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ASSESSING THE BEHAVIOR OF COLUMN-SPLICE CONNECTIONS BETWEEN CFSTS IN AXIAL TENSION

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Increasing the sustainability of construction materials is a key goal in the construction industry. As the demand for high-rise residential structures grows, new more efficient methods of achieving sustainability are required. Rendering larger structures demountable to enable the re-use of steel structural members is one solution. The aim of this study was to investigate and assess the behavior of blind bolted column-column connections between concrete filled steel tubes in axial tension. The investigation aimed to address the need for accurate design guidelines for tensile members in the composite design standards such as Eurocode 4 and encourage the use of column-column connections in large-scale demountable structures. A numerical investigation of the blind bolted column-column connection has been performed using ABAQUS. The model consisted of two concrete filled square hollow steel sections connected using sleeve plates welded to the top tube only, as well as 16 M20 Lindapter Hollo-bolts connecting the tubes. The numerical investigation revealed that the sections tested conformed with AS1170.1 regarding columns in tension. Results of the numerical investigation have been compared against experimental results for a continuous CFST and it was found that the connections performed better in tension. The parametric analysis revealed no dependence of the shape of force-displacement relationships on diameter to thickness ratio, concrete compressive strength or yield stress.

Keywords: CFST, Colum-column connection, Finite element, ABAQUS, Parametric study.

1 INTRODUCTION AND BACKGROUND

It is estimated that by 2030 the world will have run out of many raw materials used for buildings. Furthermore, the production and transport of building materials contributes significantly to greenhouse gas emissions (Gorgolewski 2006). To reduce the negative environmental impacts of construction, more sustainable and efficient methods of reusing construction materials must be employed. It is known that 65 percent of all steel in Australia goes back into the making of new steel (Bluescope 2009). Although the energy and materials savings derived from recycling steel is significant, to meet future sustainability targets recycling will not be sufficient. Therefore, reuse should be the next step in obtaining a more efficient and environmentally friendly recycling system. One of the major reasons the reuse of structural and composite steel has not gained great popularity is the difficulty in rendering structures demountable (Li *et al*. 2015). To make reuse a feasible option, innovative and standardized methods of designing connections between structural members that facilitate dismantling must be developed. Column-column connections (or column splices) are especially important in multi-story construction as they provide strength and continuity of stiffness as well as allowing for lengths of columnar members convenient for fabrication, transport and erection. At present, our understanding of blind-bolted column-column connections is limited. It is critical we increase our understanding of blind-bolted column-column connections to ensure safe design of demountable composite structures. The Eurocode design guidelines identify situations in which structural columns are placed under tensile loading yet the behavior of column-column connections under tension is not well understood. This study therefore aims to improve our understanding of the behavior of column-column connections between CFST's in axial tension to support the development of comprehensive design guidelines for demountable structures.

The potential of blind bolted connections in making demountable structures consisting of CFSTs a feasible alternative in construction has generated discussion and interest among many scholars across the world, due to the ability of these blind bolts to be installed and removed with access to only one side of a structural member, with minimal damage to the structural steel. As a result, extensive experimental investigations and numerical models have been employed to improve our understanding of the behavior of these blind bolted composite connections. There has been limited research done on the behavior of column-column connections of the form depicted in Figure 1. As such this study aims to address the gap in the literature regarding the behavior of this type of connection. However, to do so required contextualizing this research within the greater body of work on composite connections.

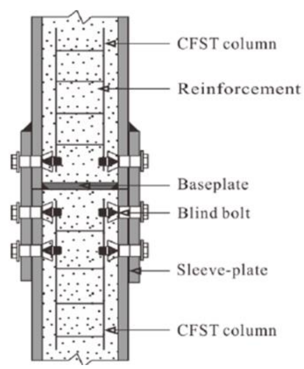

Figure 1. Blind bolted column-column splice connection between CFSTs.

2 RESEARCH METHODOLOGY

This investigation consisted of a numerical investigation on column-column connections between CFSTs validated by past experimental results of a continuous CFST in axial tension. The models developed in ABAQUS CAE for this investigation were used to conduct a parametric study to observe the influence of depth to thickness ratio, concrete strength, steel yield stress and the presence of two types of reinforcement, straight and anchored. Details of the numerical investigation were defined and a preliminary model was developed including the model geometries, mesh size, material properties, contact interactions and boundary conditions.

All components including the steel tube, concrete fill, sleeve plates and Hollo-bolts were modelled using 8-node brick elements (C3D8R). The initial mesh sizes used were approximately 20mm per element, chosen to obtain roughly 100 elements in the Hollo-bolt and 9000 elements overall. The blind bolt was modelled like the model produced by Hassan *et al*. (2014). A surface based interaction with a contact pressure model in the normal direction was used to simulate the interface between the steel and concrete fill. A tangential penalty model was used with a frictional coefficient of 0.6 and 0.3 to define the contact between steel and concrete and the contacts between steel surfaces for as well as a hard contact defining the normal behavior to ensure there was no penetration of either solid. The boundary conditions included a displacement/rotation with a single axial displacement at the top end with no rotations.

The validation process involved predicting the yield load of the column-column connections using the equation produced by Han *et al*. (2014) and checking the values from the modelling had a maximum of 10% deviation from these predicted values.

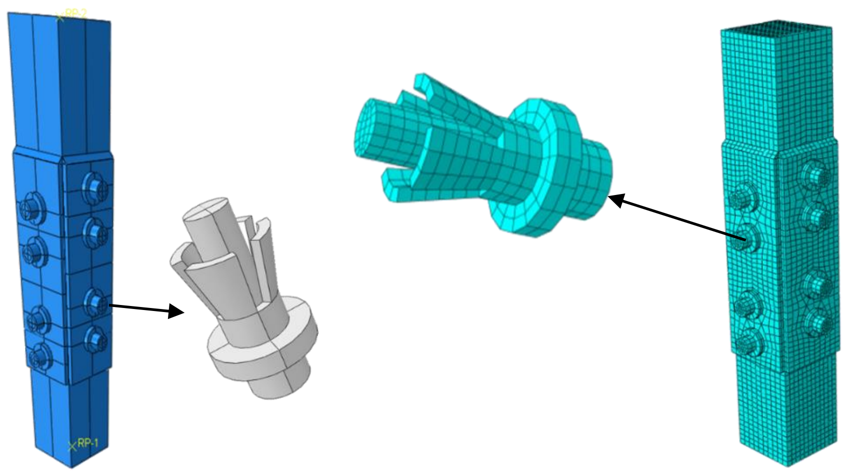

Figure 2. FE modelling of column-to-column connection for CFST columns.

3 RESULTS AND DISCUSSION

The FE model results of the different depth to thickness ratio, concrete strength, steel yield stress and reinforcement are reported here. The dimensions and material properties of 9 CFST column specimens are tabulated in Table 1.

The sleeve plates surrounding the joint were identical with dimensions of 10 × 125 × 375 mm however had slightly different bolt arrangements for specimen T-1-1 and T-1-2. The sleeve plates were essentially alternating in vertical orientation as they went around the tube. That is, sleeve plates were always 180 degree rotations of their adjacent sleeve plates. This was to ensure there was a staggered pattern of bolts as the tube depth was too low to have them all in the same plane and obtain reasonable results. The positioning of the bolts was arbitrary and the impact of their placement was not within the scope of this investigation.

Table 1. Dimensions and material properties of specimens considered in parametric study.

Specimen	***Column-D×t×L* (mm)**	**D/t**	**f_y (MPa)**	**f_c (MP)**
T-1-1	125×5×750	25	350	20
T-1-2	125×4×750	31.25	350	20
T-2-1	125×5×750	25	250	20
T-2-2	125×4×750	31.25	250	20
T-3-1	125×5×750	25	350	30
T-3-2	125×4×750	31.25	350	30
T-4-1R	250×4×750	62.5	350	30
T-4-1L	250×4×750	62.5	350	30
T-4-1	250×4×750	62.5	350	30

Table 2. Comparison of predicted and modelled yield loads.

Specimen	**Predicted (kN)**	**Modelled (kN)**	**Percentage Deviation**
T-1-1	707	714	0.9%
T-1-2	863	884	2.4%
T-2-1	505	547	8%
T-2-2	616	632	2.6%
T-3-1	707	740	4.7%
T-3-2	863	894	3.6%

Table 2 illustrates the comparison of predicted and modelled yield loads of the CFST connections. The results have revealed that the CFST connection designs tested in the parametric analysis all meet the tensile strength requirement of columns by AS1170.1-2009. The results of modelling have generally given values of the yield load higher than the predicted yield load using the equation by Han *et al.* (2014) for predicting the tensile strength of CFSTs. A table comparing the predicted and modelled yield loads as well as the percentage deviation from predicted loads is given in Table 2. The deviations are below 10% and are therefore considered not very significant. This does not necessarily mean that the Han *et al*. model is conservative as it is derived from tests on continuous CFSTs which behave differently in terms of stress development and failure. Therefore, it may be concluded from the modelling results that the yield load is higher for blind bolted connections between CFSTs than continuous tubes. The most likely reason is the extra stress transferred to the high strength blind bolts through the concrete, indicating another benefit of composite members. From the parametric analysis, the effect does not appear to have any dependence on the steel yield stress however there was some dependence of the influence of increasing concrete compressive strength on the depth to thickness ratio revealed.

3.1 Effect of Depth-to-Thickness Ratio of CFST Columns

The depth to thickness ratio was considered in this investigation. The values of axial tension at yield were both overestimated by the equation produced by Han *et al* (2014). The force at yield of T-1-1 was 714 kN while the predicted force was 707 kN. The force at yield of T-1-2 was 884 kN while the predicted force was 863 kN. The force at yield of T-1-2 was higher than the theoretical tensile capacity of a continuous CFST. The yield load of specimen T-1-1 exceeded the theoretical tensile strength of a continuous CFST which would be 712 kN, implying that there is a composite effect at work. The yield load modelled in specimen T-1-2 also exceeded the predicted yield load however not by a significant amount. It should be noted both of the yield loads are greater than half the required compressive capacity. The required compressive capacity of T-1-1 is 951 kN meaning the required tensile capacity would be 476 kN. The required compressive capacity of T-1-2 is 1,104.5 kN meaning the required tensile capacity would be 552 kN. Therefore, these designs satisfy the Eurocode guidelines and do not require reinforcement.

3.2 Effect of Steel Yield Stress of CFST Columns

Two specimens with a steel yield stress of 250 MPa and 350 MPa were modelled and it was found that the force at yield of each connection was underestimated by the equation produced by Han *et al*. (2014). The force at yield of specimen T-2-1 was 547 kN while the force predicted was 505 kN. The force at yield of specimen T-2-2 was 632 kN while the force predicted was 616 kN. There was minimal difference in the way the stresses developed in the connections as can be seen in the similarity of the shapes of the graphs to the curves based on steel yield stress of 350 MPa. It should be noted both of these values are greater than half the required compressive capacity. The compressive capacity of T-2-1 is 758 kN meaning the required tensile capacity is 379 kN. The compressive capacity of T-2-2 is 865 kN meaning the required tensile capacity is 433 kN.

3.3 Effect of Concrete Compressive Strength of CFST Columns

The effect of concrete compressive strengths of 30 MPa and 20 MPa are investigated and it was found that the force at yield of T-3-1 and T-3-2 were both underestimated by the predicted force. The force at yield of T-3-1 was 740 kN while the predicted was 707 kN and the force at yield of T-3-2 was 896 kN while the predicted force was 863kN. For specimen T-3-1 the increase in strength of the connection with 30 MPa concrete instead of 20 MPa concrete was about 33 kN, while the increase in tensile capacity of the concrete was only 9 kN. Therefore, there must be a composite effect influencing the strength of the connection. In this case the predicted strength using the equation by Han *et al*. underestimates the strength. For specimen T-3-2 the increase in strength was about 33kN while the increase in tensile capacity of concrete was only 8 kN. Therefore it can be concluded there is a composite effect on the yield strength, however this does not have any relation to the depth-thickness ratio. The compressive capacity of specimen T-3-1 is 1,088.27 kN meaning it must be able to bear loads up to 544.14 kN. The compressive capacity of specimen T-3-2 is 1,237 kN meaning it must be able to bear loads up to 618 kN.

3.4 Effect of Reinforcement of CFST Columns

The model consisted of two 250 x 4 x 750 mm CFSTs connected by 250 x 10 x 565 mm sleeve plates with 6 hollo-bolts in each sleeve plate. There were two bolts in the top tube and four bolts in the bottom tube. The aim of this parametric variation was to observe the impact of including reinforcement in the composite connection and observe whether composite effects improved, reduced or had no impact on the effectiveness of the reinforcement. A secondary goal was to observe the effect of adding anchorage to the reinforcement. It was found that for depth to thickness ratios greater than a certain value, the tensile capacity of the CFST alone was not enough, not including the strength increase by composite effects. It was found that with 4N12 reinforcement bars, column-column connections that were just below the tensile requirement could have their tensile strength increased above the tensile requirement of AS1170.1. The anchorage had minimal effect on the strength of the connections but the location of the anchor did influence the stress-distribution in the reinforcement somewhat.

4 CONCLUSION

From the parametric analysis, it can be concluded that 125 x 5 mm and 125 x 4 mm sections of CFST column-column connections consisting of two 375 mm long tubes can withstand the tensile loading required by AS1170.1. It was also found that the depth to thickness ratio had minimal effect on the shape of the force-displacement curves, and the impact of diameter-thickness ratio on strength was minimal. The composite effect did appear to increase with increasing concrete strength which validated the model by Han *et al*. The yield stress of steel and concrete strength variations did not have any effect on the shape of the curves. The greatest deviation from the predicted results was for the sections with the lower yield stress which was 8%. This was still considered insignificant within the 10% confidence bounds of this study. Hence the numerical model was shown to lie within reasonable confidence bounds when predicted using the equation produced by Han *et al*. for continuous tubes. However, it was concluded that this equation was still not sufficient to validate the model and instead experimental results from tests on specimens identical in physical characteristics to the model should be used. The depth to thickness ratios that were more problematic were found by applying AS1170.1 to determine when reinforcement was required and an expression accounting for yield stress and compressive strength was derived to assist in finding these depth-thickness ratios. It was found that with 4N12 reinforcement bars, column-column connections that were just below the tensile requirement could have their tensile strength increased above the tensile requirement of AS1170.1. The anchorage had minimal effect on the strength of the connections but the location of the anchor did influence the stress-distribution in the reinforcement somewhat.

Acknowledgments

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