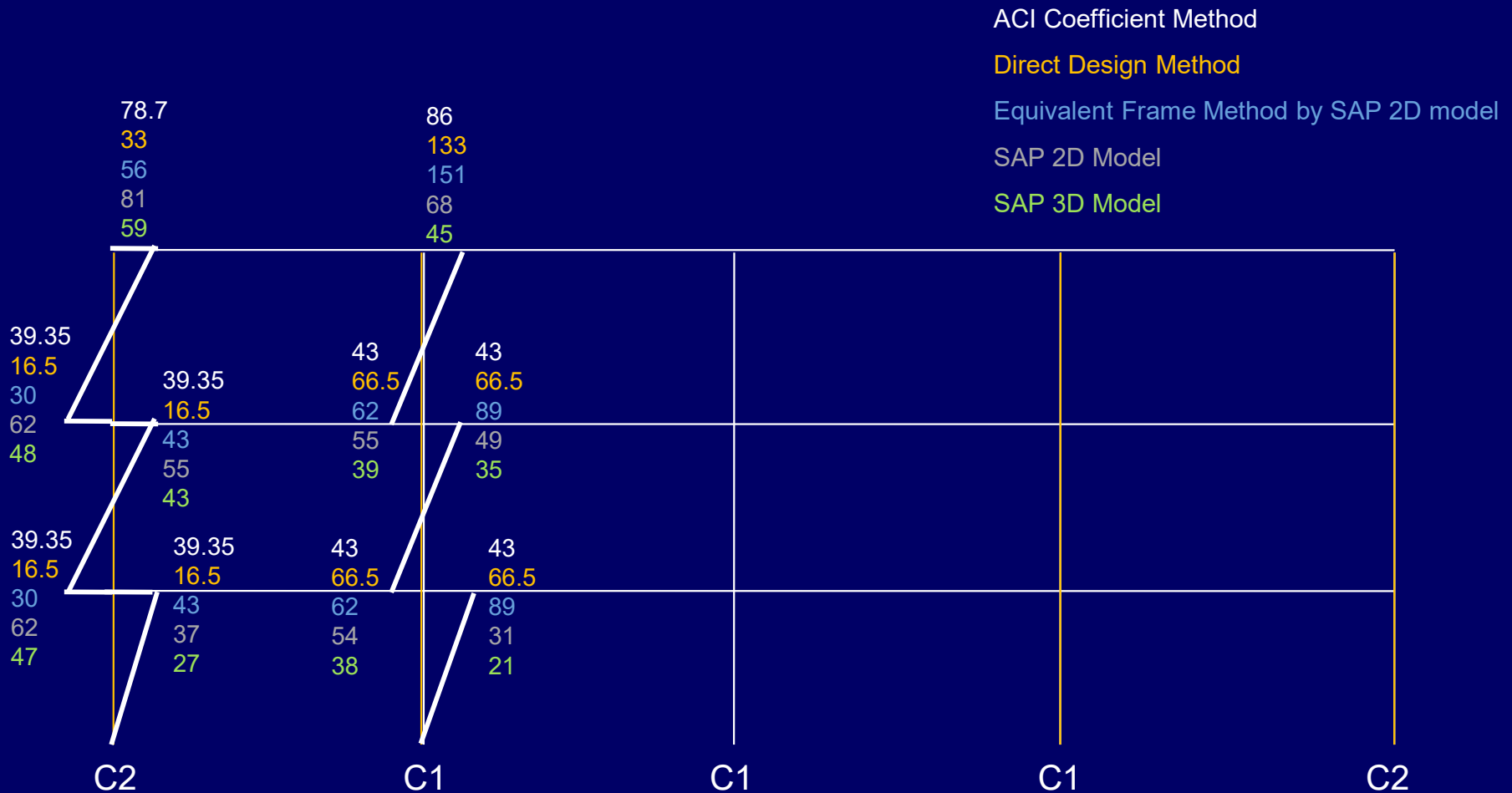
- ACI Coefficient Method
- Direct Design Method
- Equivalent Frame Method by SAP 2D model
- SAP 2D Model
- SAP 3D Model





Case Study 1

Live Load Bending Moment in Columns





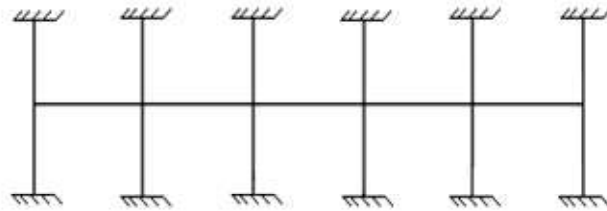
Lateral Load Analysis of RC Frames



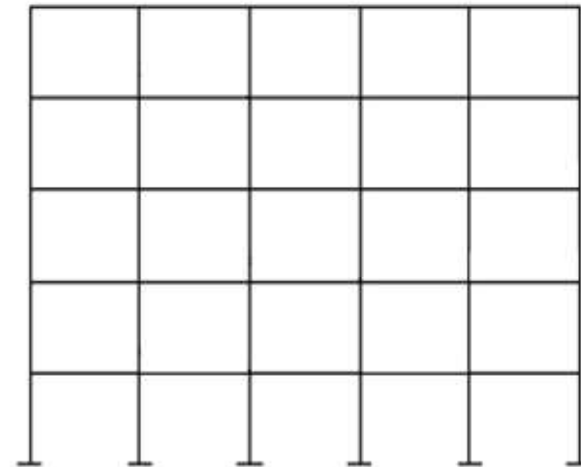
General

□ ACI Requirements for Lateral Load Analysis

- Unlike ACI section 8.9 which allows separate floor analysis for gravity loads, ACI R 8.9 states that for lateral load analysis, a full frame from top to bottom must be considered.



For Gravity Load



For Lateral Load



General

□ Methods for lateral load Analysis

1. More Accurate

- Finite Element Analysis

2. Approximate

- Portal Frame Method

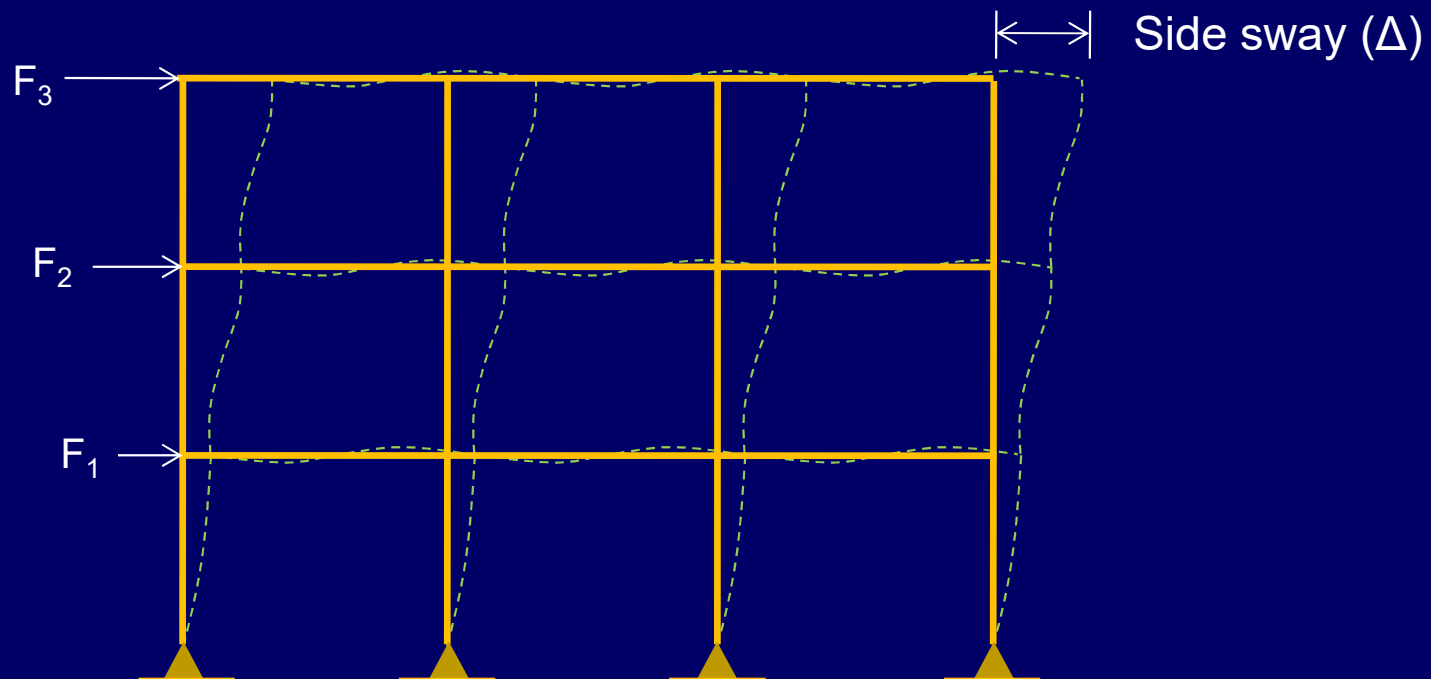
Portal Frame Method is discussed next



Portal Frame Method

□ Introduction

- This is a method used to estimate the effects of side sway due to lateral forces acting on multistory building frame.
- This method is specialized form of point of inflection method.





Portal Frame Method

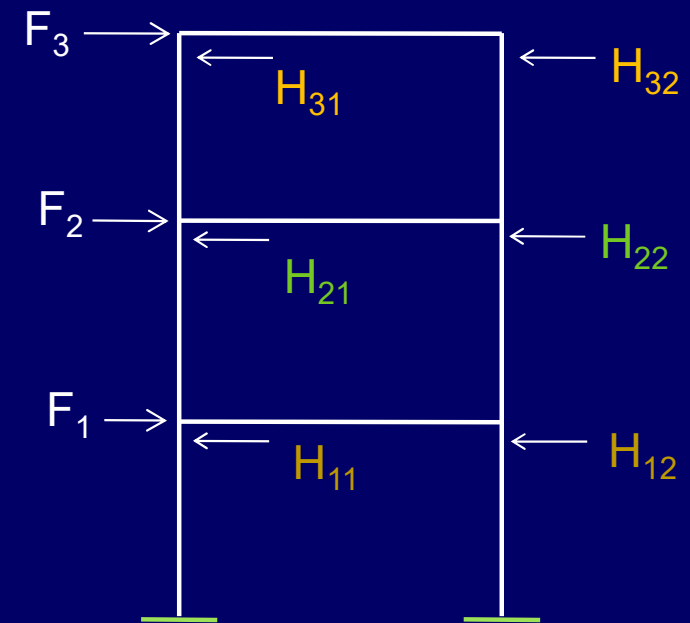
□ Propositions in Portal Frame Method

1. The total horizontal shear in all columns of a given story is equal and opposite to the sum of all horizontal loads acting above that story.
 - This proposition follows from the requirement that horizontal forces be in equilibrium at any level.

$$H_{31} + H_{32} = F_3$$

$$H_{21} + H_{22} = F_3 + F_2$$

$$H_{11} + H_{12} = F_3 + F_2 + F_1$$





Portal Frame Method

□ Propositions in Portal Frame Method

2. The horizontal shear is the same in both exterior columns. The horizontal shear in each interior column is twice that of exterior column.

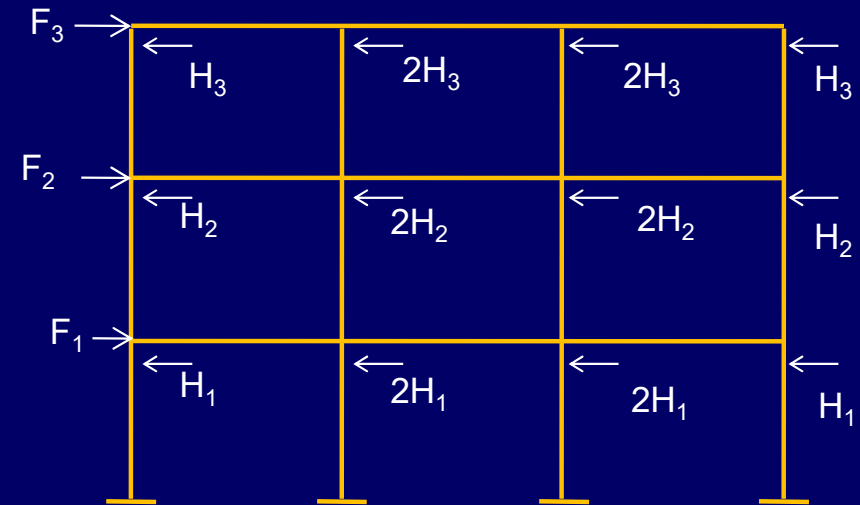
- This proposition is due to the fact that interior columns are generally more rigid than exterior columns (interior column with larger axial load will require larger cross section).

$$6 H_3 = F_3 \text{ or } H_3 = F_3 / 6$$

$$H_3 = F_3 / 2n$$

$$\text{and } 2H_3 = F_3 / n$$

Where n = no. of bays

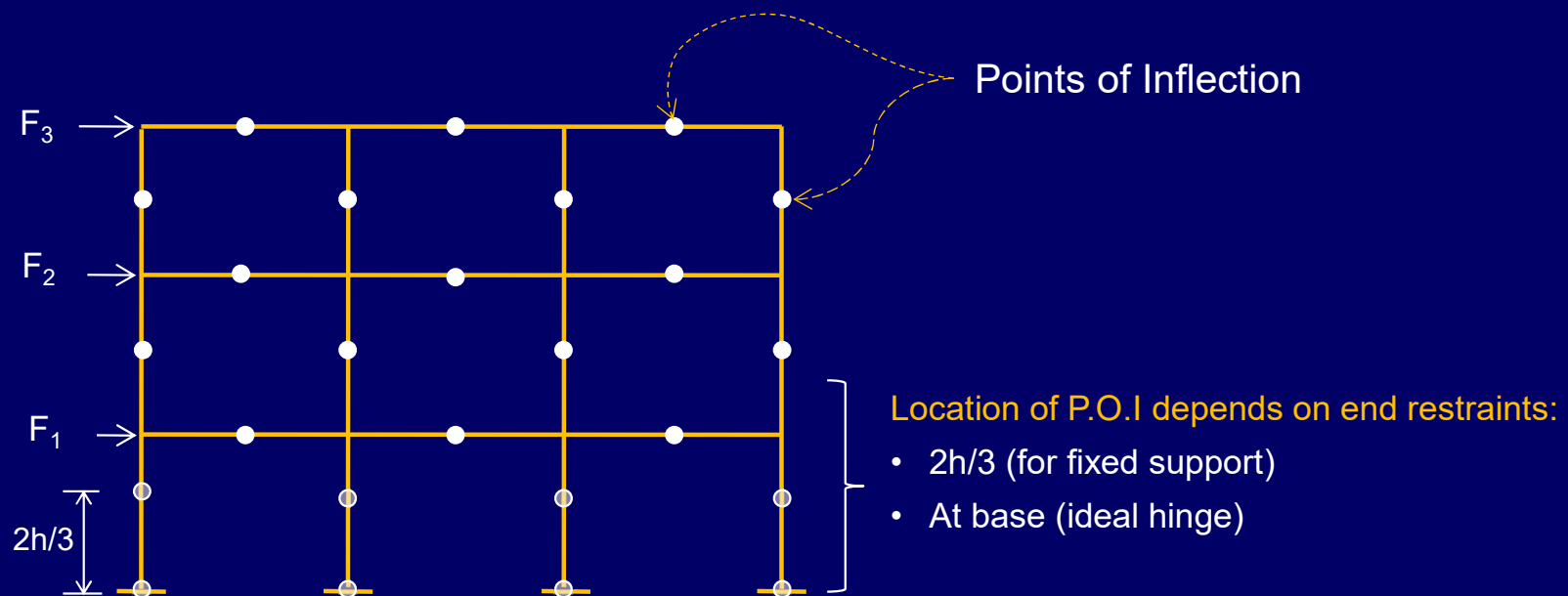




Portal Frame Method

□ Propositions in Portal Frame Method

- The inflection points of all members (columns and beams) are located midway between the joints except for bottom story.

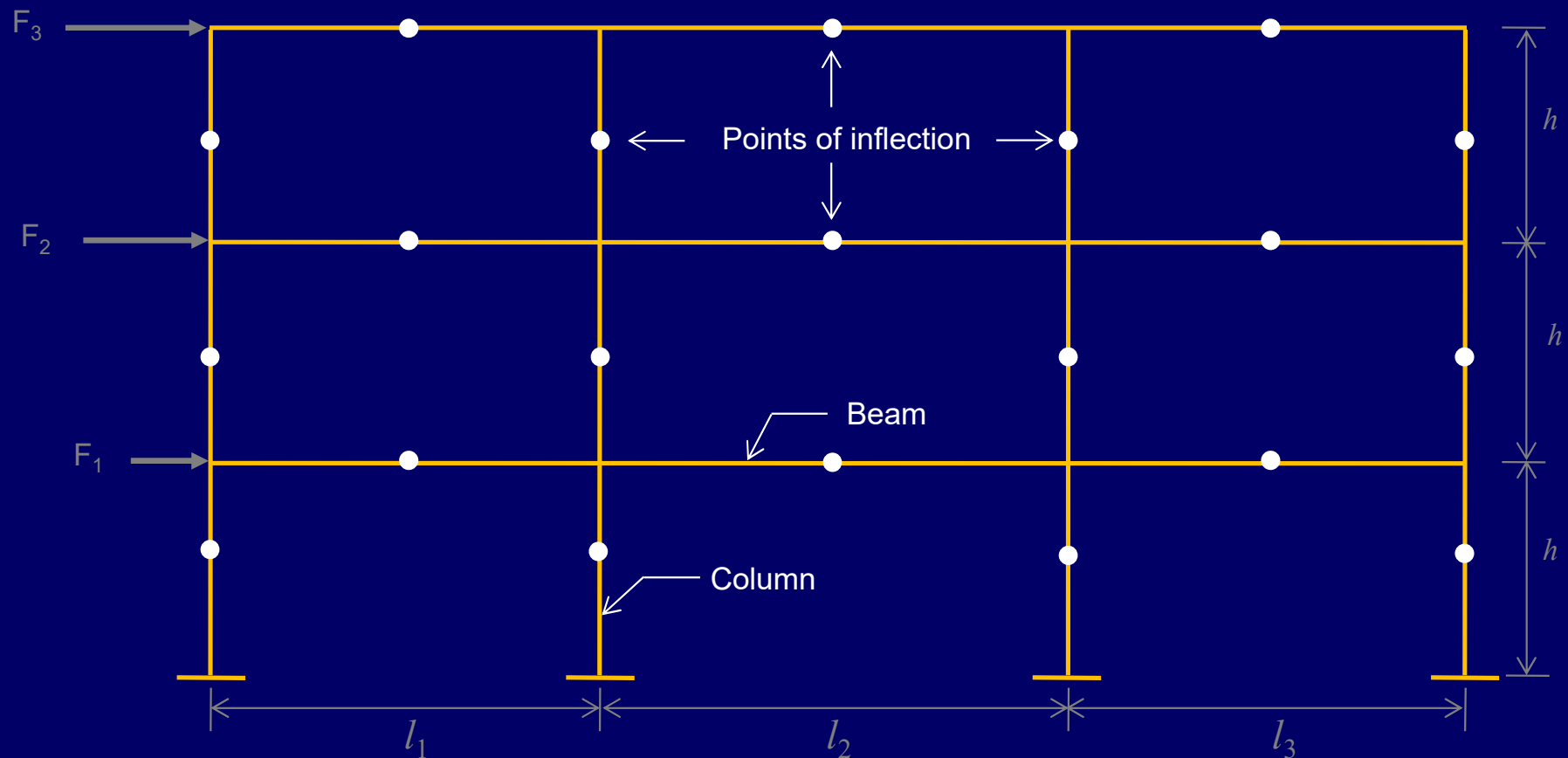




Portal Frame Method

□ Steps in Portal Frame Method

➤ Step 1: Location of Points of Inflection (preposition 3)

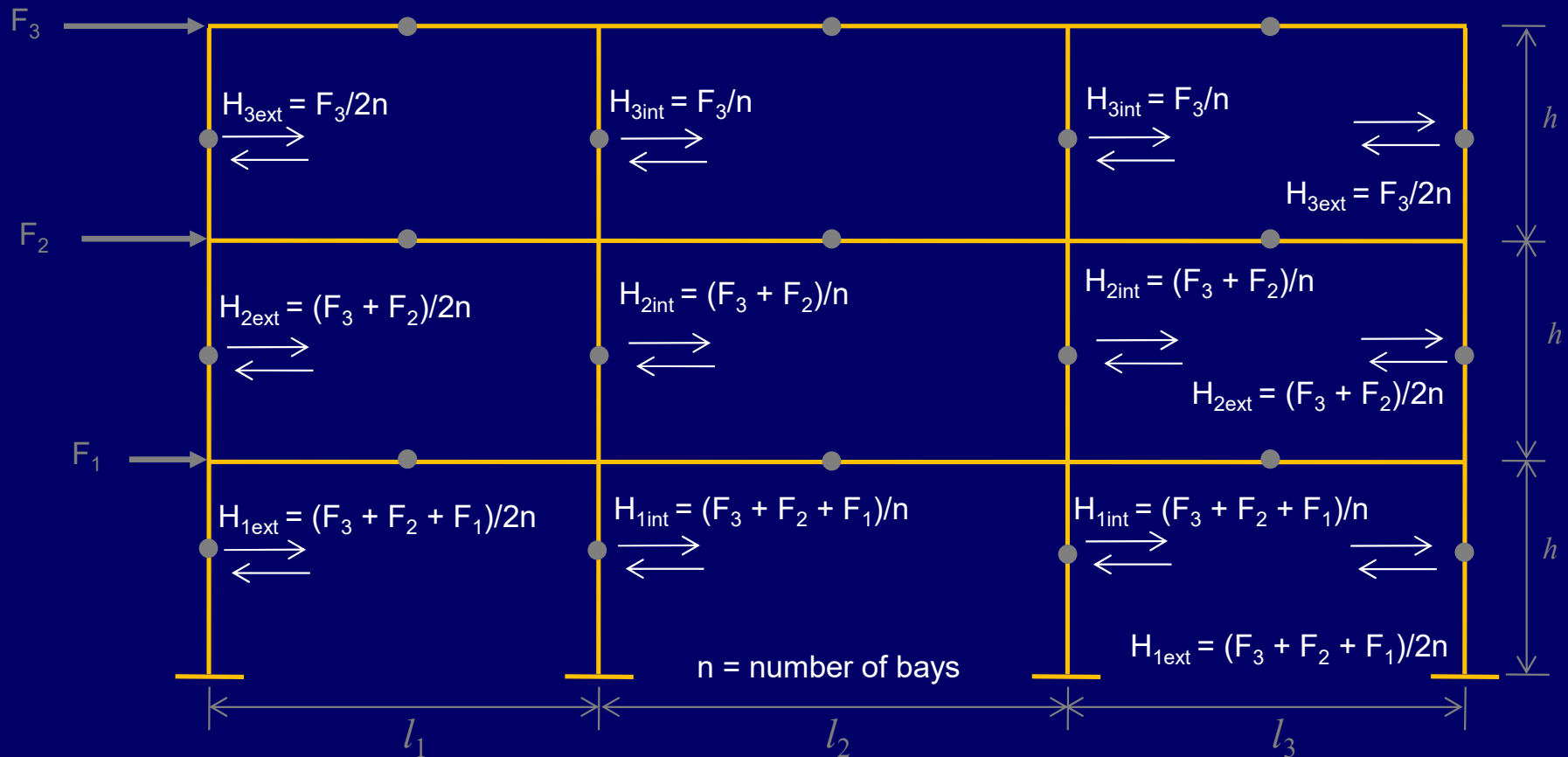




Portal Frame Method

□ Steps in Portal Frame Method

➤ Step 2: Determination of Column Shears (prepositions 2 & 3)

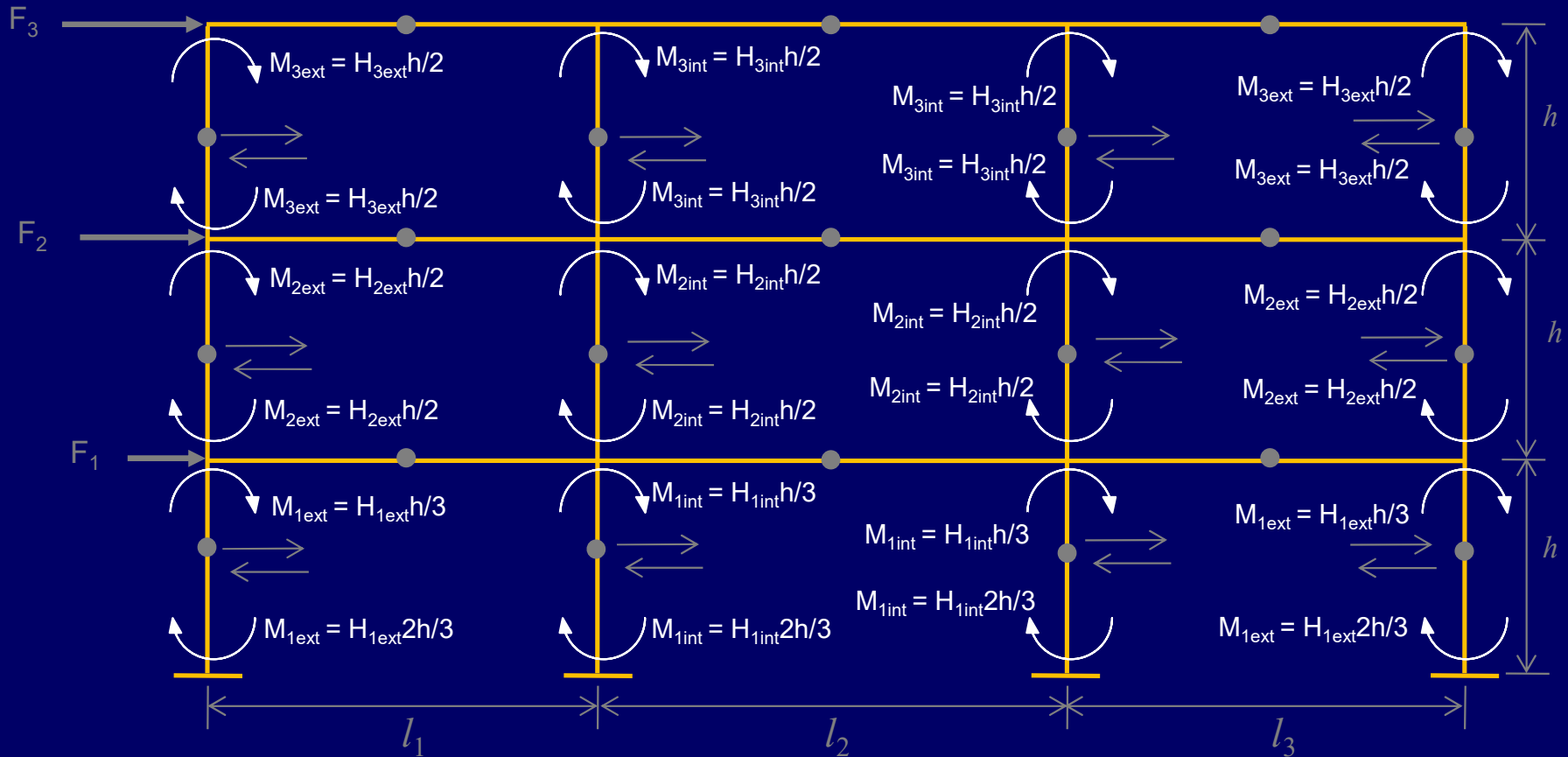




Portal Frame Method

□ Steps in Portal Frame Method

➤ Step 3: Determination of Column Moments from Statics





Portal Frame Method

□ Steps in Portal Frame Method

➤ Step 4: Determination of Beam Moments from Statics

- Beam moments at a joint can be determined from equilibrium. The beam moments to the left (M_{BL}) and right (M_{BR}) of a joint can be determined from the following formulae.

$$M_{BL} = \frac{\sum M_{col}}{m}$$

$$M_{BR} = \frac{\sum M_{col}}{m}$$

Where;

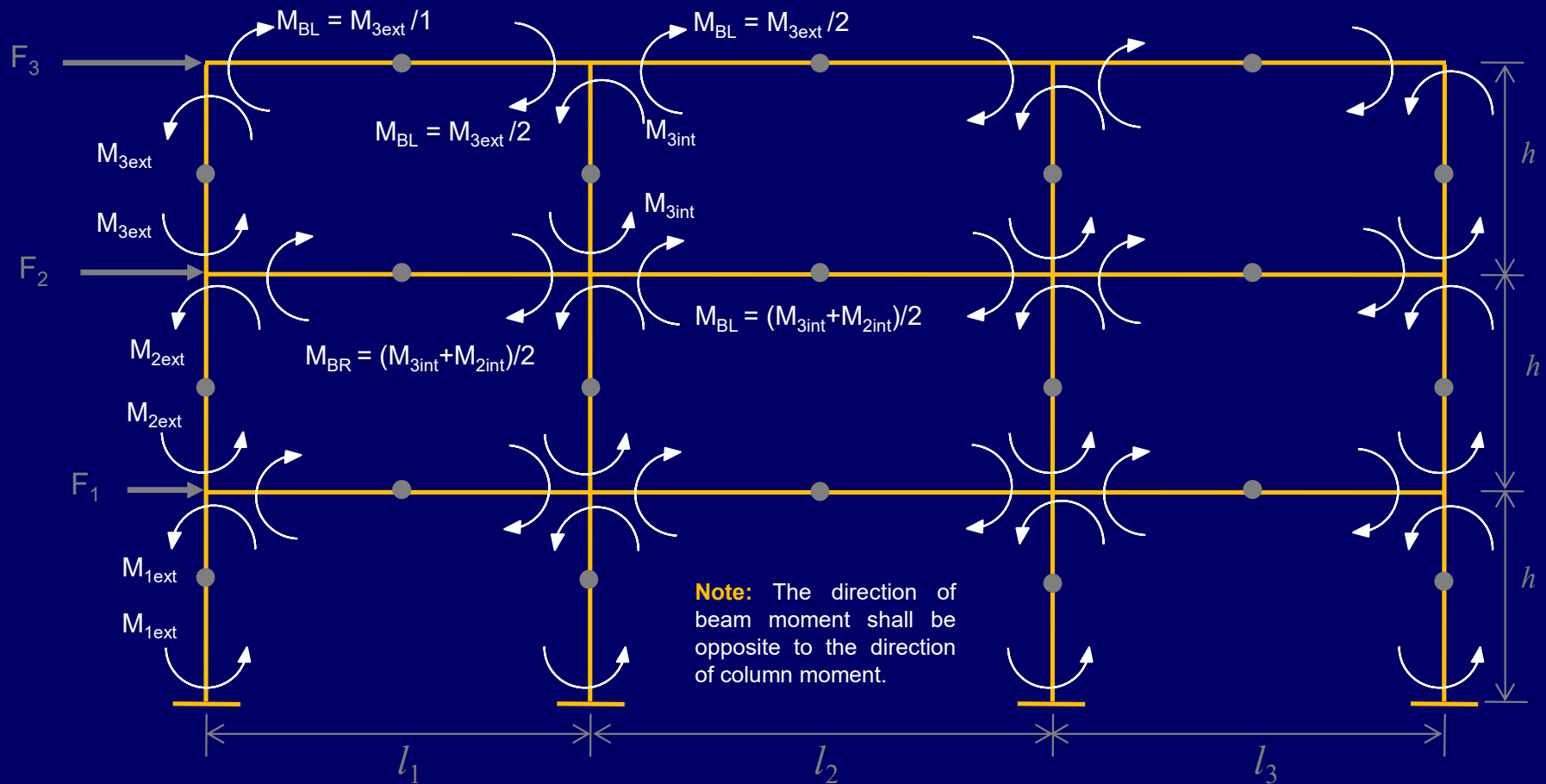
- m = number of connecting beams at a joint.
- $\sum M_{col}$ = summation of column moments at a joint.



Portal Frame Method

□ Steps in Portal Frame Method

➤ Step 4: Determination of Beam Moments from Statics

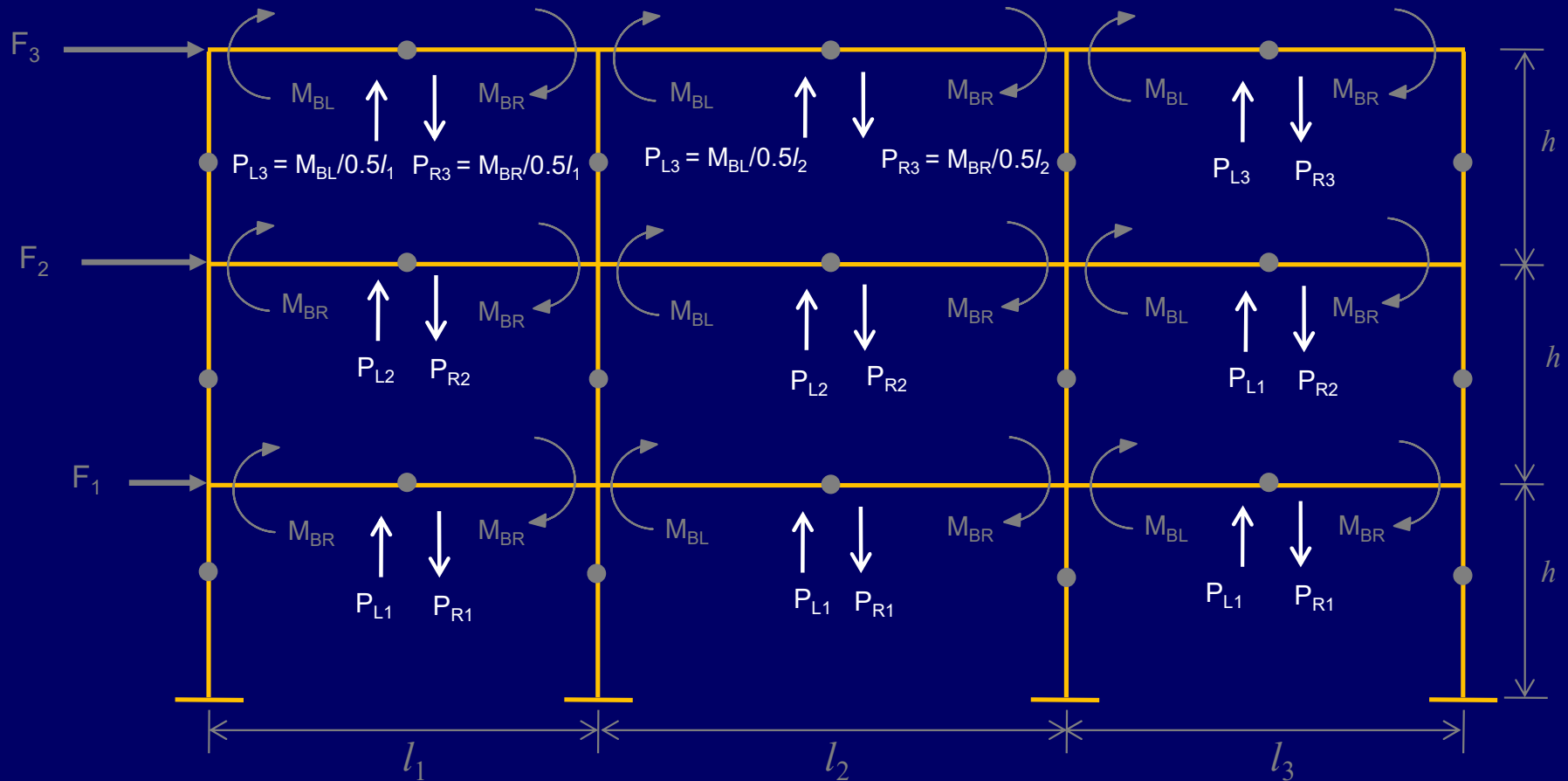




Portal Frame Method

□ Steps in Portal Frame Method

➤ Step 5: Determination of Beam Shear from Statics

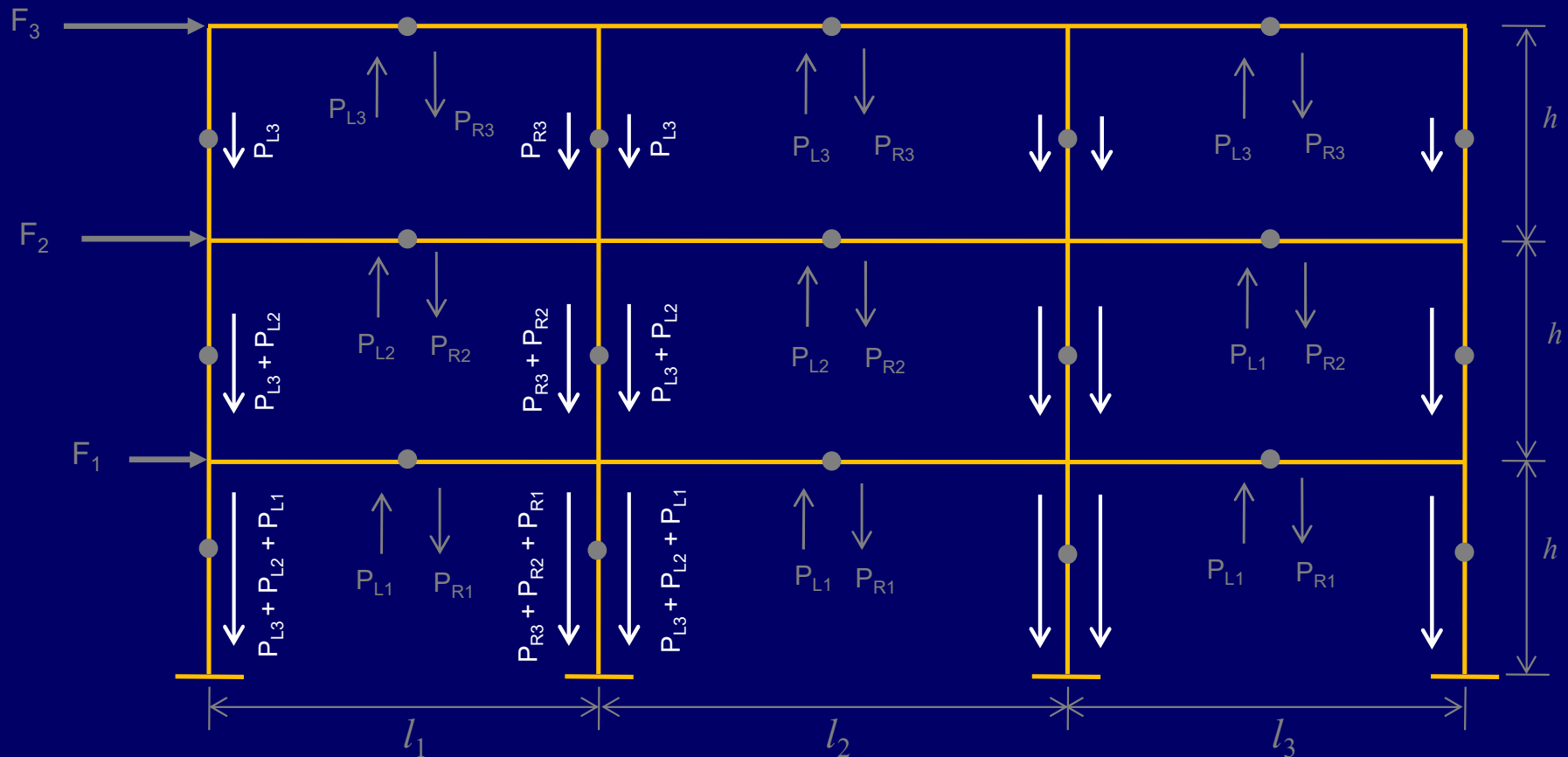




Portal Frame Method

□ Steps in Portal Frame Method

➤ Step 6: Determination of Column Axial Forces from Statics





Case Study 2

Lateral Load Analysis by Portal Frame Method and Comparison with SAP2000

The objective is to check the level of accuracy of portal method.

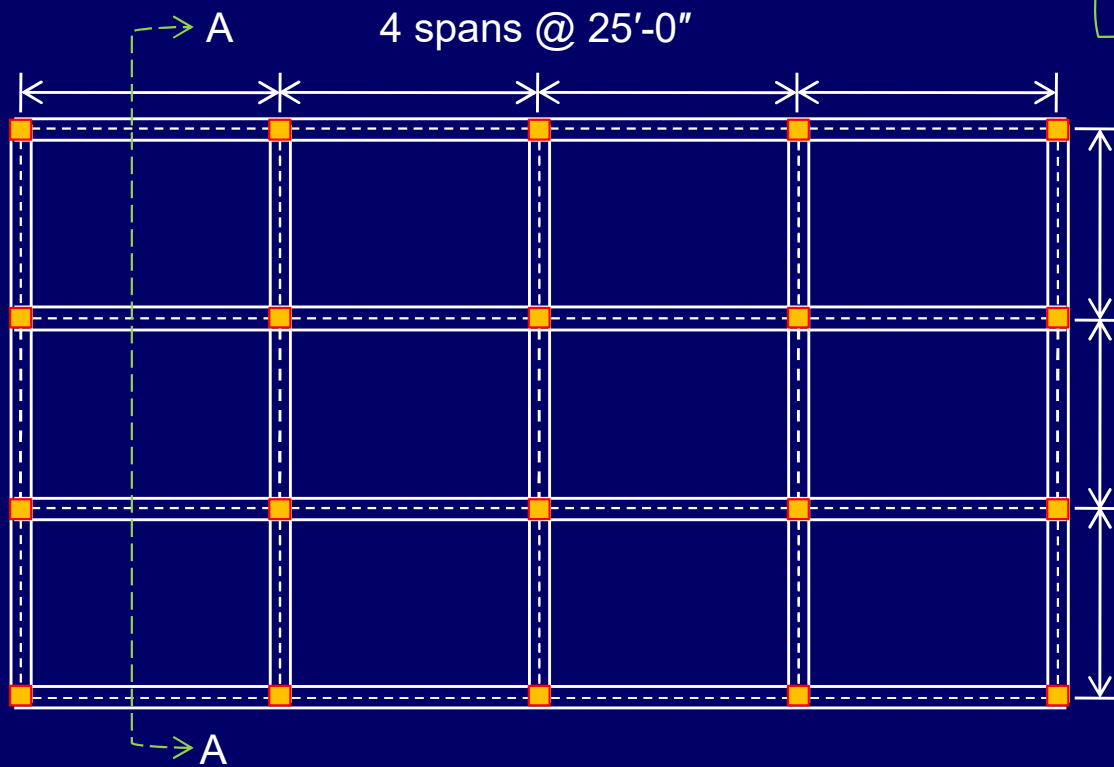


Case Study 2

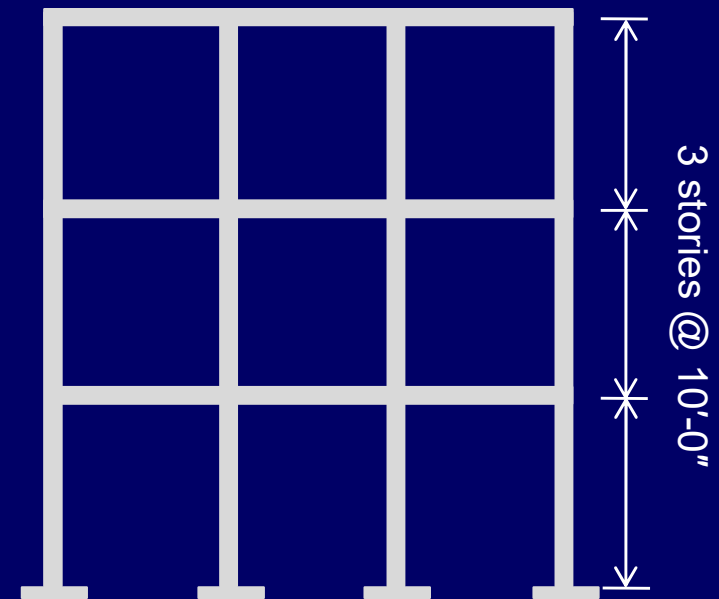
□ Geometry and Input Data

SDL = 40 psf and L = 60 psf on all floors

$f'_c = 3$ ksi and $f_y = 40$ ksi



3 spans @ 20'-0"

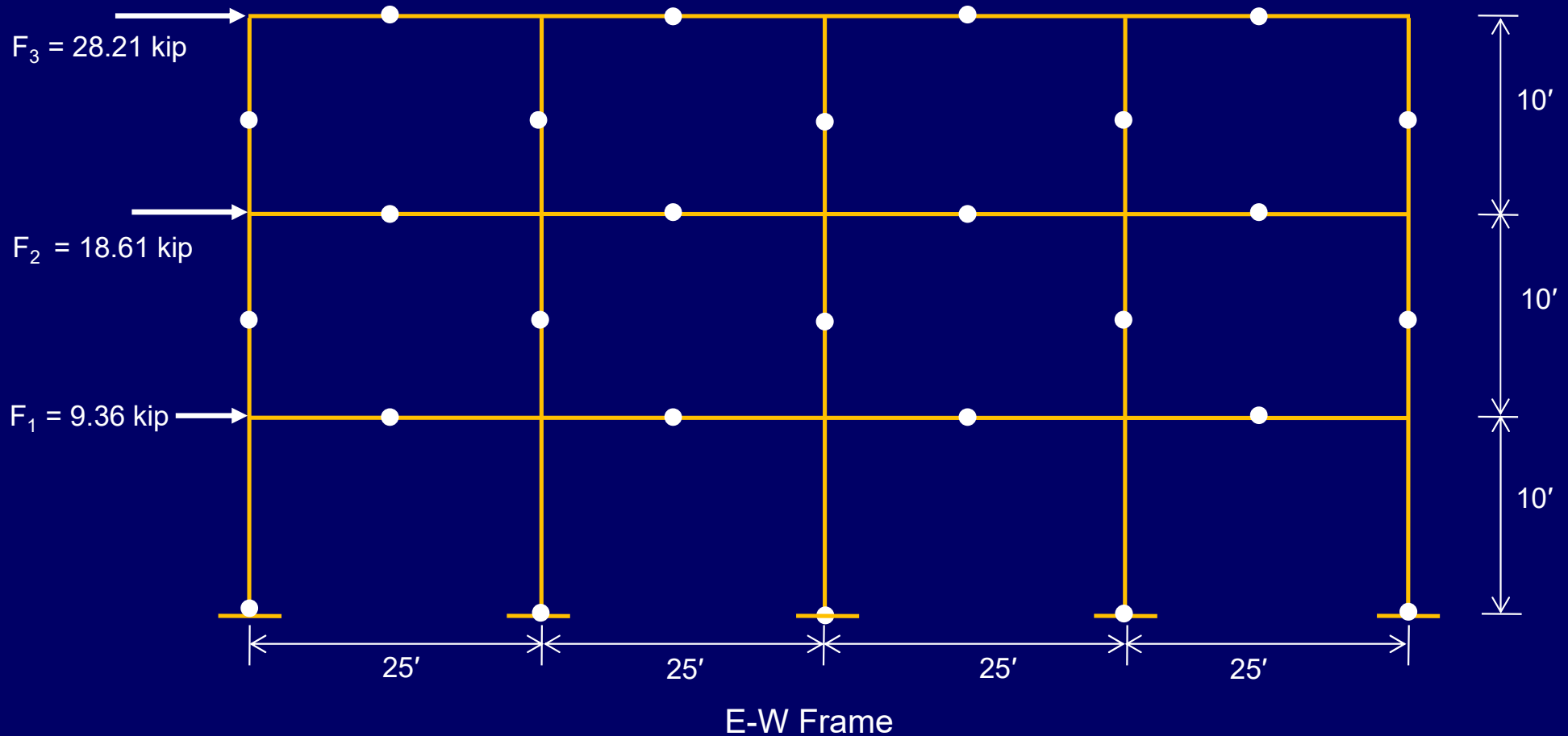


Section A-A



Case Study 2

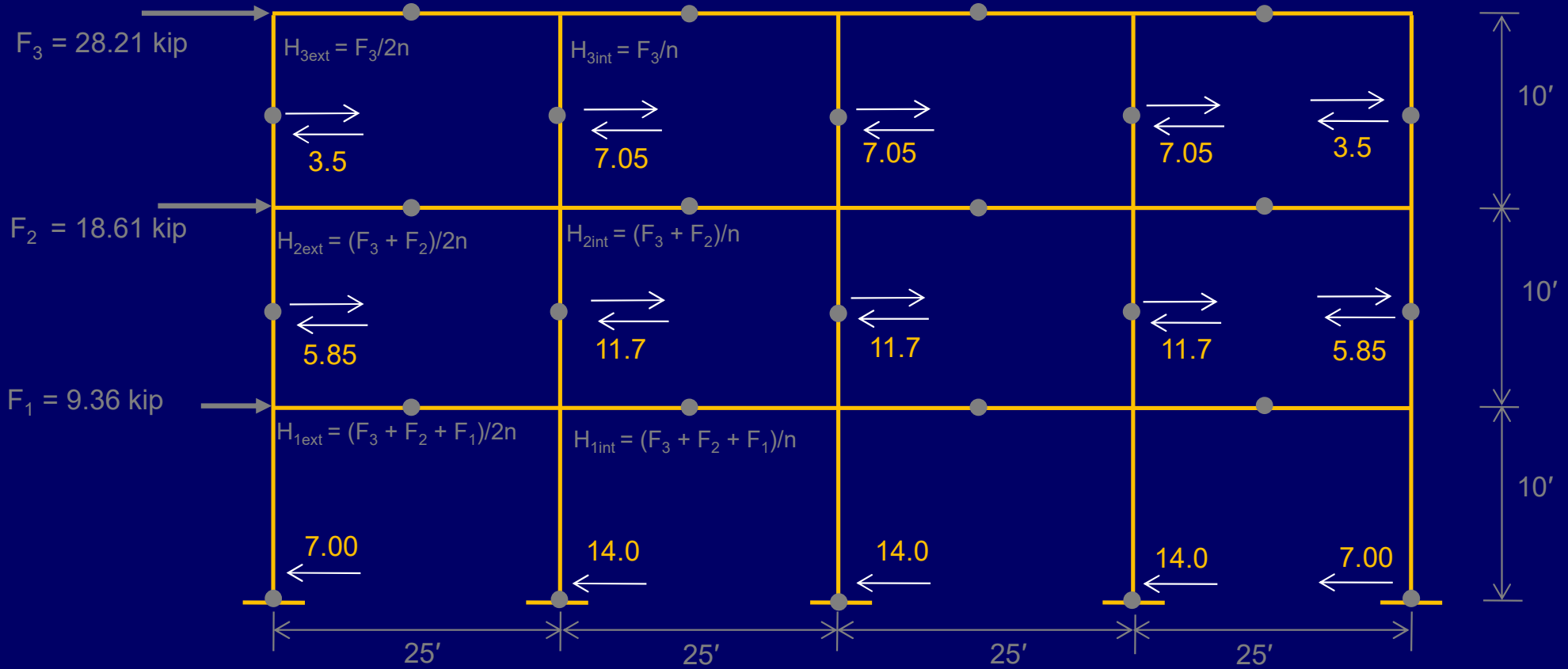
➤ Step 1 : Location of Points of Inflection





Case Study 2

➤ Step 2 : Determination of Column Shear



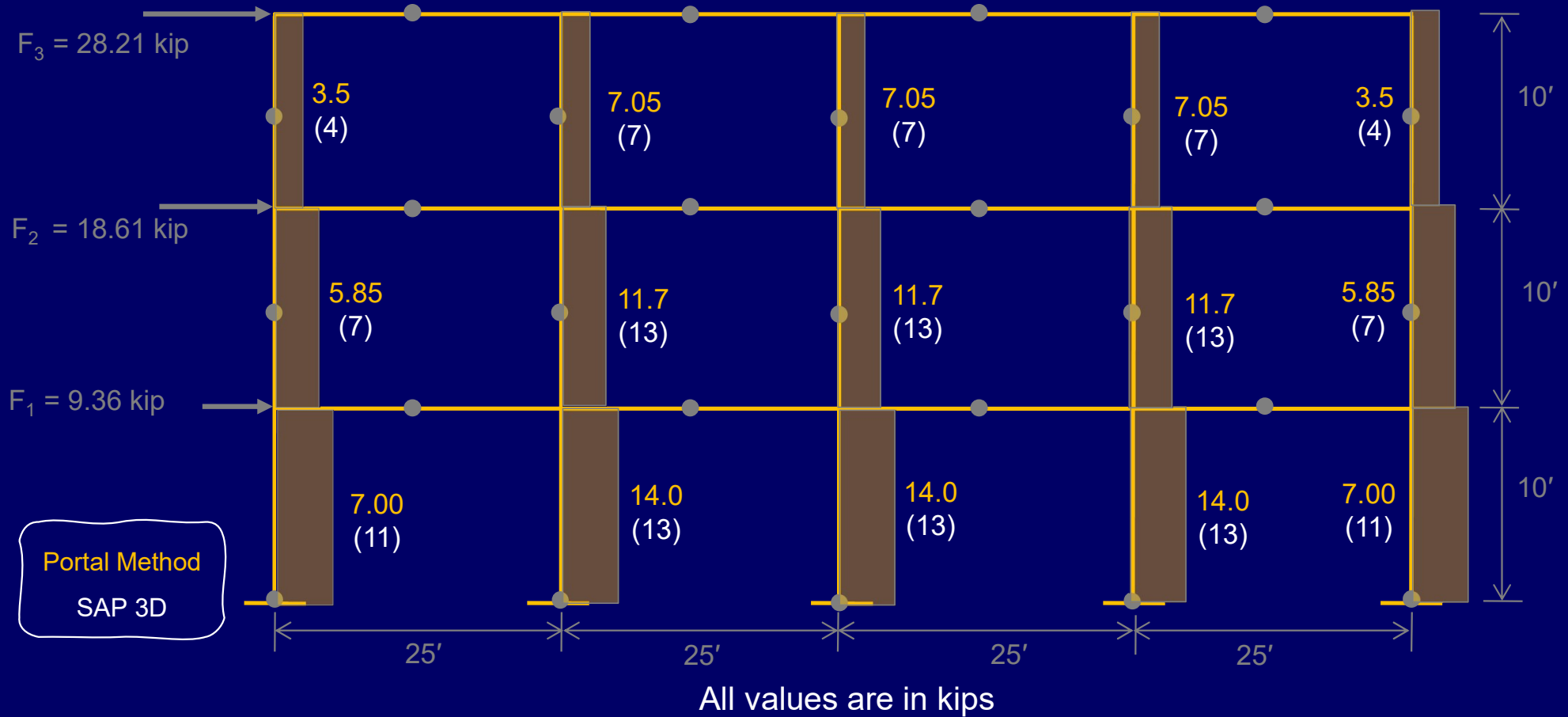
All values are in kips



Case Study 2

➤ Step 2 : Determination of Column Shear

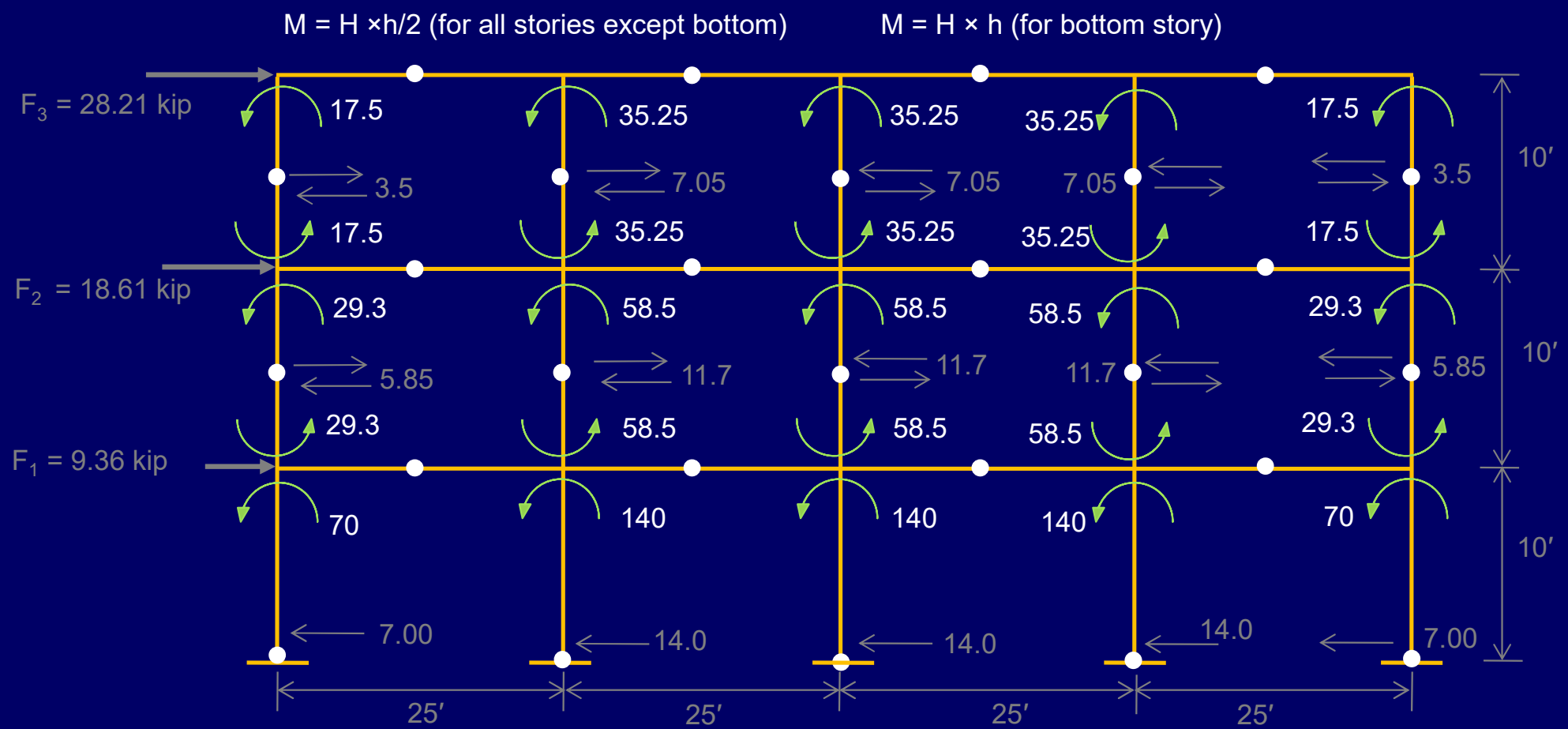
❖ Comparison with SAP2000





Case Study 2

➤ Step 3: Determination of Column Moments



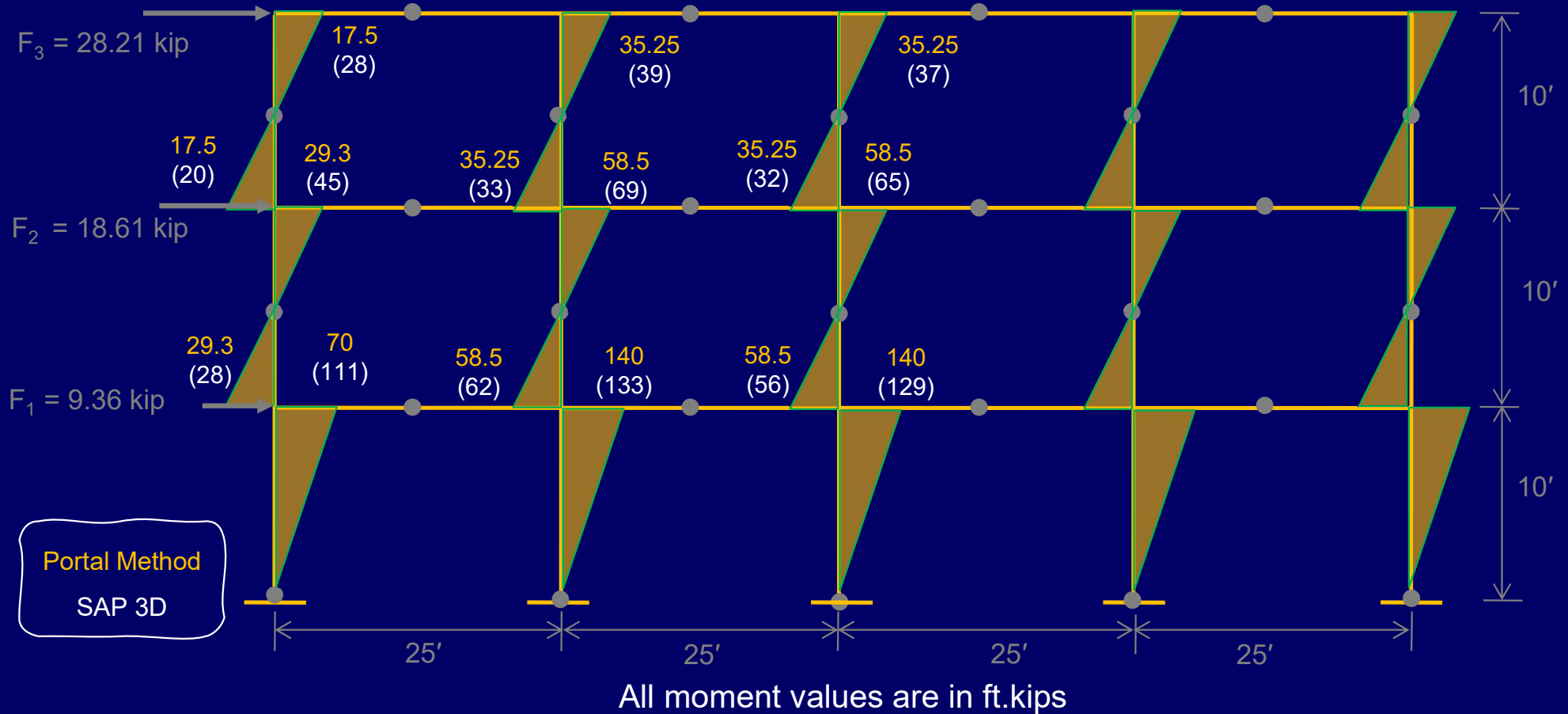
All moment values are in ft.kips



Case Study 2

➤ Step 3: Determination of Column Moments

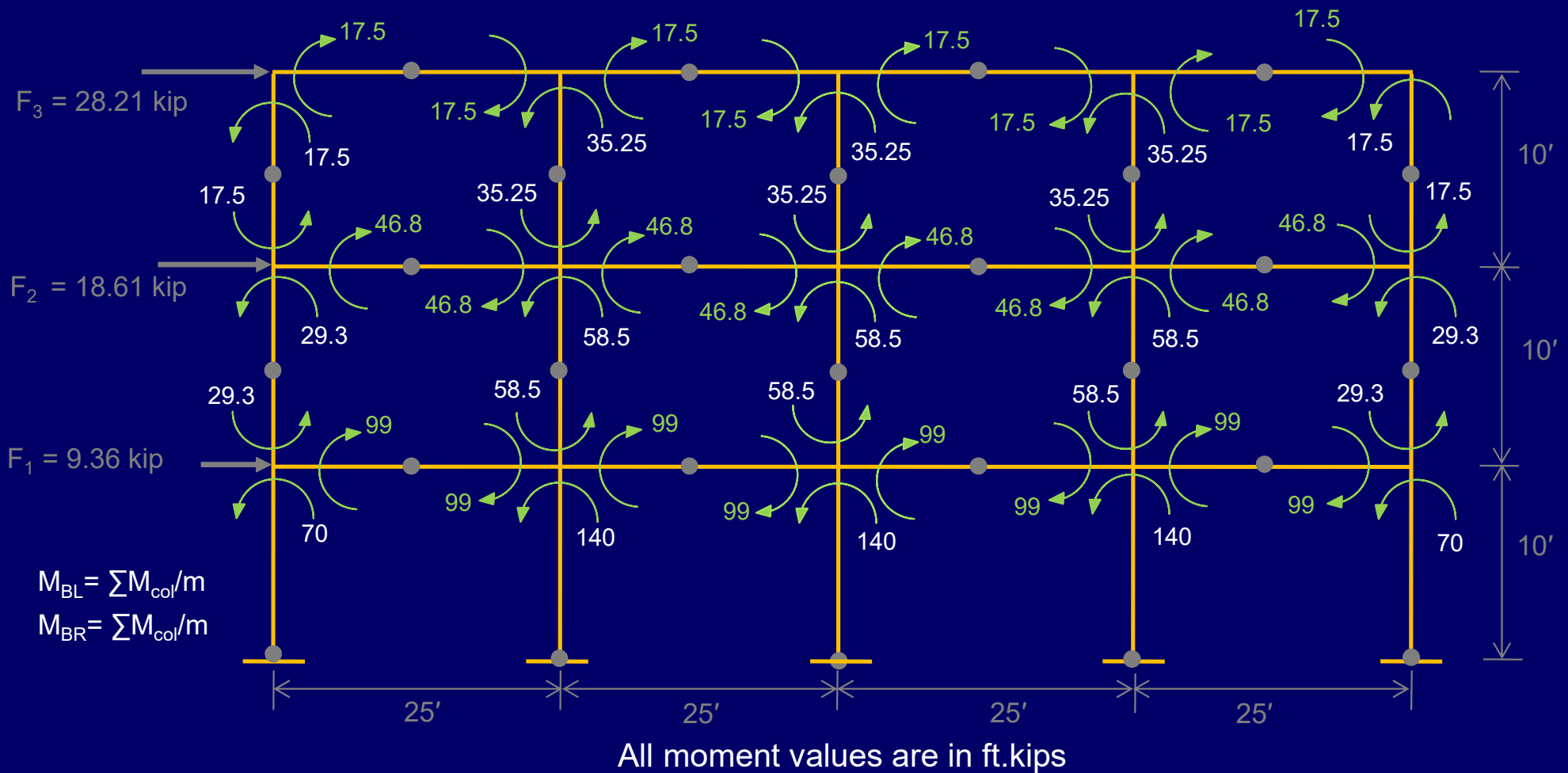
❖ Comparison with SAP2000





Case Study 2

➤ Step 4: Determination of Beam Moments

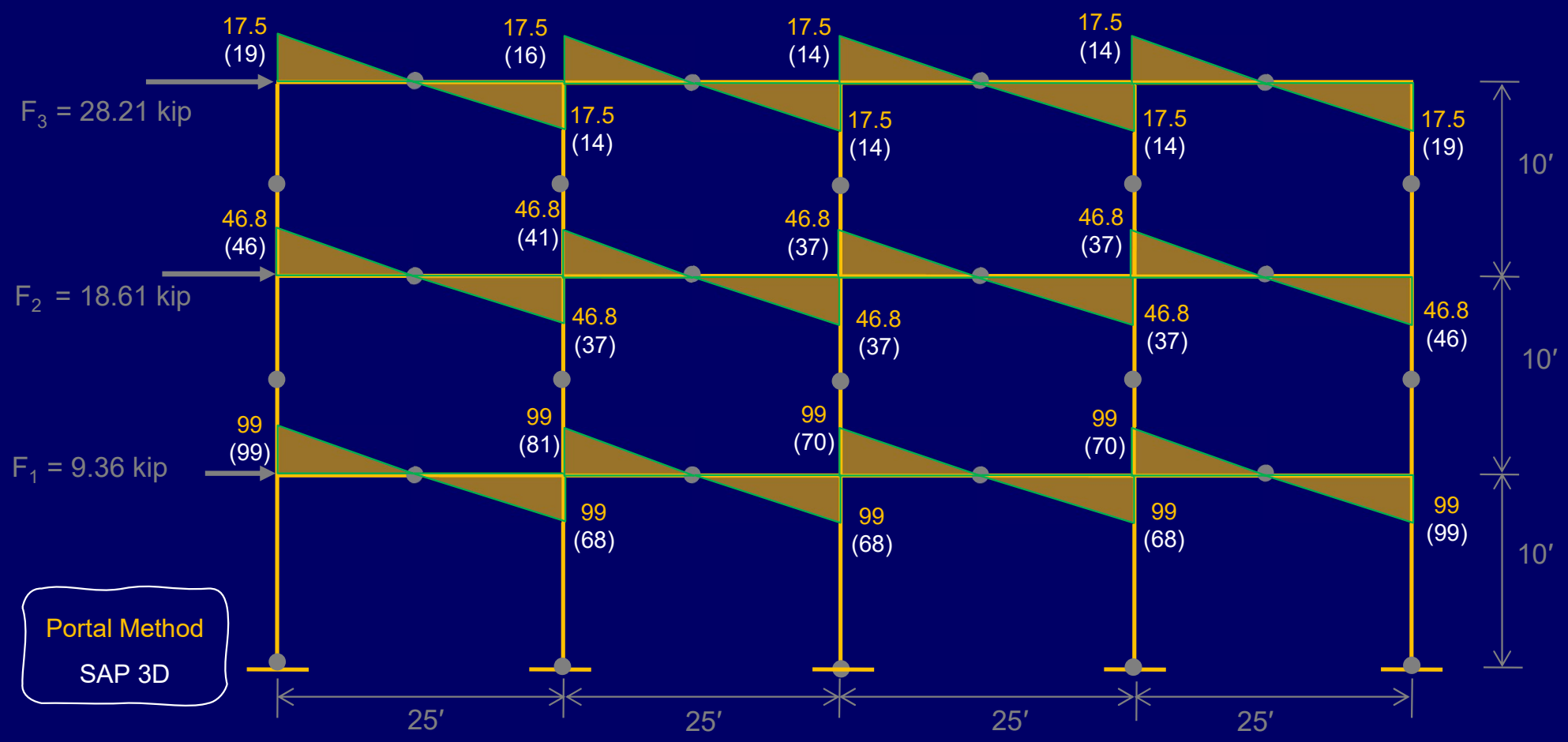




Case Study 2

➤ Step 4: Determination of Beam Moments

❖ Comparison with SAP2000



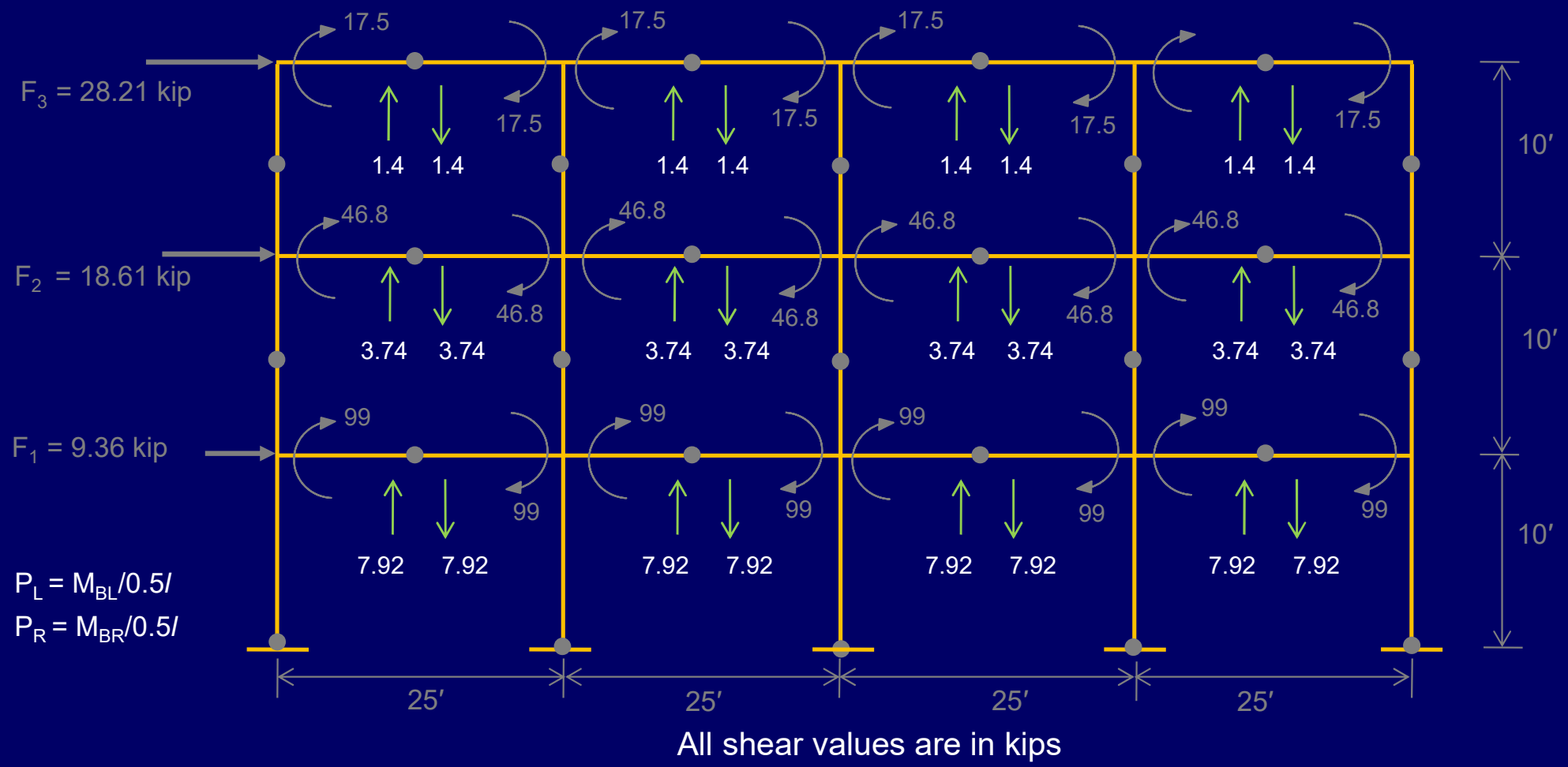
All moment values are in ft.kips

Portal Method
SAP 3D



Case Study 2

➤ Step 5: Determination of Beam Shear

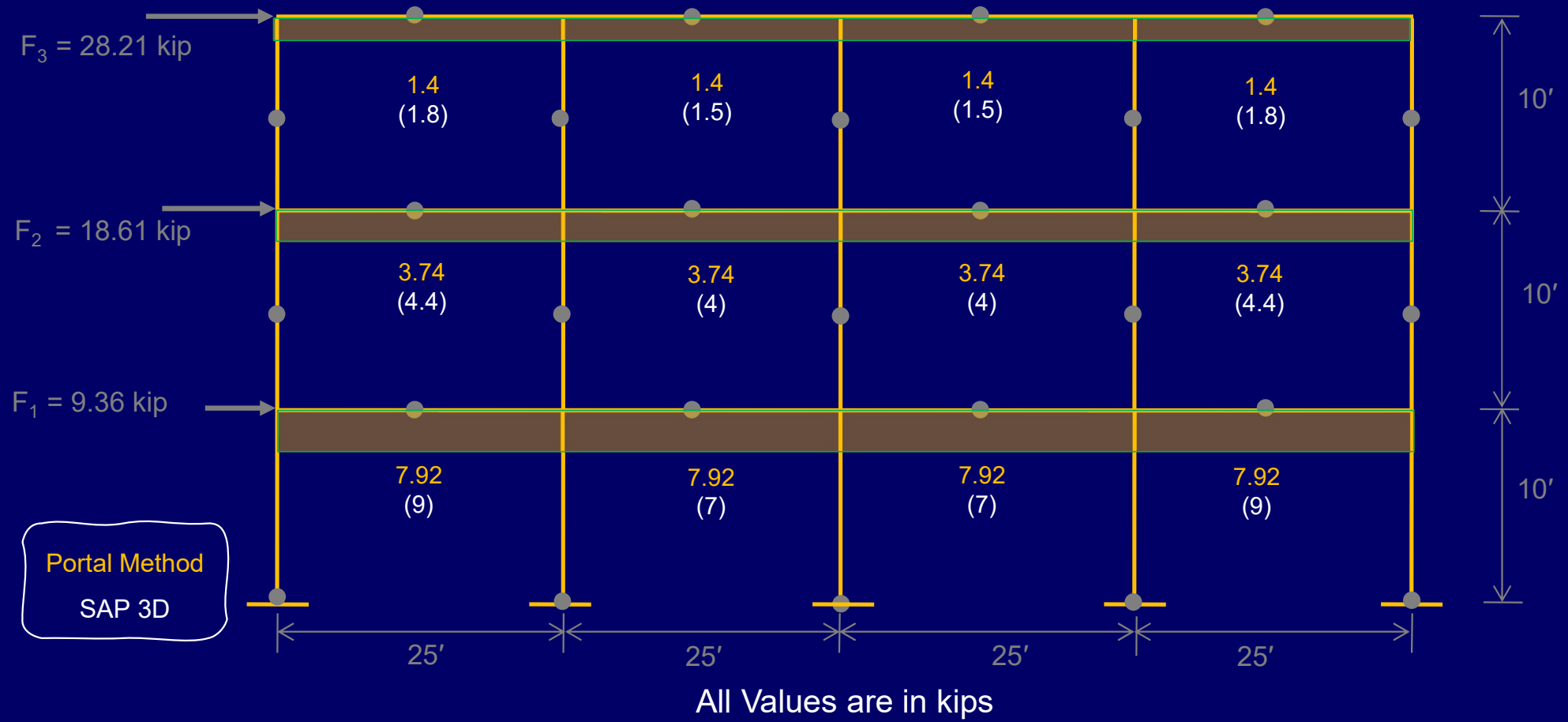




Case Study 2

➤ Step 5: Determination of Beam Shear

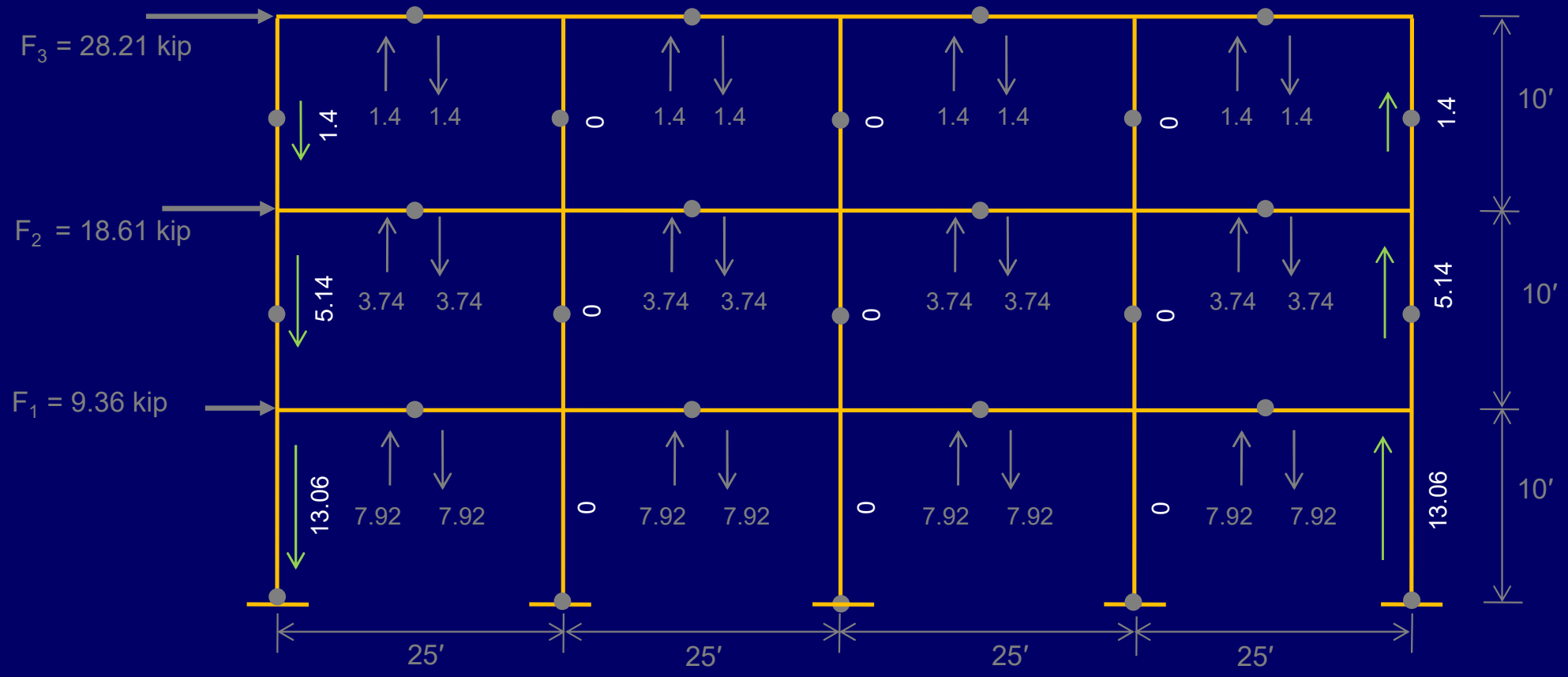
❖ Comparison with SAP2000





Case Study 2

➤ Step 6: Determination of Column Axial Forces



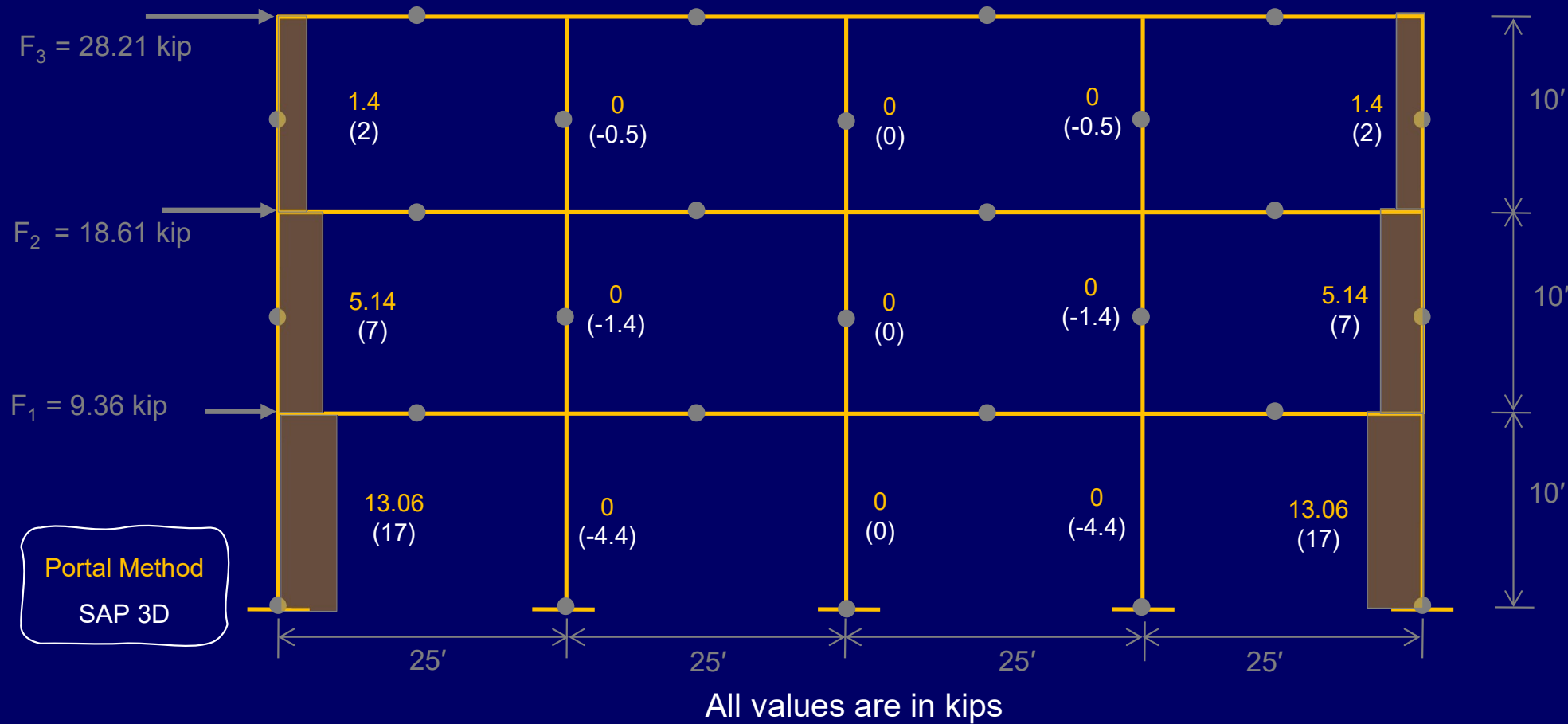
All shear values are in kips



Case Study 2

➤ Step 6: Determination of Column Axial Forces

❖ Comparison with SAP2000



Portal Method
SAP 3D



Limit Analysis

□ Introduction

- Most RC structures are designed using following approach:
 - Moments, shears, and axial forces in RC structures are found by elastic theory.
 - The actual proportioning of members is done by strength methods, in which inelastic section and member response is considered.
- Although this design approach is safe and conservative but is inconsistent to total analysis-design process.



Limit Analysis

□ Redistribution

- A frame normally will not fail when the nominal moment capacity of just one critical section is reached:
 - A plastic hinge will form at that section.
 - Large rotation at constant resisting moment will occur.
 - Load transfer to other locations (having more capacity) along the span will occur.
 - On further increase in load, additional plastic hinges may form at other locations along the span.
 - As a result, structure will collapse, but only after a significant redistribution of moments.



Limit Analysis

□ Redistribution

- Full use of the plastic capacity of reinforced concrete beams and frames requires an extensive analysis of all possible mechanisms and an investigation of rotation requirements and capacities at all proposed hinge locations.
- On the other hand, a restricted amount of redistribution of elastic moments can safely be made without complete analysis yet may be sufficient to obtain most of the advantages of limit analysis.



Limit Analysis

□ Redistribution of Moments in Continuous Flexural Members

- A limited amount of redistribution is permitted by ACI Code 6.6.5. depending upon a rough measure of available ductility, without explicit calculation of rotation requirements and capacities.



Limit Analysis

□ Redistribution of Negative Moments in Continuous Flexural Members

- The net tensile strain in the extreme tension steel at nominal strength ϵ_t given in eq. below, is used as an indicator of rotation capacity.
- The ACI Code Section 6.6.5 states “except where approximate values for moments are used, it shall be permitted to increase or decrease negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement by not more than $1000\epsilon_t$ percent, with a maximum of 20 percent”.

$$\epsilon_t = \frac{\epsilon_u(d - c)}{c}$$



Limit Analysis

□ Redistribution of Negative Moments in Continuous Flexural Members

$$\epsilon_t = \epsilon_u(d - c)/c$$

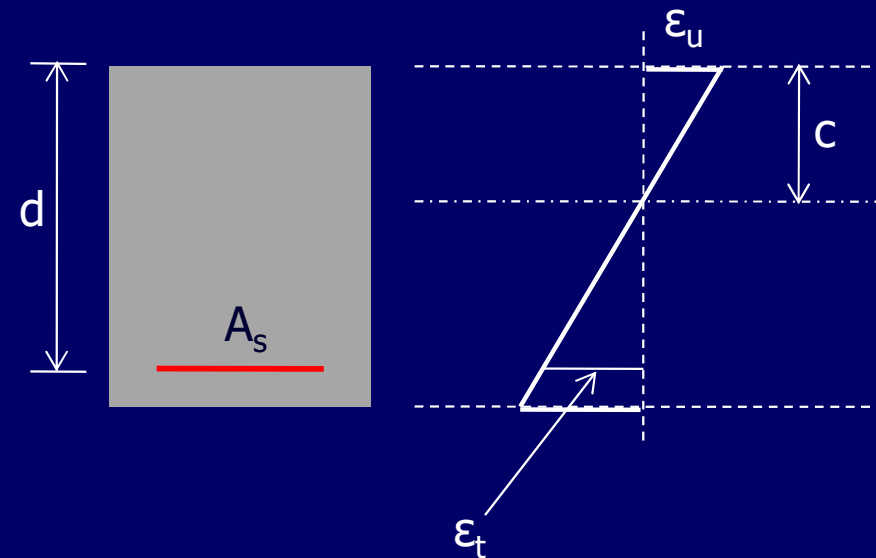
$$\epsilon_u = 0.003$$

As example, for given A_s if:

$$d = 16.5'' ; c = 4''$$

$$\epsilon_t = 0.009$$

$$1000\epsilon_t = 9 \% < 20 \%$$

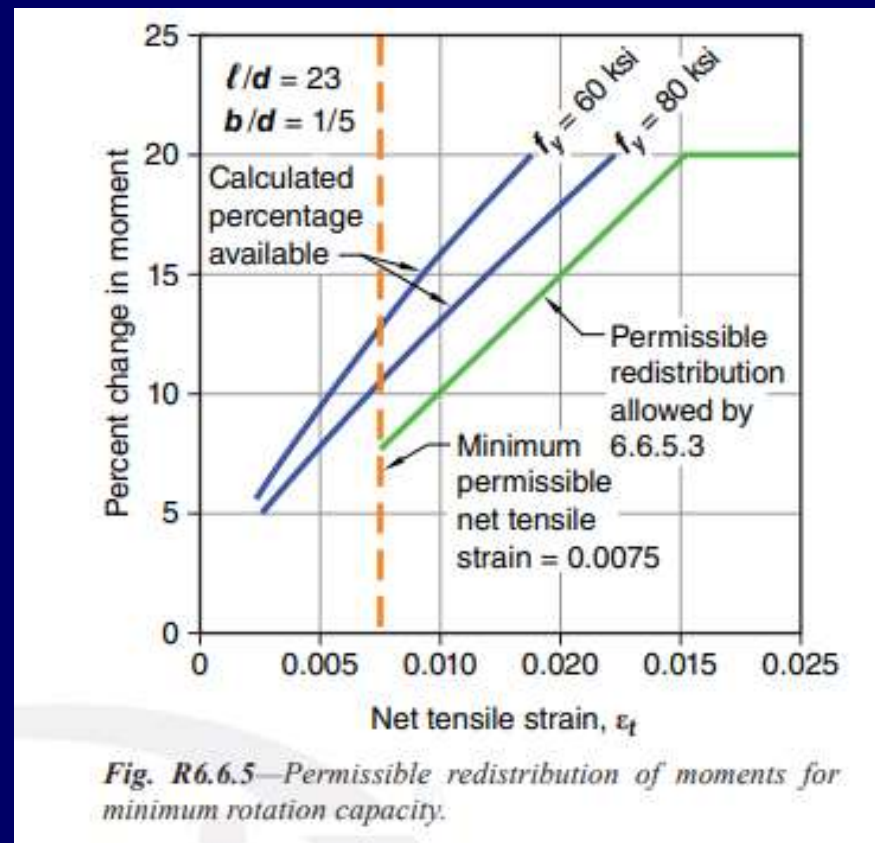




Limit Analysis

□ Redistribution of Negative Moments in Continuous Flexural Members

- Graphical representation of ACI code provision





Limit Analysis

□ Redistribution of Negative Moments in Continuous Flexural Members

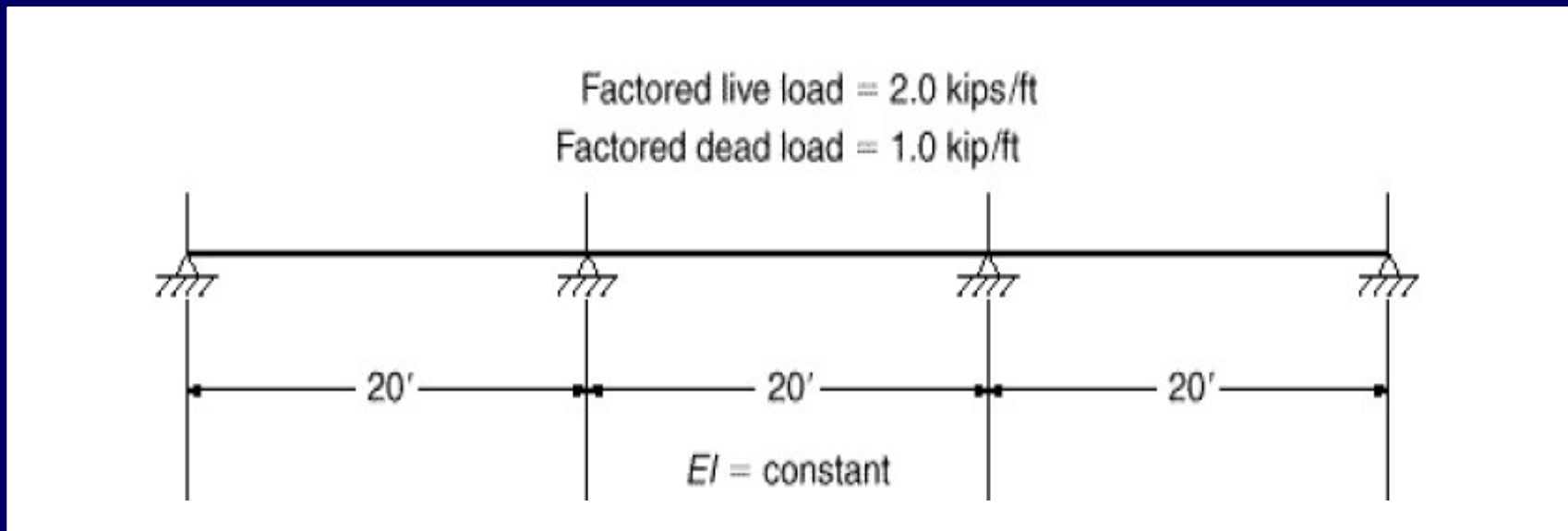
- The modified negative moments shall be used for calculating moments at sections within the spans.
- Redistribution of negative moments shall be made only when ϵ_t is equal to or greater than 0.0075 at the section at which moment is reduced (ACI 6.6.5.1).



Example 8.2

□ Problem Statement

- For the beam shown, find moment redistribution.

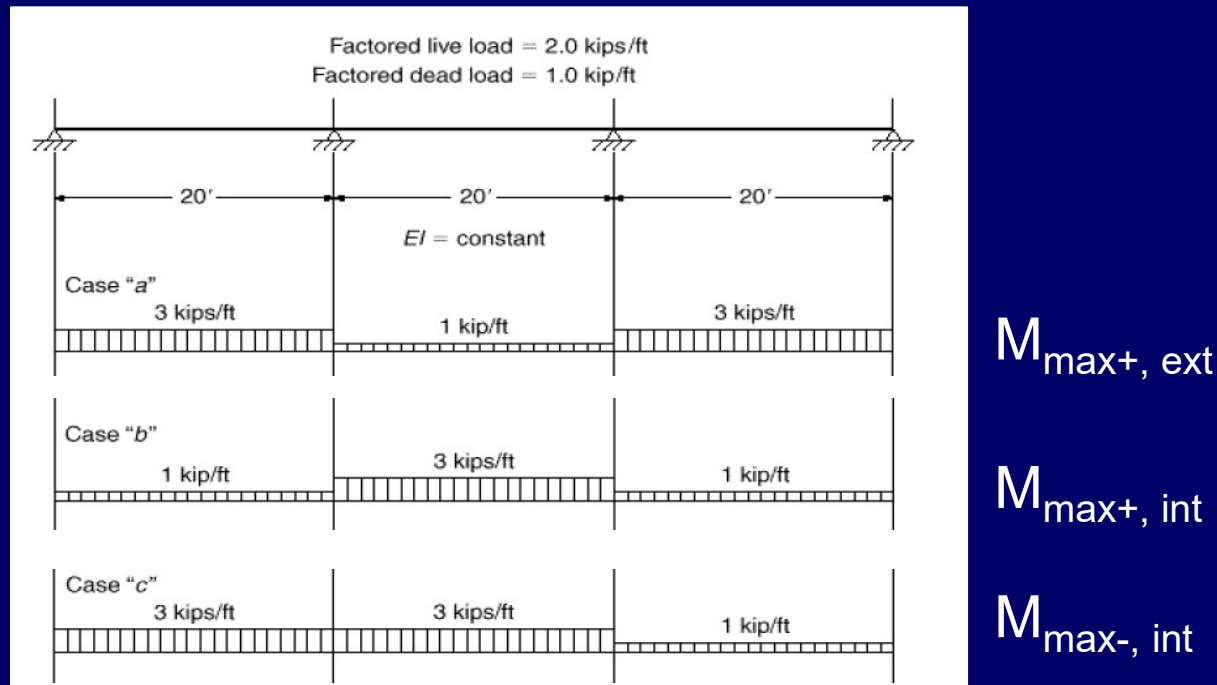




Example 8.2

□ Solution

- To obtain maximum moments at all critical design sections, it is necessary to consider three alternative loadings.
- It will be assumed that 20 % adjustment of support moment is permitted throughout.





Example 8.2

□ Solution

Decrease in exterior positive moment:

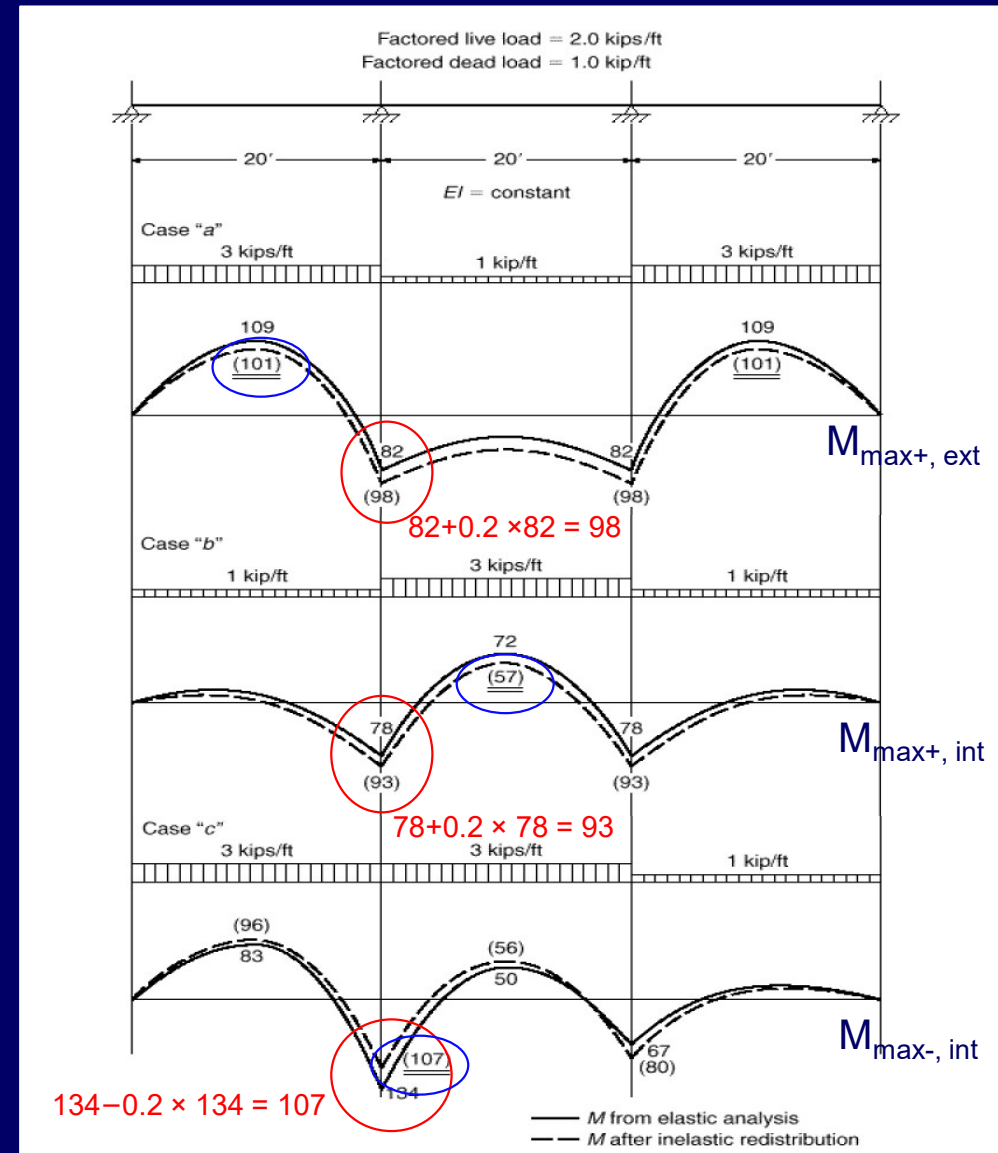
If negative moment is increased 20 %, the result is decrease in $M_{\max+, \text{ext}}$ from 109 to 101

Decrease in interior positive moment:

If negative moment is increased 20 %, the result is decrease in $M_{\max+, \text{int}}$ from 72 to 57

Decrease in interior negative moment:

If negative moment is decreased 20 %, positive moments increase in both spans.





Example 8.2

□ Conclusion on Redistribution of Moments

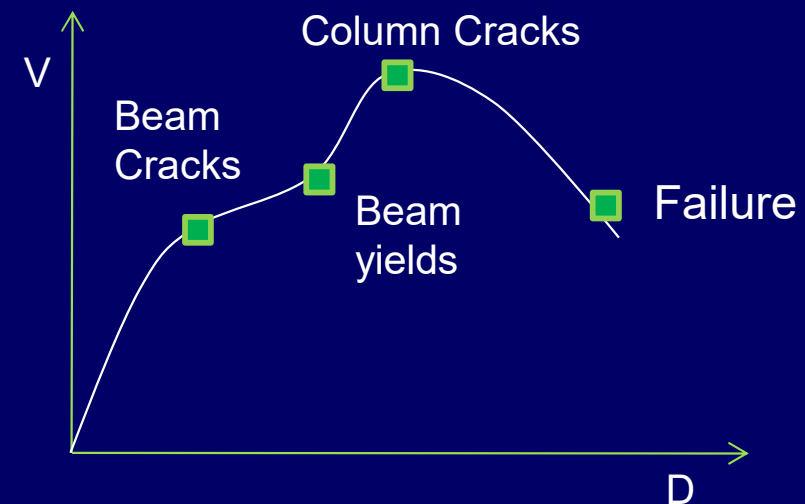
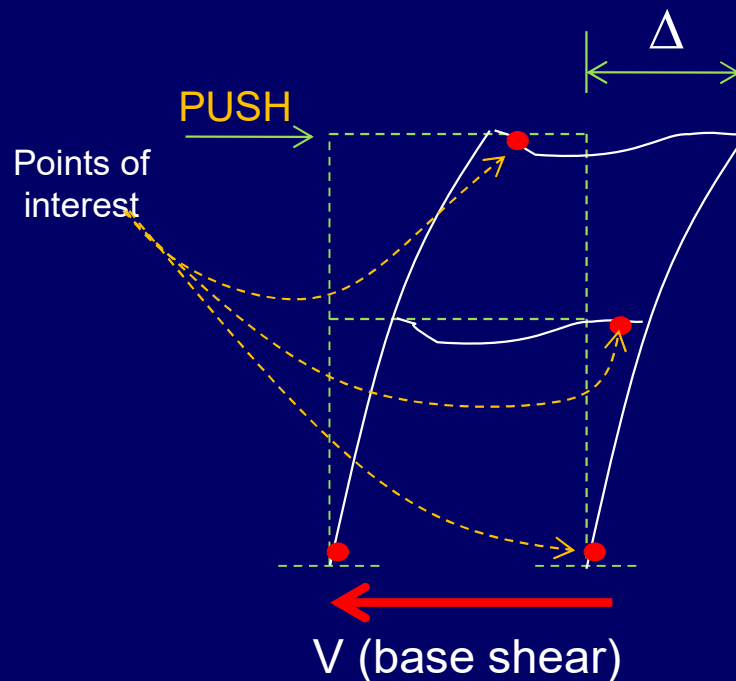
- The net result is a reduction in design moments over the entire beam.
- This modification of moments does not mean a **reduction in safety factor** below that implied in code safety provisions; rather, it means a **reduction of the excess strength** that would otherwise be present in the structure because of the actual redistribution of moments that would occur before failure.
- It reflects the fact that the maximum design moments are obtained from alternative load patterns, which could not exist concurrently.
- The result is a more realistic appraisal of the actual collapse load of the indeterminate structure.



Plastic Analysis

□ Non-Linear Static (Pushover) Analysis

- Points on the structure whose performance (when it yields, cracks or fails) is required to be monitored are selected.
- The structure is pushed at the top.

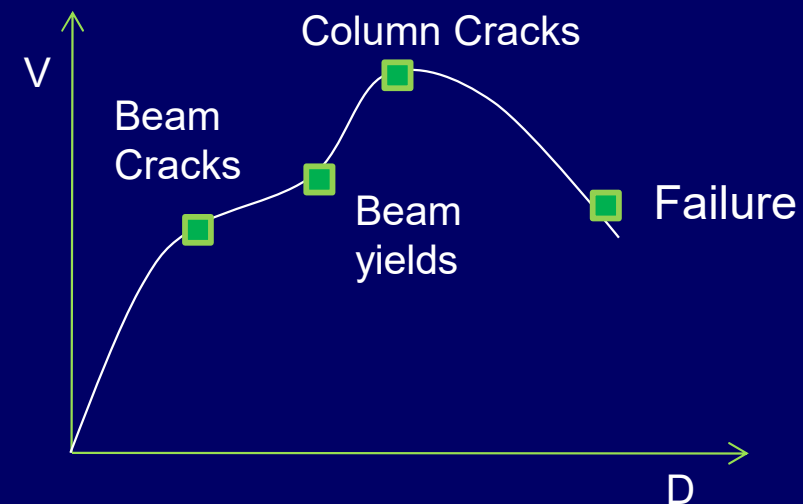
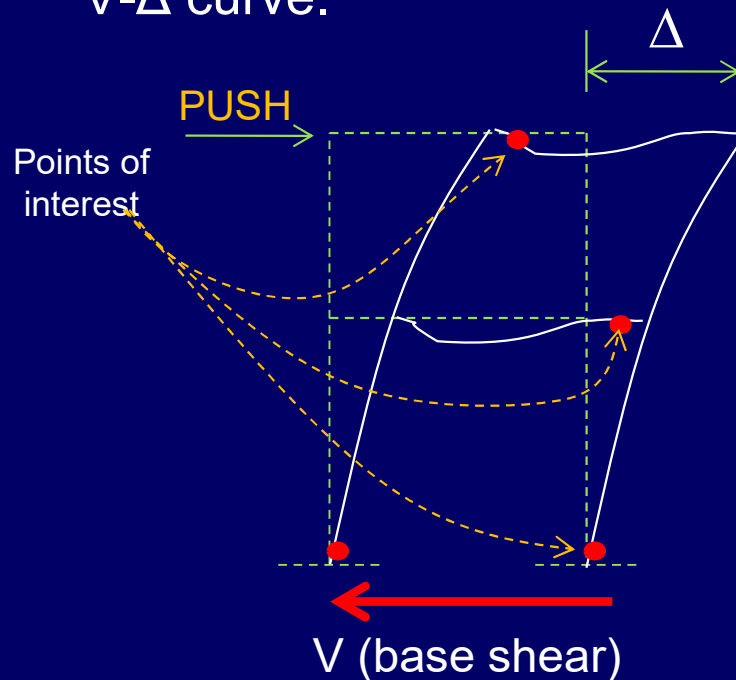




Plastic Analysis

□ Non-Linear Static (Pushover) Analysis

- Top Drift (D) and corresponding base-shear (V) is calculated and plotted on V - Δ curve.
- Structure is further pushed in steps and V - Δ curve is plotted. Also, performance of the selected points is monitored and marked on the V - Δ curve.

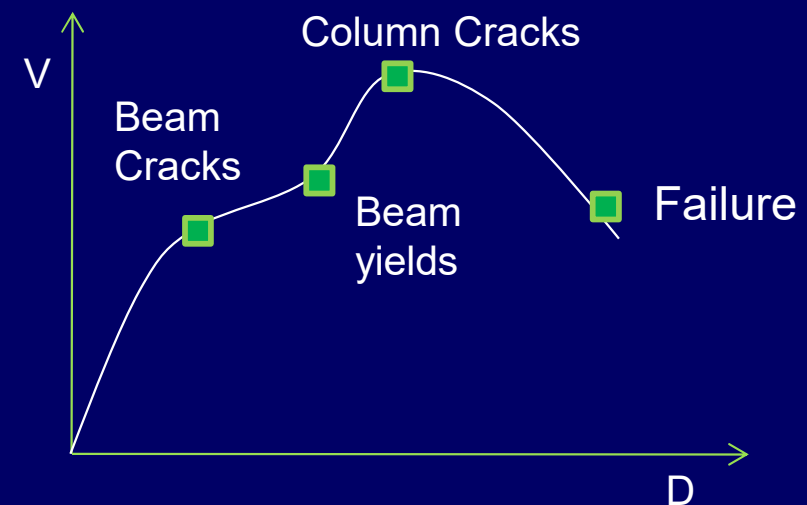
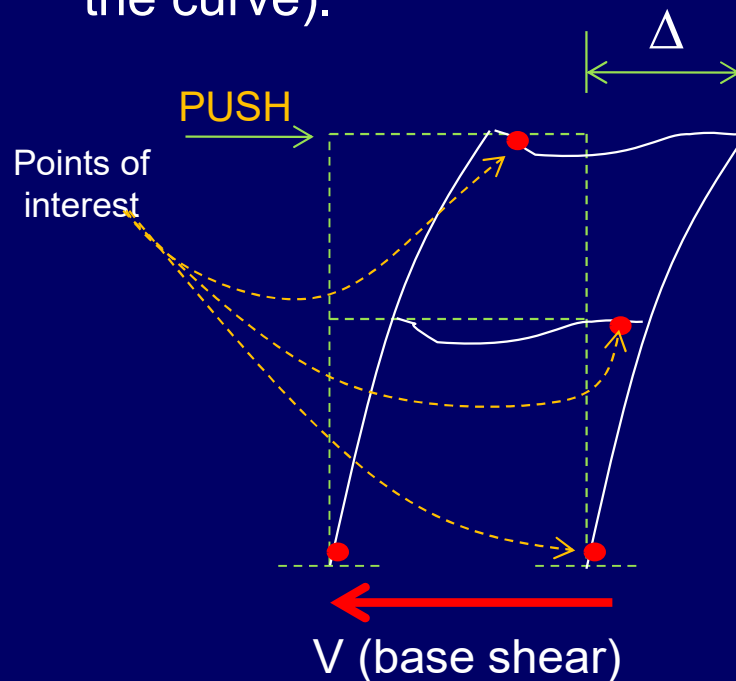




Plastic Analysis

□ Non-Linear Static (Pushover) Analysis

- Therefore, a single chart that shows the performance of the whole structure (or separate charts for all points of interests) is obtained.
- These charts can be used to identify points where the strengthening of structure is required (i.e points that fail or start to fail in the start of the curve).





Modern Analysis Tools

- In earlier times, structural analysis was largely limited to simplified models and manual calculations, focusing primarily on static loads and linear elastic behavior.
- Today, modern tools can perform analysis with full nonlinear material and geometric behavior from the elastic to plastic stages including cracking, buckling, post-buckling, P-Delta effects, contact, complete element separation, collision, and effects of falling debris.



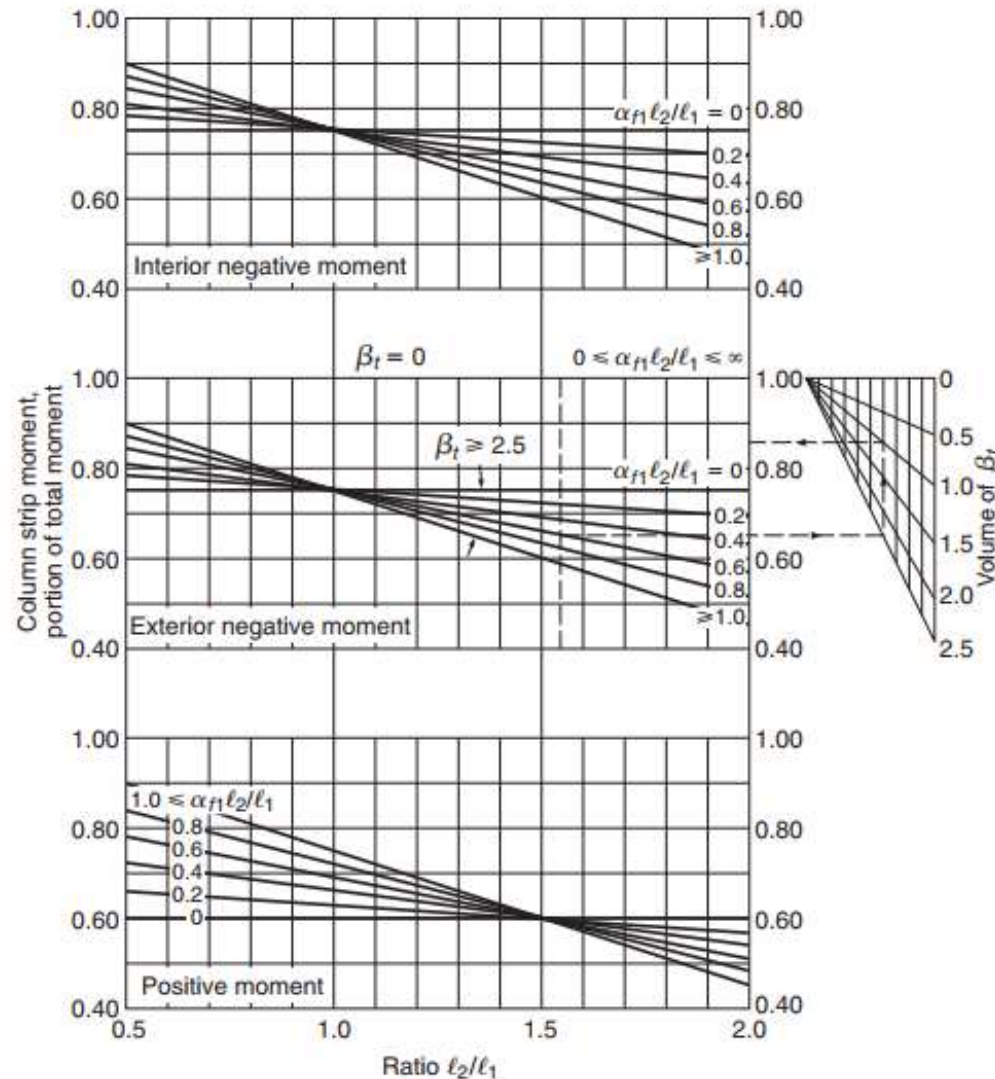
References

- Reinforced Concrete - Mechanics and Design (7th Ed.) by James MacGregor.
- Design of Concrete Structures 14th / 15th edition by Nilson, Darwin and Dolan.
- Building Code Requirements for Structural Concrete (ACI 318-19)
- Portland Cement Association (PCA 2002)



Appendix

GRAPH A.4
Interpolation charts for lateral distribution of slab moments.



Source: Design of Concrete Structures 14th / 15th edition by Nilson, Darwin and Dolan.