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EFFECT OF LOW TEMPERATURE ON THE SHEAR-FATIGUE PERFORMANCE OF REINFORCED CONCRETE BEAMS

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ABSTRACT

This paper investigates the fatigue behaviour of a reinforced concrete beam (without shear reinforcement) at low temperature (-20°C) compared to a similar beam tested near room temperature ($+16^{\circ}\text{C}$). Two large-scale steel reinforced beams ($200\text{ mm} \times 400\text{ mm} \times 4200\text{ mm}$) were fabricated and tested. The beams had temperature differentials over their depth to simulate solar radiation and in-service temperature of the bridges. The beams were cyclically loaded to failure with a stress range representing the ratio of live to dead loads found in most bridges. This study showed that low temperature increased the fatigue life of the reinforced concrete beam by 51%, and changed the mode of failure of the beam from shear fatigue failure at room temperature to flexural fatigue failure at low temperature. It was observed that the low and room temperature beams maintained 65% and 31% of their original flexural rigidity indicating that low temperature mitigated the stiffness degradation of the reinforced concrete caused by fatigue loading. The strains in the tensile reinforcement of the low temperature beam were much lower than the room temperature beam which could be due to the higher strength of concrete and reduction in concrete softening at low temperature which resulted in lower stresses in the tensile reinforcement. The findings of this study show that the contribution of concrete under either static or cyclic load becomes much higher at low temperature.

1. INTRODUCTION

In Canada, concrete bridges are the primary links of the country's surface transportation. For instance, over 55 million cars and 10 million trucks crossed the Canada-U.S. border in 2011 (Transport Canada 2012), and a large number of these vehicles passed over bridges to reach their destination. However, a significant percentage of the bridges in Canada are reaching the end of their service life. According to Statistics Canada, nation's bridges, on average, have passed almost 60 percent of their useful life as they were built in the 1960s and 70s (Cusson 2008).

The concrete bridges are degraded due to overloading and fatigue caused by increased legal load limits, corrosion, impact damage, and environmental conditions, i.e. large temperature fluctuations and freeze-thaw cycling (Kim and Heffernan 2008). For instance, Viaduc de la Concorde overpass collapsed in 2006 in Laval, Quebec. The collapse, which was due to shear failure occurred as result of concrete cracking in a zone of weakness. A study conducted by Mitchel et al. (2011) suggests that the cracking could have been initiated by shrinkage of the concrete, thermal stresses as well as vehicle impact on the expansion joint, and then the inclined cracking provided a natural path for the entry of water from the expansion joint resulting in freeze-thaw deterioration in the presence of deicing salts.

Furthermore, the development of high strength and more durable materials allows the structural concrete members to withstand higher static loads for a longer period of time; therefore, a satisfactory fatigue performance of those structural members that experience cyclic loading during their service life is essential and their fatigue strength must be accounted for at the design stage.

For the reasons outlined above, interest in the fatigue behaviour of reinforced concrete members has increased considerably in recent years. Nevertheless, the fatigue behaviour of reinforced concrete members at low temperature is still unexplored in spite of the high demand for repairing decaying infrastructure in cold regions, as well as a lack of understanding of the fatigue behaviour of reinforced concrete under severe climatic effects (Heffernan et al. 2004).

2. BACKGROUND

2.1 Fatigue Behaviour of Plain Concrete

Cyclic loading causes concrete to fail at stresses below its ultimate static strength, and the modulus of elasticity of concrete to significantly decrease due to the formation and interaction of the differently oriented microcracks. The fatigue fracture of concrete is characterized by considerably larger strains and microcracking than its fracture under static loading. (ACI Committee 215 1992, Neville 2006, Schlafli and Bruhwiler 1998).

2.2 Fatigue Behaviour of Reinforcing Steel at Room and Low Temperature

Failure of steel reinforcement under cyclic loading has three phases: crack initiation, crack propagation and unstable crack growth until brittle fracture. The failure is initiated at a micro-crack at the largest stress concentration site on the bar surface, usually at the intersection of transverse lugs and longitudinal ribs. As the stress continues to cycle, the crack gradually propagates until the crack reaches a critical length at which its propagation becomes unstable and sudden fracture occurs (Schlafli and Bruhwiler 1998, Papakonstantinou et al. 2001). No S-N relationship has been prescribed for reinforcing steel by ACI or AASHTO standards; however, a significant amount of research has investigated the fatigue behaviour of reinforcing bars to develop empirical equations that describe their S-N relationship, which are in the general form of $N\sigma_r^m=K$ (Helgason and Hanson 1974, Moss 1982, Soltani et al. 2011), where N is the number of cycles to failure, σ_r is the stress range, and m is the inverse slope of the log σ_r -log N curve and K is a constant.

Previous studies have shown that the fatigue performance of steel improves at low temperatures (Sines and Dolan 1959, Troshchenko 1971, Frost et al. 1974, Stephens et al. 1984). Stephens et al. (1984) studied constant and variable amplitude fatigue behaviour of five cast steels with carbon weight percentages of 0.23, 0.24, 0.30, 0.34 and 0.49, at room temperature and -45 °C, which falls in either the lower shelf impact CVN energy region or lower transition region of the steels. This study showed that at low temperature the fatigue limits of the steels at fatigue lives longer than 10^5 reversals increased by 24%, fatigue crack initiation was delayed by up to 2.5 times, and constant amplitude fatigue crack growth rates decreased by a factor ranging from 1.2 to 3 which resulted in longer fatigue lives of the steel at low temperature.

2.3 Fatigue Behaviour of Reinforced Concrete

There are a limited number of recent studies focusing on the fatigue behaviour of conventionally reinforced concrete beams. Some recent studies have focused on the fatigue behaviour of reinforced concrete beams strengthened with FRP sheets. For that reason, the fatigue properties of the beams reinforced with glass fibre reinforced polymer (GFRP) are included in this section. The fatigue characteristics of a composite member such as reinforced concrete not only depends on the fatigue properties of its constituent parts, but also depends on the composite action between its parts i.e. the bond between steel and concrete (Papakonstantinou et al. 2001, Heffernan et al. 2004). Papakonstantinou et al. (2001) performed fatigue tests on fourteen simply supported reinforced concrete beams (1321 mm x 152 mm x 152 mm) with and without GFRP sheets on their tensile surfaces. All the beams had transverse reinforcement to avoid shear failure. The study showed that although the fibre strengthening system increased the fatigue life of the beams, it did not change the fatigue failure mechanism, and both strengthened and non-strengthened beams failed due to the fatigue failure of the tensile reinforcement. Heffernan and Erki (2004) studied the fatigue performance of twelve three-meter beams with and without CFRP sheets at three different stress ranges, as well as three five-meter beams. The study showed that the fatigue lives of the CFRP strengthened beams were significantly increased when compared to the reinforced concrete beams without CFRP sheets due to lower stresses in the tensile reinforcement, and both strengthened and unstrengthened beams failed primarily due to the brittle fracture of the tensile reinforcement; however, the beams with CFRP sheets did not necessarily become unstable at the fracture of one of the rebars and continued to support the load cycles.

2.4 Fatigue Behaviour of Reinforced Concrete at Low Temperature

To the best of the authors' knowledge, the effect of low temperature on the fatigue behaviour and life of conventionally reinforced concrete is still unexplored. However, there is only one study that has investigated the high-cycle fatigue behaviour of CFRP-prestressed concrete T-beams at low temperature (Saeidi et al., 2010).

This paper presents an investigation into the fatigue life and behaviour of large-scale conventionally reinforced concrete beams without stirrups. The results of the fatigue tests on two reinforced concrete beams (4200mm x 400mm x 200mm) with temperature differentials at average ambient temperatures of +16 °C and -20 °C are discussed. This study is essential to understand the fatigue behaviour of large-scale reinforced concrete members, and is of particular importance to cold regions with prolonged freezing seasons.

3. EXPERIMENTAL PROGRAM

3.1 Material Tests

To determine the compressive and tensile strength of the concrete, concrete cylinders were cast together with the beams. Three compressive and three splitting tensile tests were conducted on concrete cylinders (150 mm × 300 mm) with at least 28-day age in accordance with (ASTM) C39M-12a and C496M-11, respectively. The mean compressive and splitting tensile strengths of the concrete from the cylinder tests were respectively 40.0 MPa and 4.1 MPa. Six tensile tests, in accordance with ASTM A370-12a and A615M-12, were performed on the samples of hot rolled 10M and 20M bars, from the same batch of 400 Grade steel (HR G30.18 400W) that were used in the beams as compression and tension reinforcement. The uniaxial tensile tests on the 10M and 20M bars resulted in mean yield and mean ultimate strengths of 481 MPa and 669 MPa, and 421 MPa and 531 MPa, respectively.

3.2 Beam Tests

Two reinforced concrete beams 200 mm × 400 mm × 4200 mm, were constructed to be fatigue tested at room and low temperature. The room and low temperature beams are respectively named "RT beam" and "LT beam" in this paper. Prior to casting the beams, strain gauges and type T thermocouples were attached to the tension reinforcement. Figure 1 shows the details of the reinforcement and internal instrumentation as well as test configuration of the beams.

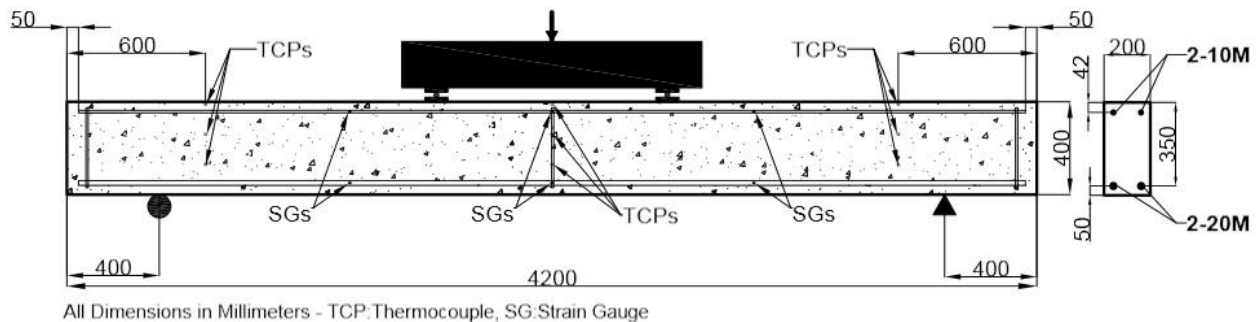


Figure 1: Internal reinforcement and test configuration (left) and cross-section of the beams (right)

To simulate solar radiation, heating pads and insulation were placed on top of the beams prior to the start of each test to warm up the beams and create a temperature differential over the depth of the beams during the tests. The average temperature, read by the thermocouples, over the cross-section, at top, middle and bottom of the beams during the fatigue tests were +50 °C, +30 °C and +20 °C (for the room temperature beams), and +10 °C, -10 °C and -20 °C (for the low temperature beams). These thermal gradients were induced over the depth of the beams in accordance with Cl.3.9.4.4 of the Canadian Highway Bridge Design Code (CSA 2006) requirements for temperature effects on superstructures. A positive temperature differential decreasing linearly by 30 °C from the top to the bottom of the superstructure of type B, i.e. deck truss systems with concrete decks; should be considered. However, the present thermal gradient is higher than the recommended thermal gradient for type A and C superstructures.

The average ambient temperature during testing of the RT and LT beam was +16 °C and -20 °C, respectively. The test room was equipped with a heating-cooling system which allowed the room temperature to be adjusted with a temperature range of -30 °C to +30 °C.

The beams were tested using a 500 kN servo-hydraulic actuator capable of applying cyclic and monotonic loading in a temperature controlled room. All of the beams (simply supported) were tested under four-point bending with a span of 3.4 m and a constant moment region of 1.0 m as shown in Figure 1. The beams were loaded by applying a sinusoidal loading pattern at a frequency of 1.0 Hz. This frequency was chosen since previous research showed that beams under cyclic loading at frequencies higher than 2.0 Hz were unable to recover fully from one load application before the arrival of the next (Emberson and Mays 1996, Barnes and Mays 1999).

Stress limits in the flexural reinforcement were chosen to give a mean stress and stress range that would result in fatigue failure of the tensile steel around 10^6 cycles. The stress limits in the tensile reinforcement required for a fatigue life of one million cycles were calculated using the Helgason equation (Helgason and Hanson 1974) and verified by performing axial fatigue tests on the 10 M rebars that remained from the same batch of steel used for compressive reinforcement. It was determined that a stress range equal to 45% of the nominal yield strength (189 MPa) of the tensile steel would give a fatigue life of approximately one million cycles. From moment-curvature calculations, the loads corresponding to the minimum and maximum stresses, 95 MPa ($0.22 f_y$) and 284 MPa ($0.67 f_y$) in the flexural reinforcement, were found to be 30 kN ($0.25 P_u$) and 90 kN ($0.75 P_u$), respectively. This ratio of loading is similar to the ratio of live to dead loads found in most bridges (Heffernan and Erki, 2004).

In order to determine changes in stiffness, deflection and strains with the number of cycles during the fatigue tests, cyclic loading was stopped periodically, approximately at every 250,000 cycles, and static tests were carried out from zero to 90 kN at 10 kN increments. An initial static test, prior to the fatigue testing of the beams, and a final static test after fatigue failure of the beams were performed on the beams as well. These two static tests are named “initial static test” and “post-failure static test” in this paper.

Load values were measured by the built-in load cell of the electric and hydraulic rams, and deflection values were measured by linear potentiometers (LP) placed at midspan. Four additional LPs were placed in the middle of the shear spans on opposite sides of the beams to measure possible rotation of the beams.

4. EXPERIMENTAL RESULTS

4.1 Fatigue Life

Comparison of the fatigue lives of the RT and LT beams, given in Table 1, shows that the fatigue life of the LT beam was 51% higher than the RT beam due to low temperature.

Table 1: Fatigue lives of the beams

Beam ID	Cycles to Failure	Failure Mode
RT	448,581	Shear cracking and concrete spalling
LT	678,134	Rupture of tensile steel

These fatigue tests also showed that the fatigue life of the reinforced concrete beam at room temperature primarily depends on the concrete and steel-concrete bond since fatigue loading of RT beam caused a) a large shear crack to develop which prevented any shear stress transfer and b) progressive deterioration of the bond between reinforcement and concrete which led to spalling of the bottom concrete cover on the tension side as shown in Figure 2. A similar mode of failure for beams without shear reinforcement have been reported by Schlaflfi and Bruhwiler, 1998. The fatigue testing of the LT beam showed that low temperature increases the shear capacity of the concrete and prevents development of any critical shear crack which results in significant improvement in the fatigue life of the reinforced concrete beam. Thus, the fatigue life of this beam depends on the stresses in the tensile reinforcement and fatigue fracture of the steel reinforcement.

4.2 Failure Mode

LT beam failed in a tension flexural mode due to the fatigue rupture of one of the tensile reinforcing bars near the midspan. However, RT beam primarily failed because of the development of a large shear crack in one of its shear spans (shear fatigue failure). The post-failure images of the beams are shown in Figure 2. The large shear crack on RT beam was first observed around 261,000 cycles and the concrete cover on the tension side of this beam in the flexural span spalled around 390,000 cycles, approximately 58,000 cycles prior to its final fatigue failure at 449,000 cycles. The tensile bars of this beam became bent shortly prior to the final fatigue failure. The fatigue failure of the tensile reinforcement on LT beam started at one of the flexural cracks that had been developed during the initial static test near the midspan of the beam. This could be attributed to high stresses and formation of stress concentration sites in the bar at the crack location. The contribution of concrete softening to the stress history of the steel at crack locations and formation of stress concentration sites at crack locations have been reported in previous studies (Papakonstantinou et al. 2001, Heffernan et al. 2004).



Figure 2: The tested beams after fatigue failure – shear failure at room temperature (left) and flexural failure by reinforcement rupture at low temperature (right)

4.3 Stiffness Degradation

Figure 3 shows the load-deflection curves of the RT and LT beams that were obtained from the periodic static tests. The load-deflection diagram of the RT beam indicates that the stiffness (slope of load-deflection response) of this beam remained unchanged up to approximately 261,000 cycles; however, the increase in the number of cycles from 261,000 cycles up to failure at 449,000 cycles caused the beam to lose almost all of its flexural rigidity. The post-failure static test showed that the stiffness of the beam decreased by 69% after fatigue failure. It should be noted that the highest load carried by this beam during the post-failure static test was around 20 kN, and the permanent residual deflection for this beam was around 5 mm at 261,000 cycles, and continued to increase to 16 mm until the fatigue failure of the beam.

LT beam, tested at low temperature, demonstrated a different behaviour: this beam maintained 89% of its flexural rigidity up to a half million cycles. Even after fatigue failure, at 678,000 cycles, the reduction in the stiffness of this beam was not as much as the RT beam, and the post-failure static test showed that its flexural rigidity reduced by 35% and 27% when compared to its initial stiffness and at 500,000 cycles, respectively. This behaviour of the reinforced concrete in maintaining its stiffness at low temperature can be attributed to the higher initial stiffness of the LT relative to the RT beam, which was around 40% higher, and better shear performance of reinforced concrete at low temperature.

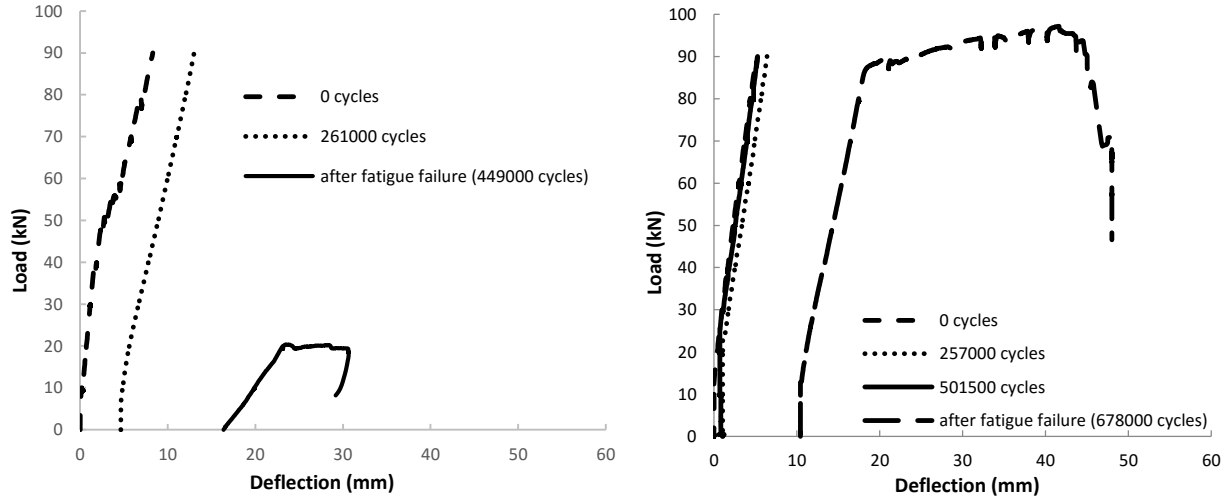


Figure 3: Load-deflection curves of RT beam (left) and LT beam (right) from the periodic static tests

4.4 Cyclic Load-Deflection Behaviour

Figure 4 shows the changes in the maximum mid-span deflection of the beams at the upper limit of the cyclic loading (90 kN) with number of cycles. The behaviour of the RT and LT beams were similar, in that they initially demonstrated a small gradual increase in deflection, after which the deflection remained stable with number of cycles, followed by a sudden increase in deflection prior to failure. During the entire test up to failure, the low temperature beam had a much lower deflection than its room temperature counterpart. The deflection of the LT beam at the maximum load was around half that of the RT beam up to failure.

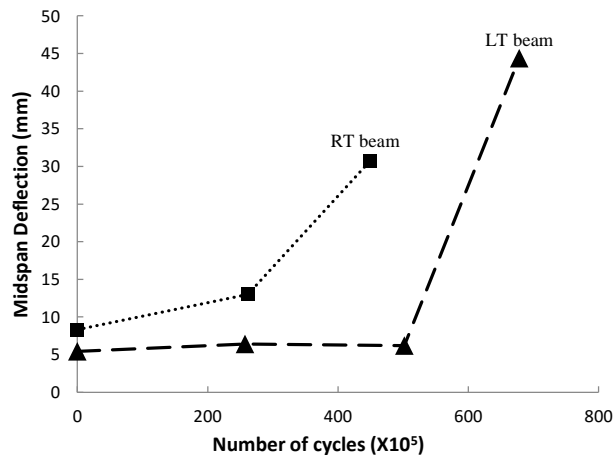


Figure 4: Maximum midspan deflection vs. number of cycles

The highest deflections of the RT and LT beams at failure were 31 mm and 44 mm, respectively. Although relatively large deflections were observed with the RT beam, this beam had lost almost all of its load capacity prior to the post-failure static test. Nevertheless, comparing the behaviour of the RT with the LT beam during the post-failure static test shows that even after fatigue failure the low temperature beam had sufficient capacity to carry the static load up to 97 kN while the room temperature beam actually was not able to support any load greater than 20 kN.

4.5 Strain in the Tensile Reinforcement

Figure 5 shows the load-strain curves that were obtained from the periodic static tests on the beams. The initial (pre-fatigue) static tests showed that steel strain in the midspan of the RT beam increased at a higher rate than the LT beam, and at 90 kN (the highest load applied on the beams during the static tests) the steel strain in the LT beam was 29% lower than the RT beam. This is indicative of the higher number of cracks in the RT beam compared to the LT beam. The number of the cracks that were observed after the initial static test in the shear and flexural spans of these two beams are given in Table 2.

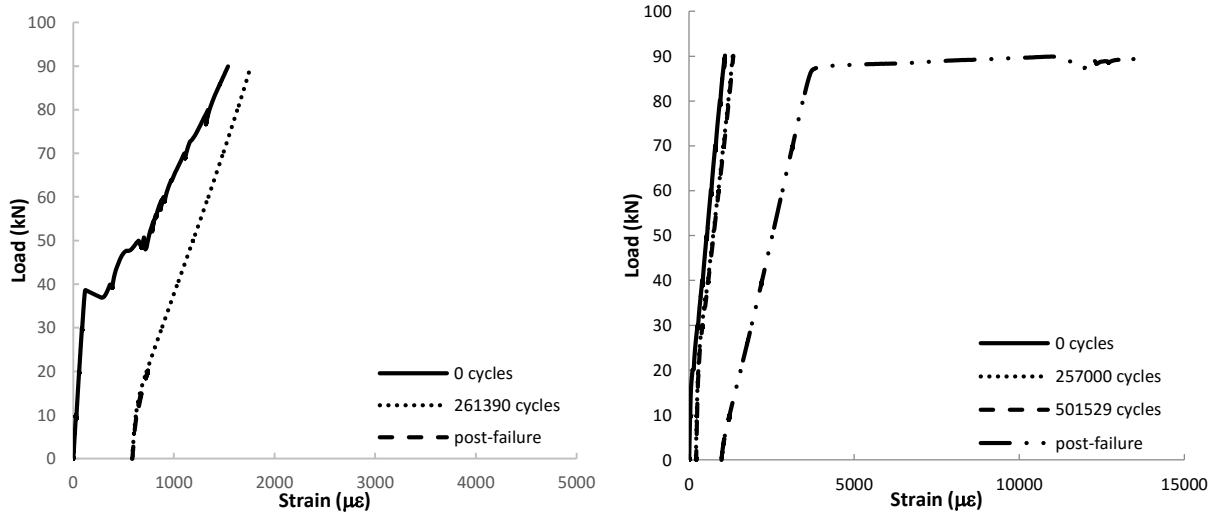


Figure 5: Load vs. strain in the tensile reinforcement (midspan) of RT (left) and LT (right) beam

Table 2: No. of cracks during the initial static test

Beam ID	Flexural Span	Shear Spans
RT	8	5
LT	6	4

The static tests performed on the RT and LT beams at approximately 260,000 cycles showed that the cyclic loading increased the residual steel strain in both RT and LT beams; however, steel strains in the tensile reinforcement of the LT beam at 30 kN and 90 kN were respectively 54% and 25% lower than the RT beam. The steel strain in the midspan of the LT beam remained almost unchanged up to 500,000 cycles; however, the post-failure static test on this beam showed that the residual steel strain following the fatigue test increased by more than three times and the elastic strain increased at a much higher rate when compared to the previous static tests as can be seen in Figure 5.

During the post-failure static tests on the LT beam, the tensile reinforcement of this beam started yielding around 87 kN and yielding continued until the beam reached its full flexural capacity and failed due to the crushing of the concrete in the compression zone. This indicates that the unruptured tensile reinforcement of the low temperature beam remained functional even after fatigue failure and allowed the beam to sustain static loads up to 75% of its ultimate flexural resistance for a relatively long time until ductile failure occurred, while the tensile reinforcement of the room temperature beam had no function after fatigue failure and the RT beam was actually not able to sustain any load; the highest load that the RT beam was able to resist after fatigue failure was 20 kN.

5. DISCUSSION

The room temperature beam experienced shear fatigue failure followed by spalling of the concrete on its tension side while the low temperature beam failed due to rupture of one of its tensile reinforcement (flexural fatigue failure). This indicates that low temperature changes the fatigue failure mode of the reinforced concrete beam without shear reinforcement, and at room temperature, the fatigue life of the reinforced concrete beam primarily depends on the concrete strength (shear capacity) and steel-concrete bond while at low temperature, the fatigue life of the beam largely depends on the stresses in the tensile reinforcement, and fatigue fracture of the steel reinforcement is the dominant factor governing failure in this beam. Flexural fatigue failure and the dominant role of stress history in tensile reinforcement have been reported in previous studies (Barnes and Mays 1999, Papakonstantinou et al. 2001, Heffernan et al, 2004).

The increase of fatigue life at low temperature can be largely attributed to the higher strength and stiffness of the concrete and lower number of cracks with smaller widths that resulted in reduced stresses in the reinforcement. The higher compressive and tensile strengths of concrete at low temperatures, i.e. 15% to 20% increase in compressive and tensile strength at -20 °C comparing to room temperature, and the lower compressive and tensile strengths of the concrete at high temperatures, approximately 15% reduction in the strengths at +50 °C comparing to room temperature, have been shown by Shoukry et al. (2011).

Nevertheless, the positive effect of low temperature on the fatigue performance of steel should not be ignored. Previous research showed that low temperature improves the fatigue properties and the fatigue strength of steel, and the improvement was attributed to the delayed fatigue crack initiation and lower propagation in steels at low temperature (Sines and Dolan 1959, Troshchenko 1971, Frost et al. 1974, Stephens et al. 1984). Therefore, the higher strength of concrete at low temperatures, and the positive effect of low temperature on the fatigue properties of steel are the two factors that improve the fatigue behaviour of reinforced concrete.

The fatigue failure of the tensile reinforcement on the low temperature beam started at a crack location near the midspan of the beam which could be due to the formation of stress concentration sites on the flexural reinforcement at that crack location. As the concrete in a reinforced concrete beam cracks in the tension zone of the beam, high stresses in the bar at crack locations will be developed and stress concentration sites form. These stress concentration sites as well as the redistribution of stresses due to concrete softening contributes to the stress history of the steel at these locations (Papakonstantinou et al. 2001, Heffernan et al. 2004).

During the fatigue tests, the deflection of the low temperature beam was around half of its room temperature counterpart indicating the better serviceability performance of reinforced concrete without shear reinforcement at low temperature. The post-failure static tests showed that the room and low temperature beams respectively maintained 31% and 65% of their initial (pre-fatigue) flexural rigidity after fatigue failure. This implies that low temperature plays a key role in maintaining the stiffness of the reinforced concrete beams without shear reinforcement. This behaviour of the reinforced concrete in maintaining its stiffness at low temperature can be attributed to the higher initial stiffness of the low temperature beam relative to the room temperature beam, which was around 40% higher, and better shear performance of reinforced concrete at low temperature. The higher modulus of elasticity of concrete at lower temperatures as well as higher effective moment of inertia due to a smaller number of cracks are the other factors that could potentially contribute to the improvement in stiffness.

The strains in the tensile reinforcement of the low temperature beam were lower than its room temperature counterpart during the initial static test, and lower residual strains due to cyclic loading were induced in the tensile reinforcement of low temperature beam. This behaviour of concrete under static and cyclic loading is indicative of higher strength of concrete and lower concrete softening at low temperature. The lower number of cracks in the low temperature beam following the initial static test also implies the higher strength of concrete at low temperature.

6. CONCLUDING REMARKS

This study is based on a limited number of test specimens and thus more research is recommended to confirm the general applicability of the findings. The following preliminary conclusions are that:

1. low temperature may change the mode of fatigue failure of reinforced concrete from shear fatigue failure to flexural fatigue failure since freezing of porewater increases the strength and fatigue resistance of the concrete,
2. low temperature can mitigate stiffness degradation in reinforced concrete beams exposed to fatigue and low temperature can play a key role in maintaining the stiffness and capacity of the reinforced concrete beams that are not transversely reinforced, and
3. the fatigue failure of reinforced concrete is usually sudden with no warning (brittle failure).

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