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EXTERNAL PRESTRESSED CFRP STRAPS FOR THE SHEAR ENHANCEMENT OF CONCRETE

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ABSTRACT

The use of fiber reinforced polymers (FRPs) for the strengthening and repair of existing concrete structures is a field with tremendous potential. The materials are very durable and hence ideally suited for use as external reinforcement. Although extensive work has been carried out investigating the use of FRPs for flexural strengthening (e.g. Meier et al., 1992) a fairly recent development is the use of these materials for the shear strength enhancement of concrete.

The current system investigates the use of post-tensioned non-laminated carbon fiber reinforced polymer (CFRP) straps as external shear reinforcement for concrete. Experiments were carried out on an unstrengthened control beam and beams strengthened with external CFRP straps. It was found that the ultimate load capacity of the strengthened beams was significantly higher than that of the control specimen.

Existing design codes and analysis methods were found to underestimate the ultimate resistance of the control specimen and the strengthened beams. Nevertheless, the modified compression field theory provided insight into possible failure mechanisms and the influence of the strap prestress level on the structural behaviour.

It is concluded that the use of these novel stressed elements could represent a viable and durable means of strengthening existing concrete infrastructure.

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INTRODUCTION

A significant number of existing concrete structures have insufficient shear capacity. There are several possible reasons for this deficiency: a reduction in strength as a result of deterioration, changes in design codes, increased traffic loadings, changes in use and poor initial design. If these structures can be strengthened then alternatives such as weight restrictions and/or demolition can be mitigated.

Work has been carried out where fiber reinforced polymers (FRPs) have been considered as passive shear reinforcement which is either internal (typically for new construction) or external (for repair and retrofit) to a structure. One key difference between FRP reinforcement and steel reinforcement is that FRPs do not yield. Hence, unlike in conventional design where the shear stirrups are assumed to yield, for FRP applications the actual strains in the shear stirrups are important.

An additional factor is that, whereas steel is isotropic, FRPs are anisotropic and the strength of the fibers in the transverse direction is low. When an FRP rod or laminate is bent into the form of a shear stirrup, significant strength reductions will occur in the corners of the element. The reduction in strength is a result of the differential stresses through the thickness of the FRP element.

The aim of the current project is to determine the feasibility of the use of non-laminated carbon fiber reinforced polymer (CFRP) straps as external post-tensioned shear reinforcement for concrete. The work focuses on aspects of the design of a system which would optimise the performance of both the concrete and the main tensile CFRP reinforcing elements. It is hoped that by providing active confinement to the concrete, and thereby subjecting the concrete to a beneficial tri-axial state of stress, an enhancement of the shear strength of a concrete beam can be achieved.

A key aspect of the proposed system is the use of carbon fiber reinforced thermoplastic tape. The tape is thin (typically between 0.12-0.16 mm thick) and is made up of high strength carbon fibers oriented in the longitudinal direction. As the tape is flexible, it can conform to numerous profiles and the high corrosion resistance of the material is well suited for external shear reinforcing systems.

Winistörfer (1999) carried out a comprehensive study of pin-loaded strap elements where the load was applied to non-laminated CFRP straps using two circular pins. When these straps are formed, layers of tape are wrapped around the pins. The outermost tape layer is fusion bonded to the next outermost layer which forms a closed ring that can carry hoop tension. However, the inner tape layers remain non-laminated. The straps were tensioned until failure. The study verified the principle that, in the region of the pin, the frictional forces generated between the tape layers are sufficient to induce force in the tape layers and to prevent the strap from unravelling. However, the relative sliding of the inner tape layers allows an equalisation of force in the layers. Strain gauge readings confirmed that each non-laminated layer carried a similar amount of force. An equivalent laminated system failed at a much lower load due to through thickness stress concentrations.

In the following, the work on the pin-loaded straps is extended to consider use as external prestressed reinforcement for concrete. For a practical application the sequence of events is as follows: layers of tape are wound around interface pad elements which are placed at the top and bottom of a beam, a strap is formed by fusion bonding the outer CFRP strap layer to the next outermost layer, the strap is then stressed and finally the load is transferred to the concrete (see Figure 1). It is noted that such a system could also be used as an anchorage element for external longitudinal sheets used for flexural strengthening. However, the focus of the current work is the shear enhancement of concrete.

FRP SHEAR SYSTEMS

For shear strengthening and repair, many of the current FRP systems consider the use of externally bonded fabrics. In these systems, either unidirectional or two dimensional fiber reinforced polymer sheets are bonded to the external surface of a beam (Drimoussis and Cheng, 1994, Täljsten, 1994). The fabric can be applied continuously along the length of the beam or in discrete locations. Furthermore, the sheets can be wrapped to suit a number of different configurations. The advantage of this system is that the fabric can readily be applied after the surface preparation of the concrete. The disadvantage is that there is a tendency for the fabric to peel off after cracking occurs and proper anchorage is essential. A further concern is that when the sheets are applied continuously along the length of a surface, any moisture in the concrete becomes trapped under the sheet and may result in unexpected deterioration.

Work has also been carried out on the use of laminated L-shape CFRP sections as shear reinforcement. An L-shape is placed on either side of the web and the horizontal portions of the L overlap at the base of the beam. For T-sections, holes are drilled through the top flange of a beam to provide additional anchorage for the laminate (Czaderski, 1998).

Many authors use existing conventional models for shear (such as the truss analogy) to predict the behaviour of the external FRP reinforcement. However, instead of assuming that the stirrups yield (as is usually the case with steel), the predicted failure is taken to be either a function of a limiting strain in the FRP sheet or strip (Chajes et al, 1995, Clarke and Waldron, 1996) or a function of an assumed bond stress which would result in the peeling of the FRP fabric (Uji, 1992). Since the modulus of elasticity of FRPs is lower than that of steel, a factor which considers the relative stiffness of the FRP when compared to steel is often included in the design equations. Triantifillou (1997) modified the approach of a limiting strain and suggested that rather than considering an arbitrary absolute strain at which failure occurs, an effective failure strain, which is a function of the stiffness of the FRP and the FRP reinforcement ratio, should be used.

In all of the aforementioned systems, the reinforcement is passive and will not influence the shear behaviour until the concrete has cracked.

The proposed use of non-laminated carbon fiber reinforced polymer straps will overcome many of the disadvantages of earlier systems. The inefficiency that is prevalent in bent FRP laminated sections, as a result of stress concentrations in the bent regions, can be mitigated by using a non-laminated strap (Winistörfer, 1999). In addition, the proposed solution is very flexible and the profile of the tensile element does not have to be specially fabricated for a particular application (such as with laminated sections). Peeling is not an issue and the closed loop system does not need to be anchored in the concrete. Another benefit of the closed loop configuration is that the unexpected reversal of load paths can be accommodated. Furthermore, the straps connect and entirely surround the concrete compression and tension zones; such connection is felt to be important for the concrete shear resistance. A rather subjective feature is that since the CFRP straps are relatively thin, the aesthetic appearance of a member strengthened using the straps is expected to be more pleasing than that of a structure strengthened with thicker laminated sections.

There are further advantages over an equivalent steel system. In particular, the CFRP is durable and light-weight. The reduced mass of a CFRP system is a key consideration for structures spanning motorways. If an external stirrup should fail, the danger to live traffic under the bridge is greatly reduced.

However, perhaps the most important feature of the system is the ability to provide active confinement to the concrete which will potentially enhance the shear capacity of the beam. This aspect will be discussed in more detail later.

SHEAR RESISTANCE OF A CONCRETE BEAM

In a paper by Collins et al. (1996) it was stated that there are approximately 43 different empirical equations in the shear design provisions of the 1995 American Concrete Institute Code. It is clear that shear is not yet fully understood and that there are conflicting ideas about possible shear mechanisms. A further complication is that the influence of external active shear reinforcement is rarely considered in design.

Failure Mechanisms

It is expected that the shear failure mechanisms will differ depending on the loading arrangement, in particular, the ratio between the shear span, a, and the effective depth of the beam, d. For fairly small and large values of *a/d* the full flexural capacity will be reached (Kotsovos, 1983, Fenwick and Paulay, 1968). For intermediate values of *a/d*, web-shear or flexure-shear failures are likely. In addition, the loading arrangement will most likely influence the portion of the shear force resisted by either the tension zone or the compression zone of the concrete (Fenwick and Paulay, 1968).

Tension Zone vs. Compression Zone

The shear resistance of a concrete beam is thought to be comprised of a combination of aggregate interlock, dowel action, the concrete compressive zone and the shear reinforcement (where present). However, the relative importance of the tensile zone of the beam (the combination of aggregate interlock and dowel action) and the concrete in the compressive zone remains unclear.

Aggregate interlock: Aggregate interlock occurs after cracking and is a result of the resistance due to the two faces of concrete on either side of the crack sliding against one another. Aggregate interlock can be an important factor and in beams tested by Fenwick and Paulay (1968) with an *a/d* ratio greater than 3 (final failure due to diagonal cracking) it was found that approximately 60% of the resistance of the beam was a result of aggregate interlock.

Although it is difficult to quantify the possible influence of active prestressed shear stirrups on the resistance due to aggregate interlock, it is expected that, for a given shear load, the presence of the straps will reduce the crack widths and increase the amount of shear transferred across a crack. Hence, if aggregate interlock is the dominant mechanism, the shear resistance should be enhanced by the straps.

Role of the concrete in the compressive zone: Kotsovos (1988) disputes the idea that aggregate interlock is a dominant mechanism. He argues that aggregate interlock must require a large shearing movement between the two crack faces. It would then be expected that localised crack branching would occur in regions where these high shearing forces are present. However, this type of crack branching has not been observed experimentally. Furthermore, on the basis of his own experimental work, he found that the crack widths were an order of magnitude greater than those considered by Fenwick and Paulay (1968) and suggests that in such a case aggregate interlock would play a much smaller role.

Kotsovos (1988) suggests that the majority, if not all, of the shear resistance at the ultimate limit state is carried by the concrete compressive zone and he proposes a model which assumes that the compressive force follows a bi-linear path from the middle cross-section to the supports. A shear failure will occur as a result of tensile stresses developing in the compressive force path and the failure modes will change depending on the ratio, *a/d*.

If the role of the concrete in the compressive zone is the most important mechanism for the shear resistance of the beam then it is expected that the presence of additional confinement should increase the capacity of the member by superposing vertical compression on the tensile stresses required in Kotsovos' model, delaying their reaching the tensile strength.

EXPERIMENTAL DETAILS

An experimental study was carried out to determine the influence of the post-tensioned straps on the shear capacity of a concrete T-beam. An unstrengthened control specimen and a strengthened beam with straps at 200 mm centres were investigated (beam T1-1).

Beam Dimensions

A beam with web width of 150 mm and a flange depth of 120 mm was chosen. The longitudinal reinforcement was 4 No. 26 mm ϕ distributed in two layers (see Figure 2). The effective depth of the beam was 425 mm and the yield strength of the steel, f_y, was 500 MPa. A small proportion of internal steel stirrups were included in the beam in order to imitate a beam with insufficient shear stirrups. The spacing of the internal steel stirrups (6 mm diameter) was 400 mm. The top flange steel was a series of 6 mm bars at 200 mm centres. To avoid the possible longitudinal splitting of the concrete small diameter confining stirrups at approximately 400 mm centres were wrapped around the 4 longitudinal bars. The concrete cover varied between 20 and 30 mm.

Concrete Parameters

The concrete used was a standard Swiss B35/25 mix supplied by a ready-mix concrete company. With this particular mix, the expected 28 day mean cube strength was approximately 40 MPa.

The maximum aggregate size of the mix is 32 mm with a grading curve corresponding to SIA 0-32 mm. The expected water/cement ratio is 0.50 and the cement content, 300 kg/m^3.

Prestressed Shear Reinforcement

The tape used for strengthening was a 0.16 mm thick by 12 mm wide carbon/nylon tape produced by Sulzer. The modulus of elasticity of the tape was approximately 130000 MPa and the strain to failure about 1%. Further details about the tape can be found elsewhere (Winistörfer, 1999). The straps were made up of 20 layers of tape. Preliminary tests on the strength of the straps when wrapped around 150 mm wide steel interface pads were carried out. Based on these results the straps were expected to fail at a load of about 100 kN.

Loading Arrangement

The control beam was designed so that a shear failure would occur before the longitudinal steel in the beam yielded. The capacity difference between a shear failure and a flexural failure represents the shear strengthening envelope.

On the basis of results described earlier (Kotsovos, 1983, Fenwick and Paulay, 1968) it appeared that a shear span over depth ratio around 2.5-3 would promote a shear failure. A shear span of 1100 mm was therefore chosen (a/d = 2.6). Flexural yield of the longitudinal reinforcement was predicted at an applied shear load of approximately 400 kN.

EXPERIMENTAL PROCEDURE

Beam Preparation

The specimens were cast in a 6.7 m long mould and two 3.35 m beams were cast at the same time. Concrete control specimens were also cast. After casting, the beams were covered with a sheet of polythene. The beams were air-cured and were removed from the formwork a week after casting.

Strap Preparation

In preparation for testing, 30 mm holes were drilled through the top flange of the concrete to accommodate the CFRP straps and to allow sufficient room to thread the tape through the holes.

The layout of the straps for beam T1-1 can be seen in Figure 2. It is recognised that, because of the shear enhancement near a support, the first stirrup is typically located a distance between *d/2* and *d* from the support. Hence in the current work an end distance of only 150 mm is conservative but the alternative spacing of 350 mm was considered to be possibly excessive. Adjacent to the load point, the spacing between the stirrups is 300 mm to allow some space on either side of the loading jack. The straps were also placed in the centre of the span. This was done to cover the possibility of a crack extending into the region between the load points and causing premature failure (Kotsovos, 1983).

A specially designed interface steel pad was grouted on to the bottom face of the beam. An equivalent interface pad was placed on a steel base plate on the top surface of the beam. The tape was wrapped around the pads until the required number of layers was achieved. The free end of the tape was fusion bonded over a length of 100 mm to the next innermost layer. In order to quantify the behaviour of the strengthened beam, the force in the straps was monitored using strain gauges that were attached to the straps. In particular, a strain gauge was placed on the outer tape layer of each strap at the mid-depth of the vertical leg. On the basis of the work on pin-loaded straps, the strain in the inner tape layers could be considered to be similar.

The straps were located at 200 mm centres and were tensioned by lifting the steel base plate on which the top pad was supported. The steel base plate had four holes with vertical screw threads to which bolts were attached. These bolts were connected to an upper plate and a hydraulic jack (supported on a frame) was used to tension the strap. The prestress force was determined by monitoring the strain in the strap and the straps were stressed to approximately 60 kN (0.6 f_{pu}). There was some difficulty in controlling the exact level of prestress hence there was some variation between the initial stress in the straps. However, the average stress through the concrete web would be approximately 2 MPa. The strain readings in the straps appeared to be stable after stressing and there were no noticeable losses due to creep and/or relaxation.

Testing

The beams were tested in four point loading. The clear span was 2700 mm and the supports were located 325 mm from the beam end. Displacements were measured at a total of 5 places along the beam (see Figure 2). The load in the jacks was also recorded.

At the time of testing, the concrete compressive cube strength was 59 MPa.

EXPERIMENTAL RESULTS

Aside from the external straps, the control specimen and the strengthened beam T1-1 were identical. As the beam T1-1 did not fail prior to yielding of the longitudinal steel, several straps were removed and the same beam was re-tested (Beam T1-2). The load deflection curves for the three tests can be found in Figure 3. In the following, the load indicated is the load on a *single* jack (this value represents the shear force in the shear span).

Control Specimen

Flexural cracking occurred at a single jack load of 60 kN and the first flexural shear cracks appeared at approximately 100 kN. Cracks continued to form until a load of approximately 305 kN when a single major shear crack opened and appeared to lead to failure.

Specimen T1-1

The first visible flexural cracks were noted at a load of 60 kN although from the change in the slope of the load deflection curve it appeared that cracking had actually started slightly earlier. The cracks occurred in between straps 3 and 4 and between straps 4 and 5 (the numbering system for the straps is shown in Figure 2). With increasing load these cracks extended. By a load of 110 kN, the first crack occurred between straps 2 and 3. This crack started a distance of 70 mm from the bottom of the beam and was inclined at an angle of approximately 70° but did not pass through a stirrup. The strain readings in stirrups 2 to 5 and 9 to 11 started to increase at a load of 150 kN (see Figure 4) and by a load of 180 kN extensive shear cracking could be seen in both spans. More shear cracks formed with increasing load. Two major cracks were forming – one at an angle of 45° and one at an angle of 25°. At a load of approximately 420 kN, flexural yield of the longitudinal bars started to occur and the test was stopped. At this stage, none of the straps had broken.

Specimen T1-2

As the beam had not failed in the test T1-1, it was possible to reuse the specimen. In particular, straps 2, 4, 9 and 11 were removed and the beam reloaded. As expected, because the concrete was already cracked, the stiffness of the beam was lower. With increasing load, the existing cracks opened up. As the beam approached a load of 405 kN (close to the load level at which the yielding of the longitudinal bars would occur), strap 3 failed catastrophically.

ANALYSIS

Prior to cracking, the predicted inclined and flexural cracking loads can be calculated from elastic theory. Although the straps may provide a slight localised horizontal stress, as well as the main vertical compression, the flexural cracking load would not be expected to increase significantly due to the presence of the straps. In contrast, the inclined cracking load is predicted to increase with increasing prestress.

To investigate the ultimate behaviour of the beams, several different analysis methods were considered; the truss analogy and the modified compression field theory.

Truss Analogy

Often the ultimate strength of a reinforced concrete beam is considered to be the sum of the resistance of the concrete, V_c, that of the stirrups, V_S, and the shear resistance due to the longitudinal prestressing V_p (where applicable). This is the basis of many design codes. In the following, the American Concrete Institute Building Code, ACI-318-95 (1998), Eurocode 2 (EC2) (1992), and the British Department of Transport Assessment code, BD44/95 (1995) are considered. An important distinction between the codes is that EC2 and ACI 318 are for new construction where BD44/95 is for assessment. Hence it is expected that BD44/95 is likely to be the less conservative. Reference should be made to the codes for comprehensive details of the code equations and conditions.

Predicted beam resistance: The expected concrete contribution for the experimental specimens was calculated using the design codes described above. The results have been plotted in Figure 5 as a function of increasing shear span. In this figure, ACI-1 refers to the ACI 318 equation that is a function of the beam depth, the beam width and the concrete compressive strength. However, the ACI code also endorses a more detailed calculation of the concrete contribution where the reinforcement ratio, the applied moment and shear force are also considered. This equation has been denoted as ACI-2. No account of any beneficial effects due to the prestress have been included.

Using the 45° truss analogy, the shear resistance of the internal steel stirrups can be calculated as:

$$V_S = \frac{f_{yv} \cdot d \cdot A_{sv}}{s_v} \qquad (1)$$

where d is the effective depth of the beam, f_{yv} is the yield stress of the steel, A_{sv} is the area of the two legs of the stirrup and s_v is the spacing.

Using this formula, the predicted shear resistance of the internal steel links was approximately 30 kN. On the same analogy, the maximum shear resistance expected to be contributed by the

CFRP straps spaced at either 200 mm or 400 mm centres would be approximately 210 kN and 105 kN respectively.

The codes were written for the design of steel shear reinforcement and thus the design equations are in terms of the yield stress of the steel stirrups. As FRPs do not yield, the code values are not representative. In the Institution of Structural Engineers (IStructE) Interim Guidance document (1999) for the design of reinforced concrete structures using FRPs (taking the material safety factors to be unity), f_{yv} in Equation 1 should be replaced by $0.0025E_{rv}$ (effectively limiting the strain in the strap to 0.0025 where E_{rv} is the modulus of elasticity of the FRP). For an unstressed strap with 20 layers (breaking load approximately 100 kN) at either 200 mm centres or 400 mm centres, the expected resistance is 45 kN or 25 kN respectively. However, again it is important to note that this guidance was developed for passive internal reinforcement.

Modified Compression Field Theory

The modified compression field theory (MCFT) is one of the few analysis methods that considers the actual strains in the concrete (Vecchio and Collins, 1986). It was therefore considered a possible means of elucidating the behaviour of the experimental beams.

The theory is based on the average properties and the conditions of compatibility and equilibrium of a cracked element. Reference should be made to Vecchio and Collins (1986) for a comprehensive overview of the methodology and assumptions.

In order to include the CFRP straps in the analysis, the straps are modelled as a uniform stress in the vertical *y* direction. This is not strictly the case as the vertical prestress will vary at different sections along the beam. The applied vertical force is a function of the vertical strain in the concrete, due to the elastic properties of the straps. Although more work needs to be carried out to confirm the relationship between the strap strains and the vertical displacements, as a first estimate, the strap behaviour can be considered to be analogous to that of an unbonded prestressed tendon anchored at the top and the bottom of the beam.

It is possible to investigate the actual distribution of shear stress through the depth of the section but in the current work the shear stress distribution is assumed to be uniform. In the first instance, the stress and strain distributions, including the strain in the longitudinal direction ε_x , are calculated assuming no moment on the section. The flexural strains are added by fixing the longitudinal strain at mid-section to be ε_x (found assuming no moment) and calculating the additional strain distribution that must be added in order to satisfy moment equilibrium. The predicted beam resistance using this method will be discussed in the next section.

DISCUSSION

The experimental and analytical results gave insight into the influence of the strap strengthening system. In particular, the capacity of the beam was significantly enhanced due to the presence of the CFRP straps. Whereas the unstrengthened control beam exhibited a fairly brittle shear failure at a load of approximately 305 kN, the testing of the strengthened beam T1-1 was stopped at a load of approximately 420 kN at the onset of the ductile yielding of the longitudinal steel.

When the load deflection behaviour of specimen T1-1 and the control specimen are compared (Figure 3), it can be seen that although the first inclined cracks appeared at similar loads, initially there was only a small loss of stiffness in specimen T1-1. It is felt that this is a result of the fact that the shear cracks did not extend due to the presence of the confined regions under the straps. This feature is significant for serviceability reasons.

The presence of the prestress also seemed to increase the angle of inclined cracking (which would be expected from Mohr's circle). There is a possibility that, if the angle of cracking becomes too steep and fewer shear stirrups carry the load, this could lead to failure at a load that is lower than expected. This would be a particular concern for flexure-shear cracking and will be the subject of future work.

As expected, the stiffness of T1-1 was greater than that of specimen T1-2. In particular, except at the final stages of the test, only the existing cracks from test T1-1 opened up and virtually no new crack formation was noted. An interesting observation is that, at the peak load, the sum of the additional strain in straps 1-5 in test T1-1 is similar to the total additional strain measured in straps 1, 3 and 5 in test T1-2 i.e. T1-1($\Delta\varepsilon_1+\Delta\varepsilon_2+\Delta\varepsilon_3+\Delta\varepsilon_4+\Delta\varepsilon_5$) $\approx$ T1-2($\Delta\varepsilon_1+\Delta\varepsilon_3+\Delta\varepsilon_5$). Hence, the total force *increment* carried by the straps was comparable in both tests.

Predicted Behaviour of Control Specimen

The control specimen failed at a higher load than expected. If the contribution of the internal steel stirrups is calculated using Equation 1 and the concrete contribution is in the region of the code values shown in Figure 5, then the beam should fail at a maximum load of about 120 kN. Yet the beam failed at 305 kN. If the calculated shear resistance of the steel stirrups is subtracted from the ultimate capacity of the control beam, then the actual concrete contribution, V_c , is around 275 kN. This value is much higher than the code predictions. The predicted beam resistance using the modified compression field theory is approximately 170 kN.

There are several possible reasons for the underestimate of the failure load. If the beam behaves like a tied arch, these sort of loads can be reached. In addition, the code equations are possibly conservative for beams with large compression flanges.

Predicted Behaviour of Strengthened Beams

Since the testing of beam T1-1 was stopped at the onset of flexural yielding, it is only possible to state a lower bound on the shear strengthening capability of the straps at 200 mm centers. However, beam T1-2 failed at a load of 405 kN due to a strap breaking and thus the presence of the straps at 400 mm centers increased the ultimate resistance of the beam by 100 kN.

If the truss analogy is used to calculate the contribution of the external CFRP straps then the predicted increases in load capacity for the strengthened beams are consistent with the experimental results. If a strain limit is imposed on the straps (as suggested in the IStructE guidance document) then both beams should have failed in shear at a load lower than that obtained in the experiments.

It is unclear whether conventional code design methods are suitable for the ultimate analysis of the strengthened beams. As discussed earlier, design codes typically do not take into account a possible increase in the concrete contribution due to the presence of the stressed shear reinforcement. In addition, as evidenced by the control specimen, the codes can be conservative for particular types of specimens and loading arrangements. Furthermore, the truss analogy is a plastic method but yet the CFRP straps will behave elastically.

Predictions using the modified compression field theory were also considered. Of particular interest is the determination of the strain in the vertical direction as this can be compared with the experimental strap strain gauge readings. However, again, a concern about applying this approach to the strengthened beams is that the modified compression field theory under-predicted the load capacity of the unstrengthened control specimen (see above).

In Figure 6, the strain in strap 3 in test T1-1 is compared with the predicted vertical strains using the modified compression field theory. The figure also includes a comparison of the strains for test T1-2 although it is important to note that this beam was precracked prior to testing. Although the general shape of the curves is similar, the modified compression theory underestimates the applied load at which a particular strain occurs. In addition, the failure load of beam T1-2 was underestimated.

Using this theory, a parameter study of the influence of the prestress levels was also carried out. In particular, the predicted resistance of the experimental beam with either no straps, unstressed straps or straps stressed to 20%, 40%, 60% or 80% has been plotted in Figure 7. The applied moment was taken to be zero and the strap spacing, 200 mm.

In the strengthened T-beams, the CFRP straps have a significant load carrying capacity, as do the concrete compressive struts. Hence, for beams with a low prestress, the concrete will not crush nor will the straps fail until fairly high shear loads (typically the failure strain of the CFRP strap controls). However, it is felt that these loads are not necessarily achievable and that local shear failure mechanisms in the concrete will potentially occur at an earlier stage. Therefore, in Figure 7, a number of indicative limits have been included. The first is a limit on the average crack width, w, of 1.5 mm. The second is a strut angle limit, θ, of 30°.

More work needs to be done in quantifying the failure limits and a certain caution is required since the MCFT predicted capacity of the unstrengthened control beam (no straps) was significantly lower than that determined experimentally. Nevertheless, a number of preliminary observations can be made. It is of interest that it is likely that the failure modes will change depending on the prestress level. For low levels of prestress the failure is likely to initiate in the concrete, at higher levels of prestress, the strap breaks. In addition, the stiffness of the structure increases with increasing prestress.

FUTURE WORK

Work continues on determining the influence of the strap prestress level and the strap spacing on the behaviour of reinforced concrete beams. As demonstrated in the parameter study, there appears to be an optimum level of prestress for the stiffness and the strength of the structure. Furthermore, the design prestress level will be connected to the philosophy of what the allowable design stress in the CFRP strap should be. On one hand a lower prestress level will ensure that there is a large reserve capacity in the CFRP shear element and that the structure will be predisposed to failure in the concrete. On the other hand, a higher prestress would provide more confinement to the concrete and a stiffer load deflection response. However, failure of a strap is brittle and sudden.

There are also conflicting requirements in determining the spacing of the straps and a more detailed analysis of the vertical prestress distribution through the concrete is the subject of an ongoing investigation. By increasing the spacing, the number of strap elements which need to be tensioned will reduce. The minimisation of the number of straps is likely to be of importance since labour costs will be a consideration. However, if the spacing is too great, unconfined regions will exist between the straps and premature failure could occur in the unconfined zones. The influence of the shear span to depth ratio will be also be a factor in determining the effectiveness of the system.

The strain gauge reading on the outer tape layer was taken as being indicative of the strains throughout the strap. However, the possible influence of any misalignment of the gauge, the position of the gauge on the strap, and any variation of strain between layers requires further consideration. Work on practical installation issues, the long-term behaviour of the CFRP straps and the development of an all-composite interface support pad continues.

CONCLUSIONS

(1) Non-laminated prestressed CFRP straps represent efficient and durable reinforcing elements for infrastructure applications.

(2) A concrete beam strengthened with CFRP straps exhibited a significantly higher load capacity than an unstrengthened beam. The presence of the straps seemed to increase the angle of cracking and resulted in a structure which was much stiffer than an equivalent unstrengthened member.

(3) The code equations underestimated the ultimate load capacity of the control specimen. The modified compression field theory provided insight into the possible behaviour of a strengthened or unstrengthened beam under loading. However, again the ultimate load capacity was underestimated.

(4) Further work is required to investigate the influence of the loading arrangement, the prestress level, and the strap arrangement, on the behaviour of a strengthened beam.

(5) The use of external prestressed CFRP straps can be an effective means to enhance the shear capacity of a concrete beam.

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APPENDIX II. NOTATION

a	shear span
A_{sv}	area of shear reinforcement
d	effective depth of the beam
E_{rv}	modulus of elasticity of the FRP
f_{pu}	ultimate stress of the strap
f_y, f_{yv}	the yield strength of the steel
s_v	spacing of the stirrups.
V_c	shear resistance of the concrete
V_p	shear resistance due to the longitudinal prestressing
V_s	shear resistance of the stirrups
w	average crack width
ε_x	longitudinal strain
ε_j	measured strain in strap j
ϕ	bar diameter
θ	average angle of concrete compressive strut

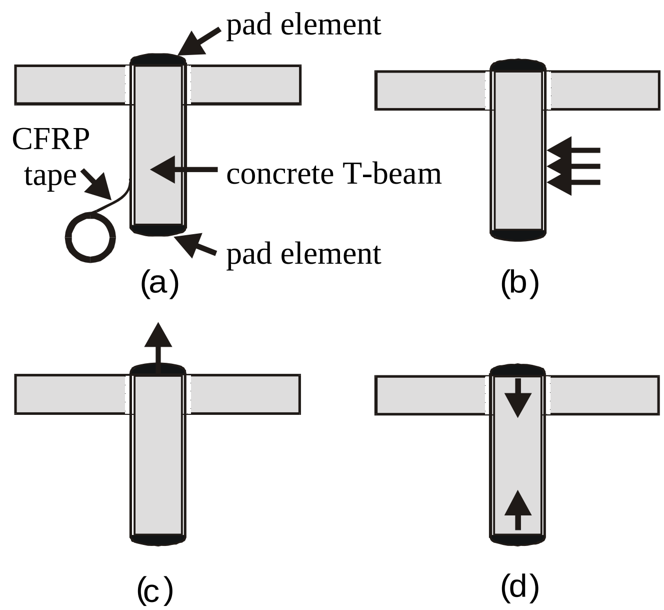

Figure 1: Practical application of non-laminated strap elements (schematic); (a) wrap beam, (b) fusion bond outer layer to second outermost layer, (c) tension strap, (d) transfer of prestress force to concrete

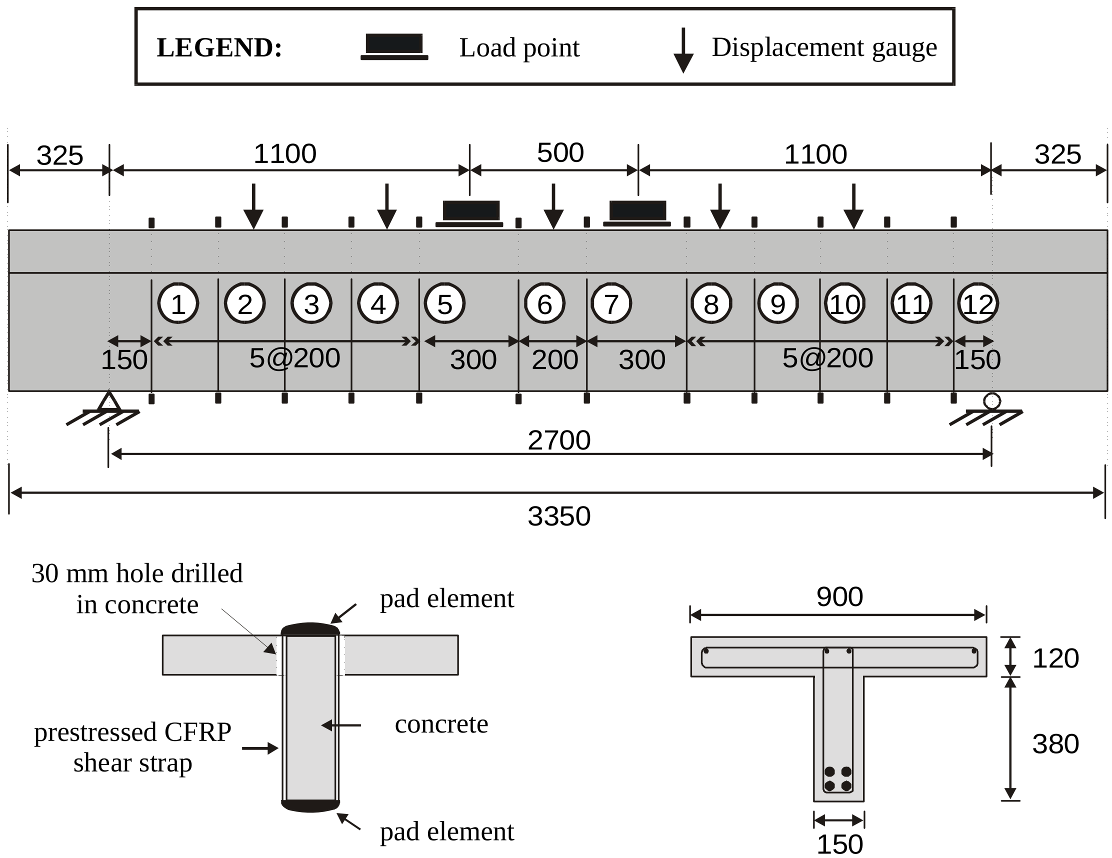

Figure 2: Beam dimensions, strap layout and loading arrangement

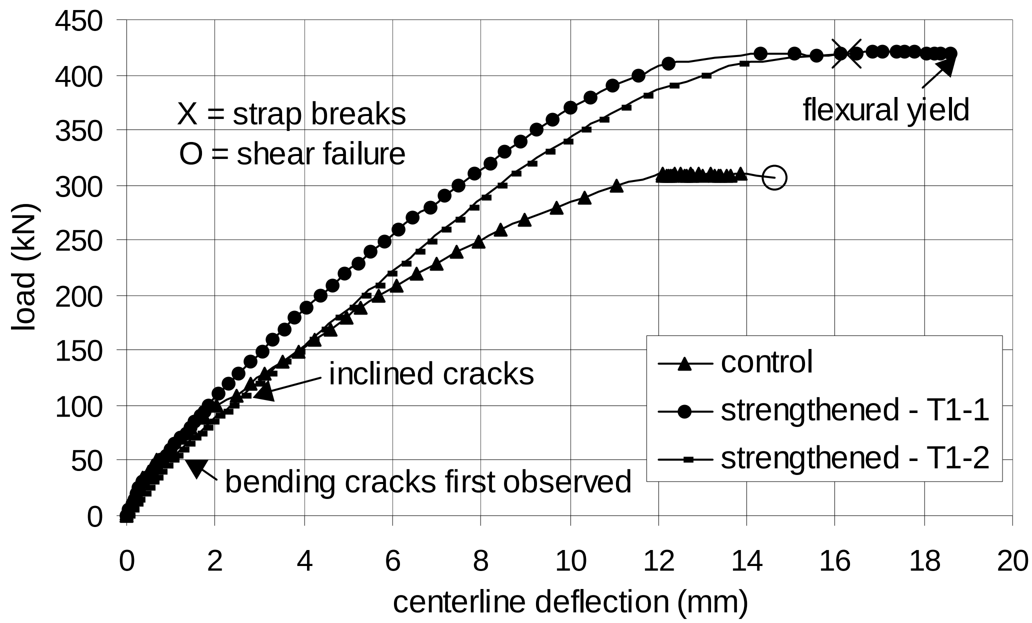

Figure 3: Load deflection curves

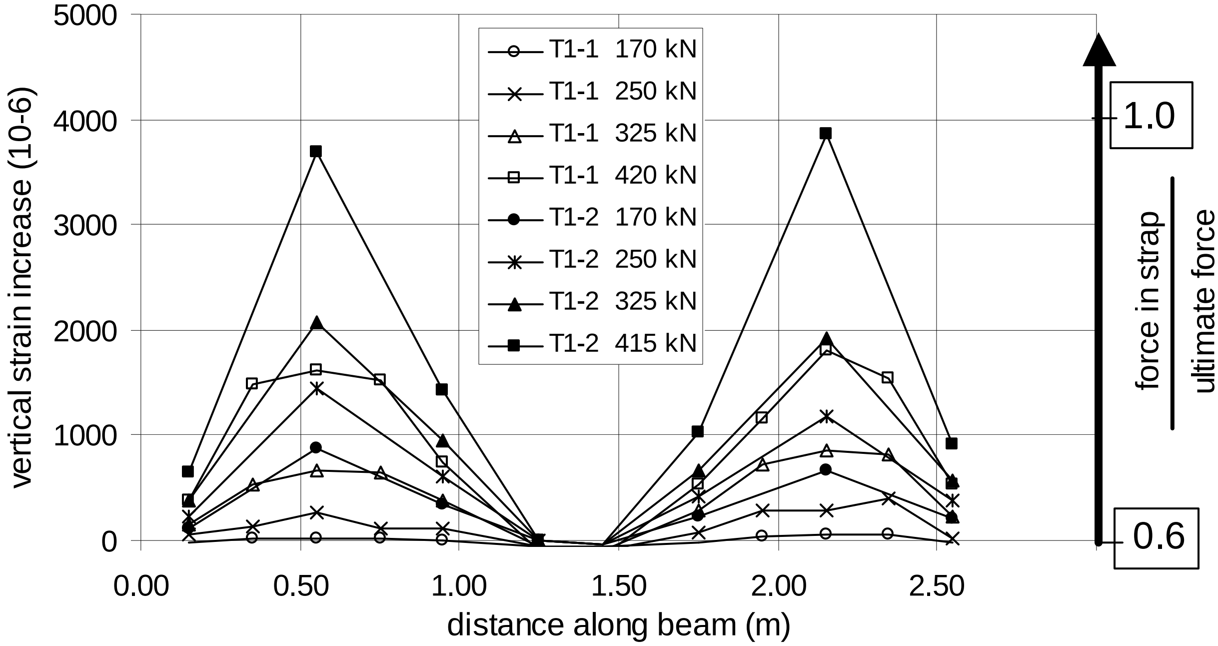

Figure 4: – Strain increase in straps for tests T1-1 and T1-2

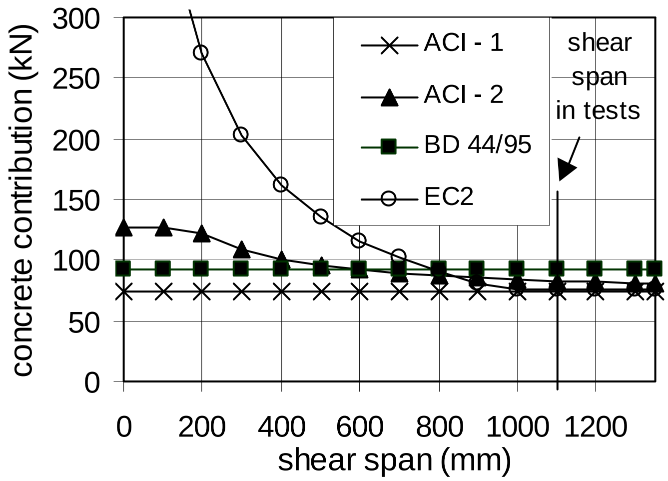

Figure 5: Predicted concrete contribution

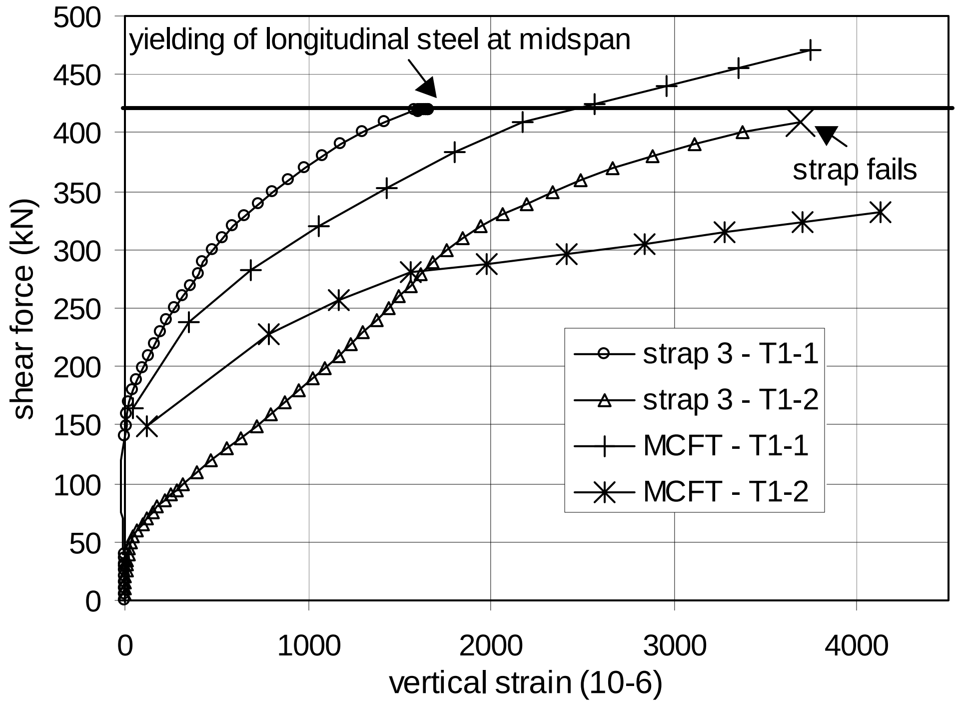

Figure 6: Comparison of vertical strains predicted using modified compression field theory and experimental strap strains

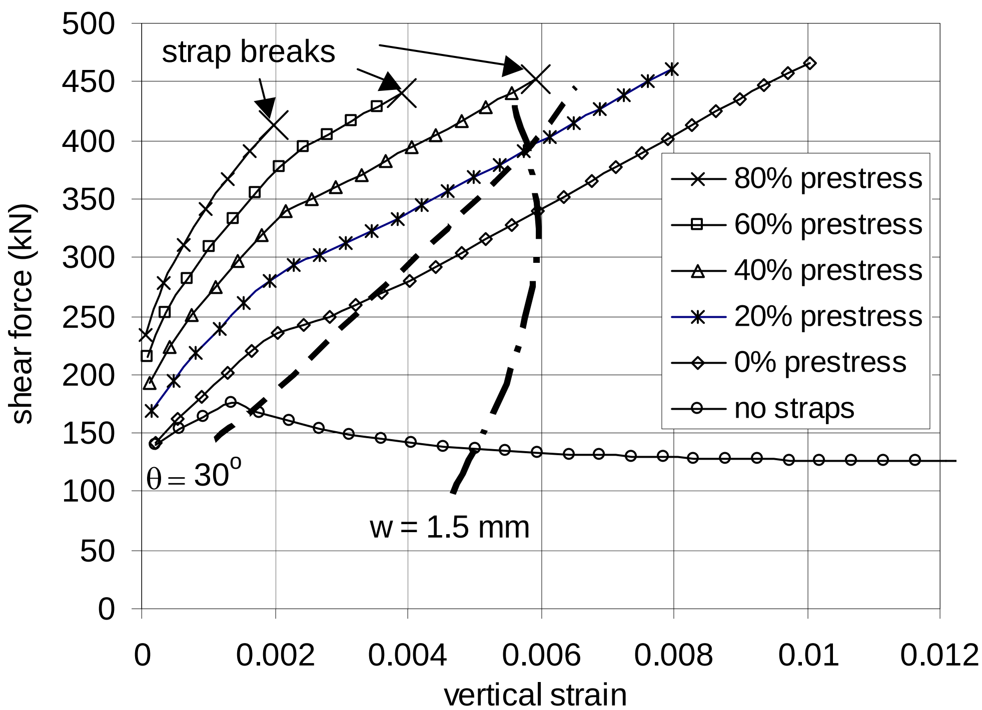

Figure 7: Effect of variation of prestress