THE SHEAR STRENGTH OF CONCRETE CONTAINING FIBRE-REINFORCED-PLASTIC (FRP) REINFORCEMENT

Tim James Ibell

Lecturer

Dept of Architecture and Civil Engineering, University of Bath, Bath, BA2 7AY, United Kingdom

Chris John Burgoyne

Lecturer

Dept of Engineering, University of Cambridge, Trumpington Street, Cambridge, CB2 1PZ, United Kingdom

Abstract

The use of fibre reinforced plastics *(FRPs) for the reinforcement of concrete structures is receiving much attention at present. Attempts are being made to understand better the behaviour of concrete containing these fibrous materials. However, much of this work is aimed at attempting to fit present design guidelines to this wholly new construction material. This paper presents push-off tests and suggests analytical techniques for the shear capacity of concrete reinforced with FRPs. As a basis for comparison, tests on specimens containing steel stirrups are also presented. It is shown that while plasticity theory may be considered appropriate for use in the steel-reinforced situation, other analysis techniques are required for FRP-reinforced specimens. This is because the brittle nature of the new materials makes them susceptible to localised stress concentrations. This means that bond characteristics of these materials are particularly important. It is concluded that the choice of an appropriate analysis technique depends on the amount of debonding which FRPs are able to undergo during shear collapse of the concrete structure.*

1. INTRODUCTION

In order to combat the corrosion of steel reinforcement in concrete structures, there are many options open to the engineer during the design phase. The use of cathodic protection methods, epoxy-coatings, galvanising or specification of stainless steel may all be considered. Alternatively, the use of *steel* reinforcement may be abandoned altogether. The use of *fibre reinforced plastics* (FRPs) is now being considered for reinforcement of concrete structures (1). These fibres hold a number of advantages over steel. Firstly, they do not corrode. Secondly, they are lighter than steel, which is useful for construction purposes. Thirdly, they exhibit an ultimate tensile strength greater than that of either reinforcing or prestressing steel. Finally, the amount of concrete cover needed to protect FRPs is reduced, leading to lighter structures overall.

Drawbacks of these materials include purely elastic, brittle behaviour under increasing load. Further, FRPs have lower stiffnesses than steel, so that larger structural deformations may be expected. FRPs are presently also expensive, and importantly, cannot easily be bent into shapes or otherwise manipulated on site.

Some work has been conducted on the shear capacity of concrete reinforced with FRPs (1)(2). This work has been aimed specifically at determining the modifications which would need to be made to *existing* codes of practice to ensure that designs using such materials would be safe. While such work is desperately needed, the question arises as to whether or not the FRPs are being used as efficiently as possible (3).

As a first step in attempting to answer this question, it was considered necessary to conduct a series of push-off tests on concrete shear blocks reinforced with steel stirrups (4), glass fibre reinforced plastic (GFRP) stirrups, and aramid fibre reinforced plastic (AFRP) helices (5). The results from such testing are discussed in terms of shear resistance, ductility, bond, and applicability of suitable theoretical models.

2. EXPERIMENTAL PROGRAMME

2.1. DETAILS OF PUSH-OFF SHEAR TESTS

Mattock (6) carried out several push-off tests on concrete shear blocks, in order to obtain the relationship between the shear and normal stress acting along slip planes in steel-reinforced concrete structures. A similar approach has been adopted here.

Figure 1(a) shows a schematic of the shear blocks used throughout all testing. Where stirrups were used as the shear reinforcement, these were fixed around the *outside* of the secondary reinforcement cages (figure 1(b)). Where the aramid helices were used, these were fixed in the mould with the central helix crossing the shear plane and two outer intersecting helices wrapped around the secondary reinforcement cages (figure 1(c)).

The target compressive cube strength of the concrete was 60MPa when tested at 7 days. The steel stirrups of diameter 4 or 6mm were found to have a yield strength of 382MPa or 393MPa respectively. The GFRP stirrups, of cross-sectional area 20mm^2 exhibited an ultimate strength of 425MPa, with a Young's Modulus of just 4GPa. The AFRP helices, of cross-sectional area 10mm^2, had an ultimate tensile capacity of 1430MPa and a Young's Modulus of about 120GPa.

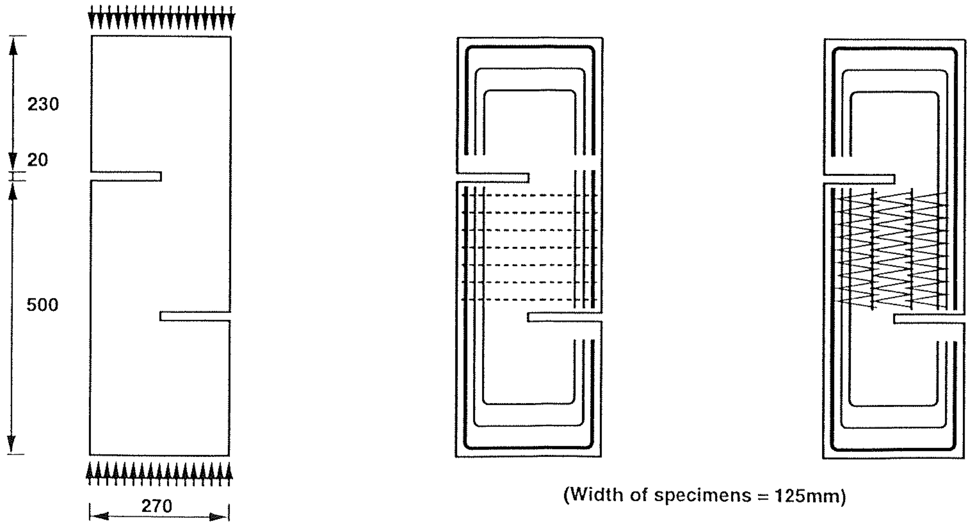

Figure 1. (a) Shear test specimens; (b) Steel and GFRP stirrup positions; (c) AFRP helix positions.

2.2. RESULTS FROM UNREINFORCED SHEAR BLOCKS

As a benchmark for all tests, two unreinforced specimens were tested initially. A catastrophic and exceptionally brittle failure was encountered in both cases. Table 1 shows details of the results obtained. In the same table are predictions from a linear elastic finite element analysis, assuming that when the tensile capacity of the concrete is exceeded at any point, collapse occurs. The agreement between results and theory is good.

2.3. RESULTS FROM STEEL-REINFORCED SHEAR BLOCKS

These tests proved to be initially brittle, followed by a ductile, plastic, energy-dissipating period thereafter. Much cracking preceded the ultimate capacity being reached. In actual fact, of the four tests conducted with steel stirrups, initial cracking along the central plane occurred at about 2/3 of the ultimate load on average. In addition, these initial cracks were orientated at about 15° to the vertical, which agreed well with elastic finite element predictions. The concrete struts between these cracks were observed to rotate during subsequent loading, as might be expected.

Table 2 provides details of the steel reinforcement and failure capacity for each specimen. Figure 2 shows schematically the ultimate state of the steel-reinforced specimens. Note the expanse of spalled concrete in the vicinity of the central crack. In every test, all steel bars yielded (strain gauges were attached to the stirrups), reflecting the reasonably ductile response of these specimens. It should, however, be noted that although prolonged crushing of concrete occurred along the central plane in a ductile manner, the ultimate capacity was reached at a vertical displacement of about 2-3mm in each case. Beyond this displacement, the sustainable load dropped off gradually. Figure 3 shows details of these load-displacement plots.

Table 1. Unreinforced specimen results.

Specimen Number	f_{cu} (MPa)	Actual Failure Load (kN)	Predicted Failure Load (kN)
1	58.8	159	164
2	68.2	155	155

Table 2. Steel-reinforced specimen results.

Specimen Number	f_{cu} (MPa)	Steel Shear Stirrups	Percent. Area of Stirrups	f_y (MPa)	Actual Failure Load (kN)
3	58.7	6 4mm diameter	0.5	382	260
4	70.8	12 4mm diameter	1.0	382	345
5	50.4	8 6mm diameter	1.5	393	408
6	62.6	11 6mm diameter	2.0	393	420

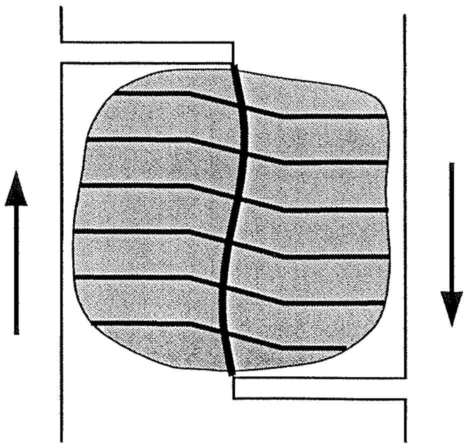

Figure 2. Spalling of steel-reinforced specimens

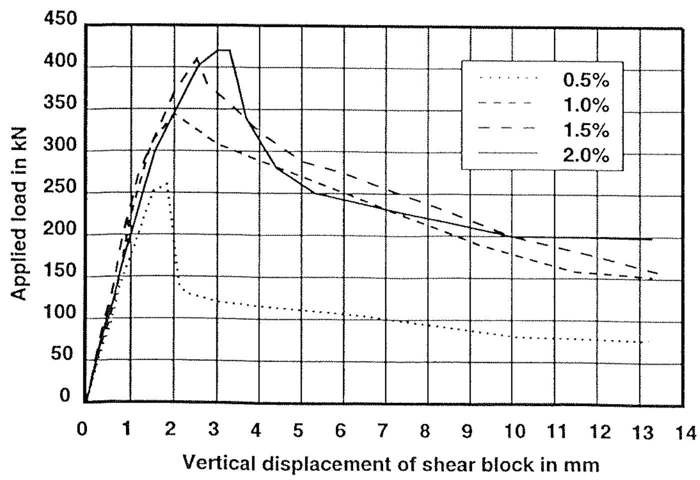

Figure 3. Displacement of steel-reinforced specimens

2.4. RESULTS FROM GFRP-REINFORCED SHEAR BLOCKS

A somewhat more brittle failure occurred in each case when reinforced with GFRP. However, unexpected ductility was exhibited beyond a vertical displacement of about 2-3mm, as had been the case for the steel-reinforced specimens. Table 3 shows details of the stirrup reinforcement and failure capacities. The form of failure encountered was not dissimilar to that of the steel-reinforced specimens (figure 2), signifying that a long section of the GFRP stirrups became unbonded, accompanied by spalling. Figure 4 shows the load-displacement plot for each specimen with GFRP stirrups. These plots highlight the initial peak load behaviour, followed by a plateau-like zone at a lower load. Increasing carrying-capacity occurred beyond this, until delamination of the stirrups caused abrupt failure.

Table 3. GFRP-reinforced specimen results

Specimen Number	f_{cu} (MPa)	GFRP Shear Stirrups	Percent. Area of Stirrups	Actual Failure Load (kN)
7	68.2	8 Stirrups	1.0	270
8	67.2	16 Stirrups	2.0	310
9	67.8	24 Stirrups	3.0	330

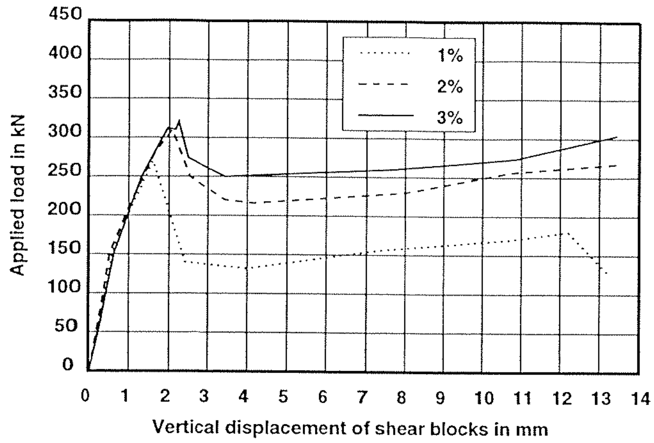

Figure 4. Displacement of GFRP-reinforced specimens

2.5. RESULTS FROM AFRP-REINFORCED SHEAR BLOCKS

An initially brittle form of failure was encountered for each of the helically-AFRP-reinforced shear block specimens. However, once again, marginal ductility (in terms of plastic energy dissipation) was displayed beyond a displacement of about 2mm and after the ultimate load had been reached. Table 4 shows details of these test results, while figure 5 shows the ultimate state of these helically-reinforced specimens. Several points are immediately apparent.

The ultimate strength of the specimens is not affected as greatly by the presence of an increasing quantity of reinforcing material as was the case for the GFRP-reinforced specimens. It is thought that this is largely due to the helical arrangement not being able to debond after initial cracking, leading to localised stress concentration and premature snapping of the AFRP helices. This is borne out by observation, where the limited spalling (figure 5) reflects such behaviour. Further, the helices seemed to snap at points where they had been tied (using thin steel wire) to the secondary reinforcement. It seems that the localised nature of debonding and stress concentration led to failure.

Figure 6 shows the load-displacement plot for each AFRP-reinforced specimen. It is clear from this plot that while *some* post-peak ductility is displayed, the previous results using GFRP (where debonding occurred over a longer section) are more desirable to a designer both from ultimate strength and behavioural standpoints. It is not implied that one material is superior to another for the shear reinforcement of concrete, but rather that it is the *geometry* and *bond* characteristics of the material that influence both strength and behaviour in shear.

Table 4. AFRP-reinforced specimen results

Specimen Number	f_{cu} (MPa)	AFRP Helical Windings	Actual Failure Load (kN)
10	49.7	7 Legs	170
11	65.1	10 Legs	165
12	49.7	14 Legs	225

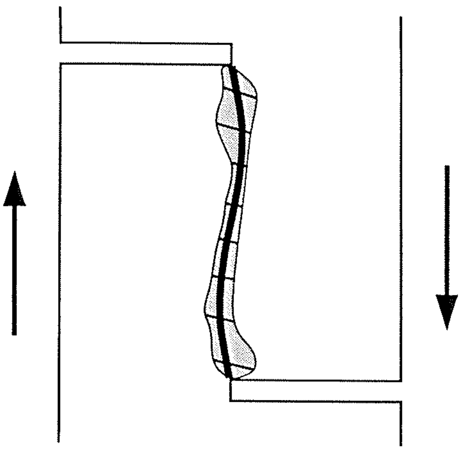

Figure 5. Spalling of AFRP-reinforced specimens

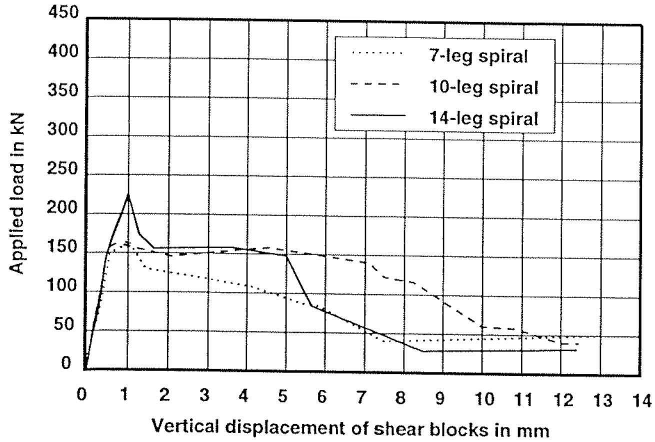

Figure 6. Displacement of AFRP-reinforced specimens

3. SHEAR ANALYSIS TECHNIQUES

Analytical tools are sought to *predict* the ultimate strength in shear of concrete reinforced with various materials. One option is the use of upper-bound plasticity theory. However, due to the relatively brittle nature of collapse of the FRP-reinforced specimens, plasticity theory seems to be only applicable to the steel-reinforced specimens, where comparatively more ductility was displayed. Other, equilibrium-based methods are probably more suitable in the case of FRP-reinforced concrete.

For the steel-reinforced specimens, a collapse mechanism could be postulated, upper-bound plasticity theory applied to this, and shear strength predictions obtained. Details of such a technique are available in Ref. (7), from which reasonably accurate shear strength predictions were obtained.

The GFRP-reinforced specimens underwent substantial spalling during collapse, with the GFRP stirrups debonding from the concrete. It would be inappropriate to adopt a plasticity approach in this case, as no 'yield' limit could be set for the GFRP material. Therefore, it seems that an equilibrium approach which considers the level of stress in the GFRP stirrups, the width of shear discontinuity and the shear slip itself would be most useful here. Such a method has been developed in Ref. (7). Again, predictions of shear strength were found to be reasonably accurate.

There was limited debonding and spalling noticeable in the tests with AFRP helical reinforcement. Further, failure occurred not by concrete reaching some stress limit, but by the AFRP snapping locally. It therefore seems that an analytical approach here could be to look at the stress levels in the AFRP only, and to infer that collapse of the concrete structure will occur when the AFRP snaps, in accordance with experimental evidence. Such a method has been developed (Ref. (7)) using a finite element approach, with encouraging results. The different analytical approaches described here seem to suggest that *ductility*, *bond characteristics* and *geometry* of the reinforcement are the important variables in determining which analysis tool to use in estimating the shear capacity of the concrete.

4. CONCLUSIONS

From the tests conducted here, the following conclusions may be drawn.

1. Steel-reinforced shear block specimens failed along a clearly defined discontinuity, initially in a brittle manner, but thereafter with some plastic deformation in a controlled manner. This suggests that the use of plasticity theory may be appropriate for such steel-reinforced specimens.

2. GFRP-reinforced specimens failed in a somewhat more brittle manner, although *some* plasticity was observed during post-peak behaviour. The debonding of the GFRP stirrups was a particularly important observation. This debonding allowed substantial deformation of the structure to occur prior to delamination of the GFRP. Plasticity theory is inapplicable as an analysis technique here due to the brittle nature of the GFRP itself. However, it appears that a method which allows for the actual shear displacements, crack sizes and GFRP stress levels is suitable for this situation where debonding has occurred.

3. AFRP helically-reinforced specimens failed in a very brittle manner. In fact, failure was due to snapping of the AFRP at relatively low load. This was probably due to the helices being tightly-wound, so that there was little chance of debonding. Stress concentrations would have built up and caused rupture. This was perhaps exacerbated by tying-wires, at the locations of which rupture usually occurred. An analytical approach in which the stress levels in the helices may be found seems sensible here. When this stress level reaches the rupture strength, the shear failure capacity of the concrete structure is assumed to be reached.

5. ACKNOWLEDGEMENTS

These tests were conducted at Cambridge University Engineering Department by Dr M.A. El-Hassan and Mr M. Rich. Their participation in this research is gratefully acknowledged, as is the technical help provided by Mr C.J. Mason, Mr V.L. Piper and Mr M.R. Touhey.

6. REFERENCES

1. Clarke, J.L. and Waldron, P., **"The reinforcement of concrete structures with advanced composites"**. *The Structural Engineer*, U.K., Vol.74, No.17, pp283-288, 3 September 1996.

2. Tottori, S. and Wakui, H., **"Shear capacity of RC and PC beams using FRP reinforcement"**. Proceedings of FRP Reinforcement for Concrete Structures, Vancouver, Canada, pp615-632, March 1993.

3. Burgoyne, C.J., **"Rational use of advanced composites in concrete"**. Technical Report CUED/D-Struct/TR167, Cambridge University Engineering Department, U.K., 1997.

4. Elhassan, M.A., **"The coefficient of friction in concrete"**. Final year project report, Cambridge University Engineering Department, 1991.

5. Rich, M., **"Shear in concrete reinforced with fibre reinforced plastics"**. Final year project report, Cambridge University Engineering Department, 1996.

6. Hofbeck, J.A, Ibrahim, I.O. and Mattock, A.H., **"Shear transfer in reinforced concrete"**. ACI Journal, Vol.66, pp119-128, February 1969.

7. Ibell, T.J. and Burgoyne, C.J., **"The use of FRPs vs steel for shear reinforcement of concrete"**. Submitted to the ACI Structures Journal.