

BEHAVIOUR OF BRIDGE DECKS REINFORCED BY GFRP

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SUMMARY: This paper describes the behaviour of a full-scale highway bridge deck model reinforced by glass fibre reinforced polymer (GFRP) reinforcements and tested under static loading conditions simulating the effect of a truck wheel load. Load-deflection behaviour, crack patterns, strain distribution and failure mode are reported. The measured values are compared to values calculated based on an analytical model developed using non-linear finite element analysis. The ultimate load carrying capacity is compared to a number of values based on international codes. The effective width and the angle of distribution of the wheel load distribution are evaluated. The effect of the reinforcement ratio and type of reinforcement on the behaviour of bridge decks are presented.

KEYWORDS: bridge, concrete, deck, deflection, GFRP, finite element, shear, slab

INTRODUCTION

Deterioration of concrete structures subjected to aggressive environmental conditions is attributed to the corrosion of the steel reinforcement. A full-scale model of three continuous bridge deck slab with double cantilevers reinforced with GFRP reinforcement was tested to failure at the University of Manitoba, Winnipeg, Manitoba, Canada. The model was built according to the reinforcement details of the Crowchild Trail Bridge constructed in 1997 in Calgary, Alberta, Canada. Each span, as well as the two cantilevers, were tested independently using a single concentrated load acting on a contact area equivalent to the area of a tire as specified by AASHTO HSS 25 (MSS 22.5) vehicle design.

A non-linear finite element model was developed to predict the behaviour of the deck slab under varying parameters. The accuracy of the analytical model was verified by comparing the predicted behaviour to two independent studies [1,2]. Based on the confidence established from the analytical model, the analysis was extended to investigate the effect of the reinforcement ratio and the type of reinforcement on the serviceability and the ultimate load carrying capacity of bridge deck slabs. The behaviour prior to cracking and after cracking, the ultimate load carrying capacity, and the mode of failure are discussed. The

measured ultimate capacity is compared to values calculated based on available international codes. The focus of this paper is on the behaviour of the cantilever portion of the deck slab.

TEST MODEL

The dimensions of the model were 7.2 x 3.0 m. The 200 mm thick deck slab was supported by four precast prestressed concrete beams spaced at 1.80 m center to center. The cantilever portion at each end of the slab had a clear span of 0.725 m as shown in Fig. 1.

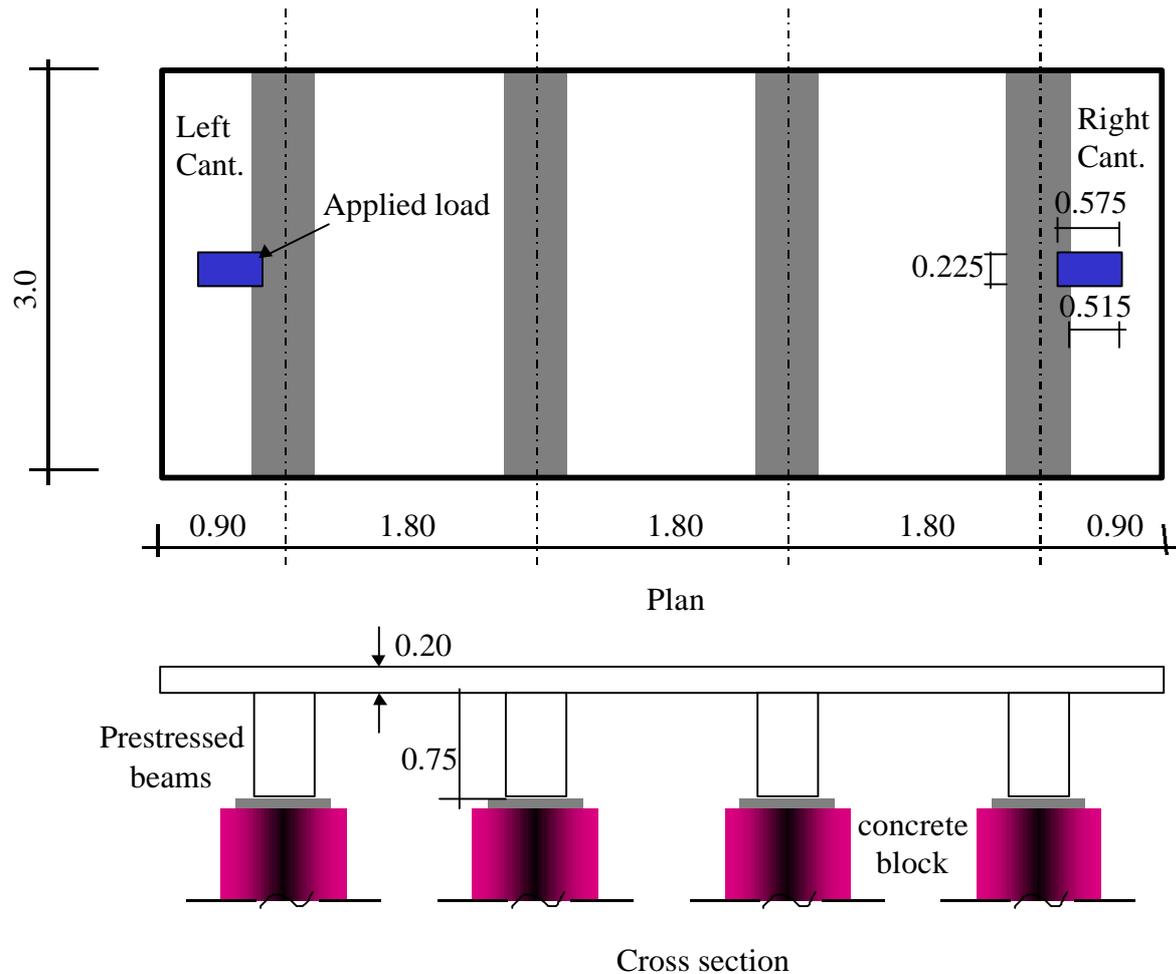


Fig.1 Schematic of the deck slab

The beams were simply supported on concrete blocks, anchored to the structural floor. Since corrosion of slab reinforcement typically starts after initiation of cracks at the tension surface of the concrete, followed by subsequent chloride ingress attacking the reinforcement, the negative moment reinforcement of the cantilevers are reinforced by GFRP bars. Two 15 diameter GFRP ISOROD bars, produced by Pultrall Inc., Quebec, Canada, were spaced at 250 mm providing a reinforcement ratio of 0.8 percent for the left cantilever. In addition, two 15 mm steel bars spaced at 150 mm were used as compression reinforcement, providing a reinforcement ratio of 0.67 percent. The tension reinforcement of the right cantilever consisted of two 15 mm diameter GFRP C-bars spaced at 250 mm, resulting in a reinforcement ratio of 0.8 percent. The C-bars were produced by Marshall Industries, Lima, Ohio, U.S.A. To study the effect of the compressive reinforcement, no tension reinforcement was used in the right cantilever. Construction of the full-scale model is shown in Fig. 2.

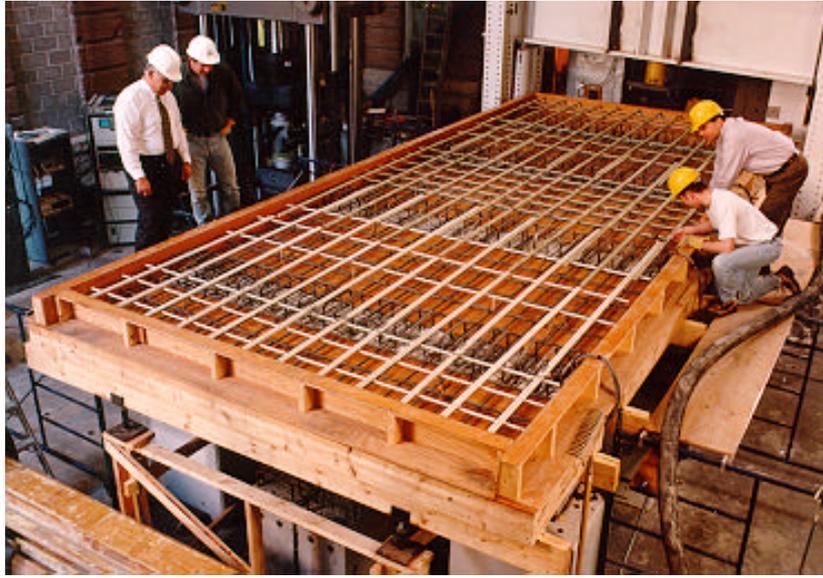


Fig.2 Slab reinforcement

Material Properties

GFRP ISOROD bars exhibit a linear tensile stress-strain behaviour with an ultimate tensile strength of 689 MPa and an elastic modulus of 41 GPa. GFRP C-bars have an ultimate tensile strength of 746 MPa and an elastic modulus of 42 GPa. The measured average compressive strength and the elastic modulus of the concrete used are 45 MPa and 30 GPa, respectively.

Testing Scheme

The cantilevers were loaded using a single concentrated load as shown in Fig. 1. The size and the location of the applied load were determined according to the AASHTO code, 1996 [3]. The load was applied through a 225 x 575 mm steel plate at a critical location that represents the closest point for a truck tire travelling near a barrier wall on the bridge. A neoprene pad was placed between the steel plate and the slab surface to simulate the tires of the truck and to avoid local crushing of the concrete surface. The load was cycled up to load levels of 200 and 400 kN to allow stabilization of the cracks at these intermediate loading stages. The instrumentation used to monitor the behaviour of the cantilever slab consisted of a combination of electrical strain gauges, PI gauges, dial gauges, demec points and linear variable differential transducers (LVDTs). The PI gauges located on the top surface of the slab were used to measure the strain and, consequently, the crack width. LVDTs and dial gauges were used to measure the deflection of the slab and of the supporting girder. Demec points and mechanical gauges were used to measure the strain of the concrete surface. One LVDT was attached to the end of the slab, parallel to the direction of the reinforcement, to measure the slip of an exposed GFRP bar located directly below the load.

Test Results

Deflection of the left cantilever was very small, less than the accuracy of the instrumentation up to an equivalent service load of 117 kN. The circumferential cracks developed at the top surface of the concrete at a load level of 330 kN caused considerable reduction of the overall stiffness, as shown in Fig. 3. The cantilever sustained a maximum load of 875 kN and failed due to crushing of concrete and extensive cracking, followed by punching shear.

The measured deflection in the right cantilever, at an equivalent service load of 117 kN, was 0.5 mm. It was also observed that the deflection values increased from 3 mm to 6 mm during the first three cycles of the 200 kN loading stage. A significant reduction in the stiffness of the cantilever occurred at a load level of 500 kN, as a result of extensive cracking of the concrete and continuous slipping of the top reinforcement as shown in Fig. 4.

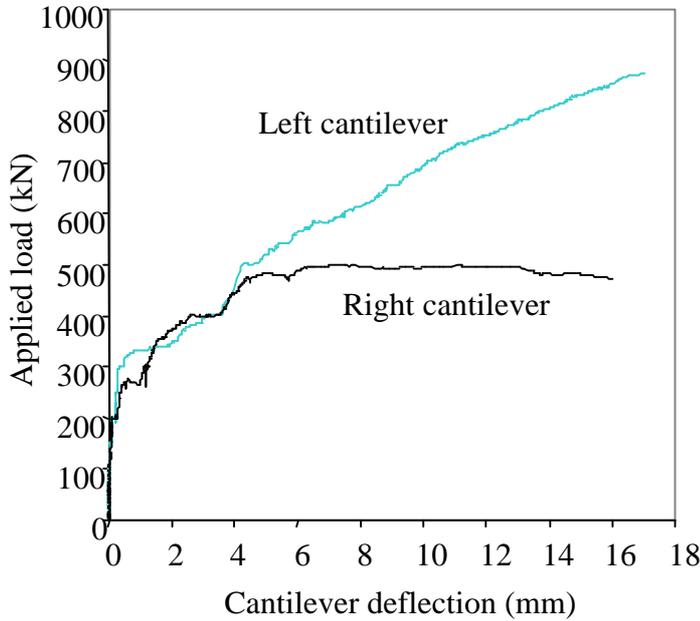


Fig.3 Load–deflection behaviour of the tested cantilevers

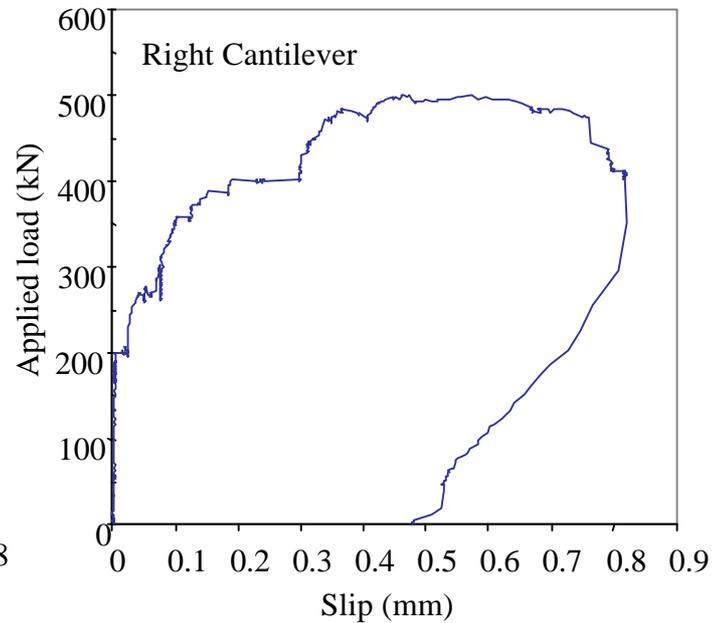


Fig.4 Load–slip behaviour of the right cantilever

Crack Pattern and Failure Mode

The first crack was observed at the top surface, within the negative bending moment zone in the adjacent slab at load values of 230 kN and 200 kN for the left and right cantilevers respectively. The left cantilever failed due to punching shear as shown in Fig. 5 at an ultimate load of 875 kN. The maximum recorded load for the right cantilever was 500 kN. Failure of the right cantilever was due to slippage of the top GFRP reinforcement. The slip at service load was approximately 0.003 mm which is significantly less than the limiting value of 0.064 mm recommended by Ehsani et al., 1996 [4].



Fig.5 Punching failure of the left cantilever

ANALYTICAL MODEL

The finite element program “Anatech Concrete Analysis Program” (ANACAP version 2.1, 1997) was used to predict the behaviour and the ultimate load carrying capacity of cantilever deck slabs. The non-linear finite element analysis accounts for biaxial and triaxial state of stresses as well as tension stiffening of the concrete. Cracking of the concrete is accounted for by using a smeared crack model for general 3-D stress states taking into consideration crack closure and re-opening under cyclic loading [5].

The capability of the finite element program to predict punching shear failure of slabs was verified by comparing the calculated behaviour to the test results of two different models. The model is a two-way square slab reinforced with NEFMAC grid type C-16 (elastic modulus of 98 GPa and ultimate strength of 1180 MPa, tested at the University of Ghent) [2] subjected to a concentric concentrated load. The second model consisted of a full-scale bridge deck reinforced with carbon fibre reinforced polymer (CFRP) with an elastic modulus of 147 GPa and an ultimate strength of 2250 MPa [1]. The predicted load-deflection behaviour of the first and the second models compared to the measured values is shown in Fig. 6 and 7, respectively. Results of the analysis suggest that the behaviour can be predicted with very good accuracy using the ANACAP software. In both cases, failure was due to crushing of the concrete, which led to punching shear failure. The experimental values were only over-estimated by 1 and 4 percent for the two models, respectively.

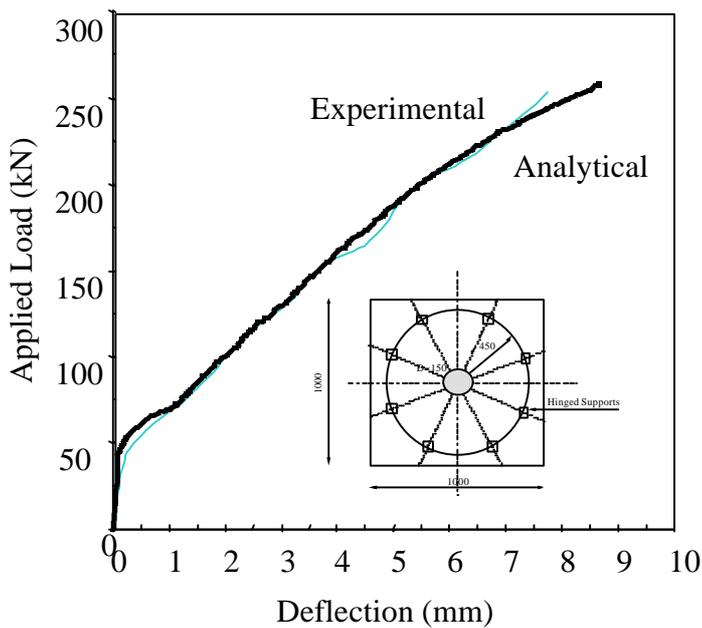


Fig.6 Load—deflection behaviour of the first model

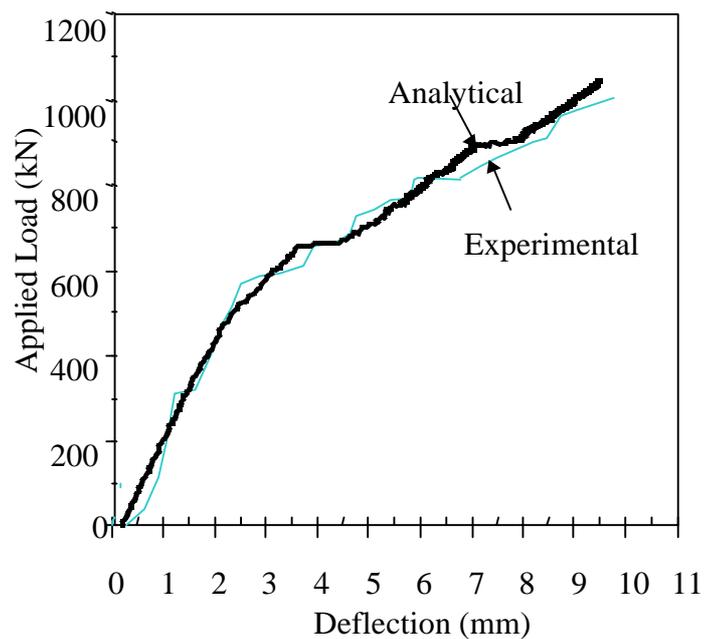


Fig.7 Load—deflection behaviour of the second model

Overall Behaviour

Based on the confidence established in the analytical model, the cantilever portion and its adjacent slab were modelled to account for the continuity effect. Due to symmetry, only one half of the slab was modelled using 20-node brick elements. The thickness of the slab was divided into three equal layers. The loading sequence of the analysis accounted for various loading stages, including the stage where the prestressed supporting beams were first installed to carry their own weight, followed by the application of fresh concrete during slab

casting. The predicted load-deflection behaviour of both left and right cantilevers compared to the experimental values is shown in Fig. 8 and 9, respectively.

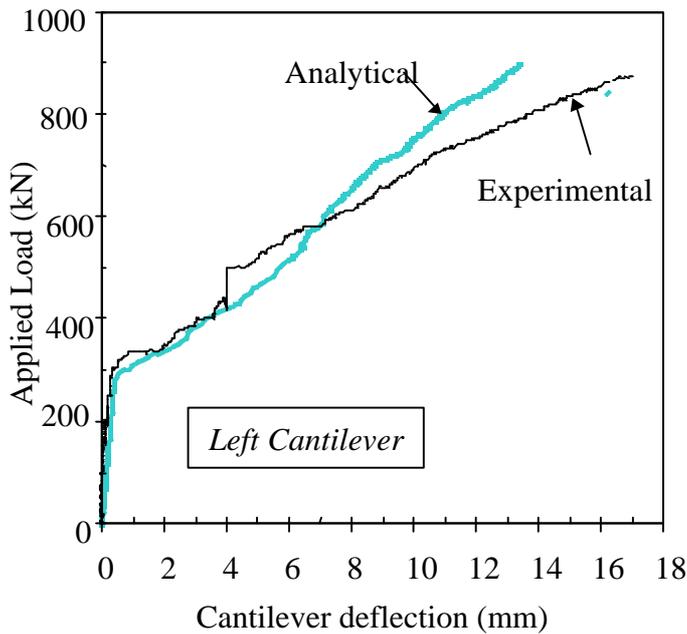


Fig. 8 Load—deflection behaviour of the left cantilever

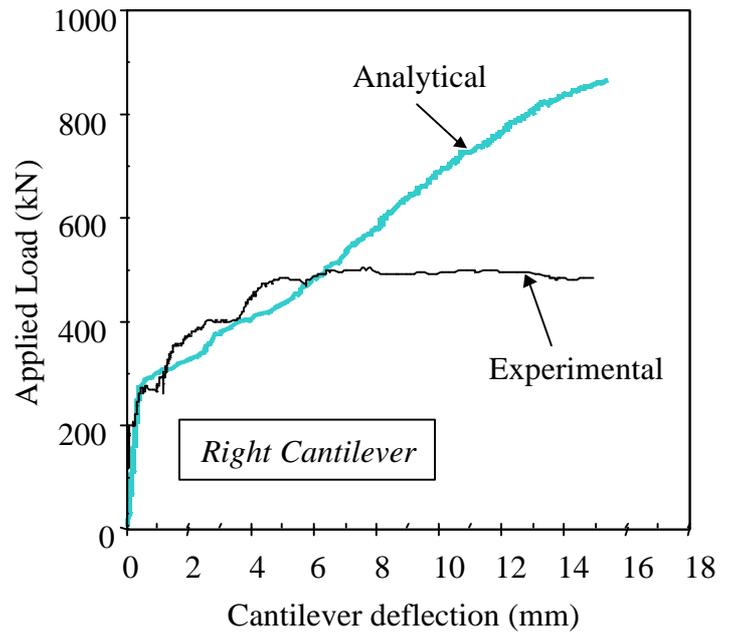


Fig. 9 Load—deflection behaviour of the right cantilever

For the left cantilever, the predicted values are in good agreement with the experimental results up to a load of 500 kN. At higher loads, the measured deflections were slightly higher than the predicted values. This may be attributed to local crushing at the interface of the slab and the supporting beam observed at the bottom surface of the cantilever during loading. Consequently, the neutral axis depth was reduced and the deflection values were increased. The failure was due to crushing of the concrete, which led to punching failure. The predicted failure load was 921 kN, which is 5 percent greater than the measured value.

For the right cantilever, the predicted deflection values were in good agreement with the measured values up to a load level of 500 kN. Under increased load levels, the measured deflection was significantly higher than the predicted values due to the large slippage that occurred in the top reinforcement and led to failure. The analysis predicted a failure load of 857 kN due to punching shear similar to that of the left cantilever. The finite element model did not capture the failure due to slippage since it assumes complete bond between the concrete and the reinforcement.

PARAMETRIC STUDY

Compression Reinforcement

Based on the analytical model, the influence of the compression reinforcement on the behaviour of the cantilever bridge deck is investigated using three different cases. In all cases, the top GFRP reinforcement ratio was set to 0.8 percent. The first two cases are the left and right cantilevers reinforced with 0.67 percent compression steel reinforcements and without compression reinforcements, respectively. The third case assumes a compression reinforcement consisting of 15.9 mm diameter GFRP bars spaced at 150 mm in the main direction, which represents an equivalent compression reinforcement ratio of 0.67 percent.

The load-deflection behaviour of the three cases is shown in Fig. 10. Results of the analysis indicate that the presence of the bottom compression steel reinforcement decreased the deflections by about 20 percent in comparison to the case without bottom compression reinforcement. Using GFRP as bottom compression reinforcement decreased the deflection values by 13 percent in comparison to the case without bottom compression reinforcements.

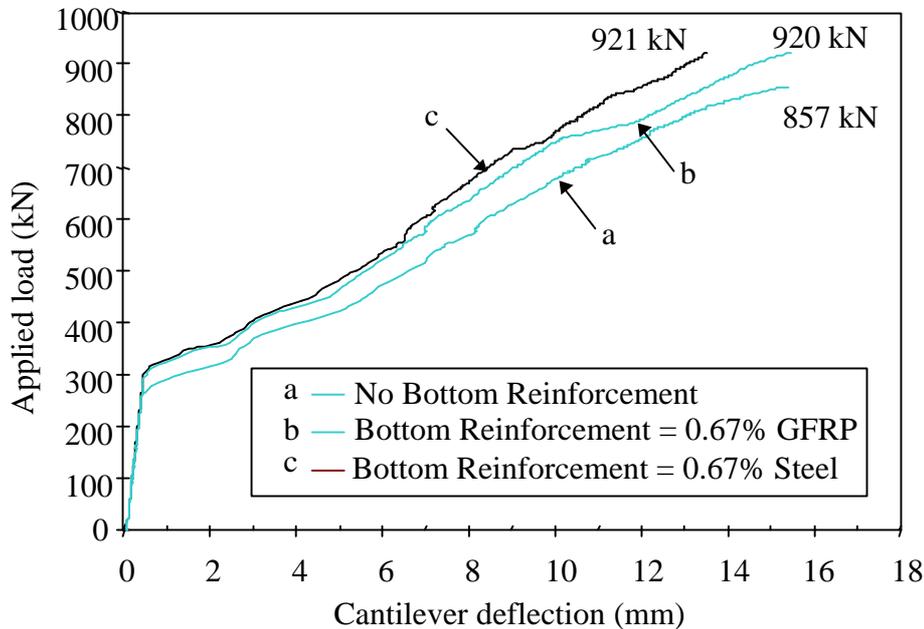


Fig. 10 Load—deflection behaviour using different types of bottom reinforcement

In all cases, failure occurred due to crushing of the concrete, which led to punching shear failure. Results of the analysis indicate that the presence of steel reinforcement increases the punching shear carrying capacity by 8 percent. Using GFRP as bottom compression reinforcement did not increase the failure load.

Tension Reinforcement

The effect of GFRP as top tension reinforcement on the behaviour of the cantilever bridge deck is studied using three different reinforcement ratios. In all cases, the bottom compression steel reinforcement was kept constant. The first case used a reinforcement ratio of 0.6 percent corresponding to the minimum recommended reinforcement ratio of GFRP that can be used for bridge decks [2]. The second case consisted of a tension top reinforcement ratio of 0.8 percent similar to the level used in the tests conducted at the University of Manitoba in 1998 and used in the Crowchild Bridge, built in Calgary, Alberta, Canada. The third case has a reinforcement ratio of 1.46 percent which is equivalent to a steel ratio of 0.3 percent and accounts for the ratio of the elastic modulus of steel to that of GFRP.

The predicted load-deflection relationships of the three cases are shown in Fig. 11. The behaviour of the cantilever slab reinforced with 0.3 percent tension steel reinforcement is also shown in Fig. 11. The analysis indicated that increasing the top reinforcement ratio resulted in a considerable decrease in the deflection. The maximum tensile strain at the top GFRP bars at failure decreased from 0.0096 to 0.0055 as the top reinforcement ratio increased from 0.6 to 1.46 percent.

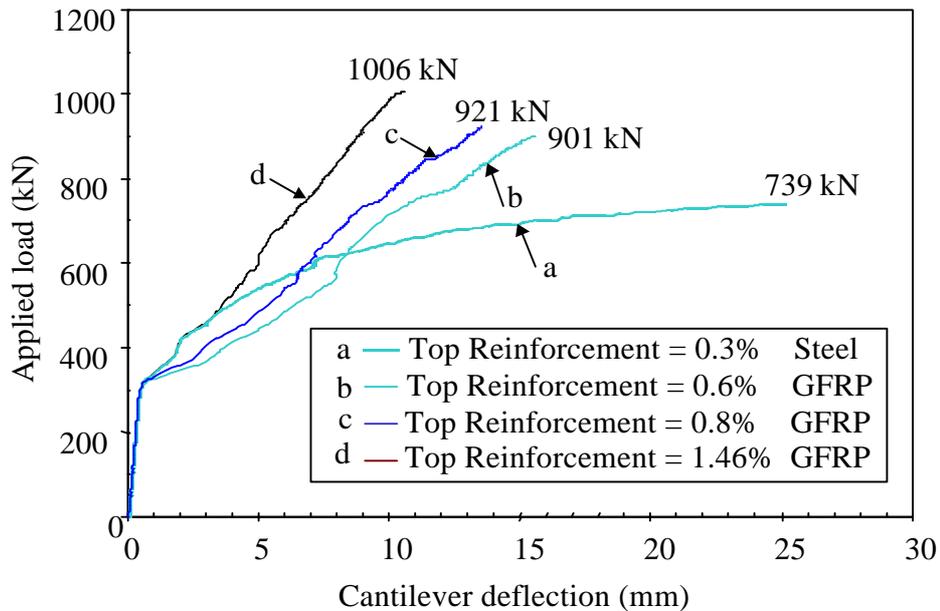


Fig. 11 Load–deflection behaviour using different reinforcement ratios

The behaviour of the case with a steel reinforcement ratio of 0.3 percent is identical to the case with GFRP reinforcement of 1.46 percent up to the load corresponding to yielding of the steel reinforcement. After yielding of the steel reinforcement, the deflection increased significantly with a small increase of the applied load, as shown in Fig. 11. The predicted failure load for the case of steel reinforcement was 739 kN due to punching shear in comparison to an ultimate load carrying capacity of 1006 kN for the cantilever reinforced with 1.46 percent GFRP bars. This significant decrease in the ultimate load carrying capacity is due to yielding of steel reinforcement which occurred at a load level of 450 kN.

The study indicates that increasing the percentage of GFRP reinforcement ratio enhanced the load carrying capacity. However, the increase in the load carrying capacity is not directly proportional to the reinforcement ratio as evidenced by the increase of 2 percent only of the ultimate capacity corresponding to an increase by 33 percent of the GFRP reinforcement ratio.

Effective Loading Width

The tensile strains in the top reinforcing bars along the length of the slab at different load levels and for both the left and right cantilevers were calculated. The effective loading width is determined using an equivalent rectangle area to match the area representing the strain distribution along the length of the slab. The same procedure is adopted for each load increment. The analysis and the experimental results indicate an average effective width of 2 m and an angle of load dispersion of 60° for the load configuration used in the test model. The predicted effective width according to OHBDC [8] and AASHTO code was 1.33 and 1.32 using an angle of dispersion of 47° . As expected, this study indicates that the codes are conservative.

Code Prediction

The punching shear capacity of cantilever bridge decks with three different bottom reinforcement configurations is predicted by the ACI code [6], CSA code [7], AASHTO code [3], OHBDC [8], Norwegian Standards (NS3473E) [9], European Standards (CEN) [10] and Japanese code [11]. All of the previous design codes except the NS3473E code and CEN code assume that the plane of failure due to punching shear is at $0.5d$ from the boundary of

the concentrated load. NS3473E code and CEN code assume that the plane of failure is at d and $1.5d$ respectively, where d is the depth of the deck slab. Fig. 12 shows the punching shear capacity as predicted by the different codes which compares well with the experimental results and the values obtained from the non-linear finite element model. However, none of these codes accounts for the enhancement in strength due to the presence or the type of the bottom compression reinforcement.

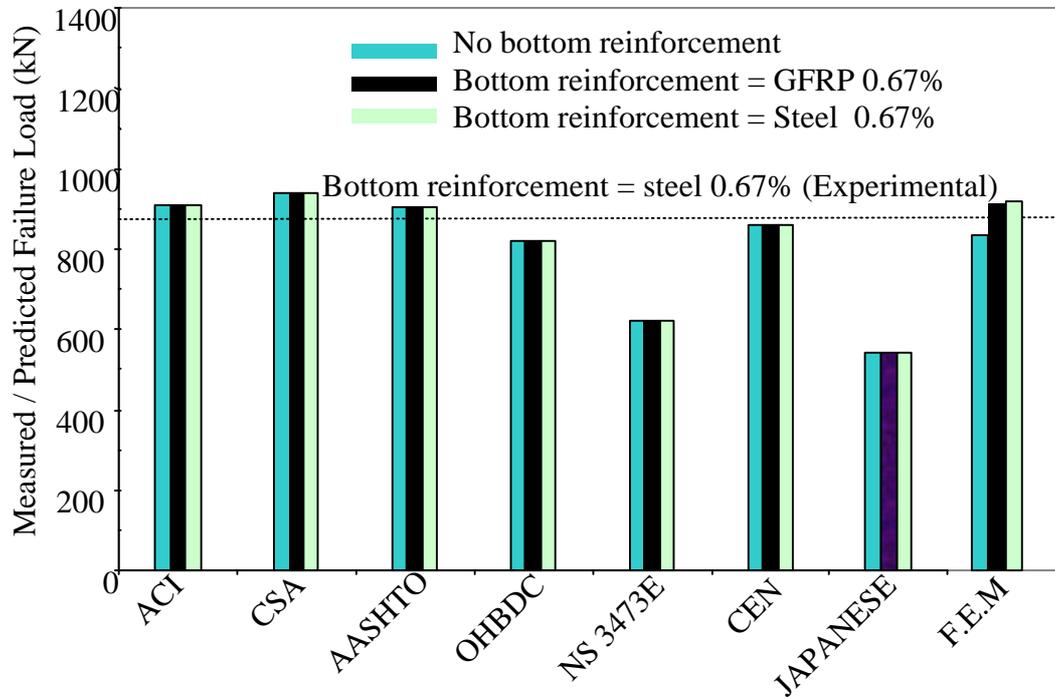


Fig. 12 Code prediction of the punching shear capacity of cantilever deck slabs

CONCLUSIONS

Based on this investigation, the following conclusions can be drawn:

1. The punching failure load of a cantilever bridge deck slab reinforced with tension GFRP reinforcement ratio of 0.8 percent used for the bridge in Calgary, Alberta, Canada, achieved an ultimate load carrying capacity equivalent to seven times the service load specified by AASHTO code.
2. At service load, the stresses in the main reinforcement for the left and the right cantilevers were equivalent to 3 and 12 percent of the ultimate strength of the GFRP bars, respectively. Consequently, the alkalinity in normal concrete should not be considered a threat to the use of GFRP reinforcement for bridge deck slabs.
3. Punching shear capacity of cantilever bridge deck slabs can be predicted with an accuracy of ± 6 percent using non-linear finite element analysis.
4. Use of bottom compression reinforcement in cantilever bridge decks increases the punching shear capacity by an average of 10 percent. It also decreases the deflection and the tensile strains at failure.
5. The ratio of main tension reinforcement has a significant effect on the punching shear capacity of the deck slab.

6. For cantilever bridge deck slabs, the effective width of distribution of the wheel load recommended by OHBDC, 1991 and AASHTO, 1996 design codes is conservative compared to the experimental values and the values predicted by the non-linear finite element analysis. A load dispersion angle of 60 degrees is recommended.
7. The values of the punching shear capacity of cantilever bridge deck slabs predicted by various design codes compared well with both the experimental results and the non-linear finite element model.

REFERENCES

1. Abdelrahman, A., Hassan, T., Tadros, G., and Rizkalla, S., (1997), "Behaviour of Concrete Bridge Deck Model Reinforced by Carbon FRP", *Proceedings of the Canadian Society for Civil Engineering Annual Conference*, Halifax, Nova Scotia, Canada, Vol.IIIb, pp. 521-526
2. Hassan, T., Abdelrahman, A. and Rizkalla, S., (1999), "Design Recommendations for Bridge Decks Reinforced by FRP" *accepted for publication in the Fourth International Symposium on Fibre-Reinforced Polymers (FRP) for Reinforced Concrete Structures (FRPRCS-4)*, Baltimore, Maryland, USA, November, 1999.
3. AASHTO, "Standard Specifications for Highway Bridges", (1996), *American Association of State Highway and Transportation Officials*, Washington D.C.
4. Ehsani, M. R., Saadatmanesh, H. and Tao, S., "Design Recommendations for Bond of GFRP Rebars to Concrete", *Journal of Structural Engineering*, V. 122, No. 3, March, 1996.
5. Gerstle, K. H., "Material Modelling of Reinforced Concrete", *IABSE Colloquium on Advanced Mechanics of Reinforced Concrete, Introductory Report*, Delft, The Netherlands, June 1981
6. American Concrete Institute, (ACI), 1995 "Building Code Requirements for Reinforced Concrete", (ACI 318M-95)" Detroit, Michigan, U.S.A.
7. Canadian Standards Association (CSA) 1994 "Design of Concrete Structures for Buildings", (CAN-A23.3 M-94), Rexdale, Ontario, Canada.
8. OHBDC, (1991), "Ontario Highway Bridge Design Code", Ministry of Transportation of Ontario, Downsview, Ontario, Canada.
9. NS3473E, "Concrete Structures, Design Rules", Norges Standardiseringsforbund, November, 1992.
10. CEN, "Comité Européen de Normalisation", (European Committee for Standardization, December, 1991).p
11. Japanese Society of Civil Engineers, (1997), "Recommendation for Design and Construction of Concrete Structures Using Continuous Fibre Reinforcing Materials", *Concrete Engineering Series 23*, Edited by A. Machida, Tokyo, Japan, 325p.