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Group combined failure in tension of extended hollo blind bolts using numerical finite element method

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Abstract

The structural hollow sections (SHS) have several advantages comparing with the open sections. However, the application of the SHS in the industry is limited due to the traditional bolt connection and cannot be used for the SHS as the inaccessibility to the inside of section. Several blind bolt systems were developed to overcome this problem and the EHB is the modified version of one of the blind bolt systems.

This project aims to investigate the behaviour of the Extended Hollo-bolt (EHB) connection under combined failure mode with a focus on the influence of the bolt pitch distance by using test data and Finite Element (FE) models and to propose an analytical model for the connection. The EHB connection is between a SHS column filled with concrete and a beam made of an open section. Two rows of EHB were embedded in concrete filled SHS and tested under tension.

FE models were created and validated by tests data and they were used to investigate the aforementioned aims. It is found that pitch distance has a significant influence on the strength of the connection as increase of the pitch distance increases the strength of the connection until a critical value. An analytical model was also proposed for the connection.

Keywords: Extended hollo blind bolt connection, Finite element analysis, Numerical modelling, Pitch distance

Introduction

Background

Bolts are one of the most common elements in construction and they consist of a threaded screw and a mating fastener which is called a nut. Bolt contains many advantages such as simple design, easy assembling and disassembling, can resist tension comparing with welding and rivet. However, the major disadvantage of the bolt is that it requires the access to both sides of the joint to assemble. This makes it difficult or sometimes impossible to use with enclosed steel section profiles, such as structural hollow sections (SHS).

To overcome these major drawbacks, blind bolt is developed, which can be assembled from only one side of the joint. Cabrera et al. [2] state many different types of blind bolts that have been developed by research groups and commercial companies. Those different types of blind bolts usually have different geometry arrangement and applies their unique on-site installation techniques.

None of blind-bolted connections currently in use can achieve a rigid connection but the researchers of University of Nottingham have introduced a new type of blind bolt, called the Extended Hollo-bolt (EHB), which has shown to have the potential to be used as rigid connection [11].

Pitrakkos and Tizani [11] introduce the EHB as a modified version of the Lindapter Hollo-bolt with distinction of containing an elongated internal bolt and the anchor head which is attached to the end (Fig. 1). The added head at the end of the extended shank in the EHB is to make use of the infill concrete and create an anchoring effect to improve the resistance of the connection [2].

Despite the research that have been done about the EHB and the potential of EHB being used as a rigid connection in SHS design, Cabrera et el. [2] concluded that there are still areas of this topic unexplored yet such as the combined failure mode and the insufficiency of the knowledge at present prevents the EHB to be a safe design of a moment-resistant connection. Cabrera et al. [2] recommends further parameter studies to be conducted for the combined failure mode as the behaviour of the connection has not been fully characterised when all of the component can deform.

Therefore, the research of this project will be focused on investigation of the group combined failure in tension of extended hollo blind bolts by using FEM modelling and the parametric study of the influence of the pitch distance on the strength of the EHB connection.

Fig. 1 The Extended Hollo-bolt (EHB) [12]

Laboratory test information

Three sets of experimental tests of the EHB connection with different pitch distances were conducted in the laboratory by Cabrera [3] in the University of Nottingham, and data of load and displacements at different locations of the connection were measured and recorded. These data were used in conducting this research project. In all of the tests, two rows of EHB were tested under a tensile load which is applied through a T-stud. The general test information is summarized in Table 1. Figure 2 demonstrates the sample configuration.

Test setup

Figure 3 demonstrates the setup of the test. Three linear potentiometers (POT) are used to measure the displacement at bolt head, bolt end and top of the column face. In addition, linear variable differential transformer (LVDT) is used to measure the displacement at the side of the column face. The test sample is placed between wood supports and restriction frames and the load is applied through a 50-mm T stud. As shown in the figure, a rod extension which is attached to the end of a EHB is used to facilitate the measurement. The load is increased gradually increased from 0 to the point where the sample fails in one of the failure modes. A picture of the test sample is given in Fig. 4. Cabrera [3] present the detailed test-setup information and test results.

Table 1 General test information

Sample ID	Bolt row	Bolt diameter	Bolt grade	SHS section (mm)	Slenderness ratio
P100-1	Double	M16	8.8	300 × 10	30
P100-2	Double	M16	8.8	300 × 10	30
P180-1	Double	M16	8.8	300 × 10	30
P180-2	Double	M16	8.8	300 × 10	30
P260-1	Double	M16	8.8	300 × 10	30
P260-2	Double	M16	8.8	300 × 10	30

Sample ID	Bolt gauge (mm)	Bolt pitch (mm)	Design concrete grade	Actual concrete grade	Embedment depth (mm)
P100-1	140	100	C40	43.57	86
P100-2	140	100	C40	41.73	86
P180-1	140	180	C40	32.5	86
P180-2	140	180	C40	32.18	86
P260-1	140	260	C40	29.67	86
P260-2	140	260	C40	29.87	86

Sample ID	Anchored length (mm)	Shank length (mm)	SHS length (mm)
P100-1	102	170	800
P100-2	102	170	800
P180-1	102	170	1050
P180-2	102	170	1050
P260-1	102	170	1050
P260-2	102	170	1050

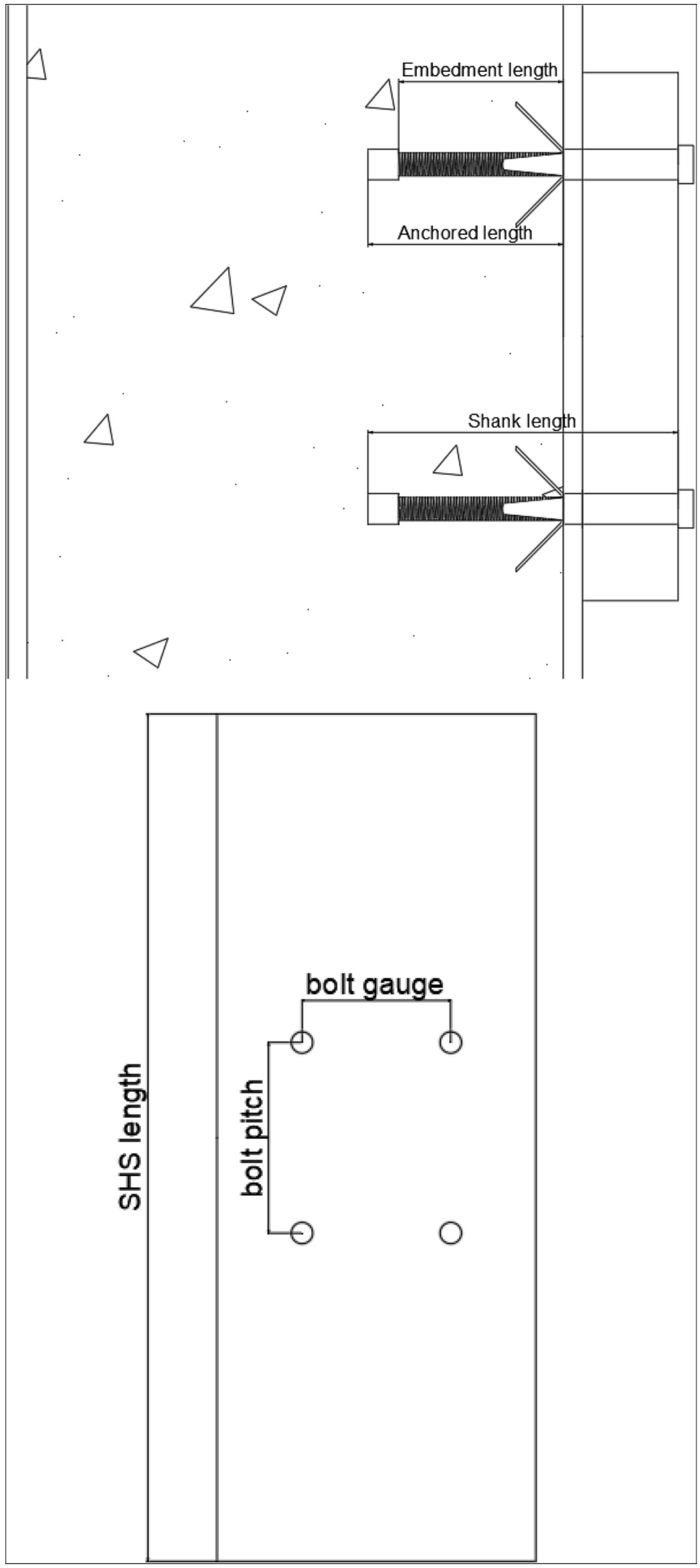

Fig. 2 Sample configuration

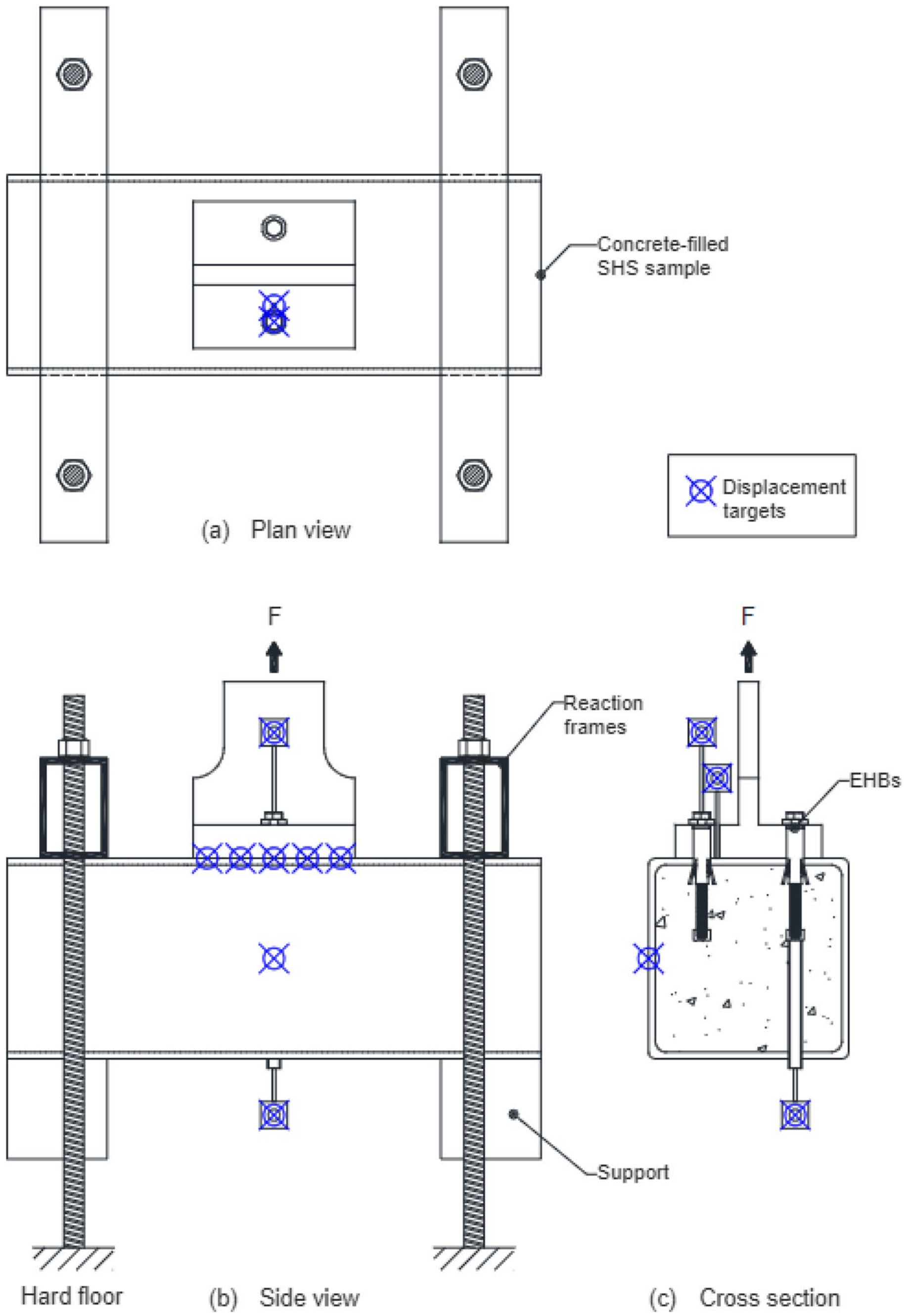

Fig. 3 Setup of the test [3]

Methods

The aim of this research project is to investigate the stiffness and strength of the EHB connection between concrete filled SHS column and open section beam under group combined failure mode, in which both bolt failure and column-face failure under tensile load occurs, by devising and using a numerical finite element (FE) model of the connection.

Fig. 4 Test of the sample [3]

In addition, to use this FE model to investigate influence of pitch distance between two rows of extended hollo-bolt (EHB) in the connection on the strength of the connection and to propose an analytical model which can be used to calculate the strength of the EHB connection.

This chosen aim is an attempt to fill the gap of knowledge identified through the literature review as the parameters of the EHB under combined failure mode is still not fully understood.

The methodology used for this project includes finite element simulation and analysis of tensile tests of EHB connection to concrete filled SHS column, parametric study of the EHB connection based on the FE model results, data analysis and proposal of analytical model by using component method.

FE modelling

Three sets of tensile tests of the EHB connection with three different pitch distance are modelled. The simulated tests are done experimentally in the laboratory by Cabrera [3] and the data which includes load, displacement of the bolt head and columns faces at different locations are measured and recorded during the test. FE models which simulate these tests were created and run to obtain the data to compare with the test data.

The failure mechanism, stress distribution, and overall connection behaviour were compared between the test observation and FE model calculation to check the correctness of the created FE models. In addition, data measured in the test and data calculated by using the FE model were compared to check and validate the accuracy of the FE model.

Parametric study and data analysis

After the FE models being validated, additional FE models with varied pitch distances were created based on the validated models as a substitution of laboratory test. The data collected from those FE models were examined and compared to investigate the influence of pitch distance on the strength of the connection.

Analytical model by component method

Analytical model which can be used to simulate and calculate the strength of the EHB connection were proposed based on component method and the results were compared with test data to check the accuracy.

The component method

Background

Proposal equations which are able to directly calculate and predict the stiffness, strength, and behaviour of a steel connection is considered to be complicated task as there are many different structural members coexist in the connection. For example, the column face, beam web and flange, endplate, and bolts and nuts. In addition, complex interactions between different parts of the connection and material and geometry non-linearity of different members in the connection will occur when the connection is subjected to different types of loads.

Analytical models

Several analytical models have been proposed by different researchers to calculate and predict the behaviour of different components of both blind bolt connection and EHB connection. In this chapter, some of the analytical models proposed for the bolt component and the column face component are presented.

Bolt component

Pitrakkos [10] studied the behaviour of the EHB under tensile load by conducting experiment tests and he proposed a spring model to represent the stiffness of the EHB. He stated that the three mechanisms, which are bolt elongation (k_{b}), expansion of the sleeve (K_{HB}) and the mechanical anchorage (K_{m}), have influences on stiffness of EHB and he suggested to use massless spring to represent the stiffness of these three mechanisms as shown in Fig. 5.

By using massless spring model with such a configuration, Pitrakko (2012) reported that the analytical model achieved a 95% accuracy predicting the behaviour of the bolt component when compared with the experimental test data.

Column face component

Mahmood [8] used the yield line method to predict the failure mode of the column face component with two rows of two bolts. He proposed six possible failure modes

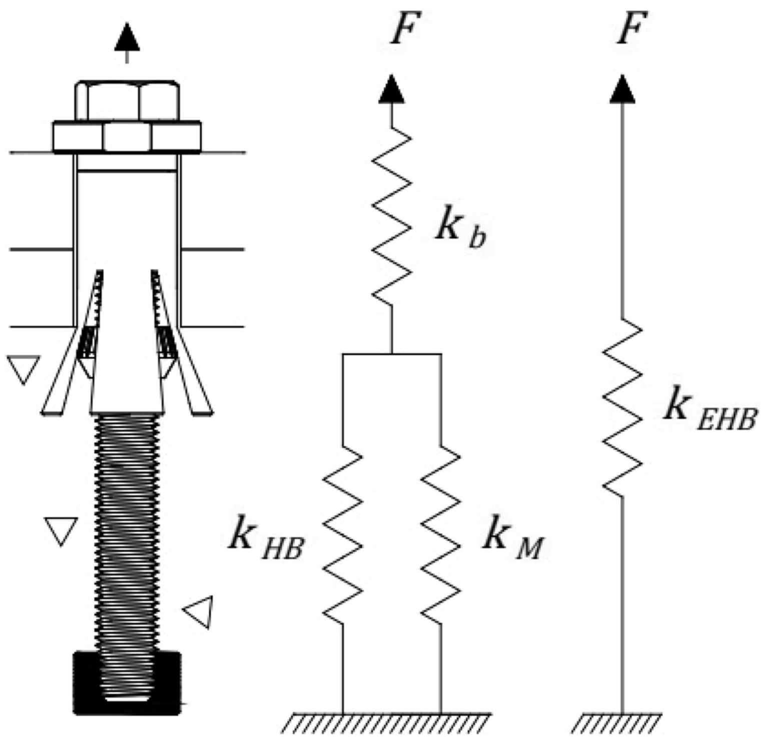

Fig. 5 Spring model for the EHB component [10]

depending on the bolt pitch distance and gauge distances as shown in Fig. 6. The mode 3 and mode 6 are idealised model for mode 2 and mode 5.

By equating the internal and external work based on the yield line patterns, Mahmood [8] derived equations which can predict the plastic resistance of the SHS column face component and they are given below:

$$Fpssingle = 2\pi Mp \times (1 + \frac{Rs + r}{Rs}) + 2Mp \times (\frac{2g - 2r}{Rs + r}) \tag{1}$$

$$Fpsdouble = 4\pi Mp \times (1 + \frac{Rs + r}{Rs}) + 4Mp \times (\frac{2g - 2r}{Rs + r}) \tag{2}$$

$$Fpscombined = 2\pi Mp \times (1 + \frac{Rs + r}{Rs}) + 2Mp \times (\frac{3p + 3g - 4r}{Rs + r}) \tag{3}$$

where.

Mp: plastic moment of resistance $= (fy \times t^2)/4$.fy: SHS plate yield strength$b$: SHS width$t$: thickness of the SHS column face

Rs: radius of the yielded area $= (\boldsymbol{b} - \boldsymbol{g} - 2\boldsymbol{r})/2$.$r$: radius of the bolt hole (for M16 EHB it is 13 mm [7])g: bolt gauge distance$p$: bolt pitch distance

Equation 1 can be used to calculate the column face plastic resistance of a single row of two bolts and Eqs. 2 and 3 can be used to calculate the column face plastic resistance of two rows of two bolts under mode 2 and mode 5 respectively. In addition, he

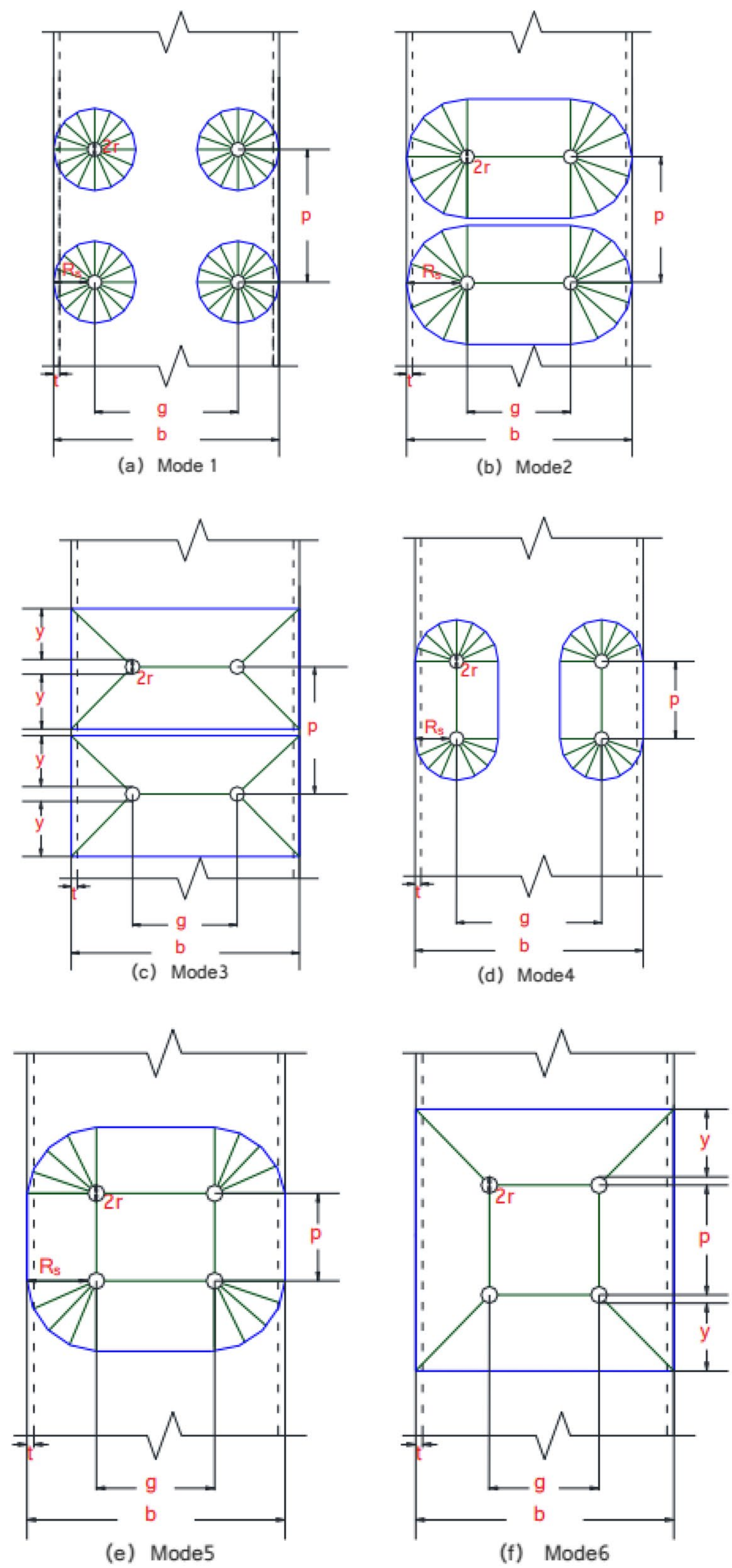

Fig. 6 Possible yield line pattern for SHS loaded at four points [8]

presented the equations for calculating the concrete contribution to the strength of the column face component. These equations are given as follows:

$$Fpa = Ac \times fct \tag{4}$$

where.

Ac: concrete cone projected area.fct: tensile strength of confined concrete = 0.1 × fcu × ϒ2fcu: concrete compression strength

ϒ2: = fy/(10μ) ≥ 1.μ is the slenderness ration of the column facefy: column face steel yielding strength

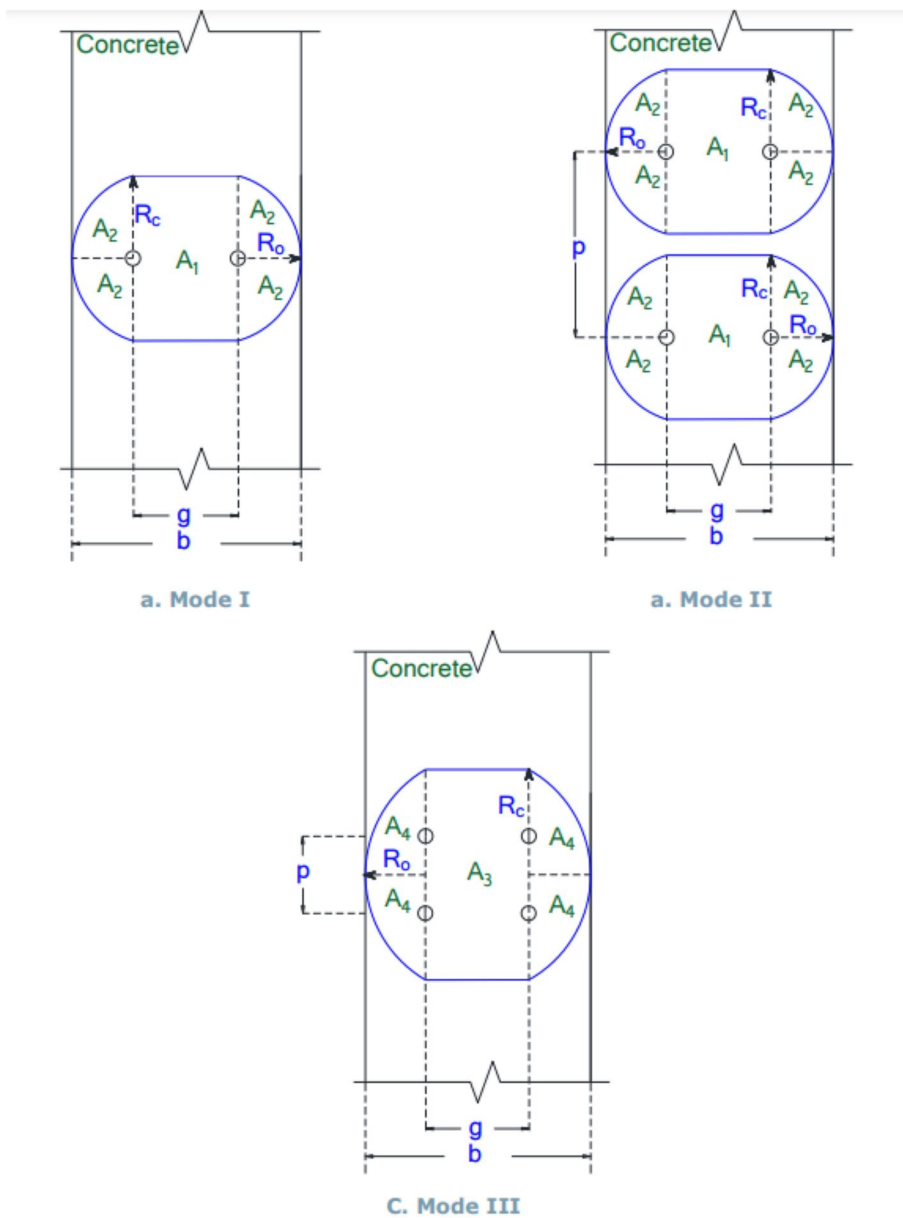

Fig. 7 Failure mode for concrete cone [8]

Three failure modes of the concrete cone were proposed by Mahmood [8] as shown in Fig. 7. The equations for calculating each of mode of the concrete cone projected area (Ac) are given below:

For mode 1:

$$Ac = 2Rc \times g + 8/3 \times Rc \times R_o \tag{5}$$

For mode 2:

$$pcrtcon = 2Rc + 0.75Lan \tag{6}$$

$$Ac = \gamma 3[2Rc \times g + 8/3 \times Rc \times Ro] \tag{7}$$

where.

Rc: radius of the concrete cone = 0.82 *Lan.*

Ro: distance from the bolt centerline to the concrete edge = $(b - g - 2t)/2$

$$\gamma 3 = \frac{4.03g + 5.37R_o}{1.64g + 2.19R_o} \tag{8}$$

γ3: correction factor

For mode 3:

$$pcrtcon = 2.39Lan \tag{9}$$

$$Ac = (2Rc + p) \times g + \frac{8}{3} \times (Rc + \frac{p}{2}) \times Ro \tag{10}$$

Equations 6 and 9 can be used to calculate the critical pitch distance where mode 2 and mode 3 change from one to the other.

The column face component strength is given as

$$Fp = (Fps + Fpa) \times \gamma 1 \tag{11}$$

where.γ1: correction coefficient = (1.1 Lan + 130)/b

Lan: anchored length of the bolt.

For the initial bending stiffness of the column face component, Mahmood [8] proposed a spring model, in which the stiffness of the component is represented by four springs which are located at each of the bolt hole. These four springs are arranged in parallel configuration and the summation of the stiffness of the four springs is equal to the component effective stiffness as shown in Fig. 8.

Mahmood [8] adapted the equation proposed by Ghobarah et al. [6], which can be used to calculate the displacement of column face at the bolt location for SHS with concrete infill as shown in Eq. (12). By using this equation, Mahmood [8] proposed an equation which can calculate the initial stiffness of the column face component for two rows of two bolts shown as Eq. (13). The value of teq and ϒf can be obtained from special tables and chart [8].

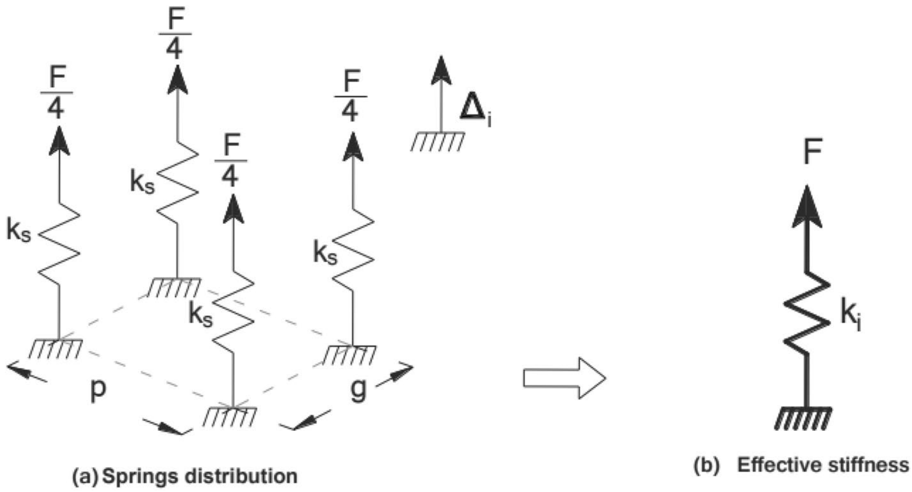

Fig. 8 Spring model for the stiffness of the component

Parameter	Range of validity
Concrete grade	C24 – C90
Column face slenderness ratio	25 – 40
Bolt gauge distance	(80 – 180)mm
Anchorage length	(80 – 112)mm
Bolt pitch distance	(120 – 280)mm
EHB	M16

Fig. 9 Validity of range for the proposed analytical model [8]

$$\Delta s = \frac{12\gamma f \times F(b - 2t)2(1 - \nu 2)}{Es \times t_{eq}^{3}} \tag{12}$$

where.γ*f*: coefficient of deflectionb: SHS widtht: SHS column face thicknessν: Poison's ratio of SHS

Es: Young modulus of elasticity for SHS.teq: equivalent column face component thickness

$$ki = \frac{\mathrm{Es} \times t_{eq}^{3}}{12 \times \gamma f \times (b - 2t)2 \times (1 - \nu 2)} \tag{13}$$

Mahmood [8] concluded that this model was only valid within the range of parameters which is given in Fig. 9.

Finite element modelling

General behaviour of the FE models

Three FE models with different pitch distances of 100 mm, 180 mm, and 260 mm were created to represent the three sets of tests done in the laboratory. In all of the FE models, the failure modes observed are the same, which is the column face of the SHS yielded before the EHB. After the initial yielding of the SHS, the column face continues to deform until the EHB starting to yield and eventually failed. The failure mode observed in the FE models is the same as the failure mode observed in the laboratory tests. Both compressive and tensile damage of the concrete elements were observed in the region near the EHB. This indicates the concrete in the region near the EHB is crushed in the FE model and the same concrete behaviour is observed in the laboratory tests.

Figure 10 demonstrates an example of the concrete compressive damage which is calculated by the FE model. It is clear that a cone shaped concrete which is around the bolt is damaged and crushed. This again agrees with the experimental observation which is reported by Mahmood [8]. In addition, Mahmood [8] stated that the measured concrete cone diameter on the surface is about 1.4 times of the anchored length (Lan). The FE model created for this project gives similar results as the shown in Fig. 11. The plastic strain develops on the column face around bolt as the column face starts to yield.

From the comparation between the FE model results and the experimental observation, it can be concluded that the FE predictions of the general behavior of EHB connection match the experimental observations.

Results and discussion

The data obtained from FE analysis and the data measured in the laboratory tests can be plotted as force–displacement diagrams for both the EHB and the column face component. It is an accurate method to check the behaviour, stiffness at different stages and ultimate strength of the EHB connection. In addition, the force displacement diagrams which are plotted with FE analysis data can be compared with the diagrams which are

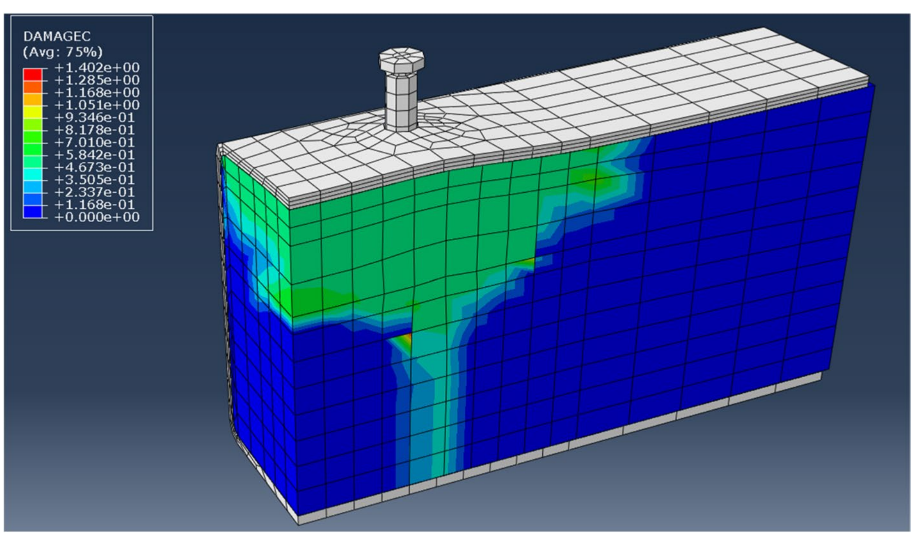

Fig. 10 Concrete damage in FE model

Table 2 EHB connection strength obtained from the FE model and the test data

Pitch distance (mm)	FE model Prediction strength (KN)	Test data strength (KN)	Error percentage
100 mm	518	535	− 3.1%
180 mm	563	570	− 1.2%
260 mm	597	585	2.0%

plotted with tests data to check the correctness and accuracy of the FE analysis and to validate the FE models. For this project, force–displacement diagrams of both EHB and column face were plotted for the three FE models and the three laboratory tests, which are correspond to each other.

Table 2 summarises the strength of the EHB connections obtained from the FE models and the test data. For all of the three FE models, the errors of the predictions of the strength of the EHB connections are less than 5% with maximum value of 3.1% and minimum value of 1.2%.

From the data analysis and force–displacement diagrams of both bolt component and column face component for the three FE models with varying pitch distances presented in this chapter, it can be concluded that the data obtained from the three FE models is accurate and within acceptable error range comparing with the experimental tests data. The behaviour of the EHB connections observed from the force–displacement diagrams of FE models match with that of the tests data. The FE models created can accurately predict the stiffness and strength of the EHB connections. As a result, the created three FE models have been validated.

Parametric study

Influence of the pitch distance on the strength of the connection

The validated FE models can be used to investigate the behaviour of the EHB connection within the valid range of parameters to achieve the aim and objectives which are investigating the influence of pitch distance of the bolt on the strength of the EHB connection. In addition to the three FE models with pitch distances of 100 mm, 180 mm and 260 mm created for the validation of the FE model, another three FE models with pitch distances of 80 mm, 140 mm and 220 mm were created. These pitch distances are chosen because they cover the pitch distances between the tested values. Together these six models cover pitch distances from 80 to 260 mm. The other parameters for these models are kept the same and they include the concrete strength which is 40 N/mm^2, bolt gauge distance which is 140 mm, the SHS size which is 300 mm × 300 mm, the column face thickness which is 10 mm, the SHS slenderness ratio which is 30, the bolt diameter which is M16 and bolt grade which is 8.8.

The strength of the connection can be represented by the column face plastic load as the column face failed and reached the ultimate limit state requirements set by Eurocode [4] before the failure of the bolts. The plastic load for each of the FE models is give in Fig. 12.

From Fig. 12, the plastic load increases with the increasing of the bolt pitch distance. The increasing of plastic load from 80 to 100 mm pitch distance and from 140 to 180 mm

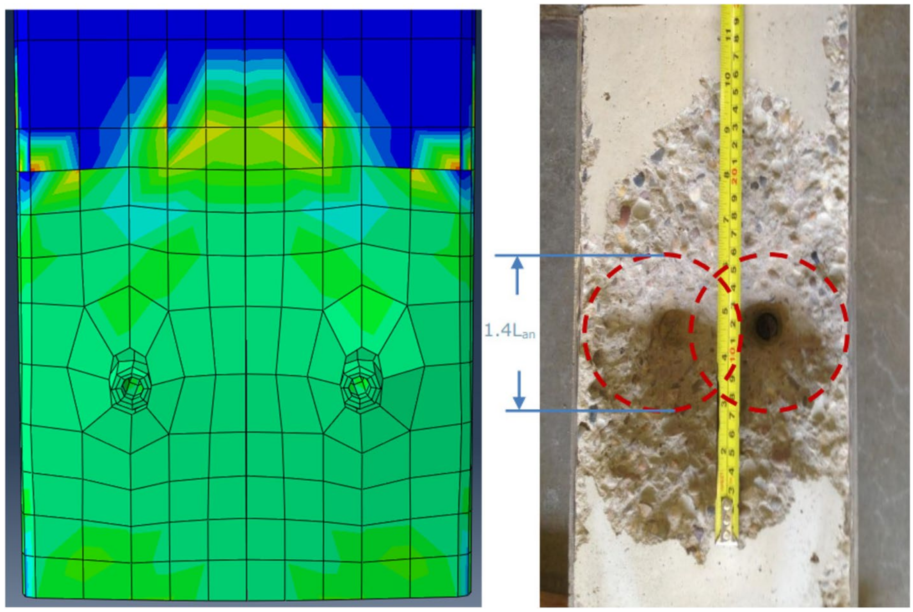

Fig. 11 Concrete damage pattern for FE (left) and test (right) [8]

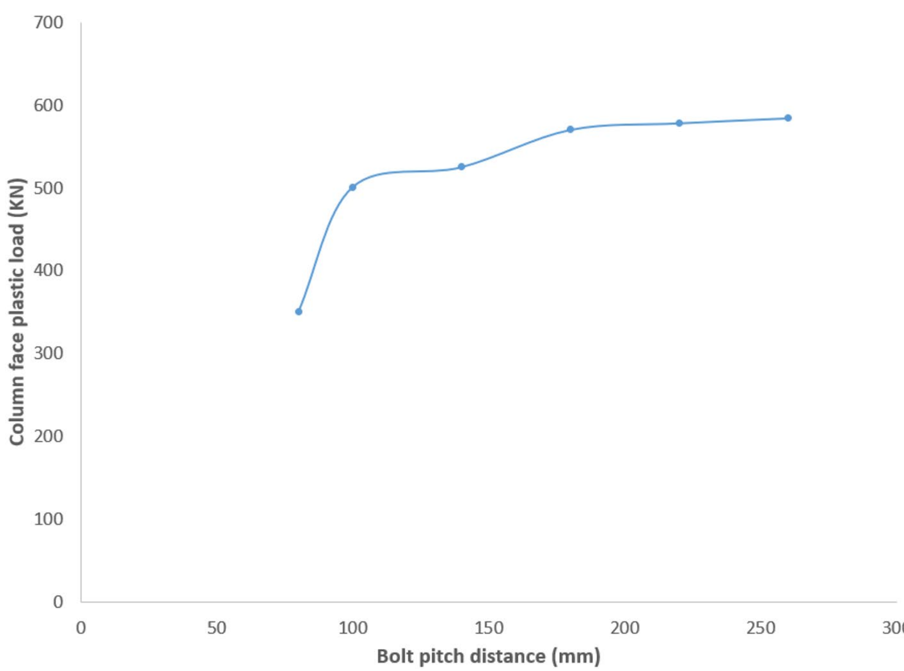

Fig. 12 Column face plastic load for different pitch distances

pitch distance is significant while the increasing from 100 to 140 mm pitch distance and from 180 to 260 mm pitch distance is slight. The minimum plastic load is 350 KN for the FE model with pitch distance of 80 mm and the maximum plastic load is 584 KN for the model with pitch distance of 260 mm. The plastic load for model with 80 mm pitch distance is significantly lower than the plastic load for all the other models and it is slightly more than half of the maximum value. This is because when two rows of the bolts are placed very closely to each other, they behaved as if they were one row of bolts since they shared the same concrete and SHS column face which contributes to the plastic load of the column face component.

The plastic load for the models with pitch distances of 100 mm and 140 mm are similar to each other while the plastic load for the models with pitch distances of 180 mm, 220 mm, and 260 mm are similar to each other. This can be explained by the failure modes proposed by Mahmood [8], which is given in the literature review. The models with pitch distances of 100 mm and 140 mm fit the description of the failure mode 5, in which two rows of bolts utilised the concrete and SHS column face combinedly and the models with pitch distances of 180 mm, 220 mm, and 260 mm fit the description of the failure mode 2, in which the two rows of bolts utilised the concrete and column face independently as the pitch distance between the two rows is large.

It is found that the change of the pitch distance from 100 to 260 mm have an insignificant affect for the initial stiffness of the connection with a difference less than 5% while there is a significant change of initial stiffness when the pitch distance change from 80 to 100 mm. The FE model prediction shows that the initial stiffness of the model with pitch distance of 80 mm is approximately half of that of the rest of the models. This again proves that when the pitch distance is relatively very small, the two rows of bolts behave as one row of bolts. Mahmood [8] mentioned that change of the pitch distance from 120 to 280 mm had an insignificant effect on the initial stiffness and it is believed that the initial stiffness of one row of bolts is half of the initial stiffness of two rows of bolts. The findings in this project agree with Mahmood's [8] observation of the influence of the pitch distance on the initial stiffness of the SHS.

Figure 13 shows the displacements of FE models with different pitch distances along the U2 direction which is the loading direction. The deformation of the column face can indicate the failure mode of the connection. In the picture of P80, the two rows of bolt are placed very closely and the failure mode of the column face is similar to that of a single row of bolts. In picture P100 and P140, the failure mode of the connections shown are similar to the failure mode 5 proposed by Mahmood [8], in which the two rows of bolts acted as a group. In picture P180, P220, and P260, the failure mode of the connections shown are similar to the failure mode 2, in which the two rows of bolts acted independently. This result matches the column face plastic load shown in Fig. 13.

Analytical model

Examining and modification of proposed analytical model

The equations, which could be used to calculate the strength of the column face component of the analytical model proposed by Mahmood [8], were examined for parametric ranges of this project. The equations are given in the "Introduction" section.

It was found that the equations for the EHB connection with one row of two bolts gave acceptable results. The equation gave a result of 293.4 KN for the column face component (Fpc) by using the parameters of this project while the FE model gave a result of 312.6 KN. The error between the analytical model and the FE model is approximately 5% which can be considered as acceptable.

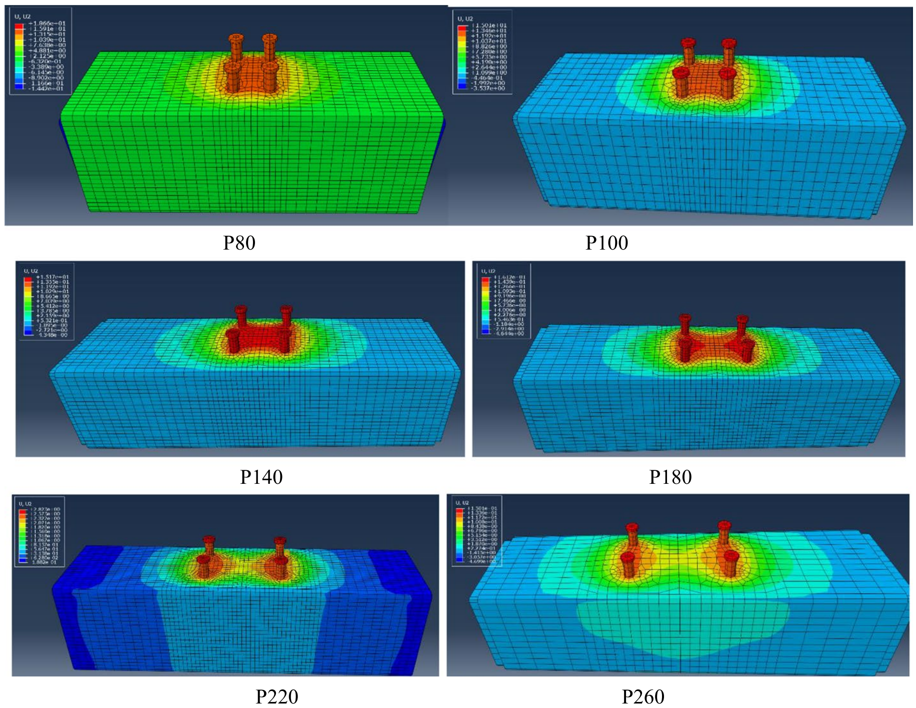

Fig. 13 Distribution of deformation for FE models with varying pitch distances

Table 3 Results of analytical model and FE model for 4 pitch distances

Pitch distances	Results of analytical model	Results of FE model	Error percentage
100	456.7	501	−8.8%
140	513.9	525	−2.1%
180	571.0	570	0.18%
220	628.2	578	8.6%

The equation, which could be used to calculate the strength of EHB connections with two rows of two bolts with a pitch distance of 220 mm under mode 2, of the analytical mode was found to give a result with relatively large error as it overestimates the strength of the column face component. The result calculated by the analytical model is 654.7 KN while the FE model and the experimental test gave a result of 584 KN.

The equation, which could be used to calculate the strength of EHB connections with two rows of two bolts under mode 3, of the analytical model was found to be able to give accurate results for 2 pitch distances of this project while was able to give results with approximately 9% error for another 2 pitch distances of this project. The results for the four pitch distances are presented in Table 6.1. The reason why only four pitch distances were considered was because according to Eq. (9), which was proposed by Mahmood [8] to calculate the critical pitch distances for mode 3, the critical pitch distance for mode 3 for this project should be 243.78 mm. Therefore, pitch distance of 260 mm does not belong to mode 3. In addition, based on the parametric studying result, the pitch distance of 80 mm behaved similar to one row of bolts due to the small value of pitch distance. As a result, pitch distance of 80 mm also doesn't belong to mode 3. The four pitch distances considered here all fit the requirements for being in mode 3.

From the results presented in Table 3, it can be observed that the analytical model gave most accurate result for the model with pitch distance of 180 mm. For the model with pitch distance of 140 mm, the result given by the analytical model is also accurate. However, when the pitch distances deviated from 180 mm, the error of the results produced by the analytical model increased.

As mention before, the by using Eq. (9) which is proposed by Mahmood [8] for calculating the critical pitch (Pcrit), the critical pitch distance for the parameters of this project would be 243.78 mm. However, based on the parametric study for this project, the critical pitch distance is 180 mm. Therefore, a modified equation is proposed to calculate the critical pitch distance for the parameters of this project as Eq. (14)

$$pcrtcon = 2Rc + 0.125Lan \tag{14}$$

where.

Rc: is = 0.82Lan.

Lan: is the anchored length of the bolt.

In order to reduce the error introduced by the equations used to calculate the column face component strength under concrete cone failure mode 2 and mode 3, the equation used to calculate $\gamma 3$ is adjusted as Eq. (15) and the new coefficient $\gamma 4$ is introduces as Eq. (16) for mode 3.γ3 is adjusted in the same as ways as how Mahmood [8] obtained this factor by substitute Ac in Eq. (10) by Eq. (7) and then solve the equation. The details are given in "Introduction" Sect. 1. However, the pitch distance used in the equation should be calculated by using Eq. (14) instead of Eq. (6) as the Eq. (14) is proposed for calculating the critical pitch distance for the parameters of this project.

The Eq. (16) is added as a correction factor to reduce the error of results between the analytical model and the FE models. Although the error of result between the analytical model and the FE models is insignificant, which is less than 10%. The introduction of Eq. (16) can further reduce the error percentage as show in Table 4. It's found that at the critical pitch distance which is 180 mm, the analytical result is closest to the FE result and the error of result increases as the pitch distance deviates from the critical pitch distance. Therefore, the Eq. (16) is derived based on how much the pitch distance deviates from the critical pitch distance.

$$\gamma 3 = \frac{3.2g + 4.4R_o}{1.8g + 1.85R_o} \tag{15}$$

Table 4 Results of analytical model and FE model for 4 pitch distances

Pitch distances	Results of analytical model	Results of FE model	Error percentage
100	493.3	501	− 1.50%
140	528.6	525	0.70%
180	571.0	570	0.18%
220	584.0	578	1.00%
260	584.1	584	0.01%

$$\gamma 4 = \frac{18 - 0.1p}{p} + 1 \tag{16}$$

where.g: is bolt gauge distance

Ro: is the distance from the bolt centreline to the concrete edge.

P: is the bolt pitch distance.

The new equation for calculating the component strength for concrete failure mode 3 is

$$Fp = (Fps + Fpa) \times \gamma 1 \times \gamma 4 \tag{17}$$

The rest equations proposed by Mahmood [8] are adapted for this project to calculate the strength of the EHB and they can be found in the "Introduction" section.

Table 4 presents the results and error percentages calculated by using the modified equations. The results calculated by the modified equations are much closer to the FE model results with no more than 2% of error for EHB connections with two rows of tow bolts acting combinedly and independently.

Overall behaviour of the component

Based on the experimental test data and the FE models results, the overall behaviour of the component in this project can be divided into four stages which can be represented by a quad-line diagram as shown in Fig. 14. This observation is similar to those reported by Mahmood [8] and Cabrera [1]. The first stage is from 0 to 0.85 of plastic load (Fp) which is obtained from the average value from the experimental test data and the FE models analysis. The second stage is from 0.85Fp to Fp which represents the decreasing of the component stiffness as it reaches plastic load. The third stage is

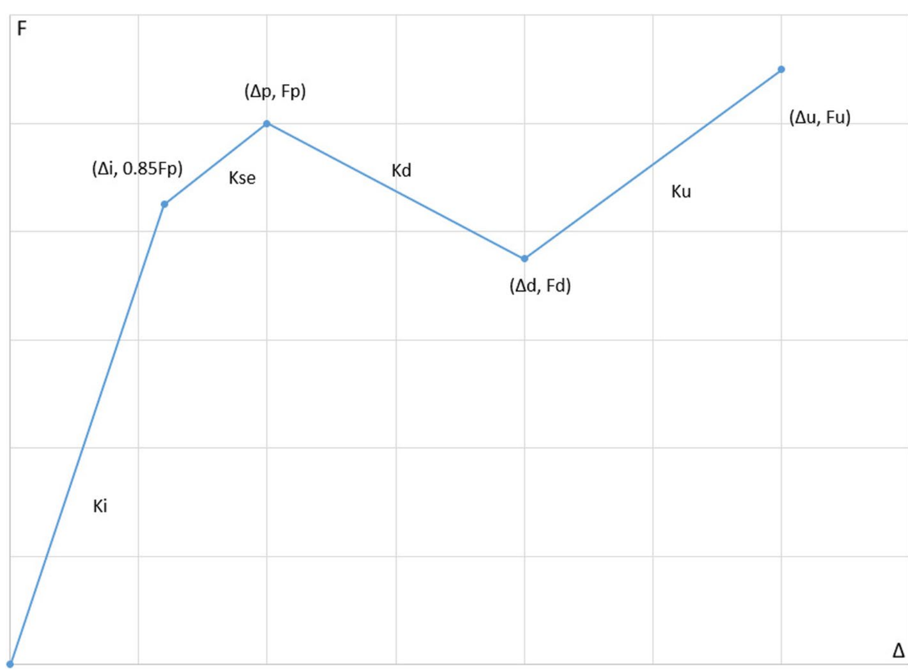

Fig. 14 Overall behaviour of the component

Table 5 Component stiffness based on test data

Pitch distances (mm)	Ki (KN/mm)	Kse (KN/mm)	Kse/Ki	Kd (KN/mm)	Kd/Ki	Ku (KN/mm)	Ku/Ki
100	379	38.2	0.101	− 8.35	− 0.02	9.02	0.024
180	302.8	34.2	0.113	− 2.89	− 0.01	5.12	0.017
260	382.5	38.15	0.1	− 8	− 0.02	4.73	0.012
Mean	-	-	0.105	-	− 0.017	-	0.018
Standard deviation	-	-	0.006	-	-		0.005

Table 6 Component displacement based on test data

Pitch distances (mm)	Δp (mm)	Δd (mm)	Δd/Δp	Fp (KN)	Fd (KN)	Fu (KN)
100	3.1	11.2	3.6	510	461	553
180	4.3	8.9	2.1	570	545	578
260	4.0	17.5	4.4	585	549	573
Mean	–	–	3.4	–	–	–
Standard divaition	–	–	0.95	–	–	–

the dropping stage, in which load dropped from Fp to Fd. The last stage is the increasing stage after the load dropped to Fd and in this stage, the SHS takes most of the load.

The stiffness of the component in each of the stages can be calculated by using the equations given below:

$$ki = \frac{0.85\mathrm{Fp}}{\Delta i} \tag{18}$$

$$kse = \frac{0.15\mathrm{Fp}}{\Delta p - \Delta i} \tag{19}$$

$$kd = \frac{Fd - Fp}{\Delta d - \Delta p} \tag{20}$$

$$ku = \frac{Fu - Fd}{\Delta u - \Delta d} \tag{21}$$

where.ki: initial stiffness of the component;

Δi: displacement at 0.85Fp;kse: secondary stiffness of the component;

Δp: displacement at Fp;

Fp: plastic strength of the component;kd: drop stiffness of the component;

Δd: displacement at Fd;

Fd: lowest drop load of the component after the plastic load;ku: final stiffness of the component;

Δu: displacement at Fu;

Fu: ultimate strength.

Tables 5 and 6 present the results calculated by using the equations given above.

From Tables 5 and 6, it is found that the value of Kse and Ku can be represented as a percentage of the stiffness of the first stage and the mean value of the ratios are used as this percentage in this project. This assumption is accepted and used by many researchers [6],MálagaChuquitaype and Elghazouli, 2010; [1, 8]. In addition, the drop displacement (Δd) can be calculated by using the mean of the ratios of drop displacement to plastic displacement (Δp).

Based on the information in Table 6, the previous equations can be written as follows:

$$ks = 0.105ki \tag{22}$$

$$ku = 0.018ki \tag{23}$$

$$\Delta d = 3.4\Delta p \tag{24}$$

In order to calculate the drop stiffness (Kd), the drop load has to be known and drop load is determined by the relation between the drop load and the plastic load based on the test data as shown in * MERGEFORMAT Fig. 15. Based on this relation, Eq. (25) can be used to calculate the drop load.

$$\mathrm{Fd} = -0.084\mathrm{Fp}^2 + 95.42\mathrm{Fp} - 26540 \tag{25}$$

The displacement at each stage can be calculated by rearranging the equations given previous as follows:

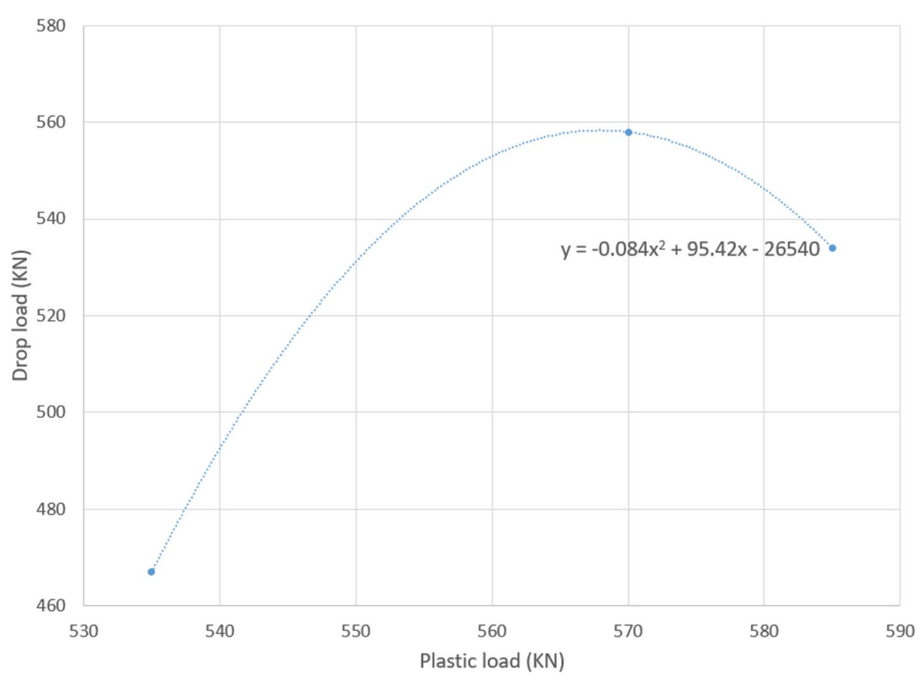

Fig. 15 Relation between plastic load and drop load

$$\Delta i = \frac{0.85Fp}{Ki} \quad (26)$$

$$\Delta p = \frac{0.15Fp}{0.105ki} + \Delta i \quad (27)$$

Since it is found that the pitch distance has insignificant influence on the initial stiffness of the EHB connection as mentioned in previous chapters and reported by other studying. The analytical model which can calculate the Ki is adapted from that proposed by Mahmood [8] and the model and equations are given in Chapter 1. The Fp can be calculated by using the modified analytical model and modified equations which are proposed in the beginning of this chapter. The Δu can be taken as 15 mm. The results calculated by using the analytical model and the test data are presented in Figs. 16, 17, and 18. The analytical model shows good prediction for the EHB connection.

Conclusions

Conclusions

Based on the study in this project, the follow conclusions were obtained:

- FE models can predict the behaviour of the EHB connection with acceptable error.
- Failures of the connection in the FE models are in the form of concrete cone crushing or separating from the main body, SHS column face yielding, and bending and EHB yielding and necking.

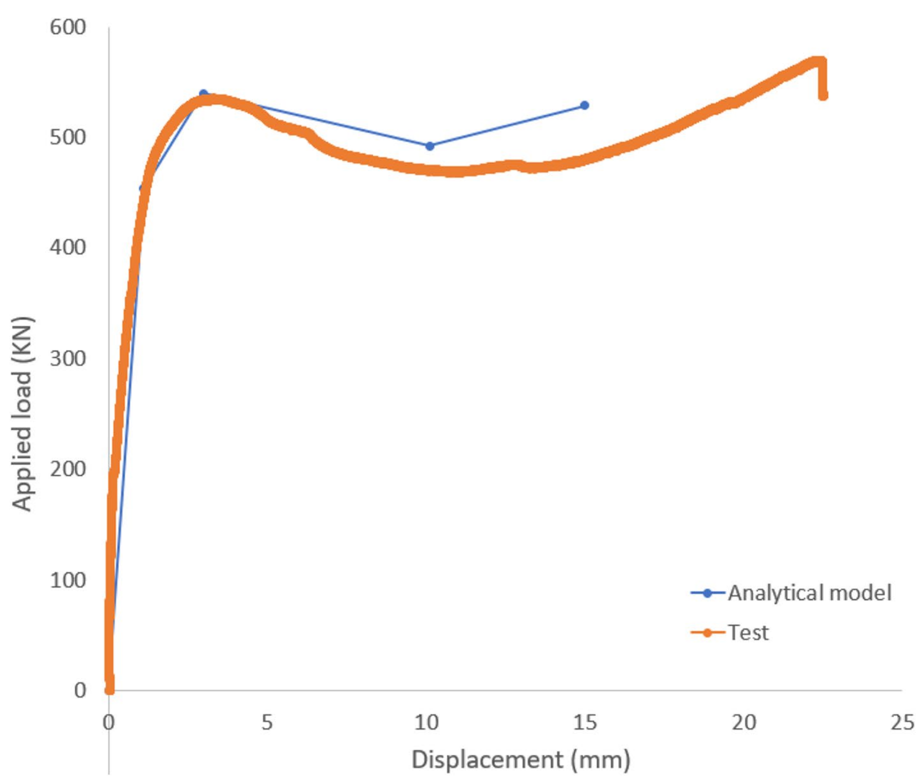

Fig. 16 Results of analytical model and test data for P100

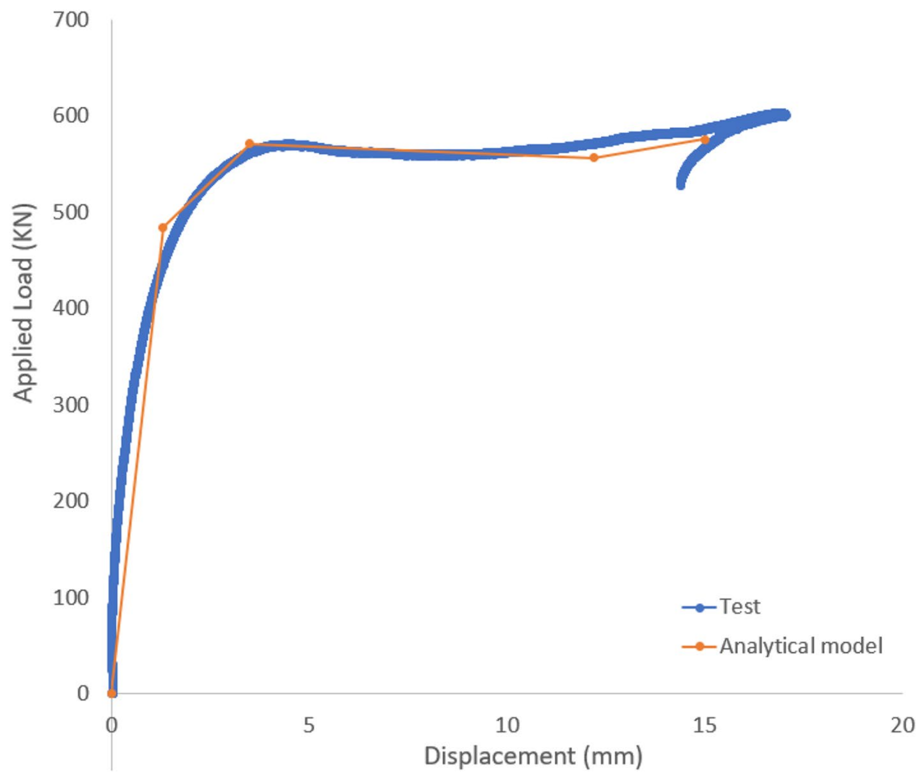

Fig. 17 Results of analytical model and test data for P180

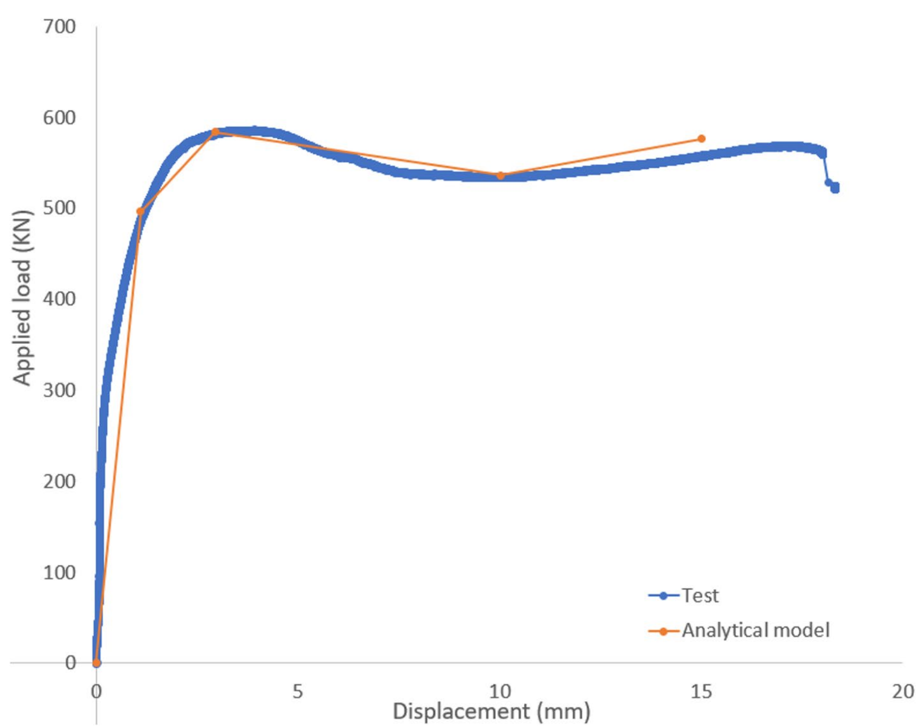

Fig. 18 Results of analytical model and test data for P260

- The strength of the connection drops when the concrete starts to crush and SH starts to yield. After the drop, the strength will increase again based on the behaviour of the SHS until the bolt failed.
- Bolt pitch distance of the EHB connection has significant influence on the strength of the connection. The strength of the connection increases with the increasing of the pitch distance until a certain point.
- When the pitch distance is very small, two rows of bolts behave similar to one row of bolts in terms of strength, stiffness, and failure mechanism.
- Bolt pitch distance presents insignificant influence on the initial stiffness of the connection with less than 5% increase when the pitch distance is increased from 100 to 260 mm for the parameters used in this project. However, when the pitch distance is very small (less than 80 mm in this project) so that two rows of bolts behave as one row of bolts, the stiffness become close to that of the one row of bolts.
- There is a critical pitch distance for the connection. When the pitch distance increased greater than the critical pitch distance, the strength of the connection does not increase significantly. The critical pitch distance varies with other parameters.
- The proposed tetra-line analytical model can predict the behaviour of the connection in terms of strength, stiffness and force–displacement relation.

Abbreviations

EHB	Extended hollo bolt
FE	Finite element
FEA	Finite element analysis
LVDT	Linear variable differential transformer
SHS	Structural hollow section
POV	Linear potentiometers

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Yongchang Guo's contribution includes writing the paper, creating, validating, and applying the finite element modelling and analysis, data analysis and presentation, proposal, and verification of the analytical model. The author read and approved the final manuscript.

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Availability of data and materials
Data used for this research project is available and provided as additional supporting documents.

Declarations

Competing interests
The author declares no competing interests.

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