Displacement Capacity of Structural Frames Incorporating Moment Resisting Anchored Blind Bolted Connections

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Abstract

Welded connections with external or internal diaphragms that are connected to the beam flanges are currently the commonly used option for achieving a moment-resisting connection to concrete filled steel tubular columns in regions of high seismicity. This is not favoured and practical in Australia because of high expenses of site welding as well as the quality control requirements that are difficult and expensive to undertake. This paper presents results from an ongoing investigation into the development of moment resisting connections using innovative anchored blind-bolts in structural frames. Using different configurations of anchored blind bolts within concrete filled square hollow section (CFSHS) columns, moment resisting connections with different levels of stiffness and strength can be achieved. The behaviour of blind bolted connections has been determined using experimental data and elaborate FE modelling. These connections have been employed in a structural frame around the perimeter of the building. The moment resisting perimeter frames provide the lateral force resisting system. The stability of the frame and the strength hierarchy of the frame are investigated considering both material and geometrical non-linearities. The drift levels obtained using a response spectrum analysis based on the bare frame properties are compared to the results from an equivalent static analysis based on AS-1170.4. Finally, the capacity curve is achieved by performing a nonlinear push-over analysis on the structure and the capacity spectrum approach is used to determine whether the capacity is adequate for various earthquake levels.

Keywords: Anchored blind bolts, Moment resisting connections, CFSHS.

1. INTRODUCTION

Concrete-filled steel tubular columns are becoming popular in multi-storey building frames due to their excellent structural capacity, good fire resistance and speed of construction (Han et al., 2010). Several attempts have been made to develop moment resisting bolted connections to tubular column sections (France et al., 1999; Lee et al., 2010; Morita, 1994; Wang et al., 2009). The design and practical use of concrete filled columns, and the benefits of using them, have been well documented (ANSI /AISC 360-10, 2010; Bergmann et al., 1995). Bolted connections have not been easily applicable to these sections due to the lack of access to their inner face. This problem could be solved using blind bolts which can be installed only from one side. However, the flexibility of the section face, especially for rectangular hollow sections, has been an obstacle to achieve to moment resisting connections. In order to increase the contribution of infill concrete in the stiffness and capacity of connections, the blind bolts have been anchored into the infill concrete. Two different types of anchored blind bolts have been developed and tested at The University of Melbourne in collaboration with Ajax Fasteners (Ajax Engineered Fasteners) and the results of these tests were presented previously (Agheshlui et al., 2012). The type of anchored blind bolt that was used in this study is depicted in Figure 1. This type of anchored blind bolt, referred to as Headed Anchored Blind Bolt (HABB), is a high tensile threaded bar with two nuts, one at the end and another one bearing on the tube wall (Figure 1). It provides anchorage to the infill concrete and bearing to the tube wall. It is relatively inexpensive to fabricate and can easily be adjusted for any required embedment length. Application of this type of bolt has been tested in concrete filled square hollow sections (CFSHS) and concrete filled circular hollow sections (CFCHS) (Agheshlui, et al., 2012). It has been concluded that connections using this type of anchored blind bolt can be classified as semi-rigid for unbraced frames with varying degrees of rigidity depending of the bolt configuration.

Figure 1. Headed Anchored Blind Bolt (HABB)

This paper discusses the feasibility and sufficiency of application of the proposed anchored blind bolted connections to structural frames in low to moderate seismicity areas. A 5-storey prototype office building, four-bay by four bay, located on a site class D in Melbourne or Sydney is designed. The lateral resistance is provided by moment resisting perimeter frames which employ anchored blind bolted connections. The frame is classified as an ordinary moment-resisting frame with limited ductility. Design loads, including dead, live, earthquake and wind are estimated based on current Australian loading specifications AS/NZS 1170.0 (2002). EC4 (European Standards, 2004) has been used for design of composite steel and concrete columns and beams. Column bases were assumed to be fixed which based on experimental results is a reasonable assumption (Hsu et al., 2004).

2. CASE STUDY: A TYPICAL OFFICE BUILDING

The floor plan of the building is shown in Figure 2. It is 33.6m x 33.6m in area with four bays of 8.4m in each direction. The longer bay spacing makes it more representative of an Australian practice. The first floor of the building is 4m high, while the others are 3.5m high.

The frame is composed of concrete-filled square hollow section (CFSHS) columns, composite steel beams with concrete slab, and anchored blind-bolted moment connections. The lateral force resisting system is a perimeter moment resisting frame with anchored blind bolted semi-rigid connections. The floor system is a 120 mm thick concrete slab which develops a composite action with beams and connections of perimeter frames and increases their strength. The connection type and its properties, including the best available estimation of the connection stiffness, are respectively illustrated in Figure 3 and Table 1. Four anchored blind bolts plus two ordinary structural bolts used at the top side of the connection provide adequate resistance against negative design moments while the two anchored blind bolts at the bottom resist the probable positive moment in the event of a moment reversal due to an extreme event. The structural bolts are connected to side stiffeners (PFC sections) and transfer the tensile force to the back of the column. As it can be seen, the detailing required for this connection is much simpler than the known welded connections.

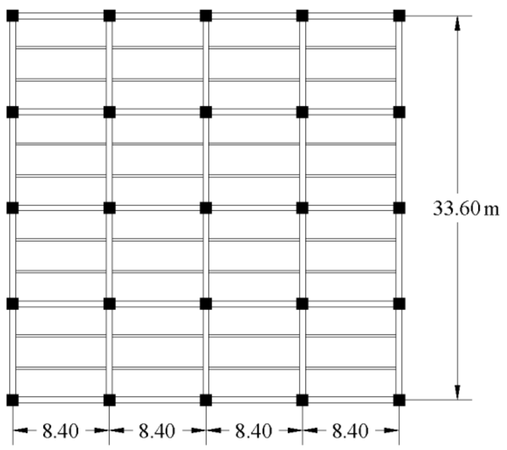

Figure 2. Plan view-Case study office building

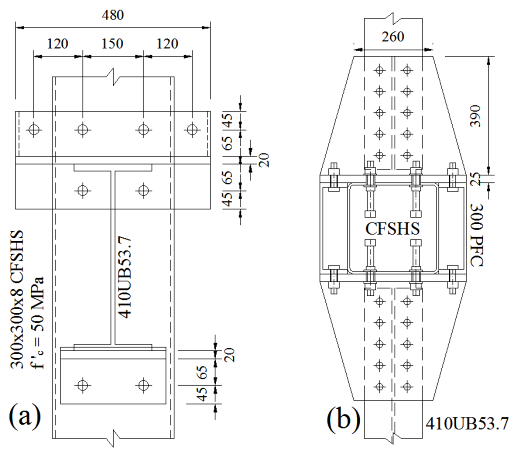

Figure 3. Connection to CFSHS columns; (a) Elevation view; (b) Plan view

Table 1. Properties of the proposed connection

Column Section	Connection Type	Elastic Stiffness (kNm/rad)	Plastic Stiffness (kNm/rad)	Serviceability moment capacity (kNm)	Ultimate moment capacity (kNm)
CFSHS	Semi-Rigid	71,600	34,730	260	384

3. LOADING AND DESIGN

The gravity loads applied to the model building included a 5kPa dead load, 3kPa live load for floors and 1.5kPa for the roof. External walls with gravity load of 2kPa were considered on perimeter frames. Earthquake loads were determined in accordance with AS 1170.4 (2007). Considering that the case study building is at an importance level of 2, in accordance with AS/NZS 1170.0 (2002), it needs to be designed for an earthquake with probability of exceedance of 1/500 ($k_p = 1$). However, as discussed in section C3.1 of the commentary to AS1170.4 (Standards Australia, 2007), designing for a return period of 2500 years is considered internationally to result in a more uniform risk of collapse when comparing regions of low to moderate seismicity with those of high seismicity. This is because the 2500 return period event is likely to be greater than the 500 year return period event by a larger factor in regions of low to moderate seismicity than in regions of high seismicity (O'Brien et al., 2010). Hence, the building studied here is designed for two different performance objectives of damage control and collapse prevention using respectively earthquakes with return periods of 500 and 2500 years. The seismic forces are applied in two perpendicular directions at the mass centre of each floor as illustrated in Table 2. A 10% eccentricity was also applied as required by AS 1170.4 (2007). The wind loads were calculated in accordance with AS1170.2 (2011). Ultimate wind pressures of 0.72 kPa windward and 0.514 kPa leeward were applied. The serviceability wind load pressures calculated for a return period of 25 years (AS/NZS 1170.0, 2002) were 0.48 kPa windward and 0.35 kPa leeward. Wind loads were then calculated and applied at each floor based on the corresponding tributary areas. The considered load combinations were: 1.35G, 1.2G+1.5Q, G+0.3Q+EQ, and 1.2G+0.4Q+W_u for stability control and strength design and W_s for serviceability control. The storey drift of 1.5% was controlled under damage control earthquake level and maximum of 0.2% storey drift was checked under serviceability wind load.

Table 2. Base shear for Damage Control and Collapse Prevention Earthquakes

Storey, i	Height H_i (m)	Seismic Weight W_i (kN)	Storey Base Shear Proportion, k_F	Storey Shear $k_F \times V_{500}$ (kN)	Storey Shear $k_F \times$V2500 (kN)
5	18	6,623	0.354	621	1,119
4	14.5	7,094	0.284	497	895
3	11	7,094	0.195	342	616
2	7.5	7,094	0.116	204	367
1	4	7,161	0.050	88	159
SUM		35,065	1	1,753	3,156

For perimeter moment resisting frames, the connections design was governed by gravity loads with the max negative moment of -295 kNm. The beam section was designed to be 410UB53.7, grade 300. In accordance with Eurocode 3 (European Standards, 2005), the connection secant stiffness (S_j) was compared to the flexural stiffness of the connected beam (EI_{bm}/L_{bm}) to define the rigidity of a beam-to-column connection. For a 410UB53.7 beam with 8.4 m span, EI_{bm}/L_{bm} is equal to 4476 kNm/radian; hence, the connection is considered to be rigid when S_j is larger than 111,900 KNm/rad ($25EI_{bm}/L_{bm}$) for unbraced frames and 35,810 KNm/rad ($8EI_{bm}/L_{bm}$) for braced frames, and is assumed to be semi-rigid when S_j is between these two values. Beams in interior frames were assumed to be simply supported and to develop a composite action with a 120 mm thick concrete slab to carry positive moments. The fundamental period of the building was found to be 1.87 seconds. The maximum storey drift under serviceability wind of 37 m/s was calculated as 5.4 mm which was less than 0.002 of the storey height (=7 mm) according to AS1170.0 (AS/NZS 1170.0, 2002). The maximum storey drift under 500 year return period earthquake (52 mm) was also observed to be less than the acceptable limit (1.5% of the storey height =52.5 mm) according to AS1170.4 (2007).

4. DESIGN PHILOSOPHY AND STRENGTH HIERARCHY

As explained in section 3, strength and serviceability designs have been conducted for the case study frame under 500 year return period earthquake. The structure will behave elastically under this level of earthquake. However, the collapse prevention performance objective also needs to be considered. The structure may enter into the nonlinear behaviour region and reach the displacement demand under a 2500 year return period earthquake by its inelastic ductile behaviour. The ductile behaviour of the frame needs to be controlled and designed to occur at predefined locations. Obviously columns must remain intact to retain the structural stability. Also, Australian practice requires long spans with large beam sections. Due to high plastic moment capacity of beams, plastic hinges are not likely to occur in beams.

Eventually, the connections become the preferable choice for the plastic joints. The source of plastic behaviour in connections is designed to be at the top T-stub. The yielding is designed to commence at about 60% of the anchored bolts capacity. Hence, the bolts that are essential for keeping the connection safe remain undamaged. This also retains the shear capacity of the connection. Therefore, the inelastic deformation is solely limited to the end-plates. FE modelling performed in ATENA program has been used to design the endplates to yield at about 60% of the anchored bolts capacity. The connection plastic behaviour was entered into a frame analysis software as a multi-linear curve and then the frame was analysed under 2500 year return period earthquake. The plastic hinges developed under this loading were determined and the stability of the structure was controlled to be satisfied. The plastic hinges did occur at the first floor's beam-column connections. The values of inter-storey stability coefficients were calculated and are presented in Table 3 in accordance to AS1170.4 (AS 1170.4, 2007). Since the calculated coefficients are between 0.1 and 0.2, the structure is stable; however the P-Δ effect needs to be considered. This has already been taken into account.

Table 3. Storey drifts and Inter-storey stability coefficients

Storey, i	Design Storey Drift (mm) 500 year return period EQ	Design Storey Drift (mm) 2500 year return period EQ	Stability Coefficient, θ_i 2500 year return period EQ
5	24.4	43.9	0.04
4	37.7	67.9	0.09
3	47.8	85.8	0.15
2	49.9	90.0	0.19
1	35.9	64.7	0.14

5. DRIFT ESTIMATION USING RESPONSE SPECTRUM METHOD

In this part, the deflections obtained using the equivalent static method according to AS1170.4 (2007) are compared to the results of the more rational method of the displacement response spectrum. The storey design drifts are presented in Table 3 and the displacement spectra compatible with the Australian earthquake standard is depicted in Figure 4 for a soil class D with 500 and 2500 year return periods. The calculated natural period of the building is 1.87 seconds which lies in the displacement controlled region. The displacement demand for both considered earthquake levels can easily be found looking at the RSD diagram (Figure 4). This displacement is the top displacement of the equivalent substitute structure which its height is calculated to be 73% of the height of the original building. Hence, the maximum displacement of the building at the roof elevation will be RSD_{max} / 0.73. This is the maximum possible displacement based on the response displacement spectrum method. These values are compared to the values obtained from equivalent lateral load method in Table 4. As it can be seen, the results from the latter are dramatically higher than the results from the former. This is because the maximum probable displacement based on response spectrum analysis is limited to RSD_{max} (Figure 4), for high period structures; whereas, the equivalent lateral load method does not introduce any upper limit for calculated displacements. Furthermore, code based static lateral load method assumes a shorter period for the case study building (1.2 seconds instead of 1.87). Therefore, a higher level of base shear is calculated for a stiffer structure and then applied to the bare frame model which is less stiff. This scales the response displacements up by a factor of $(T_{bare\ frame}/T_{code})^2$. This issue has not been addressed in AS1170.4 (2007); however, ASCE 7 (2005) permits the elastic drifts to be determined using seismic design forces based on the computed fundamental period of the structure.

Table 4. Max displacements- Calculated by response spectrum and equivalent lateral load methods

Earthquake Return Period	RSD_{max} (mm)*	Top Disp. (mm) (=RSD_{max} / 0.73)	Calculated Disp. (mm)* Equivalent lateral load method
500 years	58	79	196
2500 years	104	142	352

6. CAPACITY SPECTRUM APPROACH

The capacity spectrum (ADRS diagram) method compares the demand and the capacity of a structure. Here, the capacity of the building frame is obtained by performing a nonlinear push-over analysis on the structure. The lateral load pattern considered was in accordance to the equivalent lateral load distribution. Geometrical and material nonlinearities were considered. The Force-Displacement push-over curve was then converted to Acceleration-Displacement curve. Equations presented in Lam et al. (2004) were used to draw the ADRS diagrams, shown by the black and blue lines in Figure 5. In the nonlinear part of ADRS diagrams, which represent the velocity controlled regions, the acceleration is equal to RSV_{max}^2 / Displacement (Lam, et al., 2004). Eventually, the demand curves and capacity curve are intercepted to obtain the performance point of the structure. As it can be seen in Figure 5, the structure is capable of tolerating lateral displacements even higher than $RSD_{max,2500}$.

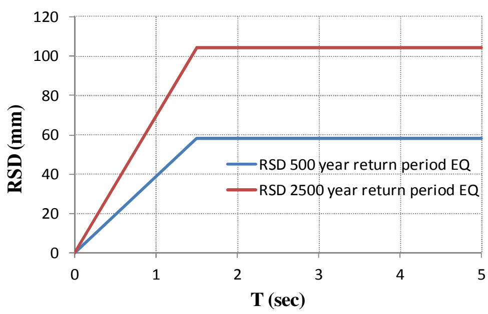

Figure 4. Code compatible displacement response spectra for soil class D

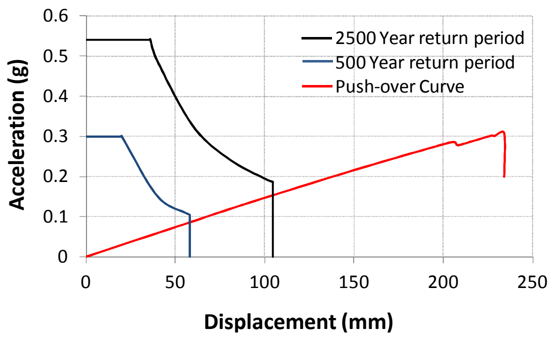

Figure 5. Capacity Spectrum for 500 and 2500 year return period earthquakes

7. CONCLUSION

A new type of bolted connections between steel beams and concrete filled square hollow section columns has been introduced. Anchored blind bolts were used to establish these connections. The connections were implemented into a typical office building model which was located in a low to moderate seismicity region and on a class D soil. The building was designed for the two performance objectives of damage control and collapse prevention for earthquakes with return periods of 500 and 2500 years. The ductility of the structure under an extreme event was designed to be provided with the inelastic behaviour of the connection T-stubs; while the bolts were not damaged. Stability of the structure against lateral seismic loads was controlled and it was concluded that a perimeter frame system using semi-rigid blind bolted connections is a viable option. Furthermore, since the equivalent lateral load method uses a shorter period to find the base shear acting on the building, it results in unrealistic displacements. This issue has been addressed in ASCE 7 (2005) by permitting the designer to use the computed fundamental period of the structure to find the base shear for the purpose of calculating displacements. Also, the code compatible response spectrum method defines a limit for the maximum displacement of the structure (RSD_{max}); whereas there is not such a limit considered when using equivalent lateral load method. Furthermore, the capacity spectrum approach was used to determine the performance points of the building for the considered lateral load scenarios. It was shown that the displacement capacity of the building determined by nonlinear push-over analysis was higher than the displacement demands defined by RSD_{max}.

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