9 FRP Reinforcement in GRC Elements

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Abstract: Corrosion of steel reinforcement is a serious problem that prohibits the development of thin reinforced concrete sections. GRC provides significantly improved tensile and impact strength compare with plain concrete and therefore is widely used to make light weight thin section elements. However, apart from some small ornamental elements or some relatively solid products, most GRC elements still need to be stiffened either by ribs and stud frames. Fibre Reinforced Polymer (FRP) reinforcement is light, strong and corrosion resistant and, hence, may well be a useful alternative to steel reinforcement. In the case of panel elements, the use of FRP can also reduce the overall thickness of the element compared with stud frame construction. A FRP reinforced GRC lintel integrated with an insulation core and a bridge permanent formwork incorporating FRP reinforced GRC based on a ‘rib and skin’ design have been developed. Work done, including the design, prototype manufacturing, in-house load capacity testing, numerical analysis and code prediction, is presented here. Results show that FRP rebars are very promising as additional reinforcement in GRC structural or secondary structural elements.

Key words: corrosion, GRC, thin section, fibre reinforced polymer (FRP), permanent formwork, lintel

Speak of the term GRC (Glass fibre reinforced concrete), it seems a commonsense that this composite material is reinforced by glass fibre other than anything else. However, apart from small ornamental elements made with spray up GRC or relatively solid products made with premix GRC, most GRC elements still need to be stiffened to achieve an economic design. Solutions include corrugated, ribbed or stud frame construction etc. Stud frame GRC panels have been proved very efficient and are wide used in cladding. They can be designed into very large scale. Projects with cladding panels up to 30 m^2 have been witnessed. While ribbed GRC elements are in a rather small scale due to the structural and weight concerns. To increase the size of this type of element, additional reinforcement is necessary. For steel reinforcement, the cover requirements would lead to increased element thickness. Stainless steel can keep the section thickness down but is expensive. FRP reinforcement is light, strong and corrosion resistant and, hence, may well be a useful alternative for steel reinforcement.

Preliminary study

To understand the design principle and mechanical performance of FRP rebar as additional reinforcement in GRC elements, premix GRC beams reinforced by FRP rebars were fabricated and followed by central point bending test.

Specimens:

FRP rebars were supplied by Jinde and Pulwell. Dimension of beam was 50×110×1200. Concrete cover was 10mm otherwise stated. GRC mix comprised 1 part of cement and 1 part of sand, 1% Flowaid SCC superplasticiser, 3% NEG ARC13PH901X fibre and at a w/c ratio of 0.36, 11 specimen beams were prepared as described in Table 1:

Table 1 Specimen description

Number	Description
None-1, None-2	control beams (no bar)
J6-1, J6-2	reinforced by one 6mm Jinde bar
P6-1, P6-2	reinforced by one 6mm Pulwell bar
J8-1, J8-2	reinforced by one 8mm Jinde bar
P8-1, P8-2	reinforced by one 8mm Pulwell bar
P8-M	reinforced by one 8mm Pulwell bar in the middle

Procedures

A wooden mould was constructed to fulfill the casting (Figure 1). FRP rebars were positioned via the holes in the baffles on both ends. The mould was stripped the next day and beams were wrapped in polythene for 28 days before testing.

Figure 1 Mould and casting

Beam was simply supported at both ends. A dial gauge with accuracy of 0.01mm was placed in the central area underneath the beam (Figure 2). Load was applied by a hydraulic jack. The accuracy of load reading was 0.5kN.

Figure 2 Test setup

Results and Discussion:

It was observed that for the majority of the beam specimens, failure starts from a single crack initiated in the central bottom area of the beams and then propagated upwards along with or without adjacent cracks occurring. The unit failed suddenly when load reached to certain point with the break of FRP bar. Failed specimen is shown in Figure 3.

Side **Bottom** **Ruptured cross section**

Figure 3 Failure pattern

The load deflection response is shown in Figure 4.

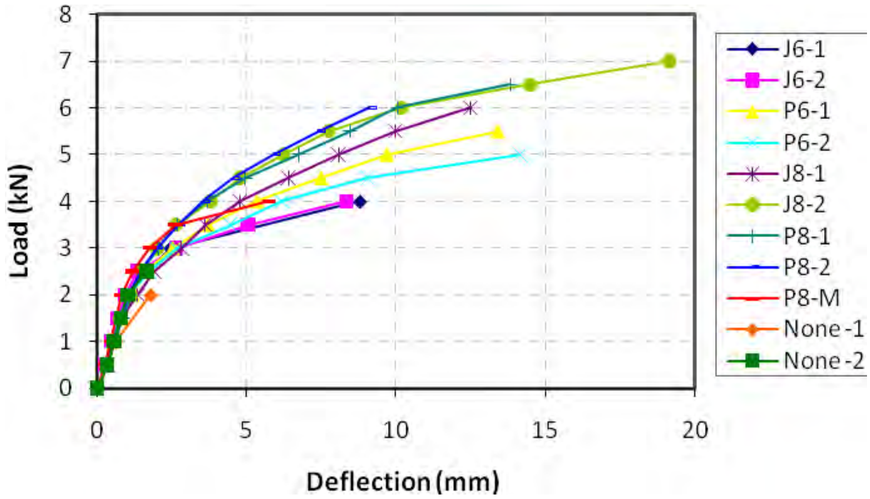

Figure 4 Load deflection correspondence

The load capacity and predicted deflection under serviceability loads are calculated according to ACI 440.1R-06 (2006). Calculation and test results are summarised in Table 2. It can be seen that the tested loads are well in accordance with the predicted values. This means the short term structural performance of the FRP reinforced GRC elements can be predicted and such prediction is also reliable.

Table 2 Code prediction and test results

FRP Bar	Diameter (mm)	Tensile Strength (MPa)	Young's Modulus (GPa)	Load capacity (kN)	Tested load (kN)	Serviceability Deflection (mm)
Unreinforced	---	---	---	1.83	2.25	1.08
Jinde 6mm	5.75	825	41	3.98	4.00	1.08
Jinde 8mm	7.75	760	41	5.07	6.50	1.08
Pulwell 6mm	6.35	840	43	4.40	5.25	1.08
Pulwell 8mm	8.00	750	43	5.30	6.25	1.08
Pulwell-middle	8.00	750	43	2.83	4.00	1.08

Compared with the unreinforced GRC elements, FRP rebar reinforcement can easily double or triple the load capacity.

Insulated FRP reinforced lintel

Prestressed precast lintel is widely used in masonry buildings around the world. They are proved to be strong and durable. A GRC lintel was made using FRP rebars as additional reinforcement with an insulation core due to the following reasons:

- Concrete itself is not a good insulation material. Steel reinforcement has high thermal conductivity.
- The mouldability and versatile finish enable the GRC lintel to achieve desirable finish and easy to integrate into the building environment.

- Reduced weight.

Design and mould

The dimension of lintel was decided to be 150×230×2000. It consists of a 15mm thick premix GRC shell and an Expanded Polystyrene (EPS) core. Two bottom inner corners of the GRC shell were chamfered 20mm to accommodate the 8 mm FRP rebar. A wood mould was constructed, the drawing and finished mould is shown in Figure 5.

Casting and testing

Premix GRC with the same mix design as mention in the previous section was used to fill the mould. To counteract the buoyancy of EPS, steel clips was applied on top of the mould, further weight was placed on top of the clips.

The finished lintel weighed only 53 kg, only one third of the weight of a solid concrete lintel.

The lintel was simply supported and a concentrated load was applied in the centre by a hydraulic jack. Central deflection was measured by a dial gauge accurate to 0.01 mm. The test setup and failure pattern is shown in Figure 6. It can be seen that diagonal cracks occurred from the bottom near both ends and propagated towards the loading point. Rather than failed in the centre area, the beam failed from one of the diagonal cracks when the FRP reinforcement broke suddenly.

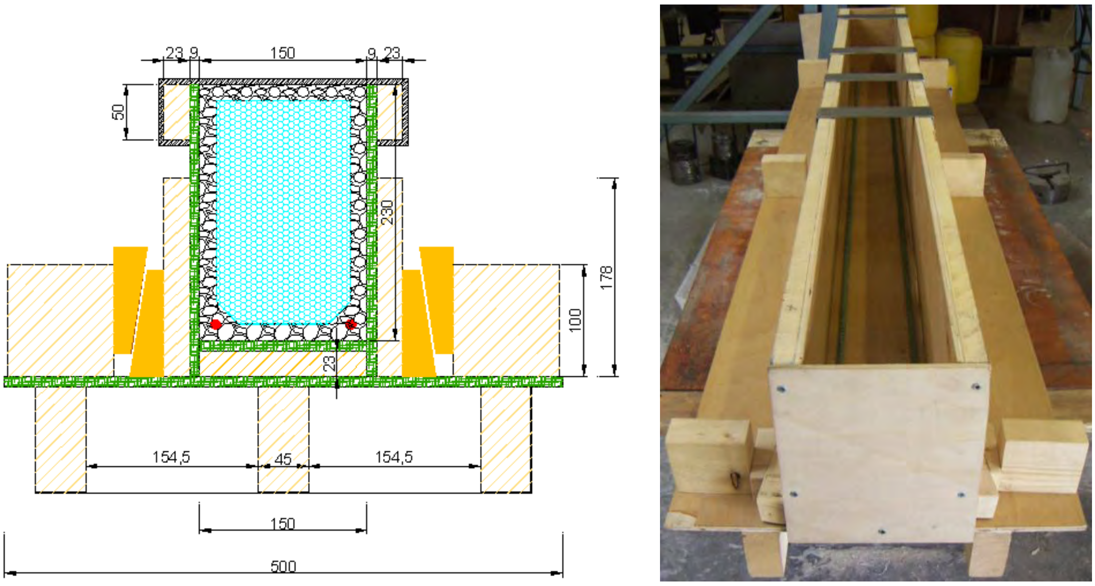

Figure 5 Mould dimension and finished mould

Figure 6 Test setup and failure pattern

Results and discussion:

The measured load deflection correspondence is shown in Figure 7. The initial part of the curve is a straight line until load reached to 12 kN. At this point, the flexural strength of GRC was back calculated to be 9.3 MPa, which denotes the LOP of the premix GRC used in this element. The load capacity of the lintel according to flexural failure was calculated to be 35.4 kN. This is much higher than the test result of 24 kN. This can be explained by the shear failure mode of the lintel rather than the flexural failure. To prevent shear failure, it is necessary to add stirrups in the lintel. This may be difficult since the majority of FRR rebars cannot bend and irregular shaped normally need to be bespoken. Nevertheless, a load at 24 kN is already much higher than the actual load then can be applied on this lintel.

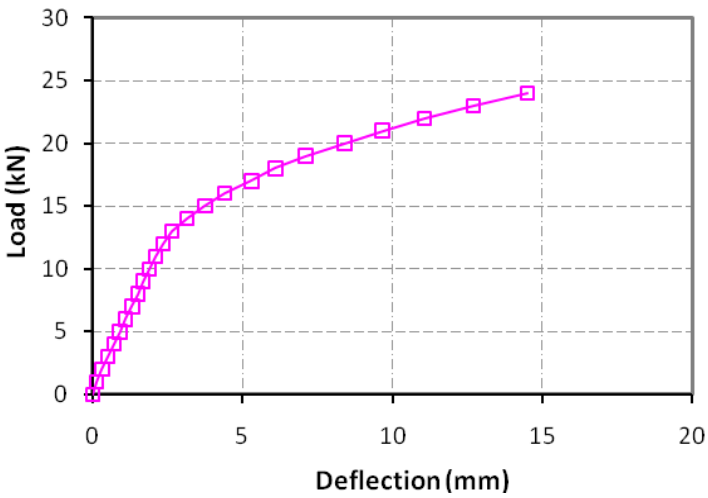

Figure 7 Load deflection correspondence

FRP reinforced GRC permanent formwork

GRC permanent formwork provides one of the most cost-effective methods for constructing bridges and viaduct decks when used in conjunction with precast beams (Harrison 1986). It is available in flat sheets, single corrugated sheets (Figure 8) and ribbed double skin sheets. These panels are usually made by using the spray-up method and the span is normally limited to around 1 m. Additional reinforcement is therefore necessary to increase the effective span. Thus, in conjunction with BCM GRC Limited, a bridge permanent formwork incorporating FRP reinforced GRC based on a 'rib and skin' design was developed.

Figure 8 GRC permanent formwork (left: flat (Shay Murtagh); right: single corrugated (BCM GRC))

Design

The following issues were considered at the design stage:

- Geometry: The general requirements for the proposed formwork were to achieve a 2 m span, 40 mm vertical clearance from the support to the concrete deck and a weight limit of 100 kg. For practical reasons, the thickness of the panel was chosen to be 12 mm.
- Strength: Flexural demand in simply supported beams necessitates compressive strength on the top and tensile strength at the bottom. The formwork needs to take advantage of the high compressive strength of GRC and high tensile strength of FRP reinforcement. Apart from flexural strength, shear strength also needs to be checked.
- Loading: The load considered included formwork self-weight, 250 mm thick fresh concrete, imposed construction load of 1.5 kN/m^2 which accounts for men, hand tools and small mechanical plant used in the placing operation such as vibrator motors etc. (Concrete-Society 1986; BS5975 1996)
- Deflection: Excessive deformation is normally the governing design criterion for permanent formwork, especially for long spans. The Highways Agency (1991) recommends a limit of 1/300 of span between supports 4 hours after completion of concreting.
- Aesthetics: it is believed that the smooth exposed surface combined with the neat line-up of ribs will provide an aesthetic result.

The first parameter to be decided was the maximum distance between ribs, which can be calculated from the section capacity by the following equation:

$$l = \sqrt{8bt^2\sigma_t / 6q} \tag{1}$$

In which, b is the unit length of GRC panel, σ_t is the design tensile strength of GRC, t is the thickness of the GRC panel, q is the design load. Based on a design tensile strength of 4 MPa, l was calculated to be 277 mm, so it was decided to use a spacing of 250 mm.

The cover to the FRP reinforcement was selected to be 10 mm based on the study of Kim (2009) on the bond behaviour between FRP rebar and GRC.

The next parameter to design was the width and depth of the ribs. Using the ACI 440.1R-06 (2006) method, it was found that deflection governs the design and a depth of 100 mm was chosen based on the deflection analysis and experience. The dimension and the isometric view of the designed unit are shown in Figure 9. The overall length of the formwork was 2.08 m. The lower part the ribs at the supports was reduced 50 mm to meet the vertical clearance requirement.

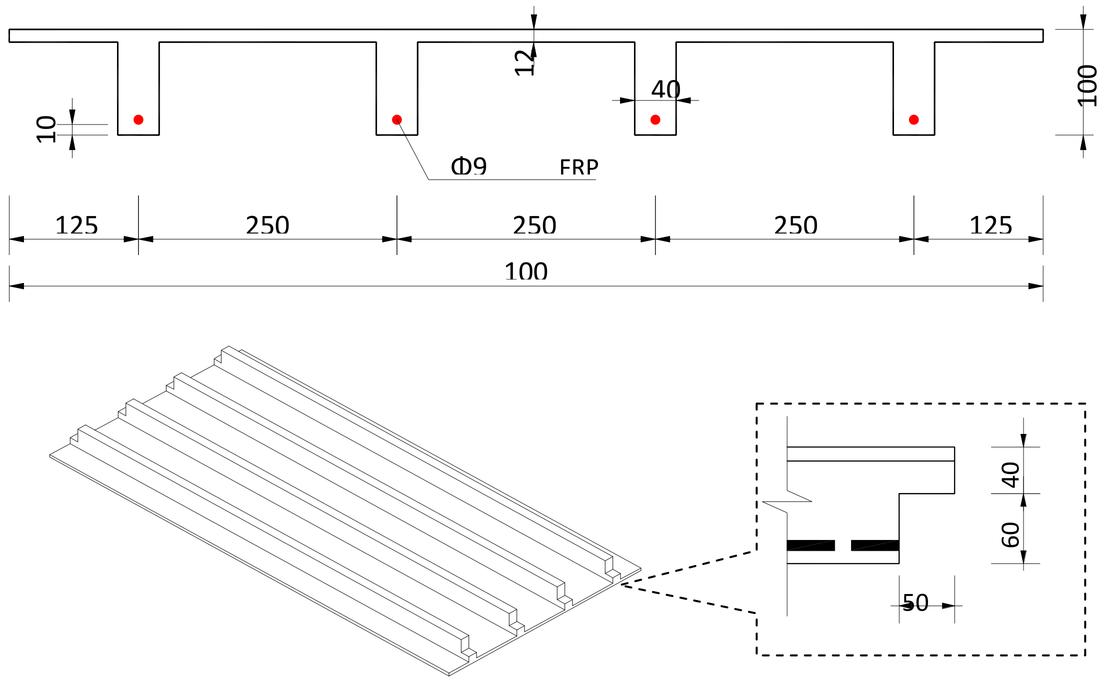

Figure 9 Error! No text of specified style in document.**Dimension and isometric view of the designed formwork**

Two types of round GFRP rebars (E = 43 GPa, nominal strength = 750 (9mm)/840 (6mm) MPa) were examined. A partial factor of safety of 1.15 was applied to the FRP tensile strength. The GRC compression failure strain was assumed to be 0.0045. It was found, for both types of reinforcement, the units will fail from the breaking of the rebars. The calculated results are shown in Table 3. The deflections and crack widths were calculated by using the ACI 440.1R-06 (2006) equations.

Table 3 Section capacity

FRP bar	Flexural capacity			Under design load	
	M (kNm)	Load (kN/m^2)	σ_t (MPa)	Deflection (mm)	Crack width (mm)
6 mm	12.86	25.72	730	9.89	0.28
9 mm	21.59	43.18	652	7.02	0.13

Prototype

A GFRP mould was fabricated at the BCM GRC. The formwork prototype was made by horizontal casting using a self compacting mix (again, same mix design as previous sections). FRP bars were positioned using GRC spacers and metal wire clips. GRC mix was delivered to the mould via a peristaltic pump (Figure 10). The density of rebar and GRC is almost identical. No external vibration was needed, therefore, the rebars were expected to remain in place and this was confirmed by the later examination on a dissected cross-section. All together four prototypes were made: two with GRC only (GRC-BF1 & 2), one with 6 mm GFRP reinforcement (GRC-BF-FRP6) and one with 9 mm GFRP reinforcement (GRC-BF-FRP9).

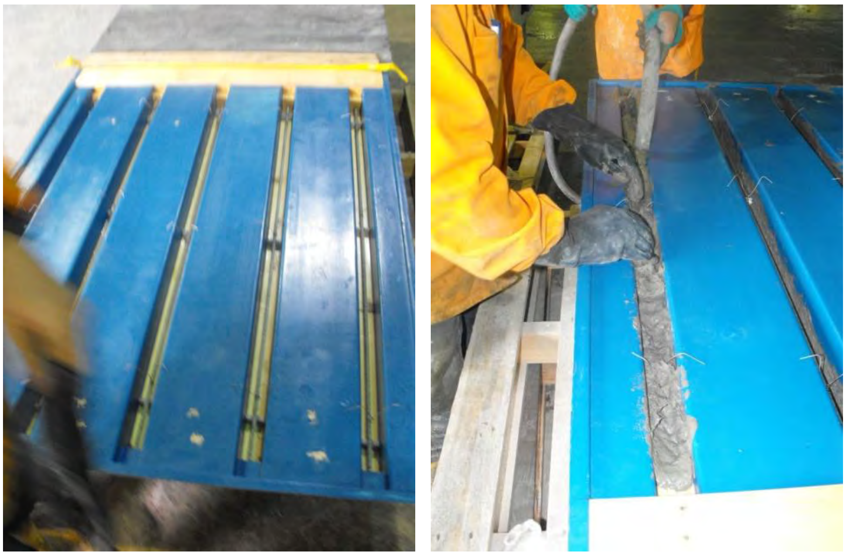

Figure 10 Formwork mould, FRP bar position and unit casting

Test results

These prototypes were tested using basic test facilities. Each unit was simply supported on two channel sections. A dial gauge (range: 20 mm, accuracy: 0.01 mm) was placed in the central area underneath the formwork. Concrete blocks were placed adjacent to the dial gauge to prevent damage if the unit failed. A simulated static UDL was applied by placing layers of concrete hollow blocks on the formwork. One layer of blocks was equivalent to a load of 1.6 kN/m^2. The deflection was recorded five minutes after each layer was added.

GRC-BF1 & GRC-BF2

Test specimens GRC-BF-1 and GRC-BF2 resisted 4 and 5 layers of concrete blocks, respectively. In both cases a single crack occurred in the middle span (Figure 11). Once the crack occurred, it propagated quickly upwards and failure was abrupt. It was noted that when the crack reached the interface of the ribs with the panel it also ran horizontally indicating a horizontal shear failure as well as flexural failure due to the tensile rupture.

GRC-BF-FRP6 & GRC-BF-FRP9

Both GRC-BF-FRP6 and GRC-BF-FRP9 specimens successfully carried twice the design load, which was equivalent to 12 layers of blocks. Evenly distributed (with approximate 250 mm intervals) fine cracks were seen on the ribs (Figure 12). In order to examine the maximum load capacity, a pallet of sand which weighed over 1 tonne was placed on top of the blocks (Figure 13). The GRC-BF-FRP9 unit withstood this additional load although with substantial deflection. The GRC-BF-FRP6 unit failed in shear at one of the supports (Figure 14). The total load resisted was 48.4 kN.

Figure 11 Failure of GRC unit **Figure 12 Evenly distributed fine cracks**

Figure 13 Maximum load capacity **Figure 14 Shear failure**

The load-deflection response is shown in Figure 15. For all 4 specimens, the initial curve is a straight line until the load reached around 4.5 kN/m^2. At this load the tensile stress in the bottom of the ribs reached the LOP of the premix GRC and the unit started cracking. After that, the plain GRC formwork quickly failed when the stress reached the MOR of the material. For the FRP reinforced formwork the rebars continued to carry the load. In this test, the rebars never reached their tensile strength. Under the design load (8 kN/m^2), the deflection measured was around 7 mm, approximately 1/300 of the span. However, at the service load of 6 kN/m^2 the deflection is only around 3.5 mm, which is approximately 1/600 of the span.

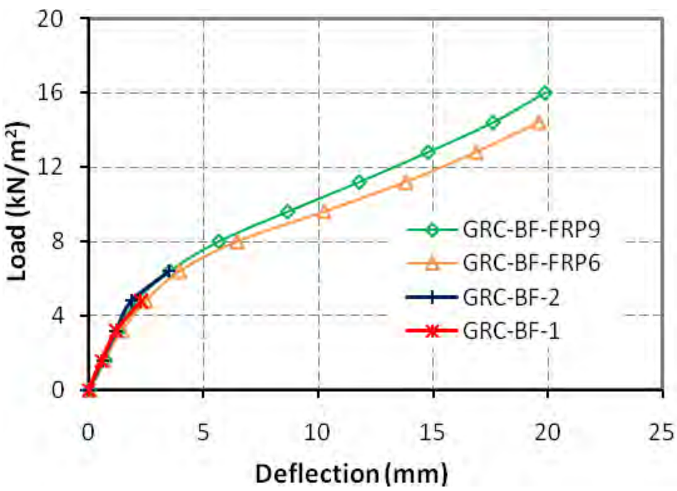

Figure 15 Load-deflection response

FE analysis

A FE analysis was conducted using a commercial software package to investigate the overall behaviour of the tested formwork.

The unit was modelled using 8-noded, linear elastic brick elements (CSD8R). FRP rebar was modelled as 2-noded linear beam in space (B31) with a circular profile. A simple linear elastic model was used to describe the material characteristics of GFRP reinforcement, which was modelled to behave linearly up to failure with a constant Young's Modulus of 43 MPa. Owing to the symmetry of both geometry and load arrangement, only a quarter of the formwork was modelled to reduce computational cost. Simple supports were used and symetrical boundaries were applied along the line of symmetry. The model contained one layer of elements in the panel and 6 layers of elements in the ribs (excluding the panel depth). Typical contour plots of stress and vertical deflection (GRC-BF-FRP9) are shown in Figure 16.

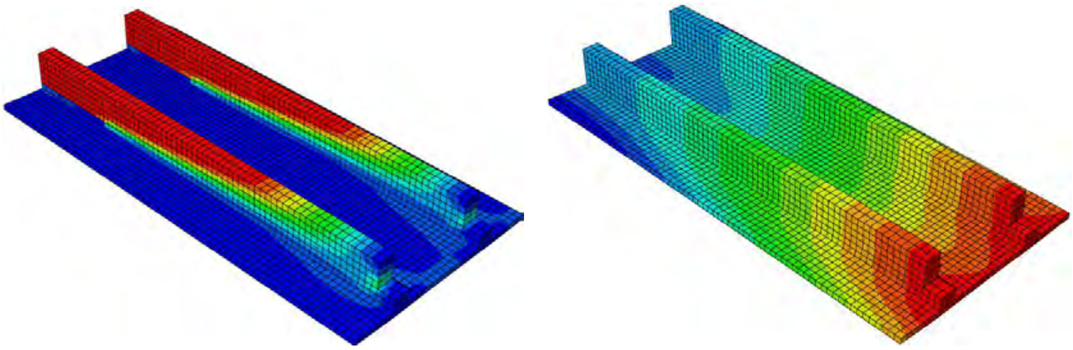

Figure 16 Stress and vertical deflection of GRC-BF-FRP9

Figure 17 and Figure 18 show the measured load-deflection curves together with the curves obtained from the numerical model. The FE analysis shows a good agreement for the predicted cracking load and the corresponding deflection for the unreinforced GRC formwork. This agreement also applies to the stage up to crakcing for the FRP reinforced formwork, however, a softer response is obtained from the numerical simulation after the elements cracked when compared to the experimental results. Code prediction of the load-deflecton response will be used to further investigate this discrepancy. It was also found that, for the FRP reinforced members, after cracking, the GRC tension stiffening mainly affects the convergence of the analysis rather than the trend of the load-deflection curve. Under the cracking load of 4.5 kN/m^2, the GRC LOP was estimated to be 6.28 MPa and this value will be used to calculate the cracking moment in the code predictions.

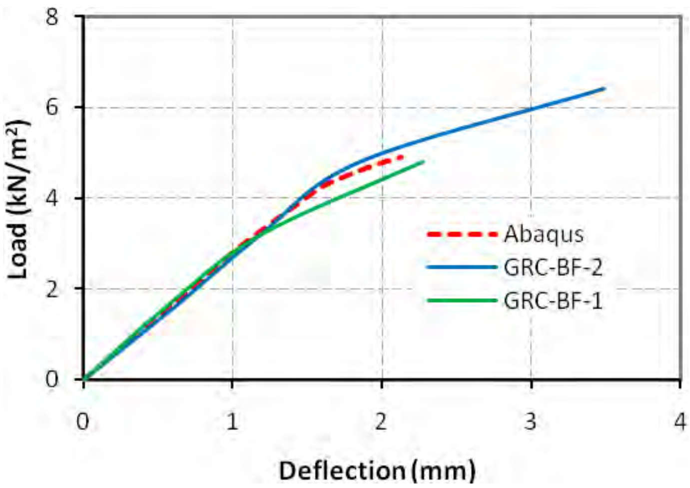

Figure 17 Predicted and measured load-deflection response of unreinforced formwork

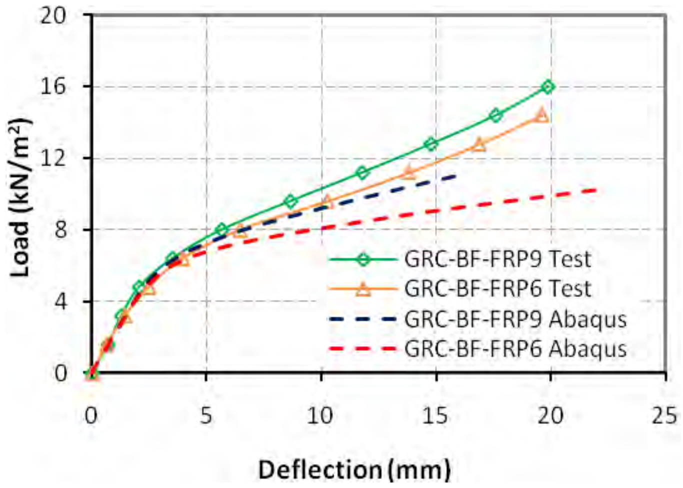

Figure 18 Predicted and measured load-deflection response of FRP reinforced formwork

Codes predictions of the load-deflection response

Due to the lower modulus of elasticity of FRP reinforcement, FRC reinforced concrete members are expected to exhibit larger deformations than steel reinforced RC for similar reinforcement ratios. FRP rebars have high tensile strength and a stress-strain behaviour that is linear up to failure. This leads, under pure bending and beyond the crack formation phase, to an almost linear load-deflection relationship, up to failure. Short-term deflections of reinforced concrete members are generally derived by applying a linear-elastic approach using an effective moment of inertia.

In this study the short-term deflection of the tested formwork was determined according to the current ACI recommendations (ACI 440.1R-06 2006) and Eurocode 2 (EN 1992-1-1:2004). These predictive equations are discussed in the following.

ACI 440.1R-06 (2006)

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g \tag{2}$$

in which I_g and I_{cr} are gross and cracked moment of inertia, M_{cr} and M_a are cracking and applied moment. ACI 440.1R-06 abandons the reliance of β_b on bond used in ACI 440.1R-03 and takes this parameter as a proportion of the ratio of reinforcement to the balanced reinforcement ratio, i.e. $\beta_b = \frac{1}{5}\left(\frac{\rho_f}{\rho_{fb}}\right)$.

Eurocode 2: EN 1992-1-1:2004

$$\delta = \beta\left(\frac{M_{cr}}{M_a}\right)^2 \delta_g + \left[1 - \beta\left(\frac{M_{cr}}{M_a}\right)^2\right] \delta_{cr} \tag{3}$$

in the above equation, the uncracked-state deflection δ_g and the cracked-state deflection δ_{cr} are calculated assuming constant uncracked and cracked sectional moments for inertia. β is a duration or repetition of load factor (1.0 for short-term loading). However, Al-Sunna (2006) proposed the use of Equation 2 with a β value of 0.5 for GFRP reinforced concrete beams to account for the different bond factor. Kim (2006) found the presence of glass fibre in the matrix did not affect the bond between GFRP reinforcement and concrete significantly. In this study, it was found that a β value of 0.7 gave reasonable results.

A comparison of experimental, numerically simulated, and code predicted load-deflection curves for the 6 mm and 9 mm GFRP reinforced formwork is presented in Figure 19Figure and figure 20. In both cases, the curves show that there is a good agreement between the experimental and the predicted values under service conditions (6 kN/m^2). Measured deflections are less than those predicted according to any of the methods considered here for values of load higher than the service load, whilst predictions from FE analysis and code of practice show good agreement. It is reasonable to suspect that the loading was not as uniformly distributed as assumed. In practice the concrete hollow blocks were placed too close to each other. As illustrated in Figure 21, for the first several layers, the resulting deflection is relatively small and the weight of the blocks is transferred downwards and can be regarded as UDL. As the layers increase, due to the increased curvature of the formwork, blocks in the top layers start pushing each other in the horizontal direction and create the ‘arch’ effect. Part of the weight is transferred towards the supports. Therefore, lower mid-span deflection is expected compared with the theoretical prediction of using UDL. Hence, this explains why the stiffness of the formwork is a bit higher than predicted.

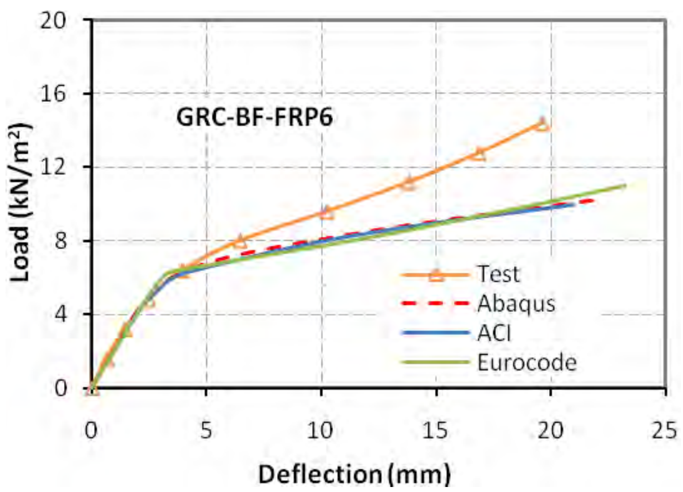

Figure 19 Comparison of load-deflection relationship among test, FE analysis and code prediction for formwork GRC-BF-FRP6

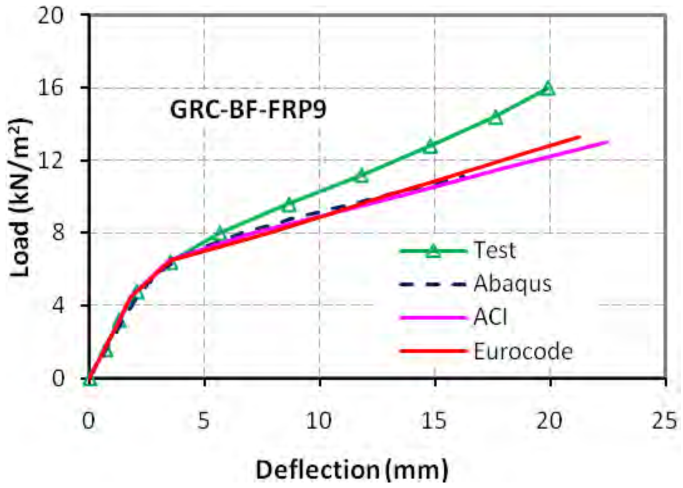

Figure 20 Comparison of load-deflection relationship among test, FE analysis and code prediction for formwork GRC-BF-FRP9

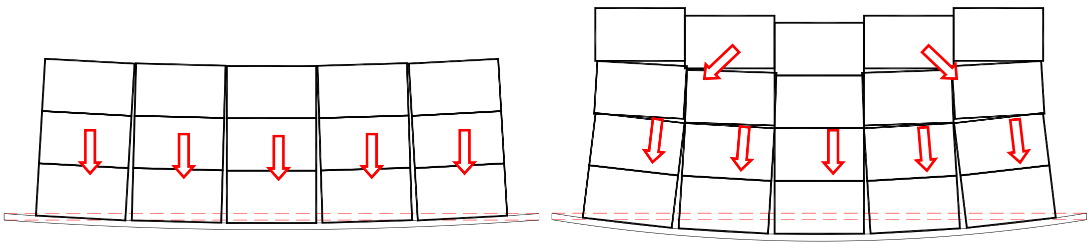

Figure 21 2 Illustration of load transferring path (out of scale for clarity)

Conclusion

FRP rebars can be used in GRC element as additional reinforcement to either increase the element span or reduce the element thickness. In the case of panel elements, the use of FRP in the 'rib and skin' design can certainly reduce the overall thickness of the element compared with stud frame construction. FRP can be a useful alternative for expensive stainless steel reinforcement. Both numerical analysis and code calculation were proved useful to predict the mechanical behaviour of FRP reinforced GRC element. The current design guideline showed good agreement with test results. However, caution needs to be taken when FRP is used in structural elements due to its brittle failure nature.

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