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INFLUENCE OF STEEL FIBRES, USED IN CONJUNCTION WITH UNCONFINED REBAR CONFIGURATIONS, ON THE STRUCTURAL PERFORMANCE OF PRECAST ELEMENTS

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Summary: A joint experimental and computational research program has been carried out to demonstrate the potential benefits of using Steel Fibre Reinforcement (SFR) within the design and manufacture of two key structural elements, namely slender walls and thin lintels with dapped ends, often adopted within the pre-cast concrete industry. The investigations specifically focus on the advantages of utilising SFR in conjunction with traditional bar reinforcement in an unconfined layout. This configuration allows cost savings in regards to precast manufacture and enjoys good performance in terms of durability and fire resistance, though its use is currently limited by the brittle mode of failure. The paper sets out to prove that the inclusion of SFR within the concrete matrix is capable of inducing a more ductile response in the structural members under consideration, therefore potentially making it possible to justify the adoption of such unconfined layouts in the design practice.

1 INTRODUCTION

Historical testing and research studies [1],[2] [3] have demonstrated that the adoption of single, centrally placed or minimum reinforcement configurations in RC wall elements, which are subjected to an eccentric axial load, results in a sudden and brittle failure mechanism. In addition, research undertaken to date [1] has also shown the ‘flexural cracking’ response of the slender RC wall elements to be critical in determining the resulting buckling behaviour and ultimate failure load of the panel. This is opposed to the more conventional assumption that the element’s capacity and response can be found by consideration of the component’s ultimate flexural capacity. This method however, has been shown to only be suitable for sections using a double layer of confined longitudinal reinforcement, where the longitudinal reinforcement ratio of this section (\( \rho = A_s / \ell t \)) is greater than 1% [3], where \( A_s \) is the cross-sectional area of reinforcement per unit length of the panel and \( t \) is the thickness of the panel. The term flexural cracking is used here to describe the situation where the concrete section at the critical location cracks in flexure (and the resulting concentrated loss of stiffness, combined with the lack of influential tension steel) controls the resulting structural behaviour and ultimate stability of the panel much more than would occur with doubly reinforced panels, where \( \rho = A_s / \ell t \geq 1% \) [4]. Hence, the axial capacity of the RC wall element becomes dependent on the element’s flexural stiffness up to and post cracking. Consequently, appropriate account now needs to be taken of the contribution of the concrete acting within both the tension and compression stress block as part of the design of the element. Further, this flexural cracking response has been shown to control the response and capacity of centrally reinforced panel elements adopting unconfined rebar configurations, up to a steel ratio of \( \rho = A_s / \ell t = 3\% \) [5].
Thus the controlling failure mechanism of the identified RC wall elements will, in part, be influenced by the formation and subsequent progression of flexural cracks in the concrete at the panel’s critical section. It follows therefore that if, as argued, the initiation and behaviour of such cracks in the concrete section can be considered to be significant when determining the structural response of such panels, the incorporation of steel fibre reinforcement should therefore be seen to substantially influence the resulting behaviour and ultimate capacity of the panel elements under consideration. This is because the use of SFR concrete mixes has been shown to bring about a number of improvements in the mechanical performance of concrete, relating to aspects such as: a delay in micro-crack propagation to a macroscopic scale, the hindrance of macroscopic crack development and an improved structural ductility [6]. Aimed at demonstrating, as well as better understanding and designing for this predicted influence, the paper summarises the results of experimental and computational analyses for the relevant panel types and SFR concrete mixes.

From the literature reviewed as part of this investigation, few resources or research studies appear to currently exist, which aid in the design of slender panel elements, using a combination of both SFR and the traditional longitudinal reinforcement configurations proposed. Aimed at improving this current situation, the paper proposes and evaluates the possible use of a computational procedure, in which ‘lumped plasticity’ is used to predict the behaviour and buckling capacity of the resulting structural members. The method has previously been shown to provide a good correlation for slender precast panel elements, albeit for test samples adopting only a traditional unconfined reinforcement configuration and a standard (C40/50 grade) concrete mix design [3]. It is believed however, that if this design method is suitably modified to account for the SFRC material behaviour, the proposed technique could also be used to derive a design capacity for the panel elements adopting the hybrid of reinforcement types considered. The method utilises a non-linear fibre hinge at the known critical cross section of the panel, in order to simulate the buckling response of the slender walls.

The second aspect of the paper considers pre-cast lintels, supported on end projections that have been reduced in height. Such ‘dapped end’ or ‘halving joint’ details are common in precast construction because they beneficially lead to a reduction in the construction depth required. The experimental investigation undertaken therefore aims to increase the understanding of the shear behaviour and capacity of these resulting discontinuity shear or ‘D-regions’, for situations in which: a centrally placed, unconfined and welded reinforcement mesh is to be used in conjunction with varying percentages of additional steel fibre content. Additionally, the structural testing undertaken will also aid in the development and verification of an analytical Strut-and-Tie Model (STM), capable of accounting for the use of such a non-traditional reinforcement strategy.

2 CURRENT LIMITATIONS OF EXISTING DESIGN METHODS IN RELATION TO UNCONFINED AND STEEL FIBRE REINFORCING STRATEGIES

2.1 Design of Eccentrically Loaded Precast RC Panels

Both the major national codes of structural design practice reviewed (ACI-318 [7], EC2 [8]) currently devote specific sections to the design and detailing of simply supported RC wall panels, subjected to an eccentric axial load. Each of the specified design standards allows for the design of such elements through the adoption of one of two possible design methods. The first of these alternatives involves the use of simplified design equations that have been empirically (or semi-empirically) derived from a limited amount of experimental data [9]. These expressions however, allow no account to be taken in regards to either the quantity or the distribution of longitudinal reinforcement. Also, the simple design equations do not currently allow for or enable the modification of the concrete material model, required in this instance to account for, and potentially take advantage of, the modified concrete behaviour due to the presence of the steel fibres within the concrete mix. In addition, the existing empirical design equations do not currently allow for design situations in which the eccentric load application is required to fall beyond the ‘kern point’ of the section. That is, the largest off-set at
which a load can be applied to a section without it developing tensile stresses. One such load case is however, investigated as part of this study in order to assess the ability of, and therefore the potential for using the proposed hybrid reinforcement configurations to resist a larger, non-standard value of load eccentricity.

One potential alternative design method however, currently available within each of the regulatory guides considered [7-8], is the consideration of the wall component as a column of an ‘equivalent’ structural width. This method, prima facie, appears to potentially offer a suitable design method, for the hybrid panels under consideration. This is because, it would enable the engineer to account for the necessary modification to the concrete material model, as well as being able to include for the longitudinal reinforcement quantity and its distribution. By using this method, one could also allow for a load applied at the larger eccentricity. However, the use of this method requires the buckling failure load of the panel element to be dependent upon, and thus determined through consideration of, the flexural capacity of the component’s cross section [3]. As defined within section 1, this is not true for the minimally and centrally reinforced panels that are the focus of this study. Therefore, neither of the existing design procedures currently available, appear suitable for the design of panels reinforced through a combination of minimum, centrally placed and unconfined longitudinal re-bar, with secondary reinforcement also provided by using a quantity of SFR.

2.2 Strut and Tie Design for D-Regions

The strut-and-tie analytical model is an extension of the Ritter-Mörsch truss analogy, with particular application to the shear design of discontinuity regions (D-Regions) in cracked reinforced elements [10]. The model assumes that structural loads are carried through a set of compressive stress fields and interconnected tensile ties. Previous studies ([11],[12]) have demonstrated that the use of steel fibre reinforcement, in conjunction with traditional longitudinal reinforcement, significantly improves the capacity of the D-regions considered within the precast structural elements. However, the past investigations do not consider the validity of adopting an STM in their design. Hence, of particular interest as part of this study is; how a traditional STM analytical model should be modified or augmented to suitably account for the behaviour and failures observed, when adopting the hybrid reinforcement proposed, within the critical structural regions?

Another important consideration in adopting the STM methodology, as part of the development of an acceptable design for the proposed precast lintel elements, is that due to the lower-bound nature of the method, a number of potential (or compliant) models are possible. However, a poorly selected and detailed strut-and-tie model may potentially result in severe damage and cracking to the element, even under service loading [13]. Because of this, the experimental investigation and validation of any potential STM analytical model is therefore considered as an essential component in the development of a design procedure for the precast dapped end beams.

3 EXPERIMENTAL INVESTIGATION

3.1 Test Samples and Experimental Arrangements

Eight 450mm wide, 100mm thick and 3000mm tall panel elements were cast adopting C40/50 grade concrete mix (500kg/m\(^3\) CEMI, 840kg/m\(^3\) Gravel<20mm, 900kg/m\(^3\) Sand<4mm, 0.8% Superplasticizer, w/c=0.36, Flow=650-700mm). Four of the samples were reinforced solely using a single, centrally placed layer of mesh reinforcement to form the unconfined reinforcement configuration illustrated in Figure 1(d). The four additional panels tested adopted an identical reinforcement configuration to that illustrated although, in these cases, an additional steel fibre content (1% by volume) was also incorporated within the specified mix design. In this way, the potential for any improved performance through the use of such a hybrid reinforcing strategy will be quantified, relative to the conventionally reinforced panels. The double hooked end type fibres used were: 50mm long, 0.75mm in diameter, had an aspect ratio of 67mm and a tensile strength greater than 1100N/mm\(^2\).
The eight panel elements were then axially tested, using the experimental setup illustrated within Figures 1(a) and (b). The testing rig used for the experiments was capable of applying a load of 4000kN, with the loading beam designed in order to ensure the transmission of a uniformly distributed load across the top of each panel at eccentricities of 17mm($t/6$) and 33mm($t/3$). The smaller of the adopted eccentricities was chosen to reflect the maximum load off-set allowed for within the major international design regulations($t/6$) investigated [7-8]. This limit on load eccentricity is commonly referred to as the ‘kern point’ and has been widely adopted as part of a number of experimental studies into the axial capacity of one-way spanning panel elements [2-5]. Additionally, a load case involving a larger eccentricity ($t/3$) has also been incorporated as part of this study, in order to investigate whether the use of SFRC in conjunction with un-confined longitudinal reinforcing steel could potentially offer an engineer the opportunity to justify the use of such panel elements for resisting such a demanding loading condition.

The top and bottom hinged support conditions were each simulated by placing a 25mm high strength steel rod on a 50mm thick bearing plate (Figure 1(c)). Displacement transducers were utilised at the locations illustrated within Figure 1(b) in order to record out-of-plane displacements at the centre and top of the panel, as well as providing a means of determining the rotation at the top of the wall. Strain readings were also taken utilising a digital portal gauge at the known critical section (i.e. the mid-span of the RC wall element). This allowed the strains induced at this section to be recorded as the axial load was incrementally increased.

As part of the secondary focus of the experimental study, four precast lintel elements were additionally cast and tested to failure. The geometry of the specimens tested and the weld mesh...
reinforcement layout adopted are illustrated within Figure 2. Because the objective of the experimental program is to study the behaviour of the D-Region of the precast lintel component, a member length of 1415mm was adopted so as to ensure that the region controlling the element’s capacity was that under investigation. All reinforcing bars used in the manufacture of the samples were 16mm in diameter, with a cover of 25mm maintained throughout. The bars were MIG welded, with all anchorage forces and requirements appropriate to the resulting welds calculated in line with the relevant EC2 provisions [8].

The testing of the beam samples in shear was undertaken using the experimental setup detailed within Figure 3, with a loading rate of 1 kN/s adopted. Bearing plates with sizes of 100x100x12.5mm were used at both the support and loading positions in order to suitably spread the applied load and thus ensure the appropriate strut propagation within the sample. Digital strain gauges were used to collect data in regards to the strains at the surface of the sample continuously during testing. The positioning of the gauges was designed so as to collect results both for the tensile region at the re-entrant corner and over the primary compression strut that will form the dap. The rosette pattern adopted allowed the angle of principal stress in the half-joint detail to be calculated and recorded throughout the loading of the specimen. Consequentially this will allow the collected data, through the application of Mohr’s circle, to be used to validate the geometry of the adopted Strut-and-Tie model (STM). The digital strain gauges used were 60mm in length, with Figure 3 identifying the end locations of this instrumentation.
3.2 Experimental Findings

Table 1 summarises the experimental failure loads observed for each of the panel elements tested. In addition Figure 4(d) details the measured relationship between the applied load and the deflection of the panel at its critical section, up until buckling failure occurred. It should be noted that the loads have been normalised (in order to allow an effective comparison of panel performance), according to the expression:

$$N_c = \frac{N}{\bar{f}_c L t}$$  \hspace{1cm} (1)

Where $N$ is the axial load applied to the panel at the set eccentricity $(kN)$, $f_c$ is the average measured concrete cylinder strength for the samples $(Nm/m^2)$, with $L$ and $t$ the width and thickness of the concrete wall elements respectively $(mm)$.

As can be seen, the inclusion of the 1% volume fraction of steel fibre reinforcement in addition to the unconfined reinforcement mesh traditionally adopted, leads to an increase in both axial load and deformation capacity of the panel. Both effects appear to be more significant within the panels, to which the load was applied at an increased eccentricity. An average increase of 12% in normalised buckling capacity was seen for panels loaded at an eccentricity of 33mm $(i/3)$, with the lateral deflection prior to failure increasing from a minimum of 10.5mm in the traditionally reinforced panel to a maximum of 20.55mm for a panel adopting the hybrid reinforcement option considered. This increased lateral deflection could also be clearly observed for the SFRC panel elements, with a distinct bowing evident prior to the failure of the wall (Figure 4(c)). For panels loaded at an eccentricity of 17mm $(i/6)$, a lesser average increase in normalised buckling capacity was recorded (9.8%). Lateral deflections of 17.51mm and 19.61mm were measured for panels SFR1 and SFR2 respectively compared to the minimum value of 11.02mm observed for Panel RC1.

The most significant difference in the behaviour of the two panel types investigated however, was perhaps associated with the buckling failure typologies observed for the hybrid and traditionally reinforced elements. In the instances where a centrally placed, unconfined reinforcement layout was solely adopted the observed failure was of a sudden, brittle and explosive nature Figure 4(a). In contrast for the cases when a 1% volume fraction of the double hooked end steel fibres was incorporated, a much more acceptable (from a structural design perspective) ductile failure resulted.

Similarly, Table 2 details the failure capacities recorded for each of the six dapped-end lintel samples fabricated. For the control samples (RCL1 and RCL2) first cracking was seen to occur at the re-entrant corner, quickly followed by flexural cracking at the mid-span. As the loading was increased however, the mid-span flexural cracking was seen to propagate at a rate greater than that which was observed at the re-entrant corners. It was then observed that both the samples exhibited a significant propagation of tensile cracking along the diagonal compressive strut. This cracking next propagated upwards towards and subsequently along the beam’s top face. The progression of this cracking was then observed to cause the brittle shear failure captured within Figure 5(a), with the concrete material forming the dap of the lintel, spalling away post failure to expose the welded mesh reinforcement. Interestingly, it was also observed that plastic hinges had formed within the longitudinal steel of the mesh, adjacent to the welded vertical bars. This perhaps indicates the potential failure mechanism for the sample.
Figure 4: Eccentrically loaded panels (a); Brittle failure of traditional RC panels (b); SFRC panel section failure (c); Increased lateral deflection of SFRC panel prior to failure (d); Experimental load-deflection curves for panels with varying eccentric load and use of SFR.

Figure 5: Brittle failure of traditional RC lintel (a); SFRC Lintel Failure (b); Experimental load-deflection curves for traditional and hybrid lintel samples (c).

Similar cracking patterns and propagation sequences were also then observed for the samples cast using a combination of a welded mesh and an additional content of steel fibre reinforcement (samples SFRL1-2). The first crack again occurred at the sample’s re-entrant corner and this was
again followed by more extensive flexural cracking at the mid-span. However, a noticeably slower and less extensive crack propagation was observed for all samples adopting a percentage content of steel fibres relative those using the more traditional mix. This provides evidence therefore that the content of steel fibres within the mix were acting as expected to provide a means of crack control. In addition to slowing crack formation the fibres also significantly reduced the level of the resulting spalling observed at failure (Figure 5(b)). Also worthy of note was that the extent of flexural cracking away from the daps appeared to significantly multiply as the fibre content in the samples was increased.

### Table 1: Panel buckling capacities

<table>
<thead>
<tr>
<th>Element Ref</th>
<th>( f_c ) (N/mm²)</th>
<th>e (mm)</th>
<th>Nᵣ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RCW1</td>
<td>37.28</td>
<td>17</td>
<td>597</td>
</tr>
<tr>
<td>RCW2</td>
<td>37.28</td>
<td>17</td>
<td>572</td>
</tr>
<tr>
<td>RCW3</td>
<td>38.48</td>
<td>33</td>
<td>336</td>
</tr>
<tr>
<td>RCW4</td>
<td>38.48</td>
<td>33</td>
<td>322</td>
</tr>
<tr>
<td>SFRW1</td>
<td>40.21</td>
<td>17</td>
<td>713</td>
</tr>
<tr>
<td>SFRW2</td>
<td>40.21</td>
<td>17</td>
<td>689</td>
</tr>
<tr>
<td>SFRW3</td>
<td>41.11</td>
<td>33</td>
<td>407</td>
</tr>
<tr>
<td>SFRW4</td>
<td>41.11</td>
<td>33</td>
<td>394</td>
</tr>
</tbody>
</table>

### Table 2: Lintel shear capacities

<table>
<thead>
<tr>
<th>Element Ref</th>
<th>( f_c ) (N/mm²)</th>
<th>Nᵣ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RCL1</td>
<td>61.28</td>
<td>190</td>
</tr>
<tr>
<td>RCL2</td>
<td>32.96</td>
<td>100</td>
</tr>
<tr>
<td>SFRL1</td>
<td>42.16</td>
<td>175</td>
</tr>
<tr>
<td>SFRL2</td>
<td>32.96</td>
<td>140</td>
</tr>
</tbody>
</table>

Figure 5(c) illustrates the load deflection behaviour recorded for each of the beam elements tested. Normalisation of loading values was undertaken in order to enable a comparison between each of the samples in relation to how efficiently the steel weight incorporated is being used within each of the designs considered, as well as to allow for the variations in concrete strength seen for the samples cast. The values were corrected according to the expression:

\[
F_\text{c} = \frac{F}{f_c \times L^2} \tag{2}
\]

As would be expected, the plots of load displacement relationship for the beam elements tested (Figure 5(c)) show that all samples had a similar elastic range. However, both samples incorporating the 1% volume of steel fibre content exhibited a much greater ductility, with the maximum deflection at the point of failure almost double that of the non-fibre samples. Such a response is indicative of the successful application of steel fibre reinforcement causing a more plastic/ductile response under loading and controlling the cracking, which would have otherwise resulted in failure. An average increase of 32.1% in normalised shear capacity was also measured for the SFRC halving joints.

### 4 PROPOSED DESIGN METHODS FOR PRECAST ELEMENTS ADOPTING HYBRID STEEL FIBRE AND UNCONFINED REINFORCEMENT CONFIGURATIONS

#### 4.1 Lumped Plasticity

Lumped plasticity idealisation is a widely adopted computational model, particularly utilised in earthquake engineering and robustness assessment, in order to determine the ultimate performance of a structural system by increasing step by step the load multiplier until failure (push-over or push-down analysis). It has been demonstrated within previous studies [1] [3] that it is possible to consider, as part of a computational assessment, the entire inelasticity of an RC panel element to be
concentrated at the critical section for the span, with this ‘lumped plasticity’ modelled through the use of a non-linear hinge (Figure 6(a)).

Such a computational model is effective for the cases considered as part of this study, because the location of the maximum moment (and thus the critical section) is known for the simply supported elements. In this representation the component’s cross section is subdivided into a number of elements or fibres, to which the appropriate material properties are then assigned (Figure 6(b)). In this way, the non-linear moment-curvature relationships and limits of the fibre hinge can then be determined for a range of axial loads (assuming plane cross sections). As such, the arrangement illustrated can therefore be used in order to provide an effective representation of system non-linearity, and consequentially, of buckling capacity.

Importantly, because the proposed computational method allows the designer to modify for the relevant concrete material model, it can therefore facilitate the incorporation within the analysis of other concrete types, such as the fibre reinforced mix adopted as part of this study. Therefore the Mander [14] model adopted for the unconfined concrete material within the traditional RC panels was replaced by the material model suggested by Al-Taan and Ezzadeen [15] (Figure 6(c)) for fibre reinforced concretes adopting a 1% fibre volume fraction. Additionally however, in order to correctly quantify the rotational capacity of a concrete member, the length of the resulting plastic hinge \( L_p \) that will be formed during loading and subsequent failure must also be accounted for. Accordingly, the hinge lengths were computed for both panel types from the expression proposed by Panagiotakos and Fardis [16] for unconfined RC panels and column elements subjected to monotonic loading:

\[
L_p = 0.18L_z + 0.021d_f f_y
\]
where $L_s = H/2$ is the shear span of the member, $d_{eff} = t/2$ (for the panels considered as part of this study) is the effective depth of the reinforcement and $f_y$ is the yield strength of that reinforcement. As can be seen from Table 2, the resulting computational predictions for both the traditionally reinforced panels and those adopting the hybrid reinforcing strategy show a good correlation with the actual experimental capacities seen. This relationship is also illustrated within Figure 6(d) which shows the least-squares best fit to slope $\theta_1 = 0.833$ and $\theta_2 = 0.846$, for the RC and SFRC hybrid panel types respectively, to be acceptably close to the $\theta = \pi/4$ ideal. The poorer correlation seen within the panels where the secondary fibre reinforcement was incorporated is likely due to the fact that a degree of calibration in relation to the length of fibre hinge is required. However, a greater number of data points would be required in order to inform how Eq 3 should be modified to account for the use of SFRC.

4.2 Design Using Strut and Tie

To aid in the development of the proposed analytical strut-and-tie model for the beam elements considered, an elastic analysis was first undertaken in order to analyse the stress flows occurring, a method strongly advocated within existing literature [10]. A 2D finite element (FE) analysis was carried out, using shell elements due to the size of the section (100mm) in relation to the size of the shells considered. These stress flows were then used in the development of a relevant STM. Additionally the outputs of the FE model were used to verify the angle of the stresses against those obtained by experimental measurement, with the angle used for the analytical STM (59°) found to lie between the maximum measured angle of principal stress (52°) and that predicted through linear computational analysis (66°). The lower bound model developed is illustrated within Figure 7 (a) and compares well to those proposed within literature [9] for concrete elements with a similar geometry and reinforcement provision. The precedent cited however, considered the response of confined concrete without a steel fibre content.

A key assumption made when arriving at the most appropriate analytical STM, was regarding the width of the critical compressive strut formed. Although the bearing plate was sized to spread loads across the full width of the beam it was assumed that the effective width was that confined by the welded mesh configuration (Figure 2). Therefore the width of concrete considered was that within the centreline of the reinforcement bars, as this was felt to best represent the 'pinching' or confining point. The design model was then used to calculate the capacity of the section, with the theoretical predictions summarised as part of Table 2. Because the experimental work conducted identified that crushing of the primary compressive strut, positioned at the support bearing plate, resulted in element failure it could therefore be considered to be critical. It follows then that the size of this strut and thus the capacity of the section is then dictated by both the angle of the strut formed and the width of the bearing plate used. The remaining struts were still subsequently assessed for adequacy however, along with checks also required to ensure the tensile capacity of the reinforcement provided would not be exceeded within any of the associated ties.

Interestingly, and as can be seen from Table 2, the proposed STM overestimates the strength of the two samples adopting the welded mesh reinforcement without any additional steel fibre content by (2-24%). This is perhaps to be expected given the brittle nature of unconfined concrete and the sudden and explosive failure observed in the testing of the element. This finding perhaps indicates that unconfined concrete elements should not be designed using STM models without a further safety factor being applied to the current strut capacity equation given within EC2 [8]:

$$\sigma_{RD,max} = 0.6\phi f_c$$

where $\sigma_{RD,max}$ is the allowable axial stress within the compressive strut, $f_c$ is the concrete cylinder
strength and $\nu' = 1 - (f'_{c} / 250)$ is a reduction factor applied for cracked compression zones within the Eurocodes. In contrast however, the STM model for samples where a 1% content of SFR, by volume was incorporated, tends to underestimate the capacity of the element by an average of 12%. This suggests that the use of standard STM design is valid for situations in which un-confined reinforcement configurations are adopted and perhaps even indicates that a beneficial factor of safety could be applied to the strut capacity expression (Eq 4) for such design cases. However, a much larger degree of testing would be required before any such conclusions or design recommendations could be provided. A potential need for such further investigation and the establishment of more appropriate correction factors is well illustrated by the comparison of actual lintel capacities to the ideal least squares correlation illustrated in Figure 7(b).

**Figure 7:** Proposed STM for the design of lintel members (a); Comparison of theoretical and experimental lintel capacities (b)

### 5 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

The paper demonstrates that the incorporation of Steel Fibre Reinforcement (SFR) has significant effects on the structural performance of both eccentrically loaded panels and shear discontinuity regions for precast elements adopting unconfined configurations for the traditional bar reinforcement. The paper also shows the effectiveness of design methods that could enable an engineer to justify the use of such hybrid reinforcing strategies in practice.

As far as the slender wall elements are concerned, the introduction of SFR was seen to increase both axial capacity and structural ductility for load eccentricities of $e=t/6$ and $e=t/3$, with a more significant improvement in the latter case. Moreover, an improved (and more acceptable) failure mechanism was observed, when compared to the sudden, brittle failure seen in the control samples. Lumped plasticity idealisation and fibre-hinge elements were shown to provide a good correlation with the experimental data relating to the singly and centrally reinforced panels adopting both traditional and SFR concrete mix alternatives. However, the computational method was found to be less effective in presence of steel fibres as secondary reinforcement, suggesting that further testing is required in order to calibrate the length of the fibre hinge.

As far as the lintels with dapped ends are concerned, it has been similarly shown that the introduction of SFR leads to increased capacity and ductility. This is believed to be because the fibres act to control cracking at the re-entrant corner, inducing a greater degree of flexural action prior to failure. The investigations conducted have also developed and validated a suitable Strut-and-Tie Model (STM) for the design of halving joint details where an unconfined steel reinforcement layout is
adopted, which however tends to overestimates the actual capacity. The findings also suggest that a modification (or safety) factor should be applied to the strut element to account for the brittle nature of the unconfined concrete without SFR. In contrast however, when a 1% volume of double-end hook SFR were introduced in the mix, the use of the STM design method could be justified, with the experimental values also indicating that a beneficial modification factor could be warranted. Also in this case, further testing would be required in order to adequately demonstrate and quantify what the value of such a beneficial factor should be.

REFERENCES