Improved Performance of Moment Resisting Connections to Concrete Filled Square Hollow Sections Using Double Headed Anchored Blind Bolts

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Abstract

Concrete filled square hollow sections (CFSHS) are becoming very popular in building construction due to their advantages in strength, ductility and slenderness. However, they are not commonly used in Australia and developing countries due to the quality and cost issues in connecting the steel beams to the CFSHS columns. Welding, which is the common method of achieving a moment resisting connection in steel construction, is not preferred in these countries and normal structural bolting cannot be used due to lack of access to the inner side of the hollow sections. A comprehensive three dimensional finite element model has been developed to simulate the experimentally observed behaviour of a full-scale sub-assemblage test of a double-T blind-bolted connection from a perimeter frame in a medium-rise office building. The sub-assemblage consisted of a concrete-filled square hollow section as the column, and two universal beams composite with a concrete slab which were connected at each side of the column using the double-T connections. Gravity loads were applied followed by cyclic loading representing the effect of an earthquake. Good agreement was obtained between the FE results and experimental results. In the test the type of blind bolts used were HABBs (headed anchor blind bolts). To further improve the behaviour of the blind bolted connection in terms of stiffness and cyclic deterioration, a modified type of blind bolt called a double headed anchored blind bolt (DHABB) was developed. The modification includes an increase in the bearing area of the blind bolts by adding an extra embedded head, and tests have been carried out to ascertain the properties of these bolts. The results of an FE analysis of the sub-assemblage in which the DHABBs are substituted for the HABBs are presented here and a comparison is made to show the improved behaviour.

Keywords: Moment resisting frame, blind bolt, numerical analysis, composite construction

INTRODUCTION

Large floor spans with open column-free spaces are usually preferred by clients and architects in building. They are expensive, so not usually an engineer's hot choice. For this reason a moment resisting frame system with concrete filled steel tubes as columns has become very popular in some countries. When concrete filled steel square hollow sections (CFSHSs) are used as the columns, they have several advantages over reinforced concrete or steel columns. The CFSHSs have a high capacity with favourable ductility. Time and cost can be saved during construction since the steel tube acts as both the formwork and reinforcement. However, the use of CFSHSs is not very popular in Australia because of difficulties associated with connections between them and steel beams. Welding is sometimes used to make these connections but in many countries welding is not preferred at site because of quality and cost issues. Furthermore, some forms of welded connections did not perform well in past earthquakes (SAC, 1995) and welds do not cope well with high localised deformations or stress concentrations.

In recent years, several studies have been conducted to overcome the problems associated with welded connections. A number of blind bolted connections have been proposed (France et al., 1999; Pitrakkos and Tizani, 2013; Yao et al., 2008). (Agheshlui, 2014) did a full scale sub-assemblage test using Headed Anchored Blind Bolts (HABBs) and obtained valuable information required to understand the connection behaviour. A finite element study conducted by (Pokharel et al., 2015) provided important information in understanding the connection behaviour in more detail. This paper presents the improved behaviour of blind bolted connections with Double Headed Anchored Blind Bolts (DHABB). Firstly, a comprehensive FE model with HABBs was prepared and verified with experimental results. Then the verified FE model is further extended by replacing HABBs with DHABBs.

EXPERIMENTAL PROGRAM

A typical office building was design in accordance with (Standards-Australia, 2007). The building was located in a low to moderate seismic region with spectral acceleration, S_a=0.08g. The span of the perimeter frame was 8.4m and that of internal frames was 12.6m in both direction. The plan dimensions of the building were 50.4x50.4m and the building was 18m tall (5 storey) as used in (Pokharel et al., 2014). It was made with CFSHSs as columns and composite Universal Beams (UBs) were connected to the CFSHSs with double T-Stubs with blind bolted connection as shown in Figure 1. The main lateral load resisting members are the perimeter frames of the building. Thus, one of the perimeter joints was selected for the sub-assemblage test.

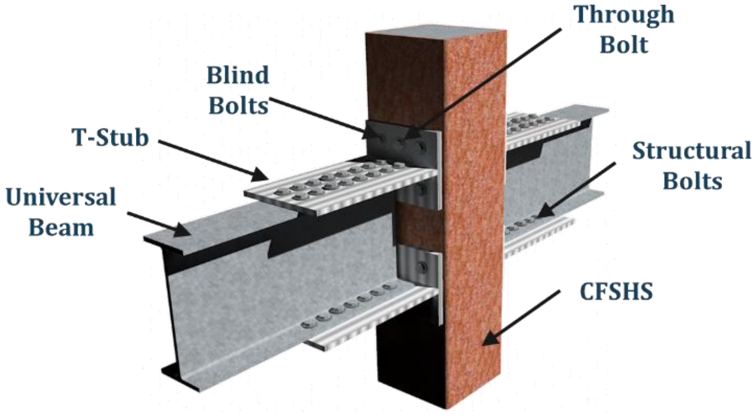
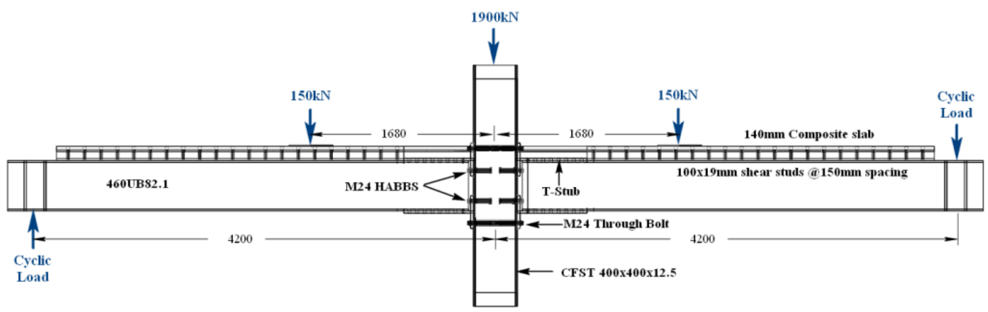


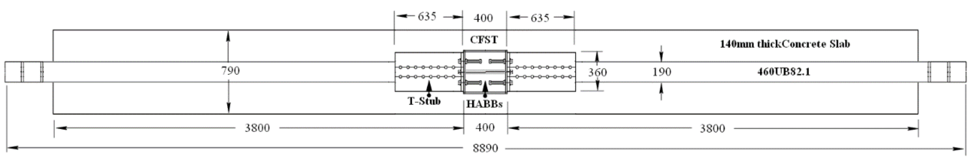
Figure 1: Connection of Beam to CFSHS using double T-Stub with blind bolts on both side

The full scale test was conducted by (Agheshlui, 2014) with a 2.2 m high CFST 400x400x12.5 as the column and a 4.2 m long 460UB82.1 composite with a 140mm thick composite slab on each side as shown in Figure 2. The slab width used represented the effective width at the connection region in accordance with Eurocode 4 (CEN, 2004). It was placed symmetrically about the column for ease of construction.

The gravity loading was selected to replicate realistically the moment distribution in the beam in the connection region and the axial load in the column prior to the application of lateral loads. This is taken to be the dead load plus 30% of live load as is usual in the loading combination that combines gravity and earthquake loading in AS1170.4 (Standards-Australia, 2007). An axial load of 1900kN was applied at the column and a gravity load of 150kN was applied to beam at a distance of 1.68m (which is 0.2L, L being the span between columns) on each side measured from the column centre-line. The gravity load was kept constant throughout the test. Cyclic loading was applied at the end of the beams. The details of the test design, setup and results can be found in (Agheshlui, 2014).



(a) Side View



(b) Top View

Figure 2: Details of test specimen

FE ANALYSIS

FE Model

A comprehensive 3D finite element model was developed using ABAQUS/Explicit. Due to the symmetry of the test, only a symmetric half of the specimen was model here. The FE model was made up of following parts:

- Square Hollow Section (SHS 400x400x12.5) – 2.2 m long
- Concrete infill to SHS – M50
- Universal beam (460UB82.1) – 2 x 4m long
- Concrete Slab – M50 – (140mm thick)
- Condeck steel sheeting (1mm thick)
- Reinforcement (Embedded in concrete slab)
- T-Stubs – 4 Nos (Top and Bottom)
- M24 Through bolts – Bolt, Sleeve and Washer –2 Nos

- M24 Headed Anchored Blind Bolts (HABB) - Bolt, Sleeve and Washer – 4 Nos
- Shear Studs - 21 Nos (19mm dia studs at 150mm centre to centre spacing)

A three dimensional eight noded element (C3D8R) was used to model the concrete, steel tube, beam, T-Stub, and bolts to improve the rate of convergence within the explicit analysis. For the profiled steel sheeting, a four noded doubly curved shell element (S4R) and a two noded linear 3-D truss element (T3D2) were used to model the sheeting and steel reinforcement.

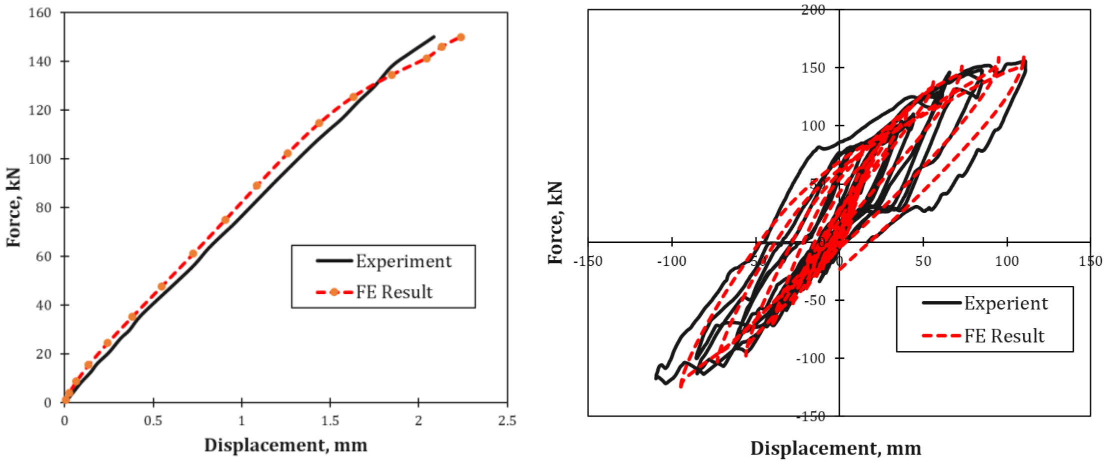
The Concrete Damage Plasticity (CDP) model was used to model the concrete as this model is ideal for cyclic loading. Isotropic damaged elasticity and multi-hardening plasticity were defined as in (Pokharel et al., 2016) to describe the irreversible damage in the CDP (Abaqus, 2012). For the steel, the combined hardening model was used which considers the Bauschinger effect under cyclic loading.

Coupon tests were conducted on material from the steel tube, beam, T-stub and threaded rods. Concrete compressive tests were conducted using standard cylinders. The mean 28 days compressive strength of concrete was found to be 46MPa. The stress strain relationship recommended by (Carreira and Chu, 1985) was used.

FE Results

As explained in the previous section, the gravity load was applied to the column at the beginning and then to the beams. Figure 3(a) shows the load vs displacement curve at the point of applying the gravity load to the beams. The experimental result is also plotted in the same figure.

After the gravity load, cyclic displacements were applied in opposite directions at the end of the beams while keeping the gravity loads constant as in the experiment. Figure 3(b) shows the result of the analysis. The Load vs displacement curve from the FE analysis and the experimental result are compared in this figure. As can be seen in the figure, the capacity and stiffness of the connections at different cycles match well.



(a) Under gravity load

(b) Under cyclic loading

Figure 3: Force vs. displacement relation curves for the FE and Experimental results

MODIFICATION OF BLIND BOLT

During pull-out of anchored blind bolts, the load from the bolt is transferred to the concrete in two ways. One is through the tangential/frictional force between the threaded rod and surrounding concrete i.e., bond; and the other is from bearing of the end head onto the concrete. Agheshlui (2014) investigated the contribution of both friction and bearing to the overall behaviour of the bolt experimentally. He concluded that the contribution from friction is very small when compared to the contribution from bearing, and thus could be neglected. The results of a FE analysis conducted in this study with friction coefficient 0.55 as recommended by (Cairns et al., 2007) also support this statement. It should also be noted that the bond is likely to deteriorate in the case of cyclic loading, so it is preferable to ignore it.

The tensile stiffness of the blind bolt can be increased by increasing the strength of the concrete. However, from FE analyses, it was found that the effectiveness of this approach decreases after a certain limiting value of the concrete strength is reached. Another way of increasing the stiffness is by increasing the bearing area of the bolt head by increasing the diameter of the end head. But the diameter of the head is limited by the diameter of the bolt hole which is 35mm for the M24 blind bolt (Fernando, 2005). To overcome this problem, additional heads could be added between the end head and head adjacent to the tube wall. Figure 4 shows the modified blind bolt, the double headed anchored blind bolt (DHABB), with one additional head between the existing heads in the embedded region. This head will be called the middle head hereafter.

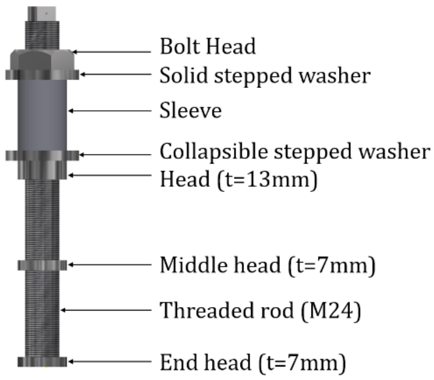


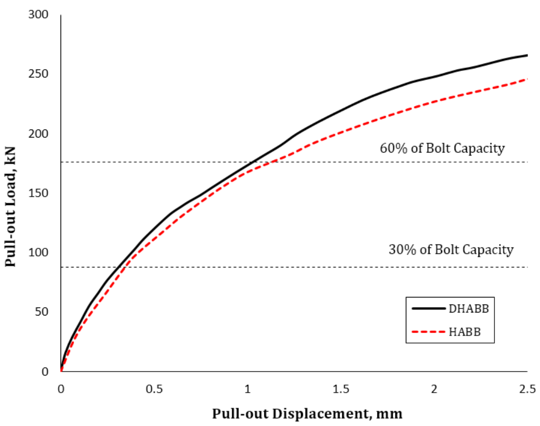
Figure 4: Double Headed Anchored Blind Bolt and its components

The location of the middle head is a crucial factor in determining the effectiveness of the DHABB. If the head is placed too close to the end head, the DHABB will behave in a similar way to a HABB with a reduced effective depth of the anchor; thus the performance would be even worse. On the other hand, if the middle head is placed too close to the tube wall, it would not be effective in transferring the load to the surrounding concrete. Therefore, the optimum position of the middle head is important. To identify the optimum position, a parametric study was conducted by varying the position of the head and it was found that the best performance was achieved when the middle head was placed at $3.2d_b$.

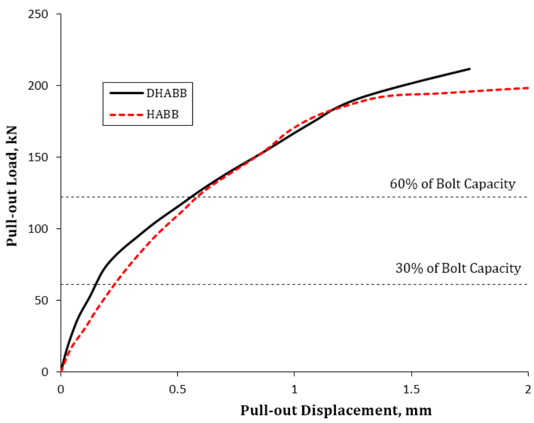
A series of experiments were conducted to determine the effectiveness of middle head in improving the tensile stiffness of blind bolts. The tests were done with different bolt sizes and B/t ratios of steel tubes.

Figure 5: Test setup for pull-out of DHABB from CFSHS 400x400x12.5

Figure 6 shows the comparison of DHABBs and HABBs in pull-out load vs displacement curves for M24 and M20 blind bolts from a CFSHS 400x400x12.5 with 50MPa concrete infill. The curves for the HABBs are the result of monotonic pull-out tests (Agheshlui et al., 2015) and the curves for the DHABBs are envelopes of cyclic pull-out tests as per (FEMA-461, 2007). As shown, the tensile stiffness of the HABBs at 60% of the bolt capacity was increased with the addition of an extra head (i.e., the stiffness of DHABB at 60% of the bolt capacity was found to be greater than that of the HABB in both cases), although it was only a marginal difference for the M20 bolts. Please note that 60% of the nominal bolt tensile capacity is used because the connection will be designed to ensure that the forces in the bolts will never exceed this level.



(a) Pull-out of M24 blind bolts



(b) Pull-out of M20 blind bolts

Figure 6: Force vs. displacement curves for the FE and Experimental results

The advantage of using DHABBs was found not only in the tensile stiffness but also in reducing cyclic deterioration under cyclic loads. Since there is no data for pull-out of HABBs under cyclic loading, finite element analysis was conducted. A 3D finite element model was developed to simulate the tensile behaviour of the DHABB and the results from this model were compared with the experimental results. The results from the FE analysis were in good agreement with the experimental results. The results are not presented here due to limited space. The verified FE model was then extended to perform the cyclic pull-out of HABBs.

Figure 7 shows the result of the FE analysis on cyclic pull-out of a M24 HABB and a M24 DHABB from a CFSHS 400x400x12.5. Here, three cycles at 30% of the ultimate tensile nominal capacity of the bolt (293kN for M24 bolts) and 5 cycles at 60% of the ultimate tensile capacity were applied. As can be seen in Figure 7, the tensile stiffness of the HABB decreases with the cyclic loading. Also, the pull-out displacement increases progressively with repetitive loadings. The residual displacement at the 60% of bolt load was found to be 0.08 mm and 0.26 mm for the DHABB and HABB respectively. The displacement was measured at the 5th and 1st cycle and subtracted to get the residual displacement. These results suggests that the DHABB performs better than HABB under monotonic and cyclic loading.

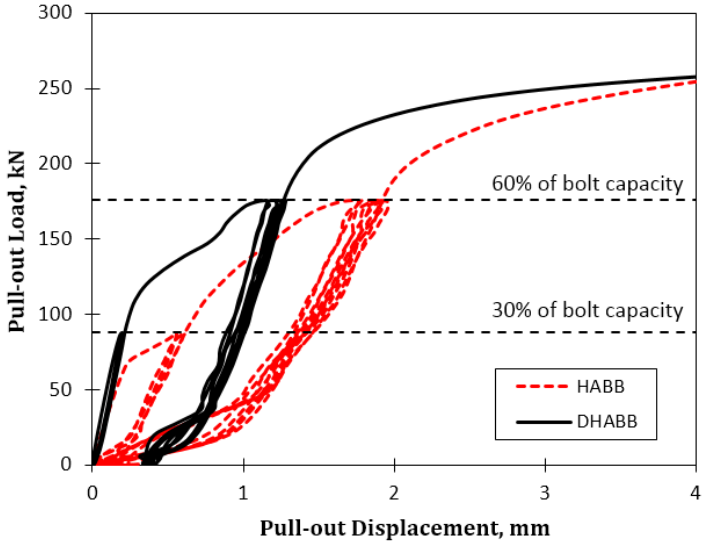


Figure 7: Load vs Pull-out displacement curves for DHABB and HABB under cyclic loading

BEHAVIOUR OF SUB-ASSEMBLAGE WITH DHABBS

After determining that some advantages could be obtained by modification of the blind bolt from a HABB to a DHABB, the effectiveness of doing this was checked for use within blind bolted connections in a structure. The same FE model of the sub-assemblage discussed in sections 2 and 3 was used to check the benefits obtained by using DHABBs in a case study building. All of the parameters were kept same as in Section 3 except for the type of blind bolt. The DHABB shown in Figure 4 was used.

The FE model described earlier has more than 250000 nodes and 768000 DOF. The stable time for the explicit analysis was in the order of 10^{-7} seconds and required many time increments to be performed to complete the solution. It took several days or even weeks to complete solution using a normal PC. Thus, to compare the behaviour of DHABBs and HABBs, a monotonic displacement was applied to the beam ends since this required much fewer computational resources than the application of cyclic displacements. Also, it had been observed previously from FE studies that the backbone curve from cyclic loading was similar to the result from monotonic loading.

Figure 8 shows the moment rotation relationships of the connection with the HABBs and DHABBs. The connection with DHABBs was found to be stiffer than the connection with HABBs. The stiffness was found to be increased by more than 12% at a moment corresponding to G + 0.3Q + EQ500. At the moment corresponding to G + 0.3Q + EQ2500, the increase in stiffness was found to be more than 15% by replacing the HABBs with DHABBs.

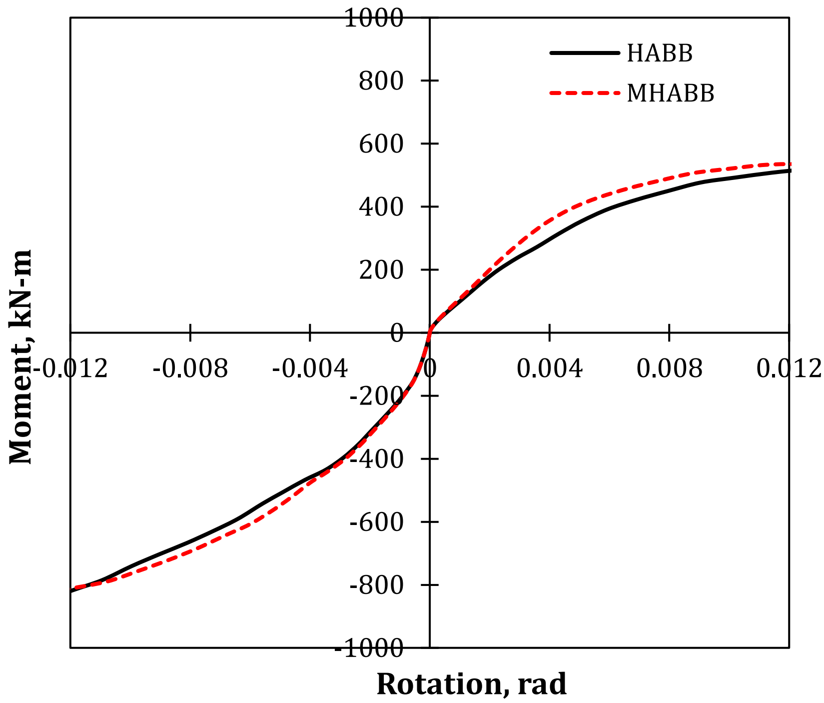


Figure 8: Moment vs. rotation curves for HABB and DHABB

CONCLUSION

The behaviour of beam-column connections using blind bolts was simulated experimentally and numerically. Concrete filled square hollow sections were used as the columns and universal beams composite with concrete slabs were connected to them using double T-stubs; each T-stub has 4-M24 blind bolts and 1-M24 through bolt. The FE results were compared with experimental results and a reasonable agreement was achieved.

The blind bolts used in the sub-assemblage test, the headed anchor blind bolts (HABBs), were modified to enable a higher stiffness to be attained and less deterioration under cyclic loads. The modification simply involved the addition of one head between the two embedded heads in the HABBs. The effectiveness of the addition of the extra head in the Double Headed Anchored Blind Bolt (DHABB) was tested experimentally and numerically. The performance of the modified blind bolt (DHABB) was found to be better than the HABB in all cases.

Finally, the modified blind bolt (DHABB) was used in a revised FE simulation of the sub-assemblage test. From the FE analysis, as expected, the connections with the DHABBs were found to be stiffer than those with HABBs.

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