FRP REINFORCEMENT FOR CONCRETE STRUCTURES: DESIGN CONSIDERATIONS

Nebojša ĐURANOVIĆ[*1], Kypros PILAKOUTAS [*2] and Peter WALDRON[*3]

ABSTRACT: This paper examines the fundamental assumption used in RC design that plane sections remain plane. Experimental results are used to demonstrate that strains remain linear within the compression zone and that strains in tension need to be considered in the light of strain stiffening. By providing evidence that no bond slip occurs, the plane sections assumption is fully verified. The use of an equivalent stress block is also justified by comparing analytical predictions with experimental.
KEYWORDS: FRP, concrete design, concrete bond, section analysis

1 INTRODUCTION

Fibre Reinforced Polymer (FRP) reinforcement is emerging as an alternative to steel re-bars in concrete structures. In cases when the likelihood of reinforcement corrosion is significant, opting for FRP bars can prove to be cost-effective by extending the life span of the structure. Apart from being much less susceptible to corrosion, FRP bars are also lighter than ordinary steel reinforcement, magnetically and electrically transparent and have significantly higher ultimate tensile strength. However, one of the main obstacles to a more widespread use of FRP reinforcement in the European construction industry is the lack of design standards. In July 1995 the Japanese Ministry of Construction published guidelines for the design of concrete structures with FRP reinforcements [1], whilst in February 1996 the American Concrete Institute published a state-of-the-art report on FRP reinforcement [2]. A task group from the EUROCRETE project has drafted a proposal for modification to the European, British and Norwegian codes in order to enable the use of those codes for RC design using FRP reinforcement [3]. In all these documents the fundamental assumptions of RC design are adopted but not proven.

This paper presents and discusses some of the results from the EUROCRETE project that are relevant for establishing the applicability of the sectional analysis approach for the design of RC structures with FRP bars. EUROCRETE is a pan-European project

[*1] Centre for Cement and Concrete, Dept. of Civil Eng., University of Sheffield, UK, Post-Doctoral Researcher.
[*2] Centre for Cement and Concrete, Manager, Dept. of Civil Eng., University of Sheffield, UK, Lecturer.
[*3] Centre for Cement and Concrete, Director, Dept. of Civil Eng., University of Sheffield, UK, Professor.

aiming to develop non-ferrous reinforcement. Full details of the measurement techniques and the instrumentation used to obtain the results presented in this paper can be found elsewhere [5].

2. VALIDATION OF SECTION ANALYSIS

The following assumptions, which are usually employed when dealing with the section analysis of steel reinforced concrete sections are examined for sections reinforced with FRP bars and shear links:

1. Plane sections remain plane, leading to a linear distribution of strains through the section.

2. Concrete/Reinforcement strain compatibility - Bond integrity.

3. Equivalent stress blocks can be used at ultimate conditions, to simulate the concrete stress-strain characteristics.

2.1 PLANE SECTIONS REMAIN PLANE

Concrete strain distribution at the midspan cross-section of the GFRP beams tested was assessed by using results from 5 horizontal displacement transducers positioned at locations as indicated in Figure 1. Displacement measurements were taken between fixed points on the concrete surface, 200mm apart. The measured displacements provide true concrete tensile strains only for loads before concrete cracking, i.e. up to about 12-15 kN. For loads greater than the cracking load, measurements below the neutral axis obviously include the crack widths inside this 200mm region. Typical results in the form of "reinforced concrete" strain are shown in Figure 2, which also includes the strain as measured on a re-bar by strain gauges. The typical strain distribution along the height of the section, as obtained by the displacement transducers is presented in Figure 1, for various load levels.

As expected, at early stages of loading, the strain distribution is linear along the entire cross-section. After exceeding the cracking load, strains are still linear above the neutral axis and almost linear below the neutral axis. The neutral axis depth shown in Figure 1, corresponds only roughly to the height reached by the cracks. This is because the depth of the neutral axis varies even within the maximum bending zone, due to the presence of discrete cracking and the effects of concrete strain stiffening.

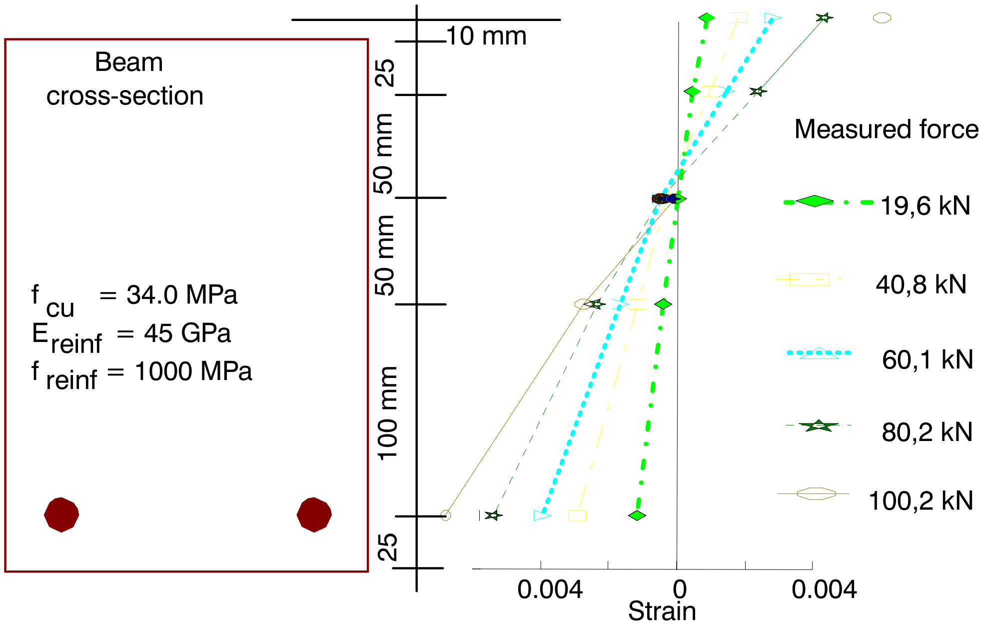

Figure 1 Strain distribution along the height of the section - test CB19

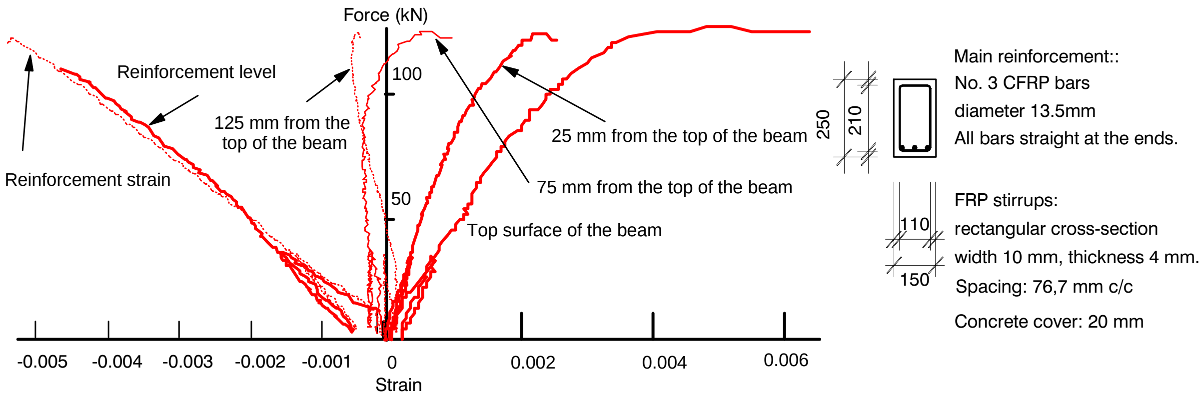

Figure 2 Reinforced concrete strain at different levels - Test CB17

Hence, care should be taken when interpreting results from both surface concrete strains, as well as from strains measured directly from strain gauges on the reinforcement. This is because the former provide an integral of surface strains over the given length - in this case 200mm. If the spacing of cracks is much lower then 200mm than the averaging effect works well, otherwise erroneous conclusions may be arrived at. The latter strains also have their problems, since the measured strains depend on the location of the strain gauge relative to the crack. If the strain gauge is located close to a crack, then higher values are expected than if the strain gauge is located away from a crack, since the stiffening effect of the concrete will reduce the strains.

In the above case, as shown in Figure 2, there appears to be no significant difference between the two strains, which seems to suggest that both values can be accepted with some confidence and that no major problems with bond occur in that region.

By further examining Figure 2, it can be observed, though, that the tensile strains lag behind the compressive strains, if the curvature obtained from the compression zone is accepted to be correct. This observation can be the result of bond slip or a result of uneven integration of concrete strains in the tensile zone. The latter can not be examined further without more detailed instrumental results. However, the former will be examined in the next section.

2. 2 STRAIN COMPATIBILITY - BOND CONSIDERATIONS

The surface of the EUROCRETE bars is designed to have the appropriate roughness so that good bond with concrete is ensured. The bond between concrete and the FRP reinforcement was monitored indirectly by electrical resistance (ER) strain gauges and bar-end LVDTs [4], in direct contact with the end of the main FRP reinforcement bar Plate 1.

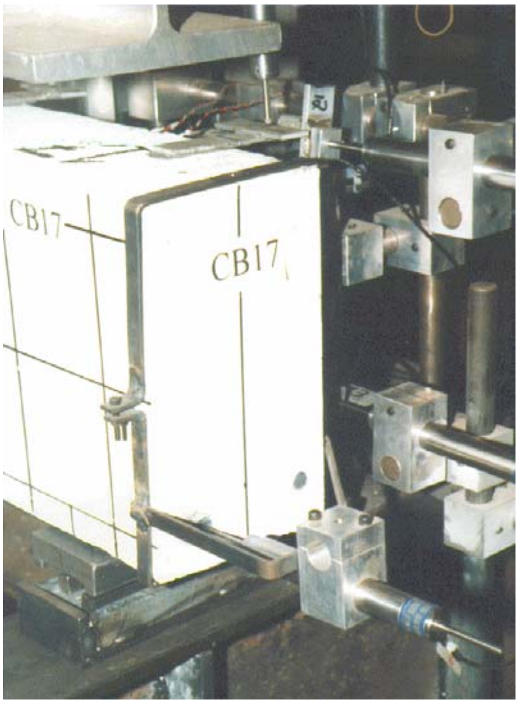

Plate 1 Arrangement for the bar-end bond-slip measurement

The maximum level of recorded FRP reinforcement strain in the first two phases of beam testing for EUROCRETE was observed in Test GB16, shown in Figure 3. The corresponding stress at this level of strain was 720 MPa i.e. around 72% of the ultimate tensile strength of GFRP bar, with an average bond stress along the shear span of about 3MPa. No splitting or other evidence of bond failure was seen in the middle of the beam. There was also no evidence of bond slip at the end of the beam as shown by the slip displacement and strain versus bond in Figure 4.

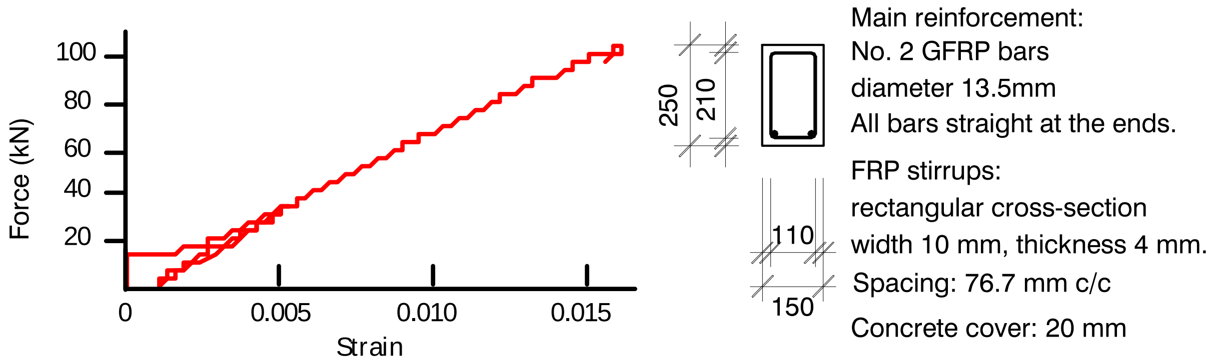

Figure 3 Strain record at the midspan of the beam - Test GB16

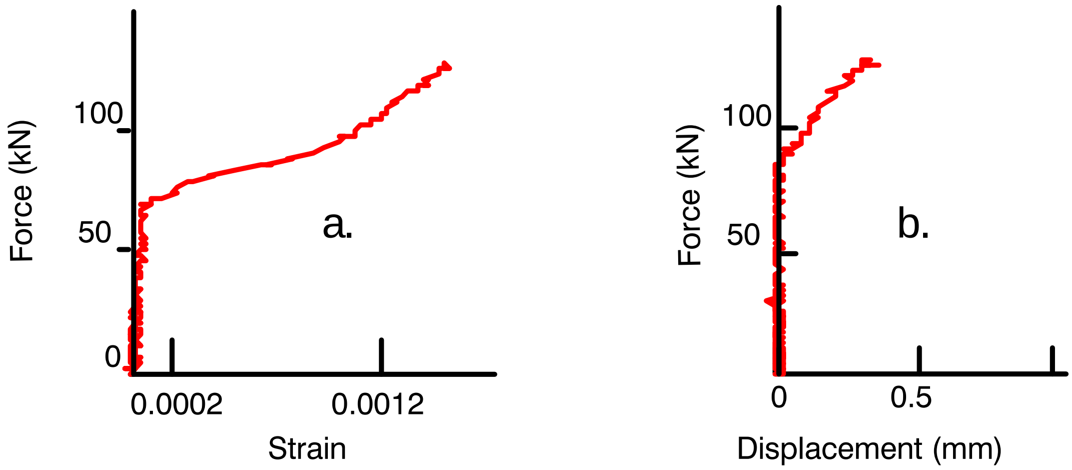

Figure 4 a. Strain recorded 75mm from the end of a CFRP main reinforcement bar and b. Relative slip between the CFRP reinforcement bar and the surrounding concrete - Test CB17

Since the Young's modulus of elasticity for the EUROCRETE Carbon FRP bars is around 116 GPa, stress in the bar in this particular case reached 210 MPa. This corresponds to an average bond stress over the last 75 mm of 9.5 MPa, which is 35% lower than the average bond strength obtained from pull-out tests. As expected, the loads at which the strain in the bar and bar slip are mobilised, do not correspond exactly to each other. The apparent bar-slip measurement seen in Figure 4, is not necessarily due to true slip between the bar and concrete, but it represents an inward movement of the bar which could be partly due to the compressibility of concrete.

In only one of the tests (CGB22 - carbon/glass composite bars) conducted so far beam failure was directly caused by the loss of bond between the FRP main reinforcement and the surrounding concrete. The main reason for the bond failure observed in test CGB22, (shown in Figure 5), was the very low compressive (18MPa) and tensile (1.7MPa) strength of the concrete used in this test. It can be seen that the end-bar stress that caused the bond failure was of the order of 260MPa, corresponding to an average bond of 5MPa over 175mm, which is higher than obtained from pull-out tests on specimens with such low concrete strength.

It can be concluded that the bond characteristics of the EUROCRETE bars are adequate for use in RC design. No differential movement between the bar and concrete is expected and, hence, strain compatibility is maintained at all times. Hence it can be concluded from the last two sections that, the plane sections remain plane assumption is valid for use with FRP reinforced sections.

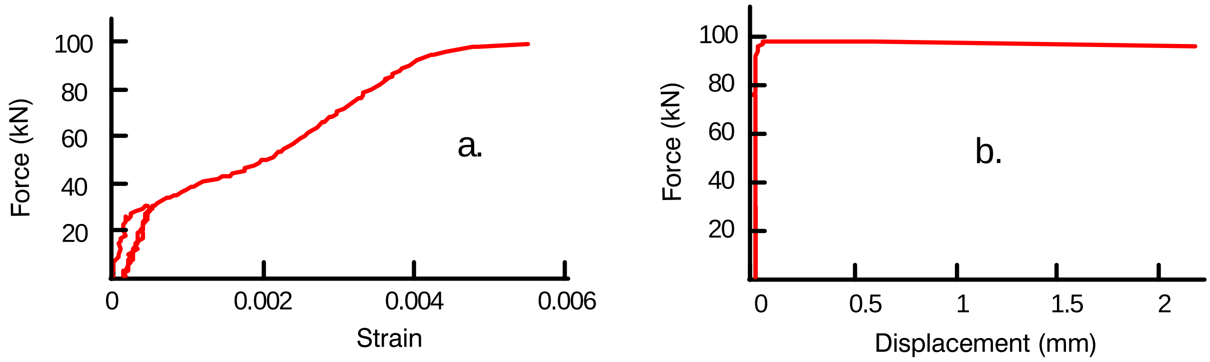

Figure 5 a. Strain recorded 175mm from the end of the CGFRP main reinforcement bar b. Relative slip between the CGFRP reinforcement bar and the surrounding concrete - Bond Failure - Test CGB22

2.3 EQUIVALENT CONCRETE STRESS BLOCK

Linearity of strains above the neutral axis near the ultimate conditions is sufficient to justify the use of an equivalent concrete stress-block. Such an approach is widely used by the codes of practice, since it simplifies section analysis calculations.

For the purposes of assessing this approach, the equivalent concrete stress-block proposed by British Standard BS 8110 [5], as shown in Figure 6, was used in all subsequent calculations.

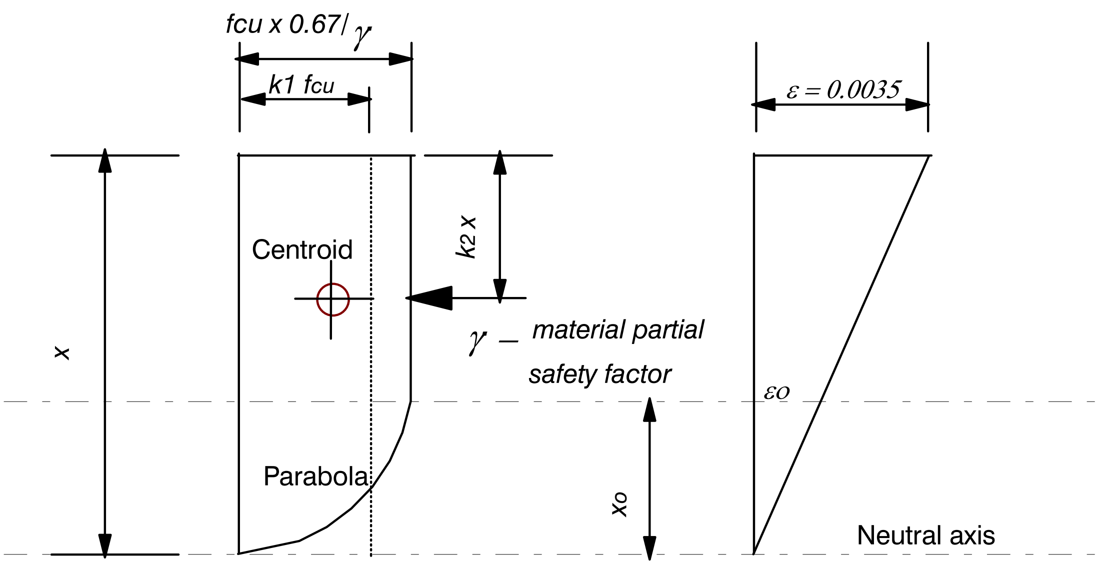

Figure 6 Concrete Stress Block and Strain Distribution as given by BS8110

3 RESULTS

Table 1 gives the results of the experimentally and analytically obtained loads and deflections for selected GFRP reinforced beams. For calculating these values all sectional analysis assumptions mentioned previously were used. For assessing serviceability conditions of reinforced concrete sections, BS8110 considers uncracked, fully cracked and partially cracked concrete sections. All of the beams presented in the table below were reinforced with Glass FRP bars and they all failed in flexure as a result of concrete compressive failure. More details about these beams can be found elsewhere [5].

Table 1 Comparison of analytical and experimental deflection results

Test No	Area of bars mm^2	f_{cu} MPa	Ultimate load Experimental kN	Ultimate load Analytical kN	Ultimate midspan deflections Experimental results (mm)	Analytical results Cracked (mm)	Analytical results Partially cr (mm)
GB1	429.4	30.0	97.8	82.9	45.8	42.7	34.7
GB5	429.4	31.2	105.1	84.86	41.4	44.4	36.4
GB9	429.4	39.8	103.6	98.1	45.4	56.5	46.4
GB10	429.4	39.8	103.0	98.1	45.4	56.5	46.4
GB13	286.3	43.4	90.6	88.1	53.4	82.1	60.8

It can be seen that the analytical predictions describe the experimental behaviour well, always giving results on the conservative side. This fully justifies the use of such an approach for the design of FRP reinforced sections.

4 CONCLUSIONS

(1) The distribution of strains through the concrete section of an FRP reinforced section is examined in detail by using surface strain and bar strain gauge results.

(2) The strain distribution in compression is shown to be linear, but the strains in tension seem to be lagging behind. This is attributed to the strain stiffening effect of concrete.

(3) The results demonstrate that no bond slip occurs in normally designed beams and, hence, this fully justifies the plane sections remain plane assumption.

(4) The use of equivalent stress block is studied and shown to give good results, always on the conservative side.

(5) Deflection predictions based on the partially cracked section give good results.

ACKNOWLEDGEMENT

Partners: Euro-Projects (LTTC) Ltd., Laing Technology Group, Sir William Halcrow and Partners Ltd., University of Sheffield, Allied Steel and Wire, Techbuild Composites Ltd., Vetrotex, DSM Resins, Statoil, Norsk Hydro and SINTEF. UK Funding Body: DTI/EPSRC, LINK Structural Composites Programme, NL Funding body SENTER. Associates: Tarmac Precast. Additional materials: Zoltek and Toray. The project has been granted EUREKA status and the assistance is acknowledged.

REFERENCES

[1] Japanese Ministry of Construction, "Guidelines for Structural Design of FRP Reinforced Concrete Building Structures", by FRP Reinforced Concrete Research Group from the Building Research Institute, July 1995

[2] ACI Committee 440, "State of the Art Report on Fiber Reinforced Plastic Reinforcement for Concrete Structures", ACI, Detroit, Feb 1996.

[3] Clarke, J.L, O'Regan, D.P. and Thirugnanendran, C., "Modification of Design Rules to Incorporate Non-Ferrous Reinforcement", Interim Report to EUROCRETE, Sir William Halcrow & Partners, London, September 1996.

[4] Duranovic N., Pilakoutas, K. and Waldron, P., "Tests on Concrete Beams Reinforced with Glass Fibre Reinforced Plastic Bars", Third International Symposium on Non-Metallic (FRP) Reinforcement for Concrete Structures, Sapporo, Japan, Oct. 1997.

[5] BS8110 - British Standards Institution, "Structural use of Concrete", 1985.