LABORATORY INVESTIGATION ON THE BEHAVIOR OF GFRP-REINFORCED CONCRETE DECK SLABS

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1 INTRODUCTION

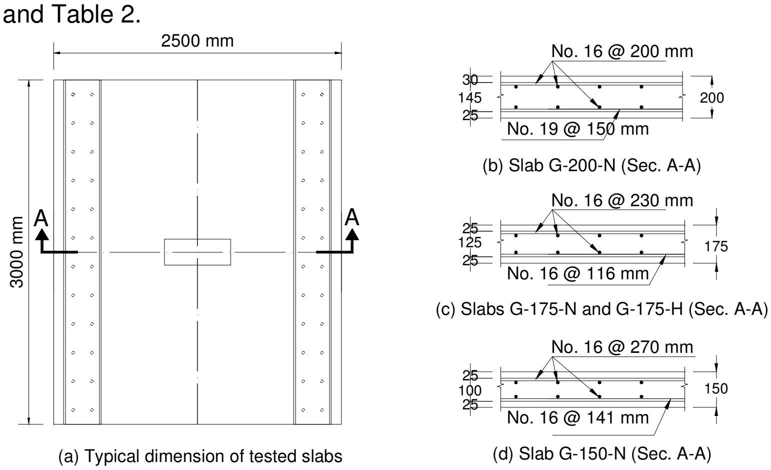
This research study is the third phase of a comprehensive experimental program conducted at the Department of Civil Engineering at University of Sherbrooke to investigate the behavior of restrained concrete bridge deck slabs reinforced with FRP bars. The first and second phases investigated the behavior of concrete bridge deck slabs under static and fatigue loads (El-Gamal et al. 2005; El-Ragaby et al. 2007). In those phases, the thickness of the slabs was 200 mm and normal strength concrete was used for all specimens. The third phase of this study, which is presented herein, investigates two additional parameters: (a) the deck slab thickness; and (b) the concrete compressive strength. This study will help for more understanding of the influence of these two parameters on the behavior of FRP-reinforced concrete bridge deck slabs. This could also assist in finding a good combination of the deck slab thickness and the concrete strength that allows the construction of more efficient and less expensive bridge deck slabs.

2 EXPERIMENTAL STUDY

2.1 Test Specimens

Four full-scale concrete deck slabs of 3000-mm long × 2500-mm wide × (150, 175, and 200-mm) thick, as shown in Figure 1, were constructed. All the slabs were reinforced with two mats of orthogonal glass FRP bars. Two different diameters (*ϕ 19 and ϕ 15.9 mm*) of sand-coated GFRP bars (V-ROD^{TM}), manufactured by Pultrall inc. (Thetford Mines, Quebec), were used. The mechanical properties of the reinforcing bars used in this study are summarized in Table 1.

For all slabs, the bottom transverse GFRP reinforcement ratio was 1.2% while the GFRP reinforcement ratio in all other directions was 0.6%. The slabs were divided into two groups. The first group was designed to investigate the effect of slab thickness, while the second group was designed to investigate the effect of concrete strength. Group 1 contained three normal strength concrete deck slabs (G-200-N, G-175-N and G-150-N) with three different thicknesses (200, 175 and 150 mm), respectively. Group 2 contained two deck slabs; G-175-N and G-175-H. The two slabs had identical dimensions and reinforcement; however high strength concrete (HSC), 64.8 MPa, was used for the G-175-H slab. The reinforcement details and concrete compressive strength of the all deck slabs are shown in Figure 1 and Table 2.



(a) Typical dimension of tested slabs

(b) Slab G-200-N (Sec. A-A)

(c) Slabs G-175-N and G-175-H (Sec. A-A)

(d) Slab G-150-N (Sec. A-A)

Fig 1 Dimensions and reinforcement details of tested slabs

Table 1 Mechanical properties of GFRP reinforcing bars

Bar Designation	Diameter (mm)	Area (mm^2)	Modulus of Elasticity (GPa)	Tensile Strength (MPa)	Ultimate Strain (%)
No. 16	15.9	198	44.6 ± 0.8	727 ± 9	1.65 ± 0.03
No. 19	19.0	283	44.5 ± 1.3	637 ± 15	1.37 ± 0.03

Table 2 Test matrix

Group	Slab	Slab Thickness(mm)	f_c' (MPa)	Reinforcement configuration	
				Bottom transverse direction	Other directions
1	G-200-N	200	49.1	No. 19 @ 150 mm	No. 16 @ 200 mm
	G-175-N	175	35.2	No. 16 @ 116 mm	No. 16 @ 230 mm
	G-150-N	150	35.2	No. 16 @ 141 mm	No. 16 @ 270 mm
2	G-175-N	175	35.2	No. 16 @ 116 mm	No. 16 @ 230 mm
	G-175-H	175	64.8	No. 16 @ 116 mm	No. 16 @ 230 mm

2.2 Test Setup and Procedure

Similar to the first two phases of this research program (El-Gamal et al. 2005 and El-Ragaby 2007) and to simulate bridge deck slabs with restrained edges, the slabs were supported on two steel girders spaced at 2 m centre-to-center. The girders were connected together by two cross frames spaced at 3 m. Each cross frame consisted of an X-shaped bracing with $55 \times 55 \times 6$ mm angles, as sown in Figure 2. Each slab was bolted to the top flange of the steel girders through two rows of holes at each edge using 25 mm-diameter steel bolts and two steel channels. In addition, a 3 mm-thick neoprene pad was used between the steel sections (channel and top flange of girder) and the concrete slab (El-Gamal et al. 2005). The deck slabs were tested up to failure using one concentrated load over a contact area of 600×250 mm to simulate the foot print of the truck wheel load; CL-625 Truck as specified in the CHBDC (CAN/CSA-S6-06) acting on the center of the slab. In all tests, the load was applied through a 60 mm-thick steel plate over 10 mm-thick neoprene pad. The load was monotonically applied at a load controlled rate of 5 kN/min. Figure 3 shows one slab during testing.

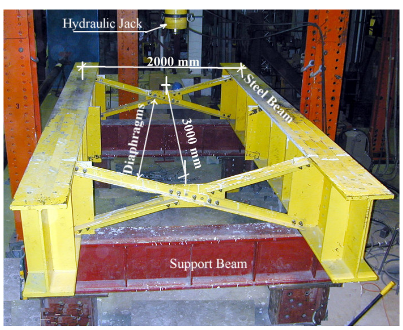


Fig. 2 The supporting girders and cross frames

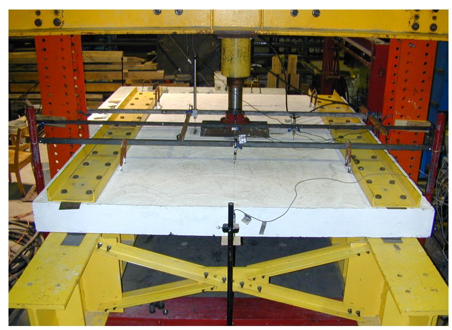


Fig. 3 Photo of one slab during testing

2.3 Test Results and Discussion

Cracking Pattern and Crack Width -- For all tested slabs, the first cracks appeared directly under the loaded area and were oriented in the longitudinal direction parallel to the supporting beams. Subsequent cracks propagated in the radial direction away from the loaded area. For group 1, the crack patterns were similar. In group 2, slab G-175-H (with high strength concrete) had more cracks on the tension face compared to slab G-175-N (with normal strength concrete). In addition, increasing the concrete strength from 35.2 MPa (G-175-N) to 64.8 MPa (G-200-H) increased the cracking load from 115 to 130 kN (13%) as listed in Table 3.

Figure 4 shows the variation of maximum measured crack width against the applied load for all tested slabs. It can be noticed that the load-crack width values of all slabs were similar except for slabs G-150-N where higher crack width values were measured. This indicates that decreasing the thickness of slabs from 200 to 175 mm did not significantly affect the load-crack width relationship. However, by decreasing the thickness to 150 mm, which is less than the minimum thickness (175 mm) allowed by the CHBDC (CAN/CSA-S6-06), wider cracks were developed.

Table 3 Summary of test results

Group	Slab	Cracking load (kN)	Ultimate load (kN)	Maximum deflection (mm)		Maximum strains in reinforcement ($\mu\varepsilon$)		Mode of failure
				Service load	Failure	Service load	Failure	
1	G-200-N	115	732	0.45	22.9	624	6690	Punching
	G-175-N	115	484	0.72	17.9	214	6224	
	G-150-N	107	362	1.49	17.1	552	4872	
2	G-175-N	115	484	0.72	17.9	214	6224	
	G-175-H	130	704	0.45	21.7	214	7566	

Deflection -- Figure 5 shows the load-deflection curves of the tested slabs. For the slabs of group 1 (G-200-N, G-175-N and G-175-N), it can be noticed that, at the same load level, as the slab thickness decreased the deflections significantly increased. At service load level (110.25 kN), decreasing the slab thickness from 200 to 175 mm for slabs G-200-N and G-175-N increased the maximum deflection from 0.45 to 0.72 mm (60% more), while decreasing the slab thickness from 175 to 150 mm for slabs G-175-N and G-150-N doubled the maximum deflection from 0.72 to 1.49 mm (107% more).

For the slabs of group 2, it can be seen that slab G-175-H (with high-strength concrete) had less deflection values, at the same load level, compared to the slab G-175-N (with normal-strength concrete). At service load level, the maximum measured deflection of the slab G75-N was 0.72 mm (about 60 % more) compared to 0.45 mm for slab G-175-H. This indicates that the concrete strength significantly enhanced the deflection behavior of the tested GFRP-reinforced deck slab.

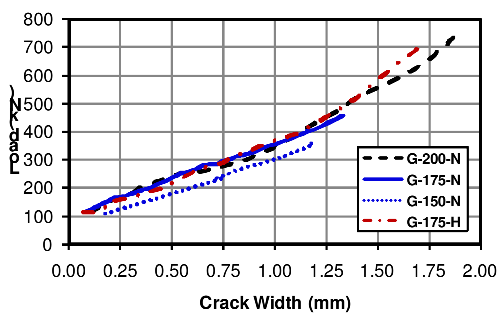


Fig. 4 Load-maximum crack width

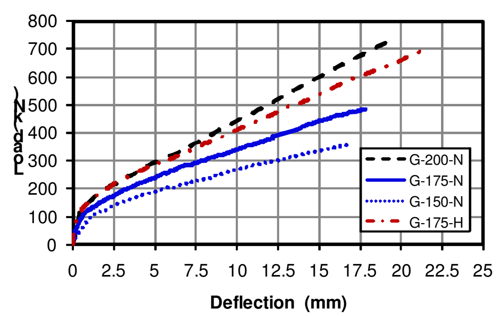


Fig. 5 Load-maximum crack width

Strains in FRP Reinforcement -- For all tested slabs, the maximum measured strains in the GFRP bars at service and factored load levels were about 4% and 15% of the ultimate strains of the GFRP bars, respectively. At failure, the maximum measured strains in the GFRP bar were less than 50 % of the ultimate strains of the GFRP bars, which is expected due to the punching shear failure.

Ultimate Capacity and Mode of Failure -- For all tested deck slabs, the failure mode was punching shear around the loaded area. For the slabs of group 1, G-200-N, G-175-N, and G-150-N, the carrying capacities were about 3.5, 2.3, and 1.7 times the factored design load (*P_f = 208.25 kN*) specified by the CHBDC. They significantly decreased with the decrease of the slab thickness. For slabs G-200-N and G-175-N, decreasing the slab thickness by 12.5% (from 200 to 175 mm) decreased the ultimate capacity by about 34%, while decreasing the slab thickness by 25% (from 200 to 150 mm) decreased the ultimate capacity by about 50%. For the slabs of group 2, the punching capacity increased with the increase of the concrete compressive strength. For slabs G-175-N and G-175-H, increasing the concrete compressive strength by 83% (from 35.4 to 64.8 MPa) increased the punching capacity by 45% (from 484 kN to 704 kN).

3 COMPARISON BETWEEN EXPERIMENTAL AND PREDICTED PUNCHING STRENGTHS

The punching shear strengths of the tested bridge deck slabs were predicted using the available models in the literature that predict the punching strength of FRP-reinforced slabs. This includes the models of the Japan Society of Civil Engineers (JSCE 1997), El-Ghandour et al. (1999), Matthys et Taerwe (2000), Ospina et al. (2003), El-Gamal et al. (2005), and ACI 440-06 (2006). The predicted punching shear strengths were compared to the experimental values as given in Table 4. It can be noticed from Table 4 that, the model proposed by El-Gamal et al. gives the superior predictions to the experimental results and yet was conservative. The average ratio of V_{exp}/V_{pred} is 1.05 with a coefficient of variation of 3.67%. On the other hand, it is evident that the punching shear model proposed by the ACI committee 440.1R in 2006 underestimates the punching strength of bridge deck slabs reinforced with FRP bars. The average ratio of V_{exp}/V_{pred} is 2.33 with a coefficient of variation of 17.9%.

Table 4 Comparison between predicted and experimental punching shear strengths

Slab	$V_{C,Exp}$ (kN)	$V_{experimental}/V_{predicted}$					
		JSCE	El-Ghandour	Matthys	Ospina et al.	El-Gamal et al.	ACI 440.1R-06
G-200-N	732	1.52	1.45	1.35	1.11	1.04	2.38
G-175-N	484	1.06	1.28	1.22	1.04	1.03	1.85
G-150-N	362	1.04	1.22	1.16	1.04	1.03	2.22
G-175-H	704	1.54	1.39	1.47	1.25	1.11	2.86
Mean		1.29	1.33	1.30	1.11	1.05	2.33
Coeff. of V. (%)		21.24	7.75	10.60	8.84	3.67	17.90

4 CONCLUSIONS

- Neither the slab thickness nor the concrete compressive strength affected the mode of failure (punching shear) of the tested slabs.
- For the slabs of group 1, decreasing the deck slab thickness by 12.5 and 25% increased the measured deflections at service load level by 60 and 230% (from 0.45 to 0.72 and 1.49 mm) and decreased the punching capacities by 34 and 51% (from 732 to 484 and 362 kN), respectively
- For the slabs of group 2, increasing the concrete compressive strength by 83% (from 35.8 to 64.3 MPa) significantly improved the performance of the deck slab. The deflection at service load level decreased by 60% and the punching capacity increased by 45%.
- The current ACI 440.1R-06 punching shear model underestimates the punching shear strength of the tested slabs. On the other hand, the model proposed by El-Gamal et al. (2005) gave the closest predictions to the experimental results and yet was conservative.

ACKNOWLEDGEMENTS

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