New *Canadian Highway Bridge Design Code* design provisions for fibre-reinforced structures[1]

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Abstract: This paper presents a synthesis of the design provisions of the second edition of the *Canadian Highway Bridge Design Code* (CHBDC) for fibre-reinforced structures. New design provisions for applications not covered by the first edition of the CHBDC and the rationale for those that remain unchanged from the first edition are given. Among the new design provisions are those for glass-fibre-reinforced polymer as both primary reinforcement and tendons in concrete; and for the rehabilitation of concrete and timber structures with externally bonded fibre-reinforced-polymer (FRP) systems or near-surface-mounted reinforcement. The provisions for fibre-reinforced concrete deck slabs in the first edition have been reorganized in the second edition to explicitly include deck slabs of both cast-in-place and precast construction and are now referred to as externally restrained deck slabs, whereas deck slabs containing internal FRP reinforcement are referred to as internally restrained deck slabs. Resistance factors in the second edition have been recast from those in the first edition and depend on the condition of use, with a further distinction made between factory- and field-produced FRP. In the second edition, the deformability requirements for FRP-reinforced and FRP-prestressed concrete beams and slabs of the first edition have been split into three subclauses covering the design for deformability, minimum flexural resistance, and crack-control reinforcement. The effect of sustained loads on the strength of FRPs is accounted for in the second edition by limits on stresses in FRP at the serviceability limit state.

Key words: beams, bridges, concrete, decks, fibre-reinforced-polymer reinforcement, fibre-reinforced-polymer sheets, prestressing, repair, strengthening, wood.

Résumé : Cet article présente une synthèse des dispositions de conception des structures renforcées de fibres telles énoncées dans la seconde édition du Code canadien sur le calcul des ponts routiers. Il aborde les nouvelles dispositions de conception pour les utilisations non couvertes dans la première édition et le raisonnement derrière les changements apportés. Parmi ces nouvelles dispositions se trouvent celles concernant les polymères renforcés de fibres de verre (« GFRP ») comme renforcement primaire et barres dans le béton ainsi que la réhabilitation des structures de béton et de bois d'œuvre au moyen de systèmes en polymère renforcé de fibres (« FRP ») externes ou de renforcement installé près de la surface (« NSMR »). Les dispositions concernant les dalles de tablier en béton renforcé de fibres (« FRC ») dans la première édition ont été réorganisées dans la seconde afin d'inclure explicitement les dalles de tablier des constructions coulées en place et précontraintes et ces dispositions sont maintenant regroupées sous les termes « dalles de tablier à encastrement externe » alors que les dalles de tablier contenant des renforcements « FRC » internes sont regroupées sous l'appellation « dalles de tablier à encastrement interne. » Les facteurs de résistance dans la seconde édition repris à partir de ceux de la première édition et dépendent de la condition d'utilisation; les « FRP » produits en

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usine et sur place ne sont plus traités ensemble. Les exigences de déformabilité stipulés dans la première édition concernant les poutres et les dalles en béton renforcé de (« FRP ») et précontraint ont été divisées dans trois sous-dispositions pour la seconde édition; elles couvrent la conception en fonction de la déformabilité, la résistance minimale en flexion, et le renforcement pour le contrôle des fissurations. L'effet des charges soutenues sur la résistance des (« FRP ») est considéré dans la seconde édition par l'imposition de limites sur les contraintes dans les (« FRP ») à l'état limite d'utilisation.

Mots-clés : poutres, ponts, béton, tabliers, armature par polymères renforcés de fibres, feuilles de polymères renforcés de fibres, précontrainte, réparation, renforcement, bois.

[Traduit par la Rédaction]

Introduction

The first edition of the *Canadian Highway Bridge Design Code* (CHBDC), published in 2000, contained design provisions for some fibre-reinforced structural components; the provisions were limited to only those applications in which the Technical Subcommittee (TSC) responsible for the provisions had confidence or for which there were documents substantiating their performance that the TSC could access. In particular, the provisions for fibre-reinforced structures in the first edition of the CHBDC, drafted mainly by 1997, were limited to fully or partially prestressed concrete beams and slabs; non-prestressed concrete beams, slabs, and deck slabs; fibre-reinforced-concrete (FRC) deck slabs; stressed wood decks; and barrier walls.

Since the first set of design provisions for fibre-reinforced structures was written, considerable research has been conducted both in Canada and elsewhere, requiring not only the revision of existing design provisions, but also new design provisions for applications not covered by the first edition of the CHBDC.

A new TSC, similar to the TSC for the first edition, was formed with members drawn from Canada and elsewhere. In addition to Canadian experts, experts from the United States, Japan, and Sweden were included in the TSC responsible for formulating the revised design provisions, which were approved in principle and expected to be published in the second edition of the CHBDC by early 2006 (see CSA 2006). The new and revised design provisions are described in the following sections, along with their rationale. It is noted that most of the subsequent headings of the paper conform to the headings of the fibre-reinforced structures section of the second edition of the CHBDC.

Durability

For bars and grids made of fibre-reinforced polymer (FRP), when used as primary reinforcement in concrete, for FRP tendons, and for FRP systems used in strengthening of concrete and timber components, the matrices are required to comprise only thermosetting polymers, except that thermoplastic polymers with proven durability may also be used with approval. The term "approval" is defined in the CHBDC as approval in writing by the regulatory authority. Thermosetting polymers are preferred over thermoplastic polymers because of the lack of experience in the use of thermoplastics in civil structural applications. Some thermoplastic polymers are indeed highly durable, but currently they are very expensive.

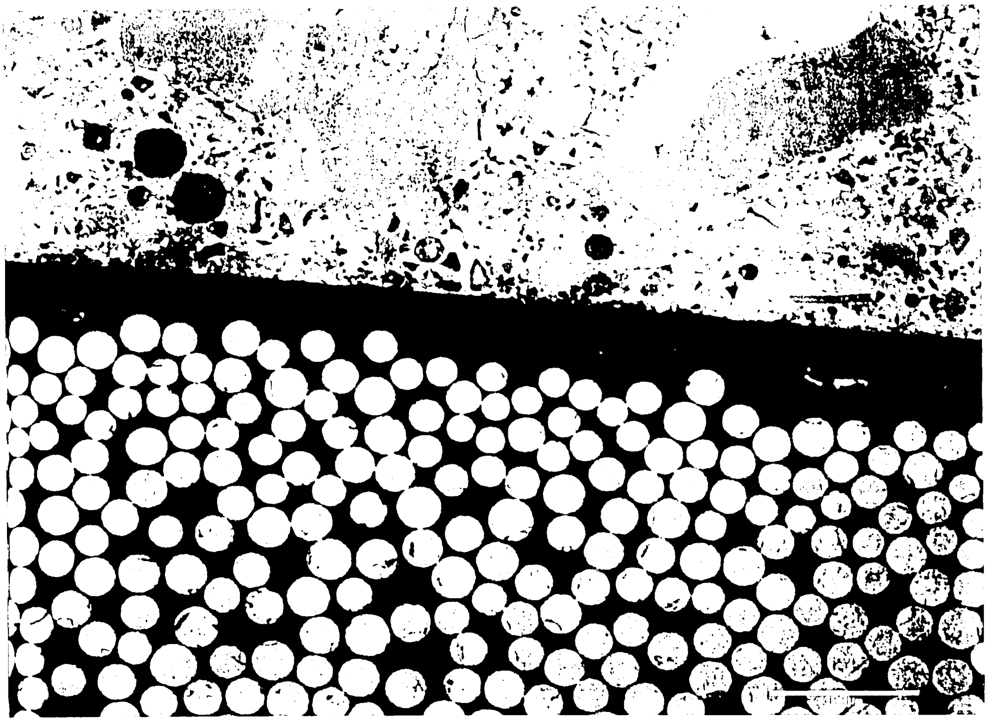
The code also requires that matrices and adhesives of FRP systems have a wet glass transition temperature (T_{gw}) of less than 20 °C plus the maximum daily mean temperature as specified elsewhere in the code.

The transition temperature (T_g) denotes the transition between the elastic and viscoelastic response of a polymer and, consequently, that of its FRP. Water uptake by polymers and their composites causes plasticization in the short term and hydrolysis in the long term, thus inducing higher levels of molecular mobility, resulting in a decrease in the glass transition temperature. An increase in moisture content results in a decrease in T_g, with the maximum decrease being seen, in the absence of other causes of deterioration, at the saturation level for moisture uptake in the polymer or its FRP. The glass transition temperature associated with moisture uptake, denoted in the code by T_{gw}, is lower than T_g. Although T_g is often taken to represent the effective operational temperature limit for the polymer and its composite, it is advisable to use T_{gw} as the effective limit in cases where water uptake is likely to be a result of normal exposure. ASTM standards D3418 (ASTM 2003) and D4065 (ASTM 2001) should be used to determine T_g, and T_{gw} should be measured at saturation.

Until recently, experts were not in full agreement about whether glass-fibre-reinforced polymer (GFRP) is stable in the alkaline environment of concrete. In a recent study (Mufti et al. 2005*a*), nine cores were taken from each of five outdoor concrete bridge structures across Canada; these structures were reinforced with GFRP bars and had been built during the last 6–8 years. Three cores from each bridge were given to each of three teams of material scientists and experts in durability for microscopic and chemical analyses. An example from this study is presented in Fig. 1, which shows a micrograph of GFRP and surrounding concrete removed from an 8-year-old structure. It can be seen that the glass fibres and the GFRP–concrete interface are intact. The findings from the analyses reported by Mufti et al. (2005*a*) have confirmed that the concerns about the durability of GFRP in alkaline concrete, based on simulated laboratory studies in alkaline solutions, are unfounded. It is mainly on the basis of this study that GFRP is now permitted as both primary reinforcement and tendons in concrete.

The other reason for permitting GFRP in these applications is the limit placed on GFRP stresses at the serviceability limit state (SLS) (Karbhari 2003; Karbhari et al. 2003; Helbling and Karbhari 2004). The maximum SLS stress permitted in GFRP is now 25% of its ultimate strength. Other studies on the durability of GFRP in concrete include those of Karbhari et al. (2001), Dejke (2001), Benmokrane et al.

Fig. 1. Micrograph of GFRP-concrete interface in a specimen removed from an 8-year-old structure.



(2002), Greenwood (2002), Karbhari (2004), Nkurunziza (2004), and Nkurunziza et al. (2005).

Permitting GFRP as the main reinforcement is already affecting bridge construction and attitude toward the use of the new materials. For example, the Ministry of Transportation of Ontario (Klement and Aly 1997), which was initially reluctant to use GFRP in its bridges, is now beginning to use it as the top reinforcement in deck slabs of bridges.

Cover to reinforcement

Some researchers have expressed the opinion that because of their high transverse coefficient of thermal expansion, FRP bars should have a larger cover than specified in the CHBDC. Extensive analysis of cores removed from GFRP-reinforced structures has confirmed that structures with small covers show no cracks despite being in service for 6–8 years (Mufti et al. 2005*c*). As discussed by Bakht et al. (2004), the reason for the absence of cracks above FRP bars in concrete structures might be that during the setting of concrete, the FRP bars are "locked" into concrete at a high temperature. The studies reported by Mufti et al. (2005*b*) also confirmed that no damage due to freeze–thaw cycles was experienced by the GFRP-reinforced structures.

Vogel (2005) examined a number of concrete beams prestressed with GFRP and carbon-fibre-reinforced-polymer (CFRP) tendons with minimum cover and subjected to the thermal gradients expected in Canada; he noted that "the flexural specimens regularly monitored during the experimental program with a handheld microscope never revealed the presence of cracks within the cover". Aguiniga (2003) made similar observations.

In light of the above discussion, the requirements for cover to FRP reinforcement have remained unchanged. It is recalled that the minimum clear cover to FRP bars is 35 mm, with a construction tolerance of ±10 mm.

Table 1. Values of ϕ_{FRP} for pultruded fibre-reinforced polymers and aramid fibre ropes.

Application	ϕ_{FRP}
AFRP reinforcement in concrete and NSMR	0.60
AFRP in externally bonded applications	0.50
AFRP and aramid fibre rope tendons for concrete and timber components	0.55
CFRP reinforcement in concrete	0.75
CFRP in externally bonded applications and NSMR	0.75
CFRP tendons	0.75
GFRP reinforcement in concrete	0.50
GFRP in externally bonded applications and NSMR	0.65
GFRP tendons for concrete components	0.50
GFRP tendons for timber decks	0.65

Note: AFRP, aramid-fibre-reinforced polymer; CFRP, carbon-fibre-reinforced polymer; GFRP, glass-fibre-reinforced polymer; NSMR, near-surface mounted reinforcement.

Resistance factors

Unlike the first edition of the CHBDC, the second edition specifies that the resistance factors for FRPs depend on the condition of use. The resistance factors (ϕ_{FRP}) for pultruded FRPs and aramid fibre rope are as listed in Table 1. The ϕ_{FRP} for an FRP made in the field is specified to be 0.75 times the corresponding value in Table 1, whereas that for an FRP made in controlled factory processes is 0.85 times the corresponding value in Table 1. The ϕ_{FRP} for bent GFRP bars resisting the effect of vehicle impact load is

Table 2. Examples of variability of tensile strengths for CFRP and AFRP bars (Machida).[a]

FRP	No. of specimens	Size (mm)	Mean failure load (kN)	Standard deviation (kN)	Coefficient of variation (%)
CFRP	507	12.5 dia.	169.4	8.70	5.1
CFRP	60	9.8 dia.	182.4	3.97	2.2
AFRP	132	20 × 3 bar	69.2	2.80	4.1
AFRP	24	8 dia.	102.2	1.60	1.6

[a]A. Machida, private communication, 1996.

to be taken as 0.75, irrespective of the method of manufacture.

Extensive tests on some Japanese FRP bars have shown that the variability of the tensile strength of factory-produced FRP bars subjected to the same environmental exposure is not large. Indeed, this variability is comparable to that of steel bars. Results from the Japanese study (Machida[4]) are summarized in Table 2, in which it can be seen that the coefficient of variation (COV) of the failure load ranges between 1.6% and 5.1%. By comparison, the COV of the strength of pultruded CFRP strips used in external rehabilitation is between 3% and 6%, which is comparable to, and even somewhat better than, that of steel plates (Karbhari 2004). However, the strength variability of FRPs produced in the field for purposes of external rehabilitation, as for example by the hand lay-up process in surface-mounted FRP sheets, is larger than the variability of factory-produced FRPs. Karbhari (2004) showed that the COV of field-produced FRPs is between 12% and 14%.

The three values of the resistance factor (ϕ_{FRP}) in the first edition of the CHBDC (0.85, 0.85, and 0.75 for aramid-fibre-reinforced polymer (AFRP), CFRP, and GFRP, respectively) were mainly drawn from Japanese recommendations for design with FRPs (JSCE 1997). Since the publication of the first edition of the CHBDC it has been recognized that the variability of the strength of FRPs is affected more by environmental exposure than by geometric properties and stress level. It is for this reason that experts in the structural use of FRP are now suggesting that the resistance factors for FRPs be specified as products of a "material factor" and an "environmental factor" (ACI Committee 440 2002; Karbhari 2000). The material factors for factory- and field-produced FRPs are denoted here by ϕ_{pul} and ϕ_{hl}, respectively; the environmental factor, by C_e.

Recent studies have confirmed that neither ϕ_{pul} nor ϕ_{hl} is affected by the type of fibre in an FRP component. Drawing from ACI Committee 440 (2002) and experimental studies conducted in California (Karbhari 2003), the TSC conservatively assumed the values of ϕ_{pul} and ϕ_{hl} to be 0.8 and 0.6, respectively, for all FRPs. The TSC arrived at values for C_e by first fixing its value for CFRP, which is the FRP least affected by environmental exposure. For CFRP, C_e could have been fixed at 1.00, but to account for minor statistical variability, a value of 0.95 was adopted.

All FRP applications, whether permitted in the second edition of the CHBDC or not, are listed in Table 3, along with the values of ϕ_{pul}, ϕ_{hl}, and C_e. The values of C_e were arrived at by scrutinizing the literature (Kaiser and Karbhari 2003; Karbhari 2003, 2004; Karbhari and Vasquez 2003) and by comparing their performance with that of CFRP. Table 3 also lists the product of material and environmental factors for both factory- and field-produced FRPs. To be consistent with the rest of the code, it was decided not to specify the material and environmental factors. Instead, the product of the two factors was specified to be the resistance factor. The applicable values of the "final" resistance factors are shown in bold in Table 3.

In the first edition of the CHBDC, the effect of stress level on the long-term strength of FRPs at the ultimate limit state (ULS) was addressed through the factor *F*, the values of which depended upon *R*, the ratio of stresses due to dead and live loads, so the effective resistance factor could be regarded as the product of *F* and ϕ. This prescription has now been replaced by placing different stress limits at the SLS for different FRPs, as described later.

Notwithstanding a different and more logical method of arriving at the resistance factors, it is necessary to compare the factors specified in the second edition of the CHBDC with those in the first edition. Table 4 provides such a comparison for typical values of *R* for different applications. It can be seen in this table that the ϕ_{FRP} values specified in Table 1 are of the same order of magnitude as the corresponding products of *F* and ϕ specified in the first edition of the code.

Note that GFRP tendons were not permitted in the first edition of the CHBDC.

Protective measures

The CHBDC requires that exposed tendons and FRP strengthening systems that are deemed to be susceptible to damage by ultraviolet rays or moisture be protected accordingly. Also, where the externally bonded FRPs are susceptible to impact damage from vehicles, ice, and debris, consideration should be given to protecting the FRP systems. According to NCHRP (2004), "Protective coating is applied for aesthetic appeal or protection against impact, fire, ultra-violet and chemical exposure, moisture, vandalism. FRP systems are usually durable to weather conditions, seawater, and many acids and chemicals. Mortar finish can provide protection against impact or fire. Weather-resistant paint of the family of urethane or fluorine or epoxide can provide protection against direct sunlight. ... The engineer

[4]Machida, A. 1996. Designing concrete structures with continuous fiber reinforcing material. Keynote paper, Proceedings of the 1st International Conference on Composites in Infrastructure, Tucson, Arizona. Unpublished.

Table 3. Calculation of resistance factors.

Application	ϕ_{pul}	ϕ_{hl}	C_e	$\phi_{pul}C_e$	$\phi_{hl}C_e$	ϕ_{FRP} in 2nd edition of CHBDC[a]	
						For factory-produced products	For field-produced products
AFRP inside concrete remaining wet after setting	0.8	0.6	0.50	0.40	0.30	N/P	N/A
AFRP surface-mounted on concrete and exposed to moisture	0.8	0.6	0.50	0.40	0.30	N/A	N/P
AFRP or aramid fibre rope as tendon exposed to moisture	0.8	0.6	0.50	0.40	0.30	N/P	N/A
AFRP inside concrete remaining dry after setting	0.8	0.6	0.75	0.60	0.45	**0.60**	N/A
AFRP surface-mounted on concrete or timber and not exposed to moisture and UV light	0.8	0.6	0.90	0.72	0.54	0.70[b]	**0.55**
AFRP or aramid rope as tendon not exposed to moisture	0.8	0.6	0.70	0.56	0.42	**0.55**	**N/A**
CFRP inside concrete or near-surface mounted	0.8	0.6	0.95	0.76	0.57	**0.75**	N/A
CFRP tendon	0.8	0.6	0.95	0.76	0.57	**0.75**	**N/A**
GFRP inside concrete remaining wet after setting	0.8	0.6	0.50	0.40	0.30	N/P	N/A
GFRP inside concrete remaining dry after setting	0.8	0.6	0.60	0.48	0.36	**0.50**	N/A
GFRP surface-mounted on concrete and not exposed to moisture	0.8	0.6	0.80	0.64	0.48	0.65[b]	**0.50**
GFRP surface-mounted on concrete and exposed to moisture	0.8	0.6	0.70	0.56	0.42	N/A	**N/P**
GFRP tendon in concrete	0.8	0.6	0.60	0.48	0.36	**0.50**	N/A
GFRP tendon for stressed wood decks	0.8	0.6	0.80	0.64	0.48	**0.65**	N/A

Note: Values in bold are the "final" resistance factors. CHBDC, *Canadian Highway Bridge Design Code* (CSA 2006).
[a]N/P and N/A, respectively, indicate "not permitted by the code" and "not applicable".
[b]not specified in CHBDC.

Table 4. Comparison of resistance factors specified in the second edition of the CHBDC with the product of F and resistance factors in the first edition.

Application	1st edition of CHBDC				
	ϕ	Typical value of R	F	ϕF	ϕ_{FRP}
AFRP reinforcement in concrete and NSMR	N/S	N/S	N/S	N/S	0.60
AFRP in externally bonded applications	N/S	N/S	N/S	N/S	0.50
AFRP and aramid fibre rope tendons for concrete components	0.85	2	0.5	0.425	0.55
AFRP and aramid fibre rope tendons for timber components	0.85	0.5	1.0	0.850	0.55
CFRP reinforcement in concrete	0.85	1	0.9	0.765	0.75
CFRP in externally bonded applications and NSMR	N/S	N/S	N/S	N/S	0.75
CFRP tendons	0.85	2	0.9	0.765	0.75
GFRP reinforcement in concrete	N/S	N/S	N/S	N/S	0.50
GFRP in externally bonded applications and NSMR	N/S	N/S	N/S	N/S	0.65
GFRP tendons for concrete components	0.75	2	0.8	0.600	0.50
GFRP tendons for timber decks	0.75	0.5	1.0	0.750	0.65

Note: CHBDC, *Canadian Highway Bridge Design Code* (CSA 2006); N/S, not specified in the code; NSMR, near-surface mounted reinforcement.

may request that the contractor provide a sample mock-up of the coating system for about a 0.1 m^2 area." It should be noted that the protective coatings might need to be renewed because of ageing or damage.

The second edition of the CHBDC forbids direct contact between CFRP and metals, as contact between carbon fibres and metals can lead to galvanic corrosion. An isolation layer of an appropriate polymer could, for example, avoid the contact between carbon fibres and steel.

Externally restrained deck slabs

To be consistent with the empirical provisions for the design of steel-reinforced deck slabs, the provisions for FRC deck slabs in the first edition have been reorganized to explicitly include deck slabs of both cast-in-place and precast construction. The "FRC deck slabs" of the first edition, has been changed to "externally restrained deck slabs" to recognize that the code now permits deck slabs without fibres, provided they contain two assemblies of FRP bars (to be discussed later). An externally restrained deck slab is defined as a deck slab that relies on external confinement, such as that provided by steel straps, for its strength.

The clause for the design of externally restrained deck slabs is divided into four subclauses to cover design provisions (*a*) of a general nature; (*b*) for full-depth cast-in-place deck slabs; (*c*) for cast-in-place deck slabs on stay-in-place formwork; and (*d*) for full-depth precast deck slabs.

A major change in the design provisions for an externally restrained deck slab is that the crack-control mesh, which was optional in the first edition of the code, is now mandatory. An externally restrained deck slab is now required to be provided with a crack-control orthogonal assembly of GFRP bars, placed near the bottom of the slab, with the cross-sectional area of the GFRP bars being at least $0.0015t$ mm²/mm in each direction, where t is the thickness of the deck slab. In addition, the spacing of transverse and longitudinal crack-control bars should not be more than 300 mm. The cross-sectional area and spacing of the specified crack-control mesh are based on recent experimental fatigue studies (Limaye et al. 2002; Mufti et al. 2002; Memon et al. 2003).

Similar to the first edition, the second edition of the CHBDC requires that the supporting beams be connected with transverse diaphragms, or cross-frames, at a spacing of not more than 8000 mm. The commentary to the code explains that the requirement for transverse diaphragms was introduced because the transverse moments induced by live load due to load transfer between the beams can, in some cases, be negative. If rigorous analysis shows that such negative moments are not induced, then the requirement for transverse diaphragms may be waived. When such moments exist, the requirement for transverse diaphragms may also be waived if adequate reinforcement is provided in the deck slab for the negative transverse moments. To illustrate this aspect, a bridge with the particulars shown in Table 5 was analyzed by the orthotropic plate program PLATO (Bakht et al. 2002*a*) for transverse negative moments due to an eccentrically placed CHBDC truck.

The maximum intensity of transverse moments is induced when the vehicle is closest to a longitudinal free edge of the deck slab. Under this loading, the maximum intensity of transverse moments in the deck slab is –6.31 kN·m/m (Fig. 2). If the vehicle loads are multiplied by a live load factor of 1.70 and by a factor 1.25 to account for the dynamic load allowance (I), the maximum intensity of transverse moment is only 13.4 kN·m/m. The very small intensity of the transverse negative moment can be accommodated by even the nominal crack-control reinforcement placed at the mid-depth of a 180 mm thick deck slab.

As in the first edition, the externally restrained deck slabs in the second edition are required to have edge beams with details approved by the code. Details for two of these edge beams are shown in Figs. 3*a* and 3*b*, in which A_s is the cross-sectional area of steel reinforcement; and A_{FRP} is that of the FRP bars. The CHBDC also permits two other edge

Table 5. Paraneters of bridge analyzed by the orthotropic plate program PLATO.

Variable	Value
Simply supported span (m)	37.00
Width (m)	21.25
Spacing of AASHTO–PCI prestressed girders BT-72 (m)	2.125
No. of girders	11
Cantilever overhang on each side (m)	1.095
Deck slab thickness (mm)	180
Composite I of girder (m⁴)	0.64
No. of intermediate diaphragms	0

Fig. 2. Transverse negative moments in a deck slab.

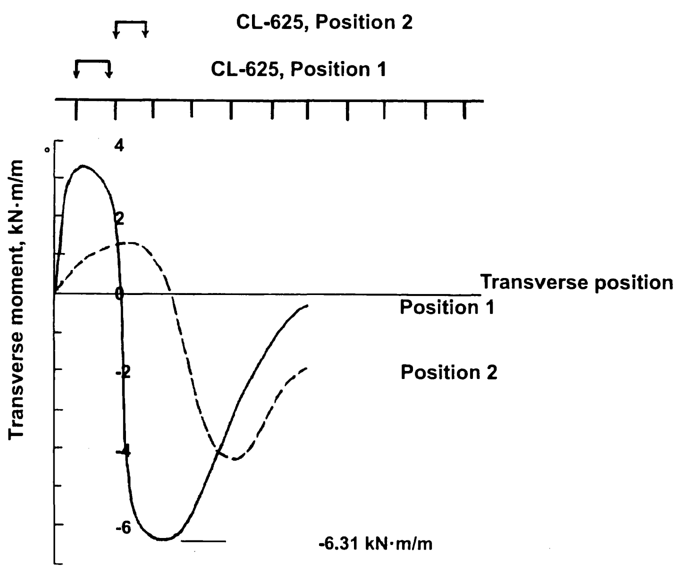
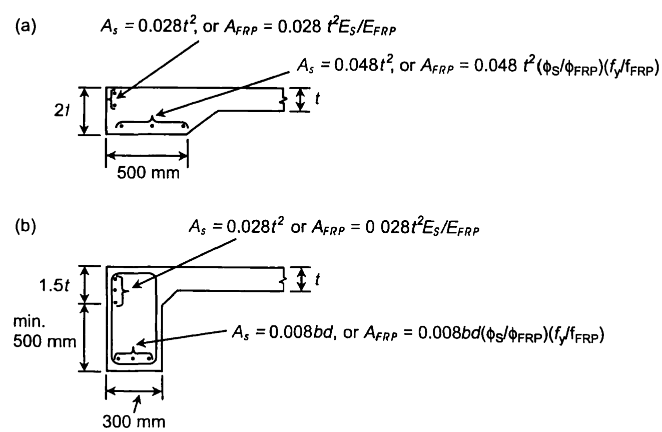
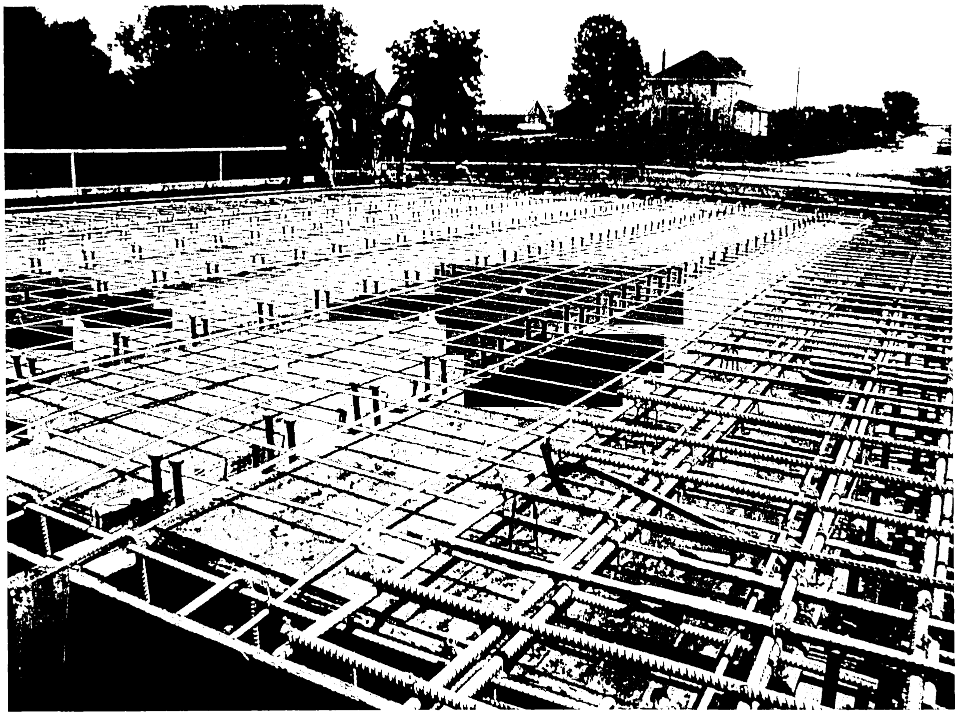


Fig. 3. Permitted edge beam details: (*a*) thickened slab and (*b*) concrete beam.



beam details. In one detail, a 300 mm length of the deck slab is thickened to 1.5 times the deck slab thickness (t), and the thickened slab is connected to a steel C200×21 channel, with its web in contact with the thickened slab and connected to it through two 22 mm diameter studs at a spacing of 300 mm. In the other permitted detail, a 300 mm length of

Fig. 4. Crack-control assembly of GFRP bars before casting of deck slab in Tama County, Iowa, USA.



the slab is similarly thickened and is connected with a steel W200×52 section by means of two 22 mm diameter studs at a spacing of 300 mm.

The draft CHBDC provisions for externally restrained deck slabs have already been used for design of a deck slab in Canada (Klowak et al. 2004) and for another in Tama County, Iowa, USA. The GFRP crack-control grid in the latter can be seen in Fig. 4 before the casting of the slab.

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

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

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

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

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

(*e*) When the deck slab is restrained by straps, the straps and their connections are similar in design to full-depth cast-in-place deck slabs.

(*f*) When the deck slab is restrained by formwork, the concept has been verified by tests on full-scale models. In addition, the cross-sectional area of the formwork (mm^2/mm) across a section parallel to the beams is at least A_f, as determined with the following equation:

$$A_f = (F_s S^2 / Et) \tag{1}$$

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

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

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

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

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

(*k*) The cast-in-place concrete is provided with a crack-control orthogonal assembly of GFRP bars, placed at its mid-depth, and the cross-sectional area of the GFRP bars in each direction is not less than $0.0015t_c$ mm^2/mm, where t_c is the depth of the cast-in-place concrete. In addition, the spacing of the transverse and longitudinal crack-control bars is not more than 300 mm.

The design provisions for externally restrained deck slabs with cast-in-place construction on stay-in-place formwork were prompted by the work of Bakht and Chu (1997) on reinforced concrete stay-in-place forms and that of Bakht et al.

Fig. 5. Cross section of the model (dimensions in millimetres).

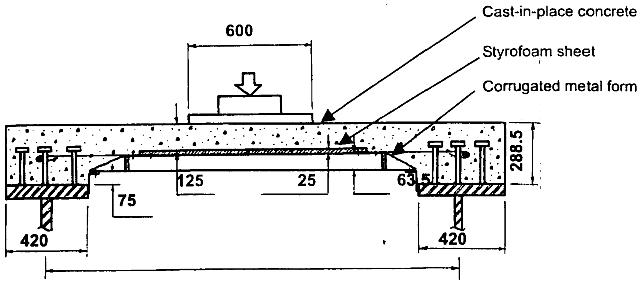
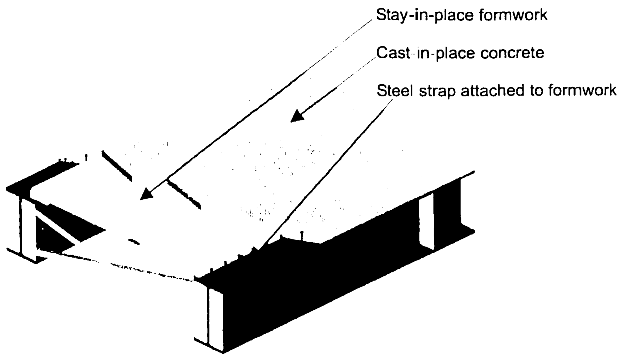


Fig. 6. Precast concrete stay-in-place formwork with cast-in-place concrete.



(2002*b*), who tested a 125 mm thick concrete slab on corrugated, thin metal forms. As shown in Fig. 5, the cast-in-place concrete was separated from the metal formwork by a 25 mm thick Styrofoam sheet. The metal form was connected to the top flanges of the girders by means of thin metal strips bonded to the ridges of the formwork at one end and to a longitudinal steel bar lying between the two rows of shear connectors at the other. As noted by Bakht et al. (2002*b*), the deck slab on the metal formwork failed under a monotonically increasing central patch load of nearly 452 kN; this load is 2.2 times larger than the factored failure load required by the CHBDC.

The two experimental studies, referenced above, confirmed that deck slabs of girder bridges could be restrained transversely by means of stay-in-place formwork, provided the formwork is tied effectively to the girders.

Bowen (2005) reported the development of a unique precast concrete formwork that contains the steel straps necessary for confining the deck slab externally. The formwork contains a GFRP crack-control mesh. The concept, illustrated in Fig. 6, was tested for a girder spacing of 2 m. Under a central patch load, the average failure load for the deck slab was 606 kN, thus confirming the validity of the concept.

The design of an externally restrained deck slab with full-depth precast concrete construction requires approval. Although externally restrained full-depth precast panels have already been used in a two-girder forestry bridge (Sargent et al. 1999), the TSC felt that more research is needed, especially about means of connecting the panels to the supporting beams, before definitive design provisions can be formulated for these deck slabs. However, to leave the door open for future innovations, this clause permits externally restrained full-depth precast deck slabs, but only after approval.

Internally restrained cast-in-place deck slabs

Deck slabs containing internal FRP reinforcement for strength are referred to in the second edition of the CHBDC as "internally restrained deck slabs". The code has provisions for designing these slabs either by taking account of their arching action or by assuming them to be acting in flexure. It is noted that the former method is also referred to as the "empirical method".

When a cast-in-place deck slab with FRP reinforcement is designed for the arching action, it is required to be designed by the same empirical method as specified for steel-reinforced deck slabs, except that the following conditions have to be satisfied in lieu of those for steel-reinforced deck slabs:

(*a*) The deck slab contains two orthogonal assemblies of FRP bars, and the clear distance between the top and bottom transverse bars is a minimum of 55 mm.

(*b*) For the transverse FRP bars in the bottom assembly, the minimum cross-sectional area (mm^2/mm) is $500d_s/E_{FRP}$, where d_s is the distance (mm) from the top of the slab to the centroid of the bottom transverse FRP bars; and E_{FRP} is the mean modulus of elasticity (MPa) of FRP bars, plates, sheets, and tendons.

(*c*) The longitudinal bars in the bottom assembly and both the transverse and the longitudinal bars in the top assembly are of GFRP, each with a minimum reinforcement ratio (ρ) of 0.0035.

(*d*) The edge-stiffening details are the same as permitted for full-depth cast-in-place deck slabs discussed earlier in reference to Figs. 3*a* and 3*b*.

The second edition of the CHBDC also permits the use of the flexural method for the design of deck slabs with internal FRP reinforcement. A number of deck slabs with FRP bars have been designed for flexure (Benmokrane et al. 2004; El-Salakawy and Benmokrane 2004; El-Gamal et al. 2005). However, as noted by Bakht et al. (2004), such slabs have more FRP reinforcement than slabs designed for arching.

Concrete beams and slabs

Deformability

In light of recent research findings, the deformability requirements of the first edition for FRP-reinforced and FRP-prestressed concrete beams and slabs have been split into three subclauses, described in the following:

- *Design for deformability* — The limit on c/d, (where c is the distance of the neutral axis from the compression face) specified in the first edition as being between 0.25 and 0.50, is replaced by requirements for minimum flexural resistance and crack-control reinforcement.
- *Minimum flexural resistance* — The factored resistance (M_r) is required to be at least 50% greater than the cracking moment (M_{cr}). This requirement may be waived if the factored resistance (M_r) is at least 50% greater than the factored moment (M_f). If the ULS design of the section is governed by FRP rupture, then M_r is required to be greater than $1.5M_f$.

The above requirement ensures that critical sections contain sufficient flexural reinforcement for there to be adequate reserve strength after the formation of initial cracks in concrete or the rupture of FRP. The rupture of FRP is allowed because some FRP bars have a very low modulus of elasticity, as a result of which the amount of reinforcement required for a T-section would be very large if it were controlled by compression failure.

- *Crack-control reinforcement* — When the maximum tensile strain in FRP reinforcement under full service loads exceeds 0.0015, cross sections of the component in maximum positive and negative moment regions are required to be so proportioned that the crack width does not exceed 0.5 mm for members subject to aggressive environments and 0.7 mm for other members, where the crack width (w_{cr}) is given by

$$[2]\qquad w_{cr} = 2\frac{f_{FRP}}{E_{FRP}}\frac{h_2}{h_1}k_b\sqrt{d_c^2+\left(\frac{s}{2}\right)^2}$$

where f_{FRP} is the stress (MPa) in the tension FRP reinforcement; h_1 and h_2 are the distances (mm) from the centroid of tension reinforcement and the extreme flexural tension surface, respectively, to the neutral axis; d_c is the distance (mm) from the centroid of the tension reinforcement to the extreme tension surface of concrete; s is the spacing (mm) of shear or tensile reinforcement; and other notation is either defined earlier or discussed below.

The value of the bond parameter (k_b) in eq. [2] is required to be determined experimentally, but in the absence of test data it may be taken as 0.8 for sand-coated and 1.0 for deformed FRP bars (El-Salakawy and Benmokrane 2004). In the calculation of d_c, the clear cover is assumed to be not greater than 50 mm. As noted by the Task Group on Crack Width (ACI Committee 440 2004), a modified version of a crack width equation by Frosch (1999) was used, and the bond parameter (k_b) was recalibrated. The value of 2 in the equation was used for predicting the maximum crack width. A value of 1.5 can be used for the mean crack width and 1.0 for the minimum crack width. The value of k_b ranges from 0.60 to 1.72, with a mean value of 1.10 and a standard deviation of 0.3.

Non-prestressed reinforcement

In the first edition, the effect of the sustained loads on the strength of FRPs was accounted for by a factor F, which depended on the ratio of stresses due to factored dead loads to stresses due to factored live loads. In light of current knowledge, this prescription has been replaced by limits on stresses in FRP induced at the SLS. It is required that maximum stress in FRP bars or grids under loads at SLS be not more than $F_{SLS}f_{FRP_u}$, where F_{SLS} is as given in Table 6.

Stress limitations for tendons

Some case histories have confirmed that under high levels ($\geq 0.55f_{FRP_u}$) of sustained stress, GFRP is prone to stress rupture and creep rupture. Other studies have confirmed that such damage mechanisms do not occur if the sustained stress level is $\leq 0.25f_{FRP_u}$. In light of this information, the following provisions have been made for stress limits for tendons.

Table 6. Values of F_{SLS}.

	F_{SLS}
AFRP	0.35
CFRP	0.65
GFRP	0.25

Table 7. Maximum permissible stresses in FRP tendons at jacking and transfer for concrete beams and slabs for both pretensioning and posttensioning systems.

Tendon	At jacking	At transfer
AFRP	$0.40f_{FRP_u}$	$0.35f_{FRP_u}$
CFRP	$0.70f_{FRP_u}$	$0.65f_{FRP_u}$
GFRP	$0.30f_{FRP_u}$	$0.25f_{FRP_u}$

For straight tendons, the maximum stresses at jacking and transfer are not permitted to exceed the values given in Table 7. For curved tendons, the maximum stresses at jacking and transfer are required to be those given in Table 7 reduced by an amount determined from tests. In addition, FRP tendons are required to be stressed to provide a minimum effective prestress of 75% of the stresses at transfer.

The maximum SLS stresses after all prestress losses are not to exceed the F_{SLS} values given in Table 6.

The maximum stress (f_{ps}) in the tendons under factored loads at ULS, computed using a method based on strain compatibility, is not permitted to exceed $\phi_{FRP}f_{FRPu}$, where the resistance factor ϕ_{FRP} is as listed in Table 1; and ϕ_{FRP} is the specified tensile strength (MPa) of an FRP bar, grid, plate, sheet, or tendon or of an aramid fibre rope.

Design for shear

For concrete beams reinforced with steel or FRP longitudinal reinforcement and steel or FRP stirrups, the factored shear resistance (V_r) is required to be computed from

$$[3]\qquad V_r = V_c + V_{st} + V_p$$

where V_c, V_{st}, and V_p are factored shear resistance provided by concrete, stirrups, and tendons, if present, respectively. The shear contribution V_{st} is denoted by V_s if the stirrups are of steel and by V_{FRP} if they are of FRP.

The contributions of V_c, V_s, and V_p are calculated according to the standard practice prescribed in the concrete section of the second edition of the CHBDC, except as follows:

(*a*) The following equation is used for calculating V_c:

$$[4]\qquad V_c = 2.5\beta\phi_c f_{cr} b_v d_{long}\sqrt{E_{long}/E_s}$$

where β is the angle of inclination (°) of the internal or external transverse reinforcement to the longitudinal axis of a member, ϕ_c is the resistance factor for concrete, f_{cr} is the cracking strength (MPa) of concrete, b_v is the effective width (mm) of the web within the depth d_{long}, E_{long} is the modulus of elasticity (MPa) of FRP or steel longitudinal reinforcement, and E_s is is the modulus of elasticity (MPa) of steel.

(*b*) The following equation is used for calculating the longitudinal strain ε_x:

$$[5] \quad \varepsilon_x = \frac{(M_f / d_{long}) + V_f - V_p + 0.5N_f - (A_{FRP} f_{po} \text{ or } A_p f_{po})}{2[E_s A_s + (E_p A_p \text{ or } E_{FRP} A_{FRP})]} \leq 0.003$$

where M_f is the factored moment (N·mm) at the section, V_f is the factored shear force (N) at the section, N_f is the factored axial load (N) normal to the cross-section occurring simultaneously with V_f, A_{FRP} is the cross-sectional area (mm^2) of FRP reinforcement, f_{po} is the stress (MPa) in tendons when the stress in surrounding concrete is zero, and A_p is the cross-sectional area (mm^2) of tendons in the tension zone.

(*c*) For the factored shear resistance carried by FRP shear reinforcement (V_{FRP}), the following is used. For components with transverse reinforcement perpendicular to the longitudinal axis, V_{FRP} is calculated from

$$[6] \quad V_{FRP} = \frac{\phi_{FRP} A_v \sigma_v \, d_{long} \cot\theta}{s}$$

When the transverse reinforcement is inclined at an angle θ to the longitudinal axis, V_{FRP} is calculated from

$$[7] \quad V_{FRP} = \frac{\phi_{FRP} A_v \sigma_v \, d_{long} (\cot\theta + \cot\alpha) \sin\alpha}{s}$$

where in eqs. [6] and [7] θ is obtained by conventional methods; the resistance factor (ϕ_{FRP}) is as given in Table 1: and σ_v is the smaller of the values obtained from the following two equations:

$$[8] \quad \sigma_v = \frac{(0.05 r / d_s + 0.3) f_{FRP\,bend}}{1.5}$$

$$[9] \quad \sigma_v = E_{vFRP} \varepsilon_v$$

in which ε_v is obtained from

$$[10] \quad \varepsilon_v = 0.0001 \left(f_c' \frac{\rho_s E_{FRP}}{\rho_{vFRP} E_{vFRP}} \right)^{0.5} \left[1 + 2 \left(\frac{\sigma_N}{f_c'} \right) \right] \leq 0.0025$$

(*d*) The minimum amount of shear reinforcement (A_{vmin}) is calculated from

$$[11] \quad A_{vmin} = 0.06 \sqrt{f_c'} \frac{b_w s}{\sigma_v}$$

where σ_v is calculated by eq. [8].

It is well known that the shear carried by concrete is smaller in FRP-reinforced concrete beams than in beams reinforced with a comparable amount of steel. Tariq and Newhook (2003) listed different equations for shear carried by concrete in FRP beams. The majority of researchers have concluded that the shear carried by concrete in FRP-reinforced beams is $(E_{FRP}/E_s)^n$ times the shear carried by concrete in steel-reinforced beams. Usually n is taken as 1/2 or 1/3. Other researchers simply assume that the shear carried by concrete in FRP-reinforced beams is half that carried by concrete in steel-reinforced beams.

Equation [3], for shear capacity, is based on the work of Machida.[4] Equation [10], for ε_v, is as specified in the JSCE (1997) design recommendations. The other equations for the calculation of shear capacity follow the procedure for concrete reinforced with steel bars given in the concrete section of the second edition of the CHBDC.

The limit on the longitudinal strain in FRP stirrups is increased from 0.002 to 0.0025 to reflect the finding that aggregate interlock can exist up to a strain of 0.003 (Priestley et al. 1996). The stress in FRP stirrups depends on the strength of the straight portion of a bent stirrup. For bent bars, the test method is specified in standard S806 (CSA 2002).

The equation for minimum shear reinforcement for FRP-reinforced beams is based on the work of Shehata (1999).

Barrier walls

The first edition of the CHBDC permitted a PL-3 barrier reinforced with GFRP bars and connected to the deck slab by means of double-headed steel bars; the code provided details of only the double-headed bars and the primary (i.e., vertical) GFRP reinforcement near the traffic face of the wall. In the second edition, the code requires that on the traffic side, the wall be provided with a GFRP grid or orthogonal assembly of GFRP bars providing factored strengths of 330 and 240 N/mm length of the wall in the vertical and horizontal directions, respectively. These strength requirements correspond to the factored strengths of No. 15 steel bars at a spacing of 220 and 300 mm, respectively. The barrier wall is required to be connected to the deck slab by means of 19 mm diameter, 500 mm long double-headed steel bars at a spacing of 300 mm. The code also requires that the spacing of the bars and anchors be reduced by half over the following lengths of the barrier wall: (*i*) 1.2 m on each side of a joint in the wall; (*ii*) 1.2 m on each side of a luminaire embedded in the wall; and (*iii*) 1.2 m from the free vertical edges of the wall.

The commentary to the second edition of the CHBDC presents details of two barrier walls (shown in Figs. 7 and 8), which were developed and tested and are approved by the ministère des Transports du Québec. A pendulum impact test was carried out on full-scale types PL-2 and PL-3 barriers reinforced with GFRP bars. The test was described by El-Salakawy et al. (2004). The strength of the bent GFRP bars connecting the walls to the deck slab is determined according to test method B.12 in ACI guide 440.3R-04 (ACI Committee 440 2004). For the designs illustrated in Figs. 7 and 8, the guaranteed tensile strength at the bend, according to test method B.12, should not be less than 400 MPa. The guaranteed tensile strength of the straight portion of the bent GFRP bars should not be less than 650 MPa.

As with the barrier explicitly permitted in the code, the spacing of the reinforcement marked with an asterisk in Figs. 7 and 8 is required to be halved within 1.0 m of the

Fig. 7. PL-3 barrier wall with GFRP bars (dimensions in millimetres).

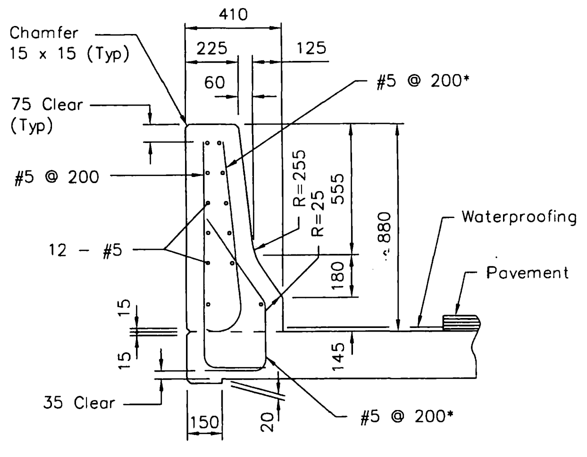
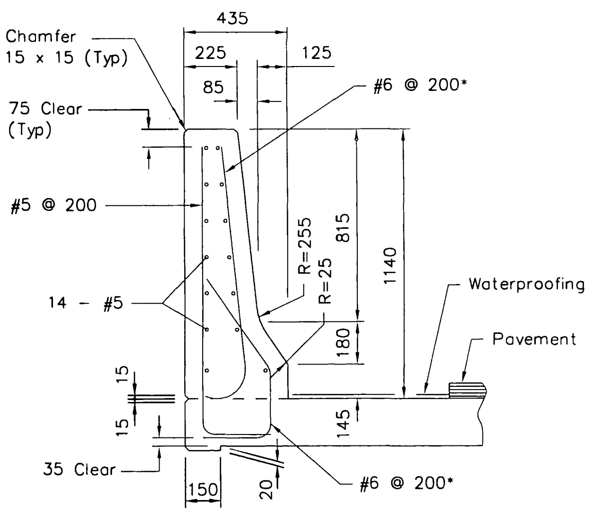


Fig. 8. PL-2 barrier wall with GFRP bars (dimensions in millimetres).



joints in the wall, 1.0 m from embedded luminaire supports, and 1.0 m from the free vertical edges of the wall.

Rehabilitation of existing concrete structures with fibre-reinforced polymer

General

The second edition of the CHBDC gives provisions for the rehabilitation of concrete structures with FRP; these provisions, which are largely based on the work of Täljsten (1994, 2004*a*, 2004*b*), are applicable to existing concrete structures having $f_c' \leq 50$ MPa and strengthened with FRP constituting externally bonded systems or near-surface mounted reinforcement (NSMR). If the concrete cover is <20 mm, NSMR is not permitted. Rehabilitation of concrete structures having f_c' of >50 MPa requires approval.

The behaviour of concrete elements strengthened with FRP is highly dependent on the quality of the concrete substrate. Corrosion-initiated cracks are more detrimental to bond-critical applications than to contact-critical applications. The code defines bond-critical applications as those applications of FRP that rely on bond to the substrate for load transfer; an example of this application is an FRP strip bonded to the underside of a beam to improve its flexural capacity. Similarly, the contact-critical applications of FRP rely on continuous intimate contact between the substrate and the FRP system. An example of contact-critical application is an FRP wrap around a circular column, which depends upon the radial pressure that it exerts on the column to improve its compressive strength.

Before a rehabilitation strategy is developed, an assessment of the existing structure or elements is required, following the requirements of the evaluation section of the CHBDC. Only those structures that have a live load capacity factor (F) of ≥ 0.5 are allowed to be strengthened. The evaluation section of the CHBDC defines F as follows for a structural component at ULS:

$$[12] \quad F = \frac{U\phi R - \Sigma\alpha_D D - \Sigma\alpha_A A}{\alpha_L(1 + I)}$$

where U is the resistance adjustment factor, depending on the category of resistance (for example, its value for the axial compression of reinforced concrete components is 1.11); ϕ is the resistance factor specified in the concrete section of the CHBDC code (0.75 for concrete); R is the nominal unfactored resistance of the component; α_D is the load factor for effects due to dead loads; D is the nominal load effect due to unfactored dead load; α_A is the load factor for force effects due to additional loads, including wind, creep, and shrinkage; A is the force effect due to the additional loads; α_L is the load factor force effect due to live loads; L is the force effect due to nominal (i.e., unfactored) live loads; and I is the dynamic load allowance,

Strengthening for flexural components

Fibre-reinforced-polymer rehabilitation systems of the externally bonded and NSMR types may be exposed to impact or fire. To prevent collapse in the event that the FRP reinforcement is damaged, the structures that are to be strengthened with FRP require a live load capacity factor (F), defined above, of >0.5. With F > 0.5, the unstrengthened structure will thus be able to carry all the dead loads and a portion of the live loads. Similar stipulations can be found in standard S806 (CSA 2002) and in ACI guide 440-2R-02 (ACI Committee 440 2002). The requirement that F be >0.5 also provides some benefits under normal service conditions: the stresses and strains in all materials, including concrete, steel, and FRP, are limited; and the risk of creep or yielding is reduced.

In addition to the conditions of equilibrium and compatibility of strains, the calculation for ULS is to be based on the resistance factors for materials of the parent component and those of the FRP (given in Table 1), the assumptions implicit in the design of the parent component, and the following additional assumptions: (*i*) strain changes in the FRP strengthening systems are equal to the strain changes in the adjacent concrete; and (*ii*) the contribution of FRP in compression are ignored.

For an externally bonded flexural strengthening system, the maximum value of the strain in the FRP is not to exceed 0.006; this conservative requirement has been formulated to

Fig. 9. Failure modes in flexure for external strengthening.

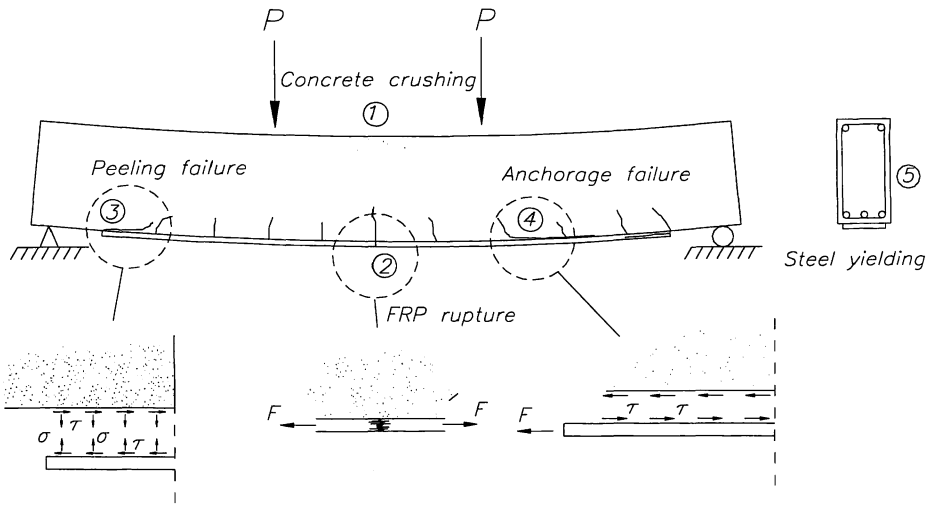
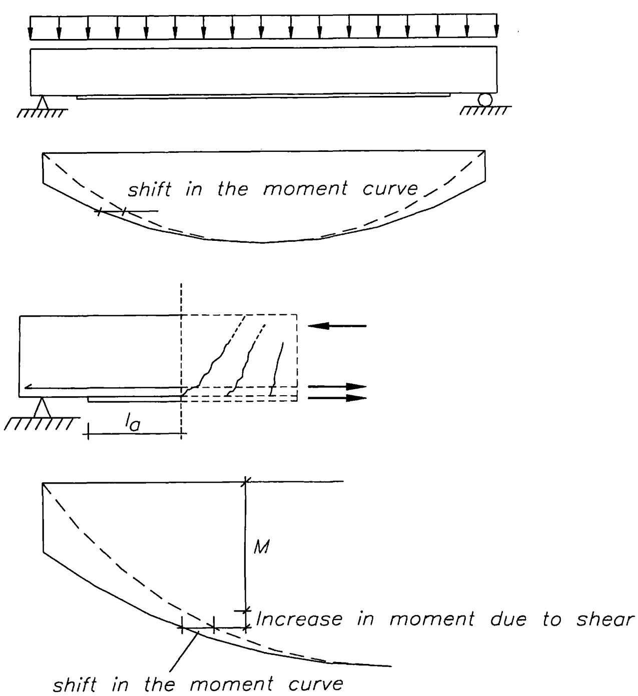


Fig. 10. Displacement of tensile force curve in relation to the moment curve.



avoid a possible failure by delamination of the FRP initiating at intermediate cracks (Täljsten 2002; Teng et al. 2002).

In the FRP strengthening of concrete components, the failure modes required to be considered are (*i*) crushing of the concrete in compression before rupture of the FRP or yielding of the reinforcing steel (mode 1 in Fig 9); (*ii*) yielding of the steel followed by rupture of the FRP in tension (mode 2 in Fig. 9); (*iii*) in the case of members with internal prestressing, additional failure modes controlled by the rupture of the prestressing tendons; (*iv*) anchorage failure (mode 4 in Fig. 9); (*v*) peeling failure or anchorage failure of the FRP system at the cut-off point (mode 3 in Fig. 9); and (*vi*) yielding of the steel followed by concrete crushing, before rupture of the FRP in tension.

For externally bonded FRP strengthening systems, the anchorage length beyond the point where no strengthening is required is not to be less than l_a, which is given by

$$[13] \quad l_a = 0.5\sqrt{E_{\mathrm{FRP}} t_{\mathrm{FRP}}}$$

where t_{FRP} is the total thickness (mm) of externally bonded FRP plates or sheets. In addition to the requirement indicated in eq. [13], the anchorage length should be at least 300 mm; otherwise, the FRP needs to be suitably anchored.

The anchorage length is of central importance if an effective strengthening design is to be achieved. A good design will always lead to concrete failure. An anchorage failure is characterized by failure mode 4 in Fig. 9. Figure 10 shows the location of the anchorage. As illustrated in this figure, the tensile force corresponding to the moment curve must be corrected because of the inclined cracking that the shear force causes.

Strengthening of compression components

When a column is strengthened with FRP, the compressive strength of the confined concrete (f'_{cc}) is determined from the following equation:

$$[14] \quad f'_{\mathrm{cc}} = f'_{\mathrm{c}} + 2f_{l\mathrm{FRP}}$$

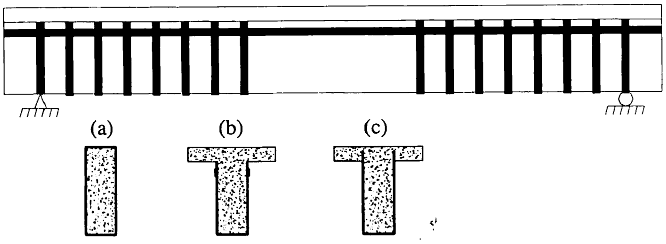
The confinement pressure due to FRP strengthening at the ULS ($f_{\ell\mathrm{FRP}}$) is determined from the following equation:

$$[15] \quad f_{l\mathrm{FRP}} = \frac{2\phi_{\mathrm{FRP}} f_{\mathrm{FRPu}} t_{\mathrm{FRP}}}{D_{\mathrm{g}}}$$

For columns with circular cross sections, D_{g} is the diameter of the column; for columns with rectangular cross sections having aspect ratios ≤1.5 and a smaller cross-sectional dimension not greater than 800 mm, D_{g} is equal to the diagonal of the cross section. For columns with other polygonal cross sections, D_{g} is equal to the diameter of the inscribed circle.

Various formulae for determining the compressive strength of FRP-confined concrete have been assessed by Teng et al. (2002), Thériault and Neale (2000), and Bisby et al. (2005). Equation [14], in conjunction with eq. [15], has been shown

Fig. 11. Anchorage of externally bonded FRP shear reinforcement: (*a*) fully wrapped section; (*b*) anchorage with horizontal strips; and (*c*) anchorage in compression zone.



to provide close but conservative estimates of the compressive strength.

The confinement pressure at ULS is required to be designed to lie between $0.1f_c'$ and $0.33f_c'$. The minimum confinement pressure is specified in order to ensure ductile behaviour of the confined section, and the maximum confinement pressure is specified in order to avoid excessive axial deformations and creep under sustained loads. The limit provided is such that the factored resistance of the FRP-confined concrete does not exceed the equivalent nominal strength of the unconfined concrete; that is, $0.8\phi_c f_{cc}' \le f_c'$.

Strengthening for shear

The shear-strengthening scheme is to be of the type in which the fibres are orientated perpendicular or at angle β to the member axis. The shear reinforcement is to be anchored by suitable means in the compression zone by one of the following schemes:

- The shear reinforcement is to be fully wrapped around the section as shown in Fig. 11*a*.
- The anchorage to the shear reinforcement near the compression flange is provided by additional horizontal strips as shown in Fig. 11*b*.
- The anchorage is provided in the compression zone as shown in Fig. 11*c*.

If none of these schemes can be provided, special provisions must be made.

For reinforced concrete members with rectangular sections or T-sections and having the FRP shear reinforcement anchored in the compression zone of the member, the factored shear resistance (V_r) is calculated from

[16] $$V_r = V_c + V_s + V_{FRP}$$

where V_c and V_s are calculated as for steel-reinforced sections; and V_{FRP} is obtained from the following:

[17] $$V_{FRP} = \frac{\phi_{FRP} E_{FRP} \varepsilon_{FRP_e} A_{FRP} d_{FRP} (\cot\theta + \cot\beta)\sin\beta}{s_{FRP}}$$

where

[18] $$A_{FRP} = 2t_{FRP} w_{FRP}$$

For completely wrapped sections,

[19] $$\varepsilon_{FRP_e} = 0.004 \le 0.75\,\varepsilon_{FRP_u}$$

For other configurations, ε_{FRP_e} is calculated from

[20] $$\varepsilon_{FRP_e} = \kappa_V\,\varepsilon_{FRP_u} \le 0.004$$

where for continuous U-shape configurations of the FRP reinforcement, the bond-reduction coefficient (κ_V) is as follows:

[21] $$\kappa_V = k_1 k_2 L_e / 11900\,\varepsilon_{FRP_u} \le 0.75$$

and

[22] $$k_1 = (f_c'/27)^{2/3}$$

[23] $$k_2 = (d_{FRP} - L_e)/d_{FRP}$$

[24] $$L_e = 23300/(t_{FRP} E_{FRP})^{0.58}$$

The value of ε_{FRP_e} is limited to 0.004 to maintain aggregate interlock in the evaluation of V_c.

For prestressed concrete components, $V_r = V_c + V_s + V_p + V_{FRP}$, for which the general theory for steel-reinforced concrete is used to calculate V_c, V_s, and V_p and the equations given above are used to calculate V_{FRP}.

For components with non-rectangular or non-T cross sections, a rigorous analysis or test should guide the design.

The spacing of FRP bands should not be more than s_{FRP} given by the following equation:

[25] $$s_{FRP} \le w_{FRP} + d_{FRP}/4$$

The total factored shear resistance subsequent to FRP strengthening (V_r) should not exceed $0.66 b_w d (f_c')^{0.5}$.

The calculation of the factored shear resistance provided by FRP shear reinforcement is similar to the calculation of the factored shear resistance provided by steel shear reinforcement, the main difference being the use of an effective strain to evaluate the stress in the FRP. The effective strain in the FRP is based on the work of Khalifa et al. (1998) and Maeda et al. (1997).

Rehabilitation of timber bridges

General

The code provisions for the strengthening of sawn timber beams and stringers are based on the simple principle that providing a defined minimal amount of FRP sheets or bars can alter the mode of failure of timber beams and stringers and enhance their load-carrying capacity in both flexure and shear.

Strengthening for flexure with glass-fibre-reinforced-polymer sheets

When the following minimum requirements for strengthening with GFRP sheets are met, the bending strengths for beam and stringer grades used for the evaluation are required to be $K_{bFRP} f_{bu}$, for which K_{bFRP} is obtained from Table 8; and the specified bending strength (f_{bu}), from the wood structure section of the CHBDC:

(*a*) The minimum fibre volume fraction of GFRP system in the direction of the span of the beam is 30%.

(*b*) The GFRP sheet on the flexural tension face of the beam covers at least 90% of the width of the beam and has a minimum thickness of 0.1 mm.

(*c*) The adhesive used for bonding the GFRP sheets to the timber beam is compatible with the preservative treatment used on the timber.

Table 8. Values of K_{bFRP}.

Grade of original beam	K_{bFRP}
Select structural grades	1.05^a or 1.1^b
No. 1	1.2
No. 2	1.5

[a]If the beam is not strengthened for shear.
[b]If the beam is also strengthened for shear.

(*d*) In the longitudinal direction of the beam, the GFRP sheets extend as close to the beam supports as possible.
(*e*) The adhesive used for bonding the GFRP sheets to the timber beam is chosen such that it is compatible with expected volumetric changes of the timber.

The values of K_{bFRP} for grades 2 and 1 sawn timber beams and stringers in Table 8 were derived from the ratios of f_{bu} for grades 1 and 2 and for select structural and grade 1 timbers, respectively, in the CHBDC. The values of K_{bFRP} for select structural grades were derived from data collected by Eden (2002), who has shown that the flexural enhancement of select structural timbers is better when the timbers are also strengthened for shear. It is for this reason that two values of K_{bFRP} are specified for select structural grades in Table 8.

The minimum amount of GFRP sheets specified in this clause has been found to be enough to strengthen the beam to at least the next higher grade, provided the sheet extends as close to the supports as possible.

Most adhesives used in the FRP strengthening of structures do not bond very well to creosote-treated timber. To determine whether an adhesive is compatible with the preservative treatment used on the timber, testing (Mufti et al. 2001; Hay 2004) should establish that bond failure occurs in the timber.

Strengthening for flexure with near-surface mounted reinforcement

As is the case for flexural strengthening with GFRP sheets, for strengthening with NSMR the bending strengths for beam and stringer grades used for the evaluation are required to be $K_{bFRP} f_{bu}$, for which K_{bFRP} is obtained from Table 8, if the following conditions are met.

(*a*) The minimum fibre volume fraction for GFRP bars is 60%.
(*b*) There are at least two bars within the width of the beam.
(*c*) The total cross-sectional area for all bars on a beam is at least 0.002 times the cross-sectional area of the timber component.
(*d*) As shown in Fig. 12, each bar is embedded in a groove, preferably with a rounded end. The depth of each groove is 1.6–2.0 times the bar diameter (d_b); the width of each groove is not less than $d_b + 5$ mm; the edge distance of the outer groove is not less than 25 mm or less than $2d_b$; and the clear spacing between grooves is not less than 25 mm or less than $3d_b$.
(*e*) Before the GFRP bars are embedded, the grooves in the beams are cleaned with pressurized air to remove any residue.
(*f*) The adhesive chosen for bonding the GFRP bars to the timber beam is compatible with the preservative treatment used on the timber and with the expected volumetric changes of the timber.

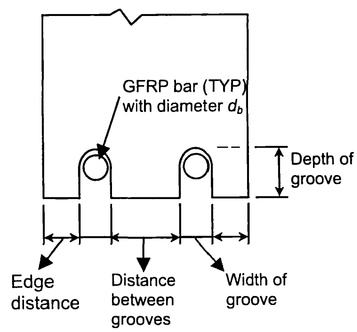


Fig. 12. Cross section of a timber beam with GFRP near-surface mounted reinforcement.

(*g*) In the longitudinal direction of the beam, the GFRP bar extends as close to the beam support as possible.
(*h*) Each GFRP bar is held in place as close to the tip of the groove as possible.

Near-surface mounted bars on sawn timber stringers, besides having all the advantages associated with sheets, have the added advantage of being protected from moisture and external abrasion, such as from ice and floating debris (Svecova and Eden 2004).

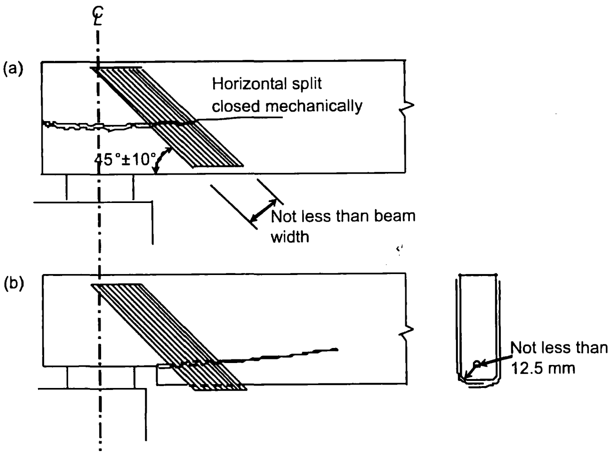
Shear strengthening with glass-fibre-reinforced-polymer sheets

When the following minimum requirements are met for shear strengthening with GFRP sheets, the shear strengths for beam and stringer grades for the evaluation are assumed to be $K_{vFRP} f_{vu}$, for which K_{vFRP} is taken as 2.0 and f_{vu} is obtained from the CHBDC:

(*a*) The minimum fibre volume fraction of GFRP sheets along their axes is 30%, and the sheets have a minimum thickness of 0.1 mm.
(*b*) Horizontal splits in beams, if present, are closed by a mechanical device before the application of the GFRP sheets.
(*c*) The GFRP sheets are at least as wide as the width of the cross section of the beam (Fig. 13*a*).
(*d*) As shown in Fig. 13*a*, the GFRP sheet is inclined to the beam axis at an angle of 45° ± 10° from the horizontal.
(*e*) The top of the inclined GFRP sheet is as close to the centerline of the beam support as possible.
(*f*) The adhesive chosen for bonding GFRP to the timber beam is compatible with the preservative treatment used on the timber and with the expected volumetric changes of the timber.
(*g*) The top of the inclined GFRP sheet extends up to nearly the top of the beam.
(*h*) The lower end of the inclined GFRP sheet extends to the bottom of the beam if there is no dap present (Fig. 13*a*). If there is a dap, the lower end is wrapped around the bottom and extends to at least half the width of the beam. In the latter case, the corner of the beam is rounded to a minimum radius of 12.5 mm to provide full contact of the sheet with the beam (Fig. 13*b*).

Recognizing that the values of F_v for sawn timber grades specified in section 9 (Wood Structures) of the first edition

Fig. 13. Elevation of timber beam with GFRP sheets for shear strengthening: (*a*) without a dap; (*b*) with a dap.



of the CHBDC were overly conservative, the second edition now specifies somewhat higher values. For example, for select structural grade Douglas-fir timber bridge stringers, the first edition had specified F_v = 0.9 MPa; in the second edition, this value has been upgraded to 1.5 MPa. The specified values of F_v in both the first and second editions are based on the statistical presence of checks, or horizontal splits, at the ends of sawn timbers.

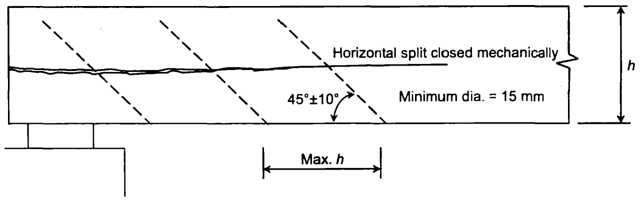
These provisions apply only when the checks are closed mechanically and the GFRP strengthening scheme is applied later. As discussed by Hay (2004) and Hay et al. (2004), the mechanical closing of a horizontal split in a timber beam involves (*i*) temporarily closing the gap with an external device, such as a grip with a hydraulic jack; and (*b*) permanently closing the gap with a lag screw. Simply closing the checks is enough to increase the shear strength of the stringers by 20%–30% above the values included in the second edition of CHBDC. The addition of FRP reinforcement will further increase the shear strength; however, in the spirit of caution, the code provisions do not reflect this extra increase in the shear strength. It is hoped that the availability of future test data will lead to further utilization of the shear strength of FRP-strengthened sawn timber beam and stringers in another edition of the code.

Shear strengthening with GFRP-embedded bars

When the following minimum requirements are met for strengthening with GFRP bars, the shear strength for beam and stringer grades for the evaluation is assumed to be $K_{vFRP} f_{vu}$, for which K_{vFRP} is taken as 2.2 and f_{vu} is obtained from the CHBDC:

(*a*) The minimum fibre volume fraction of the GFRP bars is 60%.
(*b*) Horizontal splits in beams, if present, are closed by a mechanical device before the insertion of the GFRP bars.
(*c*) As shown in Fig. 14, there are at least three GFRP bars at each end of the beam.
(*d*) The diameter of the GFRP bar (d_b) is at least 15 mm, and the minimum diameter of the hole containing a bar is d_b + 3 mm.
(*e*) The spacing of bars along the length of the beam is h ± 25 mm.

Fig. 14. Elevation of timber beam with GFRP bars for shear strengthening.



(*f*) The adhesive chosen for bonding the GFRP bars to the timber beam is compatible with the preservative treatment used on the timber and with the expected volumetric changes of the timber.
(*g*) As shown in Fig. 14, the GFRP bars are inclined to the beam axis at an angle of 45° ± 10° from the horizontal.
(*h*) The tops of the inclined GFRP bars are within 10–25 mm from the top of the beam.
(*i*) When there are daps present, the ingress of the drilled hole should be 100 ± 10 mm from the edge of the dap.

In cases where the corners of a timber stringer must be shaved to accommodate an FRP sheet extending to the bottom face of the beam, it may be found advantageous to strengthen the beam with embedded GFRP bars.

Installation of and quality control for fibre-reinforced-polymer strengthening system

Two appendices in the code cover the installation of and quality control for FRP strengthening systems. They include directions for the shipping, storage, and handling of FRP systems and guidance on the methods and details of installation.

Conclusion

This paper presented a synthesis of the design provisions of the second edition of the *Canadian Highway Bridge Design Code* for fibre-reinforced structures, including new design provisions for applications not covered by the first edition and the rationale for those that have been changed from the first edition.

References

ACI Committee 440. 2002. 440.2R-02: Design and construction of externally bonded FRP systems for strengthening concrete structures. American Concrete Institute (ACI), Farmington Hills, Mich.

ACI Committee 440. 2004. 440.3R-04: Guide test methods for fibre-reinforced polymers (FRPs) for reinforcing or strengthening concrete structures. American Concrete Institute (ACI), Farmington Hills, Mich.

Aguiniga, G. 2003. Characterization of design parameters for fiber reinforced polymer composite reinforced concrete systems. Ph.D. thesis, Texas A&M University, College Station, Tex.

ASTM. 2001. Practice for determining and reporting dynamic mechanical properties of plastics. American Society for Testing and Materials (ASTM), Philadelphia, Pa. Standard D4065.

ASTM. 2003. Test method for transition temperatures of polymers by thermal analysis (DTA or DSC). American Society for

Testing and Materials (ASTM), Philadelphia, Pa. Standard D3418.

Bakht, B., and Chu, K.-Y. 1997. Testing of a reinforced concrete stay-in-place formwork for deck slabs. *In* Proceedings of the CSCE Annual Conference, Sherbrooke, Que., 27–30 May 1997. *Edited by* K.W. Neale, P. Labossière, and G. McClure. Canadian Society for Civil Engineering, Montréal, Que. Vol. 6, pp. 257–266.

Bakht, B., Mufti, A.A., and Desai, Y.M. 2002*a*. PLATO user manual. ISIS Canada, University of Manitoba, Winnipeg, Man.

Bakht, B., Mufti, A.A., and Tadros, G. 2002*b*. Failure tests on unreinforced concrete slab on stay-in-place metal forms. ISIS Canada, University of Manitoba, Winnipeg, Man. ISIS Canada Technical Report.

Bakht, B., Mufti, A.A., and Tadros, G. 2004. Discussion of "Fibre-reinforced polymer composite bars for the concrete deck slabs of Wotton Bridge". Canadian Journal of Civil Engineering, **31**(3): 530–531.

Benmokrane, B., Wang, P., Ton-That, T.M., Rahman, H., and Robert, J-F. 2002. Durability of GFRP reinforcing bars in concrete. ASCE Journal of Composites for Construction, **6**(3): 143–155.

Benmokrane, B., El-Salakawy, E., Desgagné, G., and Lackey, T. 2004. FRP bars for bridges. Concrete International, 26(8): 84–90.

Bisby, L.A., Dent, A.J.S., and Green, M.F. 2005. Comparison of confinement models for fiber-reinforced polymer-wrapped concrete. ACI Structural Journal, **102**(1): 62–72.

Bowen, D. 2005. ArchForm design and testing of a steel free deck constructed using partial depth precast concrete panels. M.Sc. thesis, Department of Civil Engineering, University of Manitoba, Winnipeg, Man.

CSA. 2002. Design and construction of building components with fibre-reinforced polymers. Canadian Standards Association, Toronto, Ont. Standard S806-02.

CSA. 2006. Canadian highway bridge design code. 2nd ed. CSA International, Toronto, Ont. CAN/CSA-S6-00.

Dejke, V. 2001. Durability of FRP reinforcement in concrete—literature review and experiments. Ph.D. thesis, Department of Building Materials, Chalmers University of Technology, Goteborg, Sweden.

Eden, R. 2002. Strengthening of timber bridge stringers using GFRP. M.Sc. thesis, Department of Civil Engineering, University of Manitoba, Winnipeg, Man.

El-Gamal, S., El-Salakawy, E., and Benmokrane, B. 2005. Behavior of concrete bridge deck slabs with fiber-reinforced polymer bars under concentrated loads. ACI Structural Journal, **102**(5): 727–735.

El-Salakawy, E.F., and Benmokrane, B. 2004. Serviceability of concrete bridge deck slabs reinforced with FRP composite bars. ACI Structural Journal, **101**(5): 727–736.

El-Salakawy, E., Masmoudi, R., Benmokrane, B., Brière, F., and Desgagné, G. 2004. Pendulum impacts into concrete bridge barriers reinforced with glass fibre reinforced bars. Canadian Journal of Civil Engineering, **31**(4): 539–552.

Frosch, R.G. 1999. Another look at cracking and crack control in reinforced concrete. ACI Structural Journal, **96**(3): 437–442.

Greenwood, M. 2002. Creep-rupture testing to predict long-term performance. *In* Proceedings of the 2nd International Conference on Durability of Fibre-Reinforced Polymer (FRP) Composites for Construction, Montréal, Que., 29–31 May 2002. *Edited by* B. Benmokrane and E. El-Salakawy. University of Sherbrooke. pp. 203–212.

Hay, S.W. 2004. Shear strengthening of timber stringers using GFRP sheets. M.Sc. thesis, Department of Civil Engineering, University of Manitoba, Winnipeg, Man.

Helbling, C.S., and Karbhari, V.M. 2004. Investigation of synergistic hygrothermal and sustained strain effects on the durability of pultruded E-glass/vinylester composites. *In* Proceedings of the Joint American Society of Composites/American Society for Testing and Materials Committee D30 19th Technical Conference, Atlanta, Ga., 17–20 October 2004. [CD-ROM]. *Edited by* E.A. Armanios and J.R. Reeder. DEStech Publications, Lancaster, Pa. Paper AL1.

JSCE (Japan Society of Civil Engineers). 1997. Recommendations for design of structures using continuous fiber reinforcing materials. JSCE, Tokyo, Japan. Concrete Engineering Series, No. 23.

Kaiser, H., and Karbhari, V.M. 2003. Identification of potential defects in the rehabilitation of concrete structures with FRP composites. International Journal of Materials & Product Technology, **19**(6): 498–520.

Karbhari, V.M. 2000. Determination of materials design values for the use of fibre-reinforced polymer composites in civil infrastructure. Proceedings of the Institution of Mechanical Engineers, Part L: Journal of Materials: Design and Applications, **214**: 163–171.

Karbhari, V.M. 2003. Durability of FRP composites for civil infrastructure—myth, mystery or reality. Advances in Structural Engineering, **6**(3): 243–255.

Karbhari, V.M. 2004. E-glass/vinylester composites in aqueous environments: effects on short-beam shear strength. ASCE Journal of Composites for Construction, **8**(2): 148–156.

Karbhari, V.M., and Vasquez, A. 2003. Fiber-reinforced polymer composite strengthening of concrete slabs with cutouts. ACI Structural Journal, **100**(5): 665–673.

Karbhari, V., Chin, J., and Reynaud, D. (*Editors*). 2001. Gap analysis for durability of FRP composites in civil infrastructure. Civil Engineering Research Foundation, Washington, D.C. Report No. 40578.

Karbhari, V.M., Chin, J.W., Hunston, D., Benmokrane, B., Juska, T., Morgan, R., Lesko, J.J., Sorathia, V., and Reynaud, D. 2003. Durability gap analysis for FRP composites in civil infrastructure. ASCE Journal of Composites for Construction, **7**: 238–247.

Khalifa, A., Gold, W.J., Nanni, A., and Aziz, M.I. 1998. Contribution of externally bonded FRP to shear capacity of RC flexural members. ASCE Journal of Composites for Construction, **2**: 195–202.

Klowak, C., Eden, R., Mufti, A.A., Elkholy, I, Tadros, G., Bakht, B., and Loewen, E. 2004. First application of second generation steel-free deck slab for rehabilitation. *In* Proceedings of the International Conference on Bridge Maintenance, Safety and Management (IABMAS '04), Kyoto, Japan 18–22 October 2004. [CD-ROM]. *Edited by* E. Watanabe, D.M. Frangopol, and T. Utsunomiya.

Limaye, V., Mufti, A.A., Bakht, B., and Butt, S.D. 2002. Steel-free deck slabs under cyclic loading. *In* Proceedings of the 6th International Conference on Short and Medium Span Bridges, Vancouver, B.C., 31 July – 2 August 2002. *Edited by* P.H. Brett, N. Banthia, and P.G. Buckland. Canadian Society for Civil Engineering, Montréal, Que. pp. 1311–1319.

Maeda, T., Asano, Y., Sato, Y., Ueda, T., and Kakuta, Y. 1997. A study on bond mechanism of carbon fiber sheet. *In* Proceedings of the 3rd International Symposium on Non-metallic (FRP) Reinforcement for Concrete Structures, Sapporo, Japan, 14–16 October 1997. Japan Concrete Institute, Tokyo, Japan. Vol. 2, pp. 279–286.

Memon, A.H., Mufti, A.A., and Bakht, B. 2003. Fatigue investigation of concrete bridge deck slab reinforced with GFRP and steel strap. *In* Proceedings of the 6th International Symposium on FRP Reinforcement for Concrete Structures, Singapore, 8–10

July 2003. *Edited by* K.H. Tan. World Scientific, Hong Kong. Vol. 2, pp. 923–932.

Mufti, A.A., Bakht, B., Svecova, D., and Adam, T. 2001. Evaluation of a glue for bonding FRP plates on timber beams treated with creosote. *In* Proceedings of the CSCE Annual Conference, Victoria, B.C., 30 May – 2 June 2001. [CD-ROM]. *Edited by* D. Noakes. Canadian Society for Civil Engineering, Montréal, Que.

Mufti, A.A., Memon, A.H., Bakht, B., and Banthia, N. 2002. Fatigue investigation of steel-free bridge deck slabs. *Edited by* P. Balaguru, A. Namaan, and W. Weiss. American Concrete Institute, Farmington Hills, Mich. SP-206, pp. 61–70.

Mufti, A.A., Onofrei, M., Benmokrane, B., Banthia, N., Boulfiza, M., Newhook, J.P., Bakht, B., Tadros, G., and Brett, P. 2005*a*. Report on the studies of GFRP durability in concrete from field demonstration structures. *In* Proceedings of the Third International Conference on Composites in Construction (CCC 2005), Lyon, France, 11–13 July 2005. [CD-ROM].

Mufti, A.A., Onofrei, M., Benmokrane, B., Banthia, N., Boulfiza, M., Newhook, J.P., Bakht, B., Tadros, G., and Brett, P. 2005*b*. Durability of GFRP reinforced concrete in field structures. *In* Proceedings of the 7th International Symposium on Fiber-Reinforced Polymer Reinforcement for Reinforced Concrete Structures (FRPRCS-7), Kansas City, Mo., 6–9 November 2005. Edited by C.K. Shield, J.P. Busel, S.L. Walkup, D.D. Gremel, and ACI Committee 440. American Concrete Institute, Farmington Hills, Mich. SP-230-77, pp. 1361–1378.

Mufti, A.A., Onofrei, M., Benmokrane, B., Banthia, N., Boulfiza, M., Newhook, J.P., Bakht, B., Tadros, G., and Brett, P. 2005*c*. Field study of glass-fibre-reinforced polymer durability in concrete. Canadian Journal of Civil Engineering, **34**(3): 355–366.

NCHRP. 2004. Bonded repair and retrofit of concrete structures using FRP composites. National Cooperative Highway Research Program (NCHRP), Transportation Research Board, Washington, D.C. NCHRP Report 514.

Nkurunziza, G. 2004. Performance of glass FRP bars as reinforcement for concrete structures under the effect of sustained loads and elevated temperature in humid and alkaline environment. Ph.D. thesis, Department of Civil Engineering, Université de Sherbrooke, Sherbrooke, Que. [In French.]

Nkurunziza, G., Debaiky, A., Cousin, P., and Benmokrane, B. 2005. Durability of GFRP bars: a critical review of the literature. Progress in Structural Engineering and Materials, **7**: 194–209.

Priestley, M.J.N., Seible, F., and Calvi, G.M. 1996. Seismic design and retrofit of bridges. John Wiley & Sons, Inc., New York.

Sargent, D.D., Ventura, C.E., Mufti, A.A., and Bakht, B. 1999. Testing of steel-free bridge decks. Concrete International, **21**(8): 55–61.

Shehata, E.F. 1999. Fibre reinforced polymer (FRP) for shear reinforcement in concrete structures. Ph.D. thesis, Department of Civil Engineering, University of Manitoba, Winnipeg, Man.

Svecova, D., and Eden, R. 2004. Flexural and shear strengthening of timber beams using glass fibre reinforced polymer bars — an experimental investigation. Canadian Journal of Civil Engineering, **31**(1): 45–55.

Täljsten, B. 1994. Plate bonding, strengthening of existing concrete structures with epoxy bonded plates of steel of fibre reinforced plastics. Ph.D. thesis, Department of Structural Engineering, Luleå University of Technology, Luleå, Sweden. No. 1994:152D.

Täljsten, B. 2002. FRP strengthening of existing concrete structures: design guidelines. Luleå University of Technology, Luleå, Sweden.

Täljsten, B. 2004*a*. Design guideline for CFRP strengthening of concrete structures. *In* Bridge Maintenance, Safety, Management and Cost: Proceedings of the 2nd International Conference of the Association of Bridge Maintenance and Safety (IABMAS'04), Kyoto, Japan, 18–22 October 2004. *Edited by* E. Watanabe, D.M. Frangopol, and T. Utsonomiya. A.A. Balkema, Lisse, The Netherlands.

Täljsten, B. 2004*b*. FRP strengthening of existing concrete structures: design guidelines. 3rd ed. Division of Structural Engineering, Luleå University of Technology, Luleå, Switzerland.

Tariq, M., and Newhook, J.P. 2003. Shear testing of reinforced concrete without transverse reinforcement. *In* Proceedings of the CSCE Annual Conference, Moncton, N.B., 4–7 June 2003. *Edited by* M. Massièra. [CD-ROM]. Canadian Society for Civil Engineering, Montréal, Que. Paper No. GCF-340.

Teng, J.G., Chen, J.F., Smith, S.T., and Lam, L. 2002. FRP-strengthened RC structures. John Wiley & Sons, Ltd., West Sussex, UK.

Thériault, M., and Neale, K.W. 2000. Design equations for axially loaded reinforced concrete columns strengthened with fibre reinforced polymer wraps. Canadian Journal of Civil Engineering, **27**(5): 1011–1020.

Vogel, H.M. 2005. Thermal compatibility of bond strength of FRP reinforcement in prestressed concrete applications. M.Sc. thesis, Department of Civil Engineering, University of Manitoba, Winnipeg, Man.