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LIMITING SWAY FOR UNBRACED STEEL WITH COLUMN BENDING ON MINOR AXIS

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Abstract. The proposed method is presented for the design of multi-storey steel frames to limiting values of horizontal sway deflection. The frame is divided into statically determinate sub-frames by assuming points of contraflexture. Allowance for steelwork costs is then used, together with slope-deflection analysis, to derive equations for optimum design. This method is suitable for hand calculation. A series of rigid jointed unbraced steel frames was studied with column bending on minor axis. The accuracy of the design equation was found to be good by comparison with linear elastic computer analysis. Ultimate limit state and serviceability limit state were checked for the frames.

1.0 INTRODUCTION

For a frame to be classified as braced, the bracing system provided should be at least five times stiffer than the stiffness of the frame itself[1]. In Eurocode 3 (EC3) the frame is classified as braced when the bracing system reduces the horizontal displacement by at least 80%[2]. A steel frame which does not satisfy the criterion for a braced frame is classified as unbraced. Although an unbraced frame may be treated as a three-dimensional entity, it is usually idealised as a series of two-dimensional frames that resist loading (horizontal and vertical) in each plane primarily by bending action. In practice it is often arranged go that the frames are braced against horizontal displacements in one direction to simplify the behaviour and to avoid as far as possible bending action about the minor axes of the column sections. Unbraced frames may also be "sway" frames in which second-order effects need to be accounted for. The "P-(" effect (Figure 1) changes the distribution of internal moments and forces and results in a lowering of the load level at collapse. In unbraced frames, it is important to note that limitations of sway under service loading need to be satisfied, as well as the ultimate strength. These concern both the interstorey drifts and the structure as a whole. For example, the limits recommended by Eurocode 3[2] are h/300 for the interstorey drifts but $h_o/500$ for the structure as a whole, where h is the storey height and h_o is the overall height of the building.

2.0 DESIGN APPROACH FOR UNBRACED FRAMES

As loads in unbraced frames are to be resisted by bending action of the frame's members without the need of a bracing system, the most common design approach is to use rigid joints. For unbraced frames, the main design consideration is to limit sway, to avoid unacceptable deflections under service load and to avoid premature collapse by frame instability[3]. This can be done by using stiff joints and appropriate member sections. Fully welded connections are the closest approach to a truly-

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rigid joint but result in expensive fabrication costs. An extended end plate, welded to the beam and bolted to the column, provides a more reasonable form, but both welded and bolted joints are likely to require stiffening in the tension and compression zones of the column webs, and possibly in shear (see Figure 2). This may be to increase moment capacity or to reduce the sway, but this increases the fabrication cost even more. Generally, unbraced frames designed with rigid joints are not commonly adopted unless to meet an architectural requirement that no bracing system be allowed.

3.0 RANGE OF APPLICATIONS

The range of the study is for two and four bays with heights of two to eight storeys. In recognition of the unlikelihood of the frame consisting of only one longitudinal bay, the minimum number of bays in the minor axis framing was taken as two (see Figure 3). Each longitudinal bay was assumed to be 6m in length. The maximum number of longitudinal bays was taken in this study as six. The following configurations of minor-axis framing were therefore investigated:

- two-storey, two-bay
- four-storey, two-bay
- four-storey, four-bay
- four-storey, six-bay
- eight-storey, two-bay.

The limitations on frame dimensions conformed to those specified in the existing guide[4] for "wind-moment" design. In view of possible difficulty in ensuring adequate stability and stiffness, the study assumed S275 steel, rather than the higher grade material used in some of the earlier studies[5].

The arrangement of floor grids was shown in Figure 4. The floor units were assumed to span 6 m between the major-axis frames; this results in the minor-axis beams being free of significant gravity forces, the main loading being wind-moments.

4.0 DETERMINATION OF WIND FORCES

For serviceability limit states, loads were taken as unfactored. Deflection limits for a building with more than one storey are recommended by BS 5950 to be less than 1/300th of the height of the storey under consideration. Basic wind speeds were taken as the three-second gust speed estimated to be exceeded on average once in 50 years. Wind forces were calculated in accordance with CP3: Chapter V: Part 2[6], the code in use in practice at the time of the study. Wind forces were considered as horizontal point loads acting on the windward external columns at each floor level. In design, account was taken of the compressive axial forces in the leeward columns, contributed by the horizontal wind. No account was taken of wind uplift on the roof, as this would relieve the compressive axial forces in the columns.

5.0 LOAD COMBINATIONS

For serviceability limit states, loads were taken as unfactored. When considering dead load plus imposed load and wind load only 80% of the imposed load and wind load need to be considered[7]. Frames will be analysed under three load combinations as follows:-

- 1.0 Dead load plus 1.0 imposed load plus unfactored notional force
- 1.0 Dead load plus 0.8 imposed load plus 0.8 wind load
- 1.0 Dead load plus 1.0 wind load.

Deflection limits for a building with more than one storey are recommended by BS 5950 to be less than 1/300th of the height of the storey under consideration.

For ultimate limit states, loads were be taken as factored. Frames were analysed under three load combinations as follows:-

- 1.4 Dead load plus 1.6 imposed load plus factored notional force
- 1.2 Dead load plus 1.2 imposed load plus 1.2 wind load
- 1.4 Dead load plus 1.4 wind load.

6.0 DESIGN METHODOLOGY

Initially the frame was designed using a software written by Reading[8] and modified by Brown[5] which was used to design the column sections for frame bending about major axis. For minor axis design, the software was further modified by Tahir[9]. The rules to proportion the individual members to limit the sway are described below and further explained in this paper.

Two procedures are adopted to stiffen the frame:

1. sections are increased to limit the sway index to 1/300 under serviceability wind forces;
2. further increases may be made in beam sections to provide improved restraint to the columns.

7.0 DESIGN TO LIMITING SWAY DEFLECTIONS

As a result of its simplicity, the "wind-moment" approach is attractive to those who wish to continue to design by manual calculation. For rigid-jointed unbraced frames, hand methods are available to determine sway deflection[10]. One such method, is proposed to generate designs to specified limits on inter-storey sway[11]. An element of optimisation is included, which permits account to be taken of the differing efficiencies of various section shapes in providing flexural rigidity. This method has been used by Tahir[9] in designing stiffer minor-axis framing, those sections already chosen by the "wind-moment" approach being taken as lower bounds on sizes. The formulae used to limiting sway deflections are presented later in this paper.

Comparison of the more efficient Universal Beams in major-axis bending with the minor axis properties of Universal Columns (see Figure 5) showed that the former are approximately five times more efficient in providing flexural rigidity[9]. Account was taken of this difference when using the formula[9]. Their factor k3 accounts for such differences; the value taken was 4.8 (see Figure 5). The effect of this factor is to encourage the use of deeper beams to provide overall sway stiffness.

If however the formulae predicted that the optimum design required smaller columns than the "wind-moment" calculations allowed, the formulae were then used in an alternative mode. This enabled beam sections to be selected to meet the deflection limit, taking account of the rigidity of the already-chosen columns. To avoid an undue number of splices, column sections were only changed every two storeys for two and four storey frames, and two or three storeys for eight storeys frames.

For frames with the grid of Figure 4, the formulae were used in conjunction with a deflection limit of height/300 and the full unfactored wind load. The formulae are based on an assumed first-order elastic response. In the interests of research, the resulting designs were also subjected to computer analysis[12]. This was partly to check that the formulae had generated reasonably stiff designs, but it also permitted account to be taken of second-order effects. When these caused the limiting index of 1/300 to be exceeded, beam sections were further increased until the second-order (but still rigid) analysis showed this limit had been satisfied.

7.1 Design equations used

This method is presented for the design of multi-storey steel frames to limiting values of horizontal sway deflection[11]. The frame is divided into statically determinate sub-frames by assuming points of contraflexure. Allowance for steelwork costs are then used, together with slope-deflection analysis, to derive equations for optimum design. This method is suitable for hand calculation. The accuracy of the design equations was found to be good by comparison with linear elastic computer analysis.

7.2 Top storey

The subassemblage shown in Figure 6 was used to derive the design equations as stated below:

$$I_{1,2} = \frac{P_1 h_1 I_{2,2}}{P_1 h_1 + P_2 h_2} \tag{1}$$

$$I_{2,2} = \frac{(P_1 h_1 + P_2 h_2) h_1 L_2^2 I_{3,2}}{24 E \Delta B I_{3,2} - P_1 h_1^3 (L_1 + L_2)} \tag{2}$$

$$I_{3,2} = \frac{\left[P_1 h_1^3 (L_1 + L_2) + h_1 L_2 \sqrt{\dfrac{P_1 h_1 (L_1 + L_2)\left[2W_1 P_1 h_1 + W_2 (P_1 h_1 + P_2 h_2)\right]}{W_3}} \right]}{24 E \Delta B} \tag{3}$$

$$I'_{3,2} = \frac{I_{3,2}}{2} \tag{4}$$

where

P_1 is the total horizontal shear in the top storey columns,
P_2 is that in the storey below,
L_1 and L_2 are the span of the beams,
h_1, and h_2 are the height of the columns,
Δ equal the allowable sway over the storey height h_2,
B is the total width of the frame,
$I_{1,2}$ is the inertia of the upper beams in the storey,
$I_{2,2}$ is the inertia of the lower beams in the storey,
$I_{3,2}$ is the inertia of the internal designed column in the storey,
$I'_{3,2}$ is the inertia of the external designed column in the storey.

7.3 Intermediate storeys

The subassemblage shown in Figure 7 was used to derive the design equations stated below. It is assumed that the total horizontal shear is divided between the bays in proportion to the widths.

$$I_{1,2} = \frac{(P_1 h_1 + P_2 h_2) I_{2,2}}{P_2 h_2 + P_3 h_3} \tag{5}$$

$$I_{2,2} = \frac{(P_2 h_2 + P_3 h_3) h_2 L_2^2 I_{3,2}}{24 E \Delta B I_{3,2} - P_2 h_2^3 (L_1 + L_2)} \tag{6}$$

$$I_{3,2} = \frac{\left[P_2 h_2^3 (L_1 + L_2) + h_2 L_2 \sqrt{\dfrac{P_2 h_2 (L_1 + L_2)\left[W_1 (P_1 h_1 + P_2 h_2) + W_2 (P_2 h_2 + P_3 h_3)\right]}{W_3}} \right]}{24 E \Delta B} \tag{7}$$

$$I'_{3,2} = \frac{I_{3,2}}{2} \tag{8}$$

where

P_1 and P_3 are the total horizontal shear in the columns of the storeys immediately above and below,
P_2 is the total horizontal shear in all the columns of the storey being designed,
L_1 and L_2 are the span of the beams,
h_1, h_2, and h_3 are the height of the columns,
Δ equal the allowable sway over the storey height h_2,
B is the total width of the frame,
$I_{1,2}$ is the inertia of the upper beams in the storey,
$I_{2,2}$ is the inertia of the lower beams in the storey,
$I_{3,2}$ is the inertia of the internal designed column in the storey,
$I'_{3,2}$ is the inertia of the external designed column in the storey.

7.4 Bottom two storeys of a fixed base frame

The subassemblage shown in Figure 8 was used to derive the design equations stated below. The fixity of the base attracts more moment than the upper column. As a result, the design may be governed by the permissible deflection (2 of the upper storey. The effect of fixed base is more pronounced when $h_2 = h_3$, and the bottom storey column inertia ($I_{4,2}$) than has to be made equal to $I_{3,2}$) to avoid reverse taper.

$$I_{3,2} = \frac{(P_2h_2 + L_2Y)h_2^2(L_1 + L_2)}{24E\Delta_2B} \tag{9}$$

where

$$Y = \sqrt{\frac{3P_2h_3\left[W_1(P_1h_1 + P_2h_2) + 2W_2(P_2h_2 + P_3h_3)\right]}{\left(3W_3h_3(L_1 + L_2)(h_2 + h_3) - W_2L_2^2\right)}} \tag{10}$$

$$I_{1,2} = \frac{(P_1h_1 + P_2h_2)h_2L_2^2I_{3,2}}{\left(24E\Delta_2BI_{3,2} - P_2h_2^3(L_1 + L_2)\right)} \tag{11}$$

$$I_{2,2} = \frac{(P_2h_2 + P_3h_3)h_2L_2^2I_{3,2}}{\left(24E\Delta_2BI_{3,2} - P_2h_2^3(L_1 + L_2)\right)} - \frac{L_2^2I_{3,2}}{\left(6h_3(L_1 + L_2)\right)} \tag{12}$$

$$W_1 = \frac{k_{2,1}L_1^3 + k_{2,2}L_2^3 \ldots\ldots + k_{1,m}L_m^3}{2L_2^2} \tag{13}$$

$$W_2 = \frac{k_{2,1}L_1^3 + k_{2,2}L_2^3 \ldots\ldots + k_{2,m}L_m^3}{2L_2^2} \tag{14}$$

$$W_3 = \frac{\left[(k_{3,1} + k_{3,2})L_1 + (k_{3,2} + k_{3,3})L_2 + \ldots\ldots\ldots(k_{3,m} + k_{3,m+1})L_m\right]}{(L_1 + L_2)} \tag{15}$$

where

P_1 and P_3 are the total horizontal shear in the columns of the storeys immediately above and below,
P_2 is the total horizontal shear in all the columns of the storey being designed,
L_1 and L_2 are the spans of the beams,
W_1, W_2, and W_3 are the cost factors for a member of inertia $I_{i,j}$,
h_1, h_2, and h_3 are the heights of the columns,

Δ equals the allowable sway over the storey height h_2,
B is the total width of the frame,
E is Young's Modulus,
$I_{1,2}$ is the inertia of the upper beams in the storey,
$I_{2,2}$ is the inertia of the lower beams in the storey,
$I_{3,2}$ is the inertia of the internal designed column in the storey,
$I'_{3,2}$ is the inertia of the external designed column in the storey.

7.5 Parametric study

The frame arrangements studied and the dimensions and loading, are listed in Tables 1 and 2. Table 1 concern minimum wind combined with maximum gravity load and Table 2, the reverse. The wind-moment designs ('Section Designation I') are given in Table 3 and Table 6. To improve stiffness and satisfy the deflection limits, frames are designed with the proposed limiting sway formulae, and are listed in Table 4 and 7 denoted as 'Section Designation II'. Connection requirements are tabulated in Table 5 and Table 8. Table 3, 4 and 5 concern minimum wind combined with maximum gravity load and Table 6, 7, and 8, the reverse. The load-deflection (sway) behaviour for each of the frame up to the point of collapse was examined for second-order analysis at ULS.

7.6 Assessment of results

To justify the design recommendations which include proposed rule to limit sway, the frames were subject to second-order analysis accounting for the rigid nature of the joints. Software[12] was used to carry out this analysis. Generally, when the overall sway deflections were calculated, both first-order and second-order values were obtained. The resistance moment of the column sections was taken as the plastic moment about the minor axis, reduced to take account of co-existent axial force, in accordance with the usual formulae[13] given in British tables for steel sections. It should be noted that because of the shape factor about the minor axis, the attainment of the plastic moment at the end of a column will be accompanied by plastic zones of significant length away from the theoretical plastic hinge. The computer analysis does not account for the loss of stiffness resulting from partially-plastic regions. This does not invalidate the conclusions from the study because subsequent checks were made on the local behaviour of each column length as described elsewhere[9].

The results are summarised in Tables 9 and Table 10 . For frames with maximum wind combined with minimum gravity load, the overall sway slightly exceeded the index of 1/300. These frames were improved by slightly increase the second moment area of the beams and the results were shown elsewhere[9].

8.0 CONCLUSIONS

Despite the assumption of relatively stiff minor-axis connections in which the joints were considered as rigid, a straightforward extension of the previous rules for wind moment design[4] does not always result in frames of adequate overall stability. This is particularly true of frames in which floor units span between major-axis beams. In addition, the neglect of second-order effects results in the likelihood that the moment resistance of the joints will be reached below the design load level, causing a major deterioration of stiffness.

Further design rules have been developed by recognising the need to limit sway under service loading. However, for minor axis framing which extends over several bays, even these rules do not ensure adequate ultimate stability if the wind forces are low. Additional rules, relating to the minimum beam stiffness to the stiffness of the columns, have been proposed[9]. The resulting designs examined so far have adequate stability.

In view in the scope of the studies, and the problems they reveal in providing a frame of adequate resistance, it is concluded that the use of the wind-moment method "in two directions" should be restricted to low rise frames not more than eight storeys with rigid joints. Its use with frames whose minor-axis beams are little more than tie members relies on a series of rules to ensure adequate stability. In frames such as these it is more appropriate to base design on an “exact” second-order analysis, rather than to rely on the rules described earlier. These features ensure that sway deflection remains within acceptable limits, and therefore do not cause large second-order moments in the columns.

REFERENCES

[1] European Steel Design Education Programme, Structural System: Building II Group 14. Vol.21.

[2] Eurocode 3: Design of Steel Structures: ENV 1993-1-1: Part 1.1: General Rule and Rules for Buildings, CEN, Brussels (1992).

[3] Anderson, D. and Lok, T.S., “Design studies on unbraced, multi-storey frames”, The Strucutral Engineer, Vol. 61B, No. 2, June (1983), 29-34.

[4] Anderson, D., Reading, S.J. and Kavianpour, K., Wind-moment design for unbraced frames, Steel Construction Institute, (1991).

[5] Brown, N. D., Aspects of sway frame design and ductility of composite end plate connections, Ph.D Thesis, University of Warwick, (1995).

[6] CP3 : Code of basic data for the design of buildings: Chapter V: Loading : Part 2: Wind loads, British Standards Institution, (1972).

[7] British Standard Institute BS 5950: Structural Use of Steelwork in Building Part 1: Code of practice for design in simple and continuous construction: Hot-rolled Sections, British Standards Institution, London, (1990).

[8] Reading, S.J., Investigation of the wind connection method, M.Sc Thesis, University of Warwick, (1989).

[9] Md. Tahir M., Structural and Economic Aspects of the use of semi-rigid Joints in Steel Frames, Ph.D Thesis, University of Warwick, (1997).

[10] Anderson, D.; Design of multi-storey frames to sway deflection limitations steel-framed structures, Stability & strength (ed. Narayanan, R.) Elsevier Applied Science, (1987), 665-673.

[11] Anderson, D & Islam, M.A., Design of multi-storey frames to sway deflection limitations, The Structural Engineer, Vol. 57B, March 1979), 11-17.

[12] Anderson, D and Kavianpour, K., Analysis of steel frames with semi-rigid connections, Structural Engineering Review, Vol. 3, (1991), 79-87.

[13] Horne, M.R and Morris, L.J., Plastic design of low rise frames, Granada Publishing, (1981).

Table 1 Frames for minimum wind in conjunction with maximum gravity load.

Basic Frame Type	Width of Bay (m)	Height of Column Ground (m)	Elevated (m)	No. of Longi-tudinal Bays	Width of Longi-tudinal Bays (m)	Gravity Load (kN/m2) Floor L.L	D.L	Roof L.L	D. L	Basic Wind Speed (m/s)	Gound Roughness Factor
2 Storey 2 Bay	6m precast floor	6	5	2	6.0	5.0	7.5	3.75	1.5	37	4
4 Storey 2 Bay	6m precast floor	6	5	2	6.0	5.0	7.5	3.75	1.5	37	4
8 Storey 2 Bay	6m precast floor	6	5	2	6.0	5.0	7.5	3.75	1.5	37	4
4 Storey 4 Bay	6m precast floor	6	5	2	6.0	5.0	7.5	3.75	1.5	37	4
4 Storey 6 Bay	6m precast floor	6	5	2	6.0	5.0	7.5	3.75	1.5	37	4

Table 2 Frames for maximum wind in conjunction with minimum gravity load.

Basic Frame Type	Width of Bay (m)	Height of Column Ground (m)	Elevated (m)	No. of Longi-tudinal Bays	Width of Longi-tudinal Bays (m)	Gravity Load (kN/m2) Floor L.L	D.L	Roof L.L	D. L	Basic Wind Speed (m/s)	Gound Roughness Factor
2 Storey 2 Bay	6m precast floor	6	5	2	6.0	3.5	4.0	3.75	1.5	52	1
4 Storey 2 Bay	6m precast floor	6	5	2	6.0	3.5	4.0	3.75	1.5	52	1
8 Storey 2 Bay	6m precast floor	6	5	2	6.0	3.5	4.0	3.75	1.5	52	1
4 Storey 4 Bay	6m precast floor	6	5	2	6.0	3.5	4.0	3.75	1.5	52	1
4 Storey 6 Bay	6m precast floor	6	5	2	6.0	3.5	4.0	3.75	1.5	52	1

Table 3 Wind-moment design for 2, 4 and 6 bays frames considering minimum wind in conjunction with maximum gravity load.

Basic Frame Type	Section Designation (I) Universal Beam Floor	Roof		Universal Column External	Internal
2 Storey 2 Bay	1st 203x133x25	203x133x25	Up to 2nd Storey	203x203x71	203x203x71
4 Storey 2 Bay	1st 305x102x25 2nd.203x133x25 3rd. 203x133x25	203x133x25	Up to 2nd Storey	305x305x97	356x368x129
			2nd to 4th Storey	203x203x60	254x254x73
8 Storey 2 Bay	1st 457x152x52 2nd 406x140x46 3rd 406x140x39 4th 406x140x39 5th 356x127x33 6th 305x102x28 7th 203x133x25	203x133x25	Up to 3rd Storey	356x368x153	356x406x235
			3rd to 6th Storey	305x305x97	356x368x153
			6th to 8th Storey	203x203x60	254x254x89
4 Storey 4 Bay	1st 203x133x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Up to 2nd Storey	305x305x97	356x368x129
			2nd to 4th Storey	203x203x60	254x254x89
4 Storey 6 Bay	1st 203x133x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Up to 2nd Storey	305x305x97	356x368x129
			2nd to 4th Storey	203x203x60	254x254x89

Table 4 Limiting sway formulae included for 2, 4, and 6 bays frames considering minimum wind in conjunction with maximum gravity load.

Basic Frame Type	Section Designation (II) Universal Beam Floor	Roof		Universal Column External	Internal
2 Storey 2 Bay	1st 203x133x25	203x133x25	Up to 2nd Storey	203x203x71	305x305x97
4 Storey 2 Bay	1st 406x140x39 2nd.406x140x39 3rd. 356x127x33	203x133x25	Up to 2nd Storey	305x305x97	356x368x129
			2nd to 4th Storey	254x254x73	305x305x97
8 Storey 2 Bay	1st 533x210x92 2nd 533x210x82 3rd 533x210x82 4th 533x210x82 5th 457x191x67 6th 406x140x46 7th 356x127x39	203x133x25	Up to 3rd Storey	356x368x153	356x406x235
			3rd to 6th Storey	356x368x129	356x368x202
			6th to 8th Storey	254x254x89	356x368x129
4 Storey 4 Bay	1st 305x102x25 2nd 254x102x25 3rd 203x133x25	203x133x25	Up to 2nd Storey	305x305x97	356x368x129
			2nd to 4th Storey	203x203x60	254x254x89
4 Storey 6 Bay	1st 254x102x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Up to 2nd Storey	305x305x97	356x368x129
			2nd to 4th Storey	203x203x60	254x254x89

Table 5 Connection requirements for 2, 4, and 6 bays frames considering minimum wind in conjunction with maximum gravity load.

Basic Frame Type	Connection Requirements: Bending moment (kN.m) Floor	Roof	Shear force (kN) Floor	Roof
2 Storey 2 Bay	1st 24	7	1st 8	2
4 Storey 2 Bay	1st 79 2nd. 54 3rd. 33	11	1st 26 2nd. 18 3rd. 11	4
8 Storey 2 Bay	1st 232 2nd 193 3rd 172 4th 147 5th 119 6th 87 7th 51	16	1st 77 2nd 64 3rd 57 4th 49 5th 40 6th 29 7th 17	5
4 Storey 4 Bay	1st 39 2nd 27 3rd 16	5	1st 13 2nd 9 3rd 5	2
4 Storey 6 Bay	1st 26 2nd 18 3rd 11	4	1st 9 2nd 6 3rd 4	1

Table 6 Wind-moment design for 2, 4 and 6 bays frames considering maximum wind in conjunction with minimum gravity load.

Basic Frame Type	Section Designation (I): Universal Beam Floor	Universal Beam Roof		Universal Column External	Universal Column Internal
2 Storey 2 Bay	1st 356x127x33	203x133x25	Up to 2nd Storey	254x254x73	305x305x118
4 Storey 2 Bay	1st 457x191x67 2nd.406x140x46 3rd. 356x127x33	203x133x25	Up to 2nd Storey 2nd to 4th Storey	356x368x153 254x254x73	356x406x235 305x305x118
8 Storey 2 Bay	1st 610x229x113 2nd 610x229x101 3rd 533x210x82 4th 533x210x82 5th 457x191x67 6th 457x152x52 7th 406x140x39	203x133x25	Up to 3rd Storey 3rd to 6th Storey 6th to 8th Storey	356x406x287 356x368x153 254x254x89	356x406x551 356x406x287 356x368x129
4 Storey 4 Bay	1st 406x140x39 2nd 356x127x33 3rd 203x133x25	203x133x25	Up to 2nd Storey 2nd to 4th Storey	305x305x97 203x203x52	356x368x153 254x254x73
4 Storey 6 Bay	1st 356x127x33 2nd 305x102x25 3rd 203x133x25	203x133x25	Up to 2nd Storey 2nd to 4th Storey	254x254x89 203x203x46	305x305x118 254x254x73

Table 7 Proposed method to limit sway included for 2, 4 and 6 bays frames considering maximum wind in conjunction with minimum gravity load.

Basic Frame Type	Section Designation (II) Universal Beam Floor	Roof	Universal Column	External	Internal
2 Storey 2 Bay	1st 406x140x46	305x102x33	Up to 2nd Storey	305x305x97	356x368x129
4 Storey 2 Bay	1st 610x229x101 2nd.533x210x92 3rd. 457x152x74	305x102x33	Up to 2nd Storey	356x368x153	356x406x287
			2nd to 4th Storey	356x368x129	356x368x202
8 Storey 2 Bay	1st 838x292x176 2nd 838x292x176 3rd 762x267x147 4th 610x229x125 5th 610x229x125 6th 533x210x92 7th 457x191x74	305x102x33	Up to 3rd Storey	356x406x340	356x406x551
			3rd to 6th Storey	356x406x235	356x406x393
			6th to 8th Storey	356x368x129	356x406x235
4 Storey 4 Bay	1st 533x210x82 2nd 457x152x67 3rd 406x140x46	254x102x25	Up to 2nd Storey	305x305x118	356x368x153
			2nd to 4th Storey	305x305x97	356x368x129
4 Storey 6 Bay	1st 457x152x52 2nd 406x140x46 3rd 356x127x39	203x133x25	Up to 2nd Storey	305x305x97	356x368x129
			2nd to 4th Storey	254x254x73	305x305x118

Table 8 Connection requirements for 2, 4 and 6 bays frames considering maximum wind in conjunction with minimum gravity load.

Basic Frame Type	Connection Requirements Bending moment (kN.m) Floor	Roof	Shear force (kN) Floor	Roof
2 Storey 2 Bay	1st 107	30	1st 36	10
4 Storey 2 Bay	1st 318 2nd. 207 3rd. 120	38	1st 106 2nd. 69 3rd. 40	13
8 Storey 2 Bay	1st 767 2nd 615 3rd 528 4th 436 5th 340 6th 240 7th 138	43	1st 256 2nd 205 3rd 176 4th 145 5th 113 6th 80 7th 46	14
4 Storey 4 Bay	1st 159 2nd 104 3rd 60	19	1st 53 2nd 35 3rd 20	6
4 Storey 6 Bay	1st 106 2nd 69 3rd 40	13	1st 35 2nd 23 3rd 13	4

Table 9 Ultimate Limit State collapse load factor and deflection at Serviceability Limit State for rigid jointed 2, 4, and 6 bays frames (Frames design for Section Designation II; minimum wind and maximum gravity load).

Basic Frame Type	Load Case	Collapse Load Factor (2nd order)	Deflection Check 1st order	2nd. order
2 Storey 2 Bay	Load case 1	2.07	1/973	1/728
	Load case 2	1.96	1/423	1/331
	Load case 3	2.35	1/338	1/296
4 Storey 2 Bay	Load case 1	1.85	1/1207	1/968
	Load case 2	1.77	1/421	1/350
	Load case 3	2.02	1/337	1/307
8 Storey 2 Bay	Load case 1	1.84	1/1496	1/1265
	Load case 2	1.60	1/400	1/348
	Load case 3	1.82	1/309	1/289
4 Storey 4 Bay	Load case 1	1.38	1/623	1/395
	Load case 2	1.46	1/416	1/233
	Load case 3	1.74	1/329	1/286
4 Storey 6 Bay	Load case 1	1.17	1/554	1/308
	Load case 2	1.43	1/580	1/358
	Load case 3	1.85	1/365	1/365

Table 10 Ultimate Limit State collapse load factor and deflection at Serviceability Limit State for rigid jointed 2, 4, and 6 bays frames (Frames design for Section Designation II; maximum wind and minimum gravity load).

Basic Frame Type	Load Case	Collapse Load Factor (2nd order)	Deflection Check 1st order	2nd. order
2 Storey 2 Bay	Load case 1	5.66	1/4074	1/3793
	Load case 2	2.17	1/309	1/293
	Load case 3	2.02	1/248	1/239
4 Storey 2 Bay	Load case 1	6.57	1/5833	1/5526
	Load case 2	2.02	1/366	1/352
	Load case 3	1.83	1/292	1/286
8 Storey 2 Bay	Load case 1	6.48	1/6457	1/6212
	Load case 2	1.91	1/368	1/356
	Load case 3	1.72	1/294	1/289
4 Storey 4 Bay	Load case 1	3.43	1/2896	1/2658
	Load case 2	1.83	1/367	1/340
	Load case 3	1.75	1/294	1/281
4 Storey 6 Bay	Load case 1	2.68	1/2100	1/1842
	Load case 2	1.80	1/372	1/333
	Load case 3	1.85	1/297	1/278

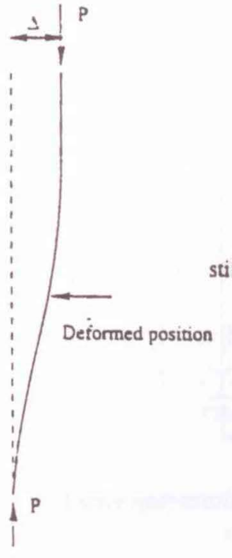

Figure 1 P-Δ effect

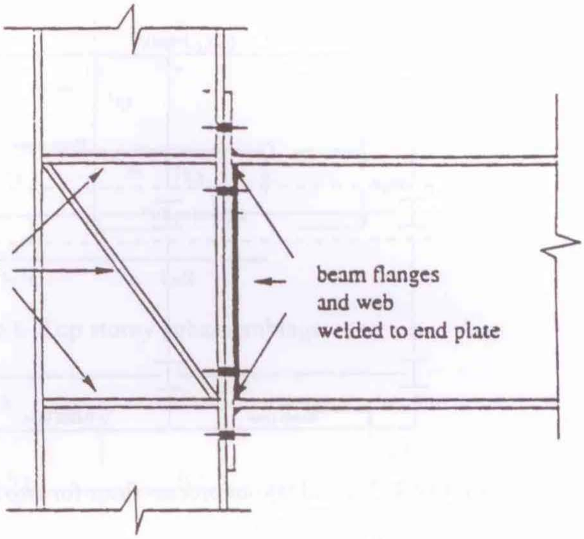

Figure 2 Extended end plate with stiffener

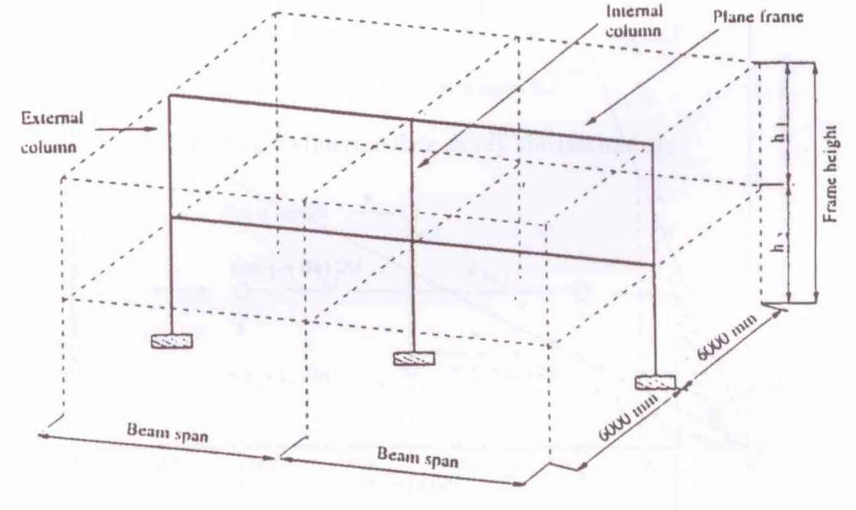

Figure 3 Typical layout for two bay frames (three dimensions)

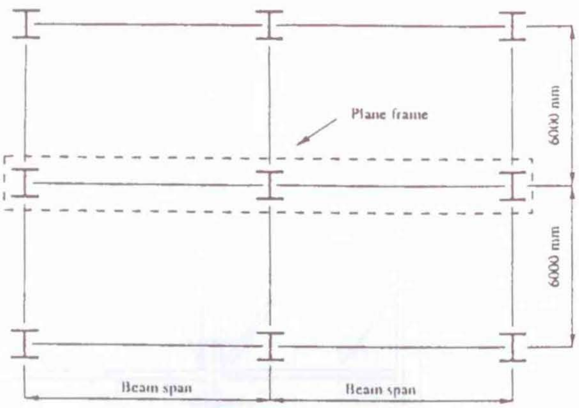

Figure 4 Typical layout precast floor for two bay frames (top view)

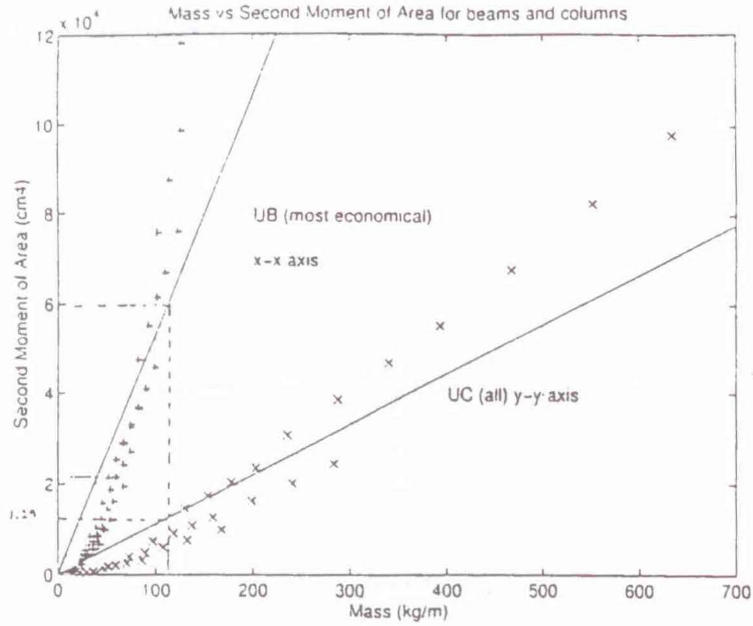

Figure 5 Comparison of $I_{x\text{-}x}$ Universal Beam and $I_{y\text{-}y}$ Universal Column to calculate optimization factor $k_3 = 4.8$

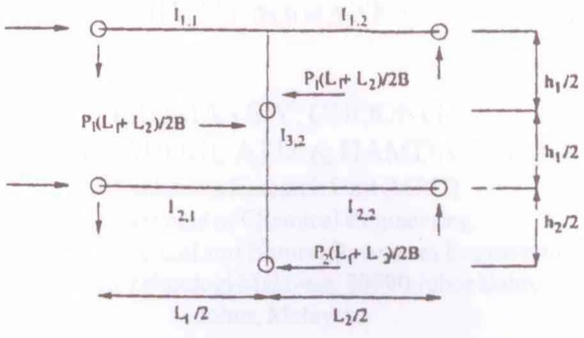

Figure 6 Top storey subassemblage

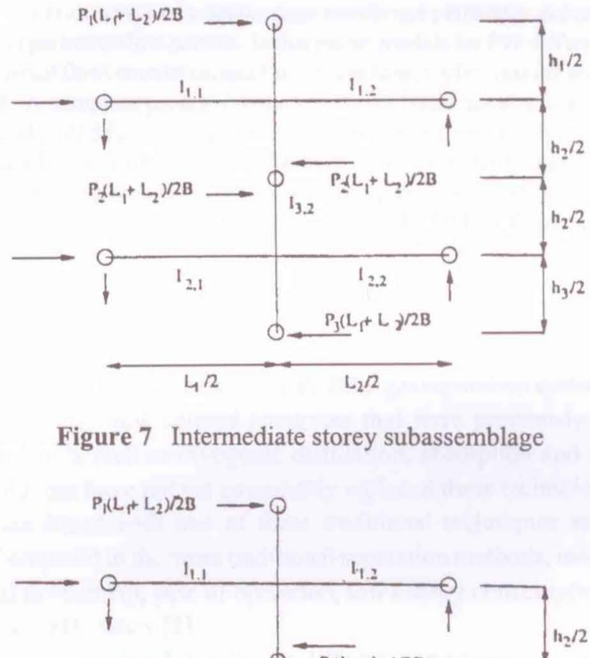

Figure 7 Intermediate storey subassemblage

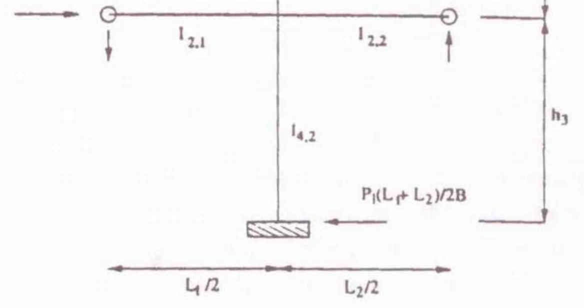

Figure 8 Subassemblage for bottom 2 storeys