The Flexural Behaviour of High-Strength Steel Beams Encased with Engineered Cementitious Composites

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The use of ECC in concrete encased high-strength steel beams promises improvements in performance in bending. In particular, the ductility of ECC may help to maintain the integrity of the casing around the top flange of a slender I-section and so prevent local buckling until the flanges reach yield strain, thereby enabling the full strength advantage of high-strength steel to be utilised. Four beams were fabricated and tested under four-point loading. Furthermore, models were made of each beam in Ahaqus and validated against the tests’ load-strain curves. Fully encased beams with ECC at the top flange showed about an 11% increase in failure strength compared to one with normal concrete, while a partially encased beam failed through lateral torsional buckling. Finite element modelling replicated the load-strain curves and showed that the steel in the ECC-encased beams reaches yield strength prior to case failure, confirming the mechanism posited.

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Nomenclature

Abbreviations:
ECC = Engineered Cementitious Composite
FE = Finite Element

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I. Introduction

Two adverse phenomena may degrade the performance of steel I-section beams under bending (or flexure). The compressive flange may suffer from local buckling resulting in loss of strength before the average axial stress in the section reaches ultimate stress, and beams may also suffer from lateral torsional buckling if not sufficiently restrained from movement out of plane (Liang, 2015). Hence, the full strength of the beam is not fully utilised.

Composite beams are a solution to these problems. Composite beams are stiffer in flexure and less likely to suffer from LTB (Rana, et al., 2018). Encasement also prevents local buckling by providing confinement to the steel plate (Liang, 2015). However, steel only retrained by concrete on one side may still buckle away from the concrete interface (Liang, 2015), an occurrence that may be caused by loss of integrity of concrete encasement through crushing in the compressive zone or shearing; again, by this point, the steel may not have reached its yield strength (Kabir, et al., 2019).

ECCs have an ability to take tension and fail in a ductile manner (Li, 2003) which can strengthen reinforced concrete beams (Yuan, et al., 2013), and a stronger bond (Lee, et al., 2016) and ability to deform an amount comparable to steel (Meng, et al., 2017) makes them a target of research in composite beam design as they can allow steel to load to yield stress and provide residual strength after steel yielding (Rana, et al., 2018).

High-strength steel can provide an improvement in strength that may outweigh economically its greater expense compared to normal constructional steel (Veljkovic & Johansson, 2004). However, a higher yield strength increases a beam’s slenderness, making local buckling more likely (Standards Australia, 2016). Meanwhile, it has been shown that composite beams using ECC and light-weight concrete as a cementitious case for steel can offer an equivalent strength for a saving in beam weight (Rana, et al., 2018). Therefore, it has been suggested that combination of these two technologies can result in further improvements with regards to weight and strength (Rana, et al., 2018) and that may be possible to prevent the local buckling of slender sections made of HSS subject to flexural loadings by retaining the confinement of the steel until it yields (Kabir, et al., 2019).

II. Aim

The aim of this project was to test and model the behaviour beams composed of a high strength steel I-section beam encased in ECC and other cementitious materials. A series of four beams with differing configurations of the cementitious case were to be subject to flexural testing under four-point loading. A finite element model of the beams was to be developed in Abaqus to compare to the experimental results.

III. Project Scope

This project was composed of two major components: experimental testing and modelling. The testing was to be carried out on a series of four composite beams. Each beam differed in the configuration of the cementitious casing in order to test the effects this has on beam behaviour. Each beam was subject to a flexural test to failure. Testing is also to be carried out of the properties of the cementitious materials used in the beams.

Using Abaqus, a finite element model is to be constructed to model the behaviour of the tested beams and of the behaviour of beams not incorporating ECC for comparison. The modelling will enable exploration of the behaviour of the bond interface between the steel and the cementitious portions of the beam.
IV. Literature Review

The following review will be divided into four parts dealing with research into the two major component materials investigated in this project, ECC and HSS, followed by research on bonding of steel to cementitious materials and finished with an overview of research into steel-concrete composite beams.

A. Engineered Cementitious Composites

Compared to normal concrete, ECC is able to sustain larger tensile strains and displays a strain-hardening behaviour (Li, 2003) due to fibre bridging limiting the opening of individual cracks and forcing new ones to form (Yuan, et al., 2013), causing the formation of small parallel cracks in place of large ones. It can deform at magnitudes compatible to steel thereby reducing separation of steel reinforcement (Li, 2003).

Material types may be varied and these have an effect on properties. PVA fibres are lower strength but higher ductility than other types (Yang & Li, 2010) and this material appears to be preferred in ECC applications for both this material aspect and lower cost (Meng, et al., 2017). Investigation into aggregates other than the common micro-silica sand (Li, 2007) have taken place, showing that size increases may have only minor degradations in structural properties and decrease shrinkage (Sahmaran, et al., 2009). Even a decrease in tensile strain capacity by 80% through use of a courser aggregate produces an ECC that can deform the same magnitude as high strength steel reinforcement while saving costs (Meng, et al., 2017). Material proportions also affect properties. Too low a water-binder ratio can eliminate strain-hardening behaviour, and there is an optimum value of the fibre content and water-binder ratio for tensile properties (Huang & Zhang, 2014).

ECC beams in bending have greater strength and can experience greater deflection than concrete beams otherwise identical in configuration (Yuan, et al., 2013), when used in composite with normal concrete, it can restrict the propagation of large cracks (Xu, et al., 2012) and decrease crack width in the normal concrete (Yuan, et al., 2014). Failure is ductile rather than brittle when beams shear (Ding, et al., 2018), and spalling does not occur after crushing (Yuan, et al., 2013).

Reinforced beams using ECC display improved shear performance compared to normal concrete (Sahmaran, et al., 2015). This improvement may be great enough that reinforced beams without stirrups will fail in bending rather than shear (Meng, et al., 2017). The strains that the reinforcement can take are also greater, possibly due to the cracking characteristics of ECC preventing formation of stress concentrations (Ding, et al., 2018). The same characteristics are also found in ECC-steel composite slabs (Mohammad, et al., 2016): ductile failure, improved strength and deflection, and also slippage between the ECC and steel occurring at higher loads than normal concrete. Self weights may also be reduced by used of ECC in such applications (Mohammad, et al., 2016).

For beams with a combination of concrete and ECC with reinforcing bars, the use of ECC on the tension side improves strength more than use on the compression side (Yuan, et al., 2013). Beams using ECC are also more resistant to the effects of corrosion than those using normal concrete (Sahmaran, et al., 2015). ECC can also enhance the effectiveness of newer reinforcing technologies such as polymer sheeting (Afefy, et al., 2015) and under cyclical loading, may allow for simplification of structural joints through removal of stirrups (Parra-Montesinos & Wight, 2000).

B. High Strength Steel

High strength steel, although its yield strength is increased compared to normal constructional steels, shows a decreased yield strain and ductility, and more brittle failure than the latter. Its strength may also be more affected by welding than normal steels (Zhao, et al., 2016).

However, it is possible to use the slenderness limits of AS 4100 to design high strength steels (690 MPa yield stress) without any special consideration, with a great conservativeness for more slender sections (Rasmussen & Hancock, 1992). On the other hand, the design methods of EC3 may not not be adequate for determining the slenderness of HSS sections without modification (Wang, et al., 2017).

There is a general trend of decreasing yield strain with increasing yield stress, and yielding behaviour of hot-rolled HSS is more similar to normal steels whereas cold rolled HSS does not show a distinct yield point, instead transitioning smoothly into the plastic stage (Wang, et al., 2017).

C. Steel–concrete bonding

The two materials in steel-concrete composites will bend different and experience slip unless there is a shear connection (Liang, 2015) such as may be provided by the steel-concrete bond. A review by Shanmugam and Lakshmi (2001) identified surface friction and confinement as the main mechanisms of steel-concrete bonding and recommend reinforcing bars to aid the latter mechanism. Hawkins (1973) demonstrates that reinforcement does increase bond strength in push-out tests, and that casting position also has an effect.

Investigation into the bonding of ECC and similar materials shows that it has generally superior bonding to steel compared to that of normal concrete (Ma, et al., 2018; Deng et al., 2018) but that this may vary by application. Bending tests with reinforcing bars indicated no effect of increased cover (Bandelt & Billington, 2016) while push out tests with flat steel indicated an effect at least to 60 mm depth (Rana, et al., 2017). The bond may be stronger.
and more resistant to slip than normal concrete (Lee, et al., 2016) or weaker for a specified slip (Rana, et al., 2017) in these respective applications.

Pull out tests with reinforcing bars indicated both an improvement with ECC and with increased cover thickness (Kanakubo & Hosaya, 2015), and push-out tests with I-section steel reported both a greater bond strength with ECC and also an increase in strength with casing depth to 110 mm (Bai, et al., 2019). An improvement in bond strength may be found for that with shallow profiled steel decking but not deeper profiles (Hossain, et al., 2016).


D. Composite Beams

Wong (1963) tested steel-concrete composite beams loaded at one-quarter span and determined that the failure is due to crushing of the concrete on the top side without yielding of the steel. Hawkins (1973) tested several different configurations of normal steel-concrete composite beams in bending and found that beams with partial encasement lose composite action due to bond failure considerable earlier than those with full encasement, and larger sections could experience steel yielding before bond failure.

Beams provided with shear connections between the concrete and steel may experience more plastic deformation before failure, experience less slip and a less likely to fail by shear than bending (Adekola, 1968). Weng et al. (2001) distinguishes shear splitting along the plane of the flanges during bending from diagonal shear and recommends shear studs to prevent this failure in the concrete casing (Weng, et al., 2002).

Yuan et al.’s (2014) modelling of normal concrete-ECC beams with steel reinforcement suggests that using an ECC layer on the tension side will increase moment capacity while use on the bottom will increase deflection capacity.

Rana et al. (2018) and Kabir et al. (2019) researched ordinary steel I-section beams encased with differing configurations of ECC and LWC, and showed that beams incorporating ECC have a greater flexural strength and deflection than those using normal concrete, and the former experience steel yielding first whereas the latter display crushing of compressive cover at initial failure.

V. Methodology

The project involved two separate components. Four physical specimens were fabricated and subject to bending tests. Two beams were built and tested at the same time: ECC25 and ECC15 first, NC100 and ECC30F second. Modelling of the beams was commenced between the testing of the first two specimens and testing of the last two, and continued through the remaining testing period and afterwards.

Detailed explanation of the methodology of each component is given in sections VI.A and VI.A

VI. Experimental Study

A. Experimental Methodology

Each of the four specimens was 3400 mm in length with a maximum width of 210 mm. Three of the beams were of 370 mm depth while ECC30F was 340 mm deep due to lacking a cementitious cover on the bottom flange.

The steel members used comprised an I-section 310 mm in height and 110 mm in width, and were fabricated from 6 mm HSS plate by welding; a section of the steel member is given in Figure 1. The steel used had a nominal yield strength of 690 MPa, and the resulting section was slender according to AS4100—1998, with non-compact flanges and a slender web. Coupon tests showed an actual yield strength of 759 MPa, for which AS4100 is still applicable (Rasmussen & Hancock, 1992).

To prepare the beam, the distortion of each of the flanges was measured to leave the less-distorted flange top-most to reduce the possibility of buckling, and eight strain gauges were attached after the surface at each attachment point was cleaned of corrosion.

The cementitious materials used were ECC, LWC and normal concrete. The ECC was based on that from Meng, Huang et al. (2017) using PVA fibre and local sand aggregate of 200 µm average and 300 µm maximum size. The LWC was based on Rana et al. (2018); the two course aggregate grades were determined by the diameter at which fifty percent of the material passed. The same condition applied to the aggregate of the normal concrete. All materials used general purpose Portland Cement as a binder; in addition, the ECC also used ASTM Class F fly ash. The mix design of each of the cementitious materials (Kabir, et al., 2019) is given in Table 1.

Cross-sections of each of the beams are given in Figure 2. All materials were hand compacted after pouring. For ECC25 and ECC15, the LWC was given one day to set before the ECC was placed. Specimens of each batch
of concrete were taken for strength testing, the values from which were used in the constitutive models. For these strength values, refer to Tables 2 to 5 in section VII.A.1.

**Table 1. Composition of ECC, LWC and normal concrete for test specimens. Mix ratios are by weight of material to weight of cement.**

<table>
<thead>
<tr>
<th></th>
<th>ECC</th>
<th>LWC</th>
<th>Normal Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>Mix ratio</td>
<td>Material</td>
<td>Mix Ratio</td>
</tr>
<tr>
<td>Cement</td>
<td>1.0</td>
<td>Cement</td>
<td>1.0</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>1.2</td>
<td>Sand</td>
<td>1.7</td>
</tr>
<tr>
<td>Sand</td>
<td>0.36</td>
<td>8 mm LWA</td>
<td>0.92</td>
</tr>
<tr>
<td>Water</td>
<td>0.56</td>
<td>14 mm LWA</td>
<td>0.92</td>
</tr>
<tr>
<td>HRWRA</td>
<td>0.01</td>
<td>Water</td>
<td>0.4</td>
</tr>
<tr>
<td>PVA Fibre</td>
<td>2.2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Each specimen was tested using four-point loading to produce a region of pure bending (Farida, et al., 2018) in the central span of 800 mm. The beam was deformed at a rate of 0.5 mm per minute. Eight strain gauges were attached to the steel member and two to the concrete casing, and seven LVDTs were used to measure the deflection of the beam and the slip between the steel beam and concrete casing; their position is shown in Figure 3.

**B. Results and Discussion**

The first beam, ECC25, displayed elastic behaviour until a load of 384.2 kN (18.5 mm deflection) at which the bottom flange began to yield. This was followed by a section of progressive yielding of the steel section until crushing of the ECC case at 589.8 kN (38.8 mm deflection) shortly after the yielding of the top flange. After a drop at this point, strength increased until a maximum strength of 596.1 kN was achieved at a deflection of 66.8 mm. Final failure of the beam occurred at a deflection of 115 mm. Removal of the case showed that final failure was due to local buckling of the top flange. Cracking occurred principally in

the moment span of the beam as shown in Figure 4.

ECC15 similarly showed yielding at the bottom flange first, at a higher load of 397.6 kN (17.2 mm displacement). Failure of the case occurred 607 kN (40.0 mm displacement), and this represented the peak strength of the beam. Final failure was also due to local buckling of the top flange and occurred at deflection of 93 mm. Moment cracking and ECC crushing were the two forms of damage experienced.

NC100 showed yielding of the bottom flange at 400.7 kN (18.0 mm deflection), and casing failure at 541.1 kN (31.3 mm deflection) before the top flange had reached yielding strain. Post-failure, there was a slight gain in strength until the maximum load was attained at 551 kN. The final loss of strength started at about 90 mm deflection, and came in the form a steep decrease in load with a still-continued increase in strain until the steel
member fractured through the bottom flange and a portion of the web, the only beam on which this behaviour occurred. Local buckling was found in the top flange. As well as moment cracking, one large diagonal shear crack was observed running from near one support as shown in Figure 6.

ECC30F showed yielding of the bottom flange at 340.3 kN (18.8 mm deflection). Maximum loading was 505.0 kN at a deflection of 39.8 mm. Immediately after this point, the beam failed through lateral-torsional buckling. Large portions of the ECC casing split entirely from the beam, and none of the casing as far as could be observed had remained bonded to the steel. ECC30F was also the only beam to show any slip between the casing and the steel. Slip commenced at about 250 kN on one end and 290 kN on the other. The slip readings are shown in Figure 5.

The load-deflection curves for all four beams are shown in Figure 7. The two fully encased beams incorporating ECC show a clear improvement in strength compared to the beam encased in normal concrete. All three beams are of about equal stiffness, but the ECC beams have a greater range during which the casing remains intact due to the superior ductility of the ECC allowing greater deformation. Compared to beams using grade 300 steel where normal concrete beams showed a much greater drop at casing failure compared to ECC ones (Rana et al., 2018), there is no marked difference in the behaviour of different types of beams in the present study.

ECC25 and NC100 showed some strain hardening after the failure of the case, and attained a higher strength in the post-failure region than at failure itself. ECC15 did not show this strain hardening and the decrease of strength as local buckling occurs in the post-failure stage occurred much earlier than in the other two fully-encased specimens. This may point to a weaker or more distorted steel section used in ECC15 as that may result in buckling at an earlier point in the post-failure stage. Despite this, ECC15 was the best performing beam by peak strength and also the stiffest. As none of the previous studies (Rana et al. 2018, Kabir et al. 2019) tested two beams differing
only by percentage of ECC at the top flange, the results have to be treated with caution. However, it is possible that there may be an optimal proportion of ECC for failure strength and stiffness.

The exception in behaviour was ECC30F. Kabir et al. (2019) tested such a beam with grade 300 steel and found it by far the weakest of the four composite sections; however, that beam still failed in flexure. The increased steel strength and taller section compared to the early study have rendered such a design unsuitable to bear loads in the unrestrained condition it was tested in. The slip data show that for a significant portion of the specimen’s loading, there was no bonding between the steel and the ECC and therefore no composite action.

The load-strain curves for all of the beams are shown in Figure 8. Analysis of the load-strain curves shows that ECC25 and ECC15 allowed the top flange to develop its full elastic strength before failure, thereby demonstrating that those configurations help prevent local buckling. On the other hand, the top flange on NC100 was found to have failed before it reached its yield strain, meaning that it had not developed its full yield strength. Local buckling of the top flange was found in all three beams when the casing was removed subsequent to testing: these tests demonstrate that local buckling occurs at or after the failure of the casing. The ECC’s characteristics are favourable when designing composite beams for allowing HSS to attain its yield strength through controlling local buckling.

VII. Numerical Study

A. Modelling Methodology

Version 16.4-2 of Abaqus was used to create 3D models of the beams tested including non-linear material behaviour. C3D8R elements (linear hexahedral) were used with sizes of 40–50mm in conformance with the sizes used in previous studies (Kabir, et al., 2019).

1. Constitutive Models

Constitutive models of each of the different materials were used to create the input for the Abaqus model, these models in turn being based upon the test results taken from the material samples from the experimental phase of the project.

The ECC constitutive model was adopted from Meng, Huang et al. (2017) and contains different curves to express the compressive and tensile stress-strain relationships. The compressive model is in turn based upon that of Zhou et al. (2015) which was developed from uniaxial compression tests. The tensile model was similarly developed from uniaxial tensile tests (Meng, Huang et al. 2017). These two models are shown in Figure 9 (a) and (b) with the parameters used in Tables 2 and 3.

LWC and normal concrete both used the same constitutive model. Compressive behaviour was simulated by the model developed by Carreira and Chu (1985) whereas tensile behaviour was limited by the fracture energy as given in Tao et al. (2013):

\[ G_F = (0.0469d_{\text{max}}^2 - 0.5d_{\text{max}} + 26)(0.1f'_{\text{c}})^{0.7} \]

where \( d_{\text{max}} \) is the maximum coarse aggregate diameter in mm, \( f'_{\text{c}} \) is the compressive strength in MPa, and \( G_F \) is returned in N/m. The curve for the compressive constitutive model and the parameters used are given in Figure 9 (c) and Table 4 respectively. All cementitious materials used a Poisson ratio of 0.2.

Figure 8. Load strain curves for strain gauges 1 to 4 on (a) ECC25, (b) ECC15, (c) NC100 and (d) ECC30F
The behaviour of the HSS was modelled using the curves from coupon testing in lieu of any empirical constitutive model. Before this was used as an input, it was converted to true stress and plastic strain. The curve used for the modelling is shown in Figure 9 (d), and the strength parameters in Table 5.

2. **Concrete Damage Plasticity**

To account for the loss of effectiveness in concrete due to the formation of cracks during plastic deformation (Awinda, et al., 2014), it is necessary to provide damage factors strains when modelling such materials (Abaqus, 2014). The CDP model used was that developed by Lubliner et al. (1989) and forms the following equations which are applicable to tensile or compressive behaviour (Tao & Chen, 2015):

\[ d = 1 - \frac{\sigma}{f} \]  

\[ \varepsilon^p = \varepsilon - \frac{f}{E} \]

**Figure 9.** Constitutive model stress-strain curves for (a) ECC in compression, (b) ECC in tension, (c) LWC in compression, and (d) HSS. Normal concrete uses the same shape curve as LWC but with different parameters.

The behaviour of the HSS was modelled using the curves from coupon testing in lieu of any empirical constitutive model. Before this was used as an input, it was converted to true stress and plastic strain. The curve used for the modelling is shown in Figure 9 (d), and the strength parameters in Table 5. The Poisson ratio used was 0.3.

2. **Concrete Damage Plasticity**

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\[ d = 1 - \frac{\sigma}{f} \]  

\[ \varepsilon^p = \varepsilon - \frac{f}{E} \]
Where for the damage factor $d$ and modified plastic strain $\varepsilon^p$, $f$ is the strength of the material and $E_0$ is the initial elastic modulus. Other parameters required to model plastic behaviour (Kmieciak & Kamiński, 2011) were input using their default values (Tysmans, et al., 2015); these are listed in Table 6.

3. Steel-concrete bonding

For LWC and normal concrete, the bonding interaction with the steel was defined by tangential behaviour using a penalty friction coefficient of 0.7. For the ECC-Steel interaction, previous studies have seen it fit to use cohesive surfaces (Abaqus, 2014), a traction-separation model defined by the following law (Camano, et al., 2003), where for uncoupled behaviour, the stresses $t_i$ in the normal ($i=n$) and shear ($i=s, t$) directions are related to the surface displacements $\delta_i$ by elastic constants $K_{ij}$:

$$
\begin{pmatrix}
 t_n \\
 t_s \\
 t_t
\end{pmatrix} =
\begin{bmatrix}
 K_{nn} & 0 & 0 \\
 0 & K_{ss} & 0 \\
 0 & 0 & K_{tt}
\end{bmatrix}
\begin{pmatrix}
 \delta_n \\
 \delta_s \\
 \delta_t
\end{pmatrix} = K \delta
$$

(4)

Also specified was a damage criterion in the form of a maximum stress beyond which damage occurs and strength is lost just as in the concrete itself. The elastic bond stiffness coefficients in the shear direction were determined using the following relationship (Henriques, et al., 2013):

$$
K_{ss} = K_{tt} = \frac{t_{\text{max}}}{S_1}
$$

(5)

Where $t_{\text{max}}$ is the maximum shear bond stress and $S_1$ is the displacement or slip at that stress. The values of these parameters were taken from Rana et al. (2017) based on their values for the pushout strength of flat steel with 30 mm cover. This gave a value of 2.9 for $K_{nn}$ (Maximum stress of 1.45 MPa at 0.5 mm slip), and the parameters for onset of damage were set at a shear stress of 1.45 MPa for ECC25 and ECC15, indicating the breaking of the chemical bond in the concrete, and 0.5 MPa for ECC30F indicating the presence of frictional bonding only. The bond coefficients and also the damage stresses in the normal direction could be found by multiplying the values for shear (Henriques, et al., 2013):

$$
K_{nn} = 100K_{ss} = 100K_{tt}
$$

(6)

4. Other considerations

Loading in the model was applied from two plates made of discrete rigid shell elements with a width of 40 mm in order to match the size of the loading points used in the experimental tests. Each end was supported on a line of nodes restrained to give a simply supported condition.

B. Results

All FE models were validated using load-displacement curves. These are shown in Figure 10.

![Figure 10. Comparison of load-strain curves from the tests and the FE models: (a) ECC25, (b) ECC15, (c) ECC30F, (d) NC100](image-url)
ECC25 showed yielding of the bottom flange at a load of 441 kN (deflection of 21.6 mm), and the failure of the case at the shortly follows the yielding of the top flange at 582 kN (deflection of 41 mm). Damage first occurred in the LWC case in the plane of the bottom flange followed by the failure of the bearing points. This was followed by the growth of damage in the moment span, also originating from the plane of the bottom flange. Damage of the ECC commenced in the plane of the top flange followed by the breakage of areas approximately 50 mm to 100 mm inwards from the loading points. The yielding of the steel commenced on the outer surfaces of the flanges and progressed from the flange into the web; this occurred earlier at the bottom.

ECC15 showed yielding of the bottom flange at commenced at 440 kN (21.6 mm deflection), and the top flange commenced yielding at 580 kN. Damage across the top of the ECC case occurred at 589 kN, with the rest of the damage pattern developing as described for ECC25, but with a greater amount of damage occurring in the shear zone. This is shown in Figure 13.

ECC30F showed yielding of the bottom flange at 414 kN (22.5 mm deflection), and of the top flange at 469 kN (28.1 mm deflection). Damage begins to occur in the ECC case at a load of 511 kN, but it not until 519 kN that the damage had rendered the entire width of the case ineffectual. The damage is restricted to the portion of the case at or above the level of the top flange, and begins at two points 50 mm to 100 mm inward from the loading points; once this had grown to about the width of the flange, failure occurred under the loading points as well. Maximum loading was 525 kN at 78.9 mm displacement.

NC100 yielded at the bottom flange at 456 kN (18.8 mm deflection), and the top flange yielded at 542 kN (31.2 mm deflection), which was before failure stress at 546 kN. Damage was concentrated first at the top between the the loading points, spreading across the full width of the case at 428 kN and spreading downwards.

C. Discussion

In general, the FE modelling proved accurate when validated against load-deflection curves, and this is probably enough to evaluate the practicability of the designs when employed for structural purposes, as the exact mechanism of plastic failure is less important so long as strength is maintained. For understanding the behaviour of the specimens at a small scale, the models have some limitations.

In no model was any local buckling observed, and a major reason for use of ECC is its control of that phenomenon. The long straight plateau found in each model after 70 mm to 100 mm deflection is a consequence of this, although it occurs well past the point when any structural applications might matter. Furthermore, no lateral-torsional buckling was observed in ECC30F: the ability to make a perfectly straight and even beam and loadings is the cause of this. Therefore, modelling is not necessarily in the best place to analyse sections which may be prone to LTB.
Each of the fully-encased beams also showed bearing failure before the occurrence of any other sort of damage. Using a plate similar to the loading plates may produce a more accurate simulation. Each beam also showed slip even though only ECC30 did in the physical tests. This suggests that the bond behaviour is not fully captured by the models used. At failure, only a small drop was observed in all of the beams. Previous simulations by Rana et al. (2018) and Kabir et al. (2019) also produced this result, and it may therefore be concluded that the mechanics of concrete failure do not transfer exactly to the model. Furthermore, all the yield loads for each point were considerably higher than those attained in the tests.

The similar performance of ECC15 and ECC25 was found in the models as well as the tests. Both beams had an almost identical stiffness and very similar failure loads; this implies it would be more economical to use ECC15 as it has a reduced amount of ECC for the same performance. ECC30F was weaker than the others as was expected from the tests and previous studies (Kabir, et al., 2019). NC100 performed the same with respect to the others as it did in the test although its yielding and failure pattern differed, again demonstrating that this type of modelling may not be sufficiently refined to investigate small-scale behaviour although it accurately replicates the load-strain curves.

From finding the points of yielding, maximum strength, and the progression of damage, it was found that the HSS yielded on the top flange before progression of damage at the top of the case had reached an advanced level. Therefore, the full elastic strength of the steel at the top flange was exhausted before failure, which when viewed in conjunction with the outcomes of the tests, provides support to the theory that keeping an intact case aids strength by allowing the full utilisation of such capacity.

The damage calculated by the models generally shows conformance in the regions in which it occurs, being mostly in the moment span, but the mechanism by which it spreads from the plane of the flanges upwards is not one that was observed in the live tests. NC100 also showed a different pattern with damage beginning at the top of the case and spreading downwards before it occurred along the bottom, the latter being what occurred first in the physical specimen.

VIII. Conclusions

Physical bending tests on four composite beams incorporating HSS and ECC were successfully carried out, and models were made of each of the beams and validated against the tests’ load-strain curves. ECC25 and ECC15 had higher strength than other configurations and were able to provide full yield strength at the top flange, thereby alleviating the problem of local buckling in the compressive zone causing a degradation in performance. NC100 failed before the top flange had reached yield strength, showing the inferior performance of normal concrete. ECC30F failed through LTB, showing that this design does not have sufficient restraint or stability by itself and is therefore not well suited to application in structural design.

The FE modelling confirmed the mechanism by which ECC25 and ECC15 failed, but was not able to replicate LTB or local buckling. Modelling also confirmed the results from the tests that ECC15 and ECC25 were not distinct in their strength parameters, even though use of ECC at the top flange produced a better result than not using it.

IX. Recommendations

The results of this project may open up further areas for research into refining this sort of composite system. Possible areas into which further study could be made include the following:

- The effect of partial encasement on LTB, and the point at which encasement is insufficient to prevent it.
- Effects of differing proportions of ECC on the top flange, to discover whether there is an optimal or minimal value for its use.

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References


**Appendices (supplementary documents)**

Appendix A. Project Schedule