Effect of concrete infill and slenderness on column face component in anchored blind-bolt connections

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Abstract

Previous research in blind-bolted connections to hollow columns have provided a wealth of experimental data but little was provided on parametric analytical models to guide their design. One of the connection components that requires further understanding is the column face bending. This paper presents the results and evaluation of experimental tests and finite element analyses for the bending behaviour of the column face component. Tests were conducted on full-scale connections to investigate the effect of concrete type and strength on the component. The results demonstrated that concrete strength has direct influence on the component strength and stiffness. The increase in concrete strength correlated with significant increase in the component strength. The increase in the component stiffness was less significant and is found to be limited by the column slenderness ratio (width/thickness). Slenderness ratio limits were defined beyond which the increase in concrete strength will have no effect on component stiffness. A formula for evaluating the confinement effect on the concrete infill is proposed. The strength and stiffness of the component were not affected when using self-compacting concrete while the use of light weight concrete reduced both. The failure mode of the component is bolt anchorage failure followed by a complete yielding of the column face plate around the bolts.

Key words: Anchored bolted connection; column face bending; concrete strength; concrete filled tubular columns

1. Introduction

The use of Steel Hollow Sections (SHS) is attractive due to their aesthetic appeal and high strength to weight ratio. However, their use is restricted due to the complex nature of the beam-column connections because of their closed form profile. It is a common practice to weld some fittings to the SHS such as fin plates, T-stubs and reverse channel which can be bolted to the beams on site (Dawe and Grondin 1990; Echeta and Owens 1981; Lu 1997;

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Málaga-Chuquitaype and Elghazouli 2010). Another alternative is to use the so-called blind bolting mechanical fasteners which can be assembled on-site as they can be tightened from one side only. Different types of blind bolts were developed such as HoloBolt (HB) (Lindapter 2011), Reverse Mechanism HoloBolt (RMH) and Extended HoloBolt (EHB) (Tizani and Ridley-Ellis 2003), Flowdrill (Flowdrill 2017) and Ajax bolt (Ajax 2012). Each type of blind bolts has specific geometric properties and requires a particular procedure for installation. The adequacy of using different types of blind bolts in connection to SHS has been investigated by many researchers and it was stated that the inherent flexibility of the column face of SHS governs the moment connection behaviour, so that the connections failed due to the excessive column face deformation (Barnett et al. 2000; France 1997; Mourad 1993). Concrete infill to the hollow sections is used to improve the connection behaviour. It results in a significant reduction in the column face deformation (France 1997; Ghobarah et al. 1996; Mourad 1993). However, the use of concrete infill was not sufficient to result in rigid or semi-rigid blind bolted connections. Gardner and Goldsworthy (1999) and (2003) have used a new concept of modifying the Ajax ONESIDED blind bolt by welding a piece of steel reinforcing bar to the end of the bolt shank that was anchored in the concrete infill. The connection strength and stiffness were improved due to the anchorage because of the reduced deformation of the tube wall. Tizani and Ridley-Ellis (2003) tried to obtain more benefits from the concrete infill by developing extended and anchored blind bolting system termed the Extended HoloBolt (EHB) (Fig. 1). The extension was achieved by simply using a longer bolt and the anchorage was achieved by using an end rounded nut. This solution proved practicable and effective. Similar solution was later used on the Ajax bolt by Yao et al.(2011).

Fig. 1: HoloBolt (HB) and Extended HoloBolt (EHB)

One of the advantages of using EHB is its rapid and easy installation, because it does not need any special skills or equipment. The installation procedure for EHB is similar to HB (Fig. 2). The first step is inserting the HB throughout the pre-drilled fixture and steel work. Then, the collar should be gripped with an open-ended spanner. Finally, the central bolt should be tightened using a torque wrench to manufacture specified value (Lindapter 2011). The tightening of the central bolt forces the cone to move inside the sleeve. During this movement, the sleeve legs start opening and tightening on the bolt hole to achieve the required clamping action.

Pull-out test data confirmed that the behaviour of the EHB is comparable with that of a standard bolt (bolt–nut–washer system). The stiffness of EHB is approximately double of the stiffness of HB. Therefore, EHB can be used to perform rigid connections, whereas HB can at best exhibits semi rigid connections (Ellison and Tizani 2004; Pitrakkos and Tizani 2013; Tizani and Ridley-Ellis 2003).

on the bolt hole to achieve the required clamping action.

Fig. 2: Installation of HolloBolt (Lindapter, 2011)

The possible modes of failure for EHB connections at the column face-bolt interface are: bolt failure, column face bending failure and combined failure (simultaneous EHB and column face failure). Pitrakkos and Tizani (2013) and Tizani and Pitrakkos (2015) performed an extensive experimental programme to study the first mode of failure. They confirmed that the pull-out resistance of the EHB is higher than the tensile capacity of the internal bolt of the EHB and that the behaviour of the EHB is comparable to standard bolt in terms of stiffness, strength and failure mechanism, which makes it suitable for use in moment-resisting connections. Other studies on the EHB included studying its fatigue behaviour (Tizani et al. 2014) and its behaviour in fire (Pascual et al. 2015). Complementary research has been carried out by Agheshlui et al. (2016) and (2016) and Oktavianus et al., (2017), in which they used a headed stud extension (sometimes with two embedded heads instead of one) as an integral part of an Ajax ONESIDE bolt. More recently, Jeddi and Sulong (2018) proposed a development to the EHB by increasing the bolt shank length and adding a second expandable sleeves. The modified bolt termed as TubeBolt. They stated that the new modification improved the bolt strength and stiffness significantly. The recent research in the area of blind bolting connections confirmed that these fasteners have the ability to provide rigid connections (Tao et al. 2017; Tao et al. 2017; Waqas et al. 2019). This finding and the advantages of blind bolting connections (easier and faster fabrication, lower cost and higher stability compared with welded connections) highlight the need for the development of design guidance for moment resisting blind bolting connections to tubular sections. The provision of such models in codes of practice can widen the use of tubular sections and such connections in

structural applications.

To date, there are few studies that had investigated the second mode of failure of the EHB connections (Mahmood et al. 2014; Mahmood et al. 2014; Mahmood et al. 2015). Therefore, there are needs for further investigations into this mode of failure to provide the knowledge required for developing a design model for the bending behaviour of the column face component of the EHB connections.

The concrete infill has significant impact on the behaviour of this type of connections by providing the anchoring resistance of the EHB. Therefore, this research aims to provide the fundamental understanding of the effect of the type and strength of concrete in addition to the slenderness ratio on the bending behaviour of the column face component in EHB connections.

2. Experimental programme

2.1. Test specimens

The experimental programme consisted of testing fifteen specimens. The specimen length is 760mm (**Fig. 3**). This length was chosen based on the findings of (Mahmood 2015) to provide sufficient distance between the specimen supports and the bolt centreline to offer clear distance for the full development of face bending. For all specimens, both of the anchorage length (L_{an}) (Fig. 4) and the bolt gauge distance are 80mm.

Fig. 3: Specimen geometry

Fig. 4: Replica EHB geometry

Since this testing programme is looking specifically at the column face bending, there is a need to isolate its behaviour. This is done through using rigid bolt contact with the flexible face (strong bolt - weak column) eliminating the flexibility of the bolts by using replica blind-bolts that are made from high-strength steel (EN24 steel). These bolts are modified version of Elamin et al. (2015). The use of these bolts prevents the bolt failure mechanism and ensure as pure as possible face bending behaviour while replicating the contact areas. The bolt component in tension has been investigated separately elsewhere (Pitrakkos and Tizani 2015; Tizani and Pitrakkos 2015). The combined behaviour of the bolt in tension and face bending will be assembled from the analytical models of the two components when fully established. The mechanical properties of the replica EHBs are presented in Table 1. The geometry of the replica EHB is identical to the EHB (Fig. 4) and the bolt extension is replaced after each test.

Table 1: Mechanical properties of EHB		
f_y (N/mm ²)	f_u (N/mm ²)	E_s (103×N/mm ²)
755	925/1075	390

Additional fittings were involved to help with the measurement of the EHB displacement inside the specimen. A threaded M16 rod attached to the rounded nut at the end of the EHB. This rod was used as a target to record the bolt displacement during the tests. To protect this rod from being in contact with concrete and eliminate any interaction between them, 20mm steel tube used to cover the threaded rod. This tube is removed after the concrete hardens. The details of the specimen setup are presented in Fig. 5.

Fig. 5: Specimen setup

The concrete was cast at room temperature. The ends of each specimen were covered by polythene bags after casting and the concrete cubes ($100\text{mm} \times 100\text{mm} \times 100\text{mm}$) were fully wrapped with polythene. Both specimens and cubes were stored in the same place until the day of testing. The average of the concrete compressive strength of three cubes on the test day represents the concrete strength in this study.

2.2. Test setup and instrumentations

During the test, a pull-out load is applied to the EHBs with a loading rate of 0.015 mm/sec . A non-contact video gauge system (Imetrum's) was employed to monitor the column face displacement at the bolt hole edge (Δ_f), bolt displacement (Δ_b) and the specimen movement at the bolts level (Δ_s) (Fig. 6). The video gauge system was proven to provide accurate results by comparing its readings with the linear potential meter readings (Pitrakkos and Tizani 2013), and was successfully used in similar studies (Elamin et al. 2015; Mahmood et al. 2014). All displacements are in the direction of the load. The net column face displacement (Δ_{fn}) and the net bolt movement (Δ_{bn}) are calculated using the following equations.

$$\Delta_{fn} = \Delta_f - \Delta_s \quad (1)$$

$$\Delta_{bn} = \Delta_b - \Delta_s \quad (2)$$

Where, Δ_f : column face displacement at the bolt hole edge, Δ_b : bolt movement, Δ_s : specimen movement at the bolts level, Δ_{fn} : net column face displacement

The column face displacement was monitored to evaluate the strength and stiffness of the column face, whereas the bolt slip was recorded to identify the pull-out point (the point at which the EHB starts pulling-out from the hole). The Digital Image Correlation (DIC) system was also used for one of the tests to capture the strain distribution in the column face. The test arrangement is illustrated in Fig. 6.

1: support 2: the sample 3: EHB 4: T3 5: pulling plate 6: load cell 7: actuator

Fig. 6: Test arrangement

2.3. Material properties

Six different concrete mixes were used in this study. The details of each mix are presented in Table 2.. MIX1, MIX2 and MIX3 were used to produce Normal Weight Concrete (NWC) with different concrete strength. MIX4 is a Light Weight Concrete (LWC) mix. MIX5 and MIX6 were used to produce Normal Weight Self Compacting Concrete (NWSCC) and Light Weight Self Compacting Concrete (LWSCC) respectively. The mechanical properties of the SHS were obtained from coupon tensile tests (three coupons were tested for each SHS) done according to EN 10002-1:2001 (BSI 2001). Table 2 summarises the material and geometrical properties for all of the tested specimens. **Error! Not a valid bookmark self-reference.** summarises the material and geometrical properties for all of the tested specimens.

Table 2: Concrete mix details (kg/m³)

	NWC			LWC	NWSCC	LWSCC
	MIX1	MIX2	MIX3	MIX4	MIX5	MIX6
Cement Type	CEM I	CEM II	CEM I	CEM I	CEM II	CEM I
Cement	235	440	642	357	480	424
Water	183	190	205	283	212	272
Fine Aggregate	941	735	625	787	818	750
Coarse Aggregate	941	1020	834	---	896	---
Lightweight Aggregates (Lytag)	---	---	---	561	---	351
Silica Fume	---	---	64	---	---	---
High Range Water Reducer	---	---	9.6	1.4	2.4	2
Ground Granulated Blast-Furnace Slag	---	---	---	79	---	100

Table 3: Material and geometrical properties for the tested specimens

	Specimen ID	Concrete mix	Concrete strength f_{cu} (N/mm ²)	Yield strength f_y (N/mm ²)	Ultimate strength f_u (N/mm ²)	Young's modulus E_s (N/mm ²)	Plate thickness t (mm)	Corner thickness t_c (mm)	Hole diameter D (mm)
Effect of concrete strength SHS 200×200×6.3	C24-1	MIX1	24	413	548	191000	6.35	7.36	26.86
	C24-2	MIX1	24	413	548	191000	6.35	7.36	26.85
	C36-1	MIX2	36	413	548	191000	6.35	7.36	26.94
	C36-2	MIX2	36	413	548	191000	6.35	7.36	26.95
	C90-1	MIX3	90	413	548	191000	6.35	7.35	26.73
	C90-2	MIX3	90	413	548	191000	6.35	7.35	26.75
	C90-3	MIX3	90	413	548	191000	6.35	7.35	26.75
Effect of concrete type SHS 200×200×8	NWC-1	MIX2	41	406	537	207000	7.99	8.72	26.74
	NWC-2	MIX2	41	406	537	207000	7.98	8.71	26.75
	LWC-1	MIX4	40	406	537	207000	7.99	8.72	26.75
	LWC-2	MIX4	40	406	537	207000	7.99	8.73	26.76
	NWSCC-1	MIX5	40	406	537	207000	7.99	8.71	26.74
	NWSCC-2	MIX5	40	406	537	207000	7.99	8.71	26.74
	LWSCC-1	MIX6	39	406	537	207000	7.99	8.71	26.75
	LWSCC-2	MIX6	39	406	537	207000	7.99	8.71	26.75

2.1. Effect of concrete strength

The applied load versus column face displacement curves for the concrete type specimens are shown in Fig. 8. The results show that there is a significant improvement in the load carrying capacity of the column face component with the increase in concrete strength. There is a noticeable improvement in the stiffness, although not as significant as the strength. The behaviour was approximately linear until about 80% of the plastic load (the plastic load is the maximum recorded load before the drop). Then it changes to non-linear until it reaches the plastic load. This was always followed by a drop in strength accompanied with clear concrete crushing sounds until the end of the test. Concrete crushing sounds, during testing, are universally taken as an observational evidence of some forms of concrete failure. These sounds were noted, during the tests, to coincide with the drop

in strength. In the tested specimens, the bolts are anchored in the concrete and, before the concrete crushes in front of the anchorage nut, the bolt pulls a concrete cone towards the column face. Once the anchorage fails, emitting the concrete crushing sounds, the bolts can pull out and bear against the inside of the tube face causing the increase in the displacement. The concrete continues to crush as the anchorage nut moves into it. This is considered as a clear indication of a progressive concrete failure associated with bolt anchorage being pulled out with associated increase in the face displacement as the bolt inner sleeve is now free to bear against the inner side of the column face.

The failure mode of this series of tests, having the strong bolt -weak column face combination, starts with anchorage failure, with associated onset of concrete crushing in front of the anchorage nut, followed by column face bending.

This scenario was observed in all the tested specimens with the load drop being larger with the higher concrete strength. For example, the strength of specimen C90-1 dropped by about 40%, whereas the strength of specimen C24-1 dropped by about 6%. The enhancement in the component plastic resistance with the increase of concrete compressive strength reflects the anchoring contribution in the component resistance. For instance, the component strength was improved by 128% by increasing the concrete compressive strength from 24 N/mm^2 to 90 N/mm^2 . This indicates that the use of low strength concrete limits the bolts anchorage resistance.

After completing each test, the column face plates were cut to examine the concrete. For all the tested specimens, the concrete near the bolts was found completely crushed (Fig. 7-a). The damaged concrete has approximately a cone shape starting from the anchor nut and extending to the concrete surface. In the transverse direction the cone size was limited by the width of the specimen and it is stopped by the column walls. The cone size at the concrete surface was about 1.4 times the bolts anchored length (L_{an}) for all concrete strengths using normal concrete (Fig. 7-b).

Fig. 7: Specimen C36-1 after testing

To understand the reason behind the drop in the strength after reaching the plastic load, two C90 specimens (C90-2 and C90-3) were tested and loaded up to specific stages in the load cycle. The test of C90-2 was stopped immediately after the strength starts dropping (point a - Fig. 8-b), and the test of C90-3 was stopped just before the maximum strength (point b - Fig. 8-b). Then, the column faces for both specimens were cut to examine the concrete around the bolts. C90-3 showed no visible cracks in the concrete surface near the bolts (Fig. 9 -a), whereas in C90-2 many concrete cracks near the bolts were evident (Fig. 9 -b). Based on these observations, it was concluded that up to the plastic load of the column face component, the bolt/concrete interaction is strong enough to share the resistance of the applied load with the column face plate. However, after the plastic strength is reached, the anchored contribution drops due to concrete crushing. This conclusion is in agreement with the FEA results presented in Section 6.

(a)

(b)

Fig. 8: Comparison of applied load versus column face displacement for C24, C36 and C90

Fig. 9: Concrete before and after the maximum strength

The drop in the component resistance beyond the plastic load represents a failure mode (first failure mode) due to the anchored failure. This would be a complete failure for design purposes. The tests, however, were continued further to investigate the full bending behaviour of the column face component. The column face strength shows slight increase the drop due to the membrane action in the SHS plate. This increase continues until the bolts are pulled-out, which represents the second mode of failure (Fig. 10).

The bolt displacement was monitored during the tests to identify the point at which the bolt starts pulling-out from its hole. The pull-out is considered to start beyond the inflection point, when the bolt displacement becomes more than the column face displacement (Fig. 11). The pull-out displacements are summarised in Table 4. It is clear that EHB could be pulled-out at lower displacement with the increase of the concrete strength and this could be due to the high loading level. Fig. 12 shows the effect of concrete strength on the pull-out displacement.

Fig. 10: Pull-out behaviour of EHB

Fig. 11: Identifying pull-out displacement (specimen C24-2)

Table 4: EHB pull-out displacements (effect of concrete strength)

	Specimens				
	C24-1	C24-2	C36-1	C36-2	C90-1
Pull-out displacement (mm)	11.628	11.889	11.177	11.192	9.648

Fig. 12: Comparison of column face displacement and bolt slip curves for C24, C36 and C90

The DIC system was used to monitor the strain distribution in the column face during the testing of C90-1. Due to the limited access to column face during the test, only half of it and its corner were monitored. Fig. 13 shows that the yield begins at the bolt hole and propagates with the increase of load to extend to the corner end at the plastic load (before the concrete anchorage failure). This indicates that the failure mechanism of the column face

component is yielding of the column plate followed by concrete anchorage failure. To identify the yield borders, the strain values on the SHS corner radius are captured including its beginning, centre and end and also at the hole edge (Fig. 14). Fig. 15 shows that there is a complete yielding of the corner at the plastic resistance of the component. Yield-line theory is considered as a simple and rapid converging method to determine upper bounds of the capacity of plates and slabs. In yield line analyses, the yield line would have been assumed to be located at the support centreline, i.e. the SHS corner (Qin et al. 2014). However, the observation from the tests showed that the yield line has actually extended to cover the full corner region.

Fig. 13: Strain distribution of the SHS at different loading stages (DIC results for specimen C90-1)

Fig. 14: Strain measurement positions

Fig. 15: Strain distribution at different locations (DIC results for specimen C90-1)

2.2. Effect of concrete type

The response of the column face component to changing the concrete type is presented in Fig. 16. The concrete strength on the day of testing was 41N/mm^2 for NWC, 40N/mm^2 for LWC, 39N/mm^2 for NWSCC and 40N/mm^2 for LWSCC. The general trend of all the specimens seems not to be affected by the concrete type. All specimens showed clear plastic point followed by drop in the strength and then there is slight climb until the test end. However, the drop after the plastic load was softer for the normal weight concrete group (NWC and NWSCC). NWSCC show quite comparable behaviour to NWC. Both specimens have approximately the same initial stiffness. The plastic load of NWC was slightly higher than that of NWSCC, but this difference could be attributed to the variation in the concrete strength on the day of testing, as it was 41N/mm^2 for NWC and 39N/mm^2 for NWSCC. The plastic load and the initial stiffness of lightweight concrete group (LWC and LWSCC) registered marked

reduction. Therefore, it is concluded that the use of self-compacting concrete or the normal weight concrete provides the same strength and stiffness for the component. However, the use of the lightweight concrete reduces the strength and stiffness of the component up to 21% and 61% respectively compared with the normal weight concrete. The bolts pull-out displacement is presented in Table 5. It is found that there is no significant change in the pull-out displacement with the change of concrete type.

In normal weight concrete, the aggregate strength is higher than that of the mortar so the fracture path travels around the aggregate. However, in light weight concrete the aggregate is weaker than the mortar and the fracture path travels through the aggregate (Bogas and Gomes 2013; FIP 1983). This means the fracture tends to extend for longer distance and consume higher energy in the case of normal weight concrete. These facts agree with the close examination of the crushed concrete around the EHBs. The size of the crushed concrete was reduced significantly with the use of lightweight concrete. The cone size reduced from $1.4L_{an}$ to $1.0L_{an}$ (Fig. 17). It is concluded that the use of self-compacting concrete or normal weight concrete provides the same strength and stiffness for the component. However, compared with the normal weight concrete, the use of lightweight concrete reduces the strength and stiffness of the component up to 21% and 61% respectively. This drop is due to the reduction in the contribution of the anchorage to the component resistance due to the reduction in the concrete cracked area. Fig. 17 shows a comparison between the size of the crushed concrete for normal, self-compacting and lightweight concrete.

Table 5: EHB pull-out displacements (effect of concrete type)

	Specimens							
	NWC-1	NWC-2	LWC-1	LWC-2	NWSCC -1	NWSCC -2	LWSCC- 1	LWSCC- 2
Pull-out displacement (mm)	11.06	11.03	11.62	11.85	11.02	10.87	11.72	11.67

(a)

(b)

Fig. 16: Comparison of applied load versus column face displacement for NWC, NWSCC, LWC and LWSCC

Fig. 17: Effect of concrete type on the size of crushed concrete

The failure mode for all specimens was a bolt anchorage failure followed by a complete yielding of the column

face plate around the bolts. Therefore, the strength of the column face component is equal to the sum of the SHS plate yielding strength and the anchorage resistance. The yield line theory was used to estimate the strength of the column flange, face and web when they act as connection faces (Cao et al. 1998; Kapp 1974; Wang et al. 2010). It is also adopted by the Eurocode in the design of steel joints (CEN 2005). Therefore, it can be implemented to propose analytical formulas to estimate the column face yield strength. The experimental evidence and the finite element results presented in section 5.4 show that the anchorage fails by pulling-out a concrete cone (Fig. 18). This kind of failure is due to the concrete tensile cracking across the failure surface (Eligehausen and Ozbolt 1990; Werner Fuchs and John 1995). Thus, the anchored pull-out strength depends on the cracked area and the concrete tensile strength.

Fig. 18: Concrete failure

3. Finite element modelling

A Finite Element (FE) model using ABAQUS 6.13-2 (Dassault 2013) was developed and validated using the experimental data presented in the previous section and experimental data from literature (Mahmood et al. 2014). In order to accurately simulate the actual behaviour of the investigated specimens, the material and geometrical properties for the entire model's components and the boundary conditions were similar to those of the tested specimens.

The advantage of symmetry along the longitudinal and transverse axis of all the investigated specimens was considered so that only quarter of each specimen is modelled to reduce computation costs. The element size of the bolt and the region around it in both concrete and SHS was 6mm and for the remaining parts of the model it was 12mm. The details of the whole specimen and the FE model are shown in Fig. 19.

Fig. 19: Model geometry

4.1. FE modelling of the square hollow section

The exact dimensions of the square hollow sections and holes size were used in the FE models. The details of all these dimensions are summarised in Table 3 and in Mahmood et al. (2014). The modelling of the corners of SHS needs special considerations because the plate thickness at the corners (t_c) is slightly larger than the nominal thickness of hollow section wall. The BSI specification for hot-finished hollow sections (BSI 1991) stated the minimum and maximum values for the radius of interior curve (R_i) and the exterior curve (R_e) for the corners. The values of R_i and R_e should not be less than half of nominal wall thickness (t), and should not be more than 1.5 and 2 of the nominal wall thickness for R_i and R_e respectively. In this study, it is found that using R_i equal to t and R_e equal to $1.65t$ can be used to model the hollow section corners with a maximum difference from the measured values equal to $0.42mm$.

ABAQUS's first order interpolation element (C3D8) with full integration was used to model the SHS. The name of each element in ABAQUS identifies its main aspects. Here C means continuum element, 3D means three dimensional element and 8 denotes the number of the nodes required to create the element (Abaqus 2013). The C3D8 element has three degrees of freedom at each node: translations in the x , y and z directions. The displacements at the eight nodes of the element are calculated, and then linear interpolation are used for other points. This element is suitable for modelling the complex nonlinear behaviour involving contact and geometrical nonlinearities (Abaqus 2013). The elastic-plastic material model was used to simulate the behaviour of the SHS. The stress/strain relationship was obtained from tensile coupon tests. The stress and strain that were obtained

from these tests are engineering values. They were converted to true values before being used in the FE model.

The linear elastic part of the stress-strain curve can be represented in the FE model by the Young's modulus (E_s) and the Poisson's ratio (ν_s). In this study, the Young's modulus for SHS was taken from the experimental test data, whereas Poisson's ratio was considered equal to 0.3. In the plastic model, the shift from elastic to plastic behaviour occurs at the yield point. The plastic behaviour for the metals can be simulated in ABAQUS by defining the true plastic strain and stress from the yield point to the ultimate point. The true plastic strain can be calculated by subtracting the yield strain from the total strain. Fig. 20 shows the decomposition of the strain into elastic and plastic strain.

Fig. 20: Decomposition of the total strain into elastic and plastic components (Abaqus 2013)

4.2. FE modelling of concrete

The C3D8 ABAQUS element (Abaqus 2013) was also used to simulate the concrete. The shape was modelled using the internal geometry of the SHS minus the embedded part of the EHB. The elastic behaviour of the concrete was simulated using the elastic model available in ABAQUS (Dassault 2013) by defining its Young's modulus and Poisson's ratio. The concrete Young's modulus (E_{cm}) is calculated using equation (3) (CEN 2004). The Poisson's ratio was considered to be 0.2 for all the concrete strengths (Logan et al. 2009; Mertol et al. 2008).

$$E_{cm} = 22000 \left(\frac{f_{cm}}{10} \right)^{0.3} \quad (3)$$

where

E_{cm} : concrete Young's modulus (N/mm^2)

f_{cm} : mean value of concrete cylinder compressive strength (N/mm^2)

$$f_{cm} = f_{ck} + 8 \quad (4)$$

where

f_{ck} : concrete characteristic compressive cylinder strength (N/mm^2)

ABAQUS provides many models for simulating the plastic behaviour of concrete such as Concrete Damage Plasticity (CDP), Drucker Prager and concrete smeared cracking. The CDP model was chosen in this study to simulate plastic behaviour of the concrete, because it has the capability to model the concrete or any brittle material. The failure mechanisms in this model are tensile cracking and the compression crushing. Furthermore, it has the capability for representing the complete inelastic behaviour of concrete in tension and compression by including damage parameters, which represent degradation of stiffness with plastic deformations. In addition, it can be applied in both ABAQUS/Standard and ABAQUS/Explicit.

The CDP model requires using appropriate parameters for defining the plasticity, compressive and tensile behaviours of concrete. The plasticity parameters are the dilation angle (φ), flow potential eccentricity (ϵ), ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress (σ_{bo}/σ_{co}), yield shape parameter (k_c) and the viscosity parameter (μ_o).

The dilation angle defines the inelastic volumetric change in granular materials due to cracking and slippage along cracked surfaces. For confined concrete a value of 55° could be used to simulate the concrete behaviour (Mercan et al. 2010). The value of yield shape parameter (k_c) should be in the range of 0.5 to 1 (Abaqus 2013). To investigate the effect of k_c on the behaviour of concrete specifically and the whole model generally, three FE analyses were performed with three different values of k_c (0.5, 0.75 and 1). It was found that for all concrete strengths between C24 and C90, k_c has a significant impact on the ultimate load carrying capacity before concrete failure. For k_c equal to 1, there is a descending trend in the load-displacement curve of the column face, which is similar to the experimental behaviour. Therefore, in this study k_c was considered to be equal to 1 for all concrete strengths. Fig. 21 shows the effect of the yield shape parameter on the column face bending behaviour for

specimen C36-1.

Fig. 21: Effect of the yield shape parameter (k_c) (specimen C36-1)

The default values of the remaining plasticity parameters that have been specified in the ABAQUS manual (Abaqus 2013) were used. These values are 0.1 for the flow potential eccentricity, 1.16 for the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress and 0 for the viscosity parameter.

There are many analytical models available in the literature for defining the stress-strain relation for concrete in tension and compression (CEN 2005; Hsu and Hsu 1994; MacGregor 1997; Nayal and Rasheed 2006). Some of these models are limited to a specific range of concrete strengths. The concrete strength in this study is between 24N/mm^2 and 90N/mm^2 . According to Eurocode 2 (CEN 2005), multi-linear model for stress-strain relation of concrete in compression is valid up to concrete strength of 105N/mm^2 . Therefore, it was chosen to simulate the concrete compressive behaviour. The details of the concrete compressive stress-strain curve are illustrated in

Fig. 22. The model assumes linear elastic behaviour up to 40% of the ultimate concrete compressive strength.

This is followed by a nonlinear ascending curve until reaching the ultimate concrete strength (f_{cm}) at ϵ_{c1} , which is followed by reduction in the concrete resistance until a strain equal to ϵ_{c2} .

Fig. 22: Concrete Compressive stress- strain curve used in this study (CEN 2004)

The stress-strain values are calculated using the following equations:

$$f_c = \left(\frac{k\eta - \eta^2}{1 + (k-2)\eta} \right) f_{cm} \quad (5)$$

$$k = \frac{1.05 E_{cm} |\epsilon_{c1}|}{f_{cm}} \quad (6)$$

$$\epsilon_{c1} = \frac{0.7(f_{cm})^{0.31}}{1000} < 0.0028 \quad (7)$$

$$\epsilon_{c2} = 0.0035 \quad \text{for} \quad f_{ck} < 50 \text{ N/mm}^2 \quad (8)$$

$$\epsilon_{c2} = 0.0028 + 0.027 \left[\frac{(98 - f_{cm})}{100} \right]^4 \quad \text{for} \quad f_{ck} \geq 50 \text{ N/mm}^2 \quad (9)$$

$$\eta = \frac{\epsilon_c}{\epsilon_{c1}} \quad (10)$$

where

f_c : concrete compressive stress at any point on the stress-strain curve (N/mm^2)

ϵ_{c1} : concrete compressive strain at the maximum stress (f_{cm})

ϵ_{c2} : Ultimate compressive strain in the concrete

ϵ_c : compressive strain in the concrete at any point along the curve

A bilinear model was used to simulate the tensile behaviour of concrete. It is linear until the ultimate tensile

strength (f_{ctm}) which is calculated using equation (11) or (12)(CEN 2004). Then, the strain softening starts so that the stress reduces linearly to zero at a total strain ϵ_{tu} of about ten times ϵ_{to} (Abaqus 2013; Carstensen 2011). However, ABAQUS (Abaqus 2013) limits the minimum value of the post-failure stress to be not less than 1% of the ultimate tensile stress to avoid potential numerical problems. A schematic plot of the stress-strain relation of

Fig. 23: Stress-strain relation of pure concrete in tension

$$f_{ctm} = 0.3 (f_{ck})^{2/3} \quad \text{for } f_{ck} \leq 50 \text{ N/mm}^2 \quad (11)$$

$$f_{ctm} = 2.12 \ln(1 + 0.1(f_{cm})) \quad \text{for } f_{ck} > 50 \text{ N/mm}^2 \quad (12)$$

Fig. 23: Stress-strain relation of pure concrete in tension

4.3. FE modelling of EHB

The EHB was modelled using the exact dimensions of the bolt used in the experimental tests. The yield strength of the EHB as presented in Table 1 is 755 N/mm^2 . The cross-sectional area of the EHB is 520 mm^2 . This makes the elastic capacity of the EHB equal to 393 kN . During the experimental tests, the ultimate load that was applied on each bolt was always less than 393 kN . Therefore, in this study the EHB was simulated as a linear elastic material using the elastic model only. The Young modulus was taken equal to $390,000 \text{ N/mm}^2$ and Poisson's ratio to 0.3 . Similar to modelling the hollow section, the C3D8 was used for the EHB.

4.4. FE modelling of contact

Contact between different parts is involved in many engineering problems. In normal contact conditions, the contacted bodies start sharing the load resistance once they touch each other. In sliding contact conditions, shear force may be created to resist the tangential movement and a suitable friction modulus should be defined. The benefits of contact simulation in FE analysis are to prevent the contacted bodies from merging or penetrating into each other and also calculate the contact bearing stresses. The contact in FE modelling is a kind of discontinuous constraint, which could transmit forces from one part to another during specific conditions. The discontinuity means that it starts transferring forces only when the parts are in contact and when they are separated there is no constraint applied.

In ABAQUS/Standard solver, contact can be simulated either by surface based or contact element based. In

this study, surface based contact was selected to define the contact between the different parts of the model. For this approach, two algorithms are available: general contact algorithm and contact pair algorithm. In general contact algorithm, ABAQUS automatically generates the constraint between the attached bodies and the user needs only to define the contact properties. However, in this study it was found that, this algorithm causes many numerical problems. Therefore, the contact pair algorithm is adopted in this study in which the user needs to define the contact properties and link the related surface manually.

Both of the normal and tangential contacts are used in the model. Only contacted surfaces needs to be defined if normal contacts are used, but in tangential contact the friction modulus should be specified. The coefficient of friction between steel and concrete was considered as 0.25 (Ellobody et al. 2006; Elremaily and Azizinamini 2001; Hu et al. 2003) and between steel and steel was considered as 0.2 (Chen et al. 2018).

The contact surfaces in the model are divided into three groups: contact between the concrete and the interior faces of the hollow section, contact between the EHB (sleeves, extension and the nut) with the concrete surrounding it, and contact between EHB sleeves and the hole in the hollow section.

4. Model validation

5.1. General Behaviour of the Column Face

The general pattern of the applied load versus the column face displacement curves from both experimental data and FE results are compared in

* (Mahmood et al. 2014) μ is the column face slenderness ratio (ratio of the width to the thickness of the column face plate)

Fig. 24: Comparison of Applied load and column face displacement (experiments versus FE analysis)

. The general patterns of the force displacement curves present close correlation between the experimental data and the FE results. Similar to the experimental data, FE results demonstrate clear identification of the plastic load before the drop in strength. The sharpness of the drop in the strength beyond the plastic load was marginally

lower in the FE results. This indicates that the FE model results in a slightly higher anchorage resistance beyond the plastic load. A possible reason for this slight difference could be related to the limited damage in the concrete model.

* (Mahmood et al. 2014) μ is the column face slenderness ratio (ratio of the width to the thickness of the column face plate)

Fig. 24: Comparison of Applied load and column face displacement (experiments versus FE analysis)

5.2. Plastic load and initial stiffness

Table 6 presents comparisons between the FE results and the experimental data. The comparisons include the plastic load and the initial stiffness. The initial stiffness for the experimental and FE results was calculated at

the end of the linear behaviour of the component (80% of the plastic load). The comparisons show that there is very good agreement between the experimental data and the FE results with a maximum difference of 2% in the plastic load and 15% in the initial stiffness.

Table 6: Experimental data and FE results (plastic load and initial stiffness)

Specimen	Plastic load (kN)			Initial stiffness (kN/mm)		
	Experimental	FE	Difference %	Experimental	FE	Difference %
C24-1	149.892	150.287	0	184	192	4
C24-2	148.015	150.287	2	188	192	2
C36-1	192.591	190.431	-1	243	259	7
C36-2	193.405	190.431	-2	236	259	10
C90-1	342.979	347.690	1	251	286	14
C90-2	348.649	347.690	0	249	286	15
$\mu 40-1^*$	147.972	151.234	2	181	189	4
$\mu 40-2^*$	147.635	151.234	2	179	189	6
$\mu 31.75-1^*$	192.592	190.431	-1	243	259	7
$\mu 31.75-2^*$	193.405	190.431	-2	236	259	10
$\mu 25-1^*$	248.814	248.814	0	241	264	10
$\mu 25-2^*$	248.365	248.814	0	286	264	-8

*(Mahmood et al. 2014), μ is the column face slenderness ratio (ratio of the width to the thickness of the column face plate)

5.3. Strain Distribution

The strain distribution in the SHS is also considered as a validation parameter. The experimental and the numerical results for the strain values at the bolts hole are presented in Fig. 25. It is clear that the FE results are in good agreement with the experimental data.

Fig. 25: Load strain curves: experimental versus FE at bolt hole (specimen C90-1)

5.4. Failure Mode

Fig. 26 presents a comparison between the damaged concrete around the anchored bolts at the end of the tests. The FE results presents similar failure mode to what was found in the experimental tests.

specimen C24-1

specimen C36-1

specimen C90-1

Fig. 26: Anchored failure: experimental versus FE

5. FE Analysis

The effect of the concrete strength on the bending behaviour of the column face component was numerically investigated using 42 FE models. The models are divided into three groups based on the column plate slenderness ratio (μ): 40, 31.75 and 25. For each group the concrete strength was varied between 25N/mm^2 to 90N/mm^2 with an increment of 5N/mm^2 .

5.1. Component plastic resistance

Fig. 27 shows that for $\mu 25$ and $\mu 31.75$ models there is a consistent rate of improvement in the plastic resistance of the column face component with the increase of the concrete strength through all the varying range. For $\mu 40$ models, the improvement in the component resistance was less with the use of high strength concrete. This finding suggests that it is always possible to improve the component resistance by increasing the concrete compressive strength, although the amount of the improvement might be less for SHS of high slenderness ratio. The idea of the development of the EHB is mainly based on improving the connection behaviour by involving the anchoring action. Increasing the compressive strength of concrete results in improving the anchoring resistance. Therefore, there was always enhancement in the connection strength with the increase of concrete compressive strength.

Fig. 27: Effect of concrete compressive strength on the component plastic resistance

5.2. Component initial stiffness

Fig. 28 shows the effect of concrete compressive strength on the component initial stiffness. It is clear that, the rate of improvement in the component initial stiffness decreases with the increase of the concrete strength.

For instance, for $\mu_{31.75}$ models, increasing the concrete strength from 25N/mm^2 to 30N/mm^2 enhanced the column face stiffness by 17%, whereas, increasing the concrete strength from 85N/mm^2 to 90N/mm^2 improved the column face stiffness by 2%. In addition, the results indicate that for each slenderness ratio there is a specific concrete strength after which the improvement in the component initial stiffness becomes minor. These concrete strengths are defined as the stiffness improvement limits and they are: 35N/mm^2 for μ_{40} , 50N/mm^2 for $\mu_{31.75}$ and 60N/mm^2 for μ_{25} . This performance is related to the limited rigidity of the SHS with use of high slenderness ratio and also to the lower improvement in the concrete modulus of elasticity in the high strength concrete zone.

Fig. 28: Effect of concrete compressive strength on the component initial stiffness

5.3. Component failure mechanism

To provide additional insight for the component behaviour, a hypothetical FE model with a very stiff column plate (40mm thickness) was analysed using concrete of 90N/mm^2 compressive strength. Fig. 29 shows the effect of using very stiff column plate on the development of the concrete strain. The concrete strain measured in front of the anchored rounded nut as this is the location of the highest recorded strain. It is clear that the use of very stiff column plate prevents the concrete failure, which means that the component strength depends on the bending resistance of the steel plate and the anchorage resistance of the bolts. This confirms that the failure mode of the component is bolts anchorage failure followed by plastification of the SHS plate.

Fig. 29: Concrete strain for very stiff and flexible column plate

5.4. SHS confinement

The ultimate strength of the concrete infill is influenced by the confining pressure of the SHS and the previous results showed that the concrete strength has clear impact on the component behaviour. Therefore, SHS confinement effect on the concrete is investigated in order to evaluate its influence on the component behaviour. The concrete stress at the component plastic load for different SHS slenderness ratios (μ) and yield strengths (f_y) were monitored using the FE model. The results are presented in Table 7. The results show that, for the same slenderness ratio, the confinement effect increased with the increase of SHS yield strength. These results are likely to be related to the improvement in the bending resistance of the column plate with the increase of its yielding strength. This means, the confinement effects is not only influenced by the slenderness ratio but it is also affected by the yield strength of the SHS. The proposed consideration here is that the confinement effects can be neglected only if the confinement index (f_y/μ) is less than 10. This is expressed in the confinement coefficient (γ) shown in Equation (13).

$$\gamma = \frac{f_y}{10\mu} \geq 1 \quad (13)$$

Where μ : column face slenderness ratio = b/t , b : width of the SHS, t : thickness of the SHS plate, f_y : yield strength of SHS plate.

Table 7: SHS confinement effect on concrete

μ	f_y (N/mm ²)	f_y/μ	f_{cu} (N/mm ²)	Concrete stress ($f_{cu FE}$) (N/mm ²)	Confinement parameters		
					$f_{cu FE}/f_{cu}$	γ equation (13)	Prediction
25	450	18.00	50	85	1.70	1.80	Consider confinement
25	400	16.00	50	75	1.51	1.60	Consider confinement
25	350	14.00	50	66	1.31	1.40	Consider confinement
25	300	12.00	50	57	1.14	1.20	Consider confinement
25	250	10.00	50	53	1.06	1.00	Consider confinement
25	200	8.00	50	50	1.00	0.80	Neglect confinement
25	450	18.00	40	69	1.71	1.80	Consider confinement
25	400	16.00	40	61	1.52	1.60	Consider confinement
25	350	14.00	40	53	1.33	1.40	Consider confinement
25	300	12.00	40	46	1.15	1.20	Consider confinement
25	250	10.00	40	41	1.03	1.00	Consider confinement
25	200	8.00	40	40	1.00	0.80	Neglect confinement
25	450	18.00	30	52	1.73	1.80	Consider confinement
25	400	16.00	30	46	1.54	1.60	Consider confinement
25	350	14.00	30	41	1.35	1.40	Consider confinement
25	300	12.00	30	35	1.16	1.20	Consider confinement
25	250	10.00	30	31	1.03	1.00	Consider confinement
25	200	8.00	30	30	1.00	0.80	Neglect confinement
30	450	15.00	50	72	1.44	1.50	Consider confinement
30	400	13.33	50	64	1.28	1.33	Consider confinement
30	350	11.67	50	56	1.11	1.17	Consider confinement
30	300	10.00	50	51	1.02	1.00	Consider confinement
30	250	8.33	50	50	1.00	0.83	Neglect confinement
30	200	6.67	50	50	1.00	0.67	Neglect confinement
30	450	15.00	40	58	1.44	1.50	Consider confinement
30	400	13.33	40	51	1.28	1.33	Consider confinement
30	350	11.67	40	44	1.11	1.17	Consider confinement
30	300	10.00	40	41	1.03	1.00	Consider confinement
30	250	8.33	40	40	1.00	0.83	Neglect confinement
30	200	6.67	40	40	1.00	0.67	Neglect confinement
30	450	15.00	30	43	1.44	1.50	Consider confinement
30	400	13.33	30	38	1.28	1.33	Consider confinement
30	350	11.67	30	33	1.11	1.17	Consider confinement
30	300	10.00	30	29	0.95	1.00	Consider confinement
30	250	8.33	30	24	0.79	0.83	Neglect confinement
30	200	6.67	30	19	0.62	0.67	Neglect confinement
40	450	11.25	50	55	1.11	1.13	Consider confinement
40	400	10.00	50	52	1.04	1.00	Consider confinement
40	350	8.75	50	50	1.00	0.88	Neglect confinement
40	300	7.50	50	50	1.00	0.75	Neglect confinement
40	250	6.25	50	50	1.00	0.63	Neglect confinement
40	200	5.00	50	50	1.00	0.50	Neglect confinement
40	450	11.25	40	44	1.11	1.13	Consider confinement
40	400	10.00	40	42	1.05	1.00	Consider confinement
40	350	8.75	40	40	1.00	0.88	Neglect confinement
40	300	7.50	40	40	1.00	0.75	Neglect confinement
40	250	6.25	40	40	1.00	0.63	Neglect confinement
40	200	5.00	40	40	1.00	0.50	Neglect confinement
40	450	11.25	30	33	1.11	1.13	Consider confinement
40	400	10.00	30	31	1.03	1.00	Consider confinement
40	350	8.75	30	30	1.00	0.88	Neglect confinement
40	300	7.50	30	30	1.00	0.75	Neglect confinement
40	250	6.25	30	30	1.00	0.63	Neglect confinement
40	200	5.00	30	30	1.00	0.50	Neglect confinement

6. Conclusions

Through this paper, both experimental and numerical techniques were used to evaluate the effect of concrete infill and slenderness ratio on the column face bending behaviour of anchored blind bolted connections. It provides valuable data, which is useful to understand the behaviour of anchored blind bolted connections. Within the range of the validity of this study, the following is concluded:

- The concrete acts compositely with the anchored bolts until it fails by forming a cone shape starting from the anchored nut and extending to the concrete surface. The size of the concrete cone is found not to be affected by the concrete infill compressive strength. The cone size at the concrete surface is equal to 1.4 and 1.0 times the anchorage length for normal and light weight concrete respectively. However, further experimental data is required to validate these relations using different anchored length.
- The failure mechanism of the component is always concrete crushing (anchorage failure) followed by yielding of the SHS plate and then bolt pull-out. The yield of the SHS plate is found to start at the bolt hole edge. At the plastic load, the yield propagates to cover the whole corners of the tubular section contrary to the common theoretical assumption which restrict yielding to the corner's centreline.
- The experimental and numerical pull-out load versus column face displacement reveals that the behaviour of the component is approximately linear until about 80% of its plastic strength. It turns to nonlinear and reaches the component plastic resistance which follows by a drop. The sharpness of the drop is found to be higher with the increase of the concrete compressive strength, which results in higher contribution of the anchoring in developing the component resistance.
- The concrete strength is found to have direct influence on the component strength and stiffness. The increase in concrete strength is associated with significant increase in the component strength throughout the whole range of the concrete compressive strength (25N/mm^2 to 90N/mm^2). However, the increase in the component stiffness is less significant and is found to be limited by the correlation between the column face slenderness ratio and the concrete strength. The correlation means there should be a balance between the concrete strength and column face slenderness ratio to obtain the optimum behaviour from the two materials. Therefore, slenderness ratio limits were defined beyond which the increase in concrete strength will have negligible effect on the component stiffness (for $\mu 40$ maximum f_{cu} is 35N/mm^2 , for $\mu 31.75$ maximum f_{cu} is 50N/mm^2 and for $\mu 25$ maximum f_{cu} is 60N/mm^2).

- The confinement effect on the concrete infill depends on the slenderness ratio (μ) and the yield strength (f_y) of the SHS. Therefore, it is recommended to neglect the confinement effect only when the confinement index (f_y/μ) is less than 10.
- The strength and stiffness of the component are not affected when using self-compacting concrete while the use of light weight concrete reduces both due to the limited anchoring capacity of lightweight concrete. This is because the fracture path travels through the light-weight aggregate rather than around them as in normal aggregate. The use of lightweight concrete reduces the component strength and initial stiffness by 21% and 61% respectively.

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