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ARTICLE

The behaviour of anchored extended blind bolts in concrete-filled tubes

Extended Hollo-Bolts (EHBs) are blind bolts that have an extended bolt shank ending in an anchor nut. When used with concrete-filled tubes, the extension and the anchor in the concrete serve to enhance significantly the performance of both connection components: bolts in tension and tube face in bending. The enhancements are a result of confining the concrete, preventing local buckling of the steel tube and allowing the blind bolt to achieve a tensile strength equal to that of standard bolt+nut fasteners. Overall, the use of the EHB results in a moment-resisting bolted connection to hollow sections which can achieve rigid behaviour under certain configurations. This paper summarizes research work done to date on such connections at the University of Nottingham. This includes experimental, numerical and analytical modelling. The aim of the work is to provide a fundamental understanding of the behaviour of anchored blind bolt connections to concrete-filled columns, leading to the specification of appropriate design rules that allow the use of such bolted moment-resisting connections in practice. The work has proposed analytical models for: connection stiffness, column face-bending strength considering both single and group behaviour of bolt rows, anchored bolts in tension and anchored bolts under combined tension and shear.

Keywords composite connections; concrete-filled steel tubes; blind bolt; Extended Hollo-Bolt

1 Introduction

Structures with open-section beams and concrete-filled tubular columns have been increasingly utilized in construction over the past few decades due to their enhanced structural performance as well as their architectural advantages. Tubular steel columns have a superior axial load-carrying capacity, higher strength-to-weight ratio and an excellent torsional resistance compared with open steel sections. However, the use of this configuration in moment-resisting construction is limited due to the difficulties in connecting the structural members as there is no access to the inner part of the tube for installing standard bolts.

Blind-bolted systems only require access to one side of the tubular section to tighten the bolt, thus overcoming fastening limitations. Connections using blind fasteners in hollow steel columns provide sufficient tying and shear resistance to satisfy structural integrity checks. However, such connections tend to have a low moment-rotation stiffness, which is usually controlled by the inherent flexibility of the column face. Therefore, the use of blind bolts in concrete-filled steel tubes represents a practical alternative for producing moment-resisting connections in tubular construction.

The Extended Hollo-Bolt (EHB) is an anchored blind fastener that was developed as a modification of the commercially available Lindapter Hollo-Bolt (HB) [1], which uses the infill concrete to create an anchoring mechanism. The tightened shape of the EHB differs from its untightened shape as the sleeve expands while the cone is pushed up during tightening (Fig. 1). Experimental studies have shown these connections to have promising prospects in practical engineering and to be an effective solution for steel structures with tubular columns.

This type of fastener is the subject of an ongoing research programme at the University of Nottingham, which is investigating its use in moment-resisting connections. This paper presents a summary of the ongoing research work done to date.

2 Background

2.1 Blind bolts

The benefits of using blind-bolted systems in steel and composite beam-to-column connections have been highlighted by many researchers. For instance, enhanced joint ductility, strength and stiffness have been reported by Loh et al. [2, 3], Liu et al. [4] and Ataei et al. [5]. In addition, satisfactory behaviour in terms of yielding, maximum strength capacity and ultimate displacement under seismic events has been highlighted by Li et al. [6], Wang et al. [7] and Waqas et al. [8].

Design guidance for simple joints using the Lindapter Hollo-bolt (HB) fastener is currently available in EC 3 [9]. Multiple authors [10–12] have investigated the HB in concrete-filled tubes, concluding that these connections do not provide the required moment resistance and rotational stiffness to be classified as moment-resisting.

Multiple modifications to commercial blind bolts have been proposed by the scientific community to expand the

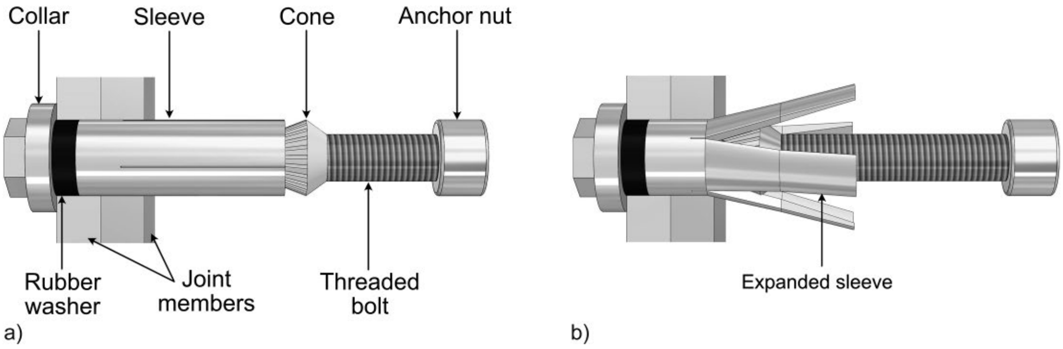


Fig. 1 Extended Hollo-Bolt (EHB) parts: a) shape before tightening, b) shape after tightening

use of blind bolts to moment-resisting connections. For instance, connections using the Double-Headed Anchored Blind Bolt (DHABB), introduced by Oktavianus et al. [13] as a modification of the Ajax-Oneside fastener [14], exhibit higher secant stiffness and lower cyclic deterioration than connections made with the original blind fastener [15]. Another example is the Thread-Fixed One-side Bolt (TFOB), which uses a similar concept to the Flowdrill system [16] that has been shown to be a good alternative to the traditional bolt and nut in engineering applications [17]. Finally, the blind fastener of interest in this article is the Extended Hollo-Bolt (EHB), modified from the HB by Tizani and Ridley-Ellis [18], which shows promising potential for use in moment-resisting connections [19–22].

The component method for different types of steel connection is detailed in EC 3 [9] and represents a simplified approach for analysing complex connections and facilitating the joint design procedure. This method breaks down the connection into a series of individual rigid and flexible components that contribute to the overall structural properties.

Models currently available for calculating the strength and stiffness of blind-bolted connections cannot be extended to the EHB due to significant differences in bolt geometry and complex contact interactions between the bolt components and the infill concrete.

The joint components that contribute to the resistance and/or rotational stiffness of the EHB joint are presented in Tab. 1 together with the availability of evaluation rules in EC 3 [9]. The identification of these components is based on the following assumptions:

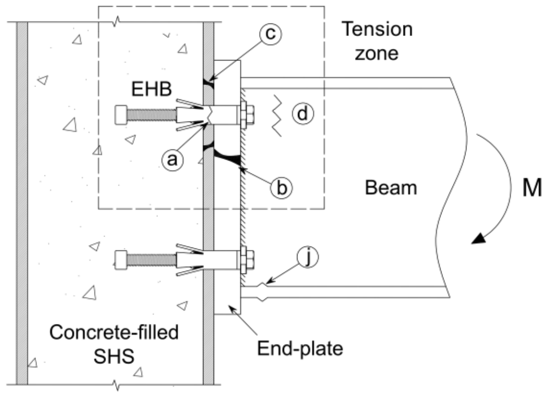
- The beam flange carries all compression and therefore the beam web in compression is not considered.
- Owing to the stiffening action of the infill concrete, the following components do not need to be considered: column face compression, side column faces compression/tension and punching shear failure around the bolt heads in compression.
- The weld components do not contribute to the rotational stiffness of the joint. However, their resistance must be checked against the existing rules available in EC 3.

2.2 EHB failure modes

Three main failure modes have been identified for the tension zone of the EHB connection: bolt failure in tension, column face failure in bending and combined failure mode (simultaneous failure of bolts in tension and column face in bending). These failure modes have also been reported for other blind bolts. For instance, the TSOB exhibits column failure for the strong column-weak bolt

Tab. 1 EHB joint components and availability of evaluation rules

Ref	Component	EC 3 coverage	Rot. stiffness contribution
Tension			
a	Bolts in tension	×	✓
b	End plate bending	✓	✓
c	Column face bending	×	✓
d	Beam web tension	✓	×
Compression			
j	Beam flange compression	✓	×



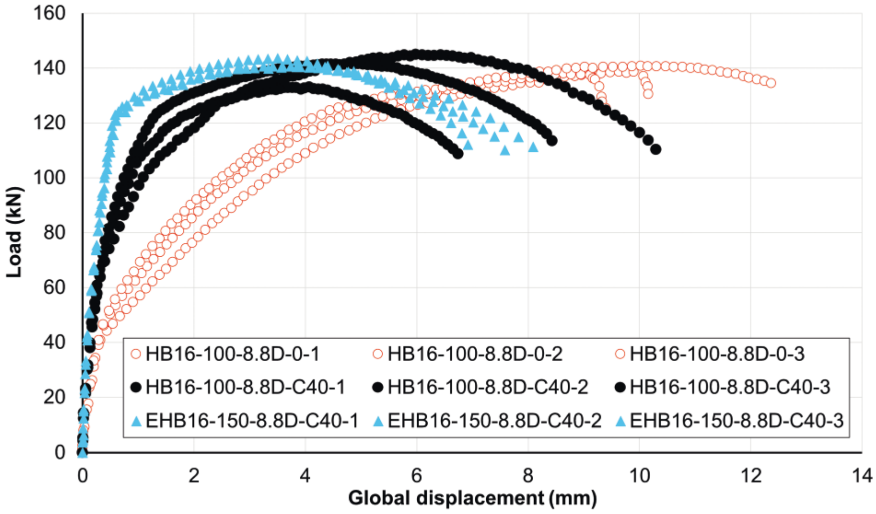


Fig. 2 Pull-out behaviour of EHB in comparison with type HB with and without infill concrete [25] (specimen key: bolt type–bolt diameter–shank length–bolt grade and batch concrete grade–sample number)

case and bolt fracture for weak column-strong bolt [23]. Similarly, the failure modes for the anchored Ajax-Oneside fastener are bolt fracture and tube wall yield accompanied by bolt pull-out, depending on the bolt location and geometrical configuration of the connection [24].

EHB bolts in tension and column face bending have been studied independently by isolating the component of interest, and these modes are described below.

3 Research work

3.1 EHB in tension

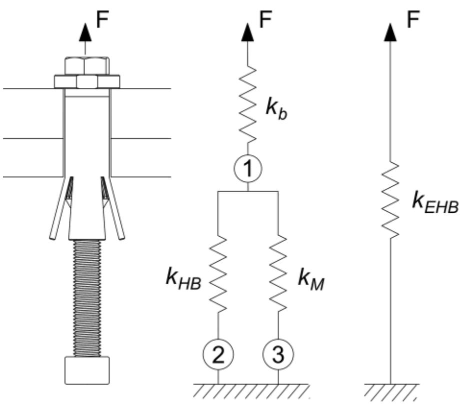
Pitrakkos [25] developed an experimental setup with a rigid column face arrangement to isolate bolt behaviour. The testing configurations included single-sided and double-sided T-stub models under different types of load. Monotonic tensile pull-out tests were performed on 16 single-sided and four double-sided T-stubs, and an analytical model was developed for this component.

The authors [25] compared the EHB global behaviour with the HB counterparts with and without infill concrete, as shown in Fig. 2. It was observed that the anchored nut distributes the applied force over the surrounding concrete and therefore stress concentrations in the expanding sleeves decrease, thus limiting their failure and eliminating concrete breakout.

The study concluded that the EHB component is similar to standard bolts as the failure mode corresponds to bolt shank necking and fracture, showing that it is able to develop the full tensile capacity of its internal bolt. In addition to the benefits of using concrete infill plus high concrete strength and bolt grade, it was found that the bolt group action does not compromise the strength of the system as the total connection strength is equal to the sum of the individual bolts.

Using the experimental results reported above, Pitrakkos [26] developed an analytical model based on a system of three massless spring elements: internal bolt elongation k_b, expanding sleeves k_{HB} and mechanical anchorage k_M. The model approximates the behaviour of the component by placing the expanding sleeves and mechanical anchorage elements in a parallel arrangement, while in series with the internal bolt elongation element (Tab. 2). The proposed model accurately predicts the response of the

Tab. 2 Spring analytical model for EHB in tension component [26]



Parallel configuration:	Series configuration:
$F_{HB,M} = F_{HB} + F_M$	$F_{EHB} = \min(F_b; F_{HB,M})$
$k_{HB,M} = k_{HB} + k_M$	$k_{EHB} = \left(\frac{1}{k_b} + \frac{1}{k_{HB,M}}\right)^{-1}$
$\delta_{HB,M} = \min(\delta_{HB}; \delta_M)$	$\delta_{EHB} = \delta_b + \delta_{HB,M}$

component and contributed to the development of a more detailed design method for the fastener.

In the table, k is the stiffness and δ the deformation capacity. The ultimate strength of the EHB component model F_u is calculated as follows (Eq. (1)):

$$F_u = f_{ub} A_s \tag{1}$$

where f_{ub} is the bolt ultimate stress and A_s the bolt tensile stress area.

3.2 Column face in bending

The column face component has been assessed under tensile pull-out load by means of a test arrangement that involves the use of a simplified rigid replica of the EHB. Mahmood [27] performed 17 pairs of tests on single-sided T-Stub EHB connections subjected to a monotonic tensile load for single and double rows of bolts. The study quantified the effect of changing concrete strength and type, tube wall slenderness ratio (defined as tube wall width-to-thickness ratio), gauge and pitch distances and bolt anchored length in the column face component performance. It was concluded that the first failure signs are caused by concrete crushing followed by bolt slippage.

Based on yield line theory, Mahmood and Tizani [28] developed a quad-linear model to estimate the plastic resistance and stiffness of the component bending behaviour based on the assumption that the strength of the column face component is equal to the sum of the tube plate strength and the anchorage resistance. The component plastic resistance is calculated as follows (Eq. (2)):

$$F_p = (F_{ps} + F_{pa})\left(\frac{1.1L_{an} + 130}{b}\right) \tag{2}$$

where:
F_{ps} SHS plate plastic resistance
F_{pa} anchorage plastic resistance
L_{an} EHB anchored length
b column face width

The EHB anchorage resistance F_{pa} is calculated as follows (Eq. (3)):

$$F_{pa} = A_c f_{tc} \tag{3}$$

where:
A_c concrete cone projected area, calculated based on the cone failure mode
f_{tc} confined concrete tensile strength, calculated from $f_{ct} = 0.1\, f_{cu}\gamma_2$, with $\gamma_2 = \dfrac{f_y}{10\mu} \geq 1$
f_{cu} characteristic concrete cube compressive strength

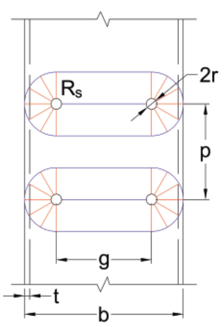
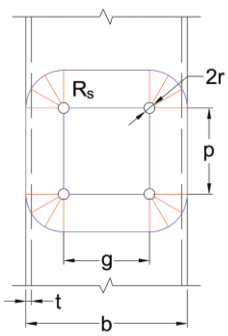
To calculate the SHS plate plastic resistance, two yield line patterns were identified as the dominant failure modes depending on the bolt pitch distance p (Tab. 3).

The proposed model captures the experimental data and numerical results with an acceptable level of accuracy, thus representing a significant step towards the development of design guidance for EHB connections on the basis of the component method.

3.3 Combined failure: bolts in tension and column face in bending

In the present study, an experimental programme was performed on eleven pairs of single-sided T-Stub EHB connections subjected to a monotonic tensile load for a single row of two EHBs. The parameters evaluated included bolt diameter, concrete strength, gauge distance,

Tab. 3 Yield line pattern modes for calculating SHS plate plastic resistance [28]

For $p > p_{crt}$:	For $p < p_{crt}$:
	
$F_{ps} = 4\pi M_p\left(1+\frac{R_s+r}{R_s}\right)+4M_p\left(\frac{2g-2r}{R_s+r}\right)$	$F_{ps} = 2\pi M_p\left(1+\frac{R_s+r}{R_s}\right)+2M_p\left(\frac{3p+3g-4r}{R_s+r}\right)$

where $p_{crt} = \frac{\pi}{3}(R_s+r)\left(1+\frac{R_s+r}{R_s}\right)+\frac{g}{3}$

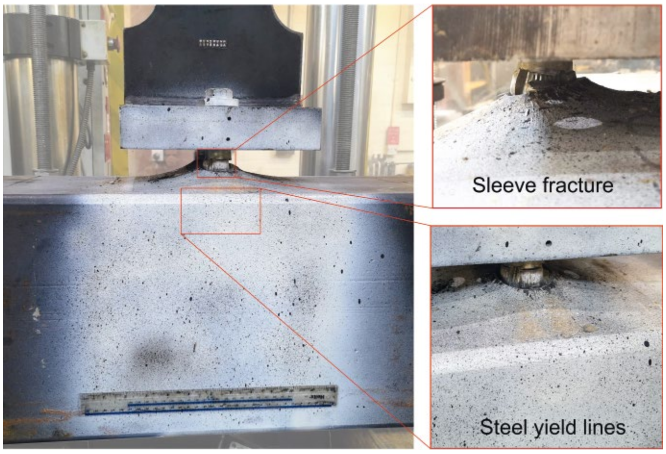


Fig. 3 Column face bending failure accompanied by sleeve fracture and bolt slippage (sample M20)

bolt shank length and tube wall slenderness ratio for the combined failure component.

It was observed that considerable deformation of the column face is generally accompanied by sleeve fracture. This failure mode had not been reported for the other components, i.e. the use of infill concrete has been demonstrated to reduce the concentration of stresses in the expanding sleeves, thus limiting their failure in the bolt component, and the simplified geometry and rigid bolt eliminated this failure mode completely in the column face component. However, in the combined component case, in configurations where the column exhibits considerable deformation and the concrete is severely damaged, the stress concentration in the area of contact between tube hole and EHB cone increases and causes sleeve fracture. Once the sleeve breaks, the bolts start slipping out of the steel section (Fig. 3).

Sleeve fracture generally occurs after column bending has reached the deformation limit of 3% of the steel section width. This limit for column face displacement at SHS joints is accepted as a strength failure criterion for SHS [29–31] and is adopted in the present study.

The average experimental results for each pair of samples are presented in Fig. 4 and Tab. 4. Fig. 4a shows that the variation in concrete strength has a significant effect not only in terms of stiffness, but also ductility, whereas the component strength remains unchanged. Reducing the concrete strength from C80 to C40 decreases the component stiffness by 38%, and by 29% when changing from C40 to C20. It was observed that a change in failure mode from column face in bending to bolt fracture occurs between concrete grades C20 and C40.

Increasing the bolt diameter from M16 to M20 increases the component strength by 29% and stiffness by 153% as shown in Fig. 4b. The failure mode also changes from bolt fracture to column face bending followed by sleeve fracture.

Fig. 4c shows that increasing the gauge distance from 80 to 140 mm increases the component stiffness by 22% and the strength by 3%. Increasing the gauge distance further to 180 mm increases the stiffness by 13%, whereas the component strength remains unchanged.

Similar to the findings of Pitrakkos [19], the change in the shank length for the parameters studied here do not seem to influence the component strength or the stiffness (Fig. 4d).

Fig. 4e shows that decreasing the tube wall slenderness ratio from 47.6 to 37.5 increases the stiffness by 22% and the strength by 30%. Further reductions in the tube wall slenderness ratio do not change the component initial stiffness (< 1% difference) or the component strength. However, a change in failure mode is observed between μ37.5 and μ30.0.

Fig. 4 shows the data collected from the experiments aimed at determining the resistance of the combined failure mode. That data will be used to establish an analytical model for this resistance and develop a machine learning algorithm that can predict such a resistance.

Tab. 4 Experimental results and failure modes (*benchmark sample)

Sample	Strength (kN)	Initial stiffness (kN/mm)	Failure
EHB16-170-140-8.8-C20-u30	300.0	111.1	Column face bending
EHB16-170-140-8.8-C40-u30*	299.0	156.3	Bolt fracture
EHB16-170-140-8.8-C80-u30	300.0	250.0	Bolt fracture
EHB20-170-140-8.8-C40-u30	386.4	400.0	Column face bending + sleeve fracture
EHB16-170-80-8.8-C40-u30	290.0	127.8	Column face bending + sleeve fracture
EHB16-170-180-8.8-C40-u30	300.0	178.6	Bolt fracture
EHB16-150-140-8.8-C40-u30	293.0	156.0	Column face bending + sleeve fracture
EHB16-190-140-8.8-C40-u30	292.6	155.7	Bolt fracture
EHB16-170-140-8.8-C40-u18.8	300.0	156.2	Bolt fracture
EHB16-170-140-8.8-C40-u37.5	300.0	156.3	Column face bending + sleeve fracture
EHB16-170-140-8.8-C40-u47.6	211.0	128.6	Column face bending + sleeve fracture

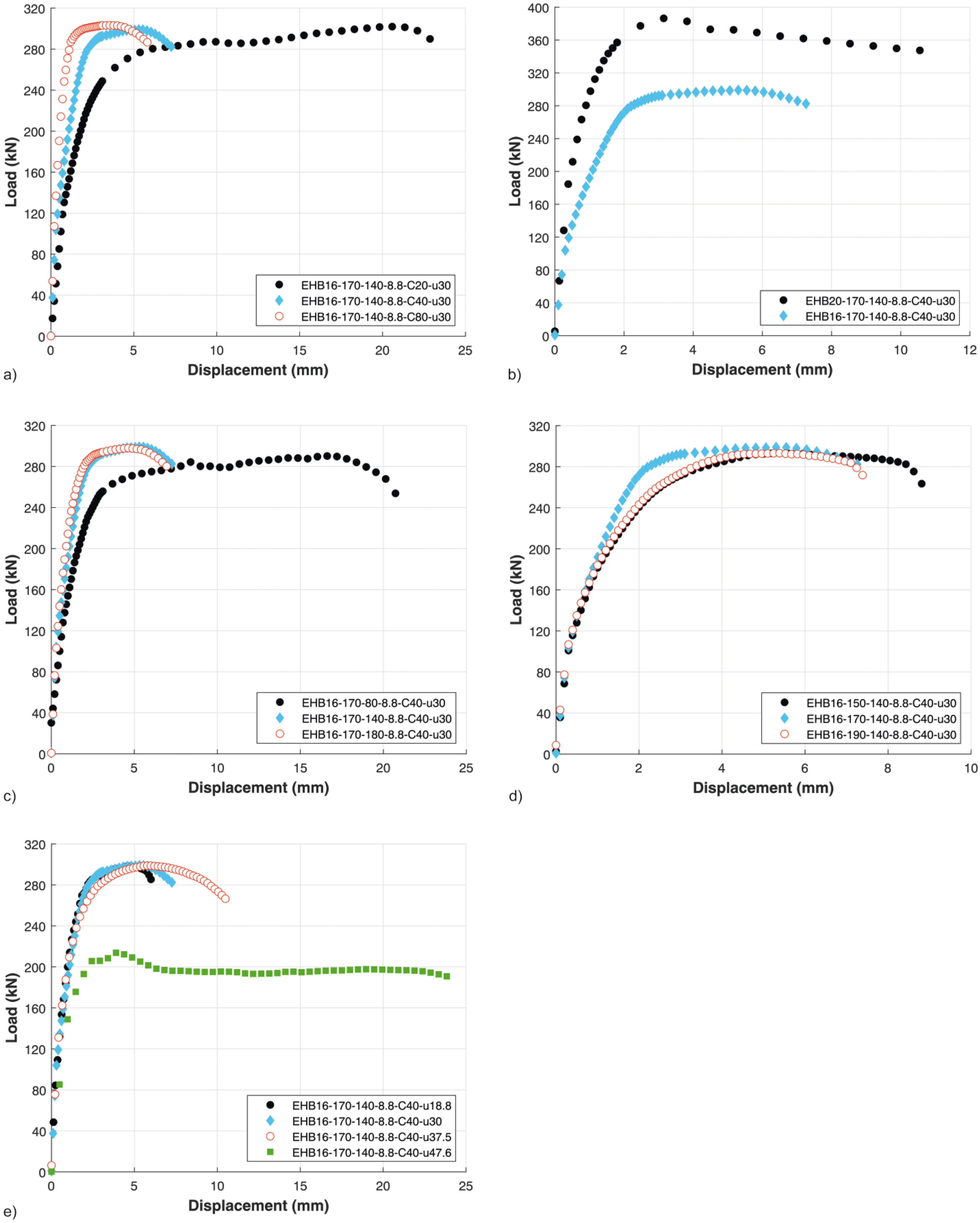


Fig. 4 Experimental results for a) concrete samples C20, C40, C80, b) bolt diameter samples M16, M20, c) gauge distance samples G80, G140, G180, d) shank length S150, S170, S190 and e) tube wall slenderness ratio samples μ18.8, μ30, μ37.5, μ47.5 (specimen key: bolt type–bolt diameter–shank length–bolt grade and batch concrete grade–sample number)

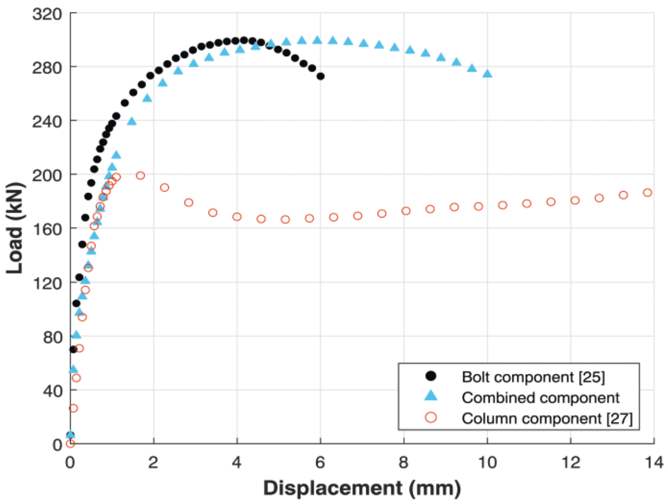


Fig. 5 Bolt, combined and column components global vs. load curves for samples with concrete grade 40 MPs, bolt diameter M16, anchored length 80 mm, tube wall slenderness ratio 37.5 and gauge distance 140 mm

4 Comparison of independent research work

The independent results from both bolt and column face components using similar geometric and material properties are described in this section for comparison. As the tests performed by Pitrakkos [25] only considered a single bolt, the results were multiplied by two in order to allow comparisons between components assuming that the use of two bolts would double the load results as long as the failure mode remains unchanged.

Fig. 5 shows that the initial stiffness of the combined component is that for the column components, whereas the ultimate strength corresponds to the ultimate capacity of the bolt. This means this sample developed the full strength of the bolt. The ductility of the combined component is 50 % higher with respect to the bolt component.

5 Research contribution and future work

The Extended Hollo-Bolt (EHB) is a blind fastener used in concrete-filled steel columns which shows promising potential for use in moment-resisting connections. A summary of the main advances regarding the EHB research work carried out to date was presented in this study. Some of the main findings are:

- Three failure modes have been identified in the combined failure component: column face in bending, bolt fracture and column face bending plus sleeve fracture. The latter has not been reported in independent studies of bolts in tension and column face in bending, indicating that further detailed studies of the combined failure mode are required for full characterization of the EHB connection.
- From the range of parameters tested, it is concluded that the most influential parameters in terms of strength are the bolt diameter and tube wall slenderness ratio, whereas the stiffness is enhanced mainly by changing the bolt diameter. These comparisons are made by changing one parameter at a time while keeping all the others constant. Combinations of the parameters studied are recommended in order to achieve a more complete understanding of the general behaviour of the EHB component.
- A comparison of the three EHB failure modes (i.e. bolt in tension, column face in bending, combined failure) for a sample with similar design properties has shown that the initial stiffness of the combined component is equivalent to the column component, whereas the ultimate strength corresponds to the ultimate capacity of the bolt.
- The chosen range of design parameters used for the experimental programme produced changes in the failure mode of the combined component. Detailed studies of these ranges are recommended for future work.

The experimental results presented in this work constitute a database for further finite element analysis, analytical modelling and machine learning algorithms, which could be combined to contribute towards the development of design guidance for EHB connections.

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