Moment resistance and rotation capacity of semi-rigid composite connections with precast hollowcore slabs

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Abstract

Semi-rigid composite connection with precast hollowcore slab is a newly developed technique with few applications in the current construction practice. The research on the structural behaviours of this new type of connection is limited with no existing method available to predict its important characteristics as moment and rotation capacities. In this paper, based on the parametric studies of 3D finite element model and full scale tests, the analytical method to calculate the moment and rotation capacity of this type of composite joints were proposed. A comparison between the proposed calculation method and the full scale test results was made, good agreement is obtained,

***Keywords**: connection, semi-rigid, moment capacity, Rotation capacity, finite element; connection; modelling*

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1. Introduction

Compared with the traditional composite floor systems like solid R.C. slab or metal profiled decking slab floor system, precast floors can save construction time, reduce cost of concrete casting, etc. Therefore, this type of flooring system has become more and more popular in the current building construction practice in the UK.

In the current design practice, the beam to column connections using the precast hollowcore slab is normally designed as pinned connections. However, the research of the Fu et al. [1] shows that, provided with enough longitudinal rebars across the column lines, the strength and stiffness of the connection can be significantly improved. Further research of the Fu et al [1] also shows that semi-rigid connection behaviour can be achieved in this type of connections. Therefore, the further application of this type of joints into construction practice is promising. Hence, the basic characteristic of this type of the connections, the moment capacity and the rotation capacity, requires the further detailed study. In the past 30 years, although the behaviour of composite connections has been extensively examined, the majority of the work was concentrated on composite connections with metal deck flooring system or RC slabs, little research has been done on this new type of connections. Therefore, the research of the authors is imperative.

Moment rotation characteristics of semi-rigid connection using metal decking slab were first investigated by Johnson and Hope-Gill [2]. They proposed the calculation method for the plastic moment capacity of the connections which only takes into account the strength of the rebar. The contribution from other components such as the bolts was neglected. On the basis of their simple formula, Johnson and Law [3] proposed a more accurate formula for the plastic moment capacity of the composite connection which takes into account the contribution of the steel beam and the steel bars. The formula for predicting the moment capacity has been improved from the original model. However, it still did not take into consideration of the contribution from some of the important factors such as the bolts in tension and over-estimated the joint strength. Ren *et al* [4] and Anderson *et al* [5] used the different springs to represent the composite connections in order to calculate the rotation stiffness. Their methods were a milestone to the study of the characteristics of the connections, which is the basis for the component method which has been widely used recently. However, the limitation is that they all ignored the effect of column stiffness and the shear studs.

Anderson *et al* [6] proposed the calculation method for the rotation capacity of the composite joints with the consideration of the elongation of the reinforcement, the slip between the slab and the steel beam, and the deformation of the bottom flange. Anderson's method is the most accurate so far to predict the rotation capacity of the composite connections. This method has been adopted by many researchers and has been validated against many tests results. Recently, the mechanical model using the component method has become more and more popular for researchers to investigate the behaviour of composite connections; the principle of this method is to divide the connection into a set of mechanically connected components, representing the behaviour of each element parts. The behaviour of each element is then described by general constitutive relations, either in stress or strain space. Finally, the general connection behaviour can be combined together from these separate element relationships by considering force equilibrium and deformation compatibility. Work by Tschemmemegg [7] Madas [8] and Rassati *et al* [9] are all based on this method. Aribert, J.M. [10] provide discussed the Influence of slip of the shear connection on composite joint behavior.

Schafer et al [11] conducted nine composite joint tests; the most important finding was the ductility of the joints was improved by a spacing of 0.7 m between the endplate connection and the first shear stud. Thus the area of the plastification in the component 'reinforcement in tension' was enlarged. The similar results were also found by Fu [1],Helmut and Hans [12].

Although extensive full-scale tests have been carried out over the last three decades, but most of them are focused on the solid concrete slab and metal decking slab. Few of them have dealt with the precast hollowcore slabs. Fu et al [1] are the first researchers to do the full scale tests of composite joints with precast hollowcore slabs. Due to the limitation of the full-scale tests results, non-linear finite elements modeling method is another attractive tool for study this form of connection. The use of finite element could explore large number of variables and potential failure modes, which could complement the experimental studies. Although there were some researches towards the modelling of composite construction, most of the work is on the modeling of composite beams and little work has been done to the composite connections. A 3-D FE model of the steel-precast composite beams was built by El-Lobody et al [13] using ABAQUS to model the behaviour of the composite beams with precast hollowcore slabs; elastic-plastic material was used for the simulation. The model was validated against the test results and good agreement is obtained. Although there were some researches towards modelling this form of composite construction, most of the work is towards the simulation of the composite beams and little work has been done on the composite connections inelastic deformations consistent and high ductility moment-resisting frames.

For designers, the difficulty in designing semi-rigid composite frames lies primarily in the non-linear behaviour of the connection, which leads to complexity in predicting the joint moment and rotation characteristics. The use of the non-linear moment-rotation curve from the test results or modelling results is too complex for designers. To solve this problem, the best way is to provide designers with a simple but accurate calculation method to predict the moment and rotation capacities. There are some researchers worked on the analytical model to predict the moment and rotation capacity of the composite joints. Bayo et al [14] used a new component-based approach to model internal and external semi-rigid connections for the global analysis of steel and composite frames. The method is based on a finite dimensioned elastic–plastic four-node joint element that takes into consideration in a congruent and complete way, its deformation characteristics including those of the panel zone and all the internal forces that concur at the joint. Braconi *et al* [15] proposed a refined component model to predict the inelastic monotonic response of exterior and interior beam-to-column joints for partial-strength composite steel–concrete moment-resisting frames. The joint typology is designed to exhibit ductile seismic response through plastic deformation developing simultaneously in the column web panel, the bolted end-plate, the column flanges and the steel rebars. The model can handle large inelastic deformations consistent and high ductility moment-resisting frames. So far, little research has been done to model the behavior of the composite connection with precast hollowcore slabs, a suitable 3-D finite element model is important.

However, no equation or design method to predict the moment and rotation capacities for the composite connection with precast hollowcore slabs is currently available. The research of this paper is focused on propose the calculation method of the moment and rotation capacities. The main difference between the precast composite joints with conventional metal decking or solid slab joints is that the longitudinal bars of this new type of connection can only be placed between the gap of the precast slabs as shown in Fig 1. Therefore, the consideration of its own features of the precast slabs was also made in the research.

2. Full scale tests

Moment resistance and rotation capacity of the composite connection with precast hollowcore slabs were firstly studied by the full-scale tests method. As it is shown in Fig1, eight full scale tests with flush endplate composite connection and precast hollowcore slab were conducted by Fu et al [1]. The variables investigated were stud spacing, degree of the shear connections, amount of the longitudinal reinforcement and slab thickness. All specimens were of cruciform arrangement as shown in Fig. 1 to simulate the internal beam-column joints in a semi-rigid composite frame. The specimen was assembled from two 3300 mm long 457×191×89kg/m (W18×7.5×60) grade S275 universal beams and one 254×254×167kg/m (W10×10×112) grade S275 universal column to form the cruciform arrangement. The beams were connected to the column flanges using 10mm thick flush end plates with two rows of M20 Grade 8.8 bolts. The steel connection was a typical connection currently used in UK practice for simple joint.

A single row of 19mm diameter headed shear studs were pre-welded to the top flange of the steel beams. Finally, two 305×102×28 kg/m (W12×4×19) grade S275 universal beams were connected to the column web to make up of the full joint arrangement. The test set up, instrumentation, test material and tests result are explicitly described in Fu et al [1]. The test results are shown in Table 1.

3. FE models and parametric study result

Apart from full scale tests, the moment resistance and rotation capacity of the connections was also studied through the 3-D finite element modelling techniques by Fu et al [17]. Using the general-purpose finite element package ABAQUS [16], a three-dimensional finite element model consisting of three-dimensional continuum (solid) elements was created as shown in Fig.2 to simulate the composite joints with precast hollowcore slabs. Using 3-D solid element, the model replicates the composite joints from the experimental program by Fu et al [1]. In order to reduce the computing time of the computer, only one side of the tests was simulated. The sizes of all the components except the precast slab are the same as the actual experimental work. For the slab, only the in-situ concrete in the center is simulated. The boundary conditions and method of loading adopted in the finite element analysis follow closely those used in the tests. Using this model, parametric studies were conducted in [18].

4. Moment resistance of the connection

The tensile strength of the concrete is ignored as the tensile force of the slabs is relatively small and can be ignored. Only the tensile strength of the longitudinal reinforce bar are considered. The effective breadth of slab over which the reinforcement may be considered to act in tension in the negative moment region is taken as the width of the in situ concrete as there is no reinforcement placed outside this zone as shown in Fig.1. The push-out test conducted by lam [20] also showed that the effective breadth around the joint is confined to the in-situ in-fill concrete portion of the slabs.

It is also shown from the test results of Fu et al [1] that the moment capacity depends on the strength of the longitudinal rebars and the ability to mobilise them. At the ultimate limit state, the transfer of the compression force through the connection relies on direct bearing of the bottom flange of the beam. Mobilization of the rebars' strength requires that the compression side of the joint is not the weak element. This requires that the bottom flange of the steel section has an adequate area, and that its slenderness is sufficiently low in order to prevent local buckling in presence of high plastic deformation.

In order to study the moment resistance of the connection,Using the model of Fu [18], parametric study of the effect of the flange thickness is conducted. In this study, four 3-D finite element models (CJ1, Flange88, Flange44 and Flange22) were built. For these four models, all other conditions were kept the same as test CJ1 of Fu [1] except that the thicknesses of the bottom flange are:17.7, 8.85mm, 4.5 mm and 2.25mm, with the b/T ratio of 5.1, 10.24, 20.5 and 41 respectively. The comparison of the moment and rotation curves between these models is illustrated in Fig. 4 and Table 2. It can be seen that there is not much difference between the Model CJ1 and Model Flange88 with correspondent thickness of 17.7 mm and 8.85mm. Their ultimate moment and rotation are almost the same. No buckling of the bottom flange at the ultimate load was observed. The failure modes of these two tests are the yielding of the longitudinal steel bar as it is shown in Fig.5 (model Flange 88 at the failure).

Result shows that the flange started to buckle when the flange thickness decreased to 4.5 mm and 2.25 mm as shown in Fig. 6 and Fig. 7. Due to the local bucking of the bottom flange, the bottom component could not balance more tensile forces in the longitudinal rebars, so, the tensile forces could not increase any further. Therefore, low moment capacity and rotation capacity were resulted. It can be seen that, the rotation stiffness is also decreased.

Base on the full-scale tests and parametric studies, a calculation method of the moment capacity for this type of connection is derived. The proposed method is based on the assumption that no local buckling of column flange and web or large deformation will occur. Otherwise, different methods should apply.

From the modelling result, it can be suggested that, in order to achieve high moment capacity and rotation capacity, the bottom flange should be thick enough to prevent yielding of the bottom flange.

The moment resistance of a composite connection is determined from plastic analysis as shown in Fig 3, where,

$\mathbf{R_r}$ is tensile resistance of the reinforcement placed within the in-situ concrete of the slab as the steel bar is only placed in the in-situ concrete as shown in Fig. 1.

$\mathbf{R_b}$ is the effective tensile resistance of a pair of upper bolts,

$\mathbf{R_f}$ is the compressive resistance of the beam bottom flange. Due to strain hardening, the bottom flange can resist stresses of up to 1.2 P_y, Py is the characteristic strength of the steel beams.

1) For $\mathbf{R_r}$, the design tensile force of the reinforcement in the slab is derived as follows:

$$\mathbf{Fs}=f_y A_s \quad (1)$$

Where,

f_y is the characteristic strength

A_s is the cross sectional area of reinforcing bars.

Fs is the total tensile force of the longitudinal bar with the consideration of strain hardening. Rather than use the yield strength of the rebars, As discussed in Fu et al [1] , results of tests CJ1 and CJ2, CJ4 and CJ5 shows that, without considering strain hardening, these four tests have different level of shear interaction. However, test results show that these four tests achieved the same level of moment capacity. This is due to the strain hardening of the longitudes bars. Therefore, it is suggested that when determining the degree of the shear interaction, the strain hardening effect of the longitudinal bar need to be considered.

Another influential factor to the mobilisation of the rebar relied on the degree of the shear connection, which is decided by the number and the capacity of the shear studs. The design resistance of headed shear studs within the hogging moment region is

$Q=nQ_n$

Where

n shear studs number

Q_n strength of the shear stud

The reading of strain gauges on the shear studs of all the tests of Fu [1] show that there is no obvious deduction in the shear stud resistance after concrete cracking. Therefore, the characteristic shear resistance for the shear studs is used here.

If $F_s>nQ_n$ The Rr=nQn Otherwise Rr=Fs

2) For $\mathbf{R_b}$,

Fig.8 to Fig. 10 is the parametric study result of Fu [18], with the different thickness of end plate, three different modes of failure of the bolted endplate were found:

Mode 1 Complete yielding of extended endplate or column flange near the bolts

Mode 2 Bolt failure with the yielding of the flange (endplate or column)

Mode 3 Bolt failure

These three modes are Correspondent to Euro code 3 Part 1.8 [21], therefore, the potential resistance of bolt row can be determined by the yield line pattern in the end plate or column flange as stated in EC3.

The proposed method assumes that :

3) For $R_f \geq R_b + R_r$,

The moment resistance of the composite connection, M is:

$$M = R_r(D + D_r - 0.5t_f) + R_b(D - D_b - 0.5t_f) \tag{2}$$

Where as it is shown in Fig. 3:

D is the depth of the beam;

D_b is the distance of the first row of bolts below the top of the beam

D_r is the distance of the reinforcement above the top of the beam

t_f is the flange thickness of the steel beam.

For $R_f < R_b + R_r$,

The position of neutral axis, $y_c = \dfrac{(R_r + R_b - R_f)}{t_w P_y}$ (3)

Where

t_w is the web thickness

p_y is the design strength of steel section.

The moment resistance of the composite connection, M is:

$$M = R_r(D + D_r - 0.5t_f) + R_b(D - D_b - 0.5t_f) - R_w \frac{y_c}{2} \quad (4)$$

Where, $R_w = y_c t_w p_y$

In order to validate the proposed calculation method, the calculation results were compared with the full scale tests as shown in Table 3. It can be seen that, the proposed method is accurate to predict the moment capacity of the connections. CJ3 and CJ8 indicate a moderate overestimation of the moment capacity provided by the proposed method. . This is because in test CJ3 premature failure of the slab crack is observed, which caused the brittle failure of the whole connections with lower moment and rotation capacity. In test CJ8, slabs thickness was increased to 250mm rather than 200mm used in the other seven tests. Therefore, D increased, so the method predicted higher moment capacity as it is presuming that the D_r is the same as the other tests. However, for this two types of slabs with different thickness, the tapered section are not identical, D_r of slab 250 thick are slightly smaller than that of the slab 200 thick. This explained why the proposed method overestimated the moment capacity.

The proposed method is based on the assumption that no local buckling of column flange and web or large deformation will occur. Otherwise, different methods should apply.

5. *Rotation capacity of the connection*

To calculate the rotation capcity, the following assumptions are made:

1. Heavy column section are used, therefore, the deformation of the column can be ignored.

2. The deformation of beam bottom flange is small and can be ignored.

The tests results from Fu [1] show that the rotation of the connection can be assumed to take place about the centre of the bottom flange. The corresponding rotation capacity is therefore obtained by taking account of the distance from the reinforcement to the centre and the beam height. Therefore the feasible method to calculate the rotation of the joints can be described as the sum of the rotation caused by the longitudinal bar and the rotation caused by the slip.

5.1 Empirical calculation method of the rotation capacity

From the research so far, no calculation method is available for the composite connection with precast hollowcore slab. The available rotation capacity of this type of connection is dependent on the mode of failure for this form of construction. For the composite joints, the deformation is provided by yielding and inelastic elongation of the slab reinforcement and slip of the shear connectors. A calculation method is proposed here for predicting the rotation capacity of this form of composite joints as shown below:

$$\theta = \frac{\Delta_r}{D + D_r} + \frac{S}{D} \qquad (5)$$

Where

D is the depth of the beam

D_r is the distance of the reinforcement above the top of the beam.

D_b is the distance of the first row of bolts below the top of the beam

Δ_r is the elongation of the longitudinal bar

S is the interface slip between the slab and the steel beam

5.2 Calculation of the elongation of the longitudinal bar

5.2.1 Calculation of L_{eff}

Tests results of Fu [1] show that, the area of the plastification in the reinforcement in tension is critical for the composite joints. It is the main source of deformation capacity for the composite connections. In order to determine the elongation of the longitudial steel bar the effective deformation lenth L_{eff} of the longitudinal steel bar need to be determined first. The geometry of a typical reinforced composite slab was show in Fig 3, where, P0 is the distance between the column face and the centre line of the first stud; P1 is the distance between the centre line of the first stud and the second stud. P2 is the distance between the centre line of the first stud and the second stud and Rr is the total force carried by the reinforcing steel.

As it is found through the full scale tests of Fu [1]and Schafer et al [11], Helmut et al [12] the spacing between the endplate connection and the first shear stud is an important factor to the ductility of the joints. The deformation capacity is influenced not only by the effective deformation length but also the ductility of the reinforcing bars in the region of the joint by tension stiffening of concrete between cracks. With the yielding of the reinforcements, the effect of tension stiffening increases significantly. This is because the bond between concrete and reinforcement transmit the strain away from the cracks. The ultimate strain is reached only in the crack due to the cracking. Thus the average strain and the deformation capacity of the imbedded reinforcement are reduced compared to the behavior of the reinforcement working alone with out concrete slab.

Fig.11 is the strain profile measurement of the longitudinal bar along the beam from the full scale test of [1]. Results show that the plasticisation area of the longitudinal reinforcement is mainly concentrated between the centre line of the column and the second stud. The strain in the other part of the steel bar is very small and can be ignored. Hence, the effective length L_{eff} after yielding is assumed to be P0+P1+ D/2 here. However, the test results of [1] and the parametric study of [18] also show that when distance from the column flange to the first stud is over 900mm, the yielding of the longitudinal bar only occurred in the range of about 500mm from the centre line of the column, rather than the whole range of P0+P1+ Dc/2.

It can also be seen from Fig 11 that the strain reading of the steel bar is not evenly distributed. This is because that the crack formed randomly on the slab as shown in Fig. 12, which is the crack pattern observed during the full scale test of [1]. It can be also seen that, all the large crack were always formed within the range of $P0+P_1+D_c/2$, which is the L_{eff} suggested in this paper. This explained the reason that the strain in this area is higher than the remaining part.

5.2.2 Calculation of ε_{smu}

1) For Full shear interaction cases

The average ultimate strain ε_{smu} of embedded reinforcement can be calculated from the ultimate value which arise from the crack and the "transmission" length Lt (Hanswille, [19]) over which bond has broken down.

Below is the calculation method for the average ultimate strain, ε_{smu} and L_t

$$\varepsilon_{smu} = \varepsilon_{sy} - \beta_t \Delta\varepsilon_{sr} + \delta\left(1 - \frac{\sigma_{sr1}}{f_{y,s}}\right)(\varepsilon_{su} - \varepsilon_{sy}) \tag{6}$$

$$L_t = \frac{k_c f_{ctm} \phi}{4\tau_{sm}\rho} \tag{7}$$

Where

βt is taken as 0.4 for short-term loading

δ is taken as 0.8 for high-ductility deformed bars.

$\Delta\varepsilon_{sr}$ is the increase in strain in the reinforcement at the crack, when the crack opens,

σs_{r1} is the stress in the reinforcement in the crack, when the first crack has formed.

The cracking moment of a composite joint is defined as the moment that causes the mean tensile strength of concrete f_{ctm} to be reached at the top fiber of the uncracked slab. σs_{rl} and $\Delta\varepsilon_{sr}$ are calculated as follows:

$$\sigma_{sr1} = \frac{f_{ctm} k_c}{\rho}\left[1 + \rho \frac{E_s}{E_c}\right] \tag{8}$$

$$\Delta\varepsilon_{sr} = \frac{f_{ctm} k_c}{E_s \rho} \tag{9}$$

$$\rho = \frac{A_s}{A_c} \tag{10}$$

where,

ρ is the longitudinal reinforcement ratio,

A_s is the area of the longitudinal bar

A_c is recommended to be the in-situ for precast hollow core slab concrete

kc is a coefficient that allows for the self-equilibrating stresses and the stress distribution in the slab prior to cracking.

$$k_c = \frac{1}{1 + \frac{h_{cs}}{2z_0}}$$

Where

h_{cs} is the thickness of the precast slab

z_0 is the vertical distance from the centroid of the uncracked unreinforced concrete flange to the neutral axis of uncracked unreinforced composite section, which is calculated ignoring the reinforcement and using the modular ratio for short-term effects, E_s/E_{cm}.

ϕ is the diameter of the reinforcing bars

τ_{sm} is the average bond stress along the transmission length. For the bond stress, a value equal to 1.8fctm is given.

As discussed above, the effective deformation length of the reinforcement is defined as P0+P1+ D/2. Therefore, the formula for calculating the elongation of the longitudinal bar is recommended as follows:

For $\rho < 0.8$ %

$$\Delta_r = 2 \times L_t \times \varepsilon_{smu} \tag{11}$$

For $\rho > 0.8$ % and $P_0+P_1<L_t$

$$\Delta_r = (\frac{D}{2} + L_t) \times \varepsilon_{smu} \tag{12}$$

For $\rho > 0.8$ % and $P_0+P_1>L_t$

$$\Delta_r = (\frac{D}{2} + L_t) \times \varepsilon_{smu} + (P_0 + P_1 - L_t) \times \varepsilon_r \tag{13}$$

In equation 13, ε_r is the average strain rather than the yield strain of the longitudinal bar. Its value is taken as the strain ε_{sh} which is the onset of the strain hardening of steel material. This is because that, as it is shown in Fig. 13 which is the comparison of the moment-strain curve of the longitudinal rebars for the eight test results in of [1]. Except CJ3, which had the premature failure of the precast slab, all the longitudinal bars developed into the strain hardening stage. In most of the remaining length of the rebar, the strain was also yield. Therefore, it is not suitable to adopt the yield strain to calculate the elongation of the reinforcement as suggested by Anderson et al. [6].

As the average value at the onset of the strain hardening for the embedded reinforcements observed in Fig 13 is about 0.016, Therefore, 0.016 is recommended. However, as explained in the early section, if the first stud spacing over 900mm, the yielding of the longitudinal bar only occurred in the range of about 500mm from the centre line of the column, in the remaining part the strain is quite small, the average yield strain of the longitudinal bar observed from tests of Fu et al [1] is ε_r= 0.002, which is the yield strain of the steel bar ε_y. Therefore, 0.002 is recommended.

2) For partial shear connections:

From Fig13 the moment-strain curves of the longitudinal bar from [1], it can be seen that, due to the failure of the stud, for test CJ3, CJ4 and CJ5, the strain of longitudinal bar will never achieve the ultimate value. Therefore, it is not suitable to use above equations (6-13) to calculate the elongation of the longitudinal bar, as it is based on the average ultimate strain of the longitudinal bar. The following formula is recommended:

$$\Delta_r = (\frac{D_c}{2} + P_0 + P_1) \times \varepsilon_r \tag{14}$$

Where

ε_r =0.016

For CJ3, as premature failure happened, the total shear force of the studs is less than the yield force of longitudinal rebars, ε_r =0.002 is used .

5.3 Calculation of the interface slip between the slab and the steel beam

It worth noting that the deformation capacity due to slip at the steel/concrete interface is also important in predicting the rotation capacity of the composite connections. In this paper, Fig. 14 presents the moment-end slip capacity curve for the composite connection from the full scale tests of [1]. An empirical method has been derived by the author against this test result.

$$S=F/(Nk). \tag{15}$$

Where

S is the slip

N is the number of the shear connectors in the hogging region

k is the stiffness of a single shear connector. For 19 mm diameter headed stud, it is taken as 100kN/mm^2 taken from the push out result of precast hollow core slab from [20]

If $F_s>nQ_n$ Then F=nQn Otherwise F=Fs

Qn is the shear capacity of one shear stud, Taken as 128kN, from the test of [20]

5.4 Validation of proposed calculation method

In this section, the calculation method of the rotation capacity of the composite connection with precast hollow core slab has been proposed. In order to validate the proposed calculation method, the calculation results were compared with the full scale test result of Fu [1]. The comparison results are shown in Table 4. It can be seen that the method is accurate enough to predict the rotation capacity of the connections.

6. Conclusions

This paper presents the study of the moment capacity and rotation capacity of semi-rigid composite connection with precast hollowcore slab. Eight full scale tests of composite joints with precast hollow core slabs were conducted with different parameters as spacing, degree of the shear connections, amount of the longitudinal reinforcement and slab thickness. The 3-D finite element model was also built to conduct the further parametric study on the structural behaviour of this type of connections. Based on the tests program and the subsequent parametric studies using the finite element model, numerical methods to predict the moment and rotation capacity of this form of composite joints is proposed. The comparison between the proposed method and full scale test results was made, good agreement was obtained.

Base on above research, the following main conclusions can be drawn:

1. The full scale tests results shows that, the elongation and the ductility of the longitudinal bars plays an important role in the moment capacity and rotation capacity of the connections.
2. The parametric study with finite element model shows that, in order to achieve high moment capacity and rotation capacity, the steel beam bottom flange should be thick enough to prevent yielding or buckling of the bottom flange.
3. Different failure modes of the steel bottom flange were discussed through the parametric study.
4. Three main failure modes for this type of bolted connections were found through the finite element modelling:

   Mode 1 Complete yielding of extended endplate or column flange near the bolts

   Mode 2 Bolt failure with the yielding of the flange (endplate or column)

   Mode 3 Bolt failure
5. The Method to calculate the moment resistance of this type of capacity was proposed by the authors with good agreement to the full scale tests.
6. The empirical formula to calculate the interface slip between the slab and steel beam has also been proposed which presents adequate accuracy.
7. The way to predict the elongation of the longitudinal bar was improved based on the full scale tests result.
8. The Method to calculate rotation capacity of the composite connection with precast hollowcore slabs was also proposed by the authors with adequate accuracy

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