

DEVELOPMENT OF NOVEL CONNECTION METHODS BETWEEN PRECAST CONCRETE PANELS

Jean Paul Vella¹, Robert Vollum¹, Andrew Jackson² and John Stehle³

¹Department of Civil and Environmental Engineering, Imperial College London, Greater London, SW7 2AZ, United Kingdom

²Laing O'Rourke, Dartford, Kent, DA2 6SN, United Kingdom

³Matter Up Ltd., Greenwich, London, United Kingdom

ABSTRACT

Laing O'Rourke (LOR) have developed and patented a novel form of connection between precast concrete panels utilising lapped headed bars, known as the "E6 joint". Headed reinforcement bars protrude from adjacent faces of precast lightweight concrete panels, such that a reduced lap length can be used in the cast-in-situ joint region, thereby decreasing the joint width, when compared to straight bar lap splices. Confining reinforcement in the form of vertical shear studs and transverse bars is also installed in the joint. The aim of this research, funded by LOR, is to study the mechanical behaviour of this novel form of connection, at the serviceability and ultimate limit states, by means of numerical analysis, experimental testing and reliability analysis. The experimental results are used to validate a three dimensional non-linear finite element (NLFEA) model of the joint. A strut-and-tie model (STM) developed by LOR and reviewed by Arup, is used to determine the E6 design joint strength. Analysis shows that the STM gives conservative results compared with numerical and experimental results whilst not fully capturing the observed joint behaviour. More refined alternative design approaches will be considered to overcome these shortcomings. Improved understanding of joint behaviour will enable the joint design to be optimised and facilitate the development of an improved simplified design method. Furthermore, a more widespread use of this system would lead to improvements in buildability, sustainability and health and safety in construction of concrete structures.

Keywords: DfMA, Precast concrete, Headed reinforcement, Lap length, Strut-and-tie.

1. Introduction

Laing O'Rourke developed the "E6 floor system" as part of their drive towards 'Design for Manufacture and Assembly' (DfMA), by which products are designed for easier and more efficient manufacturing and assembly methods to allow fast, reliable construction using high-quality prefabricated elements. In the E6 floor system, precast concrete floor planks are connected to together using a novel jointing system to form a continuous concrete floor that compares similarly to a traditional monolithic in-situ concrete slab. The system allows a high proportion of pre-manufactured elements, achieving higher quality when compared to traditional cast-in-situ concrete structures, and eliminating or reducing many in-situ trades. The use of precast elements separated by narrow joints allows a very efficient construction process in which the planks are temporarily connected by steel brackets until the joint concrete has cured, eliminating the need for traditional propping below the slabs and facilitating an extremely rapid construction programme for the structure, installation of other manufactured elements, and any follow-on trades.

In order to achieve the required high-strength connection within a narrow lap zone, the E6 joint uses headed reinforcement bars protruding from opposite faces of adjacent precast planks. Additional confining reinforcement in the form of vertical shear studs and transverse bars is also installed in the joint (Fig. 1).

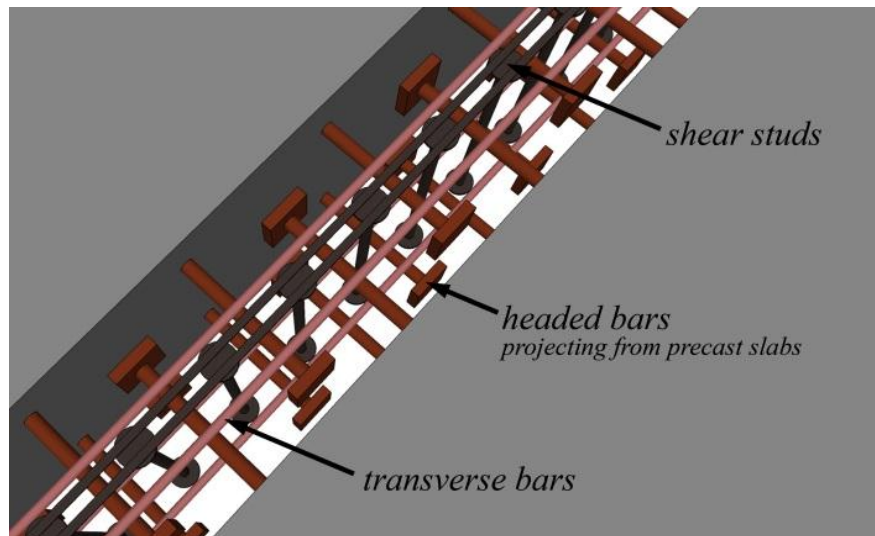


Fig. 1. E6 joint components

Before the start of this project, LOR had already conducted a suite of tests on the joint system at BRE, including small scale joint tension tests, slab bending tests and a full scale slab test on a typical structure bay. These tests demonstrated the feasibility of constructing floors with the E6 joint system and showed that the slab performance was similar to that of a monolithic slab. LOR developed a STM for the E6 joint which was subsequently reviewed by Arup. The model gives safe predictions for the joint strength, but is very conservative in some cases. However a deeper understanding of the behaviour of the E6 joint is required to enable the design model to be improved. This will also enable this system to be optimised and increase confidence in its application.

The aim of this research is therefore to study the mechanical behaviour of this connection, at the serviceability and ultimate limit states, by means of numerical analysis, experimental testing and reliability analysis. The experimental results are used to validate a three dimensional NLFEA model of the joint which will be used subsequently to refine the STM and calibrate alternative design approaches better able to capture the real behaviour of the joint.

2. Literature

A number of studies have been carried out on similar joints using lapped headed bars or U-bars, mainly with the aim for use in precast concrete bridge structures. Thompson et al. (2006a) developed an analytical model combining components of head bearing and bond to determine the capacity of headed bar laps, by calibration with results from tests on slab specimens with lap splices anchored by headed bars (Thompson et al., 2006b) and on simply supported beams with headed bars (Thompson et al., 2005). This empirical model had several limitations related to the specific parameters investigated in the study, such as minimum lap length for applicability of the model and no consideration of the beneficial effect achieved when including transverse reinforcement. The effect of transverse reinforcement was later investigated by Chun (2015), who found it to improve the anchorage of headed bars significantly. Most headed bars used in these studies had small head bearing areas, meaning that some component of bond was required to anchor the full yield strength of the headed bar. Similar tests were carried out by Li et al. (2010) using headed bars with large heads capable of anchoring the full yield strength of the bar at the head. They recommended a minimum lap length of 152mm with the same maximum spacing of headed bars to achieve satisfactory strength and ductility of the joint.

Other studies have been done on similar connections, but using lapped U-bars (Dragosavic et al., 1975, Gordon and May, 2006, He et al., 2013, Joergensen and Hoang, 2013, Ma et al., 2012, Ong et al., 2005). Ong et al., Ma et al. and He et al. developed strut-and-tie models to predict joint strengths investigated in their studies, while Joergensen and Hoang used an upper bound plasticity based model.

Most models and expressions derived from these studies are empirical and relate to specimen configurations tested in the particular study. It is therefore difficult to apply them confidently in generic applications, however, it is important to consider the relevance of the parameters included in these studies when developing a simplified model for the headed bar joint in the current research.

3. Experimental Tests

3.1 Specimen details

A series of tests were carried out in the Structures Laboratory of the Department of Civil and Environmental Engineering at Imperial College London to determine the effect of several variables on the E6 joint strength. These included small scale joint tests in direct tension and slab tests in bending. Fig. 2 shows a typical small scale joint specimen, intended to represent a joint in the tensile zone of a slab under bending. The smallest possible joint configuration, with one headed bar lapped by two headed bars in the opposite direction, was chosen to reduce cost while still giving a representative behaviour of the joint.

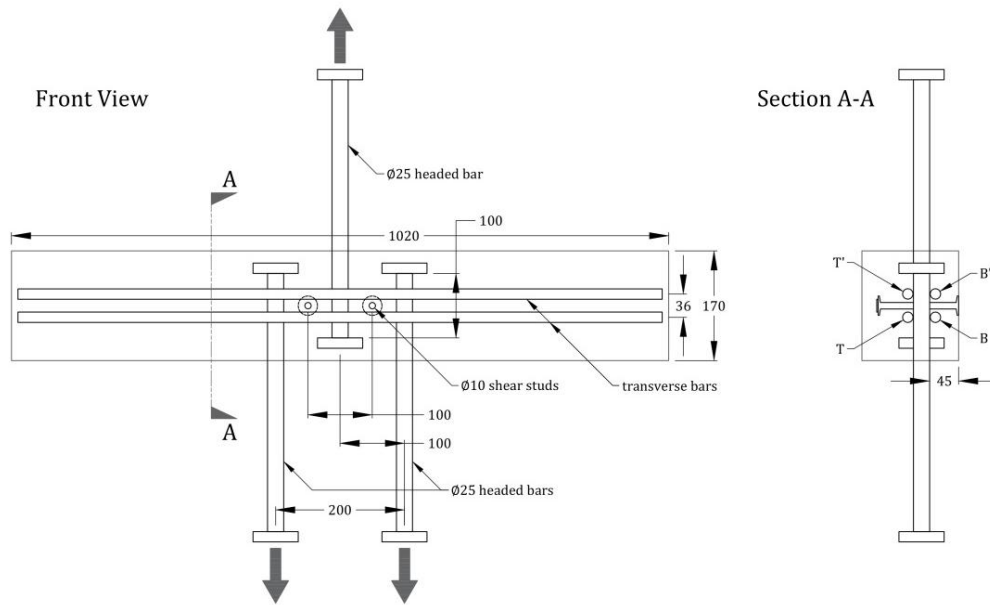


Fig. 2. E6 joint tension specimen

Headed bars were 25mm in diameter with standard 70mm square friction welded heads, giving a ratio of net head area to bar area (A_{nh}/A_b) of 9. This enables the full yield strength of the bar to be anchored at the head, without any contribution from bond. Transverse bars and shear studs were provided in the test specimens, as would be the case in joints between precast units. Table 1 summarises six small scale joint tests which demonstrate some key findings from this study, and are discussed in this article. The test IDs describe the specimens as follows:

For example, G3-28-4H20:TT'BB'-S

“ G3 ” – Test group

“ 28 ” – Concrete cylinder strength

“ 4H20 ” – Number and diameter of transverse bars

“ TT'BB' ” – Position of transverse bars as indicated in Figure 2

“ S ” – Shear studs included

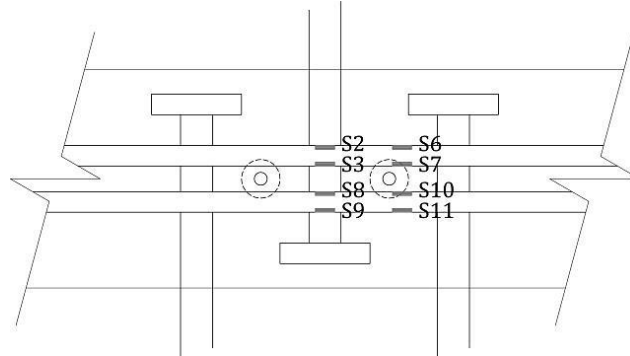
Table 1. Specimen details

Test ID	$f_{c,cyl}$ (MPa)	f_{ctm} (MPa)	Headed bar dia. (mm)	No. of transverse bars	Transverse bar dia. (mm)	Position of transverse bars*	Shear stud dia. (mm)
G1-40-2H16:TT'-S	40.4	3.6	25	2	16	T, T'	10
G1-26-2H20:TT'-S	25.6	2.4	25	2	20	T, T'	10
G1-40-2H20:TT'-S	40.4	3.6	25	2	20	T, T'	10
G1-40-2H20:TT'	40.4	3.6	25	2	20	T, T'	/
G3-28-2H20:T'B'-S	27.7	2.7	25	2	20	T', B'	10
G3-28-4H20:TT'BB'-S	27.7	2.7	25	4	20	T, T', B, B'	10
$f_{c,cyl}$ – Measured concrete cylinder strength at time of testing f_{ctm} – Measured concrete tensile splitting strength at time of testing * As indicated in Fig. 2.							

3.2 Instrumentation

A servo-hydraulic test rig was used to load the specimens in a uniaxial tension under displacement control. A load rate of 0.2mm/min was chosen, and the outputs gathered from the test rig were the load and platen-to-platen displacement. Additionally, linear variable displacement transducers (LVDTs) were placed close to the centre of the specimen to capture displacements and rotations.

Several strain gauges were fixed to the reinforcement inside the concrete. Fig. 3 shows the arrangement of the main strain gauges used with the aim to capture bending and axial forces in the transverse reinforcement.

**Fig. 3.** Strain gauge positions on transverse bars

A digital image correlation (DIC) system was also set up during testing. This consisted of two high resolution cameras capturing images of the surface of the specimen in stereo mode every 3 seconds. A random speckle pattern was sprayed onto the surface of the specimen such that the StrianMaster system (LaVision, 2014) could track the movement of the pattern on the specimen by comparing images at different load stages. Since two cameras were used, it was possible to capture 3D displacements, surface strains and crack propagation during the test.

3.3 Results

3.3.1 Load-displacement

Fig. 4 shows load-displacement plots of the specimens listed in Table 1. From these results, several observations can be made: Comparing tests G1-40-2H16:TT'-S and G1-40-2H20:TT'-S, one can see that by increasing the transverse bar size from 16mm to 20mm, very little increase in joint strength is achieved, while still achieving similar stiffness and post-peak response. The results from tests G1-26-2H20:TT'-S and G1-40-2H20:TT'-S show that increasing the concrete strength from 26MPa to 40MPa increases joint strength significantly, although the failure is much more brittle with the higher concrete strength. The absence of the shear stud in test G2-40-2H20:TT' results in a lower joint

strength and a more brittle failure when compared to test G1-40-2H20:TT'-S. The effect of changing the arrangement of the transverse bars can be observed by comparison of tests G1-26-2H20:TT'-S, G3-28-2H20:T'B'-S and G3-28-4H20:TT'BB'-S, which have similar concrete strengths. By changing the position of the transverse bars from TT' to T'B', significant increase in strength and ductility is achieved, whilst also increasing the initial stiffness similarly to joints with a higher concrete strength. By providing four bars instead of two, the strength and ductility of the joint is further increased, with the headed bar going well into the hardening branch (not fully shown in Fig. 4).

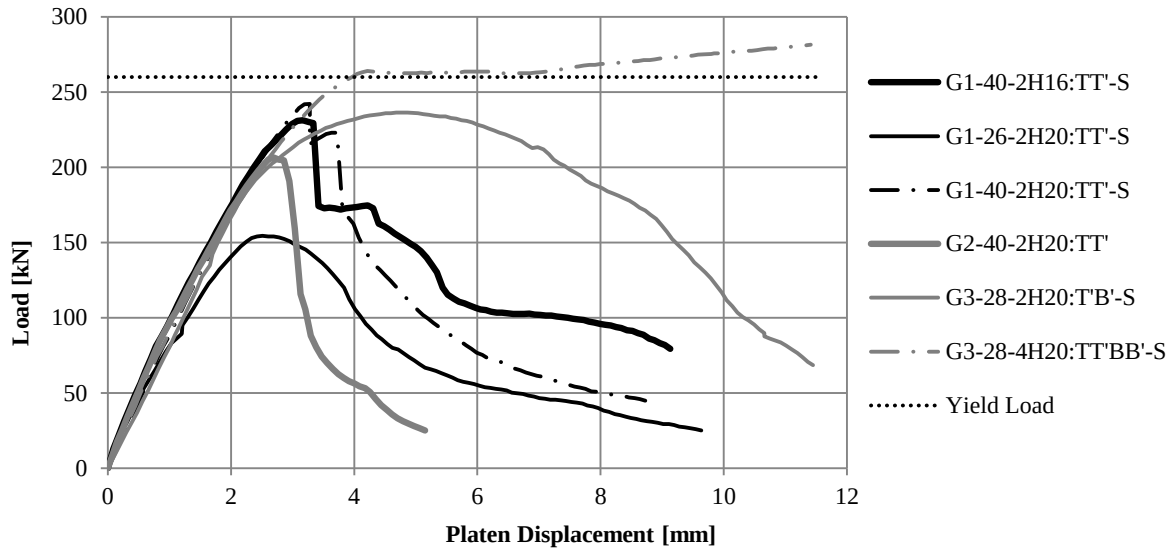


Fig. 4. Load-displacement comparison

3.3.2 Reinforcement strains

Fig. 5 shows transverse reinforcement strains from test G1-40-2H20:TT'-S, which are typical. Looking at strain gauges 2, 3, 8 and 9, one can clearly notice that the transverse bars undergo a combination of bending and axial force. This occurs as the opposing headed bars transfer bearing stresses through the concrete onto the transverse bars and, due to the proximity of the reinforcement, bending action is induced in the transverse bars.

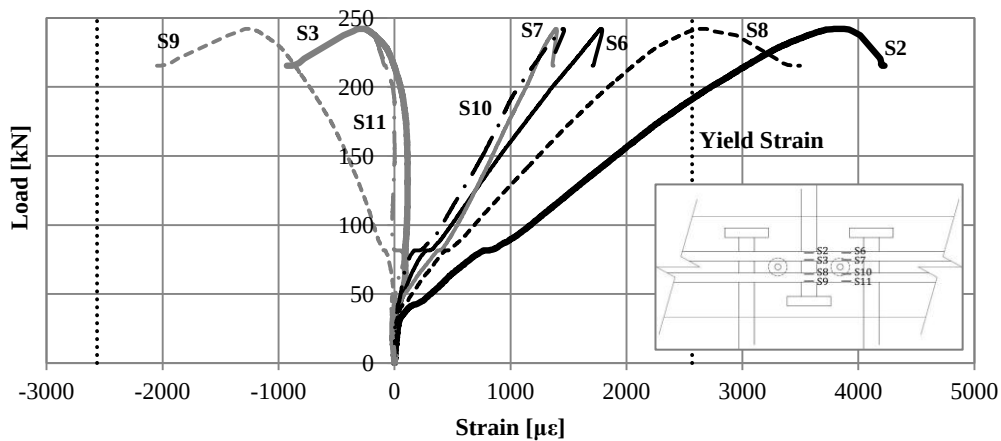


Fig. 5. Test G1-40-2H20:TT'-S transverse bar strains

3.3.3 Crack propagation

The crack pattern at the front face of test specimen G1-40-2H20:TT'-S at failure can be observed from Fig. 6. Crack 1 starts forming from the top surface at a load of around 55kN, due to some bending action induced in the specimen and also bond failure of the central headed bar. Crack 2 becomes visible at a load of 110kN, starting near the head of the central headed bar and spreading upwards,

followed by cracks 3 which propagate from crack 2, spreading downwards. At a load of about 220kN, crack 4 appears first near the head of the right headed bar and propagates quickly towards the head of the central headed bar upon further loading. This crack widens, along with the formation of crack 5 at the ultimate load of 242kN.

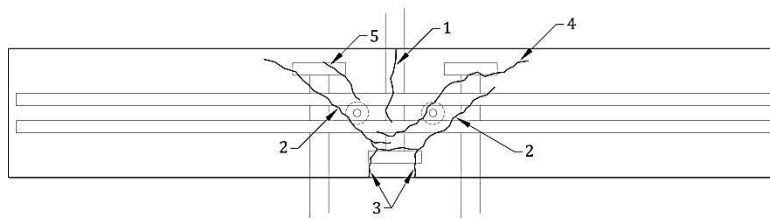


Fig. 6. Test G1-40-2H20:TT'-S crack pattern

4. Numerical Modelling

Modelling was done with the non-linear finite element package ATENA-GiD (Cervenka et al., 2014). The program GiD was used as a pre-processor for geometrical modelling and specifying the input data required for the analysis. The analysis was then run using ATENA Studio (Benes et al., 2015), in which post-processing actions were also carried out.

Taking advantage of symmetry, only half of the specimen was modelled (Fig. 7). All components were modelled with quadratic 20-noded hexahedral elements, except for the shear stud, for which a 1D truss element was used. Perfect bond was considered between all reinforcement and concrete elements as this would increase computational efficiency while still reflecting closely the real behaviour in the case of ribbed bars (Pryl and Cervenka, 2014). Static analysis was performed with a prescribed displacement of 0.02mm per load step applied to the central headed bar, while the end of the supporting headed bar was fixed in all directions.

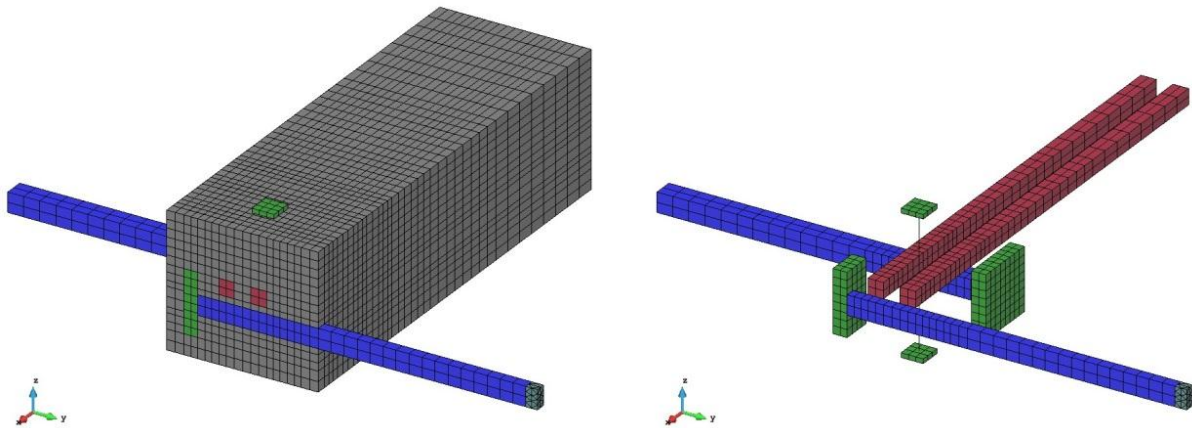


Fig. 7. Specimen 1.4 full model (left) and steel only (right)

The concrete material used in these analyses was a fracture-plastic model available in ATENA (Cervenka et al., 2013). This model combines constitutive models for tensile and compressive behaviour. The tensile fracture model employs the smeared crack approach, with the fixed crack model chosen for this study. The compressive model has a non-linear branch under loading before reaching the peak stress, followed by a linear branch upon further increase in strain. A bilinear elasto-plastic constitutive model with hardening was used for all reinforcement elements.

5. Strut-and-Tie Model

The shaded areas in fig. 8 show the geometry of the STM being used to predict the joint strength. Struts are assumed to be prismatic, with their inclination limited to 2:1. Due to the confinement provided by reinforcement, nodes are assumed to be under triaxial compression, and therefore checks

on the struts are sufficient. Where shear studs are provided, a confined concrete compressive strength according to BS EN 1992-1-1:2004 (2004) is considered, as given in Equation 1. The confinement pressure, σ_2 is considered to be given by the yield strength of the shear stud over the hatched area in fig. 8.

$$f_{ck,c} = \min(f_{ck} + 5\sigma_2, 1.125f_{ck} + 2.5\sigma_2) \quad (1)$$

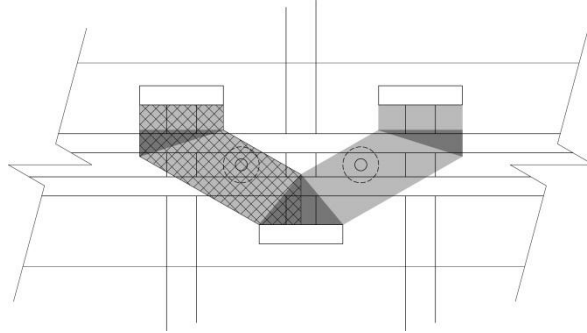


Fig. 8. Strut-and-tie model for tension

In addition to the concrete strut, the forces in the headed bars and the transverse bars are checked to ensure that their yield strength is not exceeded.

6. Discussion

Fig. 9 shows a comparison of test results with predictions from NLFEA and the unfactored STM. The results are presented in ascending order according to the maximum loads achieved in the tests, for ease of interpretation. One can immediately appreciate that the numerical results are much closer to, and follow the trend of the test data better than the STM. On average, the results presented show that the STM predictions are 32% lower than test results, whereas the NLFEA predictions are 1% lower than test results.

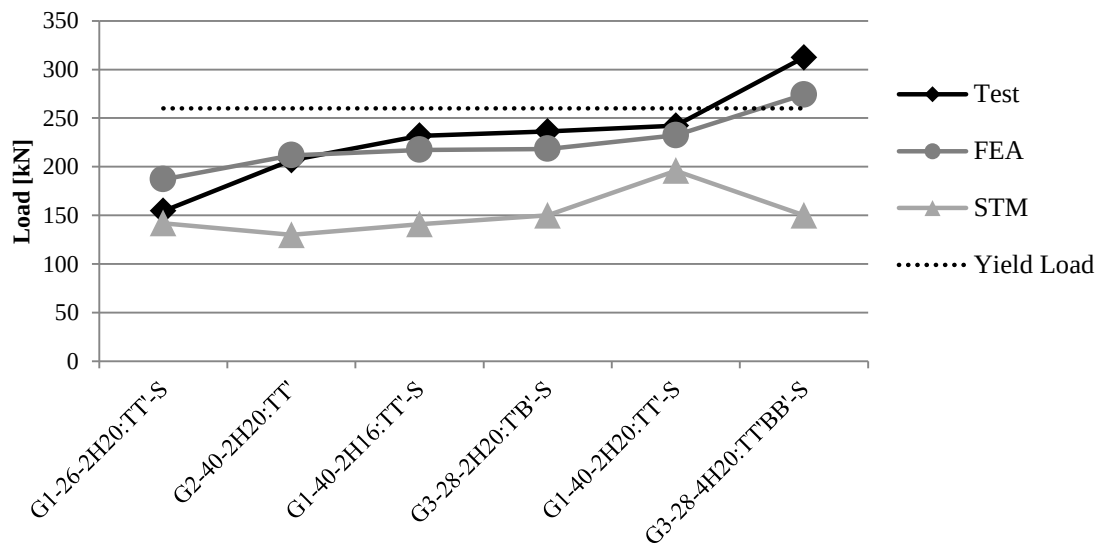


Fig. 9. Comparison of results

The main reason for this is that the STM does not capture the correct failure mode in some cases. Considering test 1.2 as an example, the STM predicts 39% lower strength than achieved in the test. In this case, the STM does not account for the contribution of the bottom transverse bar as shown in fig. 8, and suggests that axial tensile yielding of the top transverse bar is the limiting failure mechanism, while the tests and NLFEA exhibit partial yielding of the transverse bar in bending along with concrete crushing. The strain measurements from all the tests in fact show a combination of tension and bending in the transverse bars, indicating that dowel action is an important factor contributing to

joint strength. Changing the position of transverse bars from TT' to T'B' significantly increased strength and ductility particularly for specimens with low concrete strengths.

The crack pattern shown in fig. 6 does suggest a strut-and-tie type load path, although with a slightly different geometry to that suggested in fig. 8. However, STMs are typically used in much larger scale elements such as deep beams, and certain assumptions considered in these structures, such as neglect of dowel action, are conservative for small scale joints. The test results show that, although the STM is conservative and safe to use in most cases, it does not fully capture the real behaviour of the joint. More refined alternative design approaches will be considered to overcome these shortcomings.

7. Conclusions

This paper summarises research being carried out at Imperial College London on a novel connection method between precast slabs developed and patented by Laing O'Rourke, known as the "E6 joint". Six results from a suite of tests carried out are presented. The test results show that the strut and tie method is generally conservative and safe to use, but they allow more accurate understanding of the behaviour of the joint. The key observations from these tests are:

- a) Increasing the transverse bar size from 16mm to 20mm results in a small increase in joint strength
- b) A significant increase in joint strength is achieved by increasing concrete strength
- c) The shear studs increase the joint strength and ductility
- d) By providing transverse bars at the top and bottom of the main headed bars, a significant increase in joint strength and ductility is achieved, when compared to bars placed only on top.

These observations will be used to inform alternative design approaches to the STM. Improved understanding of joint behaviour and confidence in the design model will encourage designers to apply it in their structures. A more widespread use of this system would also lead to improvements in buildability, sustainability and health and safety of concrete structures, including; reduced construction time, reduced material waste, high quality control and reduced on-site labour resulting in safer construction sites.

8. Acknowledgements

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