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Feasibility study for blind-bolted connections to concrete-filled circular steel tubular columns

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Abstract. The design of structural frameworks for buildings is constantly evolving and is dependent on regional issues such as loading and constructability. One of the most promising recent developments for low to medium rise construction in terms of efficiency of construction, robustness and aesthetic appearance utilises concrete-filled steel tubular sections as the columns in a moment-resisting frame. These are coupled to rigid or semi-rigid connections to composite steel-concrete beams. This paper includes the results of a pilot experimental programme leading towards the development of economical, reliable connections that are easily constructed for this type of frame. The connections must provide the requisite strength, stiffness and ductility to suit gravity loading conditions as well as gravity combined with the governing lateral wind or earthquake loading. The aim is to develop connections that are stiffer, less expensive and easier to construct than those in current use. A proposed fabricated T-stub connection is to be used to connect the beam flanges and the column. These T-stubs are connected to the column using "blind bolts" with extensions, allowing installation from the outside of the tube. In general, the use of the extensions results in a dramatic increase in the strength and stiffness of the T-stub to column connection in tension, since the load is shared between membrane action in the tube wall and the anchorage of the bolts through the extensions into the concrete.

Keywords: concrete-filled steel tubes; blind bolts with extensions; moment-resisting composite connection.

1. Introduction

The pilot programme tests reported in this paper explore the feasibility of developing a connection between a concrete-filled circular steel tube and a composite steel-concrete beam. Circular tubes have been chosen rather than rectangular ones for a couple of reasons. Due to hoop tension, the circular concrete-filled medium-thick-walled columns are more effective at confining the concrete than the square or rectangular ones (Schneider 1998), leading to a higher degree of ductility under lateral loading. Also, the circular ones are often favoured by architects for aesthetic reasons (Bergmann *et al.* 1995). Concrete-filled tubular columns have been used extensively within moment-resisting frames in high seismic zones such as Japan. The connection region is typically stiffened by welding on either internal or external diaphragms, and the beams are then welded onto the diaphragms. This type of connection would be prohibitively expensive in Australia, because of

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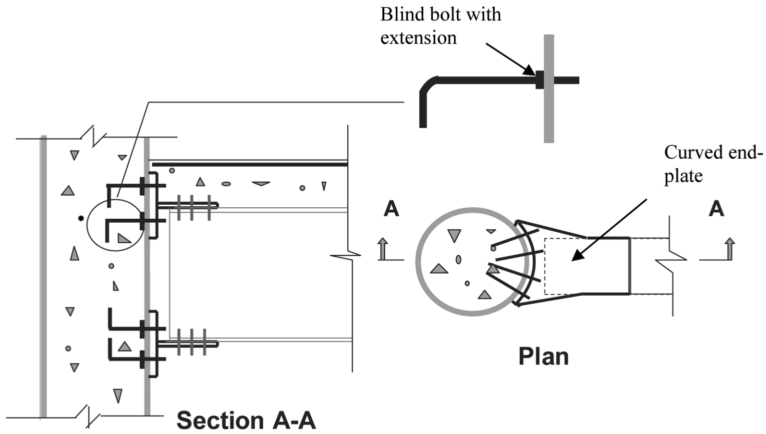


Fig. 1 Schematic of proposed connection

the large amount of site welding that is required. The preference in Australia is for welding to be carried out in the fabricating shop, where it is easier to achieve good quality control, and for the members to be bolted together on site.

Pivotal work by (Schneider and Alostaz 1998) showed that beams directly welded to concrete-filled circular steel tubes caused excessive distortion of the tube wall and eventual weld failure before the beam could reach its plastic moment capacity. However, the beam capacity could be reached if reinforcing bars were welded onto the top of the flange and anchored into the concrete within the column through holes in the tube wall. This sparked the idea, in the case of the connection being considered here, of anchoring the blind bolts into the tube, in order to improve the rigidity and strength.

A sketch of the proposed connection is shown in Fig. 1. The connection uses built-up T-stubs, consisting of a curved endplate (flange of the T-stub) and a fan-shaped horizontal web-plate, to connect the top and bottom beam flanges to the column (or just the bottom flange in the case of a semi-rigid connection). It uses shop-welding and field-bolting as discussed previously. The endplates of the built-up T-stubs are attached to the column by Ajax “One-sided Fasteners” (a type of “blind bolt”), or modified versions of these that have reinforcing bars welded on as extensions (shown as cogged extensions in Fig. 1). The blind bolts would be pre-tensioned during construction to clamp the T-stub to the tube wall. The steel tubes, which are limited to commonly available sizes with diameter to thickness ratios between 40 and 60, would be designed to take the construction loads for several storeys before being filled with concrete. The concrete would be pumped in from the bottom as outlined in (Bergmann *et al*. 1995). After the addition of concrete into the tube, the bolt extensions act to anchor the connections into the concrete. For a T-stub in tension, the tensile load would be shared between the anchored extensions and membrane action in the tube wall, rather than relying solely on the anchorage as is done in Schneider’s work. This system was developed by the authors (Gardner and Goldsworthy 1999). Others have subsequently used the same concepts but have used different types of blind bolts (Ellison and Tizani 2004).

Reinforcement is provided in the concrete slab that forms part of the top flange of the beam. This reinforcement stops at the column faces, but is continuous on both sides of the column. The contribution of the reinforcement to the tension capacity of the connection under a hogging moment is dependent on the number of shear connectors provided in the negative moment region, i.e., no shear connection, partial or full. The effect of the presence of the slab is not investigated in this paper, but will be considered later for both the semi-rigid and rigid connections when determining the moment-rotation characteristics of the overall connection.

There are several issues being addressed by the structural system proposed here. A common construction problem being encountered in Australia is that there is a demand for larger floor spans (i.e., column free spaces) in both residential and commercial buildings. This has compromised the floor stiffness and led to serviceability problems with deflections and floor vibrations. If the connections being developed in this project can be proven to have a reliably high degree of stiffness, they would be effective in significantly reducing this problem.

Another issue is that of the response of the frame to displacements caused by earthquakes. The semi-rigid type connection is being proposed for regions of low to moderate seismicity, and the rigid connection for regions of moderate to high seismicity. Australia is taken as an example of region of low to moderate seismicity. Detailing in Australia has typically catered well for gravity loading, wind loading and 500 year return period seismic loading. However, under low probability, high consequence events such as a 2500 year return period level earthquake or blast loading, the connections that are currently used may not perform as desired. Greater reliability under this type of event would be achieved if the moment-resisting connection were able to successfully undergo a reversal in the direction of the moment. Many connection details do not allow for this. Thus, even in the case of the semi-rigid connection proposed here for areas of low to moderate seismicity, the tensile behaviour of the T-stub connection between the bottom beam flange and the column is a critical consideration, leading to a more robust system overall.

In regions of high seismicity, poor performance of steel moment-resisting frame connections (especially fully welded ones) in recent earthquakes has led to a concerted international research effort to improve them. After the Northridge earthquake, valuable work by (Swanson and Leon 2000) explored the use of bolted T-stub connections of a similar type to those shown in Fig. 1, but between steel Universal Columns and steel Universal Beams. The authors showed that it was possible to create a partial-strength connection which could act as a fuse in an earthquake, by designing for the flange and web plates of the T-stub to yield simultaneously before the strength of the beam flange was reached. The rigid connection proposed here for regions of high seismicity (with T-stubs at both beam flanges) is not intended to be a partial strength connection, although that option may be explored later. The tensile behaviour of the T-stub connection to the column is clearly a primary consideration for the rigid connection being proposed here.

The pilot study experimentally investigated the stiffness and strength of the T-stub in tension. This is a key first step in building an understanding of the overall connection behaviour in bending. There are certain unique facets of the behaviour, in tension, of the connection proposed here, that have required further careful in-depth study with regards to their stiffness and strength. The key variables influencing the stiffness have been identified in a component model developed by (Gardner and Goldsworthy 2005). A brief summary of this work is given below. With regard to strength design, the main concern that is different to those usually encountered, is the potential for loss of anchorage of the extensions before the full capacity of the bolts has been reached. The T-stub connection is expected to behave well in compression, since it would be bearing against the concrete-filled steel tube, so some of the critical failure modes observed in a T-stub to Universal Column connection, such as buckling of the column web, are not a problem here.

The first series of tests in the pilot programme has investigated the effectiveness of the anchorage provided by different configurations of extensions in specimens subjected to cyclic tension. In the

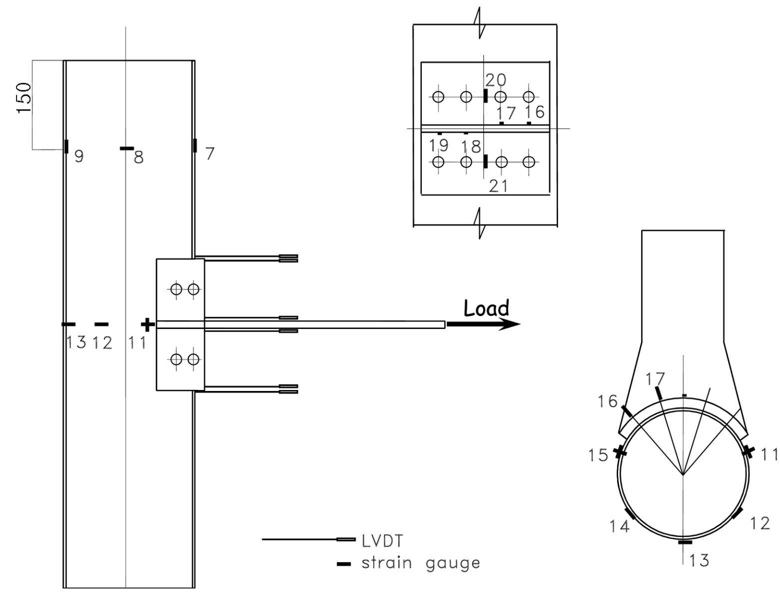


Fig. 2 First series test specimens

second series, the behaviour of a connection to a concrete-filled circular steel tube with a particular extension configuration was compared to that of a similar connection to a concrete-filled square steel tube.

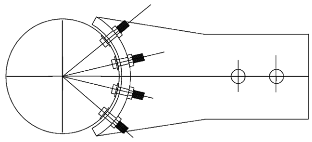
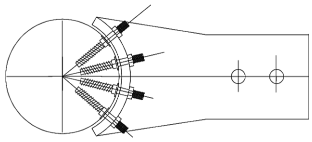
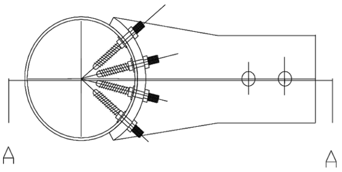
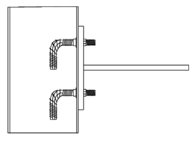
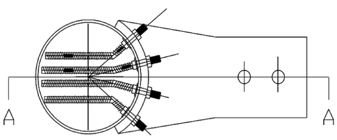
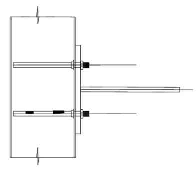
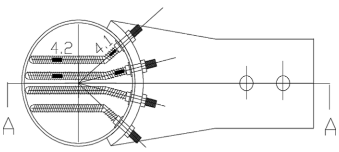
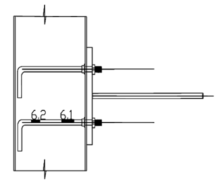
2. Test specimens

2.1 First series

The first series consisted of five tests on T-stub elements. The tests simulated the behaviour in tension, and under cyclic loading, of the T-stub connection between the beam flange and the circular concrete-filled steel tube. A typical specimen consisting of a column stub and the T-stub connection is shown in Fig. 2. The specimens are a scaled down version of a component within a realistic moment-resisting connection designed in accordance with Australian design standards for the lowest storey of a four-storey composite frame. The reduction in size was necessitated by the limited capacity of the available testing equipment (specifically the load capacity of the actuator), and was approximately two-thirds that of the prototype.

The T-stub was bolted to the circular steel (350 MPa yield strength) tubular concrete-filled column, with a diameter of 219.1 mm and a thickness of 4.8 mm, using Ajax blind bolts with or without extensions. The different configurations used for the extensions in the five tests are shown in Table 1. Commercially available Ajax blind bolts are structural bolts and have a minimum size of 16 mm. The scaled down test required 2 rows of three 12 mm bolts or 2 rows of two 16 mm bolts. The 12 mm bolt option was adopted since this gave a better representation of the geometry in the prototype. Hence, Ajax supplied Class 8.8 precision bolts instead of structural bolts for the 12 mm size. These bolts have a minimum tensile strength of 800 MPa and a 640 MPa yield strength, giving

Table 1 Bolt extensions used for each specimen

	Extensions	Plan view	Elevation
Specimen 1	none		
Specimen 2	Straight 60 mm		
Specimen 3	Cogged 60 mm		 SECTION A-A
Specimen 4	Straight 200 mm		 SECTION A-A
Specimen 5	Cogged 200 mm		 SECTION A-A

a minimum breaking load of the bolt of 67.4 kN. The bolts were tightened with a torque wrench up to 65% of the proof load of the bolt (48.9 kN) as quoted in the Ajax technical data. The strength of the concrete used to fill each column segment was approximately 30 MPa.

The extensions to the bolts were 12 mm diameter Y-type concrete reinforcing bars with a yield strength of 400 MPa. These reinforcing bars were welded in the Ajax fabrication workshop to the head of the bolt to form a complete unit.

2.2 Second series

Two specimens were tested in this series; one with a circular column that had a similar extension configuration to the third specimen of the first series (Fig. 3), and the other with a square column of similar proportions (Fig. 4).

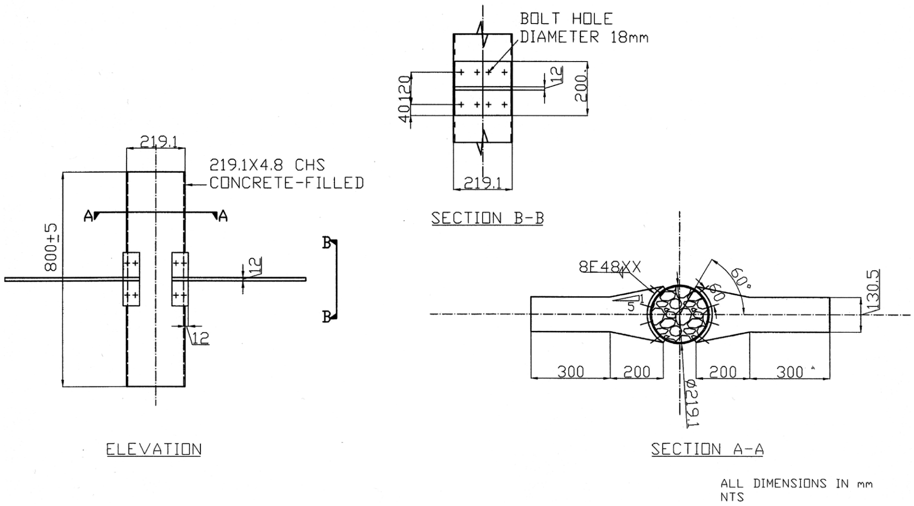


Fig. 3 Second Series with concrete-filled circular tube column

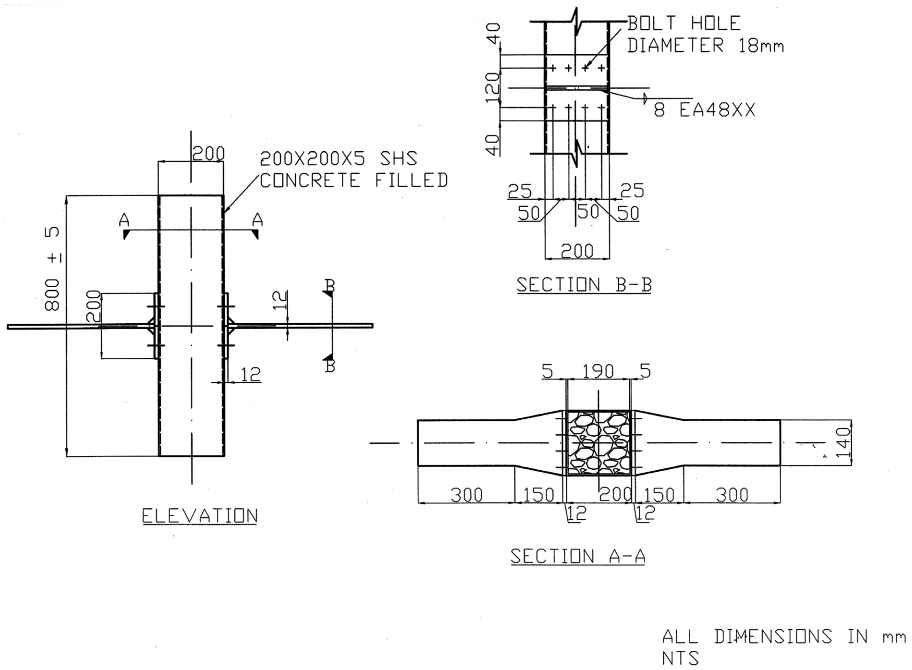


Fig. 4 Second Series with concrete-filled square tube column

Both had endplates of 12 mm thickness only, rather than the 16 mm thickness used in the first series of tests, so a greater degree of prying action was expected. In these tests, the welding technique used to form the welds between the bolt head and the extensions was greatly improved (Crawford 2001), and weld failure was not observed. Also, a different mix design was used, resulting in a characteristic concrete strength of 47.4 MPa for both specimens.

3. Experimental setup and loading

3.1 First series

The column of each specimen was inserted vertically into a specially designed bottom plate that allowed rotation but no horizontal movement. This plate was bolted through a concrete block to the laboratory strong floor. Axial loading was applied by a hydraulic jack reacting against a steel frame onto a rigid steel plate at the top of the column. (For specimens 1 to 3 the ratio of applied load to the axial strength of the column was approximately 0.4, but this was reduced to 0.2 for specimens 4 and 5. The load of 0.4 N_c is representative of axial conditions for an internal column, while 0.2 N_c simulates the axial load on an external column.) The frame was then restrained horizontally by stiff diagonal braces to the strong floor. The T-stub was attached at the centre of the column. The web-plate of the T-stub was connected through a loading plate to an MTS hydraulic actuator of 500 kN capacity, and this was used to apply cycles of tension to the connection. A schematic diagram of the experimental set-up is shown in Fig. 5.

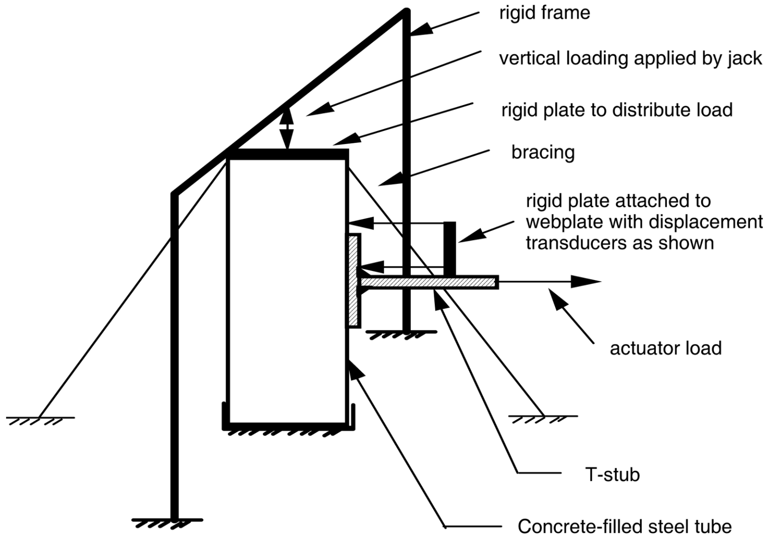


Fig. 5 Schematic of Series 1 test set-up

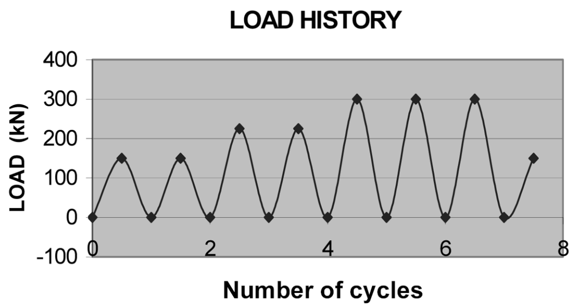


Fig. 6 Load controlled loading cycles

The horizontal load applied to the column consisted of a series of load controlled cycles of increasing tension up to 300 kN. The cycles were from zero load to the required tension load and then back to zero, as shown in Fig. 6. Displacement controlled loading was used for loads higher than 300 kN, with three cycles at 15, 20, 25, 30, and 40 mm displacement of the horizontal actuator.

Strain gauges were applied at various locations on each specimen. The location of external strain gauges is shown in Fig. 2. Gauge 10 is not shown on this drawing but is on the opposite side of the tube to gauge 8. Strain gauges 1 and 2 were attached laterally and longitudinally to the inside surface of the circular steel tube at the centre of the curved end-plate.

Fig. 2 also shows the location of the displacement transducers (LVDTs) that were mounted on the fan-shaped web-plate. The LVDTs monitored the displacement of

1) the wall of the column segment.
2) the endplate adjacent to the fillet weld connecting the fan-shaped T-stub web-plate to the endplate. The difference between this value and the deformation measured in 1) gives the outward deformation of the plate at this location relative to the tube wall.

All LVDTs were placed along the vertical centreline of the connection.

3.2 Second series

These tests were conducted with the column in a horizontal position, and were tested in the Amsler testing machine (load capacity of 1000 kN), rather than being specially mounted on the strong floor. Figs. 3 and 4 illustrate that two connections were made per specimen and were placed on opposite sides at the centre of the column. The bottom jaws of the testing machine held the web-plate of one connection, and the upper jaws held the other. Fig. 7 is a photo of the general test set-up for the square section specimen. No axial load was applied to the columns used in these specimens. Also, due to limitations of the testing machine, cyclic loading was not possible, so a monotonic test was applied by raising the upper head of the machine. This test set-up is very similar to that used in a project at the University of Nottingham (Barnett *et al.* 2000).

Fig. 7 Test set-up for Series 2 tests

LVDTs were mounted onto the web-plate as in the first series of tests. An extra displacement transducer was mounted on the web-plate to measure the displacement of the endplate at the bolt-line. The difference between this value and the deformation measured in 1) gives the outward deformation of the plate at this location relative to the tube wall. This is caused by a combination of localised bulging of the tube wall, slip of the extensions and elongation of the bolts themselves. In the test of the square tube column, it was observed that the transducer on the column was too close to the edge of the end plate since there was localised bulging of the steel column at this location. Hence the difference between the readings from 2) and 1) would have given an underestimate of the effect of the localised bending of the tube wall in this case. This problem was rectified in the test of the circular steel tube.

4. Discussion of results

4.1 Series 1 test results

4.1.1 Axial strength and stiffness

The maximum recorded load for each specimen is shown in Table 2. Specimen 1 performed poorly. Large localised strains were induced by the inside washers bearing against the tube wall, leading eventually to fracture of the tube and the bolts pulling through as shown in Fig. 8. The other specimens suffered from a variety of failure modes. The desired mode was that of the bolts reaching their full capacity and fracturing, but that was sometimes preceded by failure of the welds

Table 2 Maximum test load

Specimen	1	2	3	4	5
Maximum test load (kN)	416	495	507	500	501*
%Increase from Specimen 1		19	21.9	20.2	20.4

*bolts 3 and 6 loosened

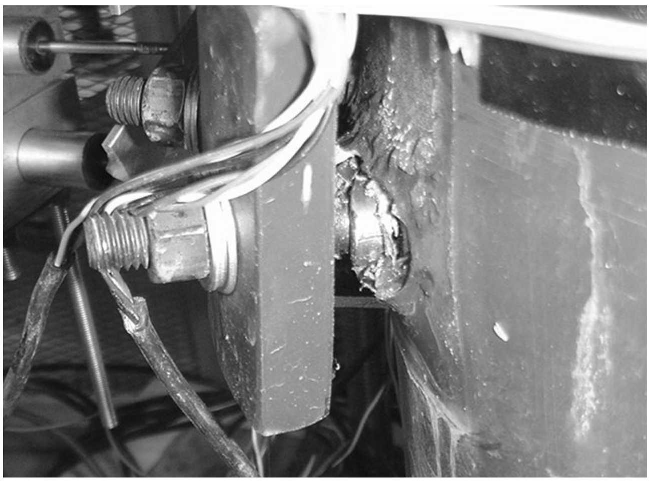


Fig. 8 Brittle failure of Specimen 1

between the bolts and the extensions, and/or anchorage failure of the extensions, particularly in Specimen 2.

The failure of the welds is clearly an undesirable mode of failure. This could be avoided if the bolt-extension units were fabricated all in one piece. The expenditure required to do this was not warranted for these pilot tests. As shown in Table 2, in all of the specimens with extensions, the increase in the total load was approximately 20%. However, it is likely that this increase would have been even higher for Specimens 3 and 4 if the weld failures had been prevented. The nominal 500 kN capacity of the actuator was only just sufficient to cause failure in Specimens 3 and 4. For specimen 5, failure was induced, of necessity, by loosening two of the bolts. Hence, the actual failure load of specimen 5 would have been higher than that recorded in Table 2.

More details of observations made during the tests, including load-displacement curves, can be found in (Gardner and Goldsworthy 2005).

With regard to stiffness, one of the key factors that contribute to the flexibility of the overall moment-resisting connection is the outward displacement of the endplate in tension at the level of the web-plate relative to the tube wall (referred to previously as the endplate displacement). Eventually, recommendations will be made to designers to ensure that the maximum load transferred to the T-stub is limited to approximately half of the total capacity of the bolts. (refer to section 4.1.2 below on “Relating the Test Results to Design”). The region of interest is hence up to a maximum total tensile load of about 300 kN in this pilot study. In Table 3 total endplate displacements are listed for the load levels of 150, 225, 300 kN, and 350 kN respectively. The extensions are clearly shown to be effective in restraining the displacement. The stiffness of the restraint is directly dependent on the anchorage provided by the extensions, with a dramatic increase from specimen 1 with no extensions to specimen 2, and significant increases thereafter.

Table 3 End plate displacements at given load levels

	Total endplate displacements (Bolt line displacements in brackets)			
Specimen	Load level (kN)			
	150	225	300	350
Series 1:				
1	0.23	1.15	3.98	6.99
2	0.11	0.32	0.67	1.22
3	0.08	0.21	0.45	0.83
4*	-	-	-	-
5	0.05	0.12	0.26	0.43
Series 2:				
Circular tube	0.4 (0.225)	0.75 (0.4)	1.15 (0.7)	1.7 (1.1)
Square tube	0.5 (0.2)	1.7 (0.5)	5.4 (1.1)	10.7 (2.0)

*This data not available for Specimen 4

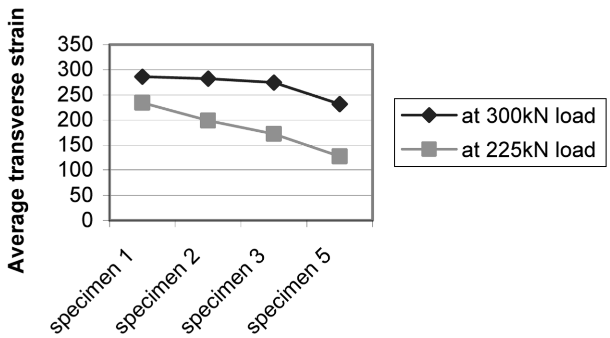


Fig. 9 Average transverse strain vs specimen type (constant load)

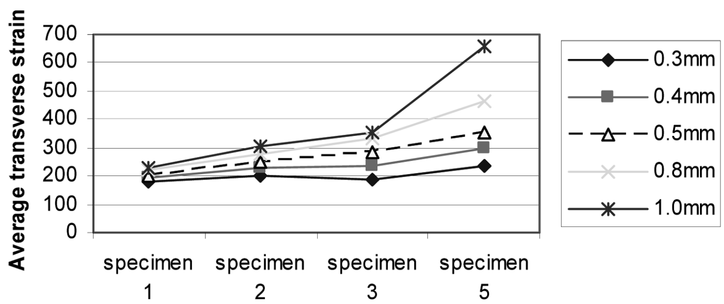


Fig. 10 Average transverse strain vs specimen type (constant displacement)

The average of the transverse (or circumferential) strains recorded by the transverse-oriented gauges at locations 11 to 15 (see Fig. 2) have been calculated. In Fig. 9 these values have been plotted for the different specimens at constant values of load (225 kN and 300 kN respectively). In Fig. 10, a further plot has been made comparing the average transverse strain calculated for the different specimens at increasing values of endplate displacement.

The trend in Fig. 9 is for the average transverse strain to decrease with improvement in the anchorage. With the improvement in anchorage it would be expected that a higher proportion of the force is attracted to the extensions, and hence less to membrane action in the tube wall.

In Fig. 10, for the same endplate displacement, the average transverse strain increased for the specimens with better anchorage. This is because better anchorage leads to a higher overall applied load for the same displacement, and hence greater overall average transverse strains. As expected, higher endplate displacements resulted in higher levels of average transverse strain for the same specimen. Also of interest is that the rate of increase of the strain is clearly higher for the better anchored specimens.

4.1.2 Relating the test results to design

In order to relate the results of this pilot study to the actual design of a frame, the design philosophy must be established, especially the relative strength hierarchy of the various elements that make up the joint (the beams, columns, and connections including components within the connections such as the T-stub). Failure modes involving excessive deformation and fracture of the tube wall, premature pull-out of the extensions, fracture of the bolt to extension weld (if one is

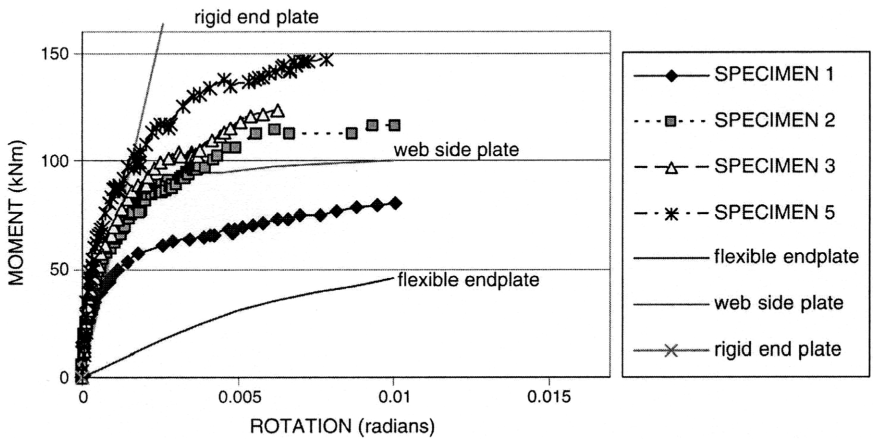


Fig. 11 Moment vs. rotation based on test results

present), or bolt fracture, must be avoided. The pilot tests have shown that the specimen without any extensions experienced excessive deformation of the tube wall, failing in the highly undesirable mode of brittle fracture of the tube wall. Although Specimen 2 was able to reach a higher failure load, the observed anchorage failure of some of the extensions gave cause for concern. Specimens 3 to 5 reached even higher levels of load, limited by weld breakages and bolt fracture. In design, if all other failure modes except bolt fracture were eliminated, a reliable estimate of the strength of the connection in tension could be made, and factors of safety could be imposed to ensure that bolt fracture did not occur under the ultimate design conditions. If the beam-column connections were designed as full-strength connections, then it would be the beams or columns that would eventually undergo plastic rotation. Thus, the moment experienced by the connection would be limited by the plastic capacity of the weakest elements, with some allowance made for over-strength.

Moment-rotation curves have been developed using the values of actuator load vs. endplate displacement obtained experimentally from the Series 1 tests, and assuming a steel-only connection. The actuator load is assumed to be one of a pair of forces forming a couple in the flanges at the end of the beam. The axis of rotation of the end of the beam is assumed to be in line with the compression force. For a particular level of force in the flanges, and the corresponding value of the couple, the recorded endplate displacement has been used, together with an appropriate beam depth, to calculate the rotation at the end of the beam. The moment-rotation curves are shown in Fig. 11. For the specimens in which effective anchorage into the concrete is maintained (Specimens 3 to 5), the initial rotational stiffness is between that of a web-side plate type connection and a rigid end-plate connection.

4.2 Series 2 test results

4.2.1 Load-displacement curves

The load-displacement curves are shown in Figs. 12 and 13 for the circular and square tubes respectively. The displacements are of

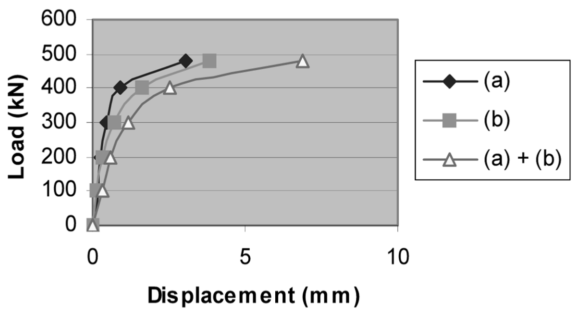


Fig. 12 Load vs. displacement for circular tube specimen

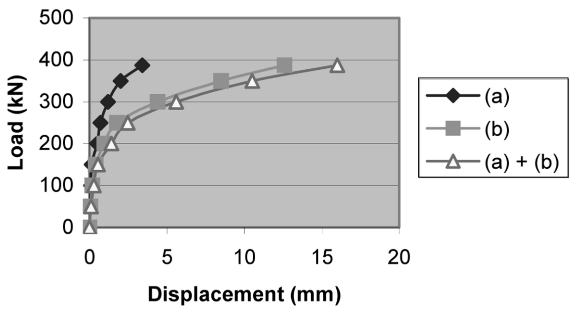


Fig. 13 Load vs. displacement for square tube specimen

(a) the endplate displacement at the bolt line relative to the tube wall
(b) the endplate displacement at the web-plate relative to the endplate displacement at the bolt line.
(c) The endplate displacement at the web-plate relative to the tube wall, [(a) + (b)]

It should be noted that no weld failures between the extension and the bolt head were observed in the Series 2 tests. The welding technique had been greatly improved using full penetration welds as well as preheating of the bolt head (to 200°C). For the circular tube, failure occurred at a load of 482 kN due to fracture of several bolts on the bottom connection. The inherent stiffness of the endplate due to its curvature resulted in a low level of deformation relative to the flat endplate attached to the square tube. The flat endplate exhibited significant flexure from an early stage (approximately 200 kN). The maximum load reached was 387 kN. Failure was initiated by fracture of several bolts in the bottom connection, followed by loss of anchorage of the extensions for several other bolts within this connection.

4.3 Comparison of the Series 1 and 2 test results

The results of the circular tube test are firstly compared with the Series 1 - Specimen 3 test. The difference between these tests is that the Series 2-circular tube endplate was 12 mm thick, and hence more flexible than the 16 mm plate used in the Series 1-Specimen 3 test. The total displacement of the endplate at the level of the web-plate relative to the tube wall, for various levels of load, has been given in Table 3. The displacement of the endplate at the bolt line relative to the tube wall is given in brackets for the Series 2 test specimens. The greater flexibility of the endplate in the Series 2-circular tube test was one obvious reason for the higher overall displacement. It also led to higher levels of prying forces in the bolts, and this provides an explanation for the high level of displacement at the bolt line in the Series 2-circular tube test. The increase in the amount of prying force also explains the lower failure load experienced by this specimen.

The Series 2-square tube specimen was designed to be as dimensionally close as possible to the Series 2-circular tube specimen. As expected, the displacement of the endplate at the web-plate relative to that at the bolt line was considerably higher for the square tube specimen, and a plastic mechanism was eventually reached with yield lines along the bolt lines and to each side of the web-plate. The flexibility and eventual plastic behaviour of this endplate resulted in very large prying forces in the bolts. Hence the failure load was 27% lower than that reached by the circular tube and the displacement of the endplate at the bolt line relative to that at the tube wall was significantly higher for this specimen. The apparent cracking of concrete within the tube, and the observed loss of anchorage of some of the extensions in this test, was an indication of the relatively poor confinement provided by the square section.

4.4 Component model

A component model of the T-stub connection element has been developed, and this gives a useful approximation to the experimentally obtained stiffness in the Series 1 tests, although for Specimen 1 the predicted stiffness is higher than that achieved experimentally. The model consists of two springs in series (k_1 and k_2) connected to two springs in parallel (k_3 and k_4). The stiffness of each spring corresponds to one source of stiffness in the test specimen; i.e., k_1 is the flexural stiffness of the curved endplate between the bolt-lines, k_2 is the axial stiffness of the bolts over the thickness of the tube wall and the endplate, k_3 is the stiffness of the anchorage of the extension into the concrete, and k_4 is the membrane stiffness of the tube wall. The effect of prying action has been ignored in this model as the end-plates in the Series 1 tests were very stiff. Details of this model, and predictions made by it, are given in (Gardner and Goldsworthy 2005). In summary, the component model is able to predict the initial connection stiffness with reasonable accuracy. However, there is considerable scope for refinement of the stiffness models used in the component model for the T-stub in tension. Further parametric studies will look at factors influencing the non-linear stiffness of the endplate including the level of pre-tension in the bolts, the geometry, the plate thickness and the amount of curvature of the plate. This, in turn, will allow a prediction of the level of prying force.

5. Conclusions

A shop-welded, field-bolted moment-resisting connection using blind bolts with extensions has been developed to connect a composite beam (steel beam with composite concrete slab) to a concrete-filled circular steel tubular column. A pilot programme has been carried out to investigate the strength and stiffness in tension of a critical component of the connection, the T-stub connection between the beam flange and the tube wall. The results of this pilot programme, consisting of two series of tests, have been reported in this paper. These results confirm the effectiveness of the extensions, especially the cogged ones, in improving the strength and stiffness of the T-stub connections in tension. The anchorage provided by the reinforcing bar extensions prevents the excessive localized column yielding or shearing, and/or bolt pull-out observed in previous blind bolt connections to steel tubes (France *et al.* 1999, Muzeau *et al.* 1999, Barnett *et al.* 2000).

From the Series 1 test results, the following conclusions can be made:

1) Extensions of the types used in Specimens 3 to 5 provide sufficient anchorage to cause fracture of the bolts. Other premature failure modes were observed in the case of Specimens 1 and 2.
2) The type of extension used in Specimen 3 would allow a connection to be made at diametrically opposed sides of the column. This arrangement has been used in the Series 2-Specimen 1 test.

3) There is a dramatic increase in the secant stiffness (for the load range from zero to 300 kN) of the connection from Specimen 1 to Specimen 2, with modest increases from Specimen 2 to 3 etc. as the anchorage improved.
4) The strain demands in the tube wall were low (less than 500 $\mu\varepsilon$) for the levels of displacement and load likely to be experienced by the specimens.

From the Series 2 test results, the following conclusions can be made:

1) The stiffness of the T-stub connection to the circular tube was markedly higher than that of the T-stub connection to the square tube, even though the proportions of the various components were similar. One obvious reason for this is that the curved end-plate has a higher flexural stiffness than the flat endplate, and, hence, the flexural deformation of the endplate between the boltline and the web-plate was smaller for the curved endplate. Also, as a result of the higher flexural stiffness of the curved endplate, the prying forces in the bolts were reduced, and, for the same level of applied load, the slip of the extensions within the concrete was smaller.
2) In the case of the circular tube, the combined effect of the anchorage and membrane action in the tube wall was sufficient to cause bolt fracture to be the failure mode.
3) In the case of the square tube, the central bolts were not well confined, and some of the extensions pulled out before the bolts fractured.

The results of the feasibility study reported here indicate that a practical implementation of the types of connections outlined is achievable. However, further analytical and experimental work is needed to establish well-defined design procedures for these new connections. Areas needing further study include the following:

1) The reliability of the stiffness and strength of the anchorage provided by the extensions under both monotonic and cyclic loading is a major concern, and this is being studied in more detail in another project (Yao *et al.* 2004).
2) Full-scale sub-assemblage tests, similar to those preformed by (Stehle *et al.* 2001) for concrete frames, are needed to calibrate the theoretical models, and to ensure that the overall system behaves in a predictable manner. The state of stress in the column region at the joint will be complicated, for example, by the presence of moments and shears created by lateral loading on the frame, and will be different to that in the simple tension tests performed here. Also, shear due to gravity loading has not been included in these tests.
3) The option of creating a ductile partial strength connection could be explored in future work.
4) The frames considered here are only required to take lateral loads in one plane. Any transverse lateral loads are assumed to be taken by structural walls.
5) The resistance to the tension force on the T-stub relies on a combination of membrane action in the tube wall and anchorage through the extensions. A study is underway at the University of Melbourne to determine the division of the tension load between these two load resistance mechanisms, and the factors that influence this.
6) A steel-only moment-rotation curve has been included in this paper, However, further studies will investigate the influence of the slab.
7) The cost of this type of connection needs to be assessed relative to other types. However, it is not just the connection, but the entire system that should be subject to these comparisons. Efficiencies achieved in the overall frame construction, and the enhanced value due to improved robustness, will need to be included, at least in a qualitative manner.

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