

BLIND-BOLTED MOMENT RESISTING CONNECTIONS TO CONCRETE-FILLED STEEL TUBULAR COLUMNS



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SUMMARY:

Despite the merits of concrete-filled steel tubes, connecting a steel I-beam to a circular hollow section column using conventional bolts is difficult due to lack of access to the inside of a hollow section. To avoid welded connections, the blind-bolted connections are designed to be easy to install to facilitate quick erection of the frames. This paper presents results from an ongoing investigation into the development of moment resisting connections using innovative blind-bolts with extensions. Results of an extensive experimental program are given to demonstrate the degree of effectiveness of these blind bolt groups. The use of either cogged or headed stud extensions enables the blind bolts to be anchored back into the concrete within the tube. Different types of moment-resisting connections are being developed, and their suitability for frames in regions of low seismicity and frames in regions of high seismicity is controlled by modelling a typical building.

Keywords: Moment resisting Connections, Concrete filled hollow sections, Anchored blind bolts

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 & Li, 2010). Several attempts have been made to develop moment resisting bolted connections to tubular column sections (France et al., 1999a, 1999b, 1999c; Lee et al., 2010a, 2010b, 2011; Wang et al., 2009; Yao et al., 2010). It has been shown that a higher degree of strength and stiffness for bolted connections to steel tubes can be achieved by filling them with concrete (France et al., 1999-c), and this also increases the axial load capacity of the column. The design and practical use of concrete filled (composite) columns, and the benefits of using them, have been well documented (ANSI /AISC 360-10, 2010; Bergmann R et al., 1995; Wang YC & Moore DB, 1997). In order to increase the contribution of infill concrete in the stiffness and capacity of the blind bolts and consequently to enhance the connection behaviour, 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). For the first type of anchorages, a cogged extension was welded to the head of a Onside blind bolt (Figure 1). This is referred to as the cogged anchored blind bolt (CABB). This was tested in concrete filled CHS columns (CFCHS). However, the difficulty and cost of fabrication of this type of blind bolt as well as the added weld breaking mode of failure, led to the development of the second type of anchored blind bolt, the Headed Anchored Blind Bolt (HABB), which is a high tensile threaded bar with two nuts, one at the end and one bearing on the tube wall (Figure 2). It provides both anchorage to infill concrete and bearing on the tube wall. It is less expensive to fabricate and can easily be adjusted for any required embedment length. This has been tested as single and groups of bolts in concrete filled SHS (CFSHS) columns. Their application within CFCHS columns is also under investigation; testes on single bolts have been completed and those on groups will be carried out within the next phase of the project. The connections built using CABB and HABB types of bolts can be classified as semi-rigid for unbraced frames with varying degrees of rigidity depending of the configuration adopted.



Figure 1. Cogged Anchored Blind Bolt (CABB)



Figure 2. Headed Anchored Blind Bolt (HABB)

2. EXPERIMENTAL PROGRAM

2.1. CABB Moment Connection to a CFCHS Column

The test specimen of the blind-bolted moment connection to a CHS column is shown in Figure 3. It is composed of a concrete-filled circular hollow section, two curved T-stubs, and a universal steel beam. The beam was deliberately chosen to be oversized so that the behaviour of the connection under extreme loads would be observed. The basic components of the connection are listed in Table 1. The curved T-stubs were fastened to the circular tube by CABBs. The CABB assembly consists of a blind bolt with a circular head, a cog extension to the bolt head, a sleeve for shear, a split stepped washer, and a stepped washer as depicted in Figure 1. The anchored extension welded to the head of the blind bolt was a short cog with 24 mm diameter N type reinforcing bars of grade 500 MPa. The grade 8.8 Ajax blind bolts have a minimum tensile strength of 800 MPa and yield strength of 640 MPa. A coupon test of a welded blind bolt with anchored extension was performed as a check on the weld capacity. It fractured at 211 kN in tension, approximately equal to the yield strength of the 24 mm extension (226 kN) and this was considered adequate for the purpose of the connection test. The circular hollow section was filled with concrete of 45 MPa target compressive strength. On the day of testing, the compressive strength of the concrete had an average value of 48 MPa, determined by compression tests on three standard cylinders. On assembly, all of the blind bolts connecting the T-stub endplates to the circular hollow section and the structural bolts connecting the T-stub flange plates to the beam were first tightened to a snug-tight condition and were then fully tensioned using the part-turn method as specified in Australian steel standard AS4100 (Standards Australia, 1998).

Table 1. Components of the blind-bolted moment connection to CFCHS column

Components	Size	Length (mm)	Grade
Column section	CHS324x6	1800	350
Beam section	460UB82.1	2300	300
Curved endplate	25 mm Thk	270	300
Web plate of T-Stub	20 mm Thk	390	300
Blind bolt	20	105	8.8
Extension	24	100	500
Structural bolt	20	75	8.8

The setup of the connection test is illustrated in Figure 3. The bottom of column was inserted into a socket plate which was bolted to the laboratory's strong structural floor by pretension. The top of the column was braced through a collar plate to the floor by four bracing members. An axial load of 800 kN (approximately 20% of the column axial capacity) was applied to the top of the column through a rigid steel plate. The loading applied to the beam end consisted of a series of load controlled cycles followed by a series of displacement controlled cycles as shown in Figure 4. Finally the specimen was loaded to failure. Six transducers were mounted on the top T-stub to measure the pullout of blind bolts, the deformation of the curved endplate and web plate, slip at the flange plate connection, and horizontal and vertical shear displacement of the T-stub relative to the column in the tension zone. Three transducers were fixed on the bottom T-stub in the compression zone to record the horizontal and vertical displacement relative to the column. An additional transducer was installed on the ground to monitor the beam end deflection. Digital photogrammetry was also used as a supplementary

approach to trace the deformation of the beam-to-column specimen and the potential relative movement between the specimen and testing rig. An initial survey was taken before the test to record the original conditions, then a series of surveys were performed after applying an axial load of 800 kN on the column, at beam end load levels of 30 kN, 60 kN, 90 kN, 120 kN, 150 kN, 180 kN, 210 kN, and at beam end deflections from 75 mm to 135 mm at increments of 10 mm. Afterwards, the specimen was continuously loaded to failure.

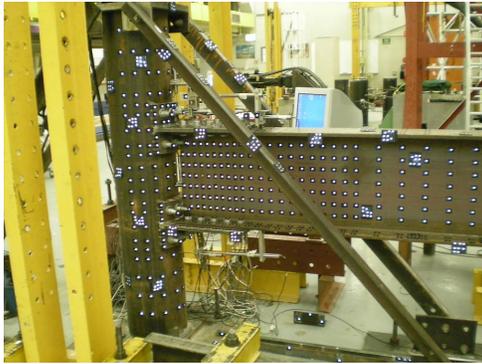


Figure 3. Test setup for the CABB connection

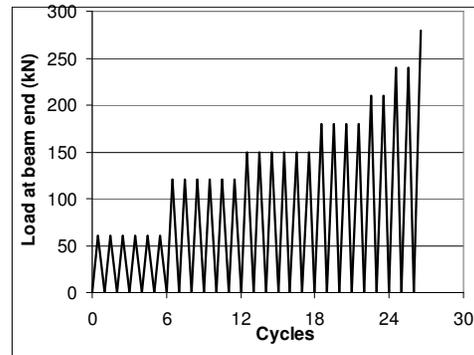


Figure 4. Loading cycles for the CABB connection

2.2. HABB Moment Connection to a CFSHS Column

The specimen constructed for testing groups of HABBs is shown in Figure 5 and the basic components of the connection are listed in Table 2. The L-stubs were fastened to the square concrete filled tube by HABBs. The SHS column was filled with concrete with an average compressive strength of 50 MPa (average of three cylinders tested on the date of the experiment). For connecting L-stubs to the column four different groups of anchored bolts were used and tested. These included groups of 3M16 HABBs, 2M20 HABBs, 3M20 HABBs and a group of 2M20 HABBs used together with 2 ordinary structural M20s which were fastened to side stiffeners. The bolt groups connected angles to the concrete-filled column which were, in turn, connected to the top and bottom beam flanges at opposite sides of the column. All the bolts were headed stud anchored blind bolts unless otherwise mentioned. For the connection with side stiffeners, these stiffeners were used to transfer the tension load to the back face of the column in order to increase strength and stiffness of the connection.

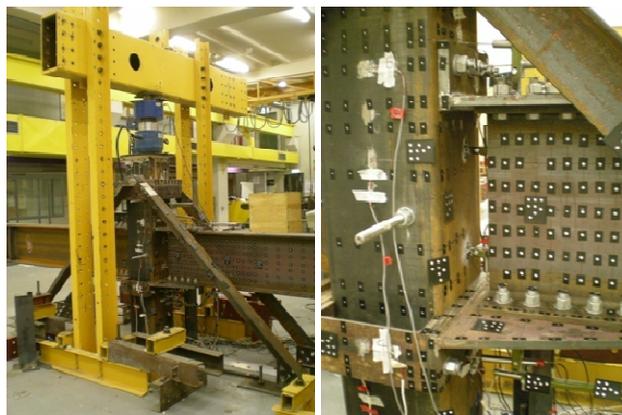


Figure 5. General view of the specimen and the connections

2.3. Pull-Out Tests of CABBs within a CFCHS

Two types of supplemental tests were conducted to investigate the anchorage performance of single blind bolts with cogged extensions. The first test examined the blind bolt bearing on the thin tube wall

without an anchored extension. It involved testing M16 and M20 blind bolts bearing on a CHS 324x6 section. The second test checked the improved response of providing anchored cogged extensions to blind bolts. It involved testing M20 HABBs situated at the bottom T-stub in the connection test that has been described in section 2.1. These anchors had not been loaded during the connection test since the compression due to bending was transferred through bearing of the endplate on the tube wall. The anchorage tests were set up as depicted in Figure 6. The concrete-filled steel tube was laid against two thrust blocks bolted to the structural floor. A coupler connected the tail of the blind bolt on one side and a high tensile threaded rod on the other side. This threaded rod passed through the reaction frame and was fixed into a gripping mechanism. The tests were run under a monotonic displacement-controlled force arrangement using a hydraulic jack with a displacement rate of approximately 1mm/minute. Four LVDTs were mounted on the threaded rod immediately after the coupler to measure the relative displacements at a predefined distance from the surface of the tube. The needles of two outer transducers were set on the tube wall, whereas those of two inner transducers were set on a reference plate that was secured to the coupler adjacent to the tube. Hence, the outwards displacement value was calculated as the difference between the average value of the outer two LVDT measurements and the inner two LVDT measurements. To monitor the strain on the steel tube, two strain gauges were mounted at 50 mm away from the edge of the hole in the longitudinal and transverse directions.

Table 2. Components of the blind-bolted moment connection to CFCHS column

Components	Size	Length (mm)	Grade
Column section	SHS300x8	1800	350
Beam section	460UB82.1	2300	300
L-Stub's vertical plate	25 mm thk	270	300
L-Stub's horizontal plate	20 mm thk	390	300
Blind bolt	16 & 20	100 (embedment)	880
Structural bolt	M20	75	8.8
Side Stiffener	300PFC40.1	140	300

2.4. Pull-Out Tests of HABBs within a CFSHS

A group of tests were conducted on blind bolts anchored within a 300×300×8 mm square hollow section. It tested anchorage performance in a grade 350 tube filled with a concrete with a 50 MPa compressive strength. The blind bolts were made of high tensile threaded bars and were of grade 8.8. The diameter of the nuts was limited by the size of the hole on the tube wall. For M20 and M16 bolts, the diameters of the circular nuts were 29 mm and 23 mm, respectively. The details of the specimens within the concrete-filled square column are listed in Table 3. For notation of specimens: M refers to a bolt installed at the middle of the tube, S refers to a bolt installed close to the tube corner, and 16 and 20 refer to the HABB diameter. The embedment length chosen for the blind bolts was the longest practicable for the dimension of the tube when the bolts are installed on two opposite faces of the tube. Two typical locations were selected for investigation of behaviour of HABB type of blind bolts (Figure 7), i) bolt installed at the middle of the tube face (centreline at 150 mm from the edge of the tube wall); ii) bolt installed close to the corner of the tube face (centreline of 75 mm from the edge of the tube wall). The test and the instrumentation were similar to those used for the circular column.

Table 3. Test specimens of HABBs anchored in a CFSHS column

Specimen	Tube size	Tube grade	Concrete (MPa)	Bolt dia. (mm)	Embedment (mm)	Location
HABB_M16	SHS300 x 8	350	50	16	100	Middle
HABB_S16	SHS300 x 8	350	50	16	100	Side
HABB_M20	SHS300 x 8	350	50	20	100	Middle
HABB_S20	SHS300 x 8	350	50	20	100	Side

2.5. Anchorage Test on Blind Bolt with Double-headed Studs

The HABBs were also tested in concrete-filled circular hollow sections with diameter of 324 mm and

thickness of 6, 8, and 10 mm. The compressive strength of the concrete infill was designated to be 45 MPa. The actual measured compressive strength of the concrete was 47.8 MPa at the day of test (average of results from three cylinders). The detailed features of the test specimens are listed in Table 4 indicating the tube size, bolt diameter, concrete strength, and embedment length. In the specimen notation, T refers to thickness of tube wall, D refers to bolt diameter, and N refers to nut. The definition of N1 and N2 is similar to one presented in section 2.4. The tests were setup in a similar way to that described previously in section 2.42.3.



Figure 6. Test setup for CABBs anchorage test



Figure 7. Blind bolts in a square hollow section

Table 4. Test specimens of blind bolts with double-headed studs anchored in CHS column

Specimen	Tube size	Concrete (MPa)	Bolt dia. (mm)	Embedment (mm)	Nut dia.(mm)
T6_D16_N1	CHS324 x 6	47.8	16	0	23
T6_D16_N2	CHS324 x 6	47.8	16	100	23
T6_D20_N1	CHS324 x 6	47.8	20	0	29
T6_D20_N2	CHS324 x 6	47.8	20	100	29
T8_D16_N1	CHS324 x 8	47.8	16	0	23
T8_D16_N2	CHS324 x 8	47.8	16	100	23
T8_D20_N1	CHS324 x 8	47.8	20	0	29
T8_D20_N2	CHS324 x 8	47.8	20	100	29
T10_D20_N1	CHS324 x 10	47.8	20	0	29
T10_D20_N2	CHS324 x 10	47.8	20	100	29

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

3.1. Moment-Rotation Curves for CABB and HABB Connections within a CFCHS and CFSHS

The moment-rotation response of the CABB connection within a CFCHS column is shown in Figure 8. The response of the connection is almost linear for moments less than 260 kNm. The ultimate moment capacity of the connection is about 600 kNm. The secant stiffness of the connection used for serviceability purposes is defined at 2/3rd of the ultimate moment (400 kNm) and is equal to 135,000 kNm/rad. The behaviour of the connection is ductile mainly due to the deformation and damage to the tube face and it shows considerable rotation before failure. The moment-rotation curves for four different groups of HABBs are depicted in Figure 9. These groups of bolts were used at the top and bottom sides of connections to the CFSHS column. Applying cyclic loading, moment-rotation curves were achieved for each of the bolt groups. During the experiment the L-stub yielded and produced extra rotation. Ultimate and serviceability moment capacities for different bolt groups as well as serviceability rotational stiffnesses obtained by deducting the yielding effect of the L-stubs are presented in Table 5. In two cases, the observed moment capacities were higher than the maximum moment achievable based on the bolt group's tensile capacity (shown in brackets in Table 5). In these cases, the ultimate moment capacity and the associated serviceability moments need to be limited with the bolt group's tensile capacity. 2/3rd of the ultimate moment capacity is taken as the serviceability limit moment. Stiffness for this level of moment has been calculated and is used as connection stiffness up to the serviceability moment limit. As it can be observed, a group of 3HABBs has not

shown a better performance compared to a group of 2HABBs. This suggests that the contribution of the bolt in the middle of the square section was insignificant.

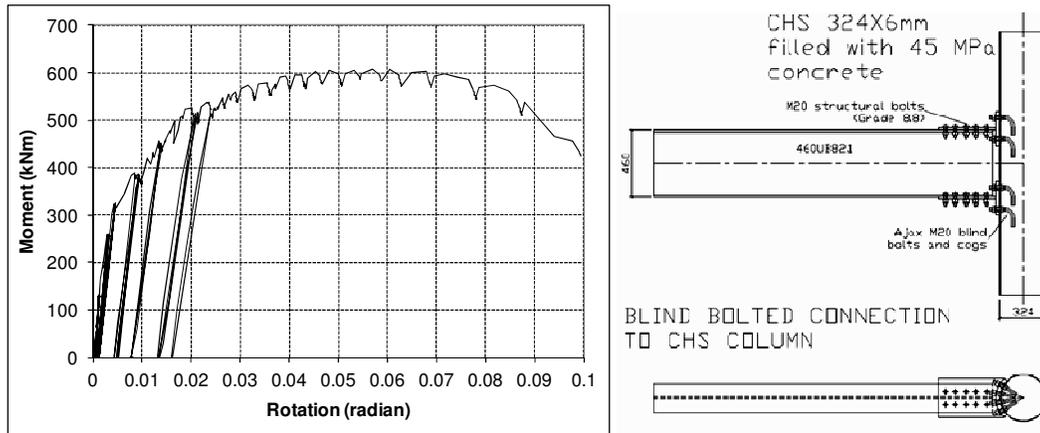


Figure 8. Moment vs. rotation of the CABB connection (shown in the right)

Table 5. Ultimate moment capacity and serviceability stiffness for HABB Connections within CFSHS

Bolt Groups	Ultimate Moment Capacity (KNm)	Serviceability Moment Capacity (KNm)	Stiffness at Serviceability Moment (KNm/rad)
2M20 HABBs+2Structural bolts	347	230	38,600
3M20 HABBs	240	160	30,900
2M20 HABBs	245 (192)	130	38,200
3M16 HABBs	200 (187)	125	23,600

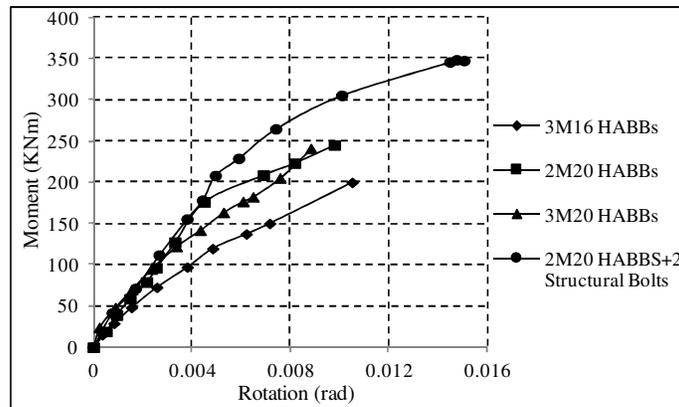


Figure 9. Moment vs. rotation curves for HABBs connections

3.2. Load versus Outwards Displacement of CABBs within CFCHS

The load versus outward displacement curves for the anchorage tests with and without cogged extensions are presented in Figure 10. Each curve represents the average values obtained by testing two nominally identical specimens. In the anchorage tests without extensions, the blind bolts were set up to directly bear against the thin tube wall through the internal split washer. The hole on the tube wall gradually opened up and the bolts were pulled out from the tube. The initial stiffness of the M16 and M20 blind bolts bearing on the 6 mm thick CHS was similar up to an applied load of 50 KN. The M16 then became more flexible due to the more concentrated stresses. In the anchorage tests with cogged extensions, the anchored extension in the tube greatly improved the blind bolt performance. The M20 CABB fractured outside the tube at a load of 217 KN. The deformation of the tube wall was very small and the overall outwards displacement was less than 3 mm at the ultimate load.

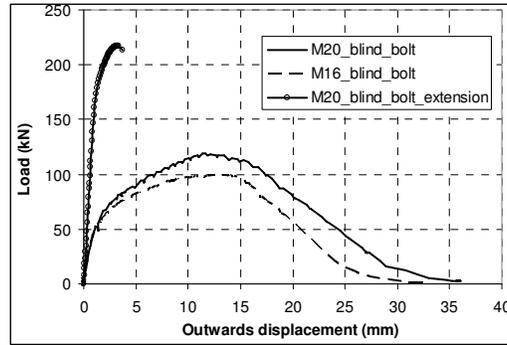


Figure 10. Anchored blind bolt with cogged extension (CABB)

3.3. Load versus Outwards Displacement of HABBs within CFCHS and CFSHS

Figure 11 to Figure 13 show the load versus outward displacement of the 16 and 20 mm diameter blind bolts with and without anchorages in the CHS324 column with tube wall thicknesses of 6, 8 and 10 mm. It is apparent that both the strength and stiffness of the connected bolts are significantly increased by providing the headed studs with 100 mm extensions into the concrete core compared with Oneseid blind bolts in specimens T6_D16_N1, T6_D20_N1, T8_D16_N1, T8_D20_N1 and T10_D20_N1. The thickness of the tube wall has a pronounced effect on the specimens with the conventional blind bolts. The strength of the connected bolt in a 10 mm thick tube reached 215 kN in specimen T10_D20_N1, whereas only 119 kN was achieved for specimen T6_D20_N1 with a 6 mm thick tube. However for HABBs, even using 6 mm thick plate ultimate tensile capacity of the threaded bar is achieved for pull-out capacity.

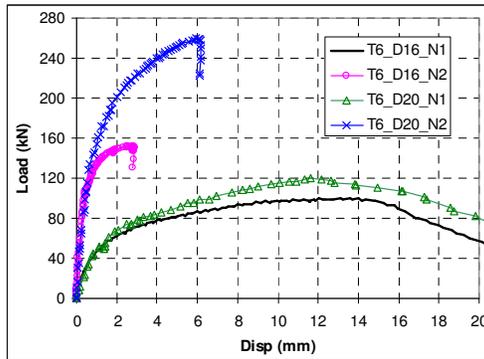


Figure 11. Blind bolts anchored in CHS324x6

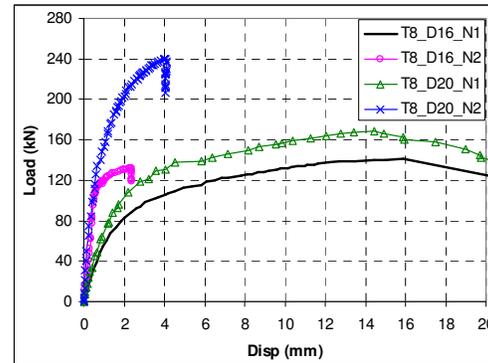


Figure 12. Blind bolts anchored in CHS324x8

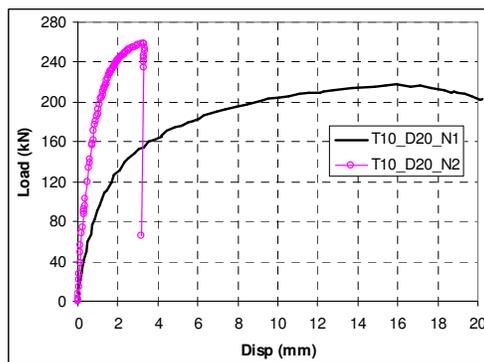


Figure 13. Blind bolts anchored in CHS324x10

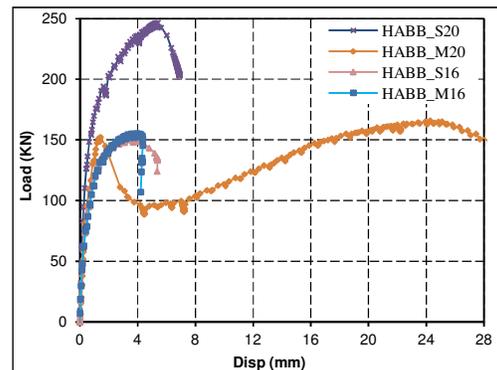


Figure 14. HABBs within CFSHS

Figure 14 show the load vs. outward displacement relationship for HABBs anchored in 50 MPa concrete. It was observed that HABBs generally failed by bolt fracture except for specimen HABB-M20 which failed by tube yielding and bolt pull-out. As a result, a drop followed by gradual increase of strength due to strain hardening was observed. The blind bolts installed adjacent to the tube corner were stiffer than those installed at the middle of the tube. Using anchored blind bolts, a high level of stiffness and strength can also be achieved for connections to CFSHS columns.

4. USE OF ANCHORED BLIND-BOLTED CONNECTIONS IN STRUCTURAL FRAMES

This section discusses the feasibility and sufficiency of application of the proposed anchored blind bolted connections to structural frames. 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. Design loads, including dead, live, earthquake and wind loads are estimated based on current Australian loading specifications AS/NZS 1170 (Standards Australia/Standards New Zealand, 2002). EC4 (European Standards, 2004) was used for design of composite steel and concrete columns and beams. Column bases were assumed to be fixed. 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 Figure 15. The longer bay spacing makes it more representative of Australian 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. The frame is composed of concrete-filled steel tubular columns, steel beams with composite slabs and blind-bolted moment connections. Two different lateral force resisting systems were studied. The first system used perimeter moment resisting frames with semi rigid connections to CFSHS columns. Based on experimental results, in CFSHS columns using HABBs semi-rigid connections are achievable, also due to clashing of bolts in perpendicular directions, only perimeter moment resisting frames are applicable. In the second, rigid connections to CFCHS columns were achieved using CABBs. Due to the circular shape of the section, use of rigid connections in orthogonal directions was possible and all frames of the structure were assumed to be moment resisting. Details of the frames are provided in Table 6.

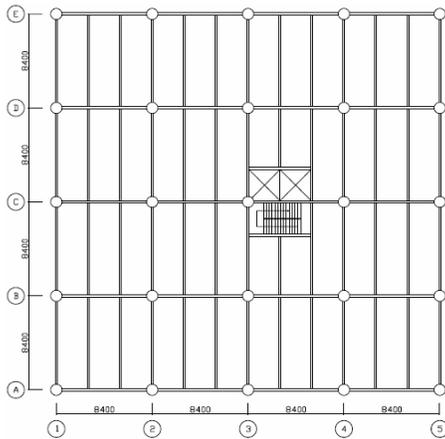


Figure 15. Plan view of a typical office building

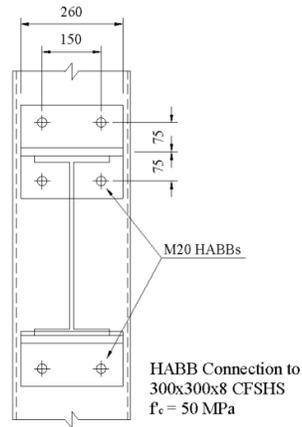


Figure 16. HABB connection to CFSHS columns

For column sections, the same sizes as the experiments were employed to be able to use the experimental results in modelling without any adjustments. For connections to CFSHS columns using HABBs, details are given in Figure 16. This is a relatively simple connection. Detail of the connection to CFCHS sections using CABBs is illustrated in Figure 8. This is a more complicated connection which provides a higher stiffness and strength and in that a curved end-plate is used and the cogged welded extensions. The beam size for the mentioned actions was designed to be 410UB53.7, grade 300. In accordance with Eurocode 3 (European Standards, 2005), connection secant stiffness (S_j) is 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). The connection is considered to be rigid when S_j is larger than $25EI_{bm}/L_{bm}$ (=111,900 KNm/rad) for unbraced frames and $8EI_{bm}/L_{bm}$ (=35,810 KNm/rad) for braced frames, pinned when S_j is less than $0.5EI_{bm}/L_{bm}$ (2238 KNm/rad), and is assumed to be semi-rigid when S_j is between these two values. Stiffness and capacity of the connections to CFSHS and CFCHS columns are tabulated in Table 6. Beams were assumed to develop a composite action with a 120 mm thick concrete slab to carry positive moments in their sagging parts. For not simply supported beams, in addition to positive mid-span moments, negative moments (as well as positive moments due to moment reversal under 2500 year return period) occur at connections that need to be carried by anchored blind bolted connections. The influence of composite action at connection areas is ignored; however, if using reliable design details, this effect could be taken into account that would considerably improve the connection's stiffness and moment capacity. In Table 7, the natural period, drift under different lateral loads, and design bending moments are presented for two different frames constructed with circular and square columns. Although all the connections of the second frame are rigid, due to lower flexural stiffness of the circular section used in here compared to square sections, the natural periods of the frames were similar. The storey drifts under serviceability wind of 37 m/s were controlled to be less than 0.002 of storey height (=7 mm) according to AS1170.0 (Standards Australia/Standards New Zealand, 2002). The values of natural period are also compared to the values calculated based on AS1170.4 (given in brackets) and it can be seen that code estimated values were considerably less than the actual period of the structures. Since horizontal static shear force due to earthquake is inversely related to the structure's period with the order of two, the higher code period produces higher shear forces, which to be conservative, have been used here. The drifts of different stories under 500 year return period earthquake were observed to be less than acceptable limit (1.5%) according to AS1170.4 (Standards Australia, 2007). Since Australia is located in a low seismicity region, the suggested earthquake loads for 500 year return period are significantly less than the ones in high seismicity regions. Therefore, in recent years there has been an interest in checking the buildings for 2500 year return period earthquake as well. The frame has also been checked for this and it has been observed that the structure will be safe under the 2500 year return period earthquake.

Table 6. Proposed Connections' stiffness and strength (units: KN-m-rad)

Frame No.	Lateral load resisting system	Column Section	S_j	Connection Type	Serviceability moment capacity	Ultimate moment capacity
1	Perimeter moment resisting frames	CFSHS	74,100	Semi-Rigid ($16.5 EI_b/L_b$)	260	384
2	All frames moment resisting	CFCHS	135,000	Rigid ($30 EI_b/L_b$)	400	600

Table 7. Period, drifts and connection moments of the sample frame using anchored bolts (units:KN-m-rad)

Column Section	Natural Period (sec)	Drift due to W_s (mm)	Max Drift(%) 500yr EQ	Max Drift (%) 2500yr EQ	Pos. Moment 500yr EQ	Neg. Moment 500yr EQ	Pos. Moment 2500yr EQ	Neg. Moment 2500yr EQ
CFSHS	1.89 (1.2)	5.4	0.57	0.97	0	-207	29	-270
CFCHS	1.95 (1.2)	4	0.51	0.94	0	-235	69	-327

5. CONCLUSION

In this paper extensive experimental results for two new types of anchored blind bolts are presented. These bolts are to be used in moment resisting connections to concrete filled steel tubular columns. The connection behaviour for different groups of HABBs and CABBs are discussed as well as single pull-out test results of these bolts within CFSHS and CFCHS. It has been revealed that the optimum number of bolts to be used in this particular CFSHS column is two and they should be used close to the corner of the section. Having a T-stub with four HABB bolts at top, a desirable level of stiffness and strength can be achieved and the connection is categorized as a moderately stiff semi rigid connection which can potentially be used in medium rise buildings with perimeter frames in low seismicity regions. Furthermore, due to the high level of confinement provided by circular sections in

CFCHS columns, groups with more numbers of bolts have shown an excellent stiffness. The connection using 8 bolts for top T-stub has reached to a very high level of stiffness and is classified as a rigid connection in unbraced frames. This type of connection can potentially be used in medium rise moment resisting frames in high seismicity regions. A typical five storey office building using concrete filled columns with proposed anchored blind bolted connections is modelled and it is shown that the proposed connections are capable of carrying typical connection moments and they also provide sufficient stiffness as lateral load resisting systems. Furthermore, from the viewpoint of strength hierarchy, connections in these types of frames are the weakest part and the T-stub can be designed to yield under an extreme event to act as a fuse. In summary, the newly developed type of fastening and connections are shown to be suitable options for solving the problem of moment resisting bolted connections to concrete filled tubular columns.

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