Low and High Temperature Performance of Near Surface Mounted FRP Strengthened Concrete Slabs

by

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ABSTRACT

Near surface mounted (NSM) FRP reinforcement has recently emerged as a promising alternative technology for strengthening concrete structures in both flexure and shear, as opposed to externally bonded FRP strengthening systems. Available research to date has focused primarily on overall member behaviour and/or the various parameters that affect the bond performance of either rectangular NSM strips or round NSM bars. No research has apparently focused on the effect of low or high temperature exposure on NSM FRP performance. It has been suggested by numerous researchers that NSM FRP reinforcement may outperform externally bonded FRP strengthening systems at elevated temperatures, but this assertion has yet to be supported by test results. An extensive review of NSM FRP technology is presented. The results of an experimental program conducted on twenty-three (23) concrete NSM FRP strengthened slab strips are presented to investigate their high (up to 200°C) and low (-26°C) temperature flexural performance. The effect of using one of two different adhesive systems (epoxy and cement-based) and two different NSM groove widths (6.4 mm and 3.2 mm) is also studied. An innovative photo imaging instrumentation technique is validated against traditional instrumentation techniques for the first time in NSM flexural testing. A numerical layer model is presented and compared against test results. It is demonstrated that low temperature exposure has no measurable negative effects on the flexural performance of the slab strips tested. From high temperature exposures, it is shown that the cementitious adhesive outperforms the epoxy adhesive system, allowing the strengthening system to remain structurally effective for more than 5 hours at 100°C under sustained loads.
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NOTATION

\( A_g \)  
gross area of concrete

\( A_{frp} \)  
area of FRP strip

\( A_p \)  
area of FRP plate

\( A_{Tfrp} \)  
transformed area of FRP

\( A_{Ts} \)  
transformed area of steel

\( A_s \)  
area of reinforcing steel

\( a_1 \)  
factor defined in Eq 2.5

\( a \)  
depth of idealized stress block

\( a_b \)  
width of NSM strip

\( b \)  
factor defined in Eq 2.5

\( b_a \)  
perimeter of groove

\( b_b \)  
height of NSM strip

\( b_{barperim} \)  
perimeter of FRP bar

\( b_f \)  
width of NSM groove

\( b_{notchperim} \)  
effective perimeter of notch according to Eq A.6 and Eq A.7

\( b_p \)  
dimension of the FRP plate parallel to the concrete surface

\( C \)  
adhesive cover according to Eq 2.8 and Eq 2.9

\( C \)  
compressive force

\( c_1 \)  
factor defined in Eq 2.5

\( c \)  
depth to neutral axis of concrete section

\( d \)  
diameter

\( d_b \)  
diameter of FRP bar
$d_f$  depth of the NSM groove  
$d_{frp}$  depth to the neutral axis of FRP  
$d_p$  dimension of the FRP plate perpendicular to the concrete face  
$d_s$  depth to neutral axis of reinforcing steel  
$d_v$  shear depth  
$E_f$  elastic modulus of FRP  
$E_p$  elastic modulus of FRP plate  
$E_s$  elastic modulus of reinforcing steel  
$f_a$  tensile strength of adhesive  
$f_{ct}$  tensile strength of concrete  
$f_c$  concrete compressive stress  
$f_c'$  concrete compressive strength  
$f_{c,\text{low}}$  concrete compressive strength at low temperature  
$f_{c,\text{room}}$  concrete compressive strength at room temperature  
$f_{fd}$  design stress of NSM reinforcement according to Eq A.1 and Eq A.2  
$f_{frp}$  stress in FRP  
$f_{FRP}$  maximum tensile stress in NSM FRP bar at onset of debonding  
$f_g$  compressive strength of grout  
$f_s$  stress in steel  
$f_{ult}$  ultimate strength of FRP  
$f_y$  yield stress of steel  
$G_a$  shear modulus of the adhesive  
$G_1, G_2, G_2'$  coefficients defined in Eq 2.8 and Eq 2.9
\( h \)  
height of section

\( I_{cs} \)  
second moment of area of strengthened concrete equivalent cracked section according to Eq A.3

\( I_{eff} \)  
effective moment of inertia of the transformed section

\( L_d \)  
development length

\( L_{per} \)  
length of the failure perimeter

\( L' \)  
total span of simply supported beam in Eq 2.5

\( l_{db} \)  
development length according to Eq A.1 and Eq A.2

\( l_{nsm} \)  
anchorage length provided for NSM

\( l_{nsm,\text{max}} \)  
anchorage length required to generate \( T_{nsm,\text{max}} \) according to Eq A.7

\( l_o \)  
unbonded length of FRP strip

\( n \)  
modular ratio

\( n_{frp} \)  
modular ratio of FRP to concrete

\( n_s \)  
modular ratio of steel to concrete

\( P \)  
applied concentrated load

\( P_{IC} \)  
maximum load in NSM strip before debonding

\( q \)  
applied uniform load

\( T \)  
tensile force

\( T_{frp} \)  
tensile force in FRP

\( T_g \)  
glass transition temperature

\( T_{nsm} \)  
characteristic anchorage force for NSM

\( T_{nsm,\text{ad}} \)  
characteristic adhesive bond failure force according to Eq A.4 and Eq A.5

\( T_{nsm,\text{max}} \)  
maximum NSM anchorage force according to Eq A.6 and Eq A.7
$T_s$  steel tensile force
$t$  thickness of failure plane in concrete
$t_a$  thickness of the adhesive layer
$t_f$  thickness of the FRP strip
$V$  difference between ultimate shear force and applied shear force at the time of FRP installation according to Eq A.3
$V_c$  shear resistance provided by concrete
$w$  width of groove according to Eq 2.8 and Eq 2.9
$x$  longitudinal coordinate according to Eq 2.4
$y$  depth to neutral axis
$y_{eff}$  distance from the centroid of the FRP strip to the neutral axis of the member
$\alpha$  factored defined in Eq 2.3
$\alpha_p$  factored defined in Eq 2.2
$\beta$  bond length factored defined in Eq 2.3
$\beta_1$  ratio of average stress in rectangular compression block to the specified concrete strength
$\varepsilon$  strain
$\varepsilon_c$  strain in concrete
$\varepsilon_{IC}$  debonding strain according to Eq 2.2
$\varepsilon_s$  strain in steel
$\varepsilon_{sy}$  yield strain of steel
$\varepsilon_{ult}$  ultimate strain
$\varepsilon_{ult}$  ultimate strain of FRP strip

$\mu$  coefficient of friction

$\sigma$  stress

$\tau$  interfacial shear stress

$\tau_{max}$  average bond strength

$\phi_f$  aspect ratio of the interface bond failure plane

$\omega$  factor defined in Eq 2.4

$\forall$  short-term modular ratio of FRP to concrete according to Eq A.3
1.1 General

Infrastructure throughout the industrialized world is showing significant and worrying signs of increasing deterioration. Numerous reports, such as the 2005 ASCE Report Card on America’s Infrastructure (ASCE 2005), paint a bleak picture of the condition of the trillions of dollars worth of existing infrastructure that literally forms the foundation of North America’s high standard of living. The Report gives an overall D grade and estimates a need for US$1.6 trillion of investment to bring infrastructure conditions to acceptable levels in the United States alone. Looking specifically at American bridges, the report states that 27.1% of the 162 000 existing bridges were structurally deficient or functionally obsolete at the time of the survey, and that an estimated investment of $9.4 billion per year would be required for the next 20 years to eliminate all current deficiencies. Clearly, with these huge infrastructure deficits, novel approaches for the design, construction and repair of infrastructure must be developed.

The use of fibre reinforced polymers (FRPs) in civil engineering applications has emerged over the past 15 years, and FRPs are now providing a number of novel approaches for both new construction, and particularly for repair and strengthening of existing structures. FRPs are composite materials that consist of high strength fibres embedded in a polymer matrix. The high strength fibres carry loads while the matrix
transfers these loads to the fibres and also protects them from environmental and mechanical damage. Fibres are typically made of carbon, glass or aramid, while commonly used polymer matrices include epoxy, vinylesters and polyesters (ISIS 2003). Some of the advantages of FRPs in infrastructure applications include their high strength to weight ratios, resistance to environmental and electrochemical attack, non magnetic properties, and ease and speed of installation. Rapid installation is particularly attractive in repair applications, and strengthening of reinforced concrete structures by externally-bonded FRP systems is now a method of choice in the concrete repair industry.

Flexural reinforcement and/or strengthening of concrete structures are two areas where FRPs have gained widespread acceptance. FRP bars have been used for internal flexural reinforcement in new concrete construction applications, replacing the use of steel reinforcing bars; this application makes use of FRPs’ non corrosive properties to eliminate many of the reinforcing bar corrosion problems invariably experienced with conventional steel reinforced concrete structures. External strengthening of reinforced concrete beams, slabs, walls and columns using bonded FRP sheets, strips or jackets has also become a popular technique, and is now a method of choice in many strengthening applications.

Existing concrete beams can be strengthened by bonding (typically unidirectional) FRP sheets to the beams’ faces to improve both flexural and shear performance, using either a wet lay-up technique or by bonding preformed pultruded FRP strips and plates with an epoxy adhesive. Despite the popularity of these externally-bonded FRP strengthening
techniques, they have a number of important limitations in practice. The main limitation is that the bond between the concrete and the FRP, which is critical for adequate performance in most cases, is often unable to develop the full tensile strength of the FRP sheet, resulting in “premature” debonding failures. The result is that design procedures for these systems often impose strain limits that can make use of FRPs uneconomical. Also, because the FRP strengthening system is located on the external face of the member in these applications, the FRP and epoxy adhesive are exposed to environmental effects, fire, and possibly vandalism.

The aforementioned drawbacks of externally-bonded FRP strengthening systems have led to the development of the near surface mounted (NSM) FRP strengthening technique. NSM strengthening is a technique in which grooves (slots) are cut into the surface of structural members, and FRP reinforcement is bonded into the grooves with an adhesive (typically an epoxy resin). This method is often able to utilize a greater proportion of the full strength of the bonded FRP because of superior bond characteristics, which help to prevent premature debonding failures. Furthermore, because the FRP strengthening system is located slightly within the member it is somewhat protected from the damage of the environment, fire and vandalism.

1.2 Objectives

With the recent emergence of NSM FRP strengthening techniques for reinforced concrete members, a large research effort has focused on both member performance for both shear and flexural strengthening applications, as well as bond specific performance. Very little
research has been reported on durability related issues with respect to NSM FRP strengthening techniques for concrete. In particular, no research apparently exists on the performance of NSM FRP strengthening systems at elevated temperatures and only one rather limited study is available on the low temperature performance of these systems. With these research gaps in mind, the aim of the research presented in the current thesis is to examine both the low temperature and high temperature performance of NSM FRP strengthening systems for concrete, with a focus on NSM techniques for flexural strengthening of reinforced concrete slabs.

The primary objectives of this research project were:

- to experimentally investigate the performance of flexural NSM FRP strengthening systems for reinforced concrete slab strips at temperatures as low as -28°C, as might realistically be experienced in bridge or parking garage slab strengthening applications in Canada;
- to experimentally investigate the performance of flexural NSM FRP strengthening systems for reinforced concrete slab strips at temperatures as high as 200°C, and to estimate the potential consequences of the observed behaviour for the fire endurance of these strengthening systems;
- to experimentally investigate the relative performance of both epoxy and cementitious adhesives for the use in flexural NSM FRP strengthening applications for reinforced concrete at room, low, and high temperatures, with a view to reducing installation costs by using lower cost adhesive systems; and
• to experimentally investigate the effects of groove size on the performance of flexural NSM FRP strengthening systems, again with a view to reducing the costs of these systems by reducing the required volumes of costly adhesives.

The secondary objectives of this research were:

• to apply a novel digital image correlation technique to analyze bond performance and curvature of flexural NSM strengthened slab strips and to correlate this data with that obtained using conventional instrumentation; and

• to develop and validate a numerical model based on a traditional sectional analysis to predict the moment-curvature behaviour of NSM FRP strengthened slab strips in bending.

1.3 Scope

The work presented within this thesis involved experimental testing of medium scale reinforced concrete slab strips strengthened with NSM carbon/vinylester FRP reinforcement, and associated modelling and analysis. Twenty-three one-way slab strips measuring 1524 mm long with a cross section of 254 mm × 102 mm were fabricated, strengthened in bending with a specific commercially available NSM FRP system and tested to failure in flexure. Pseudo-static monotonic testing was conducted at room temperature (20°C) on 8 specimens, and at low temperature (-26°C) on 8 specimens. In addition, sustained flexural load testing under a dynamic thermal regime (at temperatures up to 200°C) was performed on 7 specimens. Ancillary tests were also performed to characterize the properties of the constituent materials, including tensile tests on steel
reinforcing bars and NSM carbon FRP tapes at room temperature, and compressive tests on concrete cylinders at both room and low temperatures.

A numerical model was developed using traditional sectional analysis to generate theoretical moment-curvature plots for the unstrengthened and NSM FRP strengthened reinforced concrete slab strips, and these were compared against experimental data obtained during testing. Also, an existing bond model proposed by Seracino et al (2007a) was compared against test data from this thesis. This model was also integrated into the aforementioned numerical model.

Digital image correlation analysis was conducted to determine curvatures of slab strips during testing, and this was validated by comparison against curvatures computed on the basis of strain readings obtained using conventional PI-type strain gauges.

1.4 Outline of Thesis

Chapter 2 presents a thorough review of available literature on the behaviour of NSM FRPs in flexural strengthening applications for reinforced concrete members. The chapter begins with a detailed discussion of the existing literature for NSM FRP bars and strips, followed by a brief description of some of the existing field applications of the technology and a description of some of the previously proposed bond models for these systems. A discussion then follows covering both the low and elevated temperature performance of steel, concrete, and FRPs. Finally, available guidelines that treat the use of NSM FRP strengthening systems are highlighted.
Chapter 3 describes the experimental program that was undertaken for the purposes of this thesis. A detailed description of the testing program is given, including the test setup and instrumentation, as well as a discussion of the ancillary testing program performed to determine material properties. Also given is information on the method used for heating the slab strip specimens tested at high temperatures. The results of the experimental program are detailed in Chapter 4, along with detailed discussion of the data obtained.

Chapter 5 presents various methods of analysis to complement the experimental results obtained in Chapter 4. A detailed explanation into the process of digital image correlation is given, with emphasis placed on comparing moment curvature from traditional Pi-type strain gauges versus the digital image correlation results. Detailed calculations using available NSM FRP bond models from Seracino et al. (2007) are provided, and the results are compared against the experimental results from this thesis. Finally, a numerical layer model is described that predicts the moment curvature behaviour of both the NSM FRP strengthened and unstrengthened reinforced concrete slab strips.

Chapter 6 provides conclusions from this study along with recommendations for further work in the research area.

Appendix A presents supplementary data on available design guidelines for both NSM FRP strip and bar strengthening applications. Appendix B shows detailed calculations
for the flexural and shear strengths of the slabs strips used in this thesis. Recommendations from ACI 440.2R-07 (ACI 2007), ACI 318-05 (ACI 2005), CSA A23.3-04 (CSA 2006) and ISIS Design Manual 3 (ISIS 2001) are all considered. Appendix C shows plots comparing moment curvature results from digital image correlation versus from Pi gauges for all room temperature and low temperature slab strip tests.
CHAPTER 2

LITERATURE REVIEW

2.1 NSM Background

NSM reinforcement for strengthening concrete structures is not a new idea; the basic technique can be found in literature dating as far back as 1948 (De Lorenzis 2000), although these older applications used steel bars or rods as reinforcement and cement mortar as adhesives. However, the use of FRPs in NSM applications is a relatively new idea. In this newer technique, FRP bars or strips are used as reinforcement with either epoxy or cement-based adhesives. Research and early applications of NSM FRP strengthening began simultaneously in the late 1990’s. Early field applications include the strengthening of the Myriad Convention Center in Oklahoma City, strengthening concrete silos in Boston, and deck strengthening of a Naval Pier in San Diego (De Lorenzis et al. 2000). Early experimental work included testing of a full scale bridge in Missouri, which demonstrated the great promise of the previously unproven FRP NSM technique, with the NSM strengthening outperforming an externally bonded FRP sheet system by 10% with respect to strength gain (Alkhrdaji et al. 1999). Considerable research has since been performed to investigate the performance of different NSM strengthening systems in various applications, focusing on the effects of various system parameters, such as groove characteristics, adhesive types, and FRP shapes, on both overall member performance and bond performance.
2.1.1 NSM FRP Bars

Early experimental research studying NSM FRP technology focused primarily on the use of circular pultruded FRP bars similar to those used previously as internal reinforcement for concrete. The available literature in this area encompasses two broad testing categories, bond tests and member tests. These two areas have arisen because, in flexural strengthening applications with NSM reinforcement, as is the case with externally-bonded FRP sheets, members can typically be analyzed using the same assumptions that are used for conventional reinforced concrete members (i.e., strain compatibility analysis), with the exception that failure is often initiated by debonding failure of the NSM reinforcement by one of several related modes.

2.1.1.1 Bond Tests

Previous researchers have noted that in NSM FRP applications “bond is of primary importance, since it is the means for the transfer of stress between the concrete and the FRP reinforcement in order to develop composite action” (De Lorenzis et al. 2001). As such, numerous bond tests are reported in the literature investigating bond-specific performance issues. The two main types of bond tests reported to date are: (1) the direct pull-out test and (2) the beam pull-out test, both of which can be performed in a number of different ways. Modified beam pull-out tests are generally believed to be more representative of the true bond behaviour in real members strengthened in flexure (De Lorenzis 2001), whereas direct pull-out tests are more representative of bond behaviour in shear strengthening applications.
De Lorenzis et al. (2000, 2001 and 2002), conducted small scale beam pull-out tests on inverted T-beams to study the effects of bond length (6 to 24 bar diameters), type of FRP bar (GFRP or CFRP), surface configuration of the FRP bar (deformed or sandcoated) and size of the groove (16 to 25 mm) on the bond performance of NSM FRP bars in flexural applications. The setup was adopted from Miller (1999) and consisted of unreinforced inverted T-beams with a steel hinge and saw cut at midspan to promote cracking of the concrete up to the hinge. Grooves for the NSM were created by cutting two parallel slits into the concrete cover and chipping away the concrete from between them.

Three different failure modes were encountered during testing: epoxy splitting, concrete cracking and pull-out of the FRP bar. Epoxy splitting refers to a failure in which the epoxy adhesive cracked and split longitudinally, inducing bond failure in the specimen. Failure can also shift to cracking of the concrete surrounding the groove, depending on various parameters that remain incompletely understood. Pull-out failure refers to cases where the failure occurred at the epoxy-bar interface. It was found that deformed bars tended to have higher bond strengths due to their tendency to induce epoxy splitting failure rather than pull-out failure (which was observed for sandblasted surface finish specimens). In cases where splitting failure occurred, increasing the groove size increased the pull-out load by transferring the failure from epoxy split into the surrounding concrete. The testing also showed that the longer the bond length the higher the pull-out load. However, increased strength gains did not result in FRP rupture. For specimens with GFRP No. 4 (12.7 mm diameter) and CFRP No. 4 (12.7 mm diameter) sandblasted rods, the average bond strength decreased with increasing bond length, but
for CFRP No. 3 (9.5 mm diameter) rods the average bond strength was approximately constant.

Direct pull-out tests were conducted by De Lorenzis et al. (2002, 2004) to further investigate the bond performance of NSM FRP bars. The test variables were: adhesive type (epoxy or cement-based mortar), bond length (4, 12 or 24 times the nominal bar diameter), groove size (1.25, 1.5, 2.0 or 2.5 times the actual bar diameter) and bar surface treatment (spirally wound or ribbed). In testing reported by De Lorenzis (2002) a preformed groove was cast into the specimens during concrete placement. However, this test configuration is not realistic because it creates a smooth groove surface. In practice NSM grooves would typically be sawcut into an existing member, thus creating a rough groove surface. The smoother surface of the cast-in groove has a negative effect on the bond behaviour at the adhesive-concrete interface, thus changing the bond behaviour. This is evidenced by the fact that almost all tests with epoxy adhesive, and many with a cement-based adhesive failed by slipping at the adhesive-concrete interface.

De Lorenzis et al. (2004) reported on essentially the same tests as discussed above but with saw cut rather than preformed grooves. Failure modes reported in these tests were more similar to those reported by the other researchers mentioned previously. The results indicated that cement-based mortar adhesives had lower bond strengths than epoxy adhesives. The level of strength decrease was highly variable depending on the individual test parameters. The strength decrease was stated as being due to the lower tensile strength of the cement-based adhesive when compared to the epoxy adhesive. It
was also reported that, due to the expansive nature of the cement-based adhesive, longitudinal and transverse cracks were observed in the cover prior to testing, further degrading the bond. In general, the ultimate load of the bond increased with bond length, while the average bond strength, computed as force per unit length along the failure interface, decreased. However, for cement-based adhesives and spirally wound bars the average bond strength did not decrease. It was also found that the larger the groove size the higher the bond strength, except in cases with cement-based adhesive and spirally wound bars, for which pull-out failure at large groove sizes lowered the bond strength because of large amounts of cracking in the adhesive. The percentage of strength increase from increasing groove size decreased for larger groove sizes. The effect of bar surface configuration showed that more pronounced ribs on bars produced more brittle bond failures at lower load levels. Tests showed that the bond strength of GFRP bars was consistently lower than CFRP bars and that the longer the bonded length the more pronounced was the effect. In general it appeared that spirally wound bars yielded the highest average bond strengths. It appears that larger levels of surface deformation provide better bond strengths up to a certain point, after which bond degrades.

Hassan et al. (2004) conducted a study on reinforced concrete T-beams tested in 3 point bending looking at various bond related issues for NSM FRP bars. Some of the internal steel reinforcement was terminated before reaching midspan to promote a known location of primary flexural cracking. The effects of bonded length (150, 550, 800 or 1200 mm) and adhesive type (2 different epoxy adhesives) were investigated using CFRP ribbed NSM bars in 18 wide × 30 deep mm saw cut grooves.
Testing showed that the two epoxy adhesives performed similarly. For all beams except the ones with the shortest bond length of 150 mm, the observed failure mode was bond failure due to concrete splitting at the concrete-epoxy interface. Debonding initiated at midspan where internal steel reinforcement was terminated. The failure load increased with increasing bond length, although the strength increase between 800 and 1200 mm was smaller, at 7.5%. For the shortest bond length of 150 mm, only 6% of the tensile strength of the bar was developed. Longer bond lengths of 800 and 1200 mm were able to develop 40 to 45% of the tensile strength of the bars. Given their results, the authors concluded that rupture of the FRP reinforcement was “… not likely to occur regardless of embedment length or type of epoxy adhesive” (Hassan et al. 2004). The study also concluded that development lengths should not be less than 80 times the bar diameter to ensure proper bond strength and to limit free end slip.

Hassan et al. (2004) also created a two-dimensional finite element model to conduct parametric studies on the effect of groove spacing, groove size and edge distance on the behaviour of NSM FRP bars. These studies indicated that clear groove spacing of less than two times the bar diameter caused increased tensile stress at the concrete-epoxy interface which could cause bond failure, while clear spacing of more than two times the diameter had little effect. The increase in tensile stress was due to the overlapping stresses from the adjacent bars for closely spaced grooves. The studies also showed that the groove spacing had little effect on stresses at the FRP-epoxy interface. The minimum distance that the groove should be located from the edge of the member, termed edge
distance, was determined to be four times the diameter of the bar. The recommended groove width was found to be at least twice the bar diameter. The reader will note that while these recommendations were based almost exclusively on the results of parametric studies performed using a finite element model (i.e., little or no experimental validation), the resulting recommendations for clear groove spacing, edge distance and groove size have been adopted for design by both ACI 440.2R-07-Draft (ACI 2007) and Concrete Society TR No. 55 (The Concrete Society 2004).

2.1.1.2 Member Tests: Flexural Strengthening

Several authors have studied the overall performance of reinforced concrete beams strengthened in flexure with NSM FRP bars. Experiments on four full-scale NSM FRP strengthened reinforced concrete T-beams tested in four point bending were reported by De Lorenzis et al. (2000). One beam was left unstrengthened as a control. The second beam was strengthened using two No. 4 (12.5 mm diameter) GFRP deformed bars in two 25 × 25 mm (1 × 1”) grooves. The remaining two beams were strengthened using two CFRP No. 3 (9.5 mm diameter) or No. 4 (12.5 mm diameter) sandblasted bars in 19 × 19 mm (3/4 × 3/4”) and 25 × 25 mm (1 × 1”) grooves respectively.

Test results showed that all beams strengthened with CFRP bars failed by debonding of the NSM FRP reinforcement due to longitudinal splitting in the epoxy adhesive. In the GFRP strengthened beam, splitting was observed in both the epoxy and the concrete cover at failure. Tests showed that when bond was critical, increasing the area of NSM reinforcement did not significantly increase the strength of the member, as the beam with
No. 4 (12.5 mm diameter) CFRP bars failed at only 11% greater load than the beam with No. 3 (9.5 mm diameter) CFRP bars, corresponding to only half the NSM reinforcement area. Strength gains between 25.7% and 44.3% were observed when strengthened beams were compared to the unstrengthened control beam.

2.1.2 NSM FRP Strips and Tapes

Although the earliest laboratory and field research on NSM FRP reinforcement for strengthening concrete structures was conducted using circular FRP bars, more recent research has been carried out using rectangular (or square) NSM FRP strips or bars. The development of this new method was brought about by a desire to attain higher strains in the FRP prior to debonding failure. All other factors being equal, NSM FRP strips typically have higher average bond strengths than circular bars, due to the three dimensional distribution of bond stresses that develops in the concrete surrounding the NSM strip reinforcement in comparison to NSM bar reinforcement (Blaschko 2003). With NSM strips, the vast majority of the bond and tensile forces are kept within the plane of the surface of the concrete. With round bars, perpendicular forces, caused by radial stresses, induce tensile forces that tend to push the bar out of the concrete cover, contributing to splitting and bond failure. Furthermore, because strips have much higher ratios of perimeter to cross sectional area, the bond stresses for a given tensile force are lower (Teng et al. 2006).
2.1.2.1 Bond Tests

Numerous studies have been performed to investigate the performance of the bond between concrete and NSM FRP strips. Hassan et al. (2003) used the same flexural pull-out setup explained above and as presented by Hassan et al. (2004). Nine T-beam specimens strengthened with 1.2 mm × 25 mm strips in 5 mm wide × 25 mm deep grooves were tested to investigate the effect of bonded length (150, 250, 500, 750, 850, 950, 1050 or 1200 mm). Test results showed that bonded lengths of 150 and 250 mm failed by premature debonding at the adhesive-concrete interface providing little to no improvement in strength when compared to the unstrengthened control. However, considerable gains in strength and stiffness were observed in beams with bonded lengths of 500 and 750 mm. These beams failed by debonding of the strip from the free end. Debonding was observed at both the free ends and at the location of flexural cracks, but was governed by free end failures. For beams with bonded lengths between 850 and 1200 mm, failure was initiated by rupture of the FRP strip, indicating that the full tensile strength of the NSM FRP was utilized in these cases. Figure 2.1 shows the maximum strain achieved in the CFRP strip at failure for various bond lengths. The plot clearly shows that, for small bonded lengths, very little of the strength of the strip is utilized (less than 15%). For intermediate bonded lengths more and more of the strength was utilized (between 15% and 92%) as the bond becomes more fully developed. For the longest bonded lengths full composite action is achieved resulting in little increase in strain at failure with increasing bonded lengths of between 850 and 1200 mm. These results clearly show that it is indeed possible to utilize the full strength of the NSM for strips, as
opposed to typical results from multiple studies with NSM bars that show debonding failure regardless of bonded lengths.

Sena Cruz et al. (2004) conducted beam pull-out tests similar to those conducted by Hassan et al. (2003) to look at the effect of bond length (40, 60 and 80 mm) and concrete strength (35, 45 and 70 MPa) on the bond behaviour of NSM CFRP strips in flexural strengthening applications. Unfortunately, 60 kg/m$^3$ of hooked end steel fibres were added to the concrete mix, according to the authors to avoid shear failure of the specimen. The Authors indicated that “… for this content of fibres, only the concrete postcracking tensile residual strength is significantly affected by fibre reinforcement mechanisms. Since concrete cracking is not expected to occur in the bonding zone, the influence of adding fibres to concrete is marginal for bond behaviour” (Sena Cruz et al. 2004). In the opinion of the Author of the current thesis the above statement pertaining to fibres not having an influence in situations where cracking does not occur is indeed a reasonable argument. However, the statement that fibre addition does not affect bond behaviour, based on the work of Ezeldin et al. (1989), appears to be incorrect. Ezeldin et al. considered the bond of internal steel bars in concrete, which is a very different bond mechanism than for the case of NSM FRP strips. Thus, it may or may not be true that fibres do not have an effect on the NSM FRP bond. Given reported failures of NSM FRP strips involving cracking of the concrete at the epoxy-concrete interface, it seems reasonable to assume that fibres present in the concrete at this location could dramatically affect the failure mechanism. Sena Cruz et al. (2004) indicate that, because all test failures occurred in the adhesive, it is further proof that fibre addition has no effect on
their setup. Clearly, this assumption negates the fact that fibre addition could have strengthened the concrete in tension and resulted in epoxy failures. Detailed results of the tests have not been included because of these concerns.

Teng at al. (2006) performed direct pull-out tests to investigate the effect of bonded length on the performance of NSM FRP strengthening systems. The FRP strips had multiple strain gauges applied to their sides along their bonded length to study the strain distribution. The 2 mm x 16 mm CFRP strips were placed in grooves of 8 mm width × 22 mm depth. In the tests, two strips were bonded together side by side prior to strengthening with the same epoxy adhesive used for bonding the strips into the grooves. Strain gauges were placed between the two strips so that they did not interfere with the bond behaviour of the NSM strengthening during the test. Although the bonded length was varied from 30 mm to 250 mm, only the tests for 200 mm and 250 mm bonded lengths were reported, as other tests failed by “shear tension fracture within the concrete prism”. For bonded lengths of 200 mm and 250 mm, bond failure occurred at the CFRP-epoxy interface with the longer bond length providing the highest failure load (17.5% greater than the 200 mm bonded length) but with a lower average bond strength (6% lower). The tests also showed that as load increased, more and more of the strip became mobilized in resisting the load. This trend is as expected and has been similarly observed for externally bonded FRP sheet strengthening systems bonded to concrete.

Seracino et al. (2007b) conducted a series of push-pull direct bond tests on NSM CFRP strips in concrete prisms with the aim of developing a bond model. Thirty six tests were
conducted looking at the effects of bonded length, aspect ratio of the strip, and concrete strength. Strain gauges were placed on the NSM strip at 50 mm intervals along the groove. The Authors noted that this gauge arrangement could reduce the bond strength by 15-20% compared to specimens without gauges. This is an important conclusion, since many of the tests presented in the literature have used bonded foil strain gauges installed on the sides of the strip along the bonded length, and none of them (aside from the Seracino et al. paper) apparently consider the possible effects on bond performance.

In the Seracino et al. (2007b) study, failure modes of FRP rupture, splitting at the concrete-adhesive interface and adhesive splitting were all observed. Testing showed that increasing the width (the longer cross-sectional dimension) of the strip increased the bond strength. The increase was not only due to the greater cross sectional area, since increasing the width by 100% increased the strength by 118-157%. This was due to the better confinement of the NSM FRP strip as it penetrated deeper into the cover depth (as shown in Figure 2.2). The additional confinement resulted in a different failure plane, also shown in Figure 2.2. Increasing the strip thickness was found to only increase the load proportionally to the greater bonding area provided. The concrete strength was found to influence the bond strength. The relationship was found to correlate to the square root of the unconfined concrete compressive strength, which indicates that the failure load was related to the tensile strength of the concrete. Increases in bonded length provided higher strength. However, the research showed that these increases were smaller at longer bonded lengths, and with one groove size constant after a certain bond length.
2.1.2.2 Member Tests: Flexural Strengthening

Numerous studies have investigated the overall behaviour of reinforced concrete members strengthened in flexure with NSM FRP strips. Barros et al. (2005) tested rectangular beams in four-point bending with various amounts of internal steel and NSM FRP reinforcement. Beams were strengthened to double the unstrengthened design strength with 1.45 mm × 9.6 mm CFRP NSM laminate strips in 4 mm wide × 12 mm deep grooves. For all but one of the strengthened beams, failure occurred by concrete cover separation. The layer of concrete cover below the internal steel tensile reinforcement (which contained the NSM FRP reinforcement) detached from the beam section, and in some cases the concrete above the internal steel was also pulled away. The test of the other strengthened beam, which had the least amount of NSM reinforcement, was stopped when deflection reached 27 mm without any cover separation. Strength increases of 78% to 96% were achieved compared to the control beams. Strains ranging from 62% to 91% of the ultimate tensile strain of the strip (obtained from coupon testing) were also reported, with the highest strain level observed for the beam with the least amount of NSM strengthening. Increases in stiffness were also noted for the strengthened beams as compared to the control beams, along with increased cracking moment.

Teng et al. (2006) tested concrete beams in four-point bending with various bonded lengths (500, 1200, 1800 or 2900 mm) of NSM CFRP strips. The same strip and groove configurations were used as explained previously for the bond tests by Teng et al. (2006), including the multiple strain gauges installed between sandwiched CFRP strips. Test
results demonstrated that the beam with a 500 mm bonded length had no increased strength compared to the control, but was actually 1.25% weaker. This is not wholly unexpected since in this case the bonded length terminated within the constant moment region, meaning that failure could occur in the unstrengthened portion. The beam failed by concrete cover separation starting at the free end of the bonded FRP. Beams with bonded lengths of 1200 mm and 1800 mm provided strength gains of 30% and 90% respectively, as well as exhibiting stiffer behaviour when compared to the unstrengthened control. Both beams failed by concrete cover separation starting from the free end of the bonded FRP. From theoretical beam strength modelling the authors reported that the beams should have been close to flexural failure in their unstrengthened regions at the failure loads reported. This leads to the conclusion that bond failure at the free end of the NSM FRP reinforcement was triggered by flexural failure (or perhaps by the development of large flexural or shear-flexural cracks) in the unstrengthened region. The beam with a 2900 mm bonded length provided a strength increase of 106% compared to the control, along with demonstrating greater stiffness. In this case failure occurred by concrete crushing followed by concrete cover separation close to the maximum moment region, with debonding of the strip at the epoxy-concrete interface and splitting of the epoxy. Ultimate strains in the NSM FRP strips at failure ranged from 17 to 71% of the tensile rupture strength depending on bonded length (in general higher strains were observed for longer bonded lengths).

Because Teng at al. (2006) performed both direct bond tests and flexural beam tests with the same material, groove size and in some cases bonded length, comparison seems
reasonable between the two sets of results. However, the Authors note correctly that
direct comparison between the data sets is not possible due to fundamental differences
between the two testing methods. Specifically, debonding failures in beams are caused
primarily by concrete cracking in the cover region due to both flexural and shear effects,
rather than to pull-out failure at the strip-epoxy interface, as in the case of bond tests.

Yost et al. (2007) tested 15 reinforced concrete beams in four-point bending after
strengthening with 2.5 mm × 15 mm CFRP NSM strips in grooves of 6.4 mm wide × 19
mm deep. The effects of internal steel and NSM CFRP reinforcement ratio were
investigated. The grooves were formed by making two cuts side by side with a concrete
diamond blade to produce the desired groove width. Depending on the beam setup used
in each individual test, the observed failure mode was either concrete crushing or FRP
rupture. There was an increase in both flexural strength (10%-98%), stiffness and yield
strength (10%-47%) for all NSM FRP strengthened specimens. Also, the increase in
strength appeared to be inversely proportional to the amount of steel reinforcement
normalized to a balanced design condition (i.e. beams with less internal steel experienced
greater strengthening). Interestingly, neither premature debonding failure nor cover
separation were observed in any of the tests. This could be due to the NSM FRP being
continued past the supports (it is not clear from the source publication whether this was
the case).
2.1.3 Comparison of Strengthening Techniques for Concrete Members

In one of the earliest papers to report the use of NSM FRP reinforcement, Alkhrdaji et al. (1999) strengthened a decommissioned three span solid slab reinforced concrete bridge in flexure with NSM CRFP bars. Each span of the bridge was simply supported with a span of 7.9 m and with a total width of 8.2 m. One span was left unstrengthened to act as a control. The other spans were strengthened with either externally bonded CFRP sheets or NSM sandblasted CFRP bars. The strengthening schemes for both spans were designed to produce a 30% increased nominal flexural capacity. Each span was tested under quasi-static load cycles until failure. The NSM strengthened span failed by FRP rupture of some of the bars at the widest flexural crack. The FRP sheet strengthened deck failed by a combination of FRP rupture and peeling (debonding) of some of the sheets. The control span failed by steel yielding followed by concrete crushing as should be expected for an appropriately designed reinforced concrete (RC) structure. The NSM strengthened span was 27% stronger than the control, compared to 17% for the external FRP sheet system, which can be attributed to the debonding failure mode observed for the externally bonded system. At service load levels, both strengthened spans displayed higher stiffness than the control. The overall strength increase was less than the 30% design largely because the control was 47% stronger than anticipated. This study showed that both externally bonded sheets and NSM could be used to effectively strengthen full scale RC structures, but confirmed that the NSM strengthening technique made more effective use of the CFRP by avoiding premature peeling or bond failure.
Rizkalla et al. (2002) conducted tests on half-scale post tensioned bridge girders using various strengthening techniques. Three slabs representative of bridge slabs over intermediate pier columns were constructed, and each was tested in three different testing configurations. First the two bridge deck cantilevers were tested separately with the load applied to the end of the cantilever. Then the bridge deck center span was tested with the load applied at midspan. Before running the center span test, cracks that had developed from the previous tests were injected with a high strength epoxy resin to avoid influencing the test results. The strengthening systems used were two different NSM CFRP bars, NSM CFRP strips, externally bonded CFRP sheets and wet lay-up CFRP sheets. All strengthening was designed to achieve 30% strength gain (based on traditional strain compatibility analysis assuming perfect bond). For the NSM FRP strengthening, grooves were cut with widened wedge shapes at the ends intended to help prevent slip.

Gains in both flexural strength and stiffness were observed in all strengthened specimens pre and post steel yielding when compared against the control. All specimens except for the cantilever strengthened with externally bonded FRP strips failed by concrete crushing. The specimen with externally bonded strips failed prematurely by peeling of the strips. The externally bonded sheet application experienced the greatest strength gain, at 44% compared to the control. The NSM strip application resulted in the highest strength for the three NSM applications with a strength gain of 35% over the control. When factoring in construction costs externally bonded CFRP sheets were stated to provide the most effective strengthening technique for this particular application,
although the rationale for this statement is not clear and likely does not consider all of the previously mentioned benefits of the NSM technique.

El-Hacha et al. (2004) tested eight medium scale RC T-beams with a single load applied at midspan. One of the two groups of the internal tensile steel reinforcement ran the entire length of the beam, while the other was terminated before reaching midspan to promote flexural failure of the beam near midspan. The Author also noted that this simulated a potential field condition where the internal steel reinforcement is damaged or locally corroded. One beam was used as an unstrengthened control, while the four other beams were strengthened with either; one NSM CFRP reinforcing bar with 18 mm wide × 30 mm deep groove, two 2 mm × 16 mm NSM CFRP strip each in a 6.4 mm × 19 mm groove 75 mm apart, two 1.2 mm × 25 mm NSM CFRP strips each with in a 6.4 mm × 25 mm groove 75 mm apart and five 2 mm × 20 mm GFRP thermoplastic strips with three grooves 6.4 mm × 25 mm, 38 mm apart. In the final case, one strip was placed in the middle groove while two were placed in each of the other grooves. One of each of the remaining three beams was strengthened with two 2 mm × 16 mm externally bonded CFRP strips (one beam not damaged and one beam “extensively” damaged prior to testing) and five externally bonded GFRP thermoplastic strips. All strengthening systems used epoxy resins as adhesives. All external strengthening had U-wraps applied over the flexural FRP to improve the anchorage of the FRP strengthening systems.

Test results showed that the behaviour prior to cracking was similar for all beams including the unstrengthened control. However, after cracking the strength and stiffness
was greater for all strengthened beams compared to the control, as should be expected. For NSM FRP bars, failure was initiated by epoxy splitting, the most common failure mode reported in the literature for bars. Both NSM CFRP strip strengthening applications failed by rupture of the strip, demonstrating full composite action, while the NSM GFRP thermoplastic strip failed by concrete split failure. For all NSM strengthened beams, the number and width of cracks was reduced, thus allowing for smaller overall deflections. All beams strengthened with externally bonded FRP failed by debonding of the sheets at lower loads than the NSM strengthening applications. For the exact same CFRP strip strengthening material, the NSM application provided 4.8 times more strength gain than the externally bonded system.

Quattkebaum et al. (2005) compared the effectiveness of externally bonded CFRP strips, NSM CFRP strips and a hybrid glass/carbon mechanically fastened system for strengthening RC beams in flexure. Both fatigue and monotonic testing was conducted using single midpoint loading. Each strengthening technique was designed to provide similar axial stiffness. The same FRP strip material was used for both the externally bonded and NSM strengthening. Test results showed that the externally bonded system failed by peeling of the FRP sheets, but that both the NSM system and the hybrid system failed by concrete crushing. The NSM system was the only technique that had no apparent bond damage at the time of failure. The hybrid system had a lower failure load than the NSM due to a less effective bond transfer caused by splitting and crushing of the concrete around the fasters. Also, the system required some relative movement between the FRP and the concrete to engage the shear transfer mechanism. Interestingly, the
study also concluded that NSM performed well under fatigue loading, showing few signs of fatigue induced bond degradation.

Harrison et al. (2006) compared externally bonded CFRP sheets, NSM CFRP strips, externally bonded steel reinforced polymer (SRP) (widely known as the Hardwire™ strengthening system) and NSM stainless reinforcing steel bars as strengthening techniques for RC beams tested under midspan loading. Unfortunately, all four strengthened beams failed by concrete crushing prior to any bond failure or rupture. Strength increases in all cases were similar ranging from 46% to 51% compared to the unstrengthened control.

Aidoo et al. (2006) strengthened RC bridge girders from a decommissioned bridge built in 1961 using the same three strengthening techniques as Quattlebaum et al. (2005). The T-beams were cut from the existing structure and were tested under midspan loading either monotonically or under fatigue loading. The externally bonded FRP strengthening failed due to intermediate crack induced (IC) debonding. In IC debonding, bond failure of the applied strengthening initiates at a crack in the concrete at a location along the beam away from the free end of the strengthening. The NSM strengthened beam experienced concrete crushing followed by cover separation initiating from midspan and progressing toward one of the supports. The hybrid system failed due to concrete crushing after widespread bond damage and concrete cover separation. The study goes on to compare the strain limits present in ACI 440.2R (ACI 2007) for NSM against their test data, and it is noted that the current ACI 440 limit of 0.7-0.8 of the FRP rupture strain
appears to be too low (over conservative). The Authors also note that the concrete cover separation failure that has been observed in many flexural NSM FRP beam tests available in the literature is not considered in the current version of the ACI 440 document.

The preceding discussion indicated that, while most available studies report that NSM FRP systems, particularly NSM FRP strip systems, are able to utilize more of the strength of the FRP materials than equivalent externally bonded FRP strengthened systems (due primarily to their superior bonding characteristics), several comparison studies have suggested that NSM is a less cost effective solution (Rizkalla et al. 2002, Quattlebaum et al. 2005, Aidoo et al. 2006 and Rosenboom et al. 2007). The major cost increase for NSM stems from labor associated with groove cutting. Factors such as large amounts of epoxy required to fill grooves, high material cost and viscosity of adhesives in overhead situations can make the NSM technique more expensive when compared to externally bonded systems. However, these studies have not typically considered other potential advantages of the NSM technique such as increased durability and protection from fire and vandalism, as well as aesthetic advantages and potential to be better used in negative bending applications. With additional research into the effects of groove size, especially using NSM strips, it might be possible to significantly reduce the required groove size; thereby reducing the labor cost and the amount of epoxy needed. Also, using cementitious adhesives (or injection epoxies) rather than epoxy putty or paste adhesives could significantly decrease material costs. Furthermore, given the superior bonding characteristics of NSM strengthening, the higher costs of NSM FRP systems may be unavoidable in some applications.
2.1.4 NSM Bond Failure Modes

From the available research mentioned above, six distinct failure modes have been identified for NSM FRP strengthening. The observed failure modes are:

1. **Concrete Crushing** – In this failure mode the concrete at the top of the beam reaches its crushing strain prior to either FRP rupture or some form of bond failure. This classical reinforced concrete failure mode is entirely dependent on member geometry and reinforcement details (both internal and external), and on the ability of the FRP system to resist debonding failures.

2. **FRP Rupture** – Failure of members can occur by rupture of the FRP if the geometry is such that concrete crushing is prevented and the strengthening system has sufficient resistance to adequately delay debonding failures. The available research data indicates that FRP rupture is considerably more likely using NSM strips than NSM bars (all other factors being equal). Long bonded lengths, well anchored in regions of low moment, are apparently required to develop the necessary bond strengths to delay bond failure for beams with significant amounts of NSM FRP strengthening reinforcement.

3. **Adhesive Splitting** – High stresses, either radial stresses for bars or shear stresses for strips, can cause a failure in the adhesive adjacent to the FRP-adhesive interface, resulting in splitting of the adhesive around the FRP bar or strip. This failure mode appears to be more common for NSM bars than for strips due to more severe radial stresses caused by rounded bars. Increasing the groove width for NSM bars appears to reduce the occurrence of adhesive splitting failure.
4. **Concrete Splitting** – In this case, failure occurs in the concrete adjacent to the concrete-adhesive interface. This mode of failure occurs due to the high tensile force exerted on the concrete under transfer of force from the adhesive to the concrete member. Because of the relatively low tensile strength of concrete in comparison to the other materials involved in the bonded region, failure occurs by splitting of the concrete surrounding the adhesive filled groove. This failure mode is more common for NSM strips.

5. **Combined Splitting** – Failure can occur simultaneously by splitting of the adhesive and the concrete. Along some sections of the bond the adhesive will split, while at other locations concrete will split and pull away. The same factors as described previously for the individual failure modes conspire to cause this combined mode.

6. **Concrete Cover Separation** – The layer of concrete below the bottommost tensile reinforcing steel can pull away in its entirety from the member along a longitudinal failure plane in the concrete cover. This type of failure usually initiates at the cutoff point of the NSM strengthening in a high shear stress region (although it has also been seen in an intermediate crack-induced mode). In most cover separation failures there are multiple grooves combined with narrow member cross sections at the extreme tension fibre.

Given the sheer number and distinct nature of each failure mode it is difficult to derive rational yet simple design guidelines for the use of NSM FRP strengthening. To the Author’s knowledge, no single guideline has currently accounted for all of these various failure modes for design, and this is a clear barrier to implementation.
2.1.5 Field Applications

Although NSM FRP is still considered an emerging technology, many full scale applications and pilot projects have already been undertaken. The example applications listed below show the potential of this method in various situations:

- During 1997-98 the Myriad Convention Center in Oklahoma City, USA was strengthened using multiple strengthening techniques including externally bonded steel plates, externally bonded CFRP sheets and NSM circular FRP bars. Using an epoxy adhesive, NSM bars were placed in vertical grooves in the sides of RC joists for shear strengthening (De Lorenzis et al. 2000).

- NSM CFRP bars were used to repair six RC silos in Boston Massachusetts. The owner discovered a structural problem when large cracks started to form causing material to leak out of the silos. When field measurements were conducted it was discovered that the actual spacing of the vertical and horizontal steel reinforcing bars inside the silo walls was not as specified in the original construction drawings. In 1998, NSM CFRP bars were positioned between the existing reinforcement in both the vertical and horizontal directions on the outside of the silos. CFRP bars were placed in grooves with epoxy adhesive, first in the vertical direction and then in the horizontal direction (Nanni 2003).

- A three span simply supported RC bridge built in 1926 in Phelps County, Missouri was strengthened to remove load restrictions that had been in place since 1985. In 2002 the bridge was strengthened in the transverse direction using NSM CFRP tape (Stone et al. 2002).
• A parking garage structure in Liverpool, England was strengthened in negative bending using NSM FRP bars in 2002. After 30 years of exposure to severe chloride attack from deicing salts, internal steel reinforcement had corroded. Large cracks were also present in the structure due to structural deficiencies. The principal reasons for NSM use are telling and were stated by Farmer (2003) as:
  • protection from vandalism, traction and braking forces;
  • improved tolerance of surface irregularities; and
  • better bond strength due to removal of laitance.

2.1.6 NSM FRP Bond Models

Because many tests on NSM FRP strengthening systems in flexural applications have noted failure by one of a variety of bond failure or debonding mechanisms, many researchers have proposed bond models for use in design and analysis of NSM FRP strengthening applications. Most available models have been derived by studying only bond test results from direct pull-out type testing (Blaschko 1999, 2003, De Lorenzis 2002, 2004, Teng 2006, and Seracino 2007). While attempts have then been made to apply these models to beams, there has been little success in this area. The problems with this methodology are best explained by Teng et al. (2006):

“Note that the debonding failures observed in the flexural tests differ greatly from those found in the bond tests, as debonding in the beams is related primarily to concrete cracking in the cover region rather than pull-out failure along the NSM strip–epoxy interface. The same observation applies to previous experimental studies on the bond behaviour of NSM
round bars and strips (De Lorenzis 2002; Barros and Fortes 2004). There are several reasons for this difference: the presence of flexural and flexural-shear cracks altering the bond stress distribution, the curvature of the beam, and the dowel forces generated by the opening up of the bond cracks, phenomena which are all absent in a bond test specimen. This difference between bond tests and beam tests means that results from the former such as local bond-slip curves cannot be directly transferred into predictive models of the latter.”

Some of the reported NSM bond models are indeed based on local bond-slip curves derived empirically from bond tests (De Lorenzis et al. 2001, 2002). Many studies of member behaviour report the use of simple layer models based on simple mechanics and strain compatibility; however, these models rarely account for bond related failures in a rational way (Barros et al. 2004, Harrison et al. 2006, Kotynia 2007). A detailed discussion of this modelling technique can be found later in this thesis, where it is used to predict the performance of the beam tests reported herein. Finite element models have also been built by several researchers (De Lorenzis et al. 2004, Hassan et al. 2003, 2004, Rosenboom et al. 2007). These models seem well suited to modelling the particular test setup for which they were calibrated. However, due to the complexity of modelling the bond interface and capturing the variety of different failure modes these models are not particularly useful in predicting behavior of differing test setups.
A relatively simple, closed form expression has been proposed by Seracino et al. (2007a) to describe the performance and capacity of NSM FRP strips on bond tests such as those reported by Seracino et al. (2007a). The Seracino bond model is unique and widely applicable, since it derives a generic bond expression for both externally bonded and NSM FRP plates. The model assumes a bond failure initiated by intermediate cracks with a bond failure plane in the concrete at the concrete-adhesive interface. The assumed failure plane, which is based on post-test observations made by Seracino et al., is shown in Figure 2.3. Under these conditions, the maximum load in the NSM strip before bond failure is initiated is given by the following expression:

\[ P_{IC} = \alpha_p 0.85 \phi_f^{0.25} f_c^{0.33} \sqrt{L_{pec}(EA)_p} \begin{cases} f_{ult}A_p & \text{for FRP plates} \\ f_y A_p & \text{for metallic plates} \end{cases} \]

Eq 2.2

where:

- \( P_{IC} \) = maximum load in the strip before debonding (N)
- \( \alpha_p \) = \begin{cases} 1.0 & \text{for mean} \\ 0.85 & \text{for lower 95% confidence limit} \end{cases} 
- \( \phi_f = \frac{d_f}{b_f} \) = aspect ratio of the interface failure plane
- \( t_p = t_d = 1 \) (mm)
- \( d_f = \) depth of the groove + \( t_d \)
- \( b_f = \) length of the failure plane perpendicular to the concrete (mm)
- \( b_f = \) width of the groove + \( t_h \)
- \( L_{pec} = \) length of the failure plane parallel to the concrete surface (mm)
\[ L_{per} = 2d_f + b_f \]

= length of the failure perimeter (mm)

\[ f_c' = \text{concrete compressive strength (MPa)} \]
\[ E_p = \text{elastic modulus of FRP plate (MPa)} \]
\[ A_p = \text{area of FRP plate (mm}^2\text{)} \]
\[ f_{ult} = \text{ultimate strength of FRP plate (MPa)} \]
\[ f_y = \text{yield stress of steel plate (MPa)} \]

The value of 1 mm (for \( t_d \) and \( t_p \)) that extends the failure plane into the concrete is only an approximation. It is chosen to represent a typical failure plane as observed in testing; however it has little effect on the equation. The powers 0.25 and 0.33 are chosen based on a statistical analysis of available test data, including testing by others. The equation indicates that the most important parameters in bond strength are concrete strength, size of groove and stiffness of the FRP plate.

In another paper by Seracino et al. (2007b), a slightly different bond model is presented to predict the ultimate load in an NSM strip for intermediate crack induced debonding in a reinforced concrete beam flexural strengthening application. The model again assumes failure in the concrete adjacent to at the concrete-adhesive interface, and is again based on statistical analysis of test data (in this case on 36 push-pull bond tests using NSM CFRP strips).

\[ P_{IC} = \alpha \beta \sqrt{f_c' d_p^{1.36} b_p^{0.21}} \leq f_{ult} b_p d_p \]  

\[ \text{Eq 2.3} \]

where:

\[ \alpha, \beta, f_{ult}, b_p, d_p \]
\[
\alpha = \begin{cases}
0.19 & \text{for mean value} \\
0.16 & \text{for characteristic value} \\
1.0 & \text{for } L \geq 200 \text{ mm}
\end{cases} \\
\beta = \begin{cases}
\frac{\beta}{200} & \text{for } L < 200 \text{ mm}
\end{cases}
\]

\(d_p\) = dimension of the FRP plate parallel to the concrete face (mm)

\(b_p\) = dimension of the FRP plate perpendicular to the concrete face (mm)

This expression indicates that concrete strength and groove size are the most important parameters for bond strength.

Hassan et al. (2003) developed an analytical bond model based on the combined shear-bending model introduced by Malek et al. (1998) for externally bonded plates. This model assumes that debonding initiates due to high shear stress concentrations at the cutoff point of the NSM reinforcement, and it does not consider failure that initiates at intermediate flexural or shear-flexure cracks. The model gives a maximum expected shear stress for a given loading condition and must be compared to a maximum allowable shear stress for design (which is not known for most available NSM FRP systems).

Hence, the practical applicability of this model is limited. Three forms of the model are presented, each for a different loading case.

1. For a simply supported condition and midspan loading:

\[
\tau = \frac{t_f}{2} \left[ \frac{nP l_{o y,\text{eff}} e_{o x}}{2I_{\text{eff}}} + \frac{nP y_{\text{eff}}}{2I_{\text{eff}}} \right]
\]

Eq 2.4

where:
\( \tau = \text{interfacial shear stress} \)

\( t_f = \text{thickness of the FRP strip} \)

\[ n = \frac{E_f}{E_c} = \text{modular ratio} \]

\( E = \text{modulus of elasticity} \)

\( P = \text{applied concentrated load} \)

\( l_o = \text{unbonded length of FRP strip} \)

\( y_{\text{eff}} = \text{distance from the FRP strip to the neutral axis} \)

\( I_{\text{eff}} = \text{effective moment of inertia of the transformed section} \)

\[ \omega = \sqrt{\frac{2G_a}{t_s t_f E_f}} \]

\( t_s = \text{thickness of the adhesive layer} \)

\( G_a = \text{shear modulus of adhesive} \)

\( x = \text{longitudinal coordinate starting from cutoff point of FRP strip} \)

2. For a simply supported condition and uniformly distributed loading:

\[ \tau = \frac{t_f}{2} \left[ a_1 x + b + c \omega e^{-ax} \right] \quad \text{Eq 2.5} \]

where:

\[ a_1 = -\frac{qny_{\text{eff}}}{2I_{\text{eff}}} \]

\[ b = \frac{qny_{\text{eff}}}{2I_{\text{eff}}} (L' - 2l_o) \]

\[ c_1 = -\frac{qny_{\text{eff}}}{\omega^2 I_{\text{eff}}} + \frac{qny_{\text{eff}}l_o}{2I_{\text{eff}}} (L' - l_o) \]

\( q = \text{applied uniform load} \)

\( L' = \text{total span of simply supported beam} \)

3. For a simply supported condition and two equally-spaced concentrated loads:

\[ \tau = \frac{t_f}{2} \left[ \frac{nPy_{\text{eff}}}{I_{\text{eff}}} + \frac{nPy_{\text{eff}}l_o}{I_{\text{eff}}} \omega e^{-ax} \right] \quad \text{Eq 2.6} \]
A failure limit for the maximum shear stress was proposed based on the Mohr-Coulomb failure criterion, depending only on the properties of the concrete as:

$$\tau_{\text{max}} = \frac{f'_c f_{ct}}{f'_c + f_{ct}}$$

Eq 2.7

where:

- $f'_c$ = compressive strength of concrete
- $f_{ct}$ = tensile strength of concrete

Rizkalla and Hassan (2002, 2004) have also developed an analytical model for development length using a design chart for NSM circular FRP bars. The model works by checking two failure criteria, namely concrete splitting and adhesive splitting. The design table, shown in Figure 2.4, is based on a two dimensional finite element analysis performed by the Authors. The development length is given by:

1. To prevent concrete splitting:

$$L_d = G_1 \frac{d_f \alpha_{FRP}}{4 \mu f_{ct}}$$

Eq 2.8

2. To prevent adhesive splitting:

$$L_d = G_2 \quad \text{or} \quad G_2 \frac{d_f \alpha_{FRP}}{4 \mu f_u}$$

Eq 2.9

where:
$L_d = \text{development length (mm)}$

$G_1, G_2, G'_2 = \text{coefficients for NSM FRP bars found from Figure 2.4}$

$d = \text{diameter of FRP bar (mm)}$

$f_{\text{FRP}} = \text{maximum tensile stress in NSM FRP bar at onset of debonding (MPa)}$

$\mu = \text{coefficient of friction between NSM FRP bar and adhesive}$

$f_{ct} = \text{tensile strength of concrete (MPa)}$

$f_a = \text{tensile strength of adhesive (MPa)}$

$C = \text{adhesive cover (mm)}$

$w = \text{width of groove (mm)}$

The major limitation to this particular bond model is that the design chart is derived from finite element modelling and it thus applies to one specific NSM strengthening configuration. To be useful, the chart would have to be regenerated through finite element modelling for every different strengthening situation encountered. Furthermore, the coefficient of friction required for the model, $\mu$, is different for each different adhesive-bar configuration and would thus be difficult to use as a design parameter.

### 2.2 Material Performance at Low Temperatures

Relatively little research has been conducted on FRP strengthened reinforced concrete members at low temperatures, and there is an extreme paucity of data on the performance of NSM FRP systems for concrete at low temperatures. Although each material constituent of an FRP strengthened RC member undergoes different changes in material performance with varying temperature, the interactions between the materials may also contribute to the overall performance of the strengthened member. It is believed that the two main thermal effects on FRP strengthened members at low temperature are potential thermal incompatibility and polymer embrittlement (Green 2007). Thermal incompatibility results from the differences in the coefficients of thermal expansion.
(CTEs) of concrete, reinforcing steel, polymer resins and fibres. The coefficient of thermal expansion for carbon fibre in the direction of the fibre is in the range of -0.5 to $-0.1 \times 10^{-6}/^\circ\text{C}$ while for typical resins it is $45$ to $120 \times 10^{-6}/^\circ\text{C}$ (El-Hacha 2004). This difference, of one full order of magnitude in CTE, could induce both lateral and longitudinal thermal stresses, causing deterioration in the bond performance of NSM FRP strengthening systems. In NSM applications the bond deterioration could be worsened as compared with externally-bonded FRP sheet strengthening because a greater thickness of polymer resin is typically required for NSM systems. The coefficient of thermal expansion of concrete is sensitive to moisture content, but is usually assumed to be between $10$ and $12 \times 10^{-6}/^\circ\text{C}$ over the range of $20^\circ\text{C}$ to $-70^\circ\text{C}$ (Baumert 1995). Again, this represents a full order of magnitude difference compared to typical epoxy resins, potentially causing thermal stress and possible bond deterioration between the resin and the concrete. Polymer embrittlement is caused by an increase in the strength and stiffness of epoxies at low temperature (Täljsten et al. 2007). Failure modes involving epoxy failure, such as many of the modes involved in debonding of NSM FRP systems, become more brittle due to the adhesive’s increased stiffness. The increased stiffness may also reduce the effectiveness of the polymer resin to transfer stress between the fibres, resin and concrete (Green 2007). It is worth noting that freeze thaw cycling can also have a significant effect on the behaviour of FRP strengthened members. A complete discussion on this topic is given by Green (2007).

Concrete exhibits mild increases in compressive strength, tensile strength, elastic modulus and flexural strength at low temperature (Baumert 1995). Moisture content can
have a significant effect on the amount of increase, since the strength gain is due primarily to the formation of ice in the pores of the hydrated cement paste (Neville 1997).

The limited research available on externally-bonded FRP strengthening of concrete at sustained low temperatures suggests that there are no negative effects. The only available study into NSM FRP strengthening at low temperature is reported by Täljsten et al (2007). This study reported the results of flexural beam tests of RC beams strengthened with square NSM FRP bars. It is difficult to draw broad conclusions from the test results as the concrete strength of each sample varied from 57 to 71 MPa at room temperature. However, the results do suggest that the ultimate load capacity of the NSM FRP strengthened beams was not negatively affected at -28°C. Research into RC beams strengthened with externally bonded FRP also show an increase in ultimate strength (Baumert 1995, El-Hacha et al. 2004, Wu et al. 2006 and Green 2007). The observed increases in strength can be largely attributed to the increased compressive and tensile strength of the concrete at low temperatures.

Increased strength of FRP strengthened members failing in bond at low temperature could be partially attributed to the stronger concrete delaying bond failure in cases where concrete failure is a part of the bond failure mechanism. In NSM situations with CFRP strips, failure adjacent to the epoxy-concrete interface is a common failure mode, and this may be delayed with the higher concrete strength provided by low temperature exposure.
2.3 Material Performance at Elevated Temperature

In general, FRPs used in structural applications with reinforced concrete are not expected to perform well at elevated temperatures. Fibres in FRP materials perform significantly better at elevated temperatures than the resins that are used to bind them (Bisby et al. 2005). As thermoset resins commonly used in structural FRPs reach or exceed their glass transition temperature \( T_g \) they experience significant losses in both strength and stiffness, as well as bond properties (Kodur et al. 2007). Therefore, the \( T_g \) has been taken as a measure of the performance limit of FRP materials at high temperature. Although, the \( T_g \) has been widely assumed as the critical temperature, losses in strength, stiffness and bond properties may occur even before the material reaches its \( T_g \) (Bisby et al. 2005).

For externally bonded FRP systems or NSM applications, bond performance is typically of critical importance for the satisfactory overall performance of the strengthened member. Bisby (2003) compiled results available in the literature from various researchers to produce plots of strength, stiffness and bond loss with temperature for fibres, FRP composites and unreinforced resins. Figures 2.5 and 2.6 show the plots for fibre and bond properties respectively. From these plots it can be seen that loss of bond occurs at much lower temperatures than loss of strength for fibres, again showing the resin dominated performance of FRP systems at elevated temperature. This resin-dominated degradation of properties at elevated temperatures is of fundamental importance to the performance of NSM FRP systems at elevated temperatures, as will
become evident during the presentation of test data from the current research in Chapter 4 of this thesis.

It has been suggested by some researchers that using NSM FRP strengthening systems could provide better elevated temperature performance than externally bonded FRP systems (Blaschko et al. 1999, El-Hacha et al. 2004 and Quattlebaum et al. 2005). With the strengthening system located even slightly within the concrete cross section, the FRP is protected somewhat by the surrounding concrete cover. Although this idea has been repeatedly suggested, there is apparently no available research to either confirm or refute this claim.

For the testing conducted in the current thesis, temperatures during elevated temperature testing remained at a sufficiently low temperature as to not affect the material performance of either the concrete or the internal reinforcing steel. If detailed information into the behaviour of these materials is required it can be found in Neville (1997), Khoury (2000), Bisby et al. (2005) and ASCE (1992).

2.4 Existing Guidelines for NSM

NSM FRP strengthening can be considered a state-of-the-art or emerging strengthening technology, and as such it is only lightly covered in a few guidelines around the world. The guidelines available to the Author at the time of writing which treat the design of NSM FRP systems are; ASCE 440.2R-07-DRAFT, The American Concrete Institute’s proposed Guide for the Design and Construction of Externally Bonded FRP Systems for
Strengthening Concrete Structures (ACI 2007), CAN/CSA S6-06, The Canadian Highway Bridge Design Code (CSA 2006) and Design Guidance for Strengthening Concrete Structures using Fibre Composite Materials, Second Edition (TR 55), a report of a Concrete Society Committee (Concrete Society 2004). A brief description of guidelines’ clauses as they pertain to the use of NSM FRP reinforcement is given in Appendix A.

### 2.5 Conclusions

Tables 2.1 and 2.2 show, to the knowledge of the Author, all available results from testing on NSM FRP strips for bond and flexure with concrete respectively, which are also the focus of the current thesis. While it is extremely difficult to draw generalized conclusions from the data, since failure modes are highly dependent on the individual test setup and the specific systems used for strengthening, the table does provide a general overview of the work performed to date and the parameters that have been studied by researchers.

Comparison studies performed to date have shown that NSM FRP strengthening systems for RC structures are more effective and efficient than externally bonded FRP strengthening systems. The available research on NSM FRP also makes it is clear that NSM strips take better advantage of the strength of the FRPs than NSM FRP circular bars, and strips are thus more likely to fail by FRP rupture at higher load levels, as opposed to debonding. The ability to use smaller grooves, thus reducing the construction time and cost are also significant benefits of the NSM FRP strip technique. The research
community clearly recognizes these benefits, as the majority of the most recent research has studied NSM FRP strips, not bars. Because of the differing bond behaviour of NSM strips compared to bars, conclusions from NSM bar studies cannot be directly applied to NSM strips, and studies must be conducted looking specifically at the bond performance and durability of NSM FRP strips.

It has been shown that results from bond tests are not easily applicable to beam test results. This has meant that existing guidelines for the design of NSM FRP systems have not been thoroughly (or in some cases rationally) developed. As with any new technology in Civil Engineering, without guidelines design engineers are apprehensive about using NSM FRP in practical applications. Of the guidelines that do exist, most deal with NSM FRP bars as opposed to the more effective strips. The strain limits imposed by the codes to prevent debonding failures also appear to be somewhat arbitrary, being only based on the specific results of a few limited research studies. More experimental data that can be directly compared to look at the effects of various parameters is required so that better design guidelines can be proposed.

Of all the work completed to date, none has apparently focused on the durability of NSM FRP strengthening systems. Furthermore, while suggestions have been made in the literature that NSM should perform better in high temperature applications, no studies have proven (or disproved) this hypothesis. Finally, the use of grout adhesives has not been thoroughly investigated, nor has the effect of groove width for NSM strips been quantified.
Many studies have been published on the topic of NSM FRPs, but much work remains for the technique to gain acceptance. Although many benefits have been identified over other strengthening techniques, the variability of different studies makes it difficult to draw broad conclusions. The research presented in the current thesis represents a crucial step in expanding the available test database with a view to eventually developing rational and defensible design procedures for NSM FRP strengthening systems in a broad variety of applications.
### Table 2.1: Available direct bond test results with NSM FRP strips.

<table>
<thead>
<tr>
<th>Test Author</th>
<th>Test Type</th>
<th>$E_{frp}$ (MPa)</th>
<th>$f_{ap}$ (MPa)</th>
<th>$\varepsilon_{fult}$</th>
<th>Groove Size (mm)</th>
<th>Strip Size (mm)</th>
<th>Groove Ratio</th>
<th>Bonded Length</th>
<th>$% \varepsilon_{ult}$</th>
<th>Pull-out Force (kN)</th>
<th>Bond Stress (MPa)</th>
<th>Failure Mode</th>
<th>Concrete Strength (MPa)</th>
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<td>2740</td>
<td>0.0017</td>
<td>3.3</td>
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Table 2.1 continued

| Test Author         | Direct pull-out | | |
|---------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Seracino et al.     | 144600          | 2634            | 0.0182          | 5.0             | 12.0            | 2.9             | 10.0            | 1.7             | 1.2             | 200             | 0.59            | 45.1            | 8.77            | bond failure CFRP-Epoxy interface | 65 |
| Seracino et al.     | 162300          | 2796            | 0.0172          | 3.2             | 22.0            | 1.2             | 19.9            | 2.6             | 1.1             | 300             | 0.99            | 67.8            | 5.36            | CFRP Rupture | 33 |
| Seracino et al.     | 162300          | 2796            | 0.0172          | 3.2             | 22.0            | 1.2             | 20.0            | 2.7             | 1.1             | 200             | 0.90            | 60.7            | 7.16            | CFRP Rupture | 33 |

Table 2.2: Available flexural tests results with NSM FRP strips.

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<th>Test Author</th>
<th>$E_{frp}$ (MPa)</th>
<th>$f_{ub}$ (MPa)</th>
<th>$\varepsilon_{ub}$</th>
<th>Groove Size (mm)</th>
<th>Strip Size (mm)</th>
<th>Groove ratio</th>
<th># Strips</th>
<th>Bonded Length (mm)</th>
<th>Failure Load (kN)</th>
<th>% $\varepsilon_{ud}$</th>
<th>Failure Mode</th>
<th>FRP ratio $A_{frp}/A_x$</th>
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Figure 2.1: Maximum strain in the CFRP strip for various bonded lengths during testing by Hassan et al. (2003).

Figure 2.2: Additional confinement due to greater strip depth (Seracino et al. 2007).

Figure 2.3: Bond failure plane assumed by Seracino et al. (2007).
Figure 2.4: Design chart for NSM FRP bars (Rizkalla et al., 2002 and Hassan et al. 2004).

Figure 2.5: Variation of tensile strength of glass, aramid and carbon fibres with temperature (Bisby et al. 2005).
Figure 2.6: Variation in bond strength with temperature for various types of FRP bars with temperature (Bisby et al. 2005).

Figure 2.7: ACI 440 prescribed NSM groove dimensions (ACI 2007).
CHAPTER 3

EXPERIMENTAL PROCEDURE

3.1 General

The testing program reported in this thesis involved the design, construction, strengthening, and flexural testing of 23 medium scale steel reinforced concrete slab strips strengthened using NSM techniques with Aslan 500™ CFRP tape. Properties of this commercially available CFRP tape are provided in Table 3.1. All flexural specimens were tested in four-point bending under monotonic load. Specimens were tested under one of three exposure conditions: constant room temperature (21°C), constant low temperature (-26°C), or increasing elevated temperatures (up to 200°C) using a purpose-built load frame contained inside a large environmental testing chamber. Various ancillary tests were also conducted to characterize the properties of the constituent materials used in the fabrication of the slab strip specimens. The following sections provide details of the testing procedures used.

3.2 Variables Studied

The testing setup was chosen to specifically investigate how certain variables influence the performance of flexural NSM strengthening systems. NSM strips were chosen because, as noted in the previous chapter, many researchers have found that NSM strips are more fully able to utilize the strength of the FRP as compared to bars. Also, NSM strips use smaller grooves, resulting in less adhesive use and thus lowering costs. The effect of groove width was examined in the current study by using both 6.4 mm wide and
3.2 mm wide grooves. The effect of groove size is an important variable for the acceptance of NSM strengthening in the field. Current guidelines are based on limited experimental and parametric studies. The use of smaller grooves allows for the use of less adhesive, which lowers the cost of the system. Also, smaller grooves are faster to cut, lowering labour costs. Two different adhesives, a crack injection epoxy and a cementitious grout, were tested to determine the difference in behaviour. Cementitious adhesives are typically lower in cost than epoxy adhesives; potentially lowering the cost of the strengthening system. Epoxies have also been associated with toxic off gassing, which is not typically associated with cementitious adhesives. From an elevated temperature perspective, cementitious adhesives could possibly be more advantageous because they could provide a protective layer in high temperature situations, thus potentially improving the NSM strengthening system’s performance. Epoxy adhesives lose strength in the range of their glass transition temperature, $T_g$, which is typically much lower than the $T_g$ of the NSM strips’ matrix polymer (a vinylester in the case of the Aslan™ tape). At low temperature, thermal incompatibility between the epoxy adhesive, the concrete and the NSM strip, as well as possible epoxy embrittlement, could lead to deterioration in bond performance.

### 3.3 Slab Strip Specimens

#### 3.3.1 Design

Details of the slab strip specimens are given in Figure 3.1. The slab strip specimens were designed to simulate a scaled one-way reinforced concrete slab strengthened with an NSM FRP system. The slab strips were designed using strain compatibility according to
multiple codes and guidelines including; CSA A23.3-04 (CSA 2006), ISIS Education Modules (ISIS 2003), ACI 440.2R-07 Draft (ACI 2007) and ACI 318-05 (ACI 2005). Because the only available code or guideline dealing directly with NSM strip strengthening was ACI 440.2R-07 it was difficult to conduct a comprehensive design with anything other than the two American standards of ACI 318 and ACI 440. The goal of the slab design was to avoid crushing of the concrete or shear failure of the NSM strengthened specimens, such that the NSM strengthening system reached levels of strain sufficient to cause bond failure prior to overall failure of the members. Appendix B shows calculations for slab flexural and shear strength according to the aforementioned codes and standards. The slab’s flexural resistance was verified with and without the FRP strain limit $\varepsilon_{fult}$ suggested in ACI 440.2R-07 DRAFT because this limit is somewhat arbitrary and, given the variability of the current body of available literature noted in Chapter 2, is not yet well defined. Therefore, for testing purposes it was important to not include the currently prescribed strain limit as an upper bound for the flexural failure load.

The overall length of the specimens, 1520 ± 10 mm, was chosen to maintain a reasonable shear span length with a realistic overall slab thickness, adequate edge distances, and bonded length of NSM FRP reinforcement (again, bond failure of the NSM was the preferred failure mode), while also considering the overall weight of the specimens to facilitate their handling and testing in a space without crane access. The width of the slab strips was 250 ± 10 mm and the total depth was 100 ± 10 mm. The width was chosen so as to exceed the clear edge distance requirement (4 times the depth of the NSM groove).
specified in ACI 440.2R-07-DRAFT (ACI 2007). Minimum flexural internal steel reinforcement consisted of two D5 deformed steel bars (5 mm nominal diameter) spaced 130 ± 10 mm apart and placed on the tension side of the slab strips with 25 mm of clear concrete cover. This resulted in an internal steel reinforcement ratio of 0.34%. No internal lateral or transverse steel reinforcement was provided. The minimum steel reinforcement according to A23.3 is given by \(0.002A_g\), where \(A_g\) is the gross area of the cross section, giving 52 mm\(^2\), or a steel reinforcement ratio of 0.28%.

A 25 ± 5 mm long × 25 ± 5 mm deep notch at midspan was formed laterally across the full width of the tension face of the slab strips (see Figure 3.1) to allow a strain gauge to be applied directly to the NSM strip at midspan, thus enabling accurate strain measurements on the FRP in the constant moment region. The notch also acted as a crack initiator during testing, forcing a known location of the first flexural crack.

### 3.3.2 Fabrication

All specimens were cast in the Structural Testing Laboratories at Queen’s University, Kingston. The formwork, just prior to casting of the concrete, is shown in Figure 3.2 with internal steel reinforcement in place. The formwork was constructed using 20 mm (3/4 in.) plywood formboard. Sheets of 1220 ± 10 mm × 2440 ± 10 mm plywood with a 300 ± 10 mm extension were used to pour eight slab strip specimens on each sheet, with vertical formboard slats separating each of the slab strips. Three sheets were used to fabricate 23 beams in total. The midspan notch in each of the slab strips was formed using a 25 ± 5 mm × 25 ± 5 mm wooden block placed at midspan in the bottom of the forms. These were removed following curing of the concrete. Similar wooden blocks
were placed at either end of the slab strip forms to support the steel reinforcing bars during the casting process. These blocks remained in place throughout the testing.

The D5 deformed steel bars, used as internal reinforcement for the slab strips, were supplied in a coil from a local reinforcing bar supplier. The coil was cut into lengths of curved bar which were then manually straightened prior to placement in the slab strip forms. The bars were anchored to the wooden blocks inside the formwork using steel fencing nails; this ensured proper bar placement and held the steel in place during casting and vibration of the concrete.

Ready-mix concrete from a local concrete supplier was cast into the formwork and vibrated into place with a vibrating wand to ensure good consolidation, see Figure 3.3. The concrete had a specified 28-day compressive strength of 35 MPa, with air entrainment and with a slump of 75-100 mm. Following placement of the concrete, all slab strips were finished by screeding and toweling to achieve a smooth surface finish. Note that the slab strips were cast such that the NSM reinforcement was installed into the bottom face of the slab, as would be the case in a flexural strengthening application in positive bending. The specimens were covered with plastic drop sheet and moist cured for two days. After two days, the formwork was stripped and the slabs were carefully stacked and stored at ambient conditions in the Structural Testing Laboratory at Queen’s University.
3.3.3 NSM Strengthening

All of the slab strips, except for two unstrengthened control specimens tested at room temperature (C-RT-1 and C-RT-2), were strengthened in bending using a single strip of Aslan 500™ #2 CFRP tape placed into a central longitudinal groove in the concrete cover on the extreme tension face of the specimen. It should be noted that this provided large strengthening levels when compared against currently recommended strengthening limits, although this was the smallest CFRP strip commercially available for this type of application. For example, the predicted unfactored design failure moment of the unstrengthened slab strip, according to ACI 318-05 (ACI 2005), was 3.15 kN\cdot m, while the predicted unfactored failure moment of the strengthened slab strip was 7.08 kN\cdot m, according to ACI 440.2R-07-DRAFT (ACI 2007). This corresponds to a 265% increase in predicted unfactored flexural capacity. According to ACI 440.2 the strengthening limit in practice, assuming a one to one live to dead load ratio, is approximately 50%. However, because of the size of available strengthening material and the testing setup, no other option was available. For detailed load calculations refer to Appendix B.

Due to the cost advantage of smaller grooves, from lower material costs due to the use of less epoxy and lower labour costs due to faster groove cutting, it was necessary to investigate the influence of groove width on performance. Therefore, two different groove widths, namely 6.4 mm (1/4 in.) and 3.2 mm (1/8 in.) were used in the current experimental program (refer to Table 3.1). All grooves were cut with a tuckpointing grinder in conjunction with a diamond concrete blade (shown in Figure 3.4). The two specific groove sizes were dictated by the available widths of diamond blades. The depths
of the grooves were 21 mm ± 1 mm and 18 mm ± 1 mm for the 6.4 mm wide and 3.2 mm wide grooves, respectively. Again, the depths were dictated by the sizes of the available diamond blades. The extremely low viscosity of the adhesives used in testing, both of which were developed for crack injection applications, allowed them to flow freely within the groove surrounding the strip, providing adequate bond even with the relatively small width of the 3.2 mm groove compared to the FRP strip used for strengthening.

Each of the strengthened slab strips was turned upside-down during the groove cutting operations to allow the groove to be easily and precisely cut into the concrete cover. The wooden block at the midspan of the slab was removed, and an aluminum guide consisting of back to back angles with a slit cut into the bottom flange of one of the angles, was clamped onto the surface of the slab strip such that the slit was located over the centerline of the desired groove location (Figure 3.5). The aluminum guide ensured the proper location and longitudinal orientation of the grooves. The grinder was run along the aluminum guide, with one of the two aforementioned blade widths, to produce the grooves, and leaving approximately 25 mm of uncut concrete at both ends of the slab strips. Figure 3.6 shows selected details of the groove cutting operations, and Figure 3.1 shows the location and orientation of the grooves in the slab strip specimens.

The NSM CFRP tape was carefully cut to 1400 ± 10 mm lengths using a small hand saw. At the point where the groove was intersected by the midspan notch, the groove had to be dammed using foam strips and silicone caulking at a distance of 25 ± 5 mm from the notch (see Figure 3.7). Most structural epoxies may not require damming; however, due
to the low viscosity of the epoxy and grout used in this testing it was necessary. The NSM tape was then placed into the groove, between the foam strips, and was held firmly in place by the foam strips and silicone caulking. The resulting unbonded length of NSM FRP tape at the midspan notch was thus \( \approx 75 \) mm. The silicone was allowed to cure for 24 hours before proceeding with the groove filling and NSM bonding operations.

Two different adhesive types were used to bond the NSM CFRP tape into the grooves. Kemko® 038, an epoxy adhesive used for crack injection in concrete, was used for 14 of the slab strips. This specific product was chosen on the advice of an industry partner who has developed a novel NSM installation procedure in which NSM strips are placed in dry grooves, and the grooves are subsequently sealed and filled using pressure injection with crack injection equipment. This technique has many advantages including speed of installation due to smaller grooves, lower cost due to less wasted epoxy, clean installation using the pressure injection technique, confidence in complete filling of the groove due to the pressure and low viscosity, increased strength of the substrate concrete due to infiltration of the epoxy adhesive, and ease of overhead installation. Target 1118™, an unsanded silica fume (i.e., cementitious) grout, was used as the adhesive for 7 slab strips. This adhesive was used to compare the relative performance of polymeric versus cementitious adhesives, and to attempt to achieve better performance for NSM systems at both low and elevated temperatures. Target 1118 is also used in pressure injection applications, allowing NSM installation using conventional crack injection techniques. Thus, both of the chosen adhesives were expected to provide superior bond performance due to their high strength and low viscosity during installation, allowing for complete
filling of the confined groove space (particularly for smaller groove widths as in the case of the 3.2 mm wide groove slab strips). The detailed material properties, as provided by the manufacturers, are given for Kemko® 038 (epoxy) and Target 1118™ (grout) adhesives in Tables 3.2 and 3.3, respectively.

The Kemko adhesive is a two-part epoxy. For application, the two components were carefully measured and mixed according to the manufacturer’s instructions, and poured into the NSM groove around the strip (the epoxy had an extremely low viscosity, hence the need for groove damming). This was done while the beam was upside down, so that the groove filled by gravity in the current application (i.e., crack injection equipment was not used as would be the case in an envisioned field application). For negative moment strengthening applications, for instance flexural strengthening of a bridge deck cantilever, NSM strengthening is a well-suited technique, and the procedure used herein represents a realistic application procedure. However, for positive moment NSM flexural strengthening applications the method used herein is unrealistic and a pressure injection method would be used (as previously discussed). This would solve the problem of working with a low viscosity fluid upside down, and would also presumably provide superior bond performance due to the pressure applied during installation.

For the Target cementitious grout adhesive, the product was supplied in powder form and was mixed with water using a high shear paddle mixer and hand drill according to the manufacturer’s instructions at a ratio of 1 kg of powder to 365 mL of water. The grooved beams were thoroughly wetted prior to adhesive installation to prevent leaching of water.
from the grout into the substrate concrete, which could result in premature dehydration, shrinkage, and cracking of the grout before it had sufficiently cured. Similar to the epoxy adhesive slab strips, the low viscosity grout was poured around the strip into the grooves. The beams were covered with moist burlap and plastic drop sheets for 7 days to allow proper moist curing of the grout. Again, in an actual field application for positive bending, the grout would be installed under pressure using crack injection equipment.

3.4 Flexural Test Setup

The flexural test setup and loading apparatus for testing the slab strip specimens are shown in Figure 3.8. All slab strips were tested in four-point bending in a purpose built loading frame at Queen’s University. Four point bending was chosen to prevent the formation of a kink in the NSM strip in the notch at midspan. Specimens were tested upside down to allow for monitoring of the bond behaviour during testing using cameras as will be described in Section 3.5.2. The load beam had an overall span of $1450 \pm 5$ mm with a shear span of $510 \pm 5$ mm and a constant moment region of $430 \pm 5$ mm. The load beam consisted of two steel channel sections with a gap between them to allow a clear view of the NSM bondline from above. Circular steel rollers were used to apply point loads at known locations as shown in Figure 3.8. Three of the four support rollers were free in the horizontal direction, with one interior support being a pin support fixed in the horizontal direction, thus preventing axial loads being induced during testing. Steel bearing plates were inserted between the rollers and the slab strips’ concrete surface to avoid localized crushing of the concrete. The plates at the interior supports were $102 \pm 1$ mm wide by $10 \pm 1$ mm thick, while the plates at the end supports were $38 \pm 1$ mm wide by $13 \pm 1$ mm thick. Load was applied from above using a 490 kN hydraulic ram in a
self-reacting load frame built inside a large environmental cold room. Slabs tested at room and low temperature were loaded under displacement control at a constant loading ram stroke rate of 2 mm/min up to failure. Slabs tested at elevated temperature were tested under load control as described in the following sections. A summary of the full slab strip testing program is given in Table 3.4.

3.5 Instrumentation

The instrumentation used during flexural testing of the slab strip specimens is shown in Figure 3.8. All slab strips were instrumented during testing with conventional instrumentation consisting of: linear potentiometers (LPs), Pi-type strain gauges, and conventional foil strain gauges. In addition, two cameras were used to acquire high resolution digital images for performing digital image correlation analysis (which was later used to measure strains in the concrete and subsequently obtain beam curvatures during testing). Thermocouples were also used in tests where either low or elevated temperatures were factors.

3.5.1 Conventional Instrumentation

3.5.1.1 Linear Potentiometers

Linear potentiometers (LPs) were used to track and record vertical displacements during slab strip testing. Two LPs were placed under the loading plates at both outside supports and one was placed under the slab at midspan, giving a total of three LPs. By averaging the displacements from the end LPs and adding the midspan LP deflection, the total beam deflection could be found. This somewhat unusual testing methodology was required in
order to photograph the bonded NSM strip during testing to enable digital image correlation analysis. The linear potentiometers were calibrated using a custom fabricated calibration tool accurate to 0.001 mm. Calibrations were conducted at both room temperature (21°C) and low temperature (-26°C), and no changes in the calibration curves were discernable.

3.5.1.2 Pi-Type Strain Gauges

Three 100 mm gauge length Pi-type strain gauges were installed on one side of the slab strips to track strains over the height of the cross-section during testing. The Pi-type gauges were attached to the slab using adhesively bonded bolts. The top and bottom Pi gauges were located 11 mm from the top and bottom surfaces of the slab, respectively, and the middle Pi gauge was located at the mid-height of the cross-section. The bottom Pi gauge location corresponded to approximately the location of the centreline of the NSM strip installed in the slab strips. This configuration allowed the generation of strain profiles over the slab strips’ height, and subsequently determination of slab strip curvature during testing. Calibration of the Pi gauges was conducted using a similar calibration tool (with an accuracy of 0.001 mm) as the LPs at both room and low temperature. In the case of the Pi gauges it was found that the calibration curve changed at low temperature due to a change in the slope of the linear trend, but that linearity was maintained.

3.5.1.3 Foil Strain Gauges

Vishay® SR-4 250LW 120 Ω general purpose unidirectional strain gauges were used to monitor strains in the NSM FRP strips during testing. A single pre-wired foil gauge was
applied to the NSM strip within the notch at midspan (at the NSM tape’s centerline) in the unbonded region. All foil strain gauges were installed according to manufacturer’s instructions in accordance with Vishay Instructions Bulletin B-137 (Vishay, 2005).

For testing at low and high temperatures, strain gauge data was thermally corrected during the post processing phase using temperature correction coefficients provided by the manufacturer and using temperature data collected at the location of the strain gauges during the tests by adhesively bonded Type-T thermocouples (described below).

3.5.1.4 Thermocouples

Type-T thermocouples were used to monitor temperatures at various locations in and on the slab strip specimens during testing. During low temperature testing, a single Type-T thermocouple was placed close to the slabs to track the room temperature during the testing to ensure that it remained within the desired range of -26°C ± 2°C. During elevated temperature testing, four Type-T surface-bonded thermocouples were used to track the temperature at the outside surface of the NSM strengthening system (i.e., immediately above the CFRP tape on the extreme tension face of the slab strips). Thermocouple 1 was placed at the loaded end of the NSM system, 38 mm from midspan. Thermocouples 2 and 3 were placed at 195 mm and 390 mm toward the support from the loaded end of the NSM tape. Thermocouple 4 was adhered to the side of the CFRP tape, within the notch, to track the FRP tape’s temperature at the location of the foil strain gauge.
3.5.2 Digital Image Correlation Analysis

A novel digital image correlation technique was used to monitor strains and relative displacements that occurred during flexural testing of the slab strips. The technique basically consisted of using timelapse digital photos and computer software to track movements of pixels within the photos as the tests progressed. A detailed description of the technique can be found in Chapter 5 and more generally in White et al. (2003).

It was hoped that the technique could be used for two independent monitoring situations, hence two cameras were used. To provide visual texture to the beams surface which could subsequently be tracked using GeoPIV software, all slab strips were whitewashed on their top and side surfaces where each individual camera would be focused. The whitewash was then textured with black paint in a random stipple pattern. Two cameras were used to take stationary time-lapse photos at 5 second intervals throughout the testing. The photos were indexed to loads recorded by the data acquisition (DA) system by synchronizing the clocks of the cameras’ remote capture software and the DA computer.

One camera was located above the test setup and focused downward, perpendicular to the slab and through a gap in the loading beam, looking directly at the NSM strengthening system for one half of the slab strip (i.e., looking vertically down at one of the two NSM FRP bonded lengths). This camera was intended to be used to study bond failure mechanisms and potential relative slip between the concrete and the FRP tape (results are discussed in Chapter 5). The second camera was focused to look horizontally at the side
of the slab strip at midspan (on the side opposite the Pi-type strain gauges). This camera was used to generate strain profiles over the depth of the slabs, and to correlate with the Pi gauge data. Care was taken to ensure that the cameras were leveled and in line with the slabs to ensure accuracy in subsequent image correlation. Lines of predefined known length were drawn in the x and y planes on the beams within the frame of view of cameras, to be used later in the data analysis.

One Canon Digital Rebel XT (top camera) and one Canon Digital Rebel XTi (side camera) were used. Remote capture was accomplished using DSLR Remote Pro Version 1.3. Analysis was later conducted using GeoPIV software, which was developed by Dr. Andy Take (currently at Queen’s University) and Dr. Dave White at Cambridge University (White et al., 2003).

3.6 Slab Strip Testing Procedures

3.6.1 Room Temperature Tests

Eight slab strips were tested under room temperature conditions. These comprised:

- two unstrengthened control specimens (C-RT-1 and C-RT-2);
- two NSM CFRP tape strengthened specimens with epoxy adhesive and a 6.4 mm wide groove (E-6-RT-1 and E-6-RT-2);
- two NSM CFRP tape strengthened specimens with epoxy adhesive and a 3.2 mm wide groove (E-3-RT-1 and E-3-RT-2); and
- two NSM CFRP tape strengthened specimens with grout adhesive and a 6.4 mm wide groove (G-6-RT-1 and G-6-RT-2).
3.6.2 Low Temperature Tests

Eight slab strips were tested at low temperatures of -26°C ±2°C. These comprised:

- two unstrengthened low temperature control specimens (C-LT-1 and C-LT-2);
- two low temperature NSM CFRP tape strengthened specimens with epoxy adhesive and a 6.4 mm wide groove (E-6-LT-1 and E-6-LT-2);
- two low temperature NSM CFRP tape strengthened specimens with epoxy adhesive and a 3.2 mm wide groove (E-3-LT-1 and E-3-LT-2); and
- two low temperature NSM CFRP tape strengthened specimens with grout adhesive and a 6.4 mm wide groove (G-6-LT-1 and G-6-LT-2).

For low temperature testing, the cold room, inside which the load frame was constructed, was cooled from room temperature to a set point of -26°C ±2°C, with all of the low temperature slab strip specimens inside, and allowed to stabilize for approximately 60 hours prior to testing (to ensure that the slab strips had reached a uniform temperature of -26°C ±2°C). After stabilization, testing proceeded at a constant uninterrupted low temperature. However, it was necessary to bring the environmental chamber back to room temperature three times during the low temperature testing phase because of mechanical problems with the loading system hydraulics at such low temperatures, mechanical problems with the cold room and power outages. Each time the room was re-cooled the slabs were again allowed to stabilize for at least 60 hours at low temperature before testing continued.
3.6.3 High Temperature Tests

Due to the concerns associated with FRP strengthening at high temperatures, it was important to test NSM strengthening at elevated temperatures. A quick and simple heating technique was developed using a custom order heating blanket to heat half of the slab up to 200°C to investigate the high temperature performance of the systems. This temperature was well above the glass transition temperature of both the epoxy (measured to be 51°C) adhesive and the resin (believed to be close to 140°C) used in the NSM strip. Both epoxy and grout adhesive slabs were tested to examine their differing behaviour. Slabs were loaded under load control to a sustained load of 20 kN and then, while the load was held constant, they were heated at approximately 10°C per minute up to either 100°C or 200°C. The temperature was then held constant until failure. The 200°C temperature was chosen to be higher than the glass transition temperatures listed above. The 100°C temperature was chosen to simulate an insulated slab in a full scale fire test according to Bisby et al. 2005.

High temperature testing was conducted on seven slab strip specimens. These included:

- four NSM CFRP tape strengthened specimens with epoxy adhesive and a 6.4 mm wide groove, including:
  - three specimens tested under sustained load and heated at approximately 10°C per minute to a soak temperature of 200°C (E-6-HT-1, E-6-HT-2 and E-6-HT-3); and
  - one specimen tested under sustained load and heated at approximately 10°C per minute to a soak temperature of 100°C (E-6-FT-1)
• three NSM CFRP tape strengthened specimens with grout adhesive and a 6.4 mm wide groove, including:
  - two specimens tested under sustained load and heated at approximately 10°C per minute to a soak temperature of 200°C (G-6-HT-1 and G-6-HT-2); and
  - one specimens tested under sustained load and heated at approximately 10°C per minute to a soak temperature of 100°C (G-6-FT-1)

An Omegalux® silicone rubber fiberglass reinforced heating blanket measuring 254 mm × 610 mm was used to heat the tension face of one end of the slab strip specimens tested at elevated temperature (i.e., over the entire length of the NSM bond). The heating blanket had a watt density of 0.78 W/cm² (5 W/in²), for a total wattage of 1200 Watts. The blanket was controlled using an Omega CS-2110J-R bench top controller running in a proportional control mode. Temperature was measured by the controller using a Type-J thermocouple located under the heating blanket at its midpoint at the top surface of the FRP strengthening system (i.e., directly above the NSM FRP tape). A layer of Fiberfrax™ ceramic fibre insulation was placed on top of the heating blanket (to prevent heat loss away from the slab strip specimen) and then the entire heating assembly was lightly weighted down to keep everything in place throughout the tests.

Because the heating blanket was only available in certain predefined dimensions and it was not possible to obtain a blanket that could heat the entire bonded length of NSM in the slab strips as fabricated for the current study, and cuts were made into the concrete
cover through the NSM FRP reinforcement (i.e., perpendicular to the NSM FRP) at 152 mm from the end of the beam. This was done so that the end of the NSM reinforcement would terminate underneath the blanket and thus be within the heated area. Figure 3.9 shows the layout of the heating assembly, insulation, and control thermocouple for the slab strips in elevated temperature testing.

Three epoxy adhesive and two grout adhesive NSM FRP tape strengthened slab strips were tested under a sustained load of 20 kN using the same loading setup as previous tests. This load was chosen after all of the room and low temperature testing had been completed such that it was well above the tested strength of the unstrengthened room temperature control slab strips, but lower than the weakest of the epoxy or grout adhesive NSM strengthened slabs. This load thus ensured that the slabs would not fail immediately upon loading to the sustained level, but that they would fail once the NSM strengthening system was rendered ineffective due to the elevated temperature exposure under sustained loads. This test was designed to study the true effectiveness of the NSM systems under elevated temperature situations, rather than to verify overall member performance or to simulate a standard fire test. Simply put, the test method allowed for rational consideration of the temperatures at which the epoxy and grout adhesive NSM FRP systems would lose structural effectiveness.

The load was applied at a rate of 2 kN/min up to 20 kN, and then held constant under load control until failure (with one exception as noted below). Once the load had reached the sustained value of 20 kN, the slab strips were allowed to stabilize for two minutes,
and the heating blanket was then turned on and ramped to a hold (soak) temperature of 200°C (the maximum allowable temperature for the heating blanket).

One epoxy adhesive and one grout adhesive slab were also loaded to 20 kN in the same manner as described above but were only heated to a soak temperature of 100°C. This soak temperature was chosen to simulate the temperature that would be expected for a well insulated NSM FRP strengthening system during a standard fire, based on previous fire testing on reinforced concrete members strengthened with well insulated externally bonded FRP sheets (Bisby et al. 2005). They showed that, if the supplemental fire insulation remained intact during fire, it was possible to maintain the surface temperature of the FRP strengthened concrete member at less than 100°C for up to 4 hours in some cases. Therefore, the 100°C test was designed to simulate the behaviour of a well insulated slab strip strengthened with NSM FRP reinforcement under a standard fire scenario.

It should be noted that ancillary thermal testing was performed to determine the approximate temperatures experienced at the base of the NSM groove during a typical elevated temperature test; see Section 3.7.5 for details.

### 3.7 Ancillary Testing

The ancillary test program consisted of various standard (or slightly modified standard) tests to determine the material properties of the various constituent materials used in the flexural testing program. These tests are described in the following sections.
3.7.1 Concrete

Fifteen standard concrete cylinders with radius of 305 mm and height of 152 mm were cast at the same time as the slab strip specimens and cured under identical conditions to the slab strips. These were tested periodically during the testing phase to determine the concrete’s compressive strength. Tests of 3 cylinders each were conducted at seven and 28 days, as well as at or around the time of slab testing. Tests were conducted according to ASTM C-39M-05: *Standard test method for compressive strength of cylindrical concrete specimens* (ASTM 2005), using a Reihle 1350 kN testing machine.

Three cylinders were also left in the cold room and allowed to cool with the slab strips to approximately -26°C. These cylinders were then removed one at a time from the cold room and immediately tested (testing started within 1 min) as described above to determine the concrete compressive strength at low temperature.

3.7.2 FRP Coupons

Six tensile FRP coupons were prepared from the Aslan 500™ tape. The coupon specimens, shown in Figure 3.10, were 330 mm long and had single ply transverse fibre 102 mm × 25 mm glass FRP tabs bonded to both faces at both ends with SikaDur™ 30 epoxy putty. The glass tabs were used to allow the grips of the 600 kN Instron Satec LX 600 universal testing machine to grip the specimen without damaging the Aslan tape, thus avoiding grip failure and obtaining good tensile property data. All specimens were instrumented with a single 10 mm gauge length 120 Ω foil strain gauge applied at the centre of the strip to record tensile strain throughout the test. Specimens were tested in
displacement control at a crosshead stroke rate of 2 mm/min until failure. The test setup is shown in Figure 3.9.

### 3.7.3 Reinforcing Steel

Data on of the D5 deformed reinforcing steel used in the current study was taken from previous tests conducted by Ranger (2007) on samples from the same roll of steel that was used in the current thesis.

### 3.7.4 Mortar Cubes

Three mortar cubes measuring 50 mm × 50 mm × 50 mm were cast from the Target 1118™ grout that had been mixed as described previously. Cubes were allowed to cure for 7 days under moist burlap and plastic drop sheet. The mortar cubes were tested at the time that the first grout slab strips were tested, 71 days after casting, in a 1350 kN Reihle compressive testing machine.

### 3.7.5 High Temperature Verification

In order to verify the temperature throughout the grooves during the loaded tests, two half slabs were strengthened with either the epoxy or cementitious systems along with a series of thermocouples within the groove. This allowed an estimate of the temperatures of the loaded beams at failure. Rather than applying thermocouples to each slab’s groove, this approach was taken so that the bond strength of tested beams would not be affected by interference from the thermocouple wires. Eight thermocouples were used in each half beam section: 4 at the base of the groove and 4 on the surface of the adhesive (see Figure 3.11). At the base of the groove, Thermocouple 1 was located at the loaded end just
inside the bonded length before the notch, Thermocouples 2, 3 and 4 were spaced at 195 mm each along the groove starting from Thermocouple 1. Four surface thermocouples were placed directly above the 4 thermocouples at the base of the groove. The slabs were then heated to 200°C in the same manner as the slabs described in Section 3.4.3 and the temperature was tracked using the thermocouples.

3.7.6 Differential Scanning Calorimetry

Differential scanning calorimetry (DSC) was used to determine the glass transition temperature \( T_g \) of both the epoxy adhesive as well as the resin used in the manufacturing of the Aslan™ 500 tape. Testing was conducted at Queen’s University according to ASTM E1356-03. Three samples of both the epoxy adhesive and the FRP strip were tested and analyzed.
### Table 3.1: Manufacturer specified properties of Aslan\textsuperscript{TM} 500 #2 CFRP tape (supplied by Hughes Bros, Inc., Seward, NE).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width (mm)</td>
<td>16</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>2</td>
</tr>
<tr>
<td>Cross Sectional Area (mm(^2))</td>
<td>31.2</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>2068</td>
</tr>
<tr>
<td>Tensile Modulus (MPa)</td>
<td>124000</td>
</tr>
<tr>
<td>Ultimate Strain</td>
<td>0.017</td>
</tr>
<tr>
<td>Coefficient of Thermal Expansion</td>
<td></td>
</tr>
<tr>
<td>Transverse (°C)</td>
<td>74 to 104 \times 10^{-6}</td>
</tr>
<tr>
<td>Longitudinal (°C)</td>
<td>-9 to 0 \times 10^{-6}</td>
</tr>
</tbody>
</table>

### Table 3.2: Typical properties for KEMKO\textsuperscript{®} 038 – Regular IR (supplied by ChemCo Systems, Redwood City, CA).

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Method</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength (MPa)</td>
<td>ASTM D 638</td>
<td>62</td>
</tr>
<tr>
<td>Elongation Percent (%)</td>
<td>ASTM D 638</td>
<td>2</td>
</tr>
<tr>
<td>Compressive Yield Strength (MPa)</td>
<td>ASTM D 695</td>
<td>110</td>
</tr>
<tr>
<td>Compressive Modulus (MPa)</td>
<td>ASTM D 695</td>
<td>2758</td>
</tr>
<tr>
<td>Flexural Strength (MPa)</td>
<td>ASTM D 790</td>
<td>83</td>
</tr>
<tr>
<td>Flexural Modulus (MPa)</td>
<td>ASTM D 790</td>
<td>3792</td>
</tr>
<tr>
<td>Mixed Viscosity (Pa·s)</td>
<td>ASTM D 2393</td>
<td>0.35</td>
</tr>
</tbody>
</table>

### Table 3.3: Typical properties for Target 1118\textsuperscript{TM} Grout (supplied by Target Products Ltd., Burnaby, BC).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water to cement ratio</td>
<td>365 mL/1 kg</td>
</tr>
<tr>
<td>Compressive Strength (MPa) min 28 day</td>
<td>48</td>
</tr>
</tbody>
</table>
Table 3.4: Details of slab strip testing program.

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimen Name&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Study Variable</th>
<th>Groove Width (mm)</th>
<th>Adhesive Type</th>
<th>Test Temp. (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C-RT-1</td>
<td>Control</td>
<td>--</td>
<td>--</td>
<td>21</td>
</tr>
<tr>
<td>2</td>
<td>C-RT-2</td>
<td>Control</td>
<td>--</td>
<td>--</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>E-6-RT-1</td>
<td>NSM</td>
<td>6.4</td>
<td>Epoxy</td>
<td>21</td>
</tr>
<tr>
<td>4</td>
<td>E-6-RT-2</td>
<td>NSM</td>
<td>6.4</td>
<td>Epoxy</td>
<td>21</td>
</tr>
<tr>
<td>5</td>
<td>E-3-RT-1</td>
<td>Groove width</td>
<td>3.2</td>
<td>Epoxy</td>
<td>21</td>
</tr>
<tr>
<td>6</td>
<td>E-3-RT-2</td>
<td>Groove width</td>
<td>3.2</td>
<td>Epoxy</td>
<td>21</td>
</tr>
<tr>
<td>7</td>
<td>G-6-RT-1</td>
<td>Adhesive type</td>
<td>6.4</td>
<td>Grout</td>
<td>21</td>
</tr>
<tr>
<td>8</td>
<td>G-6-RT-2</td>
<td>Adhesive type</td>
<td>6.4</td>
<td>Grout</td>
<td>21</td>
</tr>
<tr>
<td>9</td>
<td>C-LT-1</td>
<td>Low temp. control</td>
<td>--</td>
<td>--</td>
<td>-26</td>
</tr>
<tr>
<td>10</td>
<td>C-LT-2</td>
<td>Low temp. control</td>
<td>--</td>
<td>--</td>
<td>-26</td>
</tr>
<tr>
<td>11</td>
<td>E-6-LT-1</td>
<td>Low temp. NSM</td>
<td>6.4</td>
<td>Epoxy</td>
<td>-26</td>
</tr>
<tr>
<td>12</td>
<td>E-6-LT-2</td>
<td>Low temp. NSM</td>
<td>6.4</td>
<td>Epoxy</td>
<td>-26</td>
</tr>
<tr>
<td>13</td>
<td>E-3-LT-1</td>
<td>Groove width</td>
<td>3.2</td>
<td>Epoxy</td>
<td>-26</td>
</tr>
<tr>
<td>14</td>
<td>E-3-LT-2</td>
<td>Groove width</td>
<td>3.2</td>
<td>Epoxy</td>
<td>-26</td>
</tr>
<tr>
<td>15</td>
<td>G-6-LT-1</td>
<td>Adhesive type</td>
<td>6.4</td>
<td>Grout</td>
<td>-26</td>
</tr>
<tr>
<td>16</td>
<td>G-6-LT-2</td>
<td>Adhesive type</td>
<td>6.4</td>
<td>Grout</td>
<td>-26</td>
</tr>
<tr>
<td>17</td>
<td>E-6-HT-1</td>
<td>High temp. NSM</td>
<td>6.4</td>
<td>Epoxy</td>
<td>200&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>18</td>
<td>E-6-HT-2</td>
<td>High temp. NSM</td>
<td>6.4</td>
<td>Epoxy</td>
<td>200&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>19</td>
<td>G-6-HT-1</td>
<td>High temp. adhesive type</td>
<td>6.4</td>
<td>Grout</td>
<td>200&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>20</td>
<td>G-6-HT-2</td>
<td>High temp. adhesive type</td>
<td>6.4</td>
<td>Grout</td>
<td>200&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>21</td>
<td>E-6-HT-3</td>
<td>Testing error</td>
<td>6.4</td>
<td>Epoxy</td>
<td>200&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>22</td>
<td>E-6-FT-1</td>
<td>High Temp. NSM&lt;sup&gt;b&lt;/sup&gt;</td>
<td>6.4</td>
<td>Epoxy</td>
<td>100&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>23</td>
<td>G-6-FT-1</td>
<td>High Temp. NSM&lt;sup&gt;b&lt;/sup&gt;</td>
<td>6.4</td>
<td>Grout</td>
<td>100&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>a</sup>Adhesive-Groove size-Testing temperature-Test number: C – control, E – epoxy adhesive, G – grout adhesive, 6 – 6.4 mm groove width, 3 – 3.2 mm groove width, RT – room temperature, LT – low temperature, HT – 200°C high temperature, FT - 100°C high temperature

<sup>b</sup> see Section 3.6.3
Figure 3.1: Slab strip dimensions and reinforcement details a) elevation, b) section (all dimensions in mm).

Figure 3.2: Typical slab strip reinforcement and formwork prior to casting the concrete.
Figure 3.3: Concrete casting and vibration.

Figure 3.4: Tuckpointing grinder used to create grooves for insertion of NSM CFRP tape.
Figure 3.5: Aluminum guide used during NSM groove cutting to ensure a straight and uniform cut a) upside down beam with guide, b) end view of guide on beam.
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Figure 3.6: Groove cutting using tuckpoint grinder and aluminum guide.

Figure 3.7: NSM in groove with foam dams; a) top view, b) close-up view.
CHAPTER 3: Experimental Procedure

(a)

(b) Load Beam

(c) Roller Bearing Plate
Figure 3.8: Slab Strip Test Setup a) schematic, b) end view, c) load beam gap, d) side view (all dimensions in mm).

Figure 3.9: High temperature test setup.
Figure 3.10: Aslan 500™ tensile testing coupons; a) six glass tabbed tensile coupons, b) Instron tensile testing machine.
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Figure 3.11: High temperature verification thermocouple setup.
CHAPTER 4

TEST RESULTS AND DISCUSSION

4.1 General

The experimental results of both the slab strip and ancillary testing programs are presented in this chapter. Slab strip testing is divided into sections based upon the particular exposure conditions during testing (i.e., room temperature, low temperature, or high temperature), although some of the discussion covers test results across different exposure conditions for the purposes of comparison. Further discussion of the test results, in particular with respect to photogrammetry, predictive numerical modelling, and bond modelling is presented in Chapter 5.

4.2 Ancillary Testing

4.2.1 Concrete Cylinders

The results of fifteen concrete cylinder tests are presented in Table 4.1. Compressive strength is presented for 3 cylinders at both 7 days and 28 days from casting. The specified 28-day compressive strength of 35 MPa was achieved at 7 days with an average strength of 38.2 ± 0.5 MPa (where ± 0.5 MPa is the standard deviation). The compressive strength at 28 days was determined from testing to be 41.9 ± 1.7 MPa. Table 4.1 also shows the results for cylinders tested at the time of slab strip testing at both room and low temperatures.
Of the three cylinders tested at -26°C +/- 2°C, one was not included in the calculation of the compressive strength because its capping was damaged during handling leading to an unreliable compressive strength result. The remaining two cylinders tested to a compressive strength of 51.0 ± 0.8 MPa, as compared to 43.3 ± 0.9 MPa for three cylinders tested at the same age but at room temperature. This suggests an 18% increase in compressive strength for the concrete at low temperatures, an increase that is consistent with expected compressive strength gains as reported by Neville (1997).

4.2.2 FRP Coupons

The results of six uniaxial tensile coupon tests performed on samples of the Aslan 500™ CFRP tape used for NSM strengthening of the slabs are summarized in Table 4.2 and Figure 4.1. All coupons displayed linear stress strain behaviour until failure, as expected for unidirectional CFRP composites. During the first FRP coupon test the strain gauge stopped recording prior to failure, at 1.7% strain, due to a data acquisition error (the strain gauge range was not set high enough in the software, but was fixed for subsequent tests). One of the tests showed significantly different stress-strain behaviour than all other tests (refer to Figure 4.1), this was most likely due to premature debonding of the strain gauge attached to the CFRP strip.

Discounting the erroneous coupon’s results, the average tensile strength of the CFRP tape was found to be 2780 ± 140 MPa, while the average modulus of elasticity and average ultimate strains were found to be 141 ± 4 GPa and 0.020 ± 0.001 respectively. The ultimate tensile strength ($f_{ult}$) was calculated from the ultimate load recorded during testing divided by the manufacturer’s specified cross sectional area of 31.2 mm². The
modulus of elasticity \( (E_f) \) was calculated as the secant modulus between 1000 and 3000 microstrain according to ASTM D 3039/D 3039M-00 (ASTM 2005). However, rather than calculating strains based upon load-deflection behaviour from an extensometer, as prescribed by the standard, they have been taken directly from strain readings taken by the strain gauge attached to the CFRP strip. Equation 4.1 gives the calculation of modulus of elasticity of the FRP, \( E_f \).

\[
E_f = \frac{\sigma}{\varepsilon} = \frac{\sigma_{3000} - \sigma_{1000}}{\varepsilon_{3000} - \varepsilon_{1000}}
\]

Eq 4.1

where:

\( E_f \) = modulus of elasticity of the FRP
\( \sigma_{3000}, \sigma_{1000} \) = measured stress in the CFRP at 3000 and 1000 microstrain respectively
\( \varepsilon_{3000}, \varepsilon_{1000} \) = strain in CFRP strip measured by strain gauge at 3000 and 1000 microstrain respectively

The ultimate strain in the FRP \( (\varepsilon_{ult}) \) was calculated based on \( f_{ult} \) and \( E_f \) according to:

\[
\varepsilon_{ult} = \frac{f_{ult}}{E_f}
\]

Eq 4.2

Thus the ultimate strain was calculated, rather than measured from the strain gauge, because at higher strain levels localized fiber breakages close to the strain gauge caused errors in the strain data.

4.2.3 Reinforcing Steel

The D5 deformed wire used as steel reinforcement was taken from a coil used previously by Ranger (2007) during an unrelated research project. Therefore, the results of tensile tests performed on the reinforcing steel reported in Ranger’s thesis are reported herein, and testing on the steel was not performed by the Author of the current document.
Ranger’s results showed that the steel did not display a well defined yield point, which is common and expected for cold-deformed wire such as was used in this case. A 0.2% strain offset was used to determine the yield strength as 667 ± 12 MPa. The average ultimate tensile strength was found to be 717 ± 9 MPa, and the average modulus of elasticity was found to be 194 ± 16 GPa. The ultimate tensile strain was not recorded by Ranger (2007), but a value obtained on a similar D5 bar can be taken from Fitzwilliam (2006) as 1.9%. A summary of the D5 steel tensile tests reported by Ranger (2007) is presented in Table 4.3.

4.2.4 Mortar Cubes

Three mortar cubes made of Target 1118™ grout adhesive were tested for compressive strength at 71 days from the time of casting. The average compressive strength of the mortar cubes was 61.3 ± 5.5 MPa. The results of these three tests are given Table 4.4.

4.2.5 High Temperature Verification

Heating tests were performed to determine the temperature exposures experienced by the NSM strengthening systems at both the top and bottom of the NSM grooves during transient heating tests, as described in Chapter 3. Curves of time versus temperature for both epoxy and grout adhesives during high temperature verification tests are given in Figures 4.2 and 4.3, respectively. Thermocouple locations during these tests are shown in Figure 3.11.

In the case of the epoxy adhesive heating test there was, apparently, electronic interference between the heating blanket and groove Thermocouples 1 and 2, both of
which periodically recorded nonsensical temperature data at various times during the test. This caused the temperature reading of these thermocouples to vary randomly whenever electrical power was applied to the resistive silicone rubber heating blanket. Various attempts were made to remove the erroneous data points by studying the output data and removing obviously false readings; however the overall trends recorded by Thermocouples 1 and 2 still appear strange. It does appear, however, that groove Thermocouples 3 and 4 were unaffected by this phenomenon. In general the four surface thermocouples did not correlate to each other as well as expected primarily due to the nature of the heating blanket’s construction. The blanket is made up of a metal resistive heating element coiled within a silicone rubber blanket. Because of the slightly uneven heat distribution of the silicone, the proximity of the thermocouple to the metal coil affects the temperature reading. Also, in the case of surface thermocouple 4, its location close to the free end of the heating blanket and outside the bond terminating sawcut (refer to Figure 3.11) could have resulted in lower temperature readings compared to Thermocouples 2 and 3 which were closer to the middle of the blanket.

For the grout heating test, the groove thermocouples did not seem to be affected by the electronic interference experienced above. However, surface Thermocouple 4 did display an unexpected temperature drop after about 20 minutes of heating that cannot be rationalized at this time.

### 4.2.6 DSC

Differential scanning calorimetry (DSC) was performed on both the epoxy adhesive and samples of the CFRP tape to determine their glass transition temperatures. DSC analysis
was not able to capture the $T_g$ of the FRP strip, because the characteristic heat flow signature that indicates a polymer transition was not observed during testing. This could have been due to the high fiber volume fraction in the CFRP strip, thus limiting the amount of polymer resin present in the small samples ($\approx 8$ mg each) used to conduct DSC tests. It is also possible that sizing chemicals corrupted the DSC data. Consultation with the CFRP strip manufacturer indicated that the manufacturer has not been successful in using DSC to determine $T_g$ for the vinylester matrix resin either. Rather, the manufacturer uses a Heat Distortion Temperature (HDT) to characterize high temperature mechanical performance (the HDT supplied by the manufacturer was 118°C).

The average $T_g$ of the epoxy adhesive, based on 3 identical tests, was determined to be $51.1 \pm 0.7^\circ C$. Figure 4.4 shows a typical curve showing the results of a DSC testing run for the three epoxy samples tested, as well as the points and tangents used to determine $T_g$ in accordance with ASTM 1356-03.

### 4.3 Slab Strip Specimens

All slabs tested under constant actuator stroke rate (either at room or low temperature) exhibited similar load versus deflection and/or moment versus curvature behaviour throughout the various tests. In general, slabs experienced linear load versus deflection behaviour up until first cracking, followed by a sudden drop in load and subsequent nonlinear load deflection behaviour up to failure, with a gradual reduction in flexural stiffness. Beyond first cracking, the load versus deflection behaviour was typically characterized by a series of sudden drops in load coinciding with the formation of new cracks within the slab. All tests were continued until either (1) sudden catastrophic
failure occurred (typically as a result of bond failure coincident with shear failure or concrete crushing), or (2) in some cases load reductions and rapidly increasing deflections (typically as a result of bond failure or concrete crushing) signified failure.

Crack formation and development was similar for all strengthened slabs; first a flexural crack formed at midspan within the notch, flexural cracks then formed at the internal supports (i.e., effectively the load points), and crack propagation progressed toward the compression fibre, with additional flexural cracks forming between the internal supports and midspan, and with flexural-shear cracks forming outside the internal supports. In all cases, member failures were initiated at cracks located close to one of the internal supports. The precise location of the critical crack that precipitated failure was variable from slab to slab, in some cases being located just within the constant moment region, and in some cases just outside the constant moment region. In general, strengthened slabs showed more numerous cracks than the unstrengthened controls, typical of FRP strengthened concrete slabs. Five distinct failure modes were observed in the strengthened beams, and their relative occurrence appeared to be somewhat dependent on the testing conditions (i.e., room, low, or high temperature) and NSM strengthening configuration (i.e., 6.4 mm groove width with epoxy adhesive, 3.2 mm groove width with epoxy adhesive, or 6.4 mm groove width with grout adhesive). The five failure modes observed were:

1. debonding of the CFRP strip by splitting in the adhesive and concrete;
2. debonding of the CFRP strip at the adhesive-strip interface;
3. coincident shear failure/debonding failure initiated at a large shear crack near the load point;
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4. debonding caused by NSM pullout with relative displacement at the adhesive-concrete interface; and

5. debonding caused by NSM pullout with relative displacement at the adhesive-strip interface.

The first three failure modes all occurred during either room or low temperature testing. The last two failure modes, namely the pullout failures, occurred during high temperature testing and were associated with a breakdown of the mechanical properties of the strengthening system (specifically the matrix or adhesive polymers) at high temperature; these are discussed in detail in Section 4.3.3.

Debonding of the strip caused by splitting in the adhesive and/or concrete adjacent to the NSM groove occurred in both 6.4 mm and 3.2 mm groove width epoxy adhesive tests at both room and low temperatures. This failure mode was characterized by crack formation in both the adhesive layer and the surrounding concrete, as shown in Figure 4.5.

Shear failure coincident with debonding failure occurred in 6.4 mm epoxy tests at room and low temperature and also in 3.2 mm low temperature epoxy tests. This failure mode was characterized by large flexural shear cracks that formed over one of the interior supports. This crack opened very wide at later stages in the tests and failure was sudden and catastrophic. Further discussion on the observed shear failures can be found in Section 4.3.4.
The grout adhesive changes the failure mode to debonding of the strip at the adhesive-strip interface. This occurred in all 6.4 mm groove width grout adhesive strengthened slab strips at both room and low temperatures. In this failure mode, very little cracking occurred in the adhesive layer (typically only one longitudinal crack along the centerline of the adhesive, directly above the NSM strip). In these cases, failure occurred when the bond between the FRP strip and the surrounding adhesive was lost. Post failure inspection of the specimens revealed that small amounts of the grout adhesive were adhered along portions of the failed bond plane, while some sections of the FRP strip had no grout adhesive remaining attached after failure.

### 4.3.1 Room Temperature Slab Strip Tests

Table 4.5 summarizes the results of all room temperature slab strip specimen tests, and Figures 4.6 and 4.7 show the load versus deflection and moment curvature behaviour respectively for all slab strip tests performed under room temperature conditions. All NSM FRP strengthened slab strips showed drastic strength increases compared to the unstrengthened controls. It can also be seen that all strengthened beams exhibited considerably stiffer post-cracking behaviour as compared against the controls (i.e., the unstrengthened slab strips). The 6.4 mm groove width epoxy adhesive slabs (E-6-RT-2 and E-6-RT-1) showed the highest strength increases, with ultimate loads of 33.9 kN and 34.5 kN, respectively. The 3.2 mm groove width epoxy adhesive slabs (E-3-RT-2 and E-3-RT-1) showed similar strength increases, with ultimate loads of 31.3 kN and 33.4 kN, respectively. The 6.4 mm groove width grout adhesive slabs (G-6-RT-2 and G-6-RT-1) showed slightly lower ultimate strength increases than the epoxy adhesive slab strips,
24.6 kN and 27.1 kN, respectively, although this still corresponded to a strength gain of over 100% as compared to the unstrengthened control specimens (C-RT-1 and C-RT-2), which had ultimate strengths of 12.7 kN and 11.4 kN, respectively. The lower strength of the grout adhesive NSM FRP strengthened slabs was thought to have resulted from the lower tensile and shear strength of the grout adhesive as compared to the epoxy adhesive. This lower tensile strength changed the failure mode from bond failure due to epoxy/concrete splitting to a pullout type failure due to bond failure by slip at the grout-strip interface (as discussed earlier in Section 4.3), resulting in lower ultimate member strength. The moment versus midspan FRP strain plots (Figure 4.8) show that all NSM FRP strengthened slab strips tested at room temperature displayed similar behaviour until failure. This suggests full composite action with no slipping until failure for all slab configurations.

Figures 4.6 through 4.8 all show that Slab Strip E-6-RT-2 did not demonstrate the typical first cracking behaviour characterized by the other tests, which showed a clear change in slope at cracking. This was due to accidental cracking of the slab strip during handling (but after NSM FRP strengthening), so that the first crack was already present at the time of testing. Given that the cracking load for these specimens was very low as compared with their ultimate loads (< 15% of the ultimate load in all cases), this difference in behaviour was not considered significant for examining ultimate behaviour.

### 4.3.2 Low Temperature Slab Strip Tests

Table 4.6 summarizes the results of all low temperature slab strip tests. Similar to the room temperature tests, all strengthened beams showed drastically higher ultimate loads
than the unstrengthened controls. The load versus deflection, moment versus curvature, and moment versus FRP strain plots for all low temperature specimens are shown in Figures 4.9, 4.10 and 4.11, respectively. Figure 4.9 does not show the load-deflection behaviour of the low temperature controls (C-LT-1 and C-LT-2) because of instrumentation errors in the low temperature environment (the 3 LP’s froze and did not register deflections). Slab strip E-3-LT-1 showed the same lack of first cracking behaviour as specimen E-6-RT-2 discussed previously, this behaviour was again due to a crack being inadvertently caused during handling.

For the low temperature tests, the 6.4 mm and 3.2 mm groove width epoxy adhesive (E-6-LT and E-3-LT) strengthened slab strips showed similar strengths, failing at 33.8 and 34.6 kN, and 34.8 and 36.7 kN, respectively. Also, a shear induced failure mode (described in detail in Section 4.3.4) was observed in one of the 3.2 mm groove width epoxy adhesive low temperature slabs (E-3-LT-2); a failure mode that did not occur for 3.2 mm groove width epoxy adhesive specimens in the room temperature tests (E-3-RT-1 and E-3-RT-2). Similar to the room temperature tests, at low temperature the grout adhesive strengthened slabs (G-6-LT-1 and G-6-LT-2) provided the lowest level of strengthening, with ultimate failure loads of 28.3 kN and 32.8 kN, respectively, compared to the unstrengthened controls (C-LT-1 and C-LT-2) with ultimate strengths of 13.5 kN and 13.9 kN, respectively. In general, the shape of the load deflection plots at lower load levels was consistent for all low temperature slab strip tests.
Figure 4.10 shows the moment-curvature behaviour of all of the low temperature slab strip specimens. It can been seen that all strengthened slabs show similar behaviour up to failure, exhibiting much stiffer post cracking behaviour when compared to the unstrengthened controls.

Figure 4.11, which shows the applied moment versus the FRP strain at midspan, displays similar behaviour to the room temperature slab strip results in that the behaviour of all specimens was similar and full composite action was exhibited. However, Specimen E-3-LT-2 displayed a sudden drop (of 0.04% strain) in FRP strain at 95% of the ultimate moment, which could have resulted from slip of the FRP strip inside the groove and could have signified a lack of composite action at loads very close to ultimate. Specimen E-6-LT-1 also showed a small sudden drop (of 0.01% strain) at 7 kN·m, however given the smaller nature of this drop, it is not possible to draw conclusions on its cause.

All low temperature slab strips, except the 6.4 mm groove width epoxy adhesive specimens (E-6-LT), had higher ultimate loads than the equivalent slab strips tested at room temperature; a bar chart showing failure loads is presented in Figure 4.12. The 6.4 mm groove width epoxy adhesive slab strips tested to similar ultimate strengths at both room and low temperatures. The similarity in strength for these slab strips was probably due to the shear induced failure mode which was observed for these specimens, as discussed later in Section 4.3.3, although an increase in shear strength would be expected at low temperature due to the higher compressive strength. The increased strength of the NSM FRP strengthened slab strips tested at low temperature was consistent with testing
presented by other researchers for externally bonded FRP strengthening systems using epoxy adhesives (Green 2007) and was presumed to be due to the higher concrete strength (18% in this case) realized at low temperature. No previous test data were apparently available for the performance of grout adhered FRP strengthening systems, so the data presented herein (also showing a strength increase at low temperature) are the first of their kind. In the case of specimens failing by concrete splitting bond failure, the increased concrete tensile strength at low temperature delayed the onset of the concrete splitting failure mode. It also seems reasonable to assume that, given the cementitious nature of the grout adhesive, the grout would experience similar strength gains to concrete at low temperature. This increased tensile strength would delay debonding between the grout-strip interfaces, explaining the further strength increases observed for the grout adhesive slabs at low temperatures.

Figure 4.13 shows the strength gain for the NSM FRP strengthen slab strips as compared against the control specimens for both room and low temperature tests. This figure shows that the 6.4 mm groove width epoxy adhesive slab strips displayed lower strength gains at low temperature when compared against their respective control specimens. Figure 4.13 also shows that both the 3.2 mm groove width epoxy adhesive and 6.4 mm groove width epoxy adhesive slabs tested at low temperature displayed similar strength gains compared to their equivalent room temperature tests.

Figure 4.14 shows ultimate measured FRP strains for room and low temperature strengthened specimens. The 6.4 mm epoxy at low temperature (E-6-LT) show lower
FRP strains at failure than the room temperature tests (E-6-RT). However, 3.2 mm epoxy at low temperature (E-3-LT) show higher ultimate FRP strains than the room temperature tests (E-3-RT). The 6.4 mm grout slabs show similar FRP strain at both low and room temperature. Given the variability of the tested results and the magnitude of these differences it is difficult to draw meaningful conclusions based on these test results.

### 4.3.3 High Temperature Slab Strip Tests

Figure 4.15 shows the total applied load versus time of heating curve for all slab strips tested at a hold temperature of 200°C (as described in Section 3.6.3). This figure clearly shows the unheated loading phase at a load rate 2 kN/min, followed by the constant sustained load that was maintained at 20 kN until failure. Isolated, short lived and sudden fluctuations in the applied load can be seen in this figure and are due to control errors in the hydraulic system induced by localized electromagnetic fields and voltage fluctuations during testing. These represented load fluctuations of less than 2.5 kN (i.e., less than 15% of the total applied load and were generally less than 5 seconds in duration) and as such they can reasonably be assumed not to have affected the test data. Sudden reductions in load near the end of the individual traces clearly indicate failure, which was sudden in all cases with only one or two seconds warning indicated by rapidly increasing deflections.

Results from the high temperature tests are summarized in Table 4.7. Heating time to failure is given with corresponding average surface temperatures and FRP strip temperatures within the midspan notch. With 20 kN sustained load during heating, the grout adhesive slab strips (G-6-HT-1 and G-6-HT-2) survived much longer than the
epoxy adhesive strengthened slab strips (E-6-HT-1 and E-6-HT-2), by a factor of about seven for a given temperature exposure. Indeed, the grout adhesive slab strip heated to 100°C (EG-6-FT-1) did not fail in the full 300 minutes (5 hours) of exposure, and it was subsequently, loaded while heated, at a crosshead stroke rate of 1 mm/min until failure at 27.3 kN compared to 24.6 kN and 27.1 kN for room temperature tests G-6-RT-1 and G-6-RT-2 respectively. Recorded surface temperatures vary greatly from thermocouple to thermocouple due to the construction of the blanket as described above in Section 4.2.5.

Failures of epoxy adhesive strengthened slab strips occurred by NSM FRP pullout at the concrete-epoxy interface along a smooth, clean failure surface, as shown in Figure 4.16. This failure was assumed to be due to the strength and stiffness loss in the epoxy adhesive at temperatures nearing its $T_g$. Cementitious grout adhesive slab strips failed by bond failure at the strip-grout interface, again along a smooth, clean failure surface. This failure was also likely caused by loss of strength and stiffness in the polymer matrix resin, used to produce the preformed CFRP strips, at temperatures approaching its $T_g$ (which was presumably considerably higher for the FRP matrix resin than it is for the epoxy adhesive), as opposed to the adhesive used for strengthening as was the case for the epoxy adhesive slabs.

For slab strip testing at a hold temperature of 200°C with 20 kN sustained load, failure in the epoxy adhesive slab strips occurred at 10:43 (min:sec) and 11:41 for Specimens E-6-HT-1 and E-6-HT-2, respectively. For grout adhesive slab strips E-6-HT-1 and E-6-HT-2, failures occurred after 72:59 and 76:00, respectively. FRP strain versus time of
heating and midspan deflection versus time of heating plots, shown in Figures 4.17 and 4.18, demonstrate that the failure in the epoxy adhesive slab strips (E-6-HT-1 and E-6-HT-2) occurred much earlier, with a gradual but increasing loss of NSM FRP bond and eventually complete reduction in FRP strain and increases in deflection near failure (in the structural fire engineering community this behaviour is typically referred to as “runaway” failure). The failure of the grout adhesive slab strips (G-6-HT-1 and G-6-HT-2) occurred much later in time, as previously noted. In this case, a gradual increase in deflection and trend of decreasing FRP strain over the duration of the heating (thought to be due to thermal bowing induced by heating at the tension face of the beam, as is commonly observed in standard fire tests with heating from below). The eventual failure was marked by a rapid increase in deflection similar to the epoxy adhesive slabs, but the drop in FRP strain was much more rapid than that of the epoxy slabs. The grout adhesive slab strips held the load on average 564% longer than the epoxy adhesive slab strips when heated to a hold temperature of 200°C. This was thought to be due to the lower glass transition temperature of the epoxy adhesive as compared to the vinylester matrix resin used to manufacture the CFRP strips, which would indeed explain the superior performance of the grout adhesive beams as well as the observed change in the location of the pullout failure plane. It is believed that the grout also has an insulating effect in the case of the grout adhesive slab strips, thus delaying the temperature increase in the strip, whereas in the case of the epoxy adhesive the heat begins its deteriorating effects immediately upon heating of the slab strips’ surface.
It should be noted that under sustained load the deflection gradually increased throughout the test as can been seen in Figures 4.18 and 4.21. This small increasing deflection was due primarily to two phenomena. First, as the adhesive heated there was a gradual loss of bond strength. This loss of strength was not uniform, as temperatures closer to the slab surface were higher than temperatures deeper within the groove. Second, thermal bowing also increased the deflection as the surface temperatures rose throughout the test. Thermal bowing occurs when one surface of a slab is at a higher temperature than the opposite surface. This causes the heated side to expand, increasing the curvature and deflection. In this case the heating blanket heated one surface when the opposite face of the slab was still cool, causing thermal bowing.

When the actual observed failure times (and surface temperatures) of the slab strips are compared against the verification heating tests discussed previously, and shown in Figures 4.2 and 4.3, approximate temperatures at the bottom of the groove were determined. The results of these approximate comparisons are summarized in Table 4.8. By comparison with the heating verification tests, the approximate temperature at the bottom of the groove at failure for the epoxy and grout adhesive slab strips heated to a hold temperature of 200°C were 54°C and 105°C, respectively. The average $T_g$ for the epoxy adhesive, determined through DSC as 51°C, was very close to the 54°C expected at the bottom of the groove at NSM pullout failure. This similarity in temperatures suggests that, essentially, all of the epoxy adhesive is close to the resin’s $T_g$ at time that failure occurred. This also suggests that there is some residual strength in the epoxy that is sufficient to maintain bond at temperatures close to the $T_g$ under the service load.
applied in the tests described herein. Indeed, it is not known what adhesive/matrix
strength is actually required to maintain sufficient bond properties to prevent pullout
failure under a given load, so it is likely that bond would be maintained (up to some limit)
at temperatures above $T_g$ for NSM materials subjected to service bond stresses only.
Although the $T_g$ of the Aslan 500 tape or its constituent resin matrix is not known, the
HDT of the resin used is 118°C. This is slightly higher than the 105°C approximate
temperature at the bottom of the grout groove. As with $T_g$, it is not known how bond
strength loss correlates to HDT, but the 118°C is similar to the 105°C experienced at the
base of the groove at failure.

One of the epoxy adhesive strengthened slabs (E-6-HT-3) tested at a hold temperature of
200°C was not sawcut across the bond in the same manner as the other high temperature
slab strips, as described previously in Section 3.4.3. Specimen E-6-HT-3 was tested prior
to E-6-HT-1 and E-6-HT-2 (indeed it was the first high temperature test performed for
the current research study). At the time of the first test, it was inadvertently overlooked
that, due to the size of the available silicone rubber heating pad that was used to apply
heat to the specimens, a small length of the NSM strip at the free end of the bonded
length (about 75 mm) protruded beyond the end of the heating pad. Thus, a portion of
the bonded length was not subjected to the same heating exposure as the remainder of the
NSM bond length. This resulted in a much longer ($\approx 6.5$ times longer) duration of heating
before pullout failure of the NSM FRP strip. In hindsight it is easy to see that for this
slab the bond had likely failed up to the end of the heated section early in the heating
exposure, but the bond was then held in place under service stress levels by the
approximately 75 mm of unheated adhesive. After a considerably longer heating time, the remaining unheated portion eventually did heat sufficiently to result in bond failure. While this test did not perform as designed, it does provide some interesting insights into the performance of NSM FRP bond.

Figure 4.19 shows a plot of the FRP strain at midspan versus time of heating for the uncut epoxy adhesive slab strip (E-6-HT-3) and the 2 other epoxy adhesive slab strips (E-6-HT-1 and E-6-HT-2) tested at 200°C under sustained load. It can be seen that the uncut slab strip performed similarly up to the time of failure of the other two slab strips. Indeed, a decrease in the FRP strain during the initial 10 minutes of heating was observed for the uncut slab strip, similar to the other two slab strips, but rather than failing, the strain in the uncut slab strip continued to decrease and then leveled off to an effectively constant value up until failure at more than 70 minutes of heating exposure. These results suggest that even very short bonded lengths anchored in low moment regions are capable of sustaining relatively high strains (greater than 0.4%) in the FRP strengthening. This result is significant in that it indicates that one option for providing fire protection for NSM FRP strengthening systems for reinforced concrete members could be to provide localized fire insulation and protect isolated anchorage regions only, thus resulting in an unbonded FRP strengthening system which might still remain structurally effective during a fire event. Clearly, a great deal of additional research, including full scale fire tests for validation purposes, is required before this hypothesis can be implemented in practice.
Not surprisingly, for the two heating tests performed up to hold temperatures of 100°C, both the epoxy and grout adhesives remained effective for longer than at 200°C. The epoxy adhesive slab strip (E-6-FT-1) lasted 43:40 (min:sec) before displaying similar failure behaviour as the slabs at 200°C. The midspan deflection versus time of heating exposure and midspan FRP strain versus time of heating exposure behaviour of this specimen are given in Figures 4.21 and 4.22, respectively. The grout adhesive slab strip (G-6-FT-1) had not failed after 300 min (5 hours) of heating. Since this was determined to be the upper limit for any reasonable standard fire test (i.e., structural members are rarely required to resist a standard fire for more than four hours), the load was increased until failure with the temperature held constant at 100°C. The slab eventually failed at 27.3 kN by bond failure at the FRP strip-grout interface; the same failure mode as was observed in both room temperature tests on the grout adhesive strengthening system (for which the failure loads were 24.6 kN and 27.1 kN). The similar failure load compared to room temperature testing suggests that the heating had essentially no detrimental effect on the strength of the member. Given that the 100°C temperature exposure was chosen based on the results of measured surface temperatures for FRP strengthened full-scale reinforced concrete members recorded during standard fire tests with supplementary fire insulation applied to the exterior surface of the FRP strengthening system, the current results compellingly suggest that an NSM system adhered with grout adhesive and provided with sufficient amounts of fire insulation could possibly remain structurally effective in a standard fire for up to 5 hours or more. The results also suggest that an epoxy adhesive NSM system with appropriate fire insulation could remain structurally effective in a standard fire test for up to approximately 43 min. The reader will note that
these results are preliminary and approximate, but they do show the tremendous promise of grout adhesive NSM strengthening systems in high temperature applications, or those applications in which structural effectiveness during fire is required.

4.3.4 Shear Failures: Prediction and Consequences

Given the unexpected nature of the shear failures that occurred during testing of some of the slab strip specimens (i.e., these shear failures were not expected on the basis of available design requirements for NSM FRP strengthened reinforced concrete slabs (ACI 2007)) an attempt was made to explain and better understand this behaviour. Calculations performed during the design of the testing specimen, performed according to the simplified method of CSA A23.3-04 (CSA 2004) and neglecting any contribution from the FRP strengthening system through dowel action, showed that the nominal shear resistance of the section was 26.4 kN (refer to Appendix B) which would result in a failure load for shear of 52.8 kN in the test setup used herein. Clearly, this was well above the maximum expected nominal flexural failure load of 34.8 kN (indeed the maximum load achieved for any slab strip during testing was 36.7 kN). Given these calculations it was concluded that no shear reinforcement was necessary in order to prevent shear failure prior to flexural failure. However, during testing of 6.4 mm groove width epoxy adhesive slabs at room temperature (E-6-RT-1) and 6.4 mm and 3.2 mm groove width epoxy adhesive slabs at low temperature (E-6-LT-1, E-6-LT-2 and E-3-LT-2), failures occurred, which appeared to be caused by FRP rupture coincident with global shear failure at locations of large shear cracks near the load points. Due to the violent nature of these failures, it was not possible to determine the exact sequence of events involved in the failure (i.e., whether shear failure or FRP debonding/rupture initiated the
failure). It was clear, however, that a large shear crack formed and that following the catastrophic failure the bond of the NSM FRP strengthening system had failed with debonding initiated at the location of the shear crack. It is not possible to determine if a shear failure caused the bond failure or if the bond failure caused the shear failure.

Available literature does indicate that premature shear failures have been observed in flexural tests using internal FRP reinforcement, although this problem appears not to have received much attention with respect to FRP strengthening of reinforced concrete slabs, a popular application. No available literature indicates the occurrence of premature shear failure for NSM FRP strengthened slabs. ACI 440.1R-06, a design document entitled “Guide for the design and construction of structural concrete reinforced with FRP bars”, notes that, due to the lower axial stiffness of FRPs as compared to steel reinforcement, there is typically a smaller depth to the neutral axis, and therefore the compression region of the cross section is reduced, consequently reducing the area of concrete available to resist shear (ACI 2006). This also causes larger crack widths, providing lower amounts of aggregate interlock. ACI (2006) notes available research which indicates that, in flexural members without shear reinforcement, the shear strength of the cross section is influenced by the stiffness of the tensile reinforcement. ACI (2006) also notes that dowel action of internal FRP reinforcement is assumed to be smaller than steel due to the much lower strength and stiffness of FRP bars in the transverse direction. According to the provisions of ACI 440.1R-06 (ACI 2006) for the shear strength of an FRP reinforced concrete cross section without internal shear reinforcement, the nominal shear resistance provided by the concrete would result in a
shear failure at a load of 21.6 kN for the slab strips tested herein (assuming here that the NSM FRP reinforcement may be treated in the same manner as internal FRP reinforcing bars), much lower than the 52.8 kN nominal shear resistance predicted according to CSA A23.3-04 (CSA 2004) (or the 42.8 kN found according to ACI 318-05 (ACI 2005)).

Given the relatively small amount of primary flexural steel reinforcement present in the slab strips, the lower axial stiffness of the Aslan 500™ CFRP reinforcement as compared with steel ($E_f = 141$ GPa) may have resulted in the lower than expected shear failures. As well, because of the unreasonably high levels of flexural NSM reinforcement present in the test specimens, very large cracks and curvatures were encountered during testing. These large cracks and curvatures limit the applicability of typical shear design procedures. It may also be true that these failures resulted from bond failures propagated from the large shear crack present during testing. However, given the data from internal FRP reinforcement and the similarities between NSM and internal FRP reinforcement, the phenomena of premature shear failures in NSM FRP reinforcement merits further research.

### 4.4 Effect of Adhesive Type

Figure 4.23 shows the load deflection behaviour for all 6.4 mm groove width slabs. It can be seen that the epoxy adhesive slabs had higher failure loads than the grout adhesive slabs, as previously discussed. It can also be seen that the grout adhesive slabs have lower post cracking stiffness compared to the epoxy adhesive slabs, potentially due to microcracking in the grout adhesive affecting bond performance.
4.5 Effect of Groove Width

It can be seen in Figure 4.24 that changing the groove width from 3.2 mm to 6.4 mm does not have a measurable effect on the performance of the strengthened system. Slabs performed similarly with both groove sizes. This similarity in strength brings into question the groove width requirements presented in ACI 440.2R-07.

4.6 Effect of Low Temperature

Figures 4.25, 4.26 and 4.27 show comparisons of load deflection behaviour at room and low temperatures for E-6, E-3 and G-6 specimens, respectively. It can be seen that in all cases low temperature specimens had similar or higher ultimate strengths than room temperature specimens. This suggests that there was no detrimental effect from the low temperature exposure. The higher strength of low temperature tested specimens is due to the increased concrete strength at low temperature. Also low temperature specimens show stiffer post crack behaviour, also due to the higher concrete strength.

4.7 Summary

In this chapter the results of 23 slab strip tests as well as various ancillary tests have been reported in detail. The results of the effect of groove size (6.4 mm or 3.2 mm), temperature (20°C, -26°C, 100°C or 200°C) and adhesive type (grout or epoxy) have been shown. All strengthened slab strips showed large strength gains over unstrengthened controls. Varying the groove size from 3.2 mm to 6.4 mm showed no discernable effect on strength. Epoxy adhesive slabs outperformed grout adhesives at both room and low temperature. However, grout slabs outperformed epoxy adhesives at
high temperatures by maintaining the strengthening system for much longer while exposed to heating under sustained service loads.
### Table 4.1: Results of ancillary tests on concrete cylinders.

<table>
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<th>Temperature</th>
<th>Test Number</th>
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<th>Average $f'_{c}$ (MPa)</th>
<th>Standard Deviation (MPa)</th>
</tr>
</thead>
<tbody>
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<td>7 day</td>
<td>Room</td>
<td>1</td>
<td>38.1</td>
<td>38.2</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>37.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>38.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>29 day</td>
<td>Room</td>
<td>1</td>
<td>43.3</td>
<td>41.9</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>40.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>42.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>154 days</td>
<td>Room</td>
<td>1</td>
<td>44.4</td>
<td>46.3</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>47.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>46.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>217 days</td>
<td>Low</td>
<td>1</td>
<td>N/A*</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>50.4</td>
<td>51.0</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>51.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>218 days</td>
<td>Room</td>
<td>1</td>
<td>42.3</td>
<td>43.3</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>44.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>43.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Test 1 at 217 days was not used as there was a capping failure and compressive strength test results were unreliable.

### Table 4.2: Results of ancillary unidirectional tensile coupon tests on Aslan 500™ CFRP tape.

<table>
<thead>
<tr>
<th>Test</th>
<th>$f_{ult}$ (MPa)</th>
<th>$E_f$ (GPa)</th>
<th>$\varepsilon_{ult}$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2524</td>
<td>140</td>
<td>0.018</td>
</tr>
<tr>
<td>2</td>
<td>2729</td>
<td>150</td>
<td>0.018</td>
</tr>
<tr>
<td>3</td>
<td>2876</td>
<td>139</td>
<td>0.021</td>
</tr>
<tr>
<td>4</td>
<td>2906</td>
<td>139</td>
<td>0.021</td>
</tr>
<tr>
<td>5</td>
<td>2780</td>
<td>140</td>
<td>0.020</td>
</tr>
<tr>
<td>6</td>
<td>2838</td>
<td>141</td>
<td>0.020</td>
</tr>
<tr>
<td>Average</td>
<td>2776</td>
<td>141</td>
<td>0.020</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>139</td>
<td>4</td>
<td>0.001</td>
</tr>
<tr>
<td>Manufacturer Specified*</td>
<td>2068</td>
<td>124</td>
<td>0.017</td>
</tr>
</tbody>
</table>

* As quoted by Hughes Bros. Inc. (www.hughesbros.com)
Table 4.3: Mechanical properties of D5 steel reinforcement as reported by Ranger (2007).

<table>
<thead>
<tr>
<th>Rebar</th>
<th>Tensile Yield Strength (MPa)</th>
<th>Tensile Yield Strain (%)</th>
<th>Ultimate Tensile Strength (MPa)</th>
<th>Tensile Elastic Modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>666</td>
<td>0.541</td>
<td>719</td>
<td>198.9</td>
</tr>
<tr>
<td>2</td>
<td>674</td>
<td>0.561</td>
<td>724</td>
<td>171.6</td>
</tr>
<tr>
<td>3</td>
<td>662</td>
<td>0.596*</td>
<td>713</td>
<td>181.7*</td>
</tr>
<tr>
<td>4</td>
<td>681</td>
<td>0.571</td>
<td>726</td>
<td>196.7</td>
</tr>
<tr>
<td>5</td>
<td>650</td>
<td>0.540</td>
<td>705</td>
<td>208.4</td>
</tr>
<tr>
<td>Average</td>
<td>667</td>
<td>0.553</td>
<td>717</td>
<td>193.9</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>12</td>
<td>0.015</td>
<td>9</td>
<td>15.7</td>
</tr>
</tbody>
</table>

* Specimen R3 was not used in the calculation of average yield strain and elastic modulus

Table 4.4: Results of ancillary compressive strength mortar cube tests on Target 1118™ unsanded silica fume grout adhesive.

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Test Number</th>
<th>$f'_c$ (MPa)</th>
<th>Average $f'_c$ (MPa)</th>
<th>Standard Deviation (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>71 day</td>
<td>1</td>
<td>55.3</td>
<td>61.3</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>66.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>62.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.5: Selected results of room temperature slab strip testing.

<table>
<thead>
<tr>
<th>Test Setup</th>
<th>Failure Load (kN)</th>
<th>Failure Moment (kNm)</th>
<th>FRP Failure Strain</th>
<th>% Strength Increase</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C-RT-1</td>
<td>12.7</td>
<td>3.2</td>
<td>--</td>
<td>Concrete crushing (under-reinforced)</td>
</tr>
<tr>
<td>2</td>
<td>C-RT-2</td>
<td>11.4</td>
<td>2.9</td>
<td>--</td>
<td>Concrete crushing (under-reinforced)</td>
</tr>
<tr>
<td>3</td>
<td>E-6-RT-1</td>
<td>33.5</td>
<td>8.5</td>
<td>0.01264</td>
<td>178 Shear crack induced debonding/failure</td>
</tr>
<tr>
<td>4</td>
<td>E-6-RT-2</td>
<td>34.9</td>
<td>8.9</td>
<td>0.0141</td>
<td>190 FRP debonding - epoxy/concrete split</td>
</tr>
<tr>
<td>5</td>
<td>E-3-RT-1</td>
<td>31.3</td>
<td>8.0</td>
<td>0.01125</td>
<td>160 FRP debonding - epoxy/concrete split</td>
</tr>
<tr>
<td>6</td>
<td>E-3-RT-2</td>
<td>33.4</td>
<td>8.5</td>
<td>0.01214</td>
<td>177 FRP debonding - epoxy/concrete split</td>
</tr>
<tr>
<td>7</td>
<td>G-6-RT-1</td>
<td>24.6</td>
<td>6.3</td>
<td>0.00851</td>
<td>104 FRP debonding - grout-strip interface</td>
</tr>
<tr>
<td>8</td>
<td>G-6-RT-2</td>
<td>27.1</td>
<td>6.9</td>
<td>0.00944</td>
<td>125 FRP debonding - grout-strip interface</td>
</tr>
</tbody>
</table>
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Table 4.6: Selected results of low temperature slab strip testing.

<table>
<thead>
<tr>
<th>Test Setup (Average Temperature)</th>
<th>Failure Load (kN)</th>
<th>Failure Moment (kNm)</th>
<th>FRP Failure Strain</th>
<th>% Strength Increase</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>9 C-LT-1</td>
<td>13.5</td>
<td>3.4</td>
<td>--</td>
<td>--</td>
<td>Concrete crushing (under-reinforced)</td>
</tr>
<tr>
<td>10 C-LT-2</td>
<td>13.9</td>
<td>3.5</td>
<td>--</td>
<td>--</td>
<td>Concrete crushing (under-reinforced)</td>
</tr>
<tr>
<td>11 E-6-LT-1</td>
<td>33.8</td>
<td>8.6</td>
<td>0.01199</td>
<td>147</td>
<td>Shear crack induced debonding/failure</td>
</tr>
<tr>
<td>12 E-6-LT-2</td>
<td>34.6</td>
<td>8.8</td>
<td>0.01191</td>
<td>153</td>
<td>Shear crack induced debonding/failure</td>
</tr>
<tr>
<td>13 E-3-LT-1</td>
<td>34.8</td>
<td>8.8</td>
<td>0.0141</td>
<td>154</td>
<td>Debonding - epoxy/concrete split</td>
</tr>
<tr>
<td>14 E-3-LT-2</td>
<td>36.7</td>
<td>9.3</td>
<td>0.01352</td>
<td>168</td>
<td>Shear crack induced debonding/failure</td>
</tr>
<tr>
<td>15 G-6-LT-1</td>
<td>28.3</td>
<td>7.2</td>
<td>0.00892</td>
<td>107</td>
<td>Debonding - grout-strip interface</td>
</tr>
<tr>
<td>16 G-6-LT-2</td>
<td>32.8</td>
<td>8.3</td>
<td>0.01191</td>
<td>139</td>
<td>Debonding - grout-strip interface</td>
</tr>
</tbody>
</table>

Table 4.7: Selected results of high temperature slab strip testing.

<table>
<thead>
<tr>
<th>Test</th>
<th>Temperature Setting (°C)</th>
<th>Time Heated @ Failure (min:sec)</th>
<th>Ave. Surface Temp @ Failure (°C)</th>
<th>FRP Temp @ Failure (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-6-HT-1</td>
<td>200</td>
<td>10:43</td>
<td>185</td>
<td>42</td>
</tr>
<tr>
<td>E-6-HT-2</td>
<td>200</td>
<td>11:41</td>
<td>166</td>
<td>34</td>
</tr>
<tr>
<td>G-6-HT-1</td>
<td>200</td>
<td>72:59</td>
<td>179</td>
<td>69</td>
</tr>
<tr>
<td>G-6-HT-2</td>
<td>200</td>
<td>76:00</td>
<td>197</td>
<td>54</td>
</tr>
<tr>
<td>E-6-FT-1</td>
<td>100</td>
<td>43:40</td>
<td>94</td>
<td>35</td>
</tr>
<tr>
<td>G-6-FT-2</td>
<td>100</td>
<td>No Failure @ 300:00</td>
<td>91</td>
<td>48</td>
</tr>
</tbody>
</table>
Table 4.8: Temperature correlation comparing high temperature testing to temperature verification testing.

<table>
<thead>
<tr>
<th>Test</th>
<th>Failure Time (min:sec)</th>
<th>Ave. Surface Temp (°C)</th>
<th>Heating Verification Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-6-HT-1</td>
<td>10:43</td>
<td>185</td>
<td>166</td>
</tr>
<tr>
<td>E-6-HT-2</td>
<td>11:41</td>
<td>166</td>
<td>170</td>
</tr>
<tr>
<td>G-6-HT-1</td>
<td>72:59</td>
<td>179</td>
<td>204</td>
</tr>
<tr>
<td>G-6-HT-2</td>
<td>76:00</td>
<td>197</td>
<td>202</td>
</tr>
</tbody>
</table>
**Figure 4.1:** Stress versus strain plots for Aslan 500™ coupons tested in tension.

**Figure 4.2:** Time of heating (including ramp phase) versus temperature for the epoxy adhesive heating verification test to a hold temperature of 200°C.
Figure 4.3: Time of heating (including ramp phase) versus temperature for the grout adhesive heating verification test to a hold temperature of $200^\circ$C.

Figure 4.4: Typical heat flow curve for DSC testing of epoxy adhesive showing calculation of $T_g$. 
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Figure 4.5: Failure of the NSM FRP-concrete bond by splitting of the concrete and epoxy.

Figure 4.6: Total applied load versus midspan vertical deflection for all slab strips tested at room temperature.
Figure 4.7: Applied moment versus curvature at midspan for all slab strips tested at room temperature.

Figure 4.8: Applied moment versus FRP strain at midspan for all slab strips tested at room temperature.
Figure 4.9: Total applied load versus midspan deflection for all slab strips tested at low temperature (-26°C).

Figure 4.10: Applied moment versus midspan curvature for all slab strips tested at low temperature (-26°C).
Figure 4.11: Applied moment versus FRP strain at midspan for all slab strips tested at low temperature (-26°C).

Figure 4.12: Comparison of room and low temperature (-26°C) ultimate failure loads.
Figure 4.13: Comparison of room and low temperature (-26°C) strength gain.

Figure 4.14: Comparison of room and low temperature (-26°C) ultimate FRP strain.
Figure 4.15: Total applied load versus time of heating exposure for slab strips exposed to 200°C with 20 kN sustained load (heat was applied at time 0).

Figure 4.16: Epoxy pullout failure after heating to a hold temperature of 200°C under a sustained load of 20 kN.
Figure 4.17: FRP strain at midspan versus time of heating exposure for slab strips exposed to 200°C with 20 kN sustained load.

Figure 4.18: Midspan vertical deflection versus time of heating exposure for slab strips exposed to 200°C with 20 kN sustained load.
Figure 4.19: FRP strain at midspan versus time of heating exposure for all three 6.4 mm groove width epoxy adhesive slab strips exposed to 200°C with 20 kN sustained load.

Figure 4.20: Total applied load versus time of heating exposure for slab strips exposed to 100°C with 20 kN sustained load.
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Figure 4.21: Midspan vertical deflection versus time of heating exposure for slab strips exposed to 100°C with 20 kN sustained load.

Figure 4.22: FRP strain at midspan versus time of heating exposure for slab strips exposed to 100°C with 20 kN sustained load.
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Figure 4.23: Load deflection for all 6.4 mm groove width slabs.

Figure 4.24: Load deflection for all epoxy adhesive tests.
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Figure 4.25: Comparison of load deflection for room and low temperature E-6 specimens.

Figure 4.26: Comparison of load deflection for room and low temperature E-3 specimens.
Figure 4.27: Comparison of load deflection for room and low temperature G-6 specimens.
CHAPTER 5

ANALYSIS

This chapter presents three distinct sections, each one treating a different analytical topic relevant to the current thesis. First, Section 5.1 presents details of the digital image correlation technique used in an attempt to monitor bondline deformations in the NSM FRP strengthened slab strips, as well as to monitor flexural strain distributions (and hence curvatures) over the height of the slab strips’ cross sections. Section 5.2 briefly presents and discusses a selection of currently available models for the bond of NSM FRP strips in concrete and compares the predictions of these models against the results from the experimental tests presented herein. Finally, Section 5.3 presents and discusses details of a classical plane-sections analysis which has been used to predict the moment-curvature response of the respective slab strips tested herein.

5.1 Digital Image Correlation Analysis

Digital Image Correlation Analysis is a process in which high resolution digital photographs taken at regular time intervals throughout the testing of a particular specimen, can be used to track movements (i.e., displacements) of any point within the photos’ field of view. Using specialized software (GeoPIV) developed Dr. Andy Take (currently at Queen’s University) and Dr. Dave White at Cambridge University (White et al., 2003), displacements can be tracked at any location and time throughout the test. The GeoPIV software uses particle image velocimetry (PIV) to track patches of pixels from one photograph to the next. The technical details of this process are well beyond the
scope of the current thesis, but the interested reader can find a detailed explanation in (White et al., 2003).

The technique is implemented though a multiple step process. First, cameras are carefully placed so as to focus on the desired location of measurement. In the current thesis, these locations of interest were the sides of the slab strips (to capture flexural strains) and their top surfaces (i.e., their NSM strengthened tensile faces, in an attempt to study bondline deformations and cracking patterns). Care was taken to ensure the proper orientation and positioning of the cameras so as to match the planes of movement of the slabs with the x-y plane of the photographs (out of plane movements cause problems in processing of the image data). No flash was used during testing as it interferes with the analysis software. Therefore, strong lighting with halogen construction lights was provided to consistently light the frame. Also, the auto focus was turned off during the testing as the frame conditions must stay the same between subsequent photos to allow for comparison. The slab surfaces were painted with a random textured pattern to give the GeoPIV software patches of visual uniqueness to accurately track. The texturing was random and simple to apply, giving this method an advantage over many other available digital image analysis techniques which require surface grids, markers, or uniform speckle patterns. Once all of the pretest setup had been completed, the cameras were connected to a laptop computer and images were captured at 5 sec intervals throughout the testing of each specimen using remote capture software, DSLR Remote. The internal clocks of the laptop used to remotely acquire the images and the computer logging sensor
data from the data acquisition system (used for all other conventional instrumentation) were synchronized so that the photos could later be time-correlated to the sensor data.

After all of the testing had been completed, post processing was conducted using GeoPIV in conjunction with MatLAB 6.1 and Microsoft Excel. The basic analysis method involved applying a set of pixel “patches” (i.e., areas of digital image with dimensions of 32 × 32 pixels) at specified locations within the frame. The pixel patches represent the areas which the software tracks from image to image with respect to a known initial location. The GeoPIV software then tracked the movement of these pixel patches from photo to photo throughout the testing. By placing the pixel patches at predetermined locations, it is thus possible to track both displacements and strains throughout testing. The software outputs data in terms of \( x \) and \( y \) displacements of each pixel patch, from the first picture to the last. It is then possible to turn these displacements into strains (over some assumed gauge length) if desired.

### 5.1.1 Analysis of Flexural Strains and Slab Strip Curvatures

In this analysis method, images of the side face of the slab at midspan were captured during testing (the side opposite to where the Pi gauges were installed). In the post processing phase these images were used to create artificial (virtual or optical) Pi gauges applied to the sides of the slab strips at the same height as the real Pi gauges on the opposite side. These artificial Pi gauges consisted of applying pixel patches spaced at 100 mm apart and centered on the slab midspan, over the depth of the section, as shown in Figure 5.2. The 100 mm gauge length and the patch locations over the depth had to be determined in both the true slab coordinate system as well as the image correlation.
coordinate system. To accomplish this, a known dimension (i.e., a benchmark) was needed in the images at the same z-coordinate with respect to the camera. With something of known dimensions within the images, the size of this object could be determined in pixels and the ratio between real length (mm) and optical (virtual) length (pixels) could be used subsequently to track image patch locations based on real coordinates found from image coordinates. By placing artificial (virtual) Pi gauges at the same locations on the slab strips as the real Pi gauges on their opposite side of the slab strips, it was possible to correlate and compare the readings from each, thus allowing for validation of the optical strain measurement technique for measuring flexural strains in concrete beams (to the knowledge of the Author this is the first time that such a validation has ever been performed). Based on the displacements of the pixel patches, and subsequent calculation of strains found from image analysis, the curvature was calculated assuming linear strain distributions and similar triangles. Three curvature values were calculated by comparing the virtual Pi gauges at the top and the middle, the top and the bottom and the middle and the bottom. The overall curvature was taken as the mean of these three values. This curvature was then compared to the mean of the three curvatures calculated from the real Pi gauges in the same manner as above.

The image correlation software is computationally intensive and time consuming. Because images were taken every five seconds, and tests took around 15 minutes or more, the analysis period for this many individual pictures would have been excessively long. To condense analysis time, and because the analysis is not central to the conclusions of the current research project, not every image was used. Rather, only
selected images were processed for comparison against data obtained from the conventional instrumentation. Images were taken at known load levels (actually moments) incremented by 0.5 kN·m throughout the test, and plots were made with only these points. All plots that follow show the specific points chosen for analysis.

In general, the photo image results correlate well with data collected from the Pi gauges. There are a number of possible sources of error in determining curvature either from the images or from the Pi gauges so one should expect some variability in the compared results. Some of the possible sources of error include not precisely knowing the location of the Pi gauges due to placement error, imperfect alignment of the camera, non precise conversion between image coordinate and real coordinate systems, and other possible errors present in the photo analysis technique (see White et al., 2003).

Presented in Figure 5.3 is a plot showing moment versus midspan curvature for tests E-3-RT-2 and E-3-LT-2, with both Pi gauge and photo image analysis results for curvature shown. This plot clearly shows good overall agreement between the Pi gauge data and the image data, even for the limited number of readings considered from the photo data. Each point where a photo was analyzed is shown with a marker. In both cases, the photo imaging suggests a slightly larger curvature than the Pi gauge data. However, it is not possible to say with any certainty which is the more correct reading. It is also important to recognize that the image data were obtained from the opposite side of the slab strip than then Pi gauge data. Therefore, some differences between these data are to be
expected (different crack patterns may cause different calculated curvatures on opposite sides of the slab strips).

Appendix C provides detailed and complete comparisons of moment curvature data from both photo and Pi gauge results for all room and low temperature tests. These plots show good general agreement between the Pi gauges and the image correlation analysis determined moment-curvature responses.

In general, the image correlation technique implemented through use of the GeoPIV software shows great promise. Its use has been validated for the first time herein for measuring flexural strains and curvatures in reinforced concrete members in bending. Extreme care is required to prevent out of plane displacements of the surface being monitored, and higher resolution images are needed to enable accurate measurements of the very small strains and displacements experienced in these types of members under service loads. While useful in the current study to corroborate data obtained from conventional instrumentation, the image correlation analysis remains a sidebar discussion to the research presented herein, and it is not discussed further.

5.1.2 NSM Bond Line Analysis

An attempt was made to specifically study and analyze the behaviour of the NSM strengthening system bondline using digital image correlation analysis. It was envisioned that strain profiles along the length of the bond at various load levels could be produced, thus shedding light on bond development mechanism, bond failure mechanism, and load versus bond slip response of the specific NSM and adhesive systems tested herein. The
goal was to study strain distributions along the bond as the test progressed without interfering directly with the bond as is the case when using strain gauges as has been done previously by multiple researchers (Teng et al. 2006, Seracino et al. 2007). However, once post processing of these photo sets began, it quickly became clear that this approach was not possible with the test setup used. The major problem can be attributed to out of plane movements of the beam with respect to the stationary camera. The camera was mounted on the loading frame, looking down through the loading beam at the surface of the slab, with the NSM strengthening system centered in the frame, as shown in Figure 5.1. The envisioned analysis method involved tracking the relative displacements of points along the length of the NSM strengthening in the groove; any difference in the displacement giving a strain difference. It was also hoped that the image analysis could be used to monitor the load-slip response of the NSM strips at the faces of flexural cracks. However, the slab was not stationary in the $z$-plane (i.e., out of plane from the image taken by the camera). This out of plane movement, which was a consequence of the vertical movement of the slab strip resulting from its bending, generated artificial movements (artificial contractions when the slab moved away from the camera and dilations when it moved toward the camera) in the analysis which could not be rationally accounted for or removed in the post processing phase. Furthermore, this out of plane movement was non-uniform along the length of the beam; at the location of the internal support, the slab was stationary in the $z$-plane. However, between the internal support and the external support the slab was moving away from the camera, and between the internal support and midspan the slab was moving toward the camera. All of these complicating factors made it impossible to compare any two points along the slab’s
length with any real confidence, and it was thus impossible to track strains in the NSM strip or the strip’s load-slip response.

An attempt was also made to compare the displacement of points along lines perpendicular to the slab’s length (i.e., to obtain a rough picture of the relative displacement between the NSM strip and the concrete at predefined locations along the bond line). In the experimental setup used herein, out of plane movement of the slab should not have interfered with these comparisons as all the tracked points were experiencing the same amount of artificial contractions or dilations. However, during post processing it was determined that this technique was also not practical due to the formation of large cracks in the tensile surfaces of the slab strips. The software was unable to accurately track the movements throughout the test, thus yielding indecipherable results. Furthermore, relative displacements across the width of the slab strips’ tension faces were typically of such small magnitudes that the resolution of the images taken was not sufficiently high to yield meaningful results. After much effort, the NSM bond line digital image correlation analysis was abandoned. It is likely that future refinements to the digital image correlation technique will enable researchers to accomplish the above analysis goals, but these refinements are well beyond the scope of the current thesis.

5.2 Seracino Bond Model

The most relevant and easily applicable predictive models for the bond behaviour of NSM FRP strengthening systems are those proposed by Seracino et al. (2007a). This
model is explained in detail in Chapter 2, Section 2.1.6 of the current thesis. Equation 2.2 is a generic lower bound solution for the debonding strength of FRP plate strengthened concrete. A discussion of the assumptions inherent in Equation 2.2 will be presented in this section with a comparison against tested results. Recalling information presented in Chapter 2, the relevant equation presented by Seracino et al. (2007a), for predicting intermediate crack induced debonding failures of FRP strengthened members under flexural loads, is:

\[
P_{ic} = \alpha_p 0.85 f_y^{0.25} f_c^{0.33} \sqrt{L_{pov} (EA)}_p \begin{cases} 
    f_{rupt} A_p & \text{for FRP plates} \\
    f_{y} A_p & \text{for metallic plates}
\end{cases}
\]

Eq 2.2

By dividing the maximum load by the axial rigidity of the NSM FRP strips, the maximum allowable tensile strain in the NSM FRP strengthening can be predicted. The above equation relies on modelling the bond-slip relationship of the plate-to-concrete joint as a single linear softening branch. The failure plane shown in Figure 2.3 depicts the bond failure occurring 1 mm into the concrete surrounding the NSM groove. This 1 mm assumption was chosen as an approximate representative value after physically observing testing failures in bond pullout tests. However, it was determined by Seracino et al. (2007a) that the magnitude of this assumed value had little effect on the predicted bond strength using their equation. The derivation of Equation 2.2 also assumes that the axial rigidity of the concrete is very large relative to the axial rigidity of the FRP plate and is thus ignored in the bond slip behaviour. The bond strength is a function of the aspect ratio of the failure plane. This aspect ratio has been incorporated to take into account the additional confinement provided by concrete in NSM strengthening applications. The bond strength is also a function of the concrete strength as the failure forms once the “principal tensile stress reaches the cracking strength of concrete”
(Seracino et al. 2007a). The coefficients in the equation were found using a linear regression analysis of tests performed by Seracino.

From the current study, three different bond configurations exist: (1) 6.4 mm wide grooves with an epoxy adhesive, (2) 3.2 mm wide grooves with an epoxy adhesive, and (3) 6.4 mm wide grooves with a cementitious grout adhesive. Strictly speaking, the model does not directly apply to the situation of the grout adhesive, since it assumes the bond failure plane to be located 1 mm into the substrate concrete surrounding the epoxy-filled NSM groove. However, the grout slabs failed within the grout adhesive in all cases. To compensate for this, while still using the Seracino et al. (2007) model to predict the failure strain in the NSM FRP, the failure plane was taken as 1 mm into the grout adhesive from the strip-grout interface, rather than in the surrounding concrete. Thus, for the grout adhesive beams, the groove perimeter was effectively assumed to be slightly larger (i.e., 1 mm larger) than the strip perimeter. Finally, the concrete strength was varied according to the temperature conditions, as tested, at both low and room temperature. Calculations for each of the tested systems and conditions are included below.

Using Eq 2.2 for a 6.4 mm wide groove and an epoxy adhesive, the maximum allowable FRP strain at both room temperature (20°C) and low temperature (-26°C) can be determined as follows:

**Material Properties:**

- Concrete strength at 20°C, \( f'_{c, \text{room}} \) = 46.3 MPa
• Concrete strength at -26°C, \( f_{c', low} = 51.0 \text{ MPa} \)

• Modulus of elasticity of the FRP strip, \( E_p = 141000 \text{ MPa} \)

• Cross-sectional area of the FRP strip, \( A_p = 31.2 \text{ mm}^2 \)

• Rupture strength of the FRP strip, \( f_{\text{rupt}} = 2776 \text{ MPa} \)

**Assumed Parameters:**

• Assumed depth to failure plane in concrete, \( t_b = t_d = 1 \text{ mm} \)

• Bond parameter, \( \alpha_p = 1.0 \) (for mean value)

**Groove Parameter Calculations:**

• Depth of failure plane (refer to Figure 2.3):

\[
d_f = \text{groove depth} + t_d = 22.2 + 1 = 23.2 \text{ mm}
\]

• Width of failure plane (refer to Figure 2.3):

\[
b_f = \text{groove width} + t_b = 6.4 + 2(1) = 7.4 \text{ mm}
\]

• Length of the failure perimeter:

\[
L_{per} = 2d_f + b_f = 2(23.2) + 8.4 = 54.8 \text{ mm}
\]

• Aspect ratio of failure perimeter:

\[
\phi_f = \frac{d_f}{b_f} = \frac{23.2}{8.4} = 2.8
\]

• Room temperature debonding resistance:

\[
P_{IC} = 0.85(2.8)^{0.25}(46.3)^{0.33}\sqrt{54.8(141000)(31.2)} < 2776(31.2)
\]

\[
P_{IC} = 60.4 \text{ kN} < 86.6 \text{ kN}
\]

• Room temperature debonding strain:

\[
\varepsilon_{IC} = \frac{P_{IC}}{(EA)_p} = \frac{60400}{141000(31.2)} = 0.0137
\]
• Low temperature debonding resistance:

\[ P_{IC} = 0.85(2.8)^{0.25}(51.0)^{0.33} \sqrt{54.8(141000)(31.2)} < 2776(31.2) \]
\[ P_{IC} = 62.4 \text{ kN} < 86.6 \text{ kN} \]

• Low temperature debonding strain:

\[ \varepsilon_{IC} = \frac{P_{IC}}{(EA)_p} = \frac{62400}{141000(31.2)} = 0.0142 \]

Similarly, for a 3.2 mm wide groove with an epoxy adhesive, the maximum allowable FRP strain can be predicted as follows (using the same material properties as above):

• Depth of failure plane (refer to Figure 2.3):

\[ d_f = \text{groove depth} + t_d = 17.5 + 1.0 = 18.5 \text{ mm} \]

• Width of failure plane (refer to Figure 2.3):

\[ b_f = \text{groove width} + t_b = 3.2 + 2(1.0) = 5.2 \text{ mm} \]

• Length of the failure perimeter:

\[ L_{per} = 2d_f + b_f = 2(18.5) + 5.2 = 42.2 \text{ mm} \]

• Aspect ratio of failure perimeter:

\[ \varphi_f = \frac{18.5}{5.2} = 3.6 \]

• Room temperature debonding resistance:

\[ P_{IC} = 0.85(3.6)^{0.25}(46.3)^{0.33} \sqrt{42.2(141000)(31.2)} < 2776(31.2) \]
\[ P_{IC} = 56.4 \text{ kN} < 86.6 \text{ kN} \]

• Room temperature debonding strain:

\[ \varepsilon_{IC} = \frac{P_{IC}}{(EA)_p} = \frac{56400}{141000(31.2)} = 0.0128 \]
• Low temperature debonding resistance:

\[ P_{IC} = 0.85(3.6)^{0.25}(51.0)^{0.33}\sqrt{42.2(141000)(31.2)} < 2776(31.2) \]
\[ P_{IC} = 58.3 \text{kN} < 86.6 \text{kN} \]

• Low temperature debonding strain:

\[ \varepsilon_{IC} = \frac{P_{IC}}{(EA)_{p}} = \frac{58300}{141000(31.2)} = 0.0133 \]

For the 6.4 mm wide groove with a grout adhesive, the material properties of the grout are known only at room temperature, and these are used to determine the failure load and failure strain since the failure plane is assumed to be in the concrete. The material properties are as follows:

**Material Properties:**

- Grout strength at 20°C, \( f_{g, room} = 61.3 \text{ MPa} \)
- Modulus of elasticity of the FRP strip, \( E_p = 141000 \text{ MPa} \)
- Cross-sectional area of the FRP strip, \( A_p = 31.2 \text{ mm}^2 \)
- Rupture strength of the FRP strip, \( f_{rupt} = 2776 \text{ MPa} \)

The maximum allowable FRP strain is determined as follows:

- Depth of failure plane (refer to Figure 2.3):

\[ d_f = \text{groove depth} + t_d = 16 + 1.0 = 17 \text{ mm} \]

- Width of failure plane (refer to Figure 2.3):

\[ b_f = \text{groove width} + t_b = 2.0 + 2(1.0) = 4.0 \text{ mm} \]

- Length of the failure perimeter:

\[ L_{pex} = 2d_f + b_f = 2(17) + 4 = 38 \text{ mm} \]
• Aspect ratio of failure perimeter:

\[ \phi_f = \frac{d_f}{b_f} = \frac{17}{4.0} = 4.3 \]

• Room temperature debonding resistance:

\[ P_{IC} = 0.85(4.3)^{0.25}(61.3)^{0.33}\sqrt{38(141000)(31.2)} < 2776(31.2) \]

\[ P_{IC} = 61.4 \text{ kN} \leq 86.6 \text{ kN} \]

• Room temperature debonding strain:

\[ \varepsilon_{IC} = \frac{P_{IC}}{(EA)_p} = \frac{61400}{141000(31.2)} = 0.0140 \]

It is interesting to note that the assumed failure plane in the grout gives higher ultimate strains than in the case of the 6.4 mm and 3.2 mm groove width epoxy adhesive cases even though the failure plane is much smaller. This is due to the much higher strength of the grout than the surrounding concrete. It would therefore be reasonable to check the strength with a failure plane in the concrete as well as the grout failure plane and to have the lower of the two be the governing prediction. In this case the concrete failure plane would be the same calculation as the 6.4 mm groove with epoxy adhesive case explained above. Also, since experimental testing showed the failure of the grout slabs occurred in the grout, not in the concrete, this prediction does not match the test observations. It could be that, because the grout adhesive lacks large aggregates, there is an absence of aggregate interlock from confinement providing lower friction and subsequently lower bond strengths in the grout adhesive than the surrounding concrete, even though the overall strength of the grout is higher than the surrounding concrete.
A summary of debonding strain values determined for the various groove widths and adhesive systems on the basis of Equation 2.2 is provided in Table 5.1, as well as comparisons against the maximum FRP strains recorded during the tests presented in Chapter 5, are given in Table 5.1. When comparing measured strains to calculated predictions it is important to consider that strains were measured in the notched area at midspan and this was not necessarily the failure location (although theoretically anywhere within the constant moment region should have approximately the same FRP strain if strain compatibility and perfect bond is maintained). It can be seen that values from Eq 2.2 agree closely (within 20% for epoxy slabs) with the measured results in all cases for epoxy slabs. However, the model over predicts the bond strength in most cases, which is not consistent with its purported nature as a “lower bound model”. It is likely, however, that the very large curvatures and extensive flexural and shear cracking present in the testing in the current thesis had an effect on the observed ultimate strains. Also, the prediction for grout adhesive slabs is less accurate (only within 56%) than for the epoxy adhesive slabs, which is not surprising given that the model was neither developed nor calibrated for cementitious grout adhesives.

### 5.3 Predictive Layer Model

A computer model was developed to predict the moment-curvature for both strengthened and unstrengthened slabs. The moment curvature model was based on a procedure where the slab’s depth was split into discrete layers of equal height of 0.1 mm. The steel reinforcing bars and NSM strengthening were assumed to act across one layer each at the vertical location of their respective centroids. Once the cross section had been broken into layers, an initial strain was assumed at the top of the beam (i.e., at the extreme
compression fibre). By summing the forces in each layer and equating them to zero, to maintain equilibrium of the cross-section at midspan, the height of the neutral axis for the initial extreme compression fibre strain chosen could be determined and the moment and curvature calculated. By repeating this procedure multiple times for increasing extreme compressive fibre strain values in the expected strain range, a predicted moment versus curvature curve was found. In the current analysis, the strain at the extreme compressive fibre was increased from 0.001% to 0.35% by increments of 0.001% strain. The maximum strain of 0.35% was chosen based on the strain limit for concrete crushing in design of reinforced concrete structures in Canada according to CSA A23.3-04 (CSA 2006).

Rather than using a classical elastic-plastic model for the stress-strain behaviour of the reinforcing steel, a modified Ramberg-Osgood function (see Figure 5.4), presented previously by Mattock (1973), was used to account for the fact that the small diameter bars used as internal steel reinforcement did not display a well-defined yield point. The behaviour was modelled according to Equations 5.1 and 5.2 below, where the equations’ parameters were determined for the steel bars from the same coil of reinforcement by experimental testing and non-linear least squares regression analysis by Ranger (2007).

\[
f_s = E_s \varepsilon_s \left( \frac{(1 - A)}{\left[ 1 + (B \varepsilon_s)^c \right]^\frac{1}{c}} \right) \leq f_{su} \quad \text{Eq 5.1}
\]

\[
f_s = E_s \varepsilon_s \left( 0.0078 + \frac{(1 - 0.0078)}{\left[ 1 + (279.5 \varepsilon_s)^{2.07} \right]^{\frac{1}{2.07}}} \right) \leq f_{su} \quad \text{Eq 5.2}
\]
The concrete was modelled using the Collins and Mitchell (1997) model for unconfined concrete (see Figure 5.5). The tensile strength of concrete was considered up until its cracking strength, taken as $0.5\sqrt{f'_c}$. Concrete stress was found using strains from strain compatibility and the Collins and Mitchell (1997) model, which is given by:

$$f'_c = f'_c \left[ \frac{n \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{n - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^n} \right]$$ \hspace{1cm} \text{Eq 5.3}

$$E_c = \left( 3300 \sqrt{f'_c} + 6900 \left( \frac{\gamma_c}{2300} \right)^{1.5} \right)$$ \hspace{1cm} \text{Eq 5.4}

$$n = 0.8 + f'_c / 17$$ \hspace{1cm} \text{Eq 5.5}

$$k = 0.67 + f'_c / 62$$

where:

- $f'_c$ = Compressive strength of concrete (MPa)
- $\varepsilon'_c$ = strain when $f_c$ reaches $f'_c$
- $n$ = Curve fitting factor
- $k$ = factor to increase post-peak decay in stress

The NSM FRP strips were assumed as linear elastic, with materials properties assumed based on the results of tensile coupon tests presented previously in Chapter 4 (see Figure 5.6). However, because the slab strips failed by bond failure rather than tensile rupture of the FRP, to model slabs strengthened with NSM strips a strain limit was required to be applied in the model to account for possible bond failure prior to concrete crushing. One possible way to account for the bond failure mode was to apply a uniform strain limit, as is currently suggested by ACI 440.2R-07 DRAFT (ACI 2007), of $\varepsilon_{IC} = 0.7\varepsilon_{fult}$. The
possible bond failure mode was also taken into account by limiting the FRP strain according to the Seracino bond limit (Seracino et al. 2007a) as detailed in Section 5.2.

A sensitivity analysis was undertaken to investigate the effect of elemental layer width on the results of the moment-curvature model. Layer widths of 1 mm, 0.1 mm and 0.05 mm were used and the results were compared. It was found that there was no noticeable difference in the output results (for instance, less than 1% difference for the control slabs). A 0.1 mm layer width was therefore considered as more than sufficient for the current analysis.

Given that the only difference in the model for differences in groove width and adhesive type is in the determination of the FRP strain limit to cause debonding failure, the only change in the model’s output for the strengthened slabs is the end point (i.e., behaviour up to failure is identical for a given temperature condition). As the proposed strain limit has previously been shown to provide poor agreement with grout adhesive NSM strengthening, this case has not been considered in this analysis. In the case of 6.4 mm and 3.2 mm epoxy adhesive slabs the limiting ultimate strains are either $0.7\varepsilon_{fu}$, according to the limiting strain recommendations of ACI 440, or those of Seracino listed in Table 5.1.

Clearly, the model shows drastically differing behaviour for control versus NSM FRP strengthened slabs, and the overall effects of strengthening with FRPs are, in general, well predicted. The model also shows differing behaviour for room and low temperature
specimens as the compressive strength of the concrete is higher at low temperature as discussed previously. For instance, Figure 5.7 shows the model output compared to the experimental moment versus curvature data for C-RT-1 and C-RT-2 as well as E-6-RT-1 and E-6-RT 2 at room temperature for control and strengthened slabs, respectively, with an FRP strain limit imposed according to ACI 440.2R-07 DRAFT (ACI 2007). Figure 5.8 shows the model output compared to experimental data from C-LT-1 and C-LT-2 as well as E-6-LT-1 and E-6-LT-2 at low temperature for control and strengthened slabs, respectively, again with an FRP strain limit imposed according to ACI 440. In general the model predicts the overall moment curvature behaviour well compared to test results. The model under predicts both the ultimate strength and the stiffness of the FRP strengthened slabs. The model predicts that both the room and low temperature FRP strengthened slabs will fail by bond failure limited by the ACI 440 strain limit. The model also appears to better predict the ultimate strength of the unstrengthened slab strips.

Figures 5.9 and 5.10 present comparisons of the moment versus FRP strain behaviour of the NSM FRP strengthened slab strips. The model gives the same prediction for both the 6.4 mm and 3.2 mm slab strips using the ACI strain limit as it does not consider groove geometry. For the Seracino bond limit the maximum FRP strain is dependent on the groove geometry, so the ultimate load is different for the two different cases, although as with the moment versus curvature predictions, the behaviour up to failure is exactly the same. It can be seen that the model under predicts the moment response in all cases. The strain limits applied also tend to over predict the ultimate strain as previously noted.
In general the model predicts the general trends in behaviour of moment versus curvature and moment versus FRP strain for the NSM strengthened slab strips. It is a simple method which approximates the behaviour, but choosing an ultimate failure strain to limit the FRP bond failure mode proved problematic and is a key issue in predicting failure.
Table 5.1: Summary of maximum FRP strains (i.e., FRP strains causing debonding failure) from the tests presented herein and as predicted by the Seracino et al. (2007) equations.

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Max FRP Strain</th>
<th>Max FRP Strain Test Results</th>
<th>$\varepsilon_{\text{Seracino}}/\varepsilon_{\text{test}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Eq 2.2</td>
<td>Test 1</td>
<td>Test 2</td>
</tr>
<tr>
<td>E-6-RT</td>
<td>0.0137</td>
<td>0.0126</td>
<td>0.0141</td>
</tr>
<tr>
<td>E-6-LT</td>
<td>0.0142</td>
<td>0.0120</td>
<td>0.0119</td>
</tr>
<tr>
<td>E-3-RT</td>
<td>0.0128</td>
<td>0.0112</td>
<td>0.0121</td>
</tr>
<tr>
<td>E-3-LT</td>
<td>0.0133</td>
<td>0.0141</td>
<td>0.0135</td>
</tr>
<tr>
<td>G-6</td>
<td>0.0140</td>
<td>0.0085</td>
<td>0.0094</td>
</tr>
</tbody>
</table>
**Figure 5.1:** Top view of slab for photo image analysis (i.e., view is perpendicular to the tension face of the slab strip).

**Figure 5.2:** Close-up side view of slab for photo imaging analysis to determine flexural strains and curvatures at midspan.
Figure 5.3: Comparison of moment curvature for Pi gauge and image correlation analysis results for selected members.

Figure 5.4: Modified Ramberg-Osgood function for steel reinforcement according to Mattock (1973).
Figure 5.5: Idealized concrete stress-strain behaviour according to Collins and Mitchell (1997).

Figure 5.6: Idealized stress strain behaviour for FRP.
**Figure 5.7:** Room temperature moment vs curvature results of model vs test results.

**Figure 5.8:** Low temperature moment vs curvature results of model vs test result.
Figure 5.9: Room temperature moment vs FRP strain of model vs test result.

Figure 5.10: Low temperature moment vs FRP strain of model vs test result.
CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

The testing and analysis presented in this thesis sought to investigate the room, low and high temperature performance of NSM FRP flexural strengthening, predominantly through an experimental study involving scaled flexural tests on RC slab strip specimens. An extensive literature review was also conducted to determine the current state of knowledge on the topics at hand. Also, an innovative photo imaging instrumentation process was used and validated against more traditional measuring techniques. Based on the results of the current study, the following primary conclusions can be drawn:

1. Ultimate failure loads and post cracking stiffness for all slab strips tested at low temperature were equal to or higher at than equivalent specimens tested at room temperature. This is likely due to the increased concrete strength at low temperature. It is also likely that the low temperature exposure had no detrimental effect on the bond performance of the NSM systems used.

2. The epoxy adhesive NSM strengthening system tested loses significant load carrying capacity at elevated temperatures as low as 100°C

3. All strengthened slab strips showed increases in strength and post cracking stiffness compared to unstrengthened controls, as expected

4. Changing the NSM groove width from 3.2 mm to 6.4 mm had no discernable effect on the strength or stiffness of the strengthened slab strips, suggesting that
the groove dimension limits currently recommended by ACI 440 may not be necessary.

In addition, a number of secondary conclusions can be drawn on the basis of information presented in the various chapters of this thesis. From the literature review presented in Chapter 2, the following conclusions can be drawn:

5. NSM FRP strengthening is more able to fully utilize the strength of the FRP compared to externally bonded FRP strengthening due to the superior bond performance.
6. NSM FRP strips develop higher bond average strengths than NSM FRP bars.
7. Bond test results from direct pull-out type testing do not easily apply to full scale flexural applications due to differing mechanics from one application to the other.
8. The large variety of possible failure modes makes the development of simple design procedures and strain limits difficult.
9. Virtually no research to date has focused on durability of NSM in low or high temperature applications.

With respect to general NSM flexural behaviour the following conclusions can be made on the basis of the experimental program presented in Chapters 3 and 4:

10. Failure modes of the slab strips were highly dependent on the strengthening system and exposure condition. Epoxy adhesive slab strips at room and low temperatures failed by either bond failure at the epoxy concrete interface or combined shear bond failure. Grout adhesive slab strips at room and low
temperature failed by bond failure at the grout strip interface. High temperature exposed epoxy adhesive slab strips failed by NSM pullout at the concrete-epoxy interface. While, grout adhesive slab strips at high temperature failed by NSM pullout at the strip grout interface.

11. Epoxy adhesive NSM FRP strengthened slabs had higher ultimate loads and higher post cracking stiffness than grout adhesive strengthened slabs. This is due to differences in the bond strength and failure location when using these different systems.

With respect to high temperature testing of NSM FRP strengthening on slab strips the following conclusions can be made on the basis of the experimental program presented in Chapters 3 and 4:

12. The grout adhesive NSM strengthening system tested in the current study maintained load carrying capability for more than 6 times longer at both 100°C and 200°C exposure temperatures when compared to the epoxy adhesive system tested.

13. When heating times were correlated against unloaded heating verification tests, the epoxy adhesive slab strips failed when the temperature at the bottom of the groove was at temperatures close to the measured $T_g$ of the epoxy. This suggests that there is sufficient residual strength in the epoxy to maintain bond strength at temperatures close to $T_g$. 
14. When heating times were correlated against unloaded heating verification tests, grout adhesive slab strips failed when temperatures at the groove base were 13°C below the reported heat distortion temperature.

15. Slab strips exposed to 100°C remained structurally effective at least four times longer than slab strips exposed to 200°C, as expected.

16. The grout adhesive slab strip specimen maintained full strength (i.e., room temperature strength) after 5 hours of heating exposure at 100°C.

An unexpected side note to the high temperature testing on the slab strip specimens is summarized below:

17. It appears that very short bonded lengths anchored in low moment regions are capable of sustaining relatively high strains (greater than 0.4%, or 20% of ultimate strain) in the FRP strengthening material.

From conducting validation of digital image correlation for measuring flexural curvatures in concrete members in flexure, it can be concluded that:

18. Comparison of traditional Pi gauge measurement and a new novel digital image correlation technique generally showed good agreement between the two techniques.

19. To measure small strains and displacements using digital image correlation, extreme care must be taken to ensure that photos have the required resolution and that out-of-plane displacements do not occur.
While this thesis has sought to investigate durability issues of NSM FRP strips with relation to high and low temperature, much work is left to be done in the area. The following are specific recommendations for future work:

- Further research into potential trends present in the vast amount of research available on NSM FRP strip reinforcement of concrete. This presents many practical issues as test systems and parameters vary widely. However, given the potential benefits of exploiting the available data it could be useful.
- Testing the effect of repeated freeze thaw cycling on NSM FRP reinforced concrete members (currently underway at Queen’s University).
- High temperature testing of the NSM constituent materials while under load to specifically investigate bond and material performance.
- Large scale testing of NSM FRP reinforced concrete members at high temperature, including full scale standard fire tests.
- Modelling and testing verification of heat transfer issues with relation to NSM FRP strengthened concrete members.
- Finally, the most elusive goal of current NSM research (and externally bonded FRP research also) appears to be development of a simple correlation between bond tests and flexural performance of NSM systems. However, due to the complexity of this problem, as illustrated in Chapter 2, this has not yet been achieved. Further research into this area is needed before rational, conservative, yet efficient design guidelines can be formulated.
REFERENCES


REFERENCES


References


APPENDIX A

EXISTING GUIDELINES FOR NSM

ACI 440.2R-07-Draft

The American Concrete Institute’s proposed Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures (ACI 440 2007) deals with NSM FRP strengthening in more detail than any other widely available design guideline. This document prescribes methods for calculating the flexural resistance, shear resistance, and development length for NSM FRP systems in both shear and flexural strengthening applications, and also discusses suggested installation procedures, etc.

For flexural strengthening with NSM FRP systems the Guide proposes a strain limit in the FRP between $0.6\varepsilon_{ult}$ and $0.9\varepsilon_{ult}$ with $0.7\varepsilon_{ult}$ being recommended in the absence of specific data on the particular system being used, where $\varepsilon_{ult}$ is the ultimate strain of the NSM FRP determined from direct tensile coupon tests (cl. 9.1). For round NSM bars, a minimum groove size is set at 1.5 times the diameter of the FRP bar, presumably to limit the likelihood of adhesive splitting failure based on the work of Hassan et al. (2004) (cl. 12.3). For rectangular bars or strips, the minimum groove size is given as $3.0a_b \times 1.5b_b$ where $a_b$ is the smaller bar dimension (cl. 12.3). Figure 2.7 shows the required groove dimensions prescribed by the ACI 440 document. The development length of NSM FRP reinforcement is given by ACI 440 as (cl. 12.3):
\[ l_{db} = \frac{d_b}{4(0.5\tau_{\text{max}})} \] for circular bars \hspace{1cm} \text{Eq A.1}

\[ l_{db} = \frac{a_b \cdot b_b}{2(d_b + b_b)(0.5\tau_{\text{max}})} \] for rectangular bars \hspace{1cm} \text{Eq A.2}

where:

- \( l_{db} \) = development length (mm)
- \( \tau_{\text{max}} \) = average bond strength - 6 to 9 MPa is recommended (MPa)
- \( d_b \) = diameter of FPR bar (mm)
- \( a_b \) = smallest dimension of rectangular FRP bars (mm)
- \( b_b \) = largest dimension of rectangular FRP bars (mm)
- \( f_{jd} \) = design stress of NSM reinforcement (MPa)

The minimum clear groove spacing is given as twice the depth of the groove to avoid overlapping of the tensile stresses around the NSM bars, and the clear edge distance is given as four times the depth of the groove to minimize edge effects that could accelerate debonding failures (cl. 12.3), based on the work of Hassan (2003).

Detailed calculations of the design strength according to ACI 440.2R-07 (ACI 440 2007) for the beams tested in the current thesis are illustrative and are in Appendix B. The reader is encouraged to consult this appendix for additional information.

**CAN/CSA S6-06**

For the most recent version of the Canadian Highway Bridge Design Code (CSA 2006), structural strengthening using FRPs has been included for the first time. In the S6 Code NSM FRP (referred to as NSMR – near surface mounted reinforcement) has been grouped with all other externally bonded strengthening methods and it has thus been treated in essentially the same way as other systems such as externally bonded FRP
APPENDIX A: Existing Guidelines for NSM

sheets. A strain limit of 0.006 at ultimate has been applied to all strengthening applications, externally bonded or NSMR (cl. 16.11.2.3). Strain limits also exist depending on the fibre type under sustained service loads. These are $0.35f_{ult}$ for aramid FRP, $0.65f_{ult}$ for carbon FRP and $0.25f_{ult}$ for glass FRP (cl. 16.8.3). The use of NSM is not allowed if the concrete cover to the reinforcement is less than 20 mm (cl. 16.11.1). Interestingly, section 16.12 of CAN/CSA S6-06 deals exclusively with the rehabilitation of timber bridges, and subsections 16.12.2.2 and 16.12.3.2 deal with strengthening of timber bridges using NSM GFRP bars for flexure and shear respectively.

**Concrete Society Technical Report No. 55**

Design Guidance for Strengthening Concrete Structures using Fibre Composite Materials, Second Edition (TR 55) a report of a Concrete Society Committee (Concrete Society 2004) deals with the use of NSM FRP strengthening in the United Kingdom. With respect to NSM FRP applications, the report comments on flexural strengthening, installation, inspection, research into shear strengthening and the emergence of prestressed NSM reinforcement.

Technical Report 55 treats NSM FRP in mainly the same way as externally bonded FRP strengthening systems. Section 6.4 of the Report deals exclusively with flexural strengthening using NSM reinforcement. The overall strain in the FRP at failure is limited to 0.008 which was chosen since, “such an approach alone does not fully capture the mechanics that underpin the initiation of FRP separation failure. However, it does seem prudent at present to retain a limit on the maximum design FRP strain” (The
Concrete Society 2004). Limits are also placed on the longitudinal shear stress between the FRP and the concrete at failure to prevent debonding, according to:

\[
\tau = V_{add} \forall_f A_p \left( h - y \right) / I_{cs} b_a < 0.8 \text{N/mm}^2
\]  
Eq A.3

where:

\( V_{add} \) = difference between the ultimate shear force and the applied shear force when the strengthening is installed
\( \forall_f \) = short-term modular ratio of FRP to concrete
\( = E_{sd} / E_c \)
\( y \) = depth of neutral axis of strengthened section
\( A_p \) = area of FRP plat
\( I_{cs} \) = second moment of area of strengthened concrete equivalent cracked section
\( b_a \) = perimeter of the groove
\( h \) = total depth of the section

Bond failure modes of adhesive splitting failure, concrete splitting failure and NSM separation failure (cover separation) are discussed. Anchorage design for circular bars is treated with a list of constrains given by the following;

To avoid adhesive splitting failure

\[
T_{nsm,ad} = 0.1 b_{barperim} l_{nsm} f_a \quad \text{for plain FRP bars (including spirally wound and sand coated bars)} \]
Eq A.4

\[
T_{nsm,ad} = 0.3 b_{barperim} l_{nsm} f_a \quad \text{for deformed FRP bars} \]
Eq A.5

where:

\( T_{nsm,ad} \) = characteristic adhesive bond failure force
\( b_{barperim} \) = perimeter of FRP bar
\( l_{nsm} \) = anchorage length provided for NSM bar
\( f_a \) = design adhesive tensile strength

To avoid concrete splitting failure:
APPENDIX A: Existing Guidelines for NSM

\[ T_{\text{nsm, max}} = 1.9 \sqrt{E_f A_f b_{\text{notchperim}} f_{\text{ctm}}} \]  \hspace{1cm} \text{Eq A.6}

\[ l_{\text{nsm, max}} = 4.5 \sqrt{\frac{E_f A_f}{b_{\text{notchperim}} f_{\text{ctm}}}} \]  \hspace{1cm} \text{Eq A.7}

where:

\( T_{\text{nsm, max}} \) = maximum NSM anchorage force (N)

\( l_{\text{nsm, max}} \) = anchorage length required to generate \( T_{\text{nsm, max}} \)

\( E_f \) = design FRP modulus of elasticity (N/mm\(^2\))

\( A_f \) = area of FRP (mm\(^2\))

\( b_{\text{notchperim}} \) = effective perimeter of notch (mm)

\( f_{\text{ctm}} \) = concrete tensile strength (N/mm\(^2\))

When the maximum anchorage length in not possible, the anchorage force generated is given by:

\[ T_{\text{nsm}} = T_{\text{nsm, max}} \frac{l_{\text{nsm}}}{l_{\text{nsm, max}}} \left( 2 - \frac{l_{\text{nsm}}}{l_{\text{nsm, max}}} \right) \]  \hspace{1cm} \text{Eq A.8}

where:

\( T_{\text{nsm}} \) = characteristic anchorage force for NSM

\( l_{\text{nsm}} \) = anchorage length provided for NSM

The report states that currently there is insufficient information to create specific guidelines for the use of NSM strips.
APPENDIX B

SLAB FLEXURAL AND SHEAR STRENGTH

In Appendix B detailed calculations are presented for both unfactored flexural and unfactored shear strength of the slab strips used in this thesis. Unfactored flexural strength has been calculated using strain compatibility according to ACI 318-05. Also the FRP strain limit suggested in ACI 440.2R-07-DRAFT is applied in conjunction with ACI 318 for flexural strengthening. The unfactored flexural strength is also presented using strain compatibility according to CSA A23.3-04. The strain limit suggested in ACI 440.2R-07-DRAFT is also applied in conjunction with CSA A23.3-04 for flexural strengthening. Unfactored shear strength calculations are presented according to ACI 440.1R-06, ACI 318-05 and CSA A23.3-04.

Flexural Strength

FRP Properties

- Cross sectional area of FRP strip, \( A_{frp} = 31.2 \text{ mm} \)
- Depth to neutral axis of FRP strip, \( d_{frp} = 91 \text{ mm} \)
- Elastic modulus of FRP strip, \( E_f = 124,000 \text{ MPa} \)
- Ultimate strength of FRP strip, \( f_{ult} = 2068 \text{ MPa} \)
- Ultimate strain of FRP strip, \( \epsilon_{ult} = 0.017 \)

Steel Properties

- Area of reinforcing steel, \( A_s = 64.3 \text{ mm}^2 \)
- Depth to neutral axis of reinforcing steel, \( d_s = 73 \text{ mm} \)
APPENDIX B: Slab Strip Flexural and Shear Strength

- Elastic modulus of reinforcing steel, \( E_s = 195 \, 800 \, \text{MPa} \)
- Yield stress of reinforcing steel, \( f_y = 693 \, \text{MPa} \)
- Yield stress of reinforcing steel, \( \varepsilon_{ys} = \frac{693}{195800} = 0.0035 \)

Concrete Properties

\( f'_c = 46 \, \text{MPa} \)

CSA \( E_c = 4500\sqrt{f'_c} = 30520 \, \text{MPa} \)
ACI \( E_c = 4732\sqrt{f'_c} = 32100 \, \text{MPa} \)
ACI \( \varepsilon_{ult} = 0.003 \)
CSA \( \varepsilon_{ult} = 0.0035 \)

Solving using strain compatibility ACI 440.2R-07-DRAFT and ACI 318-05

Unstrengthened Beam

\( C = T \)

\( C = 0.85f'_c\beta_lcb \)

\[ \beta_l = \begin{cases} 
0.85 & \text{if } f'_c < 4000 \, \text{psi (27.58 MPa)} \\
0.85 - 0.5 \left( \frac{f'_c - 4000}{1000} \right) & > 0.65 \text{ if } f'_c > 4000 \, \text{psi} \\
0.85 - 0.5 \left( \frac{f'_c - 27.58}{6.98} \right) & > 0.65 \text{ if } f'_c > 27.58 \, \text{MPa} 
\end{cases} \]

\( T_s = A_s f_s \)

Assuming steel yield followed by concrete crushing:
\[ \beta_i = 0.85 - 0.05 \left( \frac{46 - 27.58}{6.89} \right) = 0.72 \]

\[ C = 0.85(46)(0.72)c(254) = 7150c \]

\[ T_s = 64.3(693) = 44560 \]

using \( T = C \), solving for \( c \)

\[ c = 6.23 \text{ mm} \]

\[ \varepsilon_{ult} = \varepsilon_{ult} \left( \frac{d_s - c}{c} \right) = 0.003 \left( \frac{73 - 6.23}{6.23} \right) = 0.0322 > \varepsilon_{sy} = 0.0035 \therefore \text{ steel yields} \]

\[ a = \beta_i c = 0.72(6.23) = 4.49 \text{ mm} \]

\[ M_r = A_s f_{sy} (d_s - a/2) \]

\[ M_r = 64.3(693)(73 - 4.49/2) \]

\[ M_r = 3.15 \text{ kNm} \]

**Strengthened Beam**

\[ C = T \]

\[ C = 0.85 f'_c \beta_i cb \]

\[ \beta_i = \begin{cases} 0.85 & \text{if } f'_c < 4000 \text{ psi (27.58 MPa)} \\ 0.85 - 0.5 \left( \frac{f'_c - 4000}{1000} \right) & > 0.65 \text{ if } f'_c > 4000 \text{ (psi)} \\ 0.85 - 0.5 \left( \frac{f'_c - 27.58}{6.98} \right) & > 0.65 \text{ if } f'_c > 27.58 \text{ (MPa)} \end{cases} \]

\[ T_s = A_s f_s \]

\[ T_{fp} = A_{fp} f_{fp} \]

Assuming concrete crushing failure with steel yield prior to FRP rupture or bond failure:
\[
\frac{\varepsilon_{\text{ult}}}{c} = \frac{\varepsilon_{\text{ult}}}{d_{s} - c} = \frac{\varepsilon_{\text{ult}}}{d_{\text{frp}} - c}
\]

\[\beta_1 = 0.85 - 0.05\left(\frac{46 - 27.58}{6.89}\right) = 0.72\]

\[C = 0.85(46)(0.72)c(254) = 7150c\]

\[T_s = 64.3(693) = 44560\]

\[f_{\text{frp}} = E_{\text{frp}}\varepsilon_{\text{ult}}\left(\frac{d_{\text{frp}} - c}{c}\right)\]

\[T_{\text{frp}} = 31.2(124000)(0.003)\left(\frac{91 - c}{c}\right)\]

using \(T_s + T_{\text{frp}} = C\), solving for \(c\)

\[c = 14.67 \text{ mm}\]

\[\varepsilon_{\text{ult}} = \varepsilon_{\text{ult}}\left(\frac{d_{s} - c}{c}\right) = 0.003\left(\frac{73 - 14.67}{14.67}\right) = 0.0119 > \varepsilon_{\text{sy}} = 0.0035 \therefore \text{steel yields}\]

\[\varepsilon_{\text{frp}} = \varepsilon_{\text{ult}}\left(\frac{d_{\text{frp}} - c}{c}\right) = 0.003\left(\frac{91 - 14.67}{14.67}\right) = 0.0156 < \varepsilon_{\text{ult}} = 0.017 \therefore \text{FRP does not rupture}\]

\[a = \beta_1c = 0.72(14.67) = 10.56 \text{ mm}\]

\[M_r = A_{\text{frp}}f_{\text{sy}}\left(d_{s} - a/2\right) + A_{\text{frp}}E_{\text{frp}}\varepsilon_{\text{frp}}\left(d_{\text{frp}} - a/2\right)\]

\[M_r = 64.3(693)(73 - 10.56/2) + 31.2(124000)(0.0156)(91 - 10.56/2)\]

\[M_r = 8.19 \text{ kNm}\]

According to ACI 440.2R-07 Draft strain in the NSM strip should be limited to:

\[\varepsilon_{\text{frp}} < 0.7\varepsilon_{\text{ult}}\]

\[\varepsilon_{\text{frp}} = 0.7(0.017) = 0.0119 > 0.0156 \therefore \text{concrete does not crush prior to bond failure}\]
APPENDIX B: Slab Strip Flexural and Shear Strength

\[ C = 0.85E_c \varepsilon_c \beta c b \]

\[ C = 0.85E_c \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) \beta c b \]

\[ T_s = A_s f_s \]

\[ T_{frp} = A_{frp} E_{frp} \varepsilon_{frp} \]

\[ C = 0.85(32100)(0.0119) \left( \frac{c}{91-c} \right)(0.72) c (254) \]

\[ T_s = 64.3(693) = 44560 \]

\[ T_{frp} = 31.2(124000)(0.0119) = 46039 \]

using \( C = T_s + T_{frp} \), solving for \( c \)

\[ c = 11.05 \text{ mm} \]

\[ \varepsilon_s = \varepsilon_{frp} \left( \frac{d_s - c}{d_{frp} - c} \right) = 0.0119 \left( \frac{73 - 11.05}{91 - 11.05} \right) = 0.0092 > 0.0035 \therefore \text{ steel yields} \]

\[ \varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) = 0.0119 \left( \frac{11.05}{91 - 11.05} \right) = 0.0016 < 0.003 \therefore \text{ concrete does not crush} \]

\[ a = \beta c = 0.72(11.05) = 7.96 \text{ mm} \]

\[ M_r = A_s f_y (d_s - a / 2) + A_{frp} E_{frp} \varepsilon_{frp} (d_{frp} - a / 2) \]

\[ M_r = 64.3(693)(73 - 7.96 / 2) + 31.2(124000)(0.0119)(91 - 7.96 / 2) \]

\[ M_r = 7.08 \text{ kNm} \]

**Solving using strain compatibility ACI 440.2R-07 and CSA A23.3-04**

**Unstrengthened Beam**
APPENDIX B: Slab Strip Flexural and Shear Strength

\[ C = T \]

\[ C = \alpha_1 f'_c \beta_1 cb \]

\[ \alpha_1 = 0.85 - 0.0015 f'_c \geq 0.67 \]
\[ \beta_1 = 0.97 - 0.0025 f'_c \geq 0.67 \]

\[ T_s = A_s f_s \]

Assuming steel yield followed by concrete crushing:

\[ \alpha_1 = 0.85 - 0.0015(46) = 0.78 \]
\[ \beta_1 = 0.97 - 0.0025(46) = 0.86 \]
\[ C = 0.78(46)(0.86)c(254) = 7838c \]

\[ T_s = 64.2(693) = 44491 \]

using \( C = T \), solving for \( c \)

\[ c = 5.68 \text{ mm} \]

\[ \varepsilon_{ult} = \varepsilon_{ult} \left( \frac{d_s - c}{c} \right) = 0.0035 \left( \frac{73 - 5.68}{5.68} \right) = 0.0414 > \varepsilon_{sy} = 0.0035 \therefore \text{ steel yields} \]

\[ a = \beta_1 c = 0.86(5.68) = 4.88 \text{ mm} \]

\[ M_r = A_s f_{sy} (d_s - a / 2) \]

\[ M_r = 64.3(693)(73 - 4.88 / 2) \]

\[ M_r = 3.14 \text{ kN.m} \]

**Strengthened Beam**

\[ C = T \]

\[ C = \alpha_1 f'_c \beta_1 cb \]

\[ T_s = A_s f_s \]

\[ T_{frp} = A_{frp} f_{frp} \]

Assuming concrete crushing failure with steel yield prior to FRP rupture or bond failure:
\[
\varepsilon_{\text{ult}} = \varepsilon_{\text{ult}} \left( \frac{d_s - c}{c} \right) = \varepsilon_{\text{ult}} \left( \frac{d_{\text{frp}} - c}{c} \right)
\]

\[C = 0.78 \times 46 \times 0.86 \times 254 = 7838c\]

\[T_s = 64.3 \times 693 = 44560\]

\[f_{\text{frp}} = E_{\text{frp}} \varepsilon_{\text{ult}} \left( \frac{d_{\text{frp}} - c}{c} \right)\]

\[T_{\text{frp}} = 31.2 \times (124000) \times (0.0035) \times \left( \frac{91 - c}{c} \right)\]

using \(T_s + T_{\text{frp}} = C\), solving for \(c\)

\[c = 14.67\ \text{mm}\]

\[\varepsilon_{\text{ult}} = \varepsilon_{\text{ult}} \left( \frac{d_s - c}{c} \right) = 0.0035 \times \left( \frac{73 - 14.67}{14.67} \right) = 0.0139 > \varepsilon_y = 0.0035 \therefore \text{steel yields}\]

\[\varepsilon_{\text{frp}} = \varepsilon_{\text{ult}} \left( \frac{d_{\text{frp}} - c}{c} \right) = 0.0035 \times \left( \frac{91 - 14.67}{14.67} \right) = 0.0182 > \varepsilon_{\text{ult}} = 0.017 \therefore \text{strip ruptures}\]

Because strip ruptures before concrete crushing, must recalculate:

\[C = T\]

\[C = \alpha_t E_c \varepsilon_c \beta_t cb\]

\[C = \alpha_t E_c \varepsilon_{\text{frp}} \left( \frac{c}{d_{\text{frp}} - c} \right) \beta_t cb\]

\[T_s = A_s f_s\]

\[T_{\text{frp}} = A_{\text{frp}} E_{\text{frp}} \varepsilon_{\text{frp}}\]

\(\alpha_t\) and \(\beta_t\) must be found from ISIS Design Manual 3 (ISIS 2001)
$C = \alpha_1 (30520)(0.017)\left(\frac{c}{91-c}\right)\beta c (254)$

$C = \alpha_1 \beta_1 (131785.36)\left(\frac{c^2}{91-c}\right)$

$T_s = 64.3(693) = 44560$

$T_{frp} = 31.2 (124000)(0.017) = 65770$

$\alpha_1 \beta_1 (131785.36)\left(\frac{c^2}{91-c}\right) = 110330$

$\varepsilon_c = \varepsilon_{frp} \left(\frac{c}{d_{frp} - c}\right) = 0.017 \left(\frac{c}{91-c}\right)$

By iteration using Figure B.1 and Figure B.2 below

![Graph showing equivalent stress-block parameter $\alpha$ for concrete strengths of 20 to 60 MPa](image)

**Figure B.1:** $\alpha_1$ from ISIS (2001).
Figure B.2: $\beta_1$ from ISIS (2001).

$c = 10.34$ mm

$\alpha_1 = 0.89$

$\beta_1 = 0.71$

$a = \beta_1 c = 0.71(10.34) = 7.34$

$\varepsilon_s = \varepsilon_{fp} \left( \frac{d_s - c}{d_{frp} - c} \right) = 0.017 \left( \frac{73 - 10.34}{91 - 10.34} \right) = 0.0132 > 0.0035 \therefore$ steel yields

$\varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) = 0.017 \left( \frac{10.34}{91 - 10.34} \right) = 0.0022 < 0.0035 \therefore$ concrete does not crush

$M_r = A_s f_{sy} (d_s - a / 2) + A_{frp} E_{frp} \varepsilon_{frp} (d_{frp} - a / 2)$

$M_r = 64.3(693)(73 - 7.34 / 2) + 31.2(124000)(0.017)(91 - 7.34 / 2)$

$M_r = 8.83$ kNm

According to ACI 440.2R-07 Draft strain in the NSM strip should be limited to:
\[ \varepsilon_{fp} < 0.7 \varepsilon_{fpult} \]
\[ \varepsilon_{fp} = 0.7(0.017) = 0.0119 < 0.017 \therefore \text{NSM strip would not rupture} \]

\[ C = T \]
\[ C = \alpha_i E_c \varepsilon_c \beta_ib \]
\[ C = \alpha_i E_c \varepsilon_{fp} \left( \frac{c}{d_{fp} - c} \right) \beta_i cb \]
\[ T_s = A_s f_s \]
\[ T_{fp} = A_{fp} E_{fp} \varepsilon_{fp} \]

\( \alpha_i \) and \( \beta_i \) must be found from ISIS Design Manual 3 (2001) in Figure B.1 and Figure B.2.
\[ C = \alpha_i (30520)(0.0119)\left( \frac{c}{91 - c} \right) \beta_i c(254) \]
\[ C = \alpha_i \beta_i (92250) \left( \frac{c^2}{91 - c} \right) \]
\[ T_s = 64.3(693) = 44560 \]
\[ T_{fp} = 31.2(124000)(0.0119) = 46039 \]
\[ \alpha_i \beta_i (92250) \left( \frac{c^2}{91 - c} \right) = 90599 \]
\[ \varepsilon_c = \varepsilon_{fp} \left( \frac{c}{d_{fp} - c} \right) = 0.017 \left( \frac{c}{91 - c} \right) \]

By iteration using Figure B.1 and Figure B.2
\[ c = 12.17 \text{ mm} \]
\[ \alpha_i = 0.78 \]
\[ \beta_i = 0.67 \]
\[ a = \beta_i c = 0.67(12.17) = 4.92 \]
\[ \varepsilon_s = \varepsilon_{frp} \left( \frac{d_s - c}{d_{frp} - c} \right) = 0.0119 \left( \frac{73 - 12.17}{91 - 12.17} \right) = 0.0092 > 0.0035 \therefore \text{steel yields} \]

\[ \varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) = 0.0119 \left( \frac{12.17}{91 - 12.17} \right) = 0.0018 < 0.0035 \therefore \text{concrete does not crush} \]

\[ M_r = A_s f_{sy} (d_s - a / 2) + A_{frp} E_{frp} \varepsilon_{frp} \left( d_{frp} - a / 2 \right) \]

\[ M_r = 64.3(693)(73 - 4.92 / 2) + 31.2(124000)(0.0119)(91 - 4.92 / 2) \]

\[ M_r = 7.22 \text{ kNm} \]

**Shear Strength**

**ACI 440.1R-06**

\[ V_c = \frac{2}{5} \sqrt{f_c} b_w c \]

where:

- \( V_c \) = shear resistance provided by the concrete (N)
- \( f_c \) = concrete compressive strength (MPa)
- \( b_w \) = width of the concrete section (mm)
- \( c \) = cracked transformed section neutral axis depth (mm)

\[ n_{frp} = \frac{E_{frp}}{E_c} = \frac{124000}{32100} = 3.86 \]

\[ n_s = \frac{E_s}{E_c} = \frac{195800}{32100} = 6.1 \]

**Cracked Transformed Section**

\[ A_{frp} = n_{frp} A_{frp} = 3.86(31.2) = 120 \text{ mm}^2 \]

\[ A_{fr} = n_s A_s = 6.1(64.3) = 392 \text{ mm}^2 \]
\[ \sum A_i \bar{y}_i = 0 \]

\[ 254c \left( \frac{c}{2} \right) = 392(73 - c) + 120(91 - c) \]

\[ c = 15.7 \text{ mm} \]

\[ V_c = \frac{2}{5} \sqrt{46} (254)(15.7)/1000 \]

\[ V_c = 10.8 \text{ kN} \]

**ACI 318-05**

For SI units

\[ V_c = 0.17\sqrt{f'c b_d d} \]

\[ V_c = 0.17\sqrt{46} (254)(73)/1000 \]

\[ V_c = 21.4 \text{ kN} \]

**CSA A23.3-04**

\[ V_c = \phi \lambda \beta \sqrt{f'c b_d d_v} \]

\[ d_v = \text{greater of} \begin{cases} 0.9d = 0.9(73) = 66 \\ 0.72h = 0.72(102) = 73 \end{cases} \]

\[ d_v = 82 \]

\[ \lambda \text{ and } \phi = 1.0 \]

According to 11.3.6.2 (a) \( \beta = 0.21 \)

\[ V_c = 0.21\sqrt{46} (254)(73)/1000 \]

\[ V_c = 26.4 \text{ kN} \]
APPENDIX C

PHOTO ANALYSIS CURVATURE RESULTS

The following figures show moment curvature plots for all room and low temperature slab test comparing photo analysis and pi gauge data. Test E-6-RT-2 is not shown, since the auto focus for the camera was accidentally left on during testing of this specimen and therefore the image correlation analysis could not be conducted (the autofocus feature causes apparent pixel patch movements which corrupt the image analysis). Test G-6-LT-2 in Figure C.12 only shows photo data up to approximately 5 kNm as the camera stopped functioning, presumably due to the extreme low temperature conditions under which it was operating. In some cases there are clearly outlying results from the photo analysis nearing the end of the tests, as can be seen in Figure C.9, for example. These data points are presumed to be the result of the development of large cracks within the tracking pixel patches of the photo analysis, thus disabling the patch tracking algorithm and corrupting the image correlation analysis.
Figure C.1: E-6-RT-1 moment curvature photo vs PI gauge.

Figure C.2: E-3-RT-1 moment curvature photo vs PI gauge.
Figure C.3: E-3-RT-2 moment curvature photo vs PI gauge.

Figure C.4: G-6-RT-1 moment curvature photo vs PI gauge.
Figure C.5: G-6-RT-2 moment curvature photo vs PI gauge.

Figure C.6: C-RT-1 moment curvature photo vs PI gauge.
Figure C.7: C-RT-2 moment curvature photo vs PI gauge.

Figure C.8: E-6-LT-1 moment curvature photo vs PI gauge.
Figure C.9: E-6-LT-2 moment curvature photo vs PI gauge.

Figure C.9: E-3-LT-1 moment curvature photo vs PI gauge.
Figure C.10: E-3-LT-2 moment curvature photo vs PI gauge.

Figure C.11: G-6-LT-1 moment curvature photo vs PI gauge.
Figure C.12: G-6-LT-2 moment curvature photo vs PI gauge.

Figure C.13: C-LT-1 moment curvature photo vs PI gauge.
Figure C.14: C-LT-2 moment curvature photo vs PI gauge.