ROBUSTNESS CONSIDERATIONS IN COMPOSITE DESIGN

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Abstract

With a desire for ever increasing building heights, offset lift cores and irregular building floor plates, robustness is an important design consideration in the design of tall, steel and concrete composite buildings. In the design for robustness, the structural engineer may adopt an approach based upon identifiable accidental actions or unidentifiable accidental actions, such as specifying a minimum horizontal tying force. This paper outlines a literature review of the minimum tying force requirements in buildings outlined in the relevant Australian, American and European design codes. The case study is of a 438 metre tall tower in Doha under construction. The building provides accommodation for retail, office, hotel and residential uses over 90 levels. The typical tower floors are comprised of pre-cast slab units, with a structural topping, supported by perimeter composite steel edge beams. The tower columns are concrete filled steel tubes up to 1800 mm in diameter filled with 80 MPa high strength concrete

Introduction

In the design building of tall buildings, robustness is an important design consideration. As defined in BS EN 1991-1-7, “Robustness is the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause”. One approach adopted by international design codes is to state a minimum tying force between the structural members, which may be either a nominated load or percentage of the tower column ultimate load. In the case of a composite tall building where column loads may be substantial, the connection between the tower column and floor diaphragm and the ability of the composite floor system to transfer induced horizontal forces back to the core is critical.

This paper outlines a literature review of relevant international design codes and literature and the findings of a case study.

Review of International Standards and Literature

British Standard BS5950-Part 1 2000 Code of Practice for the Design – Rolled and Welded Sections

In BS 5950 Part 1, the restraining element is to have both sufficient strength and stiffness to prevent movement in the appropriate position or direction. In Clause 4.7.1.2, restraints to compressional members are to be capable of resisting a force not less than 1.0% of the axial force in the column. In bracing systems that provide to more than one member should be designed to resist the sum of the restraint forces from each member that they restrain, reduced by the factor kr where:
$k_r = (0.2+1/Nr)^{0.5}$ where Nr is the number of parallel restraints.

In Steel Construction Institute publication “Lateral Stability of Steel Beams and Columns – Common cases of restraint” Section 1.7 the following guidance is given:

For single restraint at any location	2.5%
Intermediate restraints	1% for any one restraint
Multiple restraints	2.5% summed over all intermediate restraints
Continuous restraint	2.5 % uniformly distributed along the member.

There is no guidance regarding restraint stiffness in this publication.

British Standard BS8110-Part 1 1997 – Structural Use of Concrete – Code of Practice for Design and Construction

In BS 8100 Part 1 Clause 3.12.3.6.1, each external column should be anchored or tied horizontally into the structure at each floor level with a tie capable of developing the greater of:

1 2.0 Ft or ($l_s/2.5F_t$) where l_s is the floor to ceiling height and F_t is the lesser of (20+4no) or 60 kN where no is the number of storeys in the structure, or

2 3% of the total design ultimate vertical load carried by the column at the level.

AS 1170 AS1170.0:2002 –Structural Design Actions: Part 0: General Principals

In AS 1170.0, Clause 6.2.3 states all parts of the structure shall be interconnected with the connections capable of transmitting 5 percent of G+ ψ Q, where ψ is a combination factor. In order to transfer this force, Clause 6.2.4 states floor diaphragms shall be designed:

- To resist required horizontal forces, and
- To have ties or struts (where used) able to distribute the required wall anchorage forces.

Australian Standard AS 4100 - 1998 Steel Structures

In AS 4100, the restraining element is to have sufficient strength to prevent movement in the appropriate position or direction.

In Clause 6.6.2, the restraint element is to be designed for the greater of:

- Restraining force as specified in Clause 6.6.1 (which refers to notional horizontal force of 0.2% times the total design vertical load as specified in Clause 3.2.4), and
- 2.5% of the maximum axial compression force in the member.

With reference to the commentary of AS4100 Clause C6.6.2, when the restraints are more closely spaced than is just required to obtain the member capacity, then the group of restraints as a whole is to transfer to 2.5% of the maximum axial compression force in the member.

With reference to the commentary of AS4100 Clause C6.6.2, restraint stiffness for centrally braced columns is satisfied with the 2.5% rule.

ANSI/AISC 360-05 Specification for Structural Buildings

In AISC 360-05, two general types of bracing systems are considered. Relative bracing controls movement of the brace point with respect to the adjacent braced points. Nodal bracing controls movement at the braced point without interaction of adjacent braced points.

With reference to Appendix 6.2, the required nodal brace strength is 1% of the required column axial load and stiffness as specified in this clause. In the case a multiple of restraints, the nodal brace strength is 1%.

Eurocode 2: Design of Concrete Structures

In EC2 Clause 9.10.2.4, each external column should be anchored or tied horizontally into the structure at each floor level with a tie capable of developing the greater of:

- $F_{tie.fac}$ per metre of façade of 20 kN/m, but not to exceed $F_{tie.col}$
- $F_{tie.col}$ of 150 kN maximum.

The above values are recommended values and are dependent on the country requirements in its National Annex

Eurocode 1: Actions on Structure

In EC1 Annex A, the approach adopted in dependent on the building classification. For tall buildings, a systematic risk assessment of the building is recommended to take into account foreseeable and unforessable hazards.

Shankar Nair (2003) "Simply Solutions to Stability Problems in the Design Office"

In this paper, simple solutions are derived for stability problems in multi-storey buildings, such as erection imperfections and horizontal floor movements due to the deformation of the lateral load resisting system. The paper recommends the connection between the column and floor diaphragm should be designed for 0.6% times the axial force in the column.

Based upon a review of the above literature, the magnitude of the minimum horizontal tying force varies significantly. This would indicate these minimum requirements were not intended for the application for a tall building and an update of current design standards would be recommended.

Therefore, in the following case study the following approach was adopted:

- Minimum horizontal tying force of 1% of the ultimate axial force in the column, or

- Bracing systems that supply positional restraint to more than one member should be designed to resist the sum of the restraint forces from each member that they restrain (1% of the ultimate axial force in the column), reduced by the factor kr where:
  $k_r = (0.2+1/Nr)^{0.5}$ where Nr is the number of parallel restraints where k_r is not less than 0.6.

Summary

Case Study: Dubai Tower Doha

Dubai Tower Doha is a 438 metre tall tower building, including a 36 metre spire, currently under construction in Doha, Qatar. The lateral stability of the tower is provided by the high strength reinforced concrete core in conjunction with composite steel and concrete outriggers and perimeter belt trusses at plant-rooms. The perimeter columns are engaged in the wind frame by the outriggers. The perimeter columns are a combination of high strength reinforced concrete and concrete filled steel tubes. The tower floors above Level 8 are comprised of pre-cast slab units, with a structural topping, supported by perimeter composite steel edge beams. The progressive set-back for floors above Level 41 is supported by a series of steel raking trusses. The basement and podium slabs are cast in situ beams and slabs. The tower foundations comprise of a pile supported raft

To maximise the available floor space, the tower design set many structural engineering challenges. With concrete cylinder design strengths up to 80MPa adopted in the design of reinforced concrete core walls and columns, concrete filled steel tubes outside current code slenderness limits, significant lateral loads to meet robustness requirements, complex load transfer paths due to column eccentricities and progressive floor set-back and architectural wall openings, new techniques for analysis and design were adopted outside normal engineering practice. An extensive review of international design standards and published papers was carried out to select the appropriate document for guidance on the above design challenges.

Figure 1: Building Elevation (Image courtesy of RMJM)

Figure 2: Building Evolution (Image courtesy of RMJM)

Structural Floor Framing

The tower floors above Level 8, except the plant room slabs, comprise of hollow core pre-cast slab units, with a structural topping, supported by perimeter composite steel edge beams.

This type of floor construction was adopted for the following economic and time benefits:

- Unpropped construction, with spans up to 12.50 metres. Therefore the requirement for secondary beams was omitted
- Reduced construction cycle time compared to insitu-concrete options
- Reduced weight and depth of the steel section relative to a non-composite design

- The natural precamber of the precast which offsets imposed load deflections
- Precast units and steel beams are manufactured off site

Based upon a review of both international design codes and reports, the composite steel edge beams were designed to Steel Construction Institute Publication P287 [6]. The steel beams are bolted to a through plate which pass through the centreline of the concrete filled steel tube.

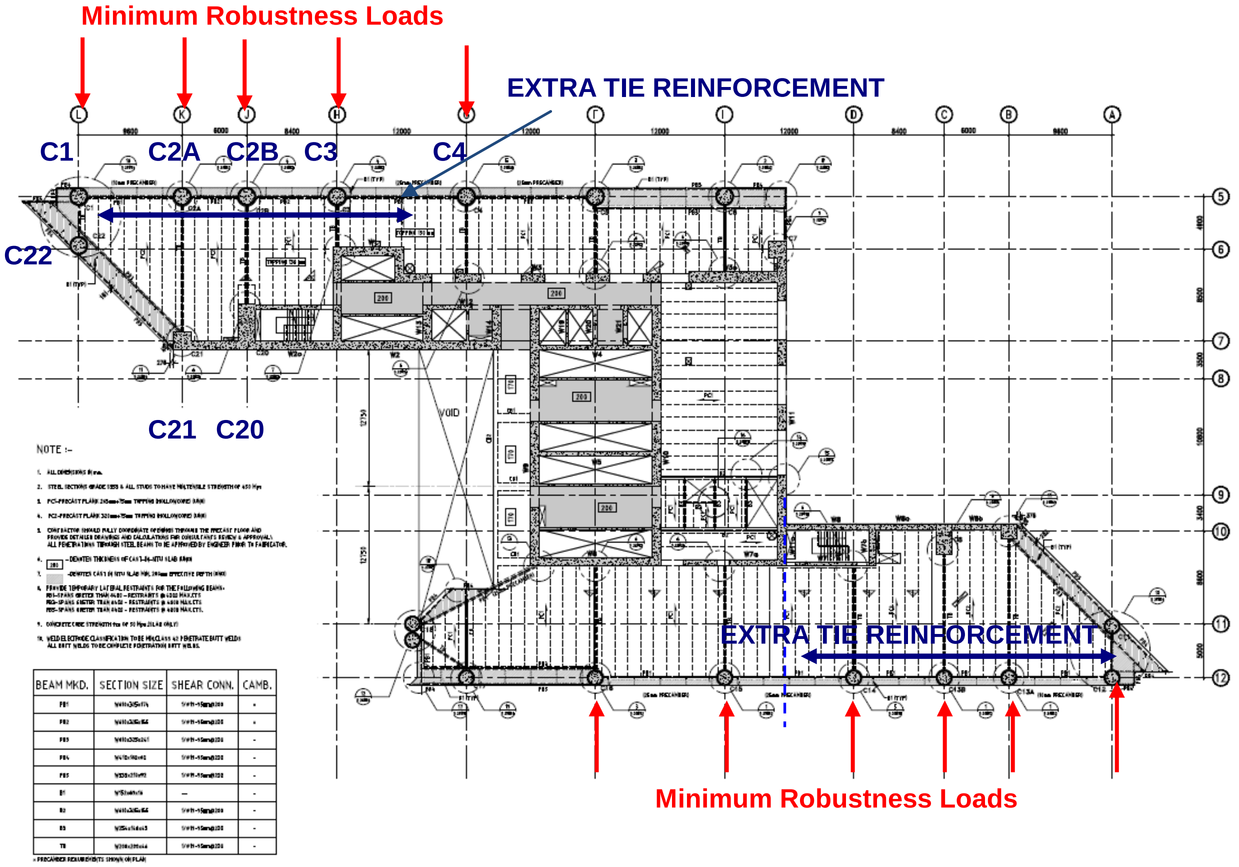

Figure 2: Structural floor framing system and robustness / diaphragm action

Robustness / Diaphragm Action

In determining the minimum tying force, a horizontal load equivalent to 1% of the vertical ultimate column load was applied to the structure at each level. At Level 23, the tower column loads and minimum tying force are summarised in Table 1.

Column Tag	DL (MN)	LL (MN)	Wu (MN)	1.2DL+1.5LL (MN)	1.2DL+0.4LL+Wu (MN)	P_{ulT} (MN)	$0.01P_{ult}$ (MN)
C1	49	7	10	69.3	71.6	71.6	0.72
C2A	50	7	5	70.5	67.8	70.5	0.71
C2B	66	10	8	94.2	91.2	94.2	0.94
C3	45	7	13	64.5	69.8	69.8	0.70
C4	51	8	14	73.2	78.4	78.4	0.78
C5	51	8	18	73.2	82.4	82.4	0.82
C20	126	19	10	179.7	168.8	179.7	1.80
C21	53	9	7	77.1	74.2	77.1	0.77
C22	58	7	13	80.1	85.4	85.4	0.85

Table 1: Column Loads at Level23

Since more than one horizontal load per level was applied, a reduction co-efficient was applied to the each 1% load. This co-efficient was dependent on the number of the columns considered to be restrained at any one time.

By considering the following two different cases:

- Column C4 tied directly to the core with a steel tie beam – with only one column restrained, a reduction co-efficient was not applied. Hence the tying force was taken as 1% of the ultimate column load
- Columns C1, C2A, C2B, C3, C22 and C20 - with a total of 6 columns considered restrained, a reduction co-efficient of 0.60 was applied. Hence the tying force was taken as 0.6% of the column load when more than one horizontal load is applied per level. This percentage was determined to be consistent with results recommended in Nair's paper.

To obtain the most unfavourable effect on the diaphragm in terms of shear and in-plane bending all the loads were applied in the same direction, with reverse action also considered. In the typical floor plate shown in Figure 2, the two wing areas act as cantilever elements supported by the transverse core walls. The resultant tension force was resisted by additional steel reinforcement within the structural topping, and in some instances on the lower levels a proportion of the tension was resisted by the perimeter steel beams and through the connections. The resulting shear force was taken by the reinforced topping slab. Refer to Figure 3 for a typical connection where fillet welds were specified around the cleat plate for additional strength in tension. In the cases where the tower column is directly opposite the core, the column is restrained by an internal steel tie beams and 5-T25 reinforcement bars in the structural topping.

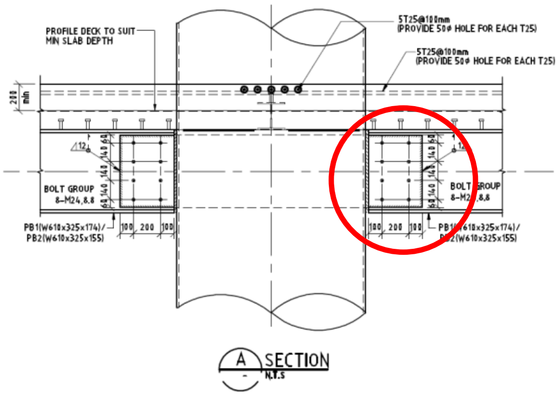

Figure 3: Typical beam connection detail to resist diaphragm action forces

A typical cross section of the hollow core planks and increased structural topping thickness and additional reinforcement at the lower levels due to increase shear is shown in Figure 4. The hollow core is opened at every second core and a transverse reinforcement bar placed similar to the detail shown in **Figure 5** to provide shear connection between the hollow core and structural topping.

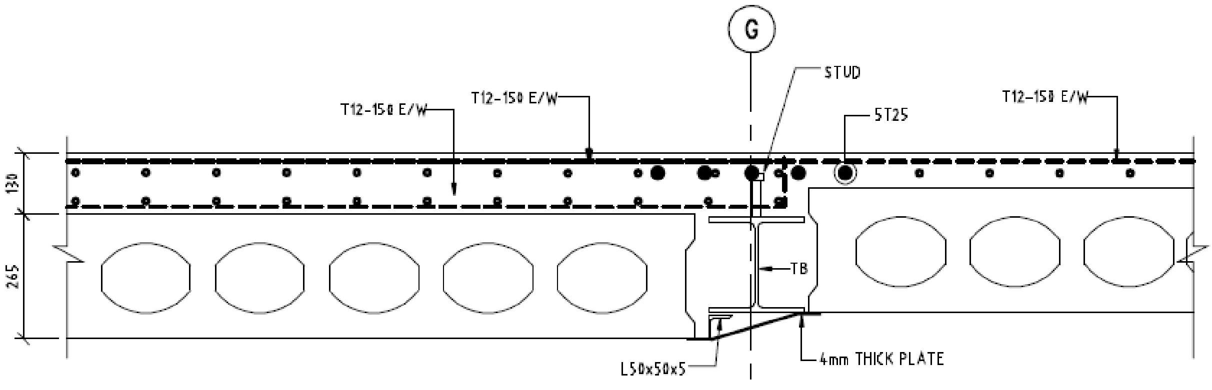

Figure 4: Typical hollow core and structural topping section

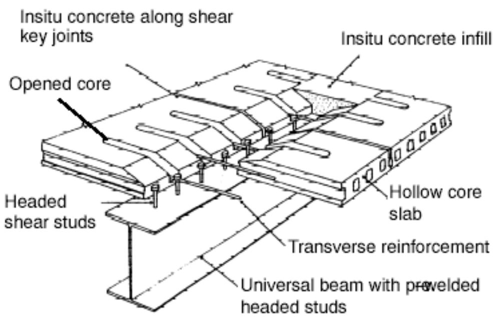

Figure 5: Hollow core / structural topping shear connection

After analysing the horizontal design loads on the structure it was established that the nominal robustness loads on the connections between vertical elements and floor systems was the governing design horizontal action for the diaphragm. Due to the high vertical loads in the columns, the robustness loads were found to be greater than the horizontal wind loads at the lower levels of the tower

Conclusion

In the design building of tall buildings, robustness is an important design consideration. The approach adopted by the structural engineer may be based upon identifiable accidental actions or unidentifiable accidental actions, such as specifying a minimum horizontal tying force.

Based upon a review of the above literature, the magnitude of the minimum horizontal tying force varies significantly. This would indicate these minimum requirements were not intended for the application for a tall building and an update of current design standards would be recommended.

In the case study presented in this paper, the following approach is adopted:

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