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The practice of blind bolting connections to structural hollow sections: A review

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Abstract. Due to aesthetic, economic, and structural performance, the use of structural hollow sections as columns in both continuous moment resisting and nominally pinned construction is attractive. Connecting the beams to these sections is somewhat problematic as there is no access to the interior of the section to allow for the tightening of a standard bolt. Therefore, bolts that may be tightened from one side, i.e., blind bolts, have been developed to facilitate the use of site bolting for this arrangement. This paper critically reviews available information concerning blind bolting technology, especially the performance of fasteners in shear, tension, and moment resisting connections. Also provided is an explanation of the way in which the results have been incorporated into design guidance covering the particular case of nominally pinned connections. For moment resisting connections, it is concluded that whilst the principle has been adequately demonstrated, sufficient data are currently not available to permit the provision of authoritative design guidance. In addition, inherent flexibilities in the connections mean that performance equivalent to full strength and rigid is unlikely to be achievable: a semi-continuous approach to frame design will therefore be necessary.

Key words: blind bolts; bolts; fasteners; hollow sections; joints; simple connections; moment connections; structural design; tubular construction.

1. Introduction

The use of structural hollow sections as columns in multi-storey steel construction is attractive due to their enhanced structural performance when compared with the capacity of open sections of a similar size when subjected to large axial forces. Hollow sections have a pleasing appearance, and when consideration of usable floor space is properly accounted for, are competitive on economic grounds.

Typically, within the UK, for multi-storey steel construction, primary structural elements are connected by the use of endplates that are shop welded to the beam and bolted to the column *in-situ.* When a hollow section replaces an open section, however, this method of connection is

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problematic, as there is rarely access to the inside of the section to allow for the tightening of the nut. Therefore, novel techniques have been developed over recent years that have increased the practicality of connection to hollow sections.

Initially, it was common practice to fully weld all connections to hollow sections. Suitable design guidance for this method of connection is available (CIDECT 1992). However, there is a reluctance to utilise site welding due to concerns over the actual making and inspection of the connection. Thus, over the years, various alternatives have been sought, some of which are listed below.

One alternative was the shop welding of seat angles to the face of the column, permitting the beam to be installed and secured by bolting to the seat angles on site (Dawe and Grondin 1990). This method, as with several other similar methods (Picard and Giroux 1977, Tabuchi *et al.* 1988), has not generally found much support in practice due to the complex design procedures involved and the intricate installation of the components. Similar methods, such as shop welding fin plates and tee sections to the face of the tube, or allowing for a splice connection with the beam, have also been investigated.

Maquoi *et al*. (1984) investigated the formation of a beam to tubular column connection by welding threaded studs to the face of the hollow section column. These studs replaced the bolt shank and head of a standard bolt, thereby allowing for installation of the beam to the connection as would be done with a frame consisting of open sections. Not surprisingly, it was found that the studs were prone to damage during delivery to site and during erection, again resulting in limited practicality.

In order to permit the use of normal forms of bolted connection, whilst avoiding poor aesthetics, complicated detailing requirements and damage to components, fasteners have been developed over recent years which may be installed and tightened from one side of a connection only, i.e., blind fasteners. These permit the erection of frames with hollow section columns to be undertaken in an identical manner to that used for frames with open sections. It is the purpose of this paper to collate and synthesise the research to date on the performance of connections to structural hollow sections using blind fasteners, with the intention of assessing the extent to which soundly based design procedures are either already available or could be made available using existing primary data as the basis.

1.1. Blind fastener varieties

Several blind fasteners are available commercially, including the Flowdrill (Flowdrill B.V. Holland), the Huck High Strength Blind Bolt (HSBB) and Huck Blind Oversized Mechanically Locked Bolt (BOM) (Huck International, USA), and the Lindapter Hollobolt (Lindapter International, UK).

The Flowdrill process involves making a hole in the face of the hollow section by a new thermal drilling technique, which significantly increases the thickness of the face of the section around the hole. While still hot, a thread is incorporated into the hole, allowing a standard bolt to be installed. This process is shown in Fig. 1. Due to the quantity of hollow section wall requiring displacement during drilling, the Flowdrilling process is currently limited to a maximum wall thickness of 12 mm.

The Lindapter Hollobolt, the Huck HSBB and the Huck BOM, however, employ sleeves around a standard bolt designed to either expand or collapse on the inside of the clearance hole, thus pulling the connected plies into contact. The Lindapter Hollobolt possesses a threaded mild steel cone and a mild steel sleeve with four equidistant slots (Fig. 2). As the bolt head is tightened, the threaded cone rides along the shank of the bolt resulting in a flaring of the steel sleeve and the four flared legs

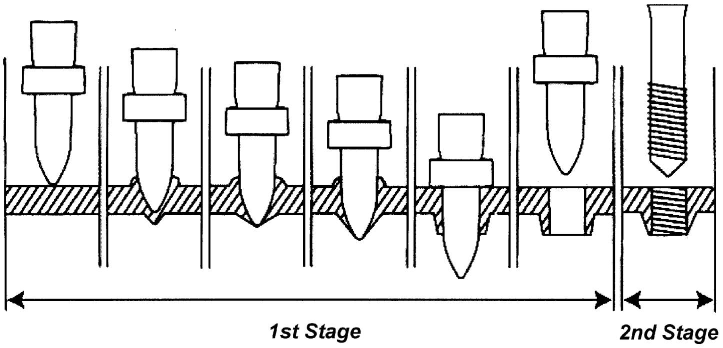

Fig. 1 The flowdrill process (Flowdrill B.V., Holland)

Fig. 2 The Lindapter Hollobolt (Lindapter International, UK)

clamping against the inside of the hole.

The Hollobolt evolved from the now superseded Hollofast, a fastener that was identical to the Hollobolt except for a knurled section at the top of the sleeve instead of a flat collar (Fig. 3). The reason why the Hollofast was superseded was due to the possibility of the insert being forced into the clearance hole on site by workers aligning holes with spanners.

The HSBB and BOM clamp the plies together by the use of collapsing mechanisms on the inside of the hole (Fig. 4). These fasteners are inserted into a clearance hole and are tightened by the use

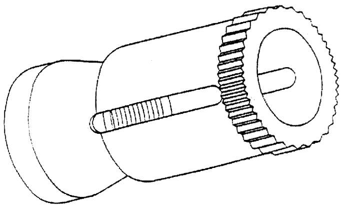

Fig. 3 The superseded Lindapter Hollofast (Lindapter International, UK)

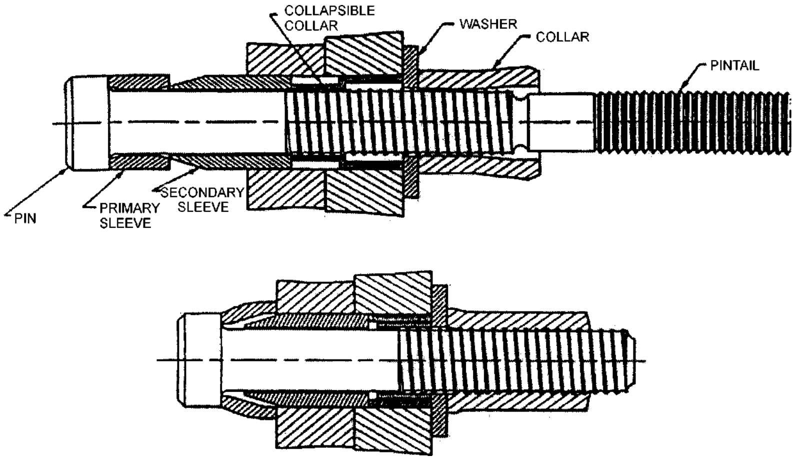

Fig. 4 The Huck HSBB before and after tightening (Huck International, Japan)

of a special tool on the pintail. As the bolt is tightened, the primary sleeve collapses over the secondary sleeve, thus forming an equivalent of a bolt nut. Installation is completed by the pintail breaking from the threaded portion of the bolt at a predetermined torque.

2. Fastener behaviour when subjected to direct shear

Direct shear tests that have been conducted on the Flowdrill, Lindapter Hollofast and Lindapter Hollobolt indicate that the capacities of blind fasteners differ from those observed with standard

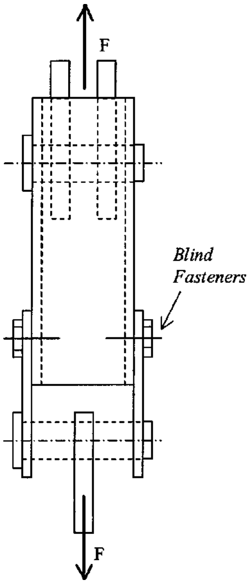

Fig. 5 Typical shear test arrangement

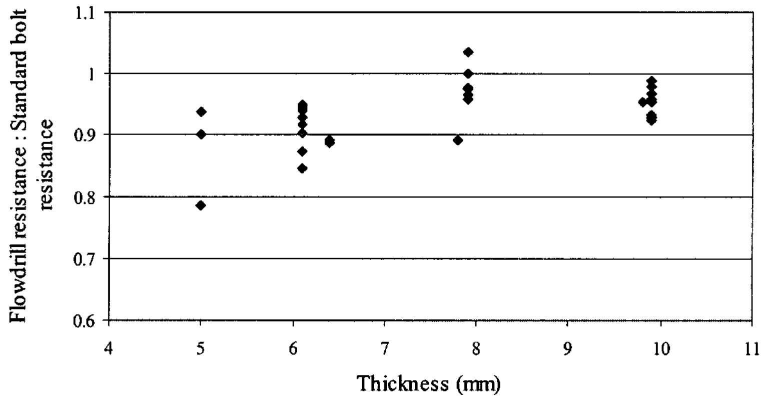

Fig. 6 Effect of plate thickness on the ratio of flowdrill strength: standard bolt strength (Ballereini *et al.* 1995)

dowel bolts, although their overall behaviour is largely the same. Typically, the blind bolt shear test arrangement consists of two plies connected to opposite sides of a hollow section by blind fasteners (Fig. 5). This arrangement allows for the determination of the capacities of the connected elements in shear, the interaction between the elements, and the influence of hollow section wall thickness on the capacity of the joint.

Ballerini *et al.* (1995) performed a series of such tests on 39 Flowdrill specimens and 22 standard dowel bolt specimens, noting the failure loads, and the slip of the bolt in the assembly. The tests were conducted using square hollow sections with wall thicknesses of 5, 6.4, 6.1, 7.9, 7.8, 9.8, and 9.9 mm, and four different bolt diameters (M12, M16, M18, and M20 grade 8.8 bolts). Two slightly different testing arrangements were employed: one using a pair of bolts in line, and one with a single bolt.

Failure loads for the Flowdrill specimens were (with two exceptions) slightly less than those obtained for standard bolts, as illustrated in Fig. 6. Statistical analysis of the results indicated that resistances were directly proportional to the thickness of the hollow section face, leading to the conclusion that the predominant failure mechanism was bearing of the section's face. Ballerini *et al.* also observed that the slip of the bolts in the assemblies increased with a decrease in hollow section thickness, again indicating that the influence of the tube was critical.

Results from direct shear tests using the Lindapter Hollofast and Hollobolt (Banks 1997(a), Banks 1997(b), Occhi 1995) have suggested that these bolts behave in a similar manner to standard dowel bolts. An experiment to determine the behaviour of a pair of grade 8.8 M20 Hollobolts in single shear between a plate and a rectangular hollow section (Banks 1997(a)) has shown that the connected plies experience extensive deformation around the clearance hole due to plate bearing, leading to a rotation of the bolt in the assembly. This rotation continues proportionally with increase in shear load until failure of the section occurs due to a pullout of the insert from the deformed holes. Furthermore, partial shear failure was observed to the sleeve of the Hollobolt.

A similar experiment (Banks 1997(b)), which was performed to determine the behaviour of two rows of grade 8.8 M20 Hollobolts in single shear (Fig. 7), produced a similar failure mechanism. Initially, the inner bolt suffered no rotation as there was no clearance hole deformation, and was subjected to pure shear only. The outer bolt was subjected to both shear and rotation caused by deformation of the hole and as the shear load increased, so did the rotation of this bolt. At a certain load, deformation of the clearance hole of the inner bolt commenced, resulting in bolt rotation. Again, failure was observed to be due to rotational pullout of the bolts from the holes with partial

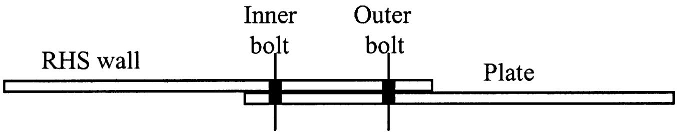

Fig. 7 Double row of bolts in shear

shear failure to the Hollobolt sleeve.

These two failure mechanisms are typical of plate bearing type failure (Owens and Cheal 1989), and therefore suggest that the shear strength of the M20 fasteners was not critical in this type of connection. The maximum loads that were achieved in these tests were 530 kN for the single row shear specimen (i.e., 265 kN per bolt) and 1102.5 kN (i.e., 275 kN per bolt) for the specimen with two rows of bolts.

The Flowdrill tests performed by Ballerini *et al.* provided a mean maximum shear resistance of 148.5 kN for an M20 bolt. This is significantly lower than that obtained in the Banks investigations. It is likely that the Hollobolt has a greater shear resistance because of its larger cross sectional area due to the expanding sleeve.

Occhi (1995(a)) performed tests using the Hollofast and Hollobolt in single shear, and recorded the failure loads of the joints. Single rows of grade 8.8 M12, M16, and M20 bolts were tested in square hollow sections of varying wall thickness. In all cases, the failure load was observed to be largely independent of wall thickness. However, Occhi did not record the failure mechanism for the joints. Therefore, it is necessary to attribute failure to either shear or bearing of the insert, on the basis that the resistance of the plate to bearing and shear failure is proportional to its thickness.

The M12 and M16 Hollofast assemblies failed at an average shear load of 87 kN and 154 kN respectively, i.e., 43.5 kN and 77 kN per bolt. The design shear capacity of the corresponding standard bolts is 31.6 kN and 58.9 kN respectively (BS 5950 1995). This represents a ratio of observed shear capacity for the Hollofast to the design shear capacity of the standard bolt of 1.4. Also, the M12 Hollobolt assemblies failed between 108 kN and 127 kN, i.e., 54 kN and 63.5 kN per bolt. This represents a ratio of observed shear capacity for the Hollobolt to nominal design shear capacity for a standard bolt of 1.8. M16 and M20 bolts possessed an average ratio of ultimate capacity to nominal design load of 1.97 and 2.25 respectively. Evidently, this is acceptable for shear connection design. The failure loads of the Hollobolt and Hollofast specimens are illustrated in Fig. 8.

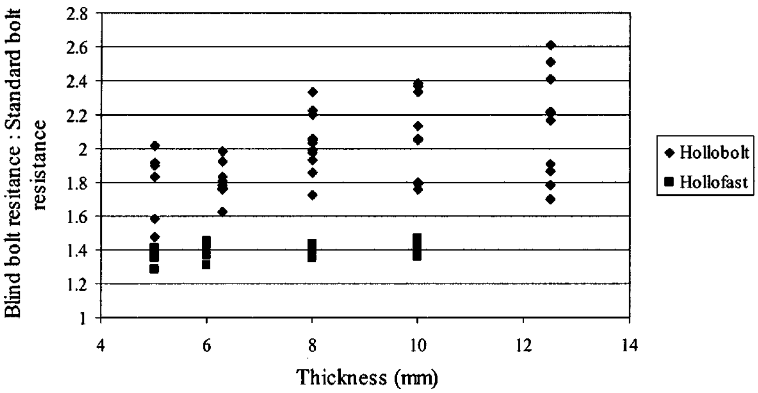

Fig. 8 Hollobolt and Hollofast shear capacities (Occhi 1995)

The observations made by Ballerini *et al.* (1995) regarding the slip at failure load of the Flowdrill assemblies was also observed with the Hollofast and Hollobolt assemblies. In all cases, the tests that were performed using a thicker hollow section wall were associated with significantly reduced overall deformation of the specimen, as expected.

It should be noted that the performance of the Huck fasteners in direct shear is not reported in the literature.

3. Fastener behaviour when subjected to direct tension

There have been several investigations (Korol *et al*. 1993, Occhi 1995, British Steel 1996) to determine the practicality of using blind bolts in connections where the bolts are predominantly subjected to axial forces and in simple nominally pinned connections where, due to the need to satisfy structural integrity criteria, tensile resistance is an issue. The relevant design guide (BS5950:1995) states that a factored tensile load of 75 kN at floors and 40 kN at roof level must be resisted to ensure that disproportionate collapse does not occur.

It is well known that a tensile load applied to a standard bolted joint will result in net tension in the connection after the bolt preload is exceeded (Owens and Cheal 1989). This tension will increase with the applied tensile load until failure of one part of the connection occurs. For high tensile steel bolts with mild steel nuts, it is likely that thread stripping of the nut will occur, prior to the yielding of the shank. Bolts with nuts of the same material grade, however, are likely to fail due to yielding of the bolt shank. It is also possible, depending upon the material characteristics and the associated plate thickness, that yielding of the connected elements will occur.

Investigations into the tensile capacity of the Huck HSBB and BOM, (Korol *et al*. 1993, Huck International), using grade 8.8 equivalent 20 mm diameter bolts, have shown that the minimum tensile strength of the HSBB and BOM are 7.3% greater and 13.9% less, respectively, than the nominal design load of a similar size standard bolt, which is 192 kN and 129 kN. Furthermore, it was reported that the minimum clamping forces exerted by the HSBB and BOM were 4% greater and 68% less, i.e., 130 kN and 40 kN, respectively than that observed with a similar size standard bolt. It may therefore be seen that the HSBB will perform adequately in tension connections when compared with standard bolts, and that there is a significant reduction in strength with the BOM. It is not possible, however, to comment further on the qualitative behaviour of these bolt types as there are no detailed reports available in the literature.

The behaviour of the Hollofast and Hollobolt when subjected to direct tension is very different to that observed with standard bolts. Tests have shown (Occhi 1995) that two failure mechanisms are likely, with the occurrence being dependent upon the thickness of the material in which the bolt is placed. Single grade 8.8 M12, M16, and M20 Hollobolts were pulled out of rectangular hollow sections possessing varying wall thickness. For a wall thickness of up to 8 mm, extensive deformation to the tube face was observed which increased with increasing applied load. This led to failure being caused by the whole insert being pulled out of the section. For larger rectangular hollow sections, i.e., tubes with a wall thickness of 8 mm and greater, a different failure mechanism was observed. Again, the chord face deformed with increasing tensile load. However, the stiffer wall, in conjunction with the sharp edges on the inside of the drilled clearance hole, led to a shear failure of the flared legs of the fastener against the side of the hole. The remainder of the sleeve, the threaded cone, and the bolt were subsequently pulled through the hole. This occurred at approximately 1.7 times the

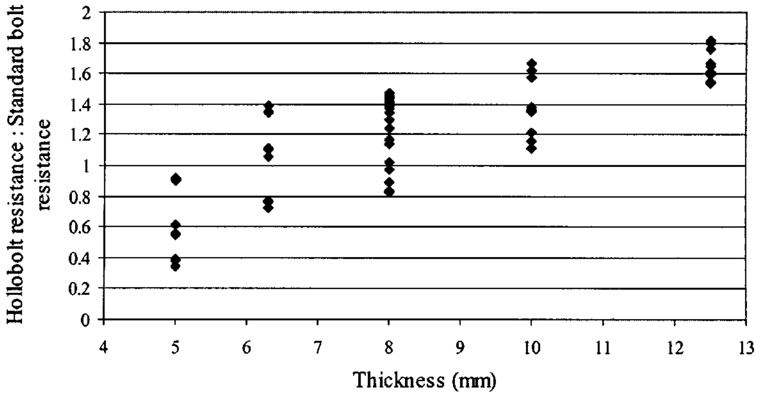

Fig. 9 Effect of plate thickness on the ratio of Hollobolt strength: standard bolt strength

nominal design tensile strength of a standard bolt. The effect of the wall thickness on the tensile capacity of the joint is shown in Fig. 9.

An analogy may be drawn between the two modes of failure that were observed in these tests and the modes of failure that are provided as design cases for moment connections to open sections using the EC3 component method (EC3 1992, SCI 1995). This method of connection design treats the joint as a series of components. Design is performed by a series of capacity checks on each component, including tensile region capacity, web buckling, and bolt shear checks. In the tension region of a connection to open sections, the design failure mechanism is dependant upon the thickness of the plating, and the tensile capacity of the bolts. In a connection with high capacity bolts and thin structural members, yielding of the section will be the predominant failure mechanism (mode 1 failure: Fig. 10(a)). An increase in plate thickness will result in some plate yielding at the failure load of the fastener (mode 2 failure: Fig. 10(b)), i.e., an interactive mode of failure. Connections with very thick plating will typically result in bolt failure only (mode 3 failure: Fig. 10(c)).

Although the geometry of the connections to hollow sections differs from that observed with open sections, and the blind bolt capacities differ from those of a standard bolt, it may be seen that the tests with wall thicknesses of less than 8 mm indicated a mode 1 failure mechanism. Similarly, the failure of the fastener due to leg shear combined with plate deformation observed with tests with a wall thickness of greater than 8 m indicated a mode 2 mechanism.

It should be noted that serviceability considerations would have limited the maximum usable tensile load in many of the above tests. Prior to ultimate failure of the joints with thin plates, the

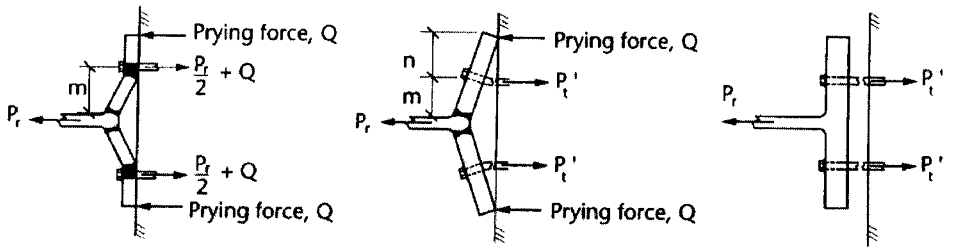

Fig. 10 Theoretical tee stub failure mechanisms (EC3 1992, SCI 1995)

hollow section chord deformation was such that serviceability limits given in appropriate design codes (EC3 1992), i.e., a 1% deformation limit of the sections chord face, were exceeded. Hence, it may be concluded that the tension capacity of the Hollobolt is not critical in direct tension connections as the tensile resistance is limited by the deformation of the chord face.

Tests performed by British Steel (1996) and Yeomans (1998) have verified that the flexibility of the hollow section face does actually limit the load carrying capacity of a tension connection. For a series of 150×150×5, 8, and 12.5 mm square hollow sections connected to a tensile testing machine by eight grade 8.8 Hollofast or grade 8.8 Hollobolt inserts (four on either side of the tube), the now superseded Hollofast failed prior to the calculated capacity of an equivalent standard bolt, albeit with excessive deformation of the hollow section face. All tests performed with the Hollobolt, however, showed that serviceability considerations of the hollow section, and not the bolt capacity, were critical.

It may be concluded, then, that the flexibility of the hollow section face will very often limit the capacity of a tension connection. This is an important consideration when assessing the extent to which connection design using Hollobolts can simply follow established procedures for dowel bolts and whether a range of connections exists for which equivalent design capacities may be achieved.

4. Simple connection design

The results of the shear tests performed by Ballerini *et al*. (1995) and Occhi (1995), and the tension tests performed by Occhi (1995), and British Steel (1996), have resulted in the production of a Design Guide allowing for the construction of simple connections to hollow sections using the Flowdrill and Hollobolt connectors. By applying a factor of safety to the empirically determined capacities, it is possible to ensure that a connection may carry both vertical loads (i.e., shear loads) and horizontal loads (i.e., loads arising from axial forces in the connected beam or from structural integrity criteria).

The Guide (British Steel 1997) provides details to allow for shear connection by following the procedural checks outlined by SCI (1991) for connections using double angle cleats or flexible end plates. Shear and wall bearing checks are required, and when necessary, structural integrity checks are provided.

With regards to structural integrity, the Guide provides two design checks. Firstly, the tensile capacities of the Flowdrill and Hollobolt fasteners, as determined in the Occhi (1995), and British Steel (1996) investigations, are modified by an appropriate factor of safety. Furthermore, analysis of the theoretical yield pattern in the wall of the section, as shown in Fig. 11, results in the equations shown in Table 1(a). The bolt capacity is also shown in this table. The nomenclature adopted is summarised in Table 1(b).

5. Moment connections

Several series of tests have been performed to ascertain the efficacy of producing moment resisting connections to structural hollow sections using blind bolts. The key results are presented below.

Korol *et al.* (1993) determined the capacities of connections using three 203×203×12.7, one

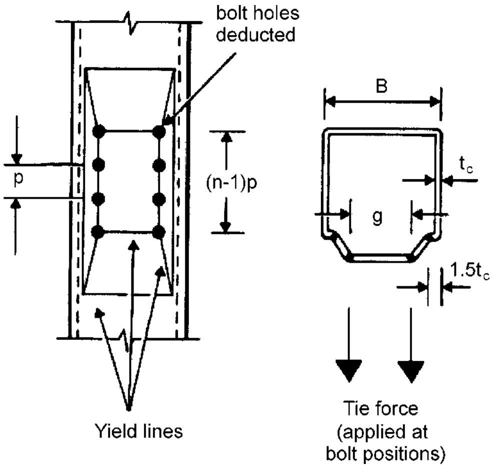

Fig. 11 Theoretical yield line pattern for limit state structural integrity criteria (British Steel 1997)

Table 1 (a) Design checks using the British Steel (1997) design guide

Bolt capacity		Hollow section face yielding capacity for structural integrity requirements
Shear requirement	$P_v \geq Q$	$Fc = \dfrac{2p_{yc}t_c^2}{1-\beta_1}[\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}]$
Bearing requirement	$P_v \geq Q$	$\eta_1 = \dfrac{(n-1)p - \frac{n}{2}d}{(B-3t_c)}$
		$\beta_1 = \dfrac{g}{(B-3t_c)}$
		$\gamma_1 = \dfrac{d}{(B-3t_c)}$

Table 1 (b) Nomenclature for Table 1(a)

P_v the local shear capacity of the column wall = smaller of 0.6 $p_{yc}A_v$ and $0.5U_{sc}A_{vnet}$		$A_v = \left[\dfrac{g}{2} + (n-1)p + e_t\right]t_c,\ [e_t \leq 5d]$
$A_{vnet} = A_v - ndt$		n = number of rows of bolts
p = bolt pitch		B = The width of the hollow section wall
d = hole diameter	g = bolt gauge	b = the bolt diameter
t_c = wall thickness	p_{yc} = hollow section design strength	U_{sc} = ultimate tensile strength of wall.
P_{bsc} = bearing capacity of the hollow section wall		

254×254×9.53, and one 254×254×11.13 mm hollow sections, connected by A325 bolts (i.e., 19 mm diameter grade 8.8 equivalent standard dowel bolt), Huck HSBB, and Huck BOM bolts. It was observed from comparison of the moment-rotation (M-Φ) relationships that the HSBB performed very similarly to the A325 bolt in the 203×203×12.7 connection, while the M-Φ curve for the BOM showed a lower stiffness and moment capacity. As the HSBB possesses a similar preload and capacity to standard bolts, and the BOM possesses a much lower preload and capacity when compared with standard bolts, the exhibited stiffnesses of these tests are as expected. Failure of the specimens connected with the A325 and HSBB bolts was principally due to local shearing of the tube wall around the underside of the bolt, at moments in the region of 190 kNm to 250 kNm (i.e., 1.01-1.33 times the nominal bending capacity of the hollow section). The tests performed with the larger hollow sections demonstrate that a larger section provides both a higher stiffness and a larger failure moment, indicating that the flexibility of the section's face is influential in the capacity of a connection of this type.

British Steel (1996) and Yeomans (1998) have recorded the M-Φ behaviour of 150×150×5, 8, and 12.5 mm hollow sections connected with both M16 and M20 grade 8.8 Hollobolts. Nine tests were performed with bolt gauges of 60 and 90 mm, and the results showed that the ultimate moment that was resisted by the connection varied between 60-145 kNm. When comparisons were made with the calculated theoretical capacities of the connection, it was shown that the mean ratio of service moment (i.e., the moment that results in a 1% deformation of the flange face length multiplied by a load factor of 1.4): calculated service moment ratio varied between 0.534 and 1.625 with a mean of 0.867. The ratios of observed ultimate moment (which is the moment required to cause a 3% deformation to the section face): calculated service moment, and observed failure moment: calculated service moment were between 0.588 and 1.992, with a mean of 1.259, and between 1.063 and 4.066, with a mean of 2.501 respectively. These load ratios have been used to indicate the effect of the face and connection flexibility on the moment capacity of the connection. It was demonstrated that the ratios became larger in proportion to the overall flexibility of the specimens. It was noted, however, that the results were inconclusive due to the Hollobolt providing variable clamping forces resulting in inconsistent connection stiffnesses.

France *et al.* (1999) performed a series of moment connection tests using 200×200×8, 10, and 12.5 mm hollow sections connected with grade 8.8 Flowdrill bolts and both flush and extended endplates. In all tests using extended endplates, it was observed that predominant failure was due to stripping of the Flowdrill threads from the hole with large deformation of the hollow section in both the tension and compression regions. Failure occurred at moments of 162 kNm and 208 kNm (i.e., 0.86-1.1 times the nominal bending capacity of the hollow section) with the thickness of the hollow section being directly proportional to the moment capacity. Tests performed using flush endplates showed a significant reduction in moment capacity compared with the extended endplate tests. The maximum moments that were resisted by the two connections were 104 kNm and 138 kNm (i.e., 0.55-0.73 times the nominal bending capacity of the hollow section), again the thicker section contributed the greater resistance moment. Failure was due to extensive yielding of the hollow section, with subsequent yielding of the bolts in the tension region of the joint.

As with tension tests, it may be seen that the flexibility of the hollow section face is often the critical factor in the performance of moment resisting connection, with loads being lower than the bolt capacity.

5.1. Rigidity of blind bolted moment connections

Several procedures exist for determining the moment capacity of a connection, including the EC3 component method, which also extends to classifying a connection as rigid, semi- rigid or nominally pinned based on its M-Φ characteristics. Due to the inherent flexibility of the face of a hollow section, it was observed in all of the tests described above, that the M-Φ behaviour should be classed as semi-rigid according to the criteria suggested by Nethercot *et al*. (1998).

The tests performed by France *et al.* demonstrated that excessive rotations occurred at high moments, mainly due to the inherent flexibility of the hollow section face. There is a need, therefore, to determine a limit at which serviceability rotations are exceeded. Nethercot *et al*. have suggested criteria that limit the amount of connection rotation at the serviceability limit state for rigid, semi-rigid, and nominally pinned connections. Determination of stiffness criteria for a given frame geometry will provide values of lower bound stiffness for a moment resisting connection and an upper bound stiffness for a nominally pinned connection. Upon classification of the connection type, it is possible to state a limit for the connection rotation. For a semi- rigid connection, this value is:

$$\theta_r = \frac{2 - r'}{6} \cdot \frac{M_{db} L}{EI}$$

where r' is the ratio of the beam moment to the connection moment at the serviceability limit state, M_{db} is the design moment of the connected beam, E, L, and I, are the elastic modulus, second moment of area, and length of the connected beam respectively.

Analysis of the connection tests performed by France *et al.* using the classification system suggested by Nethercot *et al.* demonstrates that moments were resisted far in excess of those permitted by the serviceability limit state. The analysis is summarised in Table 2, using unity for the value of r' as the specimens were loaded as a cantilever.

Furthermore, use of the Nethercot *et al.* method of classification at the ultimate limit state also indicates that the connections should be considered as semi-rigid. For this case, the rotation capacity may be expressed as:

$$\theta_u = \left[0.344 - 0.212\frac{M_{dc}}{M_{db}} + \left(\frac{M_{db} - M_{yb}}{M_{pb} - M_{yb}}\right)^2 \frac{1}{\sqrt{1 + M_{dc}/M_{db}}}\right]\frac{M_{db} L}{EI}$$

Where M_{dc} is the connection design moment, M_{pb} is the beam span ultimate moment capacity, and

Table 2 Results of analysis of moment resisting connection tests at the serviceability limit state using the criteria suggested by Nethercot *et al.* (1998)

Hollow section size	Lower bound stiffness for moment connection (kNm/rad)	Upper bound stiffness for pinned connections (kNm/rad)	Observed stiffness (kNm/rad)	Rotation capacity	Moment at rotation	Maximum moment
200×200×8	201×10^6	4.9×10^6	Approx 40×10^6	5.5 mrads	Approx 95 kNm	Approx 160 kNm
200×200×10	253×10^6	4.8×10^6	Approx 45×10^6	5.5 mrads	Approx 130 kNm	Approx 210 kNm
200×200×12.5	282×10^6	4.7×10^6	Approx 50×10^6	5.5 mrads	Approx 180 kNm	Approx 280 kNm

Table 3 Results of analysis of moment resisting connection tests at the ultimate limit state using the criteria suggested by Nethercot *et al.* (1998)

Hollow section size	Lower bound stiffness for moment connection (kNm/rad)	Upper bound stiffness for pinned connections (kNm/rad)	Observed stiffness (kNm/rad)	Rotation capacity	Moment at rotation	Maximum moment
200×200×8	298×10^6	70.58×10^6	Approx 40×10^6	54.6 mrads	Approx 160 kNm	Approx 160 kNm
200×200×10	451×10^6	7.96×10^6	Approx 45×10^6	51.9 mrads	Approx 210 kNm	Approx 210 kNm
200×200×12.5	470×10^6	8.29×10^6	Approx 50×10^6	46.0 mrads	Approx 280 kNm	Approx 280 kNm

M_{yb} is the beam span yield moment capacity. Again a further requirement for semi-rigidity is that the stiffness of the connection must lie within the stiffness of a fully connected connection and the stiffness of a nominally pinned connection, i.e.:

$$\frac{38\alpha}{(2+\alpha)}\frac{EI}{L} > k > \frac{0.67\alpha}{(2+\alpha)}\frac{EI}{L}$$

where α is the column to beam rotational stiffness ratio. As before, these equations have been used to analyse the results of the tests performed by France *et al.*, as shown in Table 3. In conducting this analysis, it has been assumed that the design moment capacity of the beam is simply the yield moment capacity divided by a factor of safety of 1.4. Furthermore, it has been assumed that the connection design moment may be taken as the moment at which the tangents of the initial and plastic rotational stiffnesses meet.

The results of these analyses indicate that it is unlikely to be possible, considering both the serviceability limit and the ultimate limit, to develop a fully moment resisting connection due to the flexibility of the hollow section face.

Hasan *et al.* (1997) have performed statistical analyses on a series of previously published moment connection tests to open sections, and stated that a connection may be classified as rigid if the ratio of moment resistance obtained from the test data to the moment for a fully rigid connection (i.e., the moment obtained from a fully rigid analysis) equals unity, and that the initial rotational stiffness, i.e., the gradient of the M-Φ curve, is at least $10^{5.05}$ kNm/rad (Fig. 12).

The moment connection tests that have been described using the Flowdrill (France *et al.* 1999), the Lindapter Hollobolt (British Steel 1996), and the Huck BOM and HSBB (Korol *et al.* 1993) have shown a maximum obtainable initial stiffness of 50×10^6 kNm/rad, i.e., significantly greater than the value required for full rigidity suggested by Hasan *et al.* However, the suggested value for the initial rotational stiffness was empirically derived from the results of tests to open sections where the full moment capacity of the connected beam was resisted by the connection at values of stiffness of greater than $10^{5.05}$ kNm/rad. It is therefore felt that this classification system is somewhat limited in accurately describing the behaviour of the complicated structural system as failure of the connection occurred at very large rotations, and at a moment that was often far smaller than the moment capacity of the connected beam.

However, it is felt that the more rigorous classification system that has been suggested by Nethercot *et al.* confirms that frames employing such connections should be treated as semi-continuous for the purpose of design (EC3 1994), since the flexibility of the joints in the frame will

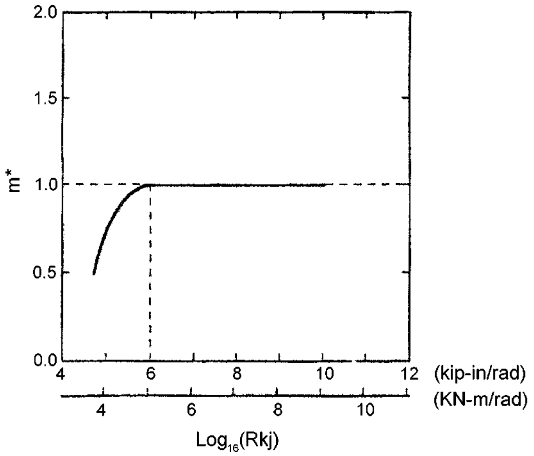

Fig. 12 Limiting criteria for full rigidity in a moment connection (Hasan *et al.* 1997)

significantly affect the bending moment distribution in the structure.

Although these results strongly suggest that these forms of connections will function as semi-rigid, it was not possible to examine the factors influencing this behaviour because the M-Φ relationship for each connection type is unique, and different section sizes, bolt types, and geometric bolt configurations have been used in each of the investigations described above, resulting in independence between the different sets of results. It is felt therefore, that further tests should be performed in order to establish a better understanding of the factors influencing moment connection behaviour to hollow sections in terms of M-Φ behaviour and the EC3 component method, and to determine further, given the inherent flexibility of the hollow section face, the applicability of using these sections in moment resisting connections. The test data reviewed herein strongly suggests that it will be necessary to adopt a semi-continuous approach, with the result that actual connection characteristics are required for use in the overall frame analysis.

6. Conclusions

It has been stated that blind bolts provide a convenient and reliable means of connecting to hollow sections when compared with early developments.

Tests regarding the performance of shear and tension connections using blind bolts to structural hollow sections have been reviewed, and the method by which the test results have led to the formulation of design guides to allow for these connections in simple construction have been discussed. Production of the design guides for nominally pinned connections in simple construction confirms that there is sufficient knowledge on the behaviour of the structural elements in the connection to allow for safe design. A summary of results for moment connections that have been obtained to date has also been presented. On this basis, it is felt that there is insufficient knowledge at present for the safe design of a semi-rigid moment connection due to lack of understanding of the fundamental behaviour of the joint. Therefore, it is suggested that additional tests should be performed, employing a previously used geometric bolt configuration and hollow section size. In order to ascertain the applicability of the EC3 component method to connections of this type, the effectiveness of hollow sections in resisting moments, and factors influencing the M-F behaviour all need to be investigated. Assessments of connection flexibility indicate that a semi-continuous approach to frame design will be necessary. Only when this is fully understood will it be possible to produce a moment connection design guide.

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References

Ballerini, M., Piazza, M., Bozzo, M., and Occhi, F. (1995), "The flowdrill system for the bolted connection of steel hollow sections. Part II: Experimental results and design evaluations for shear connections", *Costruzioni Metalliche*, No. 5.

Banks, G. (1997a), "Hollobolt joint shear tests project No. S2860", *Memo 146/RJ, British Steel Plc*., Swinden Technology Centre, Rotherham.

Banks, G. (1997b), "Hollobolt joint shear tests", *Memo 129, British Steel Plc*., Swinden Technology Centre, Rotherham.

British Steel Tubes and Pipes (1996), "Hollofast and hollobolt system for hollow section connections", CIDECT Report No. 6G-14(A)/96.

British Standards Institution (1995), "Structural use of steelwork in building, part I: code of practice for design simple and continuous construction: Hot rolled sections", London, BS 5950.

British Steel Tubes and Pipes (1997), "SHS Jointing: flowdrill and Hollobolt", Corby.

CIDECT, J., A., Packer, J., Wardenier, Y., Kurobane, D., Dutta and N., Yeomans, (1992), *Design Guide for Rectangular Hollow Section (RHS) Joints under Predominantly Static Loading*, Guide No. C03.

Dawe, J.L. and Grondin, G.Y. (1990), "W-shaped beam to RHS column connections", *Can. J. Civ., Engrg.*, **17**(3), 788-797.

Eurocode 3. (1992), *Design of Steel Structures. Part 1.1 General Rules and Rules for Buildings*, DD ENV 1993-1-1.

France, J.E., Davison, J.B. and Kirby, P.A. (1999), "Strength and rotational response of moment connections to tubular columns using flowdrill connectors", *J. Construct. Steel Res.*, **50**, 1-14.

Hasan, R., Kishi, N., Chen, W.F. and Komuro, M. (1997), "Evaluation of rigidity of extended end-plate connections", *J. Struct. Engrg.*, ASCE, **123**(12), 1595-1602.

Industrial Fastening Systems (1990), *Huck International Inc.* Irvine, California.

Korol, R.M., Ghobarah, A. and Mourad, S. (1993), "Blind bolting *W*-shape beams to HSS columns", *J. Struct. Engrg.* ASCE, **119**(12), 3463-3481.

Maquoi, R., Navean, X. and Rondal, J. (1984), "Beam-column welded stud connections", *J. Construct. Steel Res*, **3**, 3-26.

Nethercot, D., Li, T. Q. and Ahmed, B. (1998), "Unified classification system for beam to column connections", *J. Construct. Steel Res*, **45**, 39-65.

Occhi, F. (1995), "Hollow section connections using (Hollofast) hollobolt expansion bolting", *Second Interim Report 6G-16/95*, Sidercad, Italy.

Owens, G.W. and Cheal, B.D. (1989), *Structural Steelwork Connections*, Butterworths, Sevenoaks.

Picard, A. and Giroux, Y. (1977), "Rigid connections for tubular columns", *Can. J. Civ., Engrg.*, **4**(2), 134-144.

Steel Construction Institute (1991), *Joints in Simple Construction: Volume 1: Design Methods*, BCSA/SCI, Ascot.

Steel Construction Institute (1995), *Joints in Steel Construction: Moment Connections*, BCSA/SCI, Ascot.
Tabuchi, M., Kanatani, H. and Kamba, T. (1988), "Behaviour of tubular column to H- beam connections under seismic loading", *Proceedings of the Ninth World Conference On Earthquake Engrg.*, **4**, 181-186.
Yeomans, N. (1998), "Rectangular hollow section column connections using the Lindapter Hollobolt", *Proceedings of 8th International Symposium on Tubular Structures*, Singapore.