STRUCTURAL BEHAVIOR AND MODELING OF HIGH-PERFORMANCE FIBER-REINFORCED CEMENTITIOUS COMPOSITES FOR EARTHQUAKE-RESISTANT DESIGN

BY

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DISsertation

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ABSTRACT

For earthquake-resistant design, adequate concrete confinement is vital for a ductile structural response and for providing a stable energy dissipating mechanism. Since concrete materials generally exhibit quasi-brittle failure and a low tensile strength, designers of traditional reinforced concrete often specify extensive transverse reinforcement with thorough detailing to ensure that appropriate confinement to the concrete and the longitudinal reinforcing bars is provided. This approach often results in such a large amount of reinforcing steel that construction of the design can be congested, costly, and even impractical. This effect is particularly pronounced in critical shear and/or moment regions of structural concrete coupling beams and pile-wharf connections, as well as in plastic hinge regions of reinforced concrete beams, columns, and structural walls. To address this problem, the development and modeling of High Performance Fiber-Reinforced Cementitious Composites (HPFRCC) for use in key shear and/or moment regions of damage-critical structural concrete elements has been investigated.

An experimental program was conducted to further understand the behavior of HPFRCC under general multi-axial stress states, such as would be expected at various key locations in a damage-critical structural component. Concrete plate specimens comprising mixes containing from one to two percent volume fraction of hooked steel fibers and Spectra (polyethylene) fibers were tested. After exploration of these different fiber types and volume fractions, a 1.5% volume fraction of hooked steel fibers was selected as the concrete mix for more comprehensive examination, based in part on a study to create self-consolidating fiber-reinforced concrete. The stress-strain behavior of the various HPFRCC mixes was examined, and biaxial failure envelopes have been developed. The plate specimen tests showed that HPFRCC exhibits a confined compressive behavior with a significantly increased damage tolerance and deformation capacity.
Using the knowledge and behavioral trends gained from the laboratory tests of HPFRCC materials, it was possible to create a phenomenological HPFRCC finite element material model, with a smeared crack representation, that was calibrated to the experimental data. In addition to small-scale structural / material testing and modeling, the same HPFRCC hooked steel fiber mix was tested in large-scale coupling beam component tests by project partners at the University of Michigan. After completion of these large-scale tests, the material model was validated at the structural component level with their experimental coupling beam results.

Finally, a full-scale structural concrete pile-wharf connection was tested at the University of Illinois, and the behavior of this damage-critical component was thoroughly analyzed. The HPFRCC model was then implemented into the pile-wharf connection application. Overall, it was found that the increase in structural component damage tolerance through a ductile response obtained by the tensile strain-hardening and confined compressive behavior from the use of HPFRCC makes it a potentially viable solution as a replacement for some steel confinement reinforcement and as an additional shear resistance mechanism. With the development of an HPFRCC modeling tool, insight into the levels of damage experienced by structural elements can inform performance-based design decisions regarding the use of HPFRCC in critical structural components.
To the best mother a son could possibly have

Arlene Needle Foltz

1951-2011
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CHAPTER 1. INTRODUCTION

1.1 Problem

Typical concrete materials are characterized by quasi-brittle failures and a low tensile strength. In practice, reinforcing steel is added to provide a concrete member with the requisite tensile and confined compression capacities to achieve the desired strength level; however, many structural applications then require such a large amount of reinforcing steel that physical construction of the design can be congested, costly, and even impractical. For earthquake-resistant design, adequate concrete confinement is vital for a ductile structural response and for providing a stable energy dissipating mechanism. Thus, designers often specify extensive transverse reinforcement with thorough detailing to ensure that appropriate confinement to the concrete and the longitudinal bars is provided. This effect is especially pronounced in critical shear and/or moment regions, such as coupling beams, beam-column connections, and plastic hinge regions of beams, columns, and structural walls. In fact, in short-span (shear-critical) coupling beams, densely confined diagonal reinforcement cages are even required by the ACI code. The construction of such reinforcement layouts is labor intensive, which results in increased cost. High-performance fiber-reinforced cementitious composites (HPFRCCs) could potentially alleviate this problem, and others, due to its inherent ability to confine the concrete and to reduce the amount of transverse reinforcement required, while ensuring a ductile failure mechanism. Researchers have explored the use of HPFRCC in beam-columns joints, squat walls, coupling beams, and flexural members subject to high shear stress reversals (Parra-Montesinos, 2005). All of these efforts have explored the use of HPFRCC with either reduced or eliminated shear reinforcement. Additionally, Kesner and Billington (2004) explored the use of
HPFRCC for the seismic upgrading of deficient framed structures with the use of lightly reinforced precast HPFRCC infill panels. Further, Chao et al. (2009) investigated the performance of the bond behavior of reinforcing bars in HPFRCC, and found that HPFRCC can provide additional benefits to the bond of typical reinforcement. The seismic performance of structures is of paramount importance to design engineers, and the overall behavior of HPFRCC, including not only the ultimate strength capacity, but also the deformation capacity and the resistance to cover spalling of HPFRCC members, should be considered. These studies, as well as others to be discussed later, have shown the potential of HPFRCC to be a useful material to enhance the strength, stiffness, ductility, damage tolerance, and energy dissipation of structural systems, while reducing reinforcement requirements.

1.2 High-Performance Fiber-Reinforced Cementitious Composite Solution

Plain mortar and concrete without the addition of steel reinforcement lose their ability to carry load almost immediately after the formation of an initial crack. The addition of fibers into traditional fiber-reinforced concrete (FRC) may make the failures somewhat less brittle, but FRC typically does not increase the tensile strength or maintain significant residual capacity at strains beyond initial cracking. Tensile damage to FRC is localized cracking, and the tension softening deformation behavior after cracking classifies FRC as a quasi-brittle material (Fischer, 2004). As such, FRC is typically used in crack control applications, such as the control of plastic shrinkage cracking of concrete floors and thinning the design thickness of slabs on grade. A “high-performance fiber-reinforced cementitious composite” (HPFRCC) in the engineering community refers to concrete that experiences a pseudo-strain hardening behavior after initial cracking, and thus the ultimate strength is higher than the first cracking strength. (The term pseudo-strain hardening is used to differentiate between this behavior and the actual strain-
hardening of metals due to dislocation micromechanics.) The overall spectrum of fiber-reinforced concrete can be subdivided into several categories. If a multiple cracking behavior occurs, then the material is termed “tensile strain-hardening,” and if the damage localizes at a single crack, then the material is “tensile strain-softening.” The tension softening material can then be subdivided into deflection hardening and deflection softening. The tension softening / deflection softening materials can range in application from the control of plastic shrinkage cracking in concrete to higher end uses such as slabs on grade. The tension softening / deflection hardening materials are useful in particular structural applications where bending prevails. All the tension hardening materials are also deflection hardening (Naaman et al., 2007), and the HPFRCC used in the current study is a tension hardening / deflection hardening material. Figure 1-1 shows the suggested classification of FRC composites based on their tensile response. It provides additional information about the typical shapes and conditions to achieve such response in terms of the critical volume fraction of fibers. Traditional FRC falls into the tension / deflection softening category within the FRC spectrum. It should be noted that all of the FRC materials may improve the damage tolerance of a particular concrete structural application; however, traditional FRC materials would not perform as well in the damage-critical structural components, so the focus of this study is on the performance of HPFRCC.
Figure 1-1. Suggested Classification of FRC Composites from Naaman et al. (2007)

Figure 1-2 illustrates the additional typical tensile stress-strain responses of conventional FRC and HPFRCC. In Figure 1-2, $\varepsilon_{cc}$ and $\sigma_{cc}$ are the composite tensile strain and strength at first cracking, respectively, and $\varepsilon_{pc}$ and $\sigma_{pc}$ correspond to the composite peak post-cracking tensile strain and strength, respectively. The two materials have a nearly identical uncracked response during the initial ascending portion (OA). However, after initial cracking, the HPFRCC material shows a hardening portion (AB) up to relatively high strains. The strain capacity at peak stress, $\varepsilon_{pc}$, is typically equal to or exceeding 0.5% strain, which is more than twice the yield strain of standard reinforcing bars (Naaman & Reinhardt, 2006). It can be seen that regular FRC exhibits
a rapid strength decay (AB), termed strain-softening. After first cracking, multiple cracks develop throughout the HPFRCC, rather than a single localized crack in regular FRC. This portion of the HPFRCC stress-strain response (AB) is the pseudo strain-hardening behavior. After the peak stress and strain are achieved, the HPFRCC damage localizes at a single crack, and the strain-softening portions of the stress-strain curves (BC) are similar for both FRC and HPFRCC.

Figure 1-2. Comparison of Typical Stress-Strain Response in Tension of HPFRCC with Conventional FRCC (Naaman & Reinhardt, 2003)
Naaman and Reinhardt (1996) identified the key parameters to ensure a multiple cracking condition where the maximum post-cracking stress is larger than the stress at first cracking, that is $\sigma_{pc} > \sigma_{cc}$. The key parameters identified were the volume fraction of the fibers, the fiber aspect ratio (fiber length / fiber diameter), the tensile strength of the matrix, the average bond strength at the fiber matrix interface, and several coefficients relating to the orientation, efficiency, and pullout of the fibers. It can be derived that for a given fiber type, matrix, and other assumed constant conditions, a critical value of the volume fraction of the fiber reinforcement needed to guarantee pseudo-strain hardening and multiple cracking behavior can be calculated. Using composite mechanics, Naaman (1987) derived the closed-form solution in Equation (1.1) to calculate the critical volume fraction of fibers to achieve strain-hardening behavior in tension.

$$V_f \geq (V_f)_{critical} = \frac{1}{1 + \frac{\tau L}{\sigma_{mu} d} (\lambda_1 \lambda_2 \lambda_3 - \alpha_1 \alpha_2)}$$

(1.1)

Where:

- $V_f$ = Volume fraction of fibers
- $L$ = Fiber length
- $d$ = Fiber diameter
- $\frac{L}{d}$ = Fiber aspect ratio
- $\sigma_{mu}$ = Tensile strength of the matrix
- $\tau$ = Average bond strength at the fiber matrix interface
- $\alpha_1$ = Coefficient representing the fraction of bond mobilized at first matrix
cracking

\[ \alpha_2 = \text{Efficiency factor of fiber orientation in the uncracked state of the composite} \]

\[ \lambda_1 = \text{Expected pull-out length ratio (this is the average fiber length used to resist the pull out through bond stress and is equal to } \frac{1}{4} \text{ from probability considerations)} \]

\[ \lambda_2 = \text{Efficiency factor of orientation in the cracked state} \]

\[ \lambda_3 = \text{Group reduction factor associated with the number of fibers pulling-out per unit area (or density of fiber crossings)} \]

For small values of the fiber volume fraction, \( 1 - V_f \approx 1 \). Thus, Equation (1.1) can be rewritten as follows:

\[
\frac{\tau}{\sigma_{mu}} V_f \frac{L}{d} \geq \frac{1}{\lambda_1 \lambda_2 \lambda_3 - \alpha_1 \alpha_2} \quad (1.2)
\]

Equation (1.2) is a simple way to demonstrate the direct influence of the independent variables leading to the development of multiple cracking. Assuming constants for the coefficients addressing the selected fiber type and orientation (right-hand side of Equation (1.2)), it can be seen that the aspect ratio of the fiber and the ratio of bond strength to the matrix tensile strength are at least as influential as the volume fraction of the reinforcement to ensure strain-hardening behavior (Naaman & Reinhardt, 1996).

In addition to the enhancement of the tensile behavior, the compressive behavior of concrete is markedly enhanced with the addition of fibers as well, due to the passive confinement
afforded by the fibers. Both the compressive strength and deformation capacities of concrete can be improved with the use of HPFRCC. In the design of reinforced concrete structures subjected to seismic loading, large displacement reversals may be imposed on the structures. Thus, detailing of regions that could be subjected to large inelastic deformations is critical. Since regular reinforced concrete is brittle, extensive reinforcement detailing is required in regions that are more susceptible to damage, such as column bases, beam ends, hinging regions of structural walls, and coupling beams in structural wall systems. The use of HPFRCC in these final two examples is a major focus of this research.

Interest in the development of high-performance fiber-reinforced cementitious composites as a design alternative to alleviate reinforcement congestion in critical shear and/or moment regions of reinforced concrete coupled shear walls has been considered. Use of HPFRCC may allow for simplified reinforcement detailing with adequate damage tolerance through a ductile response obtained by the tensile strain-hardening and confined compressive behavior of the material. Thus, use of HPFRCC materials in large-scale structural applications could significantly reduce the amount of reinforcement required to ensure adequate performance in areas of inelastic deformation demands, while also potentially reducing labor costs and construction time delays (Parra-Montesinos, 2005). An example of the typical congestion present in a diagonally reinforced coupling beam can be seen in Figure 1-3. Due to increased material costs associated with using HPFRCC, the material has been targeted for critical regions where substantial reinforcement detailing is currently required for providing adequate behavior during earthquakes. Canbolat et al. (2005) investigated the use of HPFRCC coupling beams without diagonal reinforcing bars and with diagonal bars without transverse reinforcement around the main diagonals. The results indicated that a reduction in such reinforcement may be
achieved in HPFRCC coupling beams without compromising the shear strength, due to the additional diagonal tensile strength provided by the fibers. In fact, the HPFRCC coupling beams exhibited higher shear strength and stiffness retention than the reinforced concrete coupling beams designed according to the ACI Code. Since cast-in-place HPFRCC coupling beams could present constructability issues, the use of precast HPFRCC beams in combination with conventional reinforced concrete structural walls has been proposed. Additional HPFRCC coupling beam tests have been conducted by project colleagues at the University of Michigan, and these similarly successful results will be discussed further in Chapter 5.

![Figure 1-3. On-Site Coupling Beam from Lequesne (2011)](image)

A related experimental program reported here has been conducted to further understand the behavior of HPFRCC under general uniaxial and biaxial stress states (such as would for instance be expected at various key locations in a coupling beam). Concrete plate specimens comprising mixes containing from one to two percent volume fraction of hooked steel fibers or
Spectra (polyethylene) fibers were tested under uniaxial and biaxial compression, and failure envelopes were developed for each type of concrete. Previous test results revealed that, across both fiber types, the residual strength during uniaxial testing exceeded 60 percent of the maximum stress at as much as 2 percent strain, with the level of residual strength depending on the fiber volume fraction. Under equal biaxial compression a strength increase of greater than 50 percent over the uniaxial value was observed for both fiber types (Foltz et al., 2008). After the exploration of these different fiber types and volume fractions, a 1.5 percent volume fraction of hooked steel fibers was selected as the concrete mix for more comprehensive examination. It has been found that HPFRCC in compression exhibits about 50 percent residual stress up to a strain of 3%, as well as a shift in the failure mechanism from tensile splitting to a faulting or shear failure. Failure envelopes have been developed for each type of composite, and their stress-strain behaviors as well as failure mechanisms were observed.

Two large one-third scale tests conducted by project colleagues at the University of Michigan on coupled wall systems were recently conducted. The four-story walls utilized three HPFRCC precast coupling beams and one reinforced concrete coupling beam. Additionally, the second wall specimen implemented HPFRCC into the plastic hinge region of the wall. The test specimens were subjected to reverse cyclic loading, and the results illustrated the ability of HPFRCC to increase the shear capacity of structural walls while reducing the level of damage experienced at significant drifts. Further details of the experimental results of the large-scale tests at the University of Michigan, as well as the multi-axial tests at the University of Illinois at Urbana-Champaign will be described in Chapters 2 and 3.

From the experimental tests, the material properties were implemented into a nonlinear finite element program to demonstrate the ability to predict the behavior of HPFRCC under
multi-axial loads and in structural components. An effort was undertaken to model the multi-axial tests at the University of Illinois, and then the HPFRCC material model was extended to uses of coupling beam component tests.

Another major aspect of this research, although not directly related to HPFRCC use, was the testing of a full-scale pile-wharf connection. The background and details of the pile-wharf connection test are outlined in Chapters 7 and 8. The HPFRCC material model that was developed from the University of Illinois multi-axial tests was then extended to modeling the pile-wharf connection. While this use of HPFRCC has not been tested in a laboratory, the model allows one to assess the potential benefits of using HPFRCC on the ductility and capacity of the pile-wharf connection.

1.3 Objective and Scope

The objective of this study is to achieve the following unique contributions to the research community:

- The first study to experimentally test the biaxial behavior of a self-consolidating HPFRCC. The thesis provides failure envelopes and deformation characteristics that are not available in the literature, and should be useful for calibrating further finite element models.
- The first study to develop a finite element model for HPFRCC from its own biaxial experimental results. This thesis outlines the procedure and input parameters required to utilize commercially available software to model HPFRCC.
• The first comprehensive study on the use of an HPFRCC model, calibrated from its own experimental results, to model coupling beams and pile-wharf connections. Also, it outlines a process of implementing the HPFRCC model into historical experimental tests to perform a parametric study on the potential effect of HPFRCC on damage-critical structural components.

• The first pile-wharf large-scale connection test with realistic boundary conditions was reported in this thesis. Previous tests were not able to simulate the application of displacements, rotations, and axial load simultaneously. This realistic loading provides a deeper insight into the expected performance of pile-wharf connections during seismic events.

The scope of the process for achieving these objectives begins with the examination of the mechanical properties of HPFRCC under uniaxial and biaxial stress states. Using experimental results obtained through the laboratory tests of these HPFRCC materials, it is possible to extrapolate the energy dissipating behavior of HPFRCC to its uses in structural elements for seismic design and to formulate a material model to predict HPFRCC behavior in seismic events. A phenomenological model based on non-associated plasticity coupled with a smeared crack representation is formulated and calibrated to the experimental data. In addition to the small-scale material testing and material modeling, HPFRCC coupling beam component tests were performed by project colleagues at the University of Michigan. After completion of these large-scale tests, the material model was validated with their experimental coupling beam results. A full-scale reinforced concrete pile-wharf connection was then tested at the University of Illinois, and the HPFRCC model was then implemented into the pile-wharf connection application. It was found that the increase in structural component damage tolerance through a
ductile response obtained by the tensile strain-hardening and confined compressive behavior from the use of HPFRCC makes it a viable solution as a replacement for some steel confinement reinforcement and as an additional shear resistance mechanism. With the development of an HPFRCC modeling tool, insight into the levels of damage experienced by structural elements can inform performance based design decisions regarding the use of HPFRCC in critical structural components. Combining the knowledge and behavioral trends gained through laboratory testing of the HPFRCC materials with the vast experimental information already available in the literature, it is possible to provide the informed designer with the tools to include HPFRCC in the seismic design of damage critical structural components.

1.4 Chapter Description

This dissertation proposal is composed of four chapters as follows:

Chapter 1: Introduction – This chapter provides an overview of the research, as well as the objectives, scope, and layout of the proposed dissertation work.

Chapter 2: Background Information – This chapter provides background about HPFRCC, as well as a review of previous research pertaining to the use and testing of plain concrete, HPFRCC, and RC coupling beams. Also, it contains additional previously explored uses of HPFRCC and prior HPFRCC finite element modeling efforts.

Chapter 3: Material Testing Results – This chapter provides an overview of the conducted HPFRCC material experimental program. It describes in detail the concrete mixtures used in the study, testing techniques, and experimental results.

Chapter 4: Material Modeling – This chapter describes the material modeling effort of the HPFRCC in ATENA, a non-linear continuum finite element program. It outlines the
implementation of the HPFRCC material testing results into ATENA, as well as providing a review of the modeling results and a comparison of the results with the experimental findings.

Chapter 5: Coupling Beam Component Test Results – This chapter provides further background on the experiments conducted by previous researchers. It reviews the test setup and experimental results from previous tests to set the stage for the coupling beam modeling effort in the subsequent chapter.

Chapter 6: Coupling Beam Component Modeling – This chapter reviews the modeling of coupling beams tested by previous researchers and described in Chapter 5. In addition to an assessment of the modeling results of the coupling beams tested experimentally, this chapter also includes complementary variations of coupling beams not tested previously (e.g. previously tested HPFRCC coupling beams without fibers).

Chapter 7: Experimental Plan of Pile-Wharf Connection Test – This chapter signals a shift in the focus of the dissertation. It provides an overview of the pile-wharf connection testing program, experimental results from the literature, and details of the current experimental test setup. These details include specimen construction, loading protocol, facility capabilities, and material properties.

Chapter 8: Pile-Wharf Connection Experimental Results – This chapter investigates the results of the pile-wharf connection test. It includes a review of the visual progression of damage and a detailed analysis of the experimental results.

Chapter 9: Pile-Wharf Connection Modeling – This chapter reviews the modeling of the pile-wharf connection test in ATENA. It demonstrates the ability of ATENA to model the test specimen, but its main focus is to demonstrate the extension of the HPFRCC model to other
structural components. It reviews the modeling results of the RC pile-wharf connection and compares the results to the performance of a similar pile-wharf connection with HPFRCC.

Chapter 10: Conclusions – This chapter provides a summary of the findings and conclusions of this dissertation, as well as an outline of the research contributions and future work.
CHAPTER 2. BACKGROUND INFORMATION

In this chapter, a review of the literature pertaining to the different aspects of the current research work is given. There have been previous studies conducted concerning the multi-axial behavior of plain concrete, as well as some limited tests on the multi-axial behavior of fiber reinforced concrete. While triaxial concrete tests have been previously completed on concrete, the focus of the current research is on the biaxial and uniaxial performance of plain concrete and high-performance fiber-reinforced cementitious composites (HPFRCC). The literature review begins with a discussion of previous biaxial and uniaxial tests, setting the stage for the testing program to be discussed in Chapter 3. Testing has also been conducted on both reinforced concrete coupling beams and HPFRCC coupling beams by previous researchers. A description of their results is presented, and a discussion of their findings and the implications on the current proposed use of precast HPFRCC couplings beams and modeling is addressed. Additional coupling beam component tests have been conducted by research project colleagues at the University of Michigan, and the details of that testing program and results will be addressed in Chapter 5. Finally, previous work on the analytical modeling of HPFRCC is discussed.

2.1 Previously Conducted Multi-Axial Plain Concrete Results

Understanding the behavior of concrete under multi-axial stress states is critical to developing a universal failure criterion for concrete, thus many series of tests have been conducted on concrete under biaxial stress states over the years. The multi-axial behavior of concrete is important in design because such forces exist regularly in concrete structures, such as in shear regions of flexural members, shear walls, slabs, shells, and various containment structures. Therefore, it is essential in the creation of accurate models that characterize the
uniaxial and biaxial behavior of concrete to have an experimental testing regime that imposes the desired forces and boundary conditions on the specimen. However, a major problem when conducting the tests occurs when trying to develop a uniform biaxial stress state. Most of the discrepancies arising in the test results can be traced to unintended differences in the stress states which have been developed in the test specimen (Kupfer et al., 1969). Early biaxial tests were conducted on cube concrete specimens of varying dimensions. The initial tests ignored the biaxial tension condition, as well as the influence on the results produced by the restraining shear force on the faces of the specimens due to the test setup (Iynegar et al., 1965). This restraint inevitably results in an increase of the apparent strength in the specimen. Later investigations employed various surface treatments of the concrete or soft backing between the bearing platens and the specimen; however, the results widely varied, and the strength obtained for the equal compression case ranged from 80 percent to 350 percent of the uniaxial compression strength (Kupfer et al., 1969). A review by Hilsdorf (1965) concluded that square concrete plates subjected to in-plane loading are suitable for the determination of the biaxial strength of concrete over the entire range of biaxial stress combinations only once conventional solid bearing plates are replaced with brush bearing platens. Kupfer et al. (1969) recognized this recommendation to mitigate the effect of the confining stresses imposed by the loading platens, and replaced the solid bearing platens. The brush bearing platens consisted of a series of closely spaced steel bars that were flexible enough to accommodate transverse or lateral deformations of the concrete specimens without applying any significant restraining force. The brush bearing platens are shown in Figure 2-1. As such, a similar concept was adopted and used for the present experimental program, as outlined in Chapter 3.
Other methods to reduce friction have included the use of thin Teflon sheets with silicon grease between the loading platens and the concrete specimen. To prevent the injection of silicon grease into the specimen and any effect on the response of the specimen, paraffin was used to coat the specimen faces (Maekawa & Okamura, 1983). Wastiels (1979) warned that penetration of lubricants into the pores of the concrete can cause tensile stresses in the specimen, thereby leading to an underestimation of the strength. A finite element evaluation was conducted by Hussein and Marzouk (2000b), investigating the different levels of confinement caused by using ordinary dry solid steel testing platens, brush platens, and friction-reducing Teflon sheets. It was clear that the solid platens did not provide homogeneous stress fields and would lead to overestimating the strength of the specimen. The Teflon pads and brush platens both provided substantial improvement; however, it was ultimately found that the displacement contours for the brush supports were more uniform and the recommended method for testing (Hussein & Marzouk, 2000b).
Kupfer et al. (1969) tested all three biaxial loading scenarios: compression-compression, compression-tension, and tension-tension. The loads were applied with a constant strain rate, and the loading ratios were maintained through the use of a specially designed load distributing frame. For the tensile tests, the brush bearing platens were glued to the concrete specimens using an epoxy resin, and penetration of the glue in between the brush filaments was prevented by placing rubber cement in the voids between the filaments. It was noted that the application of the rubber cement had no measurable effect on the flexibility of the filaments.
Varying levels of increased concrete capacity due to biaxial loading have been reported by researchers. Kupfer et al. (1969) found that the maximum increase occurred when the stress ratio, $\sigma_1 / \sigma_2$, was -1/-0.5 (where $\sigma_1$ and $\sigma_2$ refer to corresponding principal stresses and compression is taken as negative); the ratio of peak applied biaxial stress to the uniaxial plate capacity was 1.27. For equal biaxial compression, it was found that the increase in the strength was 16% when compared to the uniaxial plate strength. Figure 2-2 displays the biaxial strength envelope obtained by Kupfer et al. (1969). Liu et al. (1972), Tasuji et al. (1978), and Lee et al. (2004) found similar equal biaxial compression results. As expected, biaxial tests conducted on specimens without friction reducing platens or setups experienced a maximum increase in biaxial ultimate strength of about 30 percent over the uniaxial strength, and this larger increase in biaxial strength can be attributed to the confinement of the specimens (Nawy et al., 2003). Also, several researchers have found that the relative (percentage) increase in biaxial compressive strength is higher for concrete with a lower uniaxial compressive strength (Traina & Mansour, 1991; Tasuji et al., 1978). These results were further confirmed during a biaxial testing regime on high-strength concrete (Hussein & Marzouk, 2000a). The compressive strain at maximum loading was found to be about the same for both uniaxial and biaxial compression at about 0.25% strain, and the onset of major microcracking occurred at about 75 percent of the ultimate load (Tasuji et al., 1978).
Failure modes and surfaces were found to be similar for both normal strength concrete and high-strength concrete (Hussein & Marzouk, 2000a). In fact, there was no fundamental difference in the crack patterns and failure modes due to the increase in the compressive strength of the concrete or due to the use of lightweight aggregates under multi-axial loading. Under uniaxial compression, fracture occurred due to the formation of cracks that were inclined at an angle up to 30 degrees in the direction of the applied load and perpendicular to the unloaded out-of-plane surface. Under biaxial compression ratios of 0.5 to 1.0, microcracks formed parallel to the free surface of the specimen, and failure occurred by the formation of a major tensile splitting crack along the direction of loading and in a plane parallel to the free surface of the specimen. The biaxial compression failure mode was similar under low biaxial compression ratios; however, the failure typically occurred due to the formation of a major crack along the plane of...
loading and at an angle up to 25 degrees with the free surface of the specimen. When subjected to tension-compression loading, one continuous crack formed normal to the principal tensile stress; however, at ratios with higher compressive force, some cracks were observed in the direction of compressive loading before failure. Under uniaxial tension, fracture occurred by the formation of a single crack perpendicular to the direction of loading and normal to the plane of the specimen. Biaxial tension resulted in the formation of a single crack at failure in a direction normal to the unloaded surface of the specimen and perpendicular to the maximum principal tensile stress. For equal biaxial tension, there was no preference in the direction for the formation of the fracture surface, except that the cracks were always normal to the unloaded surface (Hussein & Marzouk, 2000a). Typical biaxial failure modes of plain concrete are displayed in Figure 2-3. It should be noted that the arrows displayed are for the positive direction, so a negative loading ratio value indicates the application of compression to the specimen.
In the compression-tension region, it was found that the compressive strength decreased almost linearly as the applied tensile stress was increased (Kupfer et al., 1969; Tasuji et al., 1978). For higher strength high-performance concrete, the relative decrease in compressive strength is higher under biaxial compression-tension. At the discontinuity point, where major microcracking is initiated, it was found that the principal tensile strains were a linear function of the average applied stress, defined as \((\sigma_1 + \sigma_2)/2\). Every plate specimen failed by tensile splitting in a plane perpendicular to the direction of the principal tensile strain, indicating that the failure criterion for high-performance concrete under compression-tension is a limiting value for the tensile strain. This limiting value of tensile strain is not constant, but it increases with the degree of compression (Calixto, 2002).
In the tension-tension quadrant, some of the previous researchers observed no increase in the strength under biaxial tension. They indicated that the biaxial concrete strength was equal to its uniaxial tensile strength (Kupfer et al., 1969; Nelissen, 1972; Lee et al., 2004). Tasuji et al. (1978), however, noted a definite increase in the biaxial tensile strength of concrete when compared to its uniaxial tensile strength, on the order of 10 to 20 percent. The tensile strain at maximum loading was found to be about the same for both uniaxial and biaxial tension at about +0.015% strain, and the onset of major microcracking occurred at about 60 percent of the ultimate load in stress states involving direct tension (Tasuji et al., 1978).

Cho et al. (2004) also investigated the stress-strain relationship of reinforced concrete subjected to biaxial tension. While these specimens were mostly exploring the biaxial tensile strength of concrete with varying reinforcement ratios, they did however note that concrete subjected to biaxial tension experienced lower tensile stresses experimentally than previously proposed biaxial tension models which were largely based on the results of uniaxial tension tests. This indicates the importance of understanding the behavior of concrete under biaxial tension, and the current lack of reliable experimental data.

Lan and Guo (1999) conducted a series of tests on concrete plate specimens that compared the results of specimens under proportional monotonic compressive biaxial loading to failure, proportional repeated compressive biaxial loading to failure with complete unloading and a predetermined strain increment, and proportional repeated compressive biaxial loading to failure with partial unloading and a predetermined strain increment. For the specimens subjected to repeated loading, each cycle was loaded to the next predetermined strain increment until failure occurred in the specimen. The stress ratios investigated were 0.0, 0.25, 0.50, 0.75, and 1.0. It was found that for concrete tested under repeated biaxial loading and unloading cycles,
the failure envelope was unaffected when compared to those subjected to monotonic loads. Also, it was seen that the shape of the envelope of the stress-strain curve under repeated biaxial loads was independent of the stress ratio and was similar to the uniaxial behavior under repeated compressive loading when normalized by the failure stress and failure strain. Finally, the effect of dynamic loads on low-strength concrete cubes was explored by Yan and Lin (2007). They found that the dynamic strength increases with an increase in strain rate and lateral confinement; however, the initial tangent stiffness and fracture pattern remain independent of the strain rate.

### 2.2 Previously Conducted Fiber-Reinforced Concrete Multi-Axial Results

The initial tests on fiber-reinforced concrete subjected to multi-axial loads were conducted by Yin et al. (1989), and the experiments were performed on both steel fiber reinforced and plain concrete specimens of dimensions similar in size to those used by Kupfer et al. (1969). These plate specimens were cut from concrete blocks, similar to the tests that will be described in Chapter 3. However, testing was conducted on a “load-bifurcation” test machine, as shown in Figure 2-4, which uses two crescent shaped distribution beams, rather than four individual actuators. Since the load can only be applied to the distribution beams at three locations, the test setup limited the variety of compressive stress ratios that were applied to the specimens to 1.0, 0.5, and 0.2, as well as limiting the tests to being exclusively load-controlled; all of these specimens were epoxied to the brush platens.
Figure 2-4. Load Bifurcation Machine used by Yin et al. (1989)

Results from the experimental program indicated an increase in ductility, an increase in biaxial strength, and a shift in the failure mechanism from tensile splitting to shearing with the addition of fibers to the concrete matrix. However, Yin et al. (1989) did not find that the inclusion of fibers significantly altered the uniaxial compressive strength. Also, it was found that increasing the fiber volume fraction of 1 in. (25 mm) steel fibers from 1.0 percent to 2.0 percent produced no further increase in the biaxial strength of the fiber-reinforced concrete (Yin et al., 1989). The biaxial strength envelope normalized by the uniaxial compressive plate strength of plain concrete obtained by Yin et al. (1989) for varying fiber volumes is shown in Figure 2-5. Traina and Mansour (1991) found that steel fiber-reinforced concrete either showed an increase, decrease, or no change in uniaxial compressive strength when compared to plain concrete,
depending on the fiber type, aspect ratio, and fiber volumetric percentage. The shift in failure mechanism from a common tensile splitting failure to a shear-type failure was again observed in the fiber-reinforced concrete when compared to plain concrete (Traina & Mansour, 1991). The increase in strength and ductility, especially when loaded biaxially in compression, was further noted by Torrenti and Djebri (1995), but they also emphasized the importance of knowing the orientation of the dispersed fibers. To fully arrest cracking and to ensure more ductile behavior, a random orientation of the fibers is necessary.

Figure 2-5. Biaxial Strength Envelope for Fiber Reinforced Concrete (Yin et al., 1989)

Steel fiber-reinforced concrete was tested under biaxial tension-compression conditions by Demeke and Tegos (1994). They did not utilize the plate specimen with brush platen loading scheme; instead, they applied the tensile loads by jacks on masses of concrete cast on either end
of the specimen, and the compressive loads were applied directly, with layers of Teflon and petroleum jelly to minimize friction. It was found that steel hooked fibers doubled the uniaxial tensile strength for a 1.5 percent volume fraction, and the strength when compared to plain concrete was almost three times greater when simultaneously subjected to biaxial compression and tension for the same volume fraction. For the fiber reinforced specimens, failure occurred due to fiber pullout, with the extent of the cracked region being a function of the fiber percentage. All of the plain concrete specimens experienced an abrupt and explosive failure when one large crack appeared, dividing the specimen into two pieces (Demeke & Tegos, 1994). The triaxial behavior of steel fiber reinforced concrete was explored by Chern et al. (1992), and it was found that when compared to cylinders of plain concrete, the steel fiber reinforced concrete behaved in a more ductile manner, especially when exposed to some tensile loading.

Finally, Yin and Hsu (1995) investigated the performance of steel fiber-reinforced concrete subjected to cyclic uniaxial and biaxial loading. It was found that fibers could increase the fatigue strength when the fatigue load was in the low-cycle region because fibers can arrest the propagation of cracks at higher stress levels; however, in the high-cycle fatigue region, fibers do not help because they are ineffective at arresting the bond cracks that ultimately dictate the fatigue life of the specimens. Also, the failure mode of the specimens was unchanged when compared to monotonic tests (Yin & Hsu, 1995).

2.3 Previously Conducted Reinforced Concrete Coupling Beam Tests

Paulay (1971) reported a series of tests conducted on coupling beams with short span-to-depth ratios (1.02 and 1.29). At the time, coupling beams were typically constructed using only top and bottom flexural reinforcement, as well as some transverse reinforcement. Due to the elongation of the coupling beam caused by damage when subjected to cyclical load reversals, all
of the flexural reinforcement was found to be under tension throughout the entire span of the beam. With the flexural reinforcement subjected to tension throughout the test, the advantage of having compression steel to enhance the ductility of the coupling beam was not available. Also, it was found that the distribution of the stresses along the flexural reinforcement drastically deviates from the imposed bending moment pattern, and a region of low moment near midspan of the coupling beam was not existent. Under reversed loadings, the damage caused the coupling beam to develop into two triangular regions, with the capacity of the beam to resist shear through aggregate interlock almost entirely eliminated (Paulay, 1971). The diagonal tension failure mechanism is shown in Figure 2-6. To balance the tension forces in the top and bottom reinforcement, an equal compression force resultant had to occur between the bottom and top reinforcement, thus the internal lever arm to resist the applied load was significantly less than what would be found in a slender reinforced concrete beam. Irrespective of the amount of web reinforcement used, all of the shear force had to be transferred by the concrete between the last stirrup and the end of the beam and through dowel action of the flexural reinforcement. Once the ability to transfer shear through aggregate interlock was lost, the beams began to experience a sliding movement, engaging the flexural bars in dowel shear resistance and eventually resulting in a sliding shear failure (Paulay & Binney, 1974). Figure 2-7 shows the typical sliding shear failure of conventionally reinforced coupling beams.
Figure 2-6. Diagonal Tension Failure Mechanism (Paulay, 1971)

Figure 2-7. Sliding Shear Failure of Conventionally Reinforced Coupling Beam (Paulay, 1977)
Short coupling beams of aspect ratios 1.02 and 1.29 were then tested by Paulay and Binney (1974) utilizing diagonal bars to simulate the behavior of “cross-bracing” within the beam. In these diagonally reinforced beams, the role of the concrete for resisting forces was relatively minor; instead, the diagonal reinforcement resisted essentially all of the loading, and the behavior of the beam was governed by the behavior of the diagonal steel reinforcement, which was equally effective in tension and compression. The coupling beams eventually failed by buckling of the compression bars after the surrounding concrete had broken away. This illustrated the importance of confining the concrete within the cage of the diagonally placed group of bars to preserve the integrity of the section and to provide flexural rigidity (Paulay & Binney, 1974). Experimental verification of the behavior of diagonally reinforced coupling beams as part of a whole coupled shear wall structure was then explored by Paulay and Santhakumar (1976). Two one-quarter full-size seven-story shear walls were tested; one had conventionally reinforced coupling beams, while the other had diagonally reinforced coupling beams. The reinforcement in the structural walls of each model was identical, and the walls were designed such that yielding would occur at the base of the walls only after all of the coupling beams had yielded. This was done with consideration of the desired sequence of plastification where a designer would desire to protect the walls against permanent damage. The coupling beams were designed to have approximately the same shear capacity, and the results of the tests revealed that the ductility, energy dissipating capacity, level of damage, and overall performance were superior in the short aspect ratio diagonally reinforced coupling beam structure (Paulay & Santhakumar, 1976). Results from those tests are shown in Figure 2-8. A major advantage of the use of the more ductile coupling beams is that the majority of energy to be dissipated during a large seismic event can be dispersed in the coupling system over the full height of a structure,
rather than being concentrated in the plastic hinging region at the base of the walls and perhaps jeopardizing the gravity load carrying capacity of the wall (Paulay, 1977). Since the reinforcement in the structural walls of each specimen was the same, the degradation of the overall stiffness during similar load cycles can be attributed to the performance of the coupling beams. It was found that in the wall with diagonally reinforced coupling beams, the strength loss and stiffness degradation at any given displacement was significantly less than in the wall with conventionally reinforced coupling beams. The reason for the superior performance of the diagonally reinforced coupled wall was that the critical internal forces were carried by steel rather than concrete. Thus, the inevitable deterioration of the concrete under severe repeated reversed loading had little effect, and no significant stiffness degradation was observed in the diagonally reinforced coupled wall. Comparatively, the conventionally reinforced coupled wall experienced a sliding shear failure in each coupling beam, followed by the eventual compression zone failure in the tension wall due to spalling of the concrete coupled with buckling of the compression reinforcement on the boundary.
A comprehensive study on coupling beams with span-to-total-depth ratios of 2.5 and 5.0 was conducted by Barney et al. (1980). The primary variables explored were the arrangement of primary reinforcement, span-to-depth ratio, and the size of the confined concrete core. The three primary reinforcement layouts were conventional straight longitudinal reinforcement, diagonal bars in the hinging regions at either end of the coupling beam, and full-length diagonal reinforcement, as proposed by Paulay and Binney (1974). As previously seen, the responses of the conventionally reinforced beams were limited by sliding shear at the beam-wall intersection; however, an increased core size, achieved by increasing the width of the coupling beam, did improve the inelastic performance. The diagonal reinforcement in the hinging regions at the ends of the beams improved performance, but the increase was not significant enough to warrant
the additional complexity and cost. The fully developed diagonal reinforcement had the best strength, ductility, and energy-dissipation of all of the coupling beams tested, but the improvement was less pronounced in the long-span beams. Therefore, it was concluded that conventional bars are preferred in coupling beams with span-to-depth ratios greater than 5.0 (Barney et al., 1980).

Other coupling beam reinforcement layouts have also been explored. Tegos and Penelis (1988) conducted tests on coupling beams with inclined bars forming a rhombic truss. They found that the rhombic layout, as shown in design 3 of Figure 2-9, was in fact able to successfully prevent an explosive shear failure, despite having a decreased number of hoops. Also, there was no appreciable deterioration after reaching the maximum capacity of specimens with span-to-depth ratios of greater than 1.5 (Tegos & Penelis, 1988). Additional coupling beam tests were performed by Tassios et al. (1996) on a variety of reinforcement layouts. They investigated relatively short span-to-depth ratio coupling beams (1.0 and 1.66), and the reinforcement layouts explored were conventional, diagonal, conventional with inclined bars in the hinging region, and dowel bars. The details of the reinforcement layout, reinforcement ratio, and concrete strength of the investigated coupling beams are shown in Figure 2-9. Extreme pinching was observed in the results of their conventional and dowel coupling beams. The inclined bars seemed to diminish the pinching effect, and the diagonal bars almost eliminated it. The benefit of the diagonal reinforcement diminished as the shear span of the coupling beam increased, and at longer spans, around a 2.67 span-to-depth ratio, conventional reinforcement was then suggested (Tassios et al., 1996).

Galano and Vignoli (2000) revisited the comparison among conventionally reinforced, diagonally reinforced, and rhombically reinforced coupling beams in coupled walls. They noted
an explosive cleavage shear failure in the diagonally reinforced coupling beams, and observed that when using the rhombic configuration, this effect was avoided, as well as achieving a higher rotational ductility. They also noted the improved simplicity of using a rhombic configuration when designing relatively thin coupled walls, due to the construction difficulties associated with confinement ties along the diagonal and their placement within the wall (Galano & Vignoli, 2000).

<table>
<thead>
<tr>
<th>specimen</th>
<th>reinforcement layout</th>
<th>$f'_c$ (MPa)</th>
<th>$\varphi_t$ (mm)</th>
<th>$\varphi_d$ (mm)</th>
<th>$\rho_f$(%) total</th>
<th>$\rho_d$(%) total</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB-1A</td>
<td></td>
<td>32.8</td>
<td>12</td>
<td>-</td>
<td>0.7</td>
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</tr>
<tr>
<td>CB-1B</td>
<td></td>
<td>33.0</td>
<td>12</td>
<td>-</td>
<td>1.1</td>
<td>-</td>
</tr>
<tr>
<td>CB-2A</td>
<td></td>
<td>28.5</td>
<td>6</td>
<td>10</td>
<td>-</td>
<td>1.0</td>
</tr>
<tr>
<td>CB-2B</td>
<td></td>
<td>26.3</td>
<td>6</td>
<td>10</td>
<td>-</td>
<td>1.6</td>
</tr>
<tr>
<td>CB-3A</td>
<td></td>
<td>31.7</td>
<td>12</td>
<td>18</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>CB-3B</td>
<td></td>
<td>33.8</td>
<td>12</td>
<td>14</td>
<td>1.1</td>
<td>0.8</td>
</tr>
<tr>
<td>CB-4A</td>
<td></td>
<td>29.8</td>
<td>6</td>
<td>4Ø20</td>
<td>-</td>
<td>1.9</td>
</tr>
<tr>
<td>CB-4B</td>
<td></td>
<td>31.3</td>
<td>6</td>
<td>3Ø18</td>
<td>-</td>
<td>1.9</td>
</tr>
<tr>
<td>CB-5A</td>
<td></td>
<td>32.3</td>
<td>12</td>
<td>4Ø20</td>
<td>0.7</td>
<td>1.9</td>
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<tr>
<td>CB-5B</td>
<td></td>
<td>33.1</td>
<td>12</td>
<td>3Ø18</td>
<td>1.1</td>
<td>1.9</td>
</tr>
</tbody>
</table>

Figure 2-9. Investigated Reinforcement Layouts (Tassios et al., 1996)
Kwan and Zhao (2002) believed that previous tests of coupling beams did not properly simulate their boundary conditions by allowing unequal rotations at the two ends. Thus, they devised an experimental test set-up that required equal joint rotations, and developed a testing program of conventionally reinforced coupling beams with varying span-to-depth ratios and reinforcement layouts. The experimental results found that the diagonal bars did give a more stable hysteretic response and a much better energy dissipation capacity; however, they also found that the drift ratios were comparable to coupling beams with conventional reinforcement. They concluded that the diagonal reinforcement did not improve the deformability of the coupling beams, but their diagonally reinforced coupling beam eventually failed due to abrupt buckling of the diagonal bars, indicating insufficient confinement of the diagonal reinforcing bars (Kwan & Zhao, Cyclic Behavior of Deep Reinforced Concrete Coupling Beams, 2002).

In the design of coupling beams, ACI 318-08 (2008) requires coupling beams with a span-to-depth ratio less than 2 to be reinforced by two intersecting cages of diagonal reinforcing bars placed symmetrically about midspan of the coupling beam. These diagonal bars are then required to have a minimum of four bars in two or more layers, and these diagonal bars must be fully confined with transverse reinforcement. Additionally, the entire coupling beam cross section must be provided with substantial confining transverse reinforcement. In previous structural concrete codes, this was the same required design philosophy for all coupling beams with span-to-depth ratios less than 4. For coupling beams with span-to-depth ratios greater than 4, the provisions of special moment frames apply. Despite the almost unanimous opinion that diagonal bars provide superior seismic performance, especially for short coupling beams, it has been shown that it is difficult to design a practically constructible diagonally reinforced coupling beam with a shear stress approaching the ACI 318 limit of $10\sqrt{f'_c} \text{ [psi]} (0.83\sqrt{f'_c} \text{ [MPa]})$. 

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Harries et al. (2005) contended that the constraining geometry and confinement requirements of the individual diagonal elements are unnecessarily restrictive and they proposed the use of transverse reinforcement to satisfy the code requirements and to confine the entire cross section, while still providing adequate support and allowing versatility in the design (Harries et al., 2005). To relieve the difficulty and cost in constructing diagonally reinforced coupling beams, ACI 318-08 (2008) has a new provision which allows two options when designing coupling beams with an intermediate span-to-depth ratio between 2 and 4. The first option, shown in Figure 2-10, is similar to the previous provision and requires transverse reinforcement around the diagonal bars and modest transverse reinforcement around the entire section. The second option, displayed in Figure 2-11, does not require transverse reinforcement around the diagonal bars, but it does require a significantly larger amount of transverse reinforcement to confine the entire section. Wallace (2007) reported a coupling beam testing program on one-half scale coupling beams with diagonal reinforcement designed to accommodate maximum shear stresses of $10\sqrt{f_c} \text{ [psi]} (0.83\sqrt{f_c} \text{ [MPa]})$ and $6\sqrt{f_c} \text{ [psi]} (0.50\sqrt{f_c} \text{ [MPa]})$ for span-to-depth ratios of 2.4 and 3.33, respectively. Four tests were reported, two at each span-to-depth ratio with each of the ACI 318-08 (2008) reinforcement layout design provisions explored. The results showed that very similar force-deformation responses were obtained using the different detailing schemes at each of the aspect ratios, and these results are shown in Figure 2-12. In Figure 2-12, the plots labeled “diagonal” refer to the detail shown in Figure 2-10, with confined diagonal reinforcing, while the plots labeled “full” refer to the detail shown on the right in Figure 2-11, where the diagonal bars do not have transverse reinforcement, but the coupling beam specimen is confined with extensive transverse reinforcement. The peak shear stresses achieved the desired design
level, and the similarity in the responses indicates that the new detailing provision was adequate for this application.

Figure 2-10. Coupling Beam Reinforcement Details with Confined Diagonal Bars from ACI 318-08 (2008)
Figure 2-11. Coupling Beam Reinforcement without Confined Diagonal Bars and with Fully Confined Cross-Section from ACI 318-08 (2008)

Figure 2-12. Force-Drift Relationship for Coupling Beams with Alternate Reinforcement Layouts (Wallace, 2007)
2.4 Previously Conducted HPFRCC Coupling Beam Tests

Canbolat (2004) conducted a research project investigating the performance of four coupling beam specimens under displacement reversals. The specimens were ¾-scale with two stiff RC members to represent the structural walls. To ensure a shear-dominated response, the span-to-depth ratio was 1.0 for each coupling beam. The first specimen was RC with diagonal reinforcement, to evaluate the seismic response of coupling beams designed to the ACI code. All of the horizontal and vertical reinforcement ratios were representative of the minimum code limits to reduce their contribution to the flexural and shear capacity of the coupling beam. The width of the RC coupling beam was 8 in. (200 mm), to accommodate the width of the intersecting diagonal reinforcement cage groups.

The three subsequent HPFRCC coupling beam specimens were all precast to ease the construction process. Each beam was prefabricated with sufficient embedment length and reinforcement anchorage to ensure full moment and shear transfer with the structural walls. The second specimen utilized Spectra ultra-high molecular weight polyethylene fibers, and no diagonal reinforcement was used in this specimen – only uniformly distributed horizontal and vertical reinforcement was provided to investigate the possibility of the total elimination of diagonal reinforcement. The third specimen also consisted of Spectra ultra-high molecular weight polyethylene fibers; however, two diagonal bars were provided without confining reinforcement, corresponding to about 80% of the area of the diagonal reinforcement used in the RC specimen. This specimen was designed to demonstrate the contribution of the diagonal steel in an HPFRCC specimen. The fourth specimen used Torex twisted steel fibers and the same diagonal bars as the second Spectra specimen; however, the diagonal bars were bent at the beam-wall interface so that they would extend horizontally into the structural wall. The fiber volume
fractions were 2.0% and 1.5% for the Spectra coupling beams and the Torex coupling beam, respectively. Each of the three HPFRCC coupling beams were designed with a beam width of just 6 in. (150 mm) to increase the shear demand in the composite material (Canbolat et al., 2005). The reinforcement details of each test are shown in Figure 2-13.

The results show that the coupling beams with HPFRCC and diagonal reinforcing were able to sustain shear stresses twice as large as the RC coupling beam, and the energy dissipation capacity was also substantially greater. The HPFRCC coupling beams all experienced only minor damage at shear distortions up to 1.0%, and they retained their stiffness through many more load reversals and at more substantial deformations (Canbolat, 2004). It was ultimately found that the use of advanced fiber-reinforced cementitious materials allowed for the simplification of the construction of coupling beams, by making it possible to eliminate transverse confining reinforcement around the diagonal bars. Also, it was found that the use of precast coupling beams, with diagonal reinforcing bent near the beam-wall interface to protrude from the specimen horizontally, behaved considerably well when compared with traditional diagonally braced coupling beams (Canbolat et al., 2005).
As part of the HPFRCC coupling beam experimental testing program, Canbolat (2004) developed a damage index to quantify the damage sustained during loading of a specimen, where a value of 0.0 indicated no damage and a value of 1.0 corresponds to structural failure. The damage index makes it possible to identify different limit states, such as fiber pull-out, which has a potential for implementation into nonlinear analysis of structures. The damage index was based on previously proposed indices, but it was modified and calibrated to account for the behavior of HPFRCC before and after fiber pullout.

Figure 2-13. Reinforcement Details of Coupling Beam Test Specimens for RC (a) and HPFRCC (b,c,d) (Canbolat et al., 2005)
Based on the damage index analysis, Canbolat (2004) proposed damage index ranges for use in performance based design. For the immediate occupancy performance level, the damage index was limited to 0.40. This damage level corresponded to a shear distortion of about 1.0%, with many diagonal hairline sized cracks, requiring little or no repair to HPFRCC coupling beams. The damage index was recommended to be limited to 0.70 for the life safety performance level. At this point in testing, the fibers had pulled out, and the diagonal reinforcement was providing the residual strength and drift capacity. It is expected that repair would be costly and difficult at this performance level. From a damage index of 0.70 to 0.90, the HPFRCC coupling beams would be subjected to severe damage and shear distortions greater than 2.0%. This range would correspond to the collapse-prevention performance state, and the coupling beams would need to be replaced, and possibly the entire building would be severely damaged. Figure 2-14 displays the damage index versus the shear distortion response of the HPFRCC coupling beam with Torex fibers from Canbolat (2004), as well as the recommended performance levels. It can clearly be seen that little damage has occurred during the immediate occupancy performance level. When the fibers began to experience a pullout failure, a large jump can be seen in the damage index, and the capacity of the coupling beam relies on the diagonal reinforcing steel. After fiber pullout, Figure 2-14 shows that the damage more rapidly accumulates in the HPFRCC coupling beams. A parallel exercise could be completed to include the damage index of the RC control test specimen, and such a study would provide insight into a complete comparison of the accumulation of damage in coupling beams under reversed cyclic loading. Additional HPFRCC coupling beam tests were conducted by project colleagues at the University of Michigan, and those results are described in detail in Chapter 5.
Yun et al. (2008) conducted cyclic tests on high-performance hybrid fiber-reinforced cement composite (HPHFRCC) coupling beams with span-to-depth ratios of 1.0. The hybrid composite utilized a 0.75% polyethylene fiber volume fraction to hinder the formation of the large cracks that occur when multiple microcracks converge to form a macrocrack. Once a macrocrack develops, the 0.75% volume fraction of twisted steel fibers was used to bridge the larger crack and allow a ductile response. The idea behind a hybrid fiber system is that two or more fiber types can combine to produce a composite that exhibits a synergistic response from the benefits of each fiber type (Yun et al., 2007). These coupling beams were precast, and to
provide material continuity and proper shear and moment transfer, the coupling beams extended into each wall half of the span length and the reinforcement was fully developed. Also, of the three specimens constructed, one RC coupling beam had diagonal reinforcing, one HPHFRCC had diagonal reinforcing, and one HPHFRCC had only conventional longitudinal and transverse reinforcing. It was found that the diagonally reinforced coupling beam with HPHFRCC could sustain significantly higher loads and higher drift levels than the reinforced concrete coupling beam, as well as further confirming that HPHFRCC provides sufficient lateral confinement around the diagonal reinforcing. Further, the HPHFRCC coupling beam with only conventional reinforcement also outperformed the reinforced concrete coupling beam with diagonal reinforcement in terms of maximum load, displacement ductility, and energy absorption (Yun et al., 2008).

Kuang and Baczkowski (2009) performed an experimental program on five steel fiber-reinforced concrete coupling beams to investigate the effect of span-to-depth ratio, steel fiber content, and shear reinforcement ratio on the monotonic response of the specimens. Longer Dramix fibers, similar to those presented later in this study, were used in these specimens, and the coupling beams were cast continuously with base blocks, which were used to simulate stiff structural walls. All of the coupling beams were conventionally reinforced, and the observed experimental strengths were compared with the strengths predicted by both the British standard BS 8110 and the Eurocode 2. It was found that the steel fiber-reinforced coupling beams without any web shear reinforcement achieved three to four times the shear strength predicted by the codes of practice, thus illustrating the ability of steel fibers to effectively act as shear reinforcement beyond the code expectations. For coupling beams with transverse reinforcement, the higher transverse steel ratios resulted in higher coupling beam shear strengths; however, the
post-peak load performance for each specimen was very similar, indicating that the shear reinforcement did not have an effect on the post-peak load behavior of HPFRCC coupling beams. Finally, the study again confirmed that steel fibers increase the post-cracking tensile strength and ductility of concrete (Kuang & Baczkowski, 2009).

2.5 HPFRCC Large-Scale Coupled Wall Tests

To study the effect of HPFRCC coupling beams on overall system performance two one-third scale coupled structural wall systems were tested at the University of Michigan by Lequesne et al. (2010). Each wall was 4-stories tall, composed of a pair of T-shaped walls, coupling beams, and slabs (at the second and fourth story levels). Within each wall, three of the four coupling beams used HPFRCC, while the fourth coupling beam was made of regular reinforced concrete. Each coupling beam was designed with the intent of achieving a similar capacity; however, slight reinforcement detailing differences were employed to further explore the effect of reinforcement layout on overall ductility. Also, the first two stories of the second coupled wall specimen used HPFRCC within the structural walls themselves. This was done to compare the effect of HPFRCC in the plastic hinging region between the two wall specimens. The HPFRCC used in each specimen was the same NEES Mix #6 that was previously described in Chapter 3. The coupling beams were precast with reinforcing bars extending only horizontally from the precast specimen. It was found throughout construction of the specimen that given sufficient development of the reinforcing bars, the precast coupling beams could be easily integrated into the construction sequence of cast-in-place structural walls.

Since coupled walls are typically located near the center of a structure for use as a structural core, the walls were selected to be T-shaped. It was thought that this would provide a more accurate representation of typically constructed systems. Thus, the flanges of the T-shaped
walls were oriented toward the outer edges of the specimen, with the coupling beams connecting the narrower stem. The base of the specimen was assumed fixed during design, and this was addressed through the use of thick concrete base blocks which were then post-tensioned to the laboratory strong floor. Longitudinal reinforcement was anchored in the foundation and at the top of the wall system by ACI Code compliant screw-on mechanical anchors, and mechanical splices were used near mid-height of the wall system to reduce reinforcement congestion (Lequesne et al., 2010).

At the second and fourth stories of the specimen, a width of slab was poured to create a means through which to apply the lateral forces, as well as to observe the interaction between the slab and coupling beam interface. Since the proposal is for precast coupling beams, no special detail was designed to encourage interaction between the coupling beam and the adjacent slab. Therefore, the slab reinforcement perpendicular to the direction of loading was continuous through the structural walls but not through the coupling beams. Gravity loading was simulated through a vertical force applied to both walls by external prestressing tendons, which were anchored at the second story slab and in the base block foundation. The strands were prestressed to achieve an axial stress of 7% f’c based on the gross area of the structural walls, and the force was constant throughout the test. This level of gravity load is consistent with current design practice for structural walls and was sufficient to offset a majority of the uplift force resulting from the coupling of the walls (Lequesne et al., 2010). The loading protocol was such that the actuator at the fourth story applied a predetermined sequence of lateral displacements, generally increasing the displacement with each reversed cycle by 0.25% drift. The actuator mounted at the second story then simultaneously applied a force equal to 60% of the force applied by the top
actuator to achieve the desired displacement. Figure 2-15 displays the experimental test setup at the University of Michigan.

Figure 2-15. Large-scale Experimental Test Setup
The coupling beam details and dimensions were a main focus of this portion of the research project, so they were the first component designed for the coupled wall system. Three different coupling beam designs, shown in Figure 2-16 and described by Lequesne et al. (2009a), were explored: Bonded FRC, Debonded FRC, and RC. The bonded FRC coupling beams were different from traditional coupling beams for a number of reasons. The diagonal bars did not have any special transverse reinforcement to prevent buckling of the bars because it had been shown previously that HPFRCC can adequately confine diagonal reinforcement (Canbolat et al., 2005). Also, dowel bars were provided at the interface of the beam and wall to strengthen the connection. The dowel bars were terminated only 4 in. (100 mm) into the beam due to the high bond stress that develops between the HPFRCC and the reinforcing bars. The transverse reinforcement was distributed to provide column-type confinement to the ends of the coupling beam, extending approximately $h/2$ away from the face of the walls, where $h$ is the coupling beam height. No special confinement reinforcement was required for the remaining portion of the span, due to the confinement afforded by the HPFRCC. Throughout the rest of the beam, stirrups were provided to assist in about 60% of the shear resistance for the first wall and 35% for the second wall. The Debonded FRC had one change; the dowel bars were extended an additional 3 in. (75 mm) into the coupling beam, but the additional length was wrapped in plastic sheeting and taped to prevent the HPFRCC from bonding with the bar. This was done with the intention to distribute the flexural yielding and to prevent or delay the formation of a single failure plane due to localized rotations. The RC coupling beams were precast and had the same general design changes, except the dowel bars were extended 7.5 in. (188 mm), and the transverse reinforcement was essentially doubled in both the plastic hinging region and the
remaining span of the coupling beam. Figure 2-16 illustrates the previously described reinforcement layouts for the respective coupling beams.

Figure 2-16. Coupling Beam Reinforcement Layout from Lequesne et al. (2009a)
Each of the coupling beams were designed with similar initial stiffnesses and flexural capacities to prevent a single beam from attracting a disproportionate amount of the loading. For sizing the diagonal, longitudinal, and transverse reinforcement, the design philosophy outlined in Chapter 5 and Lequesne et al. (2010) was employed. The flexural and diagonal reinforcement were identical for the HPFRCC and RC coupling beams to provide similar stiffnesses and flexural capacities. However, the transverse reinforcement ratio was increased from 0.55% for the HPFRCC coupling beams to 1.5% to provide additional shear resistance and confinement to the RC beam.

A reinforcement detailing difference arose between the two coupled wall specimens. In the first test specimen, the coupling beam longitudinal reinforcement was terminated about 3 in. (75 mm) into the structural wall from the coupling beam-wall interface. This was done to emulate design practice, but the resulting maximum shear stress calculated from the flexural capacity of the beams was approximately \(5\sqrt{f'c} \text{ [psi]} \cdot (0.42\sqrt{f'c} \text{ [MPa]})\), and it resulted in an undesirable localization of damage along the interface between the structural wall and precast coupling beam section. All of the coupling beam reinforcement was fully developed in the second coupled-wall system, which resulted in damage occurring within the precast section and full development of the coupling beam capacity, and the corresponding peak shear stress was approximately \(9\sqrt{f'c} \text{ [psi]} \cdot (0.75\sqrt{f'c} \text{ [MPa]})\) (Lequesne et al., 2010).

The structural walls for the first specimen were designed according the ACI Building Code (ACI Committee 318, 2008), and the second specimen used HPFRCC in the first two stories, so the transverse reinforcement was reduced due to the assumed higher contribution of the concrete to the shear strength of the system. During the design of the structural walls, the
targeted system coupling ratio was 0.4. The coupling ratio is the ratio of the overturning moment to be resisted through the coupling action due to the wall axial forces to the total overturning moment resistance of the coupled wall. This targeted coupling ratio still takes advantage of the coupling action without placing excessively high axial load demands on the walls. It was assumed that the controlling mechanism of the system would be flexural hinging in the beams and at the base of the wall. The wall concrete was assumed to carry $2\sqrt{f_c'} \text{[psi]} (0.17\sqrt{f_c'} \text{[MPa]})$ for the first specimen, and the concrete shear capacity was assumed to be $4\sqrt{f_c'} \text{[psi]} (0.33\sqrt{f_c'} \text{[MPa]})$ for the second specimen to account for the contribution of HPFRCC to shear capacity. Wall transverse reinforcement, anchored by alternating 90-degree and 135-degree bends and representing a transverse reinforcement ratio of 0.45%, was provided to resist the remaining shear demand (Lequesne et al., 2010).

For confinement of the boundary elements of the structural walls, the first specimen was detailed to satisfy the minimum area and maximum spacing requirements of the 2008 ACI Building Code (ACI Committee 318, 2008), such that the spacing was limited to $1/3$ of the wall thickness. The second specimen investigated the reduction of transverse reinforcement for the boundary elements, with a spacing of $1/2$ the wall thickness in one wall, while the other had a transverse reinforcement spacing equal to the wall thickness. The final reinforcement detailing variation for the structural walls was the inclusion of dowel bars along the interface of the foundation and the HPFRCC structural walls to prevent the localization of flexural rotations in the second wall specimen. Figure 2-17 and Figure 2-18 display the reinforcement layout for the first and second coupled wall specimens, respectively.
Figure 2-17. Reinforcement Layout for Coupled Wall Specimen 1 from Lequesne et al. (2010)
Figure 2-18. Reinforcement Layout for Coupled Wall Specimen 2 from Lequesne et al. (2010)
Since the use of HPF RCC is a departure from typical design and construction practice, efforts were made to be realistic with respect to construction methods and sequencing to truly gauge the possible construction scheduling advantages recognized with the use of precast coupling beams. The construction process began with the precasting of the coupling beams, and then storing them until the rest of the wall was ready for their placement. Construction of the coupled wall began with the wooden formwork for the walls and the reinforcement cages for the foundation. The vertical reinforcement was placed into the foundation when the concrete was poured. Each floor of the wall was then constructed, and as the wall reinforcement reached the level of the coupling beam, the precast beam was put into place with an overhead crane. The beam was then supported on the formwork until the wall concrete was placed. After each level was poured, the formwork was removed and shifted up to the next level. This process was repeated for each level.

The base shear versus top story drift for each coupled wall is displayed in Figure 2-19. It can be seen that both of the coupled walls were able to achieve significant drift without appreciable strength degradation. The base shear capacity of the second wall was greater than the first, and that can be largely attributed to the full development of the coupling beam reinforcement and the presence of HPF RCC in the lower two stories of the wall. The first specimen did very well and was able to maintain about 80% of the peak overturning moment capacity to beyond 2.5% drift in both directions. However, the second specimen had an even superior behavior and was able to retain more than 80% of the peak capacity to beyond 3% drift. Also, the hysteresis loops show essentially no pinching in both experiments, and this may be attributed to the responses being governed by flexural hinging in the bases of the walls and at the ends of the coupling beams. The ability of the coupled concrete walls to achieve an overall drift...
greater than 3% without significant strength degradation is a testament to the ability of HPFRCC to resist a sizable seismic event.

![Graph showing base shear versus drift for large-scale coupled wall tests from Lequesne et al. (2010)]

**Figure 2-19. Base Shear versus Drift for Large-Scale Coupled Wall Tests from Lequesne et al. (2010)**

Figure 2-20 is a depiction of typical damage experienced by the RC coupling beam from the experimental tests at 3.3% drift. By the end of the test, the RC coupling beam had lost all ability to accommodate load and to transfer force to the adjacent structural wall. If such damage is typical of coupling beams under seismic loading, the resulting coupled wall system would effectively reduce to two separate structural walls, with the benefit of coupling action completely
eliminated. This extensive damage to reinforced concrete subjected to biaxial loading is further evidence of the need to implement a material with a greater deformational capacity into design.

Figure 2-20. RC Coupling Beam at 3.3% Drift from Large-Scale Test

Figure 2-21 shows the damage experienced by the HPFRCC coupling beam from the second coupled wall test at 3.3% drift. Lequesne et al. (2010) reported that in both specimens, diagonal-shear cracking was first observed in all four coupling beams near system drifts of 0.5%. In the first specimen, damage began to localize near the precast beam-wall interface due to the termination of coupling beam flexural reinforcement only 3 in. (75 mm) into the wall. As a result, crushing and spalling of the wall concrete near the interface with the coupling beam was observed at system drifts exceeding 2.5%. This behavior prevented the development of a more
desirable damage pattern within the precast beams. In the second specimen, where the coupling beam reinforcement was fully developed, coupling beam damage localized within the precast beam and away from the wall connection. This allowed for a better comparison of the performance of HPFRCC coupling beams relative to RC coupling beams. When reviewing the difference in performance and damage state between the RC coupling beam shown in Figure 2-20 and the HPFRCC coupling beam displayed in Figure 2-21, it is clear that HPFRCC provides a viable alternative for coupling beams and other damage critical elements.

![Figure 2-21. HPFRCC Coupling Beam at 3.3% Drift from Large-Scale Test](image)

After testing the two walls, a comparison is possible among the coupling beams with varying material and reinforcement layouts and between the performances of the two structural
wall specimens. Since the transverse reinforcement was markedly reduced in the structural walls of the second specimen, this confirms that HPFRCC is effective in providing confinement in structural wall elements. Both coupled-wall specimens performed well beyond system drifts of 2.5%, and the HPFRCC regions of the walls exhibited narrow crack spacing, reduced spalling, and improved damage tolerance. Also, utilizing precast coupling beams with reinforcement threaded through the wall boundary element proved to be a successful construction method. Figure 2-22 displays the progression of damage in the fourth story coupling beam in CW-2 from Lequesne (2011), and the multiple cracking behavior and damage capacity at large drifts is evident.

![Damage Progression of 4th Story Coupling Beam in CW-2 at Various Coupling Beam Drifts from Lequesne (2011)](image-url)
Additional experimental data has been provided by project colleagues at the University of Michigan, including coupling beam chord rotations, coupling beam axial deformations, overturning moments, and interstory drifts from the two coupled wall tests. Using this data, future analyses may be able to capture the behavior of the coupled wall specimens, both globally and locally.

2.6 Other Previously Investigated Applications of HPFRCC

Beyond just coupling beams, additional HPFRCC studies have been conducted on various other shear-dominated members or flexural members subjected to shear reversals. Several applications have found that using HPFRCC as a design alternative can increase distortion capacity, shear strength, and damage tolerance in shear critical members. Parra-Montesinos (2005) outlined several research projects that have demonstrated the versatility and applicability of HPFRCC. Parra-Montesinos et al. (2005) described a testing program using HPFRCC in reinforced concrete beam-column connections in which confinement reinforcement was fully eliminated by using Spectra fibers in a 1.5% volume fraction. Also, Parra-Montesinos (2005) described the use of HPFRCC to reduce the connection confinement reinforcement of hybrid reinforced concrete column-steel beam (RCS) connections. In RCS connections, the steel beam passes continuously through the RC column, and the connection confinement is commonly provided through overlapping U-shaped stirrups passing through holes drilled in the web of the steel beam, as well as closely spaced stirrups above and below the steel beam flanges to increase concrete bearing strength and to transfer shear in the connection. In both cases, the HPFRCC specimen with reduced connection confinement reinforcement exhibited superior performance during the test with only minor damage. Additionally, the HPFRCC materials were found to be effective in reducing slip of the reinforcing bars passing through the beam-column connections.
Kim and Parra-Montesinos (2003) investigated the use of HPFRCC in lightly reinforced low-rise walls to increase the displacement capacity when subjected to large displacement reversals. Two wall specimens were tested; one with Spectra fibers in a 1.5% volume fraction, and one with a 2.0% volume fraction of hooked steel fibers. In both specimens the reinforcement in the walls was reduced below the minimum specified standard. While typical reinforced concrete squat walls exhibit drift capacities below 1%, Kim and Parra-Montesinos (2003) reported drifts of about 2.5%, with only moderate damage up to 2.0% drift. It was estimated that the addition of fibers to the concrete matrix contributed to about 80% of the wall shear strength. Tests were also conducted by Chompreda and Parra-Montesinos (2005) on the use of HPFRCC in the plastic hinge region at the end of flexural members. The study relaxed the transverse reinforcement in the plastic hinge region of flexural members and subjected the specimens to large shear reversals. It was concluded that HPFRCC represents a viable alternative to reduce or even eliminate the transverse reinforcement in plastic hinge regions of beams.

Parra-Montesinos et al. (2006) explored the ability of HPFRCC to alleviate confinement reinforcement in the boundary regions of slender structural walls. Four wall specimens with shear span-to-wall length ratios of 3.7 were tested under large displacement reversals. One wall was constructed with regular concrete and detailed according to the 2002 ACI Building Code, while the other three specimens contained steel fibers, twisted or hooked, in the plastic hinge region of the specimen. It was found that a 2.0% fiber volume fraction allowed the elimination of boundary confinement reinforcement without any adverse effects on performance, and a denser array of smaller width cracks were observed, indicating flexure dominated behavior with no major shear-related damage. Although the reinforced concrete control wall did not fail,
significant signs of distress were evident at the final drift level. Additionally, further reverse-cyclic experiments were conducted on flexural members with HPFRCC by Parra-Montesinos and Chompreddain (2007), and it was found that HPFRCC behaved adequately. All HPFRCC test specimens, with or without transverse steel reinforcement, exhibited drift capacities equal to or greater than 4.0%, and HPFRCC was able to prevent buckling of the longitudinal reinforcement without web reinforcement until a 4.0% plastic hinge rotation.

Chao et al. (2009) undertook a testing program to evaluate the bond between reinforcing bars and FRC composites. Pullout specimens were constructed consisting of No. 5 and No. 8 (No. 16M and No. 25M) reinforcing bars embedded 4 in. (102 mm) into control plain concrete or FRC prisms having the dimensions of 6 x 6 x 4 in. (150 x 150 x 102 mm). The test parameters were fiber type, fiber volume fraction, fiber length, and loading type. Typically, bearing forces induced by mechanical interlocking between a deformed reinforcing bar and the surrounding concrete lead to inclined cracks in the concrete. These internal inclined cracks grow wider upon further tension loading, and lead to a large residual slip. Without transverse reinforcement, circumferential tensile stresses lead to the formation of splitting cracks in the concrete and ultimately to a bond failure. If significant transverse reinforcement is present, the propagation of the inclined cracks can be controlled, and the degradation of bond strength is mainly caused by concrete crushing at the toe of the reinforcing bar ribs and shearing of the concrete between the ribs. With the addition of HPFRCC, it was found that the fiber bridging effect helps to control the crack opening and propagation, thus increasing the bond strength. The bond strength in HPFRCC specimens subjected to monotonic loading was as high as 1.5 times that of the spirally reinforced specimens with a volumetric steel reinforcing ratio of 2%; therefore, fibers in HPFRCC were found to be more effective than conventional transverse reinforcement for
enhancing bond strength, as well as for crack control. Further, the conventional spiral reinforcement was inferior to the HPFRCC specimens under both unidirectional cyclic loading and fully reversed cyclic loading. The cumulative energy dissipated by the HPFRCC specimen was approximately 22 times that of the plain concrete specimen and 2.5 times that of the spirally reinforced specimen. Thus, test results suggest that the application of HPFRCC can largely reduce the development length of deformed bars in reinforced concrete members (Chao et al., 2009).

A recent study by Dinh et al. (2010) explored the performance of three different hooked steel fiber types at varying volume fractions in concrete beams without shear reinforcement. Control reinforced concrete beams without shear reinforcement were tested for comparison, and an additional reinforced concrete beam was tested with minimum shear reinforcing per the ACI code. It was found that the addition of a 0.75% fiber volume fraction improved the shear strength of beams without shear reinforcement by at least $4.0\sqrt{f'_c}$ [ksi] ($0.33\sqrt{f'_c}$ [MPa]). Also, the test results showed that when compared with the reinforced concrete beam with shear stirrup reinforcement satisfying the 2008 ACI Code, all three fiber types, when used in a volume fraction of 0.75% or greater, could be used instead of the minimum required stirrup reinforcement per the ACI Code. Overall, the potential for HPFRCC can clearly extend to uses where damage tolerance is required or reinforcement congestion is troublesome for earthquake-resistant design (Parra-Montesinos, 2005).

2.7  Previously Conducted HPFRCC Analytical Modeling

Murugappan et al. (1993) presented an analytical biaxial failure envelope model for steel-fiber-reinforced concrete based on a four-parametric model proposed by Ottosen (1977) for plain
It was proposed that the steel fibers provided a confining pressure in the direction perpendicular to the stress plane, with the confining pressure being taken as the post-cracking tensile strength of the fiber-reinforced concrete. However, there were some discrepancies between the analytical model and their experimental results, and the model was limited to the compression-compression region of behavior. Hu et al. (2003) suggested a new biaxial failure model for steel fiber-reinforced concrete based on stretching and distorting an inclined ellipse, while ensuring that the biaxial failure envelope maintains convexity; the curve avoids any inflection points to limit numerical difficulties associated with the corner effects. An isotropic hardening rule was adopted, and the yield and loading envelopes were obtained by scaling the failure envelope. The stress-strain relationship was assumed to behave in an elastoplastic manner after initial yielding, as had been observed by previous researchers. The results showed reasonable agreement with previous experiments; however, the model was calibrated without any experimental data in the tension-compression and tension-tension regions, and the model is not applicable for high-performance fiber-reinforced concrete because it does not account for pseudo-strain hardening after initial yielding.

Seow and Swaddiwudhipong (2005) proposed a unified closed-form approach to define a failure criterion for both normal strength concrete and steel-fiber-reinforced concrete, without curve-fitting to experimental data. The model was designed for regular steel fiber-reinforced concrete, and the main parameters were the fiber volume fraction, the fiber aspect ratio, the uniaxial concrete compressive strength, the fiber yield stress, and the fiber bond strength. The main shortcoming of the model is that it cannot capture the significant post-peak strain and pseudo-strain hardening of HPFRCC. Swaddiwudhipong and Seow (2006) reported an experimental program to validate the relative accuracy of the previously proposed failure
A series of plate specimens with fiber volume fractions varying from 0.5% to 1.5% were constructed and tested under biaxial loading. The model was implemented into a user subroutine in ABAQUS and was found to reasonably predict the biaxial stress-strain behavior of plate specimens. Further parametric studies were conducted on previously tested beam specimens, and the model was able to predict the failure loads of the beams within 15%, but due to the neglect of bond-slip and other parameters, there was a sizable discrepancy between the analytical and experimental strains and displacements (Swaddiwudhipong & Seow, 2006).

A cyclic analytical model for HPFRCC has recently been developed by project colleagues at the University of Michigan. Hung and El-Tawil (2009) developed a material model for HPFRCC where the material is considered homogenous during the initial stages, and crack localization is determined by a strain criterion. The uniaxial stress-strain curves are largely a curve fit from previous experimental results conducted by Liao et al. (2006) at the University of Michigan. The cyclic loading stress-strain functions from Han et al. (2003) were adopted, with the modification of using Hognestad’s parabolic function to describe the hardening behavior before the peak compressive load. Shear stiffness retention was linearly decreased as a function of the normal strain on the crack plane, and LS-DYNA was used to perform the analyses. The proposed model was investigated with a comparison to experimental data from shear walls tested under reversed cyclic loading with HPFRCC and a 1.5% volume fraction of steel twisted fibers (from Parra-Montesinos et al. (2006)). Other efforts by Hung include the modeling of two 18-story coupled wall systems. In these models, one system utilized HPFRCC in the coupling beams and the plastic hinging region of the walls, while the other model implemented traditional RC coupled walls and coupling beams. The coupling beams using HPFRCC had a reduced and simplified reinforcement layout, and when subjected to a seismic analysis it was shown that the
buildings performed comparably with respect to drift and rotation. Also, the crack patterns in the coupling beams significantly differed between the two models. The HPFRCC coupling beams were only slightly cracked, while the RC coupling beams had clearly experienced more damage, thus requiring more thorough and costly repairs. Since the material model developed by Hung and El-Tawil (2009) was not based upon a suite of experimental data, a model calibrated to the results of the multi-axial tests conducted at the University of Illinois could potentially provide a more reliable finite element result with respect to the HPFRCC material behavior and the damage evolution of HPFRCC in structural components.
CHAPTER 3. MATERIAL TESTING RESULTS

This chapter provides an overview of the experimental program (and results obtained to date) performed on plain concrete and HPFRCC material specimens under various loading states, such as uniaxial compression, biaxial compression, and indirect tension, as well as an overview of related tests performed by project colleagues at the University of Michigan on similar materials. The plain and HPFRCC specimens tested at the University of Illinois were plate specimens under multi-axial loading for material characterization, while the University of Michigan tests included preliminary HPFRCC material testing, as well as coupled wall systems with both HPFRCC and plain reinforced concrete coupling beams. Results from these tests will be used to construct failure surfaces and to inform other modeling parameters to be used later in finite element modeling.

3.1 Preliminary HPFRCC Material Testing

As part of the NEES research project titled “Innovative Applications of Damage Tolerant Fiber-Reinforced Cementitious Materials for New Earthquake-Resistant Structural Systems and Retrofit of Existing Structures,” Liao et al. (2006) initially explored and established the mechanical properties of six different high-performance fiber-reinforced concrete mixes. Each of these mixes had at least some differences from one another with respect to the mixture proportions (including the quantity of admixtures) or in the type and volume fraction of fibers. Of the six mixes, two specific ones were further explored, called as NEES Mix #4 (NM4) and NEES Mix #6 (NM6). The main objective in developing these mixes was to obtain a strain-hardening, self-consolidating concrete mix with 28-day strength of between 5 ksi (34.5 MPa) and 9 ksi (62.1 MPa). Another series of tests was conducted on specimens as described by
Sirijaroonchai (2009). These specimens were composed of a mortar mix (MM) with two different types of fibers, with fiber volume fractions of 1.0, 1.5, and 2.0 percent. Further details about the materials and mixture proportions will be described in the following section.

3.2 Materials and Mixture Proportions

All concrete test specimens were made using ASTM Type III Portland cement. Each concrete was made using class C fly ash, rather than class F. Class C fly ash was used because it has a higher early strength, and it allows the pozzolanic reactions to begin earlier. The coarse aggregate used in NM4 and NM6 was a crushed limestone, with a maximum aggregate size of $\frac{1}{2}$ in. (13 mm) and a specific gravity of about 2.70. The fine aggregate for all mixes was #16 flint silica sand supplied by the U.S. Silica Company; the fine aggregate for MM had an ASTM gradation of 30-70, while that for NM4 and NM6 had an ASTM gradation of 50-70. ADVA® Cast 530 was the polycarboxylate type superplasticizer used in each concrete mixture. For NM4 and NM6, the amount of superplasticizer in each concrete mix was prescribed, but for MM, additional superplasticizer was added when the mix proved visually to be too dry. An additional viscosity modifying admixture (VMA) was used in NM4 and NM6 to enhance the viscosity and to reduce fiber segregation in the presence of relatively high water-to-cementitious ratios (Liao et al., 2006). The VMA used was RHEOMAC® VMA 362. Between NM6 and NM4, NM6 is the more economical mix design because it reduces the amount of sand, water, superplasticizer, and VMA to be used. The water-to-cementitious material ratios for MM, NM4, and NM6 are 0.35, 0.45, and 0.43, respectively. Table 3-1 displays the proportions of each mix by weight of cement, and Table 3-2 shows the normalized mixture proportions by weight. The MM had six different mixes: 3 different volume fractions (1%, 1.5%, and 2%) and two different fiber types
(Spectra and Hooked). NM4 and NM6 only used the hooked fibers; however, NM4 explored the use of the hooked fibers at three different volume fractions (1%, 1.5%, and 2%).

Table 3-1. Mixture Proportions by Weight of Cement

<table>
<thead>
<tr>
<th>Matrix Type</th>
<th>Mortar Mix</th>
<th>NEES Mix 4</th>
<th>NEES Mix 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type III (Early age)</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Aggregates</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silica Sand (Flint)</td>
<td>1</td>
<td>2.5</td>
<td>2.2</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>-</td>
<td>1.25</td>
<td>1.2</td>
</tr>
<tr>
<td>Fly Ash Class C</td>
<td>0.15</td>
<td>0.875</td>
<td>0.875</td>
</tr>
<tr>
<td>Chemical Admixtures</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>*</td>
<td>0.0055</td>
<td>0.005</td>
</tr>
<tr>
<td>VMA</td>
<td>-</td>
<td>0.065</td>
<td>0.038</td>
</tr>
<tr>
<td>Water</td>
<td>0.4</td>
<td>0.84</td>
<td>0.8</td>
</tr>
<tr>
<td>Fibers</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Types of Fibers</td>
<td>Hooked, Spectra</td>
<td>Hooked</td>
<td>Hooked</td>
</tr>
<tr>
<td>Percent Volume Fraction</td>
<td>1.0, 1.5, and 2.0</td>
<td>1.0, 1.5, and 2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>28-Day Compressive Strength, ksi</td>
<td>8.0 (55.2)</td>
<td>5.1 (35.2)</td>
<td>5.5 (37.9)</td>
</tr>
</tbody>
</table>

*Superplasticizer added as needed when the MM was too dry
Table 3-2. Normalized Mixture Proportions by Weight (Total Weight = 1.00)

<table>
<thead>
<tr>
<th>Matrix Type</th>
<th>Mortar Mix</th>
<th>NEES Mix 4</th>
<th>NEES Mix 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type III (Early age)</td>
<td>0.392</td>
<td>0.146</td>
<td>0.155</td>
</tr>
<tr>
<td>Aggregates</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silica Sand (Flint)</td>
<td>0.392</td>
<td>0.365</td>
<td>0.342</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>-</td>
<td>0.183</td>
<td>0.187</td>
</tr>
<tr>
<td>Fly Ash Class C</td>
<td>0.059</td>
<td>0.128</td>
<td>0.136</td>
</tr>
<tr>
<td>Chemical Admixtures</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>*</td>
<td>0.0008</td>
<td>0.00078</td>
</tr>
<tr>
<td>VMA</td>
<td>-</td>
<td>0.0095</td>
<td>0.0059</td>
</tr>
<tr>
<td>Water</td>
<td>0.157</td>
<td>0.123</td>
<td>0.124</td>
</tr>
<tr>
<td>Fibers</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Types of Fibers</td>
<td>Hooked, Spectra</td>
<td>Hooked</td>
<td>Hooked</td>
</tr>
<tr>
<td>Percent Volume Fraction</td>
<td>1.0, 1.5, and 2.0</td>
<td>1.0, 1.5, and 2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>28-Day Compressive Strength, ksi</td>
<td>8.0 (55.2)</td>
<td>5.1 (35.2)</td>
<td>5.5 (37.9)</td>
</tr>
</tbody>
</table>

*Superplasticizer added as needed when the MM was too dry

Two different types of fibers were used in the present study: a hooked steel fiber and an ultra-high molecular weight polyethylene fiber. The hooked steel fibers were Dramix® RC-80/30-BP with a length of 1.2 in. (30 mm), a diameter of 0.015 in. (0.38 mm), and a tensile strength of 334 ksi (2300 MPa). The Dramix® hooked fibers were made of high strength steel and are a trademark of Bekaert. The Spectra® fibers had a length of 1.5 in. (38 mm), a diameter of 0.0015 in. (0.038 mm), and a tensile strength of 375 ksi (2580 MPa). The Spectra® fibers were made of high molecular weight polyethylene and are a trademark of Honeywell. When compared with other polymeric fibers, Spectra® fibers have a higher strength and higher elastic modulus. Table 3-3 shows a summary of the fiber properties used in this investigation, and Table 3-4 displays the weight of each type of fiber type by volume fraction.
For NM4 and NM6, a 1.5% fiber volume fraction of hooked steel fibers was used. For the MM specimens, fiber volume fractions of 1.0, 1.5, and 2.0 percent were used of each fiber type (hooked steel and Spectra®) in different specimens. Figure 3-1 shows examples of the Spectra fibers and hooked steel fibers.
3.3 Preliminary Material Property Testing at the University of Michigan

Liao et al. (2006) conducted uniaxial compression and uniaxial tension tests on NM4 and NM6 at the University of Michigan. The uniaxial compression tests on 4 in. (102 mm) diameter by 8 in. (203 mm) tall cylinders were done according to ASTM C39. Three linear variable differential transformers (LVDTs) were used to measure the strains in the specimen, and they were removed to capture post-peak response. NM4 was found to have an average 28-day compressive strength of 5.1 ksi (35.2 MPa). Direct tension tests were conducted on dog-bone shaped tensile specimens. The total length of each dog-bone specimen was 21 in. (533 mm), and the cross-section of the middle portion of the specimen, where the failure occurs, was 1 in. (25 mm) by 2 in. (51 mm). Displacement was applied to the specimen at a rate of 0.05 in./min. (1.27 mm/min.), and the elongation was measured by two LVDTs over a gauge length of 7 in. (178 mm). For NM4, the average post-cracking tensile strength was found to be 502 psi (3.5 MPa) at a 0.25% strain from direct tension tests. For NM6, the 28-day cylinder compressive strength was 5.5 ksi (37.9 MPa), and the average post-cracking tensile strength was 503 psi (3.5 MPa) at a strain of 0.45% from direct tension tests (Liao et al., 2006). More detailed information (including stress-strain plots) for this tensile behavior is presented later in Sections 3.7.3 and 3.7.4.

To explore the effect of different types of fibers and varying volume fractions, Sirijaroonchai (2009) tested 1.0, 1.5, and 2.0 percent volume fractions of both Dramix® hooked steel fibers and Spectra® ultra-high molecular weight polyethylene fibers. These fiber volume fractions were placed into the mortar mix, whose proportions are given in Table 3-1 and Table 3-2. The MM specimens were subjected to uniaxial compression, uniaxial tension, and triaxial compression. The uniaxial compression tests were conducted as previously described per ASTM
C39; however, the cylinder specimens had a 3 in. (76 mm) diameter and a 6 in. (151 mm) height. Results for several key parameters of the average test results from the uniaxial compression tests are shown in Table 3-5, with the post-peak strain at a residual strength of 40% $f'$ included to illustrate the ductility of the material. The test results showed that the addition of fibers for the HPFRCC cylinders increased both the peak strength and its corresponding strain when compared to the plain mortar specimens. More detailed information (including stress-strain plots) for this compression behavior is presented later in Sections 3.7.1 and 3.7.2. Uniaxial tensions tests were conducted on the MM specimens using the same direct tension test previously outlined; however, a reduced displacement rate of 0.025 in./min. (0.64 mm/min.) was applied. A summary of the uniaxial tension tests is displayed in Table 3-6, and a further discussion of the uniaxial tension behavior and post-peak response is provided in Section 3.7 on the overall uniaxial response of these materials.

### Table 3-5. Summary of MM Uniaxial Compression Tests from Sirijaroonchai (2009)

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>Volume Fraction (%)</th>
<th>Young's Modulus ksi (MPa)</th>
<th>Peak Maximum Strength ksi (MPa)</th>
<th>Strain (%)</th>
<th>Post-Peak Strain at 40% of $f'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spectra</td>
<td>1.00</td>
<td>5421 (37376)</td>
<td>8.01 (55.19)</td>
<td>0.31</td>
<td>1.23</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>6528 (45009)</td>
<td>7.57 (52.21)</td>
<td>0.22</td>
<td>1.79</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>4292 (29592)</td>
<td>7.24 (49.93)</td>
<td>0.27</td>
<td>1.88</td>
</tr>
<tr>
<td>Hooked</td>
<td>1.00</td>
<td>3083 (21257)</td>
<td>8.25 (56.91)</td>
<td>0.29</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>5360 (36956)</td>
<td>8.75 (60.30)</td>
<td>0.31</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>5288 (36459)</td>
<td>8.3 (57.23)</td>
<td>0.26</td>
<td>0.85</td>
</tr>
<tr>
<td>Mortar</td>
<td></td>
<td>3856 (26586)</td>
<td>6.27 (43.25)</td>
<td>0.21</td>
<td>0.43</td>
</tr>
</tbody>
</table>
Table 3-6. Summary of MM Uniaxial Tension Tests from Sirijaroonchai (2009)

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>Volume Fraction (%)</th>
<th>Mortar Mix</th>
<th>First Crack</th>
<th>Peak Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Stress ksi (MPa)</td>
<td>Strain (%)</td>
</tr>
<tr>
<td>Spectra</td>
<td>1.00</td>
<td>0.15 (1.03)</td>
<td>0.024</td>
<td>0.46 (3.15)</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>0.14 (0.96)</td>
<td>0.021</td>
<td>0.48 (3.24)</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>0.13 (0.93)</td>
<td>0.019</td>
<td>0.45 (3.09)</td>
</tr>
<tr>
<td>Hooked</td>
<td>1.00</td>
<td>0.16 (1.13)</td>
<td>0.01</td>
<td>0.51 (3.48)</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>0.19 (1.33)</td>
<td>0.013</td>
<td>0.61 (4.24)</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>0.18 (1.25)</td>
<td>0.017</td>
<td>0.58 (4.00)</td>
</tr>
<tr>
<td>Mortar</td>
<td>0.12 (0.82)</td>
<td>0.013</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Sirijaroonchai (2009) also reported a series of passive triaxial compression tests on HPFRCC specimens with Spectra® and hooked steel fibers in 1.0, 1.5, and 2.0 percent fiber volume fractions. The tests were performed by casting HPFRCC specimens inside of steel tubes with a 3 in. (76 mm) nominal diameter and a 7 in. (178 mm) height. The available level of passive confinement was controlled by changing the thickness of the tube. For this study, steel tube thicknesses were 1/16 in. (1.6 mm) and 1/8 in. (3.2 mm), which corresponded to a maximum confining pressure of 6 ksi (41 MPa) and 7.5 ksi (52 MPa), respectively. The loading was applied vertically to the end of each cylindrical HPFRCC specimen through 0.5 in. (13 mm) steel loading plates. Three LVDTs were placed along the side of the steel tube to obtain the longitudinal deformation, and two strain gauges were attached on opposite sides of the tube at mid-height. One strain gauge was oriented longitudinally, while the other was in the circumferential direction. The longitudinal strain gauge was used to compare with the LVDT results so that the friction between the specimen and the tube could be evaluated. The circumferential strain gauge reading was used to measure the expansion of the steel, and using
characteristic stress-strain curves obtained from coupon tests from the steel tubes, the circumferential strain could then be converted to circumferential stress. Then, with the circumferential stress, radius of the cylinder, and thickness of the steel tube all known, the confining pressure was computed. Details of the described triaxial compression test are displayed in Figure 3-2.

![Figure 3-2. Triaxial Test Setup (Sirijaroonchai, 2009)](image)

From the triaxial compression tests conducted by Sirijaroonchai (2009), it was found that the overall stress-strain response was not heavily influenced by the type of fiber or the volume fraction. This may be largely attributed to the heavy confinement afforded to the specimen with respect to the comparatively small confinement from the fibers. However, it should be noted that the strength of the hooked and Spectra specimens was slightly greater than the plain mortar specimens under both confining pressures. Figure 3-3 displays the average stress-strain responses of the MM HPFRCC specimens under triaxial loading. The nomenclature for the figure is as follows: TXC(Confinement Level)-(Fiber Type or Mortar). For the confinement level, “M” refers to the 7.5 ksi (52 MPa) passive confinement, where “S” denotes the 6 ksi (41
MPa) passive confinement. For the fiber type, “H”, “S”, and “M” refer to hooked fibers, Spectra fibers, and Mortar specimens, respectively.

![Graph showing average stress-strain responses](image)

**Figure 3-3. Average Stress-Strain Responses in Longitudinal Direction under Two Levels of Confining Pressure from Sirijaroonchai (2009)**

Also, the minimum volumetric strain ($\varepsilon_{v,\text{min}}$) of the Spectra specimens was double the value found for the mortar specimens. The minimum volumetric strain represents the ability of the specimen to expand laterally in the hardening regime before failure, and it is computed as the lowest value of the volumetric strain during loading. Typical specimens produce an initial negative volumetric strain, as the longitudinal strain is more pronounced than the radial strain, but as damage progresses, lateral expansions occur much more rapidly than the longitudinal deformation, and the result is an overall positive volumetric strain. Thus, if a material has a
“smaller” minimum volumetric strain, then it is able to continue to withstand an increased longitudinal deformation without the rapid progression of lateral expansion that occurs with softening of the material. Table 3-7 displays a summary of the triaxial compression test results from Sirijaroonchai (2009).

### Table 3-7. Summary of Triaxial Compression Tests from Sirijaroonchai (2009)

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>MM Volume Fraction (%)</th>
<th>Confining Pressure</th>
<th>6 ksi (41 Mpa)</th>
<th>7.5 ksi (52 Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( f_{\text{max}} ) ksi (MPa)</td>
<td>( \varepsilon_{\text{max}} ) (%)</td>
<td>( \varepsilon_{\text{v,min}} ) (%)</td>
</tr>
<tr>
<td>Spectra</td>
<td>1.00</td>
<td>21.2 (146.0)</td>
<td>0.025</td>
<td>-0.53</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>21.5 (148.1)</td>
<td>0.04</td>
<td>-0.61</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>20.5 (141.2)</td>
<td>0.045</td>
<td>-0.61</td>
</tr>
<tr>
<td>Hooked</td>
<td>1.00</td>
<td>21.5 (148.3)</td>
<td>0.015</td>
<td>-0.55</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>22.1 (152.4)</td>
<td>0.022</td>
<td>-0.55</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>22.8 (157.4)</td>
<td>0.018</td>
<td>-0.59</td>
</tr>
<tr>
<td>Mortar</td>
<td></td>
<td>18.9 (130.3)</td>
<td>0.013</td>
<td>-0.33</td>
</tr>
</tbody>
</table>

#### 3.4 UIUC Experimental Program Overview

Hooked steel fiber, Spectra fiber, and plain concrete mixes were investigated. The Dramix® hooked steel fibers are made of high strength steel by Bekaert, while the Spectra fibers are strong and durable white polyethylene fibers made by Honeywell. For the multi-axial testing regime undertaken at the University of Illinois at Urbana-Champaign, the mixes were first cast individually as 5.5 x 5.5 x 1.5 in. (140 x 140 x 38 mm) specimens, a size similar to that used in historical concrete biaxial tests (Kupfer et al., 1969; Liu et al., 1972; Tasuji et al., 1978;
Maekawa & Okamura, 1983; Yin et al., 1989; Lan & Guo, 1999; Hussein & Marzouk, 2000a; Lee et al., 2004). Three concrete mixtures were explored: the Mortar Mix (MM), NEES Mix #4 (NM4), and NEES Mix #6 (NM6). The details of each mix were previously described in Section 3.2. Specimens were cast with Spectra fibers, hooked steel fibers, or without fibers for the MM, while only hooked steel fiber and plain concrete mixes were batched for NM4 and NM6.

To explore the influence of specimen type on orientation of the fibers at casting (and therefore on eventual biaxial behavior), large 6.5 x 6.5 x 18 in. (165 x 165 x 457 mm) “loaves” of the MM were also cast, to ensure a random orientation of the fibers. These loaves were then cut and trimmed to the aforementioned 5.5 x 5.5 x 1.5 in. (140 x 140 x 38 mm) specimen size using a diamond precision saw. Upon visual inspection, it was clear that the fibers were well-dispersed and randomly oriented in the loaf specimens. The first generation of tests utilized the MM and NM4 designs, but after those individually cast specimen results were compared to the loaf specimens, the decision was made to explore only casting in loaves for the eventual NM6 design (Foltz et al., 2008). Figure 3-4 illustrates how the larger loaf specimens were cut and trimmed to the smaller individually sized specimens. For the MM specimens, 1.0, 1.5, and 2.0 percent fiber volume fractions were used for both fiber types, while only a 1.5 percent hooked steel fiber volume fraction was used for the NM4 and NM6 specimens.
Table 3-8 displays the testing specimen matrix, as well as the average uniaxial plate compressive strength. When comparing the individually cast plate specimens to those cut from large loaves, it can be seen that the uniaxial compressive strength of the individually cast plates was significantly larger. Also, during testing, the individually cast specimens experienced an abrupt failure with very limited residual strength. This indicated that the orientation of the fibers for the individually cast specimens was mostly in the plane of the loading, thus the results of the experimental program will focus on the specimens cut from the HPFRCC loaves. Further, when comparing the uniaxial plate strength of the HPFRCC loaf specimens with the cylinder specimens discussed in Section 3.3 from the University of Michigan, it was found that the uniaxial compressive strength was slightly less than the cylinder strength. This can be attributed to the effect of using the friction reducing brush platens to test the plate specimens, as seen by previous researchers (Yin et al., 1989; Lee et al., 2004). In addition to the plate specimens, 15 NM6 4 x 8 in. (102 x 203 mm) cylinder specimens were cast and tested according to ASTM C39 in pure compression for comparison with the plate uniaxial compression results. Further
discussion of the uniaxial results will be provided in Section 3.7, and the complete uniaxial
compressive plate and cylinder strengths for each type of mix are provided in Table 3-9.

Table 3-8. Specimen Testing Summary

<table>
<thead>
<tr>
<th>Mix</th>
<th>Specimen Type</th>
<th>Fiber Type</th>
<th>Average Uniaxial Plate Compressive Strength, ksi (Mpa)</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar Mix</td>
<td>Individual</td>
<td>Hooked</td>
<td>10.2 (70.1)</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Spectra</td>
<td>8.7 (60.2)</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Plain</td>
<td>8.6 (59.4)</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>Loaf</td>
<td>Hooked</td>
<td>6.6 (45.8)</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Spectra</td>
<td>5.8 (39.9)</td>
<td>18</td>
</tr>
<tr>
<td>NEES Mix #4</td>
<td>Individual</td>
<td>Hooked</td>
<td>6.3 (43.6)</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Plain</td>
<td>7.6 (52.7)</td>
<td>10</td>
</tr>
<tr>
<td>NEES Mix #6</td>
<td>Loaf</td>
<td>Hooked</td>
<td>4.9 (33.5)</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Plain</td>
<td>5.4 (37.3)</td>
<td>12</td>
</tr>
</tbody>
</table>

3.5 Specimen Preparation

The mixing protocol outlined by Liao et al. (2006) was followed when batching the specimens, under the supervision of project partner colleagues at the University of Michigan. Two specific mixing procedures for self-consolidating concrete were adopted: pouring the pre-mixed water with the chemical admixtures into the mix in several steps to provide a homogenous matrix, and reducing the coarse-to-fine aggregate ratio to provide a well-developed paste layer to fully coat the coarse aggregate. First, the water was mixed with the superplasticizing agent and viscosity modifying admixture (if needed). Then, the cement, sand, and Class C fly ash were mixed together using a concrete pan mixing machine for about 30 seconds. At this point, approximately half of the liquid solution was added to the mix until the dry components were fully mixed with the liquid components. After mixing for about one minute, half of the
remaining liquid was poured into the mix. After another minute of mixing, half of the remaining liquid was again poured into the concrete mixture. Then, after still another minute of mixing, the remainder of the liquid solution was poured into the mix. Next, all of the coarse aggregate (if needed) was added and mixed for about two minutes. Finally, fibers were slowly added incrementally to the mix. During the addition of fibers, special care was made to ensure that they did not clump, especially for the Spectra fibers. After mixing for about three additional minutes, the HPFRCC was ready to be poured. Before pouring the specimens, the slump flow test (EFNARC, 2002) was performed on the HPFRCC batch. Rather than measuring the loss of height, as in the standard ASTM slump test, the slump flow test measures the average diameter of the concrete spread in two perpendicular directions. In general, higher slump flow corresponds to an increased ability to fill formwork. The slump flows for NM4 and NM6 were 22.6 in. (575 mm) and 22.2 in. (565 mm), respectively. Although these slump flow values are in the EFNARC (2002) SF1 slump flow class, the least flowable of recommended SCC mixes, they do indicate remarkable slump-flow for a fiber reinforced concrete mix. Also, when compared to other fiber reinforced mixes with similar fiber volume fractions, the slump-flow is comparable (Liao et al., 2006). Once the HPFRCC was deemed adequate, the concrete was cast into plastic molds and placed on a vibrating table to achieve sufficient compaction. After each concrete pour, specimens were kept in their molds and covered with plastic sheets for about 24 hours. They were removed from the molds and placed into a water curing tank for at least another 28 days. All specimens were allowed to dry for at least 48 hours prior to testing. As previously noted, specimens cast as loaves were cut with a diamond precision saw to 5.5 x 5.5 x 1.5 in. (140 x 140 x 38 mm). To ensure uniform biaxial stress and strain fields, the four sides of each specimen were then ground to achieve flat edges and right-angle corners. A problem reported
during some previous testing by Maekawa and Okamura (1983) was that local damage would occur at the interface between the specimen and the platen as the result of a lower local strength due to bleeding of the concrete; however, trimming the edges of the specimens to avoid these areas proved to be effective in preventing the damage localization at the edges in this study.

3.6 Testing Procedures

The plate specimen experiments were displacement-controlled, with the ratio of principal strains varied in an effort to obtain a comprehensive understanding of the materials’ biaxial behavior. Testing was conducted using a 112 kip (500 kN) INSTRON biaxial servo-controlled hydraulic frame. A closed-loop system in displacement control was used to capture the post-peak response of the specimens, with all of the biaxial compressive loads applied simultaneously. Displacement control was provided by AC linear variable differential transformers (LVDTs) attached to each hydraulic actuator. Each axis of loading had one actuator slaved to the other (master) actuator through digital line connections. The closed-loop control of the actuators was executed using INSTRON 8500 and INSTRON 8800 controllers. Similar to what was done by previous researchers (Kupfer et al., 1969; Nelissen, 1972), frictional confinement at the edges of the test specimens by the loading platens was minimized by using brush-type loading platens. The brush platens were pin-connected to testing fixtures (including simple guide-ways to ensure planar loading), which were then in turn mounted to the load cell of each actuator. For compression (and equal biaxial compression) loading, the standard applied strain rate used was 0.01 in./min (0.25 mm/min). For intermediate targeted stress ratios, $\sigma_1/\sigma_2$, the standard applied strain rate was simply reduced in the horizontal direction to try and achieve the desired stress ratio. Figure 3-5 shows both a line drawing and a picture of the typical test setup.
All strain and displacement measurements were obtained using the non-contact Krypton K600 Coordinate Measuring Machine (DMM). The Krypton CMM can obtain the three-dimensional location of many small light-emitting diodes (LEDs) to an accuracy of +/- 0.0008 in. (0.02 mm) at a sampling rate of up to 1000 readings per second. For these tests, LEDs were placed on an overall 3 in. x 3 in. (75 mm x 75 mm) grid, with 1.5 in. (38 mm) spacings centered on the specimen. To obtain additional out-of-plane data, two 0.25 in. (6 mm) stroke LVDTs were positioned on special frames and placed such that they were touching the center of each face of a test specimen, as shown in Figure 3-5. (Early tests were conducted using 9 LEDs on the front of the specimen, with a single LVDT touching the center of the back of the specimen; however, it was later found that the out-of-plane data was less noisy when two LVDTs were used to capture this behavior.) Figure 3-6 shows the layout of the Krypton LEDs to measure 3-D deformations on the front of the specimen (left), as well as the positioning of the LVDTs on the front and rear of the specimen to obtain out of plane displacements (right).
The analog output signals from the measuring devices were connected to input channels of the data acquisition system. Four-axis control of the system was synchronized with a PC using Labview. This allowed for the synchronization of the start of each test, as well as of the load, displacement, and LVDT data collection with time. In addition, several two-axis plots were displayed with real-time updates to monitor the performance and behavior of the specimens during testing. The Krypton measuring system had its own data acquisition software, so the two sets of data were later synchronized during post-processing. Once a specimen was secured in the testing frame, it was preloaded to about 225 lbs (1 kN) in the direction(s) of loading to remove any excess flexibility in the system and to ensure proper platen contact with the specimen.
3.7 Uniaxial Test Results

This section will describe the behavior of HPFRCC and plain concrete specimens subjected to both compressive and tensile uniaxial loading. The failure modes under uniaxial compression, and the typical compressive stress-strain behavior observed during experimental tests for both plates and cylinders will be reviewed. Also, the progression of damage in HPFRCC specimens subjected to uniaxial tension will be examined, and the uniaxial tensile stress-strain behavior will be explored.

3.7.1 Uniaxial Compression Failure Mode

To eventually better understand the response of plate specimens under more complex biaxial stress states, the uniaxial failure mode and stress-strain response are first explored. Under uniaxial compression the failure modes exhibited by the plate specimens were similar to classical uniaxial plain concrete plate tests, with crack formation at an angle of 20 to 40 degrees with the axis of loading and perpendicular to the larger unloaded out-of-plane surface. Figure 3-7 shows a failure surface of a loaf NM6 plate specimen subjected to uniaxial compression, which was typical of loaf HPFRCC specimens under similar loading.
Since NM4 and NM6 were deliberately designed not to be high strength concrete mixes, crack formation and propagation occurred in the cementitious mortar (around the coarse aggregate). For plain concrete, when the material is loaded in one direction without any confinement, the largest macrocracks develop in the direction of the loading. Thus for the uniaxial compression case, crack formation was characterized by a series of vertical tensile splitting cracks, resulting in a dramatic loss of capacity for the plain concrete specimens. However, with the addition of fibers, the growth of the macrocracks are retarded, and the material ultimately fails by the interconnecting of a multitude of small microcracks along a faulting zone inclining at an angle to the direction of the applied load. This type of failure is characterized as a faulting failure (Yin et al., 1989). Since the fibers are able to arrest the propagation of macrocracks, the fiber-reinforced specimens still exhibited significant ductility beyond peak loading.
3.7.2 Uniaxial Compression Stress-Strain Behavior

Due to the displacement-controlled nature of the experiments, loading was stable beyond the maximum load on the plate specimens, and the descending branch of the stress-strain curve as obtained out to fairly large deformations. Figure 3-8 shows a comparison of the average stress-strain response from NM6 HPFRCC and plain concrete results for both plate and cylinder specimens. It can be seen that the plain specimens failed abruptly, without any significant post-peak response; however, the HPFRCC exhibited a gradual descending branch and a sustained capacity of approximately 50% of the maximum applied load out to 3% strain. This significant deformation capacity is further evidence of the energy-dissipating capability of the material. These average stress-strain curves were found by averaging the applied stress across a group of specimens for a given strain obtained from the Krypton LEDs. When processing the data, the flexibility of the experimental test setup was obtained by comparing the relationship between the stiffness found using data from the loading platens and that obtained from the Krypton targets. Since it is possible for local strain accumulation to adversely influence the data collected at individual LEDs on the surface of a specimen, once the flexibility in the test setup was corrected for in the experimental data, reliable post-peak behavior for the fiber-reinforced specimens was then also able to be obtained from the actuators.

Figure 3-8 also shows that the HPFRCC cylinder specimens were slightly stiffer than the plain concrete specimens as the loading increased, but the stiffnesses were almost identical up to about 2 ksi (13.8 MPa). The higher HPFRCC stiffness can be attributed to the hooked steel fibers arresting the propagation and development of early cracking. The plate specimens, however, had a nearly identical stiffness with or without fibers. It is expected that the addition of fibers will not affect the stiffness in the linear range, but while plain concrete may begin to
undergo damage, the HPFRCC specimens are restrained by the fibers. For the plain plate specimens, each uniaxial test failed abruptly due to concrete crushing, so the initial damage and softening is not evident as in the plain cylinders. Also, it can be seen that the average uniaxial strengths of the similar type of plain and HPFRCC specimens only slightly differed, which is consistent with the literature (Yin et al., 1989). The average uniaxial compressive strengths of the concrete and HPFRCC plate specimens were 5.4 ksi (37.3 MPa) and 4.9 ksi (33.5 MPa), respectively. The average uniaxial compressive strengths of the concrete and HPFRCC cylinder specimens were 7.2 ksi (49.7 MPa) and 6.8 ksi (46.9 MPa), respectively. The magnitude of the compressive strength of the plates can be seen to be lower than for the cylinders, which is consistent with the literature (Yin et al., 1989; Lee et al., 2004). Lee et al. (2004) attributed this to the difference in geometric shapes and sizes, the effect of a more confined end condition for the cylinder specimens, and a lower absorption capacity of failure energy in the plate specimens due to the fact that they have four edges in the axial direction.

With regard to varying the fiber volume fraction, previous test results on MM specimens showed the general trend that, as fiber volume fraction increased from 1 to 2 percent, the unconfined uniaxial compressive strength from cylinder tests slightly decreased (Foltz et al., 2008). Further, the maximum stress was achieved in both the cylinders and plate specimens under uniaxial compression at a strain of approximately 0.003 for both the plain and HPFRCC specimens.
Poisson’s ratio was also investigated under uniaxial compression. During cylinder tests at the University of Illinois at Urbana-Champaign, the diametrical strain was obtained using a specially designed extensometer, while the longitudinal strain was obtained using a linear extensometer. The cylinder tests were also conducted using displacement-control, so the post-peak behavior was captured; however, again due to the brittle nature of the plain concrete specimens, they exhibited essentially no post-peak response.

Figure 3-9 shows the average longitudinal strain versus applied stress, as well as the average diametrical strain versus applied stress, for the NM6 cylinder tests. The diametrical strain measurement was limited to 0.01 due to the gauge length of the extensometer.
Poisson’s ratio was obtained from the initial portion of the stress-strain curve (applied stress of from about 1 ksi (6.9 MPa) to 3 ksi (20.7 MPa) for each of the specimens. The lower bound for this calculation was selected to eliminate the potential for seating of the specimen during experimentation to influence the result, while the upper bound was selected as a reasonable loading before the stress-strain curve began to depart from linearity. Poisson’s ratios for the plain and HPFRCC NM6 cylinders were found to be 0.24 and 0.22, respectively, while Poisson’s ratios for the plain and HPFRCC NM6 plate specimens were found to be 0.26 and 0.22, respectively. These results are consistent with typical values for uniaxially loaded plain concrete specimens (Kupfer et al., 1969). With respect to the addition of fibers, the fact that
Poisson’s ratio remained practically unchanged after adding fiber was also observed in previous research (Yin et al., 1989). One observation that can be made about the different behavior due to the effect of fibers on the Poisson’s ratio is that the percent of peak loading at which the rapid increase in volume of the concrete occurs is much larger. In typical plain concrete, microcracks begin to form due to the lateral tensile strains, and these cracks continue to grow and coalesce until failure. This same process of microcracking eventually occurs in the HPFRCC specimens; however, it happens at larger strains, and it is accompanied by a significant post-peak deformation capacity. From Figure 3-9, a comparison between the lateral growth of the NM6 hooked fiber specimens and the NM6 plain specimens shows the ductility of HPFRCC in all directions. Both fiber and plain cylinders behaved almost identically to about 80% of the maximum loading. As damage progressed in the specimens, the fiber reinforced concrete cylinders began to deform vertically and laterally. Once the capacity of the cylinder was reached, the HPFRCC cylinders showed significant ductility in each direction, while the plain cylinder specimens failed with essentially no post-peak behavior.

Table 3-8 displays a summary of the average uniaxial compressive strengths of all of the specimen types tested in this study. Since all of the individually cast plate specimens had anomalously high compressive strengths, only the plate specimens cast as loaves were included in the summary. For comparison, Table 3-9 includes uniaxial compressive cylinder tests conducted at the University of Michigan, as well as the results of the uniaxial compressive plate and cylinder tests performed at the University of Illinois.
Table 3-9. Summary of MM and NM6 Uniaxial Compressive Strengths

<table>
<thead>
<tr>
<th>Mix</th>
<th>Fiber Type</th>
<th>Volume Fraction</th>
<th>UM Cylinders Maximum Strength ksi (MPa)</th>
<th>UIUC Plates Maximum Strength ksi (MPa)</th>
<th>UIUC Cylinders Maximum Strength ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar Mix</td>
<td>Spectra</td>
<td>1.0%</td>
<td>8.01 (55.2)</td>
<td>6.35 (43.8)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5%</td>
<td>7.57 (52.2)</td>
<td>5.39 (37.2)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.0%</td>
<td>7.24 (49.9)</td>
<td>5.64 (38.9)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Hooked</td>
<td>1.0%</td>
<td>8.25 (56.9)</td>
<td>5.23 (36.1)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5%</td>
<td>8.75 (60.3)</td>
<td>7.23 (49.9)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.0%</td>
<td>8.3 (57.2)</td>
<td>7.48 (51.6)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Plain</td>
<td>-</td>
<td>6.27 (43.3)</td>
<td>8.6 (59.4)</td>
<td>-</td>
</tr>
<tr>
<td>NM6</td>
<td>Hooked</td>
<td>1.50%</td>
<td>6.8 (46.9)</td>
<td>4.9 (33.5)</td>
<td>6.8 (46.9)</td>
</tr>
<tr>
<td></td>
<td>Plain</td>
<td>-</td>
<td>-</td>
<td>5.4 (37.3)</td>
<td>7.2 (49.7)</td>
</tr>
</tbody>
</table>

3.7.3 Uniaxial Tension Failure Mode

For plain concrete specimens subjected to direct tension, the failure consists of a single crack which extends perpendicularly to the direction of loading, resulting in the inability of the specimen to withstand further load. HPFRCC specimens exhibit a very different tensile behavior. Figure 3-10 depicts the progression of damage to an HPFRCC specimen during a direct tension test at the University of Michigan. In Figure 3-10a, the initial cracking has occurred. Rather than exhibiting a single large crack, small microcracks have begun to form. The specimen is able to continue taking increased loading after this initial cracking as more fibers are activated and more microcracks begin to form. Figure 3-10b displays the specimen around the peak tensile stress – it can be seen that it is saturated with microcracks and that some large cracks have begun to develop. During the post-peak response of the specimen, the damage localizes to a single macrocrack, and the capacity of the specimen decreases as the fibers begin
to pull out of the specimen. The crack localization stage is displayed in Figure 3-10c. The comparison between the failure mode of plain concrete and HPFRCC illustrates the benefit that the addition of fibers has on the deformation capacity and energy absorption of concrete.

![Cracking Propagation](image)

(a) First few cracks  (b) Saturated cracks  (c) Localization

Figure 3-10. Cracking Propagation for MM Spectra 1% Direct Tension Test, from Sirijaroongchai (2009)

### 3.7.4 Uniaxial Tension Stress-Strain Behavior

The response of the HPFRCC specimens is linear until first cracking. As multiple cracking develops, the stiffness of the HPFRCC specimens reduces, but they are still able to accommodate increased loading. This ability to withstand additional stress beyond cracking has been coined pseudo-strain hardening. After the specimen is saturated with microcracks, the
damage localizes in one location, and the fibers begin to pull-out. This stage of damage corresponds to the ductile post-peak response of the direct tension specimens. Figure 3-11 displays the results of direct uniaxial tension tests conducted on hooked fiber and plain MM specimens at the University of Michigan. It can be seen that the addition of fibers dramatically increased both the tensile strength and the deformation capacity. In fact, 50% of the peak tensile strength was maintained to 1% strain.

Figure 3-11. Hooked Fiber MM Direct Uniaxial Tension Test Results

Figure 3-12 shows the results of the direct uniaxial tension tests on Spectra and plain MM specimens with varying fiber volume fractions. The addition of Spectra fibers increased the tensile strength by 400% over the plain concrete specimens, and an even greater residual strength was observed than seen with the hooked fiber direct tension specimens. When comparing the
response of the two fiber types, the hooked fibers exhibited a greater tensile strength; however, the Spectra fibers showed a greater ductility. For instance, at 2% strain, the hooked fibers could withstand about 25% of the peak stress, while the Spectra fibers were able to sustain about 75% of the maximum applied tensile stress.

Figure 3-12. Spectra Fiber MM Direct Uniaxial Tension Test Results

### 3.8 Biaxial Test Results

This section will describe the behavior of HPFRCC and plain concrete specimens subjected equal and unequal compressive biaxial loading. The failure modes under biaxial compression and the typical compressive stress-strain behavior observed during experimental tests for both plain and HPFRCC plate specimens will be explored.
3.8.1 Failure Mode

Van Mier (1986) showed that the increase in ultimate compressive strength of concrete due to small confining pressures in perpendicular directions is small; however, the magnitude of the ultimate compressive strength increases rapidly with an increasing magnitude of confining pressure in one direction, while the other perpendicular confining pressure remains small. The described scenario is identical to the biaxial tests conducted during this experimental program. Further, Horri and Nemat-Nasser (1986) showed theoretically that different failure modes of brittle materials are caused by different loading conditions. Thus, Yin et al. (1989) demonstrated that the effect of adding fibers into concrete is equivalent to providing some small confining compression in the unloaded directions. Under uniaxial loading, there is a small confining pressure in each perpendicular direction, so the strength effect is minimal, but the failure mechanism is changed from splitting to faulting. However, under biaxial loading, only the confining stress in the out-of-plane direction is small, so the effect of the confinement is significant on both the strength and failure mode.

The typical failure mechanism of plain concrete specimens was by tensile splitting. Under biaxial loading, the origination of a failure surface along a plane parallel to the large unloaded surface of the specimen resulted in an abrupt failure. However, the failure mechanisms experienced by the loaf fiber specimens under biaxial loading were considerably different. As described in previous research (Yin et al., 1989), these specimens experienced a faulting or shear failure due to the formation of multiple fault planes in the specimen. Similar to in previous tests, all specimens exhibited either single shear or multiple shear failure modes (Ren et al., 2008). The single shear failure mode can be identified by a single diagonal crack inclined at about 30° to the unloaded out-of-plane surface, resulting in two triangularly shaped prisms. The multiple
shear failure mode is similar to single shear, except the specimen fails along several inclined diagonal cracks, resulting in it being divided into a few triangular pyramids. Figure 3-13 shows examples of both the single shear and multiple shear failure modes for HPFRCC, as well as the tensile splitting failure that was common in plain concrete specimens. The individually cast specimens had relatively more fibers oriented in the plane of the specimen than in the 1.5” (38 mm) out-of-plane direction. As a result, when the individually cast specimens failed brittlely, concrete exploded in the out-of-plane dimension of the specimen. This demonstrated that the fibers were not oriented to properly provide passive confinement.

![Figure 3-13. Single Shear Failure Mode (left), Multiple Shear Failure Mode (center), and Tensile Splitting Failure (right)](image)

3.8.2 Stress-Strain Behavior

The biaxial stress-strain behavior is largely dependent upon the ratio of applied axial loads. As previously described, the loading ratio was varied by altering the horizontal strain rate in the biaxial testing machine. Figure 3-14 shows the average compressive response of the leading direction of the biaxially loaded NM6 specimens, and the average uniaxial compressive
response of the plain NM6 specimens is also shown for comparison. It can be seen that a similar peak stress was obtained for loading ratios of 0.5, 0.7, and 1.0. Also, significant residual strength was maintained to very large compressive strains; in fact, more than half of the peak strength was observed at strains as large as 3%. Figure 3-15 displays the average compressive response of the biaxially loaded NM6 in the trailing (horizontal) direction. The uniaxial compressive response has been shown on both plots for reference.
When comparing the two previous figures, it can be seen that the stiffness is essentially the same in both directions of loading, further indicating a thorough and random dispersion of fibers in the loaf specimens. Also, as noted in previous research when comparing results of HPFRCC uniaxial, biaxial, and triaxial tests, Young's modulus is essentially independent of the multi-axial loading (Sirijaroonchai et al., 2010). For the NM6 plain and hooked fiber plate specimens, Young’s modulus was about 2300 ksi (15.9 GPa). Another observation is that biaxially loaded specimens at lower loading ratios, such as seen for 0.3, experienced earlier softening with respect to the applied strain in the trailing direction. This result can be attributed to the significant accumulation of damage in the leading direction during the test. Since the loadings in each direction were begun simultaneously, the specimen had undergone considerable deformation in the leading direction before obtaining a substantial strain in the trailing direction.
Also, the strain at maximum applied stress shifted to about 0.4% for specimens subjected to a biaxial loading ratio of 0.5 or greater, while the strain at maximum stress was approximately 0.3% for the uniaxial specimens. Previous research shows the strain at the onset of significant spalling for short RC columns (analogous to coupling beams) occurs at about 0.5% (Berry & Eberhard, 2003). Since HPFRCC specimens exhibit residual strengths of about 70% of the peak strength at a strain of 1% and about 50% of the peak strength at a strain of 2%, the deformation capacity of HPFRCC can dramatically improve the seismic performance of such structural elements. For structural designers, the performance of structural elements is a critical consideration; therefore, it is important that both the core and the cover of the coupling beams can withstand design basis earthquakes without undergoing significant damage or jeopardizing the load carrying capacity of the structural component. Thus, the demonstrated deformation capacity and residual strength of HPFRCC is evidence of its potential as a material to withstand seismic events while maintaining the integrity of structural elements.

### 3.9 Failure Envelope Results

Non-dimensionalized ultimate strength data are shown as biaxial stress envelopes, depicted in Figure 3-16 and Figure 3-17. Stresses are reported as fractions of the average unconfined uniaxial compressive plate strength of the specimens for the particular concrete mixture and fiber volume fraction, $\sigma_{co}$. (Average uniaxial compressive plate strengths are given in Table 3-8.) Figure 3-16 shows the biaxial failure envelopes obtained from this testing program. This figure illustrates the effect of a more random fiber orientation by plotting the average result for each tested type of fiber, concrete mixture, and specimen (loaf vs. individually cast). The averaged curves were obtained by first normalizing each specific concrete batch by its average uniaxial value, and then the data points that had a similar failure stress ratio for a
particular specimen, mix, and fiber type were averaged to create the data points for the curve. For example, the average loaf Spectra curve was made by first normalizing the 1.0, 1.5, and 2.0 percent fiber volume fraction results by their respective average uniaxial values, and then data points with similar failure stress ratios were averaged together. Each averaged data point used to generate the failure envelope represents the results of 3 to 7 experimental tests, depending on the quantity of specimens available for the particular mix. Each plot has been normalized by its average uniaxial compressive performance, so the individual specimens were not actually weaker than the loaf specimens; they just did not benefit as much from the addition of a second principal confining stress. This is consistent with the results of Tasuji et al. (1978), which indicated that the increase in biaxial compressive strength for concrete is higher for concrete with a lower uniaxial compressive strength. Additionally, Figure 3-16 displays the plain MM and plain NM4 biaxial failure envelopes, and it can be seen that the results from the plain concrete mixes even experienced a greater biaxial strength increase than the individually cast HPF RCC specimens, reconfirming the reduced effect of fibers without a random dispersion throughout the concrete. A post-experiment “autopsy” of the individually cast specimens continued to confirm that the fibers were indeed aligned in the plane of the specimen.
In Figure 3-17, a comparison of various biaxial failure envelopes from the experimental test program with those obtained by other experiments from the literature on plain concrete is shown (Kupfer et al., 1969; Hussein & Marzouk, 2000a; Nelissen, 1972; Tasuji et al., 1978). It can be seen that the benefit of a biaxial stress state on the strength of the concrete is markedly increased with the addition of fibers in the HPFRCC specimens when comparing the average
biaxial results from the loaf NM6 specimens to the historical plain concrete results. When comparing the plain NM4 biaxial failure envelope to the aforementioned historical tests, it can be seen that the results align quite well. The average individual hooked MM specimens were also included in Figure 3-17, and it shows that the individual hooked MM specimens did not experience the same increase in relative biaxial strength as the historical tests.

Figure 3-17. Biaxial Strength Envelope Comparison of NM6 Loaf, HPFRCC Individual, and Plain NM4 Specimens with Historical Tests
Table 3-10 displays a summary of the average uniaxial and equal biaxial compressive strengths for each specimen type. Again, it can be seen that the individual specimens had a much greater uniaxial strength, thus they did not experience a significant increase in strength under biaxial compression. Meanwhile, all of the loaf specimens (hooked, Spectra, and NM6) experienced a much greater benefit under biaxial loading due to the passive confinement provided by the randomly oriented fibers; in fact, a strength increase of greater than 40 percent over their respective uniaxial strengths under equal biaxial loading was observed. The only anomaly observed was that the plain NM6 specimens demonstrated a reduction in strength under biaxial loading when compared with the uniaxial strength, and only a strength increase of about 10% was experienced when subjected to a loading ratio of 0.50. This was perhaps the result of an anomalously high uniaxial compressive strength for the plain NM6 specimens.
Table 3-10. Summary of Average Uniaxial and Equal Biaxial Compressive Strength for Each Specimen Type

<table>
<thead>
<tr>
<th>Mix</th>
<th>Specimen Type</th>
<th>Fiber Type</th>
<th>Volume Fraction</th>
<th>Average Uniaxial Compressive Strength, ksi (MPa)</th>
<th>Average Equal Biaxial Compressive Strength, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar Mix</td>
<td>Individual</td>
<td>Hooked</td>
<td>1.0%</td>
<td>10.2 (70.5)</td>
<td>10.3 (71.0)</td>
</tr>
<tr>
<td></td>
<td></td>
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<td>1.5%</td>
<td>10.1 (69.4)</td>
<td>9.3 (64.4)</td>
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<td></td>
<td></td>
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<td>10.2 (70.6)</td>
<td>10.8 (74.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Spectra</td>
<td>1.0%</td>
<td>9.1 (62.8)</td>
<td>9.2 (63.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5%</td>
<td>9.0 (61.8)</td>
<td>8.7 (60.1)</td>
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<td></td>
<td></td>
<td>2.0%</td>
<td>8.1 (56.0)</td>
<td>8.5 (58.9)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Plain</td>
<td></td>
<td>8.6 (59.4)</td>
<td>10.6 (73.1)</td>
</tr>
<tr>
<td></td>
<td>Loaf</td>
<td>Hooked</td>
<td>1.0%</td>
<td>5.2 (36.1)</td>
<td>10.2 (70.2)</td>
</tr>
<tr>
<td></td>
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<td>1.5%</td>
<td>7.2 (49.9)</td>
<td>10.1 (70.0)</td>
</tr>
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<td></td>
<td></td>
<td></td>
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<td>7.5 (51.6)</td>
<td>9.5 (65.3)</td>
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<td>1.5%</td>
<td>5.4 (37.2)</td>
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</tr>
<tr>
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<td>6.5 (44.6)</td>
<td>7.4 (51.1)</td>
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<td>6.7 (46.2)</td>
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<td>Plain</td>
<td></td>
<td>5.4 (37.3)</td>
<td>4.4 (30.7)</td>
</tr>
</tbody>
</table>

3.10 Split Cylinder Test Results

Split cylinder tests were conducted on 4 in. x 8 in. (100 mm x 200 mm) cylindrical specimens according to ASTM C496. In total, two plain NM6 cylinders and three hooked fiber NM6 cylinders were tested. An extensometer was developed similar to as used by previous
researchers to obtain a measure of the transverse deformation under loading (Nanni, 1988; Graybeal, 2006). This diametrical deformation measuring apparatus was spring-loaded to ensure constant contact with the cylinder, and two linear variable differential transformers (LVDTs) were used to measure the lateral deformation during loading; also, the load data was continuously collected and monitored using Labview. With this particular test setup, it was possible to visually monitor the load versus lateral expansion plot during testing. Figure 3-18 displays the extensometer used for the testing of the split-cylinders. It was essential that the LVDTs were sufficiently sensitive to measure the smaller diametrical strains that occur even before the initial cracking of the specimen.

Figure 3-18. Split Cylinder Diametrical Deformation Apparatus and Experimental Test Setup
Applied load versus diametrical deformation was obtained throughout testing, and the results are shown in Figure 3-19. In the nomenclature used to identify the test specimens, S indicates a split-cylinder test; N indicates the NM6 concrete mixtures; NP identifies a NM6 mixture without fibers; and the number indicates the specimen number. It can be seen that the specimens containing fibers sustained more load than the plain NM6 specimens; however, SN2 achieved a substantially larger load than any of the other specimens. While the strength results were not completely consistent among the fiber specimens, all of them were at least 80% stronger than the plain NM6 specimens. Also, the residual strength was much greater for the fiber-reinforced specimens. SNP2 had lost all of its capacity before reaching 0.1 in (2.54 mm) displacement of the actuator, while SNP1 had lost over half of its capacity. None of the fiber-reinforced specimens even experienced a 50% loss of capacity by the time that the test had reached its conclusion. The testing of the specimens was halted when the local deformation of the specimen at the location of the loading platens caused a larger portion of the platen to come into contact with the specimen. Admittedly, the results from the split cylinder tensile strength test are difficult to interpret after the first matrix cracking because of the unknown stress distribution after initial cracking, as observed by Naaman et al. (2007). However, with the precision instruments to identify first cracking, and with developed relationships of other properties from previous standard ASTM tests, the split cylinder test can be used as a reliable quality control test. Further, although the split cylinder test is not a direct measure of the tensile strength of HPFRCC, it does provide results demonstrating behavior similar to other performed tests, namely a greater load carrying capacity, a markedly larger deformation capacity, and a significant residual strength.
Figure 3-19. Split-Cylinder Load versus Deformation for NM6 Cylinders

3.11 Tension Results

A suite of uniaxial and biaxial compressive experiments were conducted, and this experimental program intended to test the remaining two loading scenarios (tension-compression and tension-tension) to complete the failure envelope. The experiments were to be direct tension tests, and they were to be performed using the biaxial testing machine with the plate specimens epoxied to the brush platens, as done in previous historical tests (Kupfer et al., 1969; Tasuji et al., 1978; Maekawa & Okamura, 1983; Hussein & Marzouk, 2000a; Calixto, 2002; Lee et al., 2004; Ren et al., 2008). Unfortunately, due to general experimental obstacles, ranging from
testing machine mechanical problems to controller availability to the veteran lab technician retiring, the tensile panel tension tests were unable to be completed. Therefore, the dogbone results provided by project colleagues at the University of Michigan will be used as the HPFRCC uniaxial tensile property for material modeling purposes.
CHAPTER 4. MATERIAL MODELING

The analytical phase of the research program focuses on the creation of finite element models using the ATENA software from Cervenka Consulting, which has a main strength in the nonlinear finite element modeling of structural concrete. This chapter will provide a background of the ATENA software, and an overview of the modeling of the HPFRCC material tests. The intent is to first validate the material parameters within ATENA by modeling the material tests described in Chapter 3, then the HPFRCC model can be extended to more complex structural components, such as coupling beams and pile-wharf connections, as described in Chapter 6 and Chapter 9, respectively.

4.1 ATENA Background

Since the conducted material tests were planar, the ATENA 2D software was used for the HPFRCC modeling parameter validation portion of the current research. The ATENA program is specially designed for the nonlinear finite element analysis of concrete and reinforced concrete structural behavior. The program consists of the solution core and the user interface. The solution core has the capability to analyze 2-dimensional and 3-dimensional continuum structures, and it consists of libraries of finite elements, material models, and solution methods. The User Graphic Interface enables access to the ATENA solution core, and it provides the user with the ability to graphically model a structure and to explore a visualization of the analytical results.

While many structures may be sufficiently modeled using a linear formulation, the focus of the current research is to capture the softening and post-peak behavior of HPFRCC, thus an iterative non-linear solution scheme must be used. ATENA is suited for both the modeling of
the material nonlinear behavior and the geometric nonlinear behavior. The geometric nonlinear behavior results when the deformations of the structure are large enough to require equilibrium equations to be used on the displaced shape. To capture the behavior of the displaced shape, ATENA uses the Updated Lagrangian formulation, so the governing equations are with respect to the most recent deformed configuration.

The SBETA model is the standard concrete material model used within ATENA. The concept of the SBETA material model includes the nonlinear behavior of concrete in compression (hardening and softening), fracture of concrete in tension, biaxial failure criterion, reduction of compressive strength after cracking, tension stiffening, reduction of the shear stiffness after cracking, and both fixed and rotating crack models. Since the SBETA material model has been designed to model regular reinforced concrete and since the user defined options are limited, it will be used for the modeling of reinforced concrete structural components where further details about the concrete properties are not known. Since detailed material tests were conducted on the HPFRCC material, another material model will be used that allows the user to input specific material properties and constitutive relationships.

HPFRCC was modeled using a fracture-plastic constitutive model. Specifically, the CC3DNonLinCementitious2User model was implemented for HPFRCC. As described in detail by Cervenka et al. (2010), the fracture-plastic model combines constitutive models for tensile (fracturing) and compressive (plastic) behavior. The fracture model considers a classical orthotropic smeared crack formulation and crack band model. It uses the Rankine failure criterion with exponential softening. The plasticity model is based on the Menetrey-William failure surface, with a return mapping algorithm for the integration of constitutive equations. The model is unique in its combination of both fracture and plasticity. A combined algorithm is
used that allows for the two models to be developed and formulated separately, such that it can handle cases when the failure surfaces of both models are active. Thus, the model can handle physical changes, for instance, concrete cracking, crushing under high confinement, and crack closure. Strain decomposition, as described by De Borst (1986), is used to combine the fracture and plasticity models together, where the strain is computed as the sum of the elastic, plastic, and fracturing components. The Rankine criterion is implemented for concrete cracking, and the crack opening is determined as a function of characteristic length. The characteristic length concept as a crack band size originated in work by Bazant and Oh (1983), and the modification and approach suggested by Cervenka et al. (1995) is used in ATENA. The plasticity model for concrete crushing is based on the work by Van Mier (1986), where the ascending branch of the compressive law is based on strains and the descending branch is based on displacements to introduce mesh objectivity. While this section provided an overview of the ATENA program and its analysis approach and capabilities, the next sections will focus on the specific constitutive relationships used in the modeling of HPFRCC. Further discussion of the theory and functionality of the ATENA software is described in detail in Cervenka et al. (2010).

4.2 Modeling Parameters

The fracture-plastic model within ATENA has several variants; however, all of them are based on the same principles and theories described previously. For modeling of HPFRCC, the CC3DNonLinCementitious2User model was implemented. This particular variation of the fracture-plastic model was selected because it allows for user defined laws for the material, such as the tensile response, compressive response, softening behavior, effect of lateral compression on tensile strength, shear retention factor, and reduction of compressive strength due to lateral
tensile strain. Each of these properties was either measured during the HPFRCC material tests or reasonable relationships were assumed for HPFRCC as determined by previous researchers.

4.2.1 Material Properties

The plate material tests described in Chapter 3 are the primary source for the material properties input into the ATENA model. When defining the material properties, the first parameters to apply were the compressive strength, tensile strength, Poisson’s ratio, and the initial elastic Young’s modulus values. These values for HPFRCC were obtained from plate tests for the compressive behavior and from direct tension tests for the tensile behavior. For the plain concrete material, all of the properties were obtained from the plate tests, and default values were used for parameters that were not directly tested, such as the tensile properties. Since the data obtained to generate the average responses was recorded at a high frequency, a reduced data set was used that still captured the behavior of the material. Examples of the reduced data will be shown in later figures, where one curve will display the response for HPFRCC as obtained by the experiments, while the other plot depicts the relationship input into ATENA. It will be seen that the reduced data used to define the material properties still closely captures the response of the material. The user-input compressive stress-strain behavior is displayed in Figure 4-1.
These user defined stress-strain curves were input for both the compressive and tensile properties of the material. The compressive strength implemented into ATENA was from the plate tests. This compressive strength was used instead of the cylinder compressive strength in accordance with the guidance of the finite element program. Again, the tensile properties of the NM6 HPFRCC material were obtained from direct tension dogbone tests. These tests were conducted by project colleagues at the University of Michigan, and the full results are reported by Liao et al. (2006). Figure 4-2 displays the NM6 uniaxial tension response, as well as the tensile stress-strain characteristic behavior input into ATENA with the reduced data set.
Several additional properties are required for input into ATENA, and they include the characteristic size, $L_{th}$, and the localized strain value, $\varepsilon_{loc}$. In ATENA, the characteristic size represents the size for which the tensile and compression diagrams were valid. Since the compressive result is the average response obtained on 5.5 in. (140 mm) wide specimens, that dimension was considered the compressive characteristic size. Similarly, the width of the dogbone specimens was 2 in. (50 mm), so that dimension was considered the corresponding tensile characteristic size. The characteristic size approach is implemented to reduce the dependency of the results on the finite element mesh. The localized strain values were also obtained from the experiments. For both the compressive and tensile tests, the strain began to localize at the peak strength, and then softening ensued through the slow pullout of the fibers under continued deformations. The localized strain values were 0.0031 and 0.0046 in.
compression and tension, respectively. The strain value that is used to determine the strength is calculated using the following equations, where the tensile strain value is used as an example:

If \( \varepsilon_1^f < \varepsilon_{loc}^f \)

\[
\tilde{\varepsilon}_1^f = \varepsilon_1^f
\]

else

\[
\tilde{\varepsilon}_1^f = \varepsilon_{loc}^f + \left( \varepsilon_1^f - \varepsilon_{loc}^f \right) \frac{L}{L_{ch}} \tag{4.1}
\]

The calculation of the strain values for the compressive case and for the shear strength retention function are based on a similar strain calculation; however, the appropriate values for the characteristic size and localized strain value should be used in each case. It is important to note that \( \varepsilon_1^f \) is the tensile strain calculated from the strain tensor at the finite element integration points, while the strain \( \tilde{\varepsilon}_1^f \) is used to determine the current tensile strength from the provided stress-strain diagrams, as seen in Figure 4-1 and Figure 4-2. The use of the characteristic length represents a scaling that takes into account the difference between the experimental size and the size of the finite element. This approach ensures that the same amount of energy is dissipated when using large and small finite elements.

For the biaxial failure criterion, the failure surface reported by Menetrey and William (1995) is used within ATENA. The failure criterion is characterized by three parameters: \( f'_c, f'_t, \) and \( e \), where \( e \) is an eccentricity parameter that influences the size and shape of the failure surface. The default parameter is for an \( e \) value of 0.52. This should correspond to a failure surface which matches closely to the biaxial failure surface observed by Kupfer et al. (1969). Figure 4-3 shows a comparison of the failure envelope obtained by Kupfer et al. (1969) to three
curves produced by the ATENA model of the experimental test set up for the plain concrete experiment. It can be seen that the default $e$ parameter of 0.52 underpredicted the theoretical failure curve, and a small increase to an $e$ value of 0.53 gave significantly more favorable results. In addition to the aforementioned plots, Figure 4-3 shows the biaxial failure envelope obtained experimentally from the plain mortar specimens. It can be seen that plain mortar mix has an even further expanded failure surface, so an additional increase in the $e$ parameter would be necessary to adequately model the biaxial behavior. For the NM4 plain concrete, the default $e$ value of 0.52 would be reasonable since it experienced a reduce increase in strength under biaxial loads.
Since the results of the biaxial tests described in Chapter 3 showed an expanded failure surface for HPFRCC beyond even the plain mortar concrete, the eccentricity parameters was modified to better reflect the results from the current experimental study. It was found that an $e$ value of 0.554 gave a reasonable match with the failure surface, and Figure 4-4 shows the Menetrey-William failure surface with the appropriate $e$ value compared to the NM6 experimental result. As shown later, the failure surface resulting from an $e$ value of 0.554 is almost identical under equal biaxial compression; however, the surface is slightly conservative around the 0.2 compression ratio, while it is slightly unconservative around a 0.6 compression ratio.
ratio. Here the compression ratio is defined as the ratio of the horizontal (X-direction) displacement rate to the vertical (Y-direction) displacement rate during the biaxial plate experiments.

Figure 4-4. Experimental NM6 and Adjusted ATENA Biaxial Failure Surfaces

The compressive strength reduction after cracking due to tensile strains in the orthogonal direction was also specified. The modified compression field theory, based on the work of Vecchio and Collins (1986), established that the compressive strength should decrease when cracking occurs in the perpendicular direction. It is defined as a function of the maximum tensile
strain and specifies a reduction of the maximum uniaxial compressive strength, $f'_c$. For modeling of HPFRCC, the results of the uniaxial compressive tests were used to get relationship between the lateral strain in the specimen and the reduction of the compressive strength with respect to the maximum strength. The resulting relationship implemented into ATENA is shown in Figure 4-5.

![Figure 4-5. Compressive Strength Reduction due to Cracking](image)

In addition to the previously described material parameters, multiple cracking during the hardening phase, as well as localized cracking during softening of the HPFRCC material, can be modeled. ATENA assumes that a set of parallel planar multiple cracks will form perpendicular to the maximum principal stress when the applied stress exceeds the first cracking strength, as described by Kabele (2007). As the loading increases, additional cracks may form within the
finite element, but the direction of the initial set of cracks is fixed to be perpendicular to the principal stress direction. Cracks will then be allowed to slide if the direction of principal stress changes; however, crack opening and sliding is resisted by fiber bridging. The crack sliding phenomena is implemented using a variable shear retention factor, $\beta$. The shear retention factor is defined as a ratio of the material post-cracking shear stiffness $G^c$ to its elastic shear stiffness $G$. The value of the shear retention is affected by the fiber volume fraction, fiber shear modulus, fiber Young’s modulus, fiber diameter, and fiber cross-section shape. Within the finite element, a secondary set of cracks may form in a direction perpendicular to the primary set of cracks if the maximum normal stress in the secondary crack-normal direction also exceeds the first cracking stress. No interaction is assumed between the two sets of cracks. Once the normal cracking strain in a set of multiple cracks exceeds the level of the cracking strain capacity, a localized crack will form, and the material softening will occur. The overall strain of the representative volume of the material is then obtained as the sum of the strain in the material between the cracks, cracking strains due to multiple cracking, and cracking strains due to localized cracks.

Of the material parameters input for the HPFRCC material modeling tests, all of the properties were obtained experimentally as a part of this project except for the shear retention behavior. Since no tests were done to measure the shear retention as a function of crack opening, the model by Kabele (2007) was used, and the pertinent fiber hooked steel fiber properties were input into the model. While this may not have a large effect on the material tests, it should help to more accurately capture the pre-cracking and post-cracking behavior of HPFRCC when located in larger shear critical structural components, such as coupling beams.
4.2.2 Element Mesh

A brief mesh sensitivity study was conducted on the uniaxial specimens. Since the NM6 concrete mix had aggregate as large as 3/8 in. (9.5 mm), the minimum reasonable mesh size was considered to be 0.4 in. (10 mm). Models were run using increasingly larger mesh sizes, and it was found that for the material modeling tests, a mesh size of 0.6875 in. (17.5 mm) was adequate for convergence to a solution.

4.2.3 Loading and Boundary Conditions

For the modeling of the material tests, displacements were imposed on the specimens through elastic steel elements, and the forces were then transferred to the HPFRCC through interface elements. The interface material model was used to simulate the contact between the concrete and the brush platens. Since the main purpose of the brush platens was to reduce the confinement of the specimen to obtain the true stress-strain response, it was important to appropriately model the interface elements. The elastic normal and shear stiffness of the element were selected as small enough values to allow for the transmission of force while not restraining the edge of the specimen, and the cohesion was set to zero. This was validated during post-processing upon review of the vertical and lateral stresses and strains. For the modeling of the tension experiment, the tensile strength of the interface element was specified as being significantly larger than the tensile strength of the concrete to ensure that the failure would localize in the HPFRCC material.

For the uniaxial tests, the displacements were applied to the top brush platen, while the bottom brush platen was fixed against vertical displacement along its entire length, and the left most node of the bottom brush platen was fixed against lateral displacement. For the biaxial tests, the displacements were applied to the top, left, and right brush platens, while the bottom
platen was fixed against vertical and horizontal displacements. Figure 4-6 shows a screen shot of the uniaxial plate tests from the ATENA program. The support conditions can be seen along the bottom brush platen, and the monitoring points, which report forces, displacements, stresses, and strains at desired locations, can also be seen at various points in the model. The monitoring points are used to streamline the extraction of specific numerical values after the analysis. For these models, the stresses and strains at the middle of the specimen were of particular interest to make a comparison with the experimental results. Additionally, the reaction force required to achieve the imposed displacements, as well as the imposed deformation on the specimen were monitored. These results give a better representation of the material properties obtained during the experimental research because they can be used to calculate an average stress-strain response, as done for the experimental specimens.
4.3 Material Modeling Validation

With the material parameters completely defined for the HPFRCC material, an analytical modeling program was undertaken to validate the material model by reproducing some of the results from the suite of tests on plate specimens previously described. The plate specimens were modeled with plane quadrilateral elements. They were isoparametric elements with quadratic displacement shape functions, so there were 4 Gauss integration points in each element. These elements are suitable for plane 2-D, axisymmetric, and 3-D problems (Cervenka et al., 2010). A similar finite element analysis has been conducted by Hussein and Marzouk (2000b) on varying boundary conditions during the biaxial testing of high strength concrete.
Figure 4-7 shows an example of the results of the finite element study by Hussein and Marzouk (2000b) for HSC plate specimens subjected to uniaxial loads. It can be seen that the proper application of the boundary conditions will apply uniform stress and displacement fields in the specimens. Validating the finite element material model at the smallest component level without the inclusion of reinforcement is an important step in the analytical program.

![Stress Contours and Displacement Contours](image)

**Figure 4-7. Stress Contours (left) and Displacement Contours (right) for HSC Plate Specimens Subjected to Uniaxial Loads from Hussein and Marzouk (2000b)**

### 4.3.1 Uniaxial Modeling Results

The first step with the material test validation was to model the plain concrete specimens subjected to uniaxial loading. This step would confirm that the assumed boundary conditions were appropriate, and that at least the response of concrete without a significant post-peak response could be captured. Since the plate tests had been conducted on the plain concrete specimens, the appropriate properties were input, as previously described, with a reduced set of data points for the constitutive relationships. The main emphasis of this portion of the study was
to observe the distribution of stresses and strains in the plate specimen, as well as a comparison of the stress-strain response observed in the model to that obtained during the experimental phase. Finally, the method of failure was another element of interest.

For the loading, displacement increments of 0.001 in. (0.0254 mm) were applied to the brush platen at each load step, and a modified Newton-Raphson approach was used as the solution method. The result of the plain concrete modeling showed that the stresses were indeed uniformly distributed in the specimen without a concentration at the interface between the platen and the specimen. Additionally, it was observed that appropriate properties were input for the interface elements to ensure that the specimen was free to deform without interfacial restraint. Since brush platens were used in the experiments, it was important to model this aspect of the experiment correctly to achieve the proper stresses and strains in the material. Figure 4-8 depicts the vertical stress distribution observed in ATENA. It can be seen that a uniform vertical stress was experienced in the specimen before failure.

Figure 4-8. ATENA Uniaxial Y-Stress Distribution in Plain Concrete Specimen
The strains and stresses were recorded at an integration point near the center of the specimen, and Figure 4-9 shows a comparison of the experimental result and the modeling result. It can be seen that the analysis ended abruptly for the plain concrete specimen, which is reminiscent of the experimental failure mode.

![Figure 4-9. ATENA Plain Concrete Uniaxial Compression Stress-Strain Response Comparison](image)

The next step in the analytical program was to model the uniaxial response of the HPFRCC specimens. The same model was used as shown previously for the plain concrete specimens; however, the appropriate HPFRCC material properties were applied, as described in Section 4.2.1. The results showed that again the stresses and strains were uniformly distributed in the specimen. It was important that ATENA was able to capture the ductility inherent in the
HPFRCC material, and the comparison of the uniaxial stress-strain responses in Figure 4-10 indicates that adequate modeling of the uniaxial compressive behavior was achieved.

Next, the uniaxial tensile response was modeled. Since the tensile strength of the interface elements was now critical to the response, an arbitrarily large value was input for the parameter to ensure that the failure would occur within the specimen, rather than at the interface between the specimen and the loading platen. The same magnitude of the incremental displacement was applied, except in the opposite direction. Since the pseudo-strain hardening performance of the material is a hallmark of the HPFRCC material response, it was critical that this behavior could be repeated analytically. Figure 4-11 shows the comparison between the uniaxial tension stress-strain responses of the ATENA model and the experiments. It should be
noted that the actual experiment from which this response was obtained was on a 2 in. (50 mm) dogbone specimen, while the model is implementing the brush platen test setup. Since the tensile stress-strain response should be relatively independent of the test method, the platen experimental setup was deemed acceptable. It can be seen that after initial cracking, the model was able to achieve a pseudo-strain hardening response where an increased loading was accommodated over substantial displacements. After the peak load, damage localized as the material gradually lost its tensile load carrying capacity over large deformations.

Figure 4-11. ATENA HPFRCC Uniaxial Tension Stress-Strain Comparison
As previously described, HPFRCC exhibits a multiple cracking behavior in tension. Figure 4-12 shows the crack distribution in the model specimen, and it can be seen that a distributed crack pattern was observed, as seen in the failure of experimental tests.

![Figure 4-12. ATENA HPFRCC Uniaxial Tension Crack Distribution](image)

With the ATENA models able to adequately represent the behavior of the HPFRCC material under uniaxial conditions, the next step was to investigate its ability to reproduce ductile multi-axial behavior.

### 4.3.2 Equal Biaxial Modeling Results

The first biaxial modeling effort was to simulate the effect of equal biaxial compression, as done during the experiments. The loads were again applied through prescribed displacement increments on the top, left, and right platens. Since the platens were connected to the specimen through interface elements, and the properties of the interface elements allowed for the expansion or contraction of the specimen along the platen, it was found that the model did indeed
adequately represent the boundary conditions. An example of the prototypical model for all of the biaxial models is shown in Figure 4-13.

![Figure 4-13. ATENA Biaxial Model Prototype](image)

The biaxial responses of the specimen were recorded through four monitoring points located at an integration point closest to the center of the specimen. Two of the monitoring points recorded strain (X and Y), while the other two recorded the stresses (X and Y) in the specimen. For the equal biaxial modeling result, the stiffness of the specimen and the strength of the specimen were closely captured. However, the shape of the experimental biaxial curves varied somewhat from the uniaxial curve. It was found experimentally that when subjected to biaxial loads, the specimen gradually softened post-peak without ever really reaching a plateau. Uniaxially, however, the specimen had an initially more rapid softening post-peak, followed by a sustained plateau over large deformations. Since ATENA inputs are limited to the size of the biaxial surface and the shape of the characteristic uniaxial curve, the biaxial modeling response
had a similar shape as the uniaxial curves. This discrepancy can be seen in Figure 4-14, where the stiffness and strength of the model closely match the experimental result, but during the post-peak response, the model softens more rapidly and then sustains a greater stress at large strains. While there is some difference between the results, the model does fairly closely capture the behavior of the HPFRCC material under biaxial compression, namely the strength, stiffness, ductility, and large residual strength.

![Figure 4-14. Equal Biaxial Compression Modeling and Experimental Results](image)

The next phase of modeling was to proceed with the unequal biaxial compression loading.
4.3.3 Unequal Biaxial Modeling Results

The unequal biaxial models used the same prototypical model shown in Figure 4-13; however, the prescribed horizontal displacements were varied to achieve the desired compression ratio. The compression ratio again is defined as the horizontal strain rate to the vertical strain rate. For example, the 70% biaxial result had a vertical strain rate of 0.001 in. (0.025 mm) per load step and a horizontal strain rate of 0.0007 in. (0.018 mm) per load step. To complete the full biaxial failure envelope, the model was run at many different loading ratios, but the following figures were selected to be shown because they correspond to specific average experimental curves that were used to create the experimental biaxial failure envelope. Figure 4-15, Figure 4-16, and Figure 4-17 display a comparison of the vertical (Y) and horizontal (X) stress-strain responses from the ATENA models and the experiments outlined in Chapter 3.

![Figure 4-15. 70% Biaxial Compression Modeling and Experimental Results](image)
For the 70% compression ratio shown in Figure 4-15, the peak stress and stiffness both horizontally and vertically aligned well with the experimental results. In the vertical direction, the post-peak behavior resembled the lab test, but the horizontal modeling result did not experience the same softening before the maximum horizontal stress was achieved. This is largely a shortcoming of the modeling approach, and this was seen during each of the loading ratios. Despite the lack of pre-peak softening in the horizontal direction, the essence of the biaxial response was reasonably captured. At the 50% and 30% compression ratios, Figure 4-16 and Figure 4-17 show that the same trends were exhibited, and again the main properties of the HPFRCC, such as ductility and residual strength, were well represented.
For the 30% compression ratio shown in Figure 4-17, the modeling results less closely resembled the experimental tests. It was found that the 30% strain rate did not result in an approximate 30% force. This trend continued as the displacement rates were reduced, so to complete the biaxial envelope at lower compression ratios, prescribed forces were applied. While using load-control in the model does not provide the post-peak behavior, it does allow the user to identify the maximum loading that represents a point along the failure envelope.

4.3.4 Failure Envelope Results

For a full comparison of the experimental results to the modeling results, additional simulations to those previously described were conducted at varying biaxial loading ratios. Additionally, tension tests were simulated. A review of the literature showed that Demeke and
Tegos (1994) conducted tests on fiber reinforced concrete under compression-tension conditions, and their results showed that the compression-tension region was essentially linear from the uniaxial compressive strength to the uniaxial tensile strength; however, the uniaxial tensile strength was considerably larger than the plain concrete response. The ATENA modeling result showed a similar trend in the compression-tension region, and that can be seen in Figure 4-18.

Figure 4-18. Experimental and Modeling Result Biaxial Failure Envelopes
Figure 4-18 displays a comparison between the experimental failure envelope and the failure envelope obtained from the modeling effort. It shows that under biaxial compression, the results align well from compression ratios of 50% to equal compression. However, the model was conservative and underpredicts the biaxial strength at lower loading ratios. For the tension results, while specific experimental data was not available to validate the model under compression-tension and tension-tension, a review of the literature shows the results to be reasonable. Overall, the model has shown that it is able to capture the strength, deformation, stiffness, and ductility of HPFRCC under both uniaxial and biaxial loading. The next phase of the analytical program is to extend the use of HPFRCC to the modeling of structural components. The modeling of coupling beams and pile-wharf connections will be described in the Chapter 6 and Chapter 9, respectively.
CHAPTER 5. COUPLING BEAM COMPONENT TEST RESULTS

Coupling beam component tests have been conducted by many researchers since the 1970s. Chapter 2 highlighted some of the previously conducted tests, and it discussed briefly some of the differences. While the focus of Chapter 2 was to introduce some of the research that have been completed, this chapter is intended to more thoroughly review the results of some of the previous coupling beam component test research. These results will become particularly important during the analytical phase of the research, where reinforced concrete coupling beam components are modeled and the HPFRCC model is implemented to observe the change in behavior, and vice versa.

5.1 Reinforced Concrete Coupling Beam Tests

As outlined in Chapter 2, coupling beam component tests have been conducted on specimens with three basic reinforcement layouts: conventional reinforcement, diagonal reinforcement, and rhombic reinforcement. Examples of each reinforcement pattern from Galano and Vignoli (2000) is shown in Figure 5-1. The conventionally reinforced coupling beams typically have top and bottom longitudinal bars to resist the flexural demand, and stirrups are provided for shear resistance. They are generally designed as well confined beams, and ACI 318 (2008) restricts the reinforcement layout to shear stress demands less than $4\sqrt{f_v b_w d}$ and $l_n / h > 4$, where $b_w$ is the width of the coupling beam; $d$ is the depth of the coupling beam; $h$ is the height of the coupling beam; and $l_n$ is the clear span. Diagonally reinforced coupling beams have two intersecting groups of diagonally placed bars symmetrical about the midspan. The diagonal bars are confined with closely spaced ties to prevent buckling of the bars, and additional
longitudinal reinforcement and stirrups are included. These beams are required in coupling beams with a shear demand exceeding $4\sqrt{f'_c b_w d}$ and $l_n / h < 2$. In the intermediate aspect ratio range of 2 to 4, coupling beams are reinforced with diagonal bars; however, two confinement options exist: each diagonal can be individually confined, in addition to the perimeter of the cross-section, or the entire beam cross-section may be heavily confined, and the diagonal bars need not have confinement (ACI Committee 318, 2008). Examples of the two intermediate reinforcement layouts were shown previously in Figure 2-10 and Figure 2-11. For coupling beams with $l_n / h \geq 4$, the design must satisfy the requirements for flexural members of special moment frames. The rhombic reinforcement pattern shown in Figure 5-1 has been tested experimentally, but is not used in practice. It consists of two sets of diagonal bars that cross at each end of the coupling beam, and the idea is that the inclined orientation of the reinforcement would improve the shear resistance at the hinge location.
For the modeling effort, coupling beams were selected that were indicative of current construction practice, so the rhombic layouts were not modeled. Thus, the main focus is on coupling beams with either conventional or diagonal reinforcement patterns. Additionally, experiments for which a comprehensive report of the material properties, loading pattern, geometry, and response of the specimens were selected. The latter criterion eliminated many of the more notable coupling beam experiments since the paucity of specific information regarding the nuances of the test would make modeling the experiment subject to conjecture and assumption. The selection was narrowed to the results presented by Tassios et al. (1996) and by
Galano and Vignoli (2000). While a more thorough modeling effort of all of the coupling beams presented in the literature may have been preferred, only the coupling beams from Galano and Vignoli (2000) were investigated for this research. The following sections will outline the specific coupling beam component experiments that were selected for the modeling effort to be described in Chapter 6.

5.1.1 Galano and Vignoli (2000)

This experimental study investigated the effect of reinforcement layout and loading history through testing fifteen short coupling beams with four different reinforcement arrangements. Each reinforcement layout had specimens subjected to monotonic and cyclic loading, and the length-to-depth ratio of each specimen was 1.50. The reinforcement layouts investigated are displayed in Figure 5-1. They include conventional reinforcement, diagonal reinforcement without ties, diagonal reinforcement with confining ties, and rhombic reinforcement.

The specimens had two lateral stiff blocks to simulate the surrounding walls and to apply the loading histories. The coupling beams were tested in the vertical position under asymmetric bending and constant shear. Six steel rollers were used to constrain the specimen. Two rollers were placed laterally to prevent horizontal movements of the specimens, and the other four rollers were positioned to produce the desired loading histories. Two hydraulic actuators were attached to the specimen to impose equal and opposite rotations on the specimen. The described test setup is shown in Figure 5-2. The specimens were loaded either monotonically or cyclically to failure.
The tests were conducted until the coupling beams collapsed due to the fracture of one or more reinforcing bars or when the load was reduced by more than 2/3 of the maximum load. The strength, stiffness, ductility, and degradation of each specimen were investigated. It was found that the reinforcement layouts with diagonal bars provided a higher strength and elastic stiffness, and the conventional and rhombic reinforcement layouts had strength decreases of 7.5% and 17%, respectively (Galano & Vignoli, 2000). The conventional reinforcement showed a diagonal cracking pattern from the shear loading, and, after the yield point, only a negligible change in the shear force was accommodated. Failure eventually occurred due to the rupture of
the stirrups. The diagonally reinforced coupling beams exhibited a more ductile response at failure. After diagonal cracking was initiated, vertical cracks at the ends of the coupling beam allowed for large rotations after yielding without strength degradation. The eventual failure occurred due to crushing of the compressive strut, rather than fracture of the steel bars. The rhombic specimen carried a large amount of shear after yielding and had high rotation values; however, an abrupt failure occurred due to the fracture of the two diagonal bars. The shear versus rotation plots of the monotonically loaded specimens is shown in Figure 5-3.

![Shear-Rotation Responses of Monotonic Coupling Beams from Galano and Vignoli (2000)](image)

**Figure 5-3. Shear-Rotation Responses of Monotonic Coupling Beams from Galano and Vignoli (2000)**

The strength and stiffness degradation, as well as the energy dissipation, were examined with respect to the cyclic tests. Since earthquakes produce severe cyclic loading, it is critical that
coupling beams with significant strength and stiffness degradation be avoided, while energy
dissipating designs are favored. When reviewing the cyclic performance of the coupling beams,
it was found that the conventionally reinforced coupling beams demonstrated a lower energy
dissipating capacity due to the pinching of the hysteresis loops. The conventionally reinforced
coupling beam did exhibit comparable strength retention characteristics initially, but at larger
displacements, the strength dropped considerably. Also, the test results showed that the beams
with diagonal or rhombic reinforcement exhibited a higher rotational ductility than
conventionally reinforced coupling beams. Between the diagonal and rhombic layouts, the
difference in energy dissipation was found to be negligible, and it was the opinion of the authors
that the rhombic arrangement performed better than the diagonal reinforcement with respect to
rotational capacity and the degradation in both strength and stiffness. The shear-rotation
responses of the cyclic conventionally reinforced and diagonally reinforced coupling beams are
shown in Figure 5-4 and Figure 5-5, respectively.
Figure 5-4. Shear-Rotation Response of Cyclic Conventional RC Coupling Beam from Galano and Vignoli (2000)
The following section provides a summary of the failure modes from the reinforced concrete coupling beam tests from previous researchers.

5.1.2 Reinforced Concrete Coupling Beam Failure Modes

The failure mode of coupling beams can vary widely, depending the layout of reinforcement. FEMA 306 (Applied Technology Council, 1998) provides a “Component Damage Classification Guide” for identifying several types of coupling beam failure modes, as well as those of other concrete elements. Mohr (2007) identified six failure modes outlined in FEMA 306 that are representative of the experimental coupling beam results of previous
researchers, as well as two additional failure modes to more accurately capture the spectrum of coupling beam component test results. The selected failure modes include the following: ductile flexure, flexure/diagonal tension, flexure/sliding shear, preemptive diagonal tension, diagonal compression, and flexural compression. Ductile flexure can be identified by wide flexural cracking and spalling concentrated in the plastic hinge zone, and no buckling or fracture of the reinforcement. Flexure/diagonal tension occurs when the shear strength exceeds the flexural strength; however, as flexural cracks open, the shear strength reduces and eventually controls. It can be identified by wide diagonal cracks concentrated in one or two cracks. Flexure/sliding shear is common in coupling beams with poor construction joint details. Initially flexural yielding governs the response, but as flexural cracks join to form a single crack across a section, a sliding plane forms. Upon continued cycling, the crack opens to reduce the effect of aggregate interlock and shear friction on the sliding resistance. Preemptive diagonal tension is a brittle failure mode that results from a high flexural strength with inadequate shear reinforcement. It is characterized by one or more wide diagonal cracks that occur with little early warning of imminent failure. The previous four failure modes are described in detail in FEMA 306 (Applied Technology Council, 1998). The next two failure modes were developed by Mohr (2007) to represent some coupling beam failure modes observed by previous researchers. Diagonal compression occurs in the center of coupling beams where two diagonal bars intersect. Wide shear cracks form along the compressive strut that coincides with the diagonal reinforcement, and upon reverse cyclic loading, spalling is eventually observed at the center of the coupling beam. The failure then occurs with the buckling of the diagonal reinforcement at the spalled region in the center of the coupling beam. Flexural compression is a ductile failure mode that exhibits spalling and flexural cracking in the plastic hinge zone. Shear cracks also form
throughout the beam along the diagonal struts, and failure may occur due to the buckling of the diagonal reinforcement at the end of the coupling beam in the damage plastic hinge region.

By reviewing the description of the failure modes in the literature and by examining the photographs provided by the researchers, the following six figures show examples of the various failure modes.

Figure 5-6. Ductile Flexure Failure Modes from Kwan and Zhao (2002) Specimen MCB3 and Galano and Vignoli (2000) Specimen P16
Figure 5-7. Flexure / Diagonal Tension Failure Modes from Tassios et al. (1996) Specimen CB-1B and Kwan and Zhao (2002) Specimen CCB1

Figure 5-8. Flexure / Sliding Shear Failure Modes from Paulay (1977) Specimen 315 and Kwan and Zhao (2002) Specimen CCB12
Figure 5-9. Preemptive Diagonal Tension Failure Modes from Galano and Vignoli (2000) Specimen P01 and Kwan and Zhao (2002) Specimen MCB1

Figure 5-10. Diagonal Compression Failure Modes from Galano and Vignoli (2000) Specimen P12 and Kwan and Zhao (2002) Specimen CCB11
In the following section, the focus will shift from the behavior of reinforced concrete coupling beams to a testing program conducted on HPFRCC coupling beams. The coupling beams investigated were composed of the same NM6 that was tested multi-axially and described in Chapter 3.

5.2 HPFRCC Coupling Beam Tests

The next phase of the coupling beam modeling will focus on the HPFRCC coupling beams tested by project colleagues at the University of Michigan. Several generations of tests were conducted, as described in Chapter 2. The initial tests explored different fiber types and reduced reinforcement in coupling beams with an aspect ratio of 1.0 (Canbolat, 2004). However, a building inventory conducted by Mohr (2007) on buildings designed for the West Coast of Washington and California indicated that the average aspect ratios of conventionally reinforced coupling beams and diagonally reinforced coupling beams were 3.2 and 1.7, respectively. Lequesne (2011) reported an experimental program on diagonally reinforced HPFRCC coupling
beams with an aspect ratio of 1.75, and since this was more representative of the current state of practice, these coupling beams were selected for the modeling effort in this research.

5.2.1 Lequesne (2011)

Coupling beam tests were conducted by Lequesne (2011) on specimens with span length-to-depth ratio of 1.75 to investigate the response of less shear critical members than previous coupling beam tests. The tests were designed to show that HPFRCC can improve the shear resistance of coupling beams in three ways: 1.) preventing buckling in the diagonal reinforcement, allowing for both the tension and compression diagonal steel to resist shear, 2.) limiting damage in the coupling beam, thus maintaining the integrity of the specimen, and 3.) enhancing aggregate interlock by the fibers bridging cracks and transferring tensile stresses across cracks.

The coupling beams were precast and the same NEES Mix #6 that was described in Chapter 3 and used in this dissertation was implemented in the concrete mixture for their design and construction. These coupling beams used the same hooked steel Dramix fibers in the HPFRCC, and the properties are described in Table 3-3. The hooked steel fibers were selected over the Spectra fibers because they provide better pullout resistance. In the design of these coupling beams, a departure from the typical assumption that the behavior is controlled by the diagonal reinforcing steel was employed. Instead, the diagonal reinforcement was assumed to provide one-third of the ultimate shear capacity, with the remaining shear demand resisted by the transverse reinforcement and the HPFRCC. Basing the design approach on this assumption reduces the reliance on diagonal steel reinforcement and accounts for the improved damage tolerance of HPFRCC.
The design process of the coupling beam specimens is outlined by Lequesne (2011). The process began with the selection of the diagonal bar area and orientation. The diagonal bars were selected to resist 30-40% of the expected shear demand, and the design shear force of 
\[ 10\sqrt{f_c A_{cw}} \text{ [lb]} \left(0.83\sqrt{f_c A_{cw}} \text{ [kN]} \right), \]
where \( A_{cw} \) is the gross cross-sectional area of the coupling beam, was assumed. This was selected because it is the upper limit for the shear capacity permitted by the ACI 318 (ACI Committee 318, 2008). To relieve constructability issues with the precast coupling beams, the diagonal bars were bent to exit the precast coupling beam horizontally. This was intended to ease the process of threading the coupling beam reinforcement into the dense reinforcing cage in the boundary elements of the structural walls. Additionally, bending the diagonal bars within the clear span of the coupling beam allowed for a slight increase of the inclination of the diagonal bars from 22° to 24°. No confinement steel was provided around the diagonal bars, since it was found previously that the HPFRCC adequately confined the diagonals because no reinforcement instability was observed (2004).

The next step in the design was the selection of the longitudinal reinforcement. Since the behavior of the HPFRCC coupling beams is expected to be dominated by the formation of the plastic hinges at each end, the capacity of the coupling beam will be governed by the selection of the longitudinal reinforcement. The moment capacity was assumed to be that produced by the expected shear demand described previously \( M_u = \frac{V_u l_n}{2} \). The contribution of the diagonal reinforcement to the flexural strength was considered, and the remaining area of longitudinal steel was calculated. It should be noted that all of the longitudinal and diagonal reinforcement was fully developed into the wall to all for the reinforcement to yield at the beam-wall interface.

The remaining reinforcement to select was the transverse reinforcement. The goal of the transverse reinforcement is to transfer tension stresses, resist the opening of diagonal cracks, and
to provide confinement of the concrete core and reinforcement (2011). The specimen was divided into two distinct regions: midspan region and end region. In the midspan region, the transverse reinforcement was selected to resist approximately 40% of the shear demand. Since the diagonal reinforcement was designed to resist 30-40% of the shear, this leaves 30-40% of the shear to be resisted by the HPFRCC. In the first two specimens, CB-1 and CB-2, #3 (D10) hoops were spaced at 6 in. (152 mm), corresponding to a volumetric ratio of 0.89%. The third specimen, CB-3, also used #3 (D10) hoops, but at a spacing of 8 in. (203 mm), which corresponded to a reduced volumetric ratio of 0.67%. In the third specimen, the HPFRCC was expected to carry an increased 40-50% of the shear force. In the end region, additional transverse reinforcement was required to stabilize the bend in the diagonal reinforcement. The end region was considered to extend a distance h/2 into the coupling beam from the face of the wall. In Specimen CB-1, an inadequate amount of the transverse reinforcement was provided in this region, and a premature failure occurred. The design methodology was revised for Specimens CB-2 and CB-3, and the end region in these specimens was confined with #3 (D10) hoops at 2.75 in. (70 mm). This resulted in a volumetric reinforcement ratio of 2.9%, and it essentially satisfied the ACI 318 (ACI Committee 318, 2008) requirements for special column confinement.

The final design step was to address the transfer of forces from the precast coupling beam element to the structural wall. Two connection details were investigated: U-shaped dowel bars and straight dowel bars. The dowel bars were intended to increase the capacity of the coupling beam at the beam-wall interface and to shift the localization of damage in the coupling beam into the well confined end region of the beam. To prevent sliding shear, shear friction adopted by ACI 318 (2008) was used to calculate the area of the dowel reinforcement required across the
interface. In the calculation, the axial load and the diagonal bars forced into compression by sliding were neglected in the calculation. The precast coupling beams were embedded into the wall the distance of the concrete cover, so in this case 1 in. (25 mm). No sliding was observed at the interface of any specimens, so the design methodology was deemed sufficient. Figure 5-13 displays the details of the three reinforcement layouts developed for the testing program.

The test setup consisted of one of the wall boundary elements being bolted to the laboratory strong floor to simulate a fixed boundary condition. Lateral displacements were then imposed on the other wall boundary element through a horizontal actuator. The top wall element was restrained against rotation by steel links to impose a state of double curvature in the coupling beam. Additionally, these links provided a passive, partial restraint against the elongation of the coupling beam. The test setup can be seen in Figure 5-12.

![Figure 5-12. Coupling Beam Component Test Setup from Lequesne (2011)](image)

Figure 5-12. Coupling Beam Component Test Setup from Lequesne (2011)
Figure 5-13. Reinforcement Detail of Precast Coupling Beams with a Span-to-Depth Ratio of 1.75 from Lequesne (2011)
The load-displacement behavior of the coupling beam is presented as average shear stress versus drift. The drift is defined with the same equation that ASCE/SEI 41 (ASCE, 2007), as shown below.

\[ Drift = \frac{\Delta}{L} - \frac{\theta}{2} \]  

(5.1)

Where \( \Delta \) is the relative displacement of the coupling beam, \( L \) is the length of the coupling beam, and \( \theta \) is the rotation of the end of the coupling beam. Since the two ends of the coupling beam were assumed not to rotate, \( \theta \) was zero throughout the tests.

Specimen CB-1 displayed good energy dissipation in the early cycles, with the maximum stress achieved around 2% drift; however, the transverse reinforcement in the end region was not sufficient to resist the bursting force from the bent diagonal bar while still effectively confining the plastic hinge. The result was a failure in the plastic hinge region at about 2.2% drift. As discussed previously, this design deficiency was addressed in subsequent specimens, and the transverse reinforcement in the end regions was increased. Specimen CB-2 displayed a much more stable hysteresis behavior. The maximum shear stress achieved during testing exceeded \( 11.5 \sqrt{f_c} \) [psi] (0.87 \( \sqrt{f_c} \) [MPa]), and the specimen still performed in a stable minor with minor pinching. A capacity retention of about 80% of the peak shear force was maintained to drifts of about 5% in each direction. The shear stress versus drift response of the Specimen CB-2 can is shown in Figure 5-14.
Since cracks remained narrow throughout testing at high shear stresses, it is evident that both the stirrups and the HPFRCC were active in resisting the shear force. Plastic hinges developed near the ends of the coupling beams, and the addition of extra transverse reinforcement in the end region provided a stable flexural mechanism. Web-shear cracks formed early in the test at drifts up to about 1.0%. Beyond 1.0%, flexural-shear cracks formed near the ends of the coupling beam and continued to extend until about 1.5% drift. At 2% drift, the flexural cracks began to widen, and this mechanism continued to control the behavior. At approximately 5.5% drift, some sliding was observed along the flexural cracks in the end region.

Figure 5-14. Shear Stress versus Drift Response of Specimen CB-2 from Lequesne (2011)
which led to some spalling of the compression zone and the eventual failure of the coupling beam. Images of the coupling beam at 3.5% drift and 5.5% drift during testing are displayed in Figure 5-15.

Figure 5-15. CB-2 Damage States at 3.5% Drift (left) and 5.5% Drift (right) from Lequesne (2011)

The final specimen tested was Specimen CB-3, which explored the use of straight dowel bars and a 25% reduction of the transverse reinforcement in the midspan region. The reduction of transverse reinforcement would force the HPFRCC to resist a greater portion of the applied shear stress. Unfortunately, during testing a mishap with an instrument monitoring the slip of the
base block produced an asymmetric loading protocol. Despite the unconventional loading, the specimen showed minor pinching in the shear stress versus drift behavior, and again, 80% of the peak shear force was maintained to a drift of 5% (only in the negative direction due to the loading). Shear stresses of up to $14.0 \sqrt{f_c'}$ [psi] ($1.16 \sqrt{f_c}$ [MPa]) were achieved, which is considerably larger than the maximum $10.0 \sqrt{f_c'}$ [psi] ($0.83 \sqrt{f_c}$ [MPa]) allowed by ACI 318 (2008). The shear stresses versus drift response of Specimen CB-3 can be seen in Figure 5-16.

![Shear Stress versus Drift for Specimen CB-3 from Lequesne (2011)](image)

**Figure 5-16. Shear Stress versus Drift for Specimen CB-3 from Lequesne (2011)**

Similar to the previous tests, the response of Specimen CB-3 was dominated by flexural rotation in the end regions. Large displacement reversals were achieved, but failure ultimately
occurred with the development of sliding shear around 5% drift. Large flexural cracks were developed, as well as a large shear stress, despite the reduction of the transverse reinforcement in the midspan region. The damage states of Specimen CB-3 at 3% and 6% drift are displayed in Figure 5-16. The author noted that greater density of crack markings in Specimen CB-3 was related to increased access to the specimen from a reduction of instrumentation, rather than due to a change in the behavior of the specimen.

Figure 5-17. CB-3 Damage States at 3% Drift (left) and 6% Drift (right) from Lequesne (2011)
In each of the coupling beams, flexural hinging at the ends of the coupling beams dominated the behavior. Shear stresses exceeded \( 10.0 \sqrt{f_c} \ [\text{psi}] (0.83 \sqrt{f_c} \ [\text{MPa}]) \), and with proper confinement of the end region, the coupling beams were able to resist more than 80% of the peak shear force beyond 5% drift. It was found that precasting the HPFRCC coupling beam and embedding the reinforcement in the boundary element of the shear wall was an effective design. For the shear resistance, the diagonal reinforcement provided 20-30% of the applied shear, and it was found that assuming that HPFRCC could resist \( 5.0 \sqrt{f_c} \ [\text{psi}] (0.42 \sqrt{f_c} \ [\text{MPa}]) \) was a conservative estimate of the HPFRCC contribution to the shear capacity. Sliding shear deformations initiated along flexural cracks at about 2% drift, but the failure mechanism was resisted until rather large drifts were achieved. A final finding from the tests was that the HPFRCC coupling beams exhibited a shear stiffness of approximately \( 0.04E_A g \) at drifts greater than 1%. This value is only 10% of the recommended \( 0.4E_A g \) proposed by ASCE/SEI 41 (2007) for modeling coupling beams. The author attributes this large discrepancy to ASCE/SEI 41 not accounting for the reduction of shear stiffness due to diagonal and flexural cracking, so the reduced value is recommended (2011).

5.3 Compiled Coupling Beam Information

Since the next portion of this research program involves modeling of the coupling beams previously discussed in this chapter, the geometry, reinforcement details, and material properties are included in the following tables. These tables contain the relevant properties for creating the coupling beam models and for evaluating the model. Table 5-1 outlines the coupling beam geometry and general material properties. Table 5-2 displays more specifics about the
reinforcement layout, size, and quantity. Table 5-3 highlights some of the more relevant performance parameters of the coupling beams, such as shear strength, displacement, and stiffness.

Table 5-1. Coupling Beam Properties

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement Pattern</th>
<th>Loading</th>
<th>$E_i$ (Mpa)</th>
<th>$f_y$ (Mpa)</th>
<th>$f_y'$ (Mpa)</th>
<th>$f_u$ (Mpa)</th>
<th>b (mm)</th>
<th>d (mm)</th>
<th>L (mm)</th>
<th>L/d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Galano P01</td>
<td>Conventional</td>
<td>Mono.</td>
<td>24,400</td>
<td>48.9</td>
<td>567</td>
<td>660</td>
<td>150</td>
<td>400</td>
<td>600</td>
<td>1.50</td>
</tr>
<tr>
<td>Galano P02</td>
<td>Conventional</td>
<td>Cyclic</td>
<td>24,400</td>
<td>44.5</td>
<td>567</td>
<td>660</td>
<td>150</td>
<td>400</td>
<td>600</td>
<td>1.50</td>
</tr>
<tr>
<td>Galano P05</td>
<td>Diagonal</td>
<td>Mono.</td>
<td>24,400</td>
<td>39.9</td>
<td>567</td>
<td>660</td>
<td>150</td>
<td>400</td>
<td>600</td>
<td>1.50</td>
</tr>
<tr>
<td>Galano P07</td>
<td>Diagonal</td>
<td>Cyclic</td>
<td>24,400</td>
<td>54.0</td>
<td>567</td>
<td>660</td>
<td>150</td>
<td>400</td>
<td>600</td>
<td>1.50</td>
</tr>
<tr>
<td>Lequesne CB-2</td>
<td>Diagonal</td>
<td>Cyclic</td>
<td>-</td>
<td>52.0</td>
<td>430</td>
<td>680</td>
<td>150</td>
<td>600</td>
<td>1050</td>
<td>1.75</td>
</tr>
<tr>
<td>Lequesne CB-3</td>
<td>Diagonal</td>
<td>Cyclic</td>
<td>-</td>
<td>34.0</td>
<td>420</td>
<td>715</td>
<td>150</td>
<td>600</td>
<td>1050</td>
<td>1.75</td>
</tr>
</tbody>
</table>

* $f_c$ = mean cylinder compressive strength
* $f_y$ = yield strength based on rebar strength tests
* $f_u$ = ultimate strength based on rebar strength tests

Table 5-2. Coupling Beam Reinforcement Properties

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement Pattern</th>
<th># Primary Bars</th>
<th>$d_y$ Primary (mm)</th>
<th>$\Lambda_y$ (mm$^2$)</th>
<th>$\rho_i$</th>
<th># Mid-Bars</th>
<th>$d_y$ Mid (mm)</th>
<th>$d_y$ Vert. (mm)</th>
<th>$\Lambda_y$ (mm$^2$)</th>
<th>Spacing (mm)</th>
<th>$\rho_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Galano P01</td>
<td>Conventional</td>
<td>4</td>
<td>10</td>
<td>314.2</td>
<td>0.52%</td>
<td>2</td>
<td>6</td>
<td>8</td>
<td>100.5</td>
<td>120</td>
<td>0.84%</td>
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<td>Galano P02</td>
<td>Conventional</td>
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<td>10</td>
<td>314.2</td>
<td>0.52%</td>
<td>2</td>
<td>6</td>
<td>8</td>
<td>100.5</td>
<td>120</td>
<td>0.84%</td>
</tr>
<tr>
<td>Galano P05</td>
<td>Diagonal</td>
<td>4</td>
<td>10</td>
<td>314.2</td>
<td>0.52%</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>56.5</td>
<td>150</td>
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</tr>
<tr>
<td>Galano P07</td>
<td>Diagonal</td>
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<td>10</td>
<td>314.2</td>
<td>0.52%</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>56.5</td>
<td>150</td>
<td>0.39%</td>
</tr>
<tr>
<td>Lequesne CB-2</td>
<td>Diagonal</td>
<td>2</td>
<td>16</td>
<td>402.1</td>
<td>0.89%</td>
<td>8</td>
<td>13</td>
<td>10</td>
<td>157.1</td>
<td>70</td>
<td>2.90%</td>
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<tr>
<td>Lequesne CB-3</td>
<td>Diagonal</td>
<td>2</td>
<td>16</td>
<td>402.1</td>
<td>0.89%</td>
<td>8</td>
<td>13</td>
<td>10</td>
<td>157.1</td>
<td>70</td>
<td>2.90%</td>
</tr>
</tbody>
</table>

* $\rho_v$ = volume stirrups / volume coupling beam
* $\rho_i$ = area primary reinforcement / coupling beam cross-section
Table 5.3. Coupling Beam Performance

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_y$ (kN)</th>
<th>$V_p$ (kN)</th>
<th>$K_y$ (kN/mm)</th>
<th>$K_p$ (kN/mm)</th>
<th>$\delta_y$ (mm)</th>
<th>$\delta_p$ (mm)</th>
<th>$\delta_{0.85s}$ (mm)</th>
<th>$\mu_{0.85s}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Galane P01</td>
<td>213.0</td>
<td>222.9</td>
<td>42.2</td>
<td>9.3</td>
<td>0.84%</td>
<td>4.00%</td>
<td>5.09%</td>
<td>30.5</td>
</tr>
<tr>
<td>Galane P02</td>
<td>212.0</td>
<td>234.0</td>
<td>40.2</td>
<td>14.9</td>
<td>0.88%</td>
<td>2.62%</td>
<td>4.32%</td>
<td>15.7</td>
</tr>
<tr>
<td>Galane P05</td>
<td>200.0</td>
<td>239.3</td>
<td>39.6</td>
<td>9.8</td>
<td>0.84%</td>
<td>4.07%</td>
<td>6.54%</td>
<td>24.0</td>
</tr>
<tr>
<td>Galane P07</td>
<td>216.0</td>
<td>237.0</td>
<td>41.9</td>
<td>15.9</td>
<td>0.86%</td>
<td>2.48%</td>
<td>4.67%</td>
<td>14.9</td>
</tr>
<tr>
<td>Lequesne CB-2</td>
<td>429.0</td>
<td>655.0</td>
<td>63.1</td>
<td>21.8</td>
<td>0.65%</td>
<td>2.87%</td>
<td>4.58%</td>
<td>28.0</td>
</tr>
<tr>
<td>Lequesne CB-3</td>
<td>390.7</td>
<td>650.0</td>
<td>58.3</td>
<td>16.9</td>
<td>0.64%</td>
<td>3.66%</td>
<td>4.61%</td>
<td>48.1</td>
</tr>
</tbody>
</table>

* $V_y$ = yield strength from researcher
* $V_p$ = Peak strength from researcher
* $K_y$ = Stiffness of coupling beam at yield, $V_y/\delta_y$
* $K_p$ = Stiffness of coupling beam at peak strength, $V_p/\delta_p$
* $\delta_y$ = Drift percentage at yield displacement, $\delta_y/L$
* $\delta_p$ = Drift percentage at peak strength, $\delta_p/L$
* $\delta_{0.85s}$ = Displacement at yield
* $\delta_{0.85s}$ = Displacement at peak strength
* $\mu_{0.85s}$ = Displacement ductility at 85% peak strength
* $\mu_{0.85s}$ = Displacement ductility at 85% $V_{0.85s}/\delta_y$

The preceding chapter reviewed the experimental results of coupling beam component tests conducted by previous researchers. These results included conventionally and diagonally reinforced RC coupling beams, as well as HPFRCC coupling beams. The next phase of this research is to model these coupling beam component tests. With the tests modeled, then the HPFRCC material model developed in Chapter 3 and Chapter 4 will be extended to use in these structural component tests.
CHAPTER 6. COUPLING BEAM COMPONENT MODELING

As seen in the previous chapters, coupling beam component tests have been conducted by many researchers over the years; however, most of the results focused on the testing of RC coupling beams. Given the extensive testing on the HPFRCC material presented in Chapter 3, as well as the material model validation presented in Chapter 4, this chapter will focus on the results of implementing the HPFRCC material model into a structural component. The goal of this chapter is to model coupling beams tested by previous researchers with RC, while observing the behavior and response of the specimen. Then the same coupling beams will be modeled with the validated HPFRCC material, and the effect of adding HPFRCC to the coupling beams can be seen from the results of the parametric study. Additionally, coupling beams tested with HPFRCC will be modeled. This portion of the modeling effort will demonstrate the ability of the model to effectively represent the behavior of HPFRCC in a structural component. Finally, since the HPFRCC coupling beams did not have a control RC coupling beam, the same reinforcement layout of the HPFRCC coupling beams will be used; however, a reinforced concrete will be implemented as the material to provide the results of a control RC specimen.

6.1 Coupling Beam Modeling Parameters

The next sections will outline the modeling parameters implemented into the coupling beam models. The parameters include constitutive models, reinforcement modeling, finite element parameters, and loading and boundary conditions.

6.1.1 Constitutive Models

For the modeling of the concrete within the RC coupling beams, the default parameters for SBETA concrete within ATENA was used, given the reported compressive strength for the
respective experiments from the literature. The SBETA model is a hallmark of the ATENA program, and it includes the following effects of concrete behavior: nonlinear compressive behavior, fracture of concrete based on nonlinear fracture mechanics, biaxial strength failure criterion, reduction of compressive strength after cracking, tension stiffening, reduction of the shear stiffness after cracking, and fixed or rotated crack models. For smeared reinforcement, a perfect bond is assumed between the concrete and the reinforcement. The nonlinear behavior of the concrete in biaxial stress is described in terms of an effective axial stress and an equivalent axial strain. This approach is taken to eliminate the Poisson’s effect in the plane stress state (Cervenka, Jendele, & Cervenka, 2010). The equivalent uniaxial stress-strain diagram for the concrete is shown in Figure 6-1.

![Figure 6-1. SBETA Equivalent Uniaxial Stress-Strain Relationship from Cervenka et al. (2010)](image-url)
Tension before cracking is assumed linear elastic. After cracking, tension was modeled by the default exponential crack opening law. Concrete compression before peak stress was modeled according to the default CEB-FIP Model Code 90. After peak stress, the softening law of the concrete is linearly descending. The compression plane model is based on the assumption that the compression failure is localized in a plane normal to the direction of the compressive principal stress. The end point of the softening curve is defined by means of the plastic displacement, $w_p$. This parameter is the only deviation applied in the modeling effort from the default settings. It was found that the default value of $w_p$, which was 0.2 in. (0.5 mm), caused a premature and brittle failure of the concrete, so this parameter was increased to 0.06 in. (1.5 mm). The major advantage of formulating the concrete model in this manner to handle the softening is that the concrete model is not mesh dependant (Cervenka, Jendele, & Cervenka, 2010). The concrete compressive softening displacement law is displayed in Figure 6-2.

![Figure 6-2. SBETA Compression Softening Displacement Law from Cervenka et al. (2010)](image)

The biaxial stress failure criterion is defined according to Kupfer et al. (1969), and the shear modulus has a logarithmic reduction after cracking based on the tensile strain normal to the
crack. Similarly, the compressive strength after cracking in the direction parallel to the cracks is reduced similar to the method reported in the Modified Compression Field Theory; however, Gauss’s function is used for the reduction to allow the user to adjust this effect as desired.

Again, for each of the 20 material parameters involved in the SBETA constitutive model, only the softening compression displacement was modified to better capture the softening present in the experiments. For the HPFRCC material, the model, as described in Chapter 4, was implemented.

6.1.2 Reinforcement Modeling

Since this modeling effort is the first analysis in the current research study using reinforcement, modeling decisions were made regarding the type of reinforcement and bond models to use. In the coupling beams, reinforcement is modeled both as discrete truss elements and smeared elements. The primary reinforcement, which includes both longitudinal and diagonal reinforcing bars, is modeled discretely, while the transverse reinforcement is modeled using a smeared reinforcement approach. The use of the smeared reinforcement significantly eased the modeling process and lessened the computational effort, without compromising the results. Steel coupon test results of the reinforcement provide the parameters to define a bilinear stress-strain law with hardening for the modeled reinforcement. ATENA allows for multi-linear stress-strain laws to be used for both the discrete and the smeared reinforcement (Cervenka et al., 2010).

For the RC coupling beams, the default bond-slip model was implemented within ATENA. The default model is the CEB-FIB model code 1990 (CEB-FIP, 1990), and it defines the bond strength depending on the value of the slip between the reinforcement and the surrounding concrete. In the CEB-FIB model, the user selects whether the bar is ribbed and the
confinement conditions. Since all of the primary reinforcement in the coupling beams were within a densely confined concrete core, the bond condition was selected as “good.” This model then assumes that a slip failure occurs due to shearing of the concrete between the ribs of the deformed bar (Cervenka, Jendele, & Cervenka, 2010). The bond-slip relationship and the associated parameters values are shown in Figure 6-3.

![Bond-Slip Relationship](image)

**Figure 6-3. CEB-FIP Model Code 1990 Bond-Slip Relationship (left) with Table of Parameters (right) from Cervenka et al. (2010)**

Since prior research has been conducted on the bond performance of HPFRCC with reinforcing steel, the bond-slip experiments reported by Chao et al. (2009) were reviewed. It was found that the research did not explicitly provide a bond-slip model for HPFRCC, rather it plotted the experimental results of several different types of fibers in varying volume fractions relative to the conventionally reinforced concrete. The research reported that the bond strength of HPFRCC was about 1.5 times that of spirally reinforced specimens subjected to monotonic loading, and about 33% greater than the bond performance when subjected to unidirectional cyclic loading (Chao et al., 2009). Provided that information, the default bond model was used with an increase in the maximum bond stress and an increase in the permitted slip.
6.1.3 Finite Element Modeling Parameters

ATENA provides a finite element mesh tool to define the parameter of mesh generation. For reinforced concrete, with discrete truss elements for the reinforcing steel, the mesh is generated in two phases. First, a mesh of 3-D elements is produced within the pre-processor. In the pre-processor the bars are still maintained as geometrical objects without mesh. When the control is passed to the analysis program, the bar finite elements are generated as embedded elements within the existing mesh of 3-D finite elements. Thus, the user cannot affect the meshing of bars, since it is governed by the overall mesh of the finite elements (Cervenka et al., 2010). The use of embedded reinforcing bars is beneficial when modeling reinforced concrete because the nodes of the discrete reinforcing bar do not have to align with the nodes of the mesh. The elements used will be plane stress elements, and a smeared approach will be used to model and to apply the material properties.

For the boundary conditions of the coupling beam components, an effort was made for each coupling beam to be model the boundary conditions of the experiment. In general, the coupling beams were modeled with large loading blocks on the either side of the beam. The left coupling beam block was restrained on the top and bottom against vertical displacements, while the right edge of the right loading block was fixed against horizontal displacements. The deformations were then imposed by specifying a prescribed displacement along the bottom edge of the loading block through a steel plate. This configuration adequately represent the boundary conditions of the experiments by imposing equal and opposite rotations on each end of the coupling beam, while allowing the coupling beam to expand axially as damage progressed. For the coupling beams with axial restraint (Lequesne, 2011), completely restraining the blocks from axial expansion was found to unconservatively generate a large axial load. Thus, where the axial...
loading was provided in the literature, a corresponding axial load was applied in the model. This approach proved to be sufficient in capturing the appropriate stress and strain state in the specimen.

The coupling beam specimens were modeled with plane quadrilateral elements. They are isoparametric elements with quadratic displacement shape functions, so there are 9 Gauss integration points in each element. These elements are suitable for plane 2-D, axisymmetric, and 3-D problems (Cervenka et al., 2010). The automatic meshing feature employed by the ATENA software was implemented for each model. For the element size, since the HPFRCC had aggregate as large as 0.375 in. (9.525 mm), 0.4 in. (10.2 mm) was determined to be the minimum mesh size. A brief sensitivity study was conducted with increasing element size, and it was found that an element mesh size of 1 in. (25 mm) was found to accurately capture the results while decreasing the required computational time.

The solution parameters for ATENA implements the full Newton-Raphson solution of nonlinear equations. The main feature of the full Newton-Raphson procedure is that it continues to update the stiffness matrix with each step, while the Modified Newton-Raphson procedure continues to use the initial stiffness obtained in the first step. While it may take more computational time to update the stiffness matrix, it is believed that the convergence to the solution is superior with the full Newton-Raphson technique. The node numbers were optimized with the Sloan method, and iteration limit for each step was set to 50. The default displacement, residual, absolute residual, and energy error tolerances were used.

6.2 RC Coupling Beam Component Modeling

The first modeling effort was of the conventionally reinforced and diagonally reinforced coupling beams tested by Galano and Vignoli (2000). Many of the details of the testing program
were presented previously in Chapter 5; however, the modeling portion of this research only focused on modeling four of the coupling beams from the experimental program: conventionally reinforced with monotonic loading, conventionally reinforced with cyclic loading, diagonally reinforced with monotonic loading, and diagonally reinforced with cyclic loading. The results of the RC coupling beam models will be reviewed in the next sections, as well as the results of the coupling beams implementing the HPFRCC material model described in Chapter 4.

6.2.1 RC Coupling Beam Modeling Results

The coupling beams tested by Galano and Vignoli (2000) were modeled with the same geometry reported by the researchers. The left block was restrained on the top and bottom against vertical movement, while the right block was restrained from lateral movement. In the experiment, no comment was made about the ability of the coupling beam to expand axially or about the axial force generated during loading. The authors only mention that undesirable movements were prevented. A model was conducted with axial expansion prevented by fixing the left block from horizontal movement, and the result was a very large axial force and premature failure of the coupling beam. Since this was not what was reported by the researchers, the subsequent models were conducted without the axial restraint, and the modeling loading and boundary conditions for Specimen P01 may be seen in Figure 6-4.
Specimen P01 is a conventionally reinforced coupling beam that was loaded monotonically. It was found that the modeling results quite reasonably matched the experimental result. The yield displacement in the model occurred at 0.93% drift, while in the experiment, yielding of the reinforcement was experienced at 0.84% drift. This is well within a reasonable tolerance, and the size of the loading increment in the model may have attributed to the slight overestimation of the yield displacement. The shear force in the specimen when the primary reinforcement yielded and at the peak load were essentially identical to the experiment; however, the displacements at which the peak load occurred differed more substantially. The peak loading in the model occurred at 1.4% drift, and it did not occur in the experiment until about 4.0% drift. This may seem like a substantial difference, but upon review of the overall load-displacement responses of the model and the specimen, the overall behavior of the specimen was still closely captured. The shear versus displacement behavior of Specimen P01 for both the ATENA model and the experiment are displayed in Figure 6-5.

Figure 6-4. Specimen P01 Model with Loading and Boundary Conditions
Figure 6-5. Shear versus Displacement for Specimen P01

After yielding of the primary reinforcement, a plateau occurred in the loading. In the model, this plateau was characterized by a gradually decreasing slope and in the experiment it was gradually increasing. The slight difference in the two behaviors accounted for the rather large difference in the “displacement at peak load” evaluation metric. The model failed abruptly when some of the smeared vertical reinforcement ruptured. This type of failure is usually indicative of insufficient shear reinforcement relative to the flexural reinforcement, and the result is a diagonal tension failure. The principal stress in the specimen at imminent failure is displayed in Figure 6-6, and the wide spread heavily stressed state of the coupling beam can be seen. Overall, the behavior of the model was quite similar to the experimental result.
The next specimen investigated was Specimen P05. The area of primary reinforcement, geometry of the coupling beam, and the loading were all the same as Specimen P01; however, the primary reinforcement was placed diagonally rather than along the axis of the beam, as in the conventionally reinforced Specimen P01. The layout of the reinforcement, as well as the loading and boundary conditions are shown in Figure 6-7.
Figure 6-7. Specimen P05 Model with Loading and Boundary Conditions

The Specimen P05 model slightly over-predicted the shear strength and under-predicted the displacement at the initial yielding of reinforcement by 11% and 7%, respectively. This is an indication that the model was initially stiffer than the experimental test setup. The increase in stiffness relative to the conventionally reinforced coupling beams is most likely the result of the component of the diagonal reinforcement in the direction of loading. The ultimate load in the model was within 1% of the actual experimental value, but the same gradually descending model post-peak plateau relative the gradually ascending post-peak plateau of the specimen resulted in about a 45% error in the prediction of the displacement at peak load. While at first glance this seem like a very large discrepancy, the shear versus displacement behavior, shown in Figure 6-8, shows that the experimental response was actually rather well captured. The modeling behavior typically is characterized by a linear ascending branch followed by a sharp transition to a plateau upon yielding of the reinforcement. This differs from the experimental result which more gradually softens as the reinforcement yields. Similarly, at large displacements, the experiment
can accommodate gradual softening of the material with damage, while the modeling formulation typically demonstrates a more catastrophic event with damage.

![Shear versus Displacement for Specimen P05](image)

**Figure 6-8. Shear versus Displacement for Specimen P05**

Specimen P05 developed flexural cracks at the end of the coupling beam that allowed large rotations after yielding of the reinforcement, and diagonal cracking was present along the compressive strut. The failure eventually occurred when the tension diagonal bar fractured at the beam-wall interface. The laboratory test showed a similar performance of the coupling beam during the early stages, but in the lab, the concrete compressive strut began to soften, so the shear force gradually decreased without ever fracturing the reinforcement. Overall the response of
Specimen P05 was reasonable; however, the SBETA model does not seem to be fully emulating the softening concrete behavior with damage experienced in laboratory tests.

Figure 6-9. Principal Stress and Cracking in P05 near Failure

The next specimen investigated was Specimen P02. This coupling beam was identical to Specimen P01 except for the loading regime. For the loading of the specimen, two cycles were conducted at $1/3\delta_y$, $\delta_y$, $3\delta_y$, and $5\delta_y$, where $\delta_y$ is the applied displacement when the primary reinforcement experienced initial yielding. This loading protocol took considerably more computational time, so the mesh was increased to 2 in. (50 mm) for the cyclic models. A brief sensitivity study was conducted to monitor the effect on the results, and the change in force and displacement was found to be negligible.
In the experiment it was found that at low drift ratios that the conventionally reinforced coupling beams performed similarly to the diagonally reinforced coupling beams, but at large displacement, the strength began to decay rather rapidly. A similar behavior was seen with the ATENA model of the specimen; however, the model was not able to achieve displacements as large as in the experiment. For the model, the shear and displacement at initial yielding of the reinforcement were within 6% and 9% of the experimental result, respectively. Also, the peak shear load was essentially identical to that experiment; however, when the experiment began to experience damage and lose capacity, the model lost capacity rapidly. The envelope of the experimental shear-displacement response and the ATENA result are compared in Figure 6-10.

Figure 6-10. Shear versus Displacement for Specimen P02
The rapid strength loss of the specimen was the result of a failure plane generating at the coupling beam-wall interface. Upon reverse cyclic loading, flexural cracks formed at the beam-wall interface in each direction. At increasingly large displacements, the flexural cracks eventually coalesced to form a failure plane at the right end of the coupling beam. This mechanism is the same as previously described for the sliding shear failure mode, and the principal stress in the specimen at imminent failure is displayed in Figure 6-11. Sliding shear is a common phenomenon in coupling beams, particularly conventionally reinforced coupling beams. The early experimental work by Paulay (1971, 1974, 1977) on coupling beams made this effect better known and started the movement towards diagonal reinforcement for coupling beams with lower aspect ratios. The model may be overly pessimistic about the sliding shear failure of coupling beams due to the inability to model aggregate interlock, dowel action of the longitudinal reinforcement, or shear-friction.

Figure 6-11. Principal Stress and Cracking in Specimen P02 near Failure

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The final coupling beam modeled in this portion of the study that was tested by Galano and Vignoli (2000) is Specimen P07. This specimen is identical to the diagonally reinforced Specimen P05; however, reverse cyclic displacement were imposed on the specimen. The cyclic loading applied to this specimen followed the same displacement protocol as in the conventionally reinforced Specimen P02. The displacement and shear force at the initial yielding of the primary reinforcement were within 6% and 8% of the experimental values, respectively. Also, peak strength was predicted to within 1% of the experimental value; however, the peak value occurred at a drift of 20% less than experienced in the laboratory. The trend of the model accurately capturing the strength and performing well at the early stages of the experiment continued, but as the displacements increased, the model typically lost its capacity more rapidly than the experiment. At the $2\delta_y$ displacement level, the model began to soften relative to the laboratory test. The experiment maintained about 94% of its peak capacity, and the model only maintained 68%. At the $3\delta_y$ displacement level, the experiment had reduced to 58% of its peak capacity, and the model had reduced to 38%. While this is still less than the experimental value, the model did more accurately capture the later cycles. The applied shear force versus displacement response of the experiment and ATENA model are plotted in Figure 6-12.
The models confirmed that the cyclic behavior of the diagonally reinforced coupling beam provides more stable hysteretic behavior. When comparing the cyclic shear-displacement response of the conventionally reinforced coupling beams to the diagonally reinforced coupling beams, it can be seen that cyclic coupling beam dissipates more energy, exhibits a more ductile behavior, and a superior residual strength. Although the models did not maintain the same load capacity at large displacements as the experimental tests, these findings are consistent with the experimental results.
The damage in the diagonally reinforced coupling beam was well distributed throughout the specimen. Some stresses were localized along the beam-wall interface; however, the behavior of the specimen was dominated by the steel diagonal bars and large diagonal concrete compressive struts along the reinforcing bars. Considering the stable hysteretic behavior of the diagonally reinforced coupling beam, this was not surprising. Specimen P07 did not experience an abrupt failure in the model; instead, the reverse cyclic displacements caused widespread damage to the specimen and a reduction in load capacity without fracturing a reinforcing bar. Figure 6-13 shows the principal stresses in the specimen near the termination of model. The distributed damage and compressive struts, as well as the large stress in the diagonal bars can be seen. The next section will outline the results of implementing the HPFRCC material model into the previously modeled coupling beams.
6.2.2 RC Coupling Beam Parametric Study

The RC coupling beams by Galano and Vignoli (2000) modeled in the preceding section were next modeled with HPFRCC. Since the cyclic behavior of HPFRCC has not yet been evaluated and this is the first effort to implement the HPFRCC model into a structural component, only the monotonically loaded specimens, P01 and P05, were explored for this parametric study. The compressive strength of the concrete in P01 and P05 was 7.09 ksi (48.9 MPa) and 5.78 ksi (39.9 MPa). Both of these compressive strengths exceed the compressive strength of NM6, which was 4.76 ksi (32.8 MPa), but the modeling approach for this portion of the research was to investigate the effect of implementing the exact NM6 model developed in Chapter 4 into the RC coupling beams. The cyclic properties of the HPFRCC would be evaluated in the next phase of the modeling program when the HPFRCC coupling beams by Lequesne (2011) were investigated.

First, Specimen P01 was modeled. The NM6 HPFRCC model was applied only to the coupling beam elements, and the adjacent wall blocks maintained their original properties. The same boundary conditions, solution parameters, and loading were implemented. The evaluation of the research by Chao et al. (2009) led to the enhancement of the maximum permissible bond strength, and the remainder of the configuration of the model was identical to the version with an RC coupling beam.

The shear versus displacement response of the HPFRCC coupling beam relative to the RC coupling beam results is displayed in Figure 6-14. The marked difference in the behavior is immediately apparent. The HPFRCC coupling beam experienced a much larger load before cracking initiated in the specimen. Additionally, the capacity was increased by about 40%, while the deformation capacity of the coupling beam was increased by about 50%. These considerable
increases are with the NM6 concrete, so the material properties were not tailored to specific concrete compressive strength of the original RC coupling beams, which was 7.1 ksi (48.9 MPa). The large increase in strength may be related to the nature of the reinforcement layout of the conventionally reinforced coupling beam. Since these coupling beams do not have the reinforcement layout that promotes a strut mechanism (like diagonally reinforced coupling beams), they must rely on the transverse reinforcement to resist the shear force. Since the HPFRCC addition contributes significantly to the shear resistance of the specimen, this large increase in strength is reasonable.

![Figure 6-14. Shear versus Displacement for Specimen P01 with HPFRCC](image)

On the local behavior side of the performance review, the conventionally reinforced coupling beam still could not evade an eventual shear sliding failure. The HPFRCC increased the shear capacity of the specimen, so plastic hinges formed at the ends of the coupling beam. At
large deformations, the concrete subjected to compression in the toe of the coupling beam began to crush, and the flexural cracks at the beam-wall interface began to open substantially. Eventually, a sliding shear failure occurred at the interface; although, the coupling had been able to achieve considerably larger displacements and an increased strength capacity. The minimum principal stress in the specimen at failure is shown in Figure 6-15.

![Figure 6-15. Principal Stress in Specimen P01 with HPFRCC Failure Mode](image)

Next, Specimen P05 was modeled with the HPFRCC material model in the coupling beam. The same approach was taken as performed for the parametric study in Specimen P01, and the exact material properties from NM6 in Chapter 4 were implemented. In Figure 6-16, the shear versus displacement response of the HPFRCC model shows a similar trend in that both the strength and deformation capacities were increased. In Specimen P05, the strength capacity was increased by about 25% and the deformation capacity was increased by about 18%. The increase in strength and deformation was less pronounced than in the conventionally reinforced specimen...
most likely as a result of the reinforcement layout. In diagonally reinforced coupling beams, the behavior is largely controlled by the diagonal reinforcing bars. While the addition of the fibers did increase the shear capacity, the mechanism by which the majority of the force was resisted was still through a compressive strut aligned along the length of the reinforcement.

![Specimen P05](image)

**Figure 6-16. Shear versus Displacement for Specimen P05 with HPFRCC**

The diagonal compressive strut at peak load can be seen in the image on the left in Figure 6-17. It shows a clear diagonal compression, with some flexural cracks developing at the beam-wall interface. However, as before, Specimen P05 with HPFRCC experienced the same sliding shear failure once the damage accumulated at the beam-wall interface. The sliding shear failure was also the eventual failure mode of the HPFRCC coupling beams tested by Lequesne (2011),
and the principal stresses in Specimen P05 at failure are shown in the image on the right in Figure 6-17.

![Image](image.png)

**Figure 6-17. Specimen P05 with HPFRCC at Peak Load (left) and at Failure (right)**

The parametric study of the RC coupling beams with HPFRCC proved to be an enlightening exercise. The addition of HPFRCC increased both the strength and the deformation capacity, as expected. Additionally, the failure modes aligned well with those seen by previous researchers in HPFRCC coupling beams. The shear resistance was increased, so ductile flexural hinges formed at the end of the coupling beams. The beams did eventually fail in sliding shear, but it was with a much increased deformation capacity. The next phase of the modeling program is to model HPFRCC coupling beams by Lequesne (2011), and these results will be discussed in the next section.

### 6.3 HPFRCC Coupling Beam Component Modeling

With success in modeling coupling beams with reinforced concrete and with success in modeling the HPFRCC material tests, the next step was to model the coupling beams designed
and tested with HPFRCC. While hypothetical previous results were presented of the RC coupling beams with the HPFRCC material, these tests will serve as a better validation metric for the ability to model HPFRCC coupling beams. Lequesne (2011) reported the testing of three coupling beams; however, since Specimen CB-1 suffered an unexpected premature failure, only Specimen CB-2 and Specimen CB-3 will be modeled in this study. The coupling beams were tested cyclically; however, the modeling results will present only the monotonic coupling beam results. Since material tests were not conducted on the cyclic properties of HPFRCC, any necessary adjustments to the HPFRCC material properties were not available for the present research.

A major consideration during the modeling of these coupling beams was the application of the axial load. The test setup, shown in Figure 6-18, utilized steel links to ensure double curvature of the specimen.

![Figure 6-18. Coupling Beam Component Test Setup at U-M from Lequesne (2011)](image-url)
While the links were effective in preventing the end block from rotating, it also provided some axial restraint to the specimens. Fortunately, load cells were attached to the steel links, so the axial force was monitored throughout the testing. It was found that there was a good correlation between the axial load and the lateral displacement, and Figure 6-19 displays the axial load as a function of the axial capacity versus the peak cycle drift for each specimen. The axial capacity here was defined with the following formula:

$$P_0 = f_c' (A_g - A_{steel}) + f_y A_{steel}$$

(6.1)

Where $P_0$ is the axial capacity, $f_c'$ is the concrete compressive strength, $A_g$ is the gross area of the cross-section, $f_y$ is the yield strength of the steel, and $A_{steel}$ is the area of steel, including the axial component of the diagonal bars. An incremental axial load case based on the imposed displacement was defined in the model based on the relationships shown in Figure 6-19, and it was found that force condition in the models favorably resembled the experimental test setup.
It should be noted again that due to an error with the control instrumentation for Specimen CB-3, the specimen was loaded asymmetrically, and this resulted in a large axial force in the coupling beam. Due to the capacity of the load cell and concern about an unrealistically large axial load being generated in the coupling beam, the tension in the steel links was relieved twice. Since it is unknown how much axial strain was relieved during this process, an effort was not made to model the axial force relief during testing. While this may have affected the modeling results somewhat, the overall performance of the CB-3 coupling beam was captured, and this discrepancy is not considered problematic.

The next sections will review the results of the HPFRCC coupling beam modeling, as well as the performance of “control” RC coupling beams with the same reinforcement layout and loading and boundary conditions.
6.3.1 HPFRCC Coupling Beam Modeling Results

The HPFRCC coupling beam models had a similar boundary condition as the previous RC coupling beam models: the left wall block was restrained on the top and bottom against vertical movement, the right wall block was restrained against lateral movement, and a vertical incremental displacement was applied to the right wall block. The only real difference to the loading regime was the application of the axial load. The incremental axial load relationship was developed as a function of the vertical displacement of the specimen. A piecewise linear function was developed based on the axial force-peak cycle drift relationship shown in Figure 6-19. This function was developed such that at the appropriate axial force was applied to the specimen for the given vertical displacement. The finite element model with the loading and boundary conditions is depicted in Figure 6-20.

![Figure 6-20. Specimen CB-2 Model with Loading and Boundary Conditions](image-url)
For the modeling results, the specimens were loaded monotonically since the cyclic properties of the HPFRCC were unknown. An envelope of the cyclic response of the laboratory tests was generated, and the monotonic results were compared to the envelope. Although the HPFRCC material implemented into the experimental tests was the same NM6 described in Chapter 4, the compressive strength of the concrete in Specimen CB-2 was 7.5 ksi (52 MPa) at the time of testing. Since the compressive strength in the material model is a function of the percentage of the peak compressive strength versus strain, the compressive relationship was only modified by increasing the peak compressive strength of the HPFRCC material to match the experiment. Since the tensile strength is more a function of the fibers and the volume fraction, the tensile properties remained unchanged. The initial stiffness of the model aligned will with the laboratory test; however, the onset of damage was delayed in the model relative to the experiment. This delay in the onset of damage in the model may be related to softening of the experiment due to the cyclic loading, while the model was only loaded monotonically. The experiment began to soften, and initial yielding of the primary reinforcement occurred at a shear force of 96.4 kips (429 kN). In the model, the response of the specimen did not begin to soften until reinforcement yielded at 112 kips (499 kN). Thus, the shear force at first yielding was over predicted by about 16%, while the displacement at first yielding was under predicted by about 13%. The peak strength and displacement were much more closely predicted; the peak strength was within 3% of the laboratory test and the displacement at the peak strength was within 1% of the experiment. Finally, the displacement and strength at failure were quite close to the experimental test. The shear versus displacement response of the model relative to the envelope of the experiment is displayed in Figure 6-21.
The behavior of the coupling beam did closely resemble that of the experimental test. The image on the left in Figure 6-22 shows the X-stress in the specimen near failure. The typical large diagonal strut is evident, with a high concentration of stress located in the corner of the specimen at the location of the bend in the diagonal reinforcement. In Specimen CB-1, it was this stress that ultimately caused the premature failure of the coupling beam because the plastic hinge region was not adequately confined. Although Specimen CB-1 was not modeled explicitly in the current research study, it presents an opportunity for future researchers to further evaluate the modeling of HPFRCC coupling beams. The image on the right displays the Y-stress in the coupling beam near failure. The connection was strengthened against sliding shear through the
addition of U-shaped dowel bars at the beam-wall interface. The U-shaped bar terminated 6 in. (152 mm) into the coupling beam, and this localization of stress can be seen on the right in Figure 6-22. The subsequent beam utilized straight dowel bars in an effort to prevent this immediate drop in capacity. In any event, this Specimen CB-2 experienced the same failure mode due to the localized damage at the interface, which resembles the eventual failure in the experimental test.

Figure 6-22. X-Stress (left) and Y-Stress (right) in Specimen CB-2 near Failure

Figure 6-23 shows the shear stress in the specimen near failure. The shear stress shows a clear compressive strut along the diagonal in the coupling beam, as well as the alignment of the cracks in the beam with the shear stress.
The final coupling beam modeled was Specimen CB-3 from Lequesne (2011). It was more difficult to know what to expect from this modeling effort since there were some instrument errors in the laboratory at the time of testing. The lateral displacement imposed by the actuator and the slip of the base block were measured during testing to determine the appropriate displacement to be applied to the specimen; however, the sign convention for one of the LVDTs measuring the base block slip was incorrect. This resulted in the specimen being loaded asymmetrically, with a peak drift of about 7% in one direction and less than 4% in the other direction. Additionally, the axial force in the coupling beam was relieved on two separate occasions to prevent an unrealistically high axial load and premature failure of the specimen. Given this unknown loading regime with respect to both force and displacement, the modeling effort proceeded with the purely monotonic results and the best estimate of the axial force as reported by Lequesne (2011). It was believed that by applying the appropriate axial force for the given displacement level, the response may be similar to the experimental test. The ATENA
model with loading and boundary conditions is shown in Figure 6-24. The difference in the beam-wall connection can be seen; dowel bars extending 8 in. (203 mm) into the coupling beam were used to enhance the sliding resistance without creating an abrupt drop in moment capacity.

![Figure 6-24. Specimen CB-3 with Loading and Boundary Conditions](image)

Since the monotonic loading was applied to the model, the axial forces corresponding with the direction of loading that experienced about 7% drift was implemented. The envelope for the experimental shear-displacement result in this direction compared to the ATENA model result is plotted in Figure 6-25. As seen in Specimen CB-2, the model for Specimen CB-3 did not experience the same reduction in stiffness during the pre-peak portion of the loading as the experiment. The applied shear at initial yielding of the diagonal reinforcement was over predicted by 15%, and the displacement at initial yielding was under predicted by about 12%.
This may be due to a shortcoming in the model and/or the experimental effects of both the asymmetric loading and the unrecorded large axial force in the experiment. Also, the early softening of the experiment may have been related to the cyclic loading of the experiment compared to the monotonic loading of the model. The peak load of the model did closely capture that observed in the experiment, and it was within 4%; however, the corresponding drift at the peak load was under predicted by 25% in the model. The nature of the unknown loading in the experiment makes it difficult to more closely model the experimental response. The initial softening of the specimen could be the result of the large axial force generated, and the subsequent release of axial strain and relaxation of the specimen may have allowed the large displacement. The displacement at which a 15% reduction in the peak load occurred was predicted within about 7% by the model. Near failure, the model experienced an explosive event, and the capacity dropped rather rapidly, while the experiment demonstrated a more gradual decrease in capacity.
The typical diagonal compressive strut was seen again in Specimen CB-3, and it can be seen on the left in Figure 6-26. A major difference in the response of Specimen CB-3 relative to previous HPFRCC coupling beams is the distributed damage at the end of the coupling beam. Specimen CB-3 utilized two dowel bars to increase the shear sliding resistance, and it was assumed that the dowel bars contribution to the moment capacity would decrease linearly into the coupling beam. The idea was that this gradually decreasing capacity would prevent the creation of a localized failure plane and distribute the damage over the confined plastic hinge region. In the image on the right in Figure 6-26, the X-stress at failure can be seen, and the large distributed damage on the left end of the coupling beam is pronounced.

Figure 6-25. Shear versus Displacement for Specimen CB-3
Figure 6-26. X-Stress in Specimen at 3% Drift (left) and at Failure (right) in Specimen CB-3

Figure 6-27 displays the Y-stress on the left and the shear stress on the right. The distributed damage throughout the confined plastic hinge region can again be clearly seen. Eventually this damage resulted in a loss in capacity around 4% drift. This localized damage was very similar to the experimental result, and Specimen CB-3 at failure can be seen in Figure 5-17.

Figure 6-27. Y-Stress (left) and Shear Stress (right) in Specimen CB-3 at Failure
Although not all aspects of the HPFRCC coupling beam model identically matched the experimental results, many of the main features were captured. The peak load and displacement at peak load were consistently close to the experimental result; the distribution of damage and ductile behavior were seen; the respective failure modes were captured; and an accurate evaluation of the beam-wall interface connection details was observed.

6.3.2 HPFRCC Coupling Beam Parametric Study

The final aspect of the modeling program was to assess the response of the HPFRCC coupling beams if traditional reinforced concrete was used in the model. For the RC coupling beams, the reinforcement layout, loading conditions, and boundary conditions were all identical to those previously outlined for the HPFRCC coupling beams. The only difference was an SBETA concrete model was implemented, and the concrete softening displacement parameter was increased to 0.06 in. (1.5 mm), as done in the previous RC coupling beam models. The concrete compressive strength in the SBETA model was identical to the specified concrete compressive strength in the experiment.

The stiffness of the RC coupling beams was very similar to the initial stiffness of the HPFRCC coupling beams; however, the absence of the pseudo-strain hardening effect of the HPFRCC caused the concrete to crack at a much lower load level and earlier displacement. Once the concrete began to crack, the tension force was taken almost entirely by the reinforcement, and the reinforcement in the RC beams showed initial yielding at about a 30% lower applied shear. It may be a coincidence, but the contribution of HPFRCC to the shear resistance of the coupling beam was estimated to be about 30%, so this reduction in the RC beams would seem reasonable. Similarly, the peak shear capacity of the RC coupling beams was about 30% less and the deformation capacity at failure was almost 50% less than the HPFRCC
coupling beams. The shear versus displacement result of Specimen CB-2 and Specimen CB-3 are shown in Figure 6-28 and Figure 6-29, respectively.

**Figure 6-28. Shear versus Displacement Response of CB-2 Specimen with RC**

**Figure 6-29. Shear versus Displacement Response of CB-3 Specimen with RC**
The parametric study involving the application of RC to the HPFRCC coupling beams demonstrates several benefits of HPFRCC in the design of coupling beams. The increased shear strength from the use of HPFRCC may allow a designer to permit a larger maximum coupling beam shear stress, to reduce the reinforcement required, or to reduce the size of the coupling beam. Also, the increased deformation capacity may allow a ductile structural system performance in the event of an earthquake. A focus of this research was to provide a method to assess the performance-based design implications of adding HPFRCC to a damage-critical member, and both the laboratory tests and the subsequent modeling effort illustrate clear application of HPFRCC and the resulting enhancement in the component response. The performance of the models relative to the experimental tests is provided in a tabular format in the next section.

6.4 Compiled Coupling Beam Model Performance Assessment

The models were evaluated through several global parameters: displacement capacity, strength capacity, stiffness, and ductility. The experimental response for each of these evaluation parameters was provided previously for each coupling beam modeled in Table 5-3. Table 6-1 and Table 6-2 provide the results discussed in this chapter for each of the evaluation parameters obtained from the modeling research program, and comparisons of the ATENA results to the experimental results were also included. Table 6-1 displays the results relating to strength and stiffness, and Table 6-2 includes the results pertaining to displacement and ductility. In the tables, the subscript “e” indicates an experimental result. The same definitions of the parameters defined for Table 5-3 apply in the following two tables.
The implementation of the HPFRCC material model into structural components was shown to be a successful effort. It should be noted that if a designer wants to rely on mechanical properties similar to those reported in this study, some quality assurance and quality control measures should be implemented during construction. For instance, some care should be taken to ensure the random dispersion of fibers throughout the structural component. Additionally, several uniaxial compression, uniaxial tension, and biaxial compression specimens would be warranted to have an idea of the measure of strength and ductility of the particular mix and fiber type. One method to enhance the quality of the HPFRCC structural component is to use precast elements, as recommended in this current research program. In the preceding chapter, the ability to model HPFRCC coupling beams has been demonstrated, and additional parametric studies
were shown to illustrate the potential effect of including HPFRCC in regular RC coupling beams. Similar parametric studies could be performed with varying levels of HPFRCC strength and ductility for designers developing a unique mix. This overall modeling program is further significant because if HPFRCC is to be embraced by the design community, an HPFRCC model must be available that can be readily integrated into commercially available software. While some open-source nonlinear platforms have received considerable research attention, it will most likely be through commercially available software where the design community can conduct its own parametric studies on the effects of HPFRCC when considering it as a viable alternative to the design of damage-critical structural components.
CHAPTER 7. BACKGROUND AND EXPERIMENTAL PLAN FOR PILE-WHARF CONNECTION TESTING

Another aspect of this research focused on laboratory testing of pile-wharf connections. Admittedly, this is a rather abrupt change in focus from the previous chapters, but this aspect of the research will be linked with the HPFRCC topic in Chapter 9 where the pile-wharf connection is modeled with HPFRCC.

This chapter provides an overview of the current concerns about existing connection designs, as well as a description of previous research on the topic. Additionally, a large-scale specimen was tested in the NEES MUST-SIM facility at the University of Illinois at Urbana-Champaign, and this chapter will discuss in detail the experimental setup, testing plan, and specimen instrumentation.

7.1 Introduction

Ports are a critical infrastructure component to the economy. They are the beginning and ending points for the exporting and importing of goods, and ports employ millions of workers worldwide. However, in the United States many container ports are located in regions of high seismicity, so they are vulnerable to potentially devastating physical and economic damage. This economic damage has both short-term and long-term implications. As an example, in 1995 the Port of Kobe was the 6th busiest container port in the world when the Great Hanshin earthquake struck and caused an estimated $10 billion of damage. During the time needed for repairs, many clients had to move their business to neighboring ports, and often that business never returned. In the case of Kobe, the once flourishing port had dropped to the 17th busiest port in the world by 1997, and by 2005 it had dropped to 39th (Chang, 2000). This serves to
emphasize the importance for ports to not only avoid collapse, but also to remain mostly operational, after a large seismic event.

Most wharves on the west coast of the United States employ vertical precast-prestressed concrete piles with moment-resisting connections to the cast-in-place reinforced concrete wharf deck, as shown in Figure 7-1.

![Figure 7-1. Typical Pile Support Structure for Seismic Resistance of a Wharf (Roeder et al., 2001)](image)

It can be seen that the piles widely vary in terms of their length; however, given the typically rigid nature of the wharf, the piles will displace the same amount under a seismic event. Since the shorter piles have a considerably greater flexural stiffness, large moment and rotational demands may cause significant post-yielding damage in the shorter piles before the longer piles have even developed their full lateral capacity, as depicted in Figure 7-2. Such a structure then behaves as a ductile frame, with plastic hinges forming in the piles. Previous researchers have attempted to test the response of the shorter piles by using a testing set-up with a pinned connection at one end of a length of pile; this is to simulate the response of the pile from an inflection point to the pile-wharf connection. As will be discussed in more detail later, this
current experimental study imposed both rotations and displacements to the end of the length of pile tested, allowing for the more realistic simulation of a longer (continuous) pile length with appropriate boundary conditions.

![Idealized Port Structure under Seismic Loading](image)

**Figure 7-2. Idealized Port Structure under Seismic Loading (Jellin, 2008)**

Typical port structures are constructed by first driving the prestressed concrete piles until they achieve the necessary compressive load capacity, either through direct bearing on bedrock or until sufficient skin friction is obtained. Since this seldom coincides with the finished elevation of the wharf deck, piles extending above the finished elevation are cut to the proper elevation, while piles driven to a lower elevation are extended using what is effectively a reinforced concrete column extension (Roeder et al., 2005). The pile is then connected to the wharf deck by a variety of means, with Figure 7-3 showing three common connection details currently in use: dowel bars, embedded piles, and extended strands. For purposes of this document, the “pile-wharf interface” references the deck face closest to the pile (and not the actual end of the embedded pile, which typically extends a short distance into the wharf deck). Extended strand connections are made by crushing the concrete at the pile end to expose the
prestressing strands, and then casting the strands into the deck. If necessary, the strands may also be bent to achieve the requisite anchorage in the wharf deck. The embedded pile connection is achieved by casting the pile end a sufficient distance into the pile to develop a moment connection, while the dowel bar connection, which is the focus of this research, is constructed by grouting steel reinforcing bars (which extend into the wharf deck) into corrugated ducts located in the pile. To the knowledge of the author, the embedded pile configuration is not regularly used in practice, and for seismic designs, a dowel bar connection is typically required.

![Typical Pile-Wharf Connections](image)

*Figure 7-3. Typical Pile-Wharf Connections (Jellin, 2008)*

For dowel bar connections, the pile itself normally extends up to 3 to 4 in. (75 to 100 mm) into the bottom of the wharf deck slab (Roeder et al., 2005). Dowels are then grouted into corrugated metal ducts in the pile and embedded into the deck concrete to complete the connection. Figure 7-4 illustrates four commonly used connection details for the dowel bar
connection. Inward and outward bent bars use hooked dowel bars bent in an L-shape into the top of the wharf deck, placed in a radial or orthogonal pattern. Both of these designs can interfere with the placement of the deck reinforcement and cause difficulty in concrete placement. To ease congestion problems, the T-headed dowel bar connection and the bond bar connections are often used. The T-headed dowel bar uses straight dowels with a circular or square steel plate welded on one end of the bar to achieve mechanical development. The bond bar connection uses straight dowels with a bulbous end grouted into the pile and a T-headed bar with a bulbous end cast into the deck directly adjacent to the dowel bar – the idea is that the bulbous ends will serve as a lap splice to develop the necessary connection continuity (Stringer, 2010). The most common connection on the west coast of the United States is the T-headed dowel bar construction, and this is the type of connection explored in the current laboratory testing program. Once the connection detail is implemented, the reinforced concrete cast-in-place deck is formed and cast. An alternative deck detail can sometimes be to use precast, prestressed concrete panels spanning between cast-in-place reinforced concrete pile caps, with such a precast deck system then covered with a reinforced concrete topping slab.
As previously mentioned, a few past earthquakes have caused substantial damage to port structures, particularly at the pile-wharf connections. Common damage types have included severe cracking, connection spalling, dowel bar yielding, dowel bar buckling, dowel bar pullout, and dowel bar fracture. Figure 7-5 shows examples of such damage from the Loma Prieta (1989) and Hanshin (1995) earthquakes. In each case, the connection damage resulted from large flexural (and sometimes also shear) demands coupled with insufficient shear and other confining steel in the pile.
To better understand the capacity, ductility, and damage evolution of the common types of pile-wharf connections, and to explore possible alternative connection details, previous researchers have undertaken a variety of experimental testing programs, which will be summarized next.

7.2 Previous Pile-Wharf Connection Research

Research about the behavior of pile-wharf and pile-pile cap connections has been conducted by several researchers over the years. The following sections outline that previous research which is most relevant to the current study.

7.2.1 University of Canterbury, New Zealand

The University of Canterbury tested six pile to pile-cap connections, each with 15.7 in. (400 mm) diameter octagonal prestressed (and precast) concrete piles embedded into rectangular cast-in-place reinforced concrete pile caps. Of the six specimens tested, two utilized the...
embedded pile connection, three implemented the extended strand connection method, and only Specimen PC6 utilized the dowel bar method similar to that investigated in this study. That connection was achieved by epoxying four #6 diameter (D19) dowel bars into 21 in. (530 mm) long, 1.57 in. (40 mm) diameter drilled holes. Those bars were then anchored into the pile cap with a standard 90 degree inward bar bend, with the pile itself embedded 2 in. (50 mm) into the pile cap. Figure 7-6 shows an illustration of the Specimen PC6 connection details. Of the six specimens, PC6 performed the worst; it was the only specimen to not exceed the theoretical moment capacity, and it suffered the worst deterioration in capacity due to severe spalling of the pile. The relative superior performance of the embedded pile and extended strand connections may be attributed to their use of either ten prestressed tendons or ten prestressed tendons and ten reinforcing bars to develop the connection in the embedded pile and extended strand connections, while the dowel connection only had four dowel bars extending into the wharf deck. In fact, it was noted by the authors that the inclusion of more dowel bars would have likely increased the strength and stiffness of the connection (Joen et al., 1988).
7.2.2 University of California, San Diego

A 1997 study at the University of California, San Diego, also tested six pile-pile cap connections. The STD1 Specimen is most similar in many details to the emphasis of the current study – it utilized a 12 in. (305 mm) square precast prestressed concrete pile embedded 3 in. (75 mm) into a cast-in-place reinforced concrete pile-cap. This connection design was a standard CalTrans connection designated as a Class 70 Ton Pile Standard Plan B2-5 Alternative ‘X’, and its details are shown in Figure 7-7. The dowels for this connection were six straight #6 diameter (D19) bars embedded 20 in. (508 mm) into the pile cap (Silva et al., 1997).
The STD1 Specimen did achieve its theoretical flexural capacity, but it experienced significant strength deterioration due to spalling of the cover concrete in the pile under cyclic lateral loading (with a varying axial load). There was minimal cracking in the pile cap, and a review of the steel strain gage readings indicated that reinforcement in the pile cap never yielded, implying that the capacity of the pile cap was not reached; however, the significant reduction in the cross-section of the pile due to spalling caused a significant drop in lateral capacity.
Sritharan and Priestley (1998) tested one large-scale “pile-deck” connection specimen for the Port of Los Angeles, to examine the performance of an embedded dowel connection with T-headed bars. The connection used eight #10 diameter (D32) dowel bars with bulb ends cast into the end of the pile, with 29 in. (735 mm) embedment into the deck. Eight 26 in. (660 mm) long #9 diameter (D25) T-headed bond bars were cast next to the dowel bars, with a #5 (D16) spiral at 3 in. (75 mm) on center surrounding the entire connection reinforcement in the deck. Details of the connection are shown in Figure 7-8.

[Diagram showing the connection details]

Figure 7-8. Connection Details from Sritharan and Priestley (1998)

The specimen was constructed monolithically for the pile and the deck, so the resulting structural behavior more closely resembled an extended pile connection. The loading protocol called for an increasing reversed-cyclic loading (with no axial pile load), and it was seen that the
specimen experienced minimal deterioration in capacity after the peak load. In later cycles, a plastic hinge formed at a distance of 12 in. (305 mm) from the pile-deck interface, and the hysteretic behavior displayed full loops with almost no pinching effect and a drift in excess of 6% (Sritharan & Priestley, 1998). This may be at least partially attributable to the lack of P-Δ effects since no axial load was applied; nonetheless, it was concluded that the connection details were sufficient to provide a ductile pile-deck connection response.

In research sponsored by the Port of Los Angeles, two full-scale specimens were tested to verify structural limit states based on strains associated with certain levels of performance established by the Port of Los Angeles in the Container Wharf Seismic Code (Krier, 2006; Restrepo et al., 2007). Two new connection details were tested: one seismic and one non-seismic. The main differences between the two embedded dowel connections were the number and size of the dowel bars, the spiral reinforcement ratio, and the use of bond bars. The non-seismic pile was connected to the wharf deck by grouting four #9 (D29) dowel bars 5 ft (1.5 m) into the pile and embedded 17 in. (430 mm) into the deck. For the seismic pile, the connection was achieved by grouting eight #10 (D32) dowel bars 5 ft (1.5 m) into the pile and embedding them 29 in. (735 mm) into the wharf. In each case, the end of the dowels extending into the wharf had a 1.5 in. (38.1 mm) bulb end to enhance anchorage. Additionally, the dowel bars in this latter case were overlapped by eight #9 (D29) bulb-ended headed bond bars and two turns of the pile spiral reinforcing was exposed; this connection was very similar to that proposed by Sritharan and Priestley (1998). For the laboratory experiment, each specimen was tested with the deck above the pile in the field orientation, and lateral forces and displacements were imposed while maintaining a constantly applied axial load in the seismic pile. Figure 7-9 and Figure 7-10 show the details for the non-seismic and seismic connections, respectively.
Figure 7-9. Nonseismic Pile Pile-Deck Connection Detail from Krier (2006)

Figure 7-10. Seismic Pile-Deck Connection Detail from Krier (2006)
For this study, the structural strain limit for the Operational Level Earthquake (OLE) coincided with the peak compressive strain of 0.5% or a tensile strain in the dowel of 1%. The OLE limits were associated with deformation limits that are consistent with damage that is easily repairable. The Contingency Level Earthquake (CLE) strain limits were when the extreme concrete fiber reaches 2% strain in compression or the longitudinal reinforcement attains a tensile strain of 5%. The CLE limits are intended to allow temporary loss of operation, but repair should still be possible.

During the tests, both specimens had moderate spalling in the deck and only minor spalling in the pile. The non-seismic pile reached the OLE tensile strain limit state at 1.1% drift and the CLE tensile limit state around 3.4% drift. The testing of the non-seismic pile, which had no axial load, was concluded when the maximum displacement of the actuator was reached. Throughout the test, it was found from an analysis of the data acquired that about 70% of the applied displacement was from the rotation of the pile-deck interface at the beginning of the test, but by the end of the test, about 90% of the displacement was accommodated by the rotation of the connection. For the seismic pile, the OLE strain limit was reached at a drift ratio of 1.2%, and the CLE limit was estimated to have been reached at a drift of about 8%. The CLE limit could not be measured directly due to malfunctioning strain gauges. The seismic pile testing ended with fracture of the two dowel bars at 9.6% drift and two more dowel bars at 12.9% drift. As with the non-seismic pile, through the progression of the test, the contribution of the rotation of the pile-deck interface to the applied displacement increased from about 75% to over 90% at the end of the test (Krier, 2006). In each case, the specimens showed reserve displacement ductility capacities beyond the limits established by the structural limit states. As another part of the study, the displacement levels that corresponded to the specific strain limits were predicted,
and the experimental results showed that shallow pile embedment connection behavior matched well with the predictions (Restrepo et al., 2007).

### 7.2.3 University of Washington

To further investigate the structural behavior of pile-wharf deck connections, a study by Roeder et al. (2001) was completed at the University of Washington. This study consisted of eight pile-wharf connection specimens, each with different connection details (on a 2/3-scale specimen). The connection types investigated were extended pile, outwardly bent dowel bars, inwardly bent dowel bars, T-headed dowel bars, and bond bars. Table 7-1 outlines the full matrix of specimens experimentally investigated. Specimens 1 and 2 used the extended pile connection to simulate the case where a pile is driven below the desired elevation, while specimens 3 through 8 used a shallow pile embedment of about 2 in. (50 mm), similar to the embedment length in the current study. This provides a clear comparison between the varying connection details. It was found that the extended pile connections were less stiff in the elastic region, and after yielding there was little deterioration in the capacity of those pile-wharf connections since most of the inelastic deformation was developed as flexural yielding. In contrast, the embedded dowel connection was inherently stiffer and stronger in the elastic range, but after yielding there was a substantial reduction in resistance and a pinched hysteretic behavior. Also, it was seen that while the axial load applied to some specimens was only about 10% of their ultimate axial capacity, the addition of this axial load to the pile caused an increase in maximum resistance of the connection coupled with a much more severe and rapid deterioration of the capacity (Roeder et al., 2001).
Table 7-1. Roeder et al. (2001) Test Matrix

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Connection Type</th>
<th>Special Conditions</th>
<th>Goals of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Extended Outward Bent Dowel (Fig. 2.1b)</td>
<td>No axial load.</td>
<td>Comparison with Specimen 3 shows the behavior of extended pile connections as compared to precast pile connections.</td>
</tr>
<tr>
<td>2</td>
<td>Extended Outward Bent Dowel with spiral reinforcement</td>
<td>No axial load. Spirals around dowels in deck.</td>
<td>Comparison with Specimen 1 shows the effect of spiral reinforcement in connection zone.</td>
</tr>
<tr>
<td>3</td>
<td>Outward Bent Dowel (Fig. 2.1a)</td>
<td>No axial load.</td>
<td>Baseline connection for most comparisons.</td>
</tr>
<tr>
<td>4</td>
<td>Outward Bent Dowel (Fig. 2.1a)</td>
<td>With axial load.</td>
<td>Comparison with Specimen 3 shows the effect of axial load on connection performance.</td>
</tr>
<tr>
<td>5</td>
<td>Inward Bent Dowel (Fig. 2.2a)</td>
<td>With axial load.</td>
<td>Comparison with Specimen 4 shows the difference in between outward and inward bent details.</td>
</tr>
<tr>
<td>6</td>
<td>T-Headed Dowel Bar (Fig. 2.1c)</td>
<td>With axial load.</td>
<td>Comparisons with Specimens 4 and 5 show the difference between bent dowel and T-Headed dowel details.</td>
</tr>
<tr>
<td>7</td>
<td>Bond Bar (Fig. 2.1c)</td>
<td>With axial load.</td>
<td>Comparisons with Specimens 4 and 5 show the difference between bent dowel and Bond Bar details.</td>
</tr>
<tr>
<td>8</td>
<td>Outward Bent Dowel but lighter deck reinforcement</td>
<td>With axial load.</td>
<td>Comparisons with Specimens 3 and 4 show the effect of the substantial shear reinforcement in the deck.</td>
</tr>
</tbody>
</table>

Specimen 6 from Roeder et al. (2001) most closely resembles the specimen tested in the current experimental study. It used T-headed dowel bars, and the connection details are displayed in Figure 7-11. The pile had eleven 0.5 in. (12.7 mm) diameter grade 270 prestressing strands around the perimeter, as well as W12 wire spiral at 2 in. (50.8 mm) spacing along the length of the pile as transverse reinforcement. This is for example the same design used in wharves in Oakland, California, and it is intended to reduce reinforcement congestion by
eliminating the need for either bent dowel bars or bond bars. The specimen was subjected to reverse-cyclic lateral loading, with an axial load also applied along the pile. Crushing of the pile occurred at a drift level of 1.25%, and spalling initiated at a drift of 3.0%. By 6.0% drift, spiral and prestressing strands were exposed. The damage was concentrated at the pile-deck interface, and practically no cracking was observed above one diameter up the pile. Although the specimen experienced severe cover spalling at the pile-wharf interface, it was able to achieve its flexural capacity and accommodate large rotational demands.

Figure 7-11. Specimen 6 Connection Details from Roeder et al. (2001)
A continuation of the tests completed by Roeder et al. (2001) was then later conducted by Jellin (2008), also at the University of Washington. Specimen 9 in that study was intended to be a full-scale version of Specimen 6 tested earlier by Roeder et al. (2001), to provide a baseline of the structural behavior for comparison with the responses of subsequent Specimens 10 through 12. Each specimen had a 24-in. (610 mm) diameter, octagonal, prestressed, precast concrete pile. Each pile had eight 2-in. (50 mm) diameter ducts, in which #10 (D32) T-headed dowel bars were grouted 59 in. (1500 mm) into the pile. The piles were embedded 3 in. (75 mm) into the deck, and the T-headed dowel bars were cast 20 in. (508 mm) into the wharf deck. (As will be described later, this configuration was the model for the connection detail then used in the current experimental study at the University of Illinois.) The goal of this study was to develop a new connection detail to reduce the damage to the pile and the deck. To achieve this, a variety of methods were explored in Specimens 10 through 12. Specimen 10 used a 15-in. (380 mm) PVC sleeve, centered on the connection interface, to unbond the dowel bars; Specimen 11 used both the unbonding sleeve and a ¾-in. (19 mm) thick cotton duck bearing pad between the pile end and the deck; and Specimen 12 included the previous adjustments and added the use of a ¾-in. (19 mm) soft foam wrap around the perimeter of the embedded pile length. Figure 7-12 shows illustrations of each of the pile-deck connections explored by Jellin (2008), and Table 7-2 lists a summary of the specimens tested.

The loading protocol was reversed-cyclic displacement of the pile tip, with a constant axial load. The applied axial load was 450 kips (2000 kN), which is approximately 10% of the axial capacity of these full-scale pile specimens. During testing, all of the specimens eventually experienced significant physical damage, including pile and deck cover spalling and the exposure of transverse and longitudinal reinforcement in the pile. The standard embedded dowel
and sleeved embedded dowel connections had significant pile and deck spalling, but the addition of the bearing pad between the embedded end of the pile and the deck both reduced and delayed the onset of spalling in the pile. It was also seen that the use of the soft foam wrap essentially eliminated spalling of the wharf deck. When comparing the measured responses, specimens with bearing pads had a slightly reduced peak moment capacity, reduced stiffness, and the maximum moment was reached at higher drifts levels. Also, specimens with a bearing pad experienced substantially less degradation of their moment resistance (Jellin, 2008).
Specimen 9, which most closely resembled the specimen tested in this study, experienced initial pile spalling at a drift ratio of 1.38% and 1.45%, depending on the direction of loading. The spalling of the pile continued to progress, and around a 4% drift ratio, the spiral reinforcement was exposed up to about 8 in. (203 mm) from the pile-wharf interface. Around 5.5% drift, the spalling of the pile continued to progress, and the deck had spalled to 2 in. (50
mm) deep. By the 7% drift cycle, the dowel bars buckled, and the pile spalling could be seen to penetrate into the confined pile core. The test was concluded after three dowel bars fractured around a 8.4% drift ratio. The base moment versus drift throughout the test is shown in Figure 7-13. Considerable strength degradation can be seen at each cycle (Jellin, 2008).

![Graph](image)

**Figure 7-13. Base Moment versus Drift for Specimen 9 from Jellin (2008)**

The latest installment of tests at the University of Washington on pile-deck connections was recently completed by Stringer (2010). These specimens continued the previous naming convention, and Table 7-2 shows a summary of the experiments conducted.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Axial Load (kips)</th>
<th>Dowel Debonded Length</th>
<th>Interface Bearing Pad</th>
<th>Pile Isolation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen 9</td>
<td>450</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Specimen 10</td>
<td>450</td>
<td>15 in (centered at pile - deck interface)</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Specimen 11</td>
<td>450</td>
<td>15 in (centered at pile - deck interface)</td>
<td>¾” Cotton Duck (24” octagonal)</td>
<td>None</td>
</tr>
<tr>
<td>Specimen 12</td>
<td>450</td>
<td>15 in (centered at pile - deck interface)</td>
<td>¾” Cotton Duck (24” octagonal)</td>
<td>½” soft foam around perimeter of embedded length of pile</td>
</tr>
<tr>
<td>Specimen 13</td>
<td>450</td>
<td>15 in (centered at pile - deck interface)</td>
<td>¾” Cotton Duck (24” octagonal)</td>
<td>¾” soft foam around perimeter of embedded length of pile</td>
</tr>
<tr>
<td>Specimen 14</td>
<td>900</td>
<td>15 in (centered at pile - deck interface)</td>
<td>¾” Cotton Duck (24” octagonal)</td>
<td>¾” soft foam around perimeter of embedded length of pile</td>
</tr>
<tr>
<td>Specimen 15</td>
<td>450</td>
<td>15 in (centered at pile - deck interface)</td>
<td>½” Annular Cotton Duck (24” octagonal, 18” dia center hole)</td>
<td>¾” soft foam around perimeter of embedded length of pile</td>
</tr>
<tr>
<td>Specimen 16</td>
<td>450</td>
<td>15 in (centered at pile - deck interface)</td>
<td>½” Annular Fiber Reinforced Elastomer (24” octagonal, 18” dia center hole)</td>
<td>¾” soft foam around perimeter of embedded length of pile</td>
</tr>
</tbody>
</table>

Specimen 13 and Specimen 14 were intentionally identical to Specimen 12, to provide a baseline for comparison of the results and to explore the variability in performance of the same connection detail. Also, Specimen 14 was tested with twice the axial load of the other specimens. Specimen 15 and Specimen 16 investigated the use of an annular ¾ in. (19 mm) cotton duck bearing pad and an annular ½ in. (13 mm) random oriented fiber reinforced elastomeric bearing pad, respectively. The annular pads were cut to a 24 in. (610 mm) octagon.
to fit the cross-section of the pile, and an 18 in. (460 mm) diameter hole was cut in the center to accommodate the dowel bars. Results of these tests showed that in Specimen 14, the higher axial load caused an earlier onset of pile spalling and rapid strength degradation. The specimens with annular pads had essentially the same response, and it was seen that the annular pad delayed pile spalling, but after spalling occurred, a rapid strength loss was observed. In each specimen, it was again seen that the use of the soft foam wrap around the end of the pile eliminated deck spalling. Comparing the performance with and without bearing pads, it was found that standard embedded dowels achieved a given pile tip displacement (at the load application point, nearly 10 ft from the connection interface) through about 30% pile end rotation and 70% pile elastic and inelastic flexural action, while specimens with bearing pads achieved the same displacement through 80% to 90% end rotation of the pile (Stringer, 2010). This effect results in delayed damage of the pile and an increased resistance to strength degradation. Each of the above investigated adjustments to the standard pile-wharf connection design improve the damage resistance of the pile-wharf connection, but the increase in flexibility of the seismically designed piles may have some design implications on the system-level. In a full port structure, if the shorter seismic piles undergo larger displacements for a given load, then the longer non-seismic piles may be required to participate in the lateral resistance of the port structure in a more meaningful way.

7.2.4 Summary of Past Research

As outlined above, several researchers have investigated pile-deck connections and attempted various approaches to achieve an enhanced structural performance. It has been found that pile-deck connections exhibit essentially plastic behavior under no axial load, with a plastic hinge forming near the pile-deck interface. As an increasing axial load is applied, significant strength degradation is seen due to spalling of the thick concrete cover. This is exacerbated by
the fact that piles have more stringent (thicker) cover requirements due to the corrosive marine environment. Embedded dowel connections are preferred both due to their structural performance and also to the ease of construction in the field. While the current study will focus on a traditional embedded dowel connection with T-headed bars, previous researchers have explored the impact of including bearing pads, unbonded dowel bars at the pile-deck interface, and a foam wrap around the embedded pile length, all with favorable results. The remainder of this chapter focuses on the construction and testing of a pile with more realistic boundary conditions at the University of Illinois at Urbana-Champaign, as well as a discussion of a proposal to repair that embedded dowel pile-deck connection once damaged.

7.3 Pile-Wharf Connection Experimental Testing Program

A “grand challenge” research effort addressing seismic risk management for port systems, supported through the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES), is currently underway. The project that in part funded this research is titled “Seismic Risk Management for Port Systems,” with its main objectives focused on predicting the seismic response and damage states of key port components, estimating the effect of damage to these components on overall port capacity and revenues, and developing strategies for mitigation of possible losses through engineering design and retrofit options. More information related to all the specific tasks and the institutions involved can be found on the project website (http://www.neesgc.gatech.edu).

As part of that NEES project, the seismic performance of precast/prestressed concrete pile-wharf connections is being investigated at the University of Illinois using the Multi-Axial Full-Scale Sub-Structured Testing and Simulations Facility (MUST-SIM). While the previously discussed experimental studies by project colleagues at the University of Washington (some also
partly supported by this NEES project) have focused on developing alternative connection concepts and construction practices to better resist damage during earthquakes, the objectives of the research at the University of Illinois are instead to understand the structural performance of typical designs of vertical pile moment-resisting connections to a wharf deck (of the type that commonly have been and are being constructed), and to test a repair technique for restoring structural integrity after an earthquake. Previous tests utilized a simplified testing set-up; however, this investigation uses a unique “Loading and Boundary Condition Box” (LBCB), which utilizes six actuators in an arrangement enabling complete six degree-of-freedom control at the (non-connection) end of the pile. The LBCB loading unit allows for precise control of both forces and displacements in each degree of freedom, thereby permitting the application of equivalent loads from a computational model of a full-scale port structure (further details about the LBCB will be discussed in Section 7.3.5). Overall, the objectives of this portion of the research are to utilize the MUST-SIM small-scale facility to develop a mixed-mode loading protocol, to test a large-scale specimen to significant damage, and to repair and retest the large-scale specimen. While the last objective is not necessarily within the scope of this document, further information about the progress and results of the repaired specimen can be found at the MUST-SIM website (http://nees.uiuc.edu).

7.3.1 Experimental Plan

The experimental plan is composed of three phases: small-scale testing of a 1/5-scale rubber specimen (to validate key aspects of test control), large-scale testing of a full-scale pile-wharf connection, and large-scale re-testing of the repaired full-scale pile-wharf connection. One objective of the testing is to simulate the realistic boundary conditions that a pile may experience during a seismic event. Previous tests were able to apply an axial load while
imposing lateral displacements to the pile tip; however, a full-length pile will be subjected to moments and rotations at the pile “tip” location (which would actually be continuous), and throughout the rest of the length of the pile as well. To this end, an LBCB was used to impose a constant axial force of 90 kips (400 kN) throughout the test, while imposing lateral displacements and rotations to the end of the pile length that was tested. The axial load was selected to simulate an expected gravity loading on the wharf deck, while the rotations and displacements imposed were the result of a comprehensive modeling effort undertaken by project colleagues at Georgia Tech University. For that latter effort, nonlinear wharf structural analysis models were developed based on the Port of Oakland Berth 55/56. Profile and plan views of the modeled wharf structure are shown in Figure 7-14 and Figure 7-15, respectively.

Figure 7-14. Port of Oakland Berth 55/56 Model Wharf Profile View
From the wharf structure drawings and specifications, a 2-dimensional (transverse) translational model was developed by others using the OpenSees nonlinear finite element analysis program. The tributary mass of the deck was assigned to each node, and P-y curves were used to model soil springs. Fiber sections were developed to define the octagonal reinforced concrete pile sections, with the “nonlinear beam-column element” in OpenSees used to model the piles. The deck was modeled as rigid by connecting deck nodes through rigid beam elements. Additional details of the modeling assumptions and P-y characteristics are available in Shafieezadeh (2011). An illustration of the 2D translational model is shown in Figure 7-16, as well as a box highlighting a representative portion of the full port model to be tested in the NEES facility experimentally.
Figure 7-16. Georgia Tech OpenSees 2D Translational Model

Time-history analyses were run using the suite of ground motions provided by the SAC project. SAC was a joint venture funded by the Federal Emergency Management Agency (FEMA) that included the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and the Consortium of Universities for Research in Earthquake Engineering (CUREE). The SAC ground motions reported in Somerville et al. (1998) were selected and scaled to obtain approximate magnitudes and distances appropriate for earthquakes in the Boston, Seattle, and Los Angeles areas, for three return periods: 2%, 10%, and 50% probability of exceedance in 50 years. For test planning purposes related to the experiment
reported herein, only wharf structure analytical model results for the suite of 60 records scaled to the Los Angeles area have been considered. From all those records, one model response was selected from each of the three different hazard levels. The Imperial Valley record (LA44) was selected for the lowest hazard level; Northridge (LA18) was selected for the 10% in 50 year return period; and Kobe (LA2) was selected for the 2% in 50 year hazard, with the acceleration time histories for each of the respective ground motions shown in Figure 7-17. These selections were made rather qualitatively by reviewing the model results. Using a Response-2000 model, it was estimated that the model displacements from the Imperial Valley record would cause cracking, the Northridge record would cause yielding of the dowel bars, and the Kobe earthquake would bring the specimen to failure.

![Graphs of selected SAC Ground Motions](image)

**Figure 7-17. Selected SAC Ground Motions**

An analysis of the wharf model results from the different earthquake ground motions identified a linear relationship between the pile lateral displacement and rotations at a location of 84 in. (2135 mm) from the pile-wharf interface in the model. This location (and that simple relationship) is relevant to the experimental work because it was identified as the control point about which all rotations, displacements, and forces should be applied and calculated during testing. The linear relationship identified from the analytical models was 72.6 in. of lateral
displacement per radian of rotation. In addition to the imposed displacement and rotation on the top of the pile, a constant axial load of 90 kips (400 kN) was also applied. These imposed boundary conditions with the appropriate sign convention are displayed in Figure 7-18, as well as the typical bending moment distribution along the height of the pile. An applied shear force and moment correspond to the imposed displacement and rotation at the end of the pile. Since the magnitude of the imposed rotation was less than the free rotation of a cantilever, the result was an applied moment with the opposite sign of the moment at the pile-wharf connection, as shown Figure 7-18.
The bending moment distribution shown in Figure 7-18 was typical, but the point of contraflexure in the pile changed location throughout the testing depending on the displacement-rotation ratio used. It should be noted, however, that the relationship of the directions of deformation imposed never changed during testing. Figure 7-19 shows plots of the x-displacement record from the analytical model that was then in turn imposed at the control point.
during testing corresponding to each earthquake response. The records shown were discretized into steps to be implemented during the experimental portion of the test. A minimum of three steps was used to reach very small displacement peaks, and up to twenty steps was used to reach large displacement magnitudes.

![Figure 7-19. X-Displacement Record Imposed during Earthquake Loading](image)

After the conclusion of the earthquake record loading portion of testing, a reverse-cyclic loading protocol was undertaken. For this cyclic portion of testing, the same constant axial load was maintained, as well as the same displacement-rotation ratio. Again, the imposed ratio had to increase slightly as testing progressed to ensure that the proper relationship was maintained at the control point, but those specific amplifications will be outlined in later sections. Cycles were conducted at 2% (1.68 in. / 42.7 mm), 3.5% (2.94 in. / 74.7 mm), 5% (4.2 in. / 106.7 mm), 6.5% (5.46 in. / 138.7 mm), and 8% (6.72 in. / 170.7 mm) drift, where drift was defined as the lateral displacement divided by the length of the pile from the connection interface to the control point.
At the conclusion of the 8% drift cycle, it was determined that the specimen had been adequately damaged, and that the repair phase of the testing program could commence. Figure 7-20 displays the x-displacement history during the cyclic loading regime.

![Figure 7-20. X-Displacement Record Imposed during Cyclic Loading](image)

### 7.3.2 Test Specimen Design

The test specimen consisted of a 24-in. (610 mm), octagonal, precast, prestressed pile connected to a cast-in-place reinforced concrete wharf deck slab. The specimen was designed to closely resemble pile-wharf connections found in the Port of Oakland Berth 55/56 and the previous tests conducted at the University of Washington. The main difference between the test at the University of Illinois and the tests previously conducted at the University of Washington was the difference in the loading and boundary conditions. The University of Washington tests imposed a much larger axial load with reverse cyclic lateral displacements at the free end of the pile. At UIUC, the axial load was 1/5th the magnitude and both displacements and rotations were
imposed at the free end of the pile. Additionally, the imposed displacements and rotations were
the result of a system level modeling effort with imposed ground motions. The specimen was
cast upright to simulate actual field construction; however, testing was done with the pile
specimen inverted, as shown in Figure 7-21. The concrete pile was embedded 2 in. (50 mm) into
the cast-in-place deck, and the connection was achieved by using eight #10 (D32) T-headed
dowel bars grouted into the pile. As shown in Figure 7-21, the control point for testing was
specified at the center of the pile cross-section at a distance of 7 ft (2.1 m) from the pile-deck
interface. Through the use of the LBCB and externally-mounted string potentiometers, all
control “displacements” and “forces” imposed during testing and described hereafter are in
reference to the control point, not the end of the pile. The term “displacements” is used to
reference both the lateral displacement and the rotation at the end of the pile, while the term
“forces” refers to the axial force, lateral force, and moment on the pile.
Figure 7-21. General Pile-Wharf Connection Test Specimen Dimensions

The pile lengths (total and clear distance above soil) normally vary within a port structure, depending on the soil embankment and dredge limit, but since the deck generally acts as an almost-rigid single degree-of-freedom system, the rotational and shear demands are much greater at the shorter landside piles. As such, the pile / connection tested can be considered as representative of a landside pile, rather than of waterside piles that may have unsupported lengths greater than 50 ft (15.2 m). The shorter piles are typically detailed as seismic piles, while the much longer piles are considered non-seismic. The seismic piles are designed to provide
strength, ductility, and gravity loading capacity, while the non-seismic piles are designed primarily for gravity loading and are only detailed for limited ductility (Krier, 2006).

Details of the prestressed, precast pile used in this study are given in Figure 7-22. The pile was donated by Concrete Technology Corporation of Tacoma, WA. Specimen dimensions for the pile were 99 in. (2.5 m) in length and 24-in. (610 mm) in diameter (octagonal), and it contained eight 2-in. (50 mm) diameter corrugated metal ducts. The pile was reinforced with twenty-two 0.5 in. (12.5 mm) diameter, 270 ksi (1860 MPa) low-relaxation strands, with each strand prestressed to 31 kips (138 kN). Spiral reinforcement was W11 (0.374 in. / 9.5 mm diameter) smooth wire. Spiral pitch varied from 1 in. (25 mm) at the end of the pile to 3 in. (75 mm) along the middle of the pile, as shown in Figure 7-22.
The precast pile was embedded 2 in. (50 mm) into the cast-in-place wharf deck to model typical current design and construction practice. The prestressed strands were burnt on one end of the pile, and that end was used in the eventual connection to the LBCB. A moment connection was achieved by grouting eight #10 (D32) ASTM A706 T-headed dowel bars (donated by the Headed Reinforcement Corporation (HRC) of Fountain Valley, CA) into the corrugated ducts in the pile. The T-head was formed by welding a 3.5 in. (88 mm) square plate, with a 1 in. (25 mm) thickness, to the end of the reinforcing bar. The T-headed bars were 84 in. (2130 mm) in length, and their grouted length was 62 in. (1575 mm) to ensure full development.
The 22 in. (560 mm) remaining length was then cast into the wharf deck to complete the connection. The grout used was donated by Dayton Superior – it was a high-performance, non-shrink grout. A schematic of the grouting details are shown in Figure 7-23.

**Figure 7-23. T-Headed Bar Grouting Details**

The wharf deck was constructed of reinforced concrete, and its dimensions were 96 in. (2440 mm) in length, 66 in. (1675 mm) in width, and 30 in. (760 mm) in height. The deck depth and reinforcement ratios were modeled after as-built Port of Oakland drawings, while the other deck (plan) dimensions were as large as practical, given the constraints of the laboratory and the testing set-up. The deck longitudinal reinforcement consisted of hoops with #7 (D22) bars and #8 (D25) reinforcing bars spaced at 6 in. (150 mm) on center, and the deck transverse reinforcement consisted of hoops with #8 (D25) and #9 (D29) bars at 7.5 in. (190 mm) on center. In the wharf deck, the longitudinal reinforcement was aligned along the longer 8 ft (2.44 m) deck dimension, and the transverse reinforcement was oriented in the plane of the 5.5 ft (1.68 m) wharf deck dimension. In each case, the larger reinforcing bar was placed on the deck side away
from the pile. For shear reinforcement, #4 (D13) double-legged stirrups were used at 7.5 in. (190 mm) on center through the center 3 ft (0.91 m) of the wharf deck in the longitudinal direction. In regions further away from the pile, two double-legged stirrups were used offset by 12 in. (305 mm) in the transverse direction. Due to reinforcement congestion in the presence of the T-heads, only one double-legged stirrup was used in deck regions under the pile. Figure 7-24 displays the details of the wharf deck reinforcement. It should be noted that, depending on the view, certain of the reinforcement has been omitted for clarity.

Figure 7-24. Wharf Deck Reinforcement Details

To post-tension the test specimen to the strong floor, 3-in. (75 mm) diameter holes were cast into the deck section. The post-tensioning locations were at 12 in. (305 mm) from the north
and south edges of the specimen and 15 in. (380 mm) from the east and west edges, such that they were spaced at 36 in. (915 mm) in the transverse direction and 72 in. (610 mm) in the longitudinal direction. Additionally, to accommodate potential deflection of the wharf deck or dowel bar punch through, the specimen was placed on four 24 in. (610 mm) square steel plates of 1.25 in. (32 mm) thickness. The plates had a 3 in. (75 mm) diameter hole drilled in their centers to allow for the specimen to be post-tensioned to the strong floor. A line drawing of the tie-down locations and the steel plates can be seen in Figure 7-25.

![Figure 7-25. Strong Floor Connection Details](image)

The following section will show some images of the test specimen resulting from implementing the aforementioned design in actual construction, toward then executing the experimental plan.
7.3.3 Test Specimen Construction and Installation

The specimen was constructed with the cast-in-place deck above the precast pile. This was done with the intention of modeling actual practice in the field and to eliminate any misleading segregation of concrete aggregate during the deck pour (which might have occurred had the specimen been inverted at the time of wharf deck concrete placement). The first step in specimen construction was to design and build the wooden formwork to support the cast-in-place reinforced concrete deck. All formwork was constructed in accordance with the 2005 National Design Specification (NDS) wood design manual (American Wood Council, 2006).

Four steel W18 columns were attached to the strong floor with steel W18 sections spanning between two of the columns. For the wooden formwork, 2x4 joists were placed at 12 in. (305 mm) on center, spanning across the steel beams. A series of ¾-in. (19 mm) thick plywood panels were then placed on top of the joists for the base platform. The side forms were constructed from ½-in. (13 mm) thick plywood panels reinforced with 2x4 studs at 12 in. (305 mm) on center. To resist the hydrostatic pressure from the wet concrete, the top of the form was reinforced with 3/8-in. (9.5 mm) diameter threaded rod form ties. An octagonal cut was made in the base platform of the formwork to allow for placement of the pile, and four PVC inserts were placed to allow for post-tensioning to the strong floor. Since this research project also later contains a repair component, additional PVC inserts were placed in the deck formwork at the location of the longitudinal bars for the designed repair scheme. Since the specimen is inverted during testing, the construction phase was the last opportunity to access the (“top”) side of the wharf deck opposite of the pile to install these four bars, and grouting those retrofit bars while still in this configuration would also more closely simulate field conditions. Details of the formwork and construction plans are shown in Figure 7-26.
Figure 7-26. Formwork Details
Upon completion of the formwork, the next step was to place the precast pile vertically in the formwork. Images of the lifting process are shown in Figure 7-27 and Figure 7-28.

Figure 7-27. Pile Lifting Procedure - Step One

Figure 7-28. Pile Lifting Procedure - Step Two (left) and Step Three (right)
The lifting process was completed by first placing Dywidag bars through the pile in two of the duct locations. Steel lifting loops were then connected to a steel plate spanning between the two Dywidag bars on one end, as shown in Figure 7-27, while the other end of the Dywidag bars were connected with a steel plate and large nuts to withstand the weight of the pile during lifting. Then, the overhead crane was used to lift the pile vertically from one end, and the pile was carefully lowered into position in the formwork. PVC inserts, cut to the appropriate length and with rubber stoppers at the end, were placed in each duct to ensure that the grout for the T-headed dowel bars would remain in the duct. The lifting and placement of the pile is shown in Figure 7-28.

With the completion of the formwork and placement of the pile, the next step was to prepare the T-headed reinforcement by installing strain gages at the desired locations. YFLA-10-5LT strain gages and CN-Y epoxy adhesive (both from Tokyo Sokki Kenkyujo Co., Ltd.) were used in this process. The YFLA gauge type is designed to measure strain up to 15-20% with accurate readings, and the strain gauges were 0.39 in. (10 mm) in length with a resistance of 120Ω. Further details of the strain gage locations are provided in Section 7.3.6. The T-headed bars were then grouted into the pile ducts. In typical port construction, the longitudinal steel would be grouted into the pile in their final vertical position, so this procedure emulated common practice. The grout was mixed according to the “fluid” proportions provided by Dayton Superior. Wooden scaffolding was used to hold the bars at the appropriate elevation while the grout dried. This process is shown in Figure 7-29. After the dowels were grouted into place, the small gap between the pile and the bottom of the form was filled with latex caulk to ensure a watertight seal around the pile before subsequent casting of the wharf deck slab.
After installation of the T-headed reinforcement, the deck reinforcement could then be placed inside the formwork. The deck reinforcement cage was first pre-fabricated outside of the formwork, and then placed into the formwork with the overhead cranes in the laboratory. Details of the reinforcement layout have been provided previously in Section 7.3.2. The 3-in. (76 mm) diameter PVC inserts were placed, with confining spiral reinforcing, to allow for eventual post-tensioning to the strong floor prior to testing. Additionally, the 3-in. (76 mm) diameter PVC inserts for the repair scheme were placed. Finally, lifting inserts were positioned on the sides of the deck to allow for the specimen to later be inverted from the casting position to the testing position. Figure 7-30 displays the deck reinforcement in the formwork, with the image on the
Concrete was cast into the deck forms using a hopper and the overhead crane. Maximum size 3/8 in. (9.5 mm) aggregate was specified to minimize any problems with the concrete flowing around the reinforcing steel cage and/or with aggregate segregation. The design strength was 5 ksi (34.5 MPa) to simulate typical wharf concrete strength, and the target slump was 6 in. (152 mm) to further promote the fluidity of the concrete around the reinforcing cage. The actual properties of the deck concrete are reported in Section 7.3.4.1. Additionally, the concrete was vibrated to further minimize voids. Once the forms were filled, the concrete was troweled to a smooth and level finish. Wet burlap and plastic were used to cover the wharf deck after placing to prevent evaporation from occurring at the surface of the concrete. Further details of the wharf deck concrete mix are provided in Section 7.3.4.1. Figure 7-31 shows the deck being cast, with the various previously described components of the deck labeled in the image for clarification.
The next construction procedure for this specimen was unique from other previously tested pile-wharf connections because of the later repair component of this current research. In the field, the proposed repair calls for drilling holes through the wharf deck and grouting in 4-#9 (D29) Dywidag longitudinal bars. Those longitudinal bars are to be grouted a minimum of 15 in. (380 mm) into the wharf deck slab (after a 9 in. debonded length), and anchored near the top of the wharf with a 2 in. (50 mm) thick washer and nut. The bars are to be placed 22 in. (560 mm)
from the center of the pile in each orthogonal direction. A line drawing of the repair scheme from Liftech Consultants Inc. of Oakland, CA is depicted in Figure 7-32.

![Diagram of Pile-Wharf Connection Repair Scheme](image)

**Figure 7-32. Pile-Wharf Connection Repair Scheme**

The repair bars were grouted into place, with 2 in. (50 mm) mortar cubes also made to later test the strength of the grout (closer to the test date of the repaired specimen). In Figure 7-33, the four pockets made by the PVC inserts can be seen, as well as an example #9 (D29) Dywidag bar placed inside one of the holes in the deck. Also, the four pockets after successfully grouting 15 in. (380 mm) into the wharf deck are depicted there. PVC inserts with rubber stoppers were placed from the opposite side of the deck around the repair bars to prevent the grout from entering the intentionally unbounded portion of the repair bar near the pile-deck.
interface. And finally, strain gauges were applied at select locations on the repair bars for future strain measurements during testing of the repaired specimen.

Figure 7-33. Longitudinal Repair Bar Placement

After sufficient time had elapsed to ensure that the deck concrete had reached an adequate compressive strength, the formwork was stripped, the lifting loops were attached to the lifting inserts, and the specimen was flipped into its final inverted position for testing. To accomplish this, the specimen was first lifted vertically from the formwork. With a second overhead crane, a harness was looped around the end of the pile, such that one crane now held the specimen by the deck, while the other was attached to the pile. Gradually, one crane lowered the deck, while the other crane lifted the end of the pile; when the specimen was horizontal, the
harnesses were repositioned to complete flipping the specimen. Figure 7-34 shows several images during the specimen inversion process.

![Figure 7-34. Specimen Inverting Procedure](image)

With the specimen in the proper orientation for testing, a remaining specimen preparation task was to prepare for connection to the LBCB. To accomplish this, threaded rods were grouted into the ducts at the free end of the pile. These threaded rods were 1.5-in. (38 mm) diameter B7 steel, and they were grouted into the remaining empty 35 in. (890 mm) of the duct. A sufficient amount of the threaded rod was left extending from the end of the pile to accommodate a 4-in. (102 mm) thick steel adapter plate. The LBCB connection scheme was intentionally designed with overstrength to ensure that any damage in the pile would occur near the pile-wharf connection, rather than at the connection to the LBCB. The adapter plate consisted of a 4-in. (102 mm) thick steel plate, with eight holes drilled in it to allow the threaded rods to pass through the plate; the holes were counterbored so that the plate could flushly attach to the LBCB platen after the threaded rods were post-tensioned and the nuts were attached. The nuts were
post-tensioned with a torque wrench to a sufficient tension to prevent slip during testing of the specimen. After post-tensioning, a grinder was used to remove any excess length of the threaded rods that may have interfered with connection to the LBCB. Additional holes were drilled in the plate around its perimeter for making a bolted connection to the LBCB platen.

To confine the end of the pile, a 9 in. (230 mm) long collar of \( \frac{1}{2} \)-in. (13 mm) thick steel plates was welded around the end of the pile and to the bottom of the adapter plate. This connection was further strengthened by triangular stiffeners welded around the collar and to the adapter plate, and grout was placed through pour holes in the top of the adapter plate to fill the voids between the collar and the pile end. Weep holes were located at the top of the collar to ensure that the grout was indeed fluid enough to flow around the pile and fill all of the voids. Formwork was built under the collar to hold the grout in place while it cured. Figure 7-35 shows the threaded rods grouted into the top of the specimen (left), as well as attached to the adapter plate (right).

Figure 7-35. LBCB Connection Details
After connecting the adapter plate to the pile, the entire specimen was ready to be moved into its final testing position under the LBCB. Since the LBCB was already mounted on the strong wall, a crane could not be used to move the specimen all the way to its final location under the LBCB. Instead, the overhead laboratory crane moved the specimen by its lifting loops in the deck to a location adjacent to the final testing position, and then horizontal hydraulic jacks were used to slide the specimen along the strong floor to a location under the LBCB, as shown on the left in Figure 7-36. To permit inserting 1.25 in. (32 mm) thick steel plates under the specimen, the adapter plate was first attached to the LBCB and used to lift the specimen off of the strong floor. The steel plates were then slid under the specimen, with Hydrocal applied to both sides of the plates to ensure a uniform bearing surface during testing. These plates were placed under the specimen to have the wharf deck span a short distance between its supports and allow for the possibility of the deck to deflect or for the T-headed dowel bars to punch through, as would be allowed in typical port structures. This whole process is shown in the center image of Figure 7-36. The specimen was then post-tensioned to the strong floor by 1.5 in. (38.1 mm) diameter Dywidag bars with approximately 125 kips (550 kN) of force per bar, which was deemed to be a sufficient force to prevent any slipping between the specimen and the strong floor during testing of specimen, including under lateral load. The specimen is shown in its final position for testing on the right of Figure 7-36.
Figure 7-36. Moving Specimen into Position

Final procedures to ensure that the local coordinate system of the LBCB coincided with the desired coordinate system of the specimen still needed to occur, so the LBCB was at this point temporarily disconnected from the adapter plate. Using the non-contact dynamic measurement machine, Krypton (to be discussed in more detail in Section 7.3.6.2), the LBCB underwent a series of displacements and rotations in all six degrees of freedom. Movement of the LBCB platen was monitored relative to the coordinate system of the test specimen, and an appropriate transformation matrix was implemented into the LBCB software to align the local coordinate system of the LBCB with the specimen. After ensuring that the coordinate systems were adequately aligned, a final connection of the LBCB to the adapter plate was made.

7.3.4 Materials Characterization

Throughout the testing program, material properties were obtained to record the as-built material characteristics during testing. This section will outline the properties of each material type.
7.3.4.1 Concrete Properties

The wharf deck of the specimen was cast using a design 5 ksi (34.5 MPa) concrete mix design, with 3/8-in. (2.6 mm) limestone chips specified for the coarse aggregate (to minimize consolidation problems (e.g., honeycombing) due to the dense reinforcing steel cage). The desired concrete strength was selected to be representative of field construction practices. The water-cementitious ratio of the mix was 0.416; Table 7-3 shows the deck concrete mixture proportions, normalized by weight.

Table 7-3. Deck Concrete Mix Proportions Normalized by Weight (Total Weight = 1.00)

<table>
<thead>
<tr>
<th>Component</th>
<th>Deck Mix Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>0.149</td>
</tr>
<tr>
<td>Fine Aggregate (Sand)</td>
<td>0.265</td>
</tr>
<tr>
<td>Coarse Aggregate (3/8&quot; Chips)</td>
<td>0.469</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>0.039</td>
</tr>
<tr>
<td>Retarder</td>
<td>3 oz/cwt*</td>
</tr>
<tr>
<td>Water</td>
<td>0.078</td>
</tr>
</tbody>
</table>

* cwt = 100 lbs Cementitious (Cement + Fly Ash)

A local ready-mix concrete supplier helped to batch, deliver, and test the concrete mix. The slump of the mix was tested prior to placing the concrete – these test results found the deck concrete to have a slump of 6 in. (150 mm). After concrete placement, the wharf deck was covered with wet burlap and a plastic tarp to reduce surface evaporation from the concrete. Additionally, to collect information about the compressive strength of the concrete, 4 in. x 8 in. (102 mm x 203 mm) cylinders were cast along with the deck concrete pour. All compressive strength tests were conducted according to the ASTM C39 (ASTM C39-05, 2006) specifications; Table 7-4 displays the tabulated 7-day, 14-day, 28-day, and test day compressive strengths. For each of the sets of cylinders, three cylinders were tested to obtain the average strength, except for
the test day cylinders, where five cylinders were tested. Also, due to the disappearance of cylinders from the moist curing room, only 28-day moist cured strengths were obtained; the remainder of the cylinders were dry cured in the laboratory.

**Table 7-4. Average Cylinder Compressive Strengths**

<table>
<thead>
<tr>
<th></th>
<th>Average Cylinder Compressive Strength, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7-day Dry Cured</td>
<td>5.5 (38.0)</td>
</tr>
<tr>
<td>14-day Dry Cured</td>
<td>6.2 (43.0)</td>
</tr>
<tr>
<td>28-day Dry Cured</td>
<td>6.7 (46.0)</td>
</tr>
<tr>
<td>28-day Moist Cured</td>
<td>6.9 (47.4)</td>
</tr>
<tr>
<td>Test Day Dry Cured</td>
<td>7.4 (51.3)</td>
</tr>
</tbody>
</table>

The strength gain with respect to time is shown in Figure 7-37. It can be seen that the majority of the compressive strength for testing was achieved by 28 days after pouring the wharf deck slab concrete.
While simple cylinder compressive tests were conducted to document deck concrete strength development, more detailed cylinder tests were conducted around the time of pile-wharf connection testing. For these tests, a 600 kip (2.7 MN) MTS uniaxial servo-controlled hydraulic frame was used, with the axial strain obtained using a 6 in. (150 mm) gauge length extensometer attached to the concrete cylinders. The stress-strain results of those 5 cylinder tests are shown in Figure 7-38.

**Figure 7-37. Wharf Deck Concrete Strength Development**
Figure 7-38. Deck Deck Concrete Cylinder Compression Results at Specimen Test Date

A summary of relevant deck concrete properties is displayed in Table 7-5 for at the day of testing the pile-wharf connection (not at 28 days); $\varepsilon_c'$ is the strain corresponding to the maximum stress, and $E_c$ was obtained using the equation recommended by ACI 318 Section 8.5.1 (2008). Interestingly, using the average slope of the stress-strain curve from 1 ksi (6.9 MPa) to 4 ksi (27.6 MPa), the stiffness of the cylinders was found to be 4956 ksi (34180 MPa), which is quite close to the ACI 318 (2008) estimate.
Table 7-5. Deck Concrete Property Summary at Specimen Test Date

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_c$, ksi (MPa)</td>
<td>7.4 (51.3)</td>
</tr>
<tr>
<td>$e'_c$, in/in*10-3</td>
<td>2.13</td>
</tr>
<tr>
<td>$E_c$, ** ksi (MPa)</td>
<td>4918 (33920)</td>
</tr>
</tbody>
</table>

* Day of Testing Strength
** $57,000*$sqrt($f'_c$)

The piles were donated by Concrete Technology Corporation of Tacoma, WA, with concrete compressive strength data provided up through the 28-day value; however, cylinders were not delivered with the piles, so test-day strength is not available. The pile concrete was a 5/8 in. (15.9 mm) maximum coarse aggregate size, 8 ksi (55.2 MPa) mix with a 3 in. – 9 in. (76 mm – 230 mm) slump. Table 7-6 shows the compressive strengths from three pours used during the casting of multiple piles. Since no further information was provided, it is not possible to determine the exact concrete compressive strength that corresponds to the specific pile tested in this research study; however, a fairly narrow range of compressive strengths can be estimated from this data and then extrapolated to approximate test-day values.

Table 7-6. Pile Concrete Compressive Strength Data

<table>
<thead>
<tr>
<th>Time (Days)</th>
<th>Batch 1, ksi (Mpa)</th>
<th>Batch 2, ksi (Mpa)</th>
<th>Batch 3, ksi (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>7.2 (49.6)</td>
<td>4.8 (33.1)</td>
<td>4.0 (27.8)</td>
</tr>
<tr>
<td>14</td>
<td>9.0 (61.9)</td>
<td>10.2 (70.2)</td>
<td>8.0 (54.9)</td>
</tr>
<tr>
<td>28</td>
<td>9.7 (66.8)</td>
<td>10.6 (73.4)</td>
<td>8.4 (58.0)</td>
</tr>
</tbody>
</table>

*Provided by Concrete Technology Corporation
7.3.4.2 **Grout Properties**

Dayton Superior donated the grout used to develop (anchor) the T-headed bars in the pile ducts. Also, the same grout was used to develop the threaded rods and to fill the voids between the adapter plate collar and the pile (which were required to attach the pile to the LBCB). The specific grout used was the “Sure-Grip High Performance Grout” from Dayton Superior – its primary properties are non-shrink and high early strength performance. For mixing, the guidelines provided with the grout for a “fluid” mix were followed. Two T-heads (or threaded rods) were set with each grout pour, and along with each grout pour, three 2 in. (50 mm) cubes were cast and tested in the Forney Testing machine in the NSEL concrete laboratory. The cubes were tested at around the day of testing of the pile-wharf connection, and average tabular compressive strength data can be found in Table 7-7. The strengths of all of the cube tests were within 15% of the average value, so the grout strengths were quite consistent.

<table>
<thead>
<tr>
<th>Table 7-7. Grout Strength Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bar Type</strong></td>
</tr>
<tr>
<td>T-Headed Bars</td>
</tr>
<tr>
<td>Threaded Rods</td>
</tr>
</tbody>
</table>

7.3.4.3 **Steel Properties**

The #10 (D32) T-headed dowel bars were donated by the Headed Reinforcement Company of Fountain Valley, CA and conformed to ASTM A706 requirements. Material tests were performed on these and representative samples of all other steel reinforcement sizes used in the experimental test specimen. Tests on #10 (D32), #9 (D29), and #8 (D25) reinforcing bars were conducted in the 600 kip (2.7 MN) MTS uniaxial servo-controlled hydraulic frame, while
tests on #7 (D22) and #4 (D13) bars were performed using a 100 kip (445 kN) MTS uniaxial servo-controlled hydraulic frame. Strain measurements were obtained using a 4-in. (102 mm) gauge length clip-on extensometer. The relevant steel reinforcement material properties are provided in Table 7-8; due to the lack of a clear yield plateau in the #4 (D13) reinforcement, the 0.2% offset method was used to determine their yield properties. In all, four #10 (D32) T-headed dowel bars, two #9 (D29), two #8 (D25), two #7 (D22), and one #4 (D13) were tested. Due to difficulty in gripping the #4 (D13) bar, only one specimen could be successfully tested. Three further attempts were made to test another #4 (D13); however, the reinforcing bar continued to slip out of the grips of the testing machine.

Table 7-8. Steel Reinforcement Material Properties

<table>
<thead>
<tr>
<th>Rebar</th>
<th>( f_y )</th>
<th>( \varepsilon_{y} )</th>
<th>( f_{sh} )</th>
<th>( \varepsilon_{sh} )</th>
<th>( f_{max} )</th>
<th>( \varepsilon_{max} )</th>
<th>( f_u )</th>
<th>( \varepsilon_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td>#10 T-Headed Bars</td>
<td>68.1 (469.7)</td>
<td>0.0033</td>
<td>69.1 (476.6)</td>
<td>0.0184</td>
<td>95.6 (659.3)</td>
<td>0.156</td>
<td>84.0 (579.3)</td>
<td>0.206</td>
</tr>
<tr>
<td>#9 Bars</td>
<td>89.3 (616.1)</td>
<td>0.0060</td>
<td>89.8 (619.3)</td>
<td>0.0146</td>
<td>113.8 (784.8)</td>
<td>0.124</td>
<td>94.9 (654.5)</td>
<td>0.168</td>
</tr>
<tr>
<td>#8 Bars</td>
<td>83.1 (573.1)</td>
<td>0.0052</td>
<td>83.6 (576.6)</td>
<td>0.0212</td>
<td>104.6 (721.4)</td>
<td>0.127</td>
<td>81.6 (562.8)</td>
<td>0.173</td>
</tr>
<tr>
<td>#7 Bars</td>
<td>75.0 (517.2)</td>
<td>0.0041</td>
<td>75.7 (522.3)</td>
<td>0.0158</td>
<td>98.2 (677.2)</td>
<td>0.101</td>
<td>85.2 (587.0)</td>
<td>0.137</td>
</tr>
<tr>
<td>#4 Bars*</td>
<td>70.2 (484.1)</td>
<td>0.0046</td>
<td>***</td>
<td>***</td>
<td>107.6 (742.1)</td>
<td>0.100</td>
<td>98.1 (676.0)</td>
<td>0.138</td>
</tr>
</tbody>
</table>

*Yield Determined by 0.2% Offset Method

In Table 7-8, \( f_y \) and \( \varepsilon_{y} \) correspond to the stress and strain at yield; \( f_{sh} \) and \( \varepsilon_{sh} \) correspond to the stress and strain at the onset of strain hardening; \( f_{max} \) and \( \varepsilon_{max} \) are the maximum stress and the corresponding strain; and \( f_u \) and \( \varepsilon_u \) are the stress and strain at the rupture of the reinforcement. The stress-strain response of the T-headed bars and the various other reinforcement included in the wharf deck are shown in Figure 7-39 and Figure 7-40, respectively.
Figure 7-39. T-Headed Reinforcement Stress-Strain Response
For the prestressing in the pile, ½ in. (13 mm), 270 ksi (1860 MPa), low-relaxation strands were used, but more specific material data was not provided by the precaster. Additionally, the spiral reinforcement used for the piles was 0.1093 in.$^2$ (70 mm$^2$) W11 A82 smooth wire. This material was tested by Concrete Technology Corporation, and the ultimate stress was found to be 121 ksi (834 MPa), with an initial elastic modulus of 25,400 ksi (175,000 MPa). The material had a non-linear stress-strain response, with the initial departure from linearity occurring at a stress of about 65 ksi (at a strain of 0.0025).
7.3.5 NEES MUST-SIM Facility Overview and Test Setup

The primary way in which the current structural concrete pile-wharf connection test differs from other previous tests is the manner in which the loads and displacements are applied. The University of Illinois Multi-Axial Full-Scale Sub-Structured Testing and Simulation (MUST-SIM) Facility is one of the 15 equipment sites that form the George E. Brown Network for Earthquake Engineering Simulation (NEES). The facility possesses six Loading and Boundary Condition Boxes (LBCBs), which allow for loads and/or displacements to be applied in all six degrees of freedom. Three of the boxes are in the NSEL, and these full-scale LBCBs are used to test large-scale specimens. The LBCBs can be oriented in any configuration necessary for testing, and they may be mounted on either the strong floor or the reaction wall, as needed. The reaction wall is an L-shaped post-tensioned concrete wall, with legs measuring 50 ft (15.2 m) and 30 ft (9.1 m). Also, the wall is 28 ft (8.5 m) tall, 5 ft (1.5 m) thick, and post-tensioned to the strong floor, which is a 17 ft (5.2 m) deep box-girder on grade. The full-scale LBCBs, reaction wall, and strong floor can all be seen in Figure 7-41.
The remaining three (reduced-scale) LBCBs are located in the companion 1/5-scale laboratory and studio at the NEES MUST-SIM facility. The small-scale lab has a 1/5-scale reaction wall, as well as three 1/5-scale LBCBs. The small-scale facility allows researchers to have an opportunity to conduct valuable pre-test verifications and debugging of the loading / control protocols and software. For this experiment, a small-scale rubber specimen was constructed, and external instrumentation was added to fully emulate the control procedures to eventually be used in the large-scale laboratory. Rubber was selected as the material for the small-scale specimen because it was more forgiving than reduced-scale concrete during the process of understanding and learning the capabilities and limitations of the LBCBs. Figure 7-42
shows a picture of the small-scale facility (on the left), and an image of the rubber test specimen attached to the small-scale LBCB (on the right).

Figure 7-42. Small-Scale MUST-SIM Facility (left) and Small-Scale Specimen Attached to LBCB (right)

While it was not used in this study, the MUST-SIM facility also has the capability to conduct hybrid simulations. In hybrid simulation, displacement demands are computed from analytical modeling tools and are then imposed on the test specimen, with the model stiffness characteristics actually being updated using feedback from the experimental data. Six degree-of-freedom control of an experimental specimen coupled with the hybrid simulation capability allows for the possibility of obtaining an overall system level response by analytically modeling the system while experimentally testing just a structural component of interest.

The LBCBs are composed of six actuators connecting a loading platen to a reaction frame. The end connections of the actuators are made with pillow block spherical bearings to ensure that the movements of the actuators are not unduly restricted in any of the six degrees of freedom. With proper arrangement of the actuators within a fixed unit, a system is created that
can be calibrated to move in any coordinate system and to rotate about any given control point in space. The nominal naming convention of the six actuators is shown in Figure 7-43, as well as the global coordinate system. While the LBCB shown is a mirror image of the actual LBCB used to test the current specimen, the naming convention and global coordinate system are still the same. Two of the actuators have a ± 10 in. (250 mm) stroke, while the other four actuators have a ±5 in. (125 mm) stroke. The longer-stroke actuators are roughly aligned parallel to each other and along the x-axis, and thus they are called X1 and X2. As shown, the LBCB is attached to the strong wall, so the positive y-direction is orthogonal to the strong wall, with the positive direction being away from the wall. The single actuator aligned in this configuration is named Y1. The remaining three actuators are aligned vertically with the z-axis, and they are aptly named Z1, Z2, and Z3. In Figure 7-43, the blue reaction frame, the labeled orange actuators, and the orange loading platen are also all shown.

Figure 7-43. LBCB with Actuator Labels and Coordinate System
The overall dimensions of the reaction box are approximately 11.8 ft (3.6 m) long, 6 ft (1.8 m) wide, and 6 ft (1.8 m) tall, and the box weighs about 35 tons. The loading platen is approximately 7.2 ft (2.2 m) long and 6.2 ft (1.9 m) wide. Each individual actuator has a 225 kip (1000 kN) capacity in tension and a 310 kip (1380 kN) capacity in compression. Due to the particular configuration of the actuators, the Cartesian force and moment capacities vary for the default configuration. Table 7-9 gives a summary of the force and displacement capacities of a single LBCB acting about the default control point located at the geometric center of the bottom of the loading platen. The values given in Table 7-9 do not account for the interaction of applying multiple forces and/or moments at once. For example, if the maximum compression or tension were being applied in the z-direction, no reserve capacity in the actuators would remain to apply any moment about the y-axis at all.

Table 7-9. LBCB Force and Displacement Capacities

<table>
<thead>
<tr>
<th>Loading DOF</th>
<th>Force Capacity</th>
<th>Stroke</th>
</tr>
</thead>
<tbody>
<tr>
<td>X-Translation</td>
<td>430/660 kips (T/C)</td>
<td>± 10 in.</td>
</tr>
<tr>
<td>Y-Translation</td>
<td>215/330 kips (T/C)</td>
<td>± 5 in.</td>
</tr>
<tr>
<td>Z-Translation</td>
<td>645/980 kips (T/C)</td>
<td>± 5 in.</td>
</tr>
<tr>
<td>X-Rotation</td>
<td>7600 kip-in</td>
<td>± 16°</td>
</tr>
<tr>
<td>Y-Rotation</td>
<td>10000 kip-in</td>
<td>± 11.8°</td>
</tr>
<tr>
<td>Z-Rotation</td>
<td>7600 kip-in</td>
<td>± 16°</td>
</tr>
</tbody>
</table>

The NEES MUST-SIM facility uses four separate software programs to control an experiment. While this makes the control architecture complex, it is also very flexible and comprehensive. The architecture can be considered in a hierarchical manner, where steps, commands, and tolerance checks are looped over one another. The four programs commonly
used at the MUST-SIM facility have been termed as the Simulation Coordinator (SimCor), the LBCB Plugin, the Operation Manager (OM), and the Shore Western control software. This software is exclusively used to control the LBCB, while an entirely different system of computers is then used for data acquisition, for taking photographs, and for sharing real-time data and images with off-site collaborators. The other programs used are the data and acquisition software (NEESdaq), camera plugin, Krypton plugin, Data Turbine, Remote Data Viewer (RDV), and NEES Central. The overall software architecture is shown in Figure 7-44.

Figure 7-44. MUST-SIM Overall Software Architecture

The LBCBs are servo-controlled units produced by Shore Western Manufacturing of Monrovia, CA. The Shore Western controller is used to drive and to maintain the actuators at a given command length, but it is not capable of calculating the required sets of displacements of
the six individual actuators necessary to achieve a desired overall Cartesian LBCB position. Since the only data available are the actuator lengths and connection points of the actuator ends, the Operation Manager (OM) is used to make the transformation from actuator space to Cartesian space. A closed form solution can be obtained to determine the actuator lengths from a given Cartesian position; however, an iterative solution is required to compute the Cartesian location for given actuator lengths. Nakata et al. (2007) discusses the method used by the OM to calculate the Cartesian position using a Newton-Raphson iterative solution with a kinematic Jacobian. To successfully move the LBCB platforms smoothly in Cartesian space, the actuator commands and the corresponding feedback must be rapidly processed by the OM. The decision to use a separate computer for the OM to generate and measure the commands is due to the computational expense of the previously described procedure. The OM receives the servo error, LVDT readings, and load cell readings from each actuator, and the individual actuator readings are used to calculate the current Cartesian position of the loading platen. With the position of the loading platen known and the load cell readings, the individual actuator vectors can then be used to calculate the Cartesian forces and moments about the control point.

User input to the OM is required for all six degrees of freedom, and the OM is capable of handling both force-controlled and displacement-controlled loading schemes. While a displacement-controlled degree of freedom is straightforward, a force-controlled degree of freedom requires another iterative loop within the OM. The specimen stiffness is stored and continually updated in the OM, based on the change in force obtained for a given change in displacement command. Thus, the OM calculates the estimated change in displacement required to achieve the desired force. Iterations of the force-controlled DOFs are conducted one DOF at a time, in order to ensure that the effect of changing a single DOF is properly represented in the
specimen stiffness matrix. This procedure continues until all of the force-controlled degrees of freedom are brought within the user-defined tolerance.

While the OM is able to enforce mixed-mode commands using the LBCB throughout testing, the Cartesian displacements calculated by the OM do not necessarily reflect the displacements realized by the specimen control point. Since an equal and opposite force occurs in the LBCB any time a load is applied to a specimen, the LBCBs are susceptible to elastic deformations that must be accounted for if the desired specimen displacements are to be achieved. Since the OM is unaware of these errors or their magnitude, an additional software package is used to augment the Cartesian commands created by the OM – this software is the LBCB Plugin. Obtaining the actual specimen displacement requires a set of external sensors mounted to the specimen control point. The connection locations of these sensors in Cartesian space are input into the LBCB Plugin. The LBCB Plugin then uses the changes in the external sensor readings and the desired input displacements to compute a kinematic Jacobian defining the relationship between a change in specimen location vs. the change in external sensor readings, similar to the procedure described to compute the actuator-to-Cartesian calculations in the OM. If the measured specimen displacement(s) differ from the desired input displacement(s) by more than the user-defined tolerance, the necessary adjustment to the input displacement is computed and sent to the OM for another iteration. This process continues until all of the displacement DOFs have converged within their tolerances.

Once all of the mixed-mode criteria for a particular step have been satisfied, the LBCB Plugin then sends triggers to the data acquisition software, camera plugin, and Krypton plugin to collect what has been colloquially termed “step data.” The step data provides synchronized data from a given step collected from each instrument, as well as the LBCB command, displacement,
and force data. Previous versions of the control software used UI-SimCor for data synchronization, but with recent updates to the software packages and given that this test was not a hybrid simulation, the use of UI-SimCor was not required. An additional feature of the software packages is that the data samples collected can be simultaneously shared across the internet to the NEES Realtime Data View (RDV) for remote viewing by project colleagues and to NEES Central for additional data archiving. The previously described procedure is repeated for each individual load step until all of the steps are completed.

7.3.6 Instrumentation

One of the characteristics of a NEESR project at the MUST-SIM facility is the quantity and variety of instrumentation applied to each test specimen. Such dense data collection is critical toward making any future comprehensive modeling endeavors as robust as possible. The current test specimen was heavily instrumented with traditional instrumentation, such as strain gauges, linear variable displacement transducers (LVDTs), string potentiometers, and inclinometers. Additionally, the Krypton system, introduced previously in Section 3.6, was used as a non-contact measurement machine to obtain a dense specimen displacement field during testing. In total, 137 channels of data were logged by the DAQ system, and an additional 132 LED positions were recorded by the Krypton system throughout testing. Further specifics about types of instrumentation, DAQ setup, and application to the test specimen will be discussed in this section.

7.3.6.1 Traditional Instrumentation

Quarter-bridge strain gauges were applied densely on the #10 (D32) T-headed dowel bars and more lightly throughout the reinforcement in the deck and on the pile concrete surface. The strain gauges applied to the steel reinforcement were high-elongation, with a 0.4 in. (10 mm)
gauge length, while general purpose 1.18 in. (30 mm) strain gauges were used for the concrete surface. All strain gauges were products of Tokyo Sokki Kenkyujo Co., Ltd. (TML). When applying the strain gauges, the steel had to first be ground to remove the deformations and coating, and then sanded to a smooth surface. The gauge location was then cleaned, and the gauge was attached with the CN adhesive from TML. The gauge was then coated with polyurethane to protect from moisture, and covered with butyl rubber and metallic tape for added protection during bar placement, cage construction, and concrete placement. The wires extending from the strain gauges were then strain-relieved by using cable ties to attach the strain gauge wire to the bar. For the T-headed bars, the four bars likely to be subjected to the most strain (the extreme north and south bars) each had 9 strain gauges applied, while the center 4 dowel bars were each instrumented with four strain gauges. Figure 7-45 shows the specific T-headed dowel bar strain gauge locations.

![Strain Gauge Locations](image)

**Figure 7-45. T-Headed Dowel Reinforcement Strain-Gauge Locations**

The wharf deck was more lightly instrumented with strain gauges since little yielding was expected. Strain gauges were applied to the deck stirrups under the pile and to the deck
transverse reinforcement immediately under the pile. Additional strain gauges were applied to the longitudinal deck reinforcement on the bars closest to the pile at a location along the northern face of the pile. Plan and profile views of the deck strain gauge locations are shown in Figure 7-46 and Figure 7-47, respectively.

Figure 7-46. Deck Reinforcement Strain-Gauge Locations - Plan View
Concrete surface gauges were attached using a slightly different procedure. First, the surface of the concrete was smoothed with sandpaper, and then an epoxy base layer was applied. Once the epoxy was set, this base layer of epoxy was sanded to a smooth finish and cleaned. The concrete gauge was then secured with epoxy and allowed to set. The concrete strain gauges were essential in monitoring the overall strain condition in the specimen during the final LBCB connection process and toward understanding the behavior during elastic displacements. Ten of such concrete surface gauges were applied to the specimen. One was applied to each cardinal direction face near the top of the pile and at a location about 2 ft (610 mm) from the pile-wharf connection. An additional two strain gauges were applied to the north and south faces near the pile-wharf interface; however, it was expected that these gauges would be lost early in the testing.
due to pile spalling. The locations where the concrete strain gauges were applied are displayed in Figure 7-48.

![Figure 7-48. Concrete Strain Gauge Locations– E-W (left) and N-S (right)](image)

Inclinometers (by Spectron Systems Technology) were applied to the specimen to obtain rotations at various locations along the height of the specimen. Ten inclinometers were applied to the west and east sides of the specimen, at five locations: center of the deck, pile-wharf interface, 12 in. (305 mm) from the pile-wharf interface, 24 in. (610 mm) from the pile-wharf interface, and at the control point (84 in. (2.13 m) from the interface). These instruments were powered by a dual source direct current (DC) voltage. An example of the inclinometers used can be seen on the left in Figure 7-49.

Twelve LVDTs from TransTek were used to obtain a variety of absolute and relative displacement measurements. To attach the LVDTs, short holes were drilled in the specimen with
an impact drill, epoxy was injected into the holes, and ¼ in. (6 mm) diameter threaded rods were inserted. The LVDTs were then connected using specially constructed LVDT holders attached to the threaded rods. Four LVDTs were used to monitor any uplift, rotation, or slip of the wharf deck. Uplift of the deck was monitored using two vertically aligned LVDTs attached to the north and south faces of the wharf deck. Rotation and slip of the deck were monitored through two horizontally aligned LVDTs attached to the strong floor and monitoring locations on the west and east corners of the south deck face.

Uplift of the pile was monitored by using four spring-loaded LVDTs, where two were attached to each of the west and east faces of the pile near the pile-wharf interface. These LVDTs provided a relative measurement between the pile and wharf deck; however, due to deck spalling over the course of testing, some of the later LVDT uplift measurements became unreliable. The remaining LVDTs were used to measure the curvature of the pile at 12 in. (305 mm) and 24 in. (610 mm) from the pile-deck interface. These LVDTs were attached to the pile, with their measurements taken relative to a steel angle supported slightly above the deck. The steel angle was used to prevent deck spalling near the North and South faces of the pile from affecting these LVDT readings. The deck LVDTs were powered by a DC voltage, while the LVDTs attached to the pile all used an alternating current (AC) power supply. An example of an LVDT used for the pile curvature measurements is shown in the center of Figure 7-49.

The final type of traditional instrumentation used was string potentiometers from Celesco. In total, nine string potentiometers were used to obtain absolute measurements of the displacement of the pile at several locations. Depending on the expected range of motion, string potentiometers of different length strokes were used. To monitor any slip of the pile relative to the wharf deck, two string potentiometer bases were secured to the north end of the wharf deck
and attached to the west and east faces of the pile near the pile-wharf interface. The lateral displacement of the pile in the expected pile damage region was monitored at 12 in. (305 mm) and 24 in. (610 mm) from the pile-wharf interface by two string potentiometers. To ensure that the pile was not moving in the out-of-plane (transverse) direction, a string potentiometer was attached from the strong wall to the tip of the east face of the pile at 84 in. (2.13 m) from the interface. Each connection to the pile was achieved by epoxying threaded rods into holes drilled into the pile at locations of interest. These string potentiometers were of standard tension Celesco model PT1A. All of the previously mentioned string potentiometers were connected to the DAQ, since they were not critical to the loading of the specimen.

The final four string potentiometers were external control sensors used in part to compensate for the elastic deformation of the LBCB, so they were connected to the OM computer. Since these control sensors were critical to the loading of the specimen, either model PT101 or PT8101 high tension string potentiometers were used. The loading protocol called for the application of x-displacement and y-rotation to the end of the pile. To monitor the x-displacement of the pile, two high tension string potentiometers were aligned horizontally and attached to the west and east faces of the pile at the control point. To monitor rotation of the pile at the control point, vertically aligned string potentiometers would have ideally been connected to the specimen, but due to issues with the resolution of the instruments and the desired resolution of the applied rotation, a wider distance between the string pots was desired. The rotation control string pots were therefore attached from external knee braces on the strong wall to the LBCB loading platen. Since the connection between the LBCB platen and the adapter plate, and the connection between the adapter plate and the pile, were considered rigid, the rotations measured from the LBCB platen were taken as essentially the same as the rotations at
the control point. It was later found that the rotations achieved at the control point were not quite as large as desired, so the input rotations were amplified slightly to account for any slip in the connection to the LBCB. This will be covered in more detail in Chapter 8. The method used to calculate the actual displacement and rotation of the control point relative to the desired displacement and rotation was described previously in Section 7.3.5. All of the string potentiometers were AC powered, and an example of a string potentiometer is shown on the right in Figure 7-49.

![Figure 7-49. Inclinometer (left), LVDT (center), and String Pot (right) Examples](image)

### 7.3.6.2 Advanced Instrumentation

In addition to the traditional instrumentation described above, an advanced instrumentation system was used for this research: the Krypton K600 optical coordinate measuring machine (CMM). Since the purchase of the Krypton system by the MUST-SIM facility, the company Krypton was purchased by Metris, which was then purchased by Nikon, but in the interest of simplicity and consistency, the system will be described as the Krypton in this document. A grid of 132 light-emitting diodes (LEDs) was applied to the specimen to obtain a dense array of displacement field data during testing. The LED “targets” were hot glued to the surface of the specimen, with the Krypton system using three linear charge-coupled device
(CCD) cameras to triangulate the source of the infrared light emitted from the LEDs to locate their position in 3D space. Krypton data is stored and acquired on a separate computer than the other traditional instruments. The Krypton system is able both to record the 3D position of individual LEDs and to record the rigid body motion of a set of LEDs, all relative to a user-defined coordinate system. The LED positions can be recorded up to an accuracy of +/- 0.0008 in. (0.02 mm), at a sampling rate of up to 1000 readings per second, with the actual accuracy and frequency depending on LED distance from the Krypton camera and the number of LEDs used. The Krypton system and an image showing its method for triangulating LEDs is shown in Figure 7-50.

![Figure 7-50. Krypton Coordinate Measurement Machine](image)

The dynamic frame capabilities of the Krypton system were particularly useful when aligning the LBCB natural coordinate system to the desired coordinate system of the specimen. Once the specimen was post-tensioned to the strong floor, but before the final attachment was
made between the adapter plate and the LBCB, the LBCB coordinate system had to be transformed to a coordinate system in “specimen space”. To accomplish this, a dynamic frame was created with LEDs attached to the LBCB platen, while a measurement coordinate system was defined relative to the test specimen. Then, the LBCB underwent a series of displacements and rotations, one degree of freedom at a time, while the movement was monitored with the Krypton. Using these results, a transformation matrix was produced to adjust the LBCB motion to align with the specimen coordinate system. This procedure was then repeated, and the alignment of rotations and displacements was verified. In a similar manner, the control point of the LBCB was monitored and translated to the appropriate location within the specimen. A useful capability of the Krypton data is that each node can be considered a node of a finite element model, and thus the displacement field can be input and operated on in a similar fashion as a finite element post-processor to provide strains in quadrilateral elements. This will be further demonstrated in Chapter 8, and an image of the dense LED grid pattern is shown in Figure 7-51.
Figure 7-51. Krypton LED Grid Layout

An overall line drawing showing the location of all the aforementioned instrumentation, traditional and advanced, is displayed in Figure 7-52.
Figure 7-52. Overall Instrumentation Layout
7.3.6.3 NEES Data Acquisition System

As can be seen from the previous sections, the instruments used to obtain data from the port specimen were extensive, and collecting the data in a meaningful way is critical to the understanding and analysis conducted after the experiment. The data during testing was collected both discretely and continuously. The discrete data was collected at the conclusion of each step, which provides a manageable amount of data to analyze the response of the specimen during testing. To capture any events between converged loading steps, continuous data was also collected. The continuous data collection is more traditional; however, synchronizing the data collected on multiple computers can become burdensome during post-processing.

In total, the (non-Krypton) DAQ system logged 137 channels with the following breakdown: 24 LBCB channels (load and displacement in Cartesian and actuator space), 12 LVDT channels, 9 string potentiometer channels, 10 inclinometer channels, 68 steel strain gauge channels, 10 concrete strain gauge channels, and 4 excitation voltage channels. The DAQ unit used National Instruments (NI) hardware. Two SCXI-1001 chassis were connected in series, with signal conditioning completed by the modules and terminals connected into each chassis. The modules used were SCXI-1540, SCXI-1521b, and SCXI-1104c. These modules were connected to the SCXI-1513, SCXI-1317, and BNC-2095 terminal blocks, respectively. The chassis were connected to the DAQ computer with an NI PCI-6289 card, and the signals were monitored using the NEESdaq program built with LabView. The NEESdaq program was written such that each instrument could be input into a spreadsheet, with relevant calibration and channel name parameters provided. Then, a script automatically generates each channel within the NEESdaq program, eliminating the tedious task of inputting each individual channel by the researcher. All of the data collected is recorded locally, and will be uploaded to the NEEShub.
data repository for use by collaborators. Additionally, data from this current experimental program and the subsequent repaired pile test has been archived locally at the NEES MUST-SIM facility for future researchers.
CHAPTER 8. PILE-WHARF CONNECTION EXPERIMENTAL RESULTS

One of the major objectives of this current study is to understand the structural behavior of pile-wharf connections; thus it is critical to examine in detail the onset and progression of damage during testing, as well as to understand the global and local responses of the test specimen. The data investigated in this analysis is available for future researchers both at the NEEShub data repository and at the MUST-SIM facility at the University of Illinois.

8.1 Overall Global Behavior

The overall global behavior of the pile-wharf connection specimen was monitored throughout testing. The global behavior here is described as the applied and resulting displacements, as well as the reaction (resulting) forces and moments. As previously noted, the forces applied to the specimen were monitored by measuring the forces in the load cells located on each of the six actuators, and then the actuator forces were converted into specimen space through transformation matrices. The displacements imposed on the specimen were controlled through an iterative feedback loop between externally mounted string potentiometers and the LVDTs located on each actuator. Additional verification of these displacements will be shown in this section, where the intended displacements and rotations are compared to the actual measured values from other data acquisition instruments. The fully instrumented pile-wharf connection specimen is shown in Figure 8-1 right before testing. Many of the instruments can be seen, including the dense Krypton LED grid and the rotation control string potentiometers. The vertical reinforcing bars protruding from the wharf deck are partially pre-installed for later use in the repair during the next phase of the experimental program (after the testing described below).
Each load step had a target displacement and rotation for the center of the pile cross-section located 84 in. (2.13 m) from the pile-wharf interface. Since the lateral displacement was monitored by string pots directly attached to the control point, any error in the lateral displacement should be limited only by the inherent accuracy of the instruments (about 0.1% of
full stroke of the instrument) and the user-defined convergence tolerance, which ranged from 0.001 to 0.005 in. (0.0254 to 0.127 mm), depending on the magnitude of the displacement.

Control of the rotation, on the other hand, was subject to several assumptions. Since it was deemed that the inclinometers did not have a sufficient accuracy and precision to control the rotation, high-tension string potentiometers were used with a large lever arm to achieve the desired rotational precision. Having the instruments farther apart made attaching the string pots directly to the control point impractical, so they were attached to the loading platen of the LBCB. With the control rotations being measured from the LBCB platen, an assumption is made that the LBCB platen is rigidly attached to the control point. The connection between the specimen and the LBCB was made by first post-tensioning eight high strength threaded rods to connect the specimen to the adapter plate. Then, a perimeter of 24 high strength bolts was used to attach the adapter plate to the loading platen. It was deemed that each of these connections was sufficiently rigid for the location of the control instruments to provide accurate rotation measurements. To stiffen the 12 in. (305 mm) length of pile from the end of the pile to the control point, a 9 in. (230 mm) long welded steel plate collar was placed around the end of the pile, stiffened, and then grouted. With the aforementioned efforts, it was assumed that the relationship between the rotation of the LBCB platen and the control point was reasonably rigid for control of the specimen, and the rotation was measured with redundant instruments to verify the assumption.

Figure 8-2 displays a comparison of the lateral displacements at the control point during the Northridge earthquake loading. The target displacement is plotted and compared to lateral displacements measured by the two string potentiometers attached to the west and east faces of the specimen at the control point. An additional comparison is provided versus a Krypton LED located on the west face of the specimen at the control point. It can be clearly seen that the
external sensors all closely match the desired inputs to within 0.02 in. (0.5 mm) throughout the loading, even at small target displacements. The result shown in Figure 8-2 (from the Northridge loading) was typical of that seen throughout the entire testing program.

Figure 8-2. Control Point Lateral Displacement Comparison

The measured rotation at the control point did not as closely match the intended “input” value as the displacements did; however, they were reasonably close, and with some adjustment they were almost identical to the desired rotation. For the Imperial Valley and Northridge earthquake loadings, the input files endeavored to directly obtain a target ratio of 72.6 in. (1.84
m) of displacement per radian of rotation. It was found during post-processing after completion of the initial portion of the testing program that actual rotations at the control point were about 12% less than the desired input. This effect can be seen in Figure 8-3, which compares various rotations measured at and near the control point during the Northridge loading. The outer two curves represent the target input rotation and the corresponding rotation measured by the control string pots. As expected, these two curves align quite well; however, the other curves represent rotations nearer the control point. Those rotations are measured by three LEDs on the steel connection collar and from an inclinometer located right at the control point. It can be seen that each of these instruments showed the rotation near the control point to be slightly less than the target rotation, so an amplification of 15% was applied to the rotations in the input file of subsequent testing days.
This amplification of the rotation seemed to work quite well, and the remaining testing days show that the rotation at the control point closely matched the desired rotation through the use of an amplified input rotation. Figure 8-4 shows similar plots to those displayed in the previous figure, only now when input rotation amplification was used; the rotations obtained here on a later day of testing by the inclinometer located at the control point were 98% of the desired input rotations. Since this was well within an acceptable tolerance, no further adjustments were made. (The sample of data shown in Figure 8-4 is from the cyclic portion of the loading regimen.) At the end of each day, an assessment of the accuracy of the rotations throughout testing was made, and this quality assurance process ensured that the applied
displacements and rotations remained close to the target values, despite damage to the specimen. Regardless of any adjustments to the loading ratios, the actual values of rotation were always recorded, and the figures presented later use only the deformations experienced by the specimen.

![Figure 8-4. Control Point Rotation Comparison after Input Adjustment](image)

The complicated control scheme applied to the tip of the pile in this testing is unique to the NEES MUST-SIM facility at Illinois, and the previous plots show that the target displacements and rotations were accomplished within a reasonable tolerance throughout testing. Since the loading was based in part on the response of an analytical model subjected to ground motions, achieving a similar actual displacement and rotation at the control point in order to observe the resulting damage at the connection was a major objective of the test.
8.1.2 Global Force and Displacement Response

The force and moment response of the pile-wharf connection was a major point of interest for the test. Understanding the moment capacity at the connection for typical pile-wharf connection designs is critical to the design and construction of port structures in the field and to fully understanding the behavior of the experimental specimen.

During testing, the moment at the connection was computed from three separate contributions: moment applied to the tip of the pile, shear applied at the tip of the pile at a known lever arm, and the axial force applied at the tip of the pile at a known LBCB displacement (P-Δ). Throughout the test these individual terms were monitored, as well as the total computed connection moment. While the moment during the initial earthquake records seemed reasonable for the cross-section of the pile-wharf specimen, it was discovered between the earthquake loading and the cyclic loading regimes that some of the internal electronics associated with the X1 actuator load cell had not been working properly. Since the specimen was offset on the LBCB platen toward the X2 actuator, it was logical that the X2 actuator would contribute more to the lateral force; however, a close examination of the data revealed that the X1 actuator load cell was never reporting forces above one kip. This issue was later resolved, and the data reported from the cyclic portion of testing was without incident. The force in the X1 actuator during earthquake loading was solved for through equilibrium equations during post-processing. Since the LVDTs in each actuator were working properly, it was possible to know the position of each actuator throughout the test. With the positions of each actuator known, as well as the forces in five of the six actuators, the final X1 actuator force was computed through an equilibrium calculation by setting the Z-moment to a zero value. With all of the actuator loads and positions known, it was then possible to perform the same transformation as the Operations
Manager software by converting the values from actuator-space to specimen-space. The newly computed and corrected shear force value was then used for calculation of the connection moment.

![Figure 8-5. Connection Moment versus Lateral Displacement before and after X1 Actuator Adjustment](image)

Figure 8-5 shows both the corrected and uncorrected connection moments versus pile tip displacement. The described equilibrium procedure was validated by also performing the calculation on the cyclic loading portion and comparing the resulting forces to the actual recorded forces. This can be easily recognized in Figure 8-5 at the larger displacements, where both the computed connection moment and the recorded connection moment are almost
identical. With this method proven to be reliable, the data recorded during the earthquake loading was adjusted, and the X1 actuator force was back-solved. Figure 8-6 shows the final connection moment versus displacement plot where the connection moment from the earthquake portion of the loading is from the actuator forces with an adjusted X1 value, and the connection moment from the cyclic loading is from the actual recorded actuator load cell values.

From the connection moment versus lateral tip displacement shown in Figure 8-6, it can be seen that the moment capacity of the connection never experienced a significant degradation. The peak moment of 7,451 kip-in. was experienced in the positive displacement direction during the Northridge loading, while a maximum moment of 7,038 kip-in. in the negative displacement direction occurred during the cyclic loading. The moment capacity of the connection experienced only minor degradation over the duration of the test, with final moments at peak displacements achieving 5,707 kip-in. and 6,599 kip-in. in the positive and negative directions, respectively. This corresponds to about a 25% drop in moment capacity in the positive direction and only a 12% drop in the negative direction, relative to the maximum achieved connection moment. The smaller strength reduction in the negative direction may be due in part to the fact that the negative direction never reached its possible peak moment due to pre-existing damage in the opposite direction. The Response-2000 sectional model presented in more detail later predicted a maximum moment capacity at the connection of 7190 kip-in., which is in good agreement with the experimental results. The asymmetry in the response is due to the unsymmetrical nature of the earthquake loading during early damage. While this is less common to see in a laboratory testing environment, the realistic loading and boundary conditions of the test setup should be more similar to the connection response that may be seen during an earthquake. The progressively larger cyclic load steps were ceased when the X-actuators began
to reach their displacement capacities. Although the LBCBs have a ±10 in. (254 mm) stroke in each X-direction, some of the available actuator stroke was consumed by a rotation about the axis of the pile that was required during the connection of the specimen to the LBCB. An increase in the axial load beyond 90 kips would have likely caused the damage to the connection to be more extensive; however, it was deemed that the axial load was representative of typical port gravity loading.

Figure 8-6. Final Connection Moment versus Lateral Displacement
With the overall global behavior now briefly described, the following sections will discuss the local behavior in more detail during each phase of the loading: Imperial Valley, Northridge, Kobe, and reversed cyclic loading.

8.2 Imperial Valley Earthquake

The first record imposed on the specimen was the Imperial Valley response. As described previously in Section 7.3.1, the input lateral displacement and rotation were the results at a key pile location from a full port model subjected to a suite of earthquake records. The model was to simulate a berth in the Port of Oakland, with this modeling undertaken by project colleagues at Georgia Tech. From the suite of 60 SAC ground motions for which they ran their models, one earthquake record response was selected from each of the following probability of exceedance bins: 50% in 50 years, 10% in 50 years, and 2% in 50 years. These earthquake motions were imposed on the port specimen in increasing magnitude, so the Imperial Valley record, which was the 50% probability of exceedance in 50 years record, was imposed on the specimen first. Figure 8-7 shows the resulting lateral displacement during the Imperial Valley record at a point 84 in. (2.13 m) from the pile-wharf connection. It can be seen that the lateral displacements are rather low, with a maximum of 0.129 in. (3.3 mm) being applied. It was predicted that the test specimen would remain relatively elastic throughout this record, with only very minor cracking. The flat portion at the beginning of the record seen in Figure 8-7 corresponds to the 17 loading steps over which the axial load was incrementally applied up to the full 90 kips that was maintained throughout testing.
The next sections will describe the visual extent of damage and the local behavior experienced during the Imperial Valley earthquake loading.

### 8.2.1 Imperial Valley - Visual Damage

By design, the Imperial Valley earthquake was selected to get an understanding of the elastic stiffness of the pile-wharf connection. As a result no visual damage was observed during this portion of the loading, as expected. In fact, the displacements were so small that it was almost difficult to perceive any displacements; however, the high tension string potentiometers confirmed that the specimen did indeed experience the desired displacements and rotations.
Figure 8-8 and Figure 8-9 display views of the pile-wharf connection from fixed cameras at the peak lateral displacement during the Imperial Valley record.

In Figure 8-8 it can be seen that there were no visible signs of cracking, spalling, or distress in the specimen. The pile-wharf connection was able to accommodate all of the displacements and rotations without undergoing any damage. Figure 8-9 shows the West view of the specimen, and again there are no signs of cracking, crushing, or spalling.
While there was no visual change in the specimen throughout the Imperial Valley loading, the dense instrumentation seen in the previous two figures did provide significant detailed data about the nature of the structural response of the pile-wharf connection to this earthquake record.

8.2.2 Imperial Valley - Local Behavior

Characterizing the local structural behavior requires a thorough analysis of data from the various instruments mounted on the specimen. Throughout testing, the specimen seemed to be rotating about the connection at the wharf deck through two mechanisms: flexural behavior of the pile and concentrated connection rotation. The contribution of the pile comes in the form of elastic flexural bending (including cracking and spalling of the prestressed, precast concrete pile), and connection rotation is the term that will be used to describe the behavior below the pile-deck interface, including elongation and slip of the T-headed bars, as well as crushing of the pile at the compression toe. Therefore, to properly calculate the demand on and curvature of the
pile, the two contributions were separated by first quantifying the amount of total specimen rotation that could be attributed to rotation of the connection. An exaggerated schematic of these two contributions to specimen rotation is shown in Figure 8-10. It should be noted that the wharf deck was assumed to be essentially fixed, so it was not considered a significant contributor to the rotation of the specimen. This was confirmed through inclinometers mounted on the wharf deck.

![Figure 8-10. Connection Rotation (left) and Pile Rotation (right) Contributions to Pile-Wharf Connection Rotation (Jellin, 2008)](image)

The dense grid of Krypton LEDs provided information about the displacement of discrete points on the pile throughout the test. Using these LED coordinates, it is possible to compute strains and rotations, as well as to even generate a finite element mesh. The rotations determined from the Krypton LEDs represent the total specimen rotation, so to identify the connection rotation, it was necessary to quantify and subtract the amount of rotation that could be attributed to the pile. The amount of rotation seen by the pile due to flexure near the interface was estimated by using the values of the strain gauge readings on the T-headed dowel bars at various
locations. To obtain the rotation, the strain gauges on different dowel bars, but at a common elevation, were used to provide a strain profile at a given cut through the specimen. Admittedly, this assumption eventually becomes less reliable as strain gauges begin to enter into yielding and strain hardening, but since none of the strain gauges yielded during the Imperial Valley earthquake record, this approach can then provide some valuable insight into the percentage of rotation that could be assumed to be the result of connection rotation within the pile-wharf connection. Strain gauges were located at the interface of the pile-wharf connection, so the curvature obtained from the strain gauges at the extreme dowel bars was converted to an average rotation over the distance from the end of the pile (located two inches into the wharf deck) to a location two inches above the strain gauges along the pile. At the location two inches up from the pile-wharf interface, both a row of seven krypton LEDs and also an inclinometer were located. These instruments provided the information for total specimen rotation approximately at the interface, with the resulting values being nearly identical from each, as seen in Figure 8-11. Of the seven krypton LEDs, the extreme LEDs were used for this portion of the analysis to obtain the total rotation of specimen.
With the total rotation of the specimen obtained, the next step was to separate this rotation into contributions from the flexural behavior of the pile and the lumped rotation at the connection. The ideal way to compute this was to obtain the average flexural rotation of the pile from the strain gauge curvatures over a 4 in. (100 mm) length. Then the lumped rotation at the connection was calculated as the difference between the total rotation and the pile flexural rotation. Figure 8-12 displays a comparison of the pile flexural rotation near the interface to the total rotation measured just above the interface. The difference between the two curves can mostly be attributed to the lumped connection rotation, with the average percentage of the total rotation attributed to the connection being approximately 71%. Jellin (2008) reported a lumped
connection rotation of about 40% of the total rotation for specimens without a bearing pad; however, specimens with a bearing pad had lumped connection rotation accounting for about 80% of the total rotation. Since the tests by Jellin (2008) had a five times larger axial load, more deformation may have occurred due to the flexural behavior of the pile, and the current specimen with only a 90 kip (400 kN) axial load may have a response more similar to the specimens with a bearing pad.

![Figure 8-12. Comparison of Total Rotation and Pile Flexural Rotation at the Interface](image)

This relationship can also be seen in Figure 8-13, which shows a plot of the total rotation from the Krypton data at 2 in. (50 mm) up the pile vs. the pile flexural rotation obtained from the strain gauges. A line with a slope of 0.29 rad/rad was included to show the correlation.
Although the strain gauges did not yield during the Imperial Valley loading, it was desired to develop an alternative method to later compute the flexural rotation of the pile once the strain gauges had yielded. With this method, rather than obtaining the lumped rotation from the difference between the total rotation and the flexural pile rotation, the flexural pile rotation would be calculated from the difference between the total rotation and the lumped rotation. Since Krypton targets were available throughout testing, a method was developed using the LEDs to directly obtain the lumped connection rotation. The lumped rotation from the Krypton LEDs was calculated as shown below in Equation (8.1).
In Equation (8.1) the second term takes the average flexural curvature between the Krypton LEDs at row H to the interface and converts it to an average flexural rotation over the entire length of the pile from the embedded end (2 in. (50 mm) into the wharf deck) to the Krypton LEDs at height H. Then the connection rotation is taken as the difference between the total rotation of the Krypton LEDs at row H and the average flexural rotation of the pile. This method was conducted using LEDs at heights starting from 4 in. (102 mm) on up to 24 in. (610 mm), and the results were all very similar. Also, when compared to the lumped rotation calculated as described previously from the unyielded T-headed dowel bar strain gauges, the results also matched reasonably well. Since the method described by Equation (8.1) could continue to provide a close approximation of the lumped connection rotation even after the strain gauges yielded, it was the preferred method for use later in the test. Figure 8-14 displays a comparison of the lumped rotation of the connection calculated from the strain gauge curvatures and also from the Krypton LEDs, as described in Equation (8.1). It can be seen that the results align reasonably well, and that the method described using the Krypton LEDs to calculate the lumped rotation of the connection is a viable option for use during the post-yield region of the dowel bar strain gauges.

\[
\theta_{\text{CONNECTION}} = \theta_{\text{KRYPTON}@H} - \left( \frac{\theta_{\text{KRYPTON}@H} - \theta_{\text{KRYPTON}@\text{INTERFACE}}}{H} \right) \cdot (H + 2)
\]  

(8.1)
With the various contributions to specimen rotation near the connection interface now separated, the curvature response can be calculated. The general flexural curvature response of the pile was determined from stain gauges located throughout its height. While it was not an issue during the Imperial Valley loading, in later earthquakes when the strain-gauged dowel bars had yielded an effort was still made to continue using the strain gauges to calculate the curvature, by removing the plastic strain from the readings. Since the earthquake records each possessed small cyclic displacements over a portion of the record, the stain gauges were offset by the strain about which the gauges cycled in the post-yield regions. This will be discussed further in the next sections.
Using the material properties of the pile and the reinforcement described in detail in Section 7.3.4, a model of the pile cross-section was made in the sectional analysis program *Response-2000*. Figure 8-15 shows the moment-curvature behavior of the pile at the interface during testing compared to the predicted sectional response from *Response-2000*. Only the dowel reinforcing bars were included in the *Response-2000* model to represent conditions at the pile-wharf interface. It should be noted that the inclusion of the effect of the prestressing strands away from the interface would improve the capacity of the pile. Reviewing the figure, it is not surprising that the specimen was still linear elastic and relatively undamaged throughout the duration of the Imperial Valley loading since the maximum moment achieved during this portion of the loading was such a small percentage of the moment capacity.
As briefly described previously, the dense grid of Krypton LEDs allowed for determining a fairly full displacement field of data during testing. Using the position of each LED as the nodal displacement during a given load step, it was possible to create a “finite element grid” using planar elements. Admittedly, the specimen was not entirely planar; however, the loading and response of the specimen was essentially in a single plane, so these results provide valuable insight into the strains in the structural concrete specimen, as well as about locations of localized damage. Figure 8-16 shows the Krypton vertical strain and finite element mesh from the data analysis at the maximum displacement of 0.129 in. (3.3 mm) during the Imperial Valley loading. Each “finite element” possesses four Gauss points, and the strains reported at each Gauss point
are represented by a single element in the mesh. Thus, a square bounded by four LEDs in the experiment is represented by four elements in the mesh, and the color map indicates the strain for that particular Gauss point. To include the results of the lowest level of LEDs, artificial stationary LEDs were placed into the model directly below the interface LEDs at the end of the pile at a location 2 in. (50.8 mm) into the wharf deck. It can clearly be seen that strains are localized at the base of the specimen, as found previously through other analyses of the instrumentation data. Additionally, the strain gauges are displayed in Figure 8-16. Since several locations have two or more strain gauges, the values shown in the color map for a given strain gauge are taken as the average of the strain values at the location, unless the strain gauges were damaged. (It was found during post-processing that the strain gauges located on the north side of the specimen at 12 in. (305 mm) into the wharf deck were damaged during construction of the wharf deck. The result is that the strain gauges at this location are displaying an artificially high strain, which is shown as red in Figure 8-16.) The capability to capture such a vast displacement field throughout testing, and to process it in a meaningful and visual way, lends itself to a deeper understanding of the behavior of the entire specimen during loading, especially with respect to the typical instrumentation limitations of previous experiments.
The next section will outline the behavior and response of the pile-wharf specimen during the 10% in 50 year event (the Northridge earthquake), which was used during testing immediately after Imperial Valley.
8.3  *Northridge Earthquake*

The second earthquake record imposed on the pile-wharf specimen was the Northridge earthquake (obtained from the SAC suite of records). By design, this record was selected with the intention of yielding some of the reinforcement after entering the cracked elastic range of behavior; however, a significant residual capacity was expected. During this record, a maximum displacement of 1.687 in. (42.8 mm) was imposed on the specimen, which is about 13 times larger than the demand imposed during the Imperial Valley loading. During this largest positive displacement excursion, the north extreme dowel bars experienced first yield at a displacement of 0.813 in. (20.7 mm). The maximum negative displacement during Northridge was 0.715 in. (18.2 mm), which is about 5.5 times larger than had previously been imposed in that direction. Figure 8-17 shows the lateral displacements imposed during the Northridge loading; again the same ratio of lateral displacement to Y-rotation at the control point in the pile was enforced.
The next sections will outline the visual damage, as well as the local damage and overall structural behavior, observed during this portion of the loading regime.

8.3.1 Northridge – Visual Damage

The first visible signs of damage to the specimen occurred during the course of the Northridge loading. The first visible crack in the pile occurred at step 331, which was the first negative displacement of the specimen during this ground motion. The crack was located about 12 in. (305 mm) from the pile-wharf interface, and it can be seen in Figure 8-18. It should be briefly noted that two step-numbering conventions exist. During testing, the initial 17 steps (where the axial load was incrementally applied) were not considered when labeling cracks on
the specimen. Thus, when comparing the displacement records versus step number in this document to the step number labeled on a crack, the actual step during the earthquake loading will be 17 steps greater than seen on the crack label.

![Figure 8-18. Initial Cracking of Pile Specimen (Viewed from the Northeast)](image)

During the next large excursion (in the positive direction), on the way to the peak displacement of 1.055 in. (26.8 mm), a series of cracks developed along the height of the specimen. Additionally, at a displacement of 0.813 in. (20.7 mm), the north extreme dowel bars experienced first-yield, and initial spalling of the south face was evident. Flexural cracks opened at several discrete locations along the height of the pile, and the two largest cracks were at approximately 12 in. (305 mm) and 24 in. (610 mm) up from the pile-wharf connection and are shown in Figure 8-19.
Smaller flexural cracks also formed farther up the height of the pile at approximately 30 in. (760 mm) and 42 in. (1.07 m) from the pile-wharf connection interface. These cracks can be seen in Figure 8-20.
In addition to the flexural cracks at first yielding of the reinforcement, initial spalling occurred on the south face of the specimen, and that can be seen on the left in Figure 8-21. Initial deck cracking also became apparent at this step around the interface between the precast pile and the wharf deck, as well as on the west side of the wharf. On the right in Figure 8-21, deck cracking can be seen. The reason for the discontinuity in the crack is due to the presence of the steel bar chair on the surface of the wharf deck. During construction of the wharf deck, the specimen was inverted from the testing position, and steel wire bar chairs were used to ensure that the wharf deck reinforcing cage would be located at the proper height within the wharf deck. Over the course of testing, it was evident that the bar chair steel was providing some modest confinement / crack bridging capacity to the wharf deck, so while the wharf did crack, some local deck spalling later in the test could have been more substantial without the presence of the bar chairs.

Figure 8-21. Initial Spalling at South Face of Pile-Wharf Connection (left) and Wharf Deck Cracking (right) during Northridge

Figure 8-22 shows the images from the fixed cameras on the north and south sides of the pile-wharf connection at first yield. On the north side, flexural cracks are visible at about 12 in.
(305 mm) and 24 in. (610 mm) up from the interface, and on the south side an existing flexural crack at 12 in. (305 mm) and some minor spalling are visible.

![Image](image-url)

**Figure 8.22. North (left) and South (right) Views of Pile-Wharf Connection at First Yield during Northridge**

After the large positive displacement, the next displacement reversal was in the direction of maximum negative displacement during the Northridge earthquake. Although the maximum negative displacement of 0.715 in. (18.2 mm) was about 5.5 times larger than that experienced during the Imperial Valley loading, it was still not large enough to cause the south extreme dowel bars to yield. Since crushing occurred on the previous positive displacement excursion, during this displacement reversal some of the cracked concrete on the south face became loose and was removed from the specimen. Additionally, further wharf deck flexural cracks were visible upon the change in displacement direction. Figure 8-23 shows the cracking at the pile-wharf interface from the west and the south sides of the specimen.
Figure 8-23. Cracking at Pile-Wharf Interface during Northridge Maximum Negative Displacement

The flexural crack that first occurred on the south side of the pile at around 12 in. (305 mm) continued to grow on the way to the maximum negative displacement, almost reaching the center of the west and east faces, as shown in Figure 8-24.

Figure 8-24. Growth of Pile Flexural Crack at 12" from the Pile-Wharf Interface on the South Face during Northridge
The flexural crack at 12 in. (305 mm) along the pile continued to noticeably open during the maximum negative displacement, and Figure 8-25 shows that a maximum crack width of about 0.01 in. (0.4 mm) was observed.

Figure 8-25. Flexural Crack at 12" from the Pile-Wharf Interface at Maximum Negative Displacement during Northridge

In addition to the growth and opening of the crack at 12 in. (305 mm), flexural cracks were observed at 24 in. (610 mm), 42 in. (1.07 m), and about 54 in. (1.37 m) up from the interface on the south face of the specimen. On the left of Figure 8-26, the cracks at 24 in. (610 mm) and 42 in. (1.07 m) can be seen, while on the right the cracks at 42 in. (1.07 m) and 54 in. (1.37 m) can be observed.
Figure 8-26. New Flexural Cracks along South Face of Specimen during Northridge Maximum Negative Displacement

To maintain a standard basis for comparison of the progression of damage throughout the remainder of testing, Figure 8-27 shows the north and south views of the connection from the fixed cameras at the maximum negative displacement during the Northridge record.

Figure 8-27. North (left) and South (right) Views of the Connection at Maximum Negative Displacement during Northridge
Following the maximum negative displacement, the next displacement reversal went to a positive 1.687 in. (42.8 mm). Since this was about 60 percent larger than the previous maximum positive displacement, which resulted in yielding of the north extreme dowel bars, moderate damage was expected.

Moderate deck cracking was observed during the reversal of displacement. This phenomenon was consistent throughout the test – after a large excursion in one direction, upon unloading and displacement reversal new deck spalling was visible. While only a small part of the deck spalled during this phase of testing, cracks were visible the wharf deck about a foot away from the north face of the pile. Deck spalling can be seen from the east and west views in Figure 8-28.

![Deck Cracking and Spalling at Maximum Displacement during Northridge](image)

Figure 8-28. Deck Cracking and Spalling at Maximum Displacement during Northridge

Beyond deck spalling, the large positive displacement also caused the formation of a discrete flexural crack near the top of the specimen, at about 66 in. (1.68 mm) from the pile-wharf interface, as well as growth and opening of pre-existing cracks. The new flexural crack can be seen on the left in Figure 8-29. On the right of the figure, expansion of the flexural crack

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located at 12 in. (305 mm) up from the interface on the north face of the pile can be seen to have opened to a width of about 0.05 in. (1.3 mm).

Figure 8-29. New Flexural Crack at 66" (left) and Widening of Crack at 12" (right) during Peak Northridge Displacement

The final major physical change in the specimen during the Northridge record was the progression of damage to a moderate spalling state on the south face of the pile. The small spalled region on the south face grew to a rather substantially sized area during loading and eventually fell off of the specimen. Figure 8-30 shows the growth of the damaged region on the left, and the eventual spall on the right. After the Northridge record, the spalled region was measured and found to have a height of approximately 8 in. (203 mm) and a width of about 7 in. (178 mm), as seen in Figure 8-31.
Figure 8-30. Progression of South-Face Pile Spalling during Maximum Northridge Displacement

Figure 8-31. Measurement of Spalled Pile Region after Northridge
Figure 8-32 displays the north and south faces of the pile-wharf connection from the fixed cameras after the Northridge event, and Figure 8-33 shows a zoomed in view from the west side of the specimen after Northridge. These figures continue to provide a benchmark for the progression of damage during the test, and it can clearly be seen that Northridge caused new cracks to form, existing cracks to grow, and some moderate spalling on the south pile face.

Figure 8-32. North (left) and South (right) Views of the Pile-Wharf Connection after the Northridge Record

Figure 8-33. West View of the Pile-Wharf Connection after the Northridge Record
The Northridge event clearly caused some visible physical damage to the specimen, and the next section will investigate in more detail the local behavior of the specimen as measured by the dense array of instrumentation.

8.3.2 Northridge – Local Behavior

An analysis of the behavior of the test specimen through the Northridge record shows that there was indeed some overall softening and damage, but a majority of the rotation continued to occur at the pile-wharf connection.

Since the north strain gauges began to yield during the Northridge record, an effort was made to adjust the strain gauge values (for purposes of estimating relative rotation, actual stress, etc.) by the estimated plastic strain. This was obtained by reviewing the strain gauge values during the small displacement reversals in the latter part of the Northridge record, and obtaining the plastic offset value. The unadjusted strain gauge readings of the north and south extreme dowel bars at the interface are shown on the left in Figure 8-34, and the strain gauge values with the plastic offset are displayed on the right. It can be seen that, with the adjustment, the strain values cycle around zero strain when the loading is at zero.
With the adjusted strain gauge values, it was possible to continue computing the flexural rotation of the pile at the interface. Again using the Krypton readings to measure total average curvature, the lumped rotation at the connection was computed as the difference between the Krypton total rotation at 2in. (50.8 mm) up the pile and the pile flexural rotation at the interface computed from the strain gauges. The total rotation and the pile rotation versus step number during the Northridge record is plotted in Figure 8-35.
Since it was necessary to adjust the strain gauge values to obtain the average pile rotation at the interface, an alternative calculation of the lumped connection rotation using only Krypton LEDs was also performed, using Equation (8.1). In theory, the lumped rotation at the connection should be a fixed value, so if the method is a reasonable way to compute the connection rotation, then any row of Krypton LEDs should yield the same lumped connection rotation result. In Figure 8-36, the connection rotation was calculated by using Krypton LEDs at every row from 4 in. (102 mm) to 30 in. (762 mm) up from the connection interface. For comparison, the lumped rotation calculated from the difference between the Krypton targets and the pile flexural rotation from the strain gauges was also included. Using the Krypton LEDs did indeed produce a very

Figure 8-35. Pile Flexural Rotation and Total Rotation at the Connection during Northridge
consistent result for the lumped connection rotation, and it can be seen that, before the strain gauges yield at around step 353, the strain gauge calculation aligned well with the Krypton result. Given this, it seems that once the strain gauges have yielded significantly (and therefore the plastic offset approach may no longer be valid or easily applied), the Krypton approximation of the lumped rotation can be an adequate measure.

Figure 8-36. Lumped Connection Rotation from Krypton during Northridge

Figure 8-37 shows a plot of the flexural pile rotation versus the total pile rotation after the large excursions at the beginning of Northridge. Previously, it was found that slightly more than 70% of the total rotation could be attributed to connection rotation during the Imperial Valley loading, but after damage to the pile and some yielding of the dowel reinforcement, the amount of the total rotation attributed to the lumped connection behavior increased to about 85%.
Stringer (2010) reported that the piles with a 405 kip (2000 kN) axial load and bearing pad exhibited the same behavior; initial lumped connection rotation about 75% which increased to as high as 95% as damage progressed. A best fit line representing a 14% flexural pile rotation contribution is displayed in Figure 8-37 for reference.

![Figure 8-37. Pile Rotation versus Total Rotation during Northridge](image)

Since the strain gauges were adjusted after yield to allow for continued use of them for the curvature computation at the connection, the interface moment versus interface curvature plot shown in Figure 8-38 includes a bold portion before the plastic offset adjustment, as well as the moment-curvature response using the strain gauges with a plastic offset. For comparison to the sectional analysis from *Response-2000*, the predicted moment-curvature response is also
included. It can be seen that the peak moment aligns quite well with the predicted value when the appropriate experimentally obtained material properties are used in the analysis.

Figure 8-38. Base Moment versus Curvature through Northridge

The moment-curvature responses at 6 in. (152 mm) and 12 in. (305 mm) from the interface are shown in Figure 8-39 on the left and right, respectively. The curvatures were obtained from unadjusted strain gauge values at those locations, and again, the Response-2000 result was included for reference. When comparing the moment-curvature responses at the connection to further up the pile height, the curvature and inelastic behavior is largely limited to the connection. While there were some cracks evident in the specimen along the height, the
The majority of the visual damage did confirm that the main deformation was occurring at the interface.

![Graph showing Moment-Curvature Response at 6" (left) and 12" (right)]

Figure 8-39. Moment-Curvature Response at 6" (left) and 12" (right)

The vertical strains throughout the specimen from the dense Krypton displacement field are shown in Figure 8-40 on the deformed shape of the specimen. Also, the strain gauges are shown with a color mapping to indicate the level of strain at each location. The figure on the left shows the deformed configuration of the test specimen at the point of first yield of the north dowel bars at the interface. Around 12 in. (305 mm) from the interface, the Krypton strains are also indicating the opening of a crack, which is representative of what was observed during testing. On the right, the test specimen is shown at the peak negative displacement. The south strain gauges at the interface had not yet yielded, but a crack was opening on the south face of the pile around 12 in. (305 mm) from the interface. This was also confirmed visually.
Figure 8-40. Krypton and Strain Gauge Strains at First Yield during Northridge

Figure 8-41 shows a similar result as the previous figure; however, the results are presented from the peak Northridge displacement. This step also coincided with the maximum moment applied throughout the entire testing regime. Here it can be seen that the north extreme and north center dowel bars have yielded at the interface and at a location 6 in. (152 mm) into the wharf deck. The crack at 12 in. (305 mm) can be seen to opening and growing slightly from that shown during the first yielding of the dowel reinforcement. Overall, the Krypton system has proven to be very useful once the traditional instrumentation begins to fail during the test, and for the visualization of the overall state of the specimen throughout testing, rather than just localized readings.
The next section will provide an overview of the response of the pile-wharf connection specimen during the 2% probability of exceedance in 50 years event: the Kobe earthquake.

8.4 **Kobe Earthquake**

The final earthquake record imposed on the specimen was the Kobe earthquake. This record had a 2% probability of exceedance in 50 years, and it was selected with the intention of causing some substantial spalling, strain hardening of the reinforcement, and perhaps bringing the specimen to the brink of failure. As will be discussed in the following sections, the test specimen exhibited a substantially more ductile and resilient behavior than anticipated from the
literature and preliminary analyses, so a cyclic regime followed this final earthquake record. During the Kobe displacements, a maximum displacement of 3.535 in. (89.79 mm) was imposed in the negative direction. This amounts to almost 5 times the previous maximum displacement. Also, during that large negative displacement, the south extreme dowel bars experienced first yield at a displacement of 2.35 in. (59.7 mm). In the positive direction, the specimen was subjected to a maximum displacement of 2.058 in. (52.3 mm), which was only about 1.2 times larger than experienced during the Northridge record in that direction. Figure 8-42 shows the lateral displacements imposed on the control point of the specimen during the Kobe record, and once again, the same displacement-rotation ratio was maintained throughout the earthquake loading.

![Kobe Loading](image)

**Figure 8-42. Kobe Earthquake Lateral Displacement Record**
The following sections will provide an overview of the visual damage and an analysis of the data acquired during the Kobe earthquake record.

8.4.1 Kobe – Visual Damage

Not long after the Kobe record begins, a large negative displacement, over five times larger than previously experienced in that direction, occurs. On the way to that large negative displacement, the south dowel bars yield for the first time. The yield occurs around step 716 at a displacement of 2.35 in. (60 mm). This was much larger than the displacement at first yield for the north extreme dowel bars, and this can be attributed to the accumulation of damage to the specimen and the allowed growth of the pile specimen to maintain a constant axial force.

The Northridge earthquake had the largest displacement in the positive direction, which demonstrated moderate spalling of the pile, and some cracking of the wharf deck on the south face. As the displacement in the opposite direction began to reach substantial levels during the Kobe record, the wharf deck on the south side (tension side) began to uplift, and moderate deck spalling was observed. The width of the spalled region was about 12 in. (305 mm), and its depth went to the bottom of the embedded pile at about 2 in. (50 mm). The progression and spalling and the width of the spalled region on the south side of the pile can be seen in Figure 8-43. In Figure 8-44, it can be clearly seen that the pile deck spalling went to the bottom of the embedded pile, and on the right, the uplift was measured to be around 0.5 in. (12 mm) at the connection. This lumped rotation phenomenon is consistent with the previous findings, but with the deck concrete removed, the concentration of rotation at the pile-wharf interface is visually apparent.
In addition to the deck spalling on the south side of the specimen, the pile began to crush on the north face, and some moderate spalling of the cover concrete was observed. A height of about 5 in. (127 mm) spalled along the pile, and a region of the wharf deck to about 9 in. (230 mm) from the pile face was observed to have substantial cracking. The spalled region can be seen in Figure 8-45 on the left, and the damaged deck can be seen on the right.
At the peak negative displacement, the rotation at the pile-wharf interface was so pronounced that a ruler was slid under the south face of the pile. The standard ruler was able to extend 7 in. (178 mm) under the pile before encountering an obstruction. This can be seen in Figure 8-46, and it is further evidence of the large concentration of the rotation at the pile-wharf connection.
Upon the reversal of the displacement, uplift similar to previously seen after a large displacement caused the large cracked portion of the wharf deck on the north side of the pile to spall. The region before and after spalling can be seen in Figure 8-47. The depth of the spall went all the way to the bottom of the embedded pile, and its dimensions were about 12 in. (305 mm) wide and 10 in. (254 mm) out from the north face of the pile.

![Figure 8-47. Deck Spalling on North Side of Pile during Northridge](image)

During the Kobe loading, a relatively stable crack pattern had developed. Some of the existing cracks extended somewhat, but largely the deformation was accommodated through opening of existing cracks and lumped rotation at the connection. The main visual damage that was observed was moderate spalling of the north face of the pile, and substantial spalling of the wharf deck on both the north and south faces to a depth equal to the length of embedment of the pile. Figure 8-48 and Figure 8-49 show the north and south views of the pile-wharf connection early in the Kobe record during first yield of the south extreme dowel bars and after the record is complete, respectively. From the north side of the connection, moderate pile spalling and
substantial wharf deck spalling was evident. On the south view, some additional wharf spalling can be seen, but essentially no new cracks formed and the pile did not spall further.

![Image](image1)

**Figure 8-48. North and South Views of the Pile-Wharf Connection at Initial Yield of the South Extreme Dowel Bars during Kobe**

![Image](image2)

**Figure 8-49. North and South Views of the Pile-Wharf Connection after Kobe Record**

From the west view of the pile, the spalling of the wharf deck can be seen to have extended beyond just the north and south regions of the pile and continued around to the west
face; however, few new cracks can be seen to have originated during the Kobe record, as shown in Figure 8-50.

Figure 8-50. West View of the Pile-Wharf Connection after Earthquake Loading

The following section will provide an overview of an analysis of the data acquired during the Kobe portion of the earthquake loading.

8.4.2 Kobe – Local Behavior

During the Kobe earthquake loading, many of the trends experienced during the Northridge earthquake continued; namely, the majority of the increased deformation was accommodated through lumped rotation at the connection and the opening of existing cracks in the pile. In Figure 8-51, the total rotation of the specimen and the average flexural rotation of the pile measured by the strain gauges at the connection are shown during the Kobe record. Again, it can be seen that the flexural rotation experienced by the pile is relatively small with respect to
the total rotation of the specimen. The difference between these two values is the amount of lumped rotation that was experienced in the pile-wharf connection.

![Graph](image)

**Figure 8-51. Total Rotation and Flexural Pile Rotation at Pile-Wharf Connection during Kobe**

Since the strain gauges began to yield in the connection, as during the Northridge record, the Krypton LEDs were used to measure the lumped rotation at the connection. Again, since the value of the lumped rotation is fixed, using the Krypton LEDs at various heights should yield that same result for the connection rotation. In Figure 8-52, the connection rotations measured from the Krypton LEDs is compared to the average lumped rotation computed from the strain gauges at the connection. As expected, using the Krypton LEDs at heights ranging from 4 in. (102 mm) to 30 in. (762 mm) gave the same estimation for connection rotation. It was shown
previously that once the strain gauges began to yield, they slightly underestimated the lumped rotation relative to the Krypton calculation, and this trend continued throughout the Kobe loading.

![Figure 8-52. Lumped Rotation at Pile-Wharf Interface from Krypton LEDs and Strain Gauges during Kobe](image)

The pile flexural rotation versus the total rotation at the pile-wharf connection is plotted in Figure 8-53. Additionally, a best fit line with a slope of 15% is included. This indicates that about 85% of the total rotation is due to a concentrated connection rotation, which was similar to the value obtained during the Northridge earthquake after yielding of the extreme dowel bars. Before the dowel bars yielded, slightly more than 70% of the total rotation was attributed to the lumped connection rotation. Thus, the yielding of the dowel bars at the interface introduced a
plane of significantly reduced stiffness and altered the relative distribution of the applied rotation to be larger localized at the end of the pile.

Figure 8-53. Pile Rotation versus Total Rotation at the Connection throughout Kobe

Upon an analysis of the data after the earthquake records, the rotations and curvatures in the pile never reached substantial values, so plots of pile rotation versus the various displacement or force measures did not have any discernible trends. However, a clearly linear relationship could be seen between the x-displacement and the lumped connection rotation, and since the applied y-rotation was by definition a multiple of the x-displacement, a similar linear relationship was identified. Figure 8-54 shows the relationship between the x-displacement and the
connection rotation, as well as between the applied y-rotation and connection rotation. A best fit lines are included on each figure, and the relationships relative to the lumped rotation were found to be 99 in. per radian and 0.99 radians per radian for the x-displacement and y-rotation, respectively.

Figure 8-54. X-Displacement (left) and Y-Rotation (right) versus Connection Rotation during Kobe

A procedure similar to the post-yield Northridge record was conducted on the interface strain gauges for the Kobe record. The south extreme dowel bars experienced yielding during the large negative displacement, and the plastic offset of the strain gauge was identified during the small cycles later in the record. The resulting adjusted strain gauges values are displayed in Figure 8-55, and these values were then used obtain the curvature of the specimen at the pile-wharf interface.
The moment-curvature response of the specimen was going to be one of the key elements in deciding whether the specimen had experienced significant enough damage to warrant the end of the test and the beginning of the repair phase. Beyond visual damage and spalling, it was desired to have an actual decrease in moment capacity and an obvious degradation in strength. Figure 8-56 shows a plot of the moment-curvature response at the pile-wharf connection through the earthquake loading. A bold line is shown to indicate the response before the Kobe plastic offsets were applied to the strain gauges, while the lighter plot shows the adjusted moment-curvature behavior during Kobe. When comparing the behavior to the Response-2000 sectional prediction, it can be seen that the values of the moment capacity.
aligned rather well in both directions with the predicted moment capacity. In fact, in the positive
direction, the curvature values also align; however, in the negative direction, substantial spalling
and the use of adjusted yielded strain gauges gave larger curvatures than predicted. Also, since
the moment capacity did not experience a marked decrease, an additional reverse cyclic loading
scheme was implemented.

![Response through Kobe Loading](image)

**Figure 8-56. Moment-Curvature Response at Pile-Wharf Connection through Earthquake Loading**

The moment-curvature response further along the height of the pile was also investigated,
and Figure 8-57 shows the result at 6 in. (152 mm) and 12 in. (305 mm) from the pile-wharf
interface. Throughout the Northridge record, the pile was elastic at these locations; however,
some inelastic behavior can be seen at the 6 inch (152 mm). Although the damage is still largely
localized at the connection, this is an indication that damage has spread up the height of the pile somewhat. At the 12 inch (305 mm) location, the pile was still essentially elastic.

Figure 8-57. Moment-Curvature Response at 6" and 12" from Interface through Earthquake Loading

The final location for the moment-curvature response that was examined was at 24 in. (610 mm) from the interface. Since most of the spalling and large crack openings were below this location, it is not surprising to see that the moment-curvature shown in Figure 8-58 resembles elastic behavior.
The vertical strains from the Krypton LED displacement field and the dowel bar strains are displayed for the peak negative displacement and peak positive displacement during Kobe in Figure 8-59. During the peak negative displacement, the magnitude of the displacement compared to the undeformed position is apparent. Also, the spalling can be seen on the south west face since the Krypton LEDs were reglued to the specimen after the spalling event. Since the reglued LED targets are no longer in the same location, the strains measured relative to the undeformed configuration are no valid. For the strain gauges, it can be seen that at the negative peak, the center and extreme south dowel bars had yielded at the interface and both 6 in. (152 mm) into the wharf deck and 6 in. (152 mm) into the pile. At the maximum positive
displacement later in the Kobe record, all of the strain gauges at the interface and 6 in. (152 mm) into the wharf deck have yielded. In both of the figures, the crack opening on the tension face of the pile at 12 in. (305 mm) from the interface is evident from the Krypton strains.

![Figure 8-59. Krypton and Strain Gauge Strains at Peak Negative and Positive Displacements](image)

Since the earthquake record portion of the loading did not provide substantial enough damage to warrant the conclusion of the test and the need for a repair, a cyclic loading protocol was instituted. This loading will be discussed in the following section.

### 8.5 Cyclic Loading

A reverse cyclic loading regime was pursued at the conclusion of the earthquake loading due to further damage the specimen and to obtain a better understanding of the strength and
ductility of the pile-wharf connection test specimen. The earthquake loading had an essentially linear relationship between the imposed x-displacement and y-rotation with a value of 72.6 in. per radian. Since it was observed that the rotations at the control point that were achieved during the earthquake loading were slightly less than the desired input, a 15% amplification factor was applied to the rotations during the initial cycles. Later during the cyclic testing, large cracks began to form and open toward the top of the pile, so this amplification factor was removed and even reduced to 95% to prevent an undesirable failure location.

The cycles that were selected were 2%, 3.5%, 5%, 6.5%, and 8% drift values based on the 84 inch (2.13 m) height from the pile-wharf interface to the control point. Admittedly, percent drift may not be the appropriate term due to the applied y-rotation at the control point; however, for the sake of clarity when referencing the specific cycles, the term percent drift was used. The first 2% cycle had an x-displacement of 1.68 in. (42.7 mm), and it was selected with the intention to have a base line value for the strength and stiffness of the structure since that displacement value was less than previously experienced by the specimen in each direction during the earthquake loading. The subsequent 3.5% cycle had an x-displacement of 2.94 in. (74.7 mm), and during this level of displacement, some minor additional damage was expected. The displacement was less than previously experienced in the negative direction; however, it was almost an inch greater than any previously imposed displacement in the positive direction. Two cycles were conducted at this level. The initial plan following this step was to conduct a 5% drift cycle; however, some concern arose regarding the response of the specimen and whether the LBCB was properly behaving. It was found that one of the actuator load cells was not properly working, so this corrected. Through post-processing and equilibrium calculations, the X1 load cell values were able to be reliably computed throughout steps where the load cell was
malfunctioning. When testing resumed, repeat cycles were conducted at 2% and 3.5% to ensure that all of the equipment was properly functioning. Due to concern about large cracks near the top of the specimen, the 15% rotation amplification was removed. After reviewing the results, testing proceeded with the 5% cycle, which had an x-displacement of 4.2 in. (106.7 mm). The 5% cycle also did not have the 15% rotation amplification. The test concluded with two final cycles of 6.5% and 8%; these cycles had peak displacements of 5.46 in. (138.7 mm) and 6.72 in. (170.7 mm), respectively. These displacements were substantial, and it was found that the specimen was nearly in reverse curvature during the previous cycles. To mitigate rotation demands at the top of the specimen, the rotation was reduced from the defined relationship by 5% for the final two cycles. While it is unclear whether this measure was necessary, the specimen did not fail and was ready for the repair phase upon the conclusion of testing. The applied x-displacement history for the cyclic loading portion of the test can be seen in Figure 8-60. The slight change in slope is due to the displacement increments varying from 0.1 in. (2.5 mm) to 0.15 in. (3.8 mm) per step. For the 2% cycles and for new displacement levels, the smaller increment was used, while the larger increment was selected for the larger cycles and for displacements within the previously imposed range of motion.
The pile-wharf connection test ended with the conclusion of the 8% cycle because it was determined that the specimen was adequately damaged to warrant a repair and the maximum extension of the x-actuators in the LBCB was an additional limiting factor. The following sections will outline the visual damage and the local response of the specimen during the cyclic loading regime.

8.5.1 Cyclic – Visual Damage Progression

The cyclic test commenced with the 2% cycle to establish a baseline to observe the reduction of stiffness in the specimen since the beginning of the test. Since this displacement was less than that to which the specimen had been previously exposed, no further damage could
be seen. The subsequent cycle was to 2.94 in. (74.7 mm), and while this was less than the peak negative displacement during the Kobe record, it was almost 50% greater than the previous maximum positive displacement. The negative direction was selected to be first for each cycle because of the asymmetry of the earthquake loading. This was done in an effort to get the maximum amount of load steps within the previous range of deformation before imposing new demands on the test specimen during the cyclic loading.

Some new crack growth and spalling was observed during the 3.5% cycle. Although the negative 2.94 inch (74.7 mm) displacement was less than during the Kobe record, the north face of the pile experienced some additional spalling at a previously cracked portion of the pile. The pile can be seen in Figure 8-61 before and after the spalling was removed.

![Figure 8-61. North Face of Pile Spalling at -3.5% Drift](image)

As the displacement reversed and approached a positive 3.5% drift, this marked the first time during the loading that the specimen was being subjected to a greater deformation than during any of the previous displacements throughout the earthquake loading. It would seem logical to expect some more spalling of the south face of the pile, but since a large portion of the
wharf deck adjacent to the pile had been removed due to substantial spalling and since the lumped rotation at the connection limited the curvature experienced by the pile, no additional pile spalling was seen. However, the wharf deck on the north (tension) side of the pile showed visible signs of spalling. In some locations, the wharf deck could not be removed because the bar chairs used to position the wharf reinforcing cage held the concrete in place, but tapping on the damaged portion of the concrete acoustically revealed that it was no longer structurally contributing to the specimen. The spalled region of the wharf deck is shown in Figure 8-62. On the left, the growth of the wharf deck spalled region to the north of the pile can be observed, and on the right, a close photograph of the confined spalled region can be seen. The confined spalled region was completely disconnected from the wharf deck, but due to the manner in which the specimen was constructed, it could not be removed from the specimen. As the specimen became more damaged and a greater portion of the surrounding concrete spalled, this region was removed; however, for documentation purposes, it should be considered to have been completely spalled at this point in the test.
A second cycle at 3.5% drift was conducted during the cyclic portion of the test. Two cycles at a given drift level is a relatively common testing protocol, but it was deemed unnecessary to conduct a second cycle for the 2% drift level since the specimen had already completed several cycles at that displacement during the earthquake loading. At the peak negative displacement during the second cycle, no further damage was noticed, but the flexural crack located 12 in. (305 mm) from the interface on the south side of the pile could be seen to have extended around the entire tension face and was opening to about 0.5 mm. Figure 8-63 on the left shows the south side of the pile during the peak negative displacement, and the image on the right shows a closer view of the crack at 12 in. (305 mm) from the interface with a crack gauge for comparison.
Upon reversal of the loading, no further damage was experienced by the pile at the peak positive displacement, but the previously confined spalled region on the wharf deck by the north face of the pile was removed. With the additional cycle, some further damage was experienced by the wharf deck, and this damage made it possible to remove the spalled region. The cracked region is shown before and after removal at the peak positive displacement of the second 3.5% cycle in Figure 8-64.
At the conclusion of the two 3.5% drift cycles, the next planned level of deformation was to be two 5% cycles. Several days elapsed before the 5% cycles commenced, and when the cycles started, some anomalous behavior was observed. It could be seen that localized cracks were opening near the top of the pile, and it was noticed that the X1 actuator load cell was not reading properly. At this point, testing came to a halt and troubleshooting began. The problem with the X1 load cell was remedied, and through post-processing and equilibrium calculations, it was found that the proper X1 load cell value could be computed. When testing resumed, the cyclic loading was adjusted, and it was desired to conduct a single cycle at both 2% and 3.5% to ensure that specimen and LBCB were behaving as expected. Additionally, the previous cyclic loading used a 15% amplification on the rotation to ensure that the proper rotation was being achieved at the control point. Upon post-processing the data obtained during troubleshooting, it was found that the 15% amplification was no longer needed to obtain the desired rotation at the control point, so in the interest of not inducing a failure at the top of the specimen, the 15% amplification factor was removed for all subsequent rotations. Thus, the applied displacement-rotation ratio was 72.6 in. per radian. The 2% and 3.5% cycles were conducted and the damage and crack growth was monitored; however, no new significant visual damage was observed during these cycles, as expected.

With the instrument issues resolved, and the 2% and 3.5% levels of displacement repeated to satisfaction, the next cycle conducted was the 5% drift. The 5% drift corresponded to 4.2 in. (107 mm), and this distance was substantially larger than previously imposed on the specimen in either direction. Given the large increase in displacement, it was expected that damage would propagate throughout the specimen; however, additional spalling of the pile was not apparent and only slight growth of existing cracks occurred. It seems that the additional
displacement and rotation was accommodated largely through the opening of existing cracks and the lumped rotation at the connection. Figure 8-65 on the left and right show the views from a fixed camera on the west side of specimen at -5% and +5% drift, respectively. At this magnitude of deformation, it is now visually apparent the extent of displacement that is being imposed on the specimen. Additionally, it appears that the pile is relatively straight and the rotation is concentrated at the pile-wharf interface. Although this is what has been found throughout the analysis of the data, as the cyclic displacement increased, this behavior could be observed.

Figure 8-65. West View of Test Specimen at -5% (left) and +5% (right) Drifts
As a reference to the extent of damage at previous cycles, the next two sets of figures depict the north and south views of the pile-wharf connection from the fixed cameras. Figure 8-66 shows the connection at a -5% drift (toward the north), while Figure 8-67 displays images of the connection at a +5% drift (toward the south). No further spalling of the pile was observed during this cycle, and this may be a result of the amount of previously spalled concrete in the cover region of the pile and in the wharf deck adjacent to the pile.

Figure 8-66. North (left) and South (right) Views of Pile-Wharf Connection at -5% Drift

Figure 8-67. North (left) and South (right) Views of Pile-Wharf Connection at +5% Drift
The only real damage recognized during the 5% cycle was some moderate propagation of wharf deck damage around the pile toward the west and east sides. Over the course of the previous cycles, the wharf deck to the north and south of the pile experienced substantial spalling to a depth at least equal to the embedment of the pile, but during this cycle the damage was seen to begin spreading around to the sides of the pile. At this point, the uplift LVDTs would no longer provide reliable readings because the concrete underneath the LVDTs was damaged. As before, due to the bar chair used to elevate the wharf reinforcing cage, the damaged concrete could not be removed from the specimen yet; however, when tapping on the damaged region, it could be heard that the concrete was spalled. Photographs of the west view of pile-wharf connection at -5% and +5% drift are shown in Figure 8-68 on the left and right, respectively.

![Figure 8-68. West View of Pile-Wharf Connection at -5% (left) and +5% (right) Drifts](image)

To maintain the 1.5% drift increments, the next cycle conducted was to 6.5%, which corresponded to a displacement of 5.46 in. (139 mm) in each direction. Although the previous cycle was larger than had been previously imposed on the test specimen, the incremental
increase from 5% to 6.5% was considerable enough that some concern arose regarding the potential for a failure at the top of the specimen. Over the course of testing, some large cracks had developed in each side of the specimen around 66 in. (1.68 m) from the pile-wharf interface. To mitigate worry, it was decided that a 5% reduction of rotation would be implemented for the remaining two cycles. Thus, the displacement-rotation ratio was increased from 72.6 in. per radian to 76.4 in. per radian. It is unclear whether this modest change prevented an undesirable failure, but it did relieve some concern.

On the way to the negative peak displacement during the 6.5% cycle, prestressed strands and the extreme dowel bars were exposed for the first time on the south side of the pile. As the rotation continued to be concentrated at the connection interface, the wharf deck adjacent to the pile continued to spall and the extreme yielding of the dowel bars caused the specimen to grow along the axis of the pile to maintain a constant axial force. Due to the reference bar installed for the LVDT measurement, the view is somewhat obscured; however, the exposed prestressed strands and extreme dowel bars on the south side of the pile can be seen in Figure 8-69.

Figure 8-69. Exposed Prestressed Strands and Dowel Bars on South Side of Pile at -6.5% Drift
Also, at the peak negative displacement, the spalled region of the wharf had grown to width of about 24 in. (610 mm) and a length of about 9 in. (229 mm). The extent of the spalled wharf deck can be seen in Figure 8-70, and the image on the right shows that a tape measure was easily inserted about 3 in. (76 mm) under the pile due to the uplift caused by the rotation at the connection.

![Figure 8-70. Extent of Wharf Spalling on South Side of Pile at -6.5% Drift](image)

During the reversal of displacement toward the +6.5% peak displacement, additional spalling and opening of the north face of the pile similarly exposed the prestressing strands, transverse reinforcement, and extreme dowel bars. Photographs of the north face of the pile-wharf interface showing the exposed reinforcement, as well as the level of uplift and wharf damage can be seen in Figure 8-71. In the photograph on the right, the lumped connection rotation has again enabled a tape measure to be easily placed about 3 in. (76 mm) under pile, and the wharf deck spalling extends about 12 in. (305 mm) from the north face of the pile. A closer view of the exposed reinforcement can be seen in Figure 8-72.
The worrisome cracks near the top of the specimen began to open considerably during the 6.5% cycle. At the peak -6.5% drift, the crack at about 63 in. (1.6 m) from the interface on the south side of the pile opened to a width of about 1.5 mm, and at the peak +6.5% drift, the crack located at 66 in. (1.67 m) from the interface opened to a width of about 2 mm. Figure 8-73
identifies the location of the large cracks, while Figure 8-74 shows closer views of the two cracks at the peak 6.5% drifts.

Figure 8-73. Location of Large Cracks Near Top of Pile

Figure 8-74. Close View of Cracks on North (left) and South (right) near Top of Pile
Despite the large crack opening near the pile top and the exposure of the reinforcement, the spalling and level of damage of the pile near the connection remained relatively stable. The cracks located on either face around 12 in. (305 mm) continued to open considerably, but essentially no new pile spalling occurred during the 6.5% cycle. Figure 8-75 and Figure 8-76 show the north and south faces of the pile-wharf connection at -6.5% and +6.5% drift, respectively.

Figure 8-75. North (left) and South (right) Views of the Connection at -6.5% Drift

Figure 8-76. North (left) and South (right) Views of the Connection at +6.5% Drift
The final reverse cyclic displacement level was the 8% drift cycle. This corresponded to a displacement of 6.72 in. (171 mm). Also, due to the LVDT limits of the LBCB actuators, it was determined that if the specimen did not fail, the 8% drift cycle would mark the conclusion of the test. Considering the wide crack widths at the top of the pile during the 6.5% cycle, the same 5% rotation reduction from the previous cycle was implemented.

There was some modest crack propagation that occurred during the 8% cycles. During the -8% cycle, an additional flexural crack developed in the spalled concrete region of the south face of the pile at about 6 in. (152 mm) from the interface. This crack extended around the south face of the pile to each of the diagonal faces, and it could be seen to be opening somewhat; although, the largest crack opening still occurred at the 12 in. (305 mm) crack location. The new flexural crack, as well as the existing cracks, at -8% drift can be seen in Figure 8-77.

Figure 8-77. Cracking on South Side of Pile at -8% Drift
During the +8% drift cycle, there were not considerably more cracks, but there was some crack growth at the location near the top of the pile. Starting during the Kobe record where large negative displacements occurred, the north side of the pile continued to be more damaged visually than the south side of the specimen. This continued throughout the test, so the +8% drift cycle presented the best opportunity to see the extent of damage under the pile and to observe the maximum uplift that occurred at the interface during the concentrated rotation at the connection.

![Figure 8-78. Wharf Damage and Uplift at North Face of Pile during +8% Drift](image)

The next figures will show the photographs of the specimen at the peak 8% drift positions as seen from the various fixed cameras. These figures will provide a visual reference for the magnitude of deformation imposed on the test specimen, as well as the extent of damage, when compared to previous figures from the same perspective. Figure 8-79 shows the west view of the entire test specimen at -8% and +8% drift. Since some initial stroke of the LBCB actuators was used through z-rotation to make the connection to the test specimen, the magnitude of these displacements and rotations shown correspond to the 95% of LBCB displacement capacity. As before, other than large localized cracks in the specimen, the majority of the deformation was
accommodated through a lumped rotation at the connection. The images of the full specimen at its peak displacement allow one to really see the localized rotation at the pile-wharf interface.

Figure 8-79. West View of Specimen at -8% (left) and +8% (right) Drifts

Figure 8-80 displays a zoomed view of the pile-wharf interface at -8% and +8% drifts. Again, the concentrated rotation at the connection can be observed, but also, the extent of cover spalling at the interface. By the 8% drift cycle, the cover had completely spalled on the pile, and the transverse reinforcement was visible on both the north and south faces of the pile. Since the
cover was about 2.5 in. (64 mm), this reduced the 24 inch (610 mm) cross-section by about 20% at the connection.

![Figure 8-80. West View of Connection at -8% (left) and +8% (right) Drifts](image)

The north and south views of the pile-wharf interface at -8% drift and +8% drift are shown in Figure 8-81 and Figure 8-82, respectively. There was some additional spalling of the pile and the wharf deck, but the main development during this cycle was the addition of a few new cracks and an even more pronounced opening of the pile-wharf connection.

![Figure 8-81. North (left) and South (right) Views of the Connection at -8% Drift](image)
After the test was concluded, the test specimen was photographed with the LVDT reference bar and other instrumentation removed. With these obstructions gone, the full extent of the visual damage to the specimen could be more easily observed. Figure 8-83 shows the north face of the pile-wharf interface after testing. The extent of wharf and pile spalling cover can be seen, as well as the exposed prestressing strands, transverse reinforcement, and extreme dowel bars. The spalling of the wharf deck extended beyond the embedment length of the pile, and spalling of the pile concrete cover extended about 9 in. (229 mm) along the length of the pile. The extent of the pile spalling is significant because the repair scheme is designed to go 30 in. (762 mm) above the maximum extent of pile damage. Figure 8-84 also shows the north view of the pile-wharf interface, but with the closer view, more details of the damage can be seen.
Figure 8-83. North View of Specimen after Testing

Figure 8-84. Close View of North Face of Pile-Wharf Connection after Testing
The south view of the specimen after testing can be seen in Figure 8-85. Although the height of the deep spalling to the transverse reinforcement here is not as large as on the north face of the pile, the moderate pile spalling was also measured to be about 9 in. (229 mm) along the height of the pile.

Figure 8-85. South View of Specimen after Testing

Figure 8-86 displays the west view of the pile-wharf connection. This angle allows one to observe the reduction in the pile cross-section at the conclusion of testing. As mentioned previously, the cover spalling of the pile concrete reduced the effective cross-section of the pile at the interface by about 20%. Additionally, it can be seen that by the conclusion of testing, the wharf spalling had extended essentially all the way around the specimen; however, the deepest and most dramatic wharf spalling occurred adjacent to the north and south faces of the pile.
The previous four figures show the specimen at the conclusion of testing with the instrumentation removed. Throughout the test, loose concrete was removed from the specimen, and small pieces that were difficult to remove by hand from the connection were removed with a shop vacuum. However, lightly tapping on the specimen with a screwdriver revealed that substantially more of the concrete was damaged and at least partially spalled from the specimen. The repair scheme that was to follow this test called for the removal of this damaged concrete, leaving only the structurally sound concrete on the specimen. Additionally, to enhance bond with the concrete repair sleeve, the surface of the existing concrete specimen was to be roughened to a height of 30 in. (762 mm) beyond the damage zone, which was measured to be 9 in. (229 mm). Figure 8-87 shows the specimen ready for repair, and the extent of the damaged area at the base of the pile is evident. Also, it can be seen that the tightly spaced transverse
reinforcement performed well since the confined concrete core appears relatively intact and undamaged. The single transverse hoop seen in the photograph is the start of the repair.

![Figure 8-87. Roughened Specimen before Repair](image)

Figure 8-88 shows a closer view of the damaged area on the cleaned specimen before the repair. Considering the height of the pile and the size of the dowel bars at the connection, the 9 inch (229 mm) damage zone is considerably smaller than predicted by common plastic hinge length models. This is an effect of the rotation at the end of the pile and the slip of the dowel reinforcement. In fact, considering the nature of the rotation at the end of the pile, the term plastic hinge should be avoided, and it is more accurately described as the zone of pile damage.
The following sections will explore the extent of the slip at the interface during testing in more detail.

![Figure 8-88. Close View of Cleaned Specimen before Repair](image)

With the visual progression of the damage throughout the cyclic loading portion of the testing program outlined, the next sections will analyze the measured response of the test specimen from the instrumentation, and a comparison will be made to how the data analysis corresponds with the visually observed behavior.

### 8.5.2 Cyclic – Local Behavior

The cyclic portion of the loading finally brought the specimen to experience some strength degradation and damage. As seen by the photographs, the specimen experienced a significant amount of deformation. To be able to accommodate an 8% drift while maintaining 80% of the maximum strength capacity truly demonstrates the ductility of the connection. Admittedly, increasing the axial load would have caused more damage in the pile and increased the strength degradation, but the 90 kip axial load was selected with the intention of being representative of a typical loading on a single pile-wharf connection.
When reviewing the data from the cyclic loading portion of the test, it was realized that the Krypton malfunctioned during the 5% drift cycle, so this cycle has been omitted during the post-processing. Thus, the cycles that will be displayed are, in order: 2%, 3.5%, 3.5%, 2%, 3.5%, 6.5%, and 8%. Due to the nature of the Krypton data acquisition, the data could not be seen in real time, so it was only through a post-processing effort that the LEDs could be realized as “invisible” to the camera. Through troubleshooting, it was determined that one of the extension wires from the Krypton controller to the strober units, into which the LEDs plug, went bad during the test. The extension cable was replaced, and the testing resumed. A recommendation was placed to the MUST-SIM site to create a method enabling the visualization of the LEDs after each converged load step to ensure future tests did not experience a similar problem. As seen through the data analysis, the Krypton data has proven to be very valuable in characterizing the behavior of the specimen.

The review of the data from the cyclic loading begins with an assessment of the state of the strain gauges throughout the cyclic loading. The strain gauges had been previously used through the processing to provide a measure of the curvature or average rotation experienced by the pile. Since it was anticipated that the strain gauges would yield considerably and eventually even fail, a method was developed to use the Krypton readings to quantify the lumped rotation at the connection. With the lumped rotation computed, the difference between the total rotation obtained by the Krypton LEDs and the lumped connection rotation would provide the flexural rotation of the pile at various locations of interest. By comparing this method to the pile flexural rotation obtained from strain gauges that were properly functioning, it was concluded that it was viable procedure, and it will be used for the pile curvatures reported in this section. Figure 8-89 shows the total rotation from the Krypton LEDs at the interface and the computed average
connection rotation from the strain gauges at the interface throughout the duration of the cyclic loading. It can be seen that the strain gauges continued to work properly through the final 3.5% drift cycle; however, at the larger cycles, the strain gauges no longer provided reliable results.

![Figure 8-89. Total Rotation and Pile Rotation at the Connection during Cyclic Loading](image)

Although the strain gauges eventually failed, the relationship between the total rotation and the pile rotation during the first five cycles can be investigated. Figure 8-90 displays the total rotation versus the pile rotation, as well as a best fit line through the data. The best fit line reports that only 10% of the total rotation was due to rotation of the pile, leaving 90% of the rotation to be achieved through the concentrated rotation at the end of the pile. Over the course of the test, the percentage of the total rotation attributed to the pile has increased from 71% during the elastic cycles of the Imperial Valley record, to about 85% through the Northridge and
Kobe records, and finally to about 90% during the cyclic loading. Recognizing this relationship is critical to understanding the behavior of the pile-wharf connection, and it provides and explanation about why the substantial damage to the pile was localized over only a small height of the pile. Again, this amount of lumped connection rotation contribution, as well as the increase with damage, is consistent with the findings in piles with bearing pads from Stringer (2010).

![Figure 8-90. Pile Rotation versus Total Rotation through First Five Cycles](image)

To further demonstrate that the Krypton method to estimate the lumped rotation at the connection is valid, Figure 8-91 shows the computed lumped rotations at the connection from the Krypton targets, and it also provides a comparison to the lumped rotation obtained as the difference between the total rotation and the pile flexural rotation from the strain gauges. Again,
Figure 8-91 shows that the Krypton method aligns well with the strain gauge values until the 6.5% cycle, where the strain gauges are no longer reporting valid values.

**Figure 8-91. Comparison of the Lumped Connection Rotation from Krypton and Strain Gauges during Cyclic Loading**

Since the lumped rotation at the end of the pile was such a substantial factor in the displacements and rotations achieved by the test specimen, the relationship between the connection rotation and the imposed x-displacement and y-rotation was investigated. The relationship between the connection rotation and the control degrees-of-freedom was found to be almost perfectly linear throughout the cyclic loading. Figure 8-94 shows the connection rotation plotted against the control DOFs, and a linear best fit line found the relationships for the x-displacement and y-rotation to be 105 in./radian and 1.387 radians/radian, respectively. The
increase in each of these relationships from the values obtained during the Kobe record demonstrates an increase in the rotation at the connection relative to the pile.

![Graphs showing X-Displacement and Y-Rotation versus Lumped Connection Rotation](image)

**Figure 8-92.** X-Displacement (left) and Y-Rotation (right) versus Lumped Connection Rotation during Cyclic Loading

The lumped rotation versus the base moment in the pile is plotted on the left in Figure 8-93, and the x-displacement versus the base moment is on the right. The shapes of the two plots are strikingly similar, and this is reasonable because if the pile remains largely rigid and the imposed displacement and rotations are achieved by the specimen through connection rotation, then the relationships should be linear. The main observable difference between the two plots is that the lumped connection rotation was obtained from the Krypton data, which did not have the 5% drift cycle, while the x-displacement values were obtained by an external string pot, so it was available throughout the test.
With the examination of the connection curvature exhausted, the pile curvatures were reviewed next. Although the curvatures are relatively small from this particular experiment, the pile curvatures at the interface and 8 in. (203 mm), 12 in. (305 mm), and 24 in. (610 mm) are plotted throughout the cyclic loading in Figure 8-94. Through the 2% and 3.5% cycles, the curvatures at each location are essentially equal, but as the test entered the final two cycles at 6.5% and 8%, the curvatures at lower three locations became substantially larger. In the positive curvature direction, the order of the magnitudes decreases as the locations get farther from the interface; however, in the negative direction, the curvature over the lower 12 in. (305 mm) of the specimen seems to be almost constant.
The following two figures display the moment-curvature response of the specimen at different heights along the specimen. The pile curvatures were those obtained from previously outlined procedure with the Krypton LEDs, and the moment has been computed for the appropriate height with the pile. Also, the Response-2000 sectional analysis is provided for reference. The base moment-base curvature response in Figure 8-95 indicates that some damage did indeed occur in the pile, and the large hysteretic loops indicate a considerably larger energy dissipation when compared to previous results. Despite the accumulation of damage in the specimen, essentially no degradation of strength was observed through the cyclic loading. As the analysis progresses to locations farther away from the pile wharf interface, the moment-curvature
response seems to indicate a decreasing amount of curvature in pile, as seen at the 12-inch (305 mm) and 24-inch (610 mm) locations in Figure 8-96.

Figure 8-95. Base Moment versus Interface Curvature during Cyclic Loading

Figure 8-96. Base Moment versus Curvature at 12 in. (left) and 24 in. (right) from the Interface during Cyclic Loading
The following three figures display the vertical strains in the specimen computed from the Krypton displacement field and the strain gauge values at the 3.5%, 6.5%, and 8% drift cycles. In Figure 8-97, the pre-existing cracks around 12 in. (305 mm) from the interface can be seen to be opening on the tension side of the pile. Additionally, the strain gauges have experienced strain hardening in the interface and 6 in. (152 mm) into the wharf deck. As the test progressed, the large cracks, which were a cause for concern during testing, can be seen in both Figure 8-98 and Figure 8-99 at the peak displacements. It is unfortunate that the Krypton was unavailable during the 5% drift cycle because it would be interesting to see if the crack near the top of the pile grew from the 5% to 6.5% cycles. When comparing Figure 8-98 to Figure 8-99, it appears that the top cracks did indeed grow somewhat with the increase in displacement. When examining the strain gauges in the last two cycles, it can be seen that nearly all of the dowel gauges had entered either entered strain hardening, or the gauges were damaged.

Figure 8-97. Strains from Krypton and Strain Gauges at 3.5% Peak Drifts
Figure 8-98. Strains from Krypton and Strain Gauges at 6.5% Peak Drifts

Figure 8-99. Strains from Krypton and Strain Gauges at 8% Peak Drifts
The next sections will involve analyses of two behavioral properties of the specimen throughout the loading protocol: the global change in stiffness of the specimen and the extreme dowel bar slip.

### 8.6 Stiffness Degradation Analysis

The next analysis effort was to monitor the change in the relative stiffness of the specimen over the course of the test. During the Imperial Valley loading, the specimen remained elastic, but as the specimen continued to soften during the remaining earthquake records and cyclic loading, the relative global stiffness is a value of interest. To compute the relative degradation of stiffness, the peak-to-peak secant stiffness from the base moment versus x-displacement plot was used. While it may be more typical in a test where the end of the pile is allowed to freely to use the lateral load versus displacement, since our loading included both rotation and displacement, the moment was used to incorporate both of these effects. In Figure 8-100, the base moment versus x-displacement response during the Imperial Valley is shown. A best fit line is included to capture the elastic stiffness of the specimen. This slope of this line will be considered the base line elastic stiffness of the specimen for comparison against later cycles.
It was previously observed that during the initial large displacements through the Northridge record, a substantial change in the specimen behavior occurred. This change included yielding of the extreme dowel bars, spalling of the pile and wharf, and the formation of flexural cracks within the pile. Due to this dramatic change, the stiffness of the specimen during the Northridge earthquake is evaluated after the large initial displacements. Since the major behavioral change had already occurred, the entire Kobe record was used for the stiffness during that segment of the test. Figure 8-101 show the base moment versus x-displacement response of the specimen during post-peak Northridge and during the Kobe record. Additionally, the best fit lines of the response are included.

Figure 8-100. Base Moment versus Displacement Response during Imperial Valley
With the average stiffness obtained during the earthquake loading, the next step was to evaluate the stiffness of the specimen during the cyclic portion of the loading. The best fit lines shown in the previous two figures are included in Figure 8-102 for a reference. It can be seen in Figure 8-102 how the stiffness continued to degrade over the course of the test. Also, it should be noted that the stiffness did not substantially change from the Kobe record through the 3.5% cycles. This is as expected because the 3.5% cycle had essentially the same magnitude as the Kobe record.
Figure 8-102. Base Moment versus Displacement during Cyclic Loading

The peak-to-peak values of the cyclic loading were used to obtain similar best fit lines during the cyclic loading, and the slope of these lines were evaluated. Table 8-1 displays the numerical values of the stiffnesses at each level of loading, as well as a comparison of the stiffness to the elastic baseline value obtained during the Imperial Valley record. The most dramatic change can be seen to have occurred during the Northridge record, where the stiffness of the test specimen dropped 75%. Through the Kobe record, the 2% cycles, and the 3.5% cycles, the stiffness again reduced by about 50%, to a value of about 12% relative to the elastic stiffness. Each subsequent cycle was coupled with a reduction in the stiffness, and the final stiffness during the 8% cycle was only 5% of the elastic value. This data is telling of the amount of damage that occurred during the loading. Although the strength of the specimen did not drop substantially during the test, the stiffness of the specimen can be seen to have dramatically decreased during the test.
Table 8-1. Comparison of Specimen Stiffness during Entire Testing Program

<table>
<thead>
<tr>
<th>Cycle</th>
<th>Stiffness (k-in/in)</th>
<th>% of Elastic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imperial Valley</td>
<td>17900</td>
<td>100.0%</td>
</tr>
<tr>
<td>Northridge (Post-Peak)</td>
<td>4201</td>
<td>23.5%</td>
</tr>
<tr>
<td>Kobe</td>
<td>2066</td>
<td>11.5%</td>
</tr>
<tr>
<td>2% Drift</td>
<td>2312</td>
<td>12.9%</td>
</tr>
<tr>
<td>3.5% Drift</td>
<td>2018</td>
<td>11.3%</td>
</tr>
<tr>
<td>5% Drift</td>
<td>1557</td>
<td>8.7%</td>
</tr>
<tr>
<td>6.5% Drift</td>
<td>1187</td>
<td>6.6%</td>
</tr>
<tr>
<td>8% Drift</td>
<td>911</td>
<td>5.1%</td>
</tr>
</tbody>
</table>

The following section will include a separate investigation of the slip associated with the lumped rotation of the connection. Since this analysis will include results from the entire test, as well as a comparison to an accepted slip model, a separate section has been used to convey this information.

8.7 Connection Slip Analysis

As previously reported, the behavior of the pile-wharf connection specimen was dominated by a concentrated rotation at the end of the pile. Since the specimen was constructed with the dowel bars initially embedded in the wharf deck, any concentrated rotation must have been accompanied by some slip of the dowel bar reinforcement. If a modeling effort of the pile-wharf connection is to be pursued, it will be necessary to incorporate the lumped rotation at the end of the pile, and the appropriate dowel bar slip quantification may be critical to that effort. The Krypton LEDs closest to the pile-wharf interface were monitored throughout the test, and based upon their change in position and rotation relative to the known location of the extreme dowel bars, the amount of slip at the dowel bar location was computed. Since there was no way to measure the actual slip at the embedded end of the pile, the LEDs are neglecting any
longitudinal strain that may occur in the dowel bars over the lower two in. of the pile; however, since this is a relatively small length, the majority of the rotation below the pile-wharf interface was attributed to the lumped rotation, as discussed in the previous sections.

![Graph showing earthquake loading](image)

**Figure 8-103. Extreme Dowel Bar Slip during Earthquake Loading**

The calculated slip at the interface over the entire earthquake loading is shown in Figure 8-103. Both the north and south dowel bars are shown to demonstrate the relative slip in each bar over the course of the earthquake loading. The cursory assessment demonstrates limited slip during the Imperial Valley record, moderate slip of the north bars during the Northridge record, and significant slip of both the north and south extreme bars over the course of the Kobe record. A closer view of the bar slip during Imperial Valley is displayed in Figure 8-104. The specimen
remained purely elastic during this loading regime, so the slip, as expected, was very minimal. The initial negative slip seen in the figure is due to the application of the axial load during the first 17 loading steps.

Figure 8-104. Extreme Dowel Bar Slip during Imperial Valley Record

The Northridge loading is when the specimen first experienced yielding of the north dowel bars, and during the large positive displacement near the beginning of the record, the behavior of the specimen changed significantly. While the lumped rotation was computed to contribute 71% of the end pile rotation during Imperial Valley, after the large displacements in Northridge, the lumped rotation was found to be closer to 85% of the total rotation. This large rotation had to be achieved through the dowel bar slip, and north extreme bar can be seen to have
slipped approximately 0.25 in. (6.25 mm) during the large Northridge displacement. Additionally, there appears to be a residual slip of about 0.05 in. (1.27 mm) throughout the end of the Northridge record.

![Graph showing extreme dowel bar slip during Northridge record.](image)

**Figure 8-105. Extreme Dowel Bar Slip during Northridge Record**

Early in the Kobe record, a large negative displacement occurs, causing the south extreme dowel bar to yield considerably. Also, since 85% of the total rotation occurred at the end of the pile, a large slip of the dowel reinforcement was also observed. At the peak negative displacement, the south extreme dowel bar had slipped 0.58 in. (14.7 mm), and at the end of the Kobe record, a residual slip in both dowel bars was observed to be about 0.16 in. (4.06 mm), as seen in Figure 8-106.
Images measuring the slip of the south face of the pile during the peak negative displacement are displayed in Figure 8-107. The image on the left shows a tape measure at the pile-wharf interface, and although it is difficult to see, a close examination shows that the reading is in the 0.75 in. (19 mm) range. Since this measurement being taken at the face of the pile, it is reasonable for the tape measure to read a larger value than the computed slip of the dowel bar. The complementary image on the right displays a ruler being inserted under the uplifted side of the pile up to about 2.75 in. (70 mm). This distance corresponds roughly to the amount of cover in the specimen, so it is unclear whether further insertion was hindered by the dowel bars.
The cyclic portion of the loading followed, and the evolution of the bar slip can be seen in Figure 8-108. Since the slip was calculated from Krypton LEDs, the slip at 5% drift was not available; however, the increase in slip appears fairly linear as the cycles increase, so it could be estimated to have been about 0.55 in. (14.0 mm) for the north extreme bar and 0.45 in. (11.4 mm) for the south extreme bar. A trend emerged during the cyclic loading that the slip computed for the north bars was consistently larger than the strain for the south bars. This could be an effect of the asymmetric nature of the earthquake loading, and a review of the damage to the specimen during testing showed that the north side of the specimen consistently showed a visibly larger uplift.
Images of the north side of the pile-wharf connection are displayed in Figure 8-109. The photograph on the left shows a measurement of the uplift of the specimen at the face of the pile, and the tape measure reads about 2 in. (50.8 mm). This value is considerably larger than the estimated slip of the dowel bar, but this may be due to the measurement being taken at the face of the pile and the measurement being taken at the base of the wharf spall. Since the spalling extended below the level of the pile embedment by the end of the test, a larger reading makes sense. Considering these effects, the estimated maximum slip of the north extreme bars of 0.9 in. (22.9 mm) seems reasonable. The photograph on the right displays the tape measure being inserted underneath the uplifted pile to a distance of about 5 in. (127 mm). This is a considerable
uplift, especially since the end of the tape measure is not the ideal tool to reach the maximum distance under the pile.

Figure 8-109. Measurement of Interface Slip at +8% Drift

Zhao and Sritharan (2007) developed a model to quantify the slip that occurs in longitudinal reinforcing bars that are fully anchored into connecting concrete members. As seen in this particular experiment, neglecting the end rotation of the flexural members will cause an underestimation of deflections and an overestimation of the stiffness and section curvature. Although the model was calibrated from concrete columns and a bridge T-joint, it is definitely applicable to the current pile-wharf connection specimen. The model is a function of the bar diameter, the concrete strength, the reinforcing bar yield strength, a stiffness reduction factor, and a bond-slip relation. Using the current material properties for the dowel bars and the recommended values for the stiffness reduction factor and bond-slip relation, the bar stress versus slip is plotted in Figure 8-110. The ultimate strength of the bar is included for reference. Considering the yielding of the dowel bars and the corresponding estimated stress during the Northridge record, a bond slip value of 0.58 in. (14.7 mm) provides a reasonable result with the
experiment. Since the model seems to reasonably resemble the behavior of the specimen, it could possibly be implemented into a finite element model as a zero-length element to get a more representative behavior of the pile-wharf connection.

![Bar Stress-Slip Model for #10 Reinforcing Bar Adapted from Zhao and Sritharan (2007)](image)

**Figure 8-110. Bar Stress-Slip Model for #10 Reinforcing Bar Adapted from Zhao and Sritharan (2007)**

With the analysis of the behavior of the pile-wharf connection during the experiment concluded, the next chapter will focus on some modeling of the test specimen, as well as an example of the potential extension of the HPFRCC model to a pile-wharf connection. The comparison between the behavior of the model with reinforced concrete and with HPFRCC will demonstrate the ductility of the HPFRCC material, as well as illustrating the application of HPFRCC through modeling to damage critical structural components.
CHAPTER 9. PILE-WHARF CONNECTION MODELING

With the ATENA model having proven its ability to reasonably emulate the laboratory response of HPFRCC at the material level and the structural component level, the extension of HPFRCC to other damage-critical structural components is the final phase of the current research. The purpose of this chapter is twofold: to further demonstrate the extension of HPFRCC to the modeling of structural components and to bridge the two major experimental thrusts of this research.

9.1 Pile-Wharf Connection Modeling Parameters

The major objective of the pile-wharf connection research was to explore the structural connection behavior by constructing, testing, analyzing, repairing, and re-testing a full-scale pile-wharf connection specimen in an experimental set-up having realistic structural boundary conditions. The test specimen consisted of a 24-inch diameter, octagonal, precast, prestressed pile connected to a cast-in-place reinforced concrete wharf deck slab. The connection was made by embedding the pile two in. into the deck and then grouting T-headed bars into ducts within the pile; wharf reinforcement was placed around the connection, with the T-heads developed into the wharf. The details of this test and the experimental results were described previously in Chapter 7 and Chapter 8. It was observed that after a seismic event, the damage in port structures was often concentrated at the pile-wharf connection, and given that HPFRCC has exhibited a superior ability to withstand damage and to prevent diagonal bars in coupling beams from buckling, it was thought to be interesting to see how the implementation of the HPFRCC model into the pile-wharf connection region would affect the performance of the specimen. Since experimental data is available from the completed laboratory test, a comparison of the
response of typically constructed pile-wharf connections with those utilizing HPFRCC may be a very viable way to extend the use of HPFRCC to other damage critical structural elements. While implementing HPFRCC into the end of precast piles may pose some constructability issues that would need to be practically resolved, the parametric study could provide insight into the damage tolerance and performance improvement that HPFRCC could afford to such a structure.

9.1.1 Constitutive Models

The first step in developing the finite element model was to determine the appropriate material models to be used in the modeling effort. The precast, prestressed pile represents a significant departure from the concrete elements previously modeled in this study. Compressive concrete behavior of the concrete core is highly confined by the transverse reinforcement, so a modification of the concrete model is necessary. Previous research by Mander et al. (1988) provides a reinforced concrete model for confined concrete. The model uses the Popovics curve to represent the stress-strain relationship of both confined and unconfined concrete, and it has been verified to provide accurate results in nonlinear analysis. Stringer (2010) provided general guidelines for the modeling of pile-wharf connections and generated two stress-strain curves for the confined pile concrete using models by Leslie (1974) and Mander et al. (1988), as shown in Figure 9-1. It can be seen that each model provides similar results, and in this study, the Mander model was used for the pile concrete.
Beyond the adjustment of the concrete compressive relationship, the default parameters from the ATENA CC3DNonLinCementitious2User material were implemented. A separate unconfined model was not developed for the concrete cover, so this model may slightly overpredict the concrete strength of the pile. For the wharf deck concrete, the SBETA model was implemented since the concrete model did not require any special adjustments to the material properties.

All of the reinforcement, including prestressing steel, dowel reinforcement, wharf deck reinforcement, pile spiral steel, and connection dowels, was modeled using a bilinear constitutive relationship with hardening. The bilinear model adds a slight level of sophistication above the
elastic-perfectly plastic model, and since the material properties were known for all of the reinforcement, the bilinear model was easily implemented.

For the reinforcement, the default bond-slip model was implemented within ATENA. The default model is the CEB-FIB model code 1990, and it defines the bond strength depending on the value of the slip between the reinforcement and the surrounding concrete. In the CEB-FIB model, the user selects whether the bar is ribbed and the confinement conditions. Since all of the reinforcement was in a relatively well confined region, the bond condition was selected as “good.” For the prestressing steel, “cold-drawn wire” was selected for the type of reinforcement, while the dowels were selected as “ribbed” reinforcement. These parameters affect the values of the bond slip relationship (Cervenka, Jendele, & Cervenka, 2010). Since the prestressing should have almost no effect at the end of the pile, and the stress in the strand develops over a transfer length, the slip model for the prestressing steel was important to simulate the lab conditions. A slip model was not required for the wharf deck reinforcement or the pile spiral reinforcement because they were modeled as smeared reinforcement. The dowel bars used to connect the specimen to the LBCB were included in the model to prevent a weakness at the top of the pile. These bars were modeled to remain elastic, and a perfect bond model was used. The following section will outline the finite element modeling parameters implemented in the model.

9.1.2 Loading and Boundary Conditions

The geometry of the model was identical to the experimental specimen in an effort to most closely model the experimental test. In the laboratory, the specimen was connected to the floor through four post-tensioned threaded rods through the wharf deck. The wharf deck was designed to be large enough that the post-tensioning stress in the wharf deck would not affect the behavior of the pile-wharf connection. Thus, it was reasonable to apply the supports in the
model at the same location. Steel plates were placed on the top and bottom of the wharf deck, as done in the experiment, and each plate was restrained against vertical movement, while the bottom left steel plate was also restrained against horizontal movement.

The octagonal cross-section was simplified into three sections. The center section was a full 24 in. (610 mm) wide; however, the width of the two end sections was simplified as the average of full cross-section and the width of one of the faces of the pile. This approach provides the same area for the cross-section; however, it may lead to a slight over prediction of the bending capacity due to the increase in concrete width at the extreme fibers. A schematic of the cross-section simplification is shown in Figure 9-2 for clarity.

![Figure 9-2. Cross-Section Simplification for Pile](image)

The load cases in ATENA are incremental, so the axial load in the specimen and the prestressing forces were applied in two separate steps at the beginning of the analysis. The axial load was applied to the top of the specimen through a steel plate to prevent a stress concentration
in the model. The prestressing strands were bundled so that there is one strand in each section of the cross-section. The prestressing in the end sections represented 6 prestressing strands each. As described in Chapter 7, each prestressing strand had 31 kips (138 kN) of force, so these strands had 186 kips (827 kN) of force, and the area was appropriate for 6 strands. In the middle section, the prestressing strand represented 10 tendons, so the axial force was 310 kips (1.38 MN). The lateral displacement was applied at a location 84 in. (2.13 m) from the pile-wharf connection through a steel plate to prevent a localization of damage, and the finite element model with the described loading and boundary conditions is shown in Figure 9-3.

Figure 9-3. Pile-Wharf Model with Loading and Boundary Conditions
The models were run with a monotonic loading; although, the experimental tests were performed cyclically. Also, a rotation was not applied to the top of the specimen as done in the laboratory tests. This approach was taken because the goal of this modeling effort was not to exactly reproduce the experiment, but rather to explore the modeling of the pile-wharf connection without and with HPFRCC. A general effort was made to mimic that experimental specimen, but again perfect duplication was not the focus of this study. A comprehensive modeling effort to capture all of the nuances and complexities of the pile-wharf connection is currently being reviewed by colleagues at the University of Illinois (Caiza et al., 2010).

9.1.3 Finite Element Modeling Parameters

The finite element model mesh was varied to better capture areas of interest. In the connection region, a 1 in. (25 mm) mesh was implemented. Since the area experienced the greatest amount of deformation, a finer mesh was required to better capture the response. The two regions adjacent to the pile-wharf interface region had a 2 in. (50.8 mm) mesh. These regions included one in the wharf deck and one in the lower portion of the pile. These two areas were expected to have some damage, so a slightly finer mesh was implemented. The final three regions had a 4 in. (102 mm) mesh. These areas were expected to remain relatively undamaged, so a closely spaced mesh was unnecessary. Figure 9-4 depicts the finite element mesh for the model, as well as boxes to illustrate the different meshing regions previously described.
To be able to truly capture the response of the pile-wharf connection, it is critical to allow the connection to rotate. In a discrete finite element package, a zero-length rotational spring could be implemented in the connection to accommodate this rotation. Since ATENA is a continuum finite element program, interface elements were used at the pile-wharf connection in an effort to capture the behavior. The interface model was intended to simulate the contact between the pile and the wharf deck. In the experiment, the pile was observed lifting and separating from the wharf deck at the cold joint, so the tensile strength of the interface elements was set to zero. This parameter should allow the pile to freely rotate from the wharf deck. For the shear parameter of the interface element, the coefficient of friction was set as 0.2, and the
tangential stiffness, $K_n$, was defined as the shear modulus divided by 100. This was according to the recommended value in the ATENA Theory Manual (Cervenka et al., 2010). Also, the cohesion was set to zero since this was considered a cold construction joint. The interface model behavior in shear (left) and tension (right) as displayed in

![Figure 9-5. Interface Model Parameters in Shear (left) and Tension (right) from Cervenka et al. (2010)](image)

As in the coupling beam models, the full Newton-Raphson method was used as the solution parameter. The node numbers were optimized with the Sloan method, and iteration limit for each step was set to 50. The default displacement, residual, absolute residual, and energy error tolerances were used. The following section will outline the results of the pile-wharf connection finite element models.

### 9.2 Pile-Wharf Connection Modeling Results

For the pile-wharf connection modeling, the precast, prestressed section with standard concrete was modeled first. This specimen was modeled after the laboratory experiment outlined
in Chapter 7 and Chapter 8. As previously described, the prestressing force was applied as the first step in the model. A bond-slip model was included for the prestressing tendons to better simulate laboratory conditions and to reduce the effect of the prestressing at the critical section in the pile-wharf connection. Figure 9-6 displays the stress in the prestressing tendons, and the full development of the stress over a transfer length can be seen.

![Stress in Prestressing Strands after Application of Prestressing Load Case](image)

The reinforced concrete model predicted a maximum moment capacity at the pile-wharf connection of 6,960 kip-in, which was within 7% of the actual maximum moment observed in the experimental test of 7,451 kip-in. Although the modeled pile-wharf connection did not have
rotation applied to the end, the displacement corresponding to the peak moment occurred at 1.54 in. (39.1 mm), which was rather close to the 1.69 in. (42.9 mm) displacement corresponding to the peak moment in the experiment. It is reasonable for the experiment to have had a larger displacement at the maximum moment because the rotation applied by the LBCB in the laboratory produced a counter-clockwise moment (with respect to the images shown) at the top of the pile that would reduce the moment at the connection. Thus, to achieve the same moment at the connection, a larger displacement would be required. The modeled RC specimen did not have nearly the same ductility demonstrated in the experiment. This was due to crushing of the compressive toe of the pile, despite the efforts to provide a model for confined concrete. This is certainly a short coming of the model, and perhaps presents an opportunity for a future researcher to further investigate the creation of a continuum finite element model to more accurately predict the post-peak behavior of a pile-wharf connection.

The model displayed widespread cracking throughout the tension side of the specimen. In the visual display, cracks were filtered out that had a crack width less than 0.04 in. (1 mm), as recommended by the ATENA User’s Manual (Cervenka et al., 2010). Despite the filter eliminating many cracks initially shown by ATENA, a dense cracking pattern was still observed, and this result is shown on the left in Figure 9-7. The cracking was observed along the pile up to a distance of 60 in. (1.52 m) from the pile-wharf connection. As mentioned, the pile eventually failed due to crushing of the compressive toe of the pile. The minimum principal stresses are shown on the right in Figure 9-7, and the high compressive stress concentration can be seen in the pile. Additionally, the dowel bars are shown with their respective stress distributions. It can be seen that the dowel bars had all yielded; however, none of them experienced a premature fracture. The concrete cover crushing and the dowel bars not fracturing were consistent with the
experimental results, but once the concrete cover crushed, the model did not properly represent the ductile experimental behavior still exhibited by the concrete core at the connection.

A zoomed view of the RC pile-wharf connection model is shown in the two images in Figure 9-8 to better visualize the damage at the connection. On the left, the minimum principal stress is displayed, and on the right, the maximum principal strain is shown. Immediately it can be seen that the interface elements representing the construction joint between the pile and the wharf deck is behaving as intended. The pile is able to separate from the wharf deck without causing any tensile strains in the local finite elements. In the image on the left, the minimum principal stresses clearly show a localization of the compressive force in the concrete cover of

Figure 9-7. RC Pile-Wharf Connection Cracking Pattern (left) and Minimum Principal Stress (right)
the pile. The reduced area of compression can be seen due to the lumped connection rotation, and once the concrete cover crushed, the capacity of the pile dropped substantially. On the right in Figure 9-8, the maximum principal strains show the distributed bands of principal tension along the length of the pile. This localization of wide flexural cracks at several locations at the base of the pile is indeed reminiscent of the experimental specimen, as depicted previously throughout Chapter 8.

![Figure 9-8. RC Pile-Wharf Connection Zoomed View of Minimum Principal Stress (left) and Maximum Principal Strain (right)](image)

For the HPFRCC pile-wharf model, the HPFRCC material model was implemented into the bottom 36 in. (0.91 m) of the pile; this corresponds to the lower two regions of the pile depicted in Figure 9-4. The HPFRCC material model was limited to this portion of the pile to be consistent with the spirit of the intended use of the HPFRCC: for application in damage-critical regions. The HPFRCC pile-wharf connection model did demonstrate some of the hallmark characteristics of HPFRCC. An increased deformation capacity, as well as an increase in strength, were observed due to the shear capacity that the HPFRCC model provides to the
connection. Also, the distributed cracking behavior that was evident along the height of the RC model was limited to a small band of localized cracking at a location about 7 in. (178 mm) from the end of the pile. Since a crack filter was applied to any cracks smaller than 0.04 in. (1 mm), the small multiple cracking characteristic of HPFRCC essentially eliminated the large cracks from the specimen. The image on the left in Figure 9-9 shows the described cracking in the HPFRCC pile-wharf connection model near failure. On the right, the maximum principal strains are displayed, and it can be seen that the tensile strain in the connection is limited to a single band. Also, the dowel bar reinforcement with the stress distribution is shown in each figure, and large stresses were still developed in the bar up to failure of the model specimen. With respect to the behavior of the RC model, the HPFRCC is certainly demonstrating a greater damage tolerance, as well as strength and deformation capacity.

![Figure 9-9. HPFRCC Pile-Wharf Connection Crack Pattern (left) and Maximum Principal Strain (right)](image)
The minimum principal stresses in the HPFRCC pile-wharf connection specimen are displayed in Figure 9-10. The image on the left provides a full view of the pile-wharf connection model, while the image on the right depicts a more focused view on the connection. Near failure, the added passive confinement afforded by the HPFRCC is apparent. Minimal cracking is present in the pile, and the compression toe of the pile has a width limited to the thickness of the concrete cover. This is evident because all of the dowel bars are in tension. Despite this reduced effective area, the pile still is demonstrating enhanced shear and deformation capacities. Ultimately, failure of the HPFRCC model specimen occurred when the extreme dowel bar fractured, causing the capacity of the pile-wharf connection model to lose considerable capacity.

Figure 9-10. HPFRCC Pile-Wharf Connection Minimum Principal Stress Global View (left) and Zoomed View (right)
The connection moment versus lateral displacement response of the RC model and the HPFRCC model are presented and compared to the experimental result in Figure 9-11. Admittedly, a comparison of the model displacement response to the experimental displacement response is not completely valid because of the different boundary conditions present in the experimental test due to the applied rotation; however, it does provide a general qualitative frame of reference for the modeling result. For a given displacement, the experiment applied a rotation at the top of the pile to reduce the moment at the pile-wharf connection, so it sensible for the displacements of the experiment to be slightly larger than those demonstrated by the model. Even with that consideration, the model was not able to capture the same level of ductility displayed by the experiment. Figure 9-11 illustrates the added shear capacity that the HPFRCC provided to the damage-critical region of the pile-wharf connection model. The HPFRCC model had a peak moment capacity of about 9,400 kip-in., approximately 33% larger than the connection moment in the RC model. Also, the HPFRCC model was able to accommodate larger lumped connection rotation before the eventual failure of the model specimen. This was achieved largely due to the enhancement of the compressive strength afforded by the passive confinement of the fiber reinforcement.
Overall, the RC pile-wharf connection model captured many aspects of the experimental test, such as the lumped rotation at the connection, localized bands of principal tension, crushing of the compression toe, and the connection moment capacity was closely predicted. However, with the area of concrete in compression heavily reduced due to the lumped connection rotation, once the extreme concrete elements experienced a crushing failure, the capacity of the model dropped. This was not observed in the experiment, where the dense transverse reinforcement confined the concrete core, and large displacements were accommodated without a significant loss of capacity. When comparing the results of the RC model with the HPFRCC model, the performance indicates that HPFRCC would enhance the behavior of the pile-wharf connection.
While this may introduce some complications logistically to actually construct a precast pile where a portion of the pile is HPFRCC, the exercise of implementing the HPFRCC model into a damage-critical structural component and assessing the effect of HPFRCC on the performance of that component was the primary thrust of this portion of the research.

9.3 Other Modeling Applications

With the extension of the HPFRCC model to coupling beam components and pile-wharf connections, its use could be reasonably extended to other, potentially larger, structural applications. For example, the large-scale coupled wall specimen tested by project colleagues at the University of Michigan integrated HPFRCC coupling beams into an RC coupled wall system. That same testing program also explored the use of HPFRCC in the plastic hinging region of the wall. These two large-scale tests provide excellent opportunities to integrate the HPFRCC model into larger structural systems, with the possibility of validating the model through the available experimental results. Another possibility could include a prototype reinforced concrete dual structural system consisting of a coupled wall and a moment resisting frame. Then, critical areas where inelastic activity is expected to occur, such as at the base of the wall regions, the coupling beams, the beam ends, and the column bases, could be modeled using the HPFRCC material model. This parametric study could show the effect of incorporating the HPFRCC on the stiffness, strength, and ductility of the structure. Also, another study could be conducted exploring the influence of including HPFRCC into only some of the damage critical members. Such a parametric study could provide significant insight into the use of HPFRCC in structures by demonstrating its ability to enhance various aspects of structural performance. Whatever the case, the development of an HPFRCC model that can be implemented into commercially available software presents essentially unlimited possibilities for a designer to explore the impact
of HPFRCC on a structural design, whether it is the reduction of the required reinforcement or an impact on the global ductility of the structure.
CHAPTER 10. CONCLUSIONS

This chapter provides a summary of the findings and conclusions of this research study, as well as provides recommendations for future work.

10.1 Summary

In the current research program, the behavior of high-performance fiber-reinforced cementitious composites (HPFRCC) under multi-axial loads was studied thoroughly. An experimental program was performed on small-scale HPFRCC specimens under multi-axial loads on two different concrete mixes. Each concrete mix explored the use of two different fibers types, hooked steel fibers and Spectra fibers, as well as the effect of varying volume fractions. A final HPFRCC mix was selected, and an even more thorough investigation of the multi-axial behavior of this concrete mix was performed. Failure envelopes and constitutive relationships were developed from the small-scale experimental tests. These properties were then implemented into ATENA, a nonlinear finite element program, and modeling of the small-scale program was conducted. The modeling effort showed that the ATENA model could appropriately represent the performance of the HPFRCC material under multi-axial loads. Then, the HPFRCC material model was extended into use in structural components. Coupling beam component tests had been conducted by previous researchers on RC coupling beams and HPFRCC coupling beams. The RC coupling beams were modeled to ensure that the modeling approach was appropriately representing the experimental results, and the HPFRCC material model was implemented into the historical RC coupling beam models to perform a parametric study on the effect of HPFRCC on the coupling beam performance. Next, HPFRCC coupling beam component tests performed by project colleagues at the University of Michigan were
modeled. These coupling beams were modeled with the HPFRCC material model formulated from the material tests, and it was found to accurately capture the response of the experiments. Control RC coupling beams were also modeled with the same layout and loading as the HPFRCC coupling beams to perform another study on the effect of HPFRCC on coupling beam performance.

A separate research project was also conducted that was not directly related to the multi-axial behavior of HPFRCC. This research project involved the large-scale testing of a pile-wharf connection at the NEES MUST-SIM facility at the University of Illinois. This experiment explored the behavior of a pile-wharf connection with realistic boundary conditions. The loading protocol was developed from the results of a nonlinear port system model developed by project colleagues at Georgia Tech. The test specimen was tested with mixed mode control; displacements and rotations were applied to the end of the specimen while a constant axial load was maintained. In an effort to link the two research projects, a pile-wharf connection modeling effort was pursued. A traditional RC pile-wharf connection was modeled, and then the pile-wharf connection was modeled with the addition of HPFRCC. This effort showed the effect of HPFRCC on the pile-wharf connection, as well as the versatility of implementing the HPFRCC model into damage-critical structural components.

10.2 HPFRCC Experimental Conclusions

Several HPFRCC concrete mixes were tested in this study; however, the main findings of the small-scale experimental program can be briefly summarized as follows:

- HPFRCC specimens demonstrate a pseudo-strain hardening and multi-cracking behavior in tension after initial cracking. This is contrasted with plain concrete
and other conventional FRC, which would not perform with the same beneficial mechanical behavior.

- Under biaxial compression, the ultimate strength of HPFRCC relative to its uniaxial compressive strength is higher than for plain concrete.
- Under equal biaxial stresses, the HPFRCC specimens experience between 40% and 55% increase in strength over uniaxial compression.
- The maximum biaxial strength occurs at an intermediate compression-compression ratio. Depending on the properties of the mix, this strength increase may vary from 50% - 80% greater than the uniaxial compressive strength.
- The increase in multi-axial compressive strength relative to the uniaxial compressive strength is more pronounced for lower strength HPFRCC mixes.
- HPFRCC exhibits an enhanced deformation capacity and significant residual strength at large strains. The residual compressive strength of HPFRCC is regularly 70% of the peak compressive strength at 1% strain and 50% of the peak compressive strength at 2% strain.
- The addition of fibers did not affect the elastic modulus of the concrete mix or Poisson’s ratio.
- The addition of fibers alters the failure mechanism of the plate specimens from a splitting tension failure to a shear faulting failure.
10.3 HPFRCC Modeling Conclusions

HPFRCC material tests, RC coupling beams, HPFRCC coupling beams, and pile-wharf connections were all modeled as part of this research. The main findings of the modeling program can be summarized as follows:

- RC coupling beams from the literature can be accurately modeled with commercial nonlinear finite element programs, provided appropriate information is provided regarding the test setup, loading, boundary conditions, and material properties.

- An increase of the SBETA softening parameter was the only change from the default concrete model for the RC coupling beams.

- The cyclic models for the RC coupling beams experienced a rapid strength loss and premature sliding shear failure due to the inability of the model to capture aggregate interlock, dowel action, and other elements contributing to the shear friction mechanism.

- The parametric study of the RC coupling beams with HPFRCC showed that HPFRCC can effectively increase the shear capacity of the coupling beams by about 30% while also increasing the displacement at failure by at least 20%, depending on the reinforcement layout of the coupling beam.

- The HPFRCC coupling beam models were loaded monotonically, so the models were stiffer at low displacements than the experimental tests. This resulted in an over prediction of the force at initial yielding of the reinforcement (15%) and an under prediction of the displacement (12%) at initial yielding of the reinforcement.
• In all of the coupling beam models, the peak strength is predicted to within 4% of the experimental result. In the RC coupling beams, the shear force at initial yielding of the reinforcement was within 10%.

• The displacement at yielding of the reinforcement and at the ultimate load is under predicted by the model on average of 6% and 30%, respectively.

• Failure modes of the experimental coupling beams are predicted accurately by the model.

• The parametric study of the HPFRCC coupling beam with RC shows again that the addition of HPFRCC can increase the shear capacity of the coupling beam by about 30%.

• The application of HPFRCC to the pile-wharf connection further demonstrates the ability to implement the HPFRCC model into other structural components to evaluate its effect on the performance of a damage-critical member.

10.4 Pile-Wharf Connection Conclusions

After an examination of the behavior of the pile-wharf connection specimen, the following conclusions can be made:

• The primary mechanism through which pile displacements are accommodated was through lumped rotation at the pile-wharf connection and not through flexural action of the pile.

• At lower loads, flexural rotation of the pile accounts for about 30% of the total rotation of the pile, with 70% being attributed to a lumped connection rotation.
• As damage progresses, the flexural rotation of the pile accounts for less of the total rotation. In fact, the lumped connection rotation accounts for as much as 90% of the total rotation after significant damage occurs in the pile-wharf connection.

• Pile spalling and deck spalling lead to a decrease in the capacity of the pile-wharf connection; however, if the confined concrete core remains intact, the pile-wharf connection can accommodate large lateral deformations with only a modest reduction of capacity.

• A reduced axial load \( (0.02f'_cA_g) \) relative to other experimental tests causes the pile-wharf connection to behave similar to tests performed by previous researchers with bearing pads located at the end of the pile, with deformation localizing in the connection.

• A dense Krypton LED grid allows for the creation of unique visualization tools to review the behavior and strains throughout the specimen during testing.

10.5 Contributions

To the author’s knowledge, this research provides the following unique research contributions:

• The first study to experimentally test the biaxial behavior of a self-consolidating HPFRCC. The thesis provides failure envelopes and deformation characteristics that are not available in the literature, and should be useful for calibrating further finite element models.
• The first study to develop a finite element model for HPFRCC from its own biaxial experimental results. This thesis outlines the procedure and input parameters required to utilize commercially available software to model HPFRCC.

• This thesis contains the first comprehensive study on the use of an HPFRCC model, calibrated from its own experimental results, to model coupling beams and pile-wharf connections. Also, it outlines a process of implementing the HPFRCC model into historical experimental tests to perform a parametric study on the potential effect of HPFRCC on damage-critical structural components.

• The first pile-wharf large-scale connection test with realistic boundary conditions was reported in this thesis. Previous tests were not able to simulate the application of displacements, rotations, and axial load simultaneously. This realistic loading provides a deeper insight into the expected performance of pile-wharf connections during seismic events.

10.6 Future Work

This research program has provided a review of the multi-axial behavior of HPFRCC, as well as the performance of pile-wharf connection. In light of this research program, several topics exist that the author believes warrant further exploration. Recommendations for future work include the following:

• Biaxial tests on HPFRCC subjected to compression-tension and tension-tension have yet to be completed. This testing program would complete the biaxial failure envelope and provide further insight into the performance of HPFRCC
under complex loading, as would be expected in a damage-critical structural component.

- The cyclic properties of HPFRCC should be investigated experimentally, as well as the implementation of the cyclic properties into the nonlinear finite element model. Since seismic events cause considerable cyclic deformations, the HPFRCC model should be validated to properly model such events.

- The development of definitions of performance levels for HPFRCC associated with specific engineering demand parameters. Since HPFRCC is targeted for damage-critical components, it is important to be able to map the deformation or damage of the structural element to a specific performance level for the assessment of the HPFRCC properties. This capability could greatly further the ability of designers to make performance-based design decisions regarding the use of HPFRCC.

- Additional pile-wharf connection specimens could be tested with varying loading parameters, such as an increased axial load or a different applied displacement-rotation ratio. Also, a larger-scale model, as originally planned at the NEES MUST-SIM facility, including multiple pile-wharf specimens attached to a single wharf deck could provide valuable insight into the effect that the interaction between adjacent piles has on the global seismic response of a port structure.

- A more detailed pile-wharf connection model could be developed characterizing the behavior of the lumped connection rotation. This model could then be verified under reverse cyclic and/or earthquake loading. Additionally, the
implementation of the model into a full port structure could provide a valuable tool for the seismic assessment of existing port systems.
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