STRUCTURAL PERFORMANCE OF RETROFITTING BRIDGE DECK SLABS USING CFRP STRIPS UNDER CYCLIC LOADING

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ABSTRACT:

FRP composites have been widely used as internal and external reinforcement for concrete bridge deck slabs. However, experimental and numerical researches on the behavior of such FRP-reinforced elements in general have been limited, especially those on cyclic performance. This study evaluates the force-deformation responses of retrofitted bridge deck panels under cycling loading up to the failure. The numerical analysis is carried out in ANSYS environment. Punching shear is a common mode of failure of restraint concrete bridge deck specimens reinforced with steel or FRP composite bars under cyclic loading. Ruminating results indicates the greatest capacity achieved by FRP lay-up (0, 90). Using CFRP strip systems result in considerable upgrading of the structural capacity of deck slabs up to 24%.

KEYWORDS: Retrofitted, FRP, Deck slab, Cyclic loading

1. Introduction

Several concrete bridges were built in the entire world using GFRP composite bars as main reinforcement for their deck slabs but, little research has been carried out on concrete bridge decks retrofitted with FRP strips that can be used in emergency situation and after natural terrible phenomena like earthquakes and flood.
This research is designed to study the performance of concrete bridge deck slabs reinforced with GFRP bars and CFRP strips under cyclic loads. The research includes analytical investigation using the finite element analysis to evaluate the ultimate capacity and displacement of such deck slabs under cyclic load conditions that will be used to develop an analytical model to predict the performance of such elements.

2. Details of the analytical program

2.1. Test prototypes

The experimental program includes construction and testing of four full-size bridge deck prototypes (2500 mm width, 3000 mm length, and 200 mm thick). Three deck slab prototypes were reinforced with the same reinforcement ratios and configurations of GFRP bars and with different amount of CFRP strips and one slab prototype were reinforced with conventional steel bars as control.
Bottom and top concrete cover of 38 mm was used for all slab prototypes. The main bottom transverse GFRP reinforcement for three decks, CS-0, CS-1, and CS-2 was calculated based on the empirical design method recommended by Section 16 of the Canadian Highway Bridge Design Code (CHBDC) Clause 16.8.7.1, for internally restrained cast in place deck slabs [1]. According to this clause, a minimum FRP reinforcement area in the transverse bottom direction is set to $500ds/E_{FRP}$ where ds is the distance from the top of the slab to the centroid of the bottom transverse reinforcement in mm and E_{FRP} is the modulus of elasticity of the used FRP reinforcement in MPa. This reinforcement ratio was calculated to have the same axial stiffness as the average between the minimum and the recommended steel reinforcement ratio (0.25%) allowed by the code, Commentary C.16.8.7.1 [5]. This approach resulted in using No. 19 GFRP bars spaced at 150 mm in the bottom transverse direction with a reinforcement ratio of 1.2%. The longitudinal bottom reinforcement for the three slabs consists of No. 16 GFRP spaced at 200 mm with a reinforcement ratio of 0.6%. Different retrofitted strip

ratio for the bottom surface was used. For slab CS-1, and CS-2 CFRP strip with widths 600 mm was used in both directions (Figure 1) with different thickness as presented in Table 1. Slab CS-0, had no bottom retrofitted CFRP strips in both directions at all. The fourth slab BS-0 (control), reinforced with steel bars, and was designed according to the empirical method of the CHBDC (Section 8 – clause 8.18.4.2) [2], which recommend the use of an isotropic steel reinforcement ratio of 0.3% in all direction for the bottom and top layers. This resulted in using No. 10M spaced at 200 mm at top and bottom in both directions. Table 1 summarizes the reinforcement details of the four test prototypes.
All slabs were tested under a single cyclic concentrated load applied at the mid-span. This load was applied through a 75-mm thick steel plate that measured 250 × 600 mm, which is equivalent to the footprint of a wheel as specified by the CHBDC [2]. To avoid stress concentration, a 20-mm thick neoprene sheet was used between the steel plate and the concrete surface. To apply partial end restraint, the slab was tied to the two supporting girders (steel I-beams) spaced at 2000 mm (center-to-center) through steel bolts that were fitted into holes through the thickness of the slab. The typical dimensions of the test prototypes are shown in Figure 1.

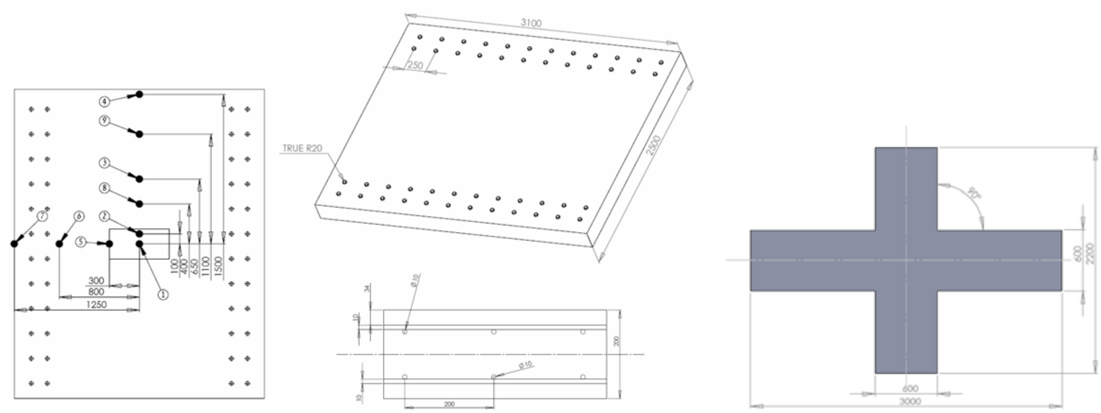

Figure 1 Geometry of considered bridge deck slab

The average 28-day concrete compressive strength based on existing test was 42 MPa and concrete tension strength assumed 3.9 MPa. The reinforcement was modeled assuming elasto-plastic behavior based on Von-Mises yield criterion with modulus of elasticity Es (200 GPa), yield strength f_y (400 MPa), and 1% hardening utilizing the work hardening hypothesis. Angle ply fiber (0/90) used to reinforce tow retrofitted slabs. Tsai Wu criterion was used to evaluate tearing mode of failure in FRP strips. Table 2 summarizes the material properties of all test prototypes.

Table 1 Specification of models

Deck Slab	**Material**		**Strip thicknesses (mm)**	**Reinforcement**	
	Bar	**Strip**		**Bottom transverse direction**	**Other direction**
CS-0	GFRP	-	-	No. 19 @ 150mm	No. 16 @ 200mm
CS-1	GFRP	CFRP	1	No. 19 @ 150mm	No. 16 @ 200mm
CS-2	GFRP	CFRP	1.5	No. 19 @ 150mm	No. 16 @ 200mm
BS-0	Steel	-	-	No. 10 @ 200mm	No. 10 @ 200mm

Due to the symmetry in cross-section of the concrete beam and loading, symmetry was utilized in the numerical modeling; in all cases only one quarter of the slab-column connections were modeled. The concrete, epoxy adhesive, CFRP strip and steel bars both longitudinal reinforcements and stirrups were modeled using appropriate elements in ANSYS software for example, FRP strips are modeled by layered, Solid-46, element to simulate mechanical characteristics of anisotropic nature of composite material. For better performance, the adhesive was considered using three layers (total thickness=1.5mm).

Table 2 Material properties

Material	Young Modulus (GPa)	Poisson's ratio	Strength (MPa)
Steel	200	0.3	401
Concrete	34	0.15	42
GFRP	45	0.3	600
CFRP	E_{11}=140, E_{22}=E_{33}=14	0.3	1500
	X^T:1500 X^C:1200	**Y^T:50**	**Y^C:250 S:70**
Adhesive	0.814	0.35	32

2.2. Validation of the model

The concrete slab was modeled using 8-node brick elements, solid65. Reinforcing bars and bolts were modeled using embedded rebar element, 2-node link element (Link8), with stiffness equal to the actual axial stiffness of each element either in tension or in compression. The load is applied as a uniformly distributed pressure over the top area through incremental processes up to the failure. Small load increments were used at the load level in the proximity of the expected cracking and failure loads to closely capture the behavior and to increase the accuracy of the analysis. Figure 2 shows comparisons between the FEM out-put and test results of a GFRP-reinforced deck slab prototype under monotonic loads only. It can be noticed that the FEM can predict the behavior, ultimate capacity, and strains with high accuracy.

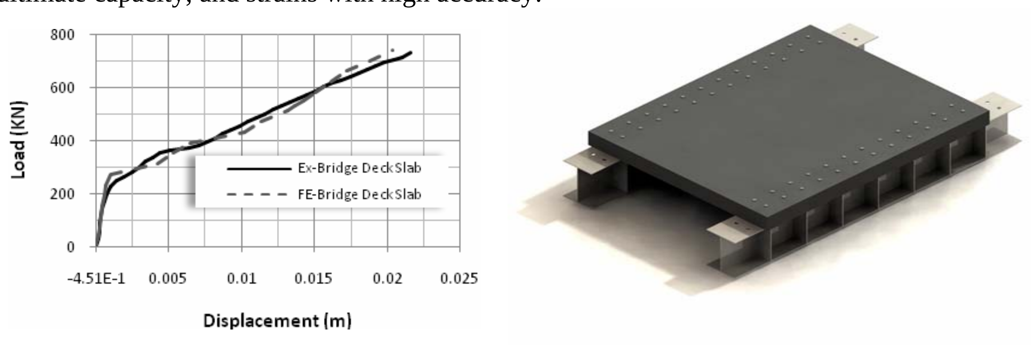

Figure 2 a: Comparisons between the FEM and test Load-Displacement relation b: the schematic view of slab

It is seen that the force-displacement behaviors of the un-retrofitted samples are approximately bilinear. The first phase shows the linear elastic behavior of the structure up to the tension cracking in concrete slabs. Since the tensile concrete is dismissed, and the stiffness is reduced; the structure enters into its second phase until the tensile steel rods yield. The collapse load is corresponded to the final deformation where the plastic stiffness becomes flatten out and the complete mechanism is formed. However, there is a difference exit between numerical and experimental results at the last stage owing to limitation of numerical modeling which is not able to consider completely cracked tested specimen behavior.

3. Analytical results

For simulating the actual condition and achieved more accurate response, retrofitted deck slabs were loaded cyclically. Loading pattern is presented in Figures 3a. In these figures, the variation of cyclic loading in time and the load-displacement alteration are plotted. In numerical analysis the applied displacement were used rather than applying load to have observation of load failure. In other words the displacement control for finding the ultimate capacity of the specimen under cyclic loading was selected to have a better access to the load-displacement on post collapse path. The cyclic loads were applied on the centre of slab and the corresponding displacement were measured at the seven locations, see Figure 1.The envelope curves as a out coming of applied cyclic loading regime versus vertical displacement were plotted in Figure 3b.

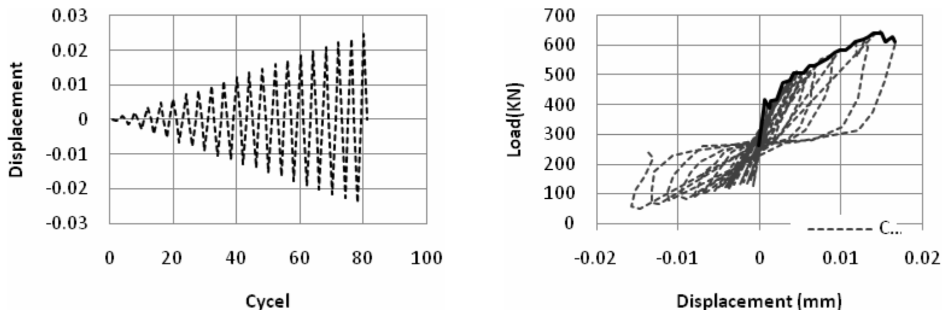

Figure 3 a: Loading pattern b: pushover response

3.1. Load-Deflection Characteristics

The load-deflection behavior of the four deck slabs at different positions is shown in Figure 4. For all slabs the deflection were measured at seven positions (Figure 1). It can be noted from Figure 4 that the deflection behavior at different positions is similar for the four deck slabs. In addition, the maximum measured deflection was recorded at position 1 near the loaded area for all tested slabs. It can be noted that the deflections decreased as the distance from the loaded area increased. Also, the deflections at different positions increased by increasing the load except at position 7 (the cantilever edges at the centre-lines of longitudinal direction of the deck slabs) where the deflections increased until load levels of 140- 150 KN for different slabs BS-0, CS-0, CS-1, and CS-2. After that and due to the rotation of the slab edges the deflections decreased (the cantilever moved up) up to failure.

Figure 5 compares the load-maximum net deflection (the deflection of the slab minus the deflection of the steel girders) for the four analytical deck slabs. The load-deflection curves for the FRP-reinforced concrete slab, CS-0 was bilinear. The first part up to the cracking loads (145.5 kN) represents the behavior of the un-cracked slab utilizing the gross inertia of the concrete cross-section, while the second part represents the cracked slab with reduced inertia. However, the load-deflection curve of the control slab (BS-0) was linear up to the cracking load (146.5 kN) then changed to non-linear behavior up to failure due to the cracking of concrete and the yielding of steel reinforcement. It can be also observed from Figure 5 that the control slab (BS-0) also had similar deflection behavior up to 75% of its ultimate capacity then the deflection increased up to failure due to the yielding of the steel reinforcement. This indicates that the reinforcement type (steel and GFRP with comparable axial stiffness) has an insignificant effect on the load-deflection behavior of the tested restrained bridge deck slabs.

The load-maximum net deflection for the two slabs retrofitted with carbon FRP strips (CS-1 and CS-2) are shown in Figure 5. The load maximum deflection curves for the tow slabs were multi-linear. The first part up to the cracking loads (168 and 169 kN) represents the behavior of the un-cracked slab utilizing the gross inertia of the concrete cross-section, while the second part represents the cracked slab with reduced inertia. However, due to the stiffness of CFRP strip, in this tow tested slabs the stiffness was reduced less than the other (BS-0 and CS-0).

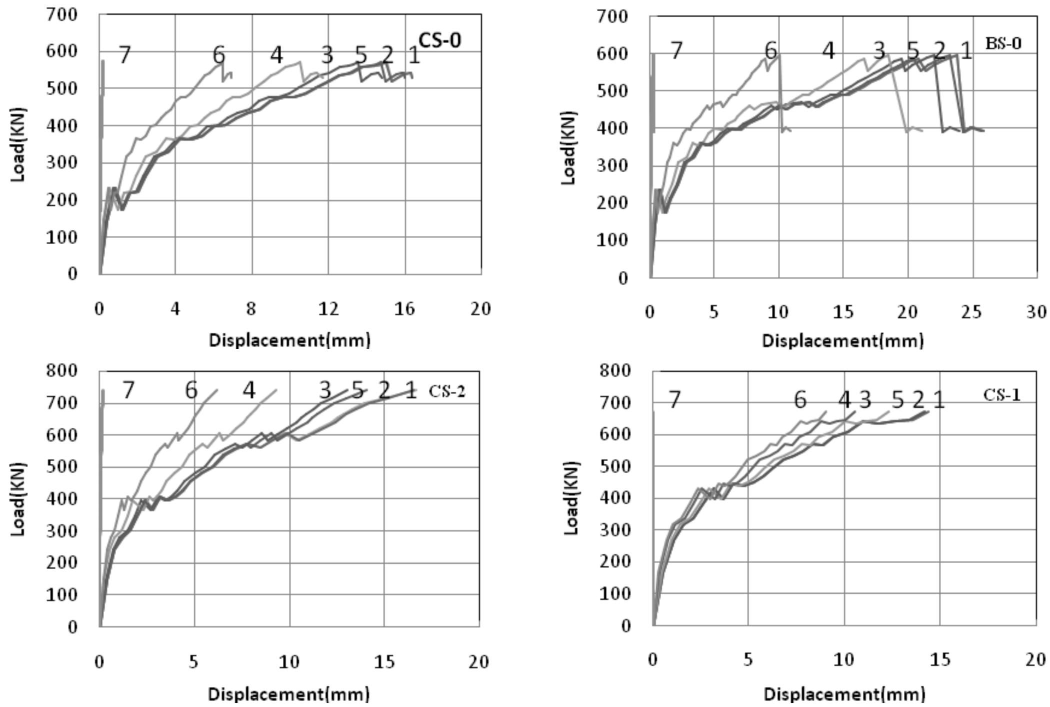

Figure 4 Load-Displacement relations of samples

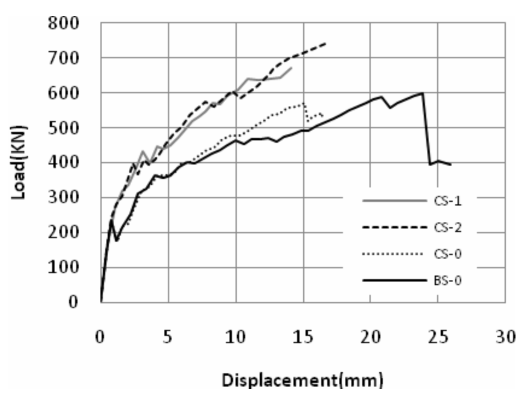

Figure 5 Comparisons between maximum net of tested deck slabs

The maximum recorded net deflections for all slabs at cracking and ultimate load levels are listed in Table 3. Figure 5 and Table 3 show that, at cracking load level, the maximum measured net deflections were 0.41, 0.41, 0.53, and 0.52 mm (difference =0.12 mm) for slabs BS-0,CS-0, CS-1, and CS-2, respectively. At failure, these values were 25.9, 16.4, 14.1, and 16.6 mm, respectively. This indicates that reinforcing and retrofitting the slabs by GFRP bars and CFRP strips did not affect the deflections at cracking load level; however it decreased the maximum recorded deflections at failure by about 35%.

Table 3 Summary of analytical results

Deck Slab	Cracking Displacement(mm)	Ultimate Displacement(mm)	Cracking Load(KN)	Ultimate Capacity(KN)	BDS CS-i /BDS St	Mode of failure
BDS CS-0	0.4	16.4	145.5	571.9	-4.2%	**Punching**
BDS CS-1	0.53	14.1	168.1	672.6	12.6%	
BDS CS-2	0.52	16.6	169.3	740.6	24.1%	
BDS St	0.41	25.9	146.5	597.1		

3.2. Deflection Profiles

The deflection profiles of the tested deck slabs were obtained along the centerlines in the transverse and longitudinal directions. The deflection profiles give a global indication of the deformation response of the slabs not only under the loaded area but also along the slab width. In addition, by using deflection profiles, the rotational response of the slabs can be specified. The deflection values at nine locations along the slabs centerlines (Figure 1) were recorded at different load stages, and were used to determine the deflection profile at each load stage.
Figures 6 and 7 show the deflection profiles along half of the longitudinal and transverse directions of slabs BS-0, CS-0, CS-1 and CS-2 at different load levels, respectively. It can be noted that the two FRP-reinforced slabs had very similar deflection profiles up to failure. Figures 6b, c, d and 7b, c, d show that, at higher load levels, the deflection increase stopped at the free edge (position 4) of the two FRP-reinforced deck slabs and the deflections increased only under the loaded area. This showed clear punching shear failure behavior for the two FRP reinforced slabs. However, for the steel-reinforced slab (BS-0, Figures 4a and 5a) the deflection at the free edge increased up to failure which indicates that this slab had a ductile punching shear failure due to the yielding of most of the steel reinforcement. This gives us an evidence of the punching shear failure behavior for all tested FRP reinforced concrete bridge deck slabs.

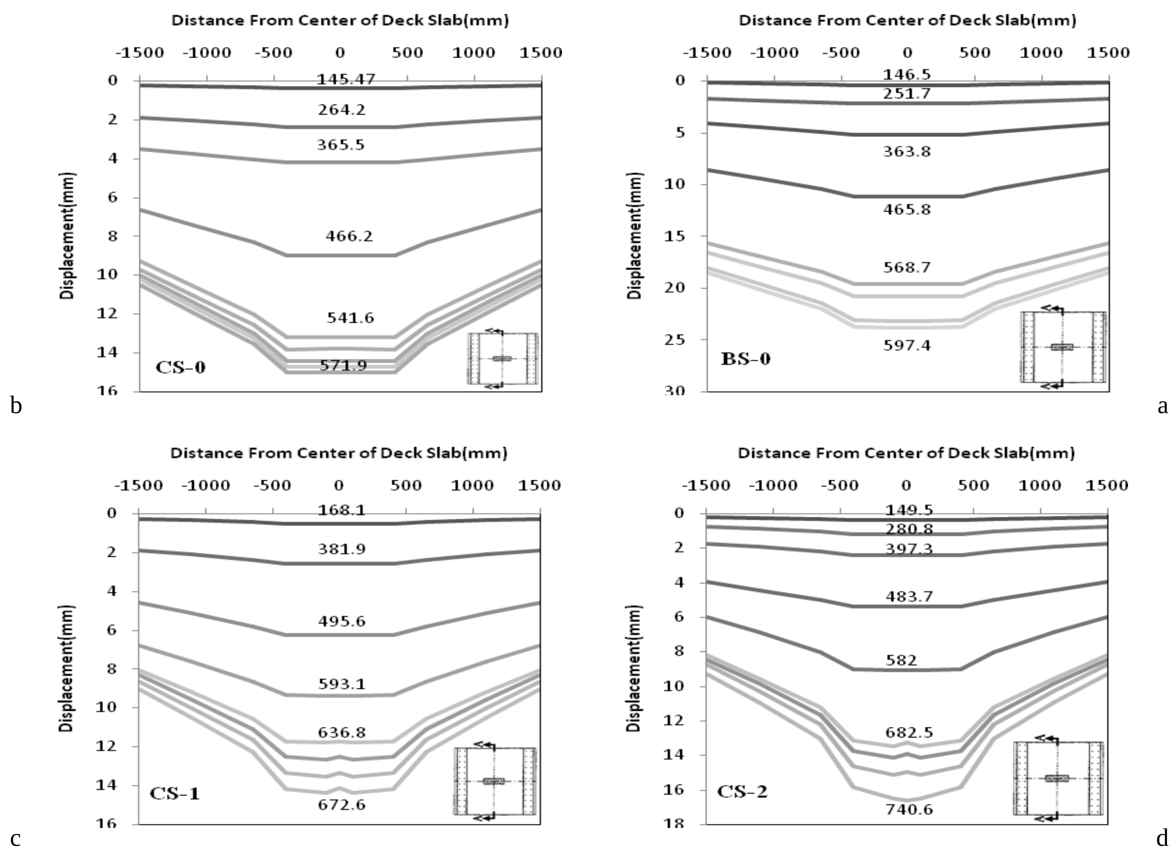

Figure 6 Deflection profile along of the longitudinal direction for slabs a: BS-0 b: CS-0 c: CS-1 and d: CS-2

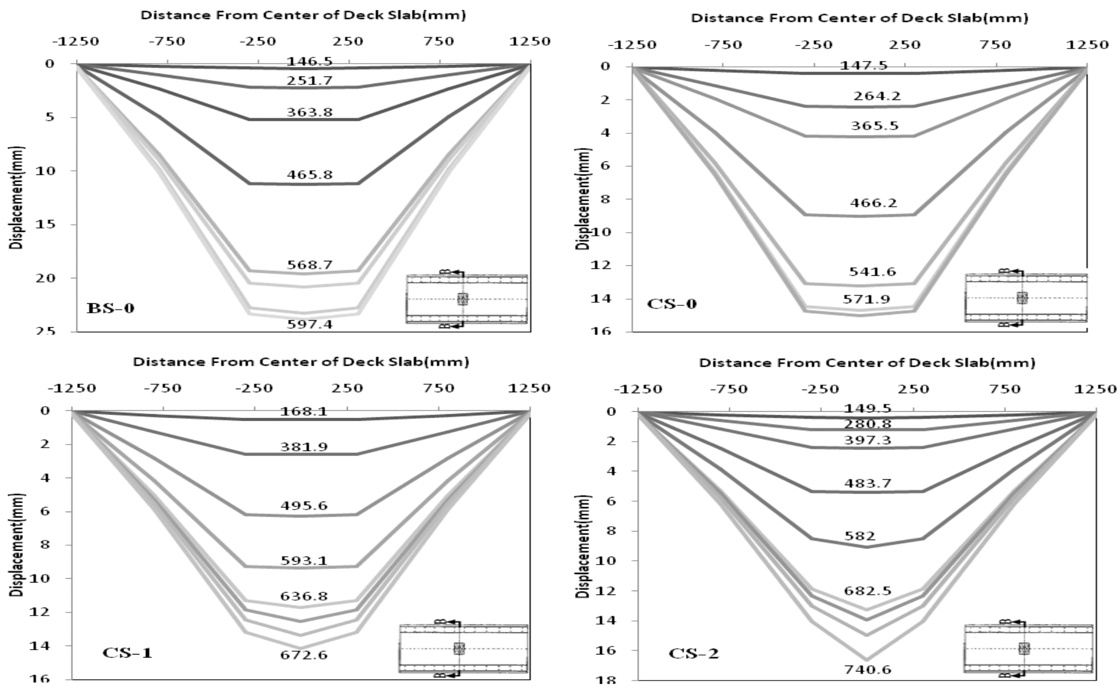

Figure 7 Deflection profile along of the transverse direction for slabs a: BS-0 b: CS-0 c: CS-1 and d: CS-2

3.3. Cracking Behavior and Ultimate Capacity

The cracking patterns on the tension surface of the four deck slabs were similar. For the four slabs, the first crack on the tension face occurred at loads higher than the service load (P_{ser} = 110.25 kN). The cracking loads were 146.5, 145.5, 168 and 169 kN for slabs BS-0, CS-0, CS-1 and CS-2, respectively. For the deck slabs, BS-0 and CS-0, the first cracks appeared directly under the loaded area and were oriented in the longitudinal direction parallel to the supporting beams. For the tow retrofitted slabs by using CFRP strips, CS-1 and CS-2, the first cracks formed near strips in unreinforced area parallel to the supporting beams.
Subsequent cracks propagated in the radial direction away from the loaded area. Before failure, roughly circular crack appeared and propagated to form the punching shear circle at the bottom face of the slabs. The final crack patterns at the top face of the four tested deck slabs were also similar. For the four deck slabs, the first crack on the top face occurred at high load levels. The first cracks appeared directly above the supporting beams and were oriented in the longitudinal direction parallel to the supporting beams then moved in circular direction to form incomplete circles. This indicates that the type of reinforcement and retrofit had no significant influence on the cracking behavior of the tested deck slabs.
The four tested deck slabs failed in punching shear around the loaded area. The top surface of the failure zone had an approximately elliptical shape passing through the comers of the loaded area. The bottom surface had an approximately circular shape with a diameter equal to the spacing between the top flanges of the two supporting girders. The failure loads were 597, 572, 673, and 741 kN for slabs BS-0, CS-0, CS-1 and CS-2, respectively. These observed values of carrying capacities are more than three times the factored design load of 208.25 kN specified by the CHBDC [6].
The difference between the ultimate capacities of the slabs CS-1 and CS-2 was more than 24%. This indicates that the retrofitted CFRP strips do have a major effect on the ultimate capacity of the three tested deck slabs.

4. CONCLUSIONS

Based on the results of this numerical investigation, the following observations and conclusions can be remarked.

1. The four tested deck slabs (BS-0, CS-0, CS-1, and CS-2) failed in punching shear failure at loads of 597, 572, 673, and 741 kN, respectively, which are more than three times the factored design load of 208.25 kN specified by the CHBDC.
2. For the four tested deck slabs, the difference in the ultimate capacities was considerable (more than 24%). This indicates that the CFRP strip (steel, GFRP, and CFRP; with approximately the same axial stiffness) has a noticeable effect on the ultimate capacity of concrete bridge deck slabs. Due to Ultimate capacity augmentation and easy installation this type off retrofitting can be used in emergency situation without take any time.
3. Deflection profile gives us an evidence of the punching shear failure behavior for all tested FRP reinforced concrete bridge deck slabs.
4. The results indicate that the type of reinforcement and retrofitted CFRP strip had no significant influence on the cracking behavior of the tested deck slabs.
5. That reinforcing and retrofitting the slabs by GFRP bars and CFRP strips did not affect the deflections at cracking load level; however it decreased the maximum recorded deflections at failure by about 35%.

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