

Effect of freeze–thaw cycles on the bond durability between fibre reinforced polymer plate reinforcement and concrete

Mark F. Green, Luke A. Bisby, Yves Beaudoin, and Pierre Labossière

Abstract: Deterioration of infrastructure is one of the most pressing concerns facing today's civil engineering community. As a result, new rehabilitation techniques, such as the external bonding of fibre reinforced polymer (FRP) plates and sheets, are being developed. To apply these rehabilitation methods in Canada, the durability of the rehabilitated structures in cold regions must be assessed. This paper examines the effects of freeze–thaw cycling on the bond between FRP and concrete. An experimental investigation was conducted using both single lap pull-off and bond beam specimens. Only uniaxial carbon FRP strips were considered. The specimens were exposed to up to 300 freeze–thaw cycles consisting of 16 h of freezing and 8 h of thawing in a water bath. After exposure, the specimens were tested to failure. The development of strain along the bond length and the failure mode are presented for both types of specimens. Load deflection curves are presented for the beam specimens. The results indicate that the bond between carbon FRP strips and concrete is not significantly damaged by up to 300 freeze–thaw cycles.

Key words: reinforced concrete, repair, rehabilitation, strengthening, fibre reinforced polymers, freeze–thaw, bond damage.

Résumé : La détérioration des infrastructures est un des soucis les plus pressants se posant à la communauté du génie civil d'aujourd'hui. En conséquence, de nouvelles techniques de réhabilitations, telles que la liaison externe de plaques et de feuilles de polymère renforcées de fibres (PRF), sont en cours de développement. Pour appliquer ces méthodes de réhabilitation au Canada, la longévité des structures réhabilitées dans les régions froides doit être évaluée. Cet article examine les effets des cycles de gel–dégel sur le lien entre les PRF et le béton. Une investigation expérimentale a été conduite en utilisant et des spécimens de blocs tirés en une étape et des spécimens de poutres liées. Seul les bandes de PRF de carbone uni-axiales ont été considérées. Les spécimens ont été exposés à jusqu'à 300 cycles de gel–dégel se composant de 16 heures de congélation et de 8 heures de dégel dans un bain d'eau. Après exposition, les spécimens ont été testés jusqu'à rupture. Le développement de déformations le long de la longueur du lien et le mode de rupture sont présentés pour les deux types de spécimens. Les courbes de déflexion due au chargement sont présentées pour les spécimens de poutres. Les résultats indiquent que le lien entre les bandes de PRF de carbone et le béton n'est pas endommagé de manière significative sous l'effet de jusqu'à 300 cycles gel–dégel.

Mots clés : béton armé, réparation, réadaptation, renforcement, polymères renforcées de fibres, gel–dégel, dommages aux liens.

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1. Introduction

In the industrialized countries of the modern age, a highly developed infrastructure is essential for economic growth and prosperity. Many structures essential to this infrastructure, especially those made of reinforced concrete, have suf-

fered severe degradation since their construction due to the combined effects of deicing salts, aggressive environments, and drastically increased live loads. One of the major problems facing the civil engineer of today is to preserve, maintain, and repair this infrastructure. Development and implementation of new, cost-effective repair and rehabilitation methods are required to prolong the useful service life of deteriorated structures.

One repair method, which has received widespread attention in the past 10 years, is external plating with adhesive bonded fibre reinforced polymer (FRP) plates and sheets. A large amount of laboratory research and field work has demonstrated that FRPs can be very useful for external flexural and shear reinforcement of reinforced concrete structures (Chajes et al. 1996). Fibre reinforced polymer plating has now been investigated for strengthening of structures all over the world (Chajes et al. 1996; Nanni 1997). One of the most pressing problems with this technique, however, is premature plate separation due to bond failure, causing cata-

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strophic failure of the reinforced concrete member. The types, causes, and mechanisms of bond failure are complex and varied, and a considerable research effort has focused on characterization of the bond forces, bond development lengths, and failure mechanisms of FRP plates bonded to concrete, in an effort to prevent these undesirable and dangerous failure modes. One area of concern is that temperature-induced stresses in the adhesive layer, due to differential thermal expansion and contraction of the FRP and substrate concrete, as well as the action of water under freeze-thaw cycling, may cause bond damage and contribute to premature bond failure. To date little, if any, research has focused on the effect of freeze-thaw cycling specifically on the anchorage of FRP plates and sheets bonded to concrete.

The objective of this paper is to investigate experimentally the effect of freeze-thaw cycling on the anchorage zone of carbon fibre reinforced polymer plates bonded to reinforced concrete beams.

2. Literature review

2.1. Fibre reinforced polymer plated beams

Fibre reinforced polymer plates and sheets offer an extremely efficient alternative to steel plates as external reinforcement for rehabilitation and strengthening applications. Since FRPs were first investigated as a plating material in Switzerland in the 1980s (Meier et al. 1992), there has been a great deal of research into their structural behaviour. Major research studies have been performed in Europe (Deuring 1993; Lane et al. 1997; Meier and Winistorfer 1995; Quantrill et al. 1995; Rostasy et al. 1992; Swamy and Mukhopadhyaya 1995; Varastehpour and Hamelin 1995), Canada (Alexander and Cheng 1996; Bizindavyi and Neale 1997; Chaallal et al. 1997; Green et al. 1997; Heffernan et al. 1996; Shehata et al. 1997) and the United States (An et al. 1991; Arduini and Nanni 1998; Chajes et al. 1996; Malek et al. 1998; Malvar et al. 1995; Nanni 1997; Plevris et al. 1995; Saadatmanesh and Ehsani 1990; Triantafillou 1998). Field applications using FRPs to strengthen reinforced concrete structures have been implemented in Europe (Steiner 1996; Nanni 1997), Japan (Ichimasu et al. 1993), and more recently North America (Labossière et al. 1997; Chajes et al. 1996).

To summarize, the overall effects of plating for flexure on a reinforced concrete beam can be stated as follows, assuming that premature failure mechanisms are avoided:

- increased flexural capacity under both service and ultimate load conditions. This is reflected in increased cracking, yield, and ultimate moments.
- finer and more evenly distributed cracks in the concrete. Also, the total sum of the crack widths is reduced in a plated beam.
- slightly increased post-cracking flexural stiffness
- decreased ductility. The decrease in ductility may be due to a change from an under-reinforced section to an over-reinforced section.

It has been shown that the load-deflection behaviour of a FRP plated reinforced concrete beam can be adequately predicted by a classical plane-sections analysis, as long as premature peeling or debonding of the FRP is avoided (Kaiser 1989; An et al. 1991).

2.2. Anchorage of fibre reinforced polymer sheets or plates

Although FRP plating has shown itself to be an excellent repair and rehabilitation technique, research indicates that, without specialized anchorage details at the ends of bonded FRP sheets, premature failure will occur because of debonding or peeling of the plate at its ends. The significance of these premature failure modes is evident in the literature, because an enormous amount of research effort has focused on determination of the types, causes, and prevention of premature failure. Premature failure modes that have been identified in the literature include plate-concrete interface failure, interface crack propagation, sheet peeling due to shear cracking, sheet peeling due to flexural cracking, concrete-shear delamination, and glue bond failure. Various attempts have been made to characterize the behaviour of these failure mechanisms so that a rational design approach can be formulated.

Premature failure modes in reinforced concrete members plated with both steel and FRP have been commented on extensively in the literature (Swamy et al. 1987; Roberts 1989; Ziraba et al. 1994; Oehlers 1989; Oehlers and Moran 1990; Triantafillou and Plevris 1992; Oehlers 1992; Ali and Oehlers 1997; Kaiser 1989). Examination of the various premature failure modes in concrete leads to the conclusion that premature failure usually initiates at the end of the plate or sheet because of the geometric singularity created by termination of the plate. This behaviour suggests that proper detailing of the plate end and supplementary plate anchorage techniques should be used. A wide variety of plate termination details and end-anchorage techniques have been studied in the literature. Some of the anchorage details studied to date include limiting the plate width-to-thickness ratio (Swamy et al. 1987), successive termination of the plate material (Wight et al. 1995), anchor bolts (Sharif et al. 1994), glued angle plates or transverse FRP wraps (Arduini et al. 1995; Erki and Heffernan 1995), installation of side plates (Ali and Oehlers 1997), and one-piece FRP I-sheets (Sharif et al. 1994). Nonetheless, in many cases, such as slab strengthening, supplemental anchorage may not be a practical option.

Because of premature failure due to delamination or peeling-off of FRP sheets bonded to concrete, it is often very difficult to develop the full strength of the composite material. Thus, many investigations have been conducted to determine required lengths of anchorage for FRPs bonded to concrete.

Chajes et al. (1996) conducted a series of FRP pull-off tests using graphite, E-glass, and aramid sheets. The major conclusions reached in this study were

- Surface preparation significantly affects bond strength. Mechanical abrasion or sandblasting, followed by a primer coat of resin, provides the best bond.
- The use of ductile adhesives (having low stiffness and high strain at failure) is not recommended.
- If the failure mode is governed by shearing of the concrete below the glue line, the ultimate bond strength is proportional to the square root of the concrete compressive strength.
- The strain distribution along the bonded length decreases at a fairly linear rate. Thus, there is a length of joint for

which no measurable increase in joint strength is observed.

Bizindavyi and Neale (1997, 1999) conducted a theoretical and experimental investigation into FRP-concrete bonded joints. After conducting over 60 single-lap pull-off tests on GFRP and CFRP sheets bonded to $150 \times 150 \times 400$ mm concrete blocks, they developed a relatively simple shear lag model that could be accurately used to determine strains along the bond for single lap joints. Some of the parameters varied included plate thickness, glue thickness, and length of joint. Experimental results were found to correlate well with theoretical values at service load levels. However, bond strains at high load levels were not accurately predicted because of cracking, which was not accounted for in the model. Empirical expressions for the transfer lengths of composite sheets as functions of the relative load levels were also developed. It was shown that the initial transfer length of the sheets is a fraction of the total bond length and that the remaining portion of the bond allows the transfer of loads in excess of service loads. Bizindavyi and Neale were able to achieve the full tensile capacity of the composite sheets in their tests.

2.3. Cold climate research on fibre reinforced polymer plated beams

The vast majority of concrete structures that require strengthening or rehabilitation are exposed to severe environmental conditions. Many of these severe environmental conditions are the result of cold climate conditions such as low temperature, freeze-thaw action, and exposure to deicing salts. Because of this, the environmental durability of both the materials and methods used in rehabilitation applications is of utmost importance, especially in aggressive climates such as those found in Canada. However, very little research has been performed relating to the environmental durability of FRP plated members.

Kaiser (1989) performed a series of freeze-thaw tests on beams plated with CFRP sheets. He found that there were no detrimental effects on the overall structural performance of beams tested after 100 cycles from $+25^{\circ}\text{C}$ to -25°C . Baumert et al. (1996) investigated the effect of extreme cold on the structural performance of FRP plated beams. These tests showed that, for CFRP plated beams exposed to a temperature difference of $+21^{\circ}\text{C}$ to -27°C , there were no adverse effects on the structural behaviour of beams when subjected to a static load. Green et al. (1997) also conducted a series of tests to investigate the freeze-thaw durability of beams strengthened with CFRP sheets. The beams were subjected to 50 freeze-thaw cycles from -18°C to 15°C . It was determined that freeze-thaw cycling did not adversely effect the strength of the concrete beams, the FRP sheets, or the FRP to concrete bond (although no specific effort was made to study bond behaviour). Tysl et al. (1998) studied the effect of surface delamination on the freeze-thaw durability of CFRP plated reinforced concrete beams. It was found that neither freeze-thaw cycling nor partial surface delamination had a diminishing effect on the overall load deflection response of the plated beams.

Very little work has been done on the effects of freeze-thaw cycling specifically on FRPs. Daniel and Ishai (1994) state that, since the fibres in FRPs are generally least sensi-

tive to the environment, thermal effects are most noticeable in matrix dominated properties, such as transverse tensile and compressive strength, and in-plane shear. Longitudinal tensile strength is not considered significantly affected by temperature effects. Tests conducted by Dutta (1988), where FRPs were subjected to 150 freeze-thaw cycles from $+23^{\circ}\text{C}$ to -40°C , however, showed that the tensile strength of glass-epoxy FRP was reduced by about 10% because of freeze-thaw cycling. Similar tests on carbon-epoxy FRPs did not show any significant reduction in tensile strength or elastic modulus due to freeze-thaw cycling. Thermal cycling did, however, produce significant degradation of off-axis properties for CFRP.

For concrete, deterioration due to freeze-thaw is caused by freezing of porewater inside the concrete. If the pores are too small, then the expansion caused by freezing can exert stresses on the concrete that crack the concrete and thus cause deterioration. Air entrainment of 7 to 8%, depending on the aggregate size, can essentially eliminate this freeze-thaw damage (Neville 1995).

2.4. Summary

A review of the literature indicates that a considerable research effort has been directed towards the use of FRPs for structural strengthening and rehabilitation applications in civil engineering. The benefits of using FRPs as opposed to steel are hard to ignore, even given their relatively high material cost. The technique of FRP plating is now widely recognized within the research community and is slowly gaining acceptance in the construction industry, with hundreds of field applications worldwide. However, until a rational design approach for the use of FRPs in construction is formulated and all of the durability concerns are addressed, the technique will never gain widespread acceptance. There is, at present, little information on the anchorage of FRPs to concrete and on the most important design factors to ensure a quality bond such that the full capacity of the composite can be developed. This paper investigates the effect of freeze-thaw cycling specifically on the anchorage and force development behaviour of a commonly used, commercially available FRP plate bonded to reinforced concrete members.

3. Experimental procedure

The experimental program conducted to determine the effects of freeze-thaw cycling on the anchorage of Sika Carbodur CFRP plates to concrete was divided into two main phases. One portion of the work, performed at the Université de Sherbrooke, used single lap CFRP-concrete joints loaded in shear. The other portion, conducted at Queen's University, used modified flexural specimens to examine similar bond parameters. The two different specimen types were crucial in order to first examine FRP-concrete joints loaded in pure shear and then joints loaded in both shear and flexure, such that the results of the study could be applicable to both shear and flexural strengthening applications.

Freeze-thaw cycling of all specimens was conducted using the cold climate testing facilities at Queen's University. Freeze-thaw cycles were applied to the blocks at a rate of one cycle per day, in accordance with ASTM C310 (1971),

Table 1. Properties of CFRP plates.

Property	Sika carbodur S512
Fibre volumetric content (%)	68
Apparent density (kg/m ³)	1 600
Modulus of elasticity (MPa)	>155 000
Tensile strength (MPa)	>2 400
Elongation at break (%)	>1.9
Thickness (mm)	1.2
Width (mm)	50

with 16 h of freezing in cold air at -18°C followed by 8 h of thawing in a warm water bath at $+15^{\circ}\text{C}$. Specimens that were not subjected to freeze–thaw cycling were stored in the structures testing laboratory at room temperature and relative humidity. The specimens were divided into groups of three, with groups subjected to 0, 50, 150, or 300 freeze–thaw cycles.

3.1. Material properties

This study considered uniaxial pultruded Sika Carbodur CFRP strips. They have a strength of 2400 MPa, a modulus of 155 000 MPa, and a thermal expansion coefficient of $1.0 \times 10^{-6}/^{\circ}\text{C}$. Table 1 summarizes the guaranteed properties of the strips as provided by the manufacturer. The binding agent used was a proprietary two-part epoxy resin, also manufactured by Sika. Manufacturer specified properties of the binding agent are given in Table 2. Because the concrete specimens used in this study came from larger studies at two different universities, there were some differences in the concrete mixes used for the bond-beams and for the pull-off specimens. Material properties of the concrete mixes are given in Table 3. Tensile tests were performed on concrete cylinders after exposure to freeze–thaw cycles, and no significant differences in tensile strength due to cycling were observed.

3.2. Pull-off specimens

For the single lap pull-off tests, a total of 12 concrete blocks were used. The blocks, fabricated at the Université de Sherbrooke, had dimensions of 150 mm \times 150 mm \times 400 mm and a specified 28-day concrete strength of 35 MPa. Once the concrete blocks had been fabricated and moist cured for 14 days, the CFRP strips were adhered to the exterior of the blocks as shown in Fig. 1. The installation of the CFRP strips was performed using a specialized technique developed by the manufacturer, specifically for this material. First the blocks were lightly sandblasted, to just expose the coarse aggregate, since adequate surface preparation is crucial for a high quality bond. Then the plates were adhered to the surface of the blocks using a specially designed epoxy putty. A putty die was used to ensure a uniform adhesive thickness. The plates were bonded to the concrete blocks over a length of 300 mm. This length of bond was chosen based on the results of preliminary studies conducted at the Université de Sherbrooke that showed 300 mm to be the minimum length of bond for which tensile failure of the CFRP strip was observed in pull-off tests and on the fact that shear failure of the small beams was undesirable. The speci-

Table 2. Manufacturer specified properties of the binding agent.

Property (23°C and 50% R.H.)	Sikadur 30
Mixing ratio A:B	3:1 by weight
Density (kg/L)	1.77
Pot life (min)	70
Tensile properties (ASTM D 638 @ 7-days)	
Tensile strength (MPa)	24.8
Elongation at break (%)	1
Tensile modulus (MPa)	4500
Flexural properties (ASTM D 790 @ 14-days)	
Modulus of rupture (MPa)	46.8
Tangent modulus of elasticity (MPa)	11 700
Shear strength (ASTM D 732 @ 14-days) (MPa)	24.8
Bond strength (ASTM C 882)	
2-days moist cure (MPa)	17.9
2-days dry cure (MPa)	20.6
4-days moist cure (MPa)	17.9
Deflection temperature (ASTM D 648 @ 7-days) ($^{\circ}\text{C}$)	47
Water absorption (ASTM D 570 @ 24-hours) (%)	0.03
Compressive strength (ASTM D 695 @ 23°C and 14-days) (MPa)	59.3
Modulus of elasticity (ASTM D 695 @ 7-days) (MPa)	26 900

Table 3. Concrete mix properties for bond-beam and pull-off specimens.

Concrete batch	28-day compressive strength (MPa)	Air entrainment (%)	Slump (mm)
Bond beams	31	6	100
Pull-off blocks	31	6	N/A

mens were then cured for at least 12 days prior to the onset of freeze–thaw cycling.

After being subjected to the appropriate number of freeze–thaw cycles, the specimens were returned to the Université de Sherbrooke where they were tested using a 90 kN MTS testing machine. Figure 2 shows the experimental apparatus used to apply a tensile load to the FRP plates such that the FRP–concrete interface was subjected to pure shear. During the course of the loading, the elongation and slip of the FRP plate were monitored with displacement transducers mounted against the blocks. The total applied load was monitored with a load cell and the strains in the FRP along the length of joint were monitored with electrical resistance strain gauges placed as shown in Fig. 1. All experimental data were recorded automatically using a data acquisition system running on a personal computer.

3.3. Bond beam specimens

In an effort to study the effects of freeze–thaw cycling on the behaviour of the FRP–concrete bond subjected to a combination of flexure and shear, as would be the case in flexural repair and strengthening applications, bond beams were

Fig. 1. Specimen configuration for pull-off tests.

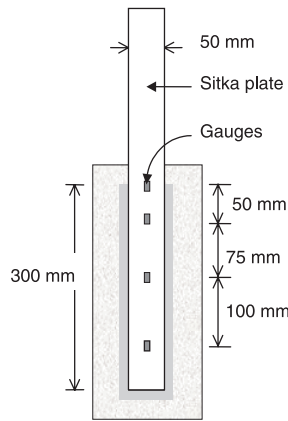


Fig. 2. Pull-off test experimental apparatus.



fabricated and tested at Queen’s University. Nine small-scale (100 mm × 150 mm × 1220 mm) reinforced concrete beams were fabricated, with overall dimensions and internal reinforcing steel layout as shown in Fig. 3. Once the beams were fabricated and cured, a single plate, 1020 mm long, was adhered to the tension face of each beam. The layout of the plate on the beams is shown in Fig. 4. Note that the plates were bonded only over a short length (300 mm) at either end of the beams and remained unbonded in the central region. This was done to promote a debonding failure mode of the FRP, rather than a tensile failure of the FRP material, when the beams were later subjected to four point bending. Installation of the plates was accomplished in the same manner as for the pull-off specimens with the exception that a plastic sheet was used as a bond breaker in the central portion of the beams. The bond breaker was removed prior to freeze–thaw cycling. The bonded FRP plates were allowed

Fig. 3. Details of beams: (a) cross-section, and (b) longitudinal details.

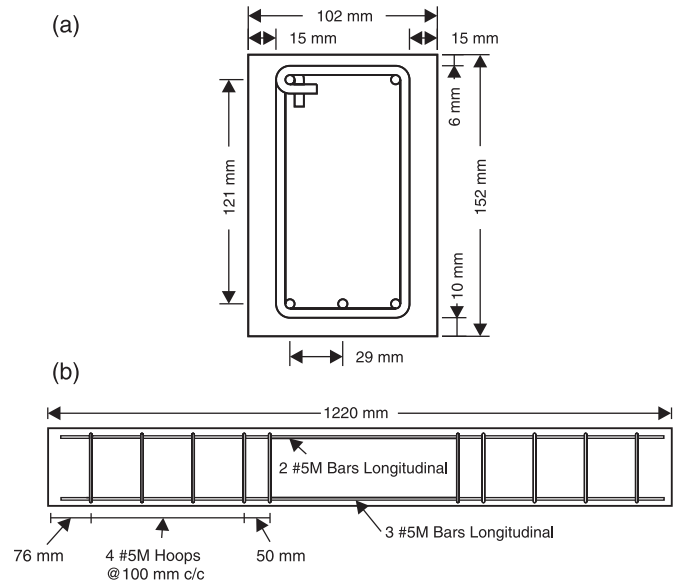
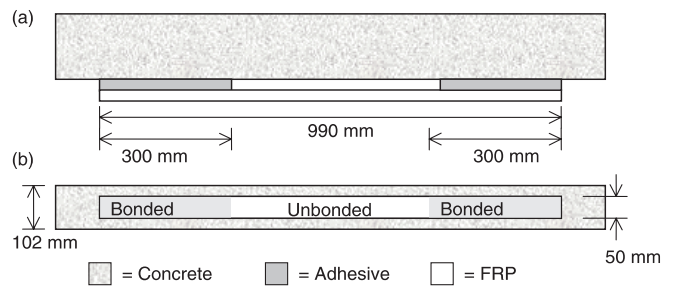


Fig. 4. Bond and FRP layout for Sika beam group: (a) side view, and (b) bottom view.



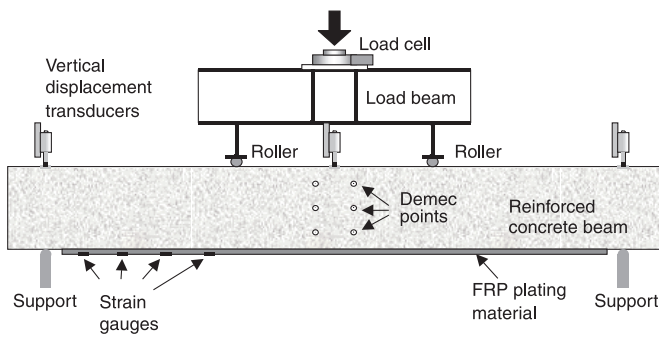
to cure for at least 12 days, at which point the beams were subjected to the same freeze–thaw cycling regime as the single lap pull-off specimens.

After being subjected to the appropriate number of freeze–thaw cycles, the beams were tested to failure in four point bending using a Reihle 9000 kN testing machine in the Structures Testing Laboratory at Queen’s University. The test setup and instrumentation are shown in Fig. 5. Test parameters monitored included (i) total vertical applied load, monitored with a load cell; (ii) midspan and support displacements, monitored using vertical displacement transducers; (iii) strains in the concrete at midspan, monitored using Demec gauges; and (iv) strains in the FRP along the length of the bond, monitored using electrical resistance strain gauges. All measurements, except the concrete strain readings, were recorded automatically using a data acquisition system running on a personal computer.

4. Results

Experimental results for both types of test specimen are discussed separately in the following two sections. The reader should keep in mind that the goal was to determine

Fig. 5. Schematic showing experimental setup for flexural tests.



the effect of cycling specifically on the bond behaviour, rather than the overall behaviour of the specimens.

4.1. Pull-off results

Table 4 presents a summary of results from pull-off tests conducted after 0, 50, 150 and 300 freeze-thaw cycles. This table presents results for ultimate load and maximum strain. It is evident that an increase in the ultimate strength of the pull-off specimens was observed with increased numbers of freeze-thaw cycles. Average strength increases for specimens subjected to 50, 150, and 300 freeze-thaw cycles, with respect to the control specimens, were 36%, 53%, and 54%, respectively. It is possible that this increase in strength is because the freeze-thaw cylinders were left in a water bath whereas the room temperature specimens were left in air for the duration of the cycling. This could contribute to enhanced curing of the substrate concrete in the specimens subjected to freeze-thaw cycling, although this hypothesis was not supported by tests on concrete cylinders subjected to freeze-thaw cycling.

After conducting a visual examination of the failure surfaces, it was noted that there were distinct differences between the control specimens and those subjected to freeze-thaw cycles. For the control specimens, the failure occurred within the substrate concrete. However, with increasing numbers of freeze-thaw cycles, the failure appeared to move into the epoxy binding agent, such that failure generally occurred at the FRP-epoxy interface. The apparent change in failure mode is shown in Fig. 6. It is thought that the change in failure surface may be due to a slight reduction in the shear modulus of the binding agent with freeze-thaw cycling, possibly reducing the magnitude of stress concentrations in the substrate concrete. This could theoretically explain both the increase in bond strength and the observed variation in failure mode. Tests specifically on the epoxy mortar would be required to validate this hypothesis.

Load-deformation curves for the composite plates were plotted and examined but are not included here. These curves did not demonstrate any significant differences in behaviour for specimens subjected to different numbers of freeze-thaw cycles.

Bond strain profiles for all specimens were also plotted and examined. Example bond strain distribution plots for one of the reference specimens at different load levels, demonstrating typical behaviour, are shown in Fig. 7. Bond strain distributions did not appear to be affected by freeze-thaw cycling. It is evident that much of the bond remains un-

Table 4. Results of pull-off tests.

Specimen name	# Freeze-thaw cycles	Ultimate load (kN)	Maximum strain in the plate ($\mu\epsilon$)
B0-Sika-1	0	35.73	3140
B0-Sika-2	0	35.44	3592
B0-Sika-3	0	30.53	1842
B50-Sika-1	50	49.54	4856
B50-Sika-2	50	43.07	4190
B50-Sika-3	50	45.73	4414
B150-Sika-1	150	49.18	—
B150-Sika-2	150	56.80	5338
B150-Sika-3	150	49.80	4486
B300-Sika-1	300	49.60	5332
B300-Sika-2	300	52.80	5104
B300-Sika-3	300	54.00	4790

used at service load levels and that strains develop near the free end of the bond only at very high load levels, once the bond has already begun to fail towards the loaded end (a horizontal portion of the bond strain distribution curve represents a debonded region of bond). This bond behaviour suggests that the critical factor in the FRP-concrete anchorage is not necessarily the required length of joint, but the allowable strain in the FRP or, possibly, the maximum allowable strain gradient at the loaded end of the joint. Once the failure condition is reached, a progressive unzipping of the bond from the loaded end towards the free end will occur. A consequence of this behaviour is that, without some form of supplementary anchorage detail, it would be very difficult to use the full strength of the composite.

Using measured bond strains, it is possible to approximate the average bond shear stress between two strain gauges using the formula

$$[1] \quad f_b = \frac{\epsilon_{n+1} - \epsilon_n}{\Delta x} E_p t_p$$

where, f_b = bond stress (MPa), ϵ_n = strain at a particular strain gauge, t_p = thickness of sheet (mm), E_p = tensile modulus of FRP sheet (MPa), and Δx = distance between gauges (MPa).

Figure 8 shows a plot of average bond shear stress as a function of the normalized applied load for one of the reference pull-off specimens. This plot again demonstrates the fact that, at least at service load levels, a large portion of the bond remains unused and that bond stress is developed near the free end only at very high load levels.

The development of bond stress did not appear to be significantly affected by freeze-thaw cycling and was similar for all specimens tested. However, it was observed that the calculated average bond stress values were generally higher for specimens exposed to increased numbers of freeze-thaw cycles. This behaviour is consistent with the higher observed ultimate loads mentioned previously.

Overall, the FRP-concrete bond tested in pure shear did not appear to be damaged by freeze-thaw cycling.

4.2. Beam test results

Results of flexural tests on the plated beams are presented in Table 5. Some of the trends indicated in this data are in-

Fig. 6. Failure surfaces for pull-off tests: (a) 0 cycles, (b) 50 cycles, and (c) 150 cycles.

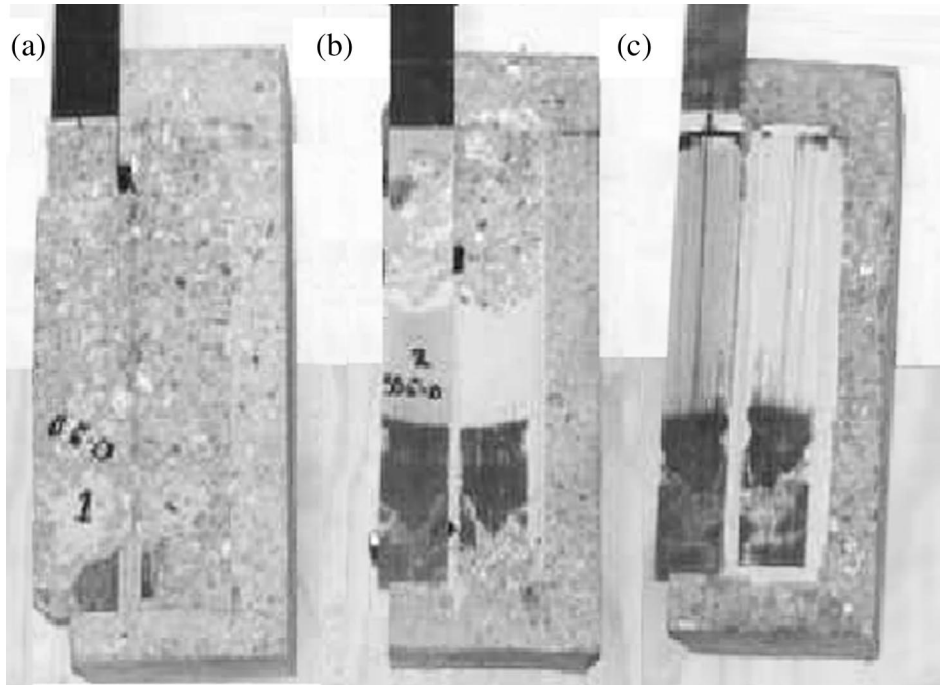
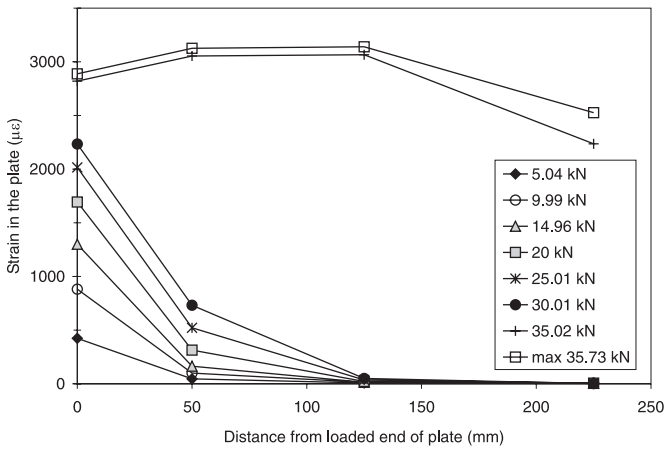


Fig. 7. Bond strain profiles for a typical pull-off specimen.



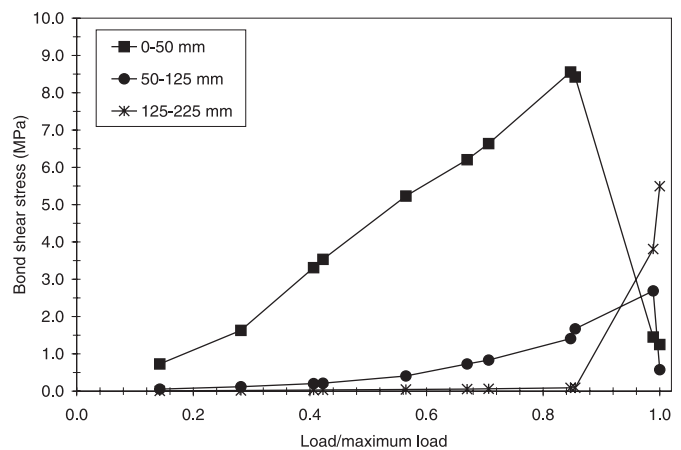
creased ultimate load and midspan deflection with increased numbers of freeze–thaw cycles, increased maximum FRP strain prior to bond damage, and increased average bond stress at failure with increasing numbers of freeze–thaw cycles. No significant trends were observed in the first crack load or deflection for this beam set. The average bond stress at failure was calculated using,

$$[2] \quad \tau_{\text{bond ave}} = \frac{\text{force in plate at loaded end}}{\text{area of bond}} = \frac{\epsilon_{p \text{ max}} E_p A_p}{L b_p}$$

In the above equation, $\tau_{\text{bond ave}}$ is the average bond stress at failure, $\epsilon_{p \text{ max}}$ is the maximum strain in the plate at failure, E_p is the elastic modulus of the plate material, A_p is the cross-sectional area of the plate material, L is the bond length, and b_p is the plate width.

Figure 9 shows load versus midspan deflection plots for three typical plated beams tested. The load–deflection curves

Fig. 8. Bond shear stress at different locations as a function of the relative load for a typical pull-off specimen.

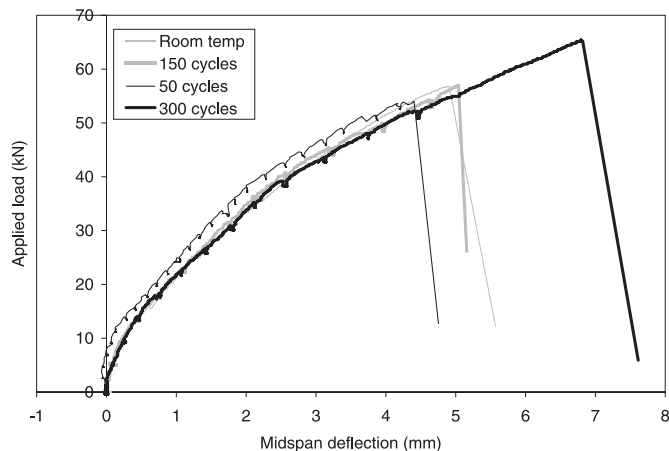


are essentially linear up until first cracking. After cracking, the curves remain essentially linear with a reduced slope, reflecting the reduced stiffness of the cracked beam section. The slope of the curve changes again at a load of about 35 kN where the internal steel yields in tension. The smoothness of the curves is interrupted occasionally by small dips; these represent intervals when the load was halted momentarily in order to take concrete strain readings. All curves display fairly similar behaviour to one another and do not appear to be affected by freeze–thaw cycling, with the exception that the beams subjected to 300 cycles carried up to 15% more load than the other beams.

Overall crack patterns and failure modes for all nine plated beams were very similar to one another and did not appear to be affected by freeze–thaw cycling. Figure 10 shows a typical crack pattern and failure mode. The failure

Table 5. Results of bond beam flexural tests.

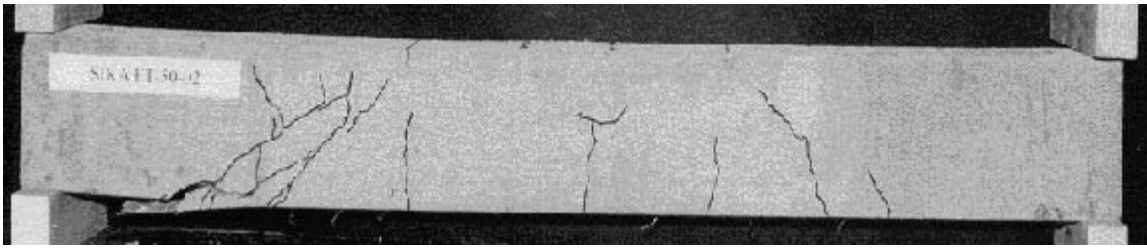
Beam name	Incident	Load (kN)	Force in plate (kN)	Midspan deflection (mm)	Max. strain in FRP plate ($\mu\epsilon$)	Ave. bond stress at failure (MPa)
RT-01	Cracking	11.9	—	0.4	—	—
	Failure	48.5	33.2	3.9	3290	2.0
RT-02	Cracking	13.1	—	0.4	—	—
	Failure	56.8	38.5	4.9	3820	2.4
RT-Ave.	Cracking	12.5	—	0.4	—	—
	Failure	52.6	35.8	4.4	3550	2.2
FT-50-01	Cracking	12.0	—	0.1	—	—
	Failure	54.0	38.7	4.4	3840	2.4
FT-50-02	Cracking	N/A	—	N/A	—	—
	Failure	54.1	37.2	4.8	3690	2.3
FT-50-Ave.	Cracking	12.0	—	0.1	—	—
	Failure	54.1	38.0	4.6	3770	2.3
FT-150-01	Cracking	9.20	—	0.2	—	—
	Failure	57.0	40.8	5.0	4050	2.5
FT-150-02	Cracking	11.1	—	0.3	—	—
	Failure	58.8	41.0	5.2	4070	2.5
FT-150-Ave.	Cracking	10.1	—	0.2	—	—
	Failure	57.9	40.9	5.1	4060	2.5
FT-300-01	Cracking	13.5	—	0.4	—	—
	Failure	65.2	51.3	6.8	5090	3.2
FT-300-02	Cracking	12.9	—	0.4	—	—
	Failure	64.2	50.1	6.9	4970	3.1
FT-300-03	Cracking	13.0	—	0.4	—	—
	Failure	59.3	43.2	5.5	4290	2.7
FT-300-Ave.	Cracking	13.1	—	0.4	—	—
	Failure	62.9	48.2	6.4	4780	3.0

Fig. 9. Load vs. deflection curves for all Sika plated flexural specimens.

mode for all plated beams essentially consisted of a peeling away of the layer of concrete between the FRP plate and the internal reinforcing steel. This failure was induced by the formation of large shear cracks in the shear region of the beams, near the termination of the plate. This type of failure is commonly referred to in the literature as a shear peeling failure mode and has been extensively studied by Oehlers and Moran (1990). As was the case for the pull-off tests, the failure surface of the FRP-concrete bond appeared to

change with increased numbers of freeze-thaw cycles. After 50 freeze-thaw cycles, the failure surface had not visibly changed as compared with the room temperature specimens and showed failure within the substrate concrete such that the FRP-concrete bond itself remained intact. After 150 cycles, peeling away of the concrete between the plate and internal reinforcement was accompanied by zones of debonding along the FRP-epoxy interface and failure within the plate material itself. After 300 cycles, larger portions of the failure surface featured FRP-epoxy debonding and failure within the FRP material. These slight changes in failure mode were difficult to reconcile, since the failure load of the beams increased with increasing numbers of freeze-thaw cycles. Again, one possible explanation for the change in failure mode is that the epoxy binding agent may be damaged by increased exposure to freeze-thaw cycling. Hence, a load re-distribution may occur at high load levels, reducing the effect of stress concentrations at the FRP-concrete interface and changing the position of the failure surface from the concrete to the FRP.

The development of strain along the bond is of some interest. Figure 11 shows a typical plot giving strain development along a single bond length for increasing applied load. The strain development did not appear to be affected by increased freeze-thaw cycling. It is evident that, at low load levels, the strains increase gradually from the free end of the bond and then increase rapidly over a very short length of joint (near the loaded end). At higher load levels, the strain

Fig. 10. Typical crack pattern and failure mode.

increases more rapidly near the free end of the bond, then flattens in the central region of the joint, and may even decrease before again increasing rapidly toward the loaded end. The flattening behaviour of the bond strain profiles at higher load levels is associated with the formation of shear cracks, which eventually cause failure, in the concrete above the glue line. These shear cracks may cause a localized debonding of the plates, which would explain the lack of strain increase in these regions.

As was the case for the pull-off specimens, no deterioration of the bond behaviour of the CFRP plates bonded to concrete due to freeze–thaw exposure was observed.

5. Discussion

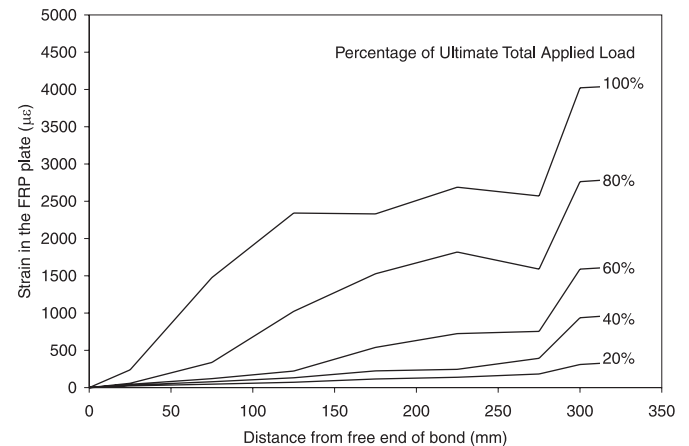
The main results to compare in this study are the maximum strain in the FRP plate and the failure modes.

Comparing Tables 4 and 5 shows that both tests were consistent in the maximum strain in the FRP plate obtained before failure. For both tests, the lowest values of maximum strain occurred for the specimens kept at room temperature and the highest for those exposed to 300 freeze–thaw cycles. In general, though, the strains reached in the beam tests were lower than those achieved in the corresponding pull-off tests. This result is expected since the pull-off tests were conducted in pure shear whereas the beam tests incorporated both flexure and shear. Because of these combined effects, higher shear stresses existed in the beams when compared to the pull-off specimens at the same strain level in the plate.

In both the pull-off tests and the beam tests, the failure changed with increasing numbers of freeze–thaw cycles. Without freeze–thaw cycles, the failure surface was totally in the substrate concrete, but as the number of cycles increased, the failure surface started to move into the adhesive and even to the plate. The most reasonable explanation is that the adhesive is affected slightly by freeze–thaw, thus reducing the shear stresses on the concrete. The plate can then take more load because these lower shear stresses allow the load to be distributed over a larger portion of the bonded surface.

6. Conclusions

This paper presents the results of a collaborative test program aimed at investigating the effects of freeze–thaw cycling on the bond between CFRP strips and concrete. The results are applicable to strengthening and repair applications and indicate that freeze–thaw cycling does not reduce the load carrying capacity of the joint between concrete and CFRP plates when the joints are loaded either in pure shear or in a combination of shear and flexure. Nevertheless, the

Fig. 11. Typical strain profiles for a single Sika bond length at different load levels (FT-150-01).

adhesive may be affected slightly by freeze–thaw exposure resulting in changes in failure modes.

7. Further work

The experimental work presented here represents a first step in the study of FRP–concrete bond durability. Studies considering a variety of other factors are required to establish adequate durability and design data. For instance, studies using deteriorated (or extensively precracked) members would be useful. An investigation into the effects of freeze–thaw cycling under sustained load is recommended. Also, tests on different epoxy binding agents under the effects of freeze–thaw cycling are needed.

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