Second generation externally restrained bridge deck slabs

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ABSTRACT: About 30 years ago, the arching action in concrete bridge deck slabs of girder bridges was initially utilized in Ontario, Canada, by reducing significantly the amount of reinforcement in these slabs. The design method for such utilization of the arching action was adopted in the American AASHTO design specifications in 1996, and in the rest of Canada in 2000 through the Canadian Highway Bridge Design Code (CHBDC). Further utilization of the arching action led to steel-free, or corrosion-free deck slabs, which contain no tensile reinforcement, and are restrained transversely by means of steel straps connected to the top flanges of the supporting girders. The concrete of the steel-free deck slabs contained synthetic fibres of short lengths. While these fibers controlled cracks due to volumetric changes in concrete, they were not effective in arresting fatigue-induced cracks, with the result that all early steel-free deck slabs developed fairly wide longitudinal cracks roughly midway between the supporting girders. Fatigue tests on full-scale models have confirmed that the safety of the deck slabs is not compromised by the wide cracks. However, these cracks appear unsightly. The 2nd edition of the CHBDC (2006) requires that all these slabs, which are now called externally restrained deck slabs, be provided with orthogonal meshes of bars of glass fibre reinforced polymer (GFRP) for controlling fatigue-induced cracks. The earlier externally restrained deck slabs with crack-control reinforcement are now referred to as those of the 1st generation, and deck slabs with the crack control reinforcement as those of the 2nd generation. The paper provides a brief history of the utilization of the arching action in bridge deck slabs, and provides details of the 2nd generation of externally restrained deck slabs.

1 INTRODUCTION

1.1 *Arching in deck slabs with steel reinforcement*

Research conducted mainly in Canada during the past three or so decades has confirmed that all concrete deck slabs, made composite with the girders, develop an internal arching action. The Ontario Highway Bridge Design Code (OHBDC) first utilized this arching action in 1979 through an empirical design method, which required that for a girder spacing of up to 3.5 m, the thickness of the deck slab be at least the greater of 175 mm and 1/15th of girder spacing, and that the slab be provided with two orthogonal assemblies of steel reinforcement, with the reinforcement ratio in each direction in each assembly being a minimum of 0.3%. In later editions of the OHBDC, the minimum thickness of the deck slab was increased to 225 mm.

For a typical 225 mm thick deck slab, the empirical method leads to 15 mm dia. steel bars at a spacing of 300 mm in each direction in each of the top and bottom assemblies. The OHBDC empirical method was adopted by the AASHTO Specifications in 1996, and by the Canadian Highway Bridge Design Code (CHBDC) in 2000, the latter permitting the lower slab thickness to girder-spacing ratio of 1/18, and also a lower reinforcement ratio of 0.2%. The lower reinforcement ratio is permitted only when the authority having jurisdiction over the bridge is satisfied that the deck slab with the smaller amount of reinforcement can be constructed satisfactorily. The clear distance between the bottom bars of the upper assembly and top bars of the lower assembly is required to be a minimum of 55 mm. In corrosive environments, the required large depths of cover demand the deck slab to have a minimum thickness of 225 mm.

1.2 *Arching in deck slabs without tensile reinforcement*

The CHBDC (2000) also permits the design of a deck slab without any tensile reinforcement, provided that it is restrained externally. In the 2nd edition of the CHBDC (2006), these slabs are referred to as externally restrained deck slabs. Elsewhere, these deck slabs are known as steel-free or corrosion-free deck slabs. In an externally restrained deck slab, which can have a thickness of 175 mm in even corrosive environments, the arching action is harnessed mainly by external transverse restraint, provided by straps, which are straps connected to the top flanges of the girders. The CHBDC design provisions require that a strap should have a minimum cross-sectional area *A*, in mm^2, given by the following equation.

$$A = (F_s \times S^2 \times S_l)/(E \times t) \quad (1)$$

where F_s = 6.0 and 5.0 MPa for the external and internal transverse panels of the slab, respectively; S is the girder spacing in mm; S_l is the strap spacing in mm; E is the modulus of elasticity of the strap material in MPa; and t is the slab thickness in mm. Of special consideration is the fact that Equation 1 relates to the axial rigidity of the strap and not to its strength.

1.3 *Role of reinforcement in arching action*

Four tests on a full-scale model of a reinforced concrete (RC) deck slab containing four different patterns of reinforcement are reported by Khanna et al. (2000). As shown in Figure 1, Segment A of the model was reinforced with two orthogonal assemblies of 15 mm dia. steel bars at a spacing of 300 mm in each direction. Segment B contained only the bottom assembly of 15 mm dia. steel bars at a spacing of 300 mm. Segment C was provided with only 15 mm dia. bottom transverse steel bars at a spacing of 300 mm. Segment D contained 25 mm dia. GFRP bottom transverse bars at a spacing of 150 mm; these bars were selected so that their axial stiffness was the same as that of the steel bars in Segment C.

When tested under a central patch load, Segments A, B, C and D of the 175 mm thick model deck slab failed in punching shear at 808, 792, 882 and 756 kN, respectively. Despite the fact that the axial strength of the GFRP bars in Segment D was about 8.6 times the strength of the bottom transverse steel bars in Segment C, the failure loads of the two segments were similar. This observation confirmed that the transverse bottom reinforcement of a deck slab serves the same function as the straps in externally restrained deck slabs. The tests also confirmed that the axial stiffness of the bottom transverse reinforcement – and not its axial strength – governs the load carrying capacity of the slab. The top assembly of bars and the bottom longitudinal bars were found to have no influence on the strength of the slab. As discussed later, these bars control fatigue-induced cracks in the slab. Benmokrane et al. (2004) have also observed that the live load strains in top transverse bars in deck slabs are very small as compared to the strains in bottom transverse bars.

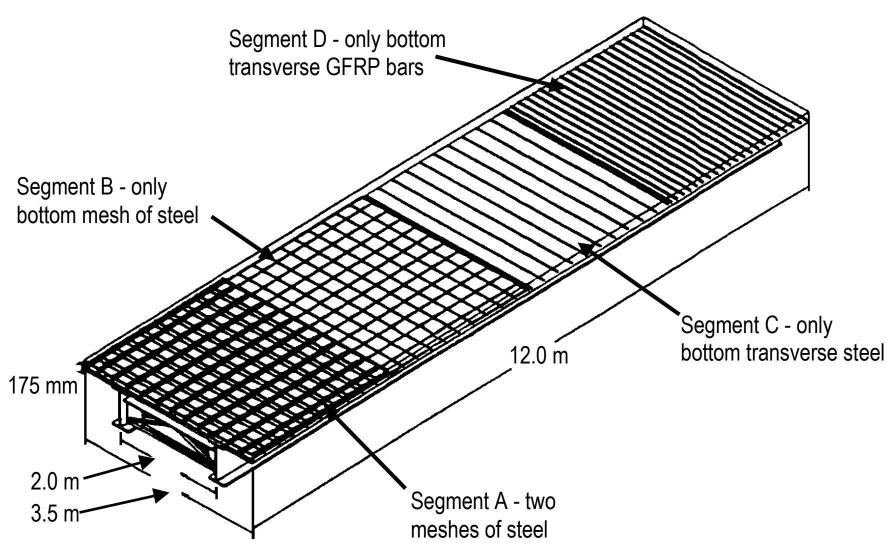

Figure 1. Details of RC deck slab with four segments.

Taking a lead from the above observations, the CHBDC (2006) extends the use of its empirical method to concrete deck slabs containing FRP reinforcement in two orthogonal assemblies. For these slabs, the bottom transverse FRP bars are required to have the same axial stiffness as the stiffness of corresponding steel bars required by the empirical method. All the remaining FRP reinforcement should have at least the same strengths as those required for the corresponding steel bars. A major advantage of using FRP reinforcement in deck slabs in corrosive environments is that the thickness of the slab can be as little as 175 mm.

2 1st GENERATION EXTERNALLY RESTRAINED DECK SLABS

2.1 *Cracking*

Six months after it was opened to traffic in December 1995, the soffit of the world's first externally restrained deck slab developed about 1 mm wide longitudinal cracks, roughly midway between the girders (Mufti et al. 1999). Other steel-free deck slabs without crack control reinforcement (Bakht and Mufti 1998) also developed full-depth cracks soon after being opened to traffic. Periodical inspection showed that the widths and pattern of these cracks have not changed significantly over time. While experimental studies have confirmed that the presence of even 1 mm wide cracks does not affect the safety of the slabs (Limaye et al. 2002), many engineers are not comfortable with wide cracks, which also appear unsightly and may give a cause for concern to the public. Following the JSCE (1997) recommendations, which are based entirely on aesthetics, it was suggested that for future externally restrained deck slabs and deck slabs with only FRP reinforcement, crack widths be limited to 0.5 mm.

Recent and current experimental studies undertaken at Dalhousie university and the universities of Manitoba and British Columbia (Limaye et al. 2002, Mufti et al. 2002, and Memon 2005) have already confirmed that (a) full-depth cracks develop in steel-free deck slabs after only a few passes of relatively light loads, (b) the widths of cracks in deck slabs with either steel or FRP reinforcement grow with fatigue-induced damage, (c) the growth of crack widths is more rapid during the initial stages of fatigue damage, and (d) the widths of cracks in steel-free deck slabs can be controlled by providing an assembly of nominal FRP reinforcement.

One of the studies discussed above involved the fatigue testing of three 175 mm thick deck slabs, each on two girders spaced at 2 m. Each slab had different crack control and transverse confining systems. The first slab contained two orthogonal assemblies of 15 mm dia. steel reinforcing bars at a spacing of 300 mm in each direction. The second slab was transversely restrained with external steel straps, and contained one orthogonal crack control assembly of 10 mm dia. carbon FRP (CFRP) bars with the transverse bars at a spacing of 200 mm and the longitudinal bars at a spacing of 300 mm. The third slab was also restrained transversely by external steel straps, but contained an orthogonal crack control assembly of 13 mm dia. GFRP transverse and longitudinal bars at spacings of 150 and 250 mm, respectively. Both the crack control assemblies were placed near the bottom of the respective slab, each with a clear cover of 40 mm.

Mufti et al. (2002) have determined that during a lifetime of 75 years, a highway bridge deck slab in North America can experience a maximum of 372 million passes of wheels, ranging in magnitudes from 1 to 16 t. Memon (2005) has presented an equation, which shows that the damage induced by all these wheel passes can be replicated in the laboratory by 173,800 cycles of a 25 t load, or by only 24 cycles of a 50 t load.

Each of the three slabs described above was subjected to successive one million cycles each of 25 t and 50 t loads. After 200,000 cycles of the 25 t load, the maximum crack widths in deck slabs with steel, CFRP and GFRP bars were 0.32, 0.38 and 0.31 mm, respectively. The study confirmed that the maximum crack widths in all the three tested slabs, after they were subjected to the lifetime damage, were well within the suggested limit of 0.5 mm. The crack widths in all the three slabs after one million cycles of the 25 t load increased to nearly 0.4 mm, indicating that the amount of crack control assemblies of FRP bars provided in the tested externally restrained deck slabs were significantly more than required to keep the crack widths within 0.5 mm.

2.2 *Safety of 1st generation externally restrained deck slabs*

The longitudinal cracks in externally confined deck slabs of the 1st generation extend the full depth of the slab. Some engineers were concerned that the shear forces due to wheel loads on one side of these cracks would lead to the failure of the slab. Limaye (2004) performed a test by placing a 40 t (~400 kN) pulsating load on one side of a full-depth longitudinal crack in an externally restrained deck slab without any crack control reinforcement. Even after 1700 cycles of the load, the 175 mm thick slab, supported on girders at a spacing of 2.0 m, showed no sign of distress; the test had to be abandoned prematurely because of time constraints on the use of the lab. As discussed by Memon et al. (2003), 6115 passes of a 40 t wheel load cause nearly the same damage as that induced by all wheel loads on a very busy bridge in 75 years. The test by Limaye (2004) removed any concern about the safety of externally restrained deck slabs without crack control reinforcement.

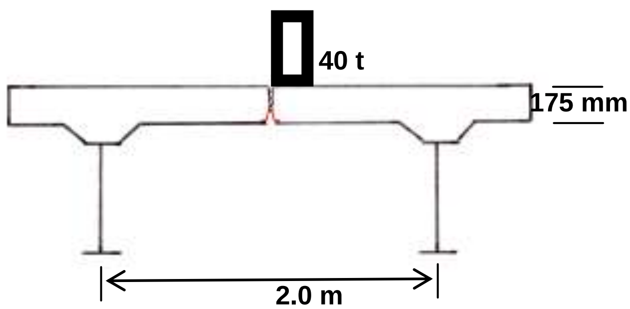

Figure 2. Cross-section of an externally restrained deck slab without crack control reinforcement.

Newhook and Gaudet (2006) have chronicled the performance of the externally restrained deck slab of the Salmon River Bridge over the last 10 years. Before the construction of the deck slab, Newhook (1997) had performed static load tests on a full-scale model of the same bridge. The crack widths were not measured during these tests. However, it was found that the strain in the transverse strap under the deck slab just before failure was nearly 1500 micro-strains. The strap strain, caused by the relative outward movement of the girders, could be accommodated by the un-reinforced slab only through the crack width. Multiplying the ultimate strap strain by the girder spacing of 2700 mm, the relative outward movement of the girders is found to be nearly 4 mm. Thus it is concluded that the crack width in the deck slab model of the Salmon River Bridge just before failure under the static, i.e. monotonically increasing, load is nearly 4 mm. So the deck slab of the Salmon River Bridge is expected to reach the end its life when widths of its longitudinal cracks approach 4 mm.

During the fatigue tests on externally restrained slabs conducted by Memon (2005), it was found that the fatigue damage in a deck slab could be quantified by both crack widths and permanent deflections of the slab; this finding is also reported by Memon et al. (2003). A corollary to this finding is that the strap strains in externally restrained deck slabs under repeated loads follow the same pattern as the crack width.

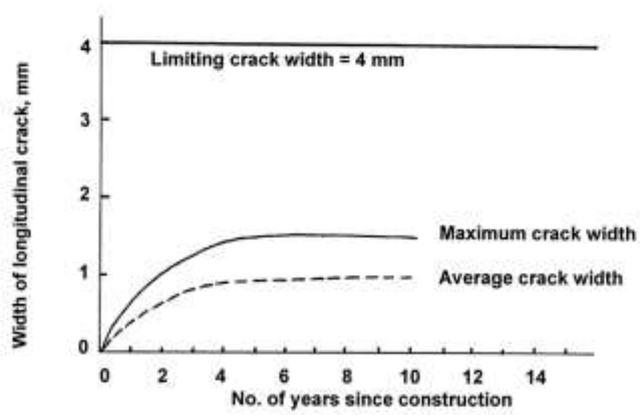

Figure 3. Widths of longitudinal cracks in externally restrained deck slab of the Salmon River Bridge plotted against time.

The widths of longitudinal cracks in the externally restrained deck slab of the Salmon River Bridge, reported by Newhook and Gaudet (2006), are plotted in Figure 3 against the age of the slab. The figure also shows the limiting crack width of 4 mm, at which crack width the deck slab is expected to fail in fatigue. It can be seen in Figure 3 that the widths of longitudinal cracks in the Salmon River Bridge have already stabilized, and that it will take much more than 100 years before the widths of these cracks approach the limiting crack width.

3 2nd GENERATION EXTERNALLY RESTRAINED DECK SLABS

The 2nd generation externally deck slab is similar to its 1st generation counterpart, but contains a grid of GFRP bars, and is free of wide longitudinal cracks. Researchers at the University of Manitoba have concluded that a bottom mat of GFRP reinforcement with a reinforcement ratio of 0.25 percent is enough to control the crack widths (Mufti and Memon, 2003).

The first second-generation externally restrained deck slab was cast in July, 2003 on one span of the multi-span Red River Bridge on the North Perimeter

Highway in Winnipeg, Manitoba; this demonstration project was a joint effort between ISIS Canada, Earth Tech (Canada) Inc., JMBT Structures Research Inc. and the Province of Manitoba.

The Red River Bridge was originally constructed in 1964 to a design loading of HS20 in accordance with the AASHTO Specifications for Highway Bridges. This ten-span bridge is 347 m long and consisted of steel plate girders, spaced at 1.8 m, and a composite, cast-in-place, steel reinforced concrete deck slab. The original concrete deck slab began to exhibit signs of deterioration in the early 1980s, and in 1985, the asphalt riding surface was replaced after significant deck patching and the installation of a waterproofing membrane. Further deterioration of the deck slab in the 1990s led to major rehabilitation of this bridge. The bridge was strengthened for current allowable loading, and the entire concrete deck slab was replaced to meet current safety requirements.

Nine spans of the bridge now contain a 225 mm thick deck slab with conventional steel reinforcement. One simply supported span contains a 2nd generation externally restrained deck slab with a thickness of 200 mm. GFRP bars reinforcement was used for both the top and bottom mats in the internal deck panels. Bundles of two 12 mm dia. carbon fibre reinforced polymer (CFRP) bars at a spacing of 200 mm were provided for transverse negative moments due to loads on the cantilever overhangs (Figure 4).

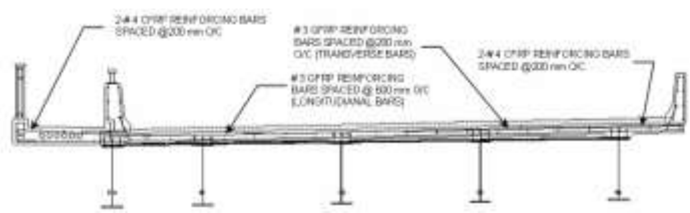

Figure 4. Cross-section of the Red River Bridge with 1st generation externally restrained deck slab.

The concrete traffic barriers in the Red River Bridge contain near inner faces 19 mm dia. and 500 mm long stainless steel bars, with a 50 mm dia. and 12 mm thick stainless steel disc welded at each end. These bars are spaced at 300 mm and have an embedment length of 175 mm into the concrete deck slab.

The existing shear studs welded to the tops of the girders were not long enough to provide full composite action between the girders and the deck slab. To compensate for this lack of length, galvanized steel haunch reinforcement was provided in the haunches; the haunch reinforcement can be seen in Figure 5**.**

Figure 5. Haunch reinforcement to compensate for inadequate length of shear studs.

After the Red River Bridge, the next major use of the 2nd generation externally restrained deck slab is on eight major bridges, currently being built to widen the floodway in the vicinity of Winnipeg, Manitoba. Three bridges with a total plan area of nearly 16,000 m^2 have already been built. The remaining five bridges, with a total plan area of nearly 26,000 m^2 are scheduled to be built in the next few years.

All externally restrained deck slabs of the 2nd generation are free of wide cracks.

The design of the deck slabs of the Red River Bridge and the Floodway bridges was done in accordance with the draft design provisions of the 2nd edition of the CHBDC, which was published in November, 2006. It is noted similar design provisions were published by American Concrete Institute (ACI) in November 2004. The CHBDC (2006) design provisions are described briefly in the following section.

4 DESIGN PROVISIONS

In order to be consistent with the empirical design provisions of reinforced concrete deck slabs in the concrete section of the code, the provisions of externally restrained deck slabs of the fibre reinforced structures section in the first edition have been reorganized to explicitly include deck slabs of both cast-in-place and precast construction.

The clause for the design of externally confined deck slabs is divided into four sub-clauses to cover design provisions: (a) of a general nature, (b) for full-depth cast-in-place deck slabs, (c) for cast-in-place deck slabs on stay-in-place formwork, and (c) for full-depth precast deck slabs.

A major change in the design provisions of externally restrained deck slabs is that the crack control reinforcement, which was optional in the 1st edition of the code, is now mandatory. The slab is required to be provided with a crack control orthogonal mesh of GFRP bars, placed near the bottom of the slab, with the area of cross section of GFRP bars being at least $0.0015t$ mm^2/mm, where t is the thickness of the deck slab in mm. In addition, the spacing of transverse and longitudinal crack control bars should not be more than 300 mm. The cross-sectional area and spacing of the specified crack control mesh are based on recent experimental fatigue studies referenced earlier.

In addition to the general requirements, externally confined deck slabs with cast-in-place construction on stay-in-place formwork are required to satisfy the following conditions:

(a) The formwork is designed by taking into account the handling and anticipated conditions during construction, with its effective span taken as the distance between the edges of the supporting beams plus 150 mm.

(b) The deflection of the formwork during construction does not exceed 1/240 of the effective span of the formwork.

(c) The ends of the formwork are supported on beams such that after placement of concrete topping, a support of at least 75 mm is provided under the lower portions of the formwork, and such support is within 25 mm of the closer edges of the supporting beams.

(d) The top flanges of all adjacent supporting beams are connected by means of either external straps or the formwork itself.

(e) When the deck slab is confined by straps, the straps and their connections are designed similarly to full-depth cast-in-place deck slabs.

(f) When the deck slab is restrained by a formwork, the concept has been verified by tests on full-scale models. In addition, the area of cross-section of the formwork, in mm^2/mm, across a section parallel to the supporting girders, is A_f given by:

$$A_f = \frac{F_s S^2}{Et}$$

where F_s is 6.0 MPa for outer panels and 5.0 MPa for inner panels, and E is the modulus of elasticity of the material of the formwork in the direction perpendicular to the supporting beams, in MPa.

(g) When the deck slab is restrained by formwork, the direct or indirect connection of the formwork to the supporting beams has been proven by full-scale tests to have shear strength in Newton/mm of at least $200A_f$.

(h) When the formwork is of precast concrete construction, it contains a crack control orthogonal grid of GFRP bars, placed at its mid-depth, with area of cross section of GFRP bars being at least $0.0015t$ mm^2/mm. In addition, the spacing of transverse and longitudinal crack control bars is not more than 300 mm.

(i) When it is of precast construction, the formwork panel has a maximum thickness of $0.5t$.

(j) When it is of precast construction, the upper surface of the formwork panel is clean and free of laitance and is roughened to an amplitude of 2 mm at a spacing of nearly 15 mm.

5 CONCLUSIONS

It has been shown that the 2nd generation externally restrained deck slab is a cost-effective solution to the problem of corrosion of conventional deck slab construction with steel reinforcement. The following summarizes the findings of the research conducted in developing the externally restrained deck slab:

a) The use of external straps leads to the highest static strength of the deck slab;

b) externally restrained deck slabs of the 2nd generation are free of wide cracks;

c) GFRP crack-control grid provides an economical solution to the problem of wide cracks; and

d) externally restrained deck slabs with GFRP bars for crack control have the best fatigue resistance and those with steel bars the worst.

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