Report on Bridge Decks Free of Steel Reinforcement

Reported by ACI Innovation Task Group 3

ACI encourages the development and appropriate use of new and emerging technologies through the publication of the ***Emerging Technology Series****. This series presents information and recommendations based on available test data, technical reports, limited experience with field applications, and the opinions of committee members. The presented information and recommendations, and their basis, may be less fully developed and tested than those for more mature technologies. This report identifies areas in which information is believed to be less fully developed, and describes research needs. The professional using this document should understand the limitations of this document and exercise judgment as to the appropriate application of this emerging technology.*

Joe Gutierrez
Chair

Gerald H. Anderson
Andrzej S. Nowak
Harold R. Sandberg
Steven L. Stroh

This document outlines procedures for the design of bridge decks free of steel reinforcement and requirements for design and installation of straps to restrain rotation of edge beams to achieve arching action in a deck slab.

Keywords: arching; bridge; composite action; corrosion; deck slab; fiber-reinforced concrete; reinforcement-free; transverse confinement; transverse constraint.

PREFACE

The concept for the design of a steel-free bridge deck slab described in this report is patented. Therefore, use of the information in this document may require payment of royalties to the owners of the patents. At the time of printing, the United States and the United Kingdom have granted a patent for the steel-free cast-in-place bridge deck slabs, with a patent pending in Canada. The steel-free precast slab is also patented in the United States, and the global patent is pending. Interested parties are invited to submit information regarding the identification of an alternative(s) to this patented item to ACI Headquarters. Your comments will receive careful consideration at a meeting of the responsible standards committee, which you may attend.

The American Concrete Institute takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this report. Users of this report are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

The inventor of the reinforcement-free bridge deck concept described in this report sponsored preparation of the report and provided reimbursement to the authors to assist in recovery of their costs and expenses related to travel to meetings; however, none of the authors received an honorarium.

ACI ITG-3-04 became effective September 10, 2004.

Conclusions from research conducted since the document was first written are included in Appendixes B and C.

CONTENTS

CHAPTER 1—INTRODUCTION

1.1—Purpose

The purpose of this document is to introduce a new technology to address corrosion-free concrete bridge deck slabs by eliminating the steel reinforcement. The concept for the design of a steel reinforcement-free bridge deck described in this document is patented. Therefore, authorization must be obtained from the owners of the patents before the information contained in this document can be used.*

Development of the steel-free bridge deck slab concept described in this report was prompted by the need for more durable bridge deck slabs. Traditionally, attempts have been made to alleviate the problem of corroding steel reinforcement by increasing the concrete cover, applying waterproofing membranes, or using epoxy-coated reinforcing steel, among other methods. While the use of these materials and methods has contributed to greater durability, the potential for deterioration due to corrosion of the reinforcing steel still exists. Eliminating the steel reinforcement in the deck slab can solve the corrosion problem.

This concept originated from recognizing that the conventional method for the design of bridge deck slabs is based on the flexural strength of the slab. The slab is assumed to be a transverse bending member supported by girders. This method leads to the need for a large amount of steel reinforcement in the deck slabs.

1.2—Scope and objectives

The scope of material presented in this document relates principally to delineating methodology for the design of reinforcement-free bridge decks. Other discussion includes guidance on special considerations for the construction and maintenance of this new type of bridge design. Appendix D provides research needs that require further investigation to facilitate continued understanding and improvement of this new emerging technology.

*For information about the patent, please contact Gamil Tadros at 43 Schiller Crescent, Calgary NW T3L 1W7, Canada, or by e-mail: gtadros@telus.net.

Publication of this document satisfies the objective of disseminating as early as possible new emerging technology to the concrete industry. This objective is satisfied through the sharing of this new knowledge and experience in implementing the new reinforcement-free bridge deck designs. This sharing of new information and experiences helps accelerate technology transfer from the research arena to practice. The discussion that follows in this section serves to provide background on the origin of the idea for the development of the reinforcement-free bridge deck.

Extensive field and laboratory testing, done mainly in Ontario, Canada, more than two decades ago, showed that the amount of reinforcement in deck slabs could be reduced substantially by taking into account the arching action that develops within the slab. The 1979 Ontario Highway Bridge Design Code (OHBDC) included an empirical design method that took the arching action into account. The AASHTO LRFD Bridge Design Specifications (1998) also include a similar method for the design of deck slabs (Reference Clause 9.7.2). Using large-scale laboratory testing, further research into the arching action of deck slabs was initiated at Dalhousie University in Nova Scotia, Canada. Researchers found that suitable harnessing of the arching action could lead to the removal of the entire tensile reinforcement. The key to removing the tensile reinforcement from the deck slab is the provision of adequate transverse restraint, which generates compressive membrane forces so that under a concentrated load, the slab fails in a localized punching shear mode rather than in flexure. The terms "transverse constraint" and "transverse confinement" have been used interchangeably in this document.

Over the past 12 years, a large number of large-scale models of steel reinforcement-free deck slabs have been tested, mainly in Nova Scotia and Ontario. This research led to the construction, in December 1995, of the world's first steel-free cast-in-place deck slab over the Salmon River, which forms part of the Trans-Canada Highway in Nova Scotia. Since then, four other Canadian bridges now incorporate concrete bridge deck slabs free of steel reinforcement. These are the Chatham Bridge in Chatham, Ontario; the Crowchild Trail Bridge in Calgary, Alberta; the Waterloo Creek Bridge in British Columbia; and the Lindquist Bridge in British Columbia, which is on a forestry road and incorporates a precast slab.

A reinforcement-free precast slab has also been used in the reconstruction of a wharf structure in Hall's Harbour, Nova Scotia. The research conducted on reinforcement-free deck slabs in Canada over the past 12 years is reported in the technical literature (Section 10.1) along with the histories of bridges incorporating the new concept. Chapter 10 is a listing of the published papers referenced in this document.

In addition to the work done in Canada, research on the reinforcement-free deck slab is also being conducted in the United States (Seible et al. 1998) and Japan (Matsui et al. 2001).

1.3—Further research needs

While the reinforcement-free bridge deck design leads to elimination of the problem of corrosion from steel reinforcement, it also raises several other concerns. Those concerns relate to the possibility that significant longitudinal cracking will be observed over time, and other types of cracks may lead to serviceability problems. Another area of concern is the potential for deterioration of the steel straps due to corrosion and the lack of their protection from a possible fire or impact. To address this issues, Appendices B, C, and D provide supplemental information and list areas of additional research that should be considered to answer the aforementioned concerns.

CHAPTER 2—DEFINITIONS AND ABBREVIATIONS

AFRP—aramid fiber-reinforced polymer.

CFRP—carbon fiber-reinforced polymer.

fibers—small-diameter filaments of materials of relatively high strength, which can be glass, carbon, aramid, or low-modulus polymer.

FRC—fiber-reinforced concrete; a fiber-reinforced composite in which the matrix is portland-cement concrete, and in which the fibers are discontinuous and uniformly and randomly distributed.

FRP—fiber-reinforced polymer; a fiber-reinforced composite with a polymeric matrix and continuous fiber reinforcement.

fiber volume fraction—the ratio of the volume of the fibers to the volume of the fiber-reinforced composite.

GFRP—glass fiber-reinforced polymer.

low-modulus fibers—fibers of thermoplastic polymer with a moduli of elasticity less than 1450 ksi (10 GPa), such as nylon, polyolefin, polypropylene, and vinylon.

matrix—the continuous material in a fiber-reinforced concrete or polymer component that contains aligned or randomly distributed fibers.

multispine bridge—a box-girder bridge in which the bottom flange is discontinuous in the transverse direction.

RC—reinforced concrete.

reinforcement—in this document, this term refers to bars that are added to concrete to enhance its tensile strength. Bars provided for only crack control are not referred to as reinforcement.

strap—a linear component of steel, FRP, or other material used to provide external transverse confinement in the reinforcement-free bridge deck slabs.

CHAPTER 3—DESIGN METHODOLOGY

The design methodology employing the concept described in this report relies mainly on transverse straps (Fig. 3.1 through 3.3), which may be made of steel, connected to the top flanges of adjacent girders for preventing their outward relative displacement; such displacement would normally occur when a load is applied on the slab between two girders. A combination of flange restraint against lateral movement and the cracking of concrete at the bottom of the slab leads to the formation of a shallow arch in the slab with the straps acting as ties. The degree of lateral restraint provided by the straps controls the relative lateral movement of adjacent girders and governs the ultimate load at which the slab fails in punching. The failure load can be several times greater

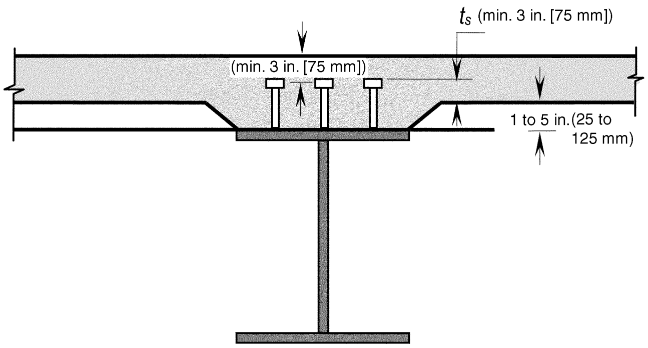

Fig. 3.1—Haunch between the deck slab and top of the supporting beam.

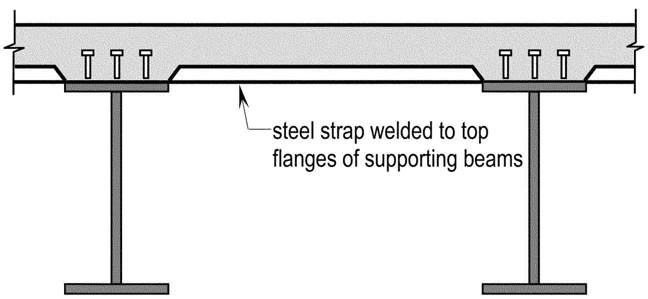

Fig. 3.2—Transverse confinement by directly connected straps.

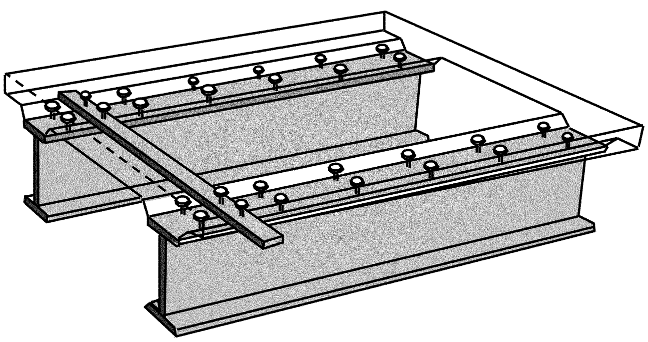

Fig. 3.3—Illustration of transverse confinement by partially studded straps.

than the load that causes flexural cracking of the slab, and is markedly greater than the failure load of an RC slab having the same dimensions. The design guidelines that follow are for cast-in-place and precast bridge deck slabs in accordance with the load and resistance factor design (LRFD) format of the AASHTO Specifications (1998). This method for designing bridge deck slabs can be modified suitably to address slabs of other structures, such as parking garages and marine wharves.

A cast-in-place or precast slab supported on beams satisfying the conditions set forth as follows need not be analyzed except for negative transverse moments due to loads on the overhangs and concrete parapets.

3.1—Composite action

The deck slab should be composite with parallel supporting beams in the positive moment regions of the beams.

The composite action of the deck slab with the supporting beams provides the necessary confinement in the longitudinal direction of the beams (Bakht, Mufti, and Jaeger 1998). The condition for the composite action in the positive moment regions should be considered in conjunction with Section 3.10, which deals with the composite action in the negative moment regions. For precast deck slabs, the longitudinal confinement can be provided within the panel itself, eliminating the need for the composite action (Mufti, Banthia, and Bakht 2001).

A deck slab supported on multispine box girders without external bracing experiences significant transverse moments under eccentric live loads. Such transverse moments cannot be dealt with by the arching action in the slab. Accordingly, neither cast-in-place nor precast deck slabs without reinforcement should be used on multispine box girders unless the box girders are prevented from excessive relative rotation by means of suitable bracing between them.

3.2—Beam spacing

The spacing of the supporting beams, *S*, should not exceed 12 ft (3.7 m).

The largest spacing of the supporting beams of RC deck slabs designed by the empirical methods of AASHTO is 13.5 ft (4.11 m). Cast-in-place and precast deck slabs have been tested in the laboratory with a beam spacing of up to 13.1 ft (3.99 m). To be conservative, however, the maximum spacing of bridge girders is limited to 12 ft (3.7 m). The spacing can be increased if approval is obtained from the authority having jurisdiction over the bridge. The approval process can be facilitated by conducting tests on full-scale models. The spacing of the girders for the first reinforcement steel-free deck slab was 8.9 ft (2.71 m). Before this slab was built, its full-scale model was tested to failure (Newhook and Mufti 1996a). Subsequently, precast deck slabs with beam spacing of 11.5 ft (3.51 m) were tested at full-scale in the laboratory and implemented in the field (Sargent, Mufti, and Bakht 1999). As noted in Chapter 7, the precast slab of a marine structure is supported on beams at a spacing of 13.1 ft (3.99 m).

The use of beam spacing greater than 12 ft (3.7 m) should be based on tests of full-scale models and not on the basis of analysis alone.

3.3—Slab thickness

The deck slab thickness *t* should be at least 6.5 in. (165 mm), and should not be less than *S*/15, where *S* is the spacing of the supporting beams, in inches (mm).

To predict the failure load of deck slabs under concentrated loads, an analytical model was developed by Mufti and Newhook (1998b). The limiting ratio of beam spacing to slab thickness required in this condition was partially established by using this method, which has been validated with experimental results. Also according to this method and verified by experiment, even a 6 in. (150 mm) thick slab provides sufficient strength to the deck slab supported on beams spaced at 6.6 ft (2.01 m) (Bakht and Lam 2000). To be conservative, however, a minimum slab thickness of 6.5 in. (165 mm) was used. Structures that are not subjected to heavy vehicular loads, such as parking garages, can have thinner slabs, but only after confirmatory tests on full-scale models have been conducted.

3.4—Diaphragms

Unless justified by rigorous analysis, the supporting beams should be connected with transverse diaphragms, or cross-frames, at a spacing of not more than 26 ft (7.9 m).

The transfer of wheel loads to beams remote from the live loads takes place primarily through the transverse flexural action of the deck slab. For most bridges, the transverse moments induced in the deck slab by this mechanism are positive, with tension occurring at the bottom of the slab. For such cases, the transverse moments are sustained by the straps at the underside of the precast and cast-in-place deck slabs. In certain cases, an eccentrically placed vehicle may induce small negative transverse moments in the deck slab, resulting in tension in the top surface of the slab that might lead to longitudinal cracks at its top surface. To accommodate such cases, this condition requires the provision of transverse diaphragms. This requirement is very conservative and can be superseded after additional analytical investigation. Another reason for providing the diaphragms, or cross-frames, is to provide additional redundancy in the bridge against failure of one of the girders. The condition for diaphragms does not apply to structures that do not carry heavy vehicular loads, such as parking garages and some marine structures.

3.5—Haunches

As illustrated in Fig. 3.1, the height of the haunch between the deck slab and the top of a supporting beam should be between 1 and 5 in. (25 and 125 mm), and the projection of the shear connecting devices in the deck slab, t_s, should be at least 3 in. (75 mm). In addition, the cover between the top of the shear connecting devices and the top surface of the deck slab should be a minimum of 3 in. (75 mm) when the slab is not exposed to moisture-containing chlorides. Otherwise, either this cover distance should be a minimum of 4 in. (100 mm), or the shear connectors should be provided with an approved coating.

The upper limit of the height of the haunch of a cast-in-place or precast slab ensures that the haunch is not sheared off under transverse loads generated by the arching action (Bakht and Lam 2000). The limit for the minimum projection of shear studs in the slab provides for an effective transfer of the interface shear from the supporting beams to the slab.

With the help of comparisons between experimental and analytical results, the effective height of the deck slab arch is the distance between its crown and the center of gravity of its transverse confining system, being the bottom transverse bars in RC slabs and straps in the cast-in-place and precast slabs (Bakht, Mufti, and Jaeger 1998). The reference Mufti and Newhook (1998b) contains a table of comparison between analytical and experimental failure loads. The effective thickness of the slab is greatly influenced by the height of the haunch.

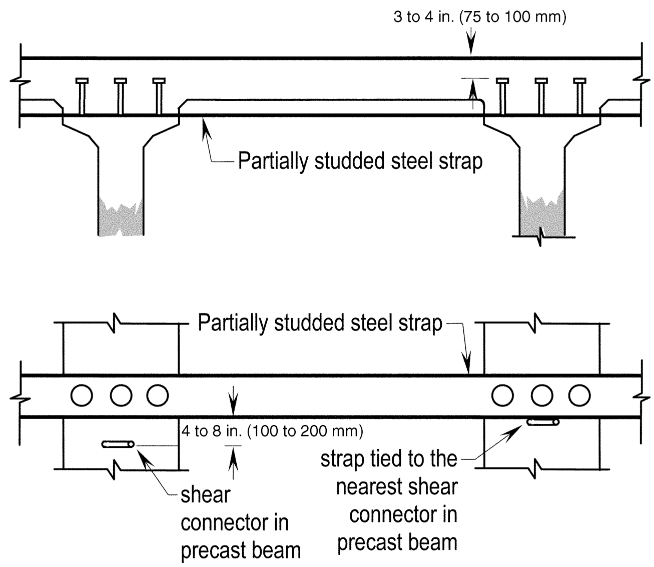

Fig. 3.4—Details of transverse confinement by partially studded straps.

Fig. 3.5—Photograph showing straps embedded in an Arch-Panel.

3.6—Transverse confinement

The top flanges of all adjacent supporting beams should be restrained by an external transverse confining system comprising straps or bars that are perpendicular to the supporting beams and that are:

1. Connected directly to the top of the flanges, as in welded steel straps (Fig. 3.2);
2. Connected indirectly, as in partially studded straps (Fig. 3.3 and 3.4);
3. Partially embedded in precast slabs (Fig. 3.5); or
4. Proven by approved full-scale laboratory testing.

The arching action in a reinforcement-free cast-in-place and precast deck slab is harnessed by transverse confinement, which occurs by restraining the relative lateral movement of the supports of the slab (the top flanges of the beams). Bakht and Lam (2000) and Bakht and Aly (1997) have shown that the transverse confinement can be provided by a number of methods, including the use of welded steel straps. In a precast slab of a forestry bridge, the transverse confinement was provided by steel straps partially embedded in the precast concrete deck slab (Fig. 3.5); these straps were provided with two or three shear studs at each of their embedded ends.

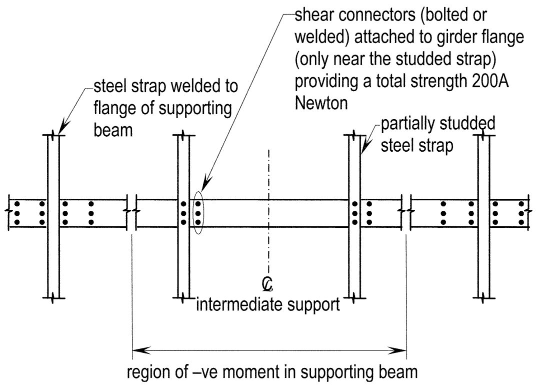

Fig. 3.6—Transverse confinement in longitudinal negative moment regions.

Guidance on testing the full-scale laboratory model of the deck slab can be found in Bakht and Lam (2000), Khanna, Mufti, and Bakht (2000), Newhook and Mufti (1996a), and Thorburn and Mufti (1995). The length of the model, measured parallel to the beams, should be at least twice the girder spacing, and the concentrated test load should be well away from the transverse free edges of the slab, especially if they are not stiffened.

3.7—Strap spacing

The spacing of straps, S_l, should not be more than 4 ft (1.2 m).

The requirement for the maximum spacing of the straps ensures that the lateral confinement is reasonably uniform along the length of the beams and also provides for the loss of a few straps. In the event a vehicle hits the underside of the bridge and more than two straps are lost, they should be replaced before permitting traffic above the damaged straps.

3.8—Strap size

Each strap should have a minimum cross-sectional area A in.^{2}, given by

$$A = \frac{F_s S^2 S_l}{Et} \qquad (3\text{-}1)$$

where the factor F_s is 0.87 ksi for outer panels and 0.73 ksi for inner panels; the girder spacing S, the strap spacing S_l, and the slab thickness t are in inches; and E is the modulus of elasticity of the material of the strap in psi.

In metric units, the cross-sectional area A in mm^2 is given by the following equation

$$A = \frac{F_s S^2 S_l}{Et} \qquad \text{(in SI)}$$

where F_s is 6.0 MPa for outer panels and 5.0 MPa for inner panels; S and S_l are in mm; t is in mm; and E is in MPa.

The requirement for the minimum cross-sectional area for the straps was developed through full-scale laboratory experiments and confirmed by analytical studies (Thorburn and Mufti 1995). For the analytical study, the method of Mufti and Newhook (1998b) was used. As suggested by E being in the denominator of Eq. (3-1), the cross-sectional area of the strap is governed by its axial stiffness and not by its strength. The value of F_s is lower for inner panels because of the substantial additional restraint provided to them by the slab in the adjacent panels, and because of bending between adjacent diaphragms in their own plane.

When the first frequency of vibration of a strap matches that of the bridge, the straps will vibrate excessively with the passage of every vehicle on the bridge. The steel straps in the Salmon River Bridge have a cross section of 4 x 1/2 in. (100 x 13 mm), with the latter dimension being the thickness. These straps vibrate visibly. Because of very low levels of stresses induced in the straps, there is no concern for the fatigue of their connections. The problem of strap vibration was eliminated in subsequent cast-in-place and precast slabs by increasing the thickness of the straps and reducing their widths.

3.9—Strap connection

The direct or indirect connection of a strap to the supporting beams should be designed to have a factored shear strength in kips of at least 29A, where A is in in.^{2} In metric units, the factored shear strength of the connection in Newtons is required to be at least 200A, where A is in mm.^{2}

The tensile force induced in a strap, even under specified failure loads, is significantly smaller than its tensile strength; therefore, the strength of its connection to the top flanges of the beams need not be as high as its tensile strength. As noted previously, the straps are required for their axial stiffness and not for their strength. Because the stresses induced in the straps are small under service loads, their fatigue resistance is not of concern. The factored connection strength is obtained by multiplying the nominal strength with the appropriate resistance factors. The effectiveness of an indirect strap connection to girders can be obtained experimentally by tests described by Bakht and Aly (1997).

3.10—Strap connection in negative moment regions

As shown in Fig. 3.6, in a negative moment region of a supporting beam, where the beam is not made composite with the slab, shear connecting devices should be provided on the beam in the vicinity of the straps, with a minimum total factored shear strength in kips (N) of 29 A (200 A), with A being in in.^{2} (mm^2). As shown in Fig. 3.4, such shear connecting devices should be within 4 in. (100 mm) of the nearest strap.

In continuous-span steel girder bridges, mechanical shear connectors are not always provided in the negative moment regions because of concerns for the fatigue resistance of the top flange in flexural tension. Even in such cases, this condition requires that some connectors be provided so that the tensile force in the straps is transferred effectively to the top flanges of the beams. Figure 3.6 shows that such connectors may be bolted to the top flanges to ensure that the fatigue category of the detail does not reduce the permissible fatigue stresses significantly.

3.11—Edge stiffening

Composite edge beams should have a suitably high flexural rigidity EI in the plane of the deck slab to stiffen the transverse edges of the deck slab. Recommended edge stiffening details are given in Section 5.1.

The arching action in the deck slab is harnessed in the longitudinal direction by making the slab composite with the beams and by providing edge beams with sufficiently high flexural stiffness in the plane of the slab (Bakht and Agarwal 1995).

3.12—Reinforcement for transverse negative moment

The deck slab should incorporate appropriate tensile reinforcement for transverse negative moments resulting from gravity loads on deck slab overhangs and horizontal loads on railings or concrete parapets.

Negative transverse moments induced by loads on the railings and those on the cantilever overhang do not generate arching action in the deck slab. When such negative moments are present, the deck slab should be suitably reinforced by transverse tensile reinforcement, which can be of coated steel, stainless steel, AFRP, CFRP, or GFRP.

The transverse negative moment reinforcement in the cantilever overhang should be extended into the adjacent panel as well. When the deck slab is subjected to uplift forces, such as wave action under a marine structure, reinforcement should be provided in the slab to resist such forces.

3.13—Reinforcement in longitudinal negative moment

For continuous-span bridges, the deck slab should contain longitudinal negative moment reinforcement in at least those segments in which the flexural tensile stresses in concrete under service loads are larger than $0.6f_{cr}$, where f_{cr} in psi is calculated in accordance with AASHTO LRFD Specifications (1998), as follows for f_c', also in psi

$$f_{cr} = 6.32(f_c')^{0.5} \tag{3-2}$$

In metric units, f_{cr} in MPa is calculated as follows for f_c', also in MPa

$$f_{cr} = 0.5(f_c')^{0.5} \quad \text{(in SI)}$$

The requirement for longitudinal negative moment reinforcement ensures that transverse cracks developing at the top of the slab are controlled.

If AFRP or GFRP is used as the longitudinal reinforcement, as noted by Tadros, Tromposch, and Mufti (1998), it should be treated as secondary reinforcement only and should not be taken into account in the calculation of the section properties of the beam in the negative moment region.

3.14—Fibers in concrete

The concrete of the slab should contain low-modulus fibers so that the residual strength index R is at least 0.15 (Appendix C).

The test method for determining the residual strength index is described in Appendix A. Discussion on the significance of this index is provided in Chapter 4.

3.15—Crack control

The slab should contain a mesh or grid of GFRP to control the width and number of cracks.

The phenomenon of cracking in cast-in-place and precast slabs, as related to the reinforcement-free concept described herein is discussed in Sections 5.4.1 through 5.4.4 The recommended approach to arrest cracking is presented in Section 5.4.4.

CHAPTER 4—MATERIALS

The design and in-service behavior of cast-in-place and precast reinforcement-free bridge deck slabs does not depend on special or exotic materials. Because the body of these deck slabs can be devoid of steel reinforcement, it is necessary to control the cracks that develop in concrete during its early life due to volumetric changes. These volumetric changes are compensated through the incorporation of low-modulus synthetic fibers.

The fiber volume fraction should be such that the post-cracking residual strength index R of the FRC is at least 0.15. The index R is the ratio of the maximum bending stress that an FRC beam can sustain after initial cracking and the initial cracking stress of the same specimen, so that $R = f_{pcr}/f_{cr}$. Both the cracking and postcracking strengths of FRC, f_{cr}, and f_{pcr}, respectively, are obtained from bending tests. If a beam specimen has no fibers, then R will be zero. When the tensile strength of FRC is greater than that of plain concrete, R is greater than 1.0. The fact that the required minimum value of R is significantly less than 1.0 confirms that the addition of the low-modulus fibers does not increase the tensile strength of the concrete. The addition of high-modulus fibers, such as carbon fibers, does increase the tensile strength of concrete, in which case the value of R is larger than 1.0. Details of the tests for determining the value of R are given in Appendix A. A similar method for determining the residual strength index is specified in ASTM C 1399-98.

Because of its predominantly compressive stress regime, the behavior of a well-confined deck slab is only slightly affected by the presence of large and frequent cracks. Agarwal and Ip (1992) tested a 9 in. (230 mm) thick RC slab with a large number of cracks wider than 0.02 in. (0.5 mm) and having a herring-bone type of pattern. By comparing the results of tests on this slab with those on a similar crack-free deck slab, they concluded that the presence of cracks does not affect the load-carrying capacity of the deck slab. Bakht and Selvadurai (1996) have also confirmed that a 6.9 in. (175 mm) thick, reinforcement-free deck slab without any fibers was able to sustain two million passes of each of 11, 15, 19, and 22 kips (49, 67, 85, and 98 kN) wheels without significant loss of stiffness. The low-modulus fibers in reinforcement-free deck slabs have practically no influence on the strength or stiffness of the slab. Pulsating load tests on two precast deck slabs designed using the concept described in this report have shown that the initiation of cracks in these slabs is related to the fiber volume fraction. Slabs with lower dosages of fibers develop cracks earlier than slabs with higher dosages (Mufti, Banthia, and Bakht 2001). As discussed in Sections 5.4 and

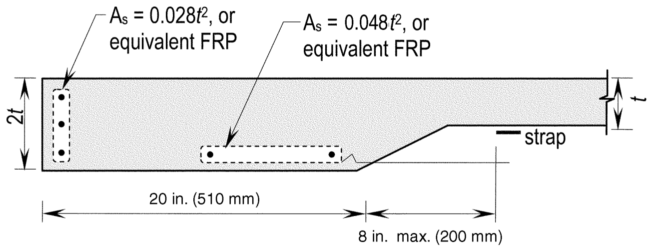

Fig. 5.1—Edge beam with thickened slab.

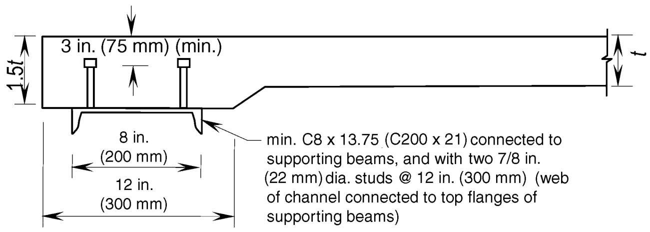

Fig. 5.2—Edge beam with composite steel channel.

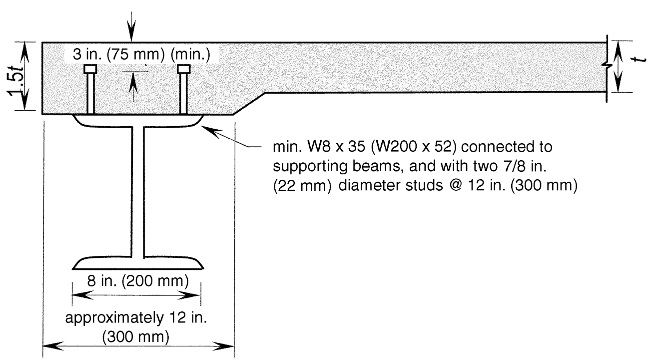

Fig. 5.3—Edge beam with composite steel I-beam.

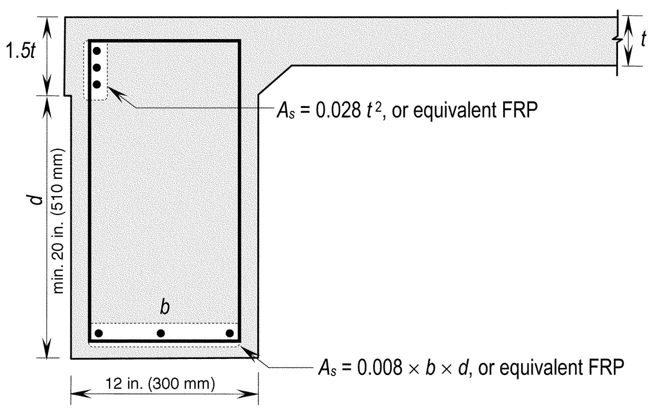

Fig. 5.4—Edge beam with reinforced concrete beam.

5.7, the presence of cracks, however, has little influence on the fatigue resistance of the slab.

CHAPTER 5—SPECIAL CONSIDERATIONS

5.1—Transverse edge stiffening

For an unsupported length of transverse edge beam less than 14 ft (4.25 m), the edge beam details can be as shown in Fig. 5.1, 5.2, 5.3, or 5.4. Engineering judgement should be exercised in formulating the details of edge beams having an unsupported length of greater than 14 ft (4.25 m).

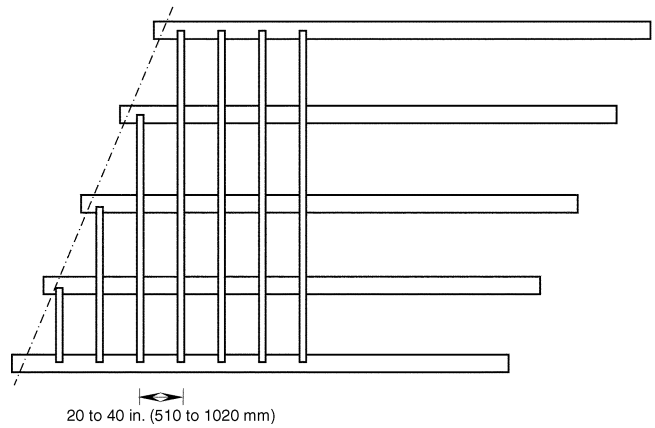

Fig. 5.5—Arrangement of straps near the end support of a skew bridge.

The edge beam details permitted above ensure the provision of a sufficiently high in-plane stiffness so that the deck slab is confined longitudinally near its transverse-free edges; these details are consistent with those permitted for RC deck slabs designed by the empirical method of CHBDC (2000). The edge stiffening details in Fig. 5.1 and 5.2 pertain to steel reinforcement and generally to equivalent FRP reinforcement that should be calculated on the basis of equivalent strength.

5.2—Skew angle

The skew angle of the bridge, while having practically no influence on the arching action of the deck slab (Bakht and Agarwal 1995), does affect the unsupported length of the edge beams. For large skew angles, it may be advantageous to reduce the unsupported length of an edge beam by connecting its midspan with a nearby girder by means of a strap. Even in skew bridges, the straps should be laid perpendicular to the supporting beams. As illustrated in Fig. 5.5, the strap lengths can be curtailed near the transverse-free edge of the slab, where edge stiffening already provides extra transverse confinement.

5.3—Concrete parapet connection

Conventionally, a concrete parapet is attached to the deck slab of a bridge by means of bent steel bars, which pass through both the wall and the deck slab. Such a connection is unsuitable when the wall is reinforced with FRP bars, which cannot be bent without incurring significant expenditure. Static and impact tests, reported by Maheu and Bakht (1994) and Klement and Aly (1997), respectively, have shown that the connection between the concrete parapet and deck slab can be made by means of double-headed tension bars. In highly corrosive environments, the tension bars can be made of stainless steel. When the deck slab is without both the cantilever overhang and tensile reinforcement, the effects of impact loads on the concrete parapet can be transferred directly to cross-frames under the deck, as illustrated in Fig. 5.6. This arrangement was used in the deck of the Salmon River Bridge, discussed in Chapter 7. Alternatively, the double-headed connection can be used instead of the conventional bent bars, as illustrated in the design examples discussed in Chapter 6.

Clause 13.7.3.1.1 of the AASHTO Specifications (1998) permits that "a crashworthy railing system may be used without further analysis and/or testing, provided that the proposed installation does not have features which are not present in the tested configuration that might detract from the performance of the tested railing system." The commentary to this clause notes the following: "When a minor detail is changed on, or improvement is made to a railing system that has already been tested and approved, engineering judgment and analysis should be used when determining the need for additional crash testing." The use of either system of connecting the concrete parapet to the deck requires the use of such judgment.

5.4—Cracking

5.4.1 *Laboratory observations*—As noted previously, many large-scale and full-scale models of reinforcement-free cast-in-place and precast slabs have been tested in Canada under monotonically increasing loads, that is, static loads. Two full-scale models were tested, in Canada, under simulated rolling wheels (Bakht and Selvadurai 1996). The effect of rolling wheels was achieved by means of four load pads, each representing the footprint of a tire. The magnitude of load on each of these pads was controlled in such a way that the load was passed sequentially from one pad to the next according to a predetermined pattern, so that the same total wheel load was maintained on the slab model during a cycle.

The concrete of one of the deck slab models contained chopped polypropylene fibers, and that of the other contained none. Both slabs were initially tested under static, monotonically increasing loads. Thereafter, each slab was subjected to two million passes each of 11.9, 16.4, 20.0, and 21.6 kips (52.9, 73.0, 89.0, and 96.1 kN) wheels. The slab with fibers developed a longitudinal crack on its underside during early stages of the dynamic testing. The plain concrete slab, however, developed a similar crack during the initial static load tests.

A significant outcome of the simulated rolling wheels was that, notwithstanding the presence of the longitudinal crack, the slabs shake down to stable elastic structures (Matsui et al. 2001). Matsui and his research colleagues in Japan have tested several full-scale models of reinforcement-free cast-in-place slabs and similar RC deck slabs under actual rolling wheels. They also observed longitudinal cracks on the underside of the slab similar to those observed in the Canadian tests. The cracks in their models ultimately bifurcated into Y shapes near the transverse end diaphragms. The Japanese tests confirmed that the RC slab developed several narrow longitudinal cracks on its underside, but the reinforcement-free cast-in-place slab developed only one predominant longitudinal crack, whose width was roughly equivalent to the sum of the widths of the cracks in the RC slab. Despite the presence of a wide crack at its soffit, the 6 in. (150 mm) thick reinforcement-free deck slab had significantly higher fatigue resistance than a 7 in. (180 mm) thick RC slab (Matsui et al. 2001).

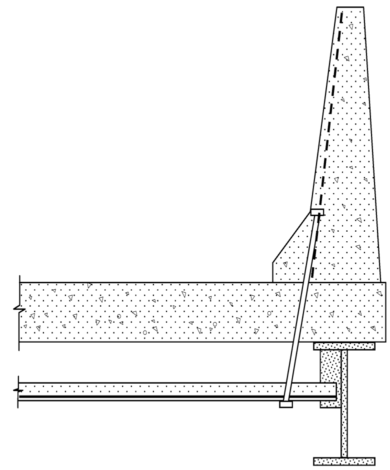

Fig. 5.6—Barrier wall connected directly to girders and cross-frames.

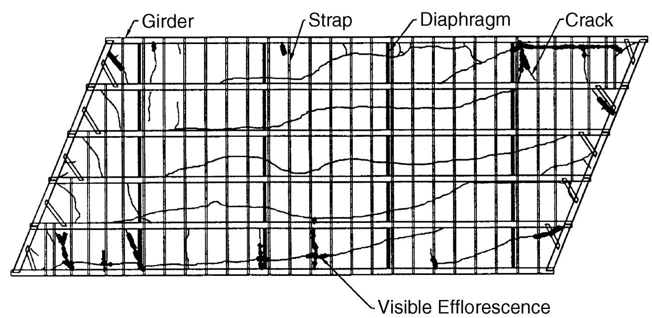

Fig. 5.7—Cracks on the underside of the ArchDeck slab of the Salmon River Bridge.

5.4.2 *Field observations*—A crack pattern survey of 14 RC bridge deck slabs is discussed by Agarwal (1990); seven of the surveyed slabs were designed for flexure and seven by the empirical method of the OHBDC (1979), which takes account of the internal arching action in the slab and leads to a smaller amount of reinforcement. Agarwal (1990) shows that all deck slabs eventually develop cracks on their soffits. The crack patterns are similar to those observed by Matsui et al. (2001) on RC slabs and are barely affected by the amount of reinforcement in the slab.

Similar to the RC slabs, the reinforcement-free cast-in-place and precast slabs without any crack-arresting material also developed cracks on their soffits. Consistent with laboratory observations, the cracks in the reinforcement-free cast-in-place slabs developed fairly early in the life of the deck. The Salmon River Bridge has both RC and reinforcement-free cast-in-place slabs; both were cast in November 1995. Inspection of the decks immediately after the 7-day initial moist-curing period revealed no cracks. A detailed visual inspection in May 1996 revealed transverse cracks on the soffit of both slabs. Because they occur in both the RC and the reinforcement-free slabs near the expansion joints, the cracks cannot be attributed to the absence of tensile reinforcement in the slabs near the expansion joints. Only the reinforcement free cast-in-place slab developed visible longitudinal cracks between each pair of adjacent girders. As can be seen

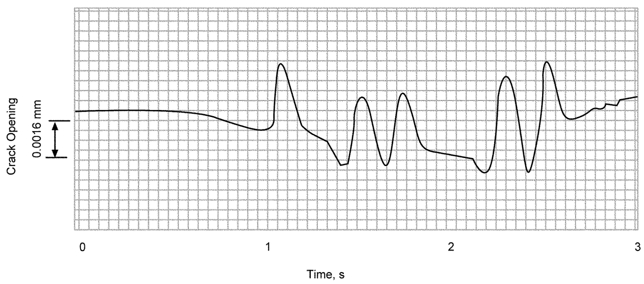

Fig. 5.8—Crack opening recorded under a fully-loaded test vehicle passing over a 30 mm (1.2 in.) high bump.

in Fig. 5.7, the longitudinal cracks stretch along the whole length of the span (Mufti, Newhook, and Khanna, 1999). These cracks, with a width of nearly 0.04 in. (1.0 mm), are clearly visible from below the bridge. The asphalt layer on the deck prevents the inspection of the top surface of the slab.

Periodic visual inspection has shown that the widths and pattern of the cracks have not changed significantly during the past 5.5 years. The Salmon River Bridge has a large number of sensors that monitor strains in various components. The data from these sensors confirm that the transverse load distribution of the bridge has not changed since the bridge was opened to traffic.

The second installation of a reinforcement-free cast-in-place slab was on the Chatham Bridge in Western Ontario. This four-span bridge has five steel girders at a spacing of 6.9 ft (2.1 m). The two outer spans incorporate a 6.9 in. (175 mm) thick cast-in-place slab with a haunch over girders, which vary from 2 to 4 in. (50 to 100 mm) in depth. The slab has a 2.7 ft (0.82 m) long overhang beyond each girder. To provide for the transverse negative moments due to loads on the cantilevers, the overhang and the adjacent outer panels of the cast-in-place slab contain grids of CFRP placed near the top surface of the slab.

The reinforcement-free panels of the slab of the Chatham Bridge also developed cracks on its soffit. The cracks, however, are quite narrow and do not follow a regular pattern. A sensitive displacement transducer, installed across a transverse crack on one of the middle two panels on the cast-in-place slab, was monitored under a test truck running over a 1.2 in. (30 mm) high bump placed just ahead of the crack. The test truck had five axles, with the steering axle weighing approximately 11 kips (49 kN) and each of the other four axles weighing approximately 22 kips (98 kN). The output from the displacement transducer was recorded on a strip-chart recorder. This output is reproduced in Fig. 5.8, in which it can be seen that the maximum crack opening under a fully loaded axle dropping from a 1.2 in. (30 mm) high bump was only approximately 0.0008 in. (0.02 mm). This observation confirms that the crack opening at the soffit of a well-confined slab is little affected by wheel loads on the deck.

5.4.3 *Mechanics of longitudinal crack opening*—A wheel load on the deck slab causes the top of the supporting girders to move outwards, inducing a tensile force in the straps or bottom transverse reinforcement. When the bottom layer of the deck slab concrete is in close vicinity of the strap or the reinforcement, it develops longitudinal cracks. The reinforcement in the RC slab helps to distribute the cracks, thus controlling their widths. A deck slab that is devoid of tensile reinforcement develops mainly one wide longitudinal crack.

Arguably, a longitudinal crack at the soffit of a reinforcement-free cast-in-place or precast deck slab should not extend into the compressive stress regime of the slab, and therefore, should not develop into a full-depth crack. The compressive stress regime, however, exists in the slab only as long as the wheel load is above the crack. This situation exists under pulsating loads (Mufti, Banthia, and Bakht 2001). Under rolling wheels, the slab at a given location rebounds immediately after the wheel has passed over it, inducing tensile forces at the sharp tip of the crack. The gradual rising of the sharp tip of a longitudinal crack at a transverse free edge of a steel-free deck slab was observed during a pulsating load test at the University of British Columbia (Mufti, Banthia, and Bakht 2001).

The appearance of cracks in reinforced concrete deck slabs with tensile reinforcement do not generally cause significant alarm because of the belief that even if the loss of concrete should occur in the slab, such as from a puncture, the mesh of reinforcement would be able to restrain a vehicle from going through the deck. The thought of a hole in a deck slab without the reassurance of tensile reinforcement in place concerns some engineers. Such fears, however, should be allayed by the long-term service performance of the deck slabs presented in this report.

Dalhousie University has conducted fatigue tests on a 6.9 in. (175 mm) thick cast-in-place deck on girders at a spacing of 6.56 ft (2.00 m). In one location, the slab had a full-depth longitudinal crack (Limaye et al. 2002). An 88 kip (391 kN) load was placed close to one side of the crack and was pulsated 1800 times. The slab did not show any measurable damage. The method described in Section 5.7.2 shows that 1800 cycles of an 88 kips (391 kN) load are equivalent 46 million passes of the AASHTO factored fatigue wheel load of 14 kips (62.3 kN) wheel, (Section 5.7.1 for details.) In a subsequent test, the same slab was tested under the same magnitude of load placed across a full-depth longitudinal crack. The slab sustained 40,000 cycles of the load without showing any noticeable signs of damage. This test is equivalent to subjecting the slab to more than 1 billion passes of the fatigue design load.

5.4.4 *Crack control*—To distribute the cracks and reduce their widths, deck slabs should be provided with a mesh of GFRP near the bottom surface (Section 3.15). In highly corrosive environments, an orthogonal mesh of 3/8 in. (10 mm) diameter GFRP bars, spaced at 12 in. (300 mm), is recommended. For example, the cast-in-place deck of the Hall's Harbour deck contains a mesh of GFRP near its top surface. The top surface of these panels is free of visible cracks, confirming the effectiveness of the GFRP grid in controlling cracks (Newhook et al. 2001). A series of pulsating load tests conducted at the Dalhousie University and University of Manitoba confirmed that a grid of nominal GFRP placed near the bottom of a cast-in-place slab controls the cracks effectively (Limaye et al. 2002; Memon, Mufti, and Bakht 2003).

5.5—Splitting stresses

When a concrete T-beam without transverse reinforcement in its top flange is tested in flexure, it is likely to fail due to the splitting stresses that cause the top flanges to separate from the web, or stem, leading to full-depth longitudinal cracks on each side of the stem. To prevent such failure, many European bridge design codes require both the transverse reinforcement and concrete in the deck slab to resist longitudinal shear at the various possible shear failure surfaces in the deck slab. Jaeger and Bakht (2001) have developed an analytical method for determining the magnitude of splitting stresses; they have shown that the tensile splitting stresses in built deck slabs are so small that they could be resisted by even the nominal transverse reinforcement in the cantilever overhangs. In internal panels, the forces causing the splitting stresses are easily resisted by the straps without requiring any additional reinforcement in the concrete.

5.6—Provisions for safety

In a slab-on-girder bridge with several girders and an RC slab, if a girder fails (for example, due to fatigue or impact from a high vehicle), the collapse of the entire structure can sometimes be prevented by the tensile reinforcement acting under large deflections as a catenary. There may be concerns that a reinforcement-free deck slab without the tensile reinforcement might not provide the same safe condition as that of a RC slab. In bridges with reinforcement-free cast-in-place or precast slabs, a degree of safety is provided by a combination of frequent intermediate diaphragms, a mesh of GFRP for crack control, and possible tensile reinforcement for transverse negative moments in the overhangs and adjacent panels. Because the substantial reinforcement for negative moments extends to the second outermost girder, it is expected that this tensile reinforcement would provide nearly the same degree of safe guard against the collapse of an external girder as an RC slab.

5.7—Fatigue resistance of deck slabs

5.7.1 *Fatigue loads*—The failure load of a deck slab under a monotonically increasing (static) load is denoted as P_s. The slab fails under a smaller load than P_s when the same load is applied many times. The failure load of a slab under N cycles is denoted as P ($P = P_s$ for $N = 1$).

The AASHTO LRFD Specifications (1998) do not provide any direct guidance on the required values of N and P for deck slabs. These specifications do, however, require that for consideration of fatigue, only one design truck should be considered, with its loads multiplied by a load factor of 0.75 (Clause 3.4.1.1), as well by ($1 + IM$), where IM, the dynamic load allowance, is 0.15 (Clause 3.6.2.1). The design truck has three axles, weighing 8, 32, and 32 kips (36, 142, and 142 kN), respectively. The factored load for the second and third axles, including IM, is 27.6 kips (123 kN). The factored wheel load, being half the axle load, is therefore 13.8 kips (61.4 kN).

Commentary Clause C3.6.1.4.2 of the AASHTO Specifications notes that the average daily traffic (ADT) in a lane is physically limited to 20,000 vehicles, and the maximum fraction of trucks in traffic = 0.20. Thus, the maximum number of trucks per day in one direction ($ADTT$) is 4000. When two lanes are available to trucks, the number of trucks per day in a single lane, averaged over the design life, ($ADTT_{SL}$) is found by multiplying $ADTT$ with 0.85, giving $ADTT_{SL}$ = 3400. If the much-lighter first axle of the design truck is ignored, the average number of wheels experienced by a deck slab per day = 6800.

Assuming a design life of 75 years, a deck slab is therefore expected to sustain 6800 × 365 × 75 (= 186 million) passes of a 13.8 kips (61.4 kN) wheel.

5.7.2 *Number of cycles versus failure load*—A given number of cycles N of a load P can be equated to N_e cycles of an experimental load P_e only on the basis of an established relationship between P and N. Matsui and his colleagues in Japan are the only researchers (Matsui et al. 2001) who have provided a P-N relationship based on rolling wheel tests on full-scale models of both RC and reinforcement-free deck slabs. Their conclusions are quantified by Eq. (5-1), which is applicable to both reinforced and unreinforced slabs

$$\log(P/P_s) = -0.07835 \times \log(N) + \log(1.52) \tag{5-1}$$

The equation gives P/P_s greater than 1.0 for N smaller than approximately 500. Matsui et al. (2001) contend that this equation is valid only for N greater than 10,000.

North American researchers (Petrou, Perdikaris, and Wang 1993) and Korean researchers (Young and Chang 1998) have presented similar relationships. Their equations, however, may not be considered highly reliable because they are based on the results of tests on small-scale models

Memon has conducted fatigue compressive tests on concrete cylinders. On the basis of his experimental evidence, he has presented the following variation to the Matsui et al. equation (Mufti et al. 2002)

$$P/P_s = 1.0 - \ln(N)/30 \tag{5-2}$$

For N greater than 10,000, Eq. (5-2) gives nearly the same results as the Matsui et al. equation. It also gives the correct result for $N = 1$. Current studies on full-scale models at Dalhousie University and the University of Manitoba are expected to confirm the validity of this equation. In the absence of a more-reliable relationship, Eq. (5-2) is used to determine equivalent number of cycles of various loads. The following notation is introduced.

P_1 and P_2 are two different wheel loads; n_1 and n_2 are the corresponding number of passes of P_1 and P_2, respectively, so that the two loads have the same damaging effect; N_1 and N_2 are the limiting number of passes corresponding to P_1 and P_2, respectively; $R_1 = P_1/P_s$; and $R_2 = P_2/P_s$.

It is assumed that the cumulative damage to the deck slab is proportional to the number of passes of a load. Thus, $N_2/N_1 = n_1/n_2$. This relationship and Eq. (5-2) lead to the following equation for n_2

$$n_2 = n_1 \times e^S \text{ where } S = (R_1 - R_2) \times 30 \tag{5-3}$$

A full-scale reinforcement-free cast-in-place slab is currently being tested in Dalhousie University under simulated rolling wheel loads. The static failure load P_s of a similar slab was found to be 221 kips (983 kN). In one test, a patch load of 88 kips (391 kN) was pulsated 1800 times on one side of a full-depth longitudinal crack. Calculating the equivalent number of cycles corresponding to the AASHTO factored wheel load of 13.8 kips (61.4 kN):

R_2 = 88/221 = 0.40

R_1 = 13.8/221 = 0.06

S = (0.40 – 0.06) × 30 = 10.2

n_2 = $1800 \times e^{10.2}$ = 48 million

Thus, it is concluded that 1800 passes of a 88 kips (391 kN) load are equivalent to 48 million passes of the required design load.

5.7.3 *Effect of reinforcement on fatigue resistance*—Perdikaris and Beim (1988) have conducted a large number of rolling wheel tests on small-scale models of deck slabs designed for bending and deck slabs designed by the Ontario empirical method, later adopted by the AASHTO LRFD Specifications. This method leads to a reduction of approximately 43% in the amount of reinforcement. These two researchers have concluded that the fatigue strength of the slabs with smaller amount of reinforcement was approximately 20 times higher.

The implication in the aforementioned conclusion is that the fatigue resistance of a concrete deck slab improves with a reduction in the amount of reinforcement. This implication has been endorsed by Matsui et al. (2001), who conducted rolling wheel tests on full-scale models of deck slabs with and without reinforcement. Their conclusions are: "The steel-free deck slabs exhibit significantly higher fatigue resistance than RC slabs with same thickness." Matsui et al. (2001) also recommend that "for new construction, a grid of minimum reinforcement be provided near the bottom face of the slab …"

Sufficient experimental evidence exists to confirm that the reinforcement-free cast-in-place and precast slabs have better fatigue resistance than slabs with two meshes of steel reinforcement. The addition of a grid of nominal reinforcement, while slightly reducing the fatigue strength of the slab, is expected to control cracks and provide additional safety against accidental damage.

CHAPTER 6—DESIGN EXAMPLES

To illustrate the design methodology, the precast slab of a three-span bridge is considered. As shown in Fig. 6.1(a), the span lengths are 80, 120, and 80 ft (24.4, 36.6, and 24.4 m). In this example, the bridge has four steel plate girders at a spacing of 10 ft (3.1 m), and the deck slab has 3 ft (0.9 m) wide overhangs beyond each of the two outer girders (left half of Fig. 6.1(b)).

The concrete parapet is a New Jersey-type concrete wall designed for Level PL3. The girders are assumed to be of weathering steel.

In another example, the four girders are prestressed concrete girders, AASHTO-PCI Bulb T-72 (right half of Fig. 6.1(b)). The cross section of the PL3 concrete parapet adopted for the bridges in both examples is shown in Fig. 6.2.

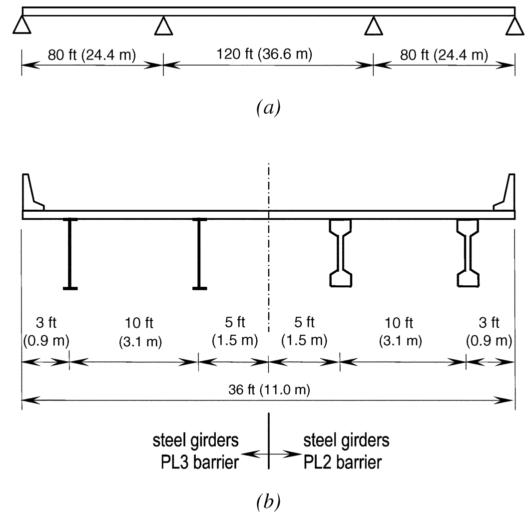

Fig. 6.1—Details of example bridges: (a) elevation; and (b) cross section.

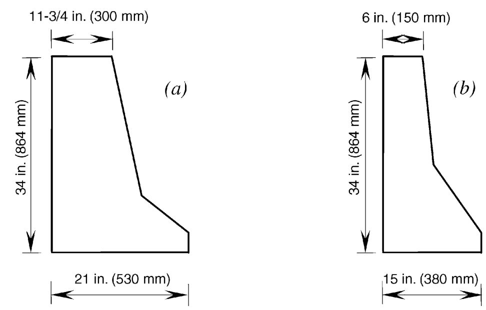

Fig. 6.2—Cross sections of barrier walls: (a) PL3 wall; and (b) PL2 wall.

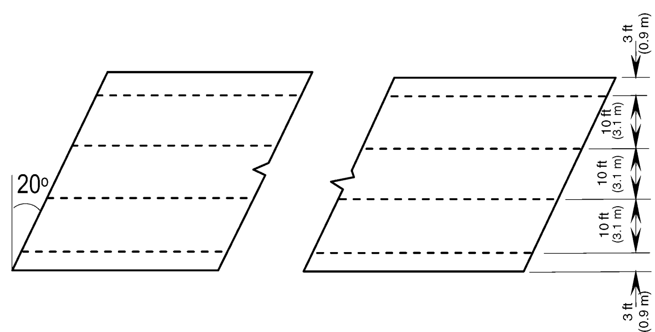

Fig. 6.3—Partial plan of example bridge.

As shown in Fig 6.3, both alternatives of the example bridge have a skew angle of 20 degrees.

Several features are common to both of the design examples:

1. The cast-in-place slab should be designed according to the provisions given in Chapter 3 of this document, and the other design details should be in accordance with the AASHTO LRFD Specifications (1998);

2. The deck slab and its components should be designed for Strength I and Extreme Event II of the AASHTO LRFD Specifications for live loading HL-93;

3. The deck slab should be cast in place and made composite with the girders;

4. The specified compressive strength of the slab concrete should be 4 ksi (28 MPa); and

5. An allowance, equivalent to a 2 in. (50 mm) thick layer of asphalt, should be made for future superimposed loads.

6.1—Common features

This section contains the design features common to the two example bridges, with steel and concrete girders, respectively.

Because the beam spacing of 10 ft (3.1 m) is within the limit of 12 ft (3.7 m) required by Section 3.2, a cast-in-place slab can be used.

If the transverse negative moments due to loads on the cantilever overhang are to be provided for by FRP reinforcement, reference should be made to ACI 440.1R-03, "Guide for the Design and Construction of Concrete Reinforced with FRP Bars."

6.1.1 *Slab thickness*—The minimum slab thickness required by Section 3.3 is the greater of 6.5 in. (165 mm) and 10.0 × 12/15 = 8 in. (200 mm). Therefore, the thickness of 8 in. (200 mm) is adopted for the cast-in-place slab.

6.1.2 *Crack control reinforcement*—To control the widths of cracks (Section 3.15), an orthogonal mesh of 1/2 in. (13 mm) diameter steel bars is selected at a spacing of 12 in. (300 mm) in each direction. In highly corrosive environments, the corrosion-resistant GFRP grid would be composed of No. 3 bars (No. 10) (with a diameter of 3/8 in. [10 mm]) at a spacing of 12 in. (300 mm) in each direction. The mesh for crack control is provided near the bottom of the slab. If the mesh is of GFRP bars, the cover to the bottom should be 1-3/8 ± 3/8 in. (35 ± 10 mm). For a mesh of steel, the concrete cover should be selected in accordance with the applicable code of practice.

6.1.3 *Haunch height*—In conformance with Section 3.5, the minimum haunch height should be 1 in. (25 mm). In certain longitudinal positions, the height of the haunch should be greater than the above minimum height (to adjust for camber).

6.1.4 *Straps*—In this example, we confine the slab transversely by partially studded straps of weathering steel. As an alternative, coated steel or stainless steel straps could be used. The largest spacing of straps permitted by Section 3.7 is 4 ft (1.2 m). A spacing of 3 ft (0.9 m) is adopted for the straps, so that $S_l = 3 \times 12 = 36$ in. The girder spacing $S = 10 \times 12 = 120$ in. As noted in Section 6.1.1, the thickness of the slab t is 8 in. The modulus of elasticity E of steel is taken as 29,000 ksi. With this data, the inch-pound equation in Section 3.8 can be rewritten as follows for the outer panels of the deck slab

$$A = 0.87 \times 120^2 \times 36/(29{,}000 \times 8) = 1.94 \text{ in.}^2 \text{ (1250 mm}^2\text{)}$$

A cross-sectional area of 1.6 in.^{2} (1030 mm^2) would have been sufficient for straps in the inner panel. Advantage is not taken of this reduced area of cross section in favor of the ease of construction. Accordingly, continuous steel straps with a 1 x 2 in. (25 x 50 mm) cross section are selected at a spacing of 3 ft (0.9 m) for both the outer and inner panels. The area of cross section of a strap, A, is 2.0 in.^{2} (1290 mm^2).

As discussed in Section 8.2, the constructibility of the cast-in-place slab can be improved by locating the straps at the transverse joints of the slab formwork. It can be shown that 1 x 2-1/2 in. (25 x 64 mm) straps at a spacing of 4 ft (1.2 m) can also satisfy the requirements of Section 3.8.

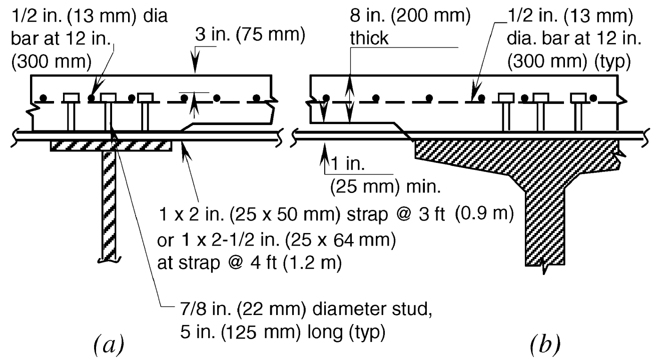

Fig. 6.4—Partial cross section of the ArchDeck slab over a girder: (a) bridge with steel girders; and (b) bridge with concrete girders.

In accordance with the requirement of Section 3.9, the minimum connection strength of a strap = 29 × 2.0 = 58 kips. Studs of 7/8 in. diameter are selected initially. The cross-sectional area of the stud is 0.6 in.^{2} AASHTO Clause 6.10.7.4.4c (1998) requires the nominal shear resistance Q_n of a stud to be calculated as follows

$$Q_n = 0.5 \times A_{SC} \times (f_c' \times E_C)^{0.5}, \tag{6-1}$$

$$\text{but not less than } A_{SC} \times F_u$$

where A_{SC} is the area of cross section of the stud (0.6 in.^{2}), f_c' is the specified compressive strength of concrete (4.0 ksi), and F_u is the specified minimum tensile strength of the stud (50 ksi). Therefore, Q_n is 30 kips.

AASHTO Clause 6.10.7.4.4a (1998) requires the factored resistance of a stud, Q_r, to be $\varphi_{SC} \times Q_n$, where the resistance factor φ_{SC} is specified by Clause 6.5.4.2 to be 0.85. Hence, $Q_r = 0.85 \times 30 = 25.5$ kips (115 kN). Three studs are selected to give a combined factored shear strength of 76.5 kips (340 kN), which is more than the required strength of 58 kips (258 kN).

The cross section of the cast-in-place slab above steel and concrete girders is shown in Fig. 6.4(a) and (b), respectively, along with the partially studded straps.

6.2—Transverse edge beams

Because of the skew angle of 20 degrees, the girder spacing of 10 ft (3.1 m) leads to the length of the transverse edge beam between girders of 10.64 ft (3.24 m). Because this length is less than 14 ft (4.25 m), any of the edge beams of Fig. 5.1 through 5.4 can be adopted.

For the first example bridge, the edge beam of Fig. 5.2 is adopted with a steel channel section C8 x 13.75. The steel of the channel is the same weathering steel as that for the girders. Although not shown herein, the steel channel is connected to the girders so that it also provides additional transverse confinement to the deck slab. The above edge-stiffening detail does not eliminate the need for cross frames

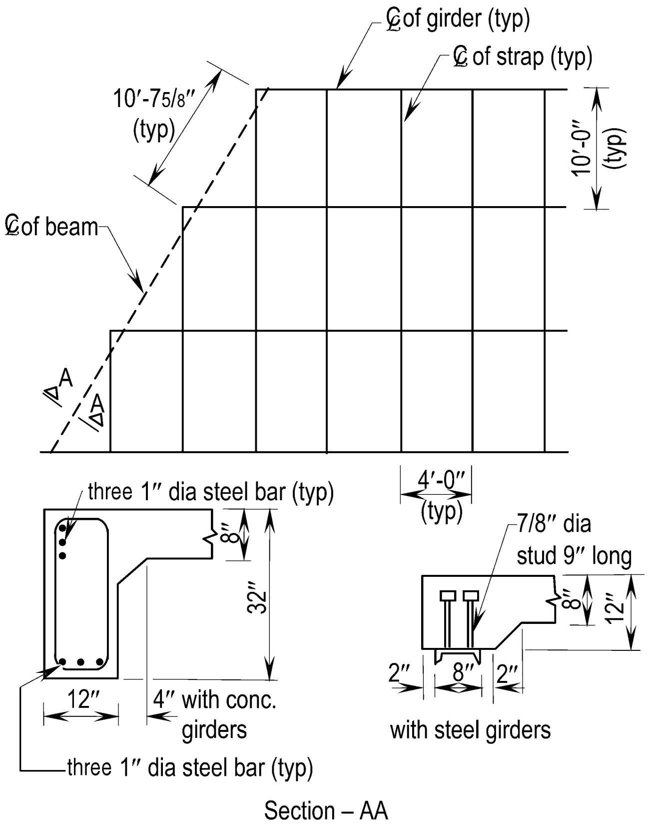

Fig. 6.5—Details of edge beams. (Note: 1 in. = 25.4 mm.)

at the bridge supports, which may be required for stability during construction.

The edge beam details of Fig. 5.4 are used for the second example bridge with concrete girders. This beam would serve both as an edge-stiffening beam and an end diaphragm. As noted in Fig. 5.4, the width of the edge beam should be 11.8 in. (300 mm), and its minimum depth = 20 + 1.5 × 8 = 32 in. (810 mm). A 12 in. (305 mm) wide and 32 in. (810 mm) deep beam is used. The area of cross section of the steel bars close to the vertical face of the beam, near its top, = 0.028 × 8^2 = 1.79 in.^{2} (1150 mm^2). This area can be provided by three 1 in. (25 mm) diameter bars. The cross-sectional area of the steel bars at the bottom of the beam = 0.008 x 12 x 20 = 1.92 in.^{2} (1240 mm^2). This area can also be provided by three 1 in. (25 mm) diameter steel bars.

Details of the two edge beams are given in Fig. 6.5. As noted in Section 5.1, the steel bars in the edge beams for the bridge with concrete girders can be replaced by GFRP bars on the basis of strength equivalence. GFRP bars with a No. 5 bar (No. 16) designation and a diameter of 5/8 in. (16 mm), with the specified tensile strength of 92 ksi (634 MPa), are used. Because the specified tensile strength of reinforcing steel is 58 ksi (400 MPa), the required areas of the GFRP bars can be obtained by dividing the aforementioned calculated areas with 635/400 = 1.59. Thus, the cross-sectional areas of the bars near the vertical face and bottom of the beam are equal to 1.12 and 1.21 in.^{2} (723 and 781 mm^2), respectively. At both of these locations, four No. 5 (No. 16) bars are selected for severely corrosive environments. Three additional 1/2 in. (13 mm) diameter bars should be provided as skin reinforcement near the vertical faces of the beams.

6.3—Parapet wall

The parapet wall design should satisfy the AASHTO requirements. For reference, it is noted that the CHBDC (2000) permits the connection of the parapet to the deck slab by means of 3/4 in. (19 mm) diameter, double-headed steel or stainless steel bars at a spacing of 12 in. (300 mm). In a recent paper by El-Salakawy et al. (2004), it has been suggested that parapets connected to the deck slab with bent GFRP bars can have the same or superior crashworthiness as parapets connected with bent steel bars.

CHAPTER 7—CASE HISTORIES

As noted previously, the concept described in this report has already been applied to six structures in Canada. Details of these structures are presented in Table 7.1.

All six structures have been discussed in the technical literature. The first application of the new deck slab was in 1995 in the Salmon River Bridge (Newhook and Mufti 1996b); the contractor for this bridge was asked to bid separately for the RC and reinforcement free cast-in-place slabs. The bid for the reinforcement-free slab was 6% higher than the bid for the RC slab. The contractor admitted that the higher bid was a result of perceived difficulty in handing concrete with fibers; his concerns were found to be groundless.

The second application of the reinforcement-free cast-in-place slab, in the Chatham Bridge, was significantly more expensive than conventional RC slabs (Aly, Bakht, and Schaefer 1997). The reason for the higher cost is attributed to the extensive use of the very expensive CFRP reinforcement in the overhangs and adjacent deck slab panels.

The reinforcement-free cast-in-place slab for the third application, the Crowchild Trail Bridge, was selected in competitive bidding against a conventional RC deck slab (Tadros, Tromposch, and Mufti 1998). The judicious use of GFRP reinforcement in this deck slab was partly responsible for its lower bid price. For the Waterloo Creek Bridge, the cost of the reinforcement-free cast-in-place deck slab was nearly the same as that for RC slabs (Bakht and Mufti 1998).

Precast deck slabs were selected for the Lindquist Bridge because they were approximately 30% cheaper than conventional RC panels (Sargent, Mufti, and Bakht 1999). The Lindquist Bridge is on an unpaved forestry road where deicing salts are never used. The temporary nature of forestry roads and the absence of deicing salts removed the need for the usual considerations of long-term durability. The deck of the wharf of the Hall's Harbour in Nova Scotia incorporates precast deck slabs (Newhook et al. 2001).

CHAPTER 8—CONSTRUCTION AND CONSTRUCTIBILITY

8.1—Connection straps

The connection of transverse confining devices (straps) to the supporting beams is achieved by utilizing partially studded straps. The straps are laid perpendicular to the girders, preferably in continuous lengths across the width of the bridge. If the width of the bridge is too large to accommo-

Table 7.1—Details of cast-in-place and precast slabs built in Canada since 1995

Structure	Girder type	Girder spacing, ft (m)	Slab thickness, in. (mm)	Fiber content, % by volume	Unique features
Salmon River Bridge, Kemptown, Nova Scotia	Steel plate	8.9 (2.7)	7.9 (200)	0.55 (8.4 lb/yd^3 [5.0 kg/m^3])	• First ArchDeck in new construction. • Welded steel wraps.
Chatham Bridge, Chatham, Ontario	Steel plate	7.0 (2.1)	6.9 (175)	0.55 (8.4 lb/yd^3 [5.0 kg/m^3])	• First ArchDeck in rehabilitation. • Welded steel wraps. • CFRP reinforcement for negative transverse moment. • Concrete parapet with GFRP and double-headed connectors.
Crowchild Trail Bridge, Calgary, Alberta	Steel plate	6.6 (2.0)	7.3 (185)	0.44 (6.7 lb/yd^3 [4.0 kg/m^3])	• First ArchDeck on continuous-span bridge. • Partially studded straps. • GFRP reinforcement for negative transverse moments. • Concrete parapet with GFRP and double-headed connectors.
Waterloo Creek Bridge, British Columbia	Precast construction	9.2 (2.8)	7.5 (190)	0.44 (6.7 lb/yd^3 [4.0 kg/m^3])	• First ArchDeck application. • Partially studded straps.
Lindquist Bridge, British Columbia	Steel plate	11.5 (3.5)	5.9 (150)	0.44 (6.7 lb/yd^3 [4.0 kg/m^3])	• First ArchPanel application. • Partially studded straps, embedded in precast panel. • Composite action through shear bulkheads. • Very large girder spacing to slab thickness ratio. • Significantly less expensive than RC alternative.
Hall's Harbour Wharf, Nova Scotia	Concrete with GFRP bars	13.1 (4.0)	5.9 (150) (at crown)	0.44 (6.7 lb/yd^3 [4.0 kg/m^3])	• First ArchPanel in a marine structure. • Partially studded straps, embedded in precast panel. • Grid of GFRP rods to resist uplift wave action forces.

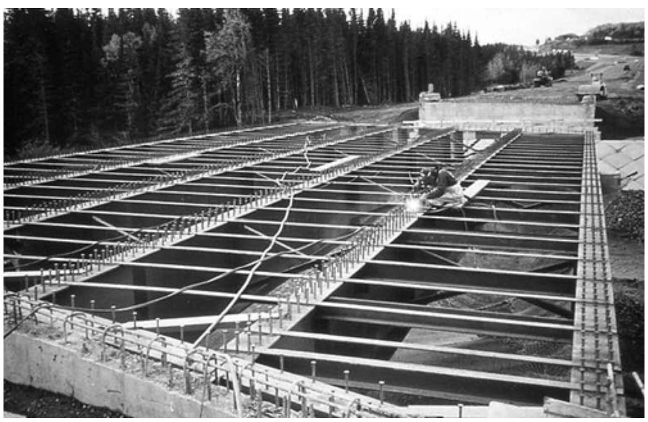

Fig. 8.1—Welded straps installed on steel girders.

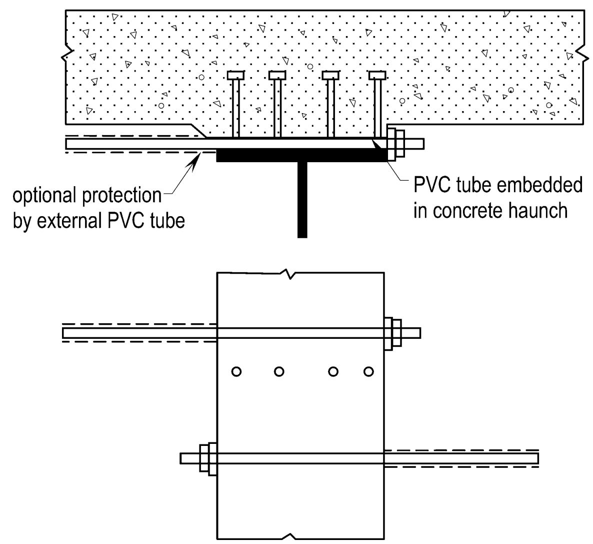

Fig. 8.2—Transverse confinement by staggered mild steel threaded rods.

date continuous straps, the straps' lengths should be staggered over the girders in such a way that the connection strengths of the straps on each side of the straddling girder is as required in Section 3.9. The straps can lie between groups of shear connectors on the girders, as shown in Fig. 8.1, which also shows that the straps are installed before the formwork is laid for the slab. To ensure that the straps do not move during construction, the straps should be tied lightly to the nearest clusters of studs on the girders. The effective transfer of the tensile forces in the straps to the girders is provided by the aggregate interlock between the shear studs on the straps and the girders.

The transverse confinement can also be provided by means of threaded mild steel rods passing through preformed holes in the haunches under the slab. Such confinement can be provided either by staggered rods (Fig. 8.2) or by continuous rods, which should contain lightly tightened restraining nuts on both sides of the girders. The same technique as used for mild steel threaded rods can also be used with FRP bars with suitably designed anchors having threads on their exposed surfaces.

Connection of the straps can also be made by means of welding. The engineer and contractor, however, should ensure that material compatibility and welding procedures will not introduce fatigue-inducing stresses in the beam flanges.

8.2—Formwork for slab

The construction of the four bridges with cast-in-place deck slabs demonstrates that the straps do not pose difficulty in stripping the formwork from the deck slabs. As shown in Fig. 8.3, there is a relatively large gap between the underside of the cast-in-place deck slab and the top of the girders that facilitates the removal of formwork panels. The constructibility of the cast-in-place deck slab can be improved by ensuring that the spacing of the straps is approximately the same as the length of formwork panels, measured in the direction of the supporting beams.

8.3—Mixing fibers

Since the addition of fibers makes concrete too stiff to mix, high-range water-reducing admixtures are always

Fig. 8.3—Gaps between slab and straps facilitate formwork removal.

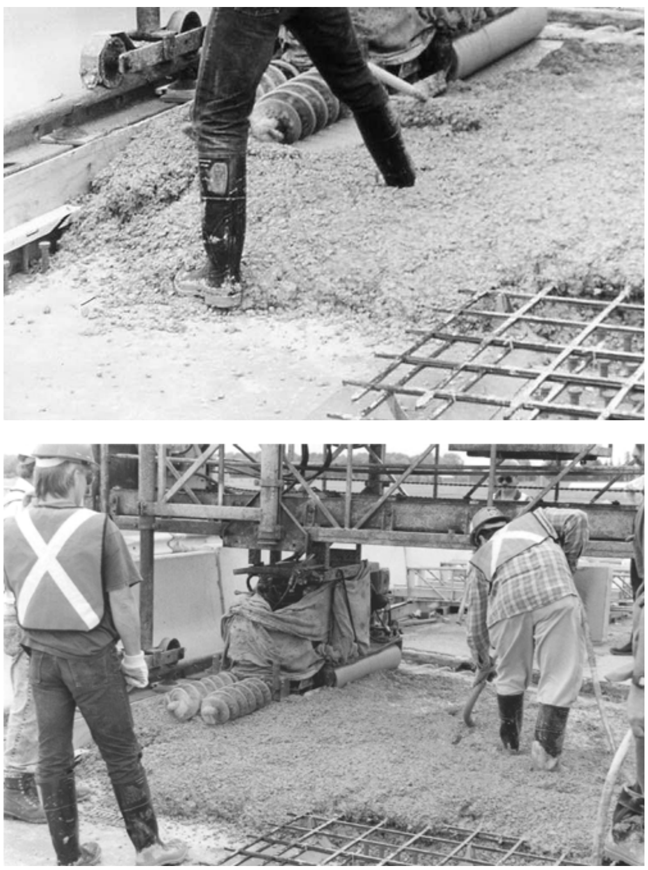

Fig. 8.4—Compacting, screeding, and finishing of the FRC of an ArchDeck slab.

added to concrete. For a given dosage of a high-range water-reducing admixture, the degree of difficulty in mixing low-modulus fibers uniformly with concrete increases when the volume of the batch approaches the capacity of the mixer. For example, a particular fiber volume fraction can be mixed easily if the mixer is only partially full, but can be very difficult to mix if the mixer is at full capacity. For this reason, a trial mixture using a mixer of the capacity to be used in construction should be used to establish the workability of the proposed FRC mixture.

Visual inspection alone may be sufficient to reject a mixture in which the fiber dispersion is highly nonuniform.

Compressive strength tests should be conducted well before start of construction, using the trial mixture recommended previously for the fiber dispersion. Fiber manufacturers usually provide reliable information about fiber dispersion in concrete corresponding to a given fiber volume fraction. Refer to Appendix C for more direction regarding the use of fibers.

8.4—Finishing FRC surfaces

Concrete with well-mixed, low-modulus fibers does not require any special treatment for finishing its surfaces. As shown in Fig. 8.4, the reinforcement-free cast-in-place deck slab of the Chatham Bridge, with CFRP grids in some panels, was compacted and finished in the usual way.

8.5—Precast installation

Precast deck slabs can be either composite or noncomposite with the supporting beams. The composite panels derive their full strength only after they have been made composite with the girders and the transverse gaps between the panels filled with grout. While placing them on the supporting beams, care should be exercised to ensure that these panels are not subjected to excessive construction loads before they attain their full strength.

The technique of making the precast slabs composite with the steel girders is the same as that employed traditionally with precast RC panels. The precast panels typically have conical holes with a bottom diameter of 11 in. (280 mm) and top diameter of 12 in. (300 mm). The holes lie directly over the top flanges of the girders and are spaced at about 40 in. (1040 mm). Each hole can accommodate a cluster of eight shear studs that are preinstalled on the girder flanges. After all of the panels are placed in position, the holes are filled with a quick-setting grout. The transverse gaps between adjacent panels are similarly filled with a suitable grout.

Noncomposite precast slabs, like their RC counterparts, are preferred in some forestry applications because of the ease of taking apart the bridge when it is no longer required.

CHAPTER 9—MAINTENANCE AND COST EFFECTIVENESS

9.1—Maintenance

Bridge deck slabs in corrosive environments require maintenance mainly because of the corrosion of the steel reinforcement that causes the concrete to spall, thereby leading to potholes on the deck. Some deck slabs are known to develop an orthogonal pattern of cracks even in benign environments; such cracks sometimes lead to holes bounded by the cracks. These severe cracks are caused by very heavy wheel loads and large volumes of steel reinforcement, with the latter leading to the fatigue failure of the steel-concrete interface. Cast-in-place and precast slabs without steel reinforcement should remain free of spalling caused by corrosion of steel. Extensive fatigue testing in Japan has confirmed that cast-in-place slabs have higher fatigue resistance than RC slabs. In light of the aforementioned information, well-designed cast-in-place and precast reinforcement-free slabs are expected to be maintenance-free. The steel straps, however, are not maintenance-free. Because of their susceptibility to corrosion, the steel straps or rods should be inspected periodically and coated when required. A gap, as called out in Fig. 3.1, between the straps and the underside of the slab, as can be seen in Fig. 8.3, is useful in permitting inspection as well as in facilitating remedial measures.

9.2—Repair

In the full-scale models reported by Bakht and Selvadurai (1996) and Newhook and Mufti (1996b), several straps were severed consecutively to study their effect on the load-carrying capacity of the cast-in-place slabs under concentrated wheel loads. The loss of even three consecutive straps did not impair the load-carrying capacity of the slab appreciably. The design provisions given in Chapter 3 for the transverse confinement are deliberately conservative to take into account the loss of a few straps. While the local corrosion of a strap does affect its axial strength significantly, it does not reduce its stiffness appreciably. Therefore, local corrosion of a strap does not pose a serious hazard to the strength of a cast-in-place or precast slab. If the steel straps corrode severely, they can be rehabilitated by welding on new straps. Threaded mild steel rods can be replaced easily.

9.3—Costs

Experience has shown that an expensive innovative concept is difficult to sell to bridge owners solely on the basis of life-cycle costs. Fortunately, the first-time costs of ArchDeck and ArchPanel slabs are of the same order of magnitude as those of their conventional counterpart, the RC slab.

The cost of an RC deck slab designed by the economical empirical method of AASHTO LRFD Specifications (1998) Section 9 is compared in the following with an ArchDeck slab for a bridge with a common girder spacing of 6.5 ft (2 m). It is assumed that the deck slab is directly exposed to salt water, for which condition the minimum cover required by AASHTO LRFD Specifications (1998) is 4 in. (100 mm). The minimum required core thickness of 4 in. (100 mm) with the top and bottom covers of 4 and 1 in. (100 and 25 mm), respectively, leads to a minimum slab thickness of 9 in. (230 mm). In accordance with the provisions of Clause 9.7.2.5 of AASHTO LRFD Specifications (1998), the RC slab is provided with a top orthogonal mesh of 1/2 in. (13 mm) diameter steel bars at a spacing of 12 in. (300 mm) in each direction, and a bottom orthogonal mesh of 5/8 in. (16 mm) diameter bars at a spacing of 12 in. (300 mm) in each direction. By using the design criteria of Chapter 4, the thickness of ArchDeck slab = 6.5 in. (165 mm). Straps of 1 x 1-1/2 in. (25 x 40 mm) cross section at a spacing of 4 ft (1.2 m) are sufficient. A mesh of 1/2 in. (13 mm) diameter steel bars at a spacing of 12 in. (300 mm) is provided for crack control at the mid-depth of the slab. The concrete is mixed with polypropylene fibers at the rate of 6.7 lb/yd^3 (4.0 kg/m^3), and a high-range water-reducing admixture at a rate of 4 L/yd^3 (5.2 L/m^3). The quantities of materials for the two slabs are listed in Table 9.1.

Table 9.1 also lists the approximate estimates of unit prices for the various materials. Using these unit costs, the costs of all the materials per ft^2 of the slab area are \$6.06 and \$5.37 for the RC and ArchDeck slabs, respectively. Notwithstanding slight variations in the assumed and prevalent unit costs and the additional royalty payment for the patented slab, the previous comparison confirms that the first-time cost of the ArchDeck and ArchPanel slabs is approximately the same as that of RC slabs designed by the AASHTO empirical method. The ArchDeck and ArchPanel slab should be significantly cheaper than the RC slabs designed by the flexural method.

The ArchDeck and ArchPanel slabs, being more durable than the conventional slabs, will prove even more economical on the basis of life-cycle costs.

Table 9.1—Comparison of material quantities in RC and ArchDeck slabs

Material	Materials for RC deck slab, /ft^2 of slab area	Materials for ArchDeck slab, / ft^2 of slab area	Approximate unit cost, U.S. \$
Concrete	0.75 ft^3	0.54 ft^3	\$5/ft^3
Steel	3.50 lb (reinforcement)	1.40 lb (reinforcement) 1.27 lb (structural steel)	\$0.66/lb (reinforcement) \$0.76/lb (structural steel)
7/8 in.-diameter studs on straps	—	0.1 No.	\$1.00 each
Polypropylene fibers, lb	—	0.17 lb	\$3.00/lb
High-range water-reducing admixture	—	0.06 L	\$3.00/L

CHAPTER 10—REFERENCES

10.1—Cited references

AASHTO, 1998, “LRFD Bridge Design Specifications,” American Association of State Highway and Transportation Officials, Washington, D.C.

Agarwal, A. C., 1990, “Load Testing of New Concrete Bridge Deck Slabs,” *Proceedings*, Third International Conference on Short and Medium Span Bridges, Toronto, Ontario, Canada, V. I, pp. 277-289.

Agarwal, A. C., and Ip, A., 1992, “Surface Cracking in the Fifteen Mile Creek Bridge,” *Proceedings*, Annual Conference on Canadian Society for Civil Engineering, Quebec City, V. I, pp. 195-205.

Aly, A.; Bakht, B.; and Schaefer, J., 1997, “Design and Construction of a Steel-Free Deck Slab in Ontario,” *Proceedings*, Annual Conference of the Canadian Society for Civil Engineering, Sherbrooke, Quebec, Canada.

ASTM C 1399, 1998, “Test Method for Obtaining Average Residual-Strength of Fiber-Reinforced Concrete,” ASTM International, West Conshohocken, Pa., 5 pp.

Bakht, B., and Agarwal, A. C., 1995, “Deck Slabs of Skew Bridges,” *Canadian Journal of Civil Engineering*, V. 22, No. 3, pp. 514-523.

Bakht, B., and Aly, A., 1997, “Testing in Isolation of Transverse Confining Systems for Steel-Free Deck Slabs,” *Canadian Journal of Civil Engineering*, V. 25, No. 4, pp. 789-796.

Bakht, B., and Lam, C., 2000, “Behavior of Transverse Confining Systems for Steel-Free Deck Slabs,” *Journal of Bridge Engineering*, ASCE, V. 5, No. 5, pp. 139-147.

Bakht, B., and Mufti, A. A., 1998, “Five Steel-Free Bridge Deck Slabs in Canada,” *Journal of the International Association for Bridge and Structural Engineering (IABSE)*, V. 8, No. 3, pp. 196-200.

Bakht, B.; Mufti, A. A.; and Jaeger, L. G., 1998, “Design Provisions for Steel-Free Deck Slabs,” *Proceedings*, 5th International Conference on Short and Medium Span Bridges, Calgary, Alberta, Canada.

Bakht, B., and Selvadurai, A. P. S., 1996, “Performance of Steel-Free Deck Slabs Under Simulated Wheel Loads,” *Second International Conference on Advanced Composite Materials in Bridges and Structures,* Montreal, Canada, pp. 767-776.

CHBDC, 2000, “Canadian Highway Bridge Design Code,” Canadian Standards Association International, Toronto.

El-Salakawy, E.; Masmoudi, R.; Benmokrane, B.; Brière, F.; and Desgagné, G., 2004, “Pendulum Impacts into Concrete Bridge Barriers Reinforced with Glass Fibre Reinforced Polymer Composite Bars,” *Canadian Journal of Civil Engineering*, V. 31, No. 4, pp. 539-552.

Jaeger, L. G., and Bakht, B., 2001, “Splitting Stresses in Concrete Flanges of Edge Girders in Bridge Decks,” *The Structural Engineer*, V. 79, No. 10, pp. 17-19.

Khanna, O. S.; Mufti, A. A.; and Bakht, B., 2000, “Experimental Investigation of the Role of Reinforcement in the Strength of Concrete Deck Slabs,” *Canadian Journal for Civil Engineering*, V. 27, No. 3, pp. 475-480.

Klement, T., and Aly, A., 1997, “Pendulum Testing of the Ontario Bridge Barrier,” Draft Paper, Research and Development Branch, Ministry of Transportation of Ontario, Toronto, Ontario, Canada.

Limaye, V.; Mufti, A. A.; Bakht, B., and Butt, S. D., 2002, “Steel-Free Deck Slabs Under Cyclic Loading,” *Proceedings*, 6th Conference on Short and Medium Span Bridges, CSCE, Vancouver, British Columbia, Canada, pp. 1311-1319.

Maheu, J., and Bakht, B., 1994, “A New Connection Between Barrier Wall and Deck Slab,” *Proceedings*, Annual Conference of the Canadian Society for Civil Engineering.

Matsui, S.; Tokai, D.; Higashiyama, H.; and Mizukoshi, M., 2001, “Fatigue Durability of Fiber Reinforced Concrete Decks Under Running Wheel Load,” *Third International Conference on Concrete under Severe Conditions*, Department of Civil Engineering, University of British Columbia, British Columbia, Canada, V. 1, pp. 982-991.

Memon, A. H.; Mufti, A. A.; and Bakht, B., 2003, “Crack Control with GFRP Bars in Steel-Free Concrete Deck Slabs,” *Proceedings*, CSCE Annual Conference, Monoton, Nebr. (CD-ROM)

Mufti, A. A.; Banthia, N.; and Bakht, B., 2001, “Fatigue Testing of Precast Arch-Panels,” *Proceedings*, Third International Conference on Concrete under Severe Conditions, Department of Civil Engineering, University of British Columbia, British Columbia, Canada, V.1, pp. 1032-1041.

Mufti, A. A.; Memon, A. H.; Bakht, B.; and Banthia, N., 2002, “Fatigue Investigation of the Steel-Free Bridge Deck Slabs,” *Concrete: Material Science to Application—A Tribute to Surendra P. Shah*, SP-206, P. Balaquru, A. Naaman, and W. Weiss, eds., American Concrete Institute, Farmington Hills, Mich., pp. 61-70.

Mufti, A. A., and Newhook, J. P., 1998b, “Punching Shear Strength of Restrained Concrete Bridge Deck Slabs,” *ACI Structural Journal*, V. 95, No. 4, July-Aug., pp. 375-381.

Mufti, A. A.; Newhook, J. P.; and Mahoney, M. A., 1999, “Salmon River Bridge Field Assessment,” *Proceedings of the 1999 Canadian Society of Civil Engineering*, Annual Conference, V. 1, pp. 51-61.

Newhook, J. P.; Bakht, B.; Mufti, A. A.; and Tadros, G., 2001, “Monitoring of Hall’s Harbour Wharf,” *Proceedings,* Annual Conference of the International Society for Optical Engineering, Newport, Calif.

Newhook, J. P., and Mufti, A. A., 1996a, “Steel-Free Concrete Bridge Deck—The Salmon River Project: Experimental Verification,” *Proceedings*, Annual Conference of the Canadian Society for Civil Engineering, Edmonton, Alberta, Canada.

Newhook, J. P., and Mufti, A. A., 1996b, “A Reinforcing Steel-Free Concrete Deck Slab for the Salmon River Bridge,” *Concrete International*, V. 18, No. 6, June, pp. 30-34.

OHBDC, 1979, “Ontario Highway Bridge Design Code,” Ministry of Transportation of Ontario, Downsview, Ontario, Canada.

Perdikaris, P. C., and Beim, S., 1998, “RC Bridge Decks Under Pulsating and Moving Loads,” *Journal of Structural Engineering*, ASCE, V. 114, No. 3, pp. 591-607.

Petrou, M. F.; Perdikaris, P. C.; and Wang, A., 1993, “Fatigue Behavior of Non-Composite Reinforced Concrete Bridge Deck Models,” *Transportation Research Record* 1460, Transportation Research Board, Washington D.C., pp. 73-80.

Sargent, D. D.; Mufti, A. A.; and Bakht, B., 1999, “Design Construction and Field Testing of Steel-Free Arch Panel Bridge Deck for Forestry Bridges,” *Proceedings of the 1999 Canadian Society for Civil Engineering Annual Conference*, V. I, pp. 95-104.

Seible, F; Karbhari, R.; Burgueno, R.; and Seaberg, E., 1998, “Modular Advanced Composite Bridge Systems for Short and Medium Span Bridges,” *Proceedings*, 6th International Conference on Short and Medium Span Bridges, Calgary, Alberta, Canada, pp. 431-441.

Tadros, G.; Tromposch, E.; and Mufti, A. A., 1998, “Superstructure Replacement of Crowchild Trail Bridge,” *Proceedings*, 5th International Conference on Short and Medium Span Bridges, Calgary, Alberta, Canada.

Thorburn, J., and Mufti, A. A., 1995, “Full-Scale Testing of Externally Reinforced FRC Bridge Decks on Steel Girders,” *Proceedings*, Annual Conference of Canadian Society for Civil Engineering, Ottawa, V. II, pp. 543-552.

Young, S. G., and Chang, S. P., 1998, “Behavior of Composite Bridge Decks Subjected to Static and Fatigue Loading,” *ACI Structural Journal*, V. 95, No. 3, May-June, pp. 249-258.

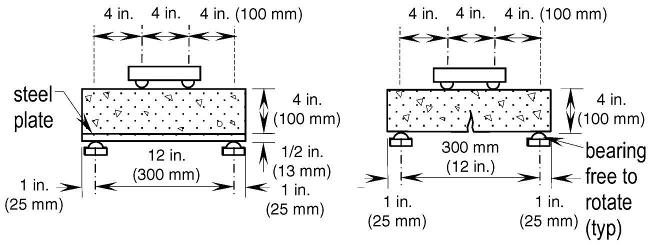

Fig. A.1—Elevations of an FRC test beam to determine its postcracking strength.

APPENDIX A—POSTCRACKING STRENGTH OF FRC TEST METHOD

The fiber volume fraction in FRC should be such that the postcracking residual strength index R of the FRC is at least 0.15, where R is given by

$$R_i = \frac{P_{pcr}}{P_{cr}} \tag{A-1}$$

where the postcracking load of FRC test beam, P_{pcr}, is obtained in accordance with the test described in the following. The cracking load of concrete, P_{cr}, shall be obtained from standard tests on at least five beams of the same size as that used in the postcracking load test.

The postcracking strength of FRC, f_{pcr}, is obtained from the average of results of tests on at least five beams, each having dimensions of 4 x 4 x 14 in. (100 x 100 x 360 mm), with a tolerance of 0.04 in. (1.0 mm) and tested according to the following procedure:

1. Before loading, the beams are moist-cured for 7 days at room temperature not exceeding 72 °F (22 °C); thereafter, the beams are kept moist up to the time of testing and are tested within 24 h;

2. Each beam is first placed on a steel plate 1/2 x 4 x 14 in. (13 x 100 x 360 mm) and simply supported with a center-to-center span of 12 in. (300 mm) and loaded at third points, as shown in Fig. A.1. The load is increased at a rate to induce a deflection of the beam at midspan of between 0.02 and 0.03 in./min (0.50 and 0.75 mm/min), until the first crack in the beam is detected either audibly or visibly, at which time the loading is immediately stopped and removed within 10 s;

3. The steel plate is then removed, and the cracked beam is retested, according to the scheme shown in Fig. A.1. and loaded to failure; and

4. The postcracking strength f_{pcr} is assumed to be equal to the measured peak total load, in Newtons, multiplied by 0.0003. The coefficient 0.0003 is derived from the dimensions of the cross-section of the beam.

APPENDIX B—CRACK CONTROL AND FATIGUE RESISTANCE OF REINFORCEMENT-FREE DECK SLABS

B.1—Purpose and scope

B.1.1 This appendix presents new research information related to the fatigue resistance of reinforced and nonreinforced bridge decks. These new research findings address the issue of transverse cracking discussed in the body of the ITG-3 document.

B.1.2 As noted in the main report, all deck slabs devoid of any reinforcement and crack control bars developed full-depth cracks having a maximum width of approximately 0.04 in. (1 mm). Researchers like Hassan et al. (2002) have expressed concern about the integrity of a slab with full-depth cracks and no embedded bars. Experimental studies reported by Limaye et al. (2002) and Bakht, Mufti, and Tadros (2003) have reassured the same researchers (Hassan and Kawakami 2003) that the presence of even 0.04 in. (1 mm) wide cracks does not affect the safety of the slabs. Engineers are, however, not comfortable with wide cracks, which may appear unsightly and may give a cause for concern to the public. Following the JSCE (1997) recommendations, which are based entirely on aesthetics, it is suggested that for future reinforcement-free deck slabs and deck slabs with only FRP reinforcement, crack widths be limited to 0.02 in. (0.5 mm).

B.1.3 Experimental studies undertaken at Dalhousie University and the Universities of Manitoba and British Columbia (Limaye et al. 2002; Mufti et al. 2002; and Memon, Mufti, and Bakht 2003) have confirmed that: a) full-depth cracks develop in reinforcement-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 initial stages of fatigue damage; and d) the widths of cracks in reinforcement-free deck slabs can be controlled by providing a mesh of nominal GFRP reinforcement.

B.1.4 The studies discussed previously, and reported by Memon, Mufti, and Bakht (2003), involved the fatigue testing of three 6.9 in. (175 mm) thick deck slabs, each on two girders spaced at 6.6 ft. (2 m). Each slab had different crack control bars and transverse confining systems. The first slab contained two orthogonal meshes of 0.6 in. (15 mm) diameter steel reinforcing bars at a spacing of 11.8 in. (300 mm) in each direction. The second slab was transversely confined with external steel straps, and contained one orthogonal crack control mesh of 0.4 in. (10 mm) diameter CFRP transverse bars at a spacing of 7.9 in. (200 mm) and CFRP longitudinal bars at a spacing of 11.8 in. (300 mm); the ratio of the volumes of CFRP bars and concrete was 0.34%. The third slab was also confined transversely by external steel straps, but contained an orthogonal crack control mesh of 0.5 in. (13 mm) diameter GFRP transverse and longitudinal bars at spacing of 5.9 and 9.8 in. (150 and 250 mm), respectively; the ratio of volumes of GFRP bars and concrete was 0.85%. Both the crack control meshes were placed near the bottom of the respective slab, each with a clear cover of 1.6 in. (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 ton (10 to 157 kN). Memon, Mufti, and Bakht (2003) have developed an equation (Eq. (5-3)), that can readily show that the damage induced by all these wheel passes on the deck slab under consideration are replicated in the laboratory by 173,800 cycles of a 25 ton

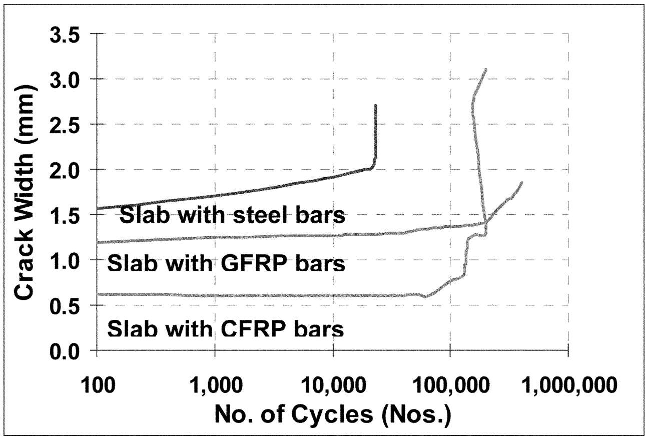

Fig. B.1—Crack width plotted against number of cycles of a 60 ton (588 kN) pulsating load. (Note: 25.4 mm = 1 in.)

(245 kN) load, or by only 24 cycles of a 50 ton (490 kN) load. Each of the three slabs described previously was subjected to successive 1 million cycles of each of 25 and 50 ton loads (245 to 490 kN). After 200,000 cycles of the 25 ton (245 kN) load, the maximum crack widths in deck slabs with steel, CFRP, and GFRP bars were 0.012, 0.015, and 0.012 in. (0.30, 0.38, and 0.30 mm), respectively. The study confirmed that the maximum crack widths in all the three tested slabs, after they were subjected to the equivalent of lifetime damage, were well within the suggested limit of 0.02 in. (0.5 mm). The crack widths in all the three slabs after 1 million cycles of the 25 ton (245 kN) load increased to nearly 0.016 in. (0.41 mm), indicating that the amount of crack control meshes provided in the tested reinforcement-free deck slabs were significantly more than required to keep the crack widths within 0.02 in. (0.5 mm).

After they were subjected to 1 million cycles of the 50 ton (490 kN) loads, the maximum crack widths in the slabs with steel, CFRP, and GFRP bars grew to approximately 0.06, 0.05, and 0.02 in. (1.5, 1.3, and 0.5 mm), respectively. By the end of this sequence of loads, each slab is estimated to have been subjected to 40,000 times the fatigue damage that it is likely to receive during its lifetime. Although at the end of this loading sequence their maximum crack widths were bigger than the proposed limit of 0.02 in. (0.5 mm), the slabs showed no sign of impending failure.

B.2—Comparative results

B.2.1 Each slab tested was subjected to the higher pulsating load of 60 tons, (588 kN) and testing continued to failure. The outcome of this last sequence of testing, presented in Fig. B.1, is instructive in comparing the fatigue resistance of slabs with bars of different materials. The RC slab with steel bars failed in punching shear after 23,162 cycles of the 60 ton (588 kN) load. The deck slab with external straps and CFRP bars failed after 198,863 cycles of the same load. The deck slab with external straps GFRP bars had the best fatigue resistance, failing at 420,684 cycles.

B.2.2 The observation that slabs with steel bars have the worst fatigue resistance and slabs with GFRP bars the best might appear surprising. After some reflection, however, it becomes obvious that the fatigue resistance of a concrete component containing bars with a much higher modulus of elasticity than that of concrete does in fact respond differently to the fatigue resistance of a component, in which both the concrete and embedded bars have similar moduli of elasticity. The interface between a hard inclusion and a soft surrounding material is clearly subjected to higher fatigue damage than would be the case if the hardness of both the inclusion and the surrounding material were nearly the same. The modulus elasticity of a steel bar, being approximately 29 million psi (200 GPa), is more than eight times larger than 3.5 million psi (24 GPa), the modulus of elasticity of normal-weight 5000 psi (35 MPa) concrete. On the other hand, the modulus of elasticity of a GFRP bar, lying between 4.4 and 6.0 million psi (30 and 41 GPa) (ISIS 2001), is much closer to that of concrete.

B.2.3 Experimental studies reported previously have confirmed the validity of the crack-control requirements presented in Section 5.4.4.

B.3—References

Bakht, B.; Mufti, A. A.; and Tadros, G., 2003, "Discussion Paper—An Experimental Investigation of Steel-Free Deck Slabs," *Canadian Journal of Civil Engineering*, V. 30, No. 4, pp. 787-789.

Hassan, A., and Kawakami, M., 2003, "Reply to discussion by Bakht et al. on 'An Experimental Investigation of Steel-free Deck Slabs,'" *Canadian Journal of Civil Engineering*, V. 30, No. 4, pp. 790-791.

Hassan, A.; Kawakami, M.; Nitani, K.; and Yoshioka, T., 2002, "An Experimental Investigation of Steel-free Deck Slabs," *Canadian Journal of Civil Engineering*, V. 29, No. 6, pp. 831-841.

ISIS, 2001, "Reinforcing Concrete Structures with Fibre Reinforced Polymers," ISIS Canada, the University of Manitoba, Winnipeg, Canada.

JSCE, 1997, "Recommendations for Design and Construction of Concrete Structures Using Continuous Fibre Reinforcing Materials," *Concrete Engineering Series* 23, Japan Society of Civil Engineers, Tokyo.

Limaye, V.; Mufti, A. A.; Bakht, B.; and Butt, S. D., 2002, "Steel-free Deck Slabs under Cyclic Loading," *Proceedings of the 6th International Conference on Short & Medium Span Bridges*, Vancouver, British Columbia, Canada, July 31-Aug. 2002, H. Brett, N. Banthia, and P. G. Buckland, eds., Canadian Society for Civil Engineering, Montreal, Quebec, Canada, pp. 1311-1319.

Memon, A. H.; Mufti, A. A.; and Bakht, B., 2003, "Crack Control with GFRP Bars in Steel-Free Concrete Deck Slabs," Proceeding of the CSCE Annual Conference, Moncton, NB. (CD-ROM)

Mufti, A. A.; Memon, A. H.; Bakht, B.; and Banthia, N., 2002 "Fatigue Investigation of Steel-Free Bridge Deck Slabs," *Concrete: Material Science to Application—A Tribute to Surendra P. Shah*, SP-206, American Concrete Institute, Farmington Hills, Mich., pp. 61-70.

Fig. C.1—Crack control GFRP bars in the deck slab of the North Perimeter Bridge in Winnipeg, Canada.

APPENDIX C—FIBERS AND CONTROL OF CRACKS DUE TO VOLUMETRIC CHANGES

The discussion about the need to incorporate fibers in the slab, as presented in Section 3.14, was derived from the provisions of the Canadian Highway Bridge Design Code (2000), which permits reinforcement-free deck slabs to be built without means of controlling fatigue-induced cracks. This code requires that *R*, a quantitative measure of controlling cracks due to volumetric changes in the early life of concrete, should not be less than 0.3.

The current document makes recommendations for the design of reinforcement-free deck slab only if it contains a mesh of GFRP bars. Because the GFRP bars also control cracks due to volumetric changes, it is quite possible that the need for fibers is eliminated. In the spirit of caution, however, the current document recommends that the lower limit of *R* be reduced to 0.15. It is important to note that a reinforcement-free deck slab with two layers of meshes of nominal GFRP bars, and no fibers, was constructed in the summer of 2003 in the Canadian Province of Manitoba. The construction of the deck slab is yet to be reported in the technical literature. A photograph of the GFRP crack control bars in the deck slab is presented in Fig. C.1. The partially studded strap can be seen next to the set of studs on the top flange of the girder. This slab is free of wide cracks.

C.1—References

CHBDC, 2000, "Canadian Highway Bridge Design Code," Canadian Standards Association International, Toronto.

APPENDIX D—RESEARCH NEEDS

D.1—Research goals

The new technology for design of reinforcement-free bridge decks is a significant departure from the philosophy and methodology that has governed the design of concrete deck slabs for over a century. Therefore, it is necessary that the safety factor that is incorporated into this new technology is sufficiently understood to ensure a predictable and safe performance of bridge decks constructed in accordance with this new method. This understanding can be enhanced through the conduct of further research to validate material and structural performance.

D.2—Areas of major R & D focus

D.2.1 *Categories*—Grouped into categories, the research thrusts recommended fall into three areas necessary to achieve a better understanding of the behavior of material and installed structure. The three areas include materials research, the need to instrument installed and new bridges to gather real-time data, and computer modeling and simulation for continued learning of the possible modes of structural behavior.

D.2.2 *Materials*—Investigations should be considered and undertaken where it may prove beneficial for incorporating new mixture proportions and concrete surface treatments for use in the reinforcement-free bridge decks. The mixture proportions should prove useful in providing acid, fire, and heat-resistant cementitious composites.

An example of the types of treatment for consideration is the treatment of cementitious materials with gaseous carbon dioxide to achieve rapid development of strength while reducing permeability tenfold. With further research, it is likely that this process could lead to the development of new materials from novel waste streams and accelerate the development of new and improved concrete admixtures. This could prove particularly useful to the new reinforcement-free bridge design because this new methodology relies solely on the strength of the concrete for its load carrying capacity.

In addition, the effects of infusing polymeric material into the surface of the concrete is another treatment that could prove useful.

Further, investigate the suitability of using carbon-fiber based grids for crack control. Another suggestion is the possibility of examining the behavior of the steel straps draped with fire retardant material. Alternatively, analyze suitability of other materials for use as straps like carbon fiber-reinforced polymer (CFRP), and carbon fiber composite cable (CFCC) tendons and strands.

D.2.3 *Assessment tools and sensors*—The national and world infrastructure are both vast and varied. Age, location, and materials of construction are major variables. The ability to remotely and inexpensively monitor and assess systems integrity and status could provide improved means for service-life prediction and defect detection to ensure operational reliability.

Therefore, efforts should be undertaken to instrument existing and new bridges constructed under the new reinforcement-free bridge design methodology. Such instrumentation can facilitate the retrieval of real-time data about the performance and make-up of concrete in a more rapid, accurate, and complete manner. Specifically, instrumentation should validate the behavior of the steel straps and their capacity to provide the lateral restraint and arching action that the new design methodology relies on.

D.2.4 *Modeling and prediction*—Advances in computational power and methods have removed barriers to predicting the evolution of a concrete structure as it interacts with its environment. Studies should be undertaken using these new advances in computational and modeling methods. The studies should focus on determining the response of the reinforcement-free bridge deck under seismic loadings. In addition, the data gathered from installed instrumentation should be used to validate the arching action and response of the constructed bridges.