EXPERIMENTAL AND NUMERICAL EVALUATION OF RAPID POST-TENSIONING OF DAMAGED REINFORCED CONCRETE GIRDERS WITH UNBONDED NEAR SURFACE MOUNTED SHAPE-MEMORY ALLOY WIRES

by

Arkabrata Sinha

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Arkabrata Sinha

Approved: Jovan Tatar, Ph.D.
Professor in charge of thesis on behalf of the Advisory Committee

Approved: Sue McNeil, Ph.D.
Chair of the Department of Civil and Environmental Engineering

Approved: Levi T. Thompson, Ph.D.
Dean of the College of Engineering

Approved: Douglas J. Doren, Ph.D.
Interim Vice Provost for Graduate & Professional Education and Dean of the Graduate College
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ABSTRACT

American Society of Civil Engineers (ASCE) classified 9.1% of the national bridges as structurally deficient in 2017. Several of the deficient concrete bridges have girder serviceability issues such as excessive cracking and deflection. Traditional external post-tensioning techniques involve high management of traffic costs and use of heavy construction equipment. To that end, a novel rapid post-tensioning technique using unbonded near-surface mounted shape-memory alloy (SMA) wires was herein evaluated. The suggested method relies on the intrinsic ability of the SMA to recover seemingly permanent deformation upon activation via heating. Mechanical characterization tests demonstrated that the SMA wires can generate 67 ksi of prestress. Within the range of SMA reinforcement ratios considered in this study (up to 0.17%), the structural-scale tests showed that post-tensioning of cracked reinforced concrete girders can reduce the crack width and residual midspan deflection by up to 370 μm (74%) and 0.06 in. (49%), respectively, and increase the ultimate moment capacity up to 83 kip-ft (45%). Finite element model of post-tensioned girders was developed in a damage mechanics framework and was validated against experimental data – the predicted yield and ultimate moment of the strengthened girders were 14% less than those observed in the experiments. The difference was due to the cumulative effect of errors embedded in the model assumptions. The model was used to evaluate the service and ultimate behavior of strengthened girders over a range of variables such as the extent of cracking in concrete, SMA reinforcement ratio, interaction between SMA and concrete (unbonded vs. bonded), and effective prestress in SMA.
The partial loss of prestress due to elastic shortening of concrete is insignificant (<1 ksi) due to a relatively low Young’s modulus of the SMA. A modest increase (up to 3 times) in the ratio of SMA reinforcement ratio resulted in an increase in the yield moment by up to 99%. Reducing the effective prestress to 80% of the applied prestress decreases the yield moment by up to 7.8% in the strengthened girder. Overall, the study demonstrated the promise of the proposed post-tensioning technique in being used as a repair and strengthening strategy for reinforced concrete structures.
Chapter 1

INTRODUCTION

Structural components, carrying flexural stresses, can suffer from serviceability issues caused by excessive cracking and deflections. Repair methods based on post-tensioning were developed to address the serviceability problems by installing external tendons on a structure to provide the uplifting force (Figure 1). Based on a similar concept, methods to post-tension concrete bridge girders implementing near surface mounted (NSM) prestressed carbon fiber reinforced polymer (CFRP) strips and bars were developed (Figure 2). The traditional post-tensioning techniques involve the use of heavy hydraulic jacking equipment and cause loss of prestressing forces due to improper application of anchorage and ambient temperature. There is a need, thus, rapid and sustainable method to address the serviceability issues in concrete bridges which would alleviate the longer construction times and losses in prestress.

According to the ASCE Report Card for 2017, approximately 9.1% of the bridges in the United States are classified as structurally deficient. Many of the deficient bridges fail to meet the serviceability limit state requirements imposed on maximum permissible deflections and crack widths. Hence, to overcome the shortcomings of the traditional post-tensioning techniques, a heat-actuated rapid repair technique involving post-tensioning using NSM SMA wires was proposed which would reduce management of traffic (MOT) costs and can be applied, if deemed necessary, during scheduled inspection and maintenance periods.
Previous attempts, to post-tension reinforced concrete girders using SMA, have not been completely successful. Previous research shows that significant losses in prestressing force have occurred due to improper anchorage, poor bond with cementitious grouts and relaxation of the SMA wires. The current research is expected to address these gaps in knowledge by both experimental and numerical methods. The novelty of the post-tensioning technique discussed in this paper is that the procedure can be carried out using relatively simpler equipment such as a heat source (Ohmic heating or open-flame torch) over intricate conventional hydraulic jacking equipment. This process can reduce the amount of construction time significantly eliminating the need for prolonged traffic obstruction and, in turn, reducing the MOT costs. The use of simple equipment also makes this an attractive option for situations that involve space constraints and difficulty of access. Although the research focuses on the application of SMA to post-tension concrete bridge girders, this technique can also be used to post-tension other structural components such as slabs and beams in buildings. The SMA can also be used in the form of strips or multi-strand cables depending on the type of structure that is being post-tensioned.

Figure 1. External post-tensioning (Structural Group, Inc. 2016)
Figure 2. (a) Near-surface mounted (NSM) FRP reinforcement; (b) Prestressed NSM FRP (El-Hacha and Soudki, 2013)

1.1 Objectives and Scope

This work seeks to verify the feasibility of using heat-induced post-tensioning via NSM SMA in existing cracked reinforced concrete girders (Figure 3) without the conventionally experienced problems with anchorage, high construction costs, long lane closure times, etc.
Figure 3. Concept of reinforced concrete girder post-tensioning with NSM SMA

The sequence and scope of the research program included experimental and numerical methods having the following specific objectives as shown in Figure 4. Materials characterization tests on SMA wires including Differential Scanning Calorimetry (DSC), the potential of generating post-tensioning forces (restrained recovery test) and tensile tests were conducted to evaluate mechanical and thermal properties of the SMA wires. Bond characterization tests were conducted on small plain concrete beam specimens to evaluate a method to transfer the post-tensioning forces to concrete. The proof-of-concept structural-scale post-tensioning tests were conducted on reinforced concrete girders to evaluate the reduction of crack widths and midspan deflection. Flexural tests were conducted on the post-tensioned girders to
study the increase in cracked stiffness and ultimate moment capacity of the strengthened girders. A 3-D numerical model using the finite element method (FEM) was developed using the results of the material characterization tests on SMA wires and was validated using the results of the proof-of-concept experiments. Subsequently, a parametric study was conducted to evaluate the effect of girder damage, ratio of SMA to internal steel reinforcement ratio, interaction with concrete (bonded vs. unbonded) and effective prestress on the strengthened girder behavior.

Figure 4. Sequence of research

1.2 Thesis Outline

The thesis is divided into six chapters. The first chapter covers the objectives and scope and sequence of the research. The second chapter discusses a brief definition of the SMA and their characteristic properties. This chapter describes the
different types of SMA, their application in previous studies and how the current research addresses the gaps in knowledge from the previous studies. The third chapter covers the experiments that have been performed to characterize the material properties of SMA and the proof-of-concept tests verifying the feasibility of the strengthening technique. The chapter has been organized by describing the material and specimen preparation before discussing the procedure and results. The fourth chapter discusses analytical calculations to estimate the partial loss of prestress using experimentally recorded camber and the stress in SMA wires when the concrete crushed in compression. The fifth chapter discusses the development and validation of a finite element model. At first, the model description has been presented followed by the material model discussion, mesh details and the results. The Control model without SMA wires and the post-tensioned model have validated using the experimental moment vs. deflection behavior. A parametric study on the post-tensioned girder by varying the tensile damage, ratio of SMA to internal steel reinforcement, bonded vs. unbonded and effective prestress. The final chapter covers the overall summary of the research and the conclusions which are drawn from it. Recommendations for future research work has also been discussed at the end of the chapter.
Chapter 2

BACKGROUND INFORMATION

This section starts with a definition of SMA and its unique properties such as shape-recovery and superelasticity. An overview of the different types of SMA and their application in civil engineering has been provided including both the shape-recovery and superelasticity property. Experimental studies on the bond characteristics between SMA and concrete have been provided in this chapter.

The final section of this chapter discusses previous finite element models that have been developed to simulate the post-tensioning of reinforced concrete girders. The study covers prestressing using different materials like steel tendons, FRP strips and SMA. The studies describe modeling techniques, choice of material properties and mesh element types for different simulations. Several parametric studies have also been included in this section.

2.1 Shape-memory Alloys

Shape memory materials (SMM) are a class of materials that can recover their original shape from significant and seemingly permanent deformation when an external stimulus (e.g., heat) is applied. SMM may be alloys, also known as shape memory alloys (SMA), or polymers – shape memory polymers (SMP). SMAs are characterized with significantly higher strength and elastic modulus than SMP, which makes them uniquely suitable for a variety of structural applications.
SMAs have two material phases: (1) the high-temperature phase – austenite (cubic crystalline structure), and (2) low-temperature phase – martensite (tetragonal, orthorhombic, or monoclinic crystalline structure). The SMA can reversibly transform from one phase to the other when heated or cooled down through the transformation temperatures. The transformations occur over a temperature range (Figure 5a). During the cooling, the transformation from austenite to martensite starts at martensitic start temperature \( (M_s) \) and is completed at martensitic finish temperature \( (M_f) \) (Figure 5a-top). Similarly, the heating transformation from martensite to austenite begins at austenitic start temperature \( (A_s) \) and is finished at austenitic finish temperature \( (A_f) \) (Figure 5a-bottom). Transformation temperatures can be changed by tailoring the molar ratio of alloy elements in an SMA. Characterization of transformation temperatures is usually performed via differential scanning calorimetry (DSC) (Lagoudas, 2008).

2.1.1 Shape-memory Effect

Martensite can take two variants: (1) twinned martensite (in which crystals are oriented in varying directions); and (2) detwinned martensite (in which one crystal orientation is dominant). Transformation from twinned to detwinned martensite is accomplished by application of external stress; the detwinning is initiated at the detwinning start stress \( (\sigma_s) \), and the process is completed at detwinning finish stress \( (\sigma_f) \) (Figure 5b). Upon the release of external loading (from some stress greater than \( \sigma_f \) to ‘zero’ stress) the deformation of martensite’s crystalline structure is retained (Figure 5c). Heating of the SMA to a temperature above \( A_f \) will then result in shape recovery through transformation of detwinned martensite to austenite (Figure 5c) – such transformation is termed reverse transformation. Cooling of SMA to a
temperature below $M_f$ triggers transformation from austenite to twinned martensite – also known as forward transformation. The described behavior, named shape-memory effect (SME), is a one-way process: subsequent heating above $A_f$ does not additionally alter the shape of SMA.

Shape-memory effect exhibited by an SMA is shown as a function of stress, strain, and temperature in Figure 5d. Cooling of austenite to a temperature below $M_f$ results in transformation from austenite to twinned martensite (Figure 5d from A to B). By applying external stress that exceeds $\sigma_f$, transformation to detwinned martensite is accomplished (Figure 5d from B to C). By releasing the applied stress, the SMA is then elastically unloaded to some permanent strain (Figure 5d from C to D). With subsequent heating of the SMA, the reverse transformation from detwinned martensite to austenite is initiated once $A_s$ is reached (Figure 5d from D to E). When the SMA is at a temperature higher than $A_f$ the full shape recovery is attained (Figure 5d point F). Once the heat source is removed, the forward transformation from austenite to martensite is accomplished once the temperature is below $M_f$.

The magnitude of transformation temperatures ($M_s, M_f, A_s, A_f$) increases with the applied load. The relationship between applied stress and transformation temperatures is schematically shown via a stress-temperature phase diagram in Figure 5c. The increase in the magnitude of transformation temperatures under stress must be considered when SMAs are used as actuators: when reverse shape recovery is initiated in a constrained detwinned SMA, the $A_s$ and $A_f$ transformation temperatures will increase as the restrained recovery stress develops in the SMA with heating. This necessitates heating the SMA above the transformation temperature obtained via DSC.
(in an unstressed state) to achieve a complete reverse transformation and maximum recovery stress.

Another important property of SMAs in actuator applications is their thermal hysteresis, which is defined as a temperature difference between the $A_s$ and $M_s$. In situations where it is desirable to retain the SMA recovery stress following removal of the heat source, it is required that the service temperature ($T_s$) which the SMA reaches upon cooling be much greater than $M_s$, or $T_s \gg M_s$. If this condition is not satisfied and $M_f$ and/or $M_s$ is/are lower than $T_s$, the SMA recovery stress could be completely lost during the cooling process due to the forward transformation (austenite to twinned martensite) (Lagoudas, 2008).
Figure 5. Thermomechanical characteristics of SMA: (a) Nitinol phase transformations; (b) transformation from twinned to detwinned martensite with external stress; (c) shape recovery of detwinned martensite through austenitic transformation; and (d) stress-strain-temperature diagram of nitinol (Lagoudas, 2008)

Figure 6. Stress-temperature phase diagram of SMA (Lagoudas, 2008)
2.1.2 Superelasticity

In addition to SME, SMAs also possess another property that allows for shape recovery – superelasticity. When the material is in the austenite phase and sufficiently large stress is applied to it, the SMA undergoes transformation to detwinned martensite. Upon release of the applied stress, the SMA returns to its original shape. The superelastic behavior is demonstrated in Figure 7. As stress is applied to the SMA, the transformation to detwinned martensite starts at stress $\sigma^{M_S}$ and is complete at stress $\sigma^{M_f}$. As the external stress is released, the transformation back to austenite is initiated and completed when stress reaches $\sigma^{A_S}$ and $\sigma^{A_f}$, respectively (Lagoudas, 2008).

Figure 7. Superelastic stress-strain behavior of SMA (Lagoudas, 2008)
2.2 Types of Shape-memory Alloys

The SMA generally contains two or three elements in their composition. The addition of a third element to *binary* SMA (SMA having two elements) results in a *ternary* SMA and can alter the properties of the base alloy. This gives the designers flexibility to choose from a large variety of SMAs, which can be applied to different problems in the industry. The third element is added to the binary SMA to modify their thermal properties like transformation temperature or the width of the thermal hysteresis. SMAs can also be classified into different categories based on the primary alloying element, mode of actuation (thermal, magnetic, etc.), operating temperatures and the desired behavior (SME or superelasticity). In the subsequent sections, the effect of the alloying element on the transformation temperature is primarily discussed, with particular emphasis on the *nickel-titanium-niobium* (NiTiNb) alloy, since that is the SMA which was used in this study (Lagoudas, 2008).

2.2.1 Nickel-Titanium (NiTi) based Alloys

The binary NiTi SMA has been widely used in different commercial applications due to its excellent SME, superelasticity, resistance to corrosion and biocompatibility. Buehler et al. (1963) studied the SME in equiatomic (50 at. % of Ni and Ti) NiTi SMA and discovered that this alloy had the highest $A_f$ out of all the NiTi based SMA. Increasing the amount of nickel by 1% over 50 at. % decreases the $A_f$ by up to 40 °C. However, decreasing the atomic percentage of nickel does not cause any change in the transformation temperatures (Lagoudas, 2008).

For an SMA to be applied in structural engineering, it needs to have the minimum response to large differences in temperature, so that it can be safely transported and installed without being accidentally actuated by the ambient
temperature. In structural engineering, the SME must be triggered when prestressing forces are required to repair damaged structures. When the SMA is used for its superelastic properties, the temperature of the SMA must be above the $A_f$ temperature. Applying stress above $\sigma^{M_f}$ will convert the SMA to detwinned martensite phase and removing the subsequent removal of the load will cause superelastic recovery due to transformation to austenite phase. Hence, when an SMA is chosen for structural applications because of their SME, the thermal hysteresis needs to be sufficiently wide to avoid accidental triggering of SME as well as loss of prestressing force (Lagoudas, 2008), (He et al., 2004).

The widening of the hysteresis loop can be achieved by adding a third element – Niobium (Nb) – to the existing binary NiTi alloy to form NiTiNb alloy. The widening of the thermal hysteresis occurs due to the partitioning of the strain into a recoverable part and an irrecoverable part. The recoverable strain is caused by the NiTi part and the irrecoverable strain is caused by the soft Nb part. Despite the widening of thermal hysteresis, the NiTiNb alloy has some residual strain when the pre-deformed alloy is actuated to trigger the SME. An important factor that affects the width of the hysteresis is the amount of Nb in the ternary NiTiNb alloy. The NiTiNb alloys generally have a wider thermal hysteresis than NiTi alloys ($M_s$ of -74.29 °C and $A_s$ of 122.5 °C). A lower Nb content increases the width of the hysteresis loop by increasing the difference between the $A_s$ and $M_s$. For example, Ti46.9Ni50.1Nb3 ($M_s$ of -68.15 °C and $A_s$ of -43.15 °C) has a wider thermal hysteresis than Ti44Ni47Nb9 ($M_s$ of -73.15 °C and $A_s$ of -23.15 °C) (Lagoudas, 2008), (Sakuma et al., 2002).

There are other applications of SMAs, where they are used as micro-actuators e.g., sensors in structural health monitoring systems. In these circumstances, it is
desirable to have a narrow mechanical hysteresis. A narrow mechanical hysteresis would make the material more sensitive to the changes in strain of the structure. The application of stress would lead to the development of strain in the SMA and causes a phase transformation which in turn would change the electrical resistance of the material.

Adding copper (Cu) to the binary NiTi alloy replaces the Ni to form NiTiCu alloys. Although the addition of Cu makes the hysteresis loop narrower, this also reduces the superelastic effect of the alloy by more than 50% when compared to the binary NiTi alloy. The Cu amount between 5 and 10 at. % is preferred for the choice of materials being used as actuators. When the Cu content is between 5 and 10 at. %, the material shows a clear phase transformation. A Cu amount greater than 10 at. % should be avoided as it tends to make the alloy lose its ductility (Lagoudas, 2008), (Sakuma et al., 2002).

Some applications of the SMA may require high Af and stable operating conditions like for example, the core region of an aircraft engine or downhole applications in the oil and gas industry. This led to the development of a new type of SMA known as the High-Temperature Shape Memory Alloy (HTSMA) which were formed by the addition of ternary elements like Palladium (Pd), platinum (Pt), Hafnium (Hf), Gold (Au) and Zirconium (Zr) to the binary NiTi alloy. The addition of these materials results in Af between 100 °C to 800 °C. The commercial application of alloys with ternary elements Pd and Pt have been limited due to the high material cost. Although alloys with ternary element Hf and Zr have a lower cost than the ones with Pt and Pd, they have lower Af. A significant limitation of HTSMAs is their lower
transformation strain (~3%) when compared to binary NiTi alloys (Lagoudas, 2008), (Otsuka and Wayman, 1999).

2.2.2 Copper (Cu)- Based Alloys

The Cu-based SMA became an attractive substitute for the NiTi SMA because of the lower cost as well as good electrical and thermal conductivity. These alloys usually have a narrower hysteresis than the NiTi alloys. The $A_f$ of these alloys are highly dependent on their composition and a change in the atomic percentage of the order of 10 is required to change the $A_f$ by 5 °C.

The Cu-Zn based alloys are characterized by good ductility and resistance to intergranular fracture when compared to other Cu-based alloys. CuZnAl, which is obtained by adding a third element Aluminum (Al) was found to have a significantly higher $M_s$ as compared to the binary alloys (e.g., increasing the Al content from 5 percentage by weight (5 wt. %) to 10 wt. % increases the Ms from -180 °C to 100 °C). CuZnAl has a transformation strain of about 3-4% and an operational temperature below 100 °C. CuZnAl alloy shows complete shape recovery and superelasticity within stress levels of approximately 200 MPa.

Another type of Cu-based alloys is the CuAlNi which is formed by adding Nickel (Ni) to the binary CuAl alloy. Compared to the CuZn alloys, CuAlNi has lower ductility and is prone to intergranular cracking which leads the alloy to fracture at a stress of about 280 MPa. The transformation strain in this class of SMA is limited to 3%, and the alloy exhibits poor response to fatigue. The benefit of adding Ni to CuAl is that it makes the alloy less sensitive to stabilization (tendency to stay in martensite phase by increasing the reverse transformation temperature) and aging (treatment of alloys at a high temperature which makes the properties change and settle very
slowly). The transformation temperatures change with the amount of Ni and Al but, the relative change in the transformation temperatures is insignificant which makes the width of thermal hysteresis stay quite constant (Lagoudas, 2008), (Otsuka and Wayman, 1999).

2.2.3 Iron (Fe)-Based Alloys

Iron-based SMAs have found significant use in structural engineering applications due to their lower cost, wider thermal hysteresis, higher elastic stiffness, and ductility when compared to most NiTi-based SMAs; however, they have lower shape recovery strain (about 4%) as compared to NiTi and Cu-based alloys. Moreover, Fe-based alloys offer better workability, weldability, and machinability which make them attractive for larger and more complex engineering structures. The most common Fe-based SMAs are FeNiCoTi and FeMnSi (Lagoudas, 2008),(Cissé et al., 2017).

2.2.4 Cobalt (Co)-Based Alloys

CoNiAl alloy can be formed by adding cobalt (Co) to NiAl or Ni to CoAl binary alloy. Their application in structural engineering is limited because they possess a very low \( A_f \) (about -26 °C) and a narrow thermal hysteresis. This alloy exhibits a transformation strain of about 4% when it is subject to thermal cycles and has stable superelastic behavior at a temperature sufficiently above the \( A_f \). CoNiAl alloy has a unique property by which it can undergo a forward transformation due to magnetic-field-induced reorientation when subjected to magnetic fields. This makes it an attractive choice for applications where magnetic actuation is required (Lagoudas, 2008), (Karaca et al., 2003).
2.3 Prior Applications of SMAs in Civil Engineering

SMAs have been developed and used since the 1960s in a wide range of applications in medical, robotics, aerospace and in the automobile industries. One of the most notable applications of SMA is in actuators. For example, SMA actuators have found applications in robotic systems that require powerful, compact and lightweight actuators. As opposed to electric, hydraulic or other actuators that lose power when they are downscaled, the SMA actuators have a high strength to weight ratio, which makes them ideal for miniaturized applications. In one of these applications, straight SMA fibers were attached between two disks in an actuator with the intention of achieving amplified mechanical displacement. The disks were kept separated by preloaded springs, but when the SMA fibers were heated, they contracted by the SME and thus achieved mechanical displacement (Figure 8a, 8b). The prospect of using SMA in structural engineering as a prestressing material by utilizing their SME and superelasticity properties have been recognized. Research is being conducted to determine the feasibility and effectiveness of such applications (Grant and Hayward, 1997).

Figure 8. SMA actuator showing: (a) the overall arrangement of the actuator; and (b) SME displayed in two disks connected by SMA fibers (Grant and Hayward, 1997)
The advent of SMA in the buildings and infrastructure started with Soroushian et al. (2001) who used Fe-based SMA rods to apply post-tensioning forces to repair damaged bridge girders. A methodology to repair the cracks caused by insufficient shear strength of a bridge girder was developed and verified in the laboratory. The implementation of this design was carried out in the field on an out-of-service bridge in Michigan (Figure 10). SMA rods were recovered in a restrained condition and the stress generated due to the restrained shape recovery was utilized to reduce the width of the shear cracks. Five SMA rods of 7/8 in. (22.2 mm) diameter was mounted externally across the cracks at an angle of 35 degrees on both sides of the beam web (Figure 9). The SMA rods were heated in a retrained condition above their $A_t$ by passing current, which resulted in a shape recovery and generation of recovery stress in the rods. Localized prestressing of the beam web via SMA rods resulted in a 40% reduction in crack width (from 0.022 in. to 0.013 in.). The rods developed recovery stress of 120 MPa (17 ksi), which was 32% lower than the maximum measured recovery stress (176 MPa or 25 ksi) in the laboratory experiments. The loss in recovery stress was explained by relatively large slippage at the anchorage (Soroushian et al., 2001).
Figure 9. Schematic diagram for SMA rods used to control of shear cracks in a bridge girder (Soroushian et al., 2001)

Figure 10. SMA rods installed in a bridge girder in Michigan to control shear cracks (Soroushian et al., 2001)

An experimental study on 1.5 ft. long concrete beams were conducted by Deng et al. (2006) to evaluate the post-tensioning potential of 6–8% prestrained NiTi (50 wt. % Ti) wires by using their SME. The NiTi wires were embedded in the reinforced
concrete beams by inserting the wires in the forms before pouring concrete. A computer-based data acquisition and control system was used to record midspan deflection of the beam and the temperature in NiTi wires (Figure 11). The SME was initiated by heating the SMA wires via Ohmic heating. The recovery stress steadily increased until the wires reached the $A_f$ temperature (40 °C). The total upward beam displacement, due to the maximum recovery stress generated by the NiTi wires, was 0.41 mm. When the wires cooled down below $A_f$ to room temperature, the transition from austenite to martensite occurred, and therefore the recovery stress dropped which caused the upward deflection of the beam to reduce from 0.41 mm to 0.36 mm (Figure 12a). The upward deflection depended on the actuation factors like the profile of current and actuation time. It was observed that a gradual increase of the current intensity from 37 A to 43 A in 150 minutes caused a greater reduction in the midspan deflection as compared to the condition where the current was increased stepwise from 0 A to 43 A over the same duration. The initial pre-strain also affects the recovery force; it was found that higher prestrain does not necessarily yield the highest recovery force (Figure 12b). A greater area of NiTi wires led to an increase in the recovery stress which increased the upward deflection (Deng et al., 2006a).

Figure 11. Experimental setup for study on NSM SMA (Deng et al., 2006b)
Li et al. (2006) experimentally evaluated the ability of SMA wires to control deflections in concrete beams. In this study, the SME of 2 mm diameter NiTi (50.8 wt. % Ni) wires was activated via Ohmic heating. In addition to this, the scope of this study also included carrying out permanent repair of the structures using CFRP after initial repair with NiTi wires. The change in resistance of the NiTi wires was used to assess the midspan deflection of the beams. At first, 1.65 ft. long beams were loaded in a three-point bending setup to induce midspan cracking (Figure 13). After inducing the cracks, the NiTi wires were actuated by passing constant currents of 14 A and 16 A. Actuation using a 16 A current generated more recovery stress than a 14 A current for the same temperature change. The test results showed a reduction of crack width from 3.98 mm to 0.65 mm following the shape recovery of NiTi wires. The specimens which had a greater NiTi reinforcement ratio were more effective at reducing the midspan deflection and closing cracks. The specimen with 0.08% NiTi reinforcement...
ratio reduced the midspan deflection by about 58% whereas the specimen with 0.16% NiTi reinforcement ratio reduced the midspan deflection by about 73%.

From this study, the possibility of using SMA as a strain sensor emerged based on the observation that change in resistance of the SMA had a linear relationship with the midspan deflection of the beams. Strengthening of the repaired beams using CFRP plates increased the load-carrying capacity of the specimens by up to 50% and hence, was considered a good permanent repair method (Li et al., 2006).

![Figure 13. Post-tensioning process showing: (a) test setup for three-point bending; and (b) schematic diagram of instrumentation for the test (Li et al., 2006)](image)

NiTiNb has a significantly wider thermal hysteresis than NiTi; thus, within the service temperatures typically experienced by civil infrastructure, the loss in the recovery stress at lower temperatures is minimal. Dommer and Andrawes (2012) conducted thermomechanical characterization of NiTiNb wires to investigate the feasibility of using NiTiNb wires for active confinement of reinforced concrete columns by wrapping the wires spirally around the columns and applying transverse...
pressure (Figure 14). The results of short-term and long-term recovery tests (Figure 15a, 15b) indicated that the NiTiNb wires could retain above 500 MPa of recovery stress over a wide range of temperatures (between -10 °C and 55 °C). The study explored resistive heating and torch heating as methods for triggering the SME. Both methods resulted in the generation of same recovery stress indicating that SME response is independent of the heating method. Based on these findings, the authors concluded that NiTiNb wires were suitable for active confinement of columns (Dommer and Andrawes, 2012).

Figure 14. Schematic diagram of active confinement using SMA spirals (Dommer and Andrawes, 2012)
In another study, Andrawes et al. (2010) compared cylinders with active
confinement provided by the SME of the NiTiNb wires, and passive confinement
provided by GFRP wraps. Some of the specimens had a combination of NiTiNb wire
and GFRP confinement. First, thermomechanical characterization of 6.4% prestrained
NiTiNb wires of 2 mm diameter was performed by conducting a restrained recovery
test. The maximum recovery stress was found to be 565 MPa at a temperature of 108
°C, but the stress reduced to 470 MPa (by about 20%) after cooling down to a
temperature of 16 °C. Evaluation of NiTiNb wires with different prestrains indicated
that the recovery stress linearly increased with the prestrain (Figure 16). The recovery

Figure 15. Thermomechanical behavior of NiTiNb wires showing: (a) Temperature vs.
Time curve; and (b) Recovery Stress vs. Time curve under different temperatures for
the long-term test (Dommer and Andrawes, 2012)
stress increased by 22% for with raise in prestrain from 2.8 to 6.4% (Shin and Andrawes, 2010).

![Graph showing recovery stress vs. prestrain for NiTiNb wires (Shin and Andrawes, 2010)](image)

Figure 16. Recovery Stress vs. Prestrain curve for NiTiNb wires (Shin and Andrawes, 2010)

Concrete cylinders were tested in compression under a displacement-controlled loading rate of 1 mm/min. The cylinders remained intact even after experiencing significant cracking and crushing (Figure 17b) because of the active pressure applied by the NiTiNb spirals. During the compression test, as the specimen was experiencing excessive deformations, the NiTiNb spiral fractured suddenly and the cylinder failed diagonally (Figure 17c). Comparing the strengthening effect of active and passive NiTiNb-GFRP wrapped specimens, it was found that active confinement imparted greater strength than the passive confinement. The comparison between the hybrid (NiTiNb-GFRP) and GFRP confined specimens show that the peak strength offered by the two were almost identical, but the ultimate strain was increased in the hybrid sample due to the ductility offered by the NiTiNb wires. This study also provided a comparative analysis of the tensile strength of spliced NiTiNb wires connected using U-clamps, sleeves and welded connections (Figure 18). The U-clamps and the welded
connections had the maximum and minimum tensile strength respectively (Shin and Andrawes, 2010).

Figure 17. Active confinement of Shape Memory alloy during different stages of compression testing: (a) before applying compression; (b) during compression; and (c) after failure (Shin and Andrawes, 2010)
Kotamala (2004) studied the feasibility of using NiTi wires to prestress concrete beams with the aim of increasing their flexural capacity. The tensile tests indicated that ultimate strength and strain at failure were affected by the detwinning load and not by the temperature of the wires. The modulus of elasticity increased significantly for the wires in austenite phase (Figure 19) (Kotamala, 2004).
In this study, prestressing was conducted on 20 in. long concrete beams which had a cross-section of 3 in. x 6 in. NiTi wires of diameter 0.119 in. with 4% prestrain were eccentrically placed in the concrete beams. Heating the NiTi wires above their $A_f$ (60-70 °C), by passing a 20 A current, generated 1335 lbf recovery force. Only 61% (810 lbf) of the recovery force was transferred to the concrete as the prestressing force. The loss in prestressing force was accredited to poor bond characteristics between the concrete and the SMA wires. However, it was observed that the overall flexural strength of the beams, prestressed with NiTi wires increased by almost 95 psi (Figure 20) (Kotamala, 2004).
The feasibility of self-post-tensioning bridge girders using the recovery stress generated from the restrained shape recovery of NiTiNb wires, when actuated via the heat of hydration of grouts, was investigated by Ozbulut et al. (2016) (Figure 21). While using in-situ repair techniques, activating the NiTiNb wires using the heat of hydration of the grouts, which are used to fill the grooves in which the wires are installed, is easier than jacking or resistive heating. The NiTiNb wires utilized in the experiments had a relatively low reverse transformation temperature of 11 °C, which was activated by the heat of hydration of cementitious grouts (temperature goes up to 53 °C). The restrained recovery tests on 10% prestrained NiTiNb wires generated sufficiently high recovery stress (above 500 MPa) which could be transferred to concrete as post-tensioning force. The amount of post-tensioning force generated can be adjusted in this self-post tensioned method by choosing grouts that generate different amounts of heat to bring about partial shape recovery (Ozbulut et al., 2015a).
The generation of the recovery stress depends on the temperature increment caused by the heat of hydration of the grout. The temperature should rise above the $A_f$ to trigger complete shape recovery. Experiments on different grouts (Table 1) demonstrated that the maximum rise in temperature was brought about by the Sika grout (53 °C). The rise in temperature by 53 °C was only sufficient to transform SMA partially to the austenite phase causing incomplete shape recovery. The bond characteristics of the NiTiNb wires with the grout was obtained by conducting pullout tests on 3.5 mm diameter wires embedded in 102 mm x 102 mm cylindrical concrete specimens. The average bond stress was found to be 1.3 MPa and there was no significant slippage as observed from the digital image correlation results. It was concluded that NiTiNb SMA wires can be used in the self-post tensioning of concrete bridge girders with grouts that can raise the temperature of the wire above $A_f$. In addition to this, the grout provides higher resistance to corrosion of the NiTiNb wires and increases the service life of the structure (Ozbulut et al., 2015a).

![Figure 21. The process of self-post-tensioning using NiTiNb SMA wires (Ozbulut et al., 2015a)](image-url)
Table 1. Temperature increase due to the heat of hydration of different grouts (Ozbulut et al., 2015a)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Grout</th>
<th>Water-Grout Ratio</th>
<th>Initial Temperature (°C)</th>
<th>Maximum Temperature (°C)</th>
<th>Temperature Increase (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Euclid</td>
<td>0.25</td>
<td>21</td>
<td>41</td>
<td>20</td>
</tr>
<tr>
<td>S2</td>
<td>Euclid</td>
<td>0.25</td>
<td>22</td>
<td>41</td>
<td>19</td>
</tr>
<tr>
<td>S3</td>
<td>Euclid</td>
<td>0.25</td>
<td>21</td>
<td>41</td>
<td>20</td>
</tr>
<tr>
<td>S4</td>
<td>Sika</td>
<td>0.24</td>
<td>21</td>
<td>48</td>
<td>27</td>
</tr>
<tr>
<td>S5</td>
<td>Sika</td>
<td>0.24</td>
<td>22</td>
<td>53</td>
<td>31</td>
</tr>
<tr>
<td>S6</td>
<td>Sika</td>
<td>0.24</td>
<td>21</td>
<td>48</td>
<td>27</td>
</tr>
<tr>
<td>S7</td>
<td>Five Star</td>
<td>0.25</td>
<td>22</td>
<td>41</td>
<td>19</td>
</tr>
<tr>
<td>S8</td>
<td>Five Star</td>
<td>0.25</td>
<td>22</td>
<td>41</td>
<td>19</td>
</tr>
<tr>
<td>S9</td>
<td>MasterFlow</td>
<td>0.27</td>
<td>22</td>
<td>41</td>
<td>19</td>
</tr>
<tr>
<td>S10</td>
<td>MasterFlow</td>
<td>0.27</td>
<td>22</td>
<td>41</td>
<td>19</td>
</tr>
<tr>
<td>S11</td>
<td>MasterFlow</td>
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<td>22</td>
<td>40</td>
<td>18</td>
</tr>
<tr>
<td>S12</td>
<td>MasterFlow</td>
<td>0.32</td>
<td>22</td>
<td>40</td>
<td>18</td>
</tr>
</tbody>
</table>

Song et al. (2006) used the SME of NiTi SMA wires to post-tension concrete specimens by using the recovery stress generated due to heating. At first, the change in resistance of the NiTi wires due to loading was used to measure the strain distribution in the concrete specimens without the help of any other external sensors. Three-point bending tests (Figure 22a) were carried out on concrete beam specimens of size 13.5 in. x 6 in. x 2 in. infused with NiTi wires of 0.015 in. diameter. Subsequently, the macro-sized cracks were closed using the post-tensioning force generated due to the shape recovery of the NiTi wires. It was concluded that structures affected by explosions and earthquakes can be rehabilitated by heating the SMA externally so that the wires undergo shape recovery to reduce the macro-sized cracks (Figure 22b) (Song et al., 2006).
The feasibility of using the SME of iron-based SMA (Fe-SMA) strips in place of CFRP strips for the prestressed strengthening of concrete structures was studied by Czaderski et al. (2014). The recovery stress of Fe-SMA varied between 250 to 300 MPa, which was relatively lower than that of NiTiNb SMA. The benefit of pre-stressing with Fe-SMA is the lower material cost and higher elastic stiffness. In this experiment, the prestressing was done by embedding the prestrained Fe-SMA in the concrete blocks and heating the SMA to induce recovery stress. The Fe-SMA strips were able to transfer a 3 MPa prestress when they were recovered. Lap-shear experiments (Figure 23) were conducted to study the bond behavior of the strips. The failure mode for ribbed Fe-SMA glued with cement-based mortar was compared to that of flat Fe-SMA without ribs and CFRP strips. It was observed that the failure in ribbed Fe-SMA was along the free length of the strip while the failure mode in flat Fe-SMA having no ribs and CFRP was sliding of the strip. Thus, it can be concluded that it is feasible to use ribbed Fe-SMA for the strengthening of concrete sections. However, CFRP strips with a cement-based mortar and Fe-SMA having no ribs cannot
be used for strengthening applications because of their poor bond quality (Czaderski et al., 2014).

The superelastic NiTi SMA restrainers were used as reinforcement for bridge decks to avoid excessive movement and unseating of the bridge girders during cyclic loadings (e.g., earthquakes). Andrawes et al. (2005) conducted a nonlinear dynamic analysis of a typical California multiple-frame reinforced concrete box girder bridge using a suite of 10 ground motion records. A comparison of performances was made using different models of both superelastic SMA restrainers and the traditional steel cable restrainers. The superelastic restrainers reduced the relative hinge displacements significantly more than the steel restrainers. The average maximum hinge opening was reduced by up to 43% for superelastic restrainers, whereas the steel restrainers reduced the hinge displacement by only 16%. The reduction in hinge opening is attributed to the high elastic strains of the superelastic elements. The superelastic restrainers also displayed significant damping characteristics. The steel restrainers performed
inadequately in most cases due to their low elastic strain limit. The study showed similar behavior in both the steel and superelastic restrainers during the first few cycles, but after the steel restrainers yield, residual strain accumulates in the steel whereas the superelastic restrainers have a re-centering ability and can recover the original length after being deformed at a strain between 6-8%. It was also observed that the maximum frame drifts did not depend on the type of restrainers (Andrawes and DesRoches, 2005).

A study was conducted by Ozbulut et al. (2016) where these NiTi multistrand cables (Figure 24) were subjected to the uniaxial tensile test under various cyclic amplitudes and loading frequencies. The performance of the cables was evaluated in terms of energy dissipation, viscous damping, secant stiffness and residual strain. The cables consisted of 7 strands of wires each having an outer diameter of 8 mm and a length of 150 mm. The test setup is shown in 25. The dissipated energy increased steadily with the increasing strain amplitude up to 7.2% (Figure 26a). The viscous damping first increased and then became constant, but a small decrease was observed when some of the strands in the cable broke (Figure 26b). The secant stiffness decreased with the increase in strain amplitude (Figure 26c). For 100 loading cycles, the maximum residual strain was found to be 1.1%. The dissipated energy decreased by 34% as the number of cycles increased. It was observed that after the 40th cycle, increasing the number of cycles did not affect the properties of the NiTi multistrand cable. The study also showed that the performance of the cable changed while varying the strain rate of loading. The change in performance occurred because the higher rate of loading did not allow the latent heat from the wires to dissipate into the atmosphere, thus changing their transformation temperatures (Ozbulut et al., 2015b).
Figure 24. Different views of multistrand NiTi cables: (a) cross-sectional view; and (b) longitudinal view (Ozbulut et al., 2015b)

Figure 25. Test setup consisting of laser extensometer, DIC camera and infrared camera (Ozbulut et al., 2015b)
The feasibility of using NiTi SMA wires as longitudinal reinforcement to reduce residual deflections by using their superelastic property was studied by Czaderski et al. (2006). SMA wires of 4 mm diameter were used to reinforce a 1.14 m long concrete beam. Tensile tests at different temperatures indicated that Young’s modulus in the martensite phase is 3.5 times lower than in the austenite phase. Subsequently, pullout tests were conducted on the wires to determine their bond strengths. Results indicated that different surface conditioning techniques (such as sandblasting) provide better bonding capability of the wires. Finally, four-point bending tests were performed on concrete beams reinforced with NiTi wires (Figure 27) and concrete beams without NiTi wires. The four-point bending test results showed that stiffness and yield strength of concrete beam reinforced with NiTi was noticeably higher in the austenite phase than martensite. The beams reinforced with NiTi wires were subjected to several deformation cycles to study the crack patterns. The stiffness of the beams decreased after deformation cycle 2 and maximum crack width of 1.1 mm was measured during the 2nd and 4th cycle. Brittle fracture occurred at the mid-span of one of the NiTi wires during the 7th cycle. Finally, after the 8th cycle,
the beam failed due to the brittle fracture of two NiTi wires. The reason for brittle failure of wires was fatigue corrosion cracking resulting from cyclic loading in presence of water and chlorides of the epoxy adhesive which was used for bonding the wires and concrete (Czaderski et al., 2014).

Saiidi et al. (2007) used NiTi as tensile reinforcement in concrete beams and studied their effectiveness in reducing the permanent deformation in the beams by using their superelasticity. In this study eight (8) beams were designed and tested, out of which four beams were reinforced with NiTi wires and the other four were reinforced with conventional steel. In addition to this, analytical studies were also conducted on four beams of size 5 ft. x 5 in. x 6 in. with a combination of high strength steel and NiTi wires, and CFRP and NiTi wires. The beams were subject to four-point loading and designed to fail in flexure. The ratio of residual displacement to maximum displacement for the beams having NiTi wires as longitudinal reinforcement.
reinforcement was 0.0333, whereas the beams with mild steel reinforcement had a ratio of 0.338. The NiTi reinforcement reduced the residual displacement by 90% when compared to mild steel (Figure 28). The measured residual strain was 7% of the maximum strain for the beams reinforced with NiTi wires, and 85% of the maximum strain for the beams reinforced with steel. Due to the low modulus of elasticity of the NiTi wire, however, the stiffness of SMA reinforced beams were lower by 38%. The analytical test results on the SMA-mild steel reinforced beams showed an increase in stiffness (by up to 25%) over SMA reinforced beams with the ability to recover deformations partially (Saiidi et al., 2007).

The superelastic property of SMA has also been used to repair and strengthen heritage structures with architectural significance. For example, Indirli and Castellano (2000) used the superelasticity of a system of pre-tensioned NiTi SMA rods connected in series with steel bars to restrain horizontal movement of a bell tower which was heavily damaged during an earthquake in Trignano, Italy in 1996 (Indirli et al., 2001).
A study of slender heritage buildings, where superelastic NiTi SMA tie bars have been installed to resist excessive deformation and prevent collapse, was carried out by the same research group. The NiTi tie bars, which ran throughout the height of the tower, were anchored at the foundation. The tie bars were used to retrofit and rehabilitate the S. Giorgio Church Bell Tower which was severely damaged in a previous earthquake in 1996. This structure was declared intact after a subsequent earthquake in 2000 (Indirli et al., 2001).

Song et al. (2006) reviewed the application of SMA as actuators, passive energy dissipaters and dampers. They also studied the application of SMA as structural rehabilitators in smart structures. Over the years, many different materials like gold, cadmium and iron-based alloys have been used for their SME. Out of these, the best thermomechanical and thermoelectrical properties were exhibited by the NiTiNb SMA. The damping behavior of SMA depended on factors like temperature, loading frequency and the number of loading cycles. It was also observed that the prestrained martensitic NiTiNb had better damping capacity than NiTiNb SMA in the austenite phase. The dissipation of energy was dependent directly on the diameter of the wire and the effect of ambient temperature was negligible (Song et al., 2005).

The SMA wires show damping property in tension as well as bending by their superelasticity. The SMA wires can be used in ground isolation systems, where isolators made of NiTi SMA wires can be installed at the junction of ground and superstructure to filter the seismic energy transferred from the ground and restore the structure to its original position. Both martensitic and superelastic NiTi wires can facilitate energy dissipation mechanisms when infused in structural elements because the wires are able to absorb the vibration due to their hysteretic stress-strain
relationship. The martensitic NiTi wires have a better damping capacity as compared to the superelastic SMAs but they must be externally heated to regain their original shape whereas the superelastic SMA, although having a lower damping capacity, has a strong re-centering property which results in lower residual strains. Figure 29 shows the use of superelastic NiTi SMA wires used in isolating systems by different researchers (Wilde et al., 2000a).

Feasibility of using base isolating systems made of NiTi SMA bars exhibiting superelastic property was investigated for elevated highway bridges by Wilde et al. (2000) (Wilde et al., 2000b). It was observed that the excitation level had a substantial effect on the effectiveness of the base isolation system. At small excitation levels, a system of NiTi bars firmly links the pier and the deck slab but at medium excitation, stress-induced forward transformation leads to relative displacement between the pier
and bridge deck. At high loading state, the SMA behaves superelastically, and there were minimal displacements observed when compared to ordinary damping systems.

A NiTi-based isolation system was investigated for its load carrying capabilities. Dolce et al. (2001) found that the isolation system can carry 600 KN force and withstand up to 180 mm deflection. The NiTi wires were wound around three stubs connected to tubes, which allowed reciprocal movement between the substructure and the superstructure. The system was very effective in filtering energy transmission. The feasibility of using the NiTi-based base isolation system as a re-centering isolation system was thus verified (Dolce and Cardone, 2001).

Casciati et al. (1998) used a finite element method to analyze static and dynamic responses of bridges to earthquakes which have been retrofitted with martensitic NiTi bars. The result of such simulations verifies the feasibility of using NiTi SMA as damping elements for bridges. The techniques of application of NiTi in different types of bridges are shown in Figure 30 (Casciati et al., 1998).

Figure 30. NiTi SMA bars used as a damping element for bridges (Casciati et al., 1998)
Anchorage made of NiTi SMA rods has been used as vibration dampeners in connections between beam-columns and other parts of the structures as shown in Figure 31. Tamai et al. (2003) found that anchorages made using NiTi rods of 20-30 mm diameter improved the response of the joints during seismic ground motion. Results obtained from numerical simulation of these connections subject to pulsating tension loading tests show that the connections have a good capability of dissipating energy and reducing vibration under severe ground motions (Tamai et al., 2003).

Martensitic NiTi tendons were used as primary load transferring elements in structures by Leon et al. (2001). The tendons achieved 4% repeatable hysteresis in rotation and sustained a strain of up to 5% without incurring any permanent damage.

![Figure 31. SMA used as connectors between structural members (Tamai et al., 2003)](image)

Superelastic SMA wires have been used for shape restoration of concrete beams by Sakai et al. (2003). The results of this study indicated that mortar beams reinforced with NiTi SMA wires could recover their shape entirely even after developing substantial cracks and deflection. Superelastic restoration of multistrand NiTi SMA cables was used to close cracks in concrete beams (Figure 32a) after being
loaded to 11000 lbf and developing significant cracks. Fourteen (14) multistrand NiTi cables of 0.125 in. diameter with 2% prestrain were used to achieve post-tensioning of a 2 ft. long concrete beam with a cross-section 4 in. x 6 in. The cables were restrained at both ends with a special type of clamps. The cables developed post-tensioning forces due to their superelastic property when the applied load was removed. After the specimen cracked at 11000 lbf load the cracks were closed by the NiTi cables as shown in Figure 32b (Sakai et al., 2003).

![Figure 32. Elastic restoration of concrete beam: (a) cracking of the beam; and (b) restoration of the crack (Sakai et al., 2003)](image)

2.4 Bond Characteristic of SMA with Concrete

The bond between SMA wires or bars and concrete is of paramount importance when anchorage is not provided. Characterization of the bond between SMAs and concrete are summarized in this section.

Billah and Alam (2016) conducted various experiments to study the bond characteristics between NiTi SMA bars and concrete. The change in bond behavior with the variation of parameters such as bar diameter, concrete strength, bonded
length, concrete cover and surface treatment (sand coating) were studied by the authors. Several cylindrical concrete samples were prepared with a 450 mm long, 0.75% prestrained NiTi bar embedded at the center. The samples were subjected to pushout tests under varying testing parameters, as shown in Figure 33. For smooth NiTi bars, the stress increased elastically up to a peak load after which the bond failed. The unloading portion of the curve descended linearly until it reached residual stress and remained constant beyond that. The slip was negligible during the loading phase and started to increase after the sample cracked during the unloading phase and large slippage was observed as the sample reached its residual load level (Figure 34). The smooth bars failed without any splitting crack indicating that there was no significant bond between smooth bars and concrete. The average residual bond strength of a NiTi bar coated with 600 μm sand was 29% and 35% higher than a bar coated with 300 μm sand, and a smooth bar respectively. Test results also indicated that the average maximum and average residual bond strength decreased with the increase in embedment length. The bond strength increased with the increase in strength of concrete. The average maximum bond strength decreased with the increase in bar diameter, whereas the residual bond strength remained quite unchanged (Billah and Alam, 2016).
The effect of various parameters such as surface conditioning (sandblasting), bar diameter, embedment length and anchorage on the effectiveness of the NSM NiTi SMA bars, used in strengthening applications, was studied by Daghash et al. (2017).
For this experiment, a C-shaped cube concrete specimen of size 305 mm and average compressive strength 38 MPa was made with a groove equivalent to twice the diameter of the bars (Figure 35a). The specimens were subjected to a pull-out test at a constant displacement rate as shown in Figure 35b. The test results were interpreted based on average bond stress and the stress-slip curves of different samples. Sandblasting was found to be an effective surface treatment technique that increased the average maximum bond strength. The increase in surface roughness improved the mechanical interlock between the bar and the epoxy resulting in increased bond strength. The bond strength decreased with the increase in bar diameter and bond length. The bond strength first increased with the increase in bond length for up to 7 times the diameter of the bar and then started to decrease. This happens due to the change in failure mode from epoxy splitting to gradual bar slippage at peak stresses. The difference in slippage between the loaded and unloaded ends of the bar at peak stresses is caused due to the change in failure mode. Moreover, providing end hooks also gave significant mechanical anchorage to the NiTi bars and increased the average bond strength by almost 33% (Daghash and Ozbulut, 2017).
Figure 35. Pullout test: (a) specimen details; and (b) test setup (Daghash and Ozbulut, 2017)

2.5 Finite Element Method Modeling of Prestressed Concrete Girders

Previously, some numerical models have been developed to simulate the post-tensioning of reinforced concrete girders. The studies are not constrained to the application of prestressing using SMAs and involve the use of using steel tendons, FRP composites.
In 2001 Fanning modeled post-tensioning of reinforced concrete girders in commercial finite element software ANSYS using a smeared cracking model of concrete with dedicated element type SOLID65. The post-tensioning was modeled by providing initial strain in the tendon, corresponding to the post-tensioning forces in the tendons followed by post-tensioning and finally loading the model up to failure. Comparison to experimental results showed that the ultimate load capacity was underestimated by the finite element method model by about 12% but the overall behavior was predicted accurately as shown in (Figure 36). The discrepancy between the numerical and experimental results was possible because of the material property of the reinforcement bars used for modeling. The reinforcement steel was assumed to have a nominal yield strength and elastic perfectly plastic behavior but, in the experiment, might have behaved differently (Fanning, 2001).

Figure 36. Comparison of load vs. midspan deflection of the numerical model and experimental beam (Fanning, 2001)

In 2003, Marovic et al. carried out a comparative study between two-dimensional and three-dimensional numerical analysis of prestressed concrete
structures using program PRECON 2D and PRECON 3D. Analysis of several numerical examples showed that both 2D and 3D programs showed good agreement between the load vs. deflection behavior of the models and the experiments. It was concluded that the 2D program can accurately model a structure that can be modeled accurately in 2D such as beams in-plane stress or plane strain condition. The 2D program takes less time and is computationally less expensive. The 3D program, on the other hand, can be used to accurately model structures in which the effects of cross-section of the structure significantly affect the results of the simulations and the structure cannot be modeled accurately using two dimensions (Marovic et al., 2003).

Vecchio et al (2006) carried out a finite element based approach to simulate the shear resistance of reinforced concrete members with unbonded longitudinal tendons. The model was formulated using truss elements as the post-tensioning tendons and a two-node bond link element connecting the tendon to concrete to simulate the friction effects of the unbonded tendons due to profile curvature and tendon wobble (Figure 37). The model was validated by comparing load capacity, cracking patterns and deformation of shear-critical beams to previously conducted experiments. The model was also successful in calculating the tendon forces at the ultimate condition within a maximum error of 24%. The increase in the tendon stress during loading influenced the response significantly whereas the behavior of the beams was not affected by accounting for the friction effect (Vecchio et al., 2006).
In 2015, Kang et al. (Kang et al., 2015) compared the structural performance of bonded and unbonded post-tensioning system on beams by developing a non-linear finite element method model which was validated using experimental data from the literature. The bonded interaction between the prestressing members and concrete was modeled using both embedded technique and surface to surface contact during post-tensioning. The unbonded condition of the prestressing reinforcement was modeled using the spring method and contact formulations by the penalty method. The comparison to experimental results showed good agreement between the response of both bonded and unbonded conditions showed good agreement by using all the interactions mentioned above (Figure 38). In addition, one-way slab members and slab column connections were modeled successfully and validated using experimental data proving the robustness of the numerical model.
Figure 38. Load vs. deflection relationship for: (a) bonded post-tensioned beam; and (b) unbonded post-tensioned beams (Kang et al., 2015)

In 2016, a parametric study was conducted on the strengthening of post-tensioned composite beams with external tendons by El-Zohairy et al. by varying various parameters like tendon length, shape, and eccentricity and degree of shear connections. The finite element method model of the composite beams (Figure 41) were validated by using experimental data, which showed a good match between the moment vs. deflection response (3.5% error in ultimate moment capacity). The ductility of the section was found to increase by using a straight tendon in place of a draped tendon. The tendons, when draped underneath the beams, showed a better strengthening than when draped parallel to the web. The tendons with higher eccentricity showed an increase in the initial camber by 25% and the increase in
stiffness by 30%. The degree of shear connection (ratio of actual number of shear connectors in the beam and the required number of shear connectors to get the full composite action) should be above 80% to ensure effective strengthening and avoiding shear-failure of the post-tensioned composite beams (El-Zohairy and Salim, 2017).

Bedon et al (2016) simulated laminated glass (LG) beams post-tensioned with mechanically anchored tendons using extended finite element method (XFEM). The glass was modeled as a brittle material using the concrete damaged plasticity method. The model was validated using experimental data from previous full-scale tests (Figure 40) and the results showed an increase in the fracture load of the beam (up to 61%) and the maximum post-fracture load due to post-tensioning. The failure mechanism observed was similar to that of a prestressed concrete beam with the yielding of steel tendons followed by the crushing of the glass. A parametric study was also performed by changing the tendon geometry, the initial prestressing force in the tendons and observing the effect of initial geometrical imperfections on the post-

Figure 39. Finite element method model showing: (a) mesh; and (b) different parts of the composite beam (El-Zohairy and Salim, 2017)
tensioning. It was observed that the increase in the tendon cross-section increased the ultimate fracture load whereas the increase in prestressing forces in the tendon increased the initial fracture load. The post-tensioning was also found to increase the residual loads after cracking (Bedon and Louter, 2016).

Building on the previous study (Bedon and Louter, 2016), the same research group studied LG beams post-tensioned with adhesively bonded steel tendons. The steel tendons were attached to the beams using a 0.1 mm (nominal) thick adhesive layer of a two-component epoxy adhesive (3M Scotch-Weld DP490). Apart from the parametric study on the effects of tendon geometry and prestressing force, the adhesive joint type and size were also studied. The adhesive thickness directly influences the stiffness and the load capacity of the beams. The beam performance was affected by the shear bonding capacity of the adhesive joint under high bending deflections with the post-tensioning effects minimizing at high deflections due to thick adhesive joints (Bedon and Louter, 2017a).
In 2017, Bedon et al. further extended their research by carrying out a finite element analysis of the bending performance of post-cracked LG beams post-tensioned with glass-FRP bars. In addition to a review of the existing research on glass-FRP post-tensioned beams, further parametric studies have been conducted to observe the effect of quasi-static loading configurations at room temperature on laminated glass beams having adhesively bonded FRP. The parameters in this study were the initial prestressing force, sectional and material properties of the CFRP tendons and the effects were the load vs. deflection response, propagation of damage and cracks in the post-failure stage as well as the stress in the adhesive joint between glass and CFRP. The increase in prestressing force improved the reaction and deflection at the ultimate along with an increase in the elastic stiffness. The change in cross-section of the CFRP tendons increased the peak load and stiffness of the beams whereas changing the material to steel having lower yield stress than the FRP showed a more ductile behavior with lower peak load (Bedon and Louter, 2017b).

Brenkus et al. (2019) developed and validated a simplified finite element method model simulating the behavior of flexural members having both pre-tensioned bonded tendons and post-tensioned unbonded tendons. The uniqueness of this model was that it consists of beam and truss elements which make it computationally inexpensive and hence provide opportunities for various parametric studies. The contact of the unbonded tendons with concrete was made using a virtual tendon and a real tendon was used to introduce the prestressing forces as predefined fields and were connected to the virtual tendons using springs with high stiffness (Figure 41). The comparison of ultimate flexural strength (within 5%), tendon stress, ultimate displacement (within 15%) and failure mode matched well with the experimental
results proving that the model was able to successfully simulate the behavior of the prestressed members having bonded and unbonded tendons under flexural loading (Brenkus et al., 2019).

In 2019, Abouali et al. developed a nonlinear 3D finite element method model simulating the flexural strengthening of reinforced concrete beams, prestressed using the shape-recovery of NSM Fe-SMA strips. The numerical model, which employed the damaged plasticity model in ABAQUS combined with a predefined prestress, was validated by comparing the load-displacement response with experimental data from Abouali et al. and they were in good agreement. The study compared the behavior of the beams strengthened with Fe-SMA strips to beams without strengthening and strengthened with CFRP strips. The beam strengthened with CFRP strip had the

Figure 41. Finite element method model showing the details of bonded and unbonded beams (Brenkus et al., 2019)
highest peak load of close which was more than the Fe-SMA strengthened beam by 25% but the ductility reduced by 40% with the CFRP strengthened beam having a brittle failure whereas the Fe-SMA strengthened beam had a ductile failure as shown in Figure 42. A parametric study showed that a low ratio of the tensile steel reinforcement to the NSM Fe-SMA reinforcement improved the ultimate moment capacity by up to 60% (Abouali et al., 2019).

Figure 42. Comparison of the load vs. deflection behavior of beams strengthened with FRP and SMA to a beam with no strengthening (Abouali et al., 2019)

2.6 Chapter Summary

Studies have been previously conducted to study the shape-recovery and superelastic properties of SMAs having various chemical compositions. The chemical composition of the SMA is mostly governed by the type of applications. The structural application mainly requires the use of Fe-SMA and NiTi SMA because of the wide range of temperatures over which they can retain the prestress (also known as service temperature). The Fe-SMA generally has a lower range of service temperature and
higher stiffness than the NiTi SMAs. Previous attempts have been made to use the shape-recovery property of the SMAs to strengthen reinforced concrete structures. The major issues in the previous studies were partial loss of prestress due to anchorage issues and due to ambient temperature. Most of the successful applications for repair and retrofitting structures involved the use of superelastic properties of the SMA. The generation of prestress using the shape-recovery property depends on a number of factors like the process of heating, the initial pre-strain, the geometrical properties of the tendons, etc. The bond behavior of the SMA varied over a range of variables like bar diameter, surface treatment, bonded length and the application of end anchorage. The numerical models of prestressed concrete beams with SMA are quite few in number and most of the models involve the use of steel tendons or FRP composites. The prestressing in the numerical models have always been carried out on undamaged girders. It has been observed that error in the simulation results occurred due to the assumption of theoretical material models in the absence of experimental material properties. Several parametric studies indicated that the bonded models displayed a higher yield and ultimate load capacity while having a lower ultimate deflection when the reinforcing material was same as the prestressing material. The increase in effective prestress increased the range of service load for the models.
Chapter 3

EXPERIMENTAL STUDY

The experimental study was conducted to learn about the behavior of the NiTiNb SMA material and to evaluate the performance of the post-tensioning technique on structural-scale reinforced concrete girders. A differential scanning calorimetry (DSC) test was conducted to determine the transformation temperatures of NiTiNb. Restrained recovery tests were then conducted on 3 SMA specimens to determine the recovery and residual stress after being heated above the $A_f$. Tensile tests were performed on: (i) 3 austenite specimens; (ii) 3 specimens after restrained recovery; and (iii) 3 specimens after prestraining and restrained recovery to study the effect of different conditions on the stress-strain behavior of NiTiNb SMA. The results of the material characterization tests were used as input parameters for the material models in numerical modeling. Bond characterization tests were performed on 12 small plain concrete beam specimens reinforced with SMA wires to determine the bond between SMA wires and fast-setting grout, and the effectiveness of anchorage devices to transfer the post-tensioning forces. The results of the bond characterization tests led to the decision of using end anchorage to transfer the post-tensioning force to the concrete. Proof-of concept tests were conducted on 4 reinforced concrete girder specimens using 8, 10 and 12 SMA wires. The girders were cracked to introduce damage, post-tensioned using anchored SMA wires and subsequently loaded up to ultimate.
3.1 Materials

3.1.1 SMA Wires

The SMA selected for this study was composed of nickel-titanium-niobium (NiTiNb). The ternary alloy was selected over a more common binary NiTi due to its wide thermal hysteresis which makes it appropriate for the range of temperatures experienced by bridges. SMA alloy 8-ft. wires measuring 0.154 in. (3.92 mm) in diameter in the prestrained state, were used in this study. The wires were supplied with a minimum of 2.5% guaranteed prestrain. Per the manufacturer, the material was reported to have a minimum $A_t$ of 60 °C in the prestrained state. The wires had a standard black oxide surface finish.

3.1.2 Concrete for Bond Characterization Specimens

Beam bond characterization specimens were made of concrete with water to cementitious material ratio (w/cm) of 0.37. The concrete mix consisted of Portland Cement Type I/II, river sand, coarse aggregate with gradation curve #89, and water in a ratio of 1:1.04:1.65:0.37. The constituents were adjusted by making corrections to account for moisture content of the aggregates. The 28-day compressive strength of 4×8 in. concrete cylinders, according to ASTM C39/C39M-18 (C09 Committee) was found to be 8000 psi.

3.1.3 Concrete for Reinforced Concrete Girder Specimens

The reinforced concrete girders were cast at the same time and from the same batch of ready-mix concrete, having cementitious material, concrete sand, coarse aggregate and water in the ratio of 1:2.73:4.04:0.49. The cementitious material consisted of a 3:1 mixture of Type I/II Portland Cement and Type C Fly ash. The
coarse aggregate consisted of a 3.7:1 mixture of Grade A gravel and pea gravel. Water reducing admixture and air-entraining admixture was used to improve workability and prevent segregation of the concrete mixture. The ratio of water to cementitious material was fixed at 0.49. The quantities of the mix were adjusted to account for the moisture content of the aggregates. The 28-day compressive strength of 4×8 in. concrete cylinders, according to ASTM C39/C39M-18, was 5000 psi.

3.1.4 Internal Steel Reinforcement

The steel bars used in reinforced girder specimens consisted of: (i) #4 deformed bars as top and bottom longitudinal reinforcement; and (ii) #3 deformed bars as stirrups. The steel had a yield stress of 69 ksi and tensile strength of 108 ksi according to the manufacturer.

3.2 Preparation of Specimens

3.2.1 DSC Test Specimens

DSC test specimens were prepared by cutting a 100 mg piece from the 2.5% prestrained wire and placing it in a DSC crucible of 20 µl capacity and after oiling the crucible to facilitate good heat transfer to the specimen.

3.2.2 Restrained Recovery Test Specimens

Three (3) SMA wires of 11 in. length were cut from an 8-ft long wire. K-Type thermocouples were attached to the wires for recording and monitoring the temperature during heating.
3.2.3  Tensile Test Specimens

Twelve (12) tensile specimens were prepared by cutting six 11-in SMA wires from the 8-ft. wire supplied by the manufacturer. Three (3) specimens used for characterization of the tensile properties of austenite were heated in an unrestrained condition above $A_f$ via Ohmic heating by passing a current of approximately 44 A until the temperature of the wire reached 200 °C. The heating initiated shape recovery and consequent transformation from martensite to austenite. Once the specimens cooled down to room temperature, strain gages were attached to the surface of the wire. Nine (9) other untransformed specimens of the same length required no special preparation and were used for tensile tests under various conditions and the recovery test on SMA. J-Type thermocouples and a 2 in. extensometer were attached to the specimens during the recovery test and the tensile test, respectively.

3.2.4  Bond Characterization Test Specimens

Beam bond characterization specimens (Figure 43) measuring 14 in. in length and 4×4 in. in cross-sectional dimensions had a 1 in. long notch of $\frac{1}{4}$ in. thickness in the middle to simulate cracked concrete. The bottom of the notch was widened to facilitate the instrumentation of sMA wires. A longitudinal groove having a cross-section of 0.4×0.4 in. was introduced throughout the length of the specimen for the placement of NSM SMA wire. The sMA wires were kept unbonded at the support by wrapping with scotch tape. The bonding matrix used for the bonded specimens was a quick-setting cementitious grout of brand SikaQuick VOH mixed with SikaLatex R in a ratio of 44:7 by weight. The test specimens were prepared based on variables like: specimens post-tensioned with SMA wire (PT) vs. specimens reinforced with austenite (AU); specimens with middle portion of SMA bonded before SMA recovery (BOB)
vs. unbonded SMA wire (UB) vs. specimens with middle portion of SMA wire bonded after SMA recovery (BOA); specimens with anchoring device (A) vs specimens without anchoring device (N). The different test specimens were denoted as shown in Figure 42: (i) Control specimen without SMA wire; (ii) BOB-PT-N; (iii) BOB-AU-N; (iv) UB-PT-A; and (v) BOA-PT-A. Standard post-tensioning strand chucks were used as anchorage devices.

Anchored specimens utilized the anchoring device consisting of strand chucks supported against the 1/8 in. thick steel plate resting on the concrete surface. Components of the strand chuck are: jaw segments, retaining ring, and the strand chuck body (Figure 45). The strand chuck body has a tapered hole which allows the jaw segments to slide in. While jaw segments are smooth on the outer surface, the inner diameter is populated with rows of fine ridges (also known as “teeth”). When the wire is placed between the jaw segments the jaw segment teeth bite into it preventing it from slipping. The retaining ring is used to keep the jaw segments together.

Figure 43. Nomenclature key to bond characterization specimens
3.2.5 Reinforced Concrete Girder Specimens

The post-tensioning experiments were conducted on structural-scale lightly reinforced concrete girders (Figure 46) designed according to ACI 318-14 (ACI Committee 318, 2019). The girders were of length 7.5 ft. and cross-section 9×16 in.
with 3-#4 steel bars as tensile reinforcement and #3 bars as stirrups placed at 4 in. center to center distances. A clear cover of 2.5 in. was maintained at the bottom of the girders to facilitate the NSM installation of the SMA wires. Except for the Control girder, the girders to be post-tensioned girders were cracked by loading in a four-point flexural setup to simulate typical damaged girder. Grooves of 0.4 in. width and 1.5 in. depth were introduced in the bottom cover of the cracked girders to accommodate the SMA wires (Figure 46b). Two SMA wires of length 8 ft. were placed in each of the grooves and kept unbonded throughout. The number of wires used for post-tensioning was kept as a test variable. The reinforced concrete girders subject to the post-tensioning tests were named as 8-SMA, 10-SMA, and 12-SMA respectively which corresponds to steel to SMA ratio of 3.95, 3.16 and 2.63, respectively. The grooves were grouted in the support region to provide an adequate bearing surface for the girders. The wires were anchored at the ends of girder using the strand chucks supported on steel bearing plates as shown in Figure 46a.
Figure 46. Details of the reinforced concrete girder specimen: (a) Elevation; and (b) Cross-section

3.3 Experimental Procedures

3.3.1 Material Characterization Tests

3.3.1.1 DSC Test

DSC experiment was performed in a nitrogen atmosphere in a Netzch DSC 214 differential scanning calorimeter. The machine was programmed to cool the
specimen to -60 °C and subsequently heating it to 300 °C at a rate of 10 °C /min. The temperature and heat flow data were recorded at a 300 Hz acquisition rate.

3.3.1.2 Restrained Recovery Test on Martensite SMA Wire

The SMA specimens were placed in the testing machine using the hydraulic grips; the sample had a grip length of 2 in. and a total gage length of 7 in. A 2-in. extensometer and Type-K thermocouple were affixed to the specimen as shown in Figure 47. The test was conducted so that the SMA wire underwent restrained shape recovery. The first phase of the test Ohmic heating of the wire by passing approximately 44 A current until the specimen reached at least 170 °C. During shape recovery, recovery load, strain and temperature were recorded. Once the specimen achieved full recovery, the current source was turned off and the specimen was allowed to cool down to room temperature to determine the loss in recovery stress associated with cooling of the wire.
3.3.1.3 Tensile Tests on Austenite SMA Wire

The specimens were placed in the 22-kip MTS universal testing machine using the hydraulic grips; the sample had a grip length of 2 in. and a total gage length of 6.5 in. A 2-in. extensometer was affixed to the specimen, in addition to the previously attached strain gage as shown in Figure 48. The experiments were performed under the constant displacement rate of 10 mm/min. The specimen was loaded until rupture. The load and strain data from the extensometer was recorded.
3.3.1.4 Tensile Tests after Restrained Recovery

Three (3) wires were gripped in an Instron UTM with a grip length of 2 in. and grip pressure of 500 psi. The wires were subjected to restrained shape recovery as described in section 3.3.1.2. The SMA wires were then allowed to cool down to room temperature while remaining restrained inside the grips. The tensile test was performed at a rate of 10 mm/min to rupture. The load and extensometer strain was measured.

3.3.1.5 Tensile Tests after Prestrain and Restrained Recovery

Three (3) wires were first loaded in a setup similar to the tensile tests and until the extensometer recorded a strain of 0.003 in./in. and another 3 wires were loaded up to a strain of 0.006 in./in.. The prestrained wires were heated to 170 °C following
procedure described in the previous sections, and subsequently allowed to cool down
to room temperature. The temperature and recovery stress were recorded using the
data acquisition system as discussed in previous sections. The tensile test was
conducted at a rate of 10 mm/min to specimen rupture. The load and extensometer
strain were recorded.

3.3.2 Bond Characterization Tests

3.3.2.1 Shape-recovery of SMA Wire

With the exception of BOB-AU-N group, all tests on bond characterization test
specimens were started by shape-recovery of SMA wire to induce post-tensioning to
the specimens. To induce shape recovery in SMA, the exposed ends of the wire were
connected to a power source. The temperature of the wire was measured with three
Type K thermocouples: two of them placed within the debonded region of SMA wire
(Figure 49), and one placed on one exposed end of the wire. During heating, the crack
opening displacement (COD) at the notch was measured with a COD clip gage affixed
to the specimen with knife edges (Figure 49).

3.3.2.2 Three-point Bending Test

Following shape-recovery of SMA wire, three-point bending tests were
conducted to assess the quality of bond between SMA wire and grout. The tests were
performed in a 22-kip MTS universal testing machine in closed-loop control under a
COD rate of 0.001 mm/min as the loading rate.
3.3.3 **Proof-of-concept Tests on Reinforced Concrete Girder**

3.3.3.1 **Cracking of Reinforced Concrete Girders**

Three (3) reinforced concrete girders, which were to be post-tensioned, were loaded in stages until they developed significant cracks and residual midspan deflection as shown in Figure 50. The girders were supported on 3/4th in. thick neoprene pads and the load was applied with a hydraulic system consisting of Enerpac pump and 100-kip actuator. The pump and the actuator were connected using hoses carrying hydraulic fluid. The load on the girder was distributed using a steel spreader beam (W 8 x 25) to achieve a four-point loading condition. SMA wires were placed in the grooves before cracking and left unanchored to prevent the development of additional strain while loading the girders.

The instrumentation was set up to collect the load, midspan deflection, support deflections and strain in the top fiber of concrete via a LabView based data acquisition system configured with National Instrument modules. The actual midspan deflection
was measured by deducting the support deflection from the measured midspan deflection. Once the cracking moment of the girders was exceeded, the cracks were traced and the corresponding loads were marked on the girders with color pens. The final crack widths after cracking the girders were measured with a concrete crack ruler.

![Test setup of cracking the reinforced concrete girders](image)

Figure 50. Test setup of cracking the reinforced concrete girders

### 3.3.3.2 Post-tensioning of Cracked Girders

The SMA wires were installed at the bottom cover of each of the three cracked girders and kept unbonded throughout the length of the girders. The number of SMA wires in each of the girders was kept as a test variable. Three of the widest cracks were instrumented with LVDTs to monitor the change in crack widths during post-tensioning (Figure 52). The strain at the top fiber of concrete on either side of the girders were instrumented with strain gages and will henceforth be referred to as SG1 to denote the strain on side 1 (to the negative z-axis) and SG2 to denote the strain on
side 2 (to the positive z-axis) as shown in section A-A (Figure 51b). Similarly, LVDTs recording midspan deflection on either side of the girder will be referred to as MD1 for midspan deflection of side 1 and MD2 for midspan deflection of side 2. The LVDTs recording support deflection will be referred to as SD1 and SD2 for the support in the negative x-direction and positive x-direction respectively. The deflection obtained by deducting the average of SD1 and SD2 from the MD1 and MD2 will henceforth be denoted as the effective midspan deflection of side 1 (EMD1) and effective midspan deflection of side 2 (EMD2). The SMA wires were instrumented with K-type thermocouple probes to measure the change in temperature of the wires. The thermocouples were denoted as T1, T2, T3, etc., where the number indicates the wire number on which they were attached. The wires were numbered starting from side 1 of the first row and proceeding in a clockwise direction up to side 1 of the second row (Figure 51b). The SMA wires were anchored at the ends of the girder using the same strand chucks which were used during the bond characterization tests. The strand chucks were made flush against the steel end plates to minimize prestressing losses as SMA wires underwent shape recovery.

The SMA wires were activated via Ohmic heating by using a power source with a rating of 100 V and 30 A (or 3000 W). The typical heating sequence of 8-SMA girder is shown in Figure 52b. At first, 4 wires in the first row (wires numbered 1-4) were heated together until the temperature of each wire exceeded 150 °C and subsequently allowed to cool down to ambient temperature. It was ensured that not more than four (4) wires were heated together at any point in time to avoid overloading the power source. The wires in the second row (wires numbered 5-8) were heated individually after providing electrical insulation by wrapping all surfaces of the
strand chucks with heat-resistant scotch tape to prevent contact with the steel end plate (Figure 51b). The heating sequence of the 10-SMA and the 12-SMA girders were also in a similar sequence where wires 1-4 are heated together first and the rest of the wires are heated individually by insulating them from each other.

The crack widths after post-tensioning were also measured and recorded manually using a concrete crack ruler.

![Figure 51](image1.png)

(a)  (b)

Figure 51. Heating procedure: (a) wires numbered 1-4; and (b) wires numbered 5-8 in 8-SMA girder

### 3.3.3.3 Four-point Flexural Tests

Four-point flexural tests were performed on the post-tensioned girders by loading them up to failure in four-point bending (Figure 52). The flexural tests were conducted to observe the change in yield and ultimate moment capacity, cracked stiffness and ductility of the girders after post-tensioning. The loading was carried out in stages: (1) the girder was first loaded beyond cracking moment and unloaded; (2) loaded beyond the yield moment and unloaded; and (3) loaded until the concrete crushed in compression. After every stage of loading, the cracks were traced and the
corresponding loads were marked on the girders with a marker pen. The load, midspan deflection, support deflections, strain at the top fiber of concrete and crack width data were recorded. The crack widths were manually measured using a concrete crack ruler after the failure of the girders.

Figure 52. Test setup of flexural test after post-tensioning the cracked girder: (a) Elevation; and (b) Cross-section view
3.4 Results and Discussion

3.4.1 Material Characterization Test

3.4.1.1 DSC Test on NiTiNb

The DSC test results (Figure 53) indicate that during heating the material from -60 °C to 300 °C, transformation from martensite to austenite ($A_s$) starts at 56 °C indicated by a change in slope of the heat, and is finished at 99 °C ($A_f$). During the cooling cycle no such spike in the heat flow is observed which indicates that the $M_s$ of the NiTiNb wire is below -30 °C. Hence, once the material is used to post-tension a damaged structure by transforming to austenite will not transform back to martensite and lose the recovery stress.

Figure 53. DSC heat flow vs. temperature of NiTiNb
3.4.1.2 Restrained Recovery Test on Martensite SMA Wires

Typical restrained recovery test results are shown in Figure 54. At the temperature is increased, the recovery stress is generated in the restrained SMA wire. The maximum recovery stress, corresponding to the maximum heating temperature, was approximately 73 ksi. The data also shows that no further recovery stress is generated beyond 150 °C. Once each wire reached at least 170 °C, the power source was turned off and wires were allowed to cool down to ambient temperature (23-25 °C) to determine the residual recovery stress. At room temperature, each wire retained approximately 92% of the maximum recovery stress (or 67 ksi). Excellent repeatability in test results was observed between the 3 replicates.

![Graph showing relationship between material temperature and generated recovery stress](image)

Figure 54. Restrained recovery test results showing relationship between material temperature and generated recovery stress
3.4.1.3 Tensile Test on Austenite SMA Wires

Figure 55 shows a typical stress-strain response of SMA wires in austenite phase. The SMA exhibits linear response characterized by an initial tangent modulus of elasticity of approximately 10,000 ksi until the material undergoes transformation to detwinned martensite. The transformation initiates at approximately 85 ksi and is completed at approximately 90 ksi. As expected, detwinned martensite phase demonstrates lower modulus of elasticity than austenite. The test ended by detwinned martensite rupture at approximately 145-150 ksi. The average strain at rupture was found to be 33.5%. Excellent repeatability in test results was observed between the 3 test replicates.

Figure 55. Stress vs. strain result of austenite SMA wire
3.4.1.4 Tensile Tests after Restrained Recovery

The SMA wire showed significant changes in the mechanical properties when subject to restrained recovery before the tensile test (Figure 56). The SMA wire has about 23% increase in the detwinning stress (up to 102 ksi) when it is subject to tensile test after inducing shape-recovery in a restrained condition. This may be due to the rearrangement of crystals when subject to the residual stress after cooling down. The detwinning start and finish stresses in the SMA wires subject to restrained shape recovery are very close. The ultimate stress remains the same in both cases, but the ultimate strain in the SMA wire subject to restrained recovery is lower because the wire had some residual stress (about 67 ksi) and zero strain at the beginning of the tensile test.

![Comparison of tensile tests of SMA wires subject to different conditions](image)

Figure 56. Comparison of tensile tests of SMA wires subject to different conditions
3.4.1.5 Tensile Tests after Prestrain and Restrained Recovery

The recovery stress increased linearly by up to 13% with the application of 0.006 in./in. prestrain before restrained shape-recovery (Figure 57) which is in agreement with previous research by Shin and Andrawes (Shin and Andrawes, 2010). The stress vs. strain response after the recovery shows that the elastic stiffness and detwinning stress increases up to 106 ksi (Figure 58). The increase in recovery stress results in the increase in the dewinning stress for the specimens with additional prestrain. The stiffness after detwinning during the strain hardening stage is also higher than the post-detwinning stiffness of the austenite SMA wire. The ultimate strain is lower than the austenite specimen and the specimen subject to the tensile test immediately after restrained recovery. This is because the wire was prestrained initially before the recovery.

Figure 57. Variation of recovery stress with prestrain
3.4.2 Bond Characterization Tests

3.4.2.1 Shape-recovery of SMA Wires

In the bond characterization tests where shape recovery-initiated post-tensioning was performed, temperature was monitored with three thermocouples: two of them placed on the portion of the SMA wire embedded in grout, and one on the exposed portion of the wire. Temperature measurements during shape recovery (Figure 59) clearly indicate that portion of SMA wire in contact with grout is heating at a lower rate than exposed portion of the wire. This is not surprising as the conductivity of concrete is on the order of magnitude of \( 1.0 \, W/mK \), while that of air is \( 0.026 \, W/mK \). Heat loss in SMA wire caused by its contact with grout meant that to reach a temperature approaching that required for complete shape recovery, the heating time had to be significantly increased over that reported in Figure 59; in each specimen the heating was stopped once the exposed portion of the SMA wire reached...
250 °C for safety reasons. It was observed that concrete surface temperature was significantly raised once the exposed portion of the wire started to reach 250 °C.

![Figure 59. Measured temperatures at different locations on the SMA wire](image)

The COD measurements during shape recovery (Figure 60) demonstrated reduction in COD as the temperature initially approached the maximum. The COD reduction was more significant in the specimens with anchored SMA wires, which indicates that the bond between the SMA wire and the grout is poor, and that anchorage plays an essential role in the transfer of post-tensioning force. However, even though the temperature was held somewhat constant upon reaching its maximum, the COD in all samples started to increase: this behavior is accredited to the development of a thermal gradient between the top and bottom surfaces of the specimen induced by heating of the wires. Such thermal gradient caused deflection of the specimen downwards which ultimately counteracted the compressive stress at the bottom of the specimen developed via post-tensioning. While the thermal gradient resulted in a change of the stress state at the notch tip from compression to tension in
the samples without anchorage, the same did not occur in the specimens with anchored SMA wires. It is expected that the thermal gradient-related crack opening would dissipate as the concrete at the bottom of the specimen cools down. However, the data collection was stopped as soon as the SMA wire temperature approached ambient temperature, so the current data does not provide evidence as to how COD would be affected upon concrete reaching its equilibrium temperature.

Figure 60. Typical change in COD with SMA shape recovery
3.4.2.2 Three-point Bending Tests

Results from typical three-point bending tests, which characterize the bond between the grout and the SMA wire, are shown in Figure 61. The relationship between applied load and COD remains linear until the crack above the notch forms and the load drops. The failure of the specimen is due to the unstable crack growth originating from the notch.

Strength, strain and slip data for all specimens is summarized in Table 2. Specimens without anchorage did not experience notable improvement in strength over control specimens. Interestingly, the specimens reinforced with austenite demonstrated slightly higher strength than that where specimens were post-tensioned with SMA wire. Post-tensioning of SMA wire may have introduced stresses that exceed the strength of the bond which would explain why BOB-PT-N group showed no increase in strength over control. Strain measurements in SMA wire at maximum load (Table 2) indicate very small strains at specimen failure; this is not surprising as given the rigidity of the test setup and relatively small depth of SMA wire in the cross-section. While no significant slip (Table 2) was observed during loading from 0 to maximum load, LVDTs clearly captured slip between maximum and “residual” load levels indicating bond failure immediately after the specimen strength was reached. It is not clear why there is an order of magnitude higher slip in BOB-AU-N when compared to BOB-PT-N.

It is apparent that presence of anchorage results in an increase of specimen strength over control group: this is thought to be due to the post-tensioning that introduces compressive stress field at the notch at the beginning of the test. The group of anchored specimens where SMA wire was bonded with grout prior to post-tensioning did not experience as great of an increase in strength as the rest of the
anchored group. A possible reason for this behavior is that post-tensioning force is distributed along the bonded region, so the effect of the post-tensioning on the stress state at the notch tip is not as drastic as it is in the unbonded anchored specimens. Strains in SMA wire at maximum load (Table 2) were small. However, an order of magnitude difference in strain measurements between bonded (BOB-PT-A and BOA-PT-A) and unbonded specimens (UB-PT-A) was observed: this is not surprising as strain is uniformly distributed along the unbonded portion of the SMA wire, while in bonded specimens localized maximum strain would develop below the notch at the measurement location. LVDT slip measurements (Table 2) in anchored specimens indicated no slip during the test most probably because most of the slip occurred during the transfer of post-tensioning.

![Typical load vs. COD behavior in bond characterization specimens](image)

Figure 61. Typical load vs. COD behavior in bond characterization specimens
Table 2. Summary of bond characterization test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Load (lbf)</th>
<th>Strain at max. load (µε)</th>
<th>Slip between 0 and max. load (in.)</th>
<th>Slip following load drop from max. to &quot;residual&quot; (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control-1</td>
<td>1583.5</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>Control-2</td>
<td>1889.1</td>
<td>68</td>
<td>0</td>
<td>0.089</td>
</tr>
<tr>
<td>BOB-AU-N-1</td>
<td>2185.2</td>
<td>374</td>
<td>0</td>
<td>0.036</td>
</tr>
<tr>
<td>BOB-AU-N-2</td>
<td>1860.9</td>
<td>68</td>
<td>0</td>
<td>0.089</td>
</tr>
<tr>
<td>BOB-PT-N-1</td>
<td>1657.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BOB-PT-N-2</td>
<td>1823.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BOB-PT-N-3</td>
<td>1797.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BOB-PT-A-1</td>
<td>1936.6</td>
<td>470</td>
<td>- lvdt used</td>
<td>- lvdt used</td>
</tr>
<tr>
<td>BOB-PT-A-2</td>
<td>2188.8</td>
<td>150</td>
<td>- lvdt used</td>
<td>- lvdt used</td>
</tr>
<tr>
<td>UB-PT-A-1</td>
<td>2806.5</td>
<td>59</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>UB-PT-A-2</td>
<td>2315.7</td>
<td>61</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>BOA-PT-A-1</td>
<td>2828.1</td>
<td>693</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Presence of anchorage has a clear influence on the post-cracking behavior of the bond characterization specimens (Figure 62). The specimens without anchorage experienced a sharp drop in load followed by a constant load level due to the friction between the SMA wire and grout. On the contrary, in the specimens with anchorage, the load picks up quickly after the initial crack formation until the test is interrupted upon COD gage reaching its maximum opening displacement of approximately 0.16 in. The increase in load is possible because anchorage allows SMA wire to carry tensile force which provides additional moment capacity to the specimen.

Once the specimens without anchorage were unloaded there was no significant change in the final crack width. Anchored specimens, however, exhibited an interesting behavior induced by superelastic recovery of SMA wire (Figure 62). Following unloading, COD returned to zero and a crack in the top of the specimen formed due to the tensile stress generated by superelastic recovery of SMA wire. Once
again, the observed behavior demonstrates the advantage of using anchorage in conjunction with NSM SMA reinforcement.

Evaluation of strain developed in SMA wire in an anchored specimen (Figure 63) shows a significant increase in strain following initial cracking, proving that following cracking SMA wire begins to provide the tensile capacity to the specimen. Upon unloading the strain recovers due to a superelastic effect and goes into compression which ultimately leads to the development of tensile stress at the top of the specimen and subsequent cracking (Figure 62 photograph no. 4).

The effectiveness of strand chucks as anchorage was also confirmed via post-mortem inspection of the anchored portion of SMA wires. Micrographs of the wire surface clearly show a well-developed pattern of bite marks (Figure 64) indicating that jaw segments were effective at preventing lateral sliding of the wire.
Figure 62. Typical load vs. COD behavior of anchored specimens (plotted here is specimen UB-PT-A-1): (1) initial crack formation; (2) maximum crack opening; (3) crack closure induced by superelastic recovery of SMA wire; (4) crack formation at the top of the specimen induced by superelastic recovery of SMA wire

Figure 63. Typical load vs. strain in SMA wire plot for anchored specimens (plotted here is specimen UB-PT-A-1); number marks 1-4 relate to photographs in Figure 64
3.4.3 Proof-of-concept tests on Reinforced Concrete Girders

3.4.3.1 Cracking of Reinforced Concrete Girders

Prior to post-tensioning, 3 reinforced concrete girders were cracked by loading them in stages. The extent of cracking the girders significantly influenced their behavior during post-tensioning. A Control girder without grooves or SMA wires was loaded up to failure before cracking the girders to be post-tensioned. The yield moment of the Control girder was found to be 44 kip-ft and the girders to be post-tensioned were cracked up to a moment which is expressed as a multiple of the yield moment of Control girder. The 8-SMA, 10-SMA and 12-SMA girders were cracked up to 0.89, 1.15 and 1.06 times the yield moment of the Control girder, respectively. A summary of the crack width ($w_{cr}$), stiffness of cracked girders ($K_{cr}$) and residual midspan deflection ($\delta$) is provided in Table 3. It is clearly seen that the 8-SMA girder was not loaded to the same extent as 10-SMA and 12-SMA girders and this affected the results of post-tensioning tests as will be seen in the subsequent sections.
Table 3. Summary of the results after cracking the girders to be post-tensioned

<table>
<thead>
<tr>
<th>Specimen</th>
<th>(w_{cr}) (µm)</th>
<th>(K_{cr}) (kip-ft/in.)</th>
<th>(\delta) (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8-SMA</td>
<td>180</td>
<td>305.5</td>
<td>0.075</td>
</tr>
<tr>
<td>10-SMA</td>
<td>640</td>
<td>299.4</td>
<td>0.175</td>
</tr>
<tr>
<td>12-SMA</td>
<td>640</td>
<td>269.6</td>
<td>0.115</td>
</tr>
</tbody>
</table>

3.4.3.2 Post-tensioning of Cracked Girders

Following cracking, the girders were post-tensioned by heating the SMA wires beyond 150 °C. The data collected during post-tensioning of the 12-SMA girder are shown as a variation of crack widths, residual midspan deflection and tensile strain on the top fiber of concrete with temperature (Figure 65). The results from the 12-SMA girder are representative of the results of all the post-tensioning tests. The results of the post-tensioning tests are summarized in Table 4. The temperature in wire number 12 (T12) was not recorded due to the malfunction of the data acquisition system. The temperature T2, T3, T4 and T11 did not reach 150 °C which implies that the peak recovery stress was not generated in these wires. The results of decreasing crack width, residual midspan deflection and the increase in tensile strain on the top of concrete could have been higher if all the wires reached were heated above 150 °C.

The number of SMA wires used for post-tensioning the cracked girders was calculated based on the stresses at the top surface of the concrete caused by the recovery of a SMA wire. The cracking stress and the stress at the top of concrete were added to provide the target stress with which the stress from the recovery of the single SMA wire (73 ksi) was compared to fix the total number of SMA wires required for post-tensioning. Post-tensioning losses were not accounted for in this calculation since...
this was used as a guide to determine the maximum number of SMA wires required to cause tension cracks in the top of concrete.

The crack widths decreased as each SMA wire was heated above 150 °C. The restrained shape-recovery of the SMA wires generated a compressive recovery force that was transferred to the concrete via end anchorage resulting in the crack closure. The maximum reduction in crack width ($\Delta w_{cr}$) were found to be up to 74% in the girder with the highest SMA reinforcement ratio (12-SMA). The final crack width (0.18 mm) conforms to the acceptable crack width limit for structures in dry air or protected by membranes (0.41 mm) and in humid and moist air or surrounded by soil (0.30 mm) as prescribed by ACI 224R-01 (ACI Committe 224, 1997).

The effective residual midspan deflection (EMD1 and EMD2) was calculated by subtracting the average of SD1 and SD2 from MD1 and MD2 respectively. The reduction of residual midspan deflection is due to the negative moment induced by the post-tensioning forces transferred via end anchorage. The maximum reduction in the residual midspan deflection ($\Delta \delta$) was 49% for 10-SMA. The reason for a greater reduction of residual deflection in 10-SMA than 12-SMA is the lower cracked stiffness and greater residual midspan deflection after cracking in 10-SMA due to the higher loading moment during cracking the 10-SMA.

The compressive stress generated at the bottom of the girders due to post-tensioning led to the development of tensile stresses on top of the girders causing an increase in the tensile strain ($\varepsilon$) on the top of concrete of up to $180 \times 10^{-6}$ in./in. in the 10-SMA girder. During the heating of some of the wires, the tensile strain in the top of concrete was found to reduce on one side of the girder while increasing on the other side. This behavior can be justified by the following reasoning; when a wire adjacent
to side 1 is heated (e.g., wire numbered 10), the SG2 decreases due to lateral bending. Similarly, when a wire adjacent to side 2 is heated (e.g., wire numbered 6), the SG1 decreases. Furthermore, imperfections in geometry during casting the girders may have led to an asymmetric position of strain gages which may have resulted in higher strain measurements on one side of the girders.

Table 4. Summary of post-tensioning test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Δw (µm)</th>
<th>Δδ (in.)</th>
<th>ε × 10⁻⁶ (in./in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8-SMA</td>
<td>90 (39%)</td>
<td>0.025 (34%)</td>
<td>77</td>
</tr>
<tr>
<td>10-SMA</td>
<td>270 (45%)</td>
<td>0.06 (49%)</td>
<td>180</td>
</tr>
<tr>
<td>12-SMA</td>
<td>370 (74%)</td>
<td>0.056 (48%)</td>
<td>121</td>
</tr>
</tbody>
</table>
Figure 65. Variation of strain on the top fiber of the middle of the girder, residual midspan deflection, and crack width with temperature of 12-SMA girder
3.4.3.3 Four-point Flexural Tests

Following post-tensioning, the girders were loaded in stages in a four-point bending setup until each specimen failed in flexure. The yield moment, ultimate moment capacity and cracked stiffness of the post-tensioned girders were compared to those of the Control girder (Figure 66). The cracked stiffness was calculated as: (i) the slope of the moment vs. deflection curve between initial cracking point and yield point in the Control girder; and (ii) the slope of the moment vs. deflection curve before yield point of the first stage of loading after post-tensioning, in the post-tensioned girder.

The moment vs. deflection curve of the girders clearly indicates up to 52% increase in the yield moment ($M_y$) of the post-tensioned girder over the Control girder. The yield moment of the post-tensioned girders was higher than the Control girder because the SMA wires carried tensile stresses in addition to the steel reinforcement. The ultimate moment capacity ($M_u$) increased by up to 45% due to the extra moment contribution of the SMA wires. The greatest increase in the ultimate moment capacity was observed in the 10-SMA girder despite the 12-SMA girder having the maximum number of SMA wires. The reason for this anomaly is that cracking and spalling of concrete had occurred near the supports of 12-SMA during loading (Figure 67). The cracked stiffness of the post-tensioned girders was compared to the cracked stiffness of the girder before post-tensioning and was found to increase by up to 31% due to the additional SMA reinforcement as well as the post-tensioning forces prior to loading. The maximum increase in strength and cracked stiffness were observed in the 10-SMA girder despite the 12-SMA girder having a higher SMA reinforcement ratio. This can be attributed to the premature failing at the support of the 12-SMA girder caused by concrete crushing due to compressive reaction forces at the support. The cracked stiffness of the Control girder (300 kip-ft/in.) was less than the cracked
stiffness of the 8-SMA (328 kip-ft/in.), 10-SMA (346 kip-ft/in.) and 12-SMA (337 kip-ft/in.) girder. The recovery of deflection after each stage of loading beyond the yield of the 12-SMA girder (about 60%) was greater than in the Control girder (53%). One of the reasons for this behavior is the superelastic property of the austenite SMA wires by which the post-tensioned girders can recover large deformations. The $M_s$ of SMA wires is below -150 °C and hence, at room temperature, the wires will be in the austenite phase and have superelastic property as seen previously in the results of the bond characterization tests (Figure 62). Due to the reasons mentioned above, the recovery of deflection after failure ($\Delta_\tau$) in the Control girder (0.79 in.) is lower than the 8-SMA (0.67 in.), 10-SMA (0.51 in.) and 12-SMA (0.59 in.). Beyond yield, the Control girder develops large residual deflections after unloading due to the inelastic strains in the yielded tensile steel. The ultimate deflections at failure of the post-tensioned girders did not change significantly over the Control girder. The deflection ductility coefficient ($\mu$) is defined by Eqn. (1):

$$\mu = \frac{\Delta_{max}}{\Delta_y}$$  

(1)

where the deflection at which the girder fails is $\Delta_{max}$; and the deflection at which the girder yields is $\Delta_y$. The deflection ductility coefficient for the Control girder was found to be 4.9 and those for the 8-SMA, 10-SMA and 12-SMA girders were 4.2, 2.5, and 2.7, respectively. The deflection ductility coefficient is greater in the Control girder. At concrete crushing, the addition of SMA wires reduces the strain in steel reinforcement and increases the depth of the neutral axis which leads to a reduction in section curvature. Hence, the ductility coefficient in the post-tensioned girders is reduced with respect to the Control girder.
Cracking was observed in all girders during each flexural test (Figure 68). The cracking pattern in the post-tensioned girders was different from the Control girder which was clearly displayed in the 12-SMA girder. The cracks in the 12-SMA girders were more closely spaced than the Control girders. The cracks in reinforced concrete originate when the concrete reaches its tensile strength and the cracks become deeper as the steel reinforcement continues to take tensile stresses. The increased tensile reinforcement in the form of SMA wires made the concrete between two consecutive cracks reach the tensile strength before the existing cracks could propagate further, resulting in closely spaced and distributed cracks in the post-tensioned girders. It was noted that the largest crack width in the Control girder was 5 mm, whereas the largest crack width in the post-tensioned girders was 3.05 mm. The cracks in post-tensioned girders are narrower because of the compressive forces acting on the girders as a result of post-tensioning prior to loading the girders. Another reason for the narrower crack widths in the post-tensioned girders might be the superelastic recovery of the SMA wires. On the other hand, the absence of post-tensioning force prior to loading and significant plastic strain in the steel reinforcement led to the formation of wider cracks in the Control girder. The effect of the SMA reinforcement ratio on the ductility and cracking behavior of concrete girders must be further studied to provide a more comprehensive and detailed insight on this issue.
Figure 66. Comparison of moment vs. deflection responses of the post-tensioned girders to the Control girder
Figure 67. Girder with grooves in the side cover showing: (a) cracking; and (b) spalling at support regions
Figure 68. Crack patterns of the reinforced concrete girders after failure. Numbers indicate load in kips
3.5 Effective Prestress and Stress at Ultimate Stage in SMA Wires

The experimental results in the previous chapter did not provide information on the partial loss of prestress and the stress in SMA wires at the ultimate condition due to instrumentation constraints. The stress in SMA at ultimate condition is important information since it governs the ultimate capacity and deflection of the girder. An analytical approach using classical mechanics have been employed in this section to calculate the partial loss in prestress due to elastic shortening of concrete, sag of SMA wires and friction between SMA wires and concrete.

3.5.1 Effective Prestress in Unbonded SMA Wires

The effective prestress in unbonded SMA wires during post-tensioning was back-calculated from the experimental data by assuming that the sum of the camber caused by the post-tensioning forces and the deflection caused by the self-weight is equal to the reduction in residual midspan deflection recorded from the experimental data using Eqn. (2):

\[ \frac{P_{\text{sma}} e L^2}{8 E_c I_{eff}} - \frac{5 w L^4}{384 E_c I_{eff}} = \Delta \delta \]  

where \( P_{\text{sma}} \) is the effective prestressing force in the SMA wires after immediate losses; \( e \) is the actual measured eccentricity of the SMA wires with respect to the neutral axis; \( E_c \) is the modulus of elasticity of concrete; \( I_{eff} \) is the effective moment of inertia of all the girders calculated by considering the reduced girder stiffness obtained from the experimental data; \( w \) is the self-weight of the girders assumed to be uniformly distributed; \( L \) is the length of the girder; and \( \Delta \delta \) is the reduction in the residual midspan deflection after the SMA wires had cooled down and the deflections in the girder had stabilized. The computed \( P_{\text{sma}} \) represents effective prestress as it
accounts for immediate losses in the effective prestress due to the elastic shortening of concrete, the initial sag of the SMA wires, and slip at the anchorage.

The above approach revealed that the effective prestress in the SMA wires is 49.3 ksi, 87 ksi and 66.7 ksi for the 8-SMA, 10-SMA and 12 SMA girders, respectively. Restrained recovery tests on SMA wires have previously indicated that maximum recovery stress of 67 ksi can be generated due to heat-induced restrained recovery. While computed effective prestress in 8-SMA and 12-SMA girders is within a realistic range (<72.5 ksi), the effective prestress of the 10-SMA girder appeared to be greater than the maximum recovery stress. This is, of course, not possible – the erroneous estimate of effective prestress might be due to the measurement errors during experiments combined with the underlying assumptions and approximations in Eqn. (2) like the actual eccentricity of the SMA wires and the effective moment of inertia of the cracked girders. The difference in the effective SMA between the 8-SMA and 10-SMA girders can be attributed to the manual installation of the wires in the girder resulting in different losses due to: (i) different sag of the wires; and (ii) different loss at the anchorage due to insufficient bearing when the wires start transforming. Even though the back-of-the-envelope calculations are rough estimates of the effective prestress they, nonetheless, suggest that the generated post-tensioning is quite significant. A numerical analysis of the system is necessary for accurate estimation of the effective prestress.

**3.5.2 Stress at Ultimate in Unbonded SMA Wires**

The stresses were calculated by considering the equilibrium of the section at ultimate under the compressive forces of concrete and the tensile force of the steel and the SMA reinforcement according to Eqns. (3) and (4):
\[0.85f'_c ab = A_s f_{sc} + A_{ps} f_{ps}\]  \hspace{1cm} (3)
\[A_{ps} f_{ps} \left( d_{ps} - \frac{a}{2} \right) = \Delta M\]  \hspace{1cm} (4)

where \(f'_c\) is the compressive strength of concrete; \(b\) is the width of the section; \(A_s\) is the area of tensile reinforcement; \(f_{sc}\) is the stress in tensile steel reinforcement calculated from the Control girder at ultimate (found to be 94.5 ksi); \(A_{ps}\) is the area of SMA reinforcement; and \(d_{ps}\) is the actual effective depth of the SMA reinforcement from top of concrete. Since, Eqn. (3) has two unknown quantities i.e. the depth of compressive stress block \((a)\) and the stress in SMA \((f_{ps})\), the moment contribution by the SMA wires was also equated to the difference in moment at ultimate of the Control girder and the post-tensioned girders. The ultimate moment capacity in the post-tensioned girders \((M_{pt})\) was compared to that of the Control girder \((M_c)\) and the additional moment \((\Delta M = M_{pt} - M_c)\) contribution due to the SMA wires was used to calculate \(a\) and \(f_{ps}\) by simultaneously solving Eqns. (3) and (4). The stresses in the strengthened girders at the ultimate were found to be 130.5 ksi, 120 ksi and 95.72 ksi for 8-SMA, 10-SMA and 12-SMA girders respectively. The increase of the stress in the SMA wires at the ultimate stage of the experiment is expected as the reinforcement ratio decreases.

3.6 Chapter Summary

The presented work has successfully completed the experimental investigation of heat-induced rapid post-tensioning in damaged reinforced concrete girders through the implementation of shape memory alloys (SMAs) wires. Near-surface mounted (NSM) unbonded NiTiNb SMA wires are used for post-tensioning of bridge girders to mitigate the conventionally experienced problems with anchorages, high construction costs, long lane closure times, long construction times, etc. A series of experiments
were performed to evaluate the proposed post-tensioning scheme: (1) DSC tests on SMA wires to determine the transformation temperatures; (2) tensile tests to determine the stress-strain behavior of SMA under several conditions; (3) restrained recovery tests to characterize the post-tensioning potential of SMA; (4) bond characterization tests on notched plain concrete specimens reinforced with SMA wires to test the bond between the SMA wires and cementitious grout; (5) proof-of-concept post-tensioning tests on damaged reinforced concrete girders to verify the feasibility of post-tensioning on structural-scale girders; (6) four-point flexural tests on post-tensioned reinforced concrete girders to observe the changes in yield moment, ultimate moment capacity, cracked stiffness and deflection ductility coefficient following post-tensioning. The main findings are summarized below:

- The DSC tests indicated that the ($A_s$) starts at 56 °C and is finished at 99 °C ($A_f$) whereas the $M_s$ is well below -30 °C indicating that the material is suitable for structural applications.

- The tensile properties of the SMA wires are highly variable and depend on initial prestrain and restraint. An initial prestrain of 0.6% increases the detwinning stress up to 106 ksi.

- Under restrained recovery test, 2.5%-prestrained SMA wires achieved maximum recovery stress of 73 ksi at 150 °C. Following cooling to ambient temperature, the wires retained about 92% of the maximum recovery stress, or 67 ksi.

- Post-tensioning of the notched bond characterization specimens via heat-induced shape-recovery of SMA was most successful in the
specimens with wires anchored with strand chucks rather than the wires bonded in grout.

- Bond characterization specimens post-tensioned with SMA wires demonstrated superelastic behavior following unloading which manifested in crack closure in the notched bond characterization specimens.

- Post-tensioning of cracked reinforced concrete girders resulted in up to 370 µm (74%) reduction in crack widths; the largest reduction was observed in the girder which was post-tensioned with the maximum number of SMA wires (12-SMA).

- The residual midspan deflection was recovered by up to 0.6 in. (49%) after the post-tensioning was completed. The post-cracking stiffness and residual midspan deflection after cracking influenced the extent of the recovery of residual midspan deflection.

- Up to 180x10⁻⁶ in./in. tensile strain developed on the top fiber of concrete which provided further evidence of the post-tensioning.

- Following post-tensioning, the cracked girders, an increase in the yield moment (up to 52%) and ultimate moment capacity (up to 45%) were observed over the Control girder.

- Grooves introduced in the side covers of girders to accommodate SMA wires should be grouted around the support region to avoid cracking and spalling of the concrete in those areas.

- The cracked stiffness of the post-tensioned girders increased by up to 31% over the cracked stiffness before post-tensioning.
Chapter 4

FINITE ELEMENT METHOD MODELING

This section involves the development of a non-linear finite element (FE) model which can simulate the flexural behavior of reinforced concrete girders post-tensioned with SMA wires. The FE model of the post-tensioned girder has been utilized to conduct a parametric study by varying different parameters such as damage during pre-loading, SMA reinforcement ratio and the interaction (bonded vs. unbonded) of the SMA wires with concrete and effective prestress. The limited number of structural-scale experiments was insufficient to study the effects of varying parameters on the behavior of the post-tensioned girders. The parametric study was conducted because to study the partial loss of prestress and stress at ultimate in the SMA wires which can be used to develop a design guideline by estimating the service moment and the ultimate capacity of the girder over the range of the studied variables.

4.1 Introduction

Several numerical studies were carried out on post-tensioning of reinforced concrete beams. Several 2-D and 3-D non-linear finite element method models were developed which can simulate different loading scenarios and failure modes using unconventional materials such as FRP, laminated glass, Fe-SMA. Previous FEM models for concrete members post-tensioned with fiber-reinforced polymer (FRP) composite or steel tendons assume undamaged (i.e., uncracked) girders which can affect the estimation of serviceability behavior of such members. The novelty of the herein developed model is that it allows simulation of the post-tensioning technique on a cracked (damaged) reinforced concrete girder using a damage mechanics framework by applying the heat-induced shape-recovery property of NSM unbonded NiTiNb
SMA wires. A non-linear elastic-plastic model of the internal steel reinforcement and NiTiNb SMA has been used in this study. The model can simulate the load-unload response and manifestation of damage in the girders during cracking, the partial loss of prestress and increase in camber during post-tensioning of the damaged girders, and the moment vs. deflection response of the girders after post-tensioning. The model is validated using the experimentally obtained data and a parametric study is conducted over a range of variables such as girder damage, SMA reinforcement ratio, interaction with concrete (bonded vs. unbonded), and effective prestress to study the strengthened system behavior.

4.2 Finite Element Method Model Development

4.2.1 Model Description

A reinforced concrete girder measuring 7.5 ft in length and having a cross-section of 9×16 in. was modeled as shown in Figure 69a, b. The internal steel reinforcement consisted of 3-#4 rebars as longitudinal reinforcement 2-#4 rebars as compression reinforcement. The effective depth of the tensile reinforcement was defined as 13.25 in. according to the measurements taken prior to the experiments on the reinforced concrete girders. The effective depth of compression reinforcement was 2.125 in. from the top of the girder. The transverse shear reinforcement consisted of #4 rebars spaced at 8 in. on-center. The rebars and stirrups were defined as truss elements with a circular cross-section with an area of 0.2 in². The internal steel reinforcement was embedded (the translation degrees of freedom of the internal steel reinforcement nodes are constrained) in the concrete girder and it was assumed that there is a perfect bond between the concrete and steel.
The SMA wires were modeled as truss elements having a circular cross-section with a diameter of 0.154 in. (3.92 mm). The SMA wires were tied to end steel bearing plates having a dimension of 9×4.5×1 in. The wires were placed in a single layer at the centroid of the two layers (14.875 in. from the top of concrete) as measured from the experiments. The interaction of the wires (bonded vs. unbonded) with the girder was kept as a test parameter.

The girder was subjected to a four-point flexural test; the loading was applied as a pressure load uniformly distributed over two 9×3×0.25 in. steel strips that were tied to the concrete so that there is no relative motion between them. Pinned and roller boundary conditions were provided at the supports. The supports and loading strips are placed on the girder as shown in Figure 69a. The post-tensioning effect was simulated by applying heat in the form of a body heat flux to the SMA wires which led to the contraction of the wires tied to the steel end bearing plates.
4.2.2 Material Models

4.2.2.1 Concrete

The concrete has been modeled using the concrete damage plasticity model (CDPM). CDPM is a continuum, plasticity-based, damage mechanics model which can effectively model quasi-brittle materials like concrete. Concrete behaves as a brittle material when the confining pressures are low. The brittle behavior of concrete
disappears when the confining pressure is sufficiently large to prevent crack propagation. Hence, the failure occurs by consolidation and collapse of the concrete microporous microstructure, leading to an overall ductile response. The plastic-damage concrete model uses a yield condition based on the yield function proposed by Lubliner et al. (Lubliner et al., 1989). The modifications proposed by Lee and Fenves (Lee and Fenves, 1998) to account for different evolution of strength under tension and compression are included. The effective stress of the yield condition reduces to the Drucker-Prager yield condition for a biaxial compression. The parameters generally used for modeling concrete in the CDP model is shown in Table 5 (Birtel and Mark, 2006) where $f_{b0}$ and $f_{c0}$ are the biaxial loading ratio and the constant K is the second stress invariant ratio.

The main failure mechanisms in the CDP model are cracking in tension and crushing in compression. This modeling technique is applied where the loading is monotonic, cyclic, or dynamic in nature. CDPM uses the concept of isotropic damaged elasticity along with isotropic tensile and compressive plasticity to model the inelastic portion of the material stress vs. strain relation. The simplified representation can capture the main features of the response of concrete.

Table 5. Summary of CDPM parameters

<table>
<thead>
<tr>
<th>Dilation Angle</th>
<th>Eccentricity</th>
<th>$f_{b0}/f_{c0}$</th>
<th>K</th>
<th>Viscosity parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>0.1</td>
<td>1.16</td>
<td>0.666</td>
<td>0.001</td>
</tr>
</tbody>
</table>
The stress vs. strain relationship for the concrete in compression was defined by the model prescribed by Collins and Mitchell (M.P Collins and Mitchell, 1991). The stress ($f_c$) is defined as a function of strain ($\varepsilon_c$) in Eqn. 5:

$$f_c(\varepsilon_{c,f}) = f'_c \frac{n(\frac{\varepsilon_{c,f}}{\varepsilon'_c})}{n - 1 + (\frac{\varepsilon_{c,f}}{\varepsilon'_c})^{nk}}$$

(5)

where, $f'_c$ is the compressive strength of concrete taken to be 5000 psi; $\varepsilon'_c$ is the strain at peak stress $f'_c$ and has a value equal to 0.00203; and n and k are constants taken to be 2.80 and 1.23 respectively (M.P Collins and Mitchell, 1991). The Poisson’s ratio of concrete was assumed to be 0.15 (Gopalakrishnan et al., 1969) for normal-weight concrete. The stress vs. strain relation of the concrete is shown in Figure 70. Under uniaxial compression the response is linear until the value of initial yield assumed at 2994 psi (by observing the stress up to which the relation is linear), in the plastic regime, the response is typically characterized by stress hardening followed by strain-softening beyond the ultimate stress.
Under uniaxial tension, the stress-strain response is assumed to follow a linear elastic relationship up to the cracking stress as denoted previously as \( f_t' \). The concrete undergoes micro-cracking beyond this point and the behavior of concrete in tension beyond the cracking stress is represented macroscopically with a softening stress-strain response, which induces strain localization in the concrete structure. The tensile stress \( (f_t) \) vs. strain relationship was considered to be linear up to the point of fracture after which the unloading model which was originally proposed by Thoronfeldt (Dere, 2017). The non-linear part beyond the fracture of the concrete is given by the Eqn. (6):

\[
f_t(\varepsilon) = f_t' \left( \frac{\varepsilon}{\varepsilon_t^\prime} \right)^{0.7+1000\varepsilon}
\]

(6)

where \( f_t' \) is the tensile strength of concrete which is calculated by the formula \( 7.5\sqrt{f_c^\prime} \) prescribed by the ACI 318-14 (ACI Committee 318, 2019); and \( \varepsilon_t^\prime \) is the strain at peak tensile stress defined as \( (f_t' / E_c) \) or, 0.00014 where \( E_c \) is the modulus of elasticity of concrete having a value of 3828 ksi (M.P Collins and Mitchell, 1991). The tensile

Figure 70. Stress vs. strain behavior of concrete in compression
stress vs. strain relation is shown in Figure 71. The unloading portion of the curve has been modified to make it steeper to obtain an equivalent area (fracture energy) under the stress-strain curve as in the model used for the finite element analysis of reinforced concrete by Chobbor et al. (Chobbor et al., 2013).

![Stress vs. strain behavior of concrete in tension](image)

Figure 71. Stress vs. strain behavior of concrete in tension

The tension and compression damage parameters are defined inside the CDPM model by inserting tables of the parameters and inelastic strain. The damage parameters in tension \( d_t \) and compression \( d_c \) are calculated according to the equations suggested by Birtel (Birtel and Mark, 2006) in Eqns. (7) and (8):

\[
d_c = 1 - \frac{f_c E_c^{-1}}{\varepsilon_c^{pl} \left( \frac{1}{b_c} - 1 \right) + f_c E_c^{-1}} \\
d_t = 1 - \frac{f_t E_c^{-1}}{\varepsilon_t^{pl} \left( \frac{1}{b_t} - 1 \right) + f_t E_c^{-1}}
\]
where $b_c = 0.7$ and $b_t = 0.4$ are constants that are obtained from pushover analysis conducted by Birtel (Birtel and Mark, 2006). The relationship between damage parameters and the stiffness during unloading shown in Figure 72.

![Figure 72](image)

Figure 72. Description of the effect of the damage parameters in: (a) compression; and (b) tension (Birtel and Mark, 2006)

### 4.2.2.2 Internal Steel Reinforcement

The internal steel reinforcement was modeled using a separate elastic and plastic material model in ABAQUS. The elastic part requires the elastic modulus and the Poisson’s ratio which is assumed to be 0.3. The plastic part requires the input of the true yield stress and plastic strain. The stress ($\sigma_s$) vs. strain ($\varepsilon_s$) relationship of the internal steel reinforcement used in this simulation is modeled according to the one proposed by Mander (Mander, 1983) as shown in Figure 73 according to Eqns. (9), (10), and (11):

\[
\sigma_s = \varepsilon_s E_s \quad \text{for} \quad 0 < \varepsilon_s < \varepsilon_y \tag{9}
\]

\[
\sigma_s = \sigma_y \quad \text{for} \quad \varepsilon_y < \varepsilon_s < \varepsilon_{sh} \tag{10}
\]
\[
\sigma_s = \sigma_{su} + (\sigma_y - \sigma_{su}) \left( \frac{\varepsilon_{su} - \varepsilon_s}{\varepsilon_{su} - \varepsilon_{sh}} \right)^P \quad \varepsilon_{sh} < \varepsilon_s < \varepsilon_{su} \tag{11}
\]
\[
P = E_{sh} \left( \frac{\varepsilon_{su} - \varepsilon_{sh}}{\sigma_{su} - \sigma_y} \right)
\]

where \( P \) is a constant obtained from Eqn. (11); \( \varepsilon_s \) is the yield strain; \( E_s \) is the elastic modulus assumed as 29000 ksi; \( E_{sh} \) is the strain hardening modulus assumed to be 1686 ksi for normal strength Grade 60 steel; the yield stress \( (\sigma_y) \) and ultimate strength \( (\sigma_{su}) \) of the steel is taken to be 69 ksi; and 108 ksi according to the information provided by the mill. The stress is assumed to increase linearly up to the yield stress as shown by Eqn. (9). Beyond the elastic limit, the yield plateau starts at \( \varepsilon_s \) and ends at \( \varepsilon_{sh} \) which was assumed to be 1\% for normal strength Grade 60 steel. The stress stays constant at \( \sigma_y \) in the plateau region as shown in equation (10). Beyond the yield plateau, the strain hardening follows the stress vs. strain relation shown in equation (11) up to the ultimate strain which has been assumed to be 10\% for normal strength Grade 60 steel.

The stress and strain are converted from nominal values to true values using Eqns. (12) and (13). The plastic strain is calculated using Eqn. (14):

\[
\sigma_{true} = \sigma_{engg} \left( 1 + \varepsilon_{engg} \right) \tag{12}
\]
\[
\varepsilon_{true} = \ln \left( 1 + \varepsilon_{engg} \right) \tag{13}
\]
\[
\varepsilon_{pl} = \ln \left( 1 + \varepsilon_{engg} \right) - \frac{\sigma_{true}}{E_s} \tag{14}
\]

where, \( \sigma_{engg} \) is the nominal stress; \( \varepsilon_{engg} \) is the nominal strain, \( \sigma_{true} \) is the true stress, \( \varepsilon_{true} \) is the true strain; and \( \varepsilon_{pl} \) is the plastic strain.
4.2.2.3 Shape-memory Alloy

The mechanical properties of the NiTiNb SMA wires were modeled using a similar elastic-plastic material model as was done for internal steel reinforcement. The experimentally obtained modulus of elasticity of 8008 ksi was assigned and a Poisson’s ratio of 0.33 was assumed. The engineering and true stress vs. strain curve of the SMA wires is shown in Figure 74. The plastic part of the material model (Figure 74 part 2 and 3) was specified by using the yield stress as 102 ksi as per the tensile test experiment (restrained recovery without any prestrain prior to recovery, section 3.4.1.4) and therein the plastic strains were calculated according to Eqns (9), (10), and (11). The shape-recovery was simulated by providing a negative thermal coefficient of expansion. The restrained recovery of strain led to the generation of recovery stress.
4.2.3 Mesh

A quadratic (20-node) 3D stress element (Figure 75a) with reduced integration, denoted as C3D20R in ABAQUS, was used to model concrete. This element was chosen over the linear (8 node) 3D element because of its lower stiffness and better performance under flexural deformations (Cook, 1981). The linear (8-node) element has a fictitious shear effect under flexure resulting in errors and hence, a higher formulation of element (20-node) was used in this case for increased accuracy. The steel rebars were modeled as quadratic (3-node) truss elements (denoted as T3D3). The SMA wires were modeled as (3-node) truss elements having an additional temperature degree of freedom (denoted as T3D3T) and was capable of simulating coupled temperature-displacement effects. The types of elements (according to the nomenclature described in this section) in the model are shown in Figure 75b.

The size of the concrete elements was determined by conducting a mesh sensitivity analysis by considering the convergence of midspan deflection and tensile...
stresses at the bottom of concrete when the simulation is run within the elastic range. The analytical closed-form solution was used to compute the deflection and tensile stress and the percent error with the simulated deflections for varying mesh sizes was studied to determine the element size. The percent error of midspan deflection decreases with the element size (Figure 76a). This indicates that a 6 in. mesh size would be sufficient to obtain an accurate solution. The percent error for tensile stress at the bottom of concrete decreases with the decrease in element size (Figure 76b). This implies that a mesh size of 2 in. is ideal for an accurate solution with respect to the stresses. Thus, it can be concluded that the tensile stress is underestimated while the displacement is overestimated by the quadratic 3D (20-node) elements. The convergence occurs at an element size of 2.5 in. beyond which the percent error does not change much. The error for the 3 in. element size was 1.36% for deflection and 5.84% for tensile stress at the bottom which is still quite small and this was chosen for the simulations. The larger element size would also reduce a significant amount of computational resources required to carry out the simulations.

Both the internal steel reinforcement and the SMA wires were assigned a mesh size of 3 in. The mesh size in the supports and loading strips was kept at 0.5 in. size since they had very small dimensions and were not points of interest in the simulations.

The simulations were run using 8 parallel central processing units (CPUs) having 8 cores each. The typical running time for a post-tensioned model complete with all the steps (cracking, unloading, post-tensioning and loading up to failure) takes between 5-8 hours to run depending on the interaction between SMA wires and concrete (unbonded vs. bonded) and the maximum time increment in all the steps are
not more than 0.01. The running time for the models increase significantly if the mesh is made finer and the maximum time increments are reduced.

Figure 75. The detailed schematic of: (a) C3D20R element; and (b) the meshed model
4.3 Scope of Parametric Study

The limited number of proof-of-concept experiments and constraints in instrumentation led to gaps in knowledge such as the effects of various parameters on the elastic losses of the prestress and the behavior of moment vs. deflection behavior of the post-tensioned girders. Hence, a parametric study was conducted to observe the effects of changing various parameters on the behavior of the model. The purpose of this study is to develop a trend to predict the partial loss of prestressing and the behavior of the girder at the ultimate condition. Several equations have been developed previously (Lee et al., 2014) to determine the partial loss of prestress and ultimate stress in SMA which depend on parameters like relative ratios of internal steel reinforcement and SMA, whether the SMA wires are bonded with concrete or kept unbonded, the effective prestress in the SMA wires, the compressive strength of the concrete, the amount of internal steel reinforcement. Previously many of these parameters have been studied in girders prestressed with FRP or steel (Unal, 2011). A

![Figure 76. Results of mesh-sensitivity analysis in terms of percentage error of: (a) simulated deflected; and (b) tensile stress at the bottom of concrete with respect to analytical calculations](image-url)
parametric study using the non-linear SMA as post-tensioning reinforcement and its interaction with steel and concrete is the focus of this study. The following parameters were varied: (i) damage during pre-loading (ii) the ratio of SMA reinforcement ratio \( \rho_{sma} = A_{sma}/bd_{sma} \) and tensile reinforcement ratio \( \rho = A_s/bd \) and; (iii) interaction with concrete (bonded and unbonded); (iv) the effective prestress. The parametric study has considered the geometrical properties of the beam, the internal steel reinforcement ratio, compressive strength of concrete and the eccentricity of the post-tensioning tendons to remain constant.

### 4.4 Model Validation

#### 4.4.1 Control Girder

Figure 77 shows the moment vs. deflection response of the Control girder which was loaded up to a moment of 58 kip-ft in a load controlled step. The loading was determined with reference to the load at which the concrete crushed in compression for the Control specimen during the experiment. The moment vs. displacement response of the girder is cut off when the strain of concrete in compression reached 0.005 in./in. which corresponded to \( d_c = 0.8 \) and that was considered as the point where the girder failed.

The uncracked stiffness of the simulated girder is much greater than that during the experiment. One of the possible reasons could be that the experimental test setup (modular strong-block testing system) was prone to uplifting during loading hence the deflections recorded were higher than the actual. Hence, the stiffness of the system was apparently reduced. In addition, the testing system was reassembled after previous use and that may have led to a reduced initial stiffness. The seating of the
testing system might not have been perfect which led to the initial discrepancy in
deflection but as the load increased, the system stabilized (Carroll and Benton, 2018).

The cracking stress \( f_r \) at the bottom of the section can be calculated from the
Eqn. (7) provided in the ACI 318-19 (ACI Committee 318, 2019). The cracking
moment was calculated from Eqn. (13):

\[
\begin{align*}
    f_r &= 7.5\sqrt{f_c'} = 532 \text{ psi} \\
    M_{cr} &= \frac{f_r(h - c_{tr})}{l_{tr}} = 18 \text{ kip - ft}
\end{align*}
\]  

where \( l_{tr} \) is the moment of inertia of the section; \( c_{tr} \) is the depth of neutral axis of the
transformed section; and \( h \) is the overall depth of the section. The cracking moment
observed in the experiment is approximately 20 kip-ft which is 5% more than the
cracking moment shown in the simulation 18.97 kip-ft. The analytical closed-form
solution indicates that the cracking moment is 18 kip-ft.

The stress distribution in the linear elastic region before cracking shows that
the neutral axis being at the center of the section as shown in Figure 78. The stress at
the bottom of the girder appears to reach maximum tensile stress of 569 psi before
cracking as can be seen from which varies from analytically obtained cracking stress
by 6.5%. This is similar to the error which was observed during mesh sensitivity
analysis. The error in the tensile stress is because of the low number of solution points
in the region of the cracking moment (e.g. the stress increases from 459 psi to 569 psi
in one single step). Reducing the maximum time increment increases the overall run
time of the simulation by a significant amount.

Beyond the cracking point, the stiffness decreases and becomes close to what
is observed in the experiment. This is because initially the modular block system in the
experiment may not have been seated properly due to reassembly and as the load
increased, the system stabilized and became close to the simulated stiffness. Beyond the yield point, the strain hardening behavior of the tensile reinforcement steel governs the stiffness of the concrete girder and hence the moment keeps on increasing along with the deflection. The yield moment of the simulation is within 0.8% of the experiment and the ultimate moment capacity of the simulation was less than the experiment by 7%. The moment capacity from the simulation is less than the analytical calculations (according to classical moment-curvature analysis) by 3.4%. The variability between the assumed steel model according to Mander and the actual behavior of the internal steel reinforcement during the post-yield phase of the experiment might have led to the difference in the ultimate moment capacity. The ultimate deflection in the simulation is less by 15%. This may be due to the fact the modular block was prone to uplifting during the experiments and hence the instrumentation may have recorded deflections which were higher than the actual.
Figure 77. Comparison of moment vs. deflection results of the experimental and the simulated Control girder

Figure 78. Normal stress distribution along the section of Control girder in the elastic region
The axial stress distribution in the rebar cage, as shown in Figure 79, clearly shows the tension in bottom internal steel reinforcement at the midspan since that is the region of the maximum flexural moment and some compressive stresses as well as tensile stress in the top reinforcement. This is because the neutral axis is above the top reinforcement at concrete crushing. By closely checking the values at the ultimate stage of loading it was observed that the stress in the tensile steel at midspan was about 90.1 ksi which is well over the yield stress of 69 ksi. The analytical calculations also show that the stress in steel at ultimate is close to 92.2 ksi which shows that the model is in very good agreement.

![Figure 79. Stress distribution in steel](image)

The damage in tension (Figure 80) starts developing from the time when the bottom fiber of the midspan reaches cracking stress. The progression of damage in the form of cracks increased as the girder was loaded beyond the yield and the depth of
the cracks started to increase and finally reached close to the top as seen from Figure 80. The damage pattern in the simulation corresponds well with the cracks observed during the experiments. Hence it is evident that the CDP model is successful in portraying the development of damage in the model.

Figure 80. Comparison of tensile damage in simulation and cracking patterns in the experiment

The unloading curve of the simulation, as shown in Figure 81, is steeper than the experiment and the residual deflection is higher. The reduction in stiffness during unloading in the simulation (about 34%) is similar to that of the experiment (about 44%). The difference in the stiffness degradation may be due to the absence of experimental data which led to the assumption of the constants in the calculation of the damage parameters while defining the concrete model in ABAQUS. Another possible reason for different unloading stiffness between the simulation and
experiment may be explained by the same reason attributed to the disparity in initial
deflections observed in case of the Control girder. The decrease in stiffness proves that
the damage mechanics are working properly to introduce damage in the model at the
end of unloading.

![Graph showing comparison of moment vs. deflection of load-unload results of experimental and simulated data of the Control girder](image)

Figure 81. Comparison of moment vs. deflection of load-unload results of experimental and simulated data of the Control girder

### 4.4.2 Post-tensioned Girders

The comparison of moment-deflection response between the experimental and simulation of the post-tensioned girder (both 8-SMA and 10-SMA) with unbonded and bonded SMA wires are shown in Figure 82a, b. The initial difference in stiffness can be attributed to the error during measuring displacement as discussed in the earlier sections (4.4.1).
The difference in yield moment is 8.1% (unbonded) and 4.2% (bonded) for 8-SMA, and 14.8% (unbonded) and 10.3% (bonded) for 10-SMA girders. The difference in ultimate moment capacity is 14.1% (unbonded) and 12% (bonded) for 8-SMA, and 14.3% (unbonded) and 9.7% (bonded) for 10-SMA girders. The differences in yield and ultimate moment between the model and experiment could be due to multiple reasons. During the experiments the wires were not perfectly unbonded throughout their length and there may have been significant friction between the wires and the concrete which may have caused partially bonded behavior. The comparison between the experiment and simulation would suggest that the wires are more towards the bonded condition. The effective depth of the internal steel reinforcement was variable as measured prior to the experiments. This may have caused an increase in the yield and ultimate moment capacity as can be seen in the experimental result of 10-SMA girder (Figure 82b). Another reason for the difference in the post-yield behavior of the models is the variability in the tensile properties of the SMA wires which are very sensitive to prestrain as demonstrated in section 3.4.1.5. A single layer of SMA wires was installed in the girder during pre-loading in the experiment. It is possible that the SMA wires may have been prestrained during the cracking stage before post-tensioning and this may have significantly changed the stress-strain behavior of the SMA wires post-shape-recovery. As per Figure 82b, the pre-loading yield point was higher in the experiment which further bolsters the possibility of the SMA wires carrying tensile stress during prestraining.

The control girder displayed some variability in the ultimate moment capacity due to the assumed internal steel reinforcement model. The additive effects of small errors from the internal steel reinforcement model and SMA models have resulted in
large errors in the validation model. The model compared quite well to the analytically calculated ultimate moment capacity (using classical moment-curvature analysis) of a bonded system with the same parameters (within 4%).

The stress in SMA wires at ultimate in the unbonded 8-SMA and 10-SMA models are 97.4 ksi and 88.4 ksi, respectively, and in the bonded models are 117 ksi and 114 ksi, respectively. The analytically calculated stress in section 3.5.2 is are higher (130 ksi and 120 ksi for 8-SMA and 10-SMA, respectively) than the unbonded and bonded models because the calculation has been carried out assuming the SMA responsible for the additional moment in the post-tensioned girder experiments. Since the experimentally obtained ultimate moments are higher than the simulations, the stress in SMA wires at ultimate is also higher. Furthermore, the bonded models having a smaller difference in stress in SMA wires at ultimate with the experiments suggest that the experimental interaction of SMA with concrete was closer to the bonded condition. As expected, the stress in SMA wires in the bonded models is higher than the unbonded models and the stress in SMA wires in the 8-SMA model is higher than the 10-SMA model due to a higher SMA ratio.

The reduction in residual midspan deflection due to the post-tensioning is 0.00496 in. and 0.00621 in. in the 8-SMA and 10-SMA, respectively. The higher reduction in midspan deflection in the model having a higher SMA reinforcement ratio is expected because of the greater post-tensioning force. The experimentally recorded values (section 3.3.3.2) are much higher than the simulation values. This may be because the deflections recorded during the experiments were not accurate.
4.5 Parametric Study Results and Discussion

4.5.1 Effects of Tensile Damage on Partial Loss of Prestress

The concrete girders are subject to different levels of damage to study the effect on the partial loss of prestress that occurs due to elastic shortening of concrete. The different average tensile damages are achieved by loading the models to different cracking moments corresponding to $0.51M_y$, $0.65M_y$, $0.79M_y$, and $1.03M_y$, where $M_y$ is the yield moment. The prestressing force was applied as a pressure loading at the end of the concrete. The centroid of the pressure loading coincided with the centroid of the SMA wires to make sure that the prestressing was applied at the correct location. The reason for using a pressure loading in place of the wires was that the wires were causing stress concentrations in the endplates and since the losses were small (<1 ksi) compared to the applied prestress, there was some error of close to 0.45 ksi in estimating the losses. The stress concentration can be avoided by refining the
mesh density of the steel end bearing plates and using elements with quadratic formulations. The partial loss of prestress (0.66 ksi) came close to the analytically calculated (0.567 ksi) value once an element with quadratic formulation was used. The accuracy is believed to further increase by reducing the mesh size which would be computationally inefficient. The wires were used to post-tension the models used for the other parametric studies since the magnitude of the elastic losses were very low (<1 ksi) and did not affect the moment vs. deflection behavior of the post-tensioned girders when loaded up to the ultimate stage.

The partial loss of prestress due to elastic shortening of concrete \( f_{es} \) was analytically calculated (excluding the self-weight of the girder) using the following Eqn. (15):

\[
f_{es} = n_{sma} \left( - \frac{P_t}{A_g} - \frac{P_t e_p}{I_g} \right)
\]  

(15)

where \( n_{sma} \) is the modular ratio of concrete and SMA; \( P_t \) is the applied post-tensioning force; \( A_g \) is the gross sectional area of the girder; \( e_p \) is the eccentricity of the SMA wires; and \( I_g \) is the gross moment of inertia. Figure 83a shows that the partial loss of prestress is not significant (<1 ksi) compared to the overall stress in the SMA wires after post-tensioning (67 ksi) or at the ultimate stage of loading the post-tensioned girders (about 100 ksi). The partial loss of the prestress increases significantly from the undamaged girder (0.8%) as the average tensile damage approaches yield (up to 1.5%). The partial loss of prestress for the undamaged girder (0.533 ksi) is very close to the analytically calculated value of 0.567 ksi (5.9% error).

Figure 83a shows the variation of partial loss of prestress with average tensile damage which demonstrates that partial loss of prestress increases with the increase in damage and becomes constant once the cracking moment approaches the yield.
moment. The camber due to the prestressing force is shown in Figure 83b and supports the conclusion that the partial loss of prestress is most significant between the cracking and yield moment. This can be explained with reference to the effective moment of inertia \( I_{\text{eff}} \) calculated using Eqn. (16) (ACI-318-19):

\[
I_{\text{eff}} = \left( \frac{M_{\text{cr}}}{M_a} \right)^3 I_g + \left( 1 - \left( \frac{M_{\text{cr}}}{M_a} \right)^3 \right) I_{\text{cr}}
\]

(16)

where \( M_{\text{cr}} \) is the cracking moment; \( M_a \) is the moment due to the applied pressure loading; and \( I_{\text{cr}} \) is the cracked moment of inertia. Figure 84 shows the variation of effective moment of inertia with the average tensile damage and the partial loss of prestress. The effective moment of inertia of the model (Figure 84a), calculated using Eqn. (16), is very close to the cracked moment of inertia when the cracking moment exceeds yield moment. The partial loss of prestress becomes constant once the effective moment of inertia approaches the cracked moment of inertia (Figure 84b). This is in agreement with Figure 83a, which demonstrated that the partial loss of prestress becomes constant as the average tensile damage approaches the damage corresponding to the yield moment. Once the effective moment of inertia approaches the cracked moment of inertia, the recovery of deflection also becomes constant.
Figure 83. Variation of: (a) partial loss of prestress; and (b) camber with average tensile damage

Figure 84. Variation of: (a) effective moment of inertia with average tensile damage; and (b) partial loss of prestress with effective moment of inertia
4.5.2 Ratio of SMA Wire and Tensile Reinforcement (Bonded and Unbonded)

This study was conducted on the undamaged girders by applying 67 ksi of prestress via heating of the SMA wires and subsequently loading the post-tensioned girders up to failure. Both bonded and unbonded conditions were considered for this study. The ratios \( \frac{\rho_{sma}}{\rho} \) considered for in this study are 0.139, 0.277, 0.554 and 0.834. These ratios were chosen based on the number of SMA wire reinforcements corresponding to 5, 10, 20 and 30 wires while the internal steel reinforcement was kept constant. The SMA reinforcement ratios for the parametric study has been chosen with reference to calculations (according to classical mechanics) to determine the maximum SMA reinforcement ratio which would result in tensile cracking on the top of concrete due to post-tensioning, assuming that the entire prestress was transferred without losses to an uncracked concrete section. The calculations showed that roughly 40 wires would be required to cause tensile cracking on top of concrete. Hence, the reinforcement ratios for the parametric study were chosen well below 40 wires.

Figure 85 shows the variation in the moment-deflection response of the post-tensioned girders with varying SMA reinforcement ratio for unbonded and bonded cases. The elastic portion of the simulation has the same for all the girders but the cracking moment increases with an increase in SMA ratio due to greater post-tensioning force. Beyond cracking, the girders with a greater SMA ratio have a higher stiffness for both the bonded and unbonded cases. Both the bonded case and unbonded models displayed an increase in the yield and ultimate moment capacity with the increase in the SMA ratio. The yield increased by up to 69% and 99%, and the ultimate moment capacity increased by up to 56% and 76% for the unbonded and bonded case, respectively. This is due to the increase in the tensile reinforcement in the form of SMA wires. Ultimate deflection decreased by up to 53% and 50%
respectively for the unbonded and bonded specimens, respectively. The increase in the
tensile reinforcement reduces the ductility of the girder since the strain in the
reinforcement is lower at the ultimate condition.

Figure 86 shows the variation of stress in the SMA wires at ultimate along the
length of the girder in bonded and unbonded models with different SMA
reinforcement ratios. The bonded girders display a higher yield moment and ultimate
moment capacity. This is due to the higher stress in the bonded SMA wires than the
unbonded wires (Figure 86). The yield and ultimate moment of the bonded models
increase by up to 11.1% over the unbonded models.

Figure 87 shows the variation of stress in reinforcement with the SMA to
internal steel reinforcement ratio in both bonded and unbonded conditions. The
ultimate deflection depends on the relative amount of SMA reinforcement with respect
to the internal steel reinforcement and also the stress in both the SMA wire and
internal steel reinforcement at the ultimate stage as seen from Figure 85. Since the
SMA ratio is low compared to the internal steel reinforcement, the deflection is
governed by the non-linear portion of the SMA stress-strain relationship (Figure 74
part 3). The strain in SMA in the bonded model is higher than the unbonded model
and hence, the deflection is greater in case of bonded models for any particular
reinforcement ratio (Figure 87). The ultimate deflection of the unbonded model having
$\rho_{sma}/\rho$ as 0.832 (almost an equal amount of SMA reinforcement compared to steel)
is greater than the bonded model because the SMA has not yielded although the steel
has exceeded the yield stress. Both the bonded and unbonded display higher stress in
SMA wires than internal steel reinforcement because the SMA wires are placed below
the internal steel reinforcement (Figure 87). As expected, the stress in both internal
steel reinforcement and SMA decreases with the increase in the reinforcement ratio in
the both the bonded and unbonded models (Figure 87a, b).

The difference in damage at ultimate stage between bonded and unbonded for
each reinforcement ratio has been shown in Table 6. As expected the damage
decreases with the increase in reinforcement ratio due to the reduction in ductility. All
the models, had a strain of above 0.005 in./in. in the internal steel reinforcement at the
ultimate stage. The bonded models had a more distributed damage than the unbonded
model due to higher ductility since the stress-strain relation of the SMA wires were
governed by the non-linear part of Figure 74 (part 2 and 3).

Figure 85. Moment vs. deflection for post-tensioned girders having varying ratios of
bonded and unbonded SMA to internal steel reinforcement ratios. Note: Full lines
indicate models post-tensioned with unbonded SMA wires and dashed lines indicate
model with bonded SMA wires
Figure 86. Distribution of stress along the length of wire for models with different SMA ratio. Note: Dashed lines show unbonded and full lines denote bonded models

Figure 87. Stress vs. reinforcement ratio for: (a) unbonded; and (b) bonded model
Table 6. Damage state for girders with different SMA reinforcement ratios

<table>
<thead>
<tr>
<th>$\rho_{sma}/\rho$</th>
<th>Bonded</th>
<th>Unbonded</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td><img src="image1.png" alt="Image" /></td>
<td><img src="image2.png" alt="Image" /></td>
</tr>
<tr>
<td>0.139</td>
<td><img src="image3.png" alt="Image" /></td>
<td><img src="image4.png" alt="Image" /></td>
</tr>
<tr>
<td>0.277</td>
<td><img src="image5.png" alt="Image" /></td>
<td><img src="image6.png" alt="Image" /></td>
</tr>
<tr>
<td>0.554</td>
<td><img src="image7.png" alt="Image" /></td>
<td><img src="image8.png" alt="Image" /></td>
</tr>
<tr>
<td>0.832</td>
<td><img src="image9.png" alt="Image" /></td>
<td><img src="image10.png" alt="Image" /></td>
</tr>
</tbody>
</table>

4.5.3 Effective Prestress

Effective prestress in SMA wires is considered to be the stress that is still remaining in the prestressing tendons after the immediate losses (elastic shortening of concrete, friction loss, sag loss and loss at anchorage). The effective prestress in the SMA wires have been considered to be 80% and 90% of the applied prestress (67 ksi) and the wires were attached to the girders in an unbonded condition. The reduction in effective prestress has been chosen with reference to the analytical calculations in section 3.5.1 where the effective prestress has been estimated using the experimentally obtained data of the recovery of residual midspan deflection. The maximum loss of prestress calculated from the experimental deflections were within 25% of the applied prestress and hence the losses up to 20% have been considered for this parametric study. The unbonded models were involved in this parametric study because the experimental program and intended application of the post-tensioning scheme involves unbonded SMA wires.
Figure 88 shows the difference in moment-deflection response of the models when subject to reduced prestress stress. The decrease in yield stress for all reinforcement ratios with the drop in prestress has been shown in Figure 89. The decrease in yield stress decreases by 4.5%, 5.5% and 7.8% for a decrease in effective prestress to 80% for $\rho_{sma}/\rho$ of 0.139, 0.277 and 0.554, respectively (Figure 89). The cracking moment and the post-cracking stiffness is increased with the increase in effective prestress for all the models. The ultimate moment capacity has not changed for any of the reinforcement ratios since the prestress does not affect the equilibrium of the model. As expected the ultimate deflection increases with the decrease in prestress for all the reinforcement ratios. The ultimate deflection decreases with an increase in the effective prestress by up to 15% for the model with $\rho_{sma}/\rho$ of 0.554 because the stress in SMA has exceeded yield and the loss in effective prestress lowers the strain in SMA significantly to govern the behavior of the girder.
Figure 88. Effect of varying prestress on models with: (a) $\rho_{sma}/\rho = 0.139$; (b) $\rho_{sma}/\rho = 0.277$; and (c) $\rho_{sma}/\rho = 0.554$
Figure 89. Variation of yield moment with effective prestress for different SMA reinforcement ratios

4.6 Estimation of Yield Moment and Ultimate Moment Capacity

The goal of the parametric study was to develop a design methodology to predict the stress in the internal steel reinforcement and SMA wires at service level and ultimate condition of the girders which, in turn, would govern the yield moment and the ultimate moment capacities of the girder. A matrix can be prepared using the results of the parametric study (increase or decrease in the stress in internal steel reinforcement and stress of SMA reinforcement at ultimate) to predict the increase or decrease in the yield moment, ultimate capacity and ultimate deflection over the range of the varied parameters. This has been left as a future scope for research. Further parametric studies on the geometrical properties of the beam, the internal steel reinforcement ratio, the compressive strength of concrete and the eccentricity of the post-tensioning tendons can provide a more refined matrix to predict the yield and ultimate moment capacities of the girder.
4.7 Chapter Summary

A finite element method model of post-tensioning cracked reinforced concrete girders was developed by implementing a damage mechanics framework in a commercial general finite element modeling software ABAQUS. A Control model without the SMA wires was developed and validated against the experimental data. The post-tensioned model was developed by adding damage to the control model and subsequently post-tensioning the Control model by adding the SMA wires. A parametric study was conducted to study the effect of variables such as girder damage, SMA reinforcement ratio, interaction with concrete and effective prestress on the behavior of the post-tensioned girders. The changes in the yield moment, ultimate moment capacity and deflection were studied from the results of the parametric study. Below is a summary of the main findings:

- The yield moment and the ultimate moment capacity of the Control girder model is within 0.8% and 7% of the experimental results. The small errors from the assumption of the concrete and steel model caused the errors in estimating the ultimate moment capacity.

- The ultimate moment of the unbonded post-tensioned is approximately 14% less than the experimental values. The small errors from the assumption of concrete and steel models and the variability in the SMA model led to the cumulative large error in the case of the post-tensioned girders.

- The partial loss of prestress is very small (within 1.5% of the applied prestress) and becomes constant as the effective moment of inertia of the damaged model approaches the cracked moment of inertia.
• Addition of a modest amount of SMA reinforcement ratio (up to 3 times) increase in the reinforcement ratio of SMA wire can increase the ultimate moment by up to 99% for the bonded model. A decrease in effective prestress to 80% reduces the yield moment and by up to 7.8%.

• Contrary to expectation, the ultimate deflection of the girders lightly reinforced with bonded SMA wires was higher than the ultimate deflection of the girders with unbonded SMA wires. This was because of the complex non-linear behavior of the SMA wires and their interaction with the internal steel reinforcement beyond yield.
Chapter 5

CONCLUSION AND RECOMMENDATION

5.1 Summary and Conclusions

Reinforced concrete girders can face serviceability issues due to excessive cracking and deflections. Traditional external post-tensioning techniques for damaged reinforced concrete girders involve long construction times, use of heavy hydraulic equipment and issues with anchorage. A novel repair technique utilizing heat-activated near-surface mounted (NSM) unbonded NiTiNb shape-memory alloy (SMA) wires was evaluated in this work as a means of alleviating the aforementioned problems with traditional post-tensioning techniques. The proposed method relies on the intrinsic ability of the SMA to recover seemingly permanent deformation upon activation via heating. The herein presented study experimentally and numerically investigated the proposed repair technique.

The research program consisted of: (i) experimental characterization of the material properties of the SMA wires; (ii) proof-of-concept experiments on damaged structural-scale (7.5 ft×9 in.×16 in.) reinforced concrete girders post-tensioned with NSM unbonded SMA wires; (iii) the development and validation of a finite element model which can simulate the behavior of the strengthening system; and (iv) parametric study evaluating the effect of multiple variables on the service and ultimate behavior.

Materials characterization of SMA wires revealed the material’s tensile properties, recovery stress potential, bonding characteristics to cementitious materials, and the viable service temperature range. The obtained material properties were used to design the structural-scale specimens which were first precracked to simulate girder
damage and, subsequently, post-tensioned and tested to ultimate failure in a four-point bending test setup.

Finite element models were developed in a damage mechanics framework to simulate the behavior of repaired reinforced concrete girders. Following model validation against the proof-of-concept experimental data, a parametric study was conducted to investigate the effect of: (i) average tensile damage on the partial loss of prestress; (ii) SMA reinforcement ratio and interaction with concrete (bonded vs. unbonded) on the yield moment, ultimate moment capacity and deflection; (iii) effective prestress on the yield moment and ductility. Following are the major findings of this research:

- SMA wires can generate 67 ksi post-tensioning stress when heated above the transformation temperature. The tensile properties of the SMA wires are significantly affected by initial prestrain.
- The bond between SMA wires and rapid-setting grouts is poor; thus, end anchorage is necessary to adequately transfer the post-tensioning forces to concrete.
- Within the experimentally evaluated range of SMA reinforcement ratios (from 0 to 0.17%), the proposed post-tensioning technique can reduce the crack widths and the residual midspan deflection of the cracked structural-scale girders by up to 370 μm (74%) and 0.06 in. (49%), respectively, and increase ultimate strength the post-tensioned girders by up to 45%.
- The model was in reasonable agreement with the experimental data could provide conservative estimates of yield and ultimate moment.
which were approximately 14% lower than experimental data. The model error (up to 14%) is due to the cumulative addition of errors from the assumptions made regarding concrete, steel, and SMA material models.

- Parametric study demonstrated that the partial loss of prestress due to the elastic shortening of concrete in an undamaged girder is very small (approximately 0.8%), but increases with girder tensile damage (i.e., cracking) up to approximately 1.5%. The yield moment of the post-tensioned girders increases with rising effective prestress, but is accompanied by slight reduction in ductility. Comparison of girders with bonded vs. unbonded SMA revealed that girders with bonded SMA reinforcement can have in greater ductility than girders with unbonded SAM due to the complex non-linear behavior of the SMA.

5.2 Recommendations and future research

The proposed post-tensioning technique shows promise for success but the construction methodology for installation of the wires in the damaged girders needs to be refined. The procedure should be such that it does not involve using huge concrete saws for performing overhead cuts in the concrete. The NSM installation can be replaced by external attachment of the SMA wires to concrete. Simple equipment such as open-flame gas torch can be used to eliminate the requirement for an unreasonably large power source to heat the SMA wires during the post-tensioning.

The presented results suggest that anchorage must be provided to ensure proper transfer of post-tensioning force. The author also recommends heat-induced post-tensioning to be conducted on unbonded SMA wires to avoid difficulties with heat
loss to grout and concrete. Since the bond between SMA wires and grout is poor, subsequent filling of the groove with grout is optional. In fact, it may be beneficial to leave the wires exposed to facilitate future structural health monitoring, inspections, and maintenance.

Fatigue resistance of the SMA wires, particularly in the anchorage regions, needs to be evaluated to ensure longevity of the repair system under cyclic loading in bridges and other structures. Additional work is required to determine the durability properties of NiTiNb SMA when exposed to the aggressive environments which are typically experienced by bridges in northern regions and in close proximity to coastal areas. This includes investigating the durability properties of the wires when exposed to a humid and corrosive environments for extended periods of time under sustained loading.

The finite element model can be further improved by introducing a bond-slip relation between the internal steel reinforcement and concrete using discrete 3D reinforcement bars. The friction between SMA wires and concrete, SMA wires sag and slippage at anchorage can be introduced to predict a more realistic behavior of the post-tensioned girders. Effect of several parameters on service and ultimate behavior – such as the compressive strength of concrete, dimensions of the girder, boundary conditions, internal steel reinforcement ratio – should be evaluated to develop design equations for the repair system.

SMAs also have the potential of being applied in structural health monitoring where the change in resistance of the SMA wires with strain may be used as a distributed sensor. Combining SMA with other advanced materials (e.g., FRP composites) may lead to further enhancement of the structural performance of repaired
structures. In conclusion, SMAs are an innovative construction material which has the potential to be applied in multiple facets of structural health monitoring, repair and strengthening.
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