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RECENT RESEARCH AND DEVELOPMENT IN SEMI-RIGID COMPOSITE JOINTS WITH PRECAST HOLLOWCORE SLABS

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ABSTRACT

Composite structure incorporating steel beams and precast hollowcore slabs is a recently developed composite floor system for building structures. This form of composite construction is so far limited to simple beam-column connections. Although the concept of semi-rigid composite joints has been widely research in the past, most of the researches have been carried out on composite joints with metal deck flooring and solid concrete slabs. Research on composite joints with precast hollowcore slabs is rather limited. As the construction industry demands for rapid construction with reduction in cost and environmental impacts, this form of composite floor system, which does not require major onsite concreting, has become very popular among the designers and engineers in the UK. In this paper, full-scale tests of beam-to-column semi-rigid composite joints with steel beam and precast hollowcore slabs are reported. Based on the tests data; the structural behaviour of these semi-rigid composite joints is discussed together with numerical and finite element modelling. Through parametric studies, an analytical model for the semi-rigid composite joints is proposed and is verified by both the experimental data and finite element model; and good agreement is obtained.

INTRODUCTION

Composite floor incorporating steel beams and precast hollowcore slabs is a recently developed system for building structures. This form of composite construction is so far limited to simple beam-column connections. Compared with the traditional composite floor systems like solid R.C. slab or metal profiled decking floor system, precast floors can save construction time, reduce cost of concrete casting, etc., therefore, it is becoming more and more popular in the current construction market in the UK. In the past decades, a large number of studies have been conducted on the behaviour of composite joints, but the majority of these researches has been conducted on composite joint between steel column and composite beam with metal deck flooring system, little research has been conducted on this type of composite connection so far. As this form of composite design becoming more and more popular by the engineers and designers in the UK, a calculation method to determine the moment and rotational capacity for semi-rigid composite joint is badly needed.

Moment rotation characteristics of semi-rigid connection with metal decking slabs were first investigated by Johnson and Hope-Gill (1972). Ren et al (1995) and Anderson et al (2000) used different springs to represent the different components of the composite connections in order to calculate the joint stiffness, which is the basis of the component method which has been widely used today. Work by

Tschemmemegg (1988), Madas (1993) and Rassati et al (2004) are all based on this method.

In order to study the moment and rotation characteristic of the composite connections with precast hollowcore slabs, the best way is by carrying out full-scale tests. However, due to the expenses and limitation of the full-scale tests, non-linear finite elements method is an attractive tool for investigating 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. Lam et al (2000) were the first to simulate the behaviour of composite girders with precast hollowcore slabs; a 2-D finite element model was built using ABAQUS (2005). A 3-D FE model of the steel-precast composite beams was built by El-Lobody and Lam (2003) 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. Bayo et al (2006) 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 dimensional elastic–plastic four-node joint element that takes into consideration the joint deformation characteristics including those of the panel zone and all the internal forces that concur at the joint. Braconi et al (2007) 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 reinforcement. The model can handle large inelastic deformations consistent and high ductility moment-resisting frames. Recently, attempt has been made by Fu et al (2007) to model the composite joint with precast hollowcore slabs using 3-D finite element method, however, the use of FE modelling is still far too complex and impractical for designers and a simple but accurate analytical method to calculate the moment and rotation capacities for this form of composite joint is badly needed. In this paper, an analytical method for calculating the moment and rotation capacity is presented and comparison with the full-scale tests result is made to validate its accuracy.

FULL SCALE TESTS

Full-scale joint tests with flush endplate composite connection and precast hollowcore slabs were conducted by Fu and Lam (2006). The main variables investigated were stud spacing, degree of the shear connections and the amount of longitudinal reinforcement. All specimens were of cruciform arrangement as shown in Figure 1 to replicate the internal beam-column joints in a semi-rigid composite frame. The specimen was assembled from two 3300 mm long 457×191×89UB grade S275 universal beams and one 254×254×167UC 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. A single row of 19mm diameter headed shear studs were pre-welded to the top flange of the steel beams. The steel connection used is a typical connection used in UK practice for simple joint. Results of all the composite joint tests are shown in Table 1 and Figure 2.

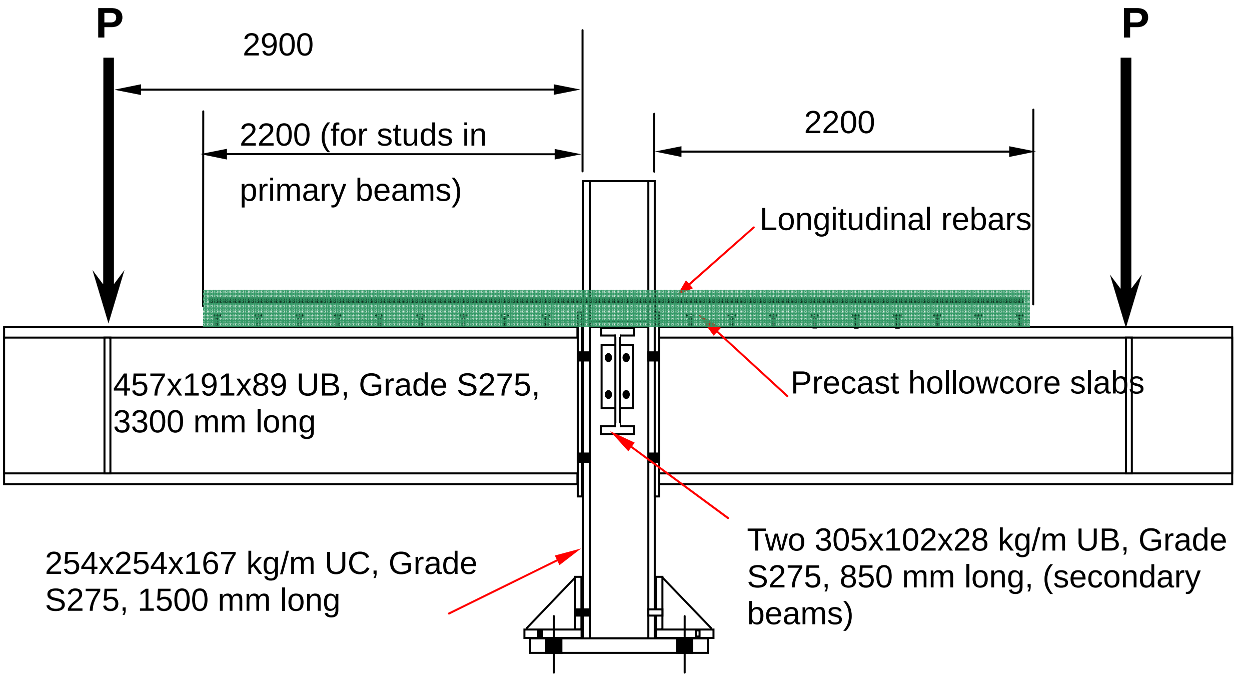

Figure 1: General arrangement of test set-up

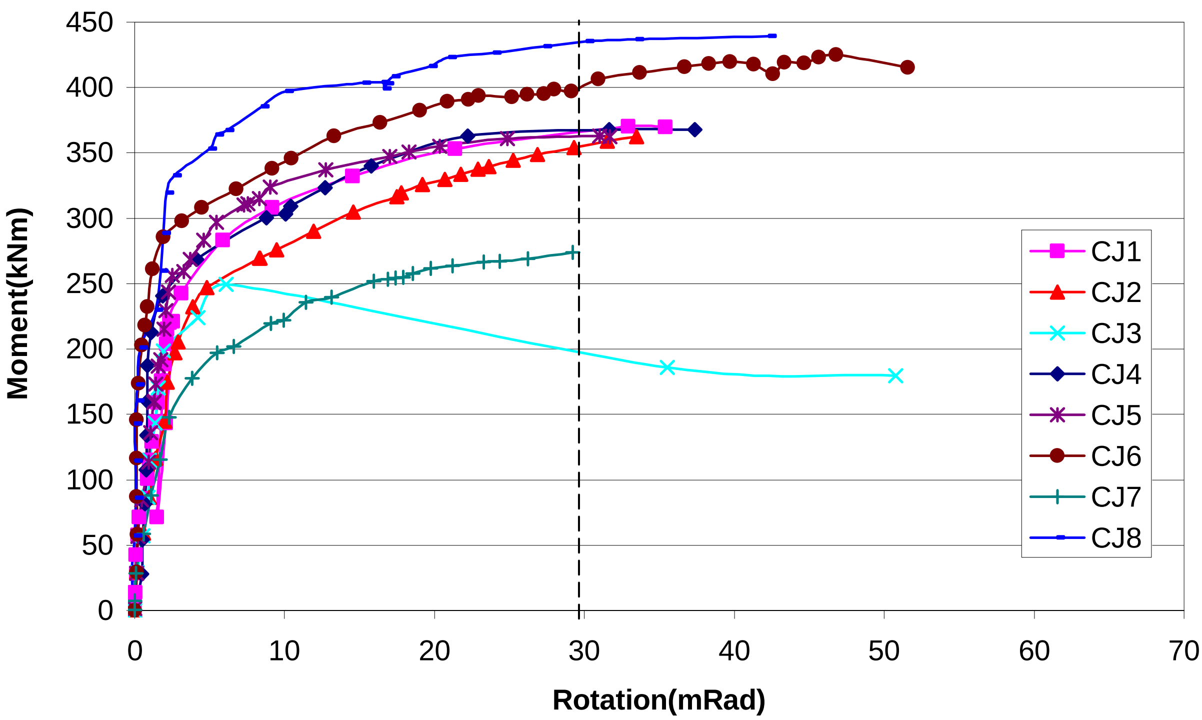

Figure 2: Moment vs. rotation curves

TABLE 1: TESTS RESULT

Reference	**CJ1**	**CJ2**	**CJ3**	**CJ4**	**CJ5**	**CJ6**	**CJ7**	**CJ8**
Moment capacity (kNm)	370	363	250	368	363	425	274	439
Rotation capacity (mRad)	35.4	33.5	6.1	37.4	31.7	46.8	30	42.3
Long. reinf. – yield (kN)	326	326	326	326	326	424	212	424
Long. reinf. – Ultimate (kN)	387	387	387	387	387	486	243	486
Shear connector capacity (kN)	896	512	256	384	384	512	256	512
Max. strain in long. reinf. ($\mu\varepsilon$)	26,000	23,000	2,031	16,000	13,706	26,000	23,000	23,000
Maximum end slip (mm)	0.34	0.8	5.8	3.5	3.5	0.84	0.4	1.6
Failure mode	RF	RF	CF & SF	CF	CF	RF	RF	RF

RF – reinforcement fracture; CF – connector fracture; SF – slab shear failure

All tests except Test CJ3 failed in a ductile manner with beam rotation well in excess of 30 mRad with the moment capacity above 0.3 Mp of the composite beams, it can be concluded that these types of joints can provide sufficient moment and rotation capacity for plastic design. Tests CJ1, CJ2, CJ6, CJ7, and CJ8 were failed due to the fracture of longitudinal reinforcement while Tests CJ3, CJ4 and CJ5 failed by fracture of the shear connectors. No yielding or buckling to the column was observed. For all the tests conducted, no bond failure between the in-situ and the precast concrete was observed, therefore it can be concluded that the in-situ and the precast hollowcore slabs were acting compositely throughout.

FINITE ELEMENT MODEL

The moment and the rotation capacity of the joints were studied using the 3-D finite element method. Using the general-purpose finite element package ABAQUS, a 3-D finite element model was built to simulate the behaviour of semi-rigid composite connection with precast hollowcore slabs. As shown in Figure 3, the model use three-dimensional solid elements to replicates the composite joint of the actual full scale test. The boundary condition and method of loading adopted in the finite element analysis followed closely to those used in the tests. The load was applied at the end of the beam as shown in Figure 3. Material nonlinearity was included in the finite element model by specifying the stress-strain curves of the material taken from the test specimens. Comparisons of the FE model with the test results are shown in Table 2 and 3 and typical moment rotation curve is shown in Figure 4. It can be seen that the model results has good agreement with the experiment data.

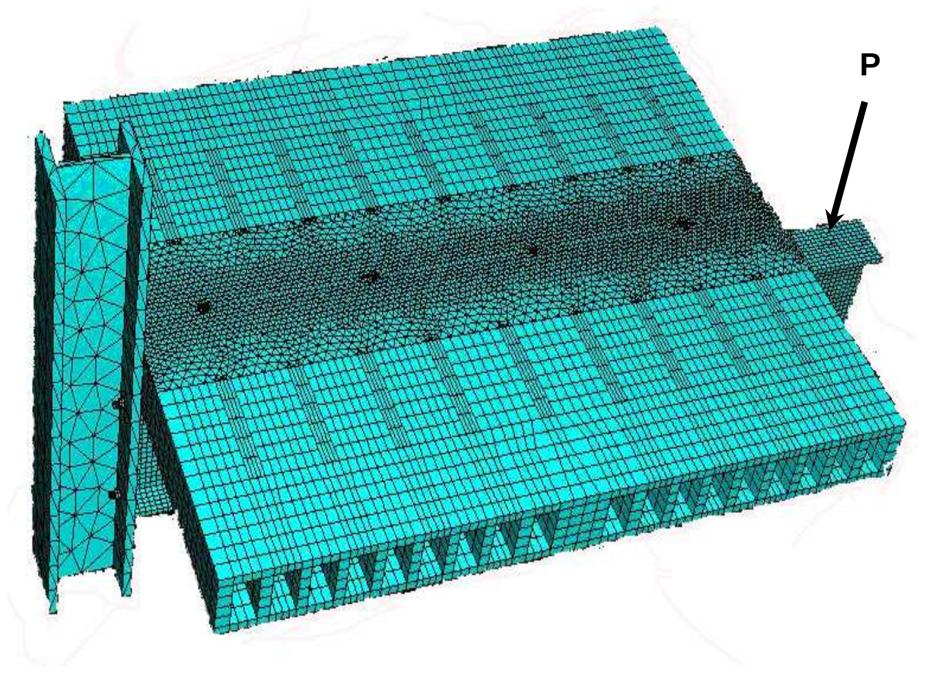

Figure 3: Finite Element Model of the Semi-Rigid Composite Joint

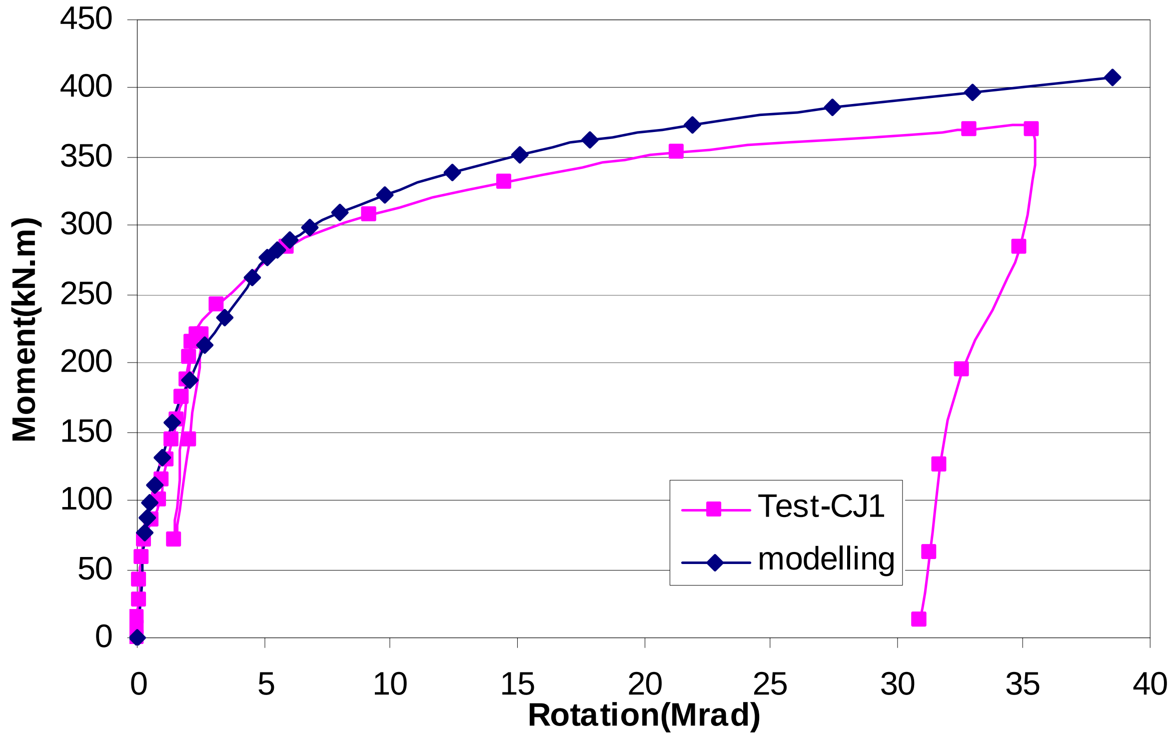

Figure 4: Comparison of Test CJ1 and the FE model

TABLE 2: COMPARISON OF MOMENT CAPACITY

Reference	CJ1	CJ2	CJ3	CJ4	CJ5	CJ6	CJ7	CJ8
Test result (kNm)	370	363	250	368	363	425	274	439
FE Model (kNm)	407	402.9	253.7	383	398	437.6	292	475

TABLE 3: COMPARISON OF ROTATION CAPACITY

Reference	CJ1	CJ2	CJ3	CJ4	CJ5	CJ6	CJ7	CJ8
Test result (mRad)	35.4	33.5	6.1	37.4	31.7	46.8	30	42.3
FE Model (mRad)	38.5	33.9	11.5	36	36.1	51.4	31.5	49.7

ANALYTICAL MODEL

Base on the full scale tests and parametric studies by Fu et al. (2007), an analytical model to calculate the moment and rotation capacity for this type of connection is derived. Figure 5 describes the force transfer mechanism for the composite joint with flush end-plates composite connection.

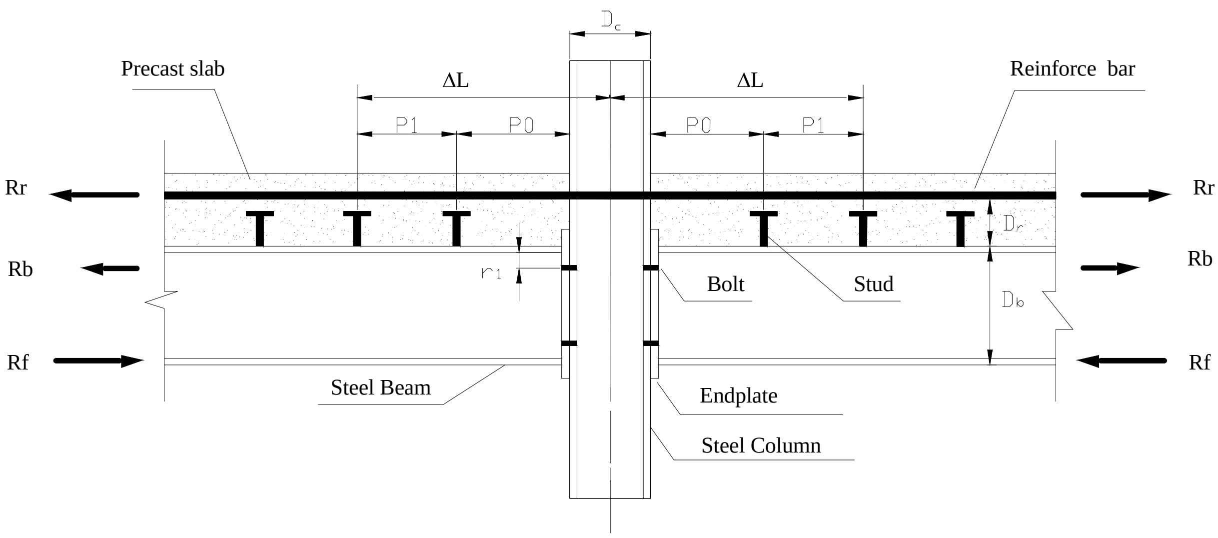

Figure 5: Load Transfer Mechanism for the Composite Joint

Tests result showed that the compression force transfer through direct bearing of the bottom flange of the beam. Due to strain hardening, it is possible for the bottom flange to resist compressive stresses of up to 1.2 times the yield strength. The tensile strength of the concrete is ignored as the tensile force of the slabs is relatively small, only the tensile strength of the longitudinal reinforcing bars was considered. A method to predict the moment capacity for this type of semi-rigid connection is proposed.

The proposed method assumes that:

For $R_f \geq R_b + R_r$,

R_f= compressive resistance of the bottom flange of the steel beam,
R_r= tensile strength of the longitudinal reinforcement,
R_b= effective tensile resistance of the bolt group.

The moment resistance of the composite connection, $M_{c,Rd}$

$$M_{c,Rd} = R_r(D_b + D_r - 0.5t_f) + R_b(D_b - r_1 - 0.5t_f) \qquad (1)$$

D_b = the depth of the beam;
r_1 = the distance of the first row of bolts below the top of the beam;
D_r = the distance of the reinforcement above the top of the beam;
t_f = the flange thickness of the steel beam.

For $R_f < R_b + R_r$,

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

t_w = the web thickness;
p_y = the design strength of steel section.
The moment resistance of the composite connection, $M_{c,Rd}$

$$M_{c,Rd} = R_r(D_b + D_r - 0.5t_f) + R_b(D_b - r_1 - 0.5t_f) - R_w\frac{y_c}{2} \qquad (2)$$

$$R_w = y_c t_w p_y$$

The comparison of the test results and the results from the proposed method above is shown in Table 4. The results showed that the moment capacity of the semi-rigid composite connections is dependent to the strength and the ability to mobilize the longitudinal reinforcing bars. The influential factor for their mobilization is depending on the degree of the shear connection, which is determined by the number and the capacity of the shear studs in the hogging moment region.

TABLE 4: COMPARISON OF MOMENT CAPACITY

Reference	**CJ1**	**CJ2**	**CJ3**	**CJ4**	**CJ5**	**CJ6**	**CJ7**	**CJ8**
Test result (kNm)	370	363	250	368	363	425	274	439
Analytical model (kNm)	365.8	365.8	284.5	365.0	366.6	422.3	274.0	446.7

The available rotation capacity 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. An analytical method is proposed for predicting the rotation capacity for this form of composite joints:

$$\phi_j = \frac{\Delta_r}{D_b + D_r} + \frac{S}{D_b} \qquad (3)$$

In order to determine the elongation of the longitudinal steel bar, the effective deformation length of the longitudinal rebar, ΔL need to be determined first. From the tests result, it showed that the yielding of the longitudinal reinforcement only occurred at the distance between the centre line of the column and the second stud if the distance between the first stud and the column flange is less than 900 mm. The strain in the other part of the rebar is relatively small. Hence, the effective deformation length is assumed to be P_0+P_1+ D_c/2 as shown in Figure 5 until the ultimate stress is reached. This demonstrates that position of the headed studs played an important role in the rotation capacity of the composite connections.

The deformation capacity is influenced not only by the effective deformation length and ductility of the reinforcing bars in the region near the joint but also by tension stiffening. When the concrete is crack and yielding of the reinforcement occurred, the effect of tension stiffening increases significantly. This is because the bond between concrete and reinforcement lowers the strain away from the cracks as shown in Figure 6.

The stress-strain relationship for embedded reinforcement provides a higher stiffness and rupture at a lower ductility than the reinforcement alone. The ultimate mean strain, ε_{smu} in embedded reinforcement, with the tension stiffening effect taken into account, which will arise from the crack over the transmission length, L_t which the bond has broken down.

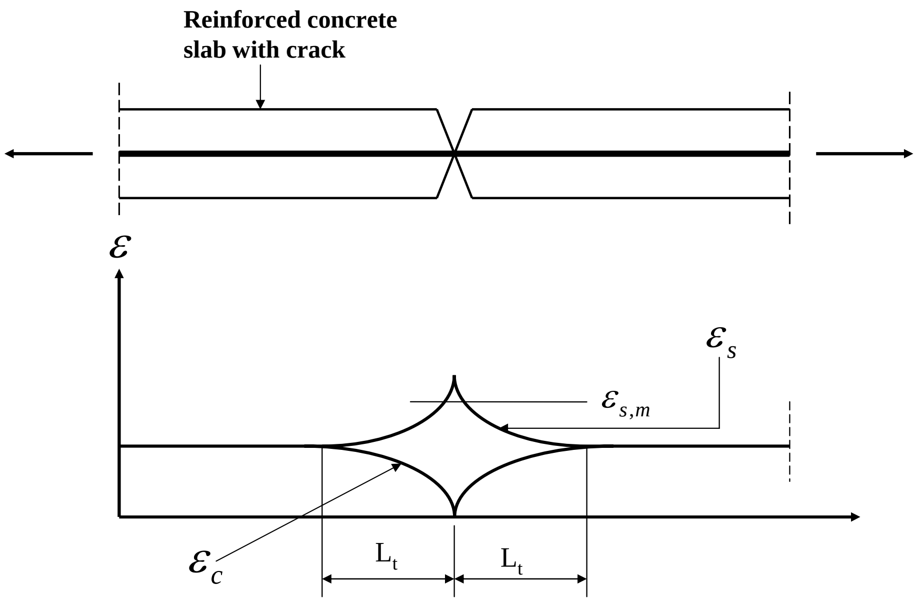

Figure 6: Strain in Cracked Reinforced Concrete

The ultimate mean strain, ε_{smu}

$$\varepsilon_{smu} = \frac{1}{2}(\varepsilon_{su} + \varepsilon_{sy}) \quad (4)$$

ε_{su} is the ultimate strain of the reinforcement

ε_{sy} is the yield strain of the reinforcement

The transmission length, L_t

$$L_t = \frac{k_c f_{ctm}\phi}{4\tau_{sm}\rho} \quad (5)$$

f_{ctm} is the tensile strength of concrete

ρ is the longitudinal reinforcement ratio where $\rho = \dfrac{A_s}{A_c}$

A_s is the area of the longitudinal bar
A_c is the area of the effective concrete slab, for composite precast hollowcore slabs, the region of the in-situ concrete infill is used.
k_c is a coefficient that allows for the self-equilibrating stresses and the stress distribution in the slab prior to cracking where $k_c = \dfrac{1}{1+\dfrac{h_{cs}}{2z_0}}$
h_{cs} is the thickness of the precast slab
z_o 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 rebars
τ_{sm} is the average bond stress along the transmission length and is taken as 1.8 f_{ctm}

For full shear connection, the formula for calculating the elongation of the longitudinal rebar, Δ_r is defined as follows:

For $\rho \leq 1.0$ %,

$$\Delta_r = \left(\frac{D_c}{2} + 2L_t\right) \times \varepsilon_{smu} \tag{6}$$

For $\rho > 1.0$ % and $P_0 \leq L_t$

$$\Delta_r = \left(\frac{D_c}{2} + 2L_t\right) \times \varepsilon_{smu} + (P_1 - L_t) \times \varepsilon_y \tag{7}$$

For $\rho > 1.0$ % and $P_0 > L_t$

$$\Delta_r = \left(\frac{D_c}{2} + P_0 + L_t\right) \times \varepsilon_{smu} + (P_1 - L_t) \times \varepsilon_y \tag{8}$$

For partial shear connection, the ultimate mean strain, ε_{smu} is taken at the on set of strain hardening if yielding of the longitudinal reinforcement can be achieved. Otherwise, ε_{smu} is taken as the yield strain, ε_y. The stress strain curve of the longitudinal rebar is shown in Figure 7.

The slip of the shear connectors can be taken directly from the standard push test. Figure 8 shows the load vs. slip curve of the 19mm headed shear stud. The correspondence shear force of the stud is taken as

$$F_s = \frac{A_s f_y}{n} \tag{9}$$

$A_s f_y$ is the maximum yield strength of the longitudinal reinforcement;
n is the total numbers of shear connector.

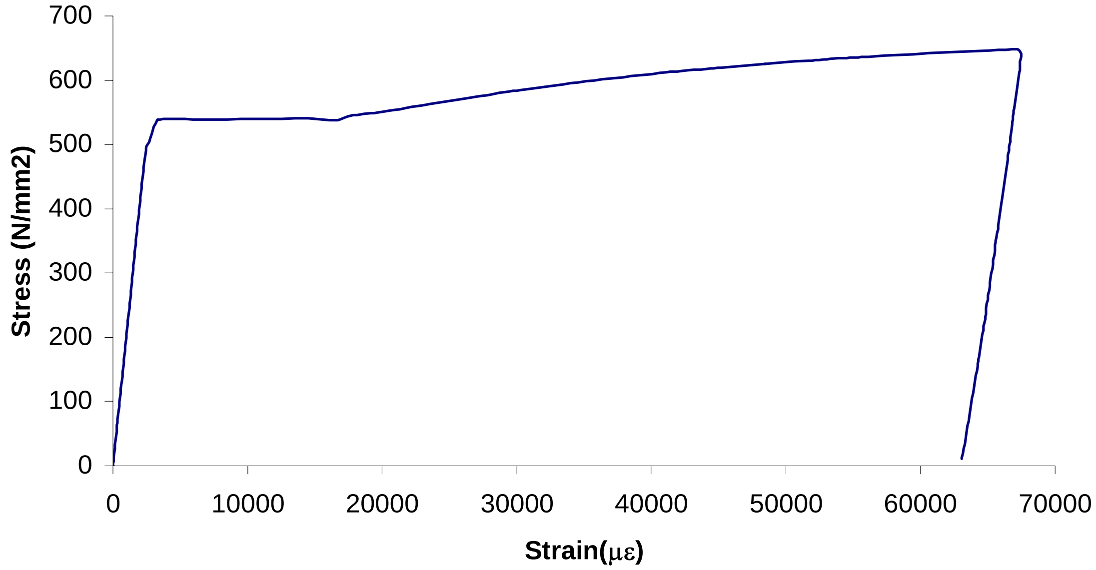

Figure 7: Stress vs. Strain Curve of the Reinforcing Bar

The comparison of the test results and the results from the analytical method for rotation capacity above is shown in Table 5. Results showed a reasonable agreement between the test results and the analytical method with the exception of CJ3 which is due to premature failure of the slabs.

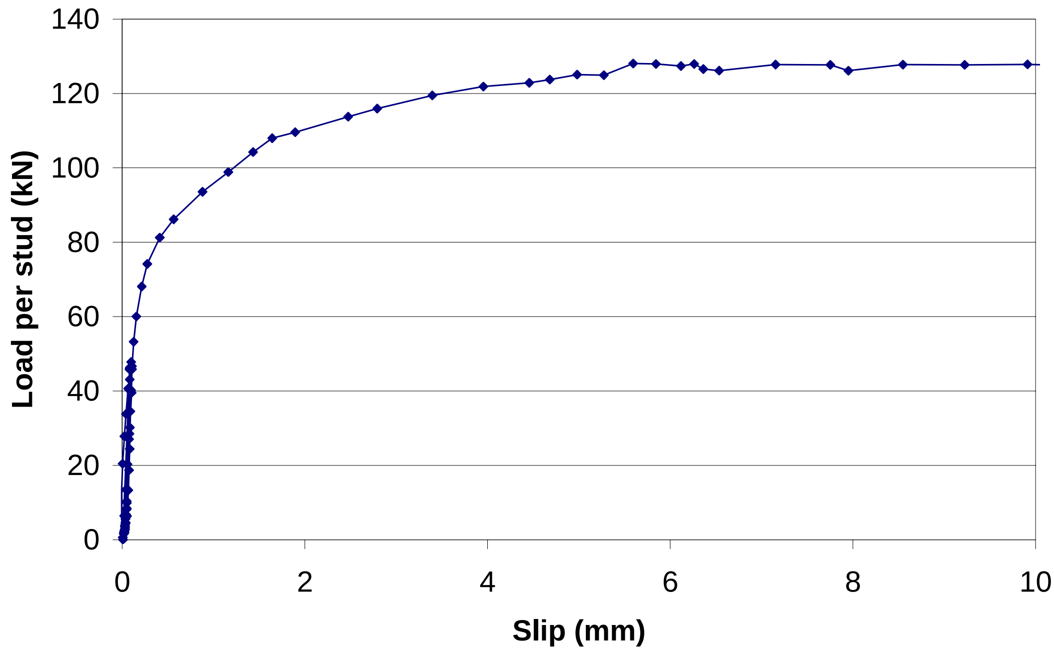

Figure 8: Load vs. Slip of 19mm Headed Shear Stud

TABLE 5: COMPARISON OF ROTATION CAPACITY

Reference	**CJ1**	**CJ2**	**CJ3**	**CJ4**	**CJ5**	**CJ6**	**CJ7**	**CJ8**
Test result (mRad)	35.4	33.5	6.1	37.4	31.7	46.8	30	42.3
Analytical method (mRad)	29.1	31.3	18.7	30.0	28.7	43.4	27.9	53.6

CONCLUSIONS

Tests program designed to study the moment and rotation capacity of the composite joints with precast hollowcore slabs has been described as well as the FE model built to investigate the structural behaviour of the composite joints. The comparison with the test results showed that the proposed model can accurately represent the overall behaviour of the composite joints. Based on the parametric studies and experimental results, an analytical method to calculate the moment and rotation capacity of the composite joints with precast hollowcore slabs were derived and good agreement has been obtained when compare with the tests results. The results show that the proposed analytical method is adequate to use for designing this form of composite joints.

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