Abstract

Fibre-reinforced polymers (FRPs) are becoming increasingly accepted in structural engineering applications. In particular, Carbon-FRP (CFRP) tendons are proving to be promising as prestressing reinforcement for concrete structures. While several studies have been conducted on CFRP-prestressed concrete beams, very little attention has been given to their long-term behaviour at low temperatures. This thesis investigates the behaviour of CFRP-prestressed concrete beams in two studies: (a) under sustained loading at low temperature, and (b) under high-cycle fatigue at low temperature. Seven 13-year-old, 4.4 m long precast concrete T-beams were tested, of which five were prestressed to various levels with CFRP tendons and two with conventional steel strands.

In the first study, three beams were exposed to \(-27\,^\circ\text{C}\) while being subjected to a sustained load of 25\% of their flexural capacity for 163 days. The sustained load produced cracking in two beams with lower prestress levels. Results were compared to those obtained from three similar beams subjected to the same sustained load at room temperature. Deflection increase under sustained load at low temperature was generally small and similar to that at room temperature. Prestressing strain had a direct relationship with temperature in the CFRP-prestressed beams.

After being subjected to sustained loading, all seven beams were tested in the second study. Only three of the five CFRP-prestressed beams were subjected to cyclic loading, one at \(-28\,^\circ\text{C}\) and two at room temperature, while only one of the two steel-prestressed beams was subjected to cyclic loading, at \(-28\,^\circ\text{C}\). Cyclic loading consisted of 3 million
cycles at a frequency of 0.85 Hz. The load range represented 21 to 42% of the flexural capacity of the CFRP-prestressed beams and 30 to 60% of that of the steel-prestressed beam. Monotonic tests were run every 1 million cycles. Finally, all seven beams were monotonically loaded to failure. All CFRP-prestressed beams survived the 3 million cycles but the steel-prestressed beam failed after 185,000 cycles. However, the CFRP-concrete bond was weakened by high prestress levels, cyclic loading, and low temperature during sustained loading and loading to failure. This resulted in bond failure at loads ranging from 69 to 91% of the full flexural capacity. Stiffness and camber gradually decreased during cyclic loading.
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Firstly, I would like to thank my supervisors, Prof. Amir Fam and Prof. Mark Green. Specifically, I am thankful to Prof. Amir Fam for his proactive style of supervision, for always being available to answer my questions and for inspiring me with his passion for experimental research. I am grateful to Prof. Mark Green for his generosity, heartening encouragement and problem-solving technical insights. Additionally, I have greatly benefited from the invaluable teachings of both supervisors through courses on reinforced concrete, prestressed concrete and fibre-reinforced polymers.

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## Notation

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<th>Description</th>
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<tbody>
<tr>
<td>AEMM</td>
<td>Age-adjusted Effective Modulus Method</td>
</tr>
<tr>
<td>AFRP</td>
<td>Aramid Fibre-Reinforced Polymer</td>
</tr>
<tr>
<td>CFCC</td>
<td>Carbon Fibre-Composite Cable</td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon Fibre-Reinforced Polymer</td>
</tr>
<tr>
<td>CMZ</td>
<td>Constant Moment Zone</td>
</tr>
<tr>
<td>CTE</td>
<td>Coefficient of Thermal Expansion</td>
</tr>
<tr>
<td>DA</td>
<td>Data Acquisition unit</td>
</tr>
<tr>
<td>DEMEC</td>
<td>Demountable Mechanical strain gauge</td>
</tr>
<tr>
<td>FRP</td>
<td>Fibre-Reinforced Polymer</td>
</tr>
<tr>
<td>GFRP</td>
<td>Glass Fibre-Reinforced Polymer</td>
</tr>
<tr>
<td>HSS</td>
<td>Hollow Structural Section</td>
</tr>
<tr>
<td>LP</td>
<td>Linear Potentiometer</td>
</tr>
<tr>
<td>M</td>
<td>Million, e.g. 3M cycles</td>
</tr>
<tr>
<td>PAN</td>
<td>Polyacrylonitrile</td>
</tr>
</tbody>
</table>

When used in beam names the following symbols refer to:

- **C**: CFRP-prestressed beams  
- **S**: Steel-prestressed beams  
- **LT**: Beams tested at Low Temperature  
- **RT**: Beams tested at Room Temperature  

- $d_b$: Tendon diameter  
- $E_c$: Modulus of elasticity of concrete  
- $E_{CFRP}$: Modulus of elasticity of CFRP tendons  
- $E_{steel}$: Modulus of elasticity of steel strands  
- $f_c'$: Specified compressive strength of concrete  
- $f_{pe}$: Effective prestress level  
- $f_{pu}$: Ultimate prestress level
<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
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<tbody>
<tr>
<td>$I_{cr}$</td>
<td>Cracked moment of inertia</td>
</tr>
<tr>
<td>$I_e$</td>
<td>Effective moment of inertia</td>
</tr>
<tr>
<td>$I_g$</td>
<td>Uncracked moment of inertia</td>
</tr>
<tr>
<td>$k_p$, $k_s$</td>
<td>Factors in deflection equation by Abdelrahman and Rizkalla (1999)</td>
</tr>
<tr>
<td>$L$</td>
<td>Free span of beam</td>
</tr>
<tr>
<td>$L_d$</td>
<td>Development length</td>
</tr>
<tr>
<td>$L_e$</td>
<td>Embedment length</td>
</tr>
<tr>
<td>$L_{fb}$</td>
<td>Flexural bond length</td>
</tr>
<tr>
<td>$L_t$</td>
<td>Transfer length</td>
</tr>
<tr>
<td>$M_{cr}$</td>
<td>Cracking moment</td>
</tr>
<tr>
<td>$M_{dc}$</td>
<td>Decompression moment due to prestressing force</td>
</tr>
<tr>
<td>$M_r$</td>
<td>Moment resistance</td>
</tr>
<tr>
<td>$M_s$</td>
<td>Service moment</td>
</tr>
<tr>
<td>$P_e$</td>
<td>Prestressing force</td>
</tr>
<tr>
<td>$y_{cr}$</td>
<td>Cracked centroidal distance</td>
</tr>
<tr>
<td>$y_e$</td>
<td>Effective centroidal distance</td>
</tr>
<tr>
<td>$y_g$</td>
<td>Uncracked centroidal distance</td>
</tr>
<tr>
<td>$\alpha_{fb}$</td>
<td>Factors in flexural bond length equation by Mahmoud et al. (1997)</td>
</tr>
<tr>
<td>$\Phi$</td>
<td>Used to refer to diameter of tendons, e.g. $8\Phi$: 8 mm in diameter</td>
</tr>
<tr>
<td>$\Delta e_{CFRP}$</td>
<td>Strain range in CFRP tendons under cyclic load</td>
</tr>
<tr>
<td>$\Delta \sigma_{CFRP}$</td>
<td>Stress range in CFRP tendons under cyclic load</td>
</tr>
<tr>
<td>$\Delta e_{steel}$</td>
<td>Strain range in steel strands under cyclic load</td>
</tr>
<tr>
<td>$\Delta \sigma_{steel}$</td>
<td>Stress range in steel strands under cyclic load</td>
</tr>
</tbody>
</table>
1.1 General

The corrosion of steel reinforcement in concrete structures costs world economies billions of dollars annually in repair and replacement (Bedard, 1992). One of the most effective solutions to overcome this problem is the use of corrosion resistant fibre-reinforced polymers (FRP) for prestressing reinforcement.

Fibre-reinforced polymers (FRP) are composite materials that typically consist of strong fibres embedded in a resin matrix. Common fibre types are glass, carbon and aramid. FRP composites can increase the service life of structures because they are non-corrosive. Additionally, FRPs are high-strength, lightweight, non-conducting and non-magnetic.

Prestressed concrete is a type of reinforced concrete in which reinforcement has been tensioned against the concrete. The high-strength prestressing tendons pre-compress the concrete prior to application of external loads. This significantly increases the external load required to crack the concrete, resulting in a stiffer member under service loads. Because prestressing can be used to control or eliminate cracking at service loads and to minimize deflections, it results in more slender structures. The most common use of prestressed concrete is in structural elements for bridges, buildings and parking structures.
Carbon-FRP (CFRP) is particularly suitable for prestressing applications due to its high modulus of elasticity, high resistance to alkalinity and excellent long-term and fatigue characteristics. CFRP tendons have successfully been used as prestressing reinforcement for a number of full-scale concrete bridges around the world.

Several experimental studies have been done on CFRP-prestressed concrete beams. However, most of these studies were carried out at room temperature. There is little information in the literature regarding the long-term behaviour of CFRP-prestressed concrete beams at low temperature, especially under sustained loading or high-cycle fatigue. The experimental program outlined in this thesis was undertaken to fill in these gaps.

This study is of particular significance to the world’s cold regions, where low temperatures persist for several months throughout the year. In the city of Yellowknife, Canada, for example, the average daily temperature is below −14 °C for 5 months every year with an average low of −31 °C in January (Environment Canada, 2009).

Furthermore, the few bridges in the world that incorporate CFRP-prestressed concrete beams are mostly located in cold regions such as Calgary, Manitoba and Michigan (Section 2.3.7). Therefore, the results of this study can be directly used to predict the long-term behaviour of these existing structures.

In this study, sustained loading represents dead loads. High-cycle fatigue represents the repeated application of live loads over the lifetime of a structure, e.g. the traffic over a bridge.
1.2 Objectives

This study aims to investigate the following aspects of the behaviour of CFRP-prestressed concrete beams by way of laboratory experiments:

1. The combined effect of sustained loading and low temperature in terms of:
   a. Ultimate capacity and failure mode
   b. Long-term deflections and prestressing strains

2. The combined effect of high-cycle fatigue and low temperature in terms of:
   a. Ultimate capacity and failure mode
   b. Fatigue life

1.3 Thesis Format

The contents of this thesis are as follows:

- **Chapter 2:** Literature review
- **Chapter 3:** Combined effect of sustained loading and low temperature on the behaviour of CFRP-prestressed concrete beams
- **Chapter 4:** Combined effect of high-cycle fatigue and low temperature on the behaviour of CFRP-prestressed concrete beams
- **Chapter 5:** Conclusions and Recommendations
- References
- Appendices
Chapters 3 and 4 are the main chapters and are presented in manuscript format. Each manuscript chapter is an independent document complete with an introduction, experimental program, results and discussion, and summary. Chapter 5 summarizes the conclusions from chapters 3 and 4 and provides recommendations for future research.
Chapter 2
Literature Review

2.1 Introduction

This chapter reviews existing literature that is directly relevant to the topic of this thesis (“Behaviour of CFRP-prestressed concrete beams under sustained loading and high-cycle fatigue at low temperature”) and the discussions that are made in the following chapters. First, CFRP is briefly introduced. Next, previous research on CFRP-prestressed concrete beams is presented with an emphasis on behaviour at low temperature and under sustained or cyclic loading. Finally, the behaviour of concrete alone is discussed under similar conditions.

2.2 CFRP

Fibre-reinforced polymers (FRP) are composite materials that typically consist of strong fibres embedded in a resin matrix. The fibres provide mechanical strength and stiffness to the composite and carry most of the applied loads. The matrix holds the fibres in place, protects them and provides for transfer of stress from fibre to fibre through shear stresses. Common fibre types are glass, carbon and aramid. Matrices are typically epoxies, polyesters, vinylesters or phenolics.
Fibre-reinforced polymer (FRP) composites are promising because they are high-strength, lightweight, non-corrosive, non-conducting and non-magnetic. They are more resistant to corrosion than steel, resulting in an increased service life of structures. They have high strength-to-weight ratios and strength properties greater than that of steel. In repair and rehabilitation, their light weight and ease of application can save labour costs. Also, FRPs can be manufactured using various unique cross-sectional shapes and material combinations. However, FRPs are relatively expensive. Therefore, their use is limited to applications where the advantages of the material can be beneficial (ACI, 2004). The market for FRPs in the construction industry has rapidly grown over the past decade and is projected to expand even further as fabrication costs decrease.

FRP prestressing tendons can be in the form of rods, bars or strands, typically made from glass, carbon or aramid fibres. Figure 2.1 depicts some typical examples. Carbon FRP (CFRP) tendons are made from polyacrylonitrile (PAN)-based or PITCH-based materials. Carbon fibres are based on the layered grapheme (hexagonal) networks present in graphite. Compared to other FRP products, carbon fibres have the highest tensile strength (up to 6,000 MPa), the highest modulus of elasticity (up to 300 GPa), and the lowest strain capacity (between 1.2 and 2%). Carbon fibre is about 5 to 10 times more expensive than glass fibre and epoxy-reinforced carbon composites are expensive to make. Some of the most widely used brands of CFRP tendons in the construction industry include Leadline by Mitsubishi Kasei (Japan), Carbon-Fibre Composite Cable (CFCC) by Tokyo Rope (Japan), Aslan 200 by Hughes Brothers (USA) and V-ROD by Pultrall (Canada).
2.3 CFRP-Prestressed Concrete Beams

This section presents the state-of-the-art on the behaviour of CFRP-prestressed concrete beams in flexure, at low temperatures and under sustained or cyclic loading. A method for calculation of development length for these beams is also introduced. This is followed by an explanation of the Hoyer effect in prestressed concrete beams. Finally, a few case studies of bridges that have CFRP as prestressing reinforcement are presented.

2.3.1 Flexural Behaviour

Fam et al. (1997) tested five I-girders reinforced using only CFRP for shear and prestressing, and one beam reinforced by conventional steel strands and steel stirrups. The beams were 1:3.6 scale models of bridges built in Manitoba, Canada. The CFRP-prestressed beams showed similar stiffness to the steel-prestressed beam from cracking up to yielding of the steel. Figure 2.2 illustrates the load-deflection behaviour of the beams, where the prestressing is CFCC in beams 1 to 3, Leadline in beams 4 and 5 and steel strand in beam 6. Harping of CFRP tendons proved practical and did not influence flexural capacity, although flexural failure could occur at the bent points.

Abdelrahman and Rizkalla (1997) conducted tests on eight beams partially prestressed by Leadline CFRP tendons and two beams partially prestressed by conventional steel strands. Cracking and deflections before and after cracking were closely monitored. Modes of failure were also studied. The findings were as follows.

- Figure 2.3 shows the load-deflection behaviour of the beams. It is observed that the two types of beams were similar in stiffness before cracking. After cracking,
however, CFRP-prestressed beams had lower stiffness and greater deflection than those prestressed with steel because of the lower elastic modulus of the CFRP tendons. When failure was controlled by crushing of concrete in the compression zone, the two types of beams had similar deflections. When the failure mode was rupture of prestressing reinforcement, on the other hand, CFRP-prestressed beams had much less deflection than their steel-prestressed counterparts.

- In the transfer zone, CFRP tendons and steel strands were found to be very similar in bond characteristics. In the flexural zone, on the other hand, CFRP tendons had lower bond strength than steel strands. This manifested itself in the lower number, greater width and larger spacing of cracks in the CFRP-prestressed beams.

Abelrahman and Rizkalla (1999) introduced simple methods to predict deflections for beams partially prestressed with CFRP reinforcement under short-term and repeated loading. These methods were then calibrated using experimental results from eight beams partially prestressed by Leadline CFRP tendons and two beams partially prestressed by conventional steel strands. The proposed equation to calculate deflections was

\[ \Delta = -k_p \frac{P_e (d_p - y_e) L^2}{E_c I_e} + k_s \frac{M_s L^2}{E_c I_e} \]  

This expression considers the deflection due to prestressing force, \( P_e \), and the deflection due to service moment, \( M_s \), due to dead and live loads. The free span of the beam is \( L \), \( d_p \) is the depth of prestressing reinforcement, \( E_c \) is the modulus of elasticity of concrete and \( k_p \) and \( k_s \) are factors that depend on the shape of the prestressing cables and the loading pattern. For a uniformly distributed load and straight prestressing tendons, the value of \( k_p \)
and $k_s$ are $1/8$ and $5/48$, respectively. The terms $y_e$ and $I_e$ are the effective centroidal distance and moment of inertia, respectively, and are calculated as follows:

$$y_e = \psi^3 y_g + (1-\psi^3)y_{cr} \leq y_g \quad (2-2a)$$

$$I_e = \psi^3 I_g + (1-\psi^3)I_{cr} \leq I_g \quad \text{Proposed by ACI Committee 435 (ACI, 1995)} \quad (2-2b)$$

where

$$\psi = \left(\frac{M_{cr} - M_{dc}}{M_s - M_{dc}}\right), \quad M_s > M_{cr} \quad (2-3)$$

Here, $y_g$ and $y_{cr}$ are uncracked and cracked centroidal distances, $I_g$ and $I_{cr}$ are uncracked and cracked moments of inertia, $M_{cr}$ is cracking moment, $M_s$ is service moment and $M_{dc}$ is decompression moment due to prestressing force.

Figure 2.4, adopted from Abdelrahman and Rizkalla (1999), illustrates the typical load-deflection response of beams partially prestressed by CFRP bars under repeated loading. This behaviour is similar to that for beams prestressed by steel, as long as the steel is in the elastic range before unloading. When the beam is loaded up to point B and then unloaded, the observed response (path BCD) is somewhere between perfectly elastic (path BAO) and perfectly inelastic (path BE). There is a residual deformation equivalent to OD which is proportional to the load at which the beam is unloaded. This behaviour was observed for all of the beams.

### 2.3.2 Behaviour at Low Temperature

Bryan and Green (1996) studied the short-term behaviour of concrete beams prestressed with 8 mm diameter Leadline CFRP tendons at low temperature. Six similar beams were tested to failure, three at room temperature (+24 °C) and three at low temperature.
(−27 °C). Table 2.1 lists the results. They concluded that the behaviour of the beams was unaffected by short-term exposure to this low temperature. Ultimate stresses and strains in the CFRP tendons exceeded those given by the manufacturer and were not affected by the low temperature. The average ultimate strain and transfer length of the tendons were 1.7% and 625 mm, respectively. Although the ultimate strain of CFRP tendons was lower than that of steel strands and the mode of failure was brittle, considerable deflection was observed before failure. This could serve as a warning sign in real applications. A simple analytical model was developed to predict the flexural response of the beams. The model predicted ultimate loads accurately but underestimated ultimate deflections.

2.3.3 Behaviour under Sustained Loading

Currier (1995) noted that long-term deflections of beams prestressed with FRPs could be estimated using conventional methods for steel-prestressed beams, but with minor adjustments. In this method, camber and deflection were separated into individual components, adjusted by a multiplier and superimposed to give final deflections. Table 2.2 shows the multipliers suggested for CFRP and aramid FRP (AFRP) prestressed beams. Three beams of each tendon type were tested to obtain these values.

Braimah et al. (2003) and Braimah (2000) investigated the long-term behaviour of CFRP pretensioned concrete beams under sustained load at room temperature. (These same beams were tested in the present study.) Of the four beams tested, three were prestressed with Leadline CFRP tendons and one with seven-wire steel strands. The beams were subjected to a sustained load of 29% of their flexural capacity for 651 days. An analytical model was developed to predict the time-dependent behaviour of the beams. Test results
indicated that the CFRP-prestressed beams had comparable or superior performance in comparison to the steel-prestressed beam. Also, the ratio of long-term to instantaneous deflection increased with prestressing force. Prestressing strain was found to decrease with time in cracked sections and remain unchanged in uncracked sections. ACI recommendations and the CEB Model Code over-estimated the measured deflections by an average of 28% and 42%, respectively.

Zou (2003) tested three series of pretensioned concrete beams, using Leadline CFRP tendons or steel strands as prestressing reinforcement. The beams were subjected to sustained loading below and above cracking load. Prestress level, sustained load and concrete strength were varied between pairs. The study concluded that beams prestressed with CFRP met serviceability criteria for deflection and cracking. It was shown that this performance improved with increasing concrete strength. Also, CFRP-prestressed and steel-prestressed beams were similar in long-term deflections in both uncracked and cracked states. Figure 2.5 shows deflection over time for one of the pairs, with sustained loading below cracking load for the first 259 days and above the cracking load later. Cracks were slightly wider for beams with CFRP compared to those with steel prestressing.

Zou (2007) developed an analytical model for predicting the long-term behaviour of beams prestressed with CFRP tendons under sustained service loads. Time-dependent concrete strains, curvature, deflection and loss of prestress were calculated. The theoretical framework used the age-adjusted effective modulus method (AEMM) and accounted for tension stiffening. Cracked and uncracked sections as well as short and long spans were considered. The model was shown to be in close agreement with
experimental results. It was concluded that the long-term performance of CFRP-prestressed beams was comparable to those with steel prestressing in terms of concrete strains, curvature and deflection. The behaviour was very similar for uncracked sections and was reasonably similar for service loads above the cracking load. CFRP tendons were judged to have satisfactory long-term performance as prestressing reinforcement in concrete beams.

### 2.3.4 Behaviour under Cyclic Loading

To determine the fatigue resistance of a beam, typically the stress range in the prestressing tendon during cyclic loading is calculated and compared with that obtained from the S-N curves for the prestressing tendon.

CFRP has very good resistance to fatigue. Figure 2.6 by Machida (1997) shows S-N curves for various types of FRP reinforcement. Clearly, CFRP is superior to other types of FRP, with stresses in excess of 1,000 MPa required to induce fatigue failure at 3 million cycles.

When used as bonded internal reinforcement, fatigue loading is more likely to affect the concrete and the bond between the concrete and the CFRP tendon than the CFRP itself (ACI, 2007). Mor et al. (1992) also concluded that the governing factor in reinforced concrete subjected to cyclic loading is the fatigue strength of bond of concrete with the reinforcement.

In uncracked members, fatigue characteristics of prestressing reinforcement and anchorages do not govern. In cracked sections, however, increased stresses between...
cracks in pre-tensioned tendons, fretting fatigue of post-tensioned tendons, and fatigue of tendon-anchorage assemblies can become considerable (ACI, 2007).

Abdelrahman et al. (1995) tested four prestressed concrete T-beams pretensioned with two types of CFRP tendons, Leadline and CFCC. The beams were 1:3.3 scale models of girders of the first Canadian smart highway bridge built in Calgary, Alberta. The objective was to study the limit state behaviour, ultimate capacities and failure modes of the beams. Two beams were monotonically loaded to failure at the start, while the other two were subjected to 2 million cycles of loading before being monotonically loaded to failure. All beams failed by rupture of cables at a higher load than expected, due to underestimation of the ultimate strength of cables. Load-deflection behaviour was bilinear and elastic up to failure (Figure 2.7a,b). The beams showed large cracks over an extensive region and large deflections before failure, which were good warning signs of impending failure. The beams exposed to cyclic loading survived 2 million cycles between cracking and 70% of cracking load with no significant effect on beam stiffness. The ultimate capacity of beams subjected to cyclic loading and the strains in CFRP tendons were very similar to the beams that did not undergo cyclic loading. Overall, the fatigue strength of beams pretensioned by FRP tendons was judged as excellent.

Dolan et al. (2000) subjected cracked CFRP-prestressed concrete beams to flexural cycles with nominal tensile stresses of $0.5\sqrt{f'_c}$ MPa at the extreme fibre. The beams gradually softened and cracking was observed after the first 100,000 cycles. Nevertheless, the beams survived 3,000,000 cycles with no fatigue failure and showed no loss of strength due to cyclic loading. The beams failed by “straw broom” rupture of CFRP tendons.
Since all tendons failed in tension at close to the predicted strength, it was concluded that the full tensile capacity of the tendons was developed.

Mertol et al. (2006) investigated the durability of concrete beams prestressed with CFRP tendons compared to those prestressed with steel wires. Fifteen beams were exposed to different mechanical and environmental conditions. Parameters included different levels of sustained stress in prestressing reinforcement, environmental exposure condition, duration of sustained loading and whether cyclic loading was applied prior to testing to failure. Cyclic loading consisted of 2 million cycles at 3 Hz, inducing stresses ranging from 65 to 75% of the guaranteed ultimate stress. The tests indicated the maximum load capacities of CFRP-prestressed beams subjected to cyclic loading were slightly lower than those tested without cycling. Also, degradation in stiffness was observed as cyclic loading progressed.

Grace (2000) tested a continuous two-span double-T bridge system, prestressed internally and externally with Leadline CFRP tendons, under static, repeated and ultimate loads. The bridge model was subjected to 15 million cycles of two repeated loads of constant amplitudes equivalent to service load at the middle of the first span and twice the service load at the middle of the second span. The effect of repeated loads on various parameters was studied before and after post-tensioning adjustment (increase in post-tensioning forces after 7.5 million cycles of repeated loads). The findings were as follows.

- The bridge stiffness gradually decreased throughout cyclic loading at a load of twice the service load before the post-tensioning adjustment. However, this effect was
diminished after post-tensioning adjustment. The bridge stiffness did not change much during cyclic loading when the applied load was equal to the service load.

- Repeated loadings did not affect the forces in the post-tensioned externally harped CFRP tendons, even with a repeated load of twice the service load. There was no rupture or change in surface texture of the CFRP tendons at deviating points in the positive or negative moment regions under 15 million cycles of repeated loads. Thus, CFRP was deemed suitable as continuous prestressing in multi-span bridges.

- In this continuous bridge model, the ultimate load capacity was 50% higher and midspan deflection at failure was 75% lower than that of a simply supported bridge using the same construction components.

### 2.3.5 Development Length

Development length ($L_d$) is the sum of the transfer length ($L_t$) and the flexural bond length ($L_{fb}$). Prestress is zero at the ends of the beam and increases over the transfer length to the effective prestress level ($f_{pe}$). Flexural bond length is the minimum length over which the stress in the tendon can increase from the effective prestress ($f_{pe}$) to the ultimate prestress ($f_{pu}$) at the location of maximum moment.

Soudki et al. (1997) determined the transfer length of 8 mm diameter Leadline CFRP tendons in prestressed concrete beams of rectangular and T-beam shapes. (These same T-beams were tested in the present study.) The strain profile along the length of the beam was measured using strain gauges on the tendons and DEMEC gauges on the concrete. The transfer length was reported as 80 times the bar diameter (650 mm) for a prestress
level of 50% of the guaranteed strength, and 90 times the bar diameter (725 mm) for a
d prestress level of 70% of the guaranteed strength. Results for rectangular and T-beams
were very similar to each other. Both long-term and instantaneous (measured at release)
transfer lengths were measured and were found to be similar to each other.

Mahmoud et al. (1997) recommended the following equation for the flexural bond length $L_{fb}$ of CFRP

$$L_{fb} = \frac{(f_{pu} - f_{pe})d_b}{\alpha_{fb}f'_c}$$

(2-4)

where $f_{pu}$ is ultimate tensile stress, $f_{pe}$ is effective prestress, $d_b$ is tendon diameter, $f'_c$ is
concrete strength at time of test and $\alpha_{fb}$ is 1.0 for Leadline and 2.8 for CFCC for N-mm
units.

2.3.6 Hoyer Effect

As in all pre-tensioned beams, CFRP tendons are stressed in a prestressing bed and
concrete is poured around the stressed tendons. At this point, the tendons are under
tensile stress and have a smaller diameter than their unstressed state due to Poisson’s
effect. After the concrete hardens, the prestressing force is released. The tensile force is
transferred to the tendon-concrete interface at the beam ends through friction. The
prestressing force is built up, starting from zero at the beam ends and increasing linearly
up to the effective prestress level ($f_{pe}$) at a distance from the beam end called the transfer
length ($l_t$). During transfer, due to the Poisson’s effect, the tendon expands to its original
diameter at the beam ends where tensile stress is zero, decreases in diameter over the
transfer length, and reaches the smallest diameter after the transfer length where tensile stress is at its maximum. This tapering of tendons creates a wedge effect that enhances the tendon-concrete bond over the transfer length and is known as the Hoyer effect (Hoyer and Friedrich, 1939).

2.3.7 Field Applications

During the late 1980’s and the 1990’s, FRP prestressing reinforcement was successfully used in several projects. These were demonstration projects used to validate the technology, to gain experience and to study long-term durability and performance. This section presents a few examples.

The BASF bridge in Ludwigshafen, Germany, was completed in 1991. The bridge is two lanes wide and 85 m long. Four CFRP tendons were used together with 16 conventional steel tendons as internal unbonded post-tensioned reinforcement (Zoch et al., 1991). Each tendon consisted of 19 CFRP strands, each strand being 12.5 mm in diameter and stressed to 70 kN. CFRP tendons were guided through ducts in the concrete that were not filled with grout.

The Beddington Trail Bridge in Calgary, Alberta, Canada was built in 1993 (Figure 2.8). It consists of two continuous spans. Each span consists of 13 precast, pretensioned concrete girders. Four girders were prestressed using CFCC cables, while two other girders were prestressed using two Leadline tendons. The girders were instrumented with fibre optic sensors and are being monitored by ISIS Canada (2009).
The Taylor Bridge in Headingly, Manitoba, Canada was built in 1998 (Figure 2.9). It has multiple spans and an overall length of 165 m. Four out of 40 girders were prestressed with CFRP and included FRP stirrups as well (ISIS Canada, 2009).

The Bridge Street Bridge in Southfield, Michigan, USA was completed in 2001 (Figure 2.10). This is a three span structure with bonded and unbonded CFRP tendons in the longitudinal and transverse directions. The bridge consists of two parallel, independent bridges that traverse the Rouge River and are over 62 m long. The first bridge consists of five equally spaced AASHTO Type III girders in each of its three spans with a continuous cast-in-place concrete deck slab. The second bridge consists of four special double-T (DT) girders in each of its three spans. The girders were pre-tensioned using Leadline tendons and post-tensioned, in the longitudinal and transverse directions, using CFCC strands (Grace et al., 2001).

### 2.4 Concrete

The long-term behaviour of a CFRP-prestressed concrete beam under sustained and cyclic loading is often governed by the behaviour of the concrete, because CFRP tendons have very low relaxation under sustained loading and very high resistance to cyclic loading. For example, in a beam that is cracked under sustained load, the gradual increase of deflection is due to creep of concrete in the compression zone.

There is a large body of literature on the creep of concrete as well as the behaviour of concrete at low temperatures and under cyclic loading. The following sections provide a short summary.
2.4.1 Behaviour at Low Temperature

Yamane et al. (1978) tested concrete specimens in air-dried conditions at various temperatures. Compressive strength was found to be higher by a factor of 1.4 at \(-30 \, ^\circ C\) and by a factor of 2.2 at \(-70 \, ^\circ C\) in comparison with specimens at +20 \, ^\circ C. For tensile strength this ratio was 1.1 at \(-30 \, ^\circ C\), rose to 1.5 at \(-50 \, ^\circ C\) and remained almost unchanged at lower temperatures. However, little change in the modulus of elasticity was observed between +20 \, ^\circ C\) and \(-70 \, ^\circ C\).

2.4.2 Behaviour under Sustained Loading (Creep)

Creep is defined as the gradual increase in strain under a sustained stress. From another point of view, if a concrete specimen is stressed and restrained such that the strain is constant, creep will cause the stress to slowly drop over time. This is called relaxation.

The amount of creep that a particular concrete specimen will exhibit is best determined by conducting tests. In a typical creep experiment, stress is applied to the concrete and then held constant over time. The strain that occurs immediately after loading is termed the “elastic strain” and the extra strain that develops over time is called “creep strain”. Figure 2.11 shows a schematic plot of the increase in strain over time.

Various factors influence creep of a concrete specimen. These include properties of the aggregate and cement, size of the specimen, stress and strength, ambient relative humidity and temperature. Generally, creep increases when humidity decreases (Neville, 1995).
Even very old concrete undergoes creep, as demonstrated by tests on 50-year-old concrete (Nasser and Neville, 1965). Also, once initiated, creep continues for a very long time, if not indefinitely. Nonetheless, the rate of creep decreases continuously. Troxell et al. (1958) made measurements of creep for as long as 30 years.

Johansen and Best (1962) conducted creep tests in the temperature range of −20 to +20 °C on concrete specimens sealed against moisture gain or loss. They cured a concrete mix for 42 days at +20 °C and 100% relative humidity and then for 3 days at test temperature and ambient humidity before loading. Test results showed the modulus of elasticity increased sharply with decreasing temperature below the freezing point of capillary water (about −2 °C). Also, in a system with ice at constant temperature, the initial rate of creep was higher than an ice-free system at constant temperature (Figure 2.12). This rate dropped quickly and approached zero, whereas the ice-free system continued to creep at a rate that was dependent on the temperature.

Turner (1980) has postulated that at temperatures between −10 and −30 °C creep is about half of the creep at 20 °C.

### 2.4.3 Behaviour under Cyclic Loading

In a concrete specimen subjected to cycles of compressive loading between stress values \( \sigma_1 (\geq 0) \) and \( \sigma_h (> \sigma_1) \), the stress-strain curve changes as cycling progresses (Figure 2.13). The shape changes from concave downwards (with a hysteresis loop on unloading) to a straight line that shifts at a decreasing rate (i.e. there is some permanent deformation) and ultimately becomes concave upwards. Failure only occurs if \( \sigma_h \) exceeds a certain
limiting value, called the *fatigue limit* or *endurance limit*. If $\sigma_a$ is lower than this value, the stress-strain curve will always remain straight and failure in fatigue will not occur (Neville, 1995).

The elastic strain increases progressively with cycling. Figure 2.14 shows this phenomenon as a reduction in the secant modulus of elasticity as fatigue life is used up (Bennett and Raju, 1969). The secant modulus of elasticity can be observed to decrease by up to 32% over the entire fatigue life of the member.

Cyclic loading below the fatigue limit actually improves the fatigue strength of concrete. This increase is likely due to the densification and compaction of concrete under the initial low-stress cycling, in the same way that moderate sustained loading enhances concrete strength (Neville, 1966).

When a prestressed concrete beam is subjected to a cyclic load that creates cracking in the beam, cracks gradually propagate. There are two theories as to why cracks propagate under cyclic loading. One hypothesis claims the bond between the coarse aggregate and the matrix progressively deteriorates over many cycles, especially if the modulus of elasticity of the aggregate is greater than the matrix. The other hypothesis states that with increasing deformation under cyclic loading, pre-existing microcracks in the concrete matrix coalesce into a single macrocrack, thus weakening the section (Toumi et al., 1997).
2.5 Links to Body of Thesis

The information learned in this literature review informs the discussions in the main body of the thesis. While most of the literature presented here provided the author with general background information for the study, several works had direct relevance to specific parts of the discussion. Table 2.3 lists the links between this chapter and Chapters 3 and 4.
### Table 2.1. Summary of test results (Bryan and Green, 1996)

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<thead>
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<th>Condition</th>
<th>Beam No.</th>
<th>Temperature ($^\circ$C)</th>
<th>$f_c'$ (MPa)</th>
<th>Cracking load (kN)</th>
<th>Ultimate load (kN)</th>
<th>Ultimate CFRP strain (%)</th>
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<td>1.74</td>
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<td>3</td>
<td>23.0</td>
<td>51</td>
<td>25.5</td>
<td>52.2</td>
<td>1.67</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>23.2</td>
<td>52</td>
<td>25.8</td>
<td>54.3</td>
<td>—</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>23.4</td>
<td>51</td>
<td>25.7</td>
<td>53.8</td>
<td>1.70</td>
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<tr>
<td>Low temperature</td>
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<td>69</td>
<td>29.8</td>
<td>55.0</td>
<td>1.76</td>
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<td></td>
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<td>30.8</td>
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<td>—</td>
<td>30.2</td>
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### Table 2.2. Suggested multipliers for FRP tendons (Currier, 1995)

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<tr>
<th>Without composite topping</th>
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<th>Aramid</th>
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<tr>
<td>Deflection due to self-weight</td>
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<td>Camber due to prestress</td>
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<td><strong>Final</strong></td>
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<tr>
<td>Deflection due to self-weight</td>
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<td>2.70</td>
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<tr>
<td>Camber due to prestress</td>
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<tr>
<td>Deflection due to applied loads</td>
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<td>4.00</td>
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</table>
Table 2.3. Links between literature review and main body of thesis

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<th>Section in Main Body</th>
<th>Subject</th>
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<td>Section 4.3.1.1.3</td>
<td>Sustained loading at low temperature</td>
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<td>2.3.3 Behaviour under Sustained Loading</td>
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<td>2.4 Concrete</td>
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<td>2.4.2 Creep</td>
<td>Section 3.3.1.3</td>
<td>Decrease in secant modulus of elasticity, Crack propagation</td>
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<tr>
<td>2.4.3 Behaviour under Cyclic Loading</td>
<td>Section 4.3.1.3</td>
<td></td>
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Figure 2.1. Sample FRP reinforcement configurations (ACI, 2004)

Figure 2.2. Midspan load-deflection behaviour of beams (Fam et al., 1997)
Figure 2.3. Midspan load-deflection behaviour of beams prestressed by CFRP and steel (Abdelrahman and Rizkalla, 1997)

Figure 2.4. Proposed model to calculate deflection of beams prestressed by CFRP under repeated load (Abdelrahman and Rizkalla, 1999)
Figure 2.5. Instantaneous and time-dependent deflections of CFRP-prestressed and steel-prestressed beams (Zou, 2003)

Figure 2.6. S-N curves for various FRP reinforcements (Machida, 1997)

CFRP: Carbon FRP, AFRP: Aramid FRP, GFRP: Glass FRP
B/Al: Boron reinforced aluminium composite
Al 2024-T3: Fibre-metal laminate with aluminium 2024-T3 alloy
Figure 2.7. Midspan load-deflection behaviour of beams prestressed by (a) CFCC and (b) Leadline (Abdelrahman et al., 1995)
Figure 2.8. Beddington Trail Bridge, Calgary, Canada
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a) Leadline tendons and CFCC strands in reinforcement cage of double-T girders

b) Installed double-T girders with external CFCC post-tensioning strands in place

Figure 2.10. Bridge Street Bridge, Southfield, Michigan, USA
(Grace et al., 2002)
Figure 2.11. Development of creep with time
(Collins and Mitchell, 1994)

Figure 2.12. Creep of concrete in frozen and unfrozen state (Johansen and Best, 1962)
Figure 2.13. Stress-strain relation of concrete under cyclic compressive loading (Neville, 1995)

Figure 2.14. Ratio of secant modulus of elasticity at given instant ($E$) to that at the start of cycling ($E_0$) over the fatigue life of concrete (Bennett and Raju, 1969)
Chapter 3
Sustained Loading at Low Temperature

3.1 Introduction

Early research on concrete girders prestressed using Carbon Fibre-Reinforced Polymer (CFRP) tendons in the 1990’s proved that this technology has great promise in terms of performance and durability. This led to applications in bridges in Canada and the USA. As discussed in Chapter 2, the performance of CFRP-prestressed concrete beams has been studied under sustained loading. However, testing was typically carried out at room temperature. To the author’s knowledge, the combined effect of sustained loading and low temperature has not been investigated for this type of prestressed system.

Braimah (2000, 2003) studied the effect of sustained loading on 2-year-old CFRP-prestressed concrete beams at room temperature. In the present study, three similar beams were tested under sustained loading at low temperature.

3.2 Experimental Program

This chapter summarizes the experimental work performed for the study. The following sections give a detailed description of the test beams, the control beams, the cold room,
the sustained loading setup, load and temperature changes, the instruments used to gather
data and the loading of test beams to failure.

3.2.1 Test Beams

Three large-scale concrete beams, referred to as C50LT, C70LT and S55LT, were tested. The designation for each beam indicates the type of prestressing reinforcement (CFRP tendons or steel strands), the level to which it was prestressed, and the temperature at which sustained loading was applied (low temperature). For example, C50LT had CFRP tendons (C) prestressed to 50% of their guaranteed ultimate strength (50) and was subjected to sustained loading at low temperature (LT). This naming system also applies to control beams (Section 3.2.2).

Figure 3.1 shows the geometry and reinforcement details of the beams. The concrete T-beams were 4.4 m long, had a 500 mm wide flange and were 300 mm deep. The cross-section consisted of a wide shallow flange at the top and a tapered web underneath. Each beam weighed approximately 640 kg. In addition to the prestressing which differed from one beam to the other, the beams also included steel stirrups at a 100 mm spacing for shear strength, a steel wire mesh in the flange for control of shrinkage cracks and two steel bars at the top of the web for stirrup support.

The concrete used for the beams was designed to have a 28-day compressive strength of 40 MPa, a 3-day (at prestress release) compressive strength of 30 MPa and a maximum aggregate size of 16 mm. Actual concrete strengths are listed in Appendix A. The CFRP prestressing tendons were Leadline manufactured by Mitsubishi Plastics, Inc. Their
properties are summarized in Table 3.1. The tendons had a diameter of 7.9 mm, a guaranteed strength of 2300 MPa, a measured ultimate strength of 3200 MPa, a measured modulus of elasticity of 187 GPa, a fibre volume ratio of 65% and an epoxy resin matrix with spiral indentations. The seven-wire steel prestressing strands were 13 mm in diameter and of grade 1860 MPa.

Table 3.2 presents the prestressing details of the test beams, including type of prestressing reinforcement, prestress level and jacking force. C50LT and C70LT were prestressed with four 8 mm CFRP tendons arranged in two layers, with two tendons in each layer. In C50LT, all four tendons were stressed to 50% of their guaranteed ultimate strength, while in C70LT these were stressed to 70%. S55LT contained two 13 mm seven-wire steel strands arranged in one layer. Both strands were stressed to 55% of their guaranteed ultimate strength, giving a total jacking force of 202 kN, similar to that of C50LT (208 kN). Figure 3.2 shows the end faces of the beams with CFRP and steel prestressing reinforcement. The centroid of the reinforcement was at the same depth for the CFRP- and steel- prestressed beams.

The beams were fabricated in 1995 and stored in the laboratory for 13 years before being tested for this study in 2008 and 2009.

3.2.2 Control Beams

Braimah (2003) experimented on four beams: C50RT, C70RT, S55RT and C60RT. The first three beams were used as controls for the sustained loading part of the present study, while the last beam was used as a control for the loading to failure part. Each control
beam was identical to its corresponding test beam of the present study in terms of materials, geometry and prestress. However, the control beams were subjected to sustained loading at room temperature as compared to low temperature for the test beams of the present study. Therefore, the names of control beams end in RT (room temperature) as compared to LT (low temperature) for the test beams. Also, the control beams were two-years old when tested while the test beams of this study were 13-years old. This, however, should not make a significant difference since the concrete strength would have stabilized within the first year.

3.2.3 Cold Room

The test beams were placed inside a cold room 5.5 m long, 3.4 m wide and 2.9 m high (Figure 3.3a). Fans connected to a refrigeration system blew cold air into the room (Figure 3.3b), keeping the temperature at $-27 \, ^\circ C$. The fans had 30 minute thaw cycles to melt the ice inside them, but the resulting temperature fluctuations created were negligible. The room was insulated to prevent the cold from escaping.

The 640 kg concrete beams were transported into the cold room using cranes, a pedal lift, a forklift and rollers. The cold room floor was fitted with plywood to avoid point loading and to create a smooth surface for the lifting machines to roll on.

3.2.4 Sustained Loading

The beams were each subjected to a sustained load of about 46 kN. The magnitude of sustained load was chosen within the service load range such that it caused cracking in
two of the beams, namely C50LT and S55LT. These beams represented partially prestressed beams. The remaining uncracked beam, C70LT, represented a fully prestressed beam.

A special setup was prepared to apply the sustained load of 46 kN in a four-point bending scheme with a constant-moment-zone of 1.33 m, one third of the 4 m span. Figure 3.4a and Figure 3.4b show the details of the setup in side view and in cross-section, respectively. Figure 3.5 shows the setup during the loading operation.

The sustained load was applied by stressing two vertical steel strands between two transverse hollow steel sections (HSS), one on top of the test beam and the other under the reaction beam, and locking the load using proper grips. The top transverse HSS acted on the longitudinal HSS which in turn applied concentrated loads to the topside of the test beam at the two loading points. The bottom transverse HSS acted on a reaction beam which exerted concentrated loads to the underside of the test beam at the beam supports. In this way, the force in the vertical steel strands was transmitted to the test beam in a four-point-bending scheme.

The two vertical steel strands were stressed simultaneously using two hydraulic jacks fed from one hand pump. These jacks were placed on jacking chairs. Adjusting screws were used to make small adjustments to the load. Helical springs located between grips and the transverse HSS at the top helped maintain a constant load in the event of slip or length change of the vertical strands. Load cells positioned between grips and the transverse HSS at the bottom were employed to measure load in the vertical strands. Upon completion of stressing, the hydraulic jacks and jacking chairs were removed while the
grips maintained the load in the vertical strands. Figure 3.6 shows all three test beams in their loaded state.

The longitudinal HSS was simply-supported on the test beam, with a roller on one side and a pin on the other. Similarly, the test beam was simply-supported on the reaction beam. The loading points were 1.33 m apart and symmetrical about the centreline, leaving 1.33 m on each side of the 4 m span.

3.2.5 Load and Temperature Changes

Figure 3.7 shows the load applied to each beam over time. The beams were moved into the cold room on different days and it took several days of adjustments to start the sustained loading. C70LT and S55LT were loaded on days 1 and 2, respectively, while C50LT was loaded on day 17. On day 24, all three test beams were unloaded and reloaded to take incremental deflection readings during loading to the sustained load level. On day 29, the loads in C70LT and S55LT were adjusted for consistency with C50LT.

Figure 3.8 shows the temperature and humidity in the cold room over time. The loading operations were performed at room temperature for convenience and safety. On day 31, the cold room fans were turned on. Days 31 to 35 represent the transition from room temperature to low temperature for the beams. At room temperature, average temperature was +28 °C. At low temperature, average temperature was −27 °C. Temperature was typically between −25 and −30 °C with five instances of up to −23 °C and one instance of
−19 °C. The average relative humidity at low temperature was 86%. The beams were kept under sustained load at low temperature for 163 days.

There was a significant loss of sustained load during the transition from room to low temperature. This was the result of complex interactions between the members in the sustained load setup that shrank during cooling to low temperature. During the sustained load period in low temperature, no significant drop in load occurred, so load was not readjusted. However, sustained load gradually dropped over time as a result of relaxation in the vertical prestressing strands and increasing sagging of the test beams under load.

3.2.6 Instrumentation

This section describes the measurement process, the instruments used and their calibration.

3.2.6.1 Measurement Process

A measurement station was arranged outside the cold room and connected to all instruments inside the cold room (Figure 3.13). This connection was achieved using long wires that passed through a small insulated duct in the cold room wall. From the measurement station, power units sent excitation voltages to the instruments inside the cold room and the output voltages came back to the measurement station. Both the excitation and output voltages were measured using a voltmeter and manually recorded at regular intervals, often every other day. These values were then converted to give measured quantities, either by interpolation from calibrated values or using formulae provided by manufacturers.
3.2.6.2 Instruments

Measured quantities include sustained load, deflection at midspan, longitudinal strain in prestressing reinforcement at midspan, temperature and relative humidity. This section describes the instruments used and how readings were made.

Loads in the vertical steel strands were measured by load cells placed in their path between grips and the transverse HSS under the beam (Figure 3.9). The loads in the two strands loading each beam were added to give the total sustained load on that beam. The load cells were made in-house. A 10-channel amplifier sent an excitation of 8 V to the load cells (Figure 3.14a). The output voltage was read using a handheld multi-meter and converted to load by interpolation from calibrated values.

Deflection was measured using mechanical dial gauges and linear potentiometers (LPs). During initial loading to the sustained load level, deflection was measured by dial gauges located at midspan and under the loading points (Figure 3.10a). Further readings were made during sustained loading at room temperature. During sustained loading at low temperature, deflection was measured by LPs placed at midspan (Figure 3.10b). A power unit sent an excitation of 10 V to the LPs (Figure 3.14b). The output voltage was read using a multi-meter and converted to deflection by interpolation from calibrated values. Figure 3.12a shows the location of the dial gauges and LPs on the beams.

Strain in the prestressing tendons/strands was measured using strain gauges attached to them when the beams were originally fabricated. The strain gauge wires came out of the top flange (Figure 3.11). In beams C50LT and C70LT, two bottom tendons and one top tendon were instrumented with strain gauges at midspan (Figure 3.12b). In S55LT, one
wire in one steel strand was instrumented with a strain gauge at midspan. The strain
gauges used were foil strain gauges, SHOWA 5 mm, type N11-FA-5-120-11. A 10-
channel switch and balance unit sent an excitation of 2 V to the strain gauges (Figure
3.14c). The output voltage was read using a voltmeter and converted to strains using a
formula given by the manufacturer.

Only change in prestressing strain from the moment of loading could be measured. The
absolute strain was unknown because the pre-existing strain in the prestressing
reinforcement (pre-strain) was not measured. To measure absolute strain, one would have
to continuously monitor prestressing strain starting just before the beam was originally
pre-tensioned (zero strain).

Temperature in the cold room was measured by a thermometer built into the inner wall
and shown on a digital display outside (Figure 3.14d). All three test beams were assumed
to have the same temperature as the air in the cold room, except during the transition from
room temperature to low temperature. During this period no readings were made for 4
days (days 31-35) to ensure the core of the concrete beams reached the temperature of the
ambient air.

Relative humidity was measured using a humidity-meter placed in the centre of the room.
A power unit sent an excitation of 5 V to the humidity-meter. The output voltage was
read using a voltmeter and converted to relative humidity using a formula given by the
manufacturer.

All instruments in the cold room were rated by manufacturers to function at -30 °C.
Appendix B lists the accuracies of the instruments used and the measurements made.
3.2.6.3 Calibration

The load cells and LPs were manually calibrated by applying known loads and displacements, respectively, and correlating the measured output voltages with the known loads or deflections. Calibration was repeated at room and low temperature, as well as before and after the sustained load period.

An extra ‘dummy’ load cell was placed inside the cold room with no load applied to it. Since this load cell was never under any load, variations in readings for this load cell were identified as error and subtracted from readings from other load cells to correct them.

For the strain gauges and humidity-meter, formulas given by the manufacturer were used. Hence, there was no need for manual calibration. The formula for strain gauges was adjusted for temperature.

3.2.6.3.1 Temperature Compensation for Strain Gauges

The strain gauges used on the prestressing tendons/strands were temperature-compensated for ordinary steel, but not for prestressing steel or CFRP. That is, the coefficient of thermal expansion of the strain gauges in the longitudinal direction was different from that of the prestressing steel strands or CFRP tendons they were attached to. As the beams were cooled from room to low temperature (+28 to −27 °C), the strain gauges shrank at a lower rate than the prestressing steel strands in S55LT and at a higher rate than the CFRP tendons in C50LT and C70LT. Thus, the strain gauges perceived some added compression in S55LT and some added tension in C50LT and C70LT. The
resulting strain changes do not represent actual strain changes in the steel strands and CFRP tendons. Thus, measured strains should be adjusted accordingly.

To measure these false perceived strain changes, strain gauges similar to those originally used in the beams were attached to a piece of prestressing steel strand and a piece of CFRP tendon with no load applied to them (Figure 3.15). Both pieces were kept at +28 °C and then suddenly placed inside the cold room at −27 °C. Strain readings were taken while the pieces cooled from room to low temperature. Figure 3.16 presents the changes in perceived strain over time. It is seen that the strain gauges on the steel strand and CFRP tendon sensed compressive and tensile strains, respectively. The difference in the initial and final strains for each piece represents the false perceived strain change. This value was 188 με (compressive) for the steel strand and 390 με (tensile) for the CFRP tendon. These values were then subtracted from all strain readings from the test beams at low temperature to correct them.

### 3.2.7 Monotonic Loading to Failure

Upon completion of sustained loading at low temperature in the cold room, C70LT was monotonically loaded to failure at the same low temperature of −27 °C. This was done using a loading machine in a different cold room (Figure 3.17). C60RT, which was a control beam with a history of sustained loading at room temperature, was monotonically loaded to failure at room temperature to compare with C70LT. This was done in a different loading machine (Figure 3.18). C60RT was chosen as the control beam for C70LT because they were very similar in materials and geometry and were both
subjected to sustained load in the same configuration. However, a few differences between the two beams are listed in Table 3.3.

Loading to failure was applied in a four-point-bending scheme with a constant-moment-zone of 500 mm as compared to the 1.33 m used in the sustained loading configuration. This was changed in order to minimize the risk of a premature shear failure by producing greater bending moments in the constant moment zone at the same level of shear.

Load was measured using load cells built into the loading machines. Deflections were recorded at midspan and at loading points using LPs. Slip of the prestressing tendons was closely monitored by measuring the change in length of tendons projecting out of the ends of the beams.

A data acquisition unit (DA) and computer were used as the processing centre for load and deflection measurements. Wires attached to the instruments were connected to ‘green cards’ which were plugged into the back of the DA. A computer program named StrainSmart interacted with the DA during tests to collect electrical signals from the instruments, convert them into measured quantities using calibration data, display them graphically in real time and store them in spreadsheets.

### 3.3 Results and Discussion

This section presents results from the sustained loading at low temperature of the three test beams and the subsequent loading to failure of one of the test beams. Results are compared with those from control beams exposed to sustained load at room temperature.
3.3.1 Sustained Loading

This section presents the load-deflection behaviour of the three test beams during initial loading as well as changes in midspan deflection and prestressing strain over the sustained load period. The beams are compared to one another and also to their corresponding control beams.

3.3.1.1 Initial Load-Deflection Behaviour

On day 24, all three test beams, having already been loaded, were unloaded and reloaded to the sustained load level at room temperature. At this point C50LT and S55LT were already cracked, while C70LT was not. This was done in multiple steps, with load and deflection at midspan measured at each step. Figure 3.19 shows the resulting load-deflection graphs for the three test beams in comparison to those of their counterparts (Braimah, 2003). Several observations can be made from close examination of Figure 3.19.

1. C50LT and S55LT both displayed bi-linear load-deflection. This was a result of both beams being cracked. Figure 3.20 shows the beams after loading. It is evident that C50LT and S55LT were cracked.

2. Since C50LT and S55LT were pre-cracked, the transition points in their load-deflection curves reflect the decompression loads, which were almost equal (about 33 kN) because of similar jacking forces. C50RT and S55RT were slightly stiffer because these responses reflected the first loading cycle for these beams with no cracking history.
3. C70LT was not cracked and as such it exhibited linear load-deflection behaviour up to the sustained load level. Figure 3.20 shows C70LT was not cracked after loading.

4. The stiffness of C70LT was higher than that of C50LT and S55LT, because the two beams had cracked under prior loading. To elaborate, C70LT, being uncracked, resisted the load using the compressive and tensile capacity of the entire concrete cross-section. This large area gives a large moment of inertia, which gives a large stiffness. C50LT and S55LT, however, were already cracked and therefore only used the compressive capacity of the concrete above the neutral axis and the tensile capacity of the CFRP tendons. This small area gives a small moment of inertia, which results in a small stiffness.

5. The stiffness of C70LT and C70RT were very similar to each other because neither beam was cracked during loading.

6. The initial midspan deflections under sustained load were 12.7, 12.5 and 6.0 mm for S55LT, C50LT and C70LT, respectively.

3.3.1.2 Deflection over Time

Figure 3.21 shows the deflection history of the low temperature test beams and the room temperature control beams over 196 days of sustained loading. Adjustments in sustained load caused sudden changes in deflection initially. C70LT and S55LT were first loaded on days 1 and 2, while C50LT was first loaded on day 17. On day 24, all three test beams were unloaded and reloaded, and deflection monitoring began for C50LT. On day 29, the loads in C70LT and S55LT were adjusted for consistency with C50LT, causing a minor change in deflections. The graph for control beam C50RT starts on the day that is
equivalent to the day test beam C50LT was first loaded. The gap between days 31 and 35
represents the transition from room to low temperature. This change in temperature
increased deflection in all beams, especially in C70LT.

The beams showed a slight increase in deflection in the early period after loading
followed by stabilization of deflection. In C50LT and S55LT, most of this increase in
deflection occurred within the first 45 days of initial loading, which ended 22 days into
the low temperature period. Beyond this point, deflection remained fairly constant with
small fluctuations over time. Both of these beams were cracked. Braimah (2003),
however, reported continuous increase in midspan deflection of their corresponding
control beams for at least 450 days. This could suggest that under sustained load at low
temperature deflection stabilizes much faster than at room temperature. This can be
explained by the fact that at low temperatures, freezing produces a high initial rate of
creep in concrete but it quickly drops to zero (Johansen and Best, 1962). A drop in
deflection was observed in C70LT between days 36 and 52. The reason for this is
unknown. However, deflection gradually increased again after day 52.

Braimah (2003) attributed the periodic changes in deflection of the control beams to
changes in ambient relative humidity and temperature in the laboratory. No similar
relationship was found for the test beams in this study.

Table 3.4 shows the midspan deflections at the start and end of the low temperature
period in contrast to those of the control beams at room temperature for an equivalent
period of time. This equivalent period for each beam starts after the same number of days
from initial loading as the low temperature period and is of the same length to allow for
consistent comparison. Generally, the changes in deflection over time were relatively small (less than 7%). For C50LT and S55LT the changes were slightly greater than those of their corresponding control beams. Both of these beams were cracked under the sustained load. C70LT actually exhibits a small drop in deflection between the start and end of the low temperature period. This is due to the unusual drop in deflection between days 36 and 52 which is later slowly recovered between days 52 and 196 but not fully.

3.3.1.3 Prestressing Strain over Time

Figure 3.22 shows the change in prestressing strain at midspan of the low temperature test beams and the room temperature control beams over 196 days of sustained loading. The plotted strains are the average of strains in the two bottom tendons for CFRP-prestressed beams and the strain in one strand for steel-prestressed beams. The graph for control beam C50RT starts on the day that is equivalent to the day test beam C50LT was first loaded.

Measurement began on day 24 when all three test beams were unloaded and reloaded. On day 29, the loads in beams C70LT and S55LT were adjusted for consistency with C50LT, causing a minor change in strains. No other load adjustments were made after this.

3.3.1.3.1 Drop during Cooling of Beams

From Figure 3.22, it is seen that the prestressing strain in all three beams dropped between days 31 and 35, during the transition from room to low temperature. This occurred because of a drop in sustained load as well as difference in the thermal expansion coefficients of the prestressing materials and concrete. Table 3.5 summarizes
the drops in prestressing strain and sustained load during this transition. The following observations can be made from this table:

- Although the drop in sustained load was similar for C70LT and S55LT, the drop in prestressing strain was greater in C70LT ($390 \mu e > 100 \mu e$). This is due to the difference in the coefficient of thermal expansion (CTE) of CFRP tendons and steel strands compared to concrete. The CTE of steel strands and concrete are similar to each other ($10 \times 10^{-6} / ^\circ C$). Therefore, when S55LT was cooled, the strands and the concrete wanted to shrink at the same rate so the prestressing force did not change much. However, the CTE of Leadline CFRP tendons in the longitudinal direction ($-0.9 \times 10^{-6} / ^\circ C$) is negative, while the CTE of concrete is positive ($10 \times 10^{-6} / ^\circ C$). Thus, when C50LT and C70LT were cooled, the tendons wanted to expand while the concrete wanted to shrink. Consequently, the concrete exerted a compressive force on the tendons through their bond and the prestressing force and prestressing strain were reduced.

- The drops in prestressing strain in C50LT and C70LT were of the same magnitude, because both beams had CFRP prestressing which shrank at the same rate during cooling. However, the drop in prestressing strain was slightly greater in C50LT than in C70LT ($450 \mu e > 390 \mu e$). This occurred because (a) C50LT experienced a greater drop in sustained load than C70LT ($5.7 kN > 3.2 kN$) and (b) C50LT was cracked under the sustained load while C70LT was not.
3.3.1.3.2  Time-Dependent Changes

Table 3.6 shows the prestressing strains at the start and end of the low temperature period in contrast to those of the control beams at room temperature for an equivalent period of time. This equivalent period for each beam starts after the same number of days from initial loading as the low temperature period and is of the same length to allow for consistent comparison. The gradual drop in sustained load over time obviously accounts for some of the loss in prestressing strain. These values are listed to enable consistent comparison between any two beams.

Close examination of Figure 3.22 and Table 3.6 provides the following observations:

- Prestressing strain was stable in C50LT but gradually decreased in C50RT, although both beams have similar drops in sustained load (1.5$kN \approx 1.3$kN). This can be explained by the fact that at low temperatures, freezing produces a high initial rate of creep in concrete but it quickly drops to zero (Johansen and Best, 1962).

- Prestressing strain was stable in both C70LT and C70RT.

- Prestressing strain gradually decreased in S55LT and S55RT. The magnitude of this change was bigger in S55LT than S55RT ($-100\mu\varepsilon > -30\mu\varepsilon$) because the drop in sustained load was bigger ($1.0kN > 0.4kN$).

- In C50LT and C70LT prestressing strain was stable, while in S55LT it decreased. This was despite the fact that S55LT had the lowest drop in sustained load among the three. This might indicate that at low temperatures, beams prestressed with CFRP lose less strain over time than those prestressed with steel.
3.3.1.3.3 Effect of Ambient Conditions

Increases and decreases in prestressing strain happened at the same time in all three test beams. This held for the control beams as well. This shows dependence on ambient conditions (i.e. relative humidity and temperature), conditions that all three test beams experienced simultaneously. The relationship between prestressing strains and relative humidity was weak (Figure 3.23). For example, the marked drops in relative humidity between days 141 and 147 or between days 181 and 188 are not accompanied by a distinct change in prestressing strain in any of the beams. However, a good direct relationship was found between prestressing strains and temperature (Figure 3.24). For example, the local peaks in temperature on days 62, 69, 89, 96 and 113 correspond to local peaks in prestressing strain in all three beams. Temperature affected prestressing strain in the following two ways:

- There was a direct relationship between temperature and the sustained load applied to the beam. This is shown in Figure 3.25 for the low temperature period. (A small range of sustained load is shown to magnify changes in load.) This direct relationship was the net result of complex length changes in different elements of the sustained load setup due to the temperature change. Therefore, a rise in temperature increased sustained load on the beam. This, in turn, increased prestressing strain.

- In C50LT and C70LT, the coefficient of thermal expansion (CTE) of the Leadline CFRP tendons in the longitudinal direction ($-0.9 \times 10^{-6} / ^\circ C$) is negative while the CTE of concrete is positive ($\approx 10 \times 10^{-6} / ^\circ C$). Therefore, a temperature increase would expand the concrete longitudinally while the CFRP tendons would want to get shorter. Consequently, the bond between the concrete and the CFRP tendons would
induce tension in the tendons, thus increasing tensile prestressing strain. Similarly, a temperature decrease would cause concrete to contract, induce compression in the CFRP tendons and decrease tensile prestressing strain. This does not apply to S55LT because the CTE of concrete is similar to that of prestressing steel \( \approx 10 \times 10^{-6} / ^\circ C \).

### 3.3.1.3.4 Changes in Different CFRP Tendons

In C50LT and C70LT one top tendon and two bottom tendons were instrumented with strain gauges. Figure 3.26 and Figure 3.27 present the strain in these tendons over the sustained load period. The following observations can be made:

- Strain in the two bottom tendons were very close to each other because both tendons were at the same level and the sustained load was transversely centered on the web.
- Strains in the top tendons were lower than strains in the bottom tendons because strain changes linearly from compressive in the top to tensile at the bottom of the beam.
- Strain changes over time in the top tendons closely followed those in the bottom tendons. This is to be expected and shows consistency of strain readings.

### 3.3.2 Loading to Failure

C70LT and C60RT were monotonically loaded to failure and their behaviour was compared. Test beam C70LT with a history of sustained load at low temperature was loaded to failure at low temperature, while control beam C60RT with a history of sustained load at room temperature was loaded to failure at room temperature.

Figure 3.28 shows the load-deflection behaviour of the two beams.
Figure 3.29 shows the cracking pattern and failure mode of control beam C60RT. The beam had a flexural compression failure by crushing of concrete at the top flange above a flexural-shear crack just outside the constant-moment-zone at a load of 143 kN (Figure 3.29a,b). This was immediately followed by a secondary failure. One CFRP tendon ruptured (Figure 3.29c,d), two other tendons slipped (Figure 3.29e,f) and the load dropped to zero.

Test beam C70LT, on the other hand, failed by slip of the CFRP tendons. Slip occurred in several stages. These stages are marked in Figure 3.28. At 104 kN, a small slip of 6 mm occurred in one bottom tendon (first slip) resulting in a small drop in load. The beam then continued to resist more load up to 116 kN, at which point all four tendons slipped by 4 to 32 mm (second slip) and the load dropped significantly. The beam was further loaded to 92 kN when an additional slip of 23 to 29 mm occurred (third slip), causing another significant load drop. Figure 3.30a shows the slip at various stages at one end of the beam. Figure 3.30b shows cracks in the midspan region after failure. Figure 3.31 shows the load-slip behaviour of the beam.

The figures demonstrate that C70LT achieved a much lower peak load than C60RT (116 kN < 143 kN), which is a 19% reduction in load capacity. Also, C70LT failed due to slip of CFRP tendons while C60RT failed in flexure since the concrete started crushing before the tendons slipped and ruptured. Thus, the bond between CFRP tendons and concrete was weaker in C70LT than in C60RT. To identify the factors contributing to this decrease in ultimate load capacity, the differences in the two beams, summarized in Table 3.3, are discussed.
Chapter 3  Sustained Loading at Low Temperature

Compared to C60RT, C70LT has a slightly higher prestress level in the bottom layer of tendons, which suggests a longer transfer length. This likely contributed to triggering the bond failure.

The other important factor was temperature during both sustained loading and loading to failure. C70LT was loaded at low temperature while C60RT was tested at room temperature. Since the transverse coefficient of thermal expansion (CTE) of CFRP tendons \( (27 \times 10^{-6} / ^\circ C) \) is about 3 times that of concrete \( (10 \times 10^{-6} / ^\circ C) \), the CFRP tendons shrink at a higher rate than the surrounding concrete. Thus, it is likely that the low temperature adversely affected bond strength in C70LT and contributed to the premature slip failure.

Clearly, the fact that C60RT was exposed to a slightly higher sustained load than C70LT \( (52.1kN > 47.2kN) \) for a significantly longer period of time \( (651 days > 163 days) \) and that it was cracked during this period did not adversely affect its ultimate strength relative to C70LT.

### 3.4 Summary

Three 13-year-old, 4.4 m long precast concrete T-beams were tested, of which two were prestressed to various levels with CFRP tendons and one with conventional steel strands. The beams were exposed to \(-27 ^\circ C\) while being subjected to a sustained load of 25% of their flexural capacity for 163 days. The sustained load produced cracking in two beams with lower prestress levels. Results were compared to those obtained from three similar beams subjected to the same sustained load at room temperature. Deflection increase
under sustained load at low temperature was generally small and similar to that at room temperature. Prestressing strain had a direct relationship with temperature in the CFRP-prestressed beams. Exposure to low temperature likely contributed to a 19% reduction in strength and a change in failure mode from flexural to bond failure.
Table 3.1: Properties of Leadline CFRP prestressing tendons  
(Mitsubishi Kasei Corp., 1993)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>Fibre</td>
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<td>Resin</td>
<td>Epoxy</td>
</tr>
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<tr>
<td>Ultimate Stress (Measured) *</td>
<td>3,200 MPa</td>
</tr>
<tr>
<td>Ultimate Strain (Guaranteed)</td>
<td>1.3 %</td>
</tr>
<tr>
<td>Ultimate Strain (Measured) †</td>
<td>1.69%</td>
</tr>
<tr>
<td>Longitudinal Thermal Expansion Coefficient</td>
<td>$-0.9 \times 10^{-6} /\degree C$</td>
</tr>
<tr>
<td>Transverse Thermal Expansion Coefficient</td>
<td>$27 \times 10^{-6} /\degree C$</td>
</tr>
<tr>
<td>Density</td>
<td>1.53 g/cm$^3$</td>
</tr>
<tr>
<td>Relaxation Ratio</td>
<td>2 to 3 %</td>
</tr>
</tbody>
</table>

* Abdelrahman and Rizkalla (1997)  
† Bryan (1994)

Table 3.2. Prestressing details of test beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Description</th>
<th>Prestress Level (%)</th>
<th>Jacking Force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C50LT</td>
<td>4-8Φ CFRP Tendons</td>
<td>50</td>
<td>208</td>
</tr>
<tr>
<td>C70LT</td>
<td>4-8Φ CFRP Tendons</td>
<td>70</td>
<td>291</td>
</tr>
<tr>
<td>S55LT</td>
<td>2-13Φ Steel Strands</td>
<td>55</td>
<td>202</td>
</tr>
</tbody>
</table>

Table 3.3. Differences between test and control beams for loading to failure

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Test Beam C70LT</th>
<th>Control Beam C60RT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress level in bottom layer of tendons</td>
<td>70% of ultimate</td>
<td>50% of ultimate</td>
</tr>
<tr>
<td>Temperature at loading to failure</td>
<td>Low ($-27 \degree C$)</td>
<td>Room</td>
</tr>
<tr>
<td>Sustained Loading</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Magnitude</td>
<td>47.2 kN</td>
<td>52.1 kN</td>
</tr>
<tr>
<td>Duration</td>
<td>163 days</td>
<td>651 days</td>
</tr>
<tr>
<td>Cracking</td>
<td>Uncracked</td>
<td>Cracked</td>
</tr>
<tr>
<td>Temperature</td>
<td>Low ($-27 \degree C$)</td>
<td>Room</td>
</tr>
</tbody>
</table>
Table 3.4. Midspan deflection over time for low and room temperature (control) beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Sustained Load at Start† (kN)</th>
<th>Deflection</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Start‡ (mm)</td>
<td>End‡ (mm)</td>
<td>Change (%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C50LT Test</td>
<td>44.5</td>
<td>14.4</td>
<td>15.2</td>
<td>+5.3</td>
<td></td>
</tr>
<tr>
<td>C50RT Control*</td>
<td>52.6</td>
<td>12.5</td>
<td>13.0</td>
<td>+3.9</td>
<td></td>
</tr>
<tr>
<td>C70LT Test</td>
<td>47.2</td>
<td>8.5</td>
<td>8.4</td>
<td>-1.5</td>
<td></td>
</tr>
<tr>
<td>C70RT Control*</td>
<td>52.1</td>
<td>8.6</td>
<td>9.1</td>
<td>+6.1</td>
<td></td>
</tr>
<tr>
<td>S55LT Test</td>
<td>46.1</td>
<td>15.7</td>
<td>15.9</td>
<td>+1.0</td>
<td></td>
</tr>
<tr>
<td>S55RT Control*</td>
<td>52.1</td>
<td>15.7</td>
<td>15.8</td>
<td>+0.6</td>
<td></td>
</tr>
</tbody>
</table>

* Control beam data courtesy of Braimah (2003)
† Start and end refer to the start and end of the low temperature period for test beams and the equivalent period for control beams.

Table 3.5. Drop in prestressing strain and sustained load during cooling of test beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Sustained Load (kN)</th>
<th>Prestressing Strain§ (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Before (1)</td>
<td>After (2)</td>
</tr>
<tr>
<td>C50LT Test</td>
<td>50.2</td>
<td>44.5</td>
</tr>
<tr>
<td>C70LT Test</td>
<td>50.4</td>
<td>47.2</td>
</tr>
<tr>
<td>S55LT Test</td>
<td>49.3</td>
<td>46.1</td>
</tr>
</tbody>
</table>

§ Prestressing strain relative to just before the beams were loaded (not absolute strain)

Table 3.6. Prestressing strain over time for low and room temperature (control) beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Sustained Load (kN)</th>
<th>Prestressing Strain§ (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Start‡</td>
<td>End‡</td>
</tr>
<tr>
<td>C50LT Test</td>
<td>44.5</td>
<td>43.0</td>
</tr>
<tr>
<td>C50RT Control*</td>
<td>52.6</td>
<td>51.3</td>
</tr>
<tr>
<td>C70LT Test</td>
<td>47.2</td>
<td>44.8</td>
</tr>
<tr>
<td>C70RT Control*</td>
<td>52.1</td>
<td>50.5</td>
</tr>
<tr>
<td>S55LT Test</td>
<td>46.1</td>
<td>45.2</td>
</tr>
<tr>
<td>S55RT Control*</td>
<td>52.1</td>
<td>51.7</td>
</tr>
</tbody>
</table>

* Control beam data courtesy of Braimah (2003)
† Start and end refer to the start and end of the low temperature period for test beams and the equivalent period for control beams.
§ Prestressing strain relative to just before the beams were loaded (not absolute strain)
Chapter 3                                                                              Sustained Loading at Low Temperature

Figure 3.1. Geometry and reinforcement details of beams [Braimah, 2003]

<table>
<thead>
<tr>
<th>Mark</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>MK1</td>
<td>8mm CFRP Rod</td>
</tr>
<tr>
<td>MK1A</td>
<td>13mm Steel Strand</td>
</tr>
<tr>
<td>MK2</td>
<td>8mm Steel Rebar</td>
</tr>
<tr>
<td>MK3</td>
<td>8mm Steel Stirrup</td>
</tr>
<tr>
<td>MK4</td>
<td>WFT102x102 MV 13.3</td>
</tr>
<tr>
<td>MK5</td>
<td>Steel Anchor Plate</td>
</tr>
</tbody>
</table>

a) C50LT  
Figure 3.2. End faces of test beams

b) C70LT

c) S55LT

a) Room interior  
b) Cooling fans

Figure 3.3. Cold room
Figure 3.4. Sustained load setup (units: mm)
Figure 3.5. Sustained load setup, beam loading in progress

Figure 3.6. Sustained load setup, beams loaded
Figure 3.7. Changes in sustained load over time

Figure 3.8. Temperature and relative humidity over time
Figure 3.9. Load cells placed between grips and transverse HSS under beams

Figure 3.10. Dial gauges (left) and LP (right) used to measure deflection
Figure 3.11. Strain gauge wires coming out of beam flange

a) Side view (units: mm)

↑: Dial Gauge or LP

b) Cross-section at midspan (CFRP-prestressed beams)

1, 2, 3: Strain Gauges

Figure 3.12: Location of instrumentation on test beams
Figure 3.13. Measurement station
Figure 3.14. Components of measurement station
a) Free pieces of steel strand and CFRP tendon

b) Strain gauge on seven-wire steel strand
c) Strain gauge on CFRP tendon

Figure 3.15. Strain gauges attached to free pieces of steel strand and CFRP tendon

Figure 3.16. Perceived strain during cooling of steel strand and CFRP tendon
Figure 3.17. Setup for monotonic loading to failure at low temperature

Figure 3.18. Setup for monotonic loading to failure at room temperature
Chapter 3  
Sustained Loading at Low Temperature

Figure 3.19. Load-deflection behaviour up to sustained load for low\(^\dagger\) and room temperature (control*) beams

\(\dagger\). All beams (including low temperature beams) were loaded at room temperature.

*. Control beam data courtesy of Braimah (2003)

C50LT  
Cracks marked

C70LT  
No cracks observed

S55LT  
Cracks marked

Figure 3.20. Cracks in beams after loading
Figure 3.21. Deflection at midspan over time for low and room temperature (control*) beams

*. Control beam data courtesy of Braimah (2003)

Figure 3.22. Prestressing strain§ at midspan over time for low and room temperature (control*) beams

§. Prestressing strain relative to just before the beams were loaded. Not absolute strain.

*. Control beam data courtesy of Braimah (2003)
Figure 3.23. Effect of relative humidity on prestressing strain§

§. Prestressing strain relative to just before the beams were loaded (not absolute strain)

Figure 3.24. Effect of temperature on prestressing strain§

§. Prestressing strain relative to just before the beams were loaded (not absolute strain)
Figure 3.25. Effect of temperature on sustained load

Figure 3.26. Prestressing strain\textsuperscript{\$} at midspan over time for C50LT

\textsuperscript{\$}. Prestressing strain relative to just before the beams were loaded (not absolute strain)
Figure 3.27. Prestressing strain§ at midspan over time for C70LT

§ Prestressing strain relative to just before the beams were loaded (not absolute strain)

Figure 3.28. Load-deflection behaviour of C70LT and C60RT (control) up to failure
a) Shear-flexure crack just outside of constant-moment-zone

b) Crushing of concrete in the flange above the crack

c) Concrete cover broken, CFRP tendons exposed

d) Close-Up of ruptured CFRP tendon

e) Initial state

f) After slip: Tendons 2 and 3 have slipped.

Figure 3.29. Failure mode of C60RT (control), flexural compression failure
Figure 3.30. Failure mode of C70LT, slip of CFRP tendons

**a)** Progressive slip of CFRP tendons at one end of the beam

- Initial state
- After first slip: Tendon 1 has slipped by 6 mm
- After second slip: Tendon 1 has slipped by 30 mm, Tendon 2 has slipped by 32 mm
- After third slip: Tendon 3 has slipped by 23 mm

**b)** Flexural and shear cracks in the midspan region

Figure 3.30. Failure mode of C70LT, slip of CFRP tendons
Figure 3.31. Load-Slip behaviour of C70LT
Chapter 4
High-Cycle Fatigue at Low Temperature

4.1 Introduction

Early research on concrete girders prestressed using Carbon Fibre-Reinforced Polymer (CFRP) tendons in the 1990’s proved that this technology has great promise in terms of performance and durability. This led to applications in bridges in Canada and the USA. As discussed in Chapter 2, the performance of CFRP-prestressed concrete beams has been studied under cyclic loading. However, testing was typically carried out at room temperature. To the author’s knowledge, the combined effect of cyclic loading and low temperature has not been investigated for this type of prestressed system.

4.2 Experimental Program

This chapter summarizes an experimental investigation into the performance of CFRP-prestressed concrete girders under the combined effects of high-cycle fatigue and low temperature. The following sections give a detailed description of the test beams, the cyclic and monotonic loadings they were subjected to, the cold room in which they were tested, the loading setup, the testing control station, the instruments used to gather data and special measures taken for testing at low temperature.
4.2.1 Test Beams

Seven large-scale concrete beams were tested. Five beams were prestressed with CFRP tendons and two beams were prestressed with steel strands. Table 4.1 presents details of the test beams, including their prestressing details and the loadings to which they were subjected. All beams had some history of sustained loading. Some beams were subjected to cyclic loading then monotonic loading to failure, while others were directly subjected to monotonic loading to failure. The designation for each beam indicates the type of prestressing reinforcement (CFRP tendons or steel strands), the level to which it was prestressed, and the temperature at which loadings were applied, as in the following examples:

- C50RT had CFRP tendons (C) prestressed to 50% of their guaranteed ultimate strength (50) and was subjected to sustained loading, cyclic loading and monotonic loading to failure at room temperature (RT).

- S55RLT had steel strands (S) prestressed to 55% of their guaranteed ultimate strength (55) and was subjected to sustained loading at room temperature and then cyclic loading at low temperature (RLT).

Figure 4.1 shows the geometry and reinforcement details of the beams. The concrete T-beams were 4.4 m long, had a 500 mm wide flange and were 300 mm deep. The cross-section consisted of a wide shallow flange at the top and a tapered web underneath. Each beam weighed approximately 640 kg. In addition to the prestressing which differed from beam to the other, the beams also included steel stirrups at a 100 mm spacing for shear
strength, a steel wire mesh in the flange for control of shrinkage cracks and two steel bars at the top of the web for stirrup support.

The concrete used for the beams was designed to have a 28-day compressive strength of 40 MPa, a 3-day (at prestress release) compressive strength of 30 MPa and a maximum aggregate size of 16 mm. Actual concrete strengths are listed in Appendix A. The CFRP prestressing tendons were Leadline manufactured by Mitsubishi Plastics, Inc. Their properties are summarized in Table 4.2. The tendons had a diameter of 7.9 mm, a guaranteed ultimate strength of 2300 MPa, a measured ultimate strength of 3200 MPa, a measured modulus of elasticity of 187 GPa, a fibre volume ratio of 65% and an epoxy resin matrix with spiral indentations. The seven-wire steel prestressing strands were 13 mm in diameter and of grade 1860 MPa.

Table 4.1 presents the prestressing details of the test beams, including type of prestressing reinforcement, prestress level and jacking force. All CFRP-prestressed beams were prestressed with four 8 mm CFRP tendons arranged in two layers, with two tendons in each layer. In C50RT and C50LT, all four tendons were stressed to 50% of their guaranteed ultimate strength, while in C70RT and C70LT these were stressed to 70%. In C60RT, the two tendons in the bottom layer were stressed to 50% and the two tendons in the top layer to 70% of their guaranteed ultimate strength, giving an average of 60% for all four tendons. All steel-prestressed beams contained two 13 mm seven-wire steel strands arranged in one layer. In S55RLT and S55LT, both strands were stressed to 55% of their guaranteed ultimate strength, giving a total jacking force of 202 kN, similar to that of C50RT and C50LT (208 kN). Figure 4.2 shows the end faces of sample beams.
with CFRP and steel prestressing reinforcement. The centroid of the reinforcement was at the same depth for both the CFRP-prestressed and steel-prestressed beams.

Table 4.3 presents the timeline of loadings applied to the test beams. All beams were fabricated in 1995. Starting in 1997 (at 2 years of age), C50RT, C70RT, C60RT and S55RLT were subjected to a sustained load of 53 kN (29% of their experimental flexural capacity) for 651 days at room temperature (Braimah, 2003). Starting in 2008 (at 13 years of age), C50LT, C70LT and S55LT were subjected to a sustained load of 46 kN (25% of their experimental flexural capacity) for 196 days at low temperature (−27 °C). This was outlined in Chapter 3. Then, C50RT, C50LT, C70RT and S55RLT were subjected to cyclic loading in 2009 (at 14 years of age) for the present study.

### 4.2.2 Applied Loads

Table 4.1 lists the types of loadings to which each beam was subjected and the respective temperature of each loading. All beams had a history of sustained loading as indicated earlier. Some beams were subjected to cyclic loading at room temperature, some were subjected to cyclic loading at low temperature and some beams were not subjected to cyclic loading at all. Finally, all beams were monotonically loaded to failure. For any given beam, sustained loading, cyclic loading and monotonic loading to failure were performed at the same temperature, with one exception: S55RLT had a history of sustained loading at room temperature from a previous study (Braimah, 2003) but it was exposed to cyclic loading at low temperature for the sake of this study. The following sections discuss each type of loading in further detail.
4.2.2.1 Sustained Loading

C50LT, C70LT and S55LT were subjected to a sustained load and kept at $-27 \, ^\circ C$ for 163 days. The sustained load represented 25% of the experimental flexural capacity for the CFRP-prestressed beams and 35% of that for the steel-prestressed beam. Details of the sustained loading procedures were explained in Chapter 3. Similarly, C50RT, C70RT, C60RT and S55RLT were subjected to a sustained load at room temperature for 651 days. The sustained load represented 29% of the experimental flexural capacity for the CFRP-prestressed beams and 40% of that for the steel-prestressed beam. See Braimah (2003) for details.

4.2.2.2 Cyclic Loading

Cyclic (fatigue) loading consisted of 3 million cycles of loading between 30 and 60 kN at a frequency of 0.85 Hz. The load range of 30 to 60 kN represented 21 to 42% of the experimental flexural capacity for the CFRP-prestressed beams and 30 to 60% of that for the steel-prestressed beam. These factors were kept constant between different beams to enable direct comparisons in which other factors are different. C50LT and S55LT were subjected to this cyclic loading at $-28 \, ^\circ C$ while C50RT and C70RT were subjected to it at room temperature.

One exception was C70LT which was accidentally overloaded at the start. It was loaded from 37 to 84 kN at 0.9 Hz for the first 50,000 cycles. However, the remaining 2,950,000 cycles were from 30 to 60 kN at 0.85 Hz, like the other beams. Although the first 50,000 cycles were relatively aggressive (84 kN vs 60 kN), the ratio of aggressive cycles to the total of 3 million cycles was relatively small (1.6%).
4.2.2.2.1 **Number of cycles:** Most studies of cyclic loading typically include 2 million loading cycles. In order to make the effects of cyclic loading more pronounced, each beam was subjected to 3 million cycles. This was considered high cyclic loading but it was also consistent with the fact that CFRP tendons are durable and have excellent fatigue performance.

4.2.2.2 **Load range:** A load range of 30 to 60 kN was used. The lower limit of 30 kN was chosen because it was below the decompression load for all beams. This ensured that with every cycle, cracks completely closed and opened. The grinding of the concrete at the crack interface made the cyclic loading rather aggressive. Figure 4.3 shows the opening of cracks and the accumulation of small concrete particles on the reaction beam below the cracks during cyclic loading. The upper limit of 60 kN was chosen such that the corresponding deflection per cycle was close to serviceability limits for floor construction specified in codes. The CSA A23.3 Code (2004) dictates that the immediate deflection due to specified live load for roof or floor construction supporting or attached to non-structural elements not likely to be damaged by large deflections should not exceed \( l_n/240 \). The term \( l_n \) for these beams is the free span of 4 m. Therefore, the upper deflection limit is \( 4m/240 = 17 \text{mm} \). The deflections per cycle for C50RT, C50LT, C70RT and S55RLT were approximately 20, 23, 15 and 20 mm, respectively. These deflections during cyclic loading were noticeable and created considerable curvature in the beams (Figure 4.4).

The load range in cyclic loading (60 – 30 = 30kN) represents a rather heavy live load, e.g. a heavy truck passing over a bridge, while the lower limit of cyclic loading (30 kN) represents the total dead load. The test beams can be considered a small-scale model of a
real girder used in a building or a bridge. Dead loads are under-represented in small-scale models because when dimensions are scaled down linearly, weight (which is directly proportional to volume) does not decrease linearly but reduces at a higher rate. To simulate a full-scale beam, dead loads should be approximately equal to live loads. Choosing the upper limit of cyclic loading as 60 kN satisfies this criteria perfectly, since dead and live load are both equal to 30 kN, as mentioned above.

4.2.2.2.3 Cycling frequency: Due to the large number of cycles required (3 million), a higher cycling frequency was preferred to save time. However, too high a frequency could cause damage to the testing machine and hydraulic lines. The tradeoff was a frequency of 0.85 Hz, the highest frequency achievable without risking damage to the lines. At this frequency, 3 million cycles would take 41 days of continuous cycling.

4.2.2.2.4 Starting cyclic loading: To start cyclic loading the following steps were taken:

1. The beam was monotonically loaded to 45 kN in stroke control. This would be the mean value of the loading range.
2. The machine was set to load control since cycling should always be between 30 and 60 kN of load regardless of the stroke.
3. Cyclic loading was started at a low frequency and small amplitude.
4. Frequency was gradually increased to 0.85 Hz and amplitude was gradually increased until the 30 to 60 kN load range was achieved. This typically occurred within the first 100 cycles.

4.2.2.2.5 Load and stroke variations: Figure 4.5 shows the load variation over a few seconds of cyclic loading. Due to the high frequency of cycling, a perfect sinusoidal
wave was not achieved. However, the minimum and maximum loads of 30 and 60 kN were attained. Figure 4.6 shows the stroke over time corresponding to the same cycles as in Figure 4.5.

4.2.2.6 Periodic Monotonic Tests: Periodically, cyclic loading was stopped and monotonic tests were run from zero load to the upper limit of cyclic loading (60 kN). Comparing results from these tests helped identify gradual changes in stiffness and camber during cyclic loading. These tests were run at the start (before any cycles), and then again at 1 million and 2 million cycles. After 3 million cycles, a similar monotonic test was run, but loading continued up to failure. The monotonic tests were conducted in stroke control at the rate of 1 mm/min in the same loading machine.

4.2.2.3 Monotonic Loading to Failure

Monotonic tests to failure were performed in the same loading machine as was used for the cyclic loading at a rate of 1 mm/min. For beams that were exposed to cyclic loading, this was done after the completion of 3 million cycles. Crack propagation patterns were marked on the surface of the beams at regular intervals.

4.2.3 Cold Room

The test beams were placed inside a cold room equipped with a refrigeration system and a loading machine capable of cyclic and monotonic loading (Figure 4.7). This made it possible to subject beams to cyclic or monotonic loading at room or low temperature.
The cold room was 9.2 m long, 3.5 m wide and 3.4 m high. For testing at low temperature, fans connected to a refrigeration system blew cold air into the room (Figure 4.8), keeping the temperature at \(-28 \pm 1 \, ^\circ\text{C}\) and relative humidity almost constant at 84%. The fans had regular thaw cycles to melt the ice inside them, but the resulting temperature fluctuations were negligible. The room was insulated to maintain a low temperature. A thick window was built into the cold room door to allow visual observation of the test from outside the cold room (Figure 4.9).

### 4.2.4 Loading Setup

A special setup was prepared to apply the cyclic or monotonic load in a four-point-bending scheme with a span of 4 m and a constant moment zone of 500 mm (Figure 4.10). The hydraulic ram pushed down on the centre of the steel spreader beam, which transferred the load to two transverse hollow structural sections (HSS) resting directly on the top flange across the full width (Figure 4.11). The loading points were 500 mm apart and symmetrical about the centreline of the beam. The test beam was supported on two points 4 m apart. The supports rested on a stiff steel reaction beam and were securely fastened to it via clamps (Figure 4.12). The spreader beam was simply-supported on the transverse HSSs, with a roller on one side and a pin on the other. Similarly, the test beam was simply-supported on the reaction beam.

During cyclic loading, the ram moved down and up as it loaded and unloaded the test beam. As the ram moved down, it created sagging in the test beam. When the ram moved up, prestress pushed the beam back up. The ram and beam moved down and up together. A load cell built into the tip of the ram measured the magnitude of exerted load.
A swivel joint consisting of a lubricated spherical head and seat was devised between the ram and spreader beam so no moment would be exerted on the ram. The hydraulic ram had a range of motion of only 154 mm. As such, the height of the supports was adjusted to make full use of this range of motion.

To prevent the beam from gradually sliding away from its original position under the loading cycles, special measures were taken. For transverse bracing, two wooden braces with guide angles were placed on the flange between the columns of the loading frame (Figure 4.13a). If the beam moved transversely to one side, the wooden braces would come in contact with the columns and metal angles fixed to the wood on either side of the flange would keep the beam from moving any closer. The ends of these wooden braces were lubricated with grease to reduce friction with the columns. For longitudinal bracing, chains connected each end of the beam with the columns of the loading frame (Figure 4.13b). If the beam moved longitudinally to one side, the tension in the chains would keep the beam from moving any further.

### 4.2.5 Control Station

All tests were managed from a control station outside the cold room (Figure 4.14a). The control station included the following components:

1. Power unit (Figure 4.14a), bottom-right.
2. Control panel for hydraulics (Figure 4.14b). This allows the user to:
   a. Turn the hydraulics on or off
   b. Turn the actuator on or off
   c. Jog the ram up or down at slow or fast speed
d. Shut the hydraulics off in case of an emergency

3. Control panel for the ram (Figure 4.14c). This allows the user to:
   a. Start, hold and finish tests
   b. Switch between load and stroke control
   c. Set load and stroke limits that automatically stop the test or shut off hydraulics when triggered to prevent overloading of specimen
   d. Set waveform parameters via the bottom display (Figure 4.14d). For cyclic tests, these include wave type, shape, amplitude, frequency and mean point. For monotonic tests, these include ramp type, amplitude and loading rate.
   e. Monitor load, deflection, number of cycles, etc. via the top display. For example, Figure 4.14e shows the minimum and maximum loads in each cycle as well as the total number of cycles so far.

4.2.6 Instrumentation

Measured quantities included load, deflection at midspan and loading points, longitudinal strain in prestressing reinforcement at midspan, slip of prestressing reinforcement at beam ends, camber, temperature and relative humidity. This section describes the instruments used and how readings were made.

Load was measured by the load cell built into the hydraulic ram (Figure 4.11).

Deflection was measured by linear potentiometers (LPs) placed at midspan and under loading points (Figure 4.15 and Figure 4.17a). LPs were installed and used only for the
periodic monotonic tests. They were removed during cyclic loading to prevent damage from millions of cycles of movement.

Strain in prestressing tendons/strands was measured by strain gauges attached to them when the beams were originally fabricated. The strain gauge wires came out of the top flange (Figure 4.16). In the CFRP-prestressed beams the two bottom tendons and one top tendon were instrumented with strain gauges at midspan (Figure 4.17b). In the steel-prestressed beams one steel strand was instrumented with a strain gauge at midspan. The strain gauges used were foil strain gauges, SHOWA 5 mm, type N11-FA-5-120-11.

Only change in prestressing strain from the moment of loading could be measured. The total strain was unknown because the pre-existing strain in the prestressing reinforcement (pre-strain) was not measured. To measure total strain, one would have to continuously monitor prestressing strain starting just before the beam was originally pre-tensioned (zero strain).

Slip of prestressing reinforcement at beam ends was monitored manually. Steel discs were glued to the end faces of the beam with epoxy (Figure 4.18a). Dial gauges were mounted on stands that attached magnetically onto these discs, with the tips of the dial gauges resting on the end of prestressing tendons/strands (Figure 4.18b). Also, for large amounts of slip, a ruler was placed next to each tendon/strand and against the concrete surface at the beam end to measure changes in the length of reinforcement protruding from the beam end (Figure 4.18c).

Camber in the prestressed beams was measured before and during cyclic loading to check for camber reduction. To measure camber, the cold room floor was used as a reference,
assuming it was perfectly flat. The vertical distance between the cold room floor and the test beam was measured at midspan \((h_m)\) and at both supports \((h_{S1} \text{ and } h_{S2})\). Camber was then found as follows:

\[
Camber = h_m - \left( \frac{h_{S1} + h_{S2}}{2} \right) \tag{4-1}
\]

To get the vertical distance at each cross-section, it was measured on either side of the top flange and then averaged. A weighted plumb-bob was hung from these points on the beam edge to serve as a guide in keeping the tape measure perfectly vertical.

Temperature in the cold room was measured by a thermometer built into the inner wall and shown on a digital display outside (Figure 4.8b). Beams were assumed to have the same temperature as the air in the cold room, because they were exposed to the desired temperature in the cold room for at least 24 hours before each test.

Relative humidity was measured using a humidity-meter placed in the centre of the room. A power unit sent an excitation of 5 V to the humidity-meter, the output voltage was read using a voltmeter and converted to relative humidity using a formula given by the manufacturer.

Appendix B lists the accuracies of the instruments used and the measurements made.

The load cell was calibrated using known pairs of load and output voltage. LPs were calibrated manually using a mechanical micrometer before being installed in position. Strain gauges were shunt calibrated.

A data acquisition unit (DA) (Figure 4.19a) and computer were used as the processing centre for load, deflection and prestressing strain measurements. Wires from instruments
were connected to cards which were plugged into the back of the DA (Figure 4.19b). A computer program named StrainSmart interacted with the DA during tests to collect electrical signals from instruments, convert them into measured quantities using calibration data, display them graphically in real time and store them in spreadsheets.

### 4.2.7 Special Measures

Testing at temperatures as low as $-28 \, ^\circ\text{C}$ required taking the following special measures:

1. Protecting self from the cold: The student had to wear a winter coat and toque to keep himself warm inside the cold room in addition to standard safety equipment (hard hat, gloves, boots, goggles). He was not to stay in the room for over 5 minutes at a time to prevent frost-bite. He was supervised by another person whenever inside the room for safety.

2. Protecting electrical equipment from the cold: All electrical instruments in the cold room were rated by manufacturers to function at $-28 \, ^\circ\text{C}$. Also, the control station was safe because it was already located outside the cold room. However, the following equipment had to be protected:

   a. Data acquisition unit (DA) and computer: These were stationed outside the cold room (Figure 4.20a). The DA was connected to the instruments inside the cold room via long wires that passed through a small insulated duct in the cold room wall (Figure 4.20b).

   b. Electrical units inside cold room: These could not be moved outside because their cables were too short. Therefore, a special ‘hot box’ with Styrofoam insulation
was built to contain and insulate these units (Figure 4.21a,b). The slots in the box for incoming and outgoing cables were filled with fibreglass insulation (Figure 4.21c). A digital thermometer was placed outside the cold room with a sensor located inside the hot box to ensure the temperature in the box was within the operating range of the units (Figure 4.21d). It was found that the heat produced by the electric units kept the air sufficiently warm.

3. Protecting hydraulic fluid from the cold: During cyclic loading hydraulic fluid constantly flowed in and out of the piston. Moving fluid that left the cold room travelled through the pump where it was re-heated. Thus, freezing of hydraulic fluid was prevented. In the same way, when the room was being cooled from room temperature to low temperature before testing at low temperature began, the hydraulic fluid had to be kept in motion to prevent it from freezing. This was achieved by removing the spreader beam and setting the ram to move up and down freely in a stroke cycle.

4. Keeping room temperature low: The door was opened for minimal amounts of time and only when entering or leaving the room. Also, the duct in the cold room wall for passing of wires was filled with fibreglass insulation (Figure 4.20b).

5. Waiting for room to cool down and for test beam to get cold to its core before testing at low temperature: The complete transition from room to low temperature took almost two days.

6. Calibrating instruments at the low temperature: The instruments were placed in a freezer at sub-zero temperature, taken out and calibrated before they could warm up to room temperature. No measurable difference was found between calibration graphs at
room and low temperature. It was assumed this would also hold for the low temperatures used in this test (−28 °C).

### 4.3 Results and Discussion

Test results are presented in this section. This includes a comparison of the load-deflection behaviour of the different beams described in Table 4.1. CFRP-prestressed and steel-prestressed beams are discussed separately.

#### 4.3.1 CFRP-Prestressed Beams

The following sections discuss differences in load capacity and cracking/decompression loads between the beams, and identify trends such as stiffness degradation, stiffening in the upper range of cyclic loading and camber reduction during cyclic loading.

##### 4.3.1.1 Load Capacity

Figure 4.22 shows the load-deflection behaviour of all CFRP-prestressed beams for monotonic loading to failure. Table 4.4 lists key values based on this figure.

##### 4.3.1.1.1 Failure Modes

The failure mode of all CFRP-prestressed beams was slip of CFRP tendons, except for C60RT which failed by flexural compression by crushing of concrete at the top flange, followed shortly by the rupture of one tendon and the slip of two other tendons. Figure 4.23, Figure 4.24, Figure 4.25, Figure 4.26 and Figure 4.27 show the failure modes for
C50RT, C50LT, C70RT, C70LT and C60RT, respectively. Figure 4.28, Figure 4.29 and Figure 4.30 show the load-slip behaviour of C50RT, C50LT and C70LT, respectively. There is no load-slip graph for C70RT because slip measurements were not made for this beam.

In theory, the ultimate moment capacity of a beam that fails in flexure is independent of its prestress level and depends only on its cross-section, reinforcement ratio and reinforcement type. Because these factors are similar in all of the CFRP-prestressed beams, it shall be assumed that if bond failure had not occurred, all beams would have achieved a load capacity equal to that of C60RT which failed in flexure at 143 kN. Thus, the load capacity of each beam is compared to this value, as listed in Table 4.4 in the form of percentages. These ratios range from 91% (9% loss) for C50RT to 69% (31% loss) for C50LT. (Note that the ratio reported for C50LT is at initial slip. The beam did resist higher loads before ultimate slip.)

In addition to developing its full flexural capacity, C60RT was also not exposed to cyclic loading or low temperature, the two main parameters studied in this chapter. Therefore, C60RT will be used as a control (reference) beam in this study.

4.3.1.1.2 Beams Tested at Room Temperature

a) C50RT vs C70RT (Effect of prestress level): Slip occurred at a lower load in C70RT than in C50RT (123kN < 130kN). The main difference was the higher level of prestress in C70RT compared to C50RT (70% > 50%). Therefore, the higher prestress level weakened the CFRP-concrete bond. This occurred because at a higher prestress level, the transfer length is longer (Soudki et al., 1997), which causes slip to occur at a lower load.
b) C50RT vs C60RT (Effect of cyclic loading):  C50RT failed at a lower load than C60RT (130kN < 143kN) with a different failure mode (bond failure vs flexural compression failure). The differences included (a) the higher prestress level in the top layer of tendons for C60RT compared to C50RT (70% > 50%), and (b) cyclic loading at room temperature for C50RT compared to no cyclic loading for C60RT. In the previous paragraph, it was established that higher prestress level weakens the CFRP-concrete bond leading to lower load capacity. This factor should have put C60RT at a disadvantage compared to C50RT, yet C50RT slipped at a lower load than C60RT. This suggests that the cyclic loading applied to C50RT weakened the CFRP-concrete bond, leading to an 8% reduction in load capacity.

c) C70RT vs C60RT (Effect of prestress level and cyclic loading):  C70RT failed at a lower load than C60RT (123kN < 143kN) with a different failure mode (bond failure vs flexural compression failure). The differences included: (a) higher prestress level in the bottom layer of tendons for C70RT compared to C60RT (70% > 50%), and (b) cyclic loading at room temperature for C70RT compared to no cyclic loading for C60RT. It was previously established that both higher prestress level and cyclic loading weaken the CFRP-concrete bond. In this case, both factors contributed to the weaker bond in C70RT compared to C60RT, leading to a total of 13% reduction in load capacity.

4.3.1.1.3 Beams Tested at Low Temperature

a) [C50LT and C70 LT] vs [C50RT, C70RT and C60RT] (Effect of low temperature during sustained loading and loading to failure):  A quick comparison of the beams tested at low temperature and at room temperature reveals that in beams tested at low
temperature there is a small initial slip (partial failure), after which greater load is resisted up to a larger ultimate slip (global failure). These initial slips occurred at 69% of flexural capacity for C50LT and 73% of flexural capacity for C70LT\(^1\) and are shown in Figure 4.22. This phenomenon is the result of the low temperature during sustained loading and loading to failure. It is not related to cyclic loading because (a) C50LT was exposed to cyclic loading at low temperature while C70LT was not exposed to any cyclic loading, yet they both exhibited this small initial slip, and (b) C50RT and C70RT were exposed to cyclic loading at room temperature while C60RT was not exposed to any cyclic loading, but none of them experienced this small initial slip.

The premature slip that occurs during loading at low temperature can be explained by the fact that the coefficient of thermal expansion of Leadline CFRP tendons in the transverse direction is greater than that of concrete (\(27 \times 10^{-6} \, ^\circ C > 10 \times 10^{-6} \, ^\circ C\)). Therefore, the CFRP tendons shrink at a higher rate than the surrounding concrete. This significantly reduces the radial pressure and friction at the CFRP-concrete interface. Hence, the CFRP-concrete bond is weaker at low temperature compared to room temperature and therefore slip occurs at a lower load.

After the initial slip of a CFRP tendon, the ends which have moved towards the middle of the beam appear to be once again gripped by the surrounding concrete, allowing the beam to continue to resist increasing load. This may be explained by the Hoyer effect (Section 2.3.6), which caused the CFRP tendons to be larger in diameter at both ends of the beam than in the middle of the beam. When the initial slip occurred, these tapered ends were

\(^1\) The ultimate load capacities of C50LT and C70LT for flexural failure were assumed to be the same as C60RT (143kN). See Section 4.3.1.1.1 and Figure 4.22.
pulled inside towards the centre of the beam and were essentially forced to fit through a void of smaller diameter. This increased the normal stress at the CFRP-concrete interface and enhanced the frictional bond between the two.

The small initial slip may be considered as failure for design purposes because: (a) It is the beginning of bond failure. (b) It disrupts the load-deflection behaviour of the beam by suddenly widening the cracks. (c) It causes an undesirable permanent deflection. In this study, loading was applied by stroke control and therefore the load dropped. If loading were applied by load control as in real life, a large non-recoverable deflection would have occurred. (d) This initial slip might be progressive and affect serviceability, i.e. once this slip occurs, further slips may occur even at service loads. Hence, to be conservative, the loads at which initial slip occurred in C50LT and C70LT will be considered the failure loads. The two beams will not be compared with regard to load at global failure because the path of their load-deflection graphs changed after the initial slip.

All CFRP-prestressed beams that failed by slip at a peak load continued to resist loads even after the large drop in load that followed. C50RT and C70LT both demonstrated a secondary slip failure at a lower peak load compared to their maximum loads. If loading of C50LT and C70RT were also continued after their primary load drop, they too would likely have showed similar secondary slip failures.

Multiple slips are favoured in engineering practice because of load sharing to redundant members in indeterminate systems. For example, if slip of a prestressing tendon occurred in one girder of a slab-and-girder bridge, adjacent girders would take the load that the first
girder is now unable to resist. This is analogous to load redistribution in steel members, although CFRP-prestressed concrete beams are not ductile like steel.

**b) C50LT vs C50RT (Effect of low temperature):** Slip occurred in C50LT at a lower load than in C50RT ($98kN < 130kN$). The main difference was that sustained loading, cyclic loading and monotonic loading to failure were carried out at low temperature for C50LT and at room temperature for C50RT. As stated before, low temperature during loading to failure weakens the CFRP-concrete bond. It is also possible that low temperature during sustained and cyclic loading weakened the CFRP-concrete bond. C50LT continued to resist greater loads after its initial slip, with ultimate slip failure occurring at about the same load as in C50RT (130 kN). This is probably a mere coincidence because the two beams followed different paths on the load-deflection graph after the small initial slip in C50LT.

c) C70LT vs C70RT (Effect of cyclic loading and low temperature): Slip occurred in C70LT at a lower load than in C70RT ($104kN < 130kN$). The differences included (a) C70RT was exposed to cyclic loading at room temperature while C70LT was not exposed to any cyclic loading, and (b) sustained loading and monotonic loading to failure were carried out at low temperature for C70LT but at room temperature for C70RT. As stated before, cyclic loading and low temperature during loading to failure weaken the CFRP-concrete bond. It is also possible that low temperature during sustained loading weakened the CFRP-concrete bond. C70LT continued to resist greater loads after its initial slip, but ultimate slip failure still occurred at a lower load than in C70RT ($116kN < 130kN$).
d) C50LT vs C70LT (Effect of prestress level and cyclic loading): Slip occurred in C50LT at a lower load than in C70LT ($98kN < 104kN$). The differences included (a) higher prestress level in C70LT compared to C50LT ($70\% > 50\%$), and (b) cyclic loading at low temperature for C50LT compared to no cyclic loading for C70LT. It was previously established that higher prestress level weakens the CFRP-concrete bond. This factor should have put C70LT at a disadvantage compared to C50LT, but C50LT slipped at a lower load than C70LT. This confirms the earlier observation that cyclic loading weakens CFRP-concrete bond. Both C50LT and C70LT continued to resist greater loads after their initial slips.

e) C70LT vs Beams in Bryan and Green (1996) (Effect of sustained loading at low temperature): Bryan and Green (1996) loaded several CFRP-prestressed concrete beams to failure after short-term exposure to low temperature (Section 2.3.2). In that study, the type of CFRP tendon and the low temperature were the same as those in the present study. Although the embedment length provided in the beams in that study (1,300 mm) was much shorter than that in the present study (1,950 mm), in that study the mode of failure was rupture of CFRP tendons in all cases without any slip, while in the present study C70LT failed by slip of CFRP tendons. The main difference between the beams in that study and C70LT of the present study is that C70LT had a history of sustained loading at low temperature.$^2$ Therefore, the sustained loading at low temperature in C70LT weakened the CFRP-concrete bond.

$^2$ It should be noted that the prestressing level in that study was slightly lower than that in this study ($60\% < 70\%$), but this still does not compensate for the large difference in embedment lengths provided.


4.3.1.1.4 Development Length and Slip Failure

Since four of the five CFRP-prestressed beams failed by slip of CFRP tendons, in this section it is determined if sufficient embedment length (the length between the beam ends and the points of maximum moment) was provided to produce enough bond strength to bring about flexural failure of the beams by rupture of the tendons. In other words, the objective is to check whether the embedment length exceeded the development lengths for the CFRP tendons.

Soudki et al. (1997) determined ranges for the transfer length \( (L_t) \) of the test beams used in this study. These ranges are 600 to 750 mm for C50RT and C50LT, 650 to 800 mm for C70RT and C70LT, and 650 to 700 mm for C60RT. The upper limit of each range is used in calculating development length, since this is the critical case for bond slip.

The flexural bond length \( (L_{fb}) \) of the tendons in the test beams can be found using the following equation proposed by Mahmoud et al. (1997):

\[
L_{fb} = \frac{(f_{pu} - f_{pe})d_b}{\alpha_{fb}f_c^{0.67}} = \frac{(3,200 - f_{pe}) \times 7.9}{1.0 \times f_c^{0.67}} \quad (4-2)
\]

where \( f_{pu} \) is ultimate tensile stress, \( f_{pe} \) is effective prestress, \( d_b \) is tendon diameter, \( f'_c \) is concrete strength at time of test and \( \alpha_{fb} \) is 1.0 for Leadline for N-mm units (Section 2.3.5).

At the time of monotonic loading to failure the beams were 13 or 14 years old (Table 4.3). For each beam, concrete strength \( (f'_c) \) at this time was calculated by using the 28-day compressive strength determined by Braimah et al. (2003), using the provision of ACI (1978) that concrete strength at one year is 1.2 times that at 28 days, and assuming this strength did not change between 1 and 14 years. Appendix A lists the results.
Soudki et al. (1997) also calculated losses at transfer for the beams used in this study. Adding long-term losses to losses at transfer, \( f_{pe} \) was calculated as 690 MPa for C50RT and C50LT, 1,035 MPa for C70RT and C70LT, and 863 MPa for C60RT. These calculations were made using the recommendations of ACI (2004) and CPCI (2007).

Thus, the development length for the beams can be calculated as follows:

**C50RT and C50LT**

\[
\begin{align*}
    f_{pe} &= 690 \text{MPa}, \quad f_c' = 48 \text{MPa} \implies L_{fb} = 1,482 \text{mm} \\
    L_d &= L_e + L_{fb} = 750 \text{mm} + 1,482 \text{mm} = 2,232 \text{mm}
\end{align*}
\]

\( (4-3) \)

**C70RT and C70LT**

\[
\begin{align*}
    f_{pe} &= 1,035 \text{MPa}, \quad f_c' = 42 \text{MPa} \implies L_{fb} = 1,398 \text{mm} \\
    L_d &= L_e + L_{fb} = 800 \text{mm} + 1,398 \text{mm} = 2,198 \text{mm}
\end{align*}
\]

\( (4-4) \)

**C60RT**

\[
\begin{align*}
    f_{pe} &= 863 \text{MPa}, \quad f_c' = 40 \text{MPa} \implies L_{fb} = 1,569 \text{mm} \\
    L_d &= L_e + L_{fb} = 700 \text{mm} + 1,569 \text{mm} = 2,269 \text{mm}
\end{align*}
\]

\( (4-5) \)

In the four-point-loading setup of this study, maximum moment occurs throughout the constant-moment-zone (CMZ). Hence, the embedment length \( (L_e) \) provided between beam ends and points of maximum moment in the loading setup is:

\[
L_e = \frac{(L_{beam} - L_{CMZ})}{2} = \frac{(4,400 - 500)\text{mm}}{2} = 1,950 \text{mm}
\]

\( (4-6) \)

Figure 4.31 shows the stress profile in CFRP tendons that can be produced by the CFRP-concrete bond in C50RT and C50LT as well as that which needs to be resisted for the tendons to rupture.
It is seen that the calculated development lengths for the test beams (2,232, 2,198 and 2,269 mm) exceed the embedment length provided (1,950 mm). Hence, theoretically, embedment length was not sufficient to cause rupture of CFRP tendons in any of the beams. However, in C60RT, one tendon did rupture. This is not surprising because: (a) there was some scatter in the transfer lengths reported by Soudki et al. (1997), (b) there was some scatter in the data and some conservatism built into the equation for flexural bond length proposed by Mahmoud et al. (1997). The difference between the calculated development lengths and the embedment length provided is within the range of error for the development length calculations.

It is believed that the actual development length was just under the provided embedment length for all beams. In C60RT, rupture of a CFRP tendon was achieved. In C50RT, C50LT, C70RT and C70LT, however, exposure to cyclic loading, and low temperature during sustained loading and loading to failure weakened the CFRP-concrete bond. This increased the development length beyond the provided embedment length in these beams and that is why they failed by slip.

### 4.3.1.2 Cracking and Decompression Load

Figure 4.32 shows a close-up of the cracking region of the load-deflection curves for all CFRP-prestressed beams during monotonic loading to failure. The loads at which stiffness changed for C50RT, C50LT, C70RT and C70LT were 25, 19, 37 and 49 kN, respectively. At monotonic loading to failure, C50RT, C50LT and C70RT had previously undergone 3 million loading cycles up to 60 kN. Therefore, the change in their stiffness was due to decompression at pre-existing cracks. C70LT, however, was
loaded for the first time and had never been cracked, so the change in its stiffness indicates the cracking load. The following observations are made:

- C50LT has a lower decompression load than C50RT ($19kN < 25kN$) because of the reduced prestress at low temperature. The coefficient of thermal expansion (CTE) of Leadline CFRP tendons in the longitudinal direction ($-0.9 \times 10^{-6} / ^{\circ} C$) is negative, while the CTE of concrete is positive ($10 \times 10^{-6} / ^{\circ} C$). Thus, when C50LT was cooled, the tendons wanted to expand while the concrete wanted to shrink. Consequently, the concrete exerted a compressive force on the tendons through their bond and some prestress was lost.

- The cracking load of C70LT is higher than the decompression load of C70RT ($49kN > 37kN$) despite being loaded at low temperature. This occurred because C70LT was uncracked and therefore, a greater load was required to overcome the tensile forces in the concrete and induce cracking.

### 4.3.1.3 Stiffness Degradation

Figure 4.33, Figure 4.34 and Figure 4.35 show the load-deflection behaviour at different stages of cyclic loading for C50RT, C50LT and C70RT, respectively. Figure 4.36 and Figure 4.37 present the average slope of these curves before and after cracking (or decompression), respectively. Figure 4.36 indicates a big drop in pre-cracking stiffness between the initial response and the response after the first 1 million cycles. However, pre-cracking stiffness seems generally stable beyond the first 1 million cycles. Figure 4.37 indicates a gradual decrease in post-cracking stiffness throughout cyclic loading. The post-cracking stiffness at initial loading was not directly compared to those during
cyclic loading because of the difference between cracking and decompression loads and the difference in the shapes of these curves.

This stiffness degradation of prestressed concrete members under cyclic loading has also been reported by Grace (2000) and Mertol et al. (2006) (Section 2.3.4). Stiffness degradation can be attributed to the following factors:

- **Reduction in tension stiffening**: Repeated loading weakens the CFRP-concrete bond. This reduces the contribution of concrete to the tensile capacity of the sections between cracks, also known as tension stiffening. Hence, the tension stiffening effect is gradually lost under cyclic loading, which results in lower stiffness as cycling progresses.

- **Reduction in modulus of elasticity of concrete, $E_c$**: The elastic strain of concrete in the compression zone increases progressively with cycling, thus reducing its secant modulus of elasticity (Section 2.4.3).

- **Reduction in effective moment of inertia, $I_e$**: Cracks propagate under cyclic loading. With the upward propagation of existing cracks, moment of inertia of cracked sections $I_{cr}$ decreases. Also, with the development of new cracks, the effective moment of inertia $I_e$, which is a value between the moment of inertia of cracked sections ($I_{cr}$) and that of uncracked sections ($I_g$), shifts towards $I_{cr}$.

### 4.3.1.4 Stiffening in Upper Range of Cyclic Loading

From Figure 4.22 it is seen that during monotonic loading to failure of C50RT, C50LT and, to a smaller extent, C70RT, there is an increase in stiffness before 60 kN with the
curve concave upwards, followed by a decrease in and stabilization of stiffness after 60 kN. This trend is also observed in the monotonic tests for these beams at 1, 2 and 3 million cycles (Figure 4.33, Figure 4.34 and Figure 4.35). This effect is not seen in C70LT and C60RT which were not subjected to cyclic loading. Therefore, it appears this phenomenon is brought about by cyclic loading, especially because 60 kN was the upper limit of cyclic loading. This ‘stiffening’ in the upper range of cyclic loading can be explained as follows: During cyclic loading between 30 and 60 kN, as load increases and approaches the upper limit of 60 kN, high tensile stress levels are created in the concrete that has not yet cracked, such that if loading continued beyond 60 kN, new cracks would form. Therefore, in the upper range of cyclic loading, the beam’s load-deflection behaviour resembles that of a new uncracked beam which is relatively stiff because it did not suffer stiffness degradation under cyclic loading.

### 4.3.1.5 Camber Reduction

Table 4.5 presents the camber of beams exposed to cyclic loading throughout cyclic loading. The following observations are made: Cambers in C70RT and S55RLT decreased immediately after the initial monotonic test. This is attributed to the formation of new cracks in the beams and the loss of concrete tensile strength. Camber in C70RT decreased further after cyclic loading. The increase in camber between 2 million and 3 million cycles is unusual and falls within the range of error for camber measurement (Appendix B). However, there was no measurable change in the camber of C50RT between 1 million and 3 million cycles.
These observations demonstrate that in CFRP-prestressed beams subjected to cyclic loading beyond cracking load, camber decreases immediately after the first cycle, continues to decrease during cyclic loading and eventually stabilizes.

4.3.1.6 Load-Strain Behaviour

Figure 4.38, Figure 4.39 and Figure 4.40 show the load-strain curves for C50RT, C50LT and C70RT, respectively, before cyclic loading. Strain data is not available from monotonic tests at 1 million, 2 million, or 3 million cycles, because the strain gauges were rated for only 100,000 reversals at 1000 με, so they all failed before 1 million cycles. Figure 4.41 and Figure 4.42 present the load-strain curves for C70LT and C60RT, which were not subjected to any cyclic loading, during monotonic loading to failure.

4.3.2 Steel-Prestressed Beam

The following sections include a discussion of the load-deflection behaviour of S55RLT before cyclic loading, its failure during cyclic loading, a prediction of its fatigue life empirically and a comparison of its fatigue life with that of CFRP-prestressed beams.

4.3.2.1 Load-Deflection Behaviour

Figure 4.43 shows the load-deflection behaviour of S55RLT up to 60 kN before cyclic loading and that of S55LT, which was not subjected to cyclic loading, up to failure. A close match is observed between S55RLT and S55LT, since the two beams have very similar geometry and prestress levels and were both monotonically loaded at low temperature. Because of these similarities, it can be assumed that in the absence of cyclic
loading S55RLT would have the same load capacity for flexural failure as S55LT (100 kN). S55LT had a flexural compression failure by crushing of concrete at the top flange above a flexural crack inside the constant-moment-zone (Figure 4.44). The steel strands had yielded prior to failure, as evidenced by the extended yield plateau in the load-deflection curve before failure (Figure 4.43).

Figure 4.43 also shows the load-deflection behaviour of C50LT before cyclic loading. A close match is observed between S55RLT and C50LT because in addition to geometric similarity and being monotonically loaded at low temperature, the two beams were also very similar in prestressing force \((202kN \approx 208kN)\) (Table 4.1). For cyclic loading, the load range applied to S55RLT was the same as that applied to C50LT and other CFRP-prestressed test beams (30 to 60 kN).

### 4.3.2.2 Failure during Cyclic Loading

S55RLT failed during cyclic loading after about 185,000 cycles. Therefore, monotonic tests after 1 million, 2 million and 3 million cycles were not possible. The failure mode was rupture of one of the seven-wire steel strands at a flexural-shear crack just outside the constant moment zone (Figure 4.45a,b). Crushing of concrete was also observed above this crack (Figure 4.45c). No slip of the steel strands was observed.

Figure 4.46 and Figure 4.47 show the stroke and load over the few hours leading up to failure. From Figure 4.46 it is seen that the beam experienced two sudden downward shifts in deflection about 2.2 and 0.7 hr before failure. These indicate that wires in the seven-wire strand ruptured one at a time. This is in agreement with the observation of
Warner and Hulsbos (1996) that: “One of the six outside wires was always the first to fail in fatigue. Successive failures occurred in other outside wires until the remaining wires were so overstressed that they failed statically.” In Figure 4.46 a gradual downward shift in deflection range is also observed, especially in the 0.7 hr before failure. This was also observed throughout cyclic loading from the very start. This could be a result of progressive yielding of steel as well as stiffness degradation as explained in Section 4.3.1. Despite the changes in deflection range, Figure 4.47 shows the applied load remained within the general 30 to 60 kN range during cyclic loading because the loading machine was set to load control.

Figure 4.46, Figure 4.47 and raw data from the test show that at the moment of failure, load dropped to 46 kN and the loading ram automatically moved down to seek additional resistance but it could not. The ram eventually ran out of stroke and stayed there, with the load at 46 kN initially and slowly dropping. The stopping of cycles and the inability of the beam to resist loads beyond 46 kN was due to the rupture of the first steel strand.

Figure 4.45b shows that all wires in the first steel strand are broken. A closer look reveals that the fracture surface of most broken wires contains a crescent-shaped fatigue crack typical of fatigue failure, and lacks the distinctive necked-down, cup-and-cone appearance of wires failing statically in tension. The two types of fracture surfaces are shown in Figure 4.48.

The magnitude of load resistance remaining after the first strand ruptured (46 kN) indicates that the second steel strand yielded but did not rupture and continued to resist load. The total load capacity of the beam was assumed to be 100 kN (Section 4.3.2.1 and
Figure 4.43). The remaining load capacity (46 kN) was approximately half of the total load capacity \( (100 \text{kN} ÷ 2 = 50 \text{kN}) \), which is a sign that about half of the reinforcement (first strand) had been lost while the other half (second strand) was still intact.

After the first strand ruptured, the ram pushed the beam down to find more load resistance. The second strand could not resist the 60 kN load and yielded further, leading to more deflection until the ram ran out of stroke. After the second strand yielded and before the ram ran out of stroke, the excessive curvature also crushed some concrete in the top flange. Upon unloading, the beam recovered some of its deflection, confirming that the second steel strand was still carrying tension.

### 4.3.2.3 Predicted Fatigue Life

To predict the fatigue life of S55RLT the stress range in the prestressing steel under cyclic loading will be found and used in the S-N curve for steel strands.

Figure 4.49 shows the load-strain curve for S55RLT before cyclic loading. The change in prestressing strain was measured by a strain gauge attached to one of the steel prestressing strands at midspan. The strain gauge ceased working at 51 kN, possibly due to damage caused by debonding of steel and concrete at that location. However, the strain can be assumed to increase linearly with load up to 60 kN (dotted line). This is valid within the elastic range only. From Figure 4.43 it can be seen that yielding begins well beyond 60 kN of load. To obtain the stress range in the steel strands under cyclic loading \( \Delta \sigma_{\text{steel}} \), strain values are read at 30 and 60 kN from Figure 4.49 to get the strain
range in the steel strands under cyclic loading ($\Delta \varepsilon_{\text{steel}}$), and this is multiplied by the modulus of elasticity ($E_{\text{steel}}$), as follows:

$$\Delta \sigma_{\text{steel}} = E_{\text{steel}} \cdot (\Delta \varepsilon_{\text{steel}}) = 195,000 \text{MPa} \times (1488 - 237) \mu \varepsilon = 244 \text{MPa}$$ (4-7)

Figure 4.50 shows the S-N curve for straight strands in prestressed concrete members suggested by Collins and Mitchell (1985). Plotting the stress range on the $y$-axis gives

$$\frac{\Delta \sigma}{f_{pu}} = \frac{244 \text{MPa}}{1860 \text{MPa}} = 0.131$$ (4-8)

which intercepts the curve at $N = 150,000$ cycles. This is in close agreement with the experimental value of 185,000, the number of cycles at which failure occurred.

### 4.3.3 Fatigue Life of CFRP vs Steel

In this section, the fatigue endurance of the CFRP tendons in C50LT and that of the steel prestressing strands in S55RLT are compared. C50LT was chosen because it underwent cyclic loading at the same low temperature ($-28 \ degree \ C$) as S55RLT and it had a similar jacking force which gives similar cracking loads to S55RLT (Figure 4.43).

Figure 4.39 shows the load-strain curve for the bottom layer of CFRP tendons in C50LT before cyclic loading. To obtain the stress range in these CFRP tendons under cyclic loading ($\Delta \sigma_{\text{CFRP}}$), one can read values at 30 and 60 kN from Figure 4.39 to get strain range in the CFRP tendons ($\Delta \varepsilon_{\text{CFRP}}$) and multiply this by their modulus of elasticity ($E_{\text{CFRP}}$). Because the two CFRP tendons in the bottom layer had different strain ranges, the average will be used. Also, the stress-strain relationship of Leadline tendons is
assumed to be linear up to 5,880 με (1,000 MPa) with an average $E_{CFRP}$ of 170 GPa (Abdelrahman, 1995).

$$\Delta \sigma_{CFRP} = E_{CFRP} \frac{\Delta \varepsilon_{CFRP,1} + \Delta \varepsilon_{CFRP,2}}{2} = 170,000 \text{MPa} \times \left(\frac{(5032 - 902) + (3858 - 699)}{2}\right) \mu \varepsilon = 620 \text{MPa} \quad (4-9)$$

When the same approach is used for C50RT using Figure 4.38, a very similar stress range is obtained, which confirms the accuracy of the readings:

$$\Delta \sigma_{CFRP} = E_{CFRP} (\Delta \varepsilon_{CFRP}) = 170,000 \text{MPa} \times (4224 - 585) \mu \varepsilon = 618 \text{MPa} \quad (4-10)$$

The stress range in the steel prestressing strands in S55RLT under cyclic loading was calculated as 244 MPa in Section 4.3.2.3.

The stress range under cyclic loading was much larger for CFRP tendons than for steel strands (620 MPa > 244 MPa). Despite this, the CFRP tendons survived 3 million cycles of loading, while the steel strands ruptured after only 185,000 cycles. Thus, it appears CFRP-prestressed concrete beams have superior fatigue performance compared to steel-prestressed concrete beams.

The shorter fatigue life of S55RLT compared to C50LT can be explained as follows:

- It is well established that CFRP has excellent fatigue performance (Section 2.3.4).
- Both beams were exposed to the same load range of 30 to 60 kN, but the load capacity of S55RLT for flexural failure was much lower than C50LT (100kN < 143kN$^3$). Therefore, S55RLT was loaded aggressively compared to C50LT since the ratio of $^3$The ultimate load capacities of C50LT and S55RLT for flexural failure were assumed to be the same as C60RT (143kN) and S55LT (100kN), respectively. See Section 4.3.1.1.1, Figure 4.22, Section 4.3.2.1 and Figure 4.43.
the upper limit of cyclic loading to load capacity was $60kN \div 100kN = 60\%$ for S55RLT, which is greater than $60kN \div 143kN = 42\%$ for C50LT.

The reason for applying the same load range of 30 to 60 kN to both C50LT and S55RLT was that they performed similarly to each other under service loads (Figure 4.43), which is the primary design criteria for prestressed beams. C50LT and S55RLT were similar in prestressing force and hence they had comparable cracking loads (25 and 31 kN). Also, the amplitude of deflection under cyclic loading from 30 to 60 kN was similar for C50LT and S55RLT (18.5 and 18 mm).

Moreover, the peak cyclic load (60 kN) is within permissible load limits for both beams from the perspective of ultimate limit state design. The 60 kN peak of the 30 to 60 kN cyclic load range represents a constant dead load of 30 kN and a repeated live load of 30 kN. In Canadian codes, the dead load factor is 1.25 and the live load factor is 1.5 (NBCC, 2005), giving a factored applied load of 82.5 kN.

$$ P_f = 1.25P_d + 1.5P_L = 1.25 \times 30kN + 1.5 \times 30kN = 82.5kN $$ \hspace{1cm} (4-11)

Given that in C60RT the crushing of concrete was followed within seconds by the rupture of a CFRP tendon, and in S55RLT failure was governed by the yielding of prestressing steel, the factored load resistances of these beams can be estimated by multiplying their load capacities obtained experimentally by the material factors for their prestressing materials. In Canadian codes, the material factor for CFRP is 0.75 (CSA S806, 2002) and the material factor for steel prestressing strands is 0.9 (CSA A23.3, 2004), giving a factored load resistance of 105 kN for C50LT and 90 kN for S55RLT.
C50LT: \[ P_r = 0.75 \times 140kN = 105kN \] (4-12)

S55RLT: \[ P_r = 0.9 \times 100kN = 90kN \] (4-13)

It is seen that the factored applied load (82.5 kN) does not exceed the factored load resistance for either beam (105 kN for C50LT and 90 kN for S55RLT).

### 4.4 Summary

After being subjected to sustained loading, all seven beams were tested in the second study. Only three of the five CFRP-prestressed beams were subjected to cyclic loading, one at −28 °C and two at room temperature, while only one of the two steel-prestressed beams was subjected to cyclic loading, at −28 °C. Cyclic loading consisted of 3 million cycles at 0.85 Hz. The load range represented 21 to 42% of the flexural capacity of the CFRP-prestressed beams and 30 to 60% of that of the steel-prestressed beam. Monotonic tests were run every 1 million cycles. Finally, all seven beams were monotonically loaded to failure. All CFRP-prestressed beams survived the 3 million cycles but the steel-prestressed beam failed after 185,000 cycles. However, the CFRP-concrete bond was weakened by high prestress levels, cyclic loading, and low temperature during sustained loading and loading to failure. This resulted in bond failure at loads ranging from 69 to 91% of the full flexural capacity. Stiffness and camber gradually decreased during cyclic loading.
Table 4.1. Details of test beams and applied loadings

<table>
<thead>
<tr>
<th>No.</th>
<th>Beam Type</th>
<th>Prestress Type</th>
<th>Jacking Level (%)</th>
<th>Jacking Force (kN)</th>
<th>Sustained Loading</th>
<th>Cyclic Loading</th>
<th>Loading to Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C50RT</td>
<td>4-8Φ CFRP Tendons</td>
<td>50</td>
<td>208</td>
<td>Room</td>
<td>Room</td>
<td>Room</td>
</tr>
<tr>
<td>2</td>
<td>C50LT</td>
<td></td>
<td>50</td>
<td>208</td>
<td>Low</td>
<td>Low</td>
<td>Low</td>
</tr>
<tr>
<td>3</td>
<td>C70RT</td>
<td></td>
<td>70</td>
<td>291</td>
<td>Room</td>
<td>Room</td>
<td>Room</td>
</tr>
<tr>
<td>4</td>
<td>C70LT</td>
<td></td>
<td>70</td>
<td>291</td>
<td>Low</td>
<td>(No)</td>
<td>Low</td>
</tr>
<tr>
<td>5</td>
<td>C60RT</td>
<td></td>
<td>50/70</td>
<td>250</td>
<td>Room</td>
<td>(No)</td>
<td>Room</td>
</tr>
<tr>
<td></td>
<td>(Control)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>S55RLT</td>
<td>2-13Φ Steel Strands</td>
<td>55</td>
<td>202</td>
<td>Room</td>
<td>Low</td>
<td>(No)</td>
</tr>
<tr>
<td>7</td>
<td>S55LT</td>
<td></td>
<td>55</td>
<td>202</td>
<td>Low</td>
<td>(No)</td>
<td>Low</td>
</tr>
</tbody>
</table>

Table 4.2. Properties of Leadline CFRP prestressing tendons
(Mitsubishi Kasei Corp., 1993)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre</td>
<td>Carbon</td>
</tr>
<tr>
<td>Resin</td>
<td>Epoxy</td>
</tr>
<tr>
<td>Fibre Volume Ratio</td>
<td>0.65</td>
</tr>
<tr>
<td>Nominal Diameter</td>
<td>7.9 mm</td>
</tr>
<tr>
<td>Cross Section</td>
<td>46.1 mm$^2$</td>
</tr>
<tr>
<td>Type</td>
<td>Spiral indentations</td>
</tr>
<tr>
<td>Elastic Modulus (Guaranteed)</td>
<td>147 GPa</td>
</tr>
<tr>
<td>Elastic Modulus (Measured)*</td>
<td>187 GPa</td>
</tr>
<tr>
<td>Ultimate Stress (Guaranteed)</td>
<td>2,250 MPa</td>
</tr>
<tr>
<td>Ultimate Stress (Measured)*</td>
<td>3,200 MPa</td>
</tr>
<tr>
<td>Ultimate Strain (Guaranteed)</td>
<td>1.3 %</td>
</tr>
<tr>
<td>Ultimate Strain (Measured)$†$</td>
<td>1.69%</td>
</tr>
<tr>
<td>Longitudinal Thermal Expansion Coefficient</td>
<td>$-0.9 \times 10^{-6}/^\circ C$</td>
</tr>
<tr>
<td>Transverse Thermal Expansion Coefficient</td>
<td>$27 \times 10^{-6}/^\circ C$</td>
</tr>
<tr>
<td>Density</td>
<td>1.53 g/cm$^3$</td>
</tr>
<tr>
<td>Relaxation Ratio</td>
<td>2 to 3 %</td>
</tr>
</tbody>
</table>

* Abdelrahman and Rizkalla (1997)
† Bryan (1994)
Table 4.3. Timeline of loadings applied to test beams

<table>
<thead>
<tr>
<th>No.</th>
<th>Beam</th>
<th>Fabrication</th>
<th>Sustained Loading</th>
<th>Cyclic Loading</th>
<th>Loading to Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>C70LT</td>
<td>Jul 2008 - Jan 2009</td>
<td>(No)</td>
<td>Jul 2009</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>C60RT</td>
<td>Dec 1997 - Sep 1999</td>
<td>(No)</td>
<td>Jul 2008</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>S55RLT</td>
<td>Dec 1997 - Sep 1999</td>
<td>Oct 2009</td>
<td>(No)</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.4. Key values from load-deflection graphs of CFRP-prestressed beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Ultimate Failure Mode</th>
<th>Initial Slip</th>
<th>Load at ... (kN)</th>
<th>Percentage of Flexural Capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Initial Slip</td>
<td>Ultimate Slip</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Partial Failure)</td>
<td>(Global failure)</td>
</tr>
<tr>
<td>C50RT</td>
<td>Slip of CFRP</td>
<td>No</td>
<td>-</td>
<td>130</td>
</tr>
<tr>
<td>C50LT</td>
<td>Slip of CFRP</td>
<td>Yes</td>
<td>98</td>
<td>130</td>
</tr>
<tr>
<td>C70RT</td>
<td>Slip of CFRP</td>
<td>No</td>
<td>-</td>
<td>123</td>
</tr>
<tr>
<td>C70LT</td>
<td>Slip of CFRP</td>
<td>Yes</td>
<td>104</td>
<td>116</td>
</tr>
<tr>
<td>C60RT</td>
<td>Flexural Compression</td>
<td>No</td>
<td>-</td>
<td>143</td>
</tr>
</tbody>
</table>

(Control)

Table 4.5. Camber of beams exposed to cyclic loading

<table>
<thead>
<tr>
<th>Beam</th>
<th>Camber (mm)</th>
<th>Initial Monotonic Test</th>
<th>At 1M Cycles</th>
<th>At 2M Cycles</th>
<th>At 3M Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Before</td>
<td>After</td>
<td>At 1M Cycles</td>
<td>At 2M Cycles</td>
</tr>
<tr>
<td>C50RT</td>
<td></td>
<td>13</td>
<td>-</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>C50LT</td>
<td></td>
<td>14</td>
<td>-</td>
<td>-</td>
<td>7</td>
</tr>
<tr>
<td>C70RT</td>
<td></td>
<td>18</td>
<td>16</td>
<td>-</td>
<td>10</td>
</tr>
<tr>
<td>S55RLT</td>
<td></td>
<td>16</td>
<td>13</td>
<td>Beam failed</td>
<td></td>
</tr>
</tbody>
</table>

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Figure 4.1. Geometry and reinforcement details of beams (Braimah, 2003)
d) CFRP-prestressed beam

g) Steel-prestressed beam

Figure 4.2. End faces of sample test beams

Figure 4.3. Dislodged concrete particles from repeated grinding at crack interface
Figure 4.4. Curvature in profile of test beam during cyclic loading
Figure 4.5. Load over time in cyclic loading (sample)

Figure 4.6. Stroke over time in cyclic loading (sample)
Figure 4.7. Beam ready for testing inside cold room

Figure 4.8. Cooling system in cold room

a) Fans  b) Control panel: Thaw cycle setter (left), Temperature setter + Thermometer (right)
Figure 4.9. Window in cold room door
Figure 4.10. Cyclic loading setup (units: mm)
Figure 4.11. Ram and spreader beam

Figure 4.12. Supports and reaction beam
a) Transversely: Wooden braces  

b) Longitudinally: Chains  

Figure 4.13. Bracing of test beam during cyclic loading
Figure 4.14: Components of control station

a) Control Station

b) Control panel for hydraulics

c) Control panel for ram

d) Bottom display showing wave settings

e) Top display showing min/max loads and number of cycles

Figure 4.14: Components of control station
Figure 4.15. LPs to measure deflection

Figure 4.16. Strain gauge wires coming out of top flange
a) Side view (units: mm)

b) Cross-section at midspan (CFRP-prestressed beams)

1, 2, 3: Strain Gauges

Figure 4.17: Location of instrumentation on test beams
a) Steel discs glued onto beam ends
b) LPs mounted on magnetic stands connected to steel discs
c) Ruler placed against beam end

Figure 4.18. Measuring slip of prestressing reinforcement at beam ends
a) Front

b) Back: Instruments connect to DA via green cards

Figure 4.19: Data Acquisition unit (DA)

a) DA, computer and control station outside of cold room  b) Wires from instruments and electric units pass through duct in cold room wall and connect to control station.

Figure 4.20. Control station outside cold room
a) Electrical units inside hot box

b) Hot box from outside

c) Holes are filled with insulating material.

d) Thermometer measures temperature inside hot box.

Figure 4.21. Hot box inside cold room
Figure 4.22. Load-deflection behaviour of CFRP-prestressed beams up to failure
End A: Initial state  
End A: After first slip, bottom tendons slipped completely.

End B: Initial state  
End B: After second slip, top tendons slipped completely.

a) Slip of CFRP tendons at both ends of beam

b) Midspan region after second slip:  
Vertical flexural cracks, horizontal slip cracks at elevation of slipped top and bottom tendons

Figure 4.23. Failure mode of C50RT: Slip of CFRP tendons
Initial state
First slip: Tendon 1 has slipped by 6 mm.
Second slip: All four tendons have slipped.

a) Slip of CFRP tendons at one end of beam

b) Midspan region after failure

Figure 4.24. Failure mode of C50LT: Slip of CFRP tendons
Initial state  
After slip: Top tendons slipped.

a) Slip of CFRP tendons at one end of beam

b) Midspan region after failure

Figure 4.25. Failure mode of C70RT: Slip of CFRP tendons
Figure 4.26. Failure mode of C70LT: Slip of CFRP tendons

a) Slip of CFRP tendons at one end of beam

b) Midspan region after failure

Figure 4.27. Failure mode of C60RT: Flexural compression failure
Figure 4.28. Load-slip behaviour of C50RT

Figure 4.29. Load-slip behaviour of C50LT
Figure 4.30. Load-slip behaviour of C70LT

Figure 4.31. Stress profile in CFRP tendons in C50RT and C50LT
Figure 4.32. Load-deflection behaviour of CFRP-prestressed beams up to cracking

Figure 4.33. Load-deflection behaviour of C50RT at different stages of cyclic loading
Figure 4.34. Load-deflection behaviour of C50LT at different stages of cyclic loading

Figure 4.35. Load-deflection behaviour of C70RT at different stages of cyclic loading
Figure 4.36. Pre-cracking stiffness at different stages of cyclic loading

Figure 4.37. Post-cracking stiffness at different stages of cyclic loading
Figure 4.38. Load-strain behaviour of C50RT before cyclic loading

Figure 4.39. Load-strain behaviour of C50LT before cyclic loading
Figure 4.40. Load-strain behaviour of C70RT before cyclic loading

Figure 4.41. Load-strain behaviour of C70LT up to initial slip failure
Figure 4.42. Load-strain behaviour of C60RT up to failure

Figure 4.43. Load-deflection behaviour of S55RLT, S55LT and C50LT
Figure 4.44. Failure mode of S55LT: Flexural compression
Figure 4.45. Failure mode of S55RLT: Fatigue rupture of steel strand
Figure 4.46. Stroke history over the few hours leading up to failure of S55RLT

Figure 4.47. Load history over the few hours leading up to failure of S55RLT
Figure 4.48. Fatigue (left) and tension (right) fracture surfaces of strands
Left: Fatigue fractures; Right: Cup-and-cone tension fractures (Rabbat et al., 1979)
(Photographs courtesy of Construction Technology Laboratories, Inc.)

Figure 4.49. Load-strain behaviour of S55RLT before cyclic loading
Figure 4.50. Suggested S-N curve for straight strands in prestressed concrete members (Collins and Mitchell, 1991)
Chapter 5
Conclusions and Recommendations

5.1 Conclusions

The behaviour of CFRP-prestressed concrete beams was investigated in two studies: (a) under sustained loading at low temperature, and (b) under high-cycle fatigue at low temperature. Seven 13-year-old, 4.4 m long precast concrete T-beams were tested, of which five were prestressed to various levels with CFRP tendons and two with conventional steel strands.

In the first study, three beams were exposed to −27 °C while being subjected to a sustained load of 25% of their flexural capacity for 163 days. The sustained load produced cracking in two beams with lower prestress levels. Results were compared to those obtained from three similar beams subjected to the same sustained load at room temperature. The following conclusions were drawn:

1. No major difference in deflection increase over time was observed under sustained loading at low temperature compared to room temperature. In both cases, the deflection increase over time was relatively small. The increase at low temperature, however, stabilized much earlier than at room temperature.

2. Prestressing strain has a direct relationship with temperature in CFRP-prestressed concrete beams.
3. The strength of the CFRP-prestressed beam that was exposed to sustained loading at low temperature (−27 °C) and then loaded to failure at the same temperature was about 19% lower than that of a similar control beam subjected to the sustained loading and subsequently loaded to failure at room temperature. A premature bond failure caused this reduction in strength. However, the test beam had a slightly higher prestress level than the control beam which likely contributed to the bond failure.

After being subjected to sustained loading, all seven beams were tested in the second study. Only three of the five CFRP-prestressed beams were subjected to cyclic loading, one at −28 °C and two at room temperature, while only one of the two steel-prestressed beams was subjected to cyclic loading, at −28 °C. Cyclic loading consisted of 3 million cycles at a frequency of 0.85 Hz. The load range represented 21 to 42% of the flexural capacity of the CFRP-prestressed beams and 30 to 60% of that of the steel-prestressed beam. Monotonic tests were run every 1 million cycles. Finally, all seven beams were monotonically loaded to failure. The following conclusions were drawn:

1. All CFRP-prestressed beams survived the 3 million loading cycles.

2. CFRP-concrete bond is weakened by high prestress levels, cyclic loading at room or low temperature, and low temperature during sustained loading and loading to failure. This resulted in bond failure at loads ranging from 69 to 91% of the full flexural capacity of the test beams.

3. Both pre-cracking and post-cracking stiffness decreased throughout cyclic loading of CFRP-prestressed beams. For pre-cracking stiffness, this decrease occurred faster initially.
4. In CFRP-prestressed beams under cyclic loading beyond the cracking load, camber decreased after the first cycle, decreased further as cycling progressed and eventually stabilized.

5. The steel-prestressed beam failed in fatigue after only 185,000 loading cycles. Although the beam was loaded at a relatively aggressive load range of 30 to 60% of its flexural capacity, this loading was within its permissible load limits. Comparing this beam to those prestressed by CFRP suggests that CFRP-prestressed concrete beams have superior fatigue performance.

The findings of this experimental study provide new insights into the long-term behaviour of CFRP-prestressed concrete beams at low temperature. It is believed that most of these findings go beyond the present scope of the literature.

5.2 Recommendations

The tests performed in this investigation were generally successful. However, the following changes in the experimental program would improve the accuracy of results and help draw more detailed conclusions:

1. Sustained loading at low temperature:
   a. Using the same concrete mix to cast the beams
   b. Testing all beams at the same age
   c. Making initial load-deflection and prestressing strain readings the very first time the beams are loaded
d. Using a loading setup in which sustained load does not decrease over time and is not affected by changes in temperature, e.g. placing heavy weights on the beams

e. Loading all beams to the same load

f. Exposing beams to low temperature immediately after they are loaded

g. Making more frequent measurements during the sustained loading period, perhaps by automating data recording using a data acquisition unit

h. Installing strain gauges at midspan on both strands in the steel-prestressed beams and on all four tendons in the CFRP-prestressed beams

i. Modelling changes in deflection and prestressing strain over time using a time-dependent analysis procedure

2. High-cycle fatigue at low temperature:

a. Testing more beams to better isolate various parameters, e.g. the effect of temperature during sustained loading, cyclic loading or loading to failure only, or the effect of sustained loading versus no sustained loading

b. Using longer beams with sufficient embedment length to cause rupture of the CFRP tendons rather than bond failure

c. Studying the effect of varying the cyclic load range on the fatigue life of beams

d. Using strain gauges that are rated to survive the 3 million loading cycles

3. General:

a. Monitoring transfer lengths over time as the beams age in the lab as well as during sustained and cyclic loading

b. Monitoring camber over time as beams age in the lab and during sustained loading
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Appendices

Appendix A: Concrete Strengths

Appendix B: Accuracy of Measurements
Appendix A: Concrete Strengths

All beams were more than one year old at the time of sustained loading, cyclic loading and monotonic loading to failure (Table 4.3). For each beam, concrete strength at this time, $f_c'(\text{testing})$, was calculated by using the 28-day compressive strength determined by Braimah et al. (2003), using the provision of ACI (1978) that concrete strength at one year is 1.2 times that at 28 days, and assuming this strength did not change beyond 1 year. Table A.1 lists the results.

Table A.1: Concrete strength of test beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Time (t) (days)</th>
<th>Concrete Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_c(t)$</td>
<td>$f_c'(28 \text{ days})$</td>
</tr>
<tr>
<td>C50RT, C50LT</td>
<td>46</td>
<td>43</td>
</tr>
<tr>
<td>C70RT, C70LT</td>
<td>39</td>
<td>37</td>
</tr>
<tr>
<td>C60RT</td>
<td>34</td>
<td>36</td>
</tr>
<tr>
<td>S55RLT, S55LT</td>
<td>28</td>
<td>36</td>
</tr>
</tbody>
</table>

*: Data courtesy of Braimah (2003)
Appendix B: Accuracy of Measurements

Table B.1 lists the accuracy of the instruments used. The accuracy of measurements was taken to be the same as the accuracy of instruments in all cases except the following:

- Deflection was measured using LPs or dial gauges mounted on steel stands that stood on the cold room floor. In addition to the inaccuracy of the instruments themselves (0.01 mm), other sources of error included movement and rotation of the instruments during tests, thermal expansion or contraction of the steel stands, vibration of the cold room floor, etc. Thus, the overall accuracy was estimated at 0.1 mm.

- Camber was found by measuring the perpendicular distance between the beam’s top flange and the cold room floor using measuring tape. Therefore, in addition to the accuracy of the measuring tape (1 mm), other sources of error included the flatness of the cold room floor and the angle between the measuring tape and the floor. Hence, overall accuracy was estimated at 2 mm.

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Instrument</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sustained Load</td>
<td>Load Cells (Chapter 3)</td>
<td>0.6 kN</td>
</tr>
<tr>
<td>Load</td>
<td>Load Cells (Chapter 4)</td>
<td>0.1 kN</td>
</tr>
<tr>
<td>Deflection</td>
<td>Linear Potentiometers (LPs)</td>
<td>0.002 mm</td>
</tr>
<tr>
<td></td>
<td>Dial Gauges</td>
<td>0.01 mm</td>
</tr>
<tr>
<td>Prestressing Strain</td>
<td>Strain Gauges</td>
<td>10 με (assumed)</td>
</tr>
<tr>
<td>Slip</td>
<td>Ruler</td>
<td>1 mm</td>
</tr>
<tr>
<td>Camber</td>
<td>Measuring Tape</td>
<td>1 mm</td>
</tr>
<tr>
<td>Temperature</td>
<td>Thermometer</td>
<td>1 °C</td>
</tr>
<tr>
<td>Relative Humidity</td>
<td>Humidity Meter</td>
<td>3.5% at 25 °C</td>
</tr>
</tbody>
</table>