

Self-prestressing Iron-based Shape Memory Alloy (Fe-SMA) Epoxy Composite for Active  
Reinforced Concrete Shear Strengthening

by

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## **Author's Declaration**

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

## Abstract

In Canada, the rapid deterioration of aging reinforced concrete (RC) structures has become a continuing issue, with more than 40% of bridges being older than 40 years old and 38% being in poor and fair condition, necessitating billions for rehabilitation (Cusson & Isgor, 2004; Lafleur, 2023). The loss of the strength and stiffness in RC bridge structures can be attributed to age and exposure, and it has been exacerbated with the increase of freight weight, traffic, extreme freezing/thawing cycles, and climate change. A concerning RC failure mode is shear due to its brittle and abrupt nature. Hence, various shear-strengthening mechanisms have been developed. Most of these mechanisms involve fiber-reinforced polymers (FRP) and are passive, acting after the structure experiences damage. Active (prestressing) mechanisms have gained notoriety due to their ability to act immediately after application, reducing crack widths and propagation. However, implementing shear prestressing is complex, often requiring expensive and impractical large jacking equipment. Smart materials such as iron-based shape memory alloys (Fe-SMAs) have the potential to be implemented in cost-efficient and simple shear strengthening and retrofitting techniques. Fe-SMAs present a thermomechanical property known as shape memory effect (SME) that allows the material to return to its undeformed shape after reaching an activation temperature, which can be done with resistive heating. If the material is restrained, the Fe-SMA has the capacity to self-prestress an element without the need of jacking tools. This project presents an experimental study on the shear strengthening feasibility and capacity of a near-surface bonded (NSB) active Fe-SMA epoxy composite. The composite consists of u-bent strips embedded into grooves filled with epoxy. After the epoxy cures, the Fe-SMA strips are heated to at least 180°C with an electric current to self-prestress the concrete. Three shear-critical RC beams were cast, with one beam being used as control, and the other two beams being shear strengthened. Two Fe-SMA ratios were assessed 0.05% and 0.1%. The strengthened beams exhibit about a 27% increase in strength, and the reduction of crack widths and stirrup stresses. The NSB Fe-SMA strips interrupt the formation and widening of diagonal cracks; however, increasing their ratio may not mean an increase in shear strength. A dense NSB Fe-SMA - concrete interface weakens the stirrup plane, creating horizontal cracks running along the top face of the beam (ends of the Fe-SMA u-wrapped strips) in the compression region and causing the separation of the side concrete cover. Additional insights on the active shear strengthening have been provided with two FEA parametric studies using Vector2 by evaluating prestress level and Fe-SMA ratio. This project assesses the shear strengthening effect of near-surface bonded (NSB) active Fe-SMA epoxy composites on the load-displacement response, crack widths, and reinforcement stresses on shear-critical RC specimens.

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## **List of Abbreviations**

|           |                                            |
|-----------|--------------------------------------------|
| ASTM      | American Society for Testing and Materials |
| CIRC      | Canadian Infrastructure Report Card        |
| CFRP      | Carbon fiber-reinforced polymer            |
| CFT       | Compression Filed Theory                   |
| CSA       | Canadian Standards Association             |
| CSA A23.3 | Design of concrete structures              |
| DSFM      | Disturbed Stress Field Model               |
| EB        | Externally bonded                          |
| ESL       | Estimated service life                     |
| FEA       | Finite element Analysis                    |
| Fe-SMA    | Iron-based shape memory alloy              |
| FRP       | Fiber-reinforced polymers                  |
| GFRP      | Glass fiber-reinforced polymer             |
| MCFT      | Modified Compression Field Theory          |
| MTO       | Ministry of Transportation of Ontario      |
| NSB       | Near-surface bonded                        |
| NSM       | Near-surface mounted                       |
| OSIM      | Ontario Structure Inspection Manual        |
| PCM       | Polymer-modified cement                    |
| RC        | Reinforced concrete                        |
| RS        | Recovery Stress                            |
| SL        | Seed load                                  |
| SMA       | Shape memory alloy                         |
| SME       | Shape memory effect                        |
| TML       | Tokyo Measuring Instruments Laboratory     |
| TPU       | Thermoplastic Polyurethane                 |
| TRS       | Transfer span                              |
| TS        | Test span                                  |
| UDL       | Uniform distributed load                   |

## List of Symbols

|                |                                                                              |
|----------------|------------------------------------------------------------------------------|
| $a$            | : depth of the equivalent rectangular compressive stress block               |
| $A$            | : austenite                                                                  |
| $A_{eq}$       | : area of the epoxy filler area                                              |
| $A_f$          | : austenite-finish-temperature                                               |
| $A_{Fe-SMA}$   | : area of the Fe-SMA prestressing material                                   |
| $a_g, a_{max}$ | : maximum nominal size of coarse aggregate                                   |
| $Al$           | : aluminum                                                                   |
| $A_s$          | : total area of tension longitudinal reinforcement (Sections 2.1, Chapter 3) |
|                | : austenite-start-temperature (Section 2.3)                                  |
| $A'_s$         | : total area of compression longitudinal reinforcement                       |
| $A_{st}$       | : bar area of transverse reinforcement                                       |
| $A_p$          | : area of prestressing tendons                                               |
| $A_v$          | : area of shear reinforcement within $s$                                     |
| $A_{v,min}$    | : required minimum transverse reinforcement area within $s$                  |
| $b, b_w$       | : width of the beam                                                          |
| $c$            | : depth of the neutral axis from the extreme compression fiber               |
| $C$            | : compression force                                                          |
| $C_c$          | : concrete compression force for flexural resistance                         |
| $C_s$          | : compression force of compression reinforcement for flexural resistance     |
| $Co$           | : cobalt                                                                     |
| $Cr$           | : chromium                                                                   |
| $Cu$           | : copper                                                                     |
| $c/c$          | : center-to-center                                                           |

|          |                                                                                                                           |
|----------|---------------------------------------------------------------------------------------------------------------------------|
| $d$      | : tension effective depth (distance between the centroid of the tension reinforcement and the furthest compression fiber) |
| $d'$     | : compression effective depth                                                                                             |
| $d_b$    | : bar diameter of longitudinal reinforcement                                                                              |
| $dM$     | : change in moment                                                                                                        |
| $d_{st}$ | : bar diameter of transverse reinforcement                                                                                |
| $dx$     | : change of width                                                                                                         |
| $d_v$    | : effective shear depth                                                                                                   |
| $E_c$    | : concrete elastic modulus                                                                                                |
| $E_p$    | : elastic modulus of prestressing tendons                                                                                 |
| $E, E_s$ | : elastic modulus of non-prestressed reinforcement                                                                        |
| $f'_c$   | : concrete compressive strength                                                                                           |
| $Fe$     | : iron                                                                                                                    |
| $F_{lc}$ | : required tension force on compression longitudinal reinforcement                                                        |
| $F_{lt}$ | : required tension force on tension longitudinal reinforcement                                                            |
| $f_{po}$ | : stress in prestressing tendons                                                                                          |
| $f_s$    | : stress on tension longitudinal reinforcement                                                                            |
| $f'_s$   | : stress on compression longitudinal reinforcement                                                                        |
| $f_y$    | : yield strength of non-prestressed steel reinforcement                                                                   |
| $F_y$    | : yield strength of Fe-SMA                                                                                                |
| $f_u$    | : ultimate tensile strength of non-prestressed steel reinforcement                                                        |
| $F_u$    | : ultimate tensile strength of Fe-SMA                                                                                     |
| $G_s$    | : specific gravity                                                                                                        |
| $h$      | : depth of the beam                                                                                                       |
| $I$      | : moment of inertia                                                                                                       |

|          |                                                                      |
|----------|----------------------------------------------------------------------|
| $k_1$    | : development length modification factor based on bar location       |
| $k_2$    | : development length modification factor based on coating            |
| $k_3$    | : development length modification factor based on concrete density   |
| $k_4$    | : development length modification factor based on bar size           |
| $\ell$   | : total length of the beam                                           |
| $\ell_1$ | : length of the transfer spans of the beam                           |
| $\ell_2$ | : length of the test span of the beam                                |
| $l_d$    | : development length of tension longitudinal reinforcement           |
| $l_{db}$ | : basic development length of compression longitudinal reinforcement |
| $l_n$    | : clear span                                                         |
| $L$      | : length                                                             |
| $M$      | : internal moment (Section 2.1)                                      |
|          | : martensite (Section 2.3)                                           |
| $M^d$    | : detwinned martensite                                               |
| $M_d$    | : terminal temperature for martensite transformation                 |
| $M_f$    | : factored applied moment (Section 2.1, Chapter 3)                   |
|          | : martensite-finish-temperature (Section 2.3)                        |
| $Mn$     | : manganese                                                          |
| $M_r$    | : factored flexural resistance                                       |
| $M_s$    | : martensite-start-temperature                                       |
| $M^t$    | : twinned martensite                                                 |
| $Ni$     | : nickel                                                             |
| $N_f$    | : factored applied axial load                                        |
| $Q$      | : first moment of the furthest area about the centroidal axis        |

|              |                                                                                    |
|--------------|------------------------------------------------------------------------------------|
| $R_1$        | : Internal reaction force                                                          |
| $R_2$        | : External reaction force                                                          |
| $s$          | : centre-to-centre spacing of transverse reinforcement along the longitudinal axis |
| $s_{Av,min}$ | : spacing of the required minimum transverse reinforcement area                    |
| $Si$         | : silicon                                                                          |
| $s_{max}$    | : maximum spacing of transverse reinforcement                                      |
| $s_z$        | : parameter for crack spacing                                                      |
| $s_{ze}$     | : equivalent value of $s_z$ accounting for aggregate size                          |
| $T$          | : applied tension force                                                            |
| $Ti$         | : titanium                                                                         |
| $T_{max}$    | : activation temperature                                                           |
| $T_s$        | : tension force of tension reinforcement for flexural resistance                   |
| $v$          | : shear stresses                                                                   |
| $V$          | : applied shear force                                                              |
| $V_a$        | : shear resistance due to aggregate interlock                                      |
| $V_{ay}$     | : vertical component of the shear resistance due to aggregate interlock            |
| $V_c$        | : shear resistance due to concrete                                                 |
| $V_{cy}$     | : vertical component of the shear concrete shear resistance                        |
| $V_d$        | : shear resistance due to dowel action                                             |
| $V_f$        | : factored applied shear load                                                      |
| $V_p$        | : shear resistance due to prestressing force                                       |
| $V_r$        | : factored shear resistance                                                        |
| $V_{r,max}$  | : factored shear resistance                                                        |
| $V_s$        | : shear resistance due to transverse reinforcement                                 |

|                     |                                                                                     |
|---------------------|-------------------------------------------------------------------------------------|
| $x$                 | : location along the length                                                         |
| $Zn$                | : zinc                                                                              |
| $\alpha_1$          | : factor accounting for stress distribution of compression block                    |
| $\beta$             | : factor of shear resistance on cracked concrete                                    |
| $\beta_1$           | : rectangular compression block depth over the neutral axis depth                   |
| $\varnothing$       | : diameter                                                                          |
| $\epsilon_{sh}$     | : strain after yielding and at the start of strain hardening                        |
| $\epsilon_{cu}$     | : maximum concrete strain at the extreme compression fiber                          |
| $\epsilon'_s$       | : strain in compression reinforcement                                               |
| $\epsilon_u$        | : ultimate tensile strain                                                           |
| $\epsilon_x$        | : mid-depth longitudinal strain                                                     |
| $\epsilon_y$        | : reinforcement strain at yielding                                                  |
| $\theta$            | : inclination angle of the diagonal compressive stresses from the longitudinal axis |
| $\lambda$           | : concrete density factor                                                           |
| $\rho$              | : reinforcement ratio of tension longitudinal reinforcement (Chapter 3)             |
|                     | : reinforcement ratio (Chapter 5)                                                   |
| $\rho'$             | : reinforcement ratio of compression longitudinal reinforcement                     |
| $\rho_{st}$         | : reinforcement ratio of transverse reinforcement                                   |
| $\phi_c$            | : concrete resistance factor                                                        |
| $\phi_s$            | : steel resistance factor                                                           |
| $\sigma_{expected}$ | : expected prestress in the concrete                                                |
| $\sigma_{R,S}$      | : recovery stress of the Fe-SMA                                                     |

# Chapter 1

## Introduction

### 1.1 Background

In Canada, reinforced concrete (RC) is one of the most utilized materials for the construction and repair of infrastructure (Cusson & Isgor, 2004). However, the deterioration of aging RC infrastructure has become a notorious issue. The 2019 Canadian Infrastructure Report Card (CIRC) states that at least 38% of bridges are in very poor to fair condition with a rehabilitation/replacement cost of about \$21 billion (BluePlan Engineering, 2019; Lafleur, 2023). A 2004 report from the National Research Council on the durability of concrete structures states that 40% of bridges are older than 40 years old and require rehabilitation or replacement (Cusson & Isgor, 2004). Considering that the estimated service life (ESL) of bridges is 50 years (BluePlan Engineering, 2019), it is possible to assume that the percentage of deficient bridges has increased considerably.

Aging bridge infrastructure may not be suitable to withstand increasing traffic loads and climate change effects. Transport and freight weight has considerably increased over the last few years, with bridges supporting higher loads than ever before (Baniya et al., 2018). Climate change has also been linked to RC deterioration. An average temperature climb of 2°C can increase the corrosion rate. RC also experiences additional stress and cracking due to concrete expansion because of extreme heating/freezing and thawing cycles (Dr. Muntasir Billah, PhD. in Lafleur, 2023). Hence, strengthening of existing structures is needed to increase performance and durability.

RC losses strength and stiffness with age and exposure, cracking of RC elements may expose interior rebar and lead to corrosion. Corroded bars increase in area and result in concrete spalling (Milde et al., 2005). Bridges are structures that are exposed to highly corrosive environments such as oxygen, moisture, saltwater, and de-icing salts. It is reported that there is a \$10 billion backlog on the maintenance, repair, or replacement of highway bridges (Cusson & Isgor, 2004). The high cost of bridge replacement and rehabilitation has led to the development of resilient materials and strengthening techniques to improve the strength and performance of RC elements.

### 1.2 Research Need

RC structures are usually designed to experience ductile failure. Shear failure is brittle and sudden, making it difficult to predict. Figure 1.1 presents the shear failure on a bridge beam. Shear behavior

on RC structures is still a complex and unresolved problem (Nadir et al., 2018). To prevent failure, different shear-strengthening mechanisms have been developed.



**Figure 1.1. Bridge Beam Shear Failure (Seim et al., 1999)**

Note. From “Health monitoring of an Oregon historical bridge with fiber grating strain sensors”, by J. M. Seim, E. Udd, W. L. Schulz, and H. M. Laylor, 1999, *Proceedings of SPIE - The International Society for Optical Engineering* 3671, 128-134. Copyright 1999 by Society of Photo-Optical Instrumentation Engineers. Reprinted with permission.

Passive mechanisms are the most common shear-strengthening technique, but they only start acting after the structure reaches a certain damage. It is mostly done by using steel or fiber-reinforced polymers (FRP) (Cladera et al., 2020; Xue et al., 2023a). Xue et al. (2023a) state that passive mechanisms do not reduce cracking nor increase serviceability. Hence, prestressing (active) mechanisms have been an attractive solution for shear strengthening. Active mechanisms start acting after implementation, proving a reduction in crack propagation and width, and increasing stiffness (Abdulkareem et al., 2023). However, shear prestressing has shown to be complex and achieved with expensive and large impractical jacking equipment (Xue et al., 2023a; Xue et al., 2023b; Czaderski et al., 2021).

The research of cost-efficient and simpler retrofitting materials and techniques has led to an interest in the utilization and development of smart materials like shape memory alloys (SMAs). SMAs possess unique thermomechanical properties that allow them to return to their original shape after undergoing large deformations. This is possible due to stress or heat induced phase transformations. The shape memory effect (SME) is a thermally activated property that allows an SMA material to recover plastic deformations. Low-cost iron-based SMAs (Fe-SMAs) have been developed for construction applications. Activating the SME of prestrained and restrained Fe-SMAs results in recovery stresses that can self-prestress RC elements by simply using electrical resistive heating (Soroushian et al., 2001; Schranz et al., 2020 Yeon et al., 2022).

The self-prestress ability of Fe-SMAs is a promising shear-strengthening mechanism to mitigate damage in RC elements without the need for expensive and impractical jacking or anchoring tools. Nevertheless, current methodologies implement complex and invasive installation processes and environment exposure. This research project investigates the performance of a non-invasive external Fe-SMA epoxy composite strengthening method to strengthen shear critical RC members. The results from this project will encourage the development of innovative, cost-efficient, and fact retrofit strategies geared towards maximizing the durability and resistance of RC elements in bridges around Canada.

### **1.3 Objectives**

This project aims to present a novel and efficient active RC shear retrofit solution using the self-prestress capability of Fe-SMAs as a bonded composite. To achieve this goal, four research objectives have been selected:

1. Characterize a commercially available Fe-SMA material and assess its stress versus strain response and its self-prestressing ability as an epoxy composite.
2. Design field-representative shear critical RC beam specimens to be self-prestressed with a bonded Fe-SMA composite.
3. Evaluate the performance of self-prestressed field-representative shear critical RC beam specimens with a bonded epoxy Fe-SMA composite.
4. Numerically model the self-prestressed RC beam specimens to conduct parametric studies of different configurations for the Fe-SMA composite.

### **1.4 Scope**

This project will provide an assessment of a near-surface bonded (NSB) epoxy-resin iron-based shape memory alloy (Fe-SMA) composite for the active retrofit of a shear-critical RC beam. Three shear-critical RC beams have been casted to assess whether the retrofit solution can shear strengthen and increase the performance of the structure. Three main concerns are assessed: load-displacement response, cracking, and yielding of transverse reinforcement.

The material characterization of the Fe-SMA material was done by conducting tensile and recovery stress tests. The applicability of an epoxy-resin Fe-SMA composite is initially evaluated with small

coupon specimens. The implementation of the Fe-SMA composites and strengthening capacity is then evaluated.

The finite element analysis (FEA) modeling of the beams is done with Vector2 to predict failure mode and load requirements. The modeled was verified with CSA A23.3 (2019) and validated with the experimental results. The calibrated models were used to perform two parametric studies to assess the effect of prestress and Fe-SMA ratio.

## **1.5 Structure**

This thesis is organized into the following seven chapters:

Chapter 1 introduces the project. This chapter presents the background and importance of RC shear retrofit innovations, as well as the research needs. Additional information about the project is available such as the objectives, scope, and structure of this thesis.

Chapter 2 provides a literature review. This chapter provides an insight into the theory behind the behavior of RC elements under shear. It presents empirical and analytical models used to predict shear strength, as well as procedures under the Canadian Design of Concrete Structures Standard (CSA A23.3). Moreover, previous research about shear strengthening techniques and Fe-SMAs is presented.

Chapter 3 introduces the experimental program of this project. This chapter shows the material characterization of Fe-SMA and the process of obtaining its recovery stress. It also shows the behavior of small composites. Additionally, this chapter presents the design procedure followed for the design of a shear-critical RC beam, the construction process of the beams, and the application of the epoxy-resin Fe-SMA composite for shear strengthening.

Chapter 4 provides the results of the experimental program and their assessment. The strengthening capacity of the Fe-SMA composite was determined. Additionally, three main concerns were evaluated: yielding of transverse reinforcement, displacement, and cracking.

Chapter 5 presents the FEA modeling of the RC beams in Vector2. The chapter shows the modeling process on Vector2. The verification and validation of the model. The chapter also presents a parametric study of the effectiveness Fe-SMA strengthening.

Chapter 6 summarizes the results of this project.

## Chapter 2

### Literature Review

The literature review will address the behavior of reinforced concrete beams under shear, existing shear strengthening mechanisms on reinforced concrete structures, as well as background on shape memory alloys and their applications in shear strengthening.

#### 2.1 Reinforced Concrete under Shear

The comprehensive understanding of the behavior of reinforced concrete under shear and its subsequent diagonal cracks has been a much-researched topic (Broujerdian et al., 2019). Over the years, various analytical shear models on reinforced concrete have been developed, but they are still unable to provide a comprehensive prediction or justification of shear strength behavior (Wight & MacGregor, 2012; Broujerdian et al., 2019). Broujerdian et al. (2019) also mention that classic solid mechanics cannot explain shear failures below the ultimate strength stresses. Hence, current design codes approach shear by calibrating empirical relations on physical models (Broujerdian et al., 2019).

Sonnenberg et al. (2003) state that shear is endured by concrete via shear compression and aggregate interlock. Shear compression refers to shear transfer along uncracked concrete. On the other hand, aggregate interlock refers to shear transfer across a crack where there is the presence of shear displacement parallel to the crack (Sonnenberg et al., 2003). In reinforced concrete, the steel reinforcement crossing the cracks contributes to shear strength through dowel action (Sonnenberg et al., 2003; Ince et al., 2007). According to Alimrani et al. (2022), direct shear transfer mechanisms can be assessed by testing push-off specimens as seen in Figure 2.1.

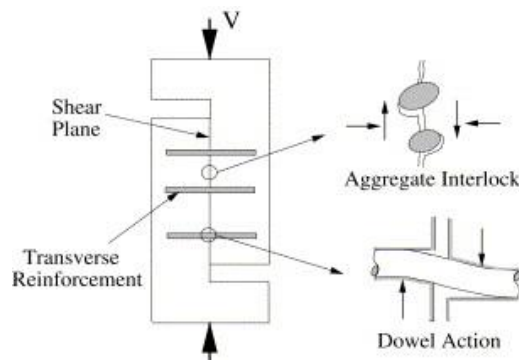


**Figure 2.1. Shear Transfer in Cracked RC Push-Off Specimen (Ince et al., 2007)**

Note. From “Size-dependent response of dowel action in R.C. members”, by R. Ince, E. Yalcin, and A. Arslan, 2007, *Engineering Structures*, 29(6), 955-961. Copyright 2007 by Elsevier. Reprinted with permission.

Birkeland and Birkeland (1966) introduce Shear Friction Theory which states that shear transfer happens due to friction along concrete-to-concrete contact. This theory has been adopted by reinforced concrete design codes and has been modified to include criteria such as dowel action, adhesion, aggregate interlock, and axial restraint (Alimrani et al., 2022; Echegaray-Oviedo et al., 2017). Push-off experimental tests are widely recognized to study shear transfer mechanisms between concrete-to-concrete interfaces (Echegaray-Oviedo et al., 2017). However, Taylor (1970) states that push-off tests may not represent the behavior of concrete in a beam since beams shear at diverse angles.

### 2.1.1 Shear in Beams

Reinforced concrete beams are designed to first resist internal moments ( $M$ ) and then shear ( $V$ ) due to the sudden and brittle nature of a shear failure. Figure 2.2 presents the primarily internal loads in a beam, resulting in the shear force ( $V$ ) being equal to the change in moment ( $dM$ ) over the change of width ( $dx$ ). Wight and MacGregor (2012) mention that the shear design does not depend on a unique criterion but many. Shear failure can depend on the loading conditions, as well as on the geometric and mechanical properties of the beams.

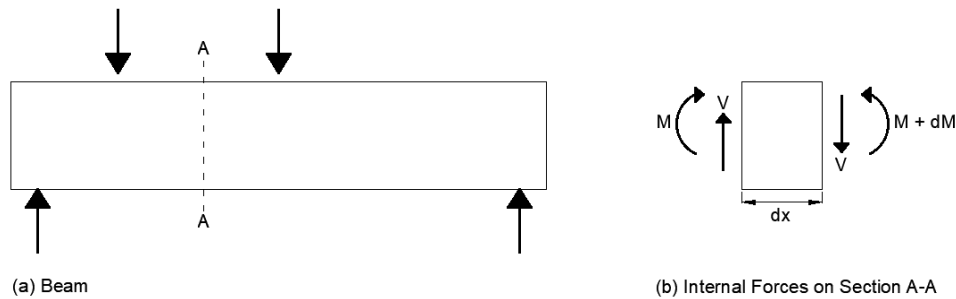


**Figure 2.2. Internal Loads in Beams**

#### 2.1.1.1 Uncracked Beam

Shear stresses ( $v$ ) on an uncracked beam can be calculated by considering the beam to be homogenous and elastic using Equation 2.1.  $Q$  is the first moment of the furthest area about the centroidal axis,  $I$  is the moment of inertia, and  $b$  is the width. Figure 2.3(a) shows the stresses acting on elements along the shear span of a beam, and Figure 2.3(b) shows the shear stress distribution in the cross-section. Shear stresses act vertically and horizontally on the elements and cancel each other. Principal stresses can be determined using Mohr's stress circle, presenting an orientation as in Figure 2.3(c) and following a trajectory as in Figure 2.3(d). The principal stress trajectory is based on the

start of cracking due to the maximum principal tension stress being larger than the tensile strength of the concrete (Wight & MacGregor, 2012).

$$v = \frac{VQ}{Ib} \quad \text{Equation 2.1}$$

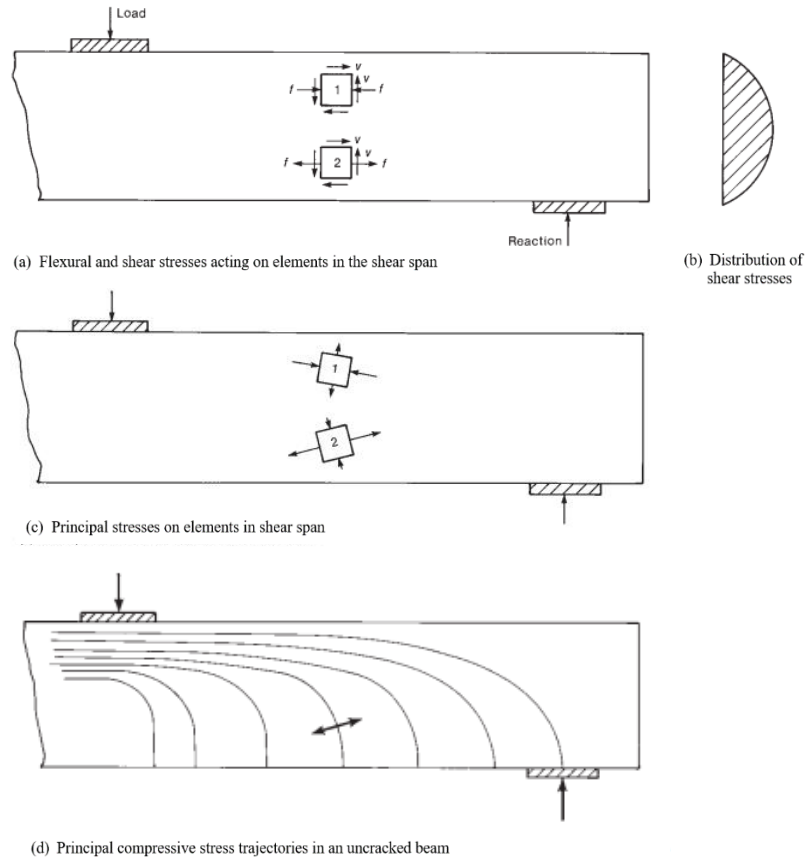


**Figure 2.3. Stresses in an Unracked Beam (adapted from Wight & MacGregor, 2012)**

Note. From *Reinforced Concrete: Mechanics and Design* (p. 245, 246), by J. K. Wight and J. G. MacGregor, 2012, Pearson Education. Copyright 2012 by Pearson Education, Inc. Reprinted with permission.

### 2.1.1.2 Cracked Beam

Cracking usually starts with vertical cracks due to flexure, as the applied load keeps increasing, inclined cracks (shear cracks) appear due to combined flexure and shear. While the cracking pattern can be estimated using the trajectory of the principal stresses, it is not perfect. The maximum principal tensile stress perpendicular to the vertical crack drops to zero, and a redistribution of stresses happens to maintain equilibrium. Hence, shear cracking cannot be predicted with principal

stresses, except when shear cracking happens before flexural cracking (Wight & MacGregor, 2012). The stress redistribution due to inclined cracks results in instability and failure (Chana, 1981).

At the appearance of inclined cracks, the aggregate interlock is the main shear transfer mechanism and dowel action rises. Chana (1981) states that Taylor (1974) shows that shear compression provides 20% - 40% of shear strength on beams without shear reinforcement, aggregate interlock provides 33% - 50%, and dowel action gives 15% - 25%. Figure 2.4 presents the internal force distribution in a beam without shear reinforcement, and Figure 2.5 shows the cracking distribution. In a sagging beam, shear compression happens at the top of a beam under flexure on its compression zone; on the other hand, aggregate interlock happens in the mid-lower region of the beam (Sonnenberg et al., 2003). Shear compression load is represented with  $V_{cy}$ , aggregate interlock with  $V_a$ , and dowel action with  $V_d$ . Additionally, the section presents a horizontal compression force (C) and a tension force (T) in the longitudinal reinforcement.

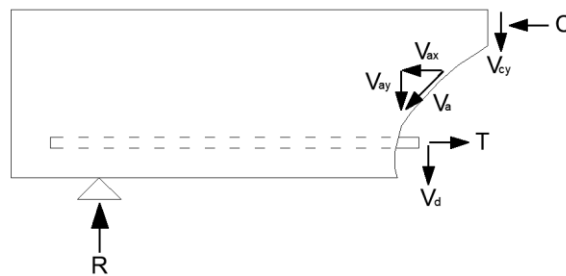


**Figure 2.4. Internal Force Distribution**

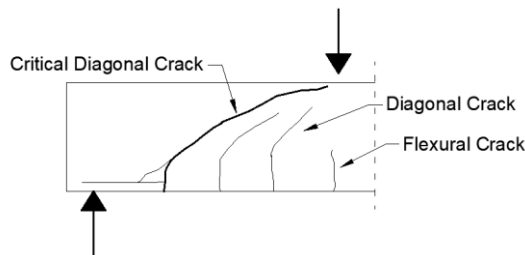


**Figure 2.5. Cracking Distribution**

Shear reinforcement is used along flexural reinforcement to prevent inclined cracks from widening. Shear reinforcement (stirrups) can be inclined or vertical. Inclined stirrups are not commonly used and are not to be utilized under cyclic loading due to the parallel cracking of the stirrup (Wight & MacGregor, 2012). Figure 2.6 shows how inclined cracks progress with flexural and shear reinforcement.

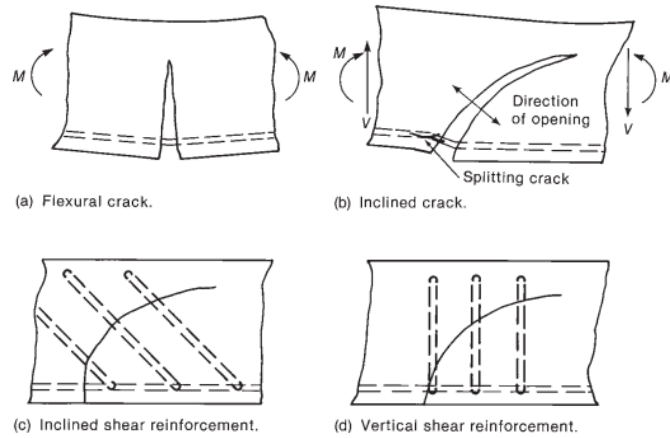


**Figure 2.6. Inclined Cracks in Beams (Wight & MacGregor, 2012)**

Note. From *Reinforced Concrete: Mechanics and Design* (p. 250), by J. K. Wight and J. G. MacGregor, 2012, Pearson Education. Copyright 2012 by Pearson Education, Inc. Reprinted with permission.

Vertical stirrups prevent the reduction of flexural capacity due to inclined cracking and help with the full development of flexural reinforcement. Before cracking, it is assumed that the small strains in the concrete and the stirrups are the same. Stirrups do not prevent inclined cracking and only start contributing to shear strength after concrete cracks. The internal forces in a reinforced concrete beam with stirrups can be seen in Figure 2.7.

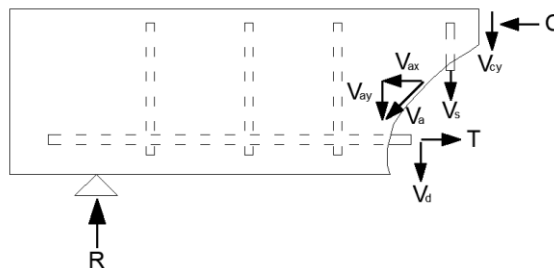


**Figure 2.7. Internal Forces in Cracked Beam with Stirrups**

With the inclusion of vertical stirrups, the shear is also transferred by the stirrups ( $V_s$ ). When the stirrups yield at the crack,  $V_s$  stays constant and the crack significantly widens, reducing  $V_{ay}$ . Therefore,  $V_d$  and  $V_{cy}$  significantly rise until flexural reinforcement splitting or concrete crushing, respectively.  $V_s$  is the only component with ductile response, so there is no clear estimate of the contribution of the other shear transfer mechanisms. In design, shear strength ( $V_r$ ) is simplified as the contribution of concrete ( $V_c$ ) and shear reinforcement ( $V_s$ ).

### 2.1.2 Truss Model

Under shear demand, beams with at least minimum shear reinforcement can be represented with “mechanical-mathematical” models such as a truss model to calculate internal forces in a cracked beam (Wight & MacGregor, 2012). Ritter (1899) proposed that shear in a reinforced concrete beam can be analyzed by modeling it as a  $45^\circ$  truss. This truss analogy would then be supported and expanded by Morsch (1902). A beam truss model is presented in Figure 2.8.

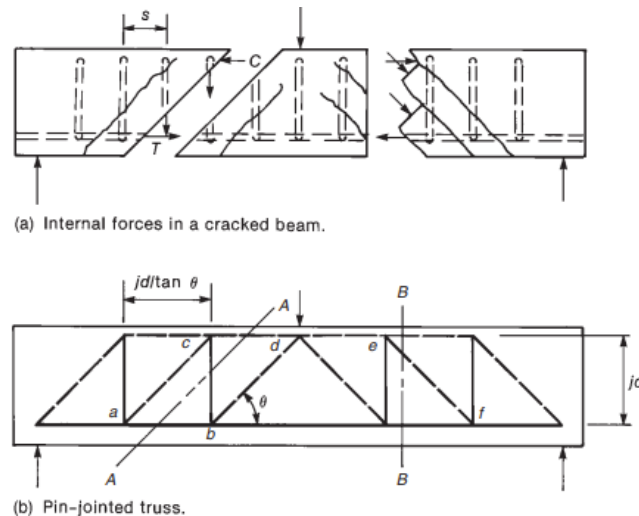


**Figure 2.8. Beam Truss Analogy (Wight & MacGregor, 2012)**

Note. From *Reinforced Concrete: Mechanics and Design* (p. 262), by J. K. Wight and J. G. MacGregor, 2012, Pearson Education. Copyright 2012 by Pearson Education, Inc. Reprinted with permission.

As seen in Figure 2.8(a), shear-inclined cracks result in compression forces at the top, tension forces in the longitudinal reinforcement and stirrups, as well as diagonal compression forces in the concrete. These forces can be translated to a determinate pin-jointed truss model with a compression cord at the top (concrete), a tension cord at the bottom (longitudinal reinforcement), vertical ties (stirrups), and  $45^\circ$  concrete struts as presented in Figure 2.8(b) (Rahal & Collins, 1999; Grandić et al., 2015). The beam truss analogy assumes that the stirrups along section A-A are lumped into tie b-c and the compression forces along section B-B are lumped in strut e-f.

#### 2.1.2.1 Simplified Plastic Truss Model

The stirrup distribution can be estimated based on the plasticity of the transverse reinforcement. A statically determinate simplified plastic truss model can be done based on Marti (1985a, 1985b) for an easier design process. It is assumed that all stirrups yield, and concrete crushing does not happen before stirrup yielding.

An RC beam with transverse reinforcement and under a distributed load can be modeled as a truss with concentrated loads on the top nodes as in Figure 2.9(a). The truss model shows diagonal struts as compression fans in discontinuity regions (D-region) and a compression field in beam regions (B-region). The angle ( $\theta$ ) of the compression field struts depends on the force load equilibrium at the compression fans. A simplified truss model is shown in Figure 2.9(b). The simplified truss model groups all stirrups as a tension tie and applies a diagonal compression strut within a length equal to  $jd / \tan \theta$ , where  $jd$  is the distance between the top and bottom cord.

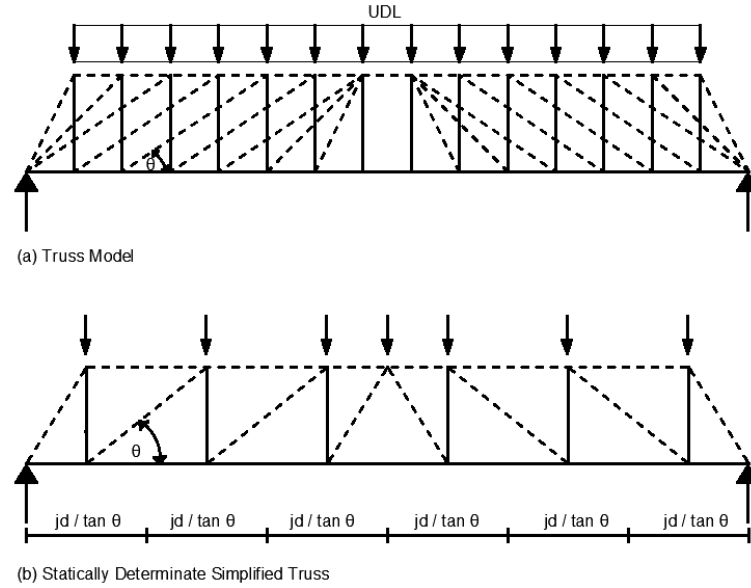


**Figure 2.9. Simplified Truss**

#### 2.1.2.2 Variable Angle ( $\theta$ ) and concrete crushing

Shear RC beams show inclined shear cracks that form angles of  $35^\circ$  to  $45^\circ$  to the horizontal. The inclination of the diagonal compression stresses can be estimated with  $\theta$ . While a reduction of  $\theta$ , decreases the required number of stirrups, it also increases the diagonal compression stresses (Wight & MacGregor, 2012). To prevent crushing of the web, it is necessary to ensure that the inclined compressive stress on the concrete is less than its cracked compressive strength. The general method in CSA A23.3 (2019) provides a variable  $\theta$  that depends on the mid-depth longitudinal strain ( $\epsilon_x$ ).

#### 2.1.3 Modified Compression Field Theory (MCFT)

The determination of the shear strength of RC elements is an ongoing discussion as there is no general rational theory behind it. Traditional shear tests on beams cannot be done under pure shear, so

they are done on elements presenting constant shear and linear changing moment, resulting in variable shear behavior along the beam (Bentz et al., 2006). Vecchio and Collins (1986) propose the Modified Compression Field Theory (MCFT) as a rational analytical model that uses stress-strain relationships to predict load-displacement responses on RC elements (Sadeghian & Vecchio, 2018). This model was verified with experimental tests of RC panels under in-plane shear and normal stresses (Vecchio & Collins, 1986).

Based on Mitchell and Collins's (1974) Compression Field Theory (CFT) for torsion and shear, the MCFT predicts the load-deformation response of a cracked RC element. The MCFT adds tension stresses acting on the concrete between the cracks, and it experimentally verifies the average stress and strain relationships on cracked concrete (Vecchio & Collins, 1986). Three relationships govern the models: equilibrium (sum of forces), compatibility (geometric), and constitutive (average stress-strain). Equilibrium is met by considering that the applied loads are resisted by the stresses in the concrete and reinforcement. Compatibility assumes that the strains in the concrete are equal to the strains in the reinforcement. Finally, it is assumed that the average stress-strain relationships for concrete and reinforcement are independent (Vecchio & Collins, 1986; Bentz et al., 2006). Figure 2.10 shows the stress-strain equations of the MCFT.

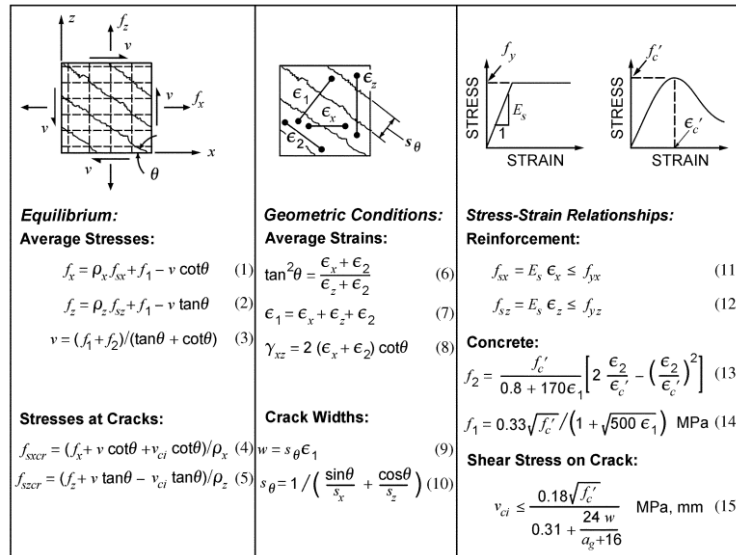


**Figure 2.10. MCFT Equations (Bentz et al., 2006)**

Note. From “Simplified Modified Compression Field Theory for Calculating Shear Strength of Reinforced Concrete Elements”, by E. C. Bentz, F. J. Vecchio, and M. P. Collins, 2006, *ACI Structural Journal*, 103(4). Copyright 2006 by American Concrete Institute (ACI). Reprinted with permission.

MCFT considers reinforcement to be uniformly distributed and perfectly bonded to the concrete. MCFT also assumes that stress states on RC elements are not affected by previous loading conditions, average stress/strain are used for regions/areas passing through several cracks, out-of-plane shear stresses in rebar are not considered, and concrete principal stresses/strains coincide (Holt et al., 2022). The model can iteratively determine the stresses that cause given strains, or vice versa, in an RC element under flexure, shear, or axial loads (Vecchio & Collins, 1986).

Solving the MCFT equations to calculate the full load-deformation response is complex and may require specialized software, making it impractical for design and code applications (Bentz et al., 2006). Bentz et al. (2006) propose a simplified MCFT for code implementation by providing simple equations to find the diagonal compressive stress angle ( $\theta$ ) and the coefficient  $\beta$  that considers the influence of tensile stresses in cracked concrete. The simplified MCFT has been able to predict accurate conservative estimates for shear strength compared to using the full MCFT equations (Bentz et al., 2006).

#### **2.1.4 Canadian Standard for the Design of Concrete Structures (CSA A23.3)**

CSA A23.3. shear provisions have been developed over the years by considering analytical and empirical models. The 45° beam truss model by Ritter (1899) was the basis for determining the shear strength contribution of transverse reinforcement. Additional experimental tests were assessed, concluding that concrete also contributes to shear strength (Rahal & Collins, 1999). The original CFT proposes a rational calculation for  $\theta$ , and the improved MCFT considers the remaining tensile stresses in between cracks.

The MCFT can provide accurate load-displacement predictions for RC elements between cracking and ultimate strength. Shear provisions on CSA A23.3 are based on the MCFT (Rahal & Collins, 1999; Bentz & Collins, 2006). The standard provides three methods for shear design: simplified or general for flexural regions or a strut-and-tie model. For this project, a shear-critical flexural region has been designed.

CSA A23.3 (2019) states that the shear strength ( $V_r$ ) of an RC concrete beam can be calculated with Equation 2.4.  $V_r$  is the addition of the concrete contribution ( $V_c$ ) due to aggregate interlock, transverse reinforcement contribution ( $V_s$ ), and the vertical component of a prestressing force ( $V_p$ ).  $V_c$  can be calculated with Equation 2.5 and  $V_s$  with Equation 2.6.

$$V_r = V_c + V_s + V_p \quad \text{Equation 2.4}$$

$$V_c = \phi_c \lambda \beta \sqrt{f'_c} b_w d_v \quad \text{Equation 2.5}$$

$$V_s = \frac{\phi_s A_v f_y d_v \cot \theta}{s} \quad \text{Equation 2.6}$$

Where  $\phi_c$  is the concrete resistance factor,  $\lambda$  is the concrete density factor,  $f'_c$  is the concrete specified compressive strength,  $d_v$  is the effective shear depth,  $\phi_s$  is the steel resistance factor, and  $s$  is the transverse reinforcement spacing.

The simplified and general method for shear design of flexural members differs in the determination of the angle of the diagonal compressive stresses ( $\theta$ ) and factor ( $\beta$ ) which reflects the shear resistance on cracked concrete. The general method is used for more accurate shear strength estimates. Bentz and Collins (2006) proposed the derivation of the simplified method from the general method. The methods and conditions can be found in Clause 11.3.6. in CSA A23.3 (2019).

The simplified method assumes a mid-depth longitudinal strain ( $\epsilon_x$ ) of  $0.85 \times 10^{-3}$  based on the flexural failure of 400 MPa reinforcement. It can be used in RC elements with reinforcement presenting a yield strength of up to 400 MPa and concrete compressive strength of up to 60 MPa.  $\theta$  is established as  $35^\circ$  while  $\beta$  is determined based on transverse reinforcement and maximum nominal size of coarse aggregate ( $a_g$ ). If a section has at least minimum transverse reinforcement,  $\beta$  is taken as 0.18. If there is no transverse reinforcement,  $\beta$  can be found with Equation 2.7 when  $a_g$  is less than 20 mm, otherwise with Equation 2.8.  $s_z$  is a parameter for crack spacing, and  $s_{ze}$  is the equivalent value of  $s_z$  (CSA, 2014).

$$\beta = \frac{230}{1000 + d_v} \quad \text{Equation 2.7}$$

$$\beta = \frac{230}{1000 + s_{ze}} = \frac{230}{1000 + \frac{35 s_z}{15 + a_g}} \quad \text{Equation 2.8}$$

For this project, the general method was used for the design of the flexural regions to obtain accurate shear strength predictions. The design of discontinuity regions is not part of the scope of this project, so a certain amount of reinforcement was selected to prevent shear failure.  $\theta$  can be found in Equation 2.9, and  $\beta$  with Equation 2.10., where  $s_{ze}$  is 300 mm if at least minimum reinforcement is present. The general method provides Equation 2.11 to calculate  $\epsilon_x$  considering the applied loads, longitudinal steel, and prestressing tendons (if applicable).

$$\theta = 29 + 7000 \epsilon_x \quad \text{Equation 2.9}$$

$$\beta = \frac{0.40}{1 + 1500 \varepsilon_x} \times \frac{1300}{1000 + s_{ze}} \quad \text{Equation 2.10}$$

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f - V_p + 0.5 N_f - A_p f_{po}}{2 (E_s A_s + E_p A_p)} \quad \text{Equation 2.11}$$

Where  $M_f$  refers to the applied moment,  $V_f$  to the applied shear, and  $N_f$  to the applied axial force. The longitudinal reinforcement area is  $A_s$ , and its elastic modulus is  $E_s$ . If applicable, the total area of the prestressing tendons is  $A_p$ , their elastic modulus is  $E_p$ , and their stress is  $f_{po}$ .

## 2.2 Shear Strengthening on Existing RC Elements

Shear strengthening in RC elements can be done using non-prestressed (passive) or prestressed (active) mechanisms (Xue et al., 2023a). Rius et al. (2019) mention that passive methods require the strengthened RC elements to reach a certain damage before starting to work. On the other hand, active methods start working after installation. Shear strengthening of beam-type RC elements can be achieved using externally anchored/bonded or near-surface-mounted techniques.

### 2.2.1 Shear Failure Identification in RC Beams

Concrete deterioration happens due to aging, flawed construction, environmental exposure, and incremental loading conditions (Broujerdian et al., 2019). Broujerdian et al. (2019) comment that initial imperfections and the propagation of cracks are the leading causes of the failure and collapse of reinforced concrete structures. One of the most concerning failure mechanisms is shear. Structures are usually designed to fail in flexure since shear failure is brittle and sudden (Abdulkareem et al., 2023).

In bridges, RC pier caps, also known as headstocks, and girders are beam-type specimens that may present critical shear cracking. The Ontario Structure Inspection Manual (OSIM) comments that concrete cracking may happen due to the applied and restraint loads, deformations, settlements, or reinforcement corrosion. Cracking starts as harmless hairline cracks, as the applied load continues to stress the reinforcement, these cracks start to widen and spread. An evaluation is required to assess the strength reduction of the structure (Ministry of Transportation of Ontario, 2008). Figure 2.11 and Figure 2.12 show shear cracks on pier caps and girders, respectively. The severity of the cracks is classified based on the recommendations from the 2010 Bridge Inspection Manual classifies the severity of cracks as presented in Table 2.1.

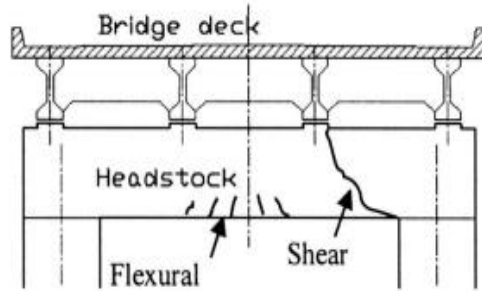


**Figure 2.11. Pier cap cracks (Nezamian & Setunge, 2007)**

Note. From “Case Study of Application of FRP Composites in Strengthening the Reinforced Concrete Headstock of a Bridge Structure”, by A. Nezamian and S. Setunge, 2007, *Journal of Composites for Construction*, 11(5). Copyright 2007 by American Society of Civil Engineers. Reprinted with permission.

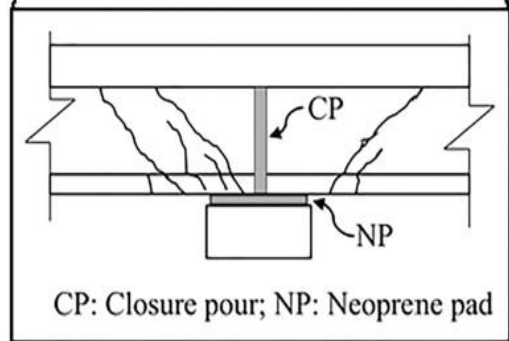
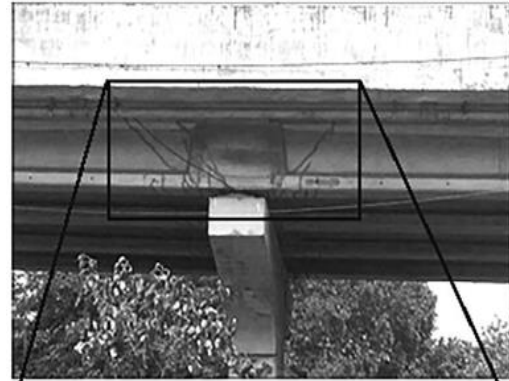


**Figure 2.12. Girder Shear Cracks (Mohandoss et al., 2021)**

Note. From “Determining bond strength of seven-wire strands in prestressed concrete”, by P. Mohandoss, R. G. Pillai, and G. Ravindra, 2021, *Structures*, 33. Copyright 2021 by Elsevier. Reprinted with permission.

**Table 2.1. Crack Severity (Public Works and Government Services Canada, 2010)**

|          |                      |
|----------|----------------------|
| Hairline | < 0.1 mm wide        |
| Narrow   | 0.1 mm – 0.3 mm wide |
| Medium   | 0.3 mm - 1.0 mm wide |
| Wide     | > 1.0 mm wide        |

## 2.2.2 Passive Mechanisms

Shear strengthening with passive mechanisms is not immediate. The strengthening material starts acting only after the element reaches a certain damage or deformation (Rius et al., 2019). The

majority of shear-strengthening mechanisms are passive (Cladera et al., 2020). Xue et al., (2023a) mention that passive methods are usually implemented by using steel plates or fiber-reinforced polymers (FRP) strips, sheets, or plates. Other methods that have been researched for RC retrofit are self-compacted concrete jacketing and steel fiber-reinforced concrete (Rius et al., 2019). Over the last few years, FRP sheets have become widely popular for non-prestressed shear-strengthening applications. However, passive mechanisms do not result in crack reduction nor serviceability or stiffness increase, so there has been a rise in the development of prestressed mechanisms (Xue et al., 2023a; El-Hacha & Soudki, 2013).

### **2.2.3 Active Mechanisms**

Active (prestressed) mechanisms start acting immediately after implementation, improving serviceability, and reducing shear crack propagation and width (Abdulkareem et al., 2023). Abdulkareem et al. (2023) mention that aggregate interlock increases due to the reduction of crack widths, leading to a decrease in the load on the stirrups. Various active mechanisms have been researched such as prestressed steel or FRP sheets or tendons (Yang et al., 2022; Xue et al., 2023a). Post-tensioning is the most common prestressing method, but the installation may be complex, needing specialized tools, and it may not be practical in limited spaces (Xue et al., 2023a; Yang et al., 2022; Czaderski et al., 2021).

### **2.2.4 External Strengthening**

External plates/strips can be implemented for the retrofit of existing RC structures. Shear strengthening on beams is done on the sides/web of the beam. These plates/strips can be made from different strengthening materials like steel or FRP which can be either bolted or bonded to the concrete surface (Oehlers, 2001). Adhesive bonding provides a full interaction bond compared to the partial interaction bond from bolting. Bolting results in slippage between the beam and the plate/strip, failing to deliver equivalent increments in strength and stiffness.

While adhesive bonding is prone to brittle failure due to mechanical debonding, bolting allows ductile failure by accommodating diagonal crack formation. Both methods act independently, and their combination may result in a stiffness increase at serviceability levels while ensuring ductility at ultimate levels. For active shear strengthening, mechanical fasteners in the web may prevent the proper performance of prestressed strips (Czaderski et al., 2021). Epoxy bonding has been a very

popular method for external strengthening due to being non-invasive, quick, and cost-efficient mechanical retrofit (Oehlers, 2001).

The effectiveness of the retrofit lies in the capacity of the adhesive to transfer the stresses between the concrete and the strengthening material (Al-Zu'bi et al., 2022). External bonding (EB) and near-surface mounted (NSM) techniques are prevalent approaches for external strengthening that employ epoxy adhesives. Wang et al. (2020) mention that bonding performance is better in NSM techniques compared to EB. EB refers to the process of bonding the material to the surface of a structure while NSM places the strengthening material into an adhesive-filled groove, limiting environmental exposure. Figure 2.13. shows EB and NSM methods.

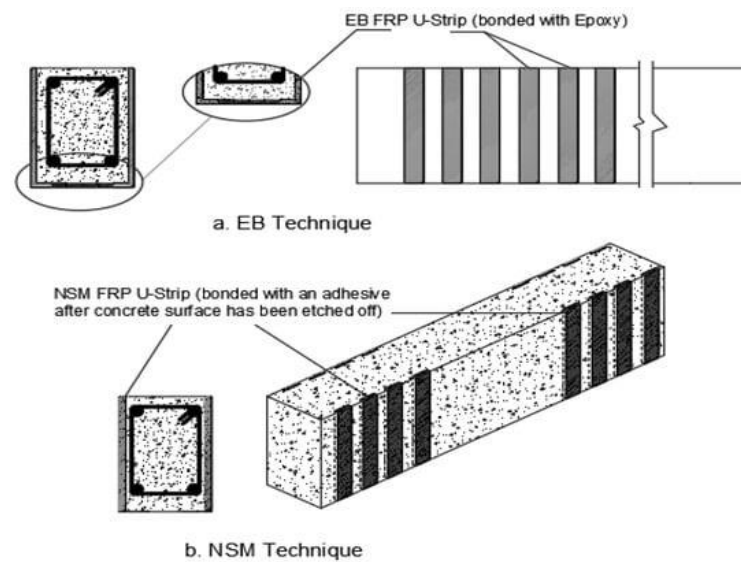


**Figure 2.13. Shear External Strengthening Techniques (Effiong & Ede, 2022)**

Note. From “Experimental Investigation on the Strengthening of Reinforced Concrete Beams Using Externally Bonded and Near-Surface Mounted Natural Fibre Reinforced Polymer Composites—A Review”, by J. U. Effiong and A. N. Ede, 2022, *Materials*, 15(17), 5848. Copyright 2022 by MDPI. CC BY 4.0.

Research with FRP has shown that EB techniques tend to delaminate, resulting in FRP not being fully utilized. NSM techniques protect the strengthening material from the environment, result in higher bonding capacity, and mitigate the propagation of crushing cracks and cover separation (Effiong & Ede, 2022). Barros et al. (2007) evaluated the load carrying capacity of RC beam specimens using different CFRP shear strengthening mechanisms. It was found that EB CFRP resulted in a strength average increase of 54% while NSM CFRP lead to an average increase of 83%.

El-Hacha & Soudki (2013) assess prestressed NSM FRP strengthening mechanisms and found that increasing the width of the groove provides greater bond strength and mitigates debonding cracks. De Lorenzis and Teng (2007) mention that Blaschko (2003) advice a groove depth and width 3 mm larger than the strip dimensions. Parreti and Nanni (2004) suggest a minimum groove width of 1.5 times the strip width and a minimum depth of 3 times the thickness. De Lorenzis and Teng (2007) also mention that groove depth may be proportional to strength increase; however, concrete may crush before the epoxy cracks when using deeper grooves.

El-Hacha & Soudki (2013) indicate the advantages of NSM compared to EB techniques. The strengthening material is not exposed to hazards nor damage, stronger bond, and smaller prepared surface, limiting built-in moisture. Yaqub et al. (2024) used NSM Fe-SMA rebars for shear strengthening of prestressed concrete I-girders, finding a shear capacity increase of 46-47%. Due to the advantages of NSM, this project will use this method for the active shear strengthening of RC beams. The width and depth of the groove are selected based on the provided limitations and the available tools. Throughout this thesis this method will be described as near-surface bonded (NSB) since the Fe-SMA is bonded to the concrete by being embedded into epoxy filled grooves.

#### 2.2.4.1 Bond Performance

Hakim et al. (2021) state that debonding happens when the strengthening reinforcement separates from the concrete. A part of the concrete adheres to the reinforcing composite. It has been found that NSM techniques may increase bond strength. Stiffer and stronger epoxy may lead to a reduction of the shear contribution of the strengthening material and rapid debonding due to higher bond stresses transferred to shorter portions. A strong local bond develops as the dimensions of the groove increases. The properties of the epoxy may influence cracking or shear failures within the epoxy. Low elasticity modulus and tensile strength result in a weak and ductile bond-slip behavior where the bond shear stresses are transmitted to a longer length on the strengthening material (Rizzo & De Lorenzis, 2009; Galati & De Lorenzis, 2009).

#### 2.2.4.2 NSM Application

NSM can be implemented by placing a strip/plate of the strengthening material in a groove and filling it with epoxy adhesive or cement mortar. Specialized epoxy adhesive may have the compressive strength to withstand a prestressing mechanism. Al-Zu'bi et al. (2022) mention that adhesives are

widely used for the rehabilitation of aging RC structures. A shear connection is formed between the concrete-epoxy-strengthening material, forming a composite.

Bonding depends on adequate adhesive selection and bonding surface preparation. It is paramount that the shear strength of the resin is at least that of concrete to ensure long-term usage (Eberline et al., 1988). The following NSM implementation process is based on Yaqub et al. (2024) and Kreit et al. (2008):

1. Cut and shape shallow grooves to the desired depth and width.
2. Clean grooves to remove any laitance and ensure adequate bonding.
3. Add a layer of epoxy adhesive into the groove.
4. Clean strengthening material to ensure no oil, dirt, or rust may affect bonding.
5. Place cleaned strips/bars of the strengthening material inside the groove and pressed.
6. Fill the groove and let the epoxy cure.
7. Level the surface.

#### 2.2.4.3 General NSM Failure Mechanisms on Shear Strengthened Beams

Jedrzejko et al. (2023a) summarizes general failure mechanisms on beams shear strengthened with NSM FRP into three categories: flexural, NSM FRP rupture, and debonding as seen in Figure 2.14. Flexural failure occurs by yielding of the longitudinal reinforcement or compressive concrete crushing. The rupture of the NSM FRP can be caused by an intersecting critical diagonal crack (Figure 2.14(a)). Out of the three failure mechanisms, debonding is the most complex and the most common for FRP composites (Jedrzejko et al., 2023a; Mostofinejad et al., 2016). The most common debonding failure modes for NSM FRP are: interfacial debonding (Figure 2.14(b)), side concrete cover separation (Figure 2.14(c)), and hybrid interfacial debonding and side concrete cover separation (Figure 2.14(d)) (Jedrzejko et al., 2023a; Jedrzejko et al., 2023b).

Interfacial debonding occurs as major diagonal cracks intercept and are interrupted by the NSM FRP. As the width of the diagonal crack increments, there is an increase of shear stresses in the NSM FRP – concrete interface. This results in the development of longitudinal cracks along the grooves that widen and cause the detachment of the NSM FRP composite from the web (Jedrzejko et al., 2023a; Jedrzejko et al., 2023b). In u-wrapped strengthening configurations, interfacial debonding

tends to happen at the end portions of the NSM FRP as shown in the experimental testing of a prestressed u-wrapped NSM FRP device for shear strengthening by Jedrzejko et al. (2023b).

Reducing the spacing of NSM FRP may lead to the delamination of the concrete side cover. The presence of epoxied grooves significantly disrupts diagonal cracking, thereby reducing the formation of critical diagonal cracks. However, a dense NSM FRP-concrete interface creates a weakened stirrup plane. Longitudinal cracks, running along length of the beam, emerge at the terminations of major diagonal cracks at the top/bottom planes, causing the side concrete covers to separate (Jedrzejko et al., 2023a; Jedrzejko et al., 2023b).

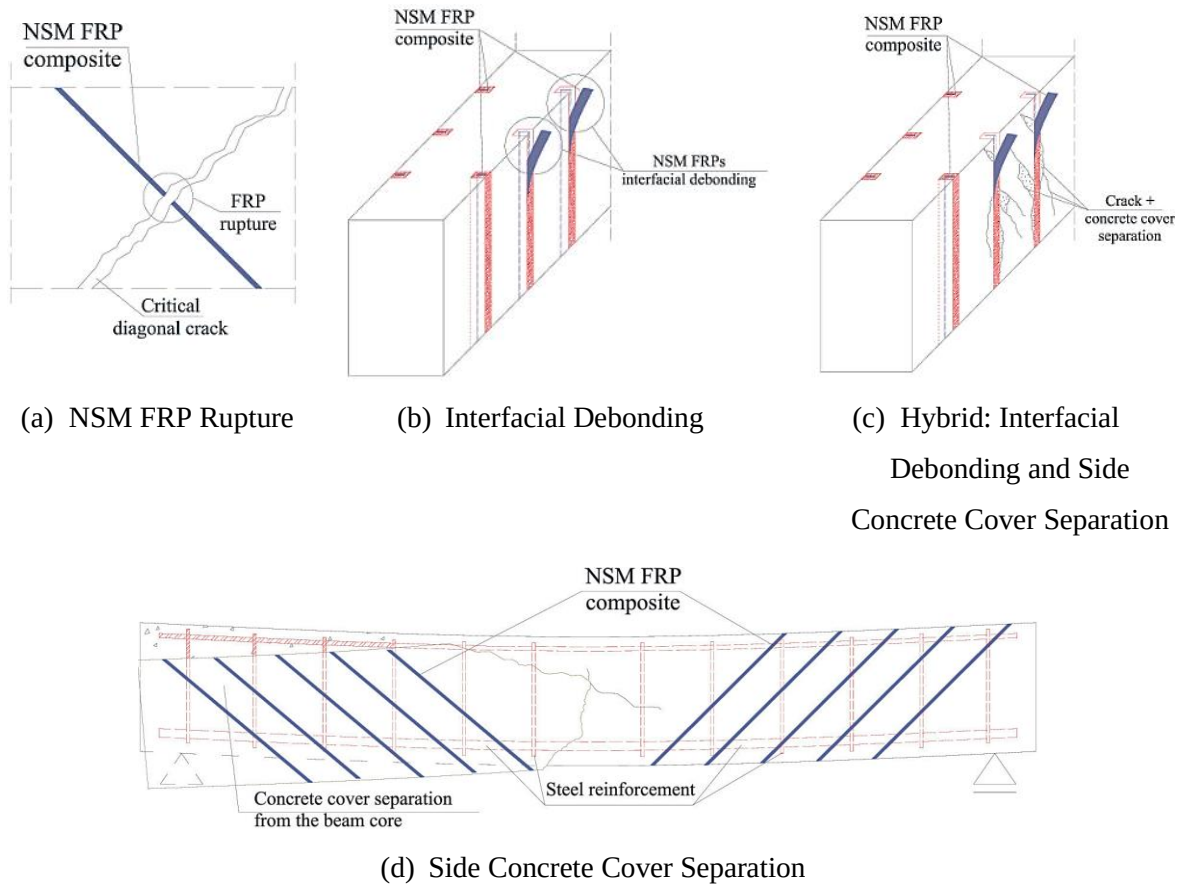


**Figure 2.14. NSM Failure Modes of Shear Strengthened Beams (Jedrzejko et al., 2023a)**

Note. From “Shear strengthening of RC beams with NSM FRP. I: Review of strength models”, by M. J. Jedrzejko, S. S. Zhang, Y. Ke, D. Fernando, and X. F. Nie, 2023, *Advances in Structural Engineering*, 26(3), 564-586. Copyright 2023 by SAGE Publications Ltd. Journals. Reprinted with permission.

## 2.2.5 Shear Strengthening of RC Beams

### 2.2.5.1 Steel

RC bridge strengthening with epoxy-bonded steel plates has been implemented all around the world, but its application in North America is limited (Eberline et al., 1988). Eberline et al. (1988) mention that the EB steel plates tend to be in the tension zones of a beam but can also be placed along its shear zones. Steel strengthening was implemented to ensure ductile failure. Adhikary et al. (2000) mention that bonded steel plates are mainly used for flexure and limited research has been done on shear strengthening.

Adhikary et al. (2000) proposes the bonding of a continuous steel plate along the web of the beam for shear strengthening as seen in Figure 2.15. This method requires less anchorage than traditional steel strengthening methods. The four-point loaded beams do not present shear reinforcement to ensure shear failure in the selected region. The plates are placed on both sides 2 mm away from the concrete surface using expansion anchors. The space between the plate and the steel was sealed with a sealing plate and filled with epoxy. The tests showed that as the depth and thickness of the plate increase, so does the shear strength and stiffness of the beam. The strengthened beams also showed more ductility than the control specimens, experiencing larger deflections before failure.

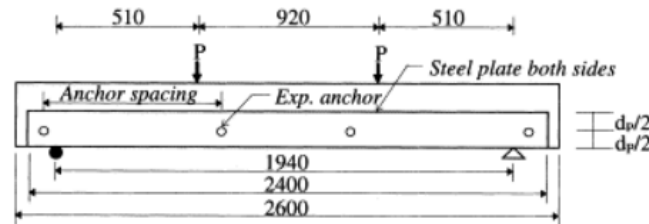


**Figure 2.15. Bonded Steel Plate Strengthening (Adhikary et al., 2000)**

Note. From “Shear strengthening of reinforced concrete beams using steel plates bonded on beam web: experiments and analysis”, by B. B. Adhikary, H. Mutsuyoshi, and M. Sano, 2000, *Construction and Building Materials*, 14(5), 237-244. Copyright 2000 by Elsevier Science & Technology Journals. Reprinted with permission.

Active strengthening mechanisms with steel have also been researched. Yang et al. (2022) assesses the shear-strengthening performance of prestressed steel strips in RC beams. The steel strips were bent into rectangular hoops around the beam and fastened with air-pump stretching equipment. Figure 2.16 shows a strengthened beam layout. The strengthened beams show increased shear capacity and ductility. The tests showed that as the strip spacing decreased, the confinement of the diagonal cracks

delayed critical shear cracking, improving performance. Ductility improved as the layering on the strips increased.

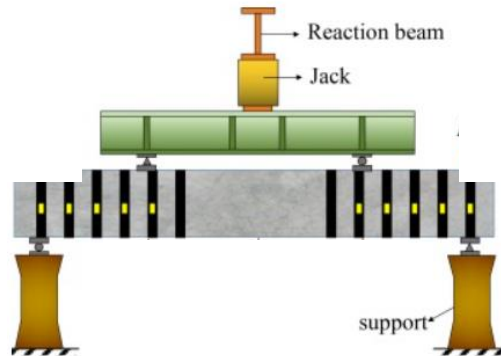


**Figure 2.16. Test Setup (adapted from Yang et al., 2022)**

Note. From “Experimental investigation on shear performance of reinforced concrete beams retrofitted by pre-stressed steel strips”, by Y. Yang, X. Chen, Y. Xue, and Y. Yu, 2022, *Structures*, 43, 767-778. Copyright 2022 by Elsevier Science & Technology Journals. Adapted with permission.

While steel strengthening solutions may be cost and labor-efficient (Yang et al., 2022), they present durability concerns. Steel-to-concrete strengthening methods have been found to experience considerable corrosion and strength loss under environmental exposure, making them a temporary solution (Eberline et al., 1988). NSM strengthening with steel bars embedded in cement mortar had been used on a bridge deck slab in Sweden (Asplund, 1949). El-Hacha & Soudki (2013) mention that steel corrosion is not solved despite NSM strengthening. Hence, exposure-resistant materials like FRP have been researched for RC strengthening.

#### 2.2.5.2 Fiber Reinforced Polymer (FRP) Composites

One of the most promising and effective materials for the repair and strengthening of RC elements is FRP. Over the last decades, FRP has been one of the most utilized materials for the strengthening of RC structures due to their corrosion resistance and high tensile strength. FRP can be EB, anchored, or NSM (Fathalla et al., 2022; Askar et al., 2022). Askar et al. (2022) provide a summary of shear-strengthening studies with FRP. FRP can be wrapped fully or u-wrapped, vertically, diagonally, or side bonded. While an increase in shear strength is noticed with bonded FRP, debonding is a common failure mechanism. Debonding decreases the potential of the FRP composite which experience lower strains under loading. Anchoring the ends of the FRP may mitigate this problem.

FRP composite bonding is one of the most common strengthening procedures, and it is usually done with carbon fiber (CFRP) or glass fiber (GFRP) as EB laminates or NSM bars (Federal

Highway Administration, 2019; Belardi & Acun, 2013). CFRP is popular for RC strengthening due to its superior mechanical properties. On the other hand, GFRP has gained interest due to its lower cost and higher corrosion resistance under alkaline exposure and energy absorption (Liew et al., 2023).

Baggio et al. (2014) test the shear strengthening of bonded FRP systems in four-point bending RC beams. Full-depth u-wrapped CFRP and GFRP sheets performed better than partial-depth u-wrapped configurations. The experimental tests showed that debonding was delayed/prevented by the inclusion of mid-width anchors. Baggio et al. (2014) present that the beams strengthened with un-anchored CFRP sheets showed a 67% increase in shear capacity and 75% if anchored, failing under flexure. On the other hand, beams with un-anchored GFRP sheets presented a 50% increase in shear strength and an additional 13% if anchored. Figure 2.17 shows the FRP for partial u-wrap configuration.

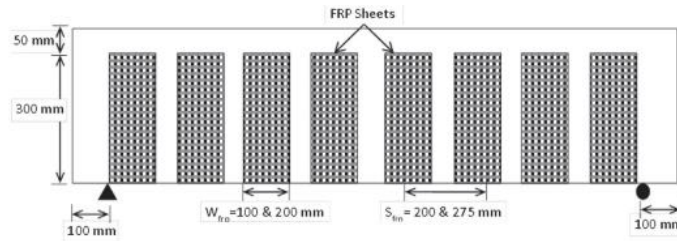


**Figure 2.17. FRP Partial U-wrap (Baggio et al., 2014)**

Note. From “Strengthening of shear critical RC beams with various FRP systems”, by D. Baggio, K. Soudki, and M. Noël, 2014, *Construction and Building Materials*, 66, 634-644. Copyright 2014 by Elsevier Science & Technology Journals. Reprinted with permission.

Liew et al. (2023) assess the shear-strengthening capacity of GFRP in RC beams under four-point bending. GFRP strips were EB with epoxy on one side of the beam at a 45° angle from the horizontal. Figure 2.18 shows the GFRP strengthening. The angle was selected to interfere with the load path from load to support, interrupting the formation of cracks. The tests showed that the strengthening only provided a 2% increase in shear capacity but resulted in considerably less crack width and propagation. Failure of the strengthened beams was due to GFRP debonding.



**Figure 2.18. GFRP Strengthening Single Side Bonding (Liew et al., 2023)**

Note. From “Shear Strengthening of Reinforced Concrete Beams Using GFRP”, by Y. X. Liew, N. Bakar, K. S. Lim, S. I. Doh, R. P. Jaya, and S. C. Chin, 2023, *The Open Civil Engineering Journal*, 17. Copyright 2023 by EUREKA SCIENCE (FZC). CC BY 4.0.

Rizzo and De Lorenzis (2009) experimentally evaluate the shear strengthening capacity of CFRP EB strips and NSM strips and bars at 90° and 45° from the horizontal. The NSM grooves were cut at both web sides of the four-point bending RC beams. NSM resulted in a higher tensile utilization of FRP and a strength increase between 22% to 44% compared to only 16% from EB laminate. Due to the higher bond fracture energy of NSM FRP compared to EB, it is estimated that the shear contribution of the stirrups and the NSM FRP are additive. The study also mentions that increasing the inclination and reducing the spacing of the NSM FRP does not increase shear strength due to specific failure modes.

The experimental tests by Rizzo and De Lorenzis (2009) also showed that the beams strengthened using NSM reinforcement failed due to the separation of the side concrete cover. Decreasing the NSM FRP spacing may not improve the shear strength of the beam and may be detrimental. While a large spacing leads to independent debonding, a close NSM FRP spacing can result in the separation of the side concrete cover. The interaction of the bond actions of adjacent NSM FRP strips can create a debonding failure pattern involving multiple NSM FRP strips. Test results exhibit that diagonal crack widths are reduced significantly in dense NSM FRP strengthening. However, post-failure inspection of the inner core, showed internal diagonal cracks. Rizzo and De Lorenzis (2009) confirms the formation of larger cracks in the inner core by comparing the strains of the stirrups in two specimens with different NSM FRP.

The majority of FRP shear-strengthening mechanisms are passive. Motavalli et al. (2010) describe the usage of CFRP tendons for prestressing. The tendons may be readjusted since they are unbonded to the concrete surface, but the application may be complex and invasive. Studies have also been done with bonded prestressed CFRP. Moshiri et al. (2021) assess the bond resistance of EB prestressed CFRP strips. Askar et al. (2022) mention that prestressed FRP is labor-intensive and expensive, and it may develop creep issues. Czaderski et al. (2021) discuss that prestressing of FRP is complicated, and it can mostly be done only with a proprietary CFRP strap system. Hence, prestressed FRP systems may not be feasible for a cost-efficient bridge-strengthening solution.

#### 2.2.5.3 Shear Strengthening Wrapping Configurations

Wrapping configurations have extensively been assessed in externally bonded FRP. FRP can be externally bonded around the section (full wrap), to the two sides and bottom of the beam (U-wrap), and to its two sides (side bonding) (Teng et al., 2002 in Chen et al., 2012) as seen in Figure 2.19(a). Full wrap improves ductility and can support higher stresses than u-wrap or side bonding

configurations due to their end anchorage (Fathalla et al., 2022). However, depending on accessibility, fully wrapped configurations may not be applicable.

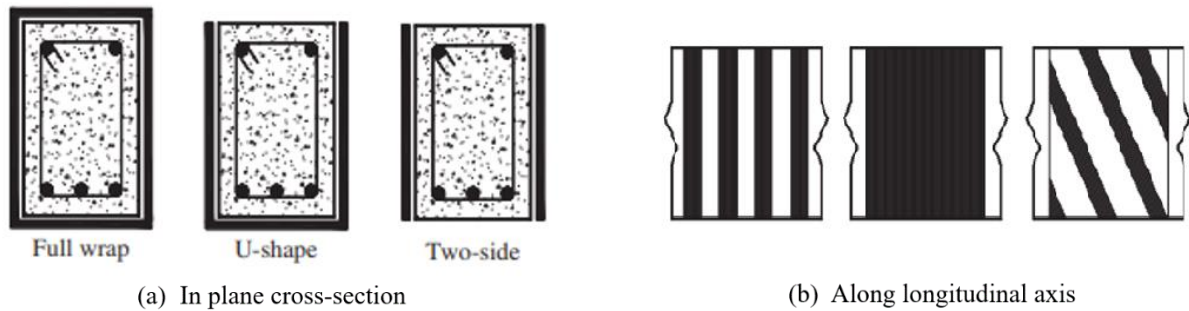


**Figure 2.19. Wrapping Configurations (adapted from Mostofinejad et al., 2016)**

Note. From “Design model for shear capacity of RC beams strengthened with two-side CFRP wraps based on effective FRP strain concept”, by D. Mostofinejad, A. T. Kashani, and A. Hosseini, 2016, *European Journal of Environmental and Civil Engineering*, 20(2), 161-179. Copyright 2016 by Taylor & Francis Informa UK Ltd - Journals. Adapted with permission.

Side bonding can be done by using perpendicular or diagonal FRP strips as presented in Figure 2.19(b). Chaallal et al. (1998) tested externally epoxy-bonded side CFRP strips for shear strengthening. The strips were placed perpendicular or diagonally to the longitudinal axis. Chaallal et al. (1998) found that beams with diagonal side strips show higher stiffness, shear strength, and less crack propagation compared to perpendicular strips. The study also notes that early failure may happen when using side strips due to concrete peeling on tension zones. Liew et al. (2023) mention that a short-spaced 45° side strip may provide the highest shear strength. Chaallal et al. (1998) conclude that the continuity of u-strips may be effective under ultimate loading by reducing the stresses on the strips. However, Mostofinejad et al. (2016) state that ultimate capacity may be affected by stress concentrations at the right corners of the u-strips.

Chen et al. (2012) presents a closed-form analytical solution for the side debonding of u-wrapped and side-bonded FRP strips. The study predicts the debonding of FRP strips along the critical shear crack. The debonding of U-wrapped FRP strips is more complex than for side strips, supporting larger shear loads than side strips. Chen et al. (2012) mention that FRP side debonding is a common failure mode on the majority of u-wrapped and almost all side bonding configurations.

#### 2.2.5.4 FRP Strengthening Limitations and Improvements

The most common failure mode for FRP strengthening is the debonding of FRP composites from concrete, resulting in a fast decrease in capacity (Mostofinejad et al., 2016). The debonding is brittle and limits the width of critical shear cracks, resulting in the stirrups not being fully utilized (yielding).

Therefore, the contribution of the transverse reinforcement to shear may be overestimated (Chen et al., 2012).

FRP research on passive shear-strengthening mechanisms has been done extensively. FRP has also been shown to be effective as a prestressed mechanism. Active (prestressed) mechanisms such as FRP tendons and sheets/plates have been researched due to their ability to act immediately after installation, increasing shear strength and a reduction of crack propagation and width (Xue et al., 2023a). Xue et al. (2023b) state that FRP prestressing is done by using post-tensioning methods that are complex, expensive, and in need of large equipment that may not be applicable for tight spaces. Hence, there is an interest in developing new active strengthening methods.

The brittleness of FRP results in limited ductility and energy absorption compared to smart materials like shape memory alloys (SMAs) (Askar et al., 2022). SMAs have gained notoriety for RC shear strengthening due to their relatively easier capacity to prestress a concrete element with their shape memory effect. Askar et al. (2022) state that using an SMA/FRP composite may improve ductility.

Xue et al. (2023a) tested shear strengthening on beams using prestressed SMA wires anchored at the ends and covered with epoxy-FRP sheets to keep the prestress. Xue et al. (2023a) state that FRP may prevent detachment of the SMA wires and counteract their loss in elastic modulus after activation. The test found that SMA/FRP composites prevent crack propagation and that only using SMA wires may hamper the formation of diagonal cracks. Xue et al. (2023a) share that the specimen shear compression failure happened on the specimen strengthened with SMA wires while the specimens with FRP/SMA composites experienced failure due to composite splitting.

Similarly, Zhu et al. (2023) performed three-point tests in RC beams and assessed the flexural strengthening capacity of NSM SMA, EB CFRP, and both. The NSM SMA test used pre-deformed SMA wires and embedded them into polymer-modified cement (PCM) inside the grooves. The SMA was heated via electricity. The beam prestressed with NSM SMA failed due to concrete crushing in compression while the strengthened beam with EB CFRP failed due to composite debonding. CFRP strengthening reduces ductility, arising safety concerns about failure identification. Therefore, SMAs are promising materials for strengthening applications, offering the potential for enhanced ductility and strength compared to FRP. SMAs may also present a cost-effective prestressing mechanism that may not require specialized large tools, but only a heating source.

## 2.3 Shape Memory Alloys (SMAs)

Progress has been made in the evolution of more durable, stronger, and lighter materials to deal with the engineering problems of today, complexity and sustainability being the most important.

Multifunctional materials have been developed to not only meet structural requirements but also to provide extra functionality such as sensing and actuation. A subgroup of these multifunctional materials is called active materials (Lagoudas, 2008).

Multifunctional active materials experience a non-mechanical input (thermal) that leads to a mechanical output (expansion), with one example being Shape memory alloys (SMAs). SMAs present a direct coupling where the input and output can be interchangeable. SMAs or “smart alloys” are materials that can return to their undeformed shape after experiencing an external stimulus (Lagoudas, 2008; Jani et al., 2014). This deformation recovery has resulted in interest in construction and active strengthening.

### 2.3.1 Background

The martensite transformation of steel was discovered by Adolf Martens in the 1890s (Patel et al., 2020). The concept of martensitic reversible transformation was first introduced by Kurdjumov and Khandros (1949), using CuZn and CuAl alloys. The term “shape-memory” was introduced by Vernon and Vernon (1941) for dental material. In 1961, NiTi (Nitinol) was accidentally discovered in the Naval Ordnance Laboratory. In 1962, it was found that NiTi presents a shape recovery capacity which was named as shape memory effect (SME). In 1965, it was determined that adding an element like Co or Fe would result in a temperature decrease for the shape memory alloy (SMA) transformation (Lagoudas, 2008; Jani et al., 2014; Patel et al., 2020).

Lagoudas (2008) mentions that the commercialization of Nitinol started after its discovery mostly in the biomedical sector, air conditioning, electronics, and valves. Major commercialization started during the 1990s, especially in the aerospace and oil industries where high-temperature conditions were expected. Janke et al. (2005) discuss that SMAs can have different implementations in civil engineering like damping, energy dissipation, vibration control, actuators, couplings, fasteners, sensors, external tensioning, active confinement, and concrete prestressing.

However, the high cost of Nitinol SMA does not make the material a feasible solution for large-scale construction. In 1982, the SME on low-cost iron-based shape memory alloys (Fe-SMAs) was discovered, making them a suitable solution for civil engineering projects (Janke et al., 2005; Cladera

et al., 2014). Czaderski et al. (2021) share that a low-cost Fe-SMA of the form FeMnSi was developed for commercialization.

### 2.3.2 Reversible Phase Transformation

SMA's are known for their deformation recovery ability after experiencing an external stimulus due to reversible martensitic transformations. The alloys tend to present two different crystal structures with their properties. SMA's can be found in a stable low-temperature phase (martensite) and a high-temperature phase (austenite). Martensite (M) and austenite (A) show distinct crystalline structures, austenite is cubic, and martensite can be orthorhombic, tetragonal, or monoclinic (Lagoudas, 2008).

Martensite presents itself in two orientation variants: twinned ( $M^t$ ) and detwinned ( $M^d$ ). Twinned refers to a “self-accommodated” martensite combination, and detwinned refers to a structure with a dominant martensite variant. SMA's also show a reversible transformation between austenite as a parent phase and martensite as a product phase, and vice versa. Transformations between phases are achieved due to shear lattice distortion (Lagoudas, 2008; Janke et al., 2005) due to properties such as shape memory effect (SME) or superelasticity (pseudoelasticity) as seen in Figure 2.20.

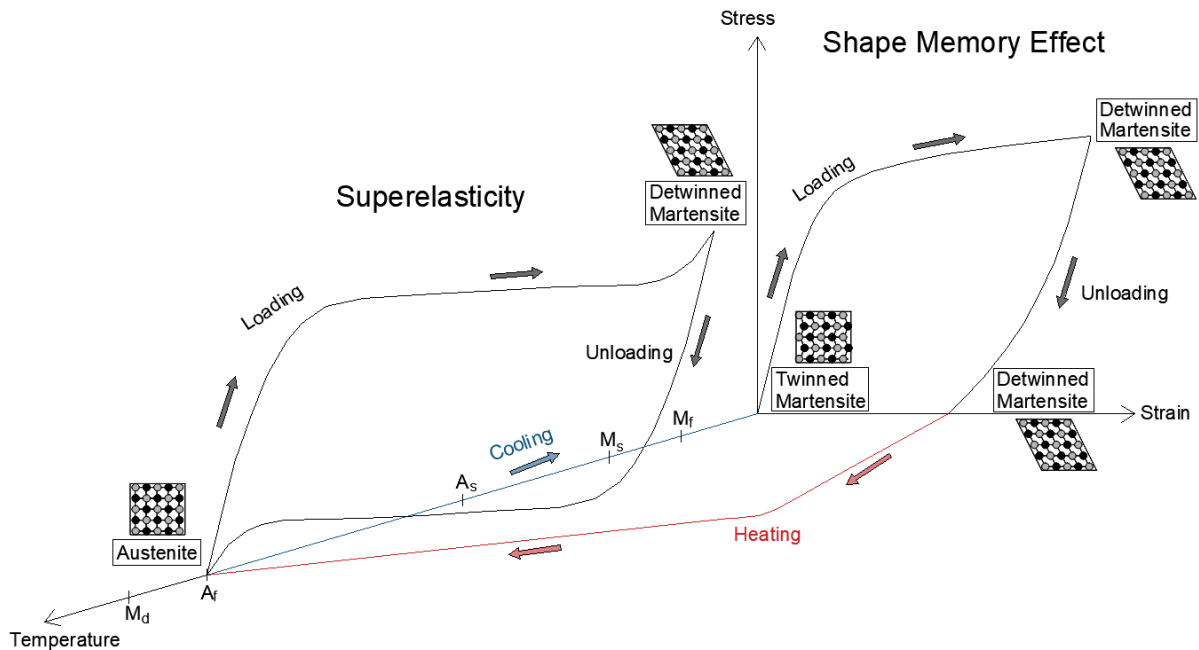


**Figure 2.20. Reversible Phase Transformations (adapted from Lagoudas, 2008)**

Note. From *Shape Memory Alloys: Modeling and Engineering Applications* (p. 7), by D. C. Lagoudas, 2008, Springer. Copyright 2008 by Springer Nature BV. Adapted with permission.

### 2.3.2.1 Superelasticity (Pseudoelasticity)

Superelasticity is a direct transformation due to stress application (Figure 2.20). It occurs on austenitic alloys under working temperatures between the austenite-finish-temperature ( $A_f$ ) and  $M_d$  which is the temperature at which the martensite transformation is no longer possible. The material will be strained when loaded, transforming from austenite to detwinned martensite. While unloading, the material will show a strain recovery, transforming from detwinned martensite to austenite (Lagoudas, 2008; Patel et al., 2020; Jani et al., 2014).

### 2.3.2.2 Shape Memory Effect (SME)

Shape memory effect (SME) is a stress-thermal induced transformation as seen in Figure 2.20. SME can happen even if the material is under load. Materials under their martensite-finish-temperature ( $M_f$ ) and stable at a twinned martensite phase can be loaded and transformed to detwinned martensite. Transformation to austenite is possible with thermal induction. Detwinned martensite starts transforming to austenite at the austenite-start-temperature ( $A_s$ ), and it is completed at the austenite-finish-temperature ( $A_f$ ). As the material cools, transformation from austenite to twinned martensite begins at the martensite-start-temperature ( $M_s$ ), and it is completed at the martensite-finish-temperature ( $M_f$ ), resulting in a strain recovery (Lagoudas, 2008; Patel et al., 2020; Jani et al., 2014). The SME has shown promise in prestressing applications. If the material is pre-strained, restrained, heated to activation, and cooled, it can create a recovery stress.

### 2.3.3 Iron-based Shape Memory Alloys (Fe-SMAs)

The most widely known and commercially available SMAs are NiTi SMAs (Nitinol). The commercial applicability of SME was developed using Nitinol. The alloy is mostly used in the aerospace, automotive, robotic, dental, and biomedical industries (Shahverdi et al., 2018; Alaneme & Okotete, 2016). However, their complex and expensive metallurgical process prevents their large-scale production and their application on structural projects (Zhang et al., 2022; Janke et al., 2005; Shahverdi et al., 2018).

Fe-SMAs are considered a more affordable option compared to NiTi SMAs (Rojab & El-Hacha, 2017). Compared to Nitinol, Fe-SMAs have a higher elastic modulus and lower SME activation temperature (Shahverdi et al., 2018). Fe-SMAs present low-cost raw materials, and their fabrication can be done with conventional steel metallurgical procedures. They can be hot/or cold-formed and produced in various shapes such as bars and strips (Zhang et al., 2022; Shahverdi et al., 2018).

Zhang et al. (2022) discuss the stress-strain response and properties of Fe-SMAs. Their stress-strain response is similar to the one from stainless steel but presents considerably higher strength. Fe-SMAs do not show a yielding plateau, so the yield strength is found using the 0.2% offset method (Shahverdi et al., 2018). Their yield strength may be a bit lower than high-strength steel and aluminum, but they show higher ultimate strength and superior ductility. Additionally, fracture happens shortly after reaching ultimate tensile strength, indicating even plasticity distribution.

Lagoudas (2008) mentions that Fe-SMAs are mainly of the form FeNiCoTi and FeMnSi, the last one being the most promising commercially for its improved SME due to the inclusion of Si. Fe-SMAs can be thermomechanically trained to achieve complete SME strain recovery. Phase transformation strains may range from 2.5% to 4.5% (Otsuka & Wayman in Lagoudas, 2008). Fe-SMAs are a good option for chloride environments and long-term usage due to having corrosion resistance close to stainless steel (Zhang et al., 2022). A low-cost Fe-SMA of the form FeMnSi has been commercialized for construction (Czaderski et al., 2021).

The price of Fe-SMAs is currently higher than structural steel, but if the production demand matches and increases, there is potential for the price to reach the same range as stainless steel (Zhang et al., 2022; Alaneme & Okotete, 2016). Additionally, Fe-SMAs can self-prestress an RC element by activating their SME with only electrical heating without using expensive hydraulic/mechanical equipment. The cost of active RC strengthening mechanisms using Fe-SMA and CFRP is comparable when considering the cost of the prestressing process and labor involved in CFRP (Zhang et al., 2022).

#### 2.3.3.1 Fe-SMA Recovery Stress

By restraining a pretrained Fe-SMA alloy, heating it to phase transformation, and leaving it to cool, the strain recovery is interrupted, and a recovery stress will be generated. Research has been done to determine this recovery stress (RS) based on different parameters such as alloy prestrain, seed load (SL), activation temperature ( $T_{max}$ ), and temperature rate (Shahverdi et al., 2018; Gu et al., 2021). Figure 2.21 shows a stress versus temperature curve of a heating-restrained pretrained Fe-SMA.

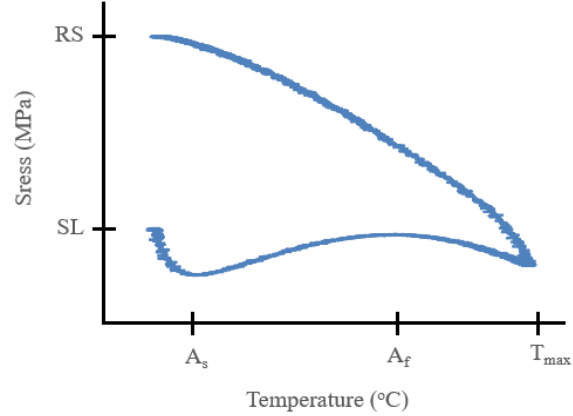


**Figure 2.21. Stress versus Temperature**

During heating, a restrained pretrained Fe-SMA sample experiences a decline in stress due to thermal expansion until it reaches  $A_s$ , after which the stress increases. The sample will experience a second stress decline due to thermal expansion after reaching  $A_f$ . During cooling, if the maximum temperature lands between  $A_s$  and  $A_f$ , the recovery stress tends to increase as the temperature increases. After  $A_f$ , the recovery stress may not increase further. Compression due to thermal expansion should be avoided to prevent the buckling of a restrained sample and damage to the machine. Hence, samples are preloaded (seed load) for material characterization testing (Shahverdi et al., 2018; Gu et al., 2021).

Shahverdi et al. (2018) and Gu et al. (2021) assess the material behavior and characterization of low-cost industrial Fe-SMA strips of the FeMnSi form. The structure of the alloy is Fe-17Mn-5Si-10Cr-4Ni-1(V, C), and it has a high elastic modulus (160000 MPa), tensile strength (1000 MPa), maximum strain (>40%), and SME recovery stress. Both studies state that while increasing the seed load increases  $A_s$ , the recovery stress is minimally influenced. The studies show that higher temperatures result in higher recovery stresses due to a larger amount of phase transformation. Phase transformation may reach a maximum at a certain temperature with recovery stresses not increasing further, or even decreasing. Hence, the SME recovery stress is influenced by the activation temperature and the amount of detwinned martensite after pretraining.

Shahverdi et al. (2018) studies the Fe-SMA recovery stress at 160°C using different prestrains. The alloy shows an increasing recovery stress from 0.5% to 2% where it plateaus. The study determines that the thickness of the specimens and the rate of heating does not influence the recovery stress. However, it should be noted that the rate of heating is not compared to equal temperatures and the recovery stresses range from 300 MPa to 350 MPa. Shahverdi et al. (2018) also suggest that the

recovery stress can be estimated with an equivalent elastic modulus of 95 GPa and the respective recovery strain. Montoya-Coronado et al. (2019) mention that after activation, the modulus of elasticity reduces significantly.

Gu et al. (2021) show that the recovery stress does not significantly change for different prestrains at low temperatures ( $<300^{\circ}\text{C}$ ). At low temperatures, SME is controlled by the activation temperature, so not all martensite generated by prestraining is transformed into austenite. On the other hand, at high temperatures ( $\geq 350^{\circ}\text{C}$ ), higher prestrains lead to higher recovery stresses. This is because the amount of transformed martensite surpasses what was formed during prestraining, resulting in the SME being controlled by the prestrain level. The study also found that the recovery stress is not influenced by the amount of time the activation temperature is maintained before cooling.

#### 2.3.3.2 Fe-SMA RC Shear Strengthening

Fe-SMAs SME has been successfully found to be able to externally strengthen RC structures by prestressing them when restrained and heated to an activation temperature and left to cool. Zhu et al. (2023) mention the majority of SMA strengthening techniques that have been researched are for flexure and use different implementation techniques such as NSM, embedment in shotcrete or mortar, and externally anchored. Xue et al. (2023a) mention that there is limited research on the active shear strengthening of RC beams using the shape memory effect of SMAs.

Zhang et al. (2022) indicate that one of the first field applications of Fe-SMA was done by Soroushian et al. (2001). Soroushian et al. (2001) repaired and strengthened an RC shear critical beam in a bridge with external post-tensioning corrective forces due to SME interrupted recovery strain. 3% prestrained Fe-SMA rods were anchored onto the web of the beam, heated with a 1000-A current (resistive heating) to  $300^{\circ}\text{C}$ , and left to cool to obtain a recovery stress of at least 120MPa. After Fe-SMA posttensioning, the average crack width was reduced by 40%. Application challenges were encountered due to the anchoring mechanism. Predicting the exact location of the internal reinforcement proved to be difficult, so the anchoring holes had to be shifted or enlarged. This resulted in slippage at the end of the anchors and a reduction in the recovery stresses. Figure 2.22 presents the anchored Fe-SMA rods for shear strengthening.



**Figure 2.22. Posttensioned Fe-SMA anchored rods (Soroushian et al., 2001).**

Note. From “Repair and Strengthening of Concrete Structures Through Application of Corrective Posttensioning Forces with Shape Memory Alloys”, by P. Soroushian, K. Ostowari, A. Nossoni, and H. Chowdhury, 2001, *Transportation Research Record*, 1770(1), 20-26. Copyright 2001 by SAGE Publications. Reprinted with permission.

Active Fe-SMA shear strengthening has been researched as a full wrap or U-wrap external retrofit solution. Zerbe (2020) assessed shear-strengthened RC beams with externally anchored Fe-SMA strips. The study showed that a full-wrap configuration is more effective and ductile than a U-wrap configuration. Full wrap resulted in a 100% strength increase while U-wrap only provided a 25% strength increase. Active Fe-SMA strengthening not only delayed cracking force and diagonal cracks but also increased shear strength compared to passive strengthening when there was sufficient prestressing force and strip spacing was at least  $d/2$ . Cracking load and shear strength rose as the Fe-SMA spacing decreased due to higher prestress distribution. Figure 2.23 shows a fully wrapped Fe-SMA specimen.



**Figure 2.23. Full Wrap Fe-SMA Strengthening (Zerbe, 2020).**

Note. From *Strengthening of Shear-Critical Reinforced Concrete Beams using Iron-Based Shape Memory Alloys* (p. 247), by L. Zerbe, 2020, University of Houston Institutional Repository. Copyright 2020 by Lara Zerbe.

Shear strengthening with full-wrap Fe-SMA strips in rectangular RC beams has been studied by Montoya-Coronado et al. (2019) and Traver et al. (2023). Montoya-Coronado et al. (2019) tested

shear-critical small-scale beams without shear reinforcement under three-point bending. The beams were shear strengthened with external mechanically wrapped Fe-SMA strips. The prestrained strips were anchored to themselves with a buckle on top and activated to 160°C. The strengthened beams failed in flexure and showed improvements in shear strength, crack propagation, and ductility due to confinement from the activated Fe-SMA strips. It was found that beams with activated Fe-SMA strips were stiffer and resulted in less deflection compared to beams with passive Fe-SMA strips.

Traver et al. (2023) tested shear-damaged beams strengthened with Fe-SMA strips anchored to themselves to prevent disadvantages from web anchoring. The study found that strengthened beams provide a higher initial strength and ductility, but cracks from previous loading were not repaired. Depending on support conditions, the beams with internal transverse reinforcement showed a shear strength improvement between 28% and 34%. The stirrup shear contribution was between 49.7% and 68.3%, and the Fe-SMA shear contribution ranged from 23.6% to 39.7%. Beams without internal stirrups presented a 109% increase in shear strength, with the Fe-SMA strips providing 50% of the strength. In existing structures, the top of the beam element may be limited, so a U-wrap Fe-SMA strengthening configuration may be applied.

Cladera et al. (2020) and Czaderski et al. (2021) have assessed U-wrap Fe-SMA configurations in shear-critical RC T-beams. Cladera et al. (2020) mechanically anchored the Fe-SMA strips in the upper web sections of the beams (Figure 2.24), increasing shear strength by 30%. Active Fe-SMA strengthening significantly impacts serviceability, increasing initial stiffness, reducing deflection, and delaying cracking. There is no significant difference in the shear strength of the beams strengthened with activated Fe-SMA strips and non-activated Fe-SMA strips. At ultimate states, stresses on the strengthening material are the same at failure. Cladera et al. (2020) also mention that the flange-web connection and ductility may be negatively impacted by web anchoring.

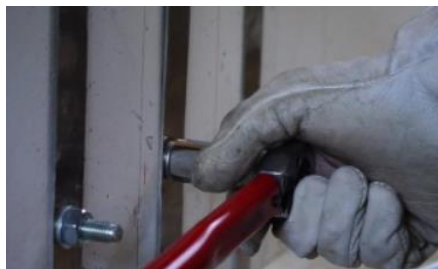


**Figure 2.24. Web Anchored Fe-SMA Strips (Cladera et al., 2020)**

Note. From “Shear strengthening of slender reinforced concrete T-shaped beams using iron-based shape memory alloy strips”, by A. Cladera, L. A. Montoya-Coronado, J. G. Ruiz-Pinilla, and C. Ribas, 2020, *Engineering Structures*, 221, 111018. Copyright 2020 by Elsevier Science & Technology Journals. Reprinted with permission.

Fe-SMAs can also be implemented using NSM techniques. Czaderski et al. (2014) mention that the usage of epoxy resin in NSM methods for Fe-SMA strengthening is not common compared to when using CFRP. The SME activation temperature of the Fe-SMA starts at about 160°C, but commercially available epoxy resins begin glass transition at around 50°C to 70°C. Hence, the NSM method in Fe-SMAs is mostly done using cement-based mortar.

Czaderski et al. (2021) assess the capacity of U-wrapped Fe-SMA ribbed bars embedded in a sprayed mortar layer to repair shear-damaged RC T-beams as seen in Figure 2.25. The bars pass through the flange for electrical heating and are fixed with polymer dowels to the concrete surface before covering them with mortar, with activation happening after curing. It is estimated that the prestress from the activated Fe-SMA bars is 300 MPa. The failure of the specimens happened due to the rupture of the transverse reinforcement and concrete crushing. There was no interface failure between the concrete and the mortar. This method proved to be a feasible active shear strengthening method as it results in a reduction in crack widths, deflections, crack propagation, stirrup stresses, and an increase in shear capacity. The improvement in shear strength may mostly be due to the additional reinforcement ratio.



**Figure 2.25. Fe-SMA Ribbed Bars Embedded into Mortar (Czaderski et al., 2021)**

Note. From “Iron based shape memory alloys as shear reinforcement for bridge girders”, by C. Czaderski, M. Shahverdi, and J. Michels, 2021, *Construction and Building Materials*, 274, 121793. Copyright 2021 by Elsevier. CC BY 4.0.

González Góez (2023) provides an insight on the shear strengthening effectiveness of Fe-SMA strips externally wrapped around push-off specimens (Figure 2.26). The tests were done at the University of Waterloo. The push off specimens show that activated Fe-SMA can self-prestress a concrete element under shear, increasing peak load and reducing crack width. Activated Fe-SMA strips may enhance shear capacity by 34% compared to only 12% using non-activated Fe-SMA strips. The tests also show that doubling the activated Fe-SMA strips increase the peak load by 57%

compared to the control specimen. Hence, activating the Fe-SMA trips to prestress a concrete element may result in enhanced strength and performance.

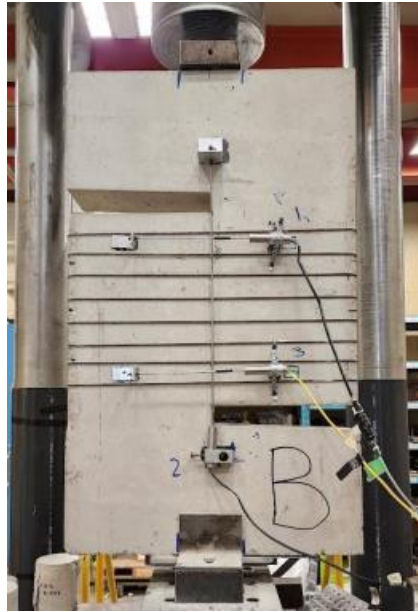


**Figure 2.26. Push-off Specimen strengthened with Fe-SMA strips (González Góez, 2023)**

Note. From *Active Retrofit of Shear Critical RC Components Using Self-Prestressing Iron-Based Shape Memory Alloys* (p. 61), by M. González Góez, 2023, UWSpace. Copyright 2023 by Miguel González Góez.

## Chapter 3

### Experimental Program

#### 3.1 Fe-SMA Material Characterization

This chapter presents the experimental program encompassing Fe-SMA material testing and the development of Fe-SMA epoxy composites for RC shear strengthening. The composite comprises Fe-SMA strips embedded in a high-strength heat-resistant epoxy resin that is NSB to the test span of the beams. Three RC beams specimens have been cast to assess the shear-strengthening potential of the Fe-SMA composite. This chapter is divided into three sections:

1. Fe-SMA material characterization,
2. Epoxy characteristics and the composite recovery stress testing,
3. Characterization of the RC materials, test matrix, and test setup of the shear-critical RC beams.

#### 3.2 Fe-SMA Material Characterization

The shear strengthening Fe-SMA strips utilized in this project are commercially available for the prestressing of concrete structures. The material is used for structural retrofit and is usually implemented as unbonded anchored tensile strips on concrete. The structure of the alloy is Fe-17Mn-5Si-10Cr-4Ni-1(V, C). The material characterization was done using an MTS C45 testing frame. To estimate the recovery stress, a power source was utilized.

##### 3.2.1 Tensile Test

The Fe-SMA was acquired in 1.5-mm thick plates. The plates are 1500 mm long and 120 mm wide. The American Society for Testing and Materials (2022a) states that specimens with a thickness lower than 7 mm can be tested using the dogbone Type I, shown in Figure 3.1. The material was cut using a waterjet to prevent the Fe-SMA from activating due to an increase of temperature during machining.

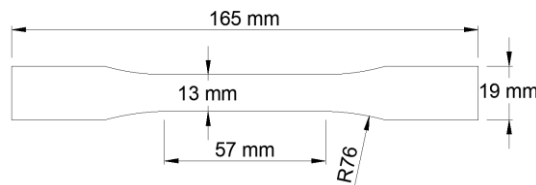


Figure 3.1. Fe-SMA Dogbone for Tensile Test

Three tensile tests were done in an MTS C45 testing frame using a crosshead speed of 1mm/min. An extensometer (50 mm) was placed in the gauge length of the dogbone to record the strains. Figure 3.2 shows the tensile test setup. The testing frame provided the stress-strain response of the Fe-SMA.

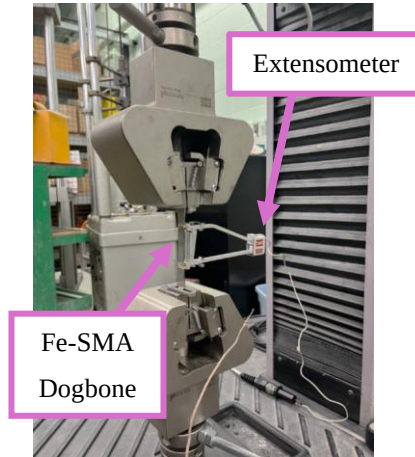


**Figure 3.2. Tensile Test in MTS C45 Testing Frame**

### **3.2.2 Recovery Stress Test**

The recovery stress tests were performed in an MTS C45 testing frame. The Fe-SMA specimens for this test were selected to be 10 mm wide and 20 mm long. The strips were prestrained at 2.5% (as recommended by the manufacturer) using the crosshead displacement at 1mm/min. Shahverdi et al. (2018) and Gu et al. (2021) suggest that a prestrain of 2% is optimal to obtain maximum recovery stresses below 200°C. The crosshead moved 3.5 mm to prestrain an unrestrained length of 140 mm. The prestraining test is presented in Figure 3.3.

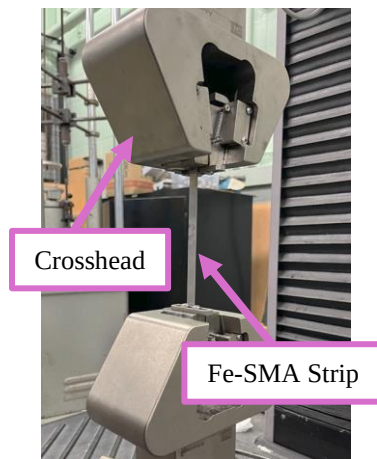


**Figure 3.3. Fe-SMA Prestrain**

To assess the recovery stress, the prestrained strips were placed in the frame and restrained by the grips. A thermocouple was placed at mid-height while the specimen was thermally heated to activation and cooled. The temperature and load change were recorded at 5 Hz to create the recovery stress curve. Multiple tests were conducted around three temperatures: 140°C, 180°C, and 220°C to assess the resultant recovery stress at different activation temperatures. The MTS C45 testing frame is not equipped to withstand electricity, so the recovery stress tests were performed with insulating heating tapes and chords.

#### 3.2.2.1 Insulating Heating Tapes

Silicon heating tapes (Figure 3.4) were used to heat the restrained Fe-SMA strip through conduction. The tapes have a maximum exposure temperature of 232°C and are mostly used indoor lab applications. The tape working conditions are 120V, 0.43 A, and 52W. The silicon tape may overheat and not function correctly if it is not regulated by a temperature-controlling device, overlaps itself, or is not in contact with a material to release the heat.



**Figure 3.4. Silicon Heating Tape**

The  $\approx 650\text{mm}$  silicon tape was wrapped around the Fe-SMA strip and secured with temperature resistant adhesive tapes ( $\leq 260^\circ\text{C}$ ). The tape was arranged so that it was not in contact with the thermocouple in the Fe-SMA strip. A voltage controller of up to 140V was used to control the heating rate of the tapes. Fe-SMA strips were attached to the excess of the tape to allow heat release and prevent overheating. Figure 3.5 shows one of the multiple wrapping configurations that were implemented.

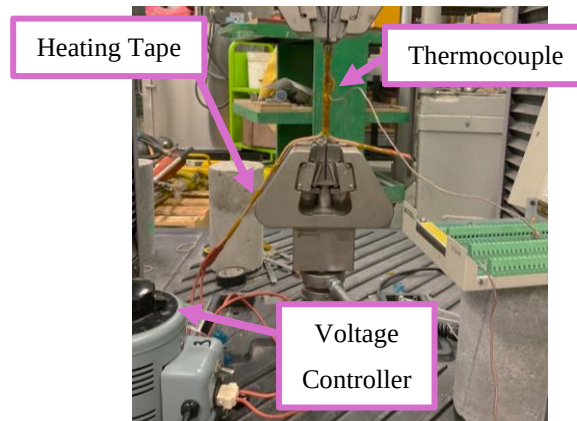
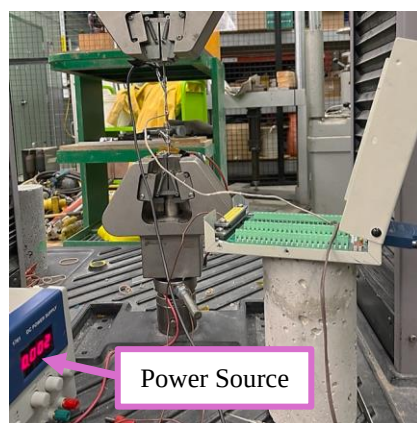


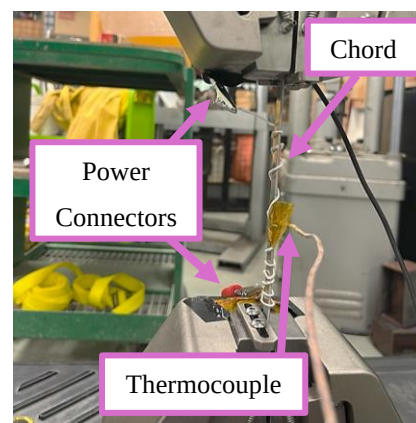
**Figure 3.5. Heating Tape Recovery Stress Test Setup**

The speed and temperature reached depended on the voltage controller. 120V was determined to not be enough to reach temperatures higher than 200°C. After trial and error, it was found that a voltage of 140V can provide faster heating rates and temperatures higher than 200°C, but reaching this temperature would take about 25 minutes. Due to the slow heating rate and melting of the silicon, other heating mechanisms were explored.

It was found that inside the silicon tape, there were small, insulated chords that could be attached to a power source and heated. The chords were wrapped around the Fe-SMA specimen avoiding direct contact with the thermocouple as presented in Figure 3.6. The power source was connected at both ends and the current was adjusted. At 3A, it was possible to heat the specimens to temperatures higher than 220°C in just a few minutes. It is important to note that the chords were delicate, breaking easily during the tests.



(a) Test Setup



(b) Powered Heating Chords

**Figure 3.6. Insulating Chord Recovery Stress Test Setup**

### **3.3 Fe-SMA Epoxy Composite**

The Fe-SMA epoxy composite is comprised of prestrained Fe-SMA strips embedded into high-strength temperature-resistant epoxy resin. The recovery stress of three Fe-SMA composites was assessed to determine the capacity of the epoxy to withstand the SME recovery stress and high temperatures.

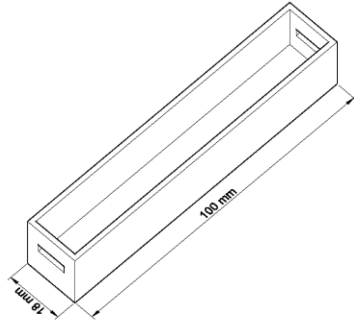
#### **3.3.1 Epoxy Resin**

A two-part high-strength temperature-resistant infusion resin system was selected to endure high temperatures ( $\approx 200^{\circ}\text{C}$ ) and recovery stresses resulting from the SME activation of the embedded prestrained Fe-SMA strips. The manufacturer specifications state that the resin has a tensile strength of 53.5 MPa, a flexural strength of 3276 MPa, and a compressive strength of 201.5 MPa. The specified glass transition temperature is  $202^{\circ}\text{C}$ . However, it was assured by the manufacturer that the material is capable of withstanding higher temperatures for short periods of time which was sufficient for activating the Fe-SMA.

The resin consists of a liquid resin (1660 cps,  $G_s = 1.17$ ) and a hardening modified amino mixture (80 cps,  $G_s = 0.96$ ) with a mixed ratio of 45 by weight. The resin has a pot life of up to 7 hours, acquiring a viscosity of 1050 cps. Curing time is estimated to be 24 hours at room temperature. Through trials, it was determined that this viscosity was not adequate for placement on a vertical surface. The manufacturer advised including silica fume with the mixed resin to increase the viscosity. It was found that a ratio of at least 2:1 by volume to the mixed resin thickened the epoxy sufficiently to be applied on vertical surfaces.

#### **3.3.2 Fe-SMA Epoxy Composite Recovery Stress**

Three Fe-SMA epoxy composite coupons were fabricated. The composites present 2.5% prestrained Fe-SMA strips (200 mm x 10 mm x 1.5 mm) embedded into a high-strength temperature-resistant epoxy resin. The epoxy resin was cast along the length of the strip using a TPU/ PLA 3D-printed cradle as seen in Figure 3.7 (a). The cradle is 1.5 mm thick. Two cradle versions were tried: 5-mm and 8-mm epoxy depth. The cradle was sanded off the specimen to prevent interference with the recovery stress test. The composite coupon is presented in Figure 3.7 (b). The manufacturer provides a variety of curing methods. For this project, the epoxy was left to cure for at least 24 hours at room temperature.



(a) Cradle



(b) Epoxy Composite Preparation

**Figure 3.7. Fe-SMA Epoxy Coupon**

Three recovery stress tests were done at three different temperatures. A thermocouple was placed inside the coupon to measure the temperature of the Fe-SMA strip, and a second thermocouple was put on the surface of the epoxy to assess the difference in temperature between the epoxy surface and interface epoxy-SMA. The recovery stress tests of the composite coupons were performed in an MTS C45 testing frame. For a rapid heating rate, the heating chords were used instead of the silicon tape. The heating chords were placed between the grips and the epoxy as presented in Figure 3.8.

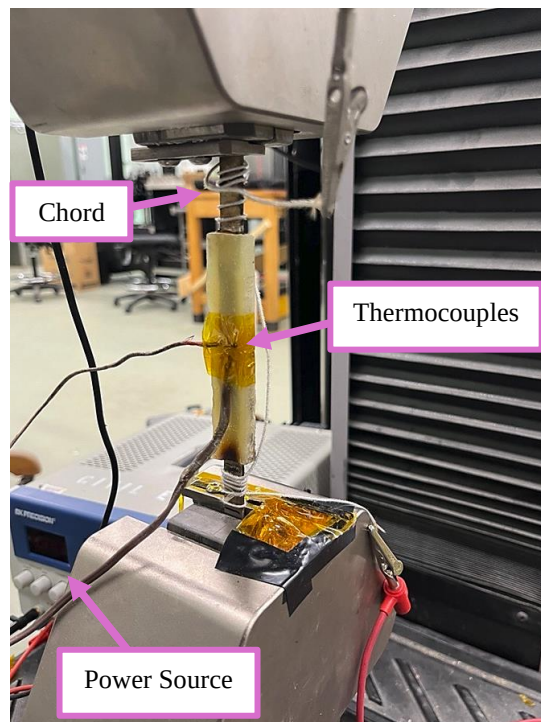


**Figure 3.8. Fe-SMA Epoxy Coupon Recovery Stress**

There were difficulties reaching temperatures higher than 100°C on the thermocouple placed on the SMA-epoxy interface. It appears that the thermocouple was influenced by the different thermal conductivity of the Fe-SMA and the epoxy. Visual inspection shows that the Fe-SMA may have reached temperatures around 200°C since the edges of the epoxy were burned (Figure 3.9). This is supported by the high recovery stresses that were recorded. It is important to note that there are no visible integrity issues in the composite.



**Figure 3.9. Fe-SMA Epoxy Coupon Recovery Stress**

### 3.4 RC Beam Specimens

Three identical 4.4-m RC beams were cast for this project. The beams are 300 mm wide and 400 mm deep. The beams were cast in two batches. The reinforcement cages were built according to the beam design presented in Section 3.4.1 and are presented in Figure 3.10(a). The RC beams will be used to compare the effect of activated Fe-SMA and spacing, as well as the adequacy of the Fe-SMA epoxy composite for shear strengthening. The capacity of the RC beams was determined based on code criteria and Vector2 FEA Models. Figure 3.10(b) shows the casting of three beams.



(a) Reinforcement cage



(b) RC beam cast

**Figure 3.10. Preparation of RC beams**

### 3.4.1 Beam Design: CSA A23.3

The materials and geometry of the beams were determined to create moderate-scale field-representative specimens. A double-curvature shear-critical loading framework was selected for the beams. The beams are under constant shear along its span and linear-changing moment. The RC beam was iteratively designed based on CSA S23.3 (2019).

#### 3.4.1.1 Loading Framework

The loading framework is based on the double-curvature beam specimens from Shekarchi (2016) that reflect pile cap girders from bridges in Texas built in the 1950s. These beam specimens are symmetrical, presenting equal but opposite linear moments and constant shear along their length. The beam specimens will have three span regions: two transfer spans (TRS) and a test span (TS). Figure 3.11 shows the loading framework, shear diagram, and moment diagram of the specimens, where  $V$  is in kN and  $L$  is in m.

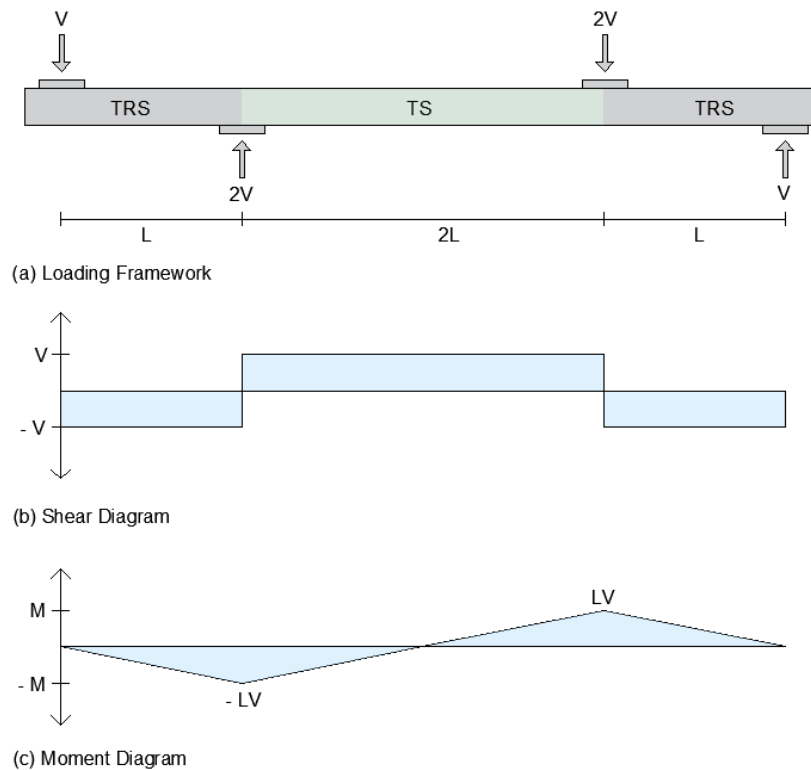


**Figure 3.11. Loading Framework**

The outside loads of the beams are expected to be half the inside loads. This project will use the test span to represent a shear-critical RC element. The transfer spans will not be part of the beam

assessment, so they will be more heavily reinforced to avoid shear failure. To ensure symmetry, the test span is twice the length of each transfer span. The adequacy of the test spans was checked with an equivalent Vector2 model.

#### 3.4.1.2 Specimen Geometry

The dimensions of the designed beam were chosen according to the available space and rebar sizes. This selection was made to ensure that crack propagation and widths could be easily observed and measured. The beam has a total length ( $\ell$ ) of 4400 mm. It presents a width ( $b$ ,  $b_w$ ) of 300 mm and a depth ( $h$ ) of 400 mm. The transfer spans are 1000 mm long ( $\ell_1$ ) and the test span ( $\ell_2$ ) is 2000 mm long. The supports and load-bearing plates are 38.1 mm thick, 254 mm long, and 300 mm wide. The beam geometry is shown in Figure 3.12.

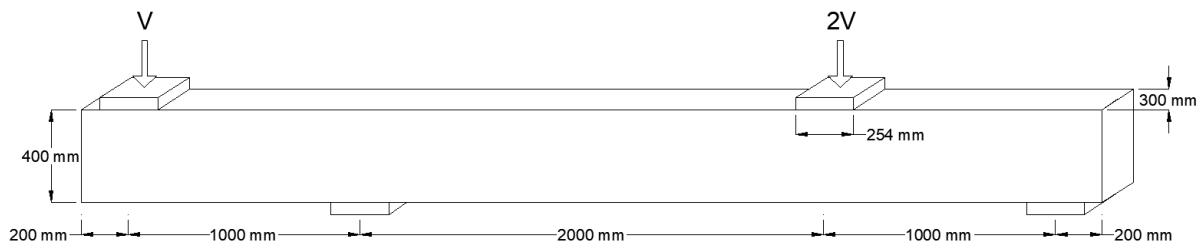


**Figure 3.12. Geometry of Specimens**

The test span length was selected to ensure that its clear span-to-depth ratio ( $l_n/h$ ) ratio complies with the classification of a flexural region (B-Region) based on CSA S23.3 (2019). The clear span-to-depth ratio of the test span is larger than 2, so it is considered a B-region and the design for shear follows Section 11.3 of CSA A23.3 (2019). On the other hand, the clear span to depth ratio of the transfer spans is less than 2, so they are classified as deep regions (D-Region). A strut-and-tie model would be appropriate for the shear design of these spans. This project does not consider the behavior of the transfer spans, so a strut-and-tie model has not been developed.

#### 3.4.1.3 Reinforcement Design

The design of the test span is done using chapters 10 and 11.3 of the CSA A23.3 (2019) – Design of Concrete Structures Standard. The test span is designed to be shear-critical based on maximum stirrup spacing and minimum reinforcement requirements. The beam is designed to be under-reinforced in flexure. Figure 3.13 present the complete reinforcement layout. The design process is presented afterwards.

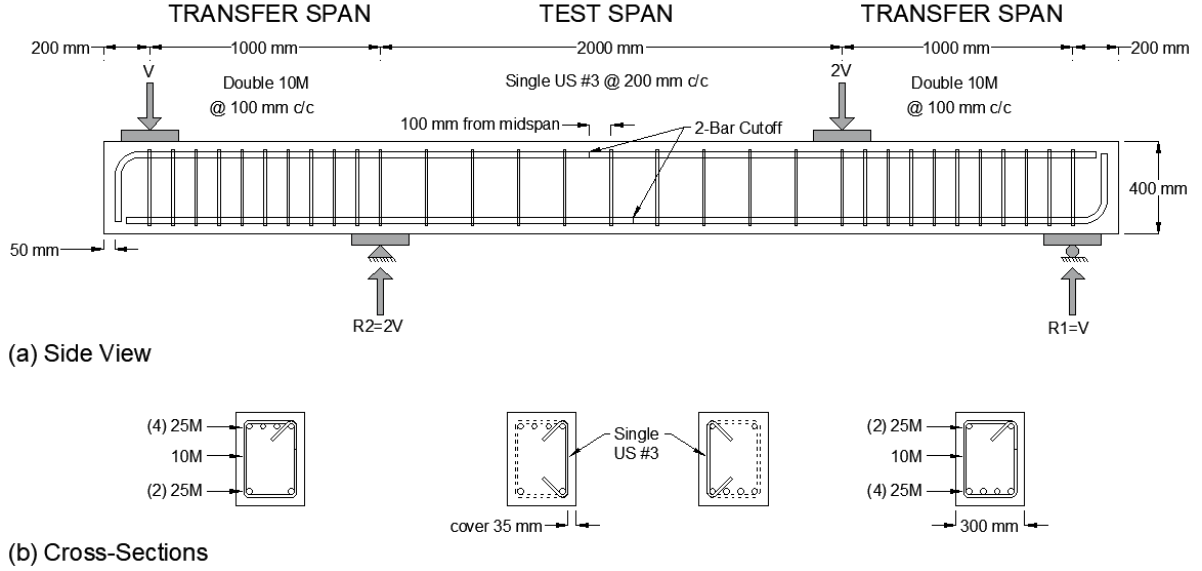


**Figure 3.13. Reinforcement Layout**

The beam specimens have four 25M bars as tension reinforcement ( $\rho = 1.94\%$ ) and two 25M bars as compression reinforcement ( $\rho' = 5.84\%$ ). Two longitudinal bars are cut 100 mm from the midspan within the tension zones. The transfer spans use double-leg 10M stirrups spaced at 100 mm c/c ( $\rho_{st} = 0.67\%$ ) while the test span uses single-leg US #3 ( $d_{st} = 9.525$  mm) stirrups spaced at 200 mm c/c ( $\rho_{st} = 0.12\%$ ). The cross-section cover is 35 mm, and the end cover is 50 mm.

#### 3.4.1.3.1 Materials

Common materials on RC beams were selected for this study. Material resistance factors for concrete ( $\phi_c$ ) and steel ( $\phi_s$ ) are equal to one due to the specimens being tested in a laboratory. Except for the US#3 single leg stirrups which are structural 60W steel, the reinforcement is 400W steel. It is assumed that both steels have a yield strength ( $f_y$ ) of 400 MPa and an elastic modulus ( $E$ ) of 200000 MPa. The maximum reinforcement strain ( $\epsilon_y$ ) is assumed to be equal to  $2 \times 10^{-3}$ , and it is calculated using Equation 3.1.

$$\epsilon_y = \frac{F_y}{E} \quad \text{Equation 3.1}$$

The beam specimens use normal-density concrete ( $\lambda = 0$ ). The concrete is considered to have a compressive strength ( $f'_c$ ) of 30 MPa with a maximum nominal aggregate size ( $a_{max}$ ) of 20 mm. The maximum concrete strain at the extreme compression fiber ( $\epsilon_{cu}$ ) is assumed to be  $3.5 \times 10^{-3}$ . The modulus of elasticity of the concrete ( $E_c$ ) can be found using Equation 3.2.

$$E_c = 4500\sqrt{f'_c} \quad \text{Equation 3.2}$$

#### 3.4.1.4 Shear Critical Test Span

The shear critical design of the test span is done based on the spacing of the minimum transverse reinforcement area ( $s_{av,min}$ ) and the maximum spacing of transverse reinforcement ( $s_{max}$ ). These spacings are calculated using Equation 3.3 and Equation 3.4, respectively.  $A_{v,min}$  is the minimum transverse reinforcement area, and  $d_v$  is the shear calculation depth.  $d_v$  can be determined with Equation 3.5, where  $d$  is the distance between the centroid of the tension reinforcement and the furthest compression fiber.

$$s_{Av,min} = \frac{A_{v,min} f_y}{0.06 \sqrt{f'_c} b_w} \quad \text{Equation 3.3}$$

$$s_{max} = \max\{0.7d_v, 600 \text{ mm}\} \quad \text{Equation 3.4}$$

$$d_v = \max\{0.9d, 0.72h\} \quad \text{Equation 3.5}$$

The bar diameter and configuration of the transverse reinforcement along the test span were selected according to the closest  $s_{av,min}$  to  $s_{max}$ . It was found that using single-leg US #3 stirrups ( $d_{st} = 9.525 \text{ mm}$ ;  $A_{st} = 71.26 \text{ mm}^2$ ),  $s_{av,min}$  would be equal to 289.10 mm, and  $s_{max}$  would be equal to 216.01 mm. Hence, the stirrup spacing ( $s$ ) along the test span was chosen to be 200 mm c/c (center-to-center), resulting in a reinforcement ratio of 0.12%.

#### 3.4.1.5 Flexural Resistance

The flexural strength of the beam was determined using Chapter 10 of the CSA A23.3 (2019). The reinforcement was designed using the highest moment demand ( $L[m] \times V[kN]$ ) as established in Figure 3.11. The beam is under-reinforced in bending to ensure ductile behavior if failing under flexure. Flexural failure is not expected as the beam has been designed to be shear-critical. The beams present a concrete cover of 35 mm along the test span.

The flexural reinforcement in the compression and tension zone is comprised of 25M bars ( $d_b = 25.2 \text{ mm}$ ;  $A_b = 500 \text{ mm}^2$ ). Four bars are used as tension reinforcement, resulting in a steel area ( $A_s$ ) of  $2000 \text{ mm}^2$ , effective depth ( $d$ ) of 342.88 mm, and a reinforcement ratio ( $\rho$ ) equal to 1.94%. On the other hand, two bars are used for compression reinforcement, leading to a compression steel area ( $A'_s$ ) of  $1000 \text{ mm}^2$ , compression effective depth ( $d'$ ) of 57.13 mm, and a compression reinforcement

ratio ( $\rho'$ ) equal to 5.84%. Figure 3.14 shows the forces acting on the cross-section of a beam under flexure.

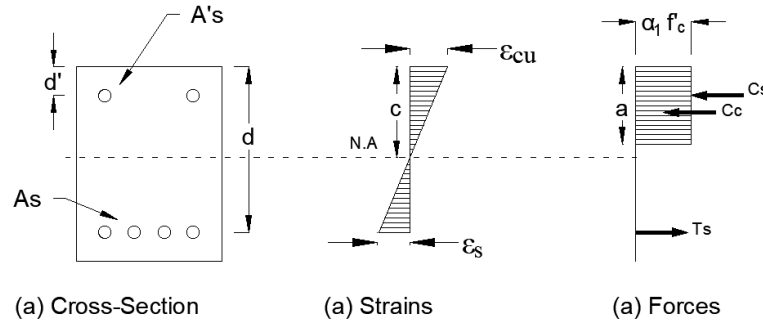


**Figure 3.14. Control Beam Cross-Section Forces**

The beam is under-reinforced, so tension reinforcement yields ( $f_s = f_y$ ) before concrete crushes. It was determined that the compression reinforcement on the beam does not yield ( $f'_s$ ). The forces developed by the tension ( $T_s$ ) and compression reinforcement ( $C_s$ ) can be calculated with Equation 3.6 and Equation 3.7, respectively, where  $c$  is the depth of the neutral axis from the extreme compression fiber.

$$T_s = \phi_s A_s f_s = \phi_s A_s f_y \quad \text{Equation 3.6}$$

$$C_s = \phi_s A'_s f'_s = \phi_s A'_s \frac{\epsilon_{cu}(c-d')}{c} E \quad \text{Equation 3.7}$$

It is assumed that concrete does not support tension, providing only a compression force ( $C_c$ ) calculated with Equation 3.8.  $a$  is the depth of the equivalent rectangular compressive stress block, and  $\alpha_1$  and  $\beta_1$  are stress distribution block factors found with Equations 3.9 and 3.10, respectively.

$$C_c = \phi_c f'_c \alpha_1 a b = \phi_c f'_c \alpha_1 \beta_1 c b \quad \text{Equation 3.8}$$

$$\alpha_1 = 0.85 - 0.0015 f'_c \geq 0.67 \quad \text{Equation 3.9}$$

$$\beta_1 = 0.97 - 0.0025 f'_c \geq 0.67 \quad \text{Equation 3.10}$$

By summing the forces,  $c$  can be calculated using Equation 3.11. The quadratic equation results in  $c$  being equal to 86.62 mm. The moment capacity was then found with Equation 3.12. The moment capacity of the beam is 238.91 kNm.

$$\phi_c f'_c \alpha_1 \beta_1 b c^2 - (\phi_s A_s f_y + \phi_s A'_s \epsilon_{cu} E) c + \phi_s A'_s \epsilon_{cu} d' E = 0 \quad \text{Equation 3.11}$$

$$M_r = C_c \left( \frac{h}{2} - \frac{a}{2} \right) + C_s \left( \frac{h}{2} - d' \right) - T_s \left( \frac{h}{2} - d \right) \quad \text{Equation 3.12}$$

#### 3.4.1.5.1 Reinforcement Checks

CSA A23.3 (2019) states that the clear distance between longitudinal bars should be the least of 1.4 times the diameter of the longitudinal bar, 1.4 the maximum aggregate size, and 30 mm. The clear distance was found to comply with CSA A23.3 (2019) at 36.72 mm. Additionally, reinforcement strains should be checked to confirm that the compression reinforcement does not yield and the tension reinforcement yields. The strain in the compression steel ( $\epsilon'_s$ ) was calculated to be equal to  $1.19 \times 10^{-3}$  using Equation 3.13. The yield of the tension steel can be confirmed with Equation 3.14.

$$\epsilon'_s = \frac{\epsilon_{cu}(c-d')}{c} \geq \epsilon_y \quad \text{Equation 3.13}$$

$$\frac{c}{d} \leq 0.8 \frac{700}{700 + f_y} \quad \text{Equation 3.14}$$

#### 3.4.1.5.2 Development Length

With the clear cover being 35 mm ( $\geq d_b$ ) and the clear spacing being larger than  $1.4d_b$ , it is possible to use the simplified equation for tension development length from CSA A23.3 (2019) Table 12.1. The tension development length ( $l_d$ ) is determined with Equation 3.15 since the beam has at least minimum transverse reinforcement.  $k_1$ ,  $k_2$ ,  $k_3$ , and  $k_4$  are factors that consider bar location, coating, concrete density, and bar size, respectively. In this case, all factors are equal to one, and the tension reinforcement development length is 828.16 mm. On the other hand, the compression reinforcement basic development length ( $l_{db}$ ) was calculated with Equation 3.16, being 443.52 mm.

$$l_d = 0.45 k_1 k_2 k_3 k_4 \frac{f_y}{\sqrt{f'_c}} d_b \geq 300 \text{ mm, where } k_1 k_2 \leq 1.7 \quad \text{Equation 3.15}$$

$$l_{db} = \max \left\{ \frac{0.24 d_b f_y}{\sqrt{f'_c}} ; 0.044 d_b f_y \right\} \geq 200 \text{ mm} \quad \text{Equation 3.16}$$

#### 3.4.1.6 Shear Resistance

The shear resistance of the test span was determined considering minimum reinforcement using single-leg US #3 stirrups. The factored shear resistance ( $V_r$ ) of the section can be estimated as the shear contribution from concrete ( $V_c$ ) and transverse reinforcement ( $V_s$ ) as presented in Equation 3.17.  $V_r$  should be less than  $V_{r,max}$  which represents the concrete crushing capacity under diagonal

compression (Bentz & Collins, 2006). Note that the control beam is not prestressed, and it does not experience axial loads, so these parameters will not be included in the equations.

$$V_r = V_c + V_s \leq V_{r,max} = 0.25 \varphi_c f'_c b_w d_v \quad \text{Equation 3.17}$$

The concrete contribution can be determined with Equation 3.18, where the cracked shear resistance is considered with the factor  $\beta$ . On the other hand, the stirrup contribution is calculated with Equation 3.19, where  $\theta$  is the angle between the diagonal compressive stress and the horizontal axis, and  $A_v$  is the total area of transverse reinforcement. An accurate determination of  $\beta$  and  $\theta$  can be made by implementing the general method.

$$V_c = \varphi_c \lambda \beta \sqrt{f'_c} b_w d_v \quad \text{Equation 3.18}$$

$$V_s = \frac{\varphi_s A_v f_y d_v \cot \theta}{s} \quad \text{Equation 3.19}$$

The general method states that Equation 3.20 can be used to find  $\beta$  and Equation 3.21 to calculate  $\theta$ . The crack spacing parameter  $S_{ze}$  is taken as 300 mm since the beams use at least minimum transverse reinforcement. The mid-depth strain ( $\epsilon_x$ ) is found with Equation 3.22, where  $M_f$  is the moment demand along the span, and  $x$  is the location.  $M_f$  is calculated using Equation 3.23, and it depends on shear demand ( $V_f$ ).

$$\beta = \frac{0.40}{1 + 1500 \epsilon_x} \times \frac{1300}{1000 + S_{ze}} \quad \text{Equation 3.20}$$

$$\theta = 29 + 7000 \epsilon_x \quad \text{Equation 3.21}$$

$$\epsilon_x = \frac{\frac{M_f}{d_v} + V_f}{2 E_s A_s} \quad \text{Equation 3.22}$$

$$M_f = \frac{L V_f x}{L} \leq M_r \quad \text{Equation 3.23}$$

It is assumed that shear failure is a concern  $d_v$  away from the face of the supports and the concentrated load application plates. The shear capacity of the along the test span of the RC beam was iteratively found to be 172.82 kN. Failure is expected to happen on the test span, so the transfer spans were considerably reinforced for shear.

The transverse spans are transversely reinforced with double-leg 10M bars ( $d_{st} = 11.3$  mm;  $A_{st} = 100$  mm<sup>2</sup>) spaced at 100 mm center-to-center (c/c). The stirrup reinforcement ratio being 0.67% which is more than 5.5 times the transverse reinforcement ratio at the test span. This reinforcement ratio is expected to ensure that the beam is shear-critical at the test span.

#### 3.4.1.6.1 Development Length

CSA A23.3 (2019) states that shear results in additional tension forces on the longitudinal reinforcement. The longitudinal reinforcement should be proportioned such that their tensile resistance is greater than the tension force on the bars in the tension ( $F_{lt}$ ) and compression ( $F_{lc}$ ) flexural regions.  $F_{lc}$  is checked at sections where  $M_f/d_v$  is less than the addition of the axial load and shear terms.  $F_{lt}$  is calculated with Equation 3.24, and  $F_{lc}$  with Equation 3.25.  $F_{lt}$  and  $F_{lc}$  were checked along the shear critical test span to confirm that the tension on the bars is lower than their resistance, ensuring that the stirrups yield before the longitudinal bars.

$$F_{lt} = \frac{M_f}{d_v} + (V_f - 0.5V_s) \cot \theta, \text{ where } V_s \leq V_f \quad \text{Equation 3.24}$$

$$F_{lc} = (V_f - 0.5V_s) \cot \theta - \frac{M_f}{d_v} \quad \text{Equation 3.25}$$

#### 3.4.1.7 Resistance

The maximum moment demand on the RC beams is estimated based on their shear resistance. The maximum shear and moment demand along the beam is 172.82 kN and 172.82 kNm, respectively. The moment demand and resistance are presented in Figure 3.15. The longitudinal bar resistance changes linearly along the development length until reaching zero at the point of cut. While the moment resistance is found to be larger than the demand, the tension bars terminate in a standard hook based on CSA A23.3 (2014) as a precaution. Hence, the development length of the tension and compression longitudinal bars are adequate under flexure.

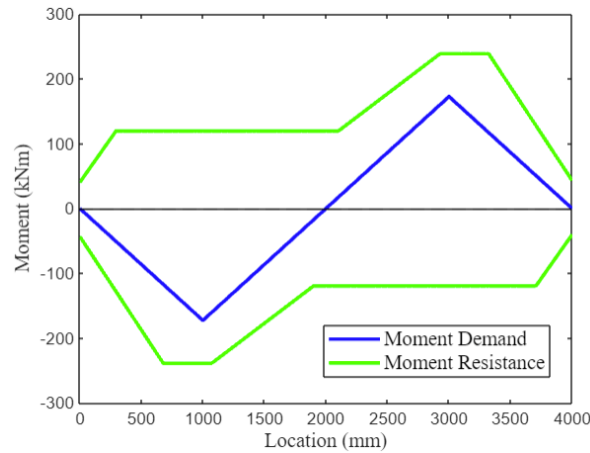


**Figure 3.15. Moment Resistant versus Moment Demand**

It is expected that the shear capacity of the transfer spans exceeds the one of the test span. The transfer spans are not part of the scope of the project and have been reinforced considerably to prevent shear failure. Therefore, shear is exclusively assessed in the test span as seen in Figure 3.16. The shear resistance is 172.82 kN  $d_v$  away from the face of the support/load plates, and it reaches 193.38 kN around the midspan.

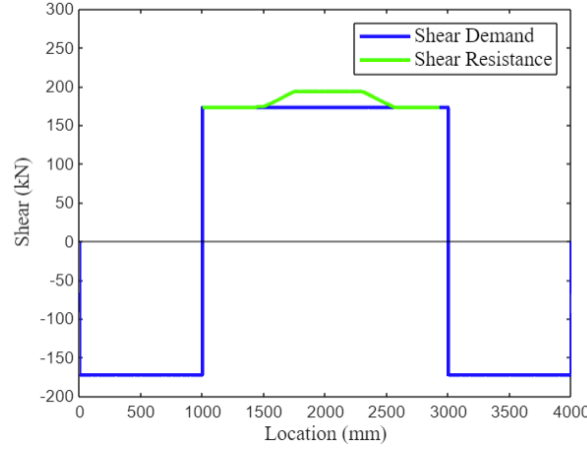


**Figure 3.16. Shear Demand versus Shear Resistance**

### 3.4.2 Test Matrix

Three RC concrete beam specimens were cast: one control beam and two active shear strengthened beams with NSB Fe-SMA strips. Two c/c spacing are considered and evaluated. Table 3.1 presents the experimental test matrix for this project. The specimens are identified with a letter representing activated (A) or non-activated (N) Fe-SMA. After which, the spacing in mm and the activation temperature in °C are added to the identification.

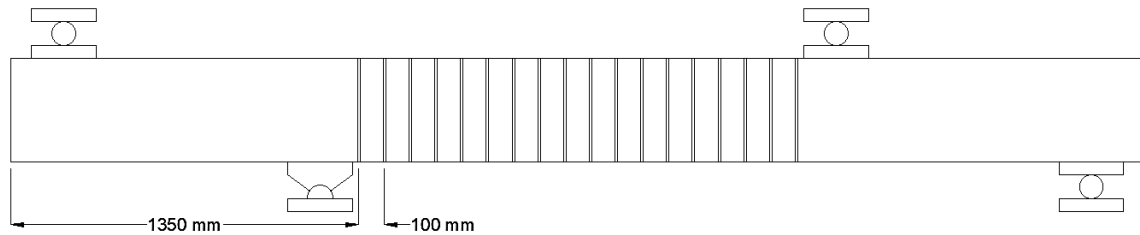
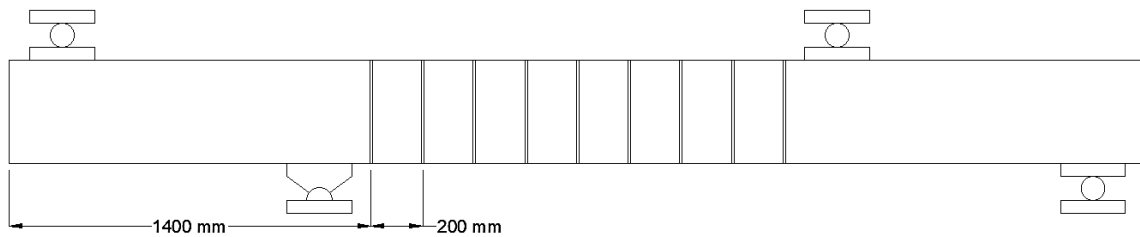
Hong et al. (2018) provide an equation to estimate the prestressing stress ( $\sigma_{\text{expected}}$ ) on the concrete if strengthened with activated NSM Fe-SMA. The prestressing force is suggested to be a function of the Fe-SMA recovery stress ( $\sigma_{R,S}$ ), the Fe-SMA strip area ( $A_{\text{Fe-SMA}}$ ), and the cement-based mortar filler area ( $A_{\text{eq}}$ ) as seen in Equation 3.26. For this project  $A_{\text{eq}}$  represents the area of the epoxy filler area. The study recommends an activation temperature of at least 160°C for optimal prestress. This equation is implemented to estimate the prestressing stress in Table 3.1.

$$\sigma_{\text{expected}} = \frac{\sigma_{R,S} \times A_{\text{Fe-SMA}}}{A_{\text{eq}}} \quad \text{Equation 3.26}$$

**Table 3.1. RC Beam Specimens Test Matrix**

| Specimen       | Fe-SMA Spacing (mm) | Fe-SMA Reinforcement Ratio (%) | SME Activation Temperature (°C) | Fe-SMA Recovery Stress (MPa) | Expected Prestress (MPa) |
|----------------|---------------------|--------------------------------|---------------------------------|------------------------------|--------------------------|
| (N-0S) Control | -                   | -                              | -                               | -                            | -                        |
| A-100S-180C    | 100                 | 0.1%                           | $\geq 180$                      | 250                          | $\approx 33$             |
| A-200S-180C    | 200                 | 0.05%                          | $\geq 180$                      | 250                          | $\approx 33$             |

The shear-strengthened beams present NSB U-wrapped Fe-SMA strips along their test span. The wrapped configuration was chosen due to the length of the Fe-SMA strips and the possibility of top surface inaccessibility. The strips are placed at an inclination of 90° due to ease of installation, and they are 1.5 mm thick and 10 mm wide. Their width and spacing were selected based on the Fe-SMA reinforcement ratio according to CSA A23.3 (2019) minimum shear reinforcement (Equation 3.3) and maximum spacing (Equation 3.4). The internal reinforcement ratio was first doubled with 0.1% Fe-SMA using a spacing of 100 mm c/c. A second spacing of 200 mm c/c was selected to represent 0.05% Fe-SMA to assess the feasibility of less strengthening material. The Fe-SMA spacings on the shear-strengthened beams are shown in Figure 3.17.

**(a) 100 mm c/c Fe-SMA****(b) 200 mm c/c Fe-SMA****Figure 3.17. Fe-SMA Spacing**

The RC beams were loaded to failure to assess strength, stirrups and tension bars stresses, displacement, and crack width and propagation. The placement and activation of the Fe-SMA on the beam specimens will provide insight into the shear-strengthening capabilities of NSB Fe-SMA epoxy composite for RC shear strengthening. The assessment will depend on Fe-SMA spacing and Fe-SMA SME activation.

### 3.4.3 Reinforcement

All reinforcement is 400W structural steel, except for the US #3 transverse reinforcement that are 60W. The longitudinal reinforcement is comprised of 25M bars. Double-leg 10M stirrups (100 mm c/c) are used as transverse reinforcement in the transfer spans, while single-leg US #3 stirrups (200 mm c/c) are used in the test span. The single-leg US #3 stirrups were alternatively placed between sides to prevent one side of the beam from being stronger than the other. The reinforcement cage is shown in Figure 3.18.

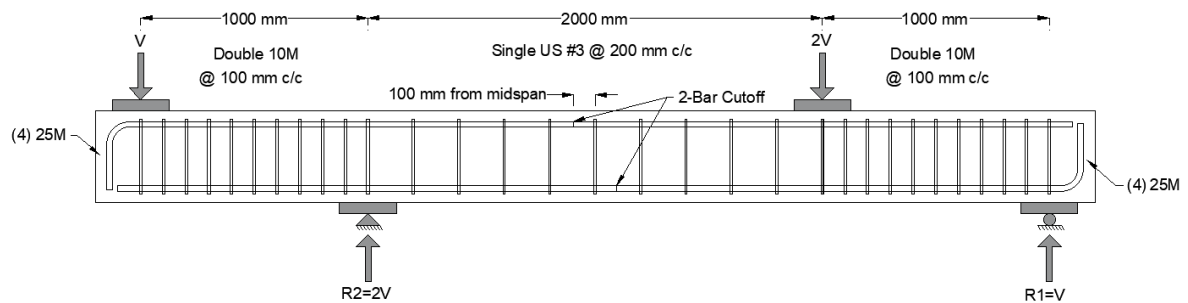


**Figure 3.18. Reinforcement Cage**

The rebar cages were built in the lab. Stirrups were manually bent or ordered pre-bent. Manually bent stirrups were considered adequate if their width and height fell within a tolerance of +10 mm, preventing a significant reduction to the concrete cover of the beams. The detailing of tensile bars and stirrups is presented in Figure 3.19. Hooks were determined based on CSA A23.3-14 Handbook Table 3.7.

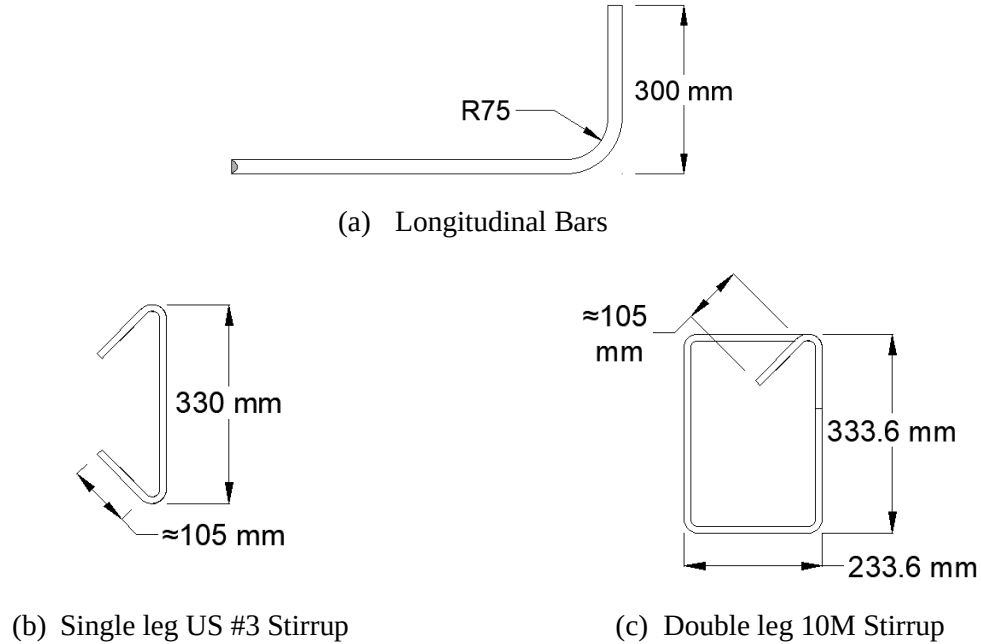


**Figure 3.19. Reinforcement Detailing**

Tensile tests were performed on the reinforcement bars based on ASTM E8 (American Society for Testing and Materials, 2022b). The tests were performed at a rate of 3 mm/min and provided results on stress-strain behavior of the rebar. An extensometer was placed midspan of the sample to ensure more accurate results. Table 3.2 summarized the material properties of the reinforcement.

**Table 3.2. Reinforcement Material Properties**

| Bar   | E<br>(GPa) | $f_y$<br>(MPa) | $\epsilon_{sh}$<br>(mm/mm) | $f_u$<br>(MPa) | $\epsilon_u$<br>(mm/mm) |
|-------|------------|----------------|----------------------------|----------------|-------------------------|
| US #3 | 200        | 500            | 0.013                      | 730            | 0.110                   |
| 10M   | 190        | 470            | 0.024                      | 580            | 0.138                   |
| 25M   | 200        | 495            | 0.012                      | 610            | 0.096                   |

#### 3.4.4 Concrete

The beams were cast in two batches using a concrete truck. For each batch, at least 25 (100 mm x 200 mm) cylinders were cast for compression testing. The cylinders were left covered for 24 hours. For each batch, three cylinders were placed in a humidity room for their 28-day compression test. The compressive strength ( $f'_c$ ) was tested using a speed between 1.62 kN/s and 2.43 kN/s (American

Society for Testing and Materials, 2023). The properties of each concrete batch are summarized in Table 3.3.

**Table 3.3. Concrete Properties**

| <b>Concrete Properties</b>                   | <b>1<sup>st</sup> Concrete Pour</b> | <b>2<sup>nd</sup> Concrete Pour</b> |
|----------------------------------------------|-------------------------------------|-------------------------------------|
| Ordered Slump (mm)                           | 127                                 | 127                                 |
| Actual Slump (mm)                            | Start: 165    End: 115              | Start: 110    End: 105              |
| Ordered $f'_c$ (MPa)                         | 30                                  | 30                                  |
| 1 <sup>st</sup> Cylinder 28-day $f'_c$ (MPa) | 37.1                                | 34.7                                |
| 2 <sup>nd</sup> Cylinder 28-day $f'_c$ (MPa) | 33.7                                | 39.4                                |
| 3 <sup>rd</sup> Cylinder 28-day $f'_c$ (MPa) | 37.3                                | 43.5                                |
| Average 28-day $f'_c$ (MPa)                  | 36.0                                | 39.2                                |

The rest of the cylinders were placed beside the beams to ensure similar curing conditions to represent each beam specimen. The beams and cylinders were placed under burlap and wetted for about 5 days. The compressive strength of three of these concrete cylinders was determined on the testing day of each respective beam. The average compressive strength of the three beam specimens is summarized in Table 3.4.

**Table 3.4. Concrete  $f'_c$  of Beam Specimens**

| <b>Specimens</b> | <b>1<sup>st</sup> Cylinder <math>f'_c</math><br/>(MPa)</b> | <b>2<sup>nd</sup> Cylinder <math>f'_c</math><br/>(MPa)</b> | <b>3<sup>rd</sup> Cylinder <math>f'_c</math><br/>(MPa)</b> | <b>Average <math>f'_c</math><br/>(MPa)</b> |
|------------------|------------------------------------------------------------|------------------------------------------------------------|------------------------------------------------------------|--------------------------------------------|
| (N-0S) Control   | 35.1                                                       | 38.4                                                       | 39.1                                                       | 37.5                                       |
| A-200S-180C      | 36.0                                                       | 36.2                                                       | 36.7                                                       | 36.3                                       |
| A-100S-180C      | 33.0                                                       | 36.0                                                       | 33.7                                                       | 34.2                                       |

### 3.4.5 NSB Fe-SMA strips

The strengthened beams present NSB Fe-SMA strips embedded into epoxy resin. NSM techniques are expected to provide better bonding, strength, and environmental protection compared to EB techniques (El-Hacha & Soudki, 2013; Effiong & Ede, 2022). Since the Fe-SMA strips are u-

wrapped, there was a concern about the material being activated and splitting from the cured epoxy resin. A rapid test was done using a small beam with a u-wrapped NSB Fe-SMA strip to confirm that the Fe-SMA would not break open from the resin (Figure 3.20).

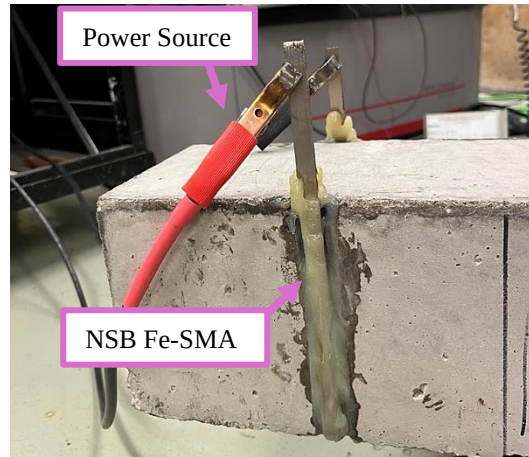


**Figure 3.20. Activated NSB Fe-SMA Strip Proof-of-Concept Test**

The embedded Fe-SMA was heated to temperatures around 200°C and left to cool for the SME. The resin did not show signs of cracking or breakage from the concrete. About 50 A were required to heat the specimen. The grooves were about 5mm deep and 20 mm wide. After this proof-of-concept test, the beam specimens were prepared.

#### 3.4.5.1 Strengthened Beam Specimens Preparation

On the strengthened beam specimens, the grooves were chosen to be 5-8 mm deep and 20 mm wide to increase the bonding area and strength. Due to heavy equipment, there were difficulties cutting exact depths so a tolerance of 3 mm is expected. The grooves were cut along the concrete surface with a diamond blade saw with a thickness of 6.35mm. The grooves were then cleaned with a compressed air gun and acetone to remove any laitance that may interfere with bonding. Due to access and tool limitations, the beams were grooved by flipping the beam.

To install the Fe-SMA, the grooves were first half-filled with the two-part high-strength temperature-resistant epoxy resin. The resin was mixed with silica fume at least two times its volume to thicken it and make it feasible for vertical surfaces. Prestrained Fe-SMA strips were bent using a bench vise to fit inside the grooves. The bent strips were gently pressed against the resin, after which

the rest of the groove was filled with more epoxy resin, embedding the Fe-SMA strips as seen in Figure 3.21. The specimens were left to cure for at least 48 hours before testing.



**Figure 3.21. NSB Fe-SMA Strips**

For the active strengthened specimens, a power source is connected to the free ends of the Fe-SMA strips to reach the SME activation temperature (Figure 3.22). The time of heating was selected to ensure that most of the Fe-SMA strip experienced at least the expected activation temperature. The power supply used for activation was set to 4V with variable amperage. The temperature is checked at the free ends and at the transverse midspan with a temperature indicator heat stick. The specimens were then left to cool for at least one hour before testing.



**Figure 3.22. Activation of NSB Fe-SMA Strip**

A 1.5 m long Fe-SMA strip was found to take seven to eight minutes to reach a temperature of at least 191°C at midspan using 40 to 50 A. If the amperage is increased to 65 A, the heating time reduces to around a minute. Heating an epoxy embedded strip was also found to take around one minute when using maximum capacity at 68.5 A. However, concerns were raised on whether the length of Fe-SMA strips would heat uniformly at rapid rates. Therefore, it was decided to gradually heat the F-SMA strips using 35 A for two minutes, 50 A for two minutes, and 65 A for about under a minute to ensure the material reaches an activation temperature of at least 191°C.

The ends of the Fe-SMA were cut to mitigate the risk of injury when failing. Note that the epoxy smokes at high temperatures and a well-ventilated area is necessary. Additionally, as the material cools, and the SME takes place, cracks appear at the surface of the epoxy on sections where the Fe-SMA strips are closest to the surface as presented in Figure 3.23.

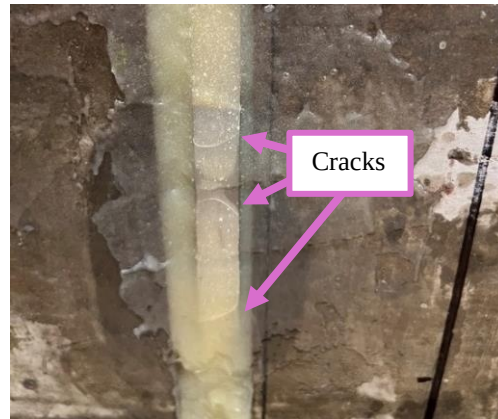


**Figure 3.23. Cracks during Cooling (SME)**

#### **3.4.6 Testing Setup**

A compression portal frame with a 2600-kN hydraulic actuator was used to test the beam specimens. Figure 3.24 shows the testing frame layout and supports. Two 4 m W250x115 beams are welded together on the flanges to be used as a spreader beam. Bending, shear, and bearing were checked to prevent failure of the spreader beam. Twelve stiffed pedestals were placed as supports, comprising of two box sections and eight I-sections. The tests setup presents two pins and five rollers. A 2D pin is placed in between the hydraulic actuator and the spreader beam while a 3D pin is placed as a support for the RC beam. The support/loading plates are 38.1 mm thick, 304.8 mm deep, and 254 mm wide.

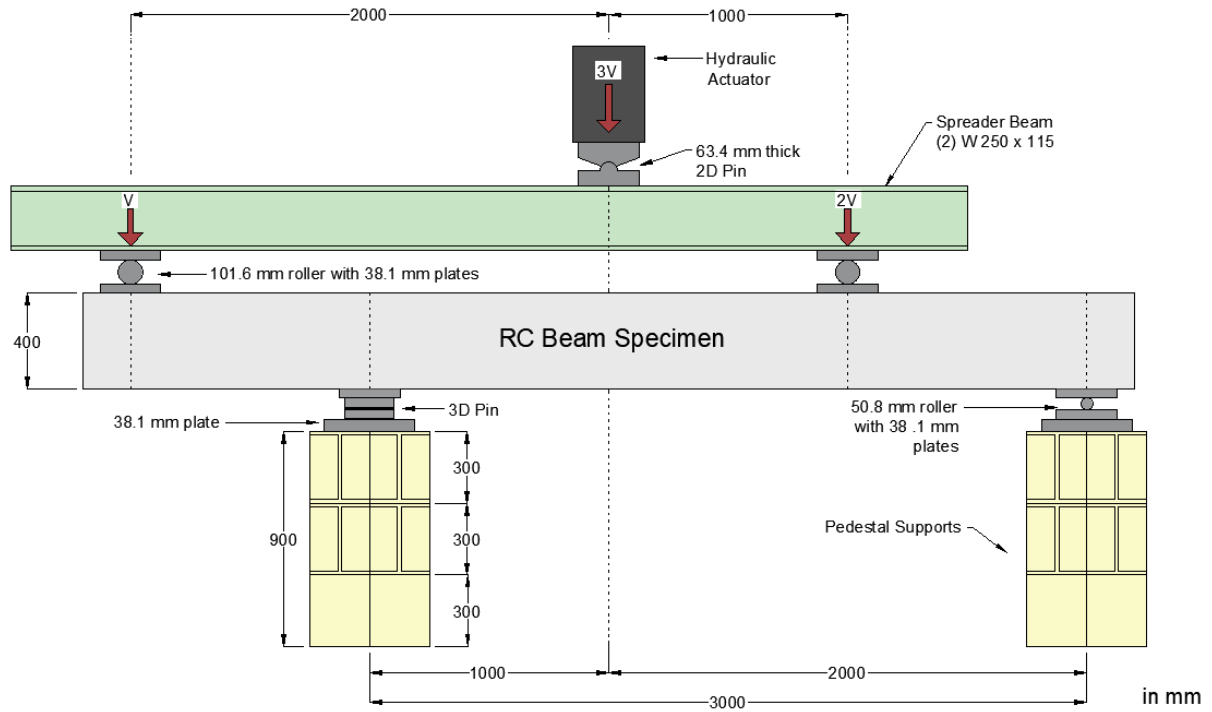


**Figure 3.24. Testing Setup**

### 3.4.7 Beam Specimen Instrumentation

Strain gauges and string potentiometers (string pots) were implemented on the beam specimens to evaluate if the strengthened beams experienced an improvement in reinforcement stresses and deformation, respectively. Five strain gauges were placed in the reinforcement at the most critical sections to check yielding of the steel, and they were selected based on the bar size. Two string pots were attached to the concrete to assess displacements at load application points. Figure 3.25 shows the location and application of the instrumentation.

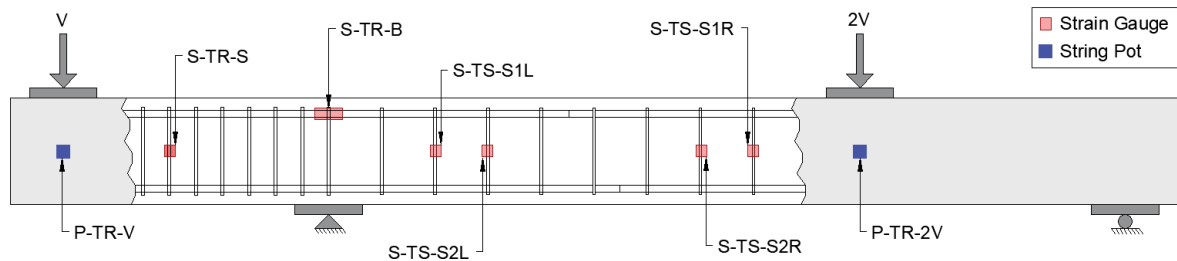


**Figure 3.25. Instrumentation Location**

The instrumentation is identified alphanumerically with the first letter being S (strain gauge) or P (string pot). TR refers to the transfer spans and TS refers to the test span. After which, the strain gauges present an S used for shear or a B for bending. Shear is assessed  $d_v$  away from the face of the support, so the identification may include a number (1 or 2) and a letter (R or L) based on the strain gauge location. On the other hand, string pots refer to their location using the load they are under (V or 2V).

#### 3.4.7.1 Strain Gauges

The strain gauges have a resistance of 120 ohms. FLKB-2 (2mm gauge length) strain gauges were used on bars with a diameter smaller than 11.3 mm, while FLAB-3 (3 mm gauge length) strain gauges were used on bars with larger diameters. Five strain gauges were placed in each beam. One strain gauge has been placed in an interior tension bar at the location of maximum moment. Another strain gauge has been placed in a stirrup on the transfer span to ensure it is not yielding. In the test span, four strain gauges were placed around  $d_v$  away from the face of the support/loading plates to check yielding. Figure 3.26 shows a strain gauge placed on a longitudinal tension bar.



**Figure 3.26. Strain Gauge**

#### 3.4.7.2 String Potentiometer (String Pots)

Two string potentiometers were used in each beam: one at the cantilever under load V (P-TR-V) and another under load 2V (P-TR-2V). The displacement at these locations can be measured using a rigid section attached/hanged at the supports to create a datum line. This method was first proposed by Boyan (2009) as mentioned in Butkovic (2022) with Perkins (2011) and Butkovic (2022) implementing it experimentally. For this project, the potentiometers are attached to an aluminium

angle that is hanged at the support locations as seen in Figure 3.27. Preliminary Vector2 models were used to estimate the displacement at the cantilever and under the inside load.

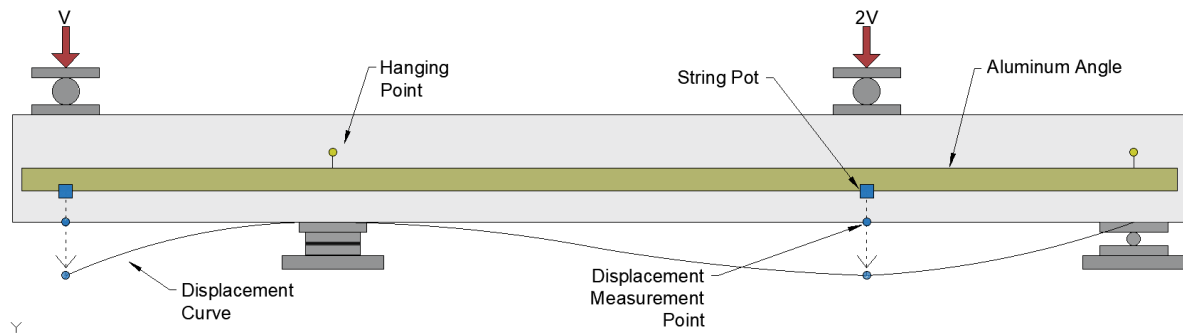


**Figure 3.27. Placement of String Potentiometers**

The aluminium angle is chained on one face of the RC beam above the supports. The measurement points are implemented with an aluminium/wood angle at the bottom of the beam. An eye bolt is used to hook the string pot. Figure 3.28 shows the chained angle with the string pots implemented on the control beam.



**Figure 3.28. Displacement Measurement Mechanism**

### 3.4.8 Loading Rate

A preliminary Vector2 model showed that the control RC beam will experience an estimated displacement of about 5 mm. Assuming that the displacement may reach about 1.5 times what is expected, and an optimal testing time is around 30 – 40 minutes, a loading rate of 0.18 mm/min was selected for the control beam. Due to multiple crack checks during the tests, the loading rate of the strengthened beams was doubled to 0.36 mm/min.

## **Chapter 4**

### **Experimental Results and Discussion**

#### **4.1 General**

This chapter presents the experimental results and discussion from the Fe-SMA material characterization and RC beam specimen testing. Tensile and recovery stress tests were done in an MTS C45 testing frame. Thermocouples were used to record respective temperature changes in the material. All beam specimens were loaded till failure. Strength, reinforcement yielding, and displacement were assessed to identify the feasibility of using NSB Fe-SMA strips embedded into epoxy for shear strengthening. This chapter is divided into three phases:

1. Fe-SMA material characterization: stress-strain behavior and recovery stress,
2. Fe-SMA epoxy composite recovery stress,
3. Fe-SMA shear strengthening effect on strength and performance of beam specimens.

#### **4.2 Fe-SMA Material Characterization**

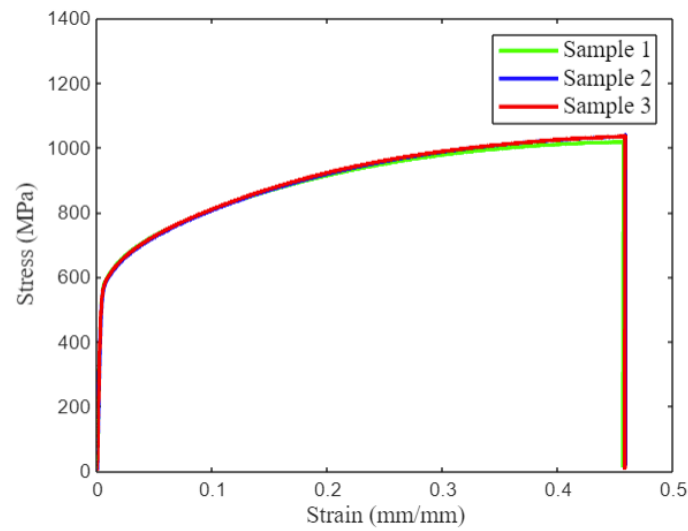
Fe-SMA of the form Fe-17Mn-5Si-10Cr-4Ni-1(V, C) was characterized by obtaining its stress-strain behavior and recovery stresses at different prestrains and temperatures. The stress-strain response was used to determine yielding, and ultimate tensile strength and strain. Two different prestrains were evaluated at three different temperatures to determine optimal conditions for good recovery stress.

##### **4.2.1 Stress-Strain Behavior**

Three ASTM Type I dogbones were tested to obtain the stress-strain behavior of the material. Figure 4.1 shows the respective stress-strain curves. The material exhibits high strength and ductility. Fe-SMA stress-strain curves do not show a clear yielding point, so the 0.2% offset method was utilized as used by Jeong et al. (2022) and Shahverdi et al. (2018). It was determined that the Fe-SMA material has an average yield strength ( $F_y$ ) of 562.96 MPa, ultimate strength ( $F_u$ ) of 1030.75 MPa, and ultimate strain ( $\epsilon_u$ ) of 0.458. The Fe-SMA characteristics are summarized in Table 4.1.

**Table 4.1. Fe-SMA Characteristics**

| Sample   | $F_y$ (MPa) | $F_u$ (MPa) | $\epsilon_u$ (mm/mm) |
|----------|-------------|-------------|----------------------|
| Sample 1 | 569         | 1017.76     | 0.457                |
| Sample 2 | 559.13      | 1038.51     | 0.458                |
| Sample 3 | 560.76      | 1035.98     | 0.458                |
| Average  | 562.96      | 1030.75     | 0.458                |

**Figure 4.1. Stress-Strain Fe-SMA**

#### 4.2.2 Recovery Stress

The Fe-SMA material is prestrained before SME activation. Four Fe-SMA specimens were prestrained at different levels. Figure 4.2 shows the prestrain response at 2%, 2.5%, 3%, and 4% prestrain. The material appears to present a deformation recovery of about 1%. Research by Shahverdi et al. (2018) and Gu et al. (2021) suggests that a 2% prestrain is enough to obtain optimal recovery stresses at temperatures below 200°C. The material manufacturer has recommended 2.5% prestrain to ensure enough phase transformation.

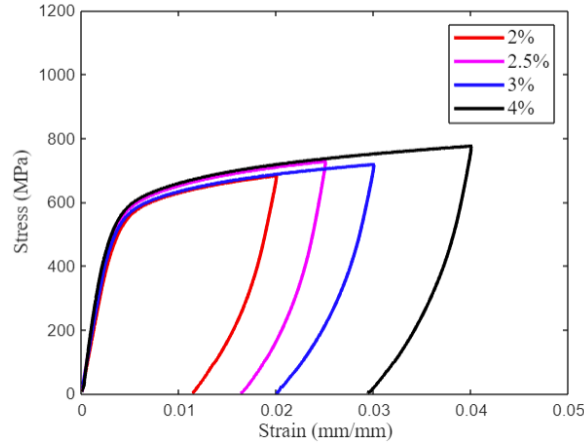


**Figure 4.2. Prestrained Fe-SMA**

To confirm that a prestrain of 2.5% (recommended by the manufacturer) results in adequate optimal recovery stresses, recovery stress tests with prestrains of 2.5% and 4% were compared. The recovery stress was evaluated around three activation temperatures: 140°C, 180°C, and 220°C as seen in Figure 4.3. A seed load of 100 MPa was selected to allow for thermal expansion to prevent buckling of the specimen and damage on the testing frame as suggested by Shahverdi et al. (2018) and Gu et al. (2021). The measured temperature showed data noise of about  $\pm 3^\circ\text{C}$ . The recovery stress tests show that the material has an austenite-start-temperature ( $A_s$ ) of about  $49.18^\circ\text{C}$ , and an austenite-finish-temperature ( $A_f$ ) of about  $159.70^\circ\text{C}$ , after which a second relaxation occurs until reaching the activation temperature.

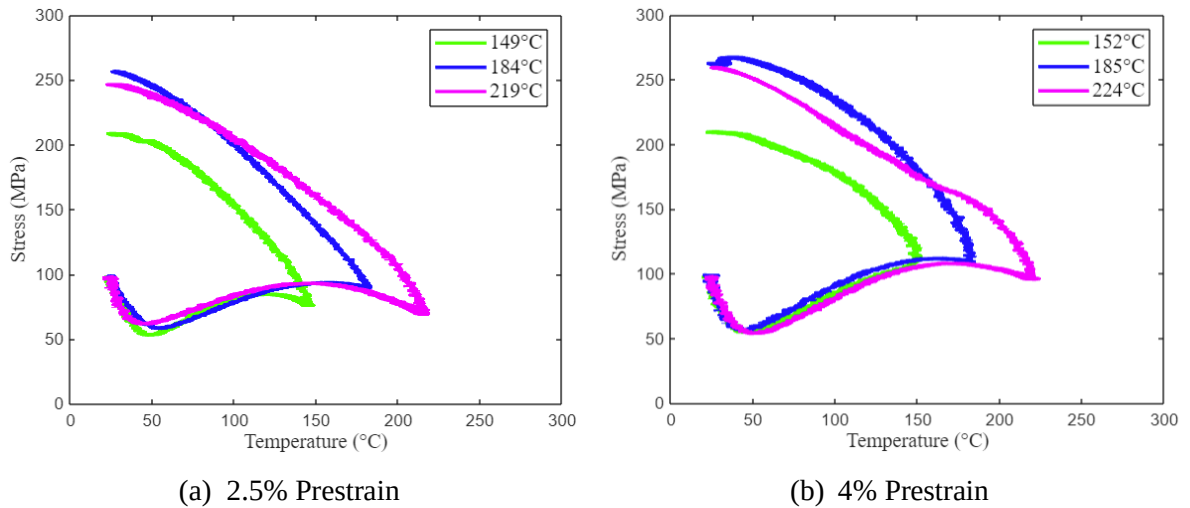


**Figure 4.3. Effect of Prestrain and Temperature on Recovery Stress**

The insulated heating tapes were used to determine the significance of the prestrain on the recovery stress of the Fe-SMA. It was difficult to obtain measurements at exact temperatures as the testing mechanism relied on manually turning off the voltage controller. In some cases, the tape continued to heat the specimens for a few seconds before the temperature decreased. Table 4.2 presents a summary of the activation temperature and the recovery stress per strain.

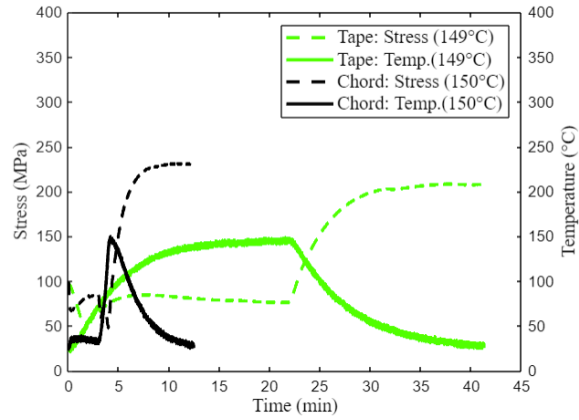
**Table 4.2. Prestrain and Temperature Effect on Recovery Stress**

| <b>Prestrain<br/>(%)</b> | <b>Temperature<br/>(°C)</b> | <b>Recovery Stress<br/>(MPa)</b> | <b>Percentage Difference<br/>(%)</b> |
|--------------------------|-----------------------------|----------------------------------|--------------------------------------|
| 2.5                      | 149                         | 208.54                           | 0.39                                 |
| 4                        | 152                         | 209.35                           |                                      |
| 2.5                      | 184                         | 256.29                           | 2.40                                 |
| 4                        | 185                         | 262.51                           |                                      |
| 2.5                      | 219                         | 246.23                           | 5.19                                 |
| 4                        | 224                         | 259.34                           |                                      |

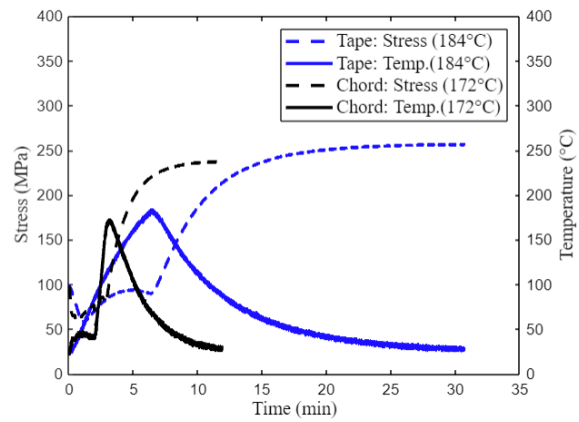
These five tests found that recovery stresses of samples with prestrains of 2.5% and 4% do not significantly change when using activation temperatures below 200°C. The percentage difference increases with temperature. A percentage difference of 5.19% is calculated when the activation temperature is around 220°C. The percentage difference in recovery stress may have also been influenced by the close but inaccurate activation temperatures. Overall, the results show that a prestrain of 2.5% with an activation temperature of around 180°C is enough for optimal recovery stress.

#### 4.2.2.1 Heating Rate

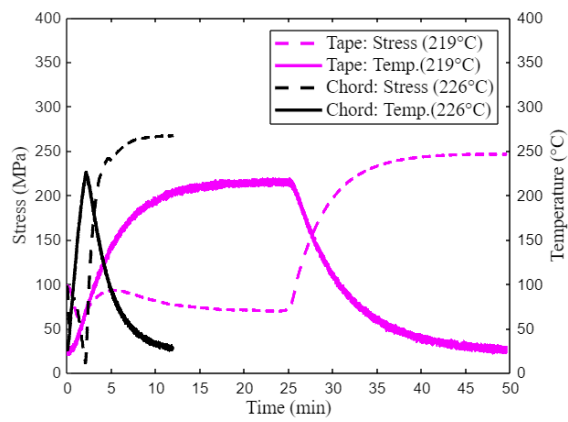
To decrease the time in reaching the activation temperature, insulating chords were used instead. Figure 4.4 shows a considerable time reduction when using the heating chords compared to the heating tapes. Reaching an exact activation temperature with the heating chords proved challenging as it relied on manually turning off a 2A power supply. Hence, close temperatures were used for comparison.



(a) Around 150°C



(b) Around 180°C



(c) Around 220°C

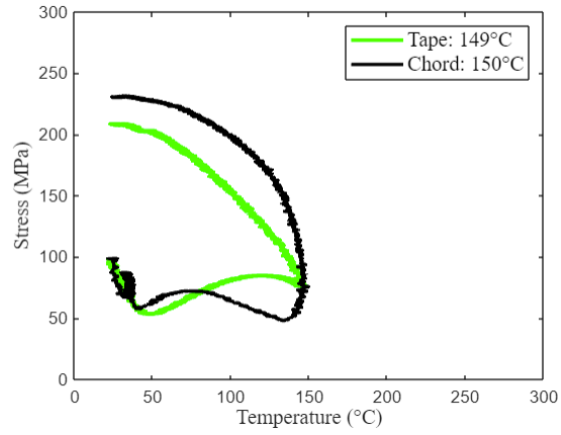
**Figure 4.4. Heating Rate**

The heating tapes were found to heat the Fe-SMA strips considerably slower than the chords. The chords showed a capacity to reach 226°C in less than 5 minutes using a 2A power source. The 150°C and 172°C activation temperature samples presented a delay at the start of the test due to breakage of the chords, resulting in longer heating times. For the tapes, it was found that a reduction in voltage increased the heating time. A voltage of 140V was needed to reach 219°C in 49.50 minutes and 184°C in 30.69 minutes. On the other hand, it was found that 120V was enough to reach 149°C in 41.26 minutes. Table 4.3 compares the recovery stress from the heating tapes and chords.

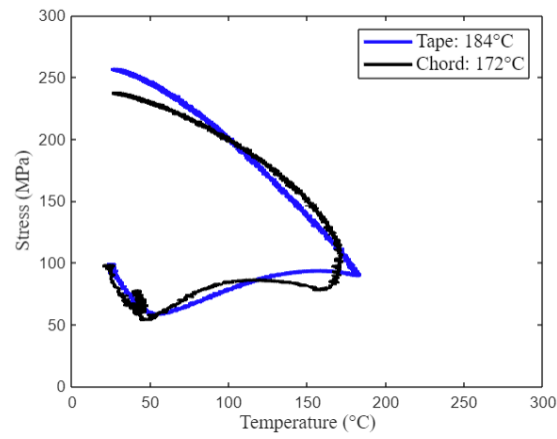
**Table 4.3. Heating Rate Effect on Recovery Stress**

| <b>Heating Method</b> | <b>Activation Temperature (°C)</b> | <b>Total Time (min)</b> | <b>Recovery Stress (MPa)</b> | <b>Percentage Difference (%)</b> |
|-----------------------|------------------------------------|-------------------------|------------------------------|----------------------------------|
| Tape                  | 149                                | 41.26                   | 208.54                       | 9.87                             |
| Chord                 | 150                                | 12.45                   | 230.18                       |                                  |
| Tape                  | 184                                | 30.69                   | 256.29                       | 7.81                             |
| Chord                 | 172                                | 11.90                   | 237.03                       |                                  |
| Tape                  | 219                                | 49.50                   | 246.23                       | 8.14                             |
| Chord                 | 226                                | 11.93                   | 267.13                       |                                  |

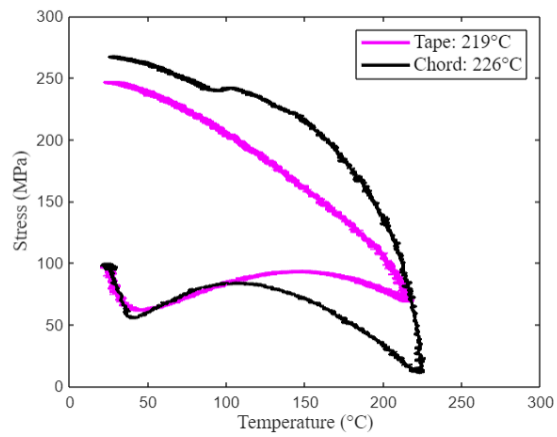
The recovery stresses between the heating tapes and chords present a percentage difference between 7.81% and 9.87%. Figure 4.5 shows how the temperature change affected the recovery stress. It is seen that the stress behavior on the material differs depending on the heating rate. A faster heating rate (chords) results in a lower austenite-finish temperature, a more pronounced second relaxation, and a convex temperature/stress curve during cooling. The recovery stress tests do not show a correlation between heating rate and recovery stress as suggested by Shahverdi et al. (2018). Note that lower temperatures tend to result in lower recovery stresses.



(a) Around 150°C



(b) Around 180°C



(a) Around 220°C

**Figure 4.5. Effect of Heating Rate on Recovery Stress**

#### 4.1.1.2 Recovery Stress Estimation

The temperature measurements presented data noise of about  $\pm 3^\circ\text{C}$  and relied on manually switching off the power supplies. The faster the heating rate, the more difficult was to reach a certain temperature. Hence, a temperature error margin of 5% was selected to estimate the recovery stress at a specific activation temperature. Previous tests showed that activating a 2.5% prestrained sample with temperatures around  $180^\circ\text{C}$  provides optimal recovery stresses. Figure 4.6 shows the recovery stress at activation temperatures  $180^\circ\text{C} \pm 9^\circ\text{C}$ .

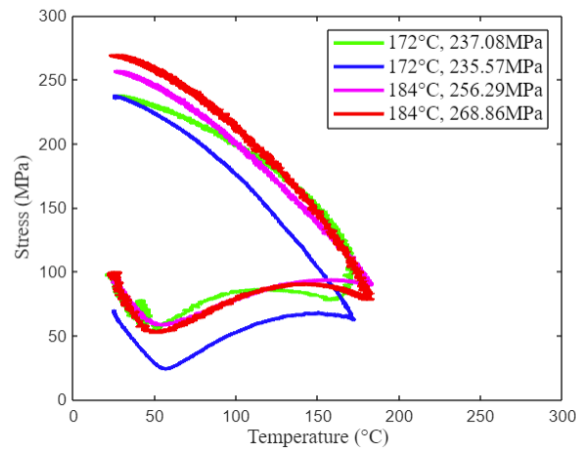
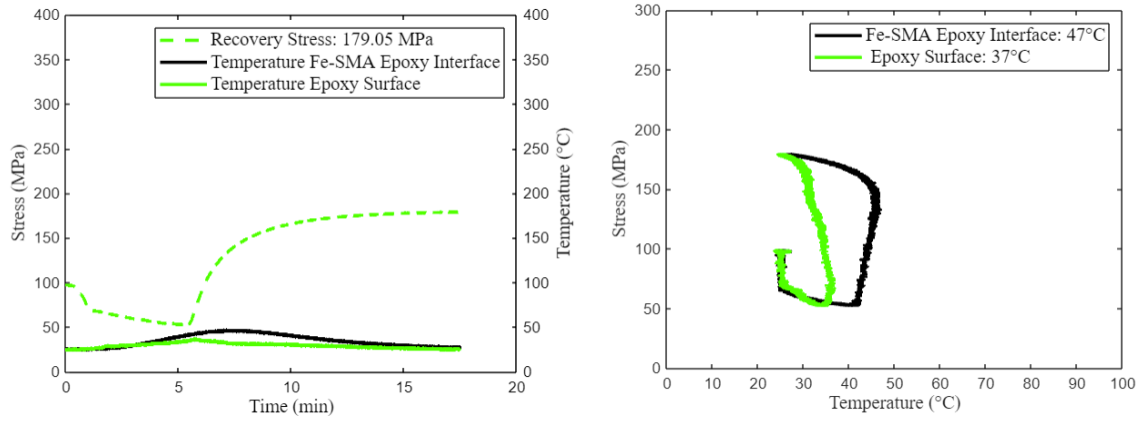


**Figure 4.6. Recovery Stress using  $180^\circ\text{C} \pm 9^\circ\text{C}$  Activation**

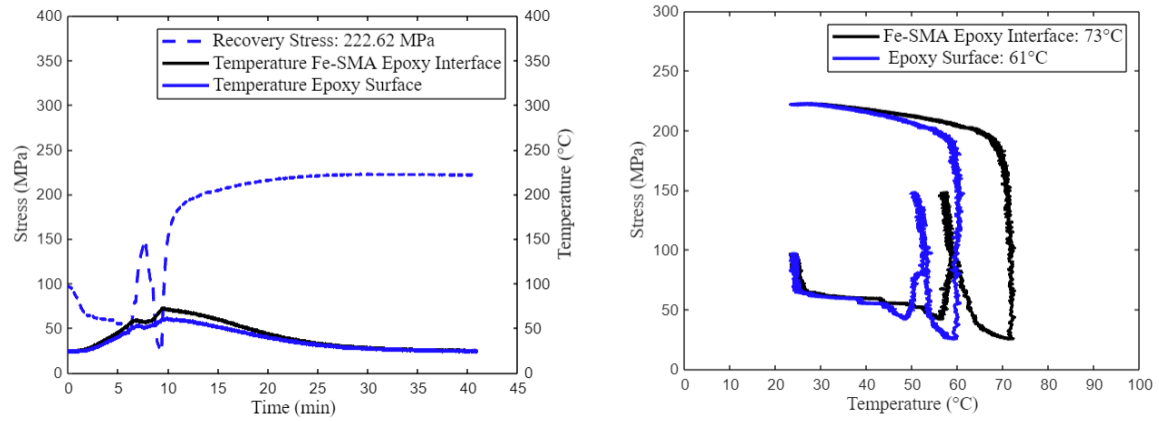
The samples with an activation temperature of  $172^\circ\text{C}$  show that seed load may not influence the resulting recovery stress as suggested by Shahverdi et al. (2018) and Gu et al. (2021). A seed load of 70 MPa and a seed of 100 MPa result in recovery stresses with a percentage difference of 0.64%. At the same activation temperature, the recovery stress presents a percentage difference of up to 4.79%. Additional tests are recommended to determine conservative recovery stress estimates. Based on the performed recovery stress tests, the material should experience an activation temperature of at least  $180^\circ\text{C}$  to ensure a recovery stress of at least 230 MPa.

### 4.3 Fe-SMA Epoxy Composite Recovery Stress

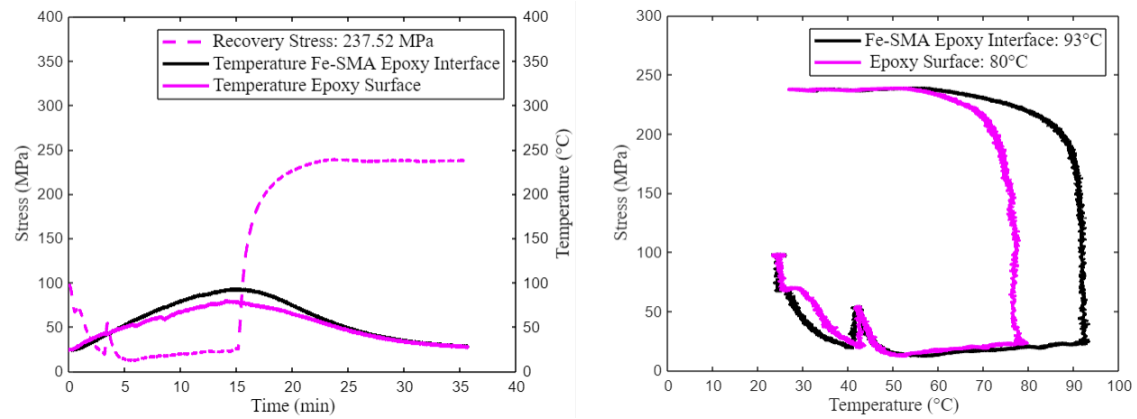
Three Fe-SMA epoxy composite samples were tested to determine their recovery stress at different temperatures. Two thermocouples were used to obtain the temperature at the Fe-SMA epoxy interface and the temperature on the epoxy surface. Figure 4.7 presents the recovery stress samples, based on both temperature measurements. It was determined that the epoxy surface experiences a heating delay compared to the Fe-SMA embedded into the epoxy.



(a) Sample I



(b) Sample II



(c) Sample III

**Figure 4.7. Recovery Stress Fe-SMA Epoxy Composite**

Temperature measurement delay is most noticeable at lower temperatures. The stress starts increasing before cooling of the interface as seen in Figure 4.7(a). The interface shows a temperature up to 13°C higher than the epoxy surface. The composite coupons show good recovery stresses, despite being activated at relatively low temperatures. It is estimated that the thermocouple at the interface does not represent the actual temperature on the Fe-SMA strip due to the different thermal conductivities of the epoxy and Fe-SMA. While the austenite-start temperature is about 49.18°C, Sample I shows a recovery stress of 179.05 MPa using an activation temperature of 47°C.

Sample II and Sample III present high recovery stresses as seen in Figure 4.7(b) and Figure 4.7(c), respectively. The magnitude of the recovery stresses may indicate that the Fe-SMA strip reached higher temperatures than the ones measured. An interface activation temperature of 93°C leads to a recovery stress of 237.52 MPa which is comparable to stresses obtained at 172°C (Figure 4.6). The heating time was found to be significantly longer as the chords (conductive heating) were placed outside the epoxy coupon. Overall, the Fe-SMA epoxy composite specimens were able to withstand high temperatures and high recovery stresses without cracking, showing the feasibility of using a Fe-SMA epoxy matrix for concrete prestress.

#### **4.4 Shear Critical RC Beam Specimens**

Three shear-critical RC concrete beams were tested till failure to determine the feasibility of a NSB Fe-SMA epoxy composite for shear strengthening. One of the beams is used as control and the other two are active shear strengthened. The results of the three beams are evaluated based on shear strength, displacement, cracking width and propagation, and reinforcement strains. Figure 4.8 shows the test setup with specimen A-200S-180C after testing.

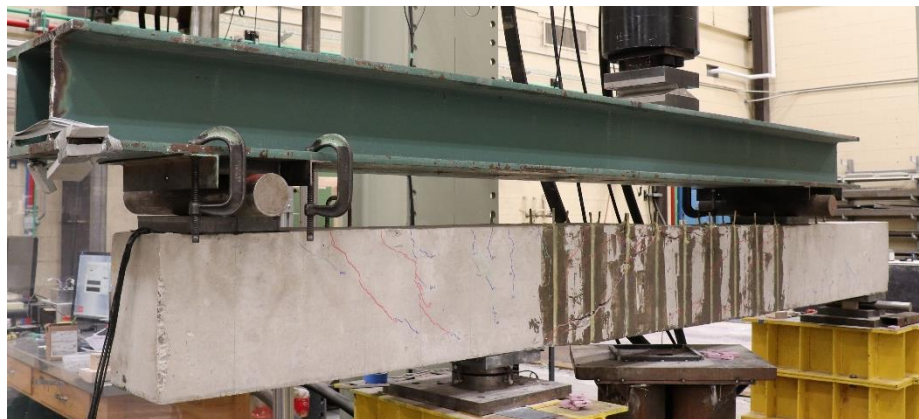


**Figure 4.8. Specimen Test Setup**

The RC beams are expected to be point symmetric with opposite but identical response. However, experimental testing showed that the section closer to the cantilever presented larger crack widths and displacements. Additionally, as specimen N-0S was loaded, the concrete under the supports was crushing (Figure 4.9), resulting in noise within the acquired data. Hence, two 1/16” thick neoprene sheets were placed in between the supports and the RC beam in the following tests. It was also noted that the spreader beam deflected during testing.



**Figure 4.9. Concrete Crushing under Support**

#### **4.4.1 Failure Modes**

Testing of the RC beams confirmed the beams were shear-critical on the test span. A summary of the failure mode of the three specimens is presented in Table 4.4. Specimen N-0S failed due to diagonal tension (shear) with a critical diagonal crack (CDC) appearing within the test span from the support closer to the cantilever (Figure 4.10). The self-weight of the RC beam is attributed to be cause of the non-symmetric response. The CDC widened and propagated until the specimen reached a shear load of about 174 kN. No major damage was observed in the transfer spans.

**Table 4.4. Specimen Failure Mode**

| <b>Specimen</b> | <b>Failure Mode</b>                        |
|-----------------|--------------------------------------------|
| N-0S            | Diagonal tension                           |
| A-200S-180C     | Diagonal tension and interfacial debonding |
| A-100S-180C     | Side concrete cover separation             |

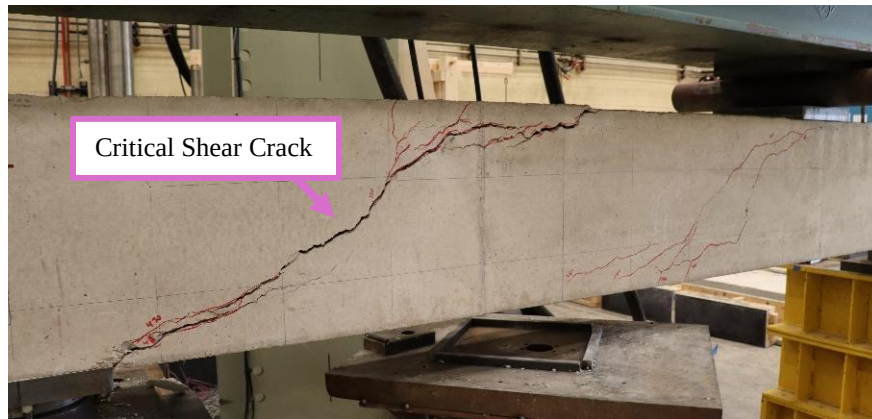
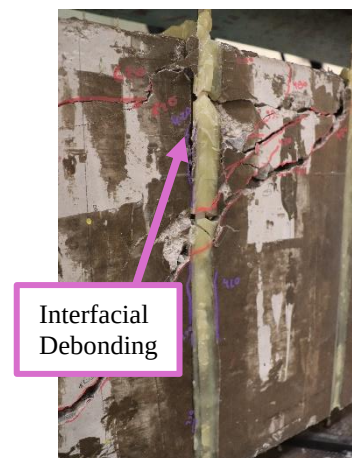


**Figure 4.10. Shear Failure in Specimen N-0S**

Similarly, specimen A-200S-180C failed due to a CDC within its test span as seen in Figure 4.11(a). The interruption of the diagonal cracks by the NSB Fe-SMA strips resulted in narrower diagonal cracks compared to specimen N-0S. The CDC crossed an NSB strip and widened, rising shear stresses in the NSB Fe-SMA – concrete interface and developing longitudinal cracks along the groove. These longitudinal cracks resulted in the detachment of the ends of the NSB Fe-SMA strip as seen in Figure 4.11(b). Interfacial debonding started becoming evident at a shear load of around 154 kN. No major damage was seen in the transfer spans. Specimen A-200S-180C failed when reaching a load of about 221 kN.



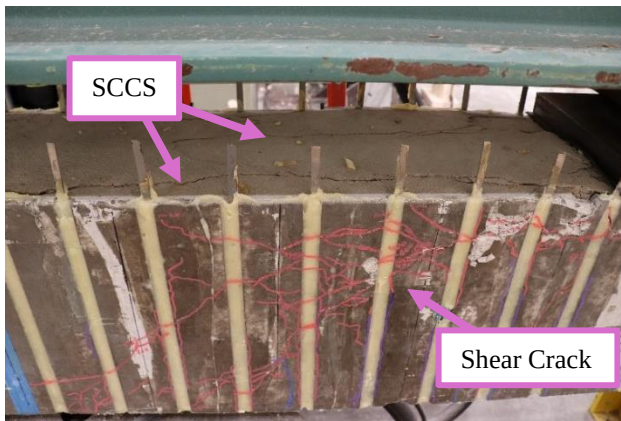
(a) Shear Failure



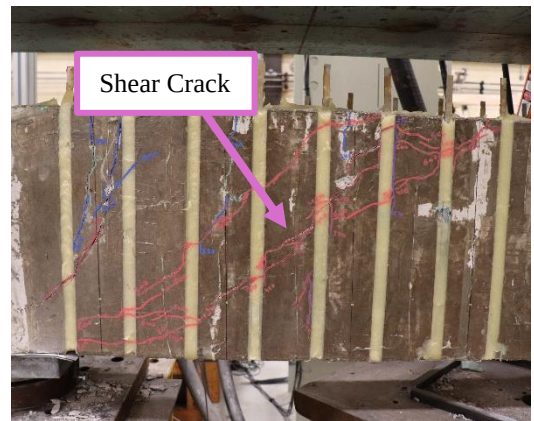
(b) Interfacial debonding

**Figure 4.11. Failure of Specimen A-200S-180C**

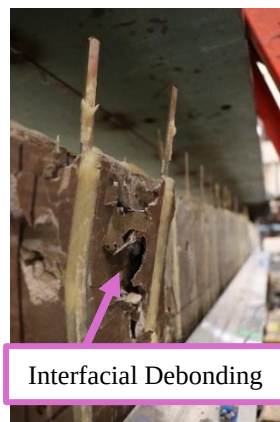
Specimen A-100S-180C appears to fail due to side concrete cover separation as shown in Figure 4.12(a). While a dense NSB Fe-SMA spacing interrupted diagonal cracks and prevented the development of CDCs (Figure 4.12(b)), it weakened the stirrup plane. An interrupted diagonal crack propagated until the loading point of the test span where the u-wrapped NSB strips terminate. From the loading point, longitudinal cracks form along the top face of the beam in the compression region, separating the side concrete cover and reaching ultimate conditions. Note that side concrete cover separation only happened at the opening of the activated u-wrapped NSB strips, preventing failure in the test span section closer to the cantilever. Post-ultimate, interfacial debonding begins on NSB strips beside the loading point as presented in Figure 4.12(c). It is also possible that the SME on the activated strips may result in the strips trying to unbend.



(a) Side Concrete Cover Separation (SCCS)



(b) Mitigation of CDCs



(c) Post-peak Interfacial Debonding

**Figure 4.12. Response of Specimen A-100S-180C**

#### 4.4.2 Shear Load and Displacement Response

The load-displacement response of the three specimens is shown in Figure 4.13. The displacement was measured under the loading points with two string pots: P-TR-V (cantilever) and P-TR-2V (inside span). A reduction in applied load was noticed when the tests were paused for specimen assessment. The active shear strengthened specimens exhibit higher strengths and displacements compared to the specimen N-0S. Table 4.5 presents a summary of the peak load of each specimen and their respective displacement. As the Fe-SMA ratio increases, the cantilever displays larger displacements than the inside span, but it does not result in an increase of shear strength.

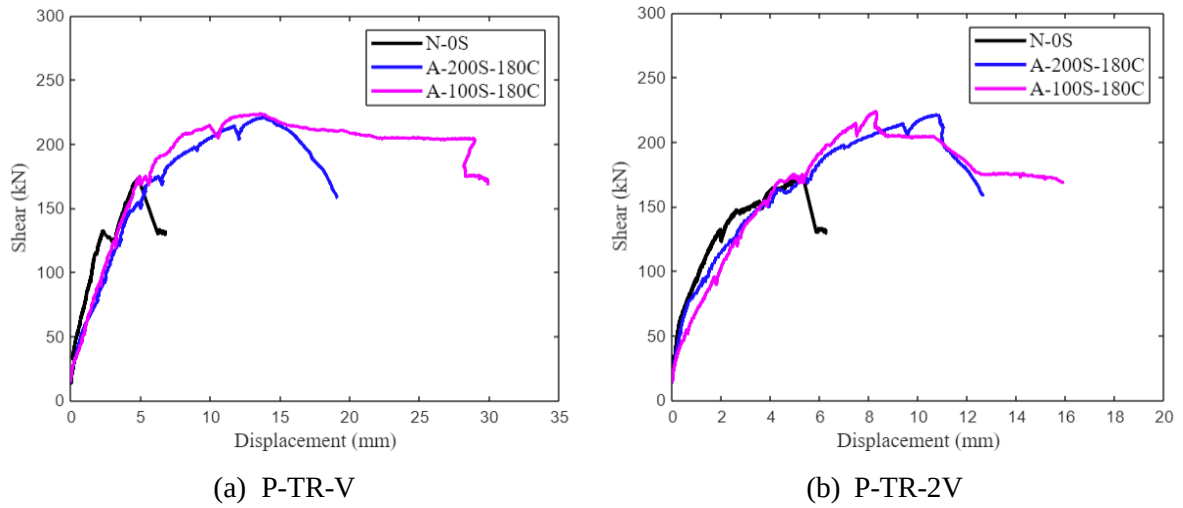


**Figure 4.13. Load-Displacement Response**

**Table 4.5. Peak Load and Displacement**

| Specimen    | Peak Load<br>(kN) | Displacement<br>(mm)                    |
|-------------|-------------------|-----------------------------------------|
| N-0S        | 174               | Cantilever: 4.83<br>Inside span: 5.31   |
| A-200S-180C | 221               | Cantilever: 13.85<br>Inside span: 11.09 |
| A-100S-180C | 224               | Cantilever: 13.58<br>Inside span: 8.25  |

The load-displacement response of the control beam (N-0S) exhibit a shear strength of about 174 kN, resulting in a percentage difference of around 0.68% compared to the estimated nominal capacity using CSA A23.3 (2019). Note that the beam experienced a sudden load of about 64 kN before testing due to the frame pump mechanism. Specimen N-0S presents a percentage difference of 9.47% between peak displacements at the cantilever and inside span, with the highest displacement happening on the later.

Active shear strengthening the test span with 0.05% NSB active Fe-SMA as in specimen A-200S-180C, leads to a strength increase of about 27%. The specimen also exhibits a 186.75% and 108.85% increase in the peak load displacement of the cantilever and inside span, respectively. The cantilever experiences larger displacements.

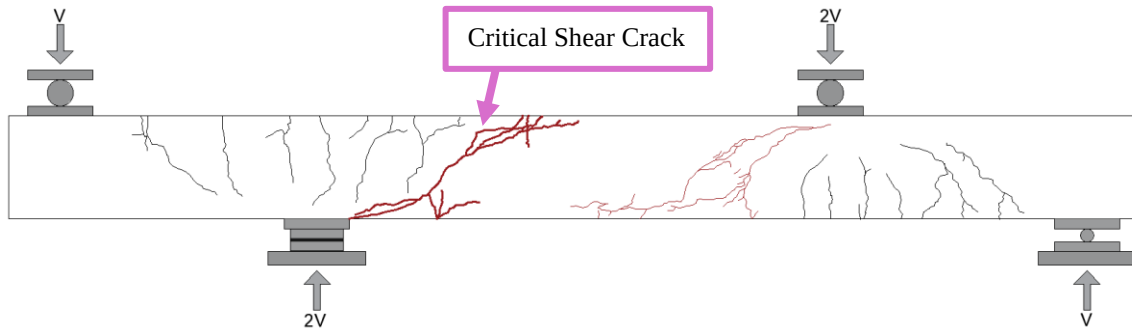
Increasing the ratio of activated NSB Fe-SMA may not result in an increase of shear strength and may be detrimental due to possible side concrete cover separation as in specimen A-100S-180C. Specimen A-100S-180C doubles the Fe-SMA ratio to 0.1%, but it only displays a shear strength increase of 1.34% compared to specimen A-200S-180C. Increasing the Fe-SMA ratio does affect the ductility of the beam: peak load displacements reduce and post-peak displacements increase. Specimen A-200S-180C exhibits a stiffer post-peak response, reaching higher displacements at higher loads compared to specimen A-100S-180C, specially at the cantilever.

#### **4.4.3 Cracking**

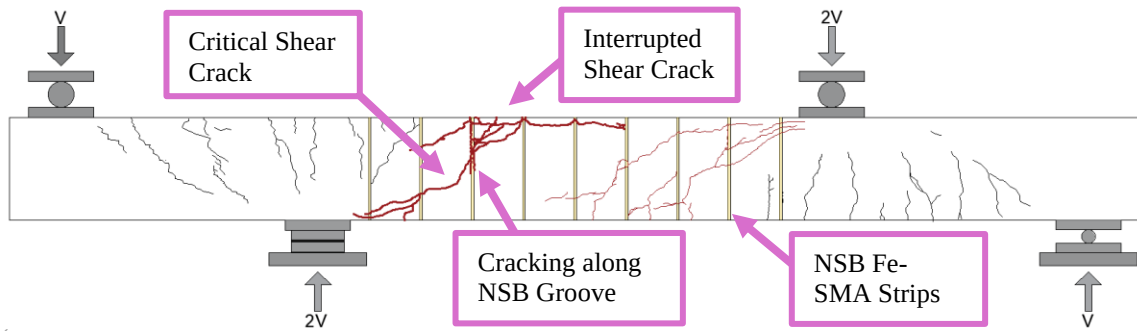
Shear crack propagation and width were recorded at multiple load steps during testing. Shear cracks appeared suddenly, propagated and widened within the test span. Figures 4.14(a), 4.14(b), and 4.14(c) display the crack pattern on specimens N-0S, A-200S-180C, and A-100S-180C, respectively. The three beams showed one side of the beam receiving more damage than the other. Specimens N-0S and A-200S-180C show a critical diagonal crack in the test span section closer to the cantilever. On the other hand, specimen A-100S-180C exhibits a wide flexural - shear crack at the support of the cantilever but fails due to side concrete cover separation at the other section of the test span. Photos of the experimental testing results can be found in Appendix A.

In the strengthened specimens, the diagonal cracks are interrupted by the NSB Fe-SMA strips and spread in between the grooves. It is noticed that the epoxy does crack and follow the propagation path. Reducing the spacing of the grooves, prevents the development of critical diagonal cracks as seen in specimen A-100S-180C. The interruption of the diagonal cracks result in an increase of shear stresses at the NSB Fe-SMA – concrete interface, forming longitudinal cracks along the grooves.

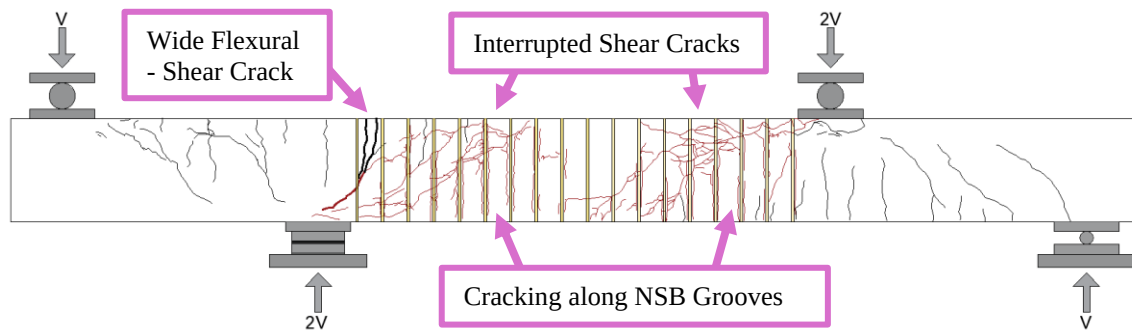
Specimen A-200S-180C exhibits considerable longitudinal cracks along the groove at the NSB strip that experienced interfacial debonding. On the other hand, in specimen A-100S-180C, multiple NSB strips present narrow longitudinal cracks along the grooves.



(a) Specimen N-0S



(b) Specimen A-200S-180C



(c) Specimen A-100S-180C

**Figure 4.14. Crack Propagation**

Table 4.6 provides a summary of the shear load and maximum shear crack widths of each specimen. Shear cracks appeared in the three specimens at a shear load of about 124 kN. Specimen N-

0S manifests the widest shear cracks per load step with critical diagonal cracks forming at a shear load of 174 kN, while specimen A-200S-180C presents the narrowest cracks.

**Table 4.6. Shear Crack Width**

| <b>Specimen</b> | <b>Shear Load<br/>(kN)</b> | <b>Maximum Shear Crack Width<br/>(mm)</b> |
|-----------------|----------------------------|-------------------------------------------|
| N-0S            | 124                        | 0.6 - 0.8 (medium)                        |
|                 | 154                        | > 1.5 (wide)                              |
|                 | 174                        | 5 - 8 (wide)                              |
| A-200S-180C     | 124                        | 0.15 - 0.2 (narrow)                       |
|                 | 154                        | 0.5 – 0.9 (medium)                        |
|                 | 174                        | 2 (wide)                                  |
|                 | 214                        | 4 (wide)                                  |
|                 | 221                        | 13 (wide)                                 |
| A-100S-180C     | 124                        | < 0.1 (hairline)                          |
|                 | 154                        | 0.25 (narrow)                             |
|                 | 174                        | 0.5 - 0.8 (medium)                        |
|                 | 214                        | 2 (wide)                                  |
|                 | 224                        | 2 (wide)                                  |

At a shear load of 124 kN, specimen A-200S-180C exhibited narrow diagonal cracks within the test span that cross the epoxy of the NSB Fe-SMA. The specimen also showed narrow longitudinal cracks alongside the groove of a NSB strip crossed by a diagonal crack. When the specimen experienced a shear load of 154 kN, interfacial debonding of the NSB Fe-SMA strip became evident. Critical diagonal cracks formed at a peak shear load of 221 kN. Flexural cracks in this specimen appeared to be less than 0.25 mm. Specimens N-0S and A-200S-180C displayed hairline and narrow flexural cracks.

As specimen A-100S-180C is loaded, hairline cracks started appearing alongside the grooves. At a shear load of 154 kN, narrow diagonal cracks (0.25 mm) formed within the test span and the cantilever. Additionally, narrow flexural cracks developed above the support closer to the cantilever, reaching widths of around 0.2 mm to 0.3 mm. At a shear load of 174 kN, a flexural crack above the cantilever support widened to 0.4 mm, and the shear crack propagation within the test span led to the cracking of the epoxy in the grooves. The diagonal cracks within the test span reached a maximum width of 2 mm at a shear load of 214 kN. At a peak shear load of 224 kN, the width of the diagonal cracks did not increase, but a wide (4 mm – 8 mm) flexural – shear crack developed above the cantilever support.

The maximum width of the shear cracks within the test span decreased by strengthening the beams with NSB active Fe-SMA strips. Specimen A-200S-180C and A-100S-180C shows a maximum shear crack width reduction of at least 60% and 84%, respectively, compared to specimen N-0S. Reducing the spacing of the shear strengthening material also reduces shear crack widths. At peak load, specimen A-100S-180C reduced the maximum diagonal crack width by around 85% compared specimen A-200S-180C.

#### **4.4.4 Reinforcement Strains**

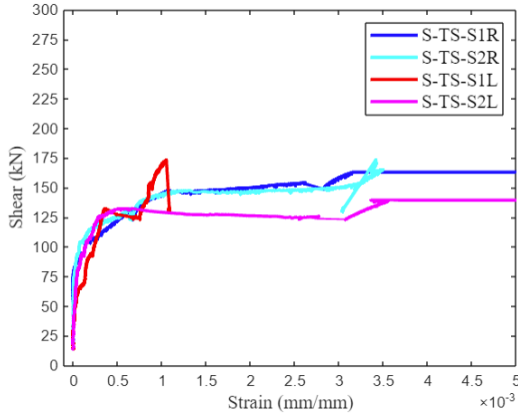
Reinforcement strains were obtained at critical locations along the beam. Four strain gages were placed in the shear-critical test span. One strain gauge was placed in the transfer span to ensure the absence of shear failure in this span. Moreover, a strain gauge in one tension longitudinal bar have been collected at the most critical moment location to determine the failure mechanism on the strengthened beams. Yielding of the reinforcement is expected to happen at a strain of about 0.0025 mm/mm.

##### **4.4.4.1 Transverse Reinforcement in Test Span**

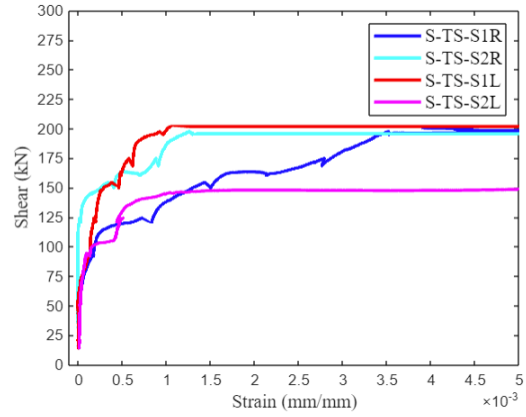
Reinforcement strains in the shear-critical test span were obtained at stirrups around  $d_v$  away from the face of the support/loading plate. Figure 4.15 presents the strains in the stirrups of the test span of each specimen. The three specimens show the most critical strain gauge being S-TS-S2L due to being located where the critical diagonal cracks appear. Strain gauge S-TS-S1L, being within  $d_v$  away from the face of the support, may not experience significant strains.

The strain gauges in specimen N-0S appear to start contributing after experiencing a shear load around 75 kN. Considerable contribution is observed at a load of around 125 kN when wide diagonal

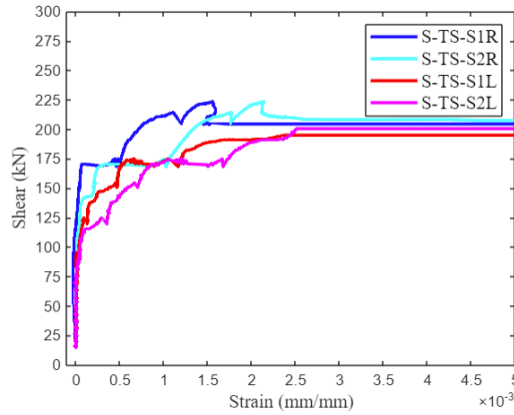
cracks develop. Similarly, specimen A-200S-180C show strain gauge contribution beginning at 75 kN, except for strain gauge S-TS-S2R which may not be in the path of a diagonal crack until about 150 kN. Strain gauges in specimen A-100S-180C show strain gauges closer to the cantilever start acting at around 100 kN while strain gauges S-TS-S1R and S-TS-S2R start contributing at around 140 kN and 175 kN, respectively.



(a) Specimen N-0S



(b) Specimen A-200S-180C



(c) Specimen A-100S-180C

**Figure 4.15. Strains at Stirrups in the Test Span**

Strain gauges on the side of the beam with smaller shear cracks in specimens N-0S and A-200S-180C yield at larger shear loads compared to strain gauges closer to critical diagonal cracks. On the other hand, strain gauges in specimen A-100S-180C yield around the same load. The short spacing between the NSB Fe-SMA strips mitigates the development of critical diagonal cracks. The epoxy in the grooves interrupt the propagation of diagonal cracks, resulting in a more scattered crack propagation in between the grooves.

The comparison of the strains at each location can be seen in Figure 4.16. The strengthened beams show a delay on the yielding of the stirrups and a reduction of strains compared to the specimen N-0S. Stirrups in specimens A-100S-180C and A-200S-180C generally show that reducing the spacing of the NSB active Fe-SMA strips leads to a reduction in the stirrup stresses. This behavior, however, is not observed in strain gauge S-TS-S1L. The interruption and scattered propagation of the shear cracks between the grooves may result in the diagonal cracks developing at different locations. Evaluating the most critical strain gauge (S-TS-S2L), it appears that strengthening the test span with 0.05% and 0.1% active Fe-SMA may result in the stirrups yielding at higher shear loads (18% and 59%, respectively).

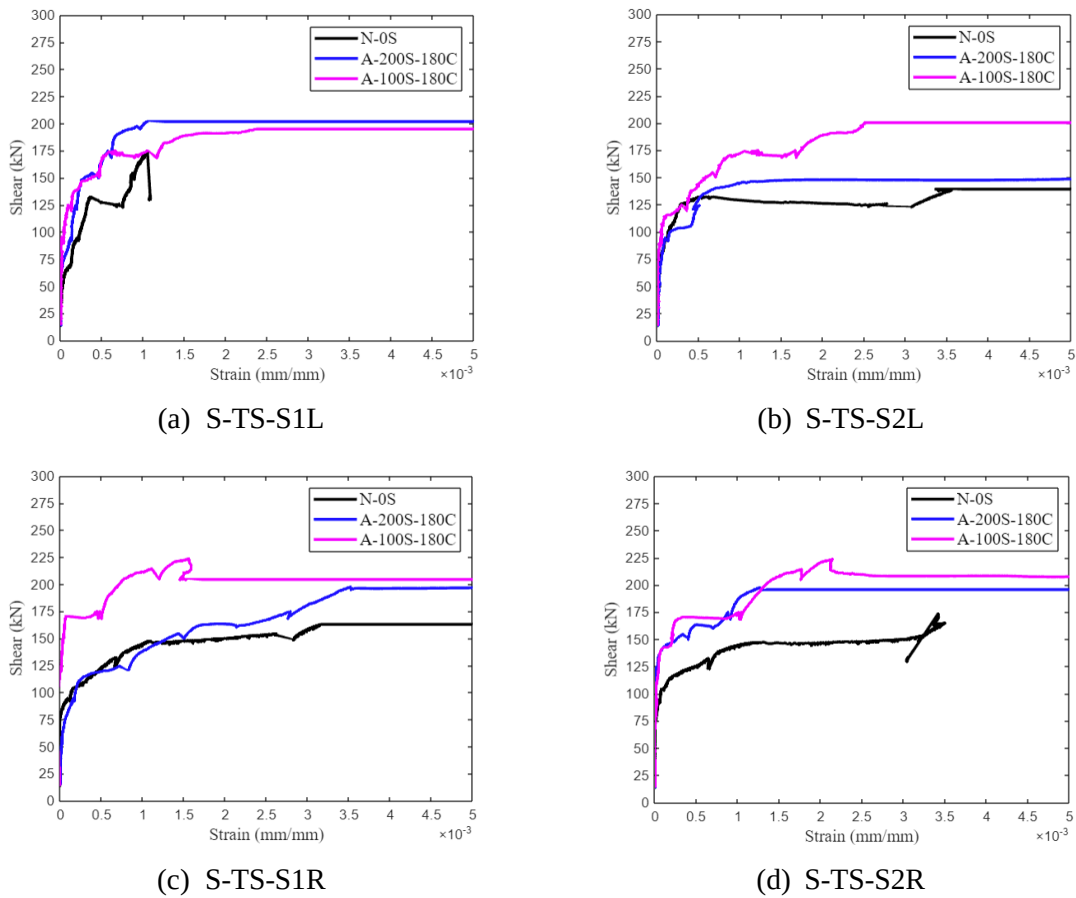


**Figure 4.16. Strain Gauges Test Span**

#### 4.4.4.2 Transverse Reinforcement in Transfer Span (Cantilever)

The strain gauge measures in the transfer span (cantilever) are displayed in Figure 4.17, confirming that these stirrups do not yield. The strengthened specimens allow for larger loads and larger strains in

the transfer spans. While the stirrup strains in specimens N-0S and A-200S-180C are similar and start acting at around 75 kN, the strain gauge in specimen A-100S-180C appears to start contributing at around 50 kN. Decreasing the spacing of the NSB Fe-SMA strips in the test span increases the stiffness of the beam, resulting in larger stresses through the unstrengthen transfer span.

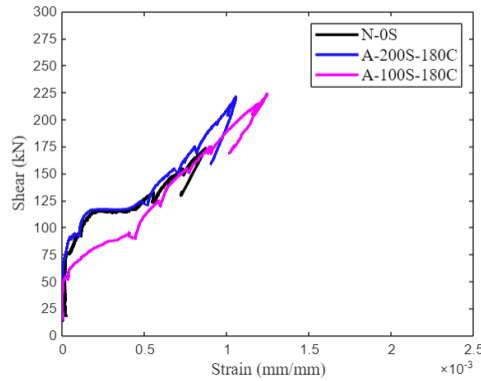


**Figure 4.17. Strain Gauge S-TR-S**

#### 4.4.4.3 Tension Longitudinal Bar

Figure 4.18 presents that the strains in one tension longitudinal bar at a maximum moment location. The strengthened beams showed the capacity to deflect at larger loads, increasing the stresses in their tension reinforcement. Specimen A-100S-180C exhibit larger strains at lower loads due to the significant deflection of its cantilever, with a wide flexural - shear crack forming on top of the cantilever support. Despite the tension reinforcement experiencing larger strains, the bars did not yield, confirming that the three specimens were still shear critical despite the considerable deflections of the strengthened beams.

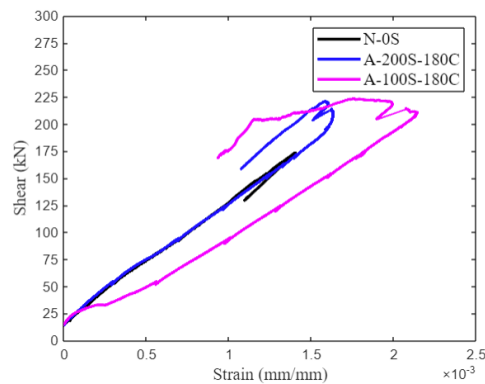


**Figure 4.18. Strain Gauge S-TR-B**

## **Chapter 5**

### **Finite Element Analysis**

#### **5.1 General**

Finite element analysis (FEA) models were developed to estimate the behavior of the three RC beam specimens. The model is further calibrated and validated using the experimental results. The beams were modeled using the non-linear two-dimensional FEA program Vector2. This chapter is divided into three phases:

1. Preliminary FEA model
2. Calibrated FEA model
3. Parametric study on prestress and Fe-SMA ratio

Preliminary FEA models were developed to estimate the strength and performance of the beam specimens. The model was verified based on the shear strength of the control beam calculated with CSA A23.3 (2019). The preliminary results were used to estimate the required testing frame capacity, strain gauge location, and expected displacement. The FEA models were then calibrated and validated to match the testing conditions of the experimental RC beam specimens. The results of the models were implemented to assess beam strength, displacement, reinforcement stresses, and crack width and propagation.

#### **5.2 Vector2**

Vector2 is a two-dimensional non-linear finite element analysis program developed by the University of Toronto to estimate the load-deformation response of reinforced concrete (RC) membrane structures (Wong et al., 2013). Wong et al. (2013) state that the program is based on the MCFT (Vecchio & Collins, 1986) and DSFM (Vecchio, 2000) which are analytical models that consider the behavior of RC elements under in-plane normal and shear stresses. Vector2 predicts a realistic cracking distribution under short-term loading conditions which can be monotonic, cyclic, and reverse cyclic. Cracked concrete is modeled using smeared rotating cracks in an orthotropic material (Wong et al., 2013).

Vector2 uses the secant stiffness method to estimate RC non-linear response with total load increments. Wong et al. (2013) include that the RC behavior modeled by Vector2 has been

corroborated with experimental tests. The program employs graphic-based processors to visualize FE model and its response. The program uses FormWorks © as its pre-processor and Augustus © as its post-processor.

### 5.3 Preliminary Model

Preliminary FEA models of the experimental RC beam specimens were done to determine the maximum load and expected displacement to select an appropriate testing frame. A preliminary model of the control RC beam was used to verify the failure mode and the shear strength of the control beam (N-0S) based on CSA A23.3 (2019). The preliminary model use default material models.

#### 5.3.1 FEA Model

The simply supported RC beams were modeled using three RC regions: two transfer spans and one test span. Supports and load bearing plates are also modeled as regions. The plates have a total thickness of 76.2 mm. For a more realistic load distribution, 38.1 mm thick unidirectional bearing regions are included between different modeled structures. A unidirectional bearing layer can develop a realistic force distribution and crack pattern by considering horizontal strains (Sadeghian & Vecchio, 2018).

In Vector2, applied loads can be implemented as nodal loads (load-controlled) or support displacements (displacement-controlled). When the model is load-controlled convergence stops at maximum load (kN). To obtain post-peak response, a displacement-controlled model should be used instead. A displacement-controlled model was achieved by including a stiff spreader beam (269 mm x 300 mm x 4000 mm) to transmit the load to the RC beam and assess displacements at the cantilever and the inside span. The built model is presented in Figure 5.1.

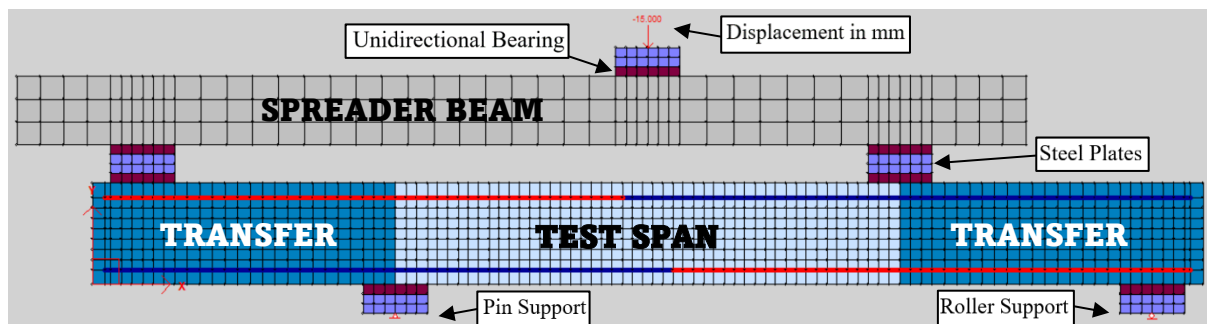


Figure 5.1. FEA Model

The RC regions diverge in their transverse reinforcement. The transverse reinforcement in the beams is input as 90° smeared reinforcement as part of each RC concrete region. The test span uses single-leg US #3 bars ( $\phi = 9.53$  mm), resulting in a reinforcement ratio of 0.12%. The transfer spans use double-leg 10M bars ( $\phi = 11.3$  mm) that represent a reinforcement ratio of 0.67%. On the other hand, the 25M ( $\phi = 25.2$  mm) longitudinal reinforcement was modeled as truss elements using default ductile steel reinforcement.

Specifics on mesh refinement are not part of the scope of the project. Default Vector2 parameters are selected for the FEA. Regions are meshed using hybrid rectangular elements. The test span of the RC beam is modeled using 40 mm x 40 mm elements while the spreader beam uses 100 mm x 100 mm elements. The rest of the regions use 50 mm x 50 mm elements. Truss elements are employed to model the longitudinal reinforcement. In summary, the model has 1161 rectangular elements, 292 truss elements, and 1329 nodes.

The displacement-controlled model applies a displacement of – 15 mm for the analysis. Experimental testing showed that self-weight considerably affects the behavior of the RC beam, resulting in critical diagonal cracks forming in the test span closer to the cantilever. Hence, self-weight was incorporated as a gravity load of 2500 kg/m<sup>3</sup> at each element of the RC beam. A pin support is incorporated under the highest expected reaction.

### **5.3.2 Materials**

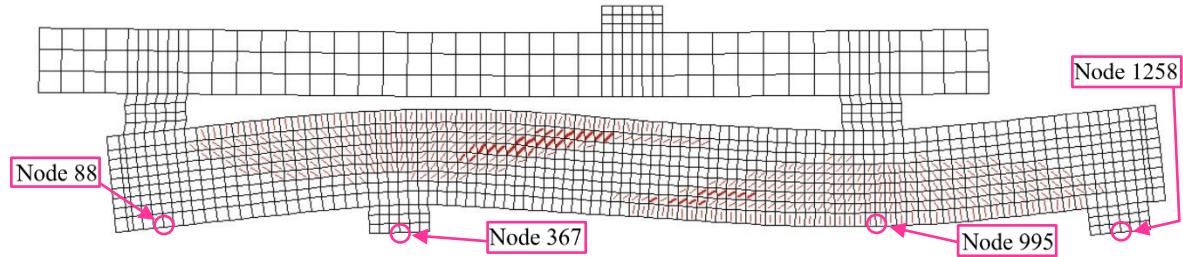
The model uses concrete with a compressive strength of 30 MPa and a maximum aggregate size of 20 mm. All structural steel present an elastic modulus of 200000 MPa. Steel reinforcement is modeled as ductile steel reinforcement with a yield strength of 400 MPa, tensile strength of 600 MPa, strain hardening onset of 10 me, and ultimate strain of 150 me.

The support/loading plates and the spreader beam are modeled structural steel. The plates are made of 44W steel with a yield strength of 300 MPa. The spreader beam uses default yield strength of 400 MPa, tensile strength of 600 MPa, and an elastic modulus of 900000 MPa to increase stiffness. The unidirectional bearing layers are modeled at 90° with an initial tangential elastic modulus of 10000 MPa.

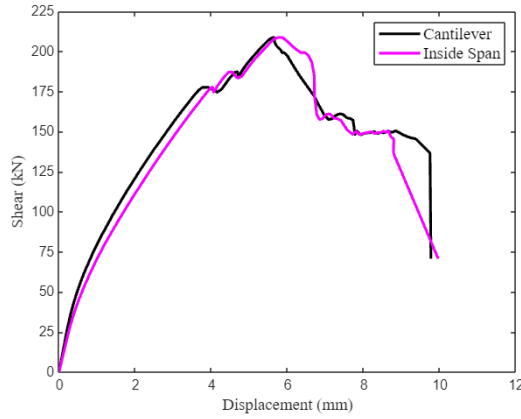
### **5.3.3 Verification**

The displacement-controlled FEA of the designed control beam was done using 200 load stages with an incremental factor of 0.005. The analysis confirmed that the RC beam fails in shear within the tests

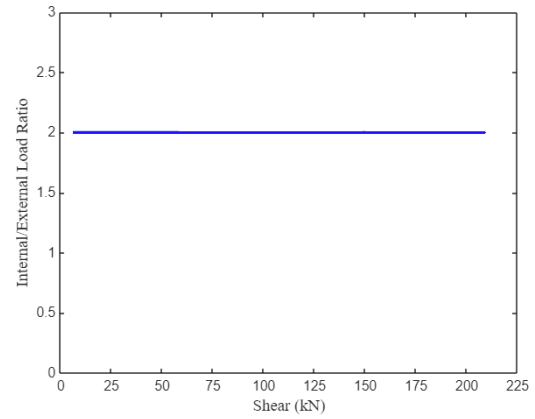
span as seen in Figure 5.2(a). The widest diagonal cracks appear in the test span closer to the cantilever due to self-weight. The control beam is expected to fail at shear load around 208.82 kN. Figure 5.2(b) shows the load-displacement response of the beam at the cantilever (Node 88) and at the inside span (Node 995). Reaction loads at Node 1258 and Node 367 confirm that the internal applied loads and double the external ones as presented in Figure 5.2(c).



(a) Displacement and Cracking Response



(b) Load-Displacement Response



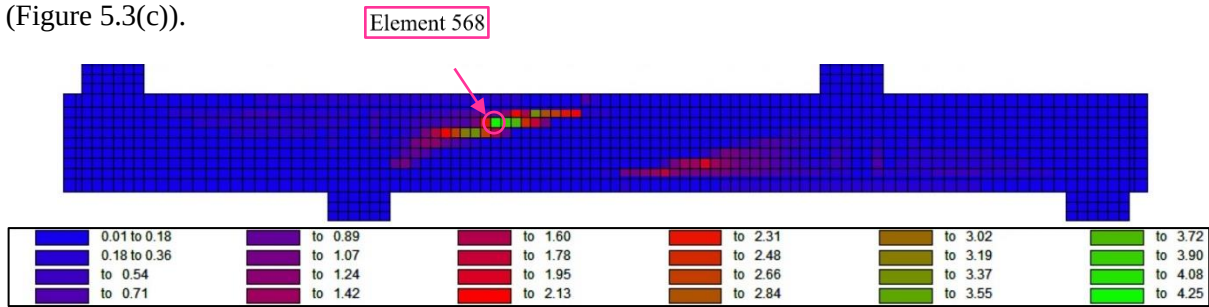
(c) Load Ratio

### Figure 5.2. Preliminary Model Verification

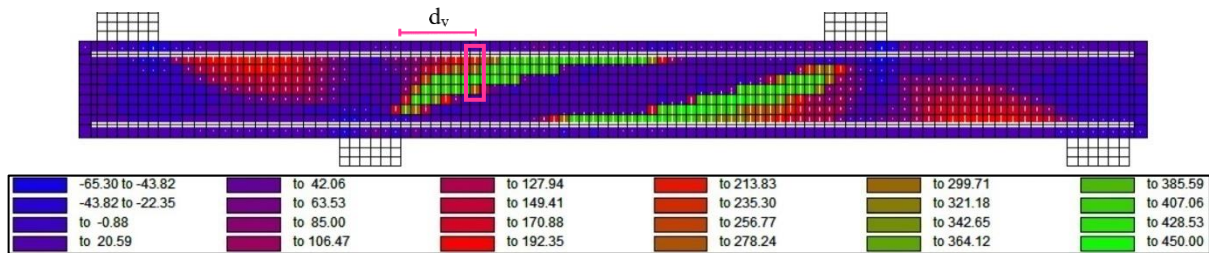
The model was further verified using the failure criteria from CSA A23.3 (2019), estimating shear failure at  $d_v$  away from the face of support/load plate. CSA A23.3 (2019) estimates failure at a shear load of 172.82 kN and a moment of 97.5 kNm. The Vector2 model predicts that the ultimate internal shear force and moment at this location are around 194.4 kN and 111.13 kNm, respectively. CSA A23.3 provides a conservative capacity prediction. The percentage difference between the results from CSA A23.3 and Vector2 model is around 13.07%. Modeling the transverse reinforcement as smeared reinforcement instead of discrete truss bars may lead to strength overestimation.

## 5.4 Critical Nodes and Elements

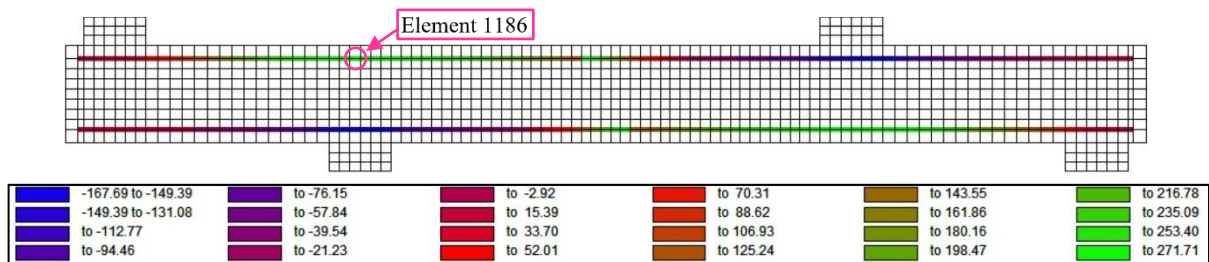
The assessment of the response of the RC beams is performed by identifying critical nodes to determine maximum displacement (Figure 5.2(a)) and critical elements to evaluate average crack width, average stirrup stresses, and longitudinal reinforcement stresses. The section of the test span closer to the cantilever experiences the most damage due to self-weight. The maximum shear crack width was found with Element 568 (Figure 5.3(a)) by considering the migration of the diagonal crack as the RC beam gets loaded. The average stresses on the stirrups are predicted with elements 620, 574, 420, and 440 as presented in Figure 5.3(b). Finally, the average stresses on the tension longitudinal reinforcement is determined with Element 1186 at the location of maximum moment (Figure 5.3(c)).



(a) Crack Width



(b) Average Stirrup Stresses



(c) Average Tension Longitudinal Reinforcement Stresses

Figure 5.3. Vector2 Critical Nodes and Elements

## 5.5 Calibrated Models

Vector2 displacement-controlled models were calibrated to determine the closest load-displacement response to the experimental beams and perform parametric analyses. The concrete compressive strength and the material properties of the reinforcement are updated to match the three RC beam specimens. Default options of the concrete material models were assessed to improve the accuracy.

### 5.5.1 Material Models

The numerical modeling was mostly done following Vector2 default monotonic constitutive stress-strain models for concrete, steel, and reinforcement materials. The structural stiffness is then calculated at each load step. Vector2 provides a variety of models that may result in discrepancies in the results (Wong et al., 2013). The load-displacement response was calibrated by changing the concrete tension stiffening and crack width checks from the default. Perfect bond is assumed between concrete and reinforcing materials.

#### 5.5.1.1 Concrete

Due to the empirical relationships between concrete materials, the non-linear stress-strain response of concrete can be predicted using Cauchy-type models (Wong et al., 2013). Compression pre-peak response is modeled using Hognestad (Parabola). Hognestad provides good results for concretes with compressive strength below 40 MPa. For a realistic load-displacement response, the additional post-peak strength and ductility in concrete due to internal stress redistribution is estimated with Modified Park-Kent, assuming concrete is adequately confined (Wong et al., 2013). Compression softening is considered with the strength-and-strained softened model of Vecchio 1992-A ( $\epsilon_1/\epsilon_2$ -Form). The model accounts for the tensile straining and transverse cracking effect, considering the softening of the uniaxial compressive strength and respective strain.

The Vector2 model does not neglect tension stresses in cracked concrete, defaulting to the Modified Bentz 2003 model for tension stiffening based on reinforcement percentage and bond properties. Modeling the control beam, it was found that Modified Bentz 2003 overestimates its shear strength. Colalillo (2012) mentions that using Izumo et al. (1992) decreases the tension stiffening prediction. Izumo et al. (1992) estimates stress-strain response using smeared cracks and provides the closest peak load prediction for specimen N-0S.

Cracked RC can withstand tensile stresses due to reinforcement bonding. Wong et al., (2013) comment that it is paramount to model tension stiffness for an accurate load-displacement response

since neglecting may result in an abrupt change in stiffness and convergence problems. Additionally, the model utilizes a linear tension softening model to include the concrete post-cracking capacity to carry tensile stresses. Considering tension softening on the FEA model may prevent errors due to the FE mesh coarseness (Wong et al., 2013).

The cracking criterion in concrete depends on stress states with the cracking strength decreasing as transverse compressive stresses increase (Wong et al., 2013). Vector2 defaults the cracking criterion to Mohr-Coulomb (Stress), estimating a failure shear stress and failure plane for a specific set of redistributed normal stresses. Wong et al. (2013) comment that Vector2 can predict the shear stress on a crack with a DSFM/MCFT method by considering stress transfer through aggregate interlock. Vector2 defaults to Basic (DSFM/MCFT).

Wong et al. (2013) state that the average compressive stresses should be reduced after reaching a maximum crack width to align with available compression softening models. For an accurate estimate of the transfer of compressive stresses in a crack, shear-slip distortions are implemented. Vector2 defaults to a crack width check of a quarter of the aggregate size. Colalillo (2012) comments that crack widths larger than 2 mm overestimate peak loads and displacements. Using a crack width check of 2 mm resulted in the closest load-displacement prediction for the control beam (N-0S). The strengthened specimens, on the other hand, provided closer estimates when using larger crack width checks. Wong et al. (2013) mentions that the crack width check is used for the analysis of shear-critical RC beams with minimum or no transverse reinforcement.

Shear slip calculation is accounted for with the stress model (Walraven) proposed by Walraven and Reinhardt (1981). The model considers the shear slip strains along a crack as in the DSFM (Wong et al., 2013). Wong et al. (2013) state that Lai (2001) establishes that Walraven provides accurate results under monotonic and cyclic loading conditions.

Vector2 defaults to a non-linear with plastic offsets hysteretic response proposed by Vecchio (1999) (Wong et al., 2013). It follows the non-linear Ramsberg-Osgood formulations for unloading in compression and tension. Reloading and unloading occur from zero strain to zero stress. The model provides non-coincident monotonic stress-strain curves that use plastic strain offsets, considering energy dissipation and internal damage under cyclic loading conditions (Wong et al., 2013).

#### 5.5.1.2 Reinforcement Steel

Reinforcement steel is modeled as ductile steel reinforcement with monotonic stress-strain behavior. Ductile steel shows linear-elastic behavior till yielding, after which the curve plateaus until the beginning of strain hardening. During strain hardening, the model increases non-linearly (Vector2 default) until ultimate conditions.

The hysteretic response in the reinforcement considers the non-linear Bauschinger Effect in the Seckin model. The Seckin model assumes that load reversal results in premature yielding of the reinforcement due to plastic straining. The model also assumes linear unloading (Wong et al., 2013).

Vector2 defaults to the Tassios model (crack slip) to account for dowel action. Dowel action is assumed to present an elastic-plastic force-displacement response. In the default model, the dowel force is deducted from the concrete stress at the crack.

#### 5.5.1.3 Fe-SMA

The Fe-SMA strips are modeled following default models for reinforcement, with the only difference being the material stress-strain behavior. Vector2 presents a material library with three simplified SMA stress-strain responses. The models are used to assess the material hysteresis under cyclic loading, only considering the effect of the material superelasticity. However, this project evaluates its shape memory effect capacity to prestress a concrete structure. Vector2 can closely obtain the Fe-SMA stress-strain response by modeling it as ductile steel reinforcement, with the prestress being included as a prestrain.

#### 5.5.1.4 Structural Steel

Support/loading application plates and the spreader beam are made of structural steel. Vector2 models structural steel as a linear-elastic material until yielding. The model presents a yielding plateau till the start of strain hardening where it increases linearly until ultimate conditions. Structural steel is modeled similarly to reinforcement steel with the only difference being the implementation of Von Mises Failure criterion to consider biaxial loading (Wong et al., 2013).

### 5.5.2 Material Properties

The material properties of the experimental beams were utilized to model them under similar conditions and determine the closest load-displacement response that aligns with the experimental

results. The selected models will then be used to perform parametric analyses and assess the effect of prestress and NSB Fe-SMA ratio. The concrete properties are summarized in Table 5.1.

**Table 5.1. Concrete Properties**

| <b>Specimen</b> | <b><math>f'_c</math><br/>(MPa)</b> | <b><math>a_{max}</math><br/>(mm)</b> |
|-----------------|------------------------------------|--------------------------------------|
| N-0S            | 37.53                              | 20                                   |
| A-200S-180C     | 36.28                              | 20                                   |
| A-200S-180C     | 34.21                              | 20                                   |

While longitudinal bars are modeled as truss elements, stirrups are modeled as smeared ductile steel reinforcement. Similarly, the Fe-SMA strips (10 mm x 1.5 mm) are implemented to the strengthened RC beams as smeared reinforcement like in González Góez (2023). The Fe-SMA strips have an equivalent diameter equal to 4.37 mm. Table 5.2 presents the reinforcement properties.

**Table 5.2. Reinforcement Properties**

| <b>Material</b> | <b><math>\rho</math><br/>(%)</b> | <b><math>d_b</math><br/>(mm)</b> | <b><math>F_y</math><br/>(MPa)</b> | <b><math>F_u</math><br/>(MPa)</b> | <b><math>E_s</math><br/>(MPa)</b> | <b><math>\epsilon_{sh}</math><br/>(me)</b> | <b><math>\epsilon_u</math><br/>(me)</b> | <b>Prestrain<br/>(me)</b> |
|-----------------|----------------------------------|----------------------------------|-----------------------------------|-----------------------------------|-----------------------------------|--------------------------------------------|-----------------------------------------|---------------------------|
| US #3           | 0.12                             | 9.525                            | 499.61                            | 728.38                            | 198110                            | 13                                         | 110                                     | -                         |
| 10M             | 0.67                             | 11.3                             | 471.33                            | 580.02                            | 190920                            | 24                                         | 138                                     | -                         |
| 25M             | -                                | 25.2                             | 495.83                            | 608.27                            | 197620                            | 12                                         | 96                                      | -                         |
| Fe-SMA          | 0.05 or 0.1                      | 4.37                             | 562.96                            | 1030.75                           | 78324                             | 7.19                                       | 458                                     | 0.42                      |

Shahverdi et al. (2018) reports that the elastic modulus ( $E_s$ ) of 2% prestrained Fe-SMA of the form Fe-17Mn-5Si-10Cr-4Ni-1(V, C) can reduce 45% after activation. Yang et al. (2021) mentions that larger austenite to martensite transformation may be the cause of the tensile stiffness loss. Activated Fe-SMA strips would ideally provide a prestress of up to 250 MPa if heated to at least 180°C. However, Hong et al. (2018) estimates that the prestress on the concrete is a function of the ratio of the area of the Fe-SMA strip over the area of the filling (Equation 3.26). The concrete would experience around 33 MPa per each NSB active Fe-SMA strip. The prestress is implemented by

calculating a prestrain equal to the ratio between the prestress and elastic modulus. Conservatively, the activated Fe-SMA strips present an elastic modulus of 78324 MPa and a prestrain of 0.42 me.

### 5.5.3 Validation

The Vector2 models are validated against the experimental tests by comparing their load-displacement response. The response of the three beam specimens is assessed based on tension stiffening and maximum crack width check. Comparisons were done between the Modified Bentz 2005 and Izumo et al. (1992) tension stiffening models, and maximum crack width checks of 2mm and 40% the maximum aggregate size (default). The strengthened beam models were also assessed with a crack width check of 10 mm and its omission.

Figure 5.4 shows the experimental and Vector2 load-displacement response of specimen N-0S. The default model overestimates the shear strength of the beam by around 35%. It also overestimates the peak displacement of the cantilever and the inside span by around 41% and 36%, respectively. Reducing the average compressive stresses when reaching a crack width limit of 2 mm results in a considerable decrease in the load-displacement response. Additional softening by using Izumo et al. (1992) further decreases strength and displacement, achieving a strength percentage difference of 9.11% and peak displacement percentage differences of 16.35% for the cantilever and 19.65% for the inside span.

The experimental and Vector2 models of the strengthened beams A-200S-180C and A-100S-180C are presented in Figure 5.5 and Figure 5.6, respectively. All models overestimate the strength of the RC beams. Note that Vector2 cannot consider failure due to the interfacial debonding of the NSB Fe-SMA strips (A-200S-180C) nor side concrete cover separation (A-100S-180C).

Models for specimen A-200S-180C underestimate its displacements (Figure 5.5). When the Fe-SMA ratio is 0.05%, increasing the crack width check and reducing the tension stiffness provides closer estimates to the actual load-displacement response and a stiffer post-peak behavior. The closest load-displacement prediction is obtained by omitting the crack width stability check. Vector2 models for specimen A-100S-180C considerably overestimate its shear strength (Figure 5.6). This could be attributed to the inability of Vector2 to model its failure due to side concrete cover separation. None of these models predict a similar post-peak stiffness to the experimental beams. At a Fe-SMA ratio of 0.1%, a crack width check of 10 mm and check omission overestimate peak displacement. The closest load-displacement response is obtained using default material models.

It has been decided that default material models will be used to assess the behavior of the Fe-SMA shear strengthened RC beams. The accuracy of these models is evaluated using percentage differences. Specimen A-200S-180C and A-100S-180C present a peak load percentage difference of 10.56% and 19.68%, respectively, compared to the default Vector2 models. The percentage difference with respect to the displacement is considerably higher with the cantilever being about 60.67% (A-200S-180C) and 47.97% (A-100S-180C). The displacement percentage difference reduces at the inside span with A-200S-180C displaying 44.57% and A-100S-180C displaying 4.97%.

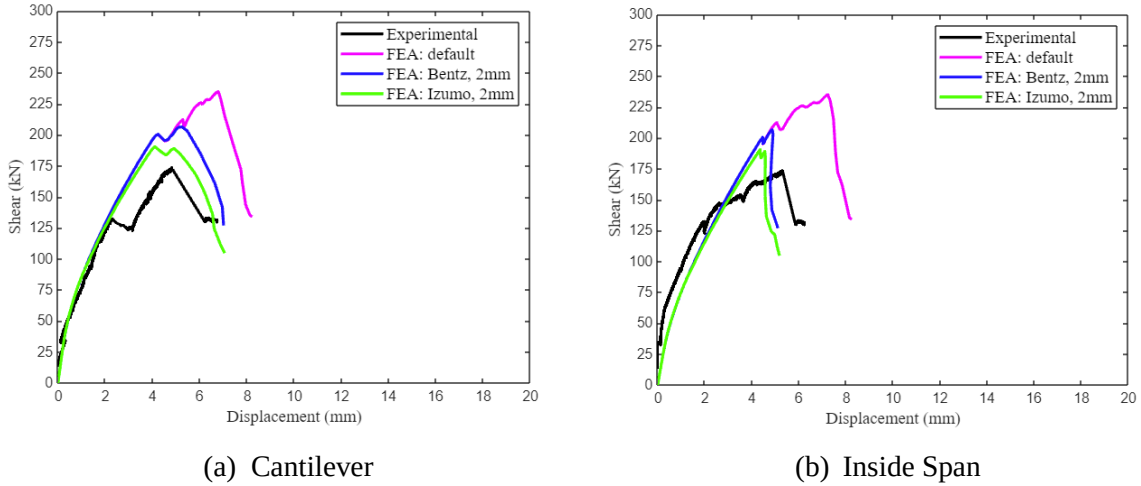


**Figure 5.4. Control Beam (N-0S)**

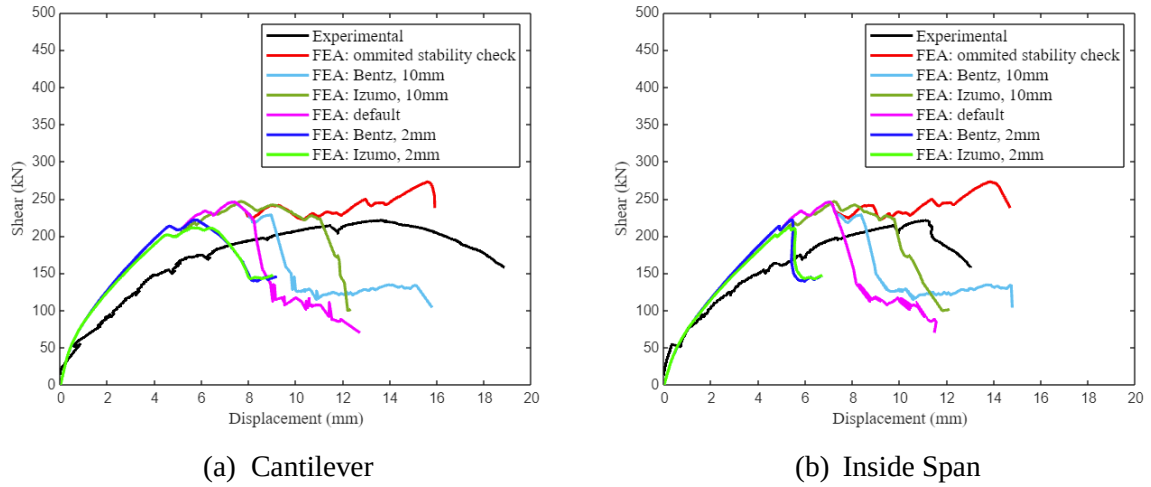


**Figure 5.5. Specimen A-200S-180C**

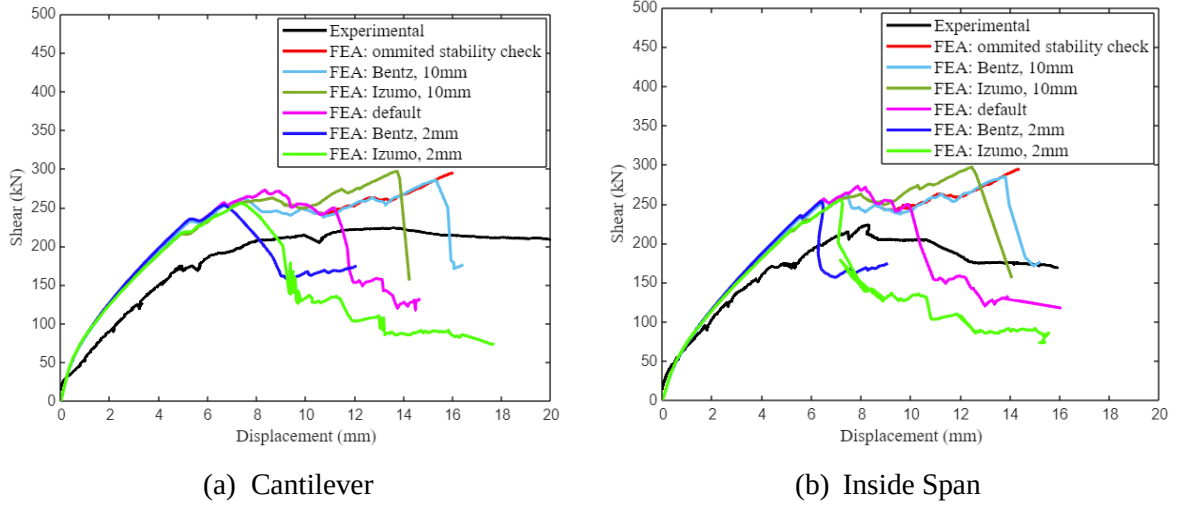


**Figure 5.6. Specimen A-100S-180C**

## 5.6 Parametric Studies

Two parametric studies were performed to evaluate how the prestress level and Fe-SMA ratio affect the behavior of strengthened shear-critical RC beams. The models assess the load-displacement response, crack width, and reinforcement stresses. Compared to a control beam (N-0S), shear strengthened RC beams present a considerable shear capacity increase, as well as diagonal crack width and reinforcement stress reduction.

The control beam is modeled using a crack width check of 2 mm and the concrete tension stiffening model of Izumo et al. (1992). On the other hand, the strengthened specimens are modeled using default material models. The Vector2 models use concrete with a compressive strength of 30 MPa and a maximum aggregate size of 20 mm. The reinforcement properties are summarized in Table 5.2. The specimens are identified alphanumerically similar to the experimental specimens. The first letter being N (non-activated Fe-SMA) or A (activated Fe-SMA), followed by the Fe-SMA spacing in mm, and the temperature of activation in °C.

### 5.6.1 Self-Prestress

Seven Vector2 models were used to assess the effect of the SME activation on the strengthened specimens (Figure 5.7). Two non-prestressed beams specimens (N-200S and N-100S) are modeled and compared to their prestressed equivalent (A-200S-180C and A-100S-180C). It is conservatively assumed that the concrete does not experience the total recovery stress from the activation of the Fe-SMA strips (250 MPa), but a ratio between the area of the strip and the embedding epoxy (~33 MPa).

The load-displacement responses of the Vector2 models are shown in Figure 5.7. The strengthened beams tend to plateau around peak. Models with higher prestress tend to start plateauing at slightly higher shear loads. The load-displacement response of non-prestressed and prestressed RC beams are similar, but there are slight differences in their post-peak-behavior. Prestress does not affect shear strength, but it appears to slightly affect displacement, with the non-activated specimens showing a larger displacements at ultimate.

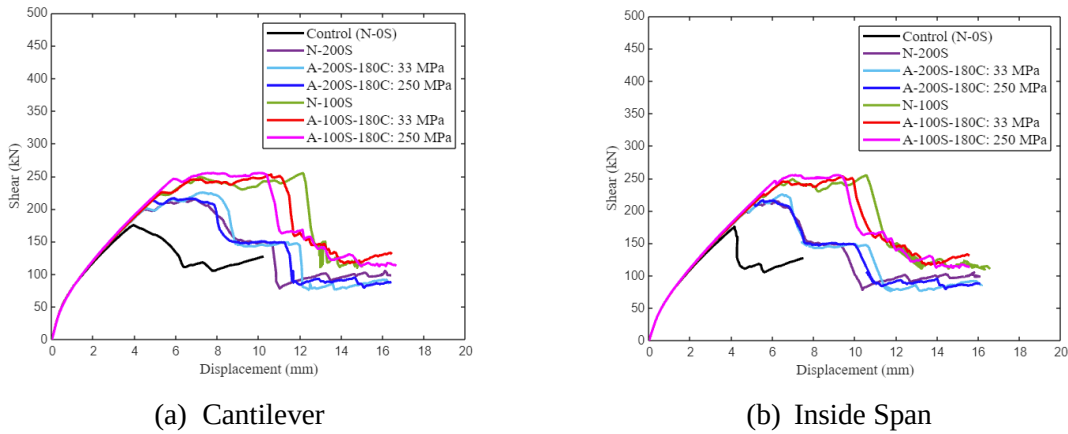


**Figure 5.7. Load-Displacement Response**

Figure 5.8 presents the average concrete cracks width till peak. Prestresses appear to have a noticeable effect in their reduction. Compared to the control beam, strengthened beams with 0.1% Fe-SMA show a decrease in crack width of 68.39%, 57.17%, and 55.12% when prestressed to 250 MPa, 33 MPa, and 0 MPa, respectively. Similarly, 0.05% Fe-SMA provide a crack width reduction of 52.76%, 46.46%, and 44.88% when using a prestress of 250 MPa, 33 MPa, and 0 MPa. Pre-peak response also show that by increasing the prestress, average crack width decreases. Prestressed beams with 33 MPa can support larger crack widths than non-prestressed or 250 MPa prestressed beams.

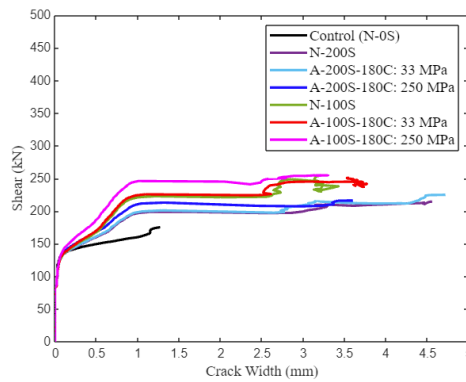


**Figure 5.8. Crack Width**

The effect of the prestress on the average reinforcement stresses until peak is presented in Figure 5.9. Figure 5.9(a) shows the average stirrup stresses at  $d_v$  away from the face of the support. As the Fe-SMA prestress increases, the average stress in the stirrups reduces. Compared to the control beam, strengthened beams with 0.05% Fe-SMA show an average stirrup decrease of 36.88%, 38.52%, and 47.85% when prestressed with 0 MPa, 33 MPa, and 250 MPa, respectively. Beams with 0.1% Fe-SMA provide a stirrup stress reduction of 42.17% on a non-prestressed beam, 44.89% when prestress is 33 MPa, and 59.88% when it is 250 MPa. The longitudinal tension bar stresses were found using a critical element at the highest moment location. Figure 5.9(b) show that the tension reinforcement does not yield and that active shear strengthening does not affect the stresses in the longitudinal reinforcement.

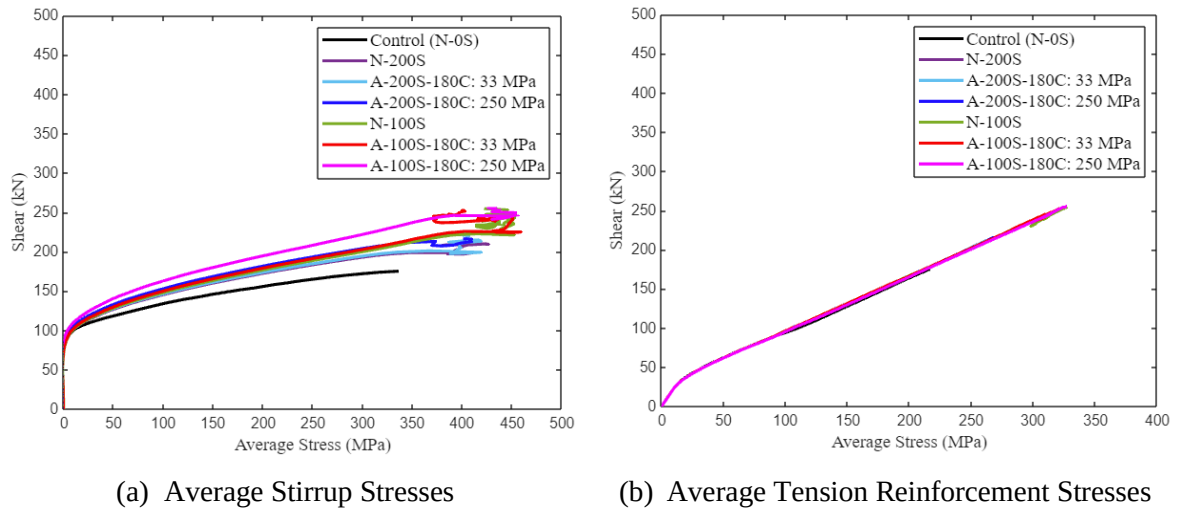


**Figure 5.9. Average Reinforcement Stresses**

### 5.6.2 Fe-SMA Ratio

The load-displacement responses of four different Fe-SMA ratios for active shear strengthening are presented in Figure 5.10. A higher Fe-SMA ratio results in larger strengths and their respective deformations. Increasing the reinforcement ratio by a quarter with 0.025% and half with 0.05% Fe-SMA increases the shear strength by 21.55% and 28.38%, respectively. The models show that using  $\geq 0.1\%$  Fe-SMA provides higher shear strength than using 0.05%. The experimental results showed that using 0.1% Fe-SMA results in failure due to side concrete cover separation at a strength of around 0.05% Fe-SMA. Vector2 does not model debonding failures.

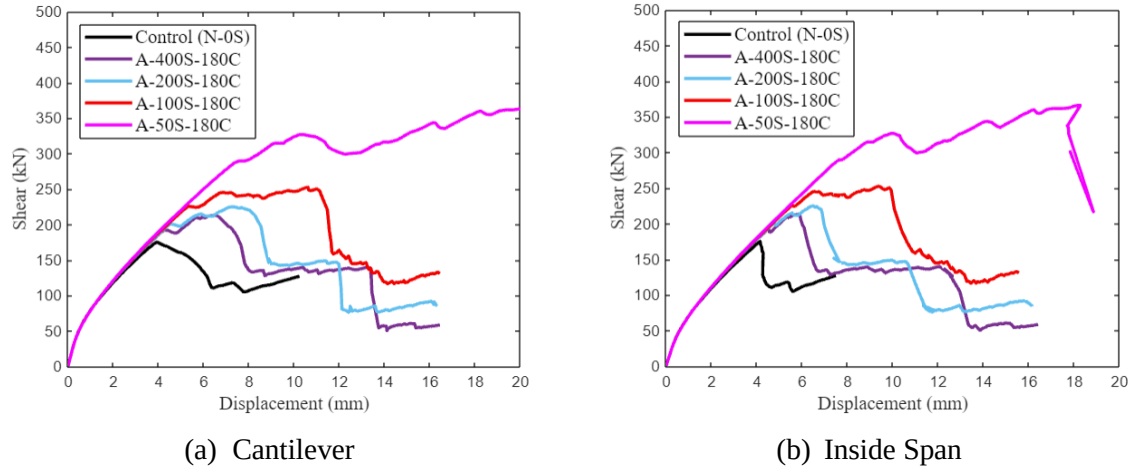


**Figure 5.10. Load-Displacement Response**

The effect of the Fe-SMA ratios in the concrete average crack width can be seen in Figure 5.11. Shear strengthening a shear-critical RC beam and increasing its Fe-SMA ratio result in the considerable reduction of crack widths. Fe-SMA ratios of 0.025% and 0.05% provide a similar response, reducing the crack width by about 37% and 46.46%, respectively. Moreover, strengthening a RC beam with 0.1% Fe-SMA may decrease crack width by 57.17%. Doubling this Fe-SMA ratio can lead to a crack width reduction of 70.08%. Beams with decreasing Fe-SMA ratios present rapid widening of the crack. An increasing Fe-SMA ratio tends to result in cracks widening at larger applied loads.

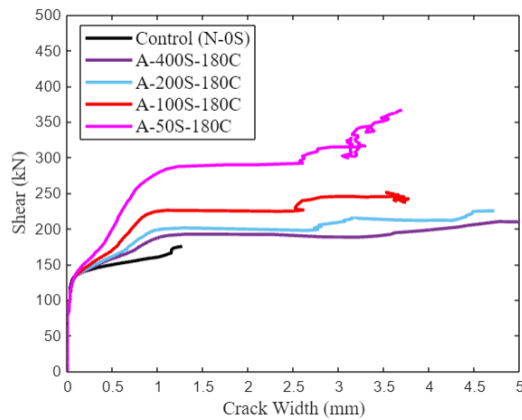
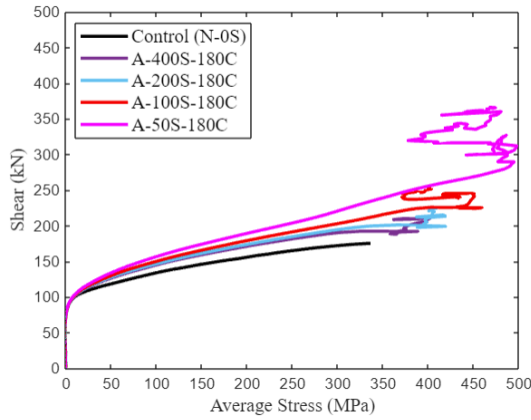


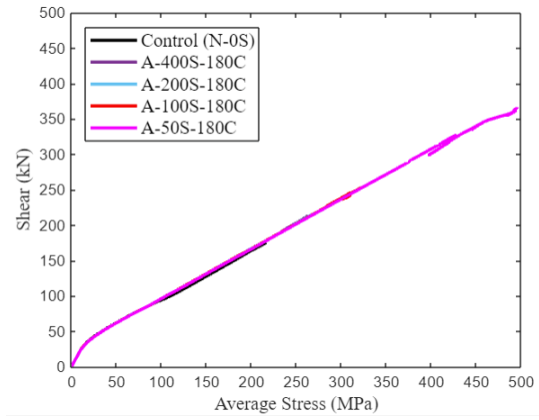
**Figure 5.11. Crack Width**

Figure 5.12 shows the effect of varying Fe-SMA ratios on average reinforcement stresses. Higher Fe-SMA ratios allows the stirrups to withstand higher loads at the same average stress (Figure

5.12(a)). Fe-SMA ratios between 0.025% and 0.05% provide a similar response. Compared to the control beam, active strengthening a RC beam can reduce the average stirrup stresses by 34.75% when using 0.025% Fe-SMA, 38.52% with 0.05% Fe-SMA, 44.88% with 0.1% Fe-SMA, and 54.14% with 0.2% Fe-SMA. As the Fe-SMA ratio increases, the test span stiffens and the beam can experience considerable bending at higher loads. Hence, stresses in the tension longitudinal reinforcement increase with an increase in Fe-SMA (Figure 5.12(b)).



(a) Average Stirrup Stresses



(b) Average Tension Reinforcement Stresses

**Figure 5.12. Average Reinforcement Stresses**

## Chapter 6

### Conclusion and Recommendations

#### 6.1 General

This project assessed the active shear strengthening effect of NSB Fe-SMA epoxy composites in shear-critical RC beams. The experimental program provided an understanding of the prestressing capacity of Fe-SMA to prestress a concrete element with resistive heating. FEA models were also performed to evaluate prestress level and Fe-SMA ratio. This research project has concluded with the following findings:

- **Enhancement of load-displacement response:** the active shear strengthening of a shear-critical RC beam with NSB Fe-SMA strips showed an increase in strength of about 27% when using a Fe-SMA ratio between 0.05% and 0.1%. The strengthened beams showed an increase in stiffness in the test spans, withstanding larger loads and deflections. Deflections at the cantilever and test span increased by at least 81% and 55%, respectively.
- **Crack width reduction and propagation:** the implementation of NSB Fe-SMA epoxy composites mitigates the development of critical diagonal cracks. The path of the crack is interrupted by the NSB Fe-SMA strips, resulting in scattered narrower cracks.
- **Delayed yielding of the transverse reinforcement:** stirrups in the test span of the strengthened beams yielded at larger loads compared to the control specimen. Stirrups are also able to withstand larger strains.
- **Self-prestress:** the effect of the Fe-SMA prestress on a concrete beam can be assessed with Vector2 FEA models. Prestressing the Fe-SMA strips reduce crack widths and average stirrup stresses. The higher the prestress, the higher the benefit.
- **Fe-SMA ratio:** increasing the Fe-SMA ratio leads to a slight increase in shear strength, delay on the stresses in the transverse and tension longitudinal reinforcement, and reduction of crack widths. However, increasing the Fe-SMA ratio may also be detrimental due to debonding failure modes. The experimental tests showed that increasing the NSB Fe-SMA ratio may weaken the stirrup plane, forming horizontal longitudinal cracks that can separate the side concrete cover.

## 6.2 Future Work

This project provides an insight of the self-prestress effect of innovative NSB Fe-SMA epoxy composites for active shear strengthening of RC structures. Further experimental research is needed to evaluate the applicability of the shear strengthening method on existing structures and measure the advantages of the prestressing effect. Hence, the following recommendations are provided:

- **Retrofit of a pre-cracked specimen:** the implementation of NSB Fe-SMA epoxy composites for shear strengthening of pre-cracked specimens can lead to an assessment of the prestress capacity to reduce or close cracks. Additionally, if temperature-resistant strain gauges are placed onto the Fe-SMA strips, the contribution of the strengthening material to the shear strength of the specimen can be estimated.
- **Prestress benefit:** non-activated NSB Fe-SMA epoxy composites can be applied in shear-critical RC beams to estimate whether prestressing the RC specimen improves performance.
- **Wrapping configuration:** u-wrapping the Fe-SMA strips may not be applicable in a specimen with limited accessibility. Hence, side bonding could be considered for NSB or EB Fe-SMA strips.
- **Alternative bonding materials:** the high-temperature high-strength epoxy utilized for this project may be too expensive to be used in construction. Therefore, cost-efficient bonding materials could be explored.

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| Institution name                                              | University of Waterloo                                                                                                      |
| Expected presentation date                                    | Aug 2024                                                                                                                    |
| Portions                                                      | Fig. 1. Shear cracks in a highway bridge girder.                                                                            |
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| Publication Title | Advances in structural engineering                                         | Publication Type | Journal                                                                                                       |
| Article Title     | Shear strengthening of RC beams with NSM FRP. I: Review of strength models | Start Page       | 564                                                                                                           |
| Date              | 01/01/1997                                                                 | End Page         | 586                                                                                                           |
| Language          | English                                                                    | Issue            | 3                                                                                                             |
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| Editor of Portion(s)                                      | Jedrzejko, M.J.; Zhang, S.S.; Ke, Y.; Fernando, D.; Nie, X.F.   | Author of Portion(s)                               | Jedrzejko, M.J.; Zhang, S.S.; Ke, Y.; Fernando, D.; Nie, X.F.              |
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| <b>Article Title</b>     | Shear strengthening of reinforced concrete beams using steel plates bonded on beam web: experiments and analysis | <b>Start Page</b>       | 237     |
|                          |                                                                                                                  | <b>End Page</b>         | 244     |
|                          |                                                                                                                  | <b>Issue</b>            | 5       |
| <b>Date</b>              | 01/01/1987                                                                                                       | <b>Volume</b>           | 14      |
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| <b>Editor of Portion(s)</b>                                      | Adhikary, Bimal Babu; Mutsuyoshi, Hiroshi; Sano, Masashi | <b>Author of Portion(s)</b>                               | Adhikary, Bimal Babu; Mutsuyoshi, Hiroshi; Sano, Masashi                                                         |
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| Author / Editor   | Institution of Structural Engineers                                                                                   | Start Page       | 767                                    |
| Date              | 01/01/2015                                                                                                            | End Page         | 778                                    |
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| Editor of Portion(s)                                      | Yang, Yong; Chen, Xin; Xue, Yicong; Yu, Yunlong | Author of Portion(s)                               | Yang, Yong; Chen, Xin; Xue, Yicong; Yu, Yunlong                                                                       |
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| Article Title     | Strengthening of shear critical RC beams with various FRP systems | Publication Type | Journal                                |
| Date              | 01/01/1987                                                        | Start Page       | 634                                    |
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| Editor of Portion(s)                                      | Baggio, Daniel; Soudki, Khaled; Noël, Martin | Author of Portion(s)                               | Baggio, Daniel; Soudki, Khaled; Noël, Martin                      |
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| Publication Title | European journal of environmental and civil engineering                                                                 | Publication Type | e-Journal                                                                                  |
| Article Title     | Design model for shear capacity of RC beams strengthened with two-side CFRP wraps based on effective FRP strain concept | Start Page       | 161                                                                                        |
| Date              | 01/01/2008                                                                                                              | End Page         | 179                                                                                        |
| Language          | English, French, Multiple languages                                                                                     | Issue            | 2                                                                                          |
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| Publication Title | Shape memory alloys : modeling and engineering applications | Country          | United States of America                                                                                          |
| Author / Editor   | Lagoudas, Dimitris C.                                       | Rightsholder     | Springer Nature BV                                                                                                |
| Date              | 01/01/2008                                                  | Publication Type | Book                                                                                                              |
| Language          | English                                                     | URL              | <a href="http://rave.ohiolink.edu/ebooks/ebc/9780387476858">http://rave.ohiolink.edu/ebooks/ebc/9780387476858</a> |

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|                   |                                                                                                                                  | Start Page       | 20                              |
|                   |                                                                                                                                  | End Page         | 26                              |
|                   |                                                                                                                                  | Issue            | 1770                            |
| Author / Editor   | National Research Council (U.S.);Transportation Research Board                                                                   | Volume           | 1770                            |
| Date              | 01/01/1974                                                                                                                       |                  |                                 |
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| Title, Description or Numeric Reference of the Portion(s) | FIGURE 17 Final setup.                                              | Title of the Article / Chapter the Portion Is From | Repair and Strengthening of Concrete Structures Through Application of Corrective Posttensioning Forces with Shape Memory Alloys |
| Editor of Portion(s)                                      | Soroushian, Parviz; Ostowari, Ken; Nossoni, Ali; Chowdhury, Habibur | Author of Portion(s)                               | Soroushian, Parviz; Ostowari, Ken; Nossoni, Ali; Chowdhury, Habibur                                                              |
| Volume / Edition                                          | 1770                                                                | Issue, if Republishing an Article From a Serial    | 1770                                                                                                                             |
| Page or Page Range of Portion                             | 20-26                                                               | Publication Date of Portion                        | 2001-01-01                                                                                                                       |

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|-------------------|--------------------------------------------------------------------------------------------------------------|------------------|----------------------------------------|
| Publication Title | Engineering structures                                                                                       | Rightsholder     | Elsevier Science & Technology journals |
| Article Title     | Shear strengthening of slender reinforced concrete T-shaped beams using iron-based shape memory alloy strips | Publication Type | Journal                                |
| Date              | 01/01/1978                                                                                                   | Start Page       | 111018                                 |
| Language          | English                                                                                                      | Volume           | 221                                    |
| Country           | United Kingdom of Great Britain and Northern Ireland                                                         |                  |                                        |

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## NEW WORK DETAILS

|                 |                                                                                                                             |                            |                        |
|-----------------|-----------------------------------------------------------------------------------------------------------------------------|----------------------------|------------------------|
| Title           | Self-prestressing Iron-based Shape Memory Alloy (Fe-SMA) Epoxy Composite for Active Reinforced Concrete Shear Strengthening | Institution Name           | University of Waterloo |
| Instructor Name | Johanna Alejandra Pinargote Torres                                                                                          | Expected Presentation Date | 2024-08-13             |

## ADDITIONAL DETAILS

|                        |     |                                                               |                                    |
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| Title, Description or Numeric Reference of the Portion(s) | Fig. 3. C) Applying a pre-specified torque by means of a torque wrench.             | Title of the Article / Chapter the Portion Is From | Shear strengthening of slender reinforced concrete T-shaped beams using iron-based shape memory alloy strips |
| Editor of Portion(s)                                      | Cladera, Antoni; Montoya-Coronado, Luis A.; Ruiz-Pinilla, Joaquín G.; Ribas, Carlos | Author of Portion(s)                               | Cladera, Antoni; Montoya-Coronado, Luis A.; Ruiz-Pinilla, Joaquín G.; Ribas, Carlos                          |
| Volume / Edition                                          | 221                                                                                 | Issue, if Republishing an Article From a Serial    | N/A                                                                                                          |
| Page or Page Range of Portion                             | 111018                                                                              | Publication Date of Portion                        | 2020-10-15                                                                                                   |



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**Iron based shape memory alloys as shear reinforcement for bridge girders**

**Author:** Christoph Czaderski,Moslem Shahverdi,Julien Michels

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## **Appendix A**

### **Experimental Testing: Crack Propagation**

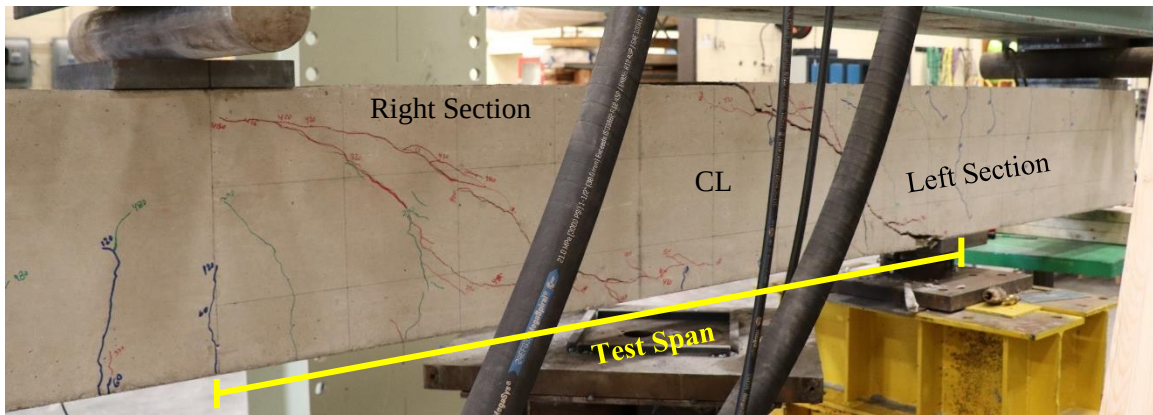


**Figure A.1. Specimen N-0S: Test Span**

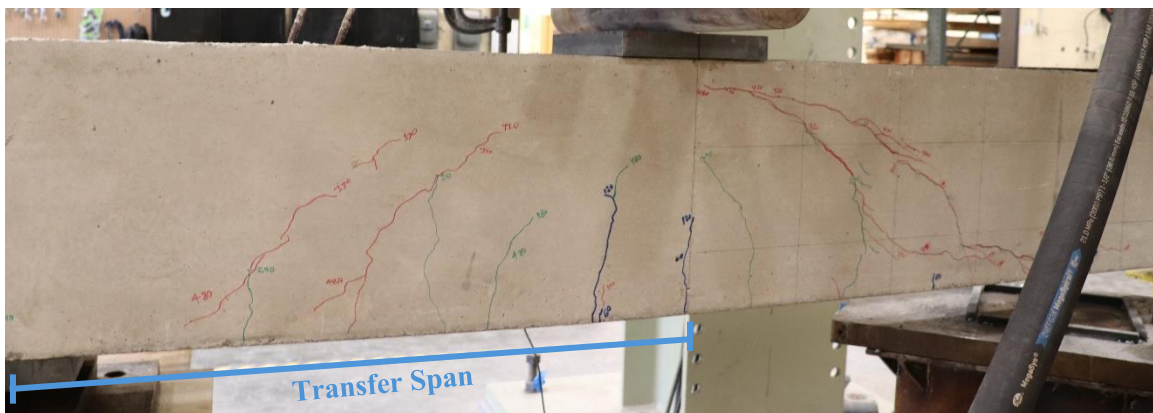


**Figure A.2. Specimen N-0S: Right Section**



**Figure A.3. Specimen N-0S: Shear Crack in Right Section**



**Figure A.4. Specimen N-0S: Left Section**

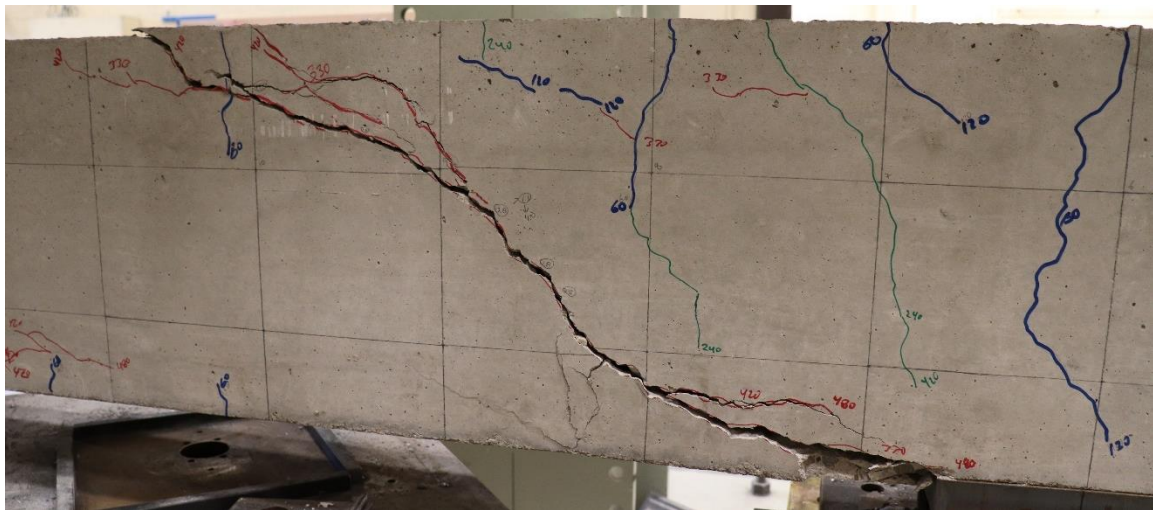


**Figure A.5. Specimen N-0S: Critical Shear Crack in Left Section**



**Figure A.6. Specimen A-200S-180C: Test Span**

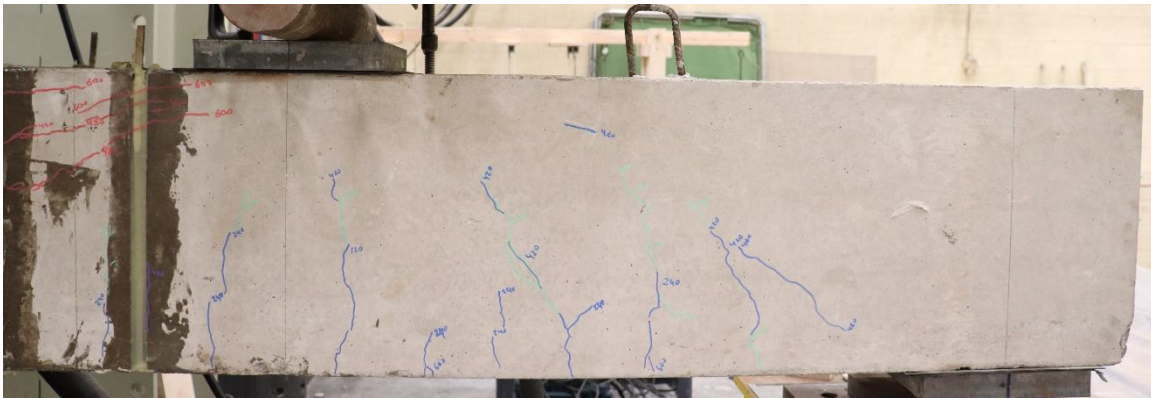


**Figure A.7. Specimen A-200S-180C: Right Section (Transfer Span)**



**Figure A.8. Specimen A-200S-180C: Shear Crack in Right Section**

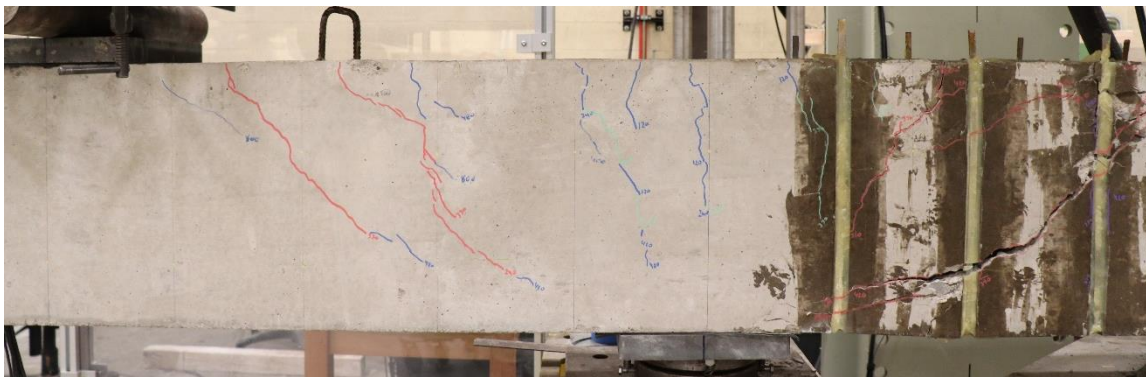


**Figure A.9. Specimen A-200S-180C: Left Section (Transfer Span)**



**Figure A.10. Specimen A-200S-180C: Critical Shear Crack in Left Section**



**Figure A.11. Specimen A-100S-180C: Test Span**

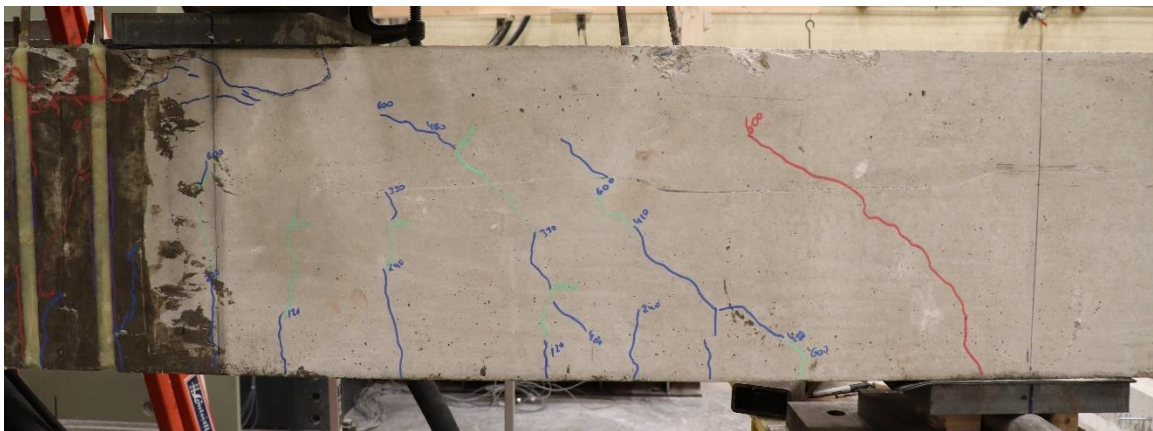


**Figure A.12. Specimen A-100S-180C: Right Section (Transfer Span)**



**Figure A.13. Specimen A-100S-180C: Shear Crack in Right Section**



**Figure A.14. Specimen A-100S-180C: Left Section (Transfer Span)**

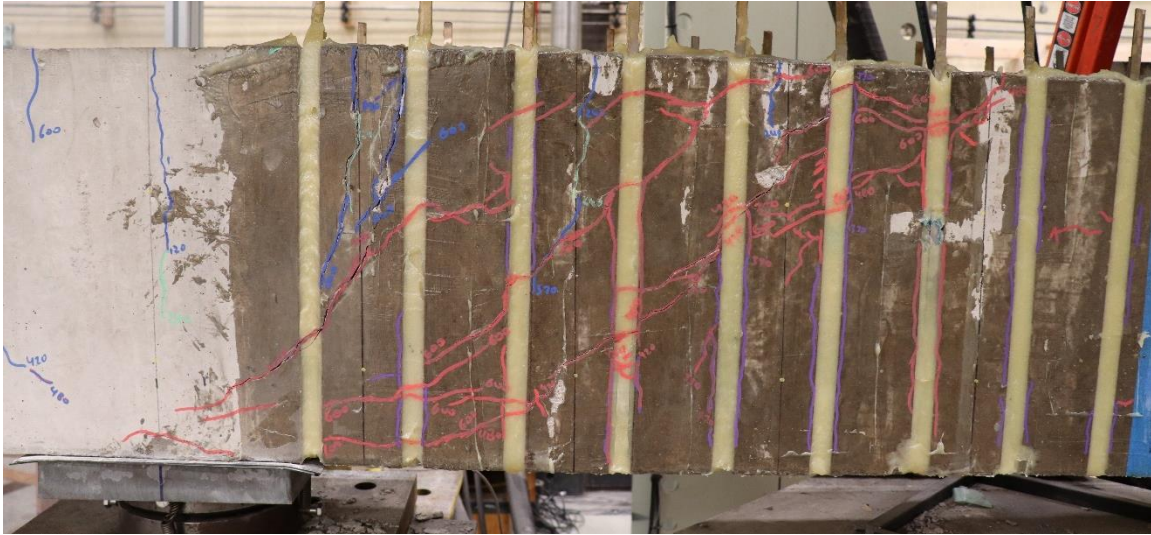


**Figure A.15. Specimen A-100S-180C: Shear Crack in Left Section**