Moment Variation Factor (C_m)

The moment variation factor is a factor relating effect of actual moment diagram to that of a constant moment diagram having ordinate equal to the maximum moment of the actual diagram.

C_m factor will be smaller in the cases where end moments are not equal or when the member is bent in reverse curvature.

According to ACI 10.12.3.1, the value of this factor is found as under:

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} \geq 0.4$$

for members without transverse loads between supports

$C_m = 1.0$ for members with transverse loads between supports

where M_1 = magnitude of smaller factored end moment on a compression member

M_2 = magnitude of larger factored end moment on a compression member

and M_1 / M_2 = Positive if the member is bent in single curvature and negative if the member is bent in reverse curvature.

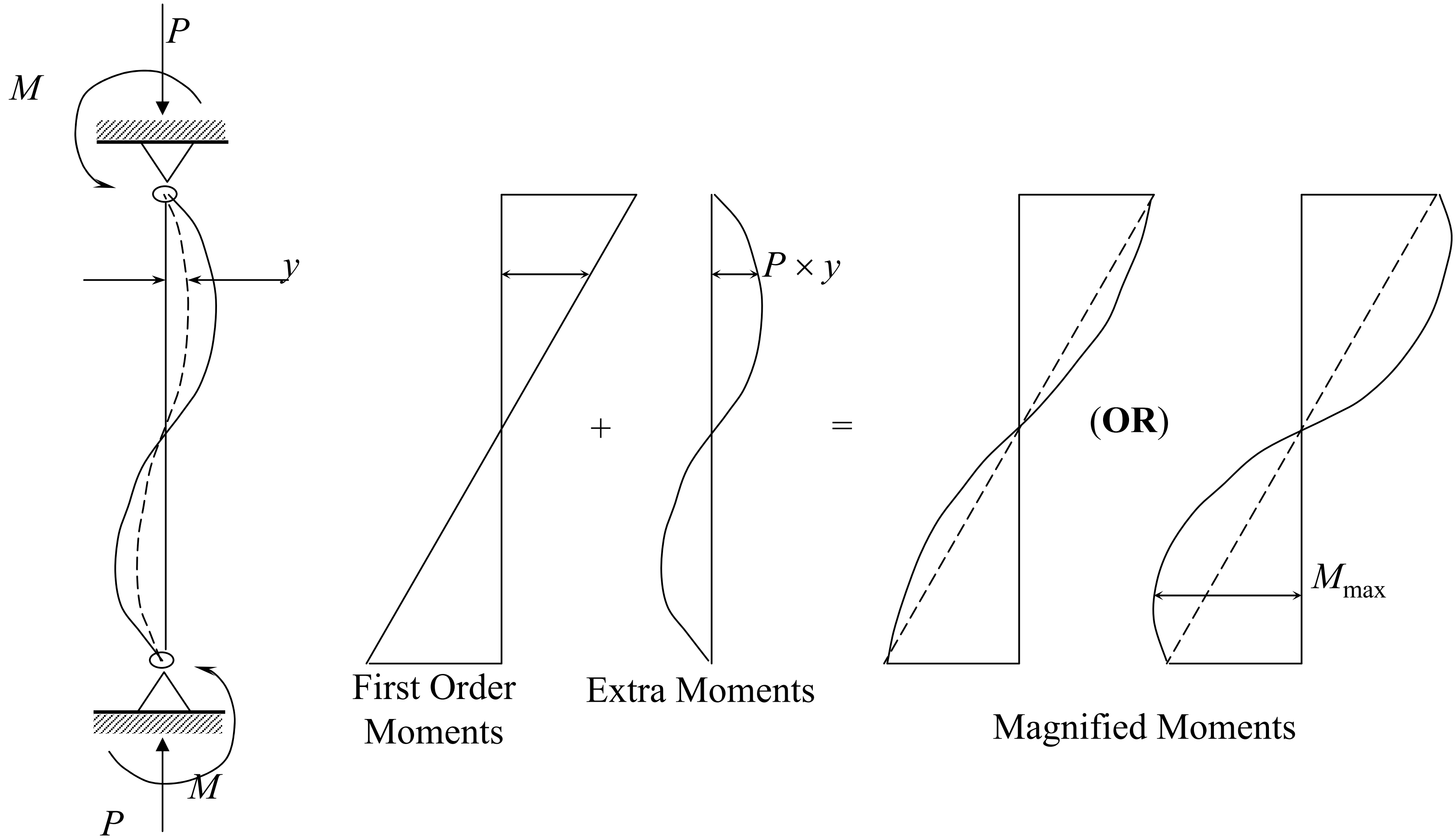


Fig. 14.38. Moment Magnification In Case Of Braced Columns.

Rankine–Jordon Formula

This formula approximately gives the reduced column strength due to slenderness effects and chances of buckling, compared with strength of a short column.

It only tries to predict the column strength for no buckling, inelastic buckling or elastic buckling and does not include the moment magnification effects.

The effect of moment is converted into an equivalent load.

Its application has significantly reduced in design and its use for selection of the trial section in case of steel columns has also been discontinued.

$$\frac{1}{P} = \frac{1}{P_{\text{crushing}}} + \frac{1}{P_c}$$

$$P = \frac{f_{ult} A}{1 + \left(\frac{f_{ult}}{\pi^2 E} \right) (k\ell / r)^2}$$

Reduced Flexural Rigidity To Be Considered For First Order Analysis

According to ACI 10.11.1, reduced moments of inertia and *E*-value, depending on the influence of axial loads, presence of cracked regions and effects of duration of the loads, are to be used for the elastic first order frame analysis.

a)	Modulus of elasticity	$4700\sqrt{f_c'}$ for normal weight concrete
b)	Moment of inertia	
	Beams	$0.35\ I_g$ ($0.70\ I_g$ of ribs for T-beams)
	Columns	$0.70\ I_g$
	Uncracked walls	$0.70\ I_g$
	Cracked walls	$0.35\ I_g$
	Flat plates and flat slabs	$0.25\ I_g$
c)	Area of cross-section	$1.0\ I_g$

(a) When sustained lateral loads like earth or water pressure act.

Case-I: β_d For Non-Sway Frames

The factor β_d is defined as the ratio of the maximum factored sustained dead load to the total factored axial load.

$$\beta_d = \frac{\text{max. factored axial sustained load}}{\text{total factored axial load in the same load combination}}$$

Case-II: β_d For Sway Frames

The factor β_d is defined as the ratio of the maximum factored sustained shear within a story to the total factored shear in that story.

$$\beta_d = \frac{\text{max. factored sustained shear}}{\text{total factored shear in that story}}$$

(b) For stability checks

The factor β_d is defined as the ratio of the maximum factored sustained axial load to the total factored axial load.

$$\beta_d = \frac{\text{max. factored sustained axial load}}{\text{total factored axial load}}$$

Reduced Flexural Rigidity To Be Considered For Determination Of Elastic Critical Buckling Load

The reduced *EI* value for the determination of elastic critical buckling load may be taken by using one of the following equations.

The first expression is more accurate and must be used when the reinforcement details are precisely known.

$$EI = \frac{E_c I_g / 5 + E_s I_{se}}{1 + \beta_d} \text{ (OR) } \frac{0.4 E_c I_g}{1 + \beta_d} \text{ (OR)}$$

$$0.25\, E_c\, I_g \quad (\text{using } \beta_d = 0.6 \text{ according to commentary})$$

I_{se} = moment of inertia of reinforcement alone about the centroidal axis of the member cross-section.

Column Moment to Be Used For Design

In case of braced columns, the maximum magnified moment may be present within the height of the column.

However, in majority of columns in sway frames, the maximum moment occurs at the ends of the columns.

Different categories of moments to be separately treated for the design of columns are as follows:

M_1 = smaller factored end moment acting on compression member.

M_2 = larger factored end moment acting on compression member.

M_{1ns} = factored moment on compression member end at which M_1 acts, due to loads that cause no appreciable side-sway, calculated using a first order elastic frame analysis.

M_{2ns} = factored moment on compression member end at which M_2 acts, due to loads that cause no appreciable side-sway, calculated using a first order elastic frame analysis.

M_{1s} = factored moment on compression member end at which M_1 acts, due to loads that cause appreciable side-sway, calculated using a first order elastic frame analysis.

M_{2s} = factored moment on compression member end at which M_2 acts, due to loads that cause appreciable side-sway, calculated using a first order elastic frame analysis.

The subscript 'ns' stands for moments due to loads that cause no appreciable sway and the subscript 's' stands for moment due to loads that cause appreciable sway.

According to ACI Code Commentary, the columns may sometimes be classified into braced or unbraced category by just inspection, during which the total lateral stiffness of all the columns in a story is compared with that of the bracing elements (like walls or shear trusses) in the story.

The lateral stiffness of a column or bracing element is the shear in the member divided by the relative lateral displacement of the ends of the column due to that shear.

According to previous code commentaries, a column may be classified as braced if the sum of the lateral stiffness for the bracing elements exceeds six times the stiffness for the columns in the direction under consideration.

The term ‘no appreciable sway’ is not defined in the ACI code or commentary.

However, the previous codes considered the columns to have no appreciable sway if the lateral deflection at the factored loads does not exceed 1/1500 of the story height.

Ordinarily a symmetrical frame subjected to gravity loads will have no appreciable shear on analysis and hence these loads may be considered as no-sway loads.

If gravity and lateral loads are applied together on symmetrical frames, story shear is developed but gravity loads may be considered to be of the no-sway type while the lateral loads may be considered sway loads.

Story Stability Index (*Q*)

The stability index is used for the calculation of sway magnification factor and also for checking the stability of the structure under gravity loads.

$$Q = \frac{\sum P_u \Delta_o}{V_u \ell_c}$$

ΣP_u = total vertical load on the story

V_u = story shear

ℓ_c = center-to-center length of column or story height

Δ_o = first order relative deflection between the top and bottom of the story due to V_u

$V_u \ell_c$ = driving moment for side-sway

$\Sigma P_u \Delta_o$ = the extra moment produced due to lateral side-sway

If the stability index is lesser than or equal to 0.05, the sway may be ignored and the story may be considered to be braced.

In computing Q, ΣP_u should correspond to the lateral loading case for which it is the greatest.

This test ($Q \leq 0.05$) would not be suitable if V_u is zero. In other words, this check is not directly applicable for the load combination that only has the gravity loads.

Story Drift Index

It is defined as Δ_o / ℓ_c and its max. value at ultimate loads is to be less than 0.004 for the comfort of the occupants.

Sway Moment Magnification

$$\delta_s = \frac{1}{1-Q} \geq 1.0 \qquad \text{Formula (I)}$$

According to some researchers, a more exact value is:
$\delta_s = 1 / (1 - 1.15Q)$.

$$\delta_s = \frac{1}{1-\dfrac{\sum P_u}{0.75\sum P_c}} \geq 1.0 \qquad \text{Formula (II)}$$

The effect of slenderness on load carrying capacity is shown in Fig. 14.39. Due to increased moment, the slender column line hits the interaction curve at a lower level, indicating lesser strength.

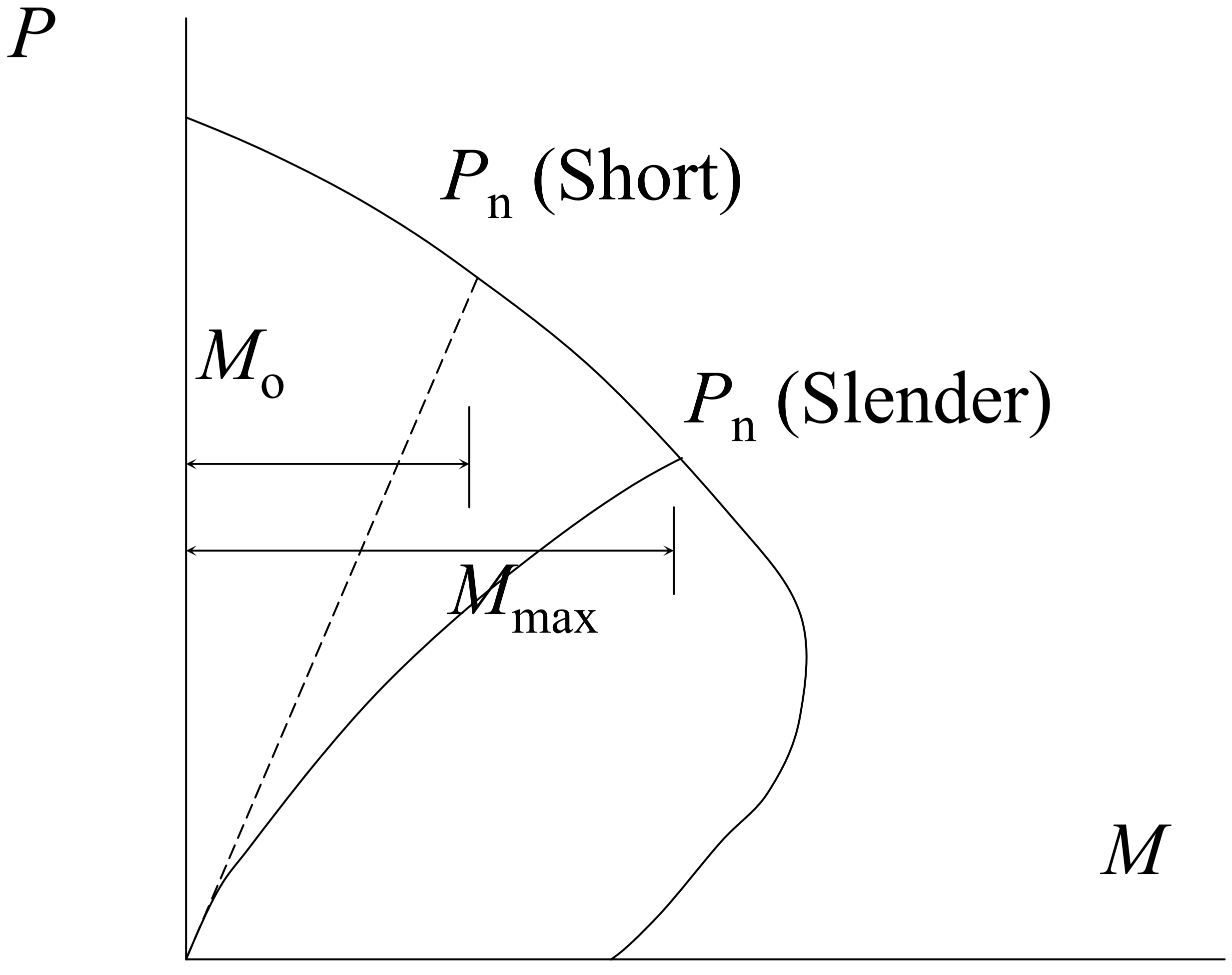


Fig. 14.39. Interaction Curve Showing Reduction In Slender Column Strength.

No–Sway Moment Magnification

$$M_c = \delta_{ns} M_2$$

δ_{ns} = no-sway moment magnification factor

$$= \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} \geq 1.0$$

M_2 = larger factored column end moment

Sway and no-sway magnifications are both to be applied on columns. However, if an individual column satisfies the following condition, only sway magnification is applied on the sway moments and both the magnifications are not required.

$$\ell_u / r \leq \frac{35}{\sqrt{\dfrac{P_u}{f'_c A_g}}}$$

$$\text{or} \qquad P_u \leq \frac{1225 f'_c A_g}{(\ell_u / r)^2}$$

The possibility of side-sway instability of the frame under gravity loads alone must be investigated as under, in addition to load cases with lateral loads:

1. The ratio of second order lateral deflections to first order lateral deflections for 1.4D + 1.7L + Lateral Load should not exceed 2.5.

2. The value of Q-factor computed using ΣP_u for 1.4D + 1.7L combination should not exceed 0.60. V_u and Δ_o may be obtained from any analysis involving lateral loads to get the ratio.

3. When δ_s is computed using ΣP_u and ΣP_c corresponding to the $1.2D + 1.6L$ combination, it must not exceed 2.5.

Second Order Analysis

According to the ACI Code, following factors must be considered during the process of second analysis:

1. Material non-linearity and cracking.
2. Effects of member curvature and lateral drift.
3. Duration of the loads.
4. Creep and shrinkage.
5. Interaction with the supporting foundation.

The results obtained from the second analysis must be within 15% of those reported in literature for indeterminate reinforced concrete structures.

If the cross-sectional dimensions used in analysis are different from the designed dimensions by more than 10%, the analysis must be repeated.

Iterative P-Δ Analysis

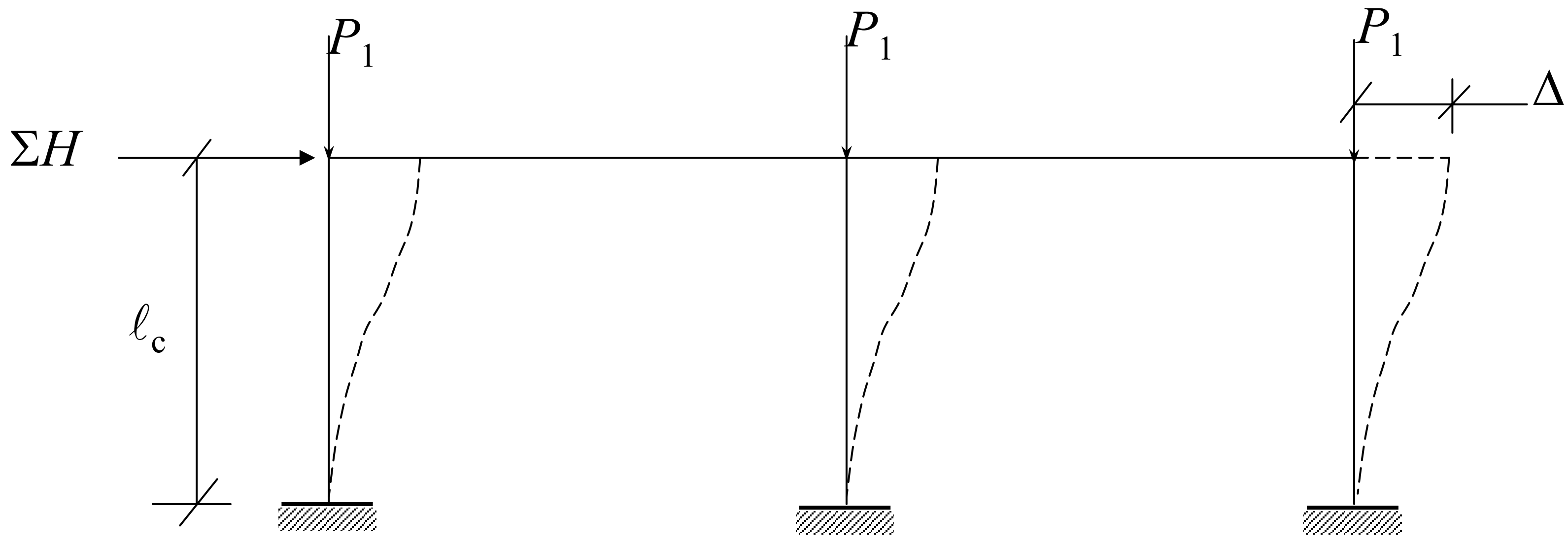


Fig. 14.40. A Typical Single Story Portal Frame Undergoing Side-Sway.

In place of applying the above moments, it is easier to apply equivalent extra horizontal shears, H_1, producing the same moment (Fig. 14.41).

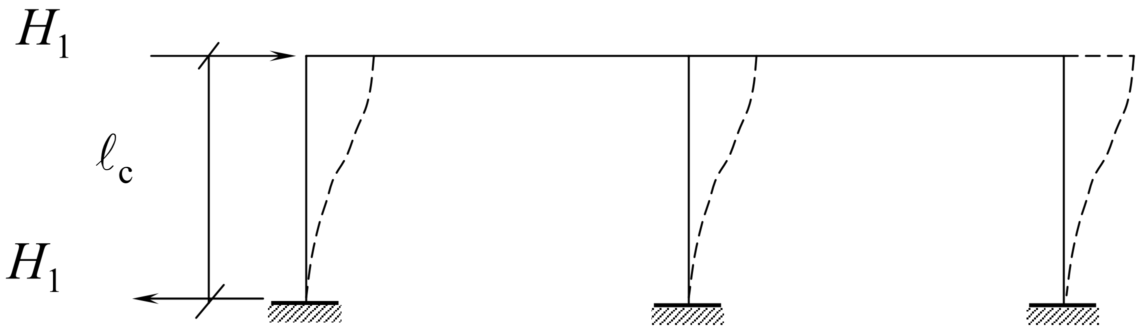


Fig. 14.41.Application of Equivalent Horizontal Shear.

$$H_1 = \frac{(\sum P)\,\Delta}{\ell_c}$$

For multistory frames, we calculate the resultant horizontal shear for the force contributed by the columns above and below a floor, as shown in Fig. 14.42.

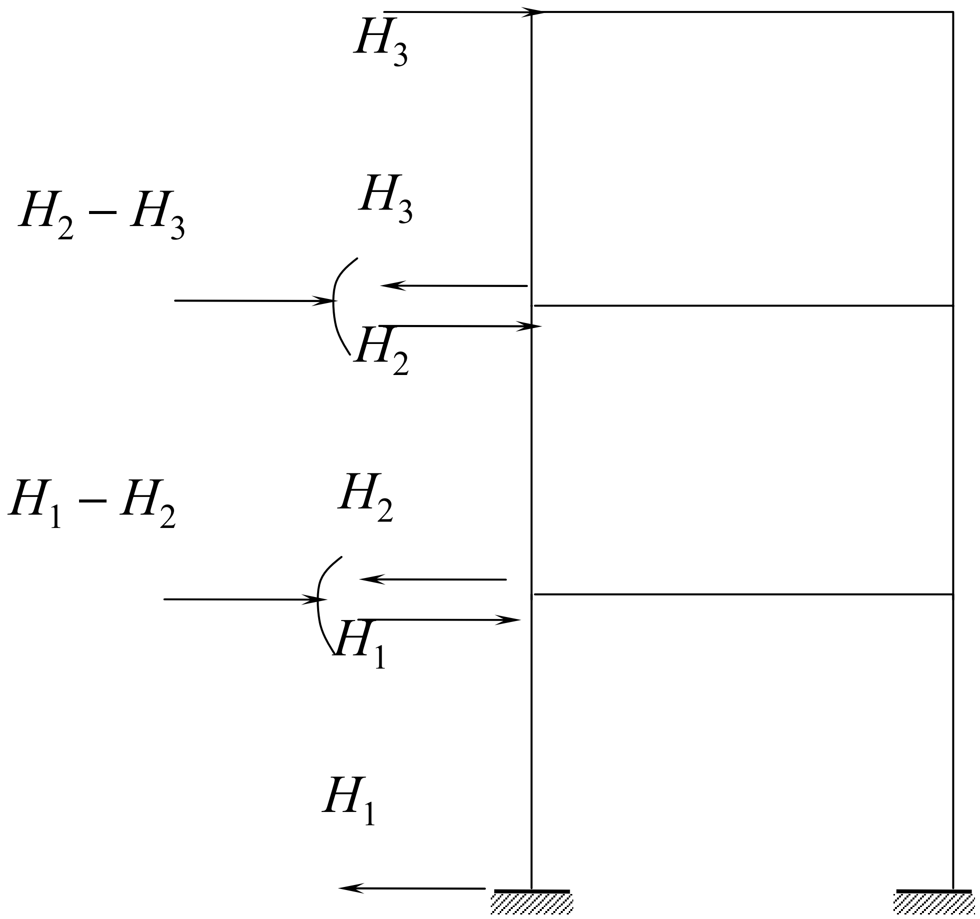


Fig. 14.42.Equivalent Horizontal Shear on Multi-story Frame.

Let j denote the floor number at which shear is to be calculated and i denote the floor number above floor-j.

$$\text{Sway force at } j = \frac{(\sum P_j)\,\Delta_j}{\ell_{c,j}} - \frac{(\sum P_{j+1})\,\Delta_{j+1}}{\ell_{c,j+1}}$$

The sway forces are added to the applied lateral loads at each floor level, and the structure is re-analyzed giving new lateral deflections and larger column moments.

If the deflections increase by more than 5%, new sway forces are computed and the structure is again analyzed using the new sway forces.

This process is continued until convergence is obtained.

One correction to the above procedure is usually applied.

The P-Δ moment diagram for a given column has the same shape as the corresponding elastic curve, while the moment diagram due to applied horizontal loads is a straight line.

As a result, the area of the real P-Δ moment diagram is larger than that of the straight-line diagram.

The increase in deflections varies from zero for a very stiff column with very flexible restraining beams to 22% for a column that is fully fixed against rotation at each end.

A reasonable average value is about 15%.

Hence, the shears to be applied in the above process are multiplied with a flexibility factor of 1.15.

Procedure For Design Of Slender Columns

Step 1: Perform first order frame analysis for gravity loads and lateral loads separately using reduced element stiffness, as defined in ACI 10.11.1.

Step 2: Combine the results for various critical load patterns and combinations. Find the factored loads and moments in the columns. Further, evaluate the lateral story drifts.

Step 3: Select a trial column section to carry the factored axial load P_u and larger end moment M_u assuming short column behavior.

Step 4: Find the unsupported length ℓ_u and the effective length factor k using the approximate values as under:

$r \approx 0.3\,h$ for rectangular columns

$k \approx 1.0$ for stocky columns restrained by flat slabs (no sway)

$k \approx 0.9$ for beam column frames (no sway)

Step 5: Check whether the column is slender by using the following criteria:

Sway Columns:

$k\ell_u / r \geq 22$ means that the column is slender.

No-Sway Columns:

$k\ell_u / r \geq 34 - 12\,(M_1 / M_2)$ means that the column is slender.

Step 6: If the slenderness is tentatively found to be important, refine the calculation of *k* based on the alignment charts.

Flexural rigidity of $0.70 E_c I_g$ for the columns and $0.35\, E_c I_g$ ($0.70\, E_c I_g$ for web portion in T-beams) for the beams may be used in place of more accurate cracked transformed sections.

Step 7: Check to see if the column is braced or unbraced. The following criteria may be used for the classification:

a) Direct bracing may be observed visually.

b) If $Q \leq 0.05$, a story is non-sway.

c) If $\Delta_o > \ell_c / 1500$, the sway is appreciable (not an ACI Code provision).

Step 8: If the column is short, leave the intermediate steps and directly go to step no. 17. Similarly, if the column is slender but braced, leave steps 9 to 12 and go to step no. 13.

Step 9: If $k\ell_u / r \geq 100$, moment magnifier method must not be used. Either increase the column dimensions or perform actual second order analysis and go to step no.16.

Step 10: Compute sway magnification factor, δ_s, as follows:

$$\delta_s = \frac{1}{1-Q} \geq 1.0 \qquad \text{if } Q \geq 0.05$$

If δ_s calculated by the above formula is greater than 1.5, use the formula given below:

$$\delta_s = \frac{1}{1 - \dfrac{\sum P_u}{0.75 \sum P_c}} \geq 1.0$$

If δ_s calculated by the above expression is greater than 2.5, perform second order analysis and go to step no. 18.

Step 11: Calculate the magnified moment as follows:

$$M_1 = M_{1ns} + \delta_s M_{1s}$$
$$M_2 = M_{2ns} + \delta_s M_{2s}$$

Step 12: If $\ell_u / r \leq \dfrac{35}{\sqrt{\dfrac{P_u}{f_c' A_g}}}$ for a particular sway or no-sway column, leave steps 13 to 15 and let $\delta_{ns} = 1.0$.

Step 13: Calculate the equivalent uniform moment factor, C_m, considering the column to be braced.

Step 14: Calculate β_d, EI and P_c for the trial column considering it to be braced.

Step 15: Calculate δ_{ns} individually for each column.

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} \geq 1.0$$

Step 16: Check for minimum moment and eccentricity using ACI 10.12.3.2.

$M_{2,min} = P_u\,(15 + 0.03h)$ about each axis separately.

If $M_{2,min} > M_2$, either $C_m = 1$ or the calculation for C_m should be based on the computed end moments, M_1 and M_2.

Calculate the design moment as follows:

$$M_c = \delta_{ns} M_2$$

Step 17: Check the adequacy of the column to resist axial load P_u and magnified moment M_c using the procedure for a short column. Revise the column section and reinforcement, if necessary.

Step 18: Satisfy the overall stability under gravity loads using ACI 10.13.6 provisions.

The possibility of side-sway instability of the frame under gravity loads alone must be investigated as under, in addition to load cases with lateral loads:

a) The ratio of second order lateral deflections to first order lateral deflections for (1.4D + 1.7L + Lateral Load) should not exceed 2.5.

b) The value of Q-factor computed using ΣP_u for (1.4D + 1.7L) combination should not exceed 0.60. V_u and Δ_o may be obtained from any analysis involving lateral loads to get the ratio.

c) When δ_s is computed using ΣP_u and ΣP_c corresponding to the *(1.2D + 1.6L)* combination, it must be positive and must not exceed 2.5.

Example 14.8: Design the column shown as member no. 30 in the frame given in Fig. 14.44, considering bending only in one direction and the given pattern live loads. All beams have a rib size of 375 × 525 mm, outer columns have size of 375 × 375mm while the inner columns have a size of 450 × 450mm. The outer columns are reinforced by 8–#19 bars. f_c' = 20 MPa, f_y = 420 MPa. Service dead load is 31.0 kN/m, service live load is 12.3 kN/m and the shown lateral loads are due to unfactored wind without directionality factor. Frames are at 5m c/c distance. Ignore the high values of lateral displacements.

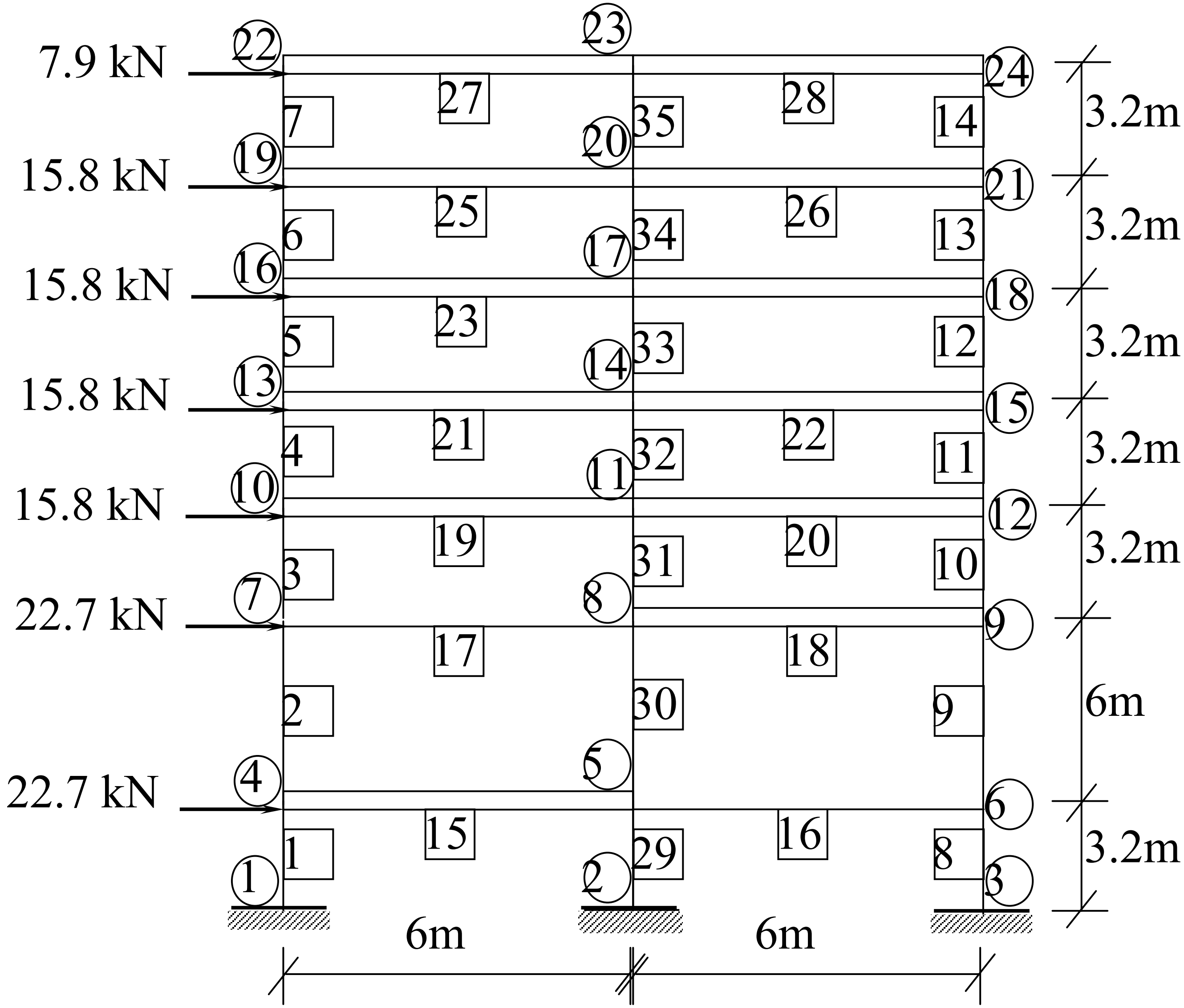


Fig. 14.44. Frame of Example 14.7.

For the unfactored forces, considering a lower value of β_d equal to 0.25, the results of the analysis are as under:

Member 30

Type Of Force	Wind	Dead	Live
Axial force (kN)	0.046	1180	429
Shear force (kN)	47.4	0	0.10
Lower end moment (kN-m)	+141	0	+4.9
Upper end moment (kN-m)	–143	0	+5.5
Type of bending	Reverse	–	Single
Lateral deflection within member (mm)	2.12	0	0.583

Member 2

Type Of Force	Wind	Dead	Live
Axial force (kN)	–76.3	525	173
Shear force (kN)	23.2	7.67	1.44
Lower end moment (kN-m)	+69.2	–23.3	–6.96
Upper end moment (kN-m)	–70.2	+22.7	+1.68
Type of bending	Reverse	Reverse	Reverse

Member 9

Type Of Force	Wind	Dead	Live
Axial force (kN)	76.3	525	210
Shear force (kN)	23.2	7.67	1.54
Lower end moment (kN-m)	+69.2	+23.3	+2.27
Upper end moment (kN-m)	–70.2	–22.7	–6.97
Type of bending	Reverse	Reverse	Reverse

Joint 5

Horizontal displacement = 7.20 mm
(due to horizontal loads)

Joint 8

Horizontal displacement = 40.00 mm
(due to horizontal loads)

Solution:

Step 1: The first order frame analysis for gravity loads and lateral loads has already been carried out.

The following simplified load combinations will be used here:

1. $U = 1.2\,D + 1.6\,L + 1.6\,L_r$
2. $U = 1.2\,D + 0.8\,W$
3. $U = 1.2\,D + 1.3\,W + 1.0\,L + 1.0\,L_r$
   (no directionality factor for wind)

DESIGN FOR GRAVITY LOAD COMBINATION

Step 2: $P_u = 1.2 \times 1180 + 1.6 \times 429$

$= 2103$ kN

$M_{ns1} = 1.6 \times 4.9 = 7.84$ kN-m

$M_{ns2} = 1.6 \times 5.5 = 8.80$ kN-m (single curvature)

$M_{s1} = 0$

$M_{s2} = 0$

Step 3: Trial size: 450×450mm

Step 4:

$$\ell_u = 6.0 - 0.525 = 5.475\text{m}$$
$$k = 0.9$$
$$r \approx 0.3\,h = 0.3 \times 450 = 135\text{mm}$$

Step 5: $k\ell_u / r = 0.9 \times 5475 / 135 = 36.5$

$$34 - 12\,(M_1 / M_2) = 34 - 12 \times (7.84 / 8.80) = 23.3$$

$k\ell_u / r \geq 22$ means that the column is slender for sway conditions.

$k\ell_u / r \geq 34 - 12\,(M_1 / M_2)$ means that the column is slender for no-sway conditions.

Step 6:

$$\psi_A = \psi_B = \frac{\dfrac{E \times 0.7 \times 450 \times 450^3}{12 \times 6000} + \dfrac{E \times 0.7 \times 450 \times 450^3}{12 \times 3200}}{2 \times \dfrac{E \times 0.7 \times 375 \times 525^3}{12 \times 6000}} = 1.09$$

k for no-sway condition $= 0.78$

k for sway condition $= 1.33$

Step 7: For gravity loads, the column is no-sway.

Step 8: Go to step no. 13.

Step 13:

$$C_m = 0.6 + 0.4\,(M_1 / M_2) \geq 0.4$$

$$= 0.6 + 0.4\,(7.84 / 8.80)$$

$$= 0.96$$

Step 14: $\beta_{\mathrm{d}} = \dfrac{\text{factored dead load}}{\text{factored total load}}$

$$= \frac{1.2\times 1180}{1.2\times 1180 + 1.6\times 429} = 0.674$$

$$E_{\mathrm{c}} = 4700\sqrt{f_c'} = 4700\ \sqrt{20} = 21{,}019\ \text{MPa}$$

$$EI = \frac{0.4E_c I_g}{1+\beta_d}$$

$$= \frac{0.4\times 21{,}019\times 450^4/12}{1+0.674} = 17{,}163\times 10^9\ \text{N-mm}^2$$

$$P_{\mathrm{c}} = \frac{\pi^2 EI}{(k\ell_u)^2} = \frac{\pi^2\times 17{,}163\times 10^9}{(0.78\times 5475)^2}\times\frac{1}{1000} = 9{,}288\ \text{kN}$$

Step 15: $\delta_{ns} = \dfrac{C_m}{1 - \dfrac{P_u}{0.75 P_c}}$

$$= \frac{0.96}{1 - \dfrac{2103}{0.75 \times 9288}} = 1.375$$

Step 16: $M_{2,\text{min}} = P_u\,(15 + 0.03h)$

$$= 2103\,(15 + 0.03 \times 350) / 1000 = 59.94 \text{ kN-m}$$

$M_2 = 59.94$ kN-m and $C_m = 1.0$

$$M_c = \delta_{ns}\, M_2$$
$$= 1.375 \times 59.94 = 82.41 \text{ kN-m}$$

Step 17: $P_u = 2103$ kN

$M_u = 82.41$ kN-m

$$\frac{P_u}{A_g} = \frac{2103 \times 1000}{450 \times 450} = 10.39 \text{ MPa}$$

$$\frac{M_u}{A_g h} = \frac{82.41 \times 10^6}{450^3} = 0.91 \text{ MPa}$$

$$\gamma = \frac{450 - 20 - 25 - 2 \times 40}{450} = 0.72 \approx 0.75$$

$\rho < 0.01$ (Use $\rho = 0.01$)

Step 18: Will be carried out after checking for all other combinations.

DESIGN FOR GRAVITY AND WIND LOAD COMBINATIONS

$$U = 1.2\,D + 1.3\,W + 1.0\,L + 1.0\,L_r$$
$$U = 1.2\,D + 0.8\,W$$

Step 2:

$$P_u = 1.2 \times 1180 + 1.0 \times 429 = 1845 \text{ kN}$$
$$M_{ns1} = 1.0 \times 4.9 = 4.90 \text{ kN-m}$$
$$M_{ns2} = 1.0 \times 5.5 = 5.50 \text{ kN-m} \quad \text{(single curvature)}$$
$$M_{s1} = 1.3 \times 141.0 = 183.3 \text{ kN-m}$$
$$M_{s2} = 1.3 \times 143.0 = 185.9 \text{ kN-m}$$

The load and moment are both significantly lesser for $U = 1.2\ D + 0.8\ W$ combination than this combination and hence needs not to be considered here.

Step 3: Trial size: 450×450mm

Steps 4-6: The result for column number 30 is the same as in the first combination.

For columns 2 and 9, acting as sway columns, we get the following results.

$$\ell_u = 6.0 - 0.525 = 5.475\text{m}$$

$$\psi_A = \psi_B = \frac{\dfrac{E\times 0.7\times 375\times 375^3}{12\times 6000} + \dfrac{E\times 0.7\times 375\times 375^3}{12\times 3200}}{2\times \dfrac{E\times 0.7\times 375\times 525^3}{12\times 6000}} = 1.05$$

k for sway condition = 1.32

Step 7: $\Sigma P_u = 1845 + 1.2 \times 2 \times 525 + 1.0 \times 173 + 1.0 \times 210 = 3488$ kN

$\Delta_o = 1.3 \times (40.00 - 7.20) = 42.64$ mm

$V_u = 1.3 \times 93.8 = 121.94$ kN

$\ell_c = 6000$ mm

$$Q = \frac{\left(\sum P_u\right)\Delta_o}{V_u \ell_c} = \frac{3488\times 42.64}{121.94\times 6000} = 0.203$$

> 0.05 $\therefore$ **Sway is to be considered.**

Steps 8-10:

$$\delta_s = \frac{1}{1-1.15Q} = 1.30$$

β_d = 0 for wind load combination.

E_c = 21,019 MPa (as before)

To check the second formula, calculate P_c for all the three columns included in the sway.

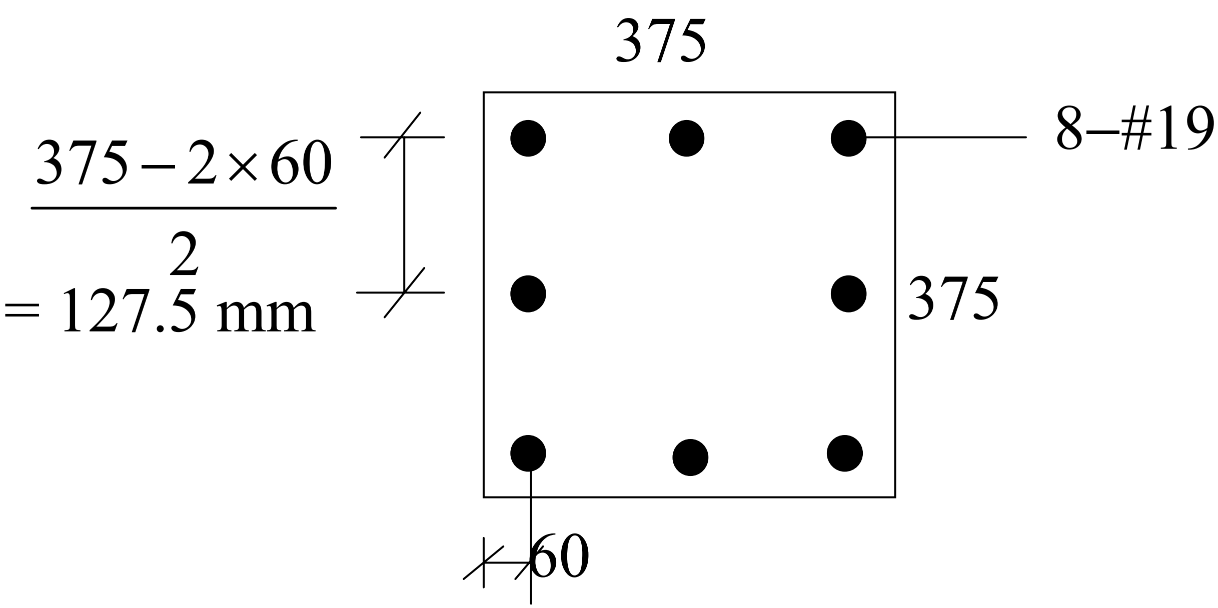
Interior Column $EI = \dfrac{0.4 E_c I_g}{1+\beta_d}$

$$= \frac{0.4 \times 21{,}019 \times 450^4 / 12}{1+0} = 28{,}730 \times 10^9 \text{ N-mm}^2$$

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} = \frac{\pi^2 \times 28{,}730 \times 10^9}{(1.33 \times 5475)^2} \times \frac{1}{1000} = 5{,}347 \text{ kN}$$

Exterior Columns

For the exterior columns, the steel reinforcement is known and more exact formula for *EI* may be used.



Referring to the previous figure, we get,

$$E_s I_{se} = 200{,}000 \times 6 \times 284 \times 127.5^2$$
$$= 5540 \times 10^9 \text{ N-mm}^2$$

$$EI = \frac{0.2E_c I_g + E_s I_{se}}{1+\beta_d}$$

$$= \frac{0.2 \times 21{,}019 \times 375^4/12 + 5540 \times 10^9}{1+0} = 12{,}468 \times 10^9 \text{ N-mm}^2$$

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} = \frac{\pi^2 \times 12{,}468 \times 10^9}{(1.32 \times 5475)^2} \times \frac{1}{1000} = 2{,}356 \text{ kN}$$

$$\Sigma P_c = 5347 + 2 \times 2356 = 10{,}059 \text{ kN}$$

$$\delta_s = \frac{1}{1 - \frac{\sum P_u}{0.75 \sum P_c}} \geq 1.00$$

$$= \frac{1}{1 - \frac{3488}{0.75 \times 10{,}059}} = 1.86$$

(significantly higher than the first formula)

< 2.5 (OK)

Step 11: Calculate the magnified moment as follows:

$$M_1 = M_{1ns} + \delta_s M_{1s}$$

$$= 4.9 + 1.30 \times 183.3 = 243.2 \; kN\text{-}m$$

$$M_2 = M_{2ns} + \delta_s M_{2s}$$

$$= 5.5 + 1.30 \times 185.9 \text{ (wind can have both directions)}$$

$$= 247.2\ kN\text{-}m > M_{2,min} \quad \text{(OK)}$$

Step 12: $\ell_u / r = \dfrac{5475}{0.3 \times 450} = 40.6$

$$\frac{35}{\sqrt{\dfrac{P_u}{f'_c A_g}}} = \frac{35}{\sqrt{\dfrac{1845 \times 1000}{20 \times 450^2}}} = 51.9$$

$$\ell_u / r \leq \frac{35}{\sqrt{\dfrac{P_u}{f'_c A_g}}}$$

$\therefore$ No-sway magnification is not required together.

$\delta_{ns} = 1.0.$

Step 16: $M_c = \delta_{ns} M_2$

$= 247.2\ kN\text{-}m$

Step 17: $P_u = 1845\ \text{kN}$

$M_u = 247.2\ \text{kN-m}$

$$\frac{P_u}{A_g} = \frac{1845 \times 1000}{450 \times 450} = 9.11\ \text{MPa}$$

$$\frac{M_u}{A_g h} = \frac{247.2 \times 10^6}{450^3} = 2.71\ \text{MPa}$$

$$\gamma = \frac{450 - 20 - 25 - 2 \times 40}{450} = 0.72 \approx 0.75$$

$$\rho \cong 0.0235$$

$$A_{st} = \rho A_g = 0.0235 \times 450^2 = 4759 \; mm^2$$

(Use 8–#29)

The resulting steel is shown in Fig. 14.46. The ties may be designed as usual.

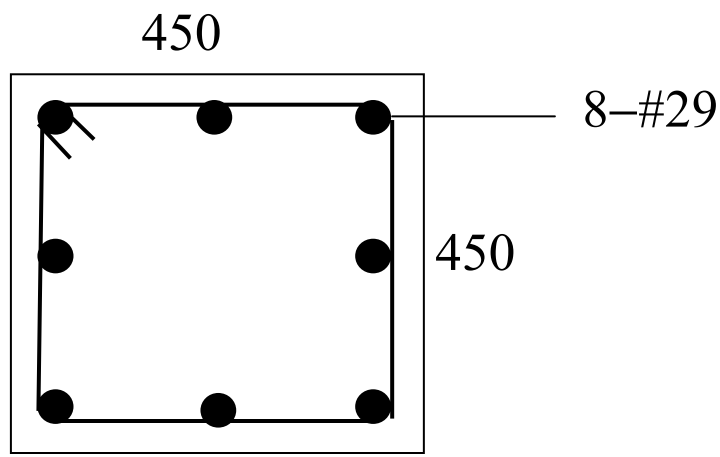


Fig. 14.46. Reinforcement Details For Column of Example 14.8.

$$\text{Clear gap between bars} = \frac{450 - 2 \times 40 - 2 \times 10 - 3 \times 29}{2}$$

$$= 132 \text{ mm} < 150 \text{ mm}$$

Step 18: $\Sigma P_u = 1.4(1180 + 52 + 525) + 1.7(429 + 173 + 210) = 4502.4$ kN

For $\Delta_o = 32.80$ mm, $V_u = 121.94$ kN

$\ell_c = 6000$ mm

$$Q = \frac{\left(\sum P_u\right)\Delta_o}{V_u \ell_c} = \frac{4502.4 \times 32.8}{121.94 \times 6000}$$

$$= 0.202 < 0.60$$

Hence the frame is stable for the gravity loads.