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Corrosion-resistant FRP reinforcement for bridge deck slabs

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This paper discusses the beneficial influence of compressive membrane action in fibre reinforced polymer (FRP) reinforced in-plane restrained slabs in bridge deck slabs and the improved service performance when arching action occurs. Bridge deck slabs that are exposed to extreme environmental conditions can experience severe corrosion damage. Expansive corrosion in steel reinforcement significantly reduces the design life and durability of concrete structures; for example, on one short section of the M1 in Northern Ireland, nearly £1 million was spent last year on the maintenance and repair of bridges due to corrosion. Corrosion-resistant composite reinforcement such as basalt fibre reinforced polymer (BFRP) and glass fibre reinforced polymer (GFRP) provides a durable alternative to reinforcing steel. In this research, two BFRP reinforced slabs and two GFRP reinforced slabs were constructed using high-strength concrete with a target cube compressive strength of 65 N/mm^2. The slabs represented typical full-scale dimensions of a real bridge deck slab 475 mm wide by 1425 mm long and 150 mm deep. The service and ultimate behaviour of the slabs are discussed and the results are compared with the relevant design guidelines.

Notation

A	cross sectional area of the slab
b	breadth of the slab section
d	effective depth of the reinforcement
d_1	available depth for arching
E_c	modulus of elasticity of concrete
f_{ck}	cylinder compressive strength of concrete
$f_{ck,cube}$	cube compressive strength of concrete
f_y	yield/rupture strength of the reinforcement
h	depth of the slab
K	axial stiffness
K_r	in-plane stiffness
L_e	half the span of the arch length
L_r	half the span of the rigidly restrained arch
M_a	arching moment of resistance
M_{bx}	flexural bending moment
M_r	arching moment resistance
P	applied load
P_a	arching load capacity
P_{ACI}	ultimate failure load predicted using the ACI 440.1R-06 guideline
P_b	flexural bending capacity
P_{EC}	ultimate failure load predicted using Eurocodes
P_{QUB}	arching strength of restrained slabs predicted by compressive membrane action theory proposed by Queen's University Belfast
P_t	measured ultimate load of test model
R	McDowell's non-dimensional parameter (elastic deformation)
u	McDowell's non-dimensional parameter (deflection)
w	deflection under the load point
ε	strain
ε_c	idealised plastic strain of concrete
ρ_y	reinforcement ratio

1. Introduction

Research by Wallbank (1989) estimated that the UK spent approximately £616·5 million to repair 10% of the country's entire bridge stock due to damage as a result of steel corrosion. The study further suggested that the total cost to repair the entire stock would be much higher. A recent study in the USA (ASCE, 2009) estimated that the authorities may need \$8·3 billion/year to repair all the structurally deficient bridges due to corrosion damage, and the indirect cost such as traffic delays and loss of productivity may exceed \$83 billion.

Several steel protection methods have been trialled to reduce the damage caused by corrosion, namely less permeable and higher quality concrete, cathodic steel protection, epoxy coated reinforcement and waterproofing of deck slabs. However, the reliability of these methods has been a concern as they have not been proved in the long term (Clarke, 1999; Keesler and Powers, 1988). It is possible to construct highly durable concrete structures using corrosion-resistant fibre reinforced polymer (FRP) bars that replace steel. The lower modulus of elasticity and brittle failure of FRP bars have raised concerns over the serviceability and the failure mode of FRP reinforced concrete structures, but when arching action occurs the structural performance is similar to steel reinforced structures. In the case of Thompson's Bridge, the basalt fibre reinforced polymer (BFRP) reinforced slab had slightly less deflection than the equivalent steel reinforced slab (Taylor *et al.*, 2013). There is currently design guidance in the USA in the form of ACI 440.1R-06 (ACI, 2006) but the lack of design guidance for FRP reinforced structures in Europe has discouraged the choice of FRP bars to replace steel.

Nevertheless, FRP has been used in the construction industry for strengthening and retrofitting of existing structures. The past two decades have seen FRP being used routinely to replace steel plates for strengthening of concrete structures. Several research studies have investigated the behaviour of simply supported FRP reinforced slabs, beams and parapet walls (Benmokrane *et al.*, 2004; Michaluk *et al.*, 1998; Ospina and Bakis, 2006). However, a very limited number of studies have investigated the behaviour of FRP reinforced in-plane restrained slabs, such as those typically found in practice. Previous research studies (Kirkpatrick *et al.*, 1986; Rankin and Long, 1998; Taylor *et al.*, 2001) on steel reinforced restrained flexural members have found that in-plane restrained slabs fail by concrete crushing due to compressive membrane action (CMA) and almost independent of the reinforcement percentage. If the CMA in restrained slabs can be exploited, it would be possible to construct high-strength FRP reinforced deck slabs using a lower percentage of reinforcement with high performance concrete.

This research assessed the behaviour of glass fibre reinforced polymer (GFRP) and BFRP reinforced in-plane restrained slabs under service and ultimate loads and the influence of CMA. The choice of GFRP and BFRP bars was based on their cost competitiveness, durability and lighter weight compared to other corrosion-resistant materials.

2. Background to CMA

Slab and beam bridge decks are one of the most common types in Europe and elsewhere (El-Gamal *et al.*, 2007). The deck slabs in slab and beam bridges such as in M-beam, Y-beam and steel girder bridges are in-plane restrained. In-plane restrained slabs inherit the beneficial influence of CMA or arching action provided that they meet the span/depth criteria and required lateral stiffness (Taylor *et al.*, 2001; Tong and Batchelor, 1971). Stiffer end restraint to in-plane expansion can induce a higher degree of CMA (Figure 1) thus further increasing the ultimate strength above that predicted by flexural theory.

Tests by Ockleston (1955) on a three-storey hospital building demonstrated that in-plane restrained slabs failed at a much higher load than predicted by yield line analysis, which was considered an upper bound prediction. Design codes (BS EN 1992-2:2005 (BSI, 2005) and ACI, 2006) often consider the flexural capacity as the first ultimate limit state check, but this underestimates the real strength of in-plane restrained slabs as it does not account for the benefits of arching action. The existence of arching strength had been empirically acknowledged by Turner (Faulkes, 1974) and also in a Russian design code (Gvodzev, 1939). The Russian design code suggested a lower amount of reinforcement for restrained slabs than the amount required for those without in-plane restraint. However, the tests by Ockleston (1955) provided a better understanding of CMA, which led to further research studies.

The research of Ockleston (1955) created an interest among researchers to understand the behaviour of in-plane restrained slabs. Early research studies by Wood (1961), Christiansen (1963), Liebenberg (1966) and Park and Gamble (1980) developed theories for CMA in restrained slabs but they were semi-empirical. Research into CMA at Queen's University Belfast (QUB) started in the 1980s to investigate the influence of various parameters on the behaviour of in-plane restrained steel reinforced concrete slabs. The research studies by Kirkpatrick *et al.* (1986), Rankin and Long (1998) and Taylor *et al.* (2001) at QUB have influenced the modification of the Northern Ireland bridge design specification (Department of

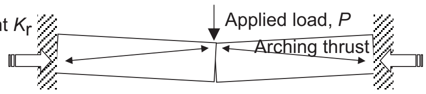

Figure 1. Arching analogy. The figure shows the arching action in in-plane restrained slabs

Regional Development, 1986). The addendum to the Northern Ireland bridge design specification currently recommends 0·6% steel reinforcement for top and bottom longitudinal reinforcement for Y-beam type bridge deck panels when the slabs satisfy CMA criteria. These research studies have further contributed to the Highways Agency code BD 81/02 (HA, 2007), which has also included guidelines to design restrained deck slabs using arching theory from QUB. Although the behaviour of steel reinforced in-plane restrained concrete slabs has been studied for many years, there has been less research on the behaviour of in-plane restrained FRP reinforced slabs and in particular BFRPs, which have shown slightly better durability characteristics than GFRPs.

3. Experimental programme

Two GFRP and two BFRP reinforced slabs were tested. The slabs had similar dimensions to the slabs tested by Taylor *et al.* (2001) as typical of a full-scale bridge deck slab in practice and for a comparison. High strength concrete with a target cube concrete strength ($f_{ck,cube}$) of 65 N/mm^2 was used in experimental investigation, as previous research (Taylor *et al.*, 2001) had shown an increase in slab ultimate capacity with increasing concrete strength.

3.1 Test slabs

Previous research by Kirkpatrick *et al.* (1986) suggested that the enhanced strength due to arching action can be considered to be equivalent to an amount of reinforcement if the slab satisfies the criteria for the arching effect. Therefore, it was decided to use 0·6% GFRP and 0·6% BFRP bars for the tests, as recommended by the Northern Ireland bridge design specification. Two slabs discussed in this paper were constructed using BFRP bars and the other two were constructed using GFRP bars with 0·6% reinforcement for the effective area of the section. The details of the test slabs are given in Table 1 and mix design details are given in Table 2.

The GFRP and BFRP bars were tested for their ultimate tensile strength and modulus of elasticity using the test method proposed by Castro and Carino (1998) for composite bars. The tensile strength and modulus of elasticity for the 12 mm dia. GFRP and BFRP bars are given in Table 3.

3.2 Test set-up

A steel frame was used to provide the lateral restraint to the slabs. The frame provided similar restraint to that in a typical slab and beam bridge deck with in-plane stiffness (K_r) of 855 kN/mm. The in-plane stiffness of 855 kN/mm is equivalent to 70% of the rigid restraint stiffness as described by Rankin and Long (1998), and in a real bridge deck is provided by the bending stiffness about the minor axes of the supporting beams in combination with the surrounding area of unloaded slab. The test arrangement of the slab is shown in Figure 2.

3.3 Test procedure

The test slabs were allowed to cure for 28 d under adequate moisture conditions using wet hessian to ensure full curing. Control sample concrete cubes were taken at the time of casting and tested in accordance with BS EN 12390 Part 3 (BSI, 2009) for compressive strength. The test set-up is shown in Figure 3. The test slabs were loaded using a 25 mm wide knife edge line load applied at the mid span to represent an equivalent axle load of up to 600 kN using an accurately calibrated hydraulic actuator. The deflection at mid span was measured using two 50 mm linear variable displacement transformers (LVDTs) positioned directly below the applied load at the soffit and 25 mm from the side face. Another pair of 25 mm LVDTs were used to monitor the in-plane movement of the steel frame.

Test slabs[a]	Provided condition	Concrete strength	Balanced reinforcement: %	Rationale
G-0·6%-12-125 T&B	0·6% of 12 mm bar at 125 mm spacing in two layers	68·1	1·08	Northern Ireland design specification recommends 0·6% steel reinforcement for restrained slabs. Maximum allowable spacing according to Eurocode is 300 mm.
G-0·6%-16-300 T&B	0·6% of 16 mm bar at 125 mm spacing in two layers	65·7	1·09	
B-0·6%-12-125 T&B	0·6% of 12 mm bar at 300 mm spacing in two layers	69·3	0·57	
B-0·6%-16-300 T&B	0·6% of 16 mm bar at 300 mm in two layers	66·1	0·54	

[a]Naming convention: G-0·6%-16-300 T&B, type of reinforcement-amount of reinforcement – rebar size – rebar spacing – top and bottom reinforcement

Table 1. Details of the slabs tested by the authors

Concrete	Cement	GGBS	Super plasticiser	Total water	20 mm aggregate	10 mm aggregate	Sand
Ordinary	450·0	—	9·0[a]	175·0	639	547	639

[a]2% by mass of cement

Table 2. Concrete mix constituents (kg/m^3 concrete) for target strength of 60–70 N/mm^2

A test load, equivalent to one third of the predicted failure load was applied to pre-crack the slab and the recovery in deflection was measured at the end of each test load. After the removal of all load, vibrating wire strain gauges (VWSGs) were fixed perpendicular to the cracks. Strain (equivalent to crack width) was measured using VWSGs at each increment of load. Electrical resistance strain gauges were also used to measure strain at the rebar–concrete interface at the mid span and support region at each increment of load.

4. Results and discussion

4.1 Serviceability behaviour of FRP reinforced slabs

The behaviour of the FRP reinforced slab under service load is of interest due to the perceived drawbacks with the lower elastic modulus of FRP bars compared to steel. Therefore, cracking load, crack pattern, mid-span deflection and the stress in FRP bars at service load are discussed in detail.

All of the test slabs showed the same stiffness up to the first crack, as shown in Figure 4, independent of the type of reinforcement and modulus of elasticity. Deflections of the test slabs at the service load level shown in Table 4 indicate that the test slabs did not exceed the recommended (BS EN 1992-2:2005) span/250 (BSI, 2005). A wheel load of 150 kN by the tandem load system model 1 suggested by Eurocode 1 (BS EN 1991-3:1995) (BSI, 1995) was considered as the service load.

The first crack on all the slabs occurred during the test load and further cracks parallel to the first crack were observed at the soffit under subsequent load increments. Similarly, cracks were noticed at the top surface of the slab adjacent to the restrained edges. Both BFRP and GFRP reinforced slabs showed similar crack patterns. The maximum crack width at the service load of 150 kN was 0·33 mm (Figure 5). The current design code limits the crack width at service load to 0·3 mm in steel reinforced concrete slabs to protect the steel from corrosion. However, FRP reinforced slabs have no such requirements other than for aesthetic reasons as FRP bars are resistant to corrosion. The American Concrete Institute (ACI) guideline (ACI, 2006) and Canadian design code (CSA, 2006) allow larger crack widths up to 0·5 mm for FRP reinforced flexural members.

The strain measured on the reinforcing bars of the slabs showed that the strain was up to 20% of the rupture strain at the service load of 150 kN (Table 4). The difference noticed between BFRP and GFRP strain values can be attributed to the difference in the elastic modulus of BFRP and GFRP. The strain on FRP bars can be compared with the Canadian highway bridge design code (CHBDC) (CSA, 2006) recommendation that limits the maximum strain on FRP bars to 25% of the ultimate rupture strain at service load level. Eurocode 2 does not provide any recommendation to limit the strain on FRP bars. Therefore, the CHBDC recommendations have been used as a limiting strain value for the FRP bars at service load.

4.2 Ultimate load behaviour

Brittle behaviour of FRP bars can cause catastrophic structural failure due to FRP rupture. The ductility of steel bars ensures a more gradual energy dissipation in steel reinforced concrete slabs. Therefore, the steel reinforced concrete slabs are often designed with less than the balanced amount of reinforcement.

	Tensile tests loading rate 0·2 kN/s			Manufacturer's reported values loading rate 1 kN/s		
Reinforcement	Tensile strength: MPa	Elastic modulus: GPa	Ultimate strain: με	Tensile strength: MPa	Elastic modulus: GPa	Ultimate strain: μS
12 mm GFRP	682·0	67·4	10 120·0	>1000·0	>60·0	16 666·0
12 mm BFRP	920·0	54·0	17 037·0	1200·0	50·0	24 000·0

Table 3. FRP material properties

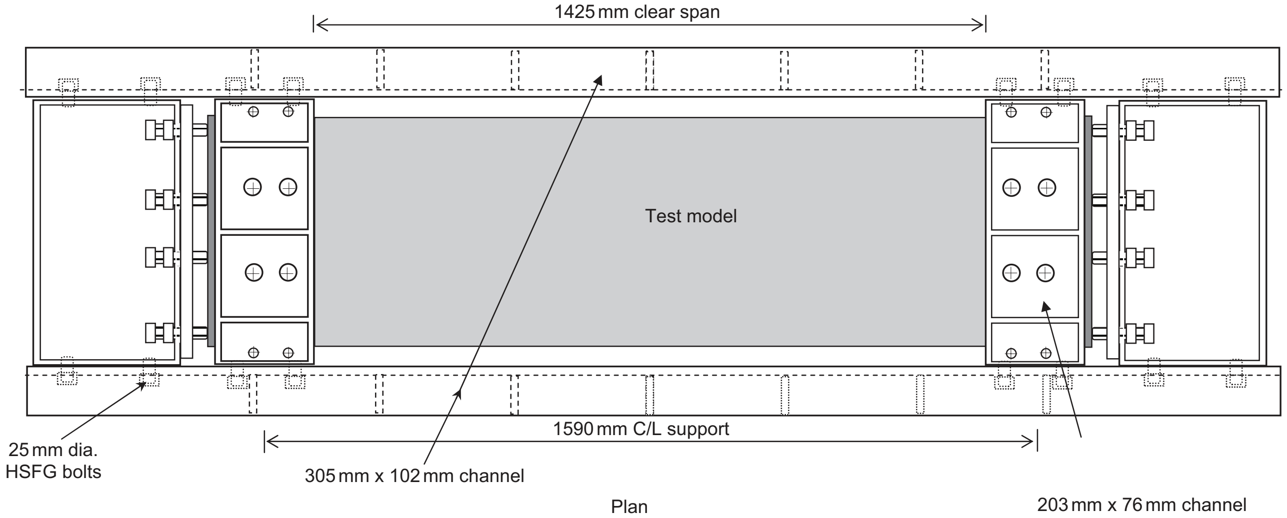

Figure 2. Plan view of the test arrangement to test in-plane restrained slabs; CL – centre line; HSFG – high-strength friction grip

At the balanced amount of reinforcement the slabs can be expected to fail by simultaneous steel and concrete failure. Both FRP bars and concrete are brittle materials, so the concrete crushing failure mode is more desirable in FRP reinforced concrete structures (ACI, 2006). As it has been established that in-plane restrained slabs generally fail by concrete crushing due to the influence of CMA, replacing steel with FRP bars in restrained slabs does not require any additional provision as a concrete crushing failure mechanism occurs.

Typical failure modes of the test slab are shown in Figure 6. All the test slabs failed by concrete crushing at a far higher load than flexural and shear predictions using Eurocode 2 (BSI, 2005). However, the test results showed a good correlation with the arching theory proposed for in-plane restrained slabs by QUB (Taylor *et al.*, 2002). The failure load prediction indicates that significant CMA occurred and improved the service and ultimate behaviour of FRP reinforced in-plane restrained slabs.

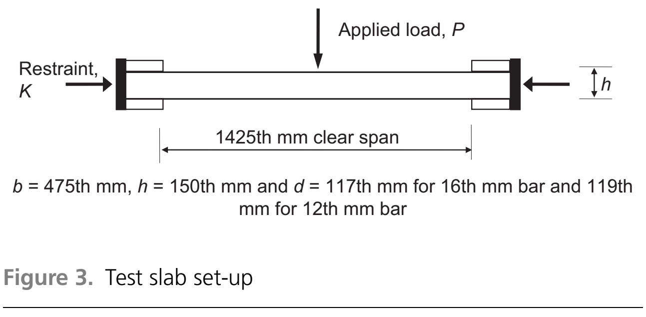

Figure 3. Test slab set-up

5. Prediction of failure load using yield line theory and arching theory

Arching theory developed at QUB demonstrated good accuracy in predicting the strength of in-plane restrained slabs by taking into account the strength enhancement due to arching action. The theory was developed based on a three-hinge analogy, which takes into account the stiffness of the external restraint and the influence of the concrete strength to predict the ultimate strength capacity of the slab. The contact depth takes into account the influence of the concrete strength to predict the ultimate strength capacity of the slabs. QUB arching theory provides a far more accurate prediction of the strength of in-plane restrained slabs compared with Eurocode 2 (BSI, 2005), particularly in high-strength concrete slabs, as Eurocode 2 does not consider the influence of CMA. A comparison of strength predicted by arching theory (P_{QUB}) and Eurocodes (P_{EC}) for the test slabs is given in Table 5.

Design codes, such as Eurocode 2, assume that the strength of concrete slabs is based on pure flexure and shear capacities. Equations 1 and 2 show the calculation of moment capacity of a restrained slab using the Eurocode 2 method.

1. $$M_{\mathrm{b}} = \rho_{\mathrm{y}} \times f_{\mathrm{y}}[1 - 0{\cdot}5(\rho_{\mathrm{y}} \times f_{\mathrm{y}}/f_{\mathrm{ck}})] \times bd^2$$

2. $$P_{\mathrm{b}} = M_{\mathrm{b}} \times 8/L$$

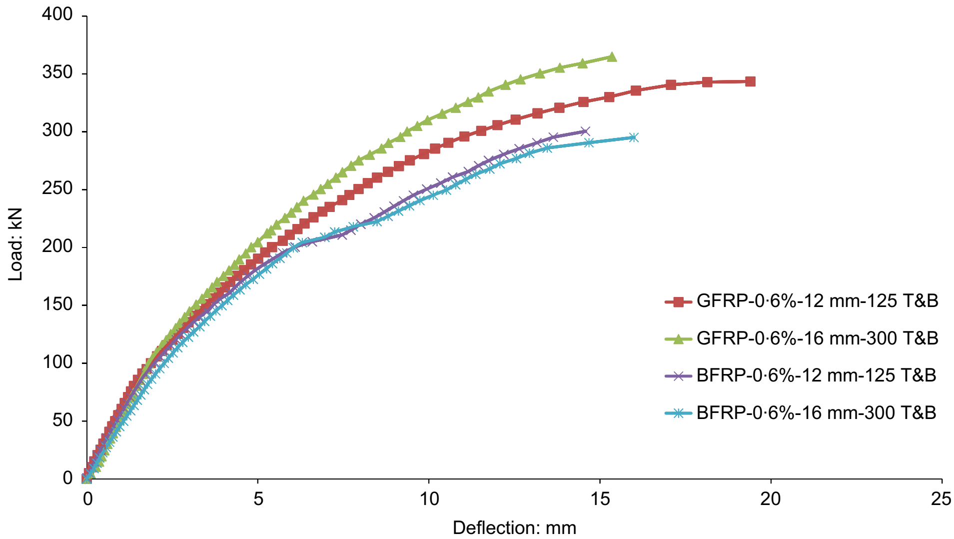

Figure 4. Load against deflection behaviour of the test slabs

From Table 5, it can be noticed that the strength capacity determined from pure flexure and rotational restraint for restrained slabs provides a highly conservative prediction that can lead to providing far more reinforcement than is needed. In the case of GFRP-0·6%-16-125 T&B the Eurocode 2 prediction was less than half the real ultimate load.

The contribution of the internal arching moment as a result of in-plane restraint is not considered in the Eurocode 2 flexural design equations. The shift of the neutral axis at the mid span and at the restrained edge in opposite directions causes the formation of an arching thrust (Figure 1). The enhancement of the ultimate capacity due to arching action is not considered in most of the design codes. Some acknowledge the enhanced strength capacity but use an empirical design method. Research at QUB proposed a method to evaluate quantitatively the arching contribution. The method of Rankin (1982) was based on the equation of McDowell *et al.* (1956) by extracting two non-dimensional parameters R and u (Equations 3 and 4).

3. $$R = \varepsilon_c \times L_r^2 / 4 \times d_1^2$$

4. $$u = w / 2d_1$$

The arching resistance of a fully rigid in-plane restrained slab can be calculated using Equations 5 and 6.

If $R > 0{\cdot}26$, then

5. $$M_r = 0{\cdot}3615 / R$$

Else, if $0 < R < 0{\cdot}26$, then

6. $$M_r = 4{\cdot}3 - 16{\cdot}1\left(3{\cdot}3 \times 10^{-4} + 0{\cdot}1234R\right)^{1/2}$$

Test slab	Concrete strength: MPa	Deflection at 150 kN	Maximum strain at 150 kN: % of ultimate strain	Crack width at 150 kN: mm	Deflection at failure: mm	Failure load: kN
GFRP-0·6%-12-125 T&B	68·1	L/407	20%	0·33	19·4	343·5
GFRP-0·6%-16-300 T&B	65·7	L/445	17%	0·31	15·4	364·9
BFRP-0·6%-12-125 T&B	69·3	L/385	10%	0·33	14·6	300·4
BFRP-0·6%-16-300 T&B	66·1	L/356	16%	0·28	16·0	295·1

Table 4. Summary of the tests

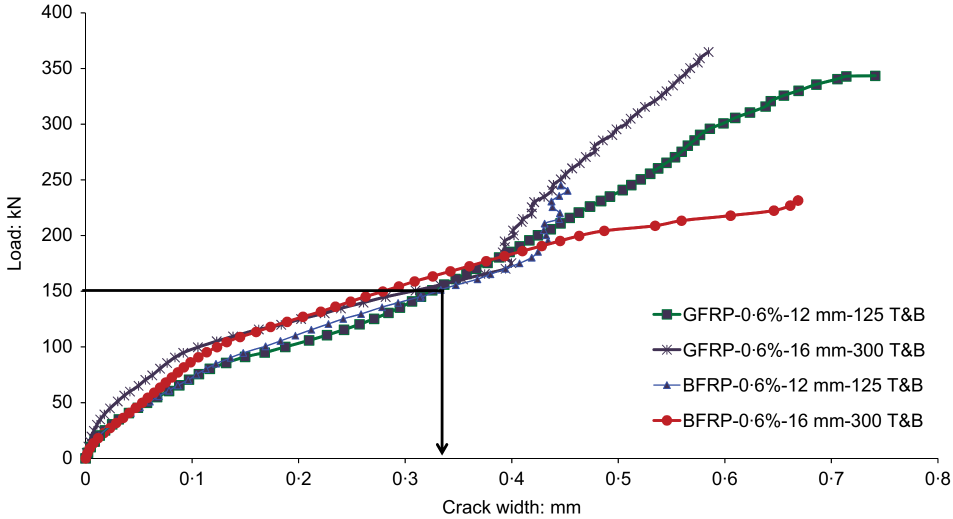

Figure 5. Crack width expansion with load increment

The arching moment (Equation 7) of the elastic arch can be calculated using the rigid moment M_r calculated in Equation 6. Finally the arching strength can be calculated using Equation 9.

$$M_a = 0{\cdot}168 \times b \times f_{ck,cube} \times d_1^2 \times M_r \times L_e/L_r \tag{7}$$

where

$$L_r = L_e[E_c A/K_r L_e + 1]^{1/3} \tag{8}$$

Figure 6. Crack on top surface and failure mode of FRP reinforced restrained slabs

Strength due to arching resistance

$$P_a = M_a \times 4/L_e \tag{9}$$

Table 5 shows that the failure load predicted by arching theory gives a better correlation than the strength predicted by normal flexural strength calculations. The standard flexural equation is too conservative to predict the strength of in-plane restrained slabs.

6. Comparison to existing design criteria

Eurocode 2 (BSI, 2005) provides recommendations for steel reinforced concrete structures. Allowances are given for minimum deflection and maximum crack widths to classify steel reinforced concrete structures as serviceable. Minimum reinforcement ratio, longitudinal bending capacity and shear resistance are recommended for ultimate limit state. However, Eurocode 2 does not provide service limit or ultimate limit state recommendations for FRP reinforced structures. Therefore, the recommendations for steel reinforced concrete slabs were adopted with few inclusions from ACI (2006) and CHBDC (CSA, 2006) to examine the service limit and ultimate limit state behaviour of GFRP and BFRP reinforced slabs.

The FRP bars are corrosion resistant and do not require stringent crack control measures recommended for steel reinforced structures. The ACI (2006) guideline and CHBDC (CSA, 2006) recommend increasing the maximum crack width limit to 0·5 mm where the crack width is limited to 0·3 mm for steel structures. Therefore, the test slabs shall be compared for the maximum crack width of 0·5 mm. Steel reinforced decks are

Test slab	Boundary conditions	Concrete strength: N/mm^2	P_t: kN	P_{EC}: kN	P_{QUB}: kN	P_t/P_{EC}	P_t/P_{QUB}
GFRP-0·6%-12-300 T&B	In-plane restrained	68·1	343·5	193·6	294·2	1·77	1·17
GFRP-0·6%-16-125 T&B	In-plane restrained	65·7	364·9	170·0	272·6	2·14	1·34
BFRP-0·6%-12-300 T&B	In-plane restrained	69·3	300·4	259·4	284·5	1·16	1·06
BFRP-0·6%-16-125 T&B	In-plane restrained	66·1	295·1	227·6	275·3	1·07	1·07
S7a	Simply supported	91·0	50·0	48·4	48·4	1·03	1·02
S8a	In-plane restrained	100·1	183·0	64·6	153·0	2·83	1·20

[a]Slabs tested by Taylor *et al.* (2001)

Table 5. Comparison of strength predicted by arching action theory

designed to fail by yielding of steel reinforcement, by providing a less than balanced amount of reinforcement. However, FRP reinforced structures are designed to fail by concrete crushing, as failure due to FRP rupture is considered more catastrophic. Therefore, it requires limiting the allowable stress on FRP bars at service load level to prevent FRP rupture. However, in slabs with restraint typical of that in a beam and slab bridge, the arching action tends to induce compression failure in both steel and FRP reinforced deck slabs.

The comparison given in Table 5 shows the significant influence of CMA to enhance the strength of in-plane restrained slabs. Furthermore, the slabs discussed in this paper were reinforced with the same amount of reinforcement recommended by the addendum to the Northern Ireland bridge design code for steel reinforced restrained slabs. The provision of the same amount of FRP reinforcement (by area) and the similar performance of the FRP reinforced slabs further demonstrated the improved behaviour of restrained slabs compared to the established perception of deck slabs in beam and slab bridges. Although the ACI (2006) guideline recommends more than a balanced amount of reinforcement to achieve concrete crushing failure, the failure mode of the restrained slabs indicates that GFRP and BFRP reinforced slabs fail by concrete crushing due to the effect of CMA even when they were reinforced with less than a balanced amount of reinforcement.

The arching phenomenon in steel reinforced restrained slabs with varying concrete compressive strength was investigated by Taylor *et al.* (2001) by comparing two slabs in which one was simply supported (S7) and the other was simply supported but with in-plane restraint (S8). The provision of the in-plane restraint in slab S8 enhanced the strength of the slab by more than three times that of the simply supported slab with no in-plane restraint. In addition to the above comparison, Taylor and Mullin (2006) also demonstrated the similar behaviour in equivalent GFRP and steel reinforced slabs due to the benefits of CMA.

7. Conclusions

The following conclusions can be drawn from the tests carried out on BFRP and GFRP reinforced slabs. Eurocode 2 was used to evaluate the maximum allowable deflection of the test slabs and recommendations from CHBDC (CSA, 2006) were considered for the allowable maximum crack width and reinforcement stress at service load level as Eurocode 2 does not include FRP. The comparison of the test results with the design code recommendations show that both GFRP and BFRP reinforced slabs satisfy the service load level requirements. The maximum deflection at service load level was less than $L/250$ and crack width was less than 0·4 mm.

For ultimate limit state, the slabs need to carry a tandem load system model I wheel load of 150 kN and also need to demonstrate a non-catastrophic failure. Concrete crushing failure mode is a preferred failure mode for FRP reinforced structures (ACI, 2006). All four slabs failed by concrete crushing and carried load in excess of 150 kN. The failure loads were higher than the load predicted by both Eurocode 2 and ACI guidelines.

The predictions using the flexural theory from Eurocode 2 and ACI guidelines give highly conservative failure loads for in-plane restrained slabs. The test slabs in this research demonstrated strengths far in excess of the flexural theory predictions using the current standards. The variation noticed in ultimate failure loads between BFRP and GFRP reinforced slabs can be attributed to the different rupture strength of bars. This study demonstrates that the FRP reinforcement can be a durable alternative in bridge decks due to the beneficial influence of CMA. However, the provision of appropriate guidelines can improve the appreciation of GFRP and BFRP bars to replace steel in bridge deck slabs such as deck slabs in beam and slab bridges.

7.1 Practical relevance and potential applications

Most of the concrete slabs in beam and slab bridges are constructed using steel reinforcement. Although steel is a popular reinforcing material, the steel reinforced structures exposed to corrosive conditions require constant monitoring for their serviceability due to the damage caused by steel corrosion. This research demonstrates the ability of corrosion-resistant FRP bars to replace steel. Although FRP bars have a lower stiffness and exhibit brittle behaviour, the research presented in this paper proves that the GFRP and BFRP bars can be good alternatives to steel in restrained slabs, such as deck slabs in beam and slab bridges. The test results show that the slabs reinforced with FRP bars, equivalent to steel satisfy the service and ultimate limit state requirements recommended by design codes. Successful application of corrosion-resistant reinforcement in civil infrastructure can contribute to more economical and durable bridges, as structures reinforced with FRP bars require minimal maintenance. The practice of using corrosion-resistant reinforcement can significantly reduce the cost of repairing bridge structures.

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REFERENCES

ACI (American Concrete Institute) (2006) ACI 440.1R-06. Guide for the design and construction of concrete reinforced with FRP bars. ACI, Michigan, USA.

ASCE (American Society of Civil Engineers) (2009) Report card for America's infrastructure. ASCE, Reston, VA, USA.

Benmokrane B, El-Salakawy E, Cherrak Z and Wiseman A (2004) Fibre reinforced polymer composite bars for the structural concrete slabs of a Public Works and Government Services Canada parking garage. *Canadian Journal of Civil Engineering* **31(5)**: 732–748.

BSI (1995) BS EN 1991-3:1995: Eurocode 1: Actions on structures. Part 2: Traffic loads on bridges. BSI, London, UK.

BSI (2005) BS EN 1992-2:2005: Eurocode 2: Design of concrete structures. Part 2: Concrete bridges – design and detailing rules. BSI, London, UK.

BSI (2009) BS EN 12390-3:2009: Testing hardened concrete. Compressive strength of test specimens. BSI, London, UK.

Castro PF and Carino J (1998) Tensile and non-destructive testing of GFRP bars. *Journal of Composites for Construction, ASCE* **2(1)**: 17–27.

Christiansen K (1963) The effect of membrane stresses on the ultimate strength of interior panel of reinforced concrete slab. *Structural Engineer* **41(8)**: 261–265.

Clarke JL (1999) *Fibre-Reinforced Plastic Reinforcement for Concrete*. The Concrete Society, London, UK, Current practice sheet no. 114.

CSA (Canadian Standards Association) (2006) CAN/CSA-S6-06. Canadian Highway Bridge Design Code. CSA, Ontario, Canada.

Department of Regional Development (DRD) (1986) *Design of M-Beam Bridge Decks, Amendment No. 3 to Bridge Design Code*. Northern Ireland Road Service Headquarters, Belfast, Northern Ireland, 11.1–11.5.

El-Gamal S, El-Salakawy E and Benmokrane B (2007) Influence of reinforcement on the beahviour of concrete bridge deck slabs reinforced with FRP bars. *Journal of Composites for Construction, ASCE* **11(5)**: 449–458.

Faulkes KA (1974) *The Design of Flat Slab Structures – An Historical Survey*. University of New South Wales, Australia.UNICIV report no. R-129

Gvodzev AA (1939) Comments on clause 33 of the Russian code for reinforced concrete. *Stroitelnaya Promyshlennest* **17(3)**: 51–58 (in Russian).

HA (Highways Agency) (2007) BD81/02: Use of compressive membrane action in bridge decks. *Design Manual for Roads and Bridges*. HA, London, UK, Vol. 3, Section 4, part 20

Keesler R and Powers R (1988) *Corrosion of epoxy coated rebars – Keys Segmental Bridge, Monroe County*. Report no. 88-8A. Corrosion Research Laboratory, Florida Department of Transportation, Gainesville, FL, USA.

Kirkpatrick J, Long A and Rankin G (1986) The influence of compressive membrane action on serviceability of beam and slab bridge decks. *Structural Engineer* **64b(1)**: 6–9.

Liebenberg A (1966) *Arch Action in Concrete Slabs (Parts 1 and 2)*. Bulletin No. 40. National Building Research Institute, Pretoria, South Africa, CSIR research report 234, p. 55.

McDowell E, McKee K and Sevin E (1956) Arching action theory of restrained masonry walls. *Journal of Structural Division, Proceedings of ASCE* **82**: 915–933.

Michaluk CR, Rizkalla SH, Tadros G and Benmokrane B (1998) Flexural behaviour of one-way concrete slabs reinforced by fiber reinforced plastic reinforcements. *ACI Structural Journal* **95(3)**: 353–365.

Ockleston A (1955) Load tests on a 3-storey RC building in Johannesburg. *Structural Engineer* **33(10)**: 304–322.

Ospina CE and Bakis CE (2006) Indirect crack control procedure for FRP reinforced concrete beams and one way slabs.

Proceedings of the Third International Conference on FRP Composites in Civil Engineering (CICE 2006), Miami, FL, USA

Park R and Gamble P (1980) *Reinforced Concrete Slabs*. John Wiley, London, UK.

Rankin G (1982) *Punching Failure and Compressive Membrane Action in Reinforced Concrete Slabs*. PhD thesis, Queen's University Belfast, UK

Rankin G and Long A (1998) Arching action strength enhancement in laterally restrained slab strips. *Proceedings of ICE – Structures and Buildings* **122(4)**: 461–467.

Taylor SE and Mullin B (2006) Arching action in FRP reinforced concrete slabs. *Construction and Building Materials* **20**: 71–80.

Taylor SE, Rankin GIB and Cleland DJ (2001) Arching action in high-strength concrete slabs. *Proceedings of ICE – Structures and Buildings* **146(4)**: 353–362.

Taylor SE, Rankin GIB and Cleland DJ (2002) *Guide to Compressive Membrane Action*, Technical paper no. 3. Concrete Bridge Development Group, Berkshire, UK.

Taylor SE, Sonebi M, Robinson D and Grattan S (2013) Application and structural behaviour of self-consolidating concrete used in Thompson's bridge deck. *Proceedings of the Fifth North American Conference on the Design and Use of Self-Consolidating Concrete (SCC), Chicago, USA*

Tong PY and Batchelor BdV (1971) *Compressive Membrane Enhancement in Two-way Bridge Slabs*. American Concrete Institute, Michigan, USA, SP 30–12.

Wallbank E (1989) *The Performance of Concrete in Bridges – A Survey of 200 Highway Bridges*. HMSO, London, UK.

Wood RH (1961) *Plastic and Elastic Design of Slabs and Plates*. Thames and Hudson, London, UK.

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