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WIND-MOMENT DESIGN FOR UNBRACED FRAMES WITH BENDING ABOUT BOTH COLUMN AXES

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ABSTRACT

Analysis and design of steel unbraced frames bending on both axes were performed with emphasis on stability and deflection checks. Wind-moment design is proposed to improve the stability and stiffness. The performance of the frames was checked for collapse load level at Ultimate Limit State (ULS) for 2nd order anaysis and the deflection limits at Serviceability Limit State checked for 1st order analysis. The investigation demonstrated that the frames should be restricted to less than four storeys.

INTRODUCTION

Steel frames with bending about both the major and the minor axes of the column sections are usually designed on the basis that beam-to-column connections are either pinned or rigid. However, the actual behaviour will usually fall somewhere between these extremes, as recognised by the concept of semi-rigid design permitted by some design codes including Eurocode 3[1]. The connection behaviour is then represented by a moment-rotation (M-Φ) curve, relating the moment M transmitted by the connection to the relative rotation Φ between the beam and the adjacent column. This means that all connections, including connections connected to column web, will possess some moment capacity and some rotational stiffness. However, uncertainty concerning the behaviour of connections attached to a column web make this configuration an uncommon one for joints designed to be moment-resistant.

For the design of unbraced sway frames at ULS, it is possible to use advanced methods based on interaction of elastic buckling characteristics and the reduction in stiffness due to plasticity[1,2]. The joints are assumed to be rigid and full-strength. However, other less-sophisticated methods apparently based on purely elastic behaviour are also available. One approach, termed the "wind-moment" or "wind-connection" method[3], is often used in the U.K. The method is known as "Type 2 Construction" in the U.S.A[4].

WIND-MOMENT DESIGN FOR UNBRACED FRAMES BENDING ABOUT MAJOR COLUMN AXIS

The use of the "wind-moment" design method in conjunction with BS 5950 Part 1[2] is now well-established for major-axis framing. This method effectively employs semi-rigid connections which are also "partial-strength" relative to the connected beams; a range of standard joints of this type are now available[5]. The existing rules for "wind-moment" design have been shown to provide adequate resistance to frames with major-axis sway. In its simplest form the "wind-moment" method[3] assumes:

(i) under gravity load, the connections act as pins Figure 1a; this means that the beam members are designed as simply supported with no moments transferred to the column, other than nominal "eccentricity" moments;.

(ii) under wind load, the connections behave as rigid joints, with points of contraflexure at the mid-height of columns and mid-length of beams Figure 1b.

Members are proportional initially to resist gravity load. The internal forces and moments due to gravity load and wind Figure 2a and Figure 2b are then combined in appropriate load cases. The design at the ultimate limit state is completed by amending the initial section sizes and other details for the members and connections, to withstand combined load effects.

The advantage of the method is its simplicity. The frame is considered as statically determinate with internal moments and forces not dependent on the relative stiffness of the members. The need to repeat the analysis to correspond to changed section sizes is thereby avoided. The beam sections generally have the same size for all the floors since the mid-span moment due to gravity load usually controls the design, thus simplifying the construction of the building. In contrast, for fully continuous construction with rigid joints, the beam sections tend to be different at the various floor levels[6]. A further advantage is that connections usually do not require the web stiffening often associated with rigid joints; this is because they are designed for moments due to wind loading only. As a result, fabrication costs are reduced and designers have greater freedom in the positioning and size of beams which frame into the column web.

For serviceability, sway deflections are calculated assuming connections are rigid. Second-order analysis due to the "P-Δ" effect is not included in the calculation. It is assumed that these can be accounted for by using effective column lengths greater than the true lengths, for axes about which sway can occur.

The method described above has been used extensively, and design rules consistent with BS 5950: Part 1: 1990[2] have been published[3]. These were developed in conjunction with an analytical study of typical frames designed by the method[7]. Despite its widespread use the method cannot be fully accepted as a generally applicable approach. The scope of the rules was therefore restricted to that of the study. In particular they apply to steelwork which can be idealised as a series of unbraced plane frames which are effectively braced against out-of-plane sway at roof level and each floor level as shown in Figure 3. Within each plane frame the column sections should be oriented such that loads in the plane of the frame tend to cause bending about the major axes. This represents an unwelcome restriction on the forms of structure to which the "wind-moment" method can be applied. Studies were therefore required to verify the method when the structure can sway about both column axes.

WIND-MOMENT DESIGN FOR UNBRACED FRAMES BENDING ABOUT MINOR COLUMN AXIS

The "wind-moment" approach has been further extended by the present author to the design of unbraced frames with bending about both column axes, as described elsewhere[8]. For frames which sway about the minor axis, the following are of particular concern:

1. the form of the minor-axis connection, which must provide reasonable moment resistance and stiffness;
2. the stiffness and stability of the frame against minor-axis sway, which will be influenced by the low flexural rigidity exhibited even by Universal Columns bent in this way;
3. in frames supporting precast units, the minor axis beams may remain as little more than tie members even when designed for wind moments, with consequent absence of appropriate stiffness to ensure frame stability at ultimate load and reasonable deflections in service.

The methodology is similar to that used previously on major-axis framing[7,9], namely the design of a series of frames by the proposed rules, followed by "exact" analysis[10] to check the designs. The "exact" analysis is described by Kavianpour as a second order elasto-plastic approach, based on the matrix displacement method. The analysis includes the behaviour of the connections by including an estimation of the secant stiffness of each connection obtained from the M-Φ characteristics. These in turn may be obtained from tests, mathematical expressions or analytical models[11]. Plasticity in members is taken into account by a plastic hinge idealization. The effect of axial force in frame stiffness is included by using stability functions with modifications made to the slope-deflection equations; small-deflection theory is assumed.

RANGE OF APPLICATIONS

The range of the study is summarised in Table 1. The frames ranged in height from 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 (Figure 4a and 4b)

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[3] 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[9].

Table 1 : Range of "wind-moment" study on minor axis

	Minimum wind	Maximum wind
Number of storeys	2 to 8	2 to 8
Number of bays	2 to 6	2 to 6
Bay width	6 m	6 m
Storey height (bottom)	6 m	6 m
Storey height (elsewhere)	5 m	5 m
Dead load on floors	3.50 kN/m^2	5.00 kN/m^2
Imposed load on floors	4.00 kN/m^2	7.50 kN/m^2
Dead load on roof	3.75 kN/m^2	3.75 kN/m^2
Imposed load on roof	1.50 kN/m^2	1.50 kN/m^2
Basic wind speed	37 m/s	52 m/s
Basic steel grade	S275	S275

Two arrangements of floor grids were considered. The first arrangement consists of floor units assumed to span 6 m between the major-axis frames as illustrated in Figure 5a. This results in the minor-axis beams being free of significant gravity forces, the main loading being wind-moments. The second grid assumed composite floors spanning only 3m (Figure 5b) with the result that the minor axis beams act as primary beams in support of the floor; substantial minor-axis beam sections are then needed to resist gravity forces and the inherent stability of the minor axis framing is significantly increased. The limitations on loading conformed to those in the existing recommendations[3]. Frames were designed for combinations of maximum gravity load with minimum wind forces, and vice-versa.

LOAD COMBINATIONS

For serviceability limit states design 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 taken into account[2]. Frames were 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.

The deflection limits for a building exceeding one storey high as recommended by BS 5950 are 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.

DESIGN METHODOLOGY

Initially the structure was designed by the wind-moment method assuming bending about the major axis of the columns. Computer software written by Reading[7] and modified by Brown[9] was used to design the column sections for frame bending about this axis. The modification by Brown was to change the effective length for minor axis buckling from 0.85L to 1.0L, in accordance with the published rules for wind-moment design[3]. For minor axis design, the software was further modified by the present author.

In the case of floor grids illustrated in Figure 5a, the minor axis beams are designed to resist only those moments due to either notional horizontal forces or wind. Small beam sections may then result. Second-order "exact" analyses[10] then show inadequate sway stability for the ULS design loads. Two procedures are then adopted to stiffen the frame:

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

A detailed description of the procedures is described in reference[8]. For floor grids arranged in Figure 5b the minor axis beams act as primary beams and therefore, carry substantial gravity loads. Their designs and proposed form of connections are discussed in the foregoing paragraphs.

WIND-MOMENT DESIGN FOR BEAMS IN MINOR-AXIS FRAMING

The beams for the floor grids illustrated in Figure 5a are considered as tie beams which carry no gravity load except their own dead weight. This is because the floor units span directly between the major-axis framing and the minor axis beams may be positioned clear of the underside of the floor. In such circumstances the minor-axis beams resist bending moments M only due to the notional horizontal forces or due to wind. By being clear of the underside of the floor, the minor axis beam is laterally unrestrained. The member should therefore be designed as follows:

1. As the notional horizontal forces are a device to allow for initial out-of-plumb (and are therefore not real forces), the appropriate maximum slenderness is that corresponding to a member carrying only wind forces in addition to its own self weight. Hence, from clause 4.7.3.2 of BS5950 Part 1[2], the effective slenderness ratio should not exceed 250; in view of the end restraint, the slenderness ratio is taken as $0.85L/r_y$;
2. $M_{cx} \geq M$, where M_{cx} is the moment capacity about major axis. For a compact section, $M_{cx} = p_y S$ but $\leq 1.2 p_y Z$ where for semi-compact section, $M_{cx} = p_y Z$ where p_y is the design strength, S is the plastic modulus of the section about the relevant axis, and Z is the corresponding elastic modulus;
3. the moments due to notional horizontal forces and wind cause double curvature in the minor-axis beams as shown in Figure 6. The need to check for lateral torsional buckling under this moment distribution is discussed below.

The design of the beams for floor grids illustrated in Figure 5b was made in accordance with the recommendations for major-axis framing[3].

CHECK FOR MAXIMUM SLENDERNESS

Tie beams transmit wind forces and notional horizontal forces which in turn are distributed between bays. In this manner the tie beams are acting as horizontal struts. In accordance to BS 5950 clause 4.7.3.2, the maximum slenderness for members resisting self weight and wind loads only, should not exceed 250. For this study the minimum beam section for tie beams was a 203x133x25UB. The resulting slenderness ratio (λ) is equal to 165 based on $0.85L/r_y$ for a 6 m span beam. The maximum slenderness ratio calculated for section 254x102x25UB is 238, not exceeding the limit recommended by the design code.

CHECK FOR LATERAL TORSIONAL BUCKLING

The tie beams do not carry gravity load, therefore can be considered as laterally unrestrained. In this case, m is taken to be equal to 0.43 due to the double curvature effect. The studies have shown that lateral torsional bucking is not critical[8].

WIND-MOMENT DESIGN FOR COLUMNS IN MINOR AXIS FRAMING

For floor grids shown in Figure 5a, it is likely that the worst situation for frame stability will arise with the structure fully-loaded. For "internal" minor-axis framing (Figure 4b), with equal bay widths and loading, the only bending moments in the columns are due to horizontal loads, and it may therefore be expected that the column moments will be in double-curvature bending. In the design of minor-axis framing for this study, an equivalent uniform moment factor m_y of 0.43 was adopted for the overall buckling check conforming to clause 4.8.3.3.1 specified in BS 5950. In view of the earlier design recommendations for major-axis framing[3], it is proposed that m_x be taken as unity. For calculation of the buckling resistance moment the effective length is taken as 1.0 L. The effective length of the column for resistance to axial load is influenced by sway about the minor axis and should be taken as 1.5 L[3].

Patterned loading should be considered in the design although it is unlikely to be critical in some cases. Patterned loading induces out-of-balance moments in the columns, and therefore be checked for biaxial bending. Patterned loading was developed by loading the beam on one side of the column, with the factored maximum gravity load (1.4 Dead load + 1.6 Imposed load) and the other side with the factored dead load only (1.4 Dead load). For floor grids illustrated in Figure 5a, the unbalanced moment acted about the major axis of the columns whereas wind or notional forces contributed moment about the minor axis. A second pattern to induce the maximum out-of-balance moments about the minor axis was also considered for the floor grids shown in Figure 5b. Figure 7 shows the two arrangements of loading for such grids.

FRAME GRIDS FOR COMPOSITE FLOORS

The recommendations described by Kavianpour[10] for limiting sway and improving restraint in columns are primarily intended to avoid inadequate sway stability that would arise with grids arrangement illustrated in Figure 5a. However, they can also be applied to frames with grids arrangements shown Figure 5b. All frames must possess adequate stiffness under serviceability loading. The recommendations represent good practice based on engineering judgement and may, in any case, be automatically satisfied by beams from the grids concerned, because these members support substantial gravity loads.

ASSESSMENT OF RESULTS

The dimensions and loading for the various frame arrangements studied in the programme are listed in Table 2 to Table 5. The data presented in Table 2 and Table 3 takes into consideration minimum wind load combined with maximum gravity load. Table 4 and Table 5 display data for the case of maximum wind load combined with minimum gravity load, vice-versa of Table 2 and Table 3. The wind-moment designs given in these tables are noted as "Section Designation I". To justify the design recommendations, the frames were subjected to a first-order analysis and accounting for rigid and semi-rigid joints. A software[10] was used to carry out this analysis.

To ensure local column stability, checks on overall buckling and local capacity were made in accordance with Clause 4.8.3.3.1 and 4.8.3.2(a) of BS 5950[2]. The moments and forces used were those given by the analysis at the design load levels for ULS. As the results were regarded as "exact", equivalent uniform moment factors were calculated from the distribution of bending moments revealed in the columns. The resulting comparisons against unity are termed 'Stability Factors'. For the overall buckling check, the minor axis moment of resistance was taken as the yield moment p_yZ_y, and not the plastic moment. For the local capacity check, the moment capacity was taken as the less of the minor axis plastic moment and $1.2\,p_yZ_y$. For comparison purposes the designs were analysed assuming that they were rigid and semi-rigid joints frames.

The results for frames design for minimum wind in conjunction with maximum gravity load for rigid and semi-rigid joints are summarised in Table 6 and Table 7. Frames subjected to maximum wind loading in conjunction with minimum gravity load are designed in accordance to the proposed rules[8]. The rules improve the wind moment design, providing adequacy in the overall stability and limiting the sway index to 1/300. Otherwise, where NA is shown (Table 6 & Table 7), this indicates that:

- in the case of Section Designation I, the design possessed inadequate overall stability;
- in the case of deflections, the overall sway exceeded the index limit of 1/300.

CONCLUSIONS

The study on unbraced frames bending about both column axes showed that it was not possible to design frames only by wind-moment analysis to provide adequate resistance. Proposed rules[8] are needed to improve the design. Despite the assumption of relatively stiff minor-axis connections in which the sole source of flexibility is associated with the beam end-plate, a straightforward extension of the previous rules for wind moment design[3] 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 this case the minor-axis beams, not being heavily loaded, may be of small section size and therefore too flexible to ensure overall frame stability. 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.

In the second case, when flooring consists of composite slabs, the minor axis beams will necessarily resist significant gravity load. This results in increased section sizes for those members and the wind-moment designs are therefore much more stable. Even so, it will be necessary in some cases to further increase section sizes, to avoid excessive sway under service load. On the basis of limited results, a multiplier of 1.5 to correct for joint flexibility is reasonable for most frames.

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" plus proposed rules[8] should be restricted to low rise frames not more than four storeys. The author has more confidence in the use of the method for frames having secondary beams (in the minor-axis direction) of a reasonable size and stiffness. Its use with frames whose minor-axis beams are little more than tie members (Fig. 5(a)) relies on a series of rules[8] to ensure adequate stability.

REFERENCES

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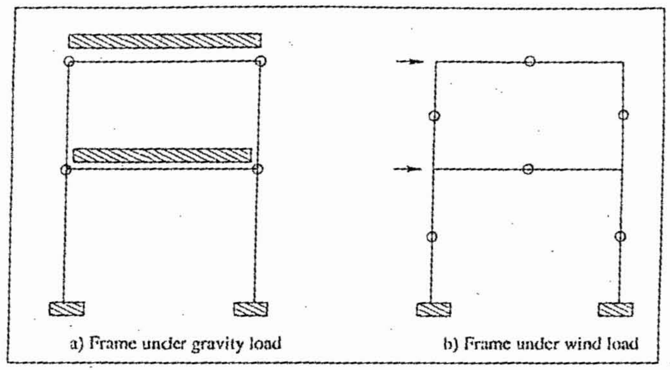


Figure 1 : Frame idealisation for "wind-moment" method

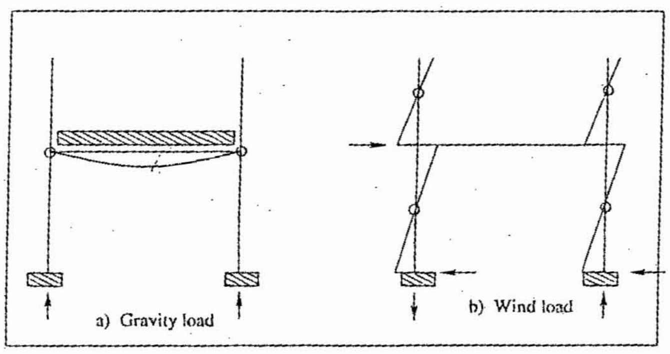


Figure 2 : Internal moments and forces according to "wind-moment" method

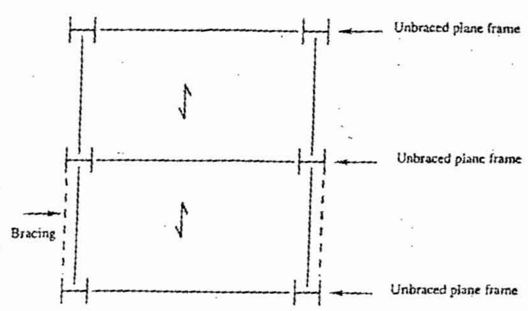


Figure 3 : Plane braced against out-of-plane sway

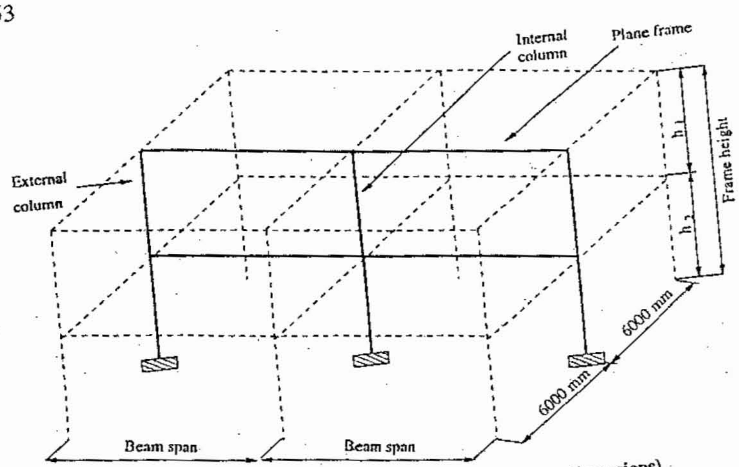


Figure 4(a): Typical layout for two bay frames (three dimensions)

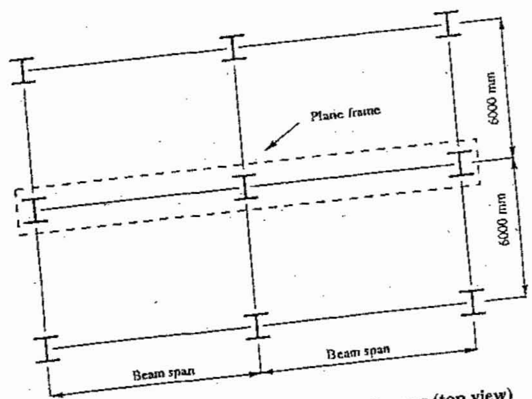
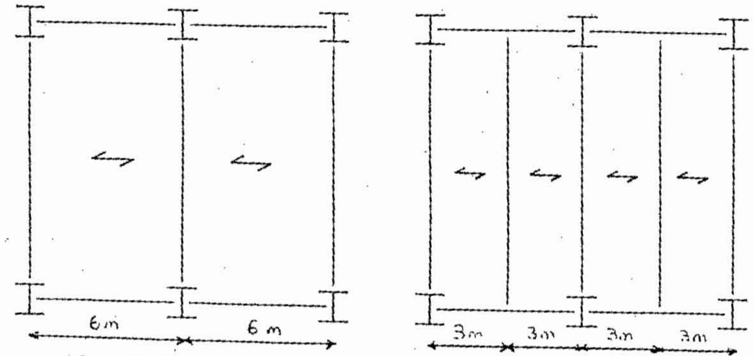


Figure 4(b): Typical layout for two bay frames (top view)



a) Precast units

b) Composite floor

Figure 5 : Floor grids

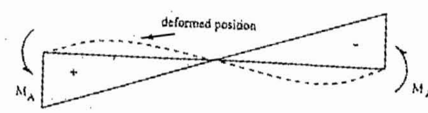
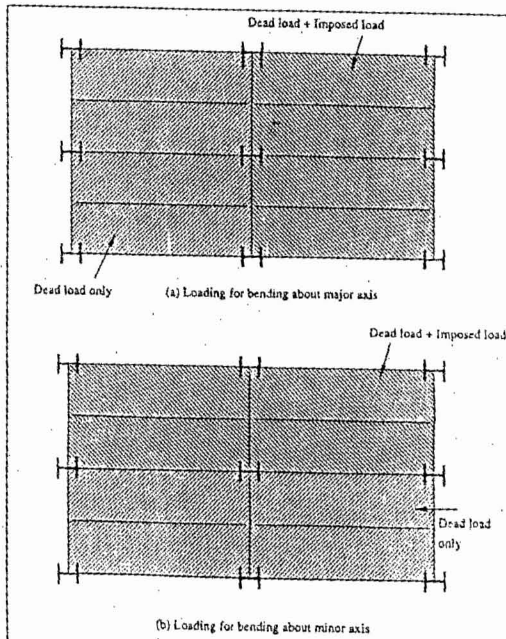


Figure 6 : Bending moment diagram contribute to double curvature in the minor-axis beam



Dead load only

Dead load only

Figure 7 : Arrangements of pattern loading

Table 2 : Wind-moment design for 2 bay frames considering minimum wind in conjunction with maximum gravity load (Grid 1, precast floor at 6m span)

Basic Frame Type	Section Designation (I)					Connection Requirements			
	Universal Beam			Universal Column		Shear force		Bending moment Universal Colum	
	Floor	Roof		External	Internal	Floor	Roof	Floor	Roof
2 Storey 2 Bay	1st 203x133x25	203x133x25	Up to 2nd Storey	203x203x71	203x203x71	1st 24	7	1st 8	2
4 Storey 2 Bay	1st 305x102x25 2nd.203x133x25 3rd.203x133x25	203x133x25	Up to 2nd Storey 2nd to 4th Storey	305x305x97 203x203x60	356x368x129 254x254x73	1st 79 2nd 54 3rd 33	11	1st 26 2nd 18 3rd 11	4
8 Storey 2 Bay	1st 457x152x52 2nd 406x140x46 3rd 406x140x39 4th 406x140x39 5th 356x127x33 6th 305x102x28 7th 203x133x25	203x133x25	Up to 3rd Storey 3rd to 6th Storey 6th to 8th Storey	356x368x153 305x305x97 203x203x60	356x406x235 356x368x153 254x254x89	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 203x133x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Up to 2nd Storey 2nd to 4th Storey	305x305x97 203x203x60	356x368x129 254x254x89	1st 39 2nd 27 3rd 16	5	1st 13 2nd 9 3rd 5	2
4 Storey 6 Bay	1st 203x133x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Up to 2nd Storey 2nd to 4th Storey	305x305x97 203x203x60	356x368x129 254x254x89	1st 26 2nd 18 3rd 11	4	1st 9 2nd 6 3rd 4	1

Table 3 : Wind-moment design for 2 bays frames considering minimum wind in conjunction with maximum gravity load (Grid 2, composite floor at 3m span)

Basic Frame Type	Section Designation (I)					Connection Requirements			
	Universal Beam			Universal Column		Shear force		Bending moment Universal Colum	
	Floor	Roof		External	Internal	Floor	Roof	Floor	Roof
2 Storey 2 Bay	1st 533x210x82	356x171x45	Up to 2nd Storey	254x254x73	305x305x97	1st 61	27	1st 174	69
4 Storey 2 Bay	1st 533x210x82 2nd.533x210x82 3rd.533x210x82	356x171x45	Up to 2nd Storey 2nd to 4th Storey	305x305x97 254x254x73	356x368x129 254x254x89	1st 108 2nd 86 3rd 69	29	1st 183 2nd 177 3rd 174	69
8 Storey 2 Bay	1st 533x210x82 2nd 533x210x82 3rd 533x210x82 4th 533x210x82 5th 533x210x82 6th 533x210x82 7th 533x210x82	356x171x45	Up to 3rd Storey 3rd to 6th Storey 6th to 8th Storey	356x368x153 305x305x118 254x254x73	356x406x235 356x368x153 254x254x89	1st 239 2nd 206 3rd 191 4th 167 5th 142 6th 115 7th 84	34	1st 251 2nd 222 3rd 196 4th 188 5th 182 6th 177 7th 172	69
4 Storey 4 Bay	1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Up to 2nd Storey 2nd to 4th Storey	305x305x97 254x254x73	356x368x129 254x254x89	1st 79 2nd 68 3rd 59	27	1st 183 2nd 177 3rd 174	69
4 Storey 6 Bay	1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Up to 2nd Storey 2nd to 4th Storey	305x305x118 254x254x73	356x368x129 254x254x89	1st 79 2nd 68 3rd 59	27	1st 183 2nd 177 3rd 174	69

Table 4 : Wind-moment design for 2 bay frames considering maximum wind in conjunction with minimum gravity load (Grid 1, precast floor at 6m span)

Basic Frame Type	Section Designation (I) Universal Beam Floor	Roof		Universal Column External	Internal	Connection Requirements Shear force Floor	Roof	Bending moment Universal Colum Floor	Roof
2 Storey 2 Bay	1st 356x127x33	203x133x25	Up to 2nd Storey	254x254x73	305x305x118	1st 107	30	1st 36	10
4 Storey 2 Bay	1st 457x191x67 2nd. 406x140x46 3rd. 356x127x33	203x133x25	Up to 2nd Storey 2nd to 4th Storey	356x368x153 254x254x73	356x406x235 305x305x118	1st 318 2nd 207 3rd 120	38	1st 106 2nd 69 3rd 40	13
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	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 406x140x39 2nd 356x127x33 3rd 203x133x25	203x133x25	Up to 2nd Storey 2nd to 4th Storey	305x305x97 203x203x52	356x368x153 254x254x73	1st 159 2nd 104 3rd 60	19	1st 53 2nd 35 3rd 20	6
4 Storey 6 Bay	1st 356x127x33 2nd 305x102x25 3rd 203x133x25	203x133x25	Up to 2nd Storey 2nd to 4th Storey	254x254x89 203x203x46	305x305x118 254x254x73	1st 106 2nd 69 3rd 40	13	1st 35 2nd 23 3rd 13	4

Table 5 : **Wind-moment design for 2 bay frames considering maximum wind in conjunction with minimum gravity load (Grid 2, composite floor at 3m span)**

Basic Frame Type	Section Designation (I) Universal Beam Floor	Roof	Universal Column	External	Internal	Connection Requirements Shear force Floor	Roof	Bending moment Universal Colum Floor	Roof
2 Storey 2 Bay	1st 457x152x60	356x171x45	Up to 2nd Storey	254x254x73	305x305x118	1st 120	45	1st 111	69
4 Storey 2 Bay	1st 457x191x67 2nd.457x152x60 3rd.457x152x60	356x171x45	Up to 2nd Storey 2nd to 4th Storey	356x368x153 254x254x89	356x3406x235 305x305x118	1st 331 2nd 221 3rd 134	52	1st 172 2nd 140 3rd 115	69
8 Storey 2 Bay	1st 610x229x113 2nd 610x229x101 3rd 533x210x92 4th 533x210x82 5th 457x191x67 6th 457x152x60 7th 457x152x60	356x171x45	Up to 3rd Storey 3rd to 6th Storey 6th to 8th Storey	356x406x287 356x368x177 254x254x89	356x406x551 356x406x287 356x368x129	1st 780 2nd 628 3rd 541 4th 449 5th 353 6th 254 7th 151	57	1st 316 2nd 257 3rd 232 4th 206 5th 178 6th 150 7th 120	69
4 Storey 4 Bay	1st 457x152x60 2nd 457x152x60 3rd 457x152x60	356x171x45	Up to 2nd Storey 2nd to 4th Storey	305x305x97 203x203x60	356x368x153 254x254x73	1st 172 2nd 117 3rd 76	33	1st 126 2nd 111 3rd 104	69
4 Storey 6 Bay	1st 457x152x60 2nd 457x152x60 3rd 457x152x60	356x171x45	Up to 2nd Storey 2nd to 4th Storey	305x305x118 254x254x73	356x368x129 254x254x89	1st 119 2nd 84 3rd 59	28	1st 104 2nd 105 3rd 111	69

Table 6 : ULS collapse load factor and deflection at SLS for rigid jointed frames.

Frame type	Width of Bay (m)	Frame Identification	Load Case	Collapse Load Factor (2nd order)	Deflection Check 1st (order)
2 storey 2 bay	6 composite floor	Frame 1	LC 1 LC 2 LC 3	1.83 2.20 2.64	1/2245 1/1014 1/815
4 storey 2 bay	6 composite floor	Frame 2	LC 1 LC 2 LC 3	1.83 1.83 2.28	1/1743 1/614 1/492
8 storey 2 bay	6 composite floor	Frame 3	LC 1 LC 2 LC 3	N/A	N/A
4 storey 4 bay	6 composite floor	Frame 7	LC 1 LC 2 LC 3	1.83 2.16 2.64	1/1680 1/1173 1/942
4 storey 6 bay	6 composite floor	Frame 8	LC 1 LC 2 LC 3	1.83 2.21 2.65	1/2333 1/2386 1/1479
2 storey 2 bay	6 precast floor	Frame 4	LC 1 LC 2 LC 3	N/A	N/A
4 storey 2 bay	6 precast floor	Frame 5	LC 1 LC 2 LC 3	N/A	N/A
8 storey 2 bay	6 precast floor	Frame 6	LC 1 LC 2 LC 3	N/A	N/A
4 storey 4 bay	6 precast floor	Frame 9	LC 1 LC 2 LC 3	N/A	N/A
4 storey 6 bay	6 precast floor	Frame 10	LC 1 LC 2 LC 3	N/A	N/A

Table 7 : ULS collapse load factor and deflection at SLS for semi-rigid jointed frames.

Frame type	Width of Bay (m)	Frame Identification	Load Case	Collapse Load Factor (2nd order)	Deflection Check 1st (order)
2 storey 2 bay	6 composite floor	Frame 1	LC 1 LC 2 LC 3	1.31 1.59 1.89	1/840 1/537 1/512
4 storey 2 bay	6 composite floor	Frame 2	LC 1 LC 2 LC 3	1.41 1.46 1.68	1/1250 1/476 1/400
8 storey 2 bay	6 composite floor	Frame 3	LC 1 LC 2 LC 3	N/A	N/A
4 storey 4 bay	6 composite floor	Frame 7	LC 1 LC 2 LC 3	1.31 1.53 1.89	1/605 1/511 1/515
4 storey 6 bay	6 composite floor	Frame 8	LC 1 LC 2 LC 3	1.29 1.59 1.90	1/636 1/686 1/727
2 storey 2 bay	6 precast floor	Frame 4	LC 1 LC 2 LC 3	N/A	N/A
4 storey 2 bay	6 precast floor	Frame 5	LC 1 LC 2 LC 3	N/A	N/A
8 storey 2 bay	6 precast floor	Frame 6	LC 1 LC 2 LC 3	N/A	N/A
4 storey 4 bay	6 precast floor	Frame 9	LC 1 LC 2 LC 3	0.70 0.74 0.93	N/A
4 storey 6 bay	6 precast floor	Frame 10	LC 1 LC 2 LC 3	0.56 0.68 0.93	N/A