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DESIGN AND ANALYSIS OF A CASE-STUDY LOW-RISE BUILDING WITH MOMENT-RESISTING COMPOSITE FRAMES

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Abstract

Although composite frames with moment-resisting connections are an excellent choice for multi-storey building structures, they are not widely used in practice in Australia due to lack of appropriate practical design methods. This paper presents a practical design method for a prototype five-storey building composed of composite columns, beams and moment resisting connections. The objective of this study is to investigate the viability of moment-resisting frames with composite columns under lateral loading (wind and earthquake actions) as well as gravity load. The paper covers both the strength and drift demands on low-rise buildings in regions of low to medium seismicity. The effects of site soil type, column base fixity, rigidity of beam-to-column connection, and loading of external façade system on the frame behaviour are examined. The proposed method enables the design of the connections, beams and columns for rigid and semi-rigid composite frames that are to be used in engineering practice.

Keywords: Composite frame, moment-resisting connection, lateral load, drift, semi-rigid connection

1. Introduction

Moment-resisting composite frames have not received widespread application in practice due to the perceived complexity of analysis required and the lack of reliable information on the moment-rotation characteristics of the connections as required by design specifications. Lack of moment bolted connections in the past has also hampered the use of such composite systems. The recent development of blind bolts with extensions has overcome this shortcoming (Yao et al. 2008). In 2004, a new version of the Australian bridge design standard AS5100 was issued for bridge design (Standards Australia 2004). It included design guidance for composite columns and beams. Although the objective of AS5100 was to provide nationally acceptable requirements for the design of road and rail, bridges, the specific content of Part 6 on steel and composite construction is also applicable for building construction.

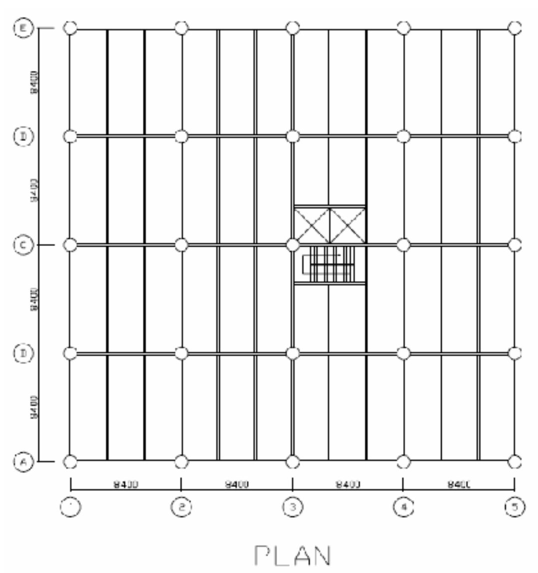
Extensive research has been carried out to investigate the actual behaviour of semi-rigid connection and to assess the performance of structural frames with semi-rigid connections (Leon 1990, Xiao et al. 1996, Rodrigues et al. 1998, Liew et al. 2000, Hensman and Nethercot 2001, de S Vellasco 2006). A little effort was devoted to develop simplified practical methods to design semi-rigid and rigid composite frames (Leon and Ammerman 1990, Cabrero and Bayo 2005, Wong et al. 2007). This paper aims to investigate the performance of moment-resisting frames with composite columns under lateral loading (wind and earthquake actions) as well as gravity load.

2. Prototype building

A prototype building with 5-storey, four-bay by four bay is designated as an office building located on a site with soil type of class D in Melbourne or Sydney. The building structure is classified as an ordinary moment-resisting frame with limited ductility. Resistance to the lateral forces is provided primarily by rigid frame action in the both directions. The moment-resisting frames are composed of concrete-filled steel tubular columns, steel beams with composite slabs and blind-bolted T-stub rigid connections (see Figure 1). The design of blind-bolted moment connections for the composite frame is part of a separate study.

The floor plan of the prototype building is 33.6m x 33.6m in area with four bays of 8.4 m in each direction as shown in Figures 2 and 3. The longer bay spacing makes it more representative of Australia practice. The first floor of the building has a height of 4 m, while the rest of the stories have a height of 3.5 m. All interior and exterior frames are moment-resisting frames in both directions. The prototype building had a composite floor system using profiled metal sheeting. The sheeting has sufficient strength to support the wet concrete during construction, and no propping of the floor system is required.

Figure 1 Blind-bolted moment connection



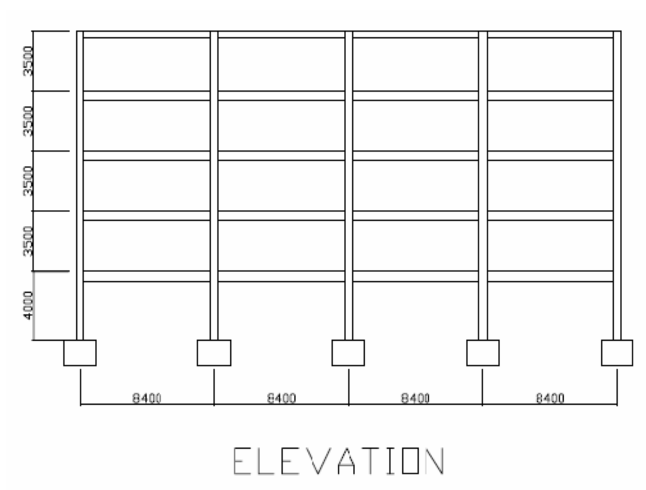


Figure 2 Build plan

Figure 3 Building elevation

3. Design procedure

3.1 Loading conditions

Design loads are estimated based on current Australian loading specifications AS/NZS 1170 (Standards Australia/Standards New Zealand 2002). Dead loads include self weight of structural components, ceiling, services, partitions, etc. Live load of 3 kPa is adopted for the office floor areas. Wind loads are applied to the building assuming this office building is located in Region A1 (non-cyclonic). The building is classified as importance level 2 with return periods of 500 years for the strength limit state of wind and earthquake actions. For serviceability limit state, 20 years return period of wind speed is assumed.

For the building site in region A1, regional wind speed is 45 m/s for ultimate limit state and 37 m/s for serviceability limit state. By applying relevant factor of wind direction, terrain/height, shielding, and topograph, the design wind speed is designated to be 55.84 m/s. Therefore, the design wind pressure is 1.05 kPa on the windward side and 0.56 kPa on the leeward side. The wind action on the edge frames and interior frames can be obtained accordingly.

For this prototype building, the earthquake load effects are determined using the equivalent lateral force procedure outlined in AS1170.4-2007 (Standards Australia 2007) with base shear calculated using Equation 1. The earthquake loads and load effects are based on equivalent static lateral forces acting at each floor, distributed over the height of the building as shown in Equation 2. The sum of these forces is equal to the design base shear of the structure, with their distribution depends on the fundamental period of the structure.

$$V = \left\lfloor K_p Z C_h(T) S_p / \mu \right\rfloor W_t \qquad (1)$$

where K_p = 1.0 (for p=1/500 year earthquake)
Z = 0.08 (Melbourne/Sydney)
$T = 1.2$ s (moment-resisting steel frame)
$C_h(T)$ = 1.65 (class D soil) or 0.9 (class B soil)
S_p = 0.77 (OMF, limited ductile)
$\mu = 2$ (OMF, limited ductile)

$$F_i = \frac{W_i h_i^k}{\sum_{j=1}^{n}\left(W_i h_i^k\right)} V_t = \frac{W_i h_i^{1.35}}{\sum_{j=1}^{n}\left(W_i h_i^{1.35}\right)} V_t \qquad (2)$$

where F_i = horizontal static design force at the ith level
W_i = seismic weight of the structure at the ith level
h_i = height of level i above the base of the structure

The earthquake loading on the same building with heavy façade and light façade are calculated respectively. The lateral loads can be distributed to the edge frames and interior frames. All loads are factored and combined to provide design limit state loads.

The non-structural components such as infill walls, parapet walls, façade etc. are incorporated in seismic weight of the structure to obtain the equivalent lateral force under seismic actions. The scrutiny on detailed examination of the effect of non-structural components on the building structure is not within the scope of this study.

3.2 Frame analysis

The three dimensional building structure can be simplified as two dimensional composite moment frames based on the assumption that composite floors behave as rigid diaphragms and the perimeter composite moment frames work together with the interior frames in resisting the lateral load. The interior frames along grid lines B, C, and D are selected for frame analysis.

Frames subjected to gravity, wind, and earthquake loads must be designed to provide the necessary strength requirements and must satisfy the drift limits under various load combinations. Frames designed to sustain earthquake loading should also be designed and detailed to provide adequate ductility.

3.3 Design for strength

Structural analysis program SPACE GASS (Integrated Technical Software 2007) is used to obtain the preliminary moments and axial forces in the beams and columns due to both gravity and lateral loads. The moments and axial forces are then factored and combined for a preliminary selection of the beams and columns. An elastic second order analysis of the frame is performed and the member designs are checked for the various load combinations. The analysis is refined until final member sizes are selected. The detailed calculations of section and member capacities for the composite columns and composite beams, axial force and moment interaction for the concrete-filled steel tube and strength & stiffness of the composite beams at the sagging and hogging moment areas can be obtained in AS5100 Part 6. The structural stiffness is formulated based on centre line dimensions and rigid connections between beams and columns. It is found that the combinations of 1.2G + 1.5Q and G + 0.4Q +E govern the design of the columns and the beams.

3.4 Drift limits

Inter-storey drift is one of the significant parameters in assessing the performance of a structure under wind and earthquake loads. The control of drift is to ensure satisfactory occupancy requirements under wind loads and to minimize non-structural damage during earthquakes. The earthquake standard AS1170.4 recommends that inter-storey drift at the ultimate limit state shall not exceed 1.5% of the storey height for each level under earthquake loads. The design storey drift should be factored in accordance with Equation 3. Also, the standard AS/NZS 1170.0 recommends that the interstorey drift should not exceed $h_x/500$ when the building is subjected to specified wind for serviceability.

$$d_i = d_{ie} \frac{\mu}{S_p} \tag{3}$$

where d_i = design storey drift at the ith level

d_{ie} = storey drift at the ith level determined by an elastic analysis

The building is checked for the drift limit requirement and the design is refined until it meets the code requirement. The final member sizes for the interior frames are listed in Table 1. Analysis shows that the structural design is controlled by drift limits rather than strength requirement for the lower floors, in particular, the ground floor with the pin based columns.

These structural members are used in the following scenarios to explore the effects of connection rigidity, column base conditions, and façade types.

Table 1 Member sizes for interior frames

Level	Composite Column	Composite Beam
5	CHS 323.9 x 6	460 UB 67.1
4	CHS 323.9 x 6	460 UB 67.1
3	CHS 406 x 9.5	460 UB 67.1
2	CHS 406 x 9.5	460 UB 67.1
1	CHS 508 x 12.5	610 UB 101

4. Composite frame with rigid connection

4.1 Rigid frame with pin base

It is assumed that columns in the frame are pinned at the base. All the beam-to-column connections are rigid within the composite frame. The frames with heavy façade and light façade are studied under the load combinations of 1.2G+1.5Q, 1.2G+0.4Q+W, and G+0.4Q+E. The lateral load of earthquake induces moment reversals at some of the connections at level 1. Under these conditions, the slab can transfer very large forces to the column by bearing if the slab contains reinforcement around the column in the two principal directions. However, brittle failure modes of crushing of the concrete and buckling of the slab reinforcement should be prevented.

The inter-storey drift at levels 1 to 5 are 19.8 mm. 11.2 mm, 10.3 mm, 15.5 mm, 7.0 mm under the combination of G+0.4Q+E and 5.8 mm, 2.9 mm, 2.1 mm, 3.3 mm, 1.0 mm for the case of G+0.4Q+Ws. These inter-storey drifts are within the drift limits required by the code AS/NZS1170 which are listed in Table 2. If the heavy façade of concrete panels are replaced by light façade of curtain wall and glazing, the lateral drifts of interior frame with light façade under G+0.4Q+E are reduced to 17.0 mm, 9.7 mm, 8.9 mm, 12.7 mm, and 6.2 mm for levels 1 to 5 respectively. The reduction of frame drift is about 12% compared to the structure with heavy facade.

The ground floor is critical in the design for the drift limits. In the frame analysis, the beams provide about 49% of storey drift and the columns provide the remaining 51% of storey drift. At the ground floor, the design of columns and beams are governed by the drift requirement. However in the upper floors, the design of the frame is controlled by strength. The contribution of column and beam flexure to the storey drift can be estimated by Equation 4 for the ground level and Equation 5 for the upper levels for the cases of frames pinned at the base.

Table 2 Inter-storey drift limits

Level	Ultimate limit state (d_{ie})	Wind serviceability ($h_x / 500$)
5	20 mm	7 mm
4	20 mm	7 mm
3	20 mm	7 mm
2	20 mm	7 mm
1	23 mm	8 mm

$$\delta_i = \frac{\left(\sum V\right)_i h_i^2}{12}\left(\frac{1.5}{\sum(EI_b / L_b)_i} + \frac{4}{\sum(EI_c / h_i)_i}\right) \tag{4}$$

$$\delta_i = \frac{\left(\sum V\right)_i h_i^2}{12}\left(\frac{1}{\sum (EI_b / L_b)_i} + \frac{1}{\sum (EI_c / h_i)_i}\right) \tag{5}$$

where $\left(\sum V\right)_i$ =storey shear for level i

h_i = storey height for level i

I_c, I_b =moment of inertia of columns and beams

L_b = bay length

4.2 Rigid frame with fixed base

The column base fixity has great influence on the frame behaviour, in particular, the storey drift at the first level. The inter-storey drift for rigid frame with fixed base are 4.9 mm, 9.8 mm, 10.2 mm, 14.4 mm, and 6.9 mm for levels 1 to 5 under the combined load of G+0.4Q+E. The storey drift at the level 1 is only 4.9 mm compared to 19.8 mm for the frame with pinned base. Therefore, the column and beam size of the frame can be adjusted to the smaller dimensions as listed in Table 3. The inter-storey drifts for the frame with reduced member sizes are updated to 23.0 mm, 19.2 mm, 19.8 mm, 14.4 mm, and 7.0 mm for levels 1 to 5. These values meet the requirement of drift limits. The first storey drift of the frames with rigid base can be estimated by Equation 6.

Table 3 Member sizes for interior frames with fixed base

Level	Composite Column	Composite Beam
5	CHS 323.9 x 6	460 UB 67.1
4	CHS 323.9 x 6	460 UB 67.1
3	CHS 323.9 x 6	460 UB 67.1
2	CHS 323.9 x 10	460 UB 67.1
1	CHS 323.9 x 10	460 UB 67.1

$$\delta_i = \frac{\left(\sum V\right)_i h_i^2}{12}\,\frac{1}{1+\dfrac{\sum \dfrac{EI_c}{h_i}}{6\sum \dfrac{EI_b}{L_b}}}\left(\frac{1}{1.5\sum \dfrac{EI_b}{L}} + \frac{1}{\sum \dfrac{EI_c}{h_i}}\right) \tag{6}$$

5. Composite frame with semi-rigid connection

From practical point of view, the main difference between the design of unbraced rigid frame and semi-rigid frame is the contribution of the beam-to-column connection to the lateral drift and moment distribution. In order to meet drift limit requirements, it is necessary to adjust the stiffness of the columns, beams, and connection to achieve the optimal distribution of resistance to the drift. Normally, it is reasonable to distribute about equally to beams, columns and connection. However in low rise frames, it may be advantageous to let columns provide the majority of the resistance to drift. For instance, 40 to 50% assigned to columns and the rest divided equally between the beams and connections. In order to limit the drifts, the use of fixed column base is imperative in the design of semi-rigid frame. Therefore, attention should be paid to the detailing of the foundations and the column bases.

5.1 Semi-rigid frame with pin base

The frame models of interior and exterior frames with pin base are altered to include the semi-rigid connections. The initial stiffness of the beam-to-column connection is changed to one of four values, 1000 kNm/radian, 10,000 kNm/radian, 100,000 kNm/radian and 100,000 kNm/radian in the composite frame. As expected, the stiffness of the semi-rigid connection affects the internal force distribution in the frame elements (beams and columns). The beam bending moments are reduced and moments at the mid-span are increased compared to the case of rigid connection. It is favourable to utilize the section strength capacity of the composite beams at the mid-span and reduce the demand at the negative bending area. However, semi-rigid connections increase the storey drift dramatically as shown in Table 4 under the combined load of G+0.4Q+E. Subsequently large column and beam sizes for the ground floor need to be used in order to limit the drift. These semi-rigid composite frames need to be combined with concrete core or braced core to achieve the requirement of lateral displacement.

It is observed that the response of a frame with semi-rigid connection will get closer to that of the frame with rigid connections as the connections in the frame become stiffer. The required rigidity of the connection that behaves like a rigid connection is related to the stiffness ratio of the column to the beam (K_c / K_b) joined by the connection. A typical range of the stiffness of the rigid connection varies from 400,000 kNm/rad to 4500,000 kNm/rad.

The contributions of flexure of columns, beams and connection rotations to the storey drift can be estimated by Equation 7 for ground level and Equation 8 for upper levels for the pin-based frames with semi-rigid connections. It is worthwhile to note that Equations 4 to 8 are close-form solutions for solving storey deflection based on assumption that the inflection points of the beams are at mid-span in elastic analysis. For the inelastic seismic effect, this is considered by the factor of μ / S_p in Equation 3.

$$\delta_i = \frac{\left(\sum V\right)_i h_i^2}{12}\left(\frac{1.5}{\sum (EI_b / L_b)_i} + \frac{4}{\sum (EI_c / h_i)_i}\right) + \frac{V h_i^2}{12 \sum K_{con}} \quad (7)$$

$$\delta_i = \frac{\left(\sum V\right)_i h_i^2}{12}\left(\frac{1}{\sum (EI_b / L_b)_i} + \frac{1}{\sum (EI_c / h_i)_i}\right) + \frac{V h_i^2}{12 \sum K_{con}} \quad (8)$$

where K_{con} = stiffness of each connection at level i

Table 4 Inter-storey drift of pin-based frame with semi-rigid connections

	Inter-storey drift (mm)			
	1000 kNm/rad	10,000 kNm/rad	100,000 kNm/rad	1000,000 kNm/rad
Level 5	295.9	66.0	6.1	3.0
Level 4	312.2	85.0	10.8	6.3
Level 3	338.4	111.3	15.4	8.8
Level 2	369.3	141.8	21.9	12.5
Level 1	453.3	203.3	44.1	31.0

5.2 Semi-rigid frame with fixed base

For the case of semi-rigid frame with fixed base, the storey drift demand can be met with connection stiffness of 10,000 kNm/radian or greater. For example, the inter-storey drift for the fixed base frame with connection stiffness of 100,000 kNm/radian are 8.2 mm, 16.0 mm, 16.4 mm, 19.4 mm, and 9.7 mm for level 1 to 5 under the combined load of G+0.4Q+E. It shall be noted that although a great effect of the base fixity on frame drift is observed; such degree of fixity of column base may not be achieved easily in practice as most of the footings are not perfectly rigid.

6. Conclusions

A five storey moment-resisting composite frame has been designed and analysed in accordance with current Australian standards. Within the present scope of investigation, the following conclusions can be made.

- For low-rise building structure, drift limit controls the design of the ground floor. For the upper floors, the moment at negative bending region control the composite beam design.

- The use of semi-rigid connection in the composite frame tends to increase lateral deflections substantially, albeit the moment at the negative bending are reduced.

- The fixity of column base has great influence on the storey drift and moment distribution of edge columns.

- The composite moment-resisting frames can provide substantial reserve capacity, reliable force distribution mechanisms, and certain degree of ductility. In addition, they can also provide benefits at the service load level by reducing deflection and vibration control.

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