ABSTRACT

STRAHLER, MEGHAN ANN. Behavior of Hollow Precast Concrete Cylinder Piles with Steel or FRP Transverse Reinforcement. (Under the direction of Drs. Gregory W. Lucier and Rudolf Seracino).

Precast concrete cylinder piles have been used extensively in marine environments since the 1950s. Concrete cylinder piles contain transverse steel reinforcement, which reinforces the hoop direction of the hollow cylindrical cross-section. This reinforcement is protected from corrosion entirely by concrete cover. Corrosion of this steel reinforcement can cause severe deterioration, reducing the service life of the piles, and consequently the service life of the entire structure.

An alternative to mild steel reinforcement is fiber reinforced polymer (FRP) reinforcement. FRP reinforcement is known to be a corrosion resistant material, and it is proposed that replacing traditional mild steel transverse reinforcement with corrosion-resistant transverse FRP reinforcement would permit reduced cylinder pile wall thicknesses, since concrete cover would no longer govern over structural design requirements. A reduced wall thickness would allow for a lighter pile, which would make precast concrete cylinder piles more competitive against alternative foundation types such as steel pipe piling and drilled shafts. Additionally, the noncorrosive behavior of FRP transverse reinforcement could significantly extend the design service life of concrete cylinder piles, as compared to piles with mild steel reinforcement.

The primary objective of the research presented in this thesis is to examine the feasibility of replacing steel transverse reinforcement in precast concrete cylinder piles with fiber reinforced polymer (FRP) transverse reinforcement. In particular, the research investigated the effects of including transverse FRP reinforcement on axial load capacity, transverse load capacity, and
longitudinal shear capacity. Longitudinal cracks, particularly in members with hollow cross-sections, can potentially reduce the moment capacity of that member.

Current industry design methods and procedures were investigated, and a thorough literature review of previous research on the use of transverse FRP reinforcement for hollow and solid concrete specimens was conducted. Six scaled concrete hollow cylinder pile segments were designed, fabricated, tested, and analyzed to evaluate the potential for using FRP transverse reinforcement. Four full-scale pile segments were spun-cast for the research, two with transverse steel reinforcement and two with transverse FRP reinforcement. One of the FRP-reinforced segments was produced with a reduced wall thickness. The full-scale pile segments were tested in the laboratory under regimens of concentric axial load, eccentric axial load, and direct transverse splitting. Segments were loaded under eccentric axial load both before and after splitting to investigate the effects of longitudinal cracking on moment capacity. Recommendations were developed for the design of precast prestressed concrete cylinder piles reinforced with transverse FRP, including recommendations for future study. Recommendations on the effect of longitudinal cracking on the moment capacity of hollow cylinder piles were also proposed.
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Behavior of Hollow Precast Concrete Cylinder Piles with Steel or FRP Transverse Reinforcement

by
Meghan Ann Strahler

A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the degree of Master of Science

Civil Engineering

Raleigh, North Carolina
2018

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_______________________________
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DEDICATION

To my husband Brandon, the bones that kept me braced.
BIOGRAPHY

Meghan Ann Strahler was born and raised in Pittsburgh, PA. In December 2013, she graduated summa cum laude with a Bachelor of Science in Civil Engineering from the University of Pittsburgh. After graduation she gained experience working full-time in industry as an Engineer in Training (EIT) performing transportation structures design. In August 2016, Meghan returned to school full-time to pursue a Master of Science in Civil Engineering degree with a focus in Structural Engineering and Mechanics at the North Carolina State University. She plans to continue her career as a bridge engineer following the completion of her graduate degree.
ACKNOWLEDGMENTS

I would like to acknowledge the Precast/Prestressed Concrete Institute (PCI) for sponsoring this research, which was funded by a Daniel P. Jenny Research Fellowship. I would like to thank Steve Seguirant for chairing the PCI industry advisory group, which provided considerable guidance and direction. I would also like to thank J.P. Binard, Chair of the PCI Piling Committee, for sharing his substantial knowledge on prestressed concrete piles. Peter Finsen and the Georgia/Carolinas PCI Chapter provided generous financial support for travel to PCI Committee meetings.

I would also like to acknowledge and thank the companies and groups within the precast concrete industry who supported this research. AltusGroup, Inc. provided generous financial support, including donation of the CFRP grid used in the scaled compression tests, and donation of the full-scale FRP-reinforced pile segments. Bayshore Concrete Products Corporation fabricated the full-scale pile segments, and offered fantastic access to their plant and personnel, providing great insight into the spin casting process. I would also like to thank Sigma DG Corporation and ALP Supply for donating some of the transverse FRP reinforcement used in the scaled compression tests.

I would like to extend my sincere gratitude to my advisor, Dr. Gregory Lucier, for his extensive guidance and continuous enthusiasm for the duration of this program. I deeply appreciate the opportunity he provided me to participate in graduate research. His belief in the potential of students is tremendous. I would also like to express my appreciation to Dr. Sami Rizkalla and Dr. Rudolf Seracino for their participation in my advisory committee, and for their direction and wisdom throughout the course of my studies.
I am greatly indebted to the staff at the Constructed Facilities Laboratory. Jerry Atkinson and Johnathan McEntire provided extensive technical support and expertise, and the experimental portion of my research could not have been completed without their support. I would also like to thank Juliet Swinea and Oscar Santoyo for their assistance with my experimental testing.

I am very fortunate and grateful for the relationships I have developed with fellow students at the Constructed Facilities Laboratory. In particular I would like to thank Dr. Armita Mohammadian and Dr. Omar Khalafalla. Their encouragement strengthened me, and I cherish their friendships.

Finally, I would like to thank my family for their endless love and support.
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CHAPTER 1: Introduction

1.1 Background

A pile is a deep foundation element with a high aspect ratio that transfers forces from a structure above into the ground. Piles are used in weak soil layers that are unsuitable for shallow foundations, and they may extend through deep water in marine environments. Precast, prestressed concrete cylinder piles are a special type of precast concrete pile with a hollow circular cross-section that have been used extensively in marine environments since the 1950s (Precast/Prestressed Concrete Institute, 2004). Typical concrete cylinder piles have diameters that range from 36” to 66” and wall thicknesses that vary from approximately 5” to 9”. Concrete cylinder piles may be cast in long lengths, or they may be cast in short segments (typically 8’ to 16’) that are subsequently joined together using a segmental construction process. With this process, longitudinal tendons are run through ducts or voids cast into the walls of the individual pile segments, and are post-tensioned and grouted, joining together the individual segments into a full-length pile. A specialized technique known as spin-casting may also be used to produce hollow cylinder piles, later joined together with the segmental construction process. Spun-cast pile segments are manufactured by placing zero-slump concrete into a steel form that is rapidly-rotating about its longitudinal axis. The concrete is distributed along the length of the form, consolidated by centrifugal force, and often roller-compacted on the inside surface. Spin-casting results in a high-density concrete with low permeability, allows for extremely low water-to-cement ratio concrete mixtures, and can easily provide high concrete strengths of up to 10 ksi or more.
Precast prestressed concrete cylinder piles provide exceptional resistance to axial and lateral loads due to their large diameters and corresponding high moments of inertia, which also provide significant global buckling resistance. High axial load capacity means fewer piles are required in a foundation, reducing construction time. Construction time can be further improved with the precast production process since pile segments can be fabricated before the final required pile lengths are determined. Segmental construction also allows for very long piles, with lengths of 150’ or greater. The high concrete strength and low permeability of spun-cast piles provide excellent durability in service, even in severe marine environments (Precast/Prestressed Concrete Institute, 2004).

The 17.6 mile long Chesapeake Bay Bridge-Tunnel (CBBT) in Virginia, built from 1960 to 1964, and the 23.8 mile long causeway across Lake Pontchartrain in Louisiana, built in 1955, are two of the most famous examples of structures founded on concrete cylinder piles. The CBBT contains nearly five-thousand 54” diameter spun-cast cylinder piles, and the causeway contains nearly ten-thousand 54” diameter spun-cast cylinder piles. A more recent structure, the Inner
Harbor Navigation Channel (IHNC) Lake Borgne Surge Barrier in Louisiana, completed in 2013 and part of the greater New Orleans Hurricane and Storm Damage Risk Reduction System, used over one-thousand 66” diameter spun-cast piles. It should be noted that all three of these structures are located in harsh marine environments.

Figure 1-2: Typical Concrete Cylinder Pile Foundation (Hartman, Castelli, & Malhotra, 2007)

Concrete cylinder piles contain steel reinforcement in the form of longitudinal post-tensioning tendons and transverse reinforcement, which reinforces the hoop direction of the hollow cylindrical cross-section. Corrosion of this steel reinforcement in prestressed concrete piling can cause severe deterioration. As a result, the service life of the piles, and consequently the service life of the entire structure, may be reduced. In post-tensioned cylinder piles produced with the segmental process, the longitudinal tendons are often well-protected from corrosion by the grout (which may contain corrosion inhibitors), the option to use plastic post-tensioning ducts, and the substantial concrete cover over the reinforcement. Transverse reinforcement, however, is protected
from corrosion entirely by the concrete cover, and necessarily sits closer to the outer concrete surface than the longitudinal tendons. The American Association of State Highway and Transportation Officials (AASHTO) and many state Departments of Transportation (DOTs) require minimum concrete cover requirements for corrosion protection. As a result, cylinder pile wall thickness is often governed by cover requirements instead of structural requirements. Stated otherwise, the wall thickness required to meet cover requirements is often not required for structural performance.

An alternative to mild steel reinforcement is fiber reinforced polymer (FRP) reinforcement, a composite material made of fibers embedded in a polymeric resin. Fiber types include continuous aramid fiber (AFRP), carbon fiber (CFRP), glass fiber (GFRP), and basalt fiber (BFRP). FRP reinforcement is known to be a corrosion resistant material, and substantial literature exits on this topic (ACI Committee 440, 2015). It should be noted that a detailed review of the corrosion resistance of FRP reinforcement is outside the scope of this thesis. It is proposed that replacing traditional mild steel transverse reinforcement with corrosion-resistant transverse FRP reinforcement would permit reduced cylinder pile wall thicknesses, since concrete cover would no longer govern over structural design requirements. A reduced wall thickness would allow for a lighter pile, which would make precast concrete cylinder piles more competitive against alternative foundation types such as steel pipe piling and drilled shafts. Additionally, the noncorrosive behavior of FRP transverse reinforcement could significantly extend the design service life of concrete cylinder piles, as compared to piles with mild steel reinforcement. Cylinder piles are typically designed to resist loads through skin friction, so a reduction in wall thickness should not negatively impact their potential to develop a resistance mechanism with the surrounding soils.
1.2 Objective

The primary objective of the research presented in this thesis is to examine the feasibility of replacing steel transverse reinforcement in precast concrete cylinder piles with fiber reinforced polymer (FRP) transverse reinforcement. In particular, the research aims to investigate the effects of including transverse FRP reinforcement on axial load capacity, transverse load capacity, and longitudinal shear capacity. Longitudinal cracks, particularly in member with a hollow cross-section, can potentially reduce the moment capacity of that member.

1.3 Program Scope

The experimental work outlined below was completed in order to achieve the research objectives:

- Current industry design methods and procedures were investigated, and a thorough literature review of previous research on the use of transverse FRP reinforcement for hollow and solid concrete specimens was conducted.
- Six scaled concrete hollow cylinder pile segments were designed, fabricated, tested, and analyzed in order to evaluate the potential for using FRP transverse reinforcement, and to demonstrate the impacts (if any) of FRP reinforcement on axial load capacity.
- Four full-scale pile segments were spun-cast for the research, two with transverse steel reinforcement and two with transverse FRP reinforcement. One of the FRP-reinforced segments was produced with a reduced wall thickness.
- The full-scale pile segments were tested in the laboratory under regimens of concentric axial load, eccentric axial load, and direct transverse splitting. Segments were loaded under eccentric axial load both before and after splitting to investigate the effects of longitudinal cracking on moment capacity.
Recommendations were developed for the design of precast prestressed concrete cylinder piles reinforced with transverse FRP, including recommendations for future study. Recommendations on the effect of longitudinal cracking on the moment capacity of hollow cylinder piles were also proposed.

1.4 Overview of Thesis

The background, objective, and scope of the research are introduced in Chapter 1. A literature review which outlines previous research related to precast cylinder piles, the use of transverse FRP, and current industry design methods is in presented in Chapter 2. Chapter 3 describes the scaled compression tests portion of the research, including fabrication of the test specimens, test setup and instrumentation, and analysis and discussion. The full-scale tests portion of the research, including test setups and instrumentation, and analysis and discussion, are presented in Chapter 4. The findings of the research and recommendations for future work are summarized in Chapter 5.
CHAPTER 2: Literature Review

2.1 Introduction

This chapter provides an overview of the design process, fabrication, and installation of prestressed precast cylinder piles. Additionally, this chapter reviews published research relevant to transverse reinforcement in circular concrete members.

2.2 Design Process of Prestressed Precast Cylinder Piles

2.2.1 Historic Design and Current Applicable Design Guides

Design methods for prestressed concrete piles have been developed by the precast concrete industry through a process of trial and error. Historically, prestressed piles were designed to primarily resist axial loads and were subjected to little or no bending moment. Acceptable standard practice was developed based on experience, and generally included adhering to minimum requirements. Such minimum requirements included limiting the concentric service axial load on the section, providing a minimum axial compressive stress of 700 psi, and satisfying requirements for minimum spiral reinforcement (Precast/Prestressed Concrete Institute, 2004). Allowable concentric service loads for piles that were fully embedded, laterally supported, and subjected only to minor bending moments due to accidental eccentricities, were first published in the 1970s based on the following equation. Although this formula is of historical importance and serves as a useful serviceability check, modern design codes impose strength design methods for pile design.

\[
f_a \leq 0.33 f'_c - 0.27 f_{pc}
\]  

(Eq. 20.5.1.1-1, Ch. 20, PCI Bridge Design Manual, 2004)

Pile design has evolved through experience and industry practice, and as a result, current pile design procedures have not yet been fully incorporated into a single design specification. Existing references for the design of prestressed concrete piles include the Precast/Prestressed Concrete Institute (PCI) publication “Precast Prestressed Concrete Piles” (Chapter 20 of the
September 2004 version of the *PCI Bridge Design Manual*, published separately); ACI 318-11: Building Code Requirements for Structural Concrete; ACI 543R-12: Guide to Design, Manufacture, and Installation of Concrete Piles; the American Association of State and Highway Transportation Officials LRFD Bridge Design Specifications (AASHTO LRFD); and PCI’s “Recommended Practice for Design, Manufacture, and Installation of Prestressed Concrete Piling” (1993, n.d.). PCI’s “Precast Prestressed Concrete Piles” presents common practice and details for the design of prestressed concrete piles, and does not itself constitute a design standard; instead, AASHTO LRFD governs for the design of prestressed bridge piles. Specifically, design requirements are given in Article 5 "Concrete Structures", which concerns structural design requirements, and Article 10.7 "Driven Piles", which concerns geotechnical design requirements. Because AASHTO LRFD addresses pile design under the general category of concrete structures, PCI’s “Precast Prestressed Concrete Piles” outlines the method of applying AASHTO LRFD requirements to the specific design of prestressed concrete piles. It should be noted that although state DOTs adhere to the AASHTO LRFD Specifications, some state agencies may have developed provisions that supplement or supersede specific AASHTO LRFD requirements.

### 2.2.2 Design Considerations

The ultimate capacity of a pile must be checked for both structural and geotechnical failure. Geotechnical failure limit states include bearing capacity failure, excessive settlement, and excessive horizontal displacement. Failure of the pile-soil system often governs the strength of the pile design (Precast/Prestressed Concrete Institute, 2004). A geotechnical engineer must be involved in the pile design process to ensure that geotechnical design requirements are fulfilled. This thesis focuses on the structural design of prestressed concrete piles, and geotechnical design considerations are outside of the research scope.
2.2.3 Structural Design Procedure

AASHTO LRFD Article 5.13.4.4 "Precast Prestressed Piles" presents several design provisions specific to prestressed piles, however, beyond this article, the structural design for prestressed piles is based on the more general axial, flexural, and shear design provisions within Article 5. The structural design of precast prestressed concrete piles must consider allowable stress design, strength design, confinement requirements, and concrete cover requirements. These requirements are summarized in the following paragraphs.

2.2.3.1 Allowable Stress Design

As with many precast concrete members, the stresses experienced by prestressed piles during handling and installation are often greater than the stresses the pile will experience once in service. This is especially true for driving stresses. During design, it is important to verify that the driving force required to develop the design capacity of the pile can be achieved without damaging the pile. The project geotechnical engineer should provide guidance on installation requirements. More background on driving stresses is provided in the Section 2.3.2 which discusses the installation of prestressed precast cylinder piles. AASHTO LRFD Article 5.13.4.4.2 requires a minimum concrete strength of 5.0 ksi at the time of driving, although this strength is easily exceeded by modern pile producers.

Prestressed piles should be handled, stored, and transported in a manner to prevent structural damage to the pile. AASHTO LRFD provides stress limits for handling and transportation and erection stresses. Adequate measures to ensure stresses are kept within allowable limits include lifting piles at predetermined points and using appropriately-spaced dunnage. To prevent cracking during handling and transportation, AASHTO LRFD Article 5.13.4.4.3 “Reinforcement” requires that a minimum effective prestress of 700 psi after losses be
provided. Because the chance of developing tensile stresses during driving increases with pile length, prestress levels up to 1200 psi are sometimes used (PCI Committee on Prestressed Concrete Piling, n.d.). AASHTO LRFD also limits the allowable service stress in prestressed concrete piles after installation. AASHTO LRFD stress limits for transportation and erection, handling, driving, and service stresses are summarized in the Table 2-1.

<table>
<thead>
<tr>
<th>Stress Limit Description</th>
<th>Allowable Stress Limit (ksi)</th>
<th>AASHTO LRFD Article</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temporary Stresses before Losses - Handling Stresses, Tension (compute stresses using a 50% increase in pile weight for impact)</td>
<td>$0.158\lambda\sqrt{f'_{ci}}$</td>
<td>Table A5.9.4.1.2-1, A5.13.4.1</td>
</tr>
<tr>
<td>Temporary Stresses before Losses - Handling Stresses, Compression (compute stresses using a 50% increase in pile weight for impact)</td>
<td>$0.65f'_{ci}$</td>
<td>A5.9.4.1.1, A5.13.4.1</td>
</tr>
<tr>
<td>Driving Stresses - Compression</td>
<td>$\varphi_{da}(0.85f'<em>{c} - f</em>{pe})$</td>
<td>A10.7.8</td>
</tr>
<tr>
<td>Driving Stresses - Tension, Normal Environments</td>
<td>$\varphi_{da}(0.95f'<em>{c} + f</em>{pe})$</td>
<td>A10.7.8</td>
</tr>
<tr>
<td>Driving Stresses - Tension, Severe Corrosive Environments</td>
<td>$\varphi_{da}f_{pe}$</td>
<td>A10.7.8</td>
</tr>
<tr>
<td>Service Stresses after Losses - Compression due to Effective P/S and Permanent Loads</td>
<td>$0.45f'_{c}$</td>
<td>Table A5.9.4.2.1-1</td>
</tr>
<tr>
<td>Service Stresses after Losses - Compression due to Effective P/S, Permanent Loads, and Transient Loads, as well as during Shipping and Handling</td>
<td>$0.60\varphi_{w}f'_{c}$</td>
<td>Table A5.9.4.2.1-1</td>
</tr>
<tr>
<td>Service Stress, Tension - Moderate Corrosive Conditions</td>
<td>$0.19\lambda\sqrt{f'_{c}} \leq 0.6$</td>
<td>Table A5.9.4.2.2-1</td>
</tr>
<tr>
<td>Service Stress, Tension - Severe Corrosive Conditions</td>
<td>$0.0948\lambda\sqrt{f'_{c}} \leq 0.3$</td>
<td>Table A5.9.4.2.2-1</td>
</tr>
</tbody>
</table>
Where $\lambda$ is the concrete density modification factor specified in Article 5.4.2.8, $f'_{ci}$ is the specified compressive strength of concrete at the time of initial prestressing (ksi), $\varphi_{da}$ is the resistance factor specified in Table A10.5.5.2.3-1 and Article 5.5.4.2.1, $f'_c$ is the specified compressive strength of concrete (ksi), $f_{pe}$ is the effective prestressing stress in concrete (ksi), and $\varphi_w$ is the reduction factor specified in Article 5.7.4.7.2.

### 2.2.3.2 Strength Design

In addition to meeting allowable stress limits, prestressed piles must also satisfy strength design requirements provided in AASHTO LRFD. In accordance with AASHTO LRFD Article 5.13.4.1, a pile that does not have adequate lateral support to prevent buckling, as is the case for a prestressed pile which extends through water, must be designed as a column. Slenderness effects that may amplify an applied moment must also be considered. Shear capacity must also be considered, although it usually does not govern for typical pile designs.

Interaction between axial and bending forces must be considered for the column design of a prestressed member. AASHTO LRFD provides equations for computing resistance under pure axial load and pure flexure, but the specifications do not provide explicit direction concerning resistance for combined axial loads and flexure. However, Article 5.7.2.1, which falls under the Article 5.7 “Design for Flexural and Axial Force Effects,” states “factored resistance of concrete components shall be based on the conditions of equilibrium and strain compatibility”. The strain compatibility method can be used to determine the strength of a member for a chosen prestressing pattern by computing concrete and prestressing steel strains at various levels in a given cross-section for an assumed neutral axis, applying stress-strain relationships, and requiring equilibrium of forces. This process can be used to generate interaction diagrams by iterating the assumed neutral axis location. Capacities listed on commonly-used interaction diagrams are limited by the
maximum axial compressive capacity specified in AASHTO LRFD Article 5.7.4.4 and are reduced by the appropriate resistance factors given in AASHTO LRFD Article 5.5.4.2. The intent of limiting the maximum axial compressive capacity is to account for unintentional eccentricity. Once generated, an interaction diagram can be used to check multiple load cases.

As mentioned above, when piles are not laterally supported a designer must consider slenderness effects in order to prevent buckling. When $\frac{k}{r}$ is less than 100, where $l_u$ is the unsupported length, $r$ is the radius of gyration, and $k$ is the effective length factor, the factored axial load and a magnified factored moment should be compared to the capacity determined by the strain compatibility procedure. When $\frac{k}{r}$ is greater than 100, a more detailed second-order analysis must be used instead of the approximate moment magnification procedure. AASHTO LRFD provides procedures for calculating magnified moment in Articles 5.7.4.3 and 4.5.3.2.2, including equations that adjust the flexural rigidity, $EI$, to account for cracking and time-dependent effects. These equations were developed for reinforced concrete columns and consequently, may overestimate the amount of cracking for prestressed piles; however, they are acceptable for the conservative design of prestressed concrete compression members (Precast/Prestressed Concrete Institute, 2004).

To facilitate design, PCI has developed a spreadsheet program to calculate interaction diagrams for precast, prestressed concrete piles (Precast/Prestressed Concrete Institute, 2015). The program considers slenderness effects, and can be customized for pile size, concrete strength, and prestressing strand size and pattern. Allowable stress design checks are also performed. Additionally, pile producers typically provide interaction diagrams for the specific cross-sections and reinforcement patterns they offer, aiding designers in the selection of an appropriate pile size.
Also mentioned above, shear must be considered in the design of piles, although it does not usually govern. At a minimum, the confinement reinforcement requirements outlined in the next section must be met. AASHTO LRFD shear provisions are presented in Article 5.8. These provisions were developed for flexural members, however, they are also applied to compression members, but this application can be cumbersome (Precast/Prestressed Concrete Institute, 2004). The “General Procedure” method presented in Article 5.8.3.4.2 is based on Modified Compression Field Theory (MCFT), a comprehensive behavioral model for the response of cracked reinforced concrete subject to shear forces, and it applies to prestressed concrete piles. Guidance is provided to calculate two of the required shear provision parameters for circular cross-sections. The effective web width, \( b_v \), may be taken equal to the diameter of the section (reduced for the presence of ducts). The effective shear depth, \( d_v \), is equal to the distance between the resultants of the tensile and compressive forces due to flexure. For circular members, \( d_v \) may be taken as \( 0.9d_e \), where \( d_e \) is defined as shown below:

\[
d_e = \frac{D}{2} + \frac{D_r}{\pi}
\]

(Eq. C5.8.2.9-2, AASHTO LRFD, 2014)

![Figure 2-1: Illustration of Terms for Circular Sections (Fig. C5.8.2.9 from AASHTO LRFD, 2014)](image)

The shear capacity of hollow circular cross-sections is not specifically addressed by AASHTO LRFD. Limited studies investigating the shear strength of hollow circular reinforced concrete sections are reviewed in Section 2.5. It is also worth noting that cylinder piles which are
designed for vessel impact may have their void filled after installation to prevent puncture or collapse of the section.

2.2.3.3 Confinement Reinforcement

AASHTO LRFD Article 5.13.4.4.3 specifies prescriptive confinement reinforcement for precast prestressed concrete piles, which must be provided along the entire length of the pile. This continuous wire spiral provides lateral confinement for solid piles and is intended to control cracks that may form due to handling, driving, or design loads. AASHTO LRFD Article 5.8.2.8 “Design and Detailing Requirements” limits the yield strength for this transverse reinforcement to 75 ksi.

For piles not greater than 24.0" in diameter, AASHTO LRFD specifies the following minimum requirements: spiral wire not less than W3.4 (A_{sp} = 0.034 in^2); spiral reinforcement at the ends of piles having a pitch of 3.0” for approximately 16 turns; the top 6.0” of pile having five turns of additional spiral winding at 1.0” pitch; and for the remainder of the pile, the strands enclosed with spiral reinforcement with not more than 6.0” pitch.

For piles greater than 24.0" in diameter, AASHTO LRFD specifies the following minimum requirements: spiral wire not less than W4.0 (A_{sp} = 0.04 in^2); spiral reinforcement at the end of the piles having a pitch of 2.0” for approximately 16 turns; the top 6.0” having four additional turns of spiral winding at a 1.5” pitch; and for the remainder of the pile, the strands must be enclosed with spiral reinforcement having not more than a 4.0” pitch. These minimum requirements are illustrated in Figures 2-2 and 2-3.
Additional confinement reinforcement requirements for Seismic Zones 2, 3, and 4 are presented in AASHTO LRFD Article 5.13.4.6. The equations for the minimum spiral volumetric ratio, $\rho_s$, required in Seismic Zones 3 and 4 were derived from what are commonly referred to as the “ACI spiral equations”. ACI 318 Section 10.9.3 determines the required steel spiral reinforcement with an assumption that in the event the concrete cover spalls off, the strength enhancement from the confined concrete core will be sufficient to replace the strength lost from the spalled concrete. It should be noted that the ACI spiral equations were developed for solid pile sections. Additional research is needed on hollow piles during seismic events, and extrapolation of these equations to hollow pile sections is not recommended. In general, however, hollow piles are not typically recommended or used in areas of moderate to high seismicity.

As mentioned in Section 2.2.1, although state DOTs adhere to the AASHTO LRFD Specifications, some agencies may have developed provisions that supplement or supersede specific AASHTO requirements. It should be emphasized that the prescriptive spiral reinforcement...
requirements outlined above are minimum requirements, and in practice, cylinder piles are commonly fabricated with a continuous spiral pitch as tight as 2”.

In addition to what is outlined above, there are some additional requirements for the design of cylinder piles. Cylinder piles can experience significant bursting forces during driving, particularly when subjected to hard driving. PCI’s “Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling” (1993, n.d.) has found that providing about 1% spiral reinforcement in the top 12” of a pile has successfully prevented splitting of the cylinder pile head for piles up to 54” in diameter. Due to the possible formation of a soil plug and internal pressure from driving, additional spiral reinforcement should also be provided at the pile tip to protect against bursting (Hartman, Castelli, & Malhotra, 2007). AASHTO LRFD Article 5.13.4.4.1 states that additional precautionary measures, such as venting, should be taken to reduce the potential for widespread internal pressure to develop. It is further recommended that test piles be driven in advance of production piles using the planned driving equipment to confirm that cylinder piles can be driven as designed without cracking (Precast/Prestressed Concrete Institute, 2004).

2.2.3.4 Concrete Cover Requirements

Concrete cylinder piles are often used in harsh marine environments where corrosion of the steel reinforcement is a concern. The longitudinal post-tensioning tendons are protected from corrosion by grout (which sometimes contains corrosion inhibitors), plastic post-tensioning ducts (if used), and concrete cover to the reinforcement. The transverse reinforcement is only protected from corrosion by its concrete cover. AASHTO LRFD Table A5.12.3-1 specifies a minimum concrete cover of 2” for precast prestressed piles for main reinforcement. For precast prestressed piles, no distinction is made between noncorrosive and corrosive environments. Cover
requirements may be reduced by a 0.8 modification factor for water-to-cement ratios ≤ 0.40. (Water-to-cement ratios for spun-cast piles can be as low as 0.24). AASHTO LRFD states that the concrete cover for ties and stirrups may be 0.5” less than the concrete cover required for the main reinforcing steel, although spiral reinforcement is not specifically mentioned. “Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling” (1993, n.d.) recommends more conservative concrete cover values of 2” for normal environments and 2.5” for marine or similar corrosive environments.

Some state DOTs require a greater concrete cover than that specified by AASHTO LRFD. For example, Florida DOT (FDOT) provides a set of requirements specifically for cylinder piles. FDOT also provides standard design drawings for 54” and 60” diameter precast concrete cylinder piles. Cylinder pile cross-sections for 54” and 60” diameter precast concrete cylinder piles from the FDOT standard drawings are presented in Figures 2-4 and 2-5.

Figure 2-4: 54” Precast/Post-Tensioned Concrete Cylinder Pile (FDOT Design Standards, Index 20654)
As can be seen in the figures, an 8.125” wall thickness is required for a 54” cylinder pile and a 7.5” wall thickness is required for a 60” cylinder pile; FDOT does permit a reduced cover requirement for 54” cylinder piles that are spun cast if certain chloride ion penetration requirements are met, as described in the notes in the figure above. The FDOT prescribed wall thicknesses with minimum cover requirements control over the minimum 5.0” minimum wall thickness specified in AASHTO LRFD Article 5.13.4.4.1. As with the FDOT designs, the selection of cylinder pile wall thickness is often controlled by concrete cover requirements, not by structural performance requirements.

Concrete cover requirements for several other coastal state DOTs have been investigated and are summarized in Table 2-2. It is worth noting that these requirements may be for marine environments or substructure components in general, and are not always exclusive to cylinder piles.
<table>
<thead>
<tr>
<th>State DOT</th>
<th>Design Document(s)</th>
<th>Concrete Cover Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>FDOT (Florida)</td>
<td>FDOT Structures Design Guide (SDG) Table 1.4.2-1, Design Standards Index 20654, Index 20660, Index 22654, Index 22660</td>
<td>3&quot; for prestressed piling for slightly, moderate, or extremely aggressive environments; 2&quot; for spun cast cylinder piling for slightly or moderately aggressive environments; 2&quot; for spun cast cylinder piling for extremely aggressive environments that have a documented chloride ion penetration apparent diffusion coefficient with a mean value of 0.005 in²/yr or less, otherwise 3&quot;</td>
</tr>
<tr>
<td>NCDOT (North Carolina)</td>
<td>NCDOT Structures Unit Management (SMU) Manual</td>
<td>no state specific requirements</td>
</tr>
<tr>
<td>VDOT (Virginia)</td>
<td>VDOT Modifications to AASHTO LRFD Bridge Design Specifications, Sheet 8 of 14</td>
<td>2.5&quot; for principal reinforcement and 2&quot; for stirrups and ties for other components, normal conditions; 3.5&quot; for principal reinforcement and 3&quot; for stirrups and ties for other components, for corrosive or marine environments</td>
</tr>
<tr>
<td>LaDOTD (Louisiana)</td>
<td>LaDOTD Bridge Design and Evaluation Manual (BDEM), Section 5.12.3</td>
<td>3&quot; for substructure components</td>
</tr>
<tr>
<td>ALDOT (Alabama)</td>
<td>ALDOT Structural Design Manual</td>
<td>no state specific requirements</td>
</tr>
<tr>
<td>NJDOT (New Jersey)</td>
<td>NJDOT Design Manual for Bridges and Structures</td>
<td>no state specific requirements</td>
</tr>
<tr>
<td>TxDOT (Texas)</td>
<td>TxDOT Bridge Design Manual – LRFD</td>
<td>no state specific requirements</td>
</tr>
<tr>
<td>SCDOT (South Carolina)</td>
<td>SCDOT Bridge Design Manual Figure 15.3-2, SCDOT Geotechnical Design Manual, Section 16.3</td>
<td>2.25&quot; for prestressed concrete piles; cylinder piles are currently being evaluated for inclusion in the pile types used by SCDOT</td>
</tr>
<tr>
<td>MDOT (Mississippi)</td>
<td>MDOT Bridge Design Manual</td>
<td>no state specific requirements</td>
</tr>
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</table>
Table 2-2 (continued)

<table>
<thead>
<tr>
<th>Agency</th>
<th>Specification</th>
<th>Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>NYSDOT (New York)</td>
<td>NYSDOT Bridge Manual, Section 15.3</td>
<td>2&quot; for precast piles; 3&quot; for precast piles exposed to sea water; 1.5&quot; for post-tensioned cylinder piles, centrifugally cast, no slump concrete, exposed to sea water</td>
</tr>
<tr>
<td>MassDOT (Massachusetts)</td>
<td>LRFD Bridge Manual</td>
<td>no state specific requirements</td>
</tr>
<tr>
<td>GDOT (Georgia)</td>
<td>GDOT Bridge and Structures Design Manual, Section 4.2.2.4</td>
<td>no specific requirements for piles; only square prestressed concrete piles to be specified for Bridge Foundation Investigation (BFI) report</td>
</tr>
<tr>
<td>RIDOT (Rhode Island)</td>
<td>Rhode Island LRFD Bridge Design Manual, Section 5.9.1</td>
<td>no state specific requirements</td>
</tr>
<tr>
<td>DelDOT (Delaware)</td>
<td>DelDOT Bridge Design Manual, Table 205.12.3-1</td>
<td>3&quot; for precast prestressed concrete piles</td>
</tr>
<tr>
<td>ConnDOT (Connecticut)</td>
<td>ConnDOT Bridge Design Manual</td>
<td>no state specific requirements</td>
</tr>
<tr>
<td>DDOT (District of Columbia)</td>
<td>DDOT Design and Engineering Manual, Section 16.7.4</td>
<td>no state specific requirements</td>
</tr>
<tr>
<td>MDSHA (Maryland)</td>
<td>MDSHA Book of Standards – For Highway &amp; Incidental Structures</td>
<td>no state specific requirements</td>
</tr>
</tbody>
</table>

2.3 Installation of Prestressed Precast Cylinder Piles

2.3.1 Overview of Installation

The stresses imposed on a pile during handling and installation are usually more severe than stresses experienced in service. Each pile must be able to structurally withstand being driven to the required bearing capacity. The equipment used for prestressed pile installation includes hammers, leads, driving helmets, and pile cushioning. The specific properties of the equipment, such as hammer weight, vary significantly among different models of equipment. The driving
stresses a pile experiences are directly (but not exclusively) related to the specific driving equipment chosen.

The project geotechnical engineer should provide guidance on pile installation criteria. Soil conditions and local experience should guide selection of the driving equipment. Furthermore, dynamic analysis, such as wave equation analysis, should be used to predict the stresses induced in a pile during driving prior to installation. This analysis should be conducted to ensure driving stresses will not exceed allowable values. Dynamic testing can then be deployed during installation of a test pile to monitor the actual driving stresses.

Driving equipment may be supported on barges or by temporary pile-supported work platforms when piles are driven in water. A template of steel beams or similar may be used to align piles during driving. Smaller-diameter piles installed in water often need to be stayed until they are connected into the final structure to protect against bending from wave action, currents, and accidental impact (Precast/Prestressed Concrete Institute, 2004). Piles should not be pulled into position to correct misalignment, as this can cause structural damage due to induced bending; this is especially true for underwater installations where the lever arm is especially long.

Cylinder piles can sometimes be very difficult to drive (Hannigan, Goble, Likins, & Rausche, 2006). Jetting, which involves the use of a water jet near the tip or perimeter of the pile to loosen the soil and reduce friction along the pile-soil interface to ease pile installation, is sometimes used to aid driving. Internal jets should be avoided for cylinder piles since a broken jet could pressurize the inside of the pile, causing it to burst. If required, external jets or predrilling can be used for cylinder piles (Hartman et al., 2007).
The following sections describe types of pile damage that may occur during installation, including longitudinal and transverse cracking. During the course of the literature review, little documentation was found on allowable crack widths for prestressed piling.

2.3.2 Driving Stresses and Transverse Cracking

Transverse cracking during pile driving can occur due to the complicated phenomenon of reflected tensile stresses. When the hammer strikes the head of a pile, a compressive stress wave is produced which propagates down the length of the pile. Depending on the soil resistance at the pile tip, this wave can reflect back up the pile, creating either a tensile stress wave or a compressive stress wave of slightly reduced intensity. If there is very hard soil resistance at the pile tip, the wave will be reflected back up the pile as a compressive stress wave. Once this compressive stress wave reaches the pile head it will be reflected back towards the pile tip as a tensile stress wave. However, if there is soft soil resistance at the pile tip, the compressive stress wave is reflected back up the pile as a tensile stress wave. The net longitudinal stress in the pile is the algebraic sum of the self-weight, the applied prestressing, any applied external loads, and all of the stress waves simultaneously propagating along the length of the pile. The pile is subjected to net tensile stress (ie: can crack) when the magnitude of the reflected tensile stress wave exceeds the magnitude of any simultaneously propagating compressive stress waves by a level large enough to offset the effects of prestressing and self-weight.

In addition to the soil resistance at the pile tip, the formation of a critical tensile stress depends on the magnitude and length of the initial compressive stress wave relative to the length of the pile. For a stress wave that is longer than twice the pile length, the reflected tensile stress wave will overlap the propagating compressive stress wave for the entire length of the pile, and
the net stress experienced by the pile will be reduced. As a result, transverse cracks can often occur in longer piles (50’ or more in length) that encounter soft soil resistance (Smith, 1960).

The amplitude of the stress wave depends on the hammer’s mass and impact velocity, cushioning material and thickness, pile characteristics, and soil resistance. A hammer that strikes a pile at a very high velocity creates a stress wave of high amplitude because the stress developed in the pile is proportional to the hammer’s velocity. Additionally, the period of the stress wave depends in part on the length of time the hammer is in contact with the pile head. A longer contact time results in a longer stress wave period, which reduces the driving stress (Precast/Prestressed Concrete Institute, 2004).

For these reasons, to reduce the likelihood of damaging a pile during installation it is generally preferable to use a heavier hammer with a lower impact velocity as opposed to a lighter hammer with a higher impact velocity. A lower impact velocity is achieved by using a shorter stroke (fall height) (ACI Committee 543, 2012). The likelihood of pile damage can be further prevented by using a soft, thick cushion which increases the length of time the hammer is in contact with the pile head and helps to uniformly distribute the force from the hammer (Precast/Prestressed Concrete Institute, 2004). In particular, these measures should be taken during early driving when soft soil resistance is generally encountered and when driving through soft soil is anticipated.

### 2.3.3 Longitudinal Cracks and Splitting

Longitudinal cracks in a concrete pile may develop for several reasons. Release of the prestressing tendons causes expansion of the tendon diameter, which creates radial expansion stresses, especially near the ends of the piles. These radial stresses may produce longitudinal cracks originating near the ends of the tendons. Compressive stresses that travel down the pile during driving cause the pile to expand due to Poisson’s effect. As a result, hoop stresses may develop
and cause longitudinal cracking (Hartman et al., 2007). The expansion of freezing water inside the pile void can also cause cracking.

When hollow cylinder piles are driven in water, or in extremely soft, semi-fluid soils, dynamic stresses from the hammer greatly increase the pressure of the water or saturated soils within the pile. If this pressure exceeds the bursting capacity of the pile, longitudinal splitting may occur. This hydraulic ram effect is known as “water hammer”. Longitudinal cracks from water hammer can often be avoided by providing weep holes to relieve the pressure during driving.

Another possible cause of longitudinal cracks is plugging of the pile tip. A “mud plug” at the pile tip may cause tensile hoop stresses to develop that exceed the tensile strength of the concrete, causing longitudinal cracking. These cracks can be controlled by appropriate transverse reinforce and periodically cleaning out the pile tip during driving (Hartman et al., 2007). Material can be cleaned by using high-pressure air, a combination of steam and water, or by drilling within the driven pile (ACI Committee 543, 2012). The void at the pile tip may also be covered with a sealed (watertight) plywood plug at the start of driving to help prevent the internal buildup of soft material (including water) during initial driving.

### 2.3.4 Other Types of Pile Damage

Additional causes of pile damage that may occur during driving include torsional cracking and spalling at the pile head or pile tip. Diagonal cracks can result from the application of a twisting moment about the longitudinal axis of the pile. This may occur due to misalignment of the hammer with the pile head, or due to excessive restraint from a driving helmet that fits too tightly and prevents the pile from freely rotating during driving (Hartman et al., 2007). Misalignment of the hammer with the pile head, inadequate spiral reinforcement, insufficient cushioning, and insufficient concrete cover may all attribute to compressive stress concentrations that result in
spalling at the pile head (Hartman et al., 2007). Furthermore, situations of high driving resistance where a pile does not bear evenly on the ground may result in eccentric loading that causes stress concentrations and results in spalling at the pile tip.

2.4 Previous Research on Confinement of Circular Concrete Members

2.4.1 Overview of Confinement

When a solid concrete column is loaded under axial compression, it will shorten longitudinally and expand laterally due to Poisson’s effect. Transverse reinforcement, traditionally in the form of steel ties or spirals, restrains this expansion. The transverse reinforcement, which is in tension, places the concrete core in a state of triaxial compression. As a result, the compressive strength of the concrete is enhanced, there is an increase in the strain that corresponds to the maximum compressive strength, and there is increased ductility because the concrete column can sustain large deformations without a significant loss of strength. A comparison of the stress-strain curve for confined versus unconfined concrete is shown in Figure 2-6.

Figure 2-6: Confined and Unconfined Concrete Stress–Strain Curve (Mander et al., 1988a)
Researchers have proposed many models to describe the stress-strain curve of confined concrete. Considere (1903) first proposed the concept of using spiral reinforcement in concrete columns. Initial research into this idea was undertaken in the early 1900s. Several studies performed by Richart et al. (1928, 1929, 1934) investigated the effects of using lateral fluid pressure and spiral steel reinforcement in concrete cylinders under axial compression. These studies concluded that a confining material which restrained lateral expansion of concrete resulted in an increase in concrete compressive strength and increased member ductility. From these investigations, the following equations were proposed for the maximum confined concrete strength, $f_{cc}'$, and strain at peak compressive strength, $\varepsilon_{cc}$:

\[
\begin{align*}
    f_{cc}' &= f_{co}' + k_1 f_l \\
    \varepsilon_{cc} &= \varepsilon_{co} \left( 1 + k_2 \left( \frac{f_l}{f_{co}'} \right) \right)
\end{align*}
\]

Where $f_{co}'$ is the unconfined compressive strength, $f_l$ is the lateral confining stress, and $\varepsilon_{co}$ is the unconfined strain that corresponds to $f_{co}'$. The coefficients $k_1$ and $k_2$ are confinement effectiveness parameters which were determined to be 4.1 and 5.

Since Richart et al., many more concrete confinement models have been developed, including models proposed by Kent and Park (1971), Sheik and Uzmeri (1980), and Mander et al. (1988a, 1988b). Most confinement models have been proposed based on theory and experimentation with solid cross-sections under axial compression. It has been established that circular spirals are much more effective at confining concrete than are rectilinear ties.

One of the more widely adopted models is that proposed by Mander et al. (1988b). This model, which can be used for circular or rectangular cross-sections, defines an effective lateral confining stress which reduces the lateral confining stress provided by the transverse reinforcement. This effective lateral confining stress depends on the arching action of the concrete.
between discrete ties, so the effectiveness with which a concrete core can be confined depends heavily on the configuration of the transverse reinforcement. The arching action is illustrated in Figure 2-7.

![Figure 2-7: Effectively Confined Core for Circular Hoop Reinforcement (Mander et al., 1988a)](image)

The equations developed by Mander et al. to calculate the maximum confined concrete strength, $f'_{cc}$, and strain at peak compressive strength, $\varepsilon_{cc}$ are provided below:

\[
f'_{cc} = f'_{co} \left( -1.254 + 2.254 \sqrt{1 + \frac{7.94f'_l}{f'_{co}} - 2 \frac{f'_l}{f'_{co}}} \right)
\]

\[
\varepsilon_{cc} = \varepsilon_{co} \left[ 1 + 5 \left( \frac{f'_{cc}}{f'_{co}} - 1 \right) \right]
\]

Where $f'_l$, the effective lateral confining stress, is given by:

\[
f'_l = f_l k_e
\]

\[
f_l = \frac{1}{2} k_e \rho_s f_{yh}
\]
Where \( \rho_s \) is the ratio of the volume of the transverse confining steel to the volume of the confined concrete core and \( k_e \) is a confinement effectiveness coefficient which is equal to the area of the effectively confined concrete core to the area of the concrete core enclosed by the centerline of the spiral or hoop. For circular hoops,

\[
k_e = \frac{(1 - \frac{s'}{2d_s})^2}{1 - \rho_{cc}}
\]

And for circular spirals,

\[
k_e = \frac{\left(1 - \frac{s'}{2d_s}\right)}{1 - \rho_{cc}}
\]

Where \( s' \) is the clear vertical spacing between spiral or hoop bars, \( d_s \) is the center-to-center diameter of the spiral or hoop, and \( \rho_{cc} \) is the ratio of the area of the longitudinal steel to the area of concrete core.

### 2.4.2 Confinement of Hollow Steel Reinforced Circular Concrete Members

Although the confined behavior of solid cross-sections is generally well understood, limited research has been undertaken on the effects of confinement for hollow cross-sections. Most of the experimental research conducted has focused on the flexural behavior and ductility of hollow columns under cyclic (seismic) loading. Consequently, there appears to be limited literature discussing the fundamental confinement effect for hollow members. Research that has been conducted on hollow concrete cylindrical members is summarized below.

One of the most noteworthy investigations of hollow circular columns with a single layer of transverse reinforcement was conducted by Zahn et al. (1990). Zahn et al. conducted an analytical and experimental study to investigate the available flexural strength and ductility of hollow circular columns with one layer each of longitudinal and spiral reinforcement placed near
the outside face of the cross-section. Six columns with varying wall thickness and axial load ratios were tested under axial compression and a cyclically varying lateral load. Cross-sections of the test specimens are shown in Figure 2-8. Mander et al.’s model was used to calculate the confined concrete properties, and a theoretical moment-curvature analysis which assumed the concrete to be in a state of biaxial confinement (confined by circumferential stress and axial stress but no radial stress) was found to satisfactorily predict the experimental results.

The results of this investigation found that the ductility of the columns depended significantly on the location of the neutral axis at the flexural strength. Columns with a low axial load, a moderate longitudinal steel percentage, and a thick wall were found to behave in a ductile manner. For these columns, the neutral axis at the flexural strength was located close to the inside face of the wall and resulted in a small longitudinal strain in the unconfined region of the concrete compression zone. On the other hand, columns with high axial load, a high longitudinal steel percentage, and a thin wall were found to behave in a brittle manner. For these specimens, the neutral axis was located away from the wall, toward the centroid of the section, which resulted in high longitudinal compressive strain on the inside wall face and rapid deterioration of the concrete near the unconfined inside face of the section.

![Figure 2-8: Cross-Sections of Circular Hollow Column Specimens (Zahn et al., 1990)]
In a more recent study, Hoshikuma and Priestley (2000) tested two circular hollow reinforced concrete columns to investigate seismic behavior based on flexural ductility capacity. The test specimens had different longitudinal reinforcement ratios, one layer of transverse reinforcement placed near the outside face of the section, and were tested under axial and cyclic lateral loading. The specimens, which were modeled after typical columns used for major bridges in California, had 60” outer diameters 5.5” thick walls. The wall thickness-to-section diameter ratio tested by Hoshikuma and Priestley was much smaller than the ratio for Zahn et al.

Failure of the test specimens occurred suddenly when concrete on the inside face of the wall crushed. It was concluded that ductile behavior could be achieved when the neutral axis was kept within (or near) the inside face of the concrete wall because this prevented high compressive strains from forming at the inside face. The specimen with the lower amount of longitudinal reinforcement experienced greater ductility than the specimen with the higher amount of longitudinal reinforcement. These results agree with the findings by Zahn et al. The inside face strain of the concrete was 0.005 at crushing; therefore, the authors proposed an ultimate strain of 0.005 for inside face concrete, with a safe design limit of 0.0035.

Results of the testing also found that the transverse reinforcement did not reach the yield strain when the columns failed. This suggests that the hollow columns with one layer of reinforcement were not effectively confined. Moment-curvature analyses which assumed the concrete to be biaxially confined were performed using Mander et al.’s model. The reduced pressures in the transverse steel, which did not reach yield, were accounted for in the model. The analytical results overall showed good agreement with the experimental results.

Lee et al. (2015) also investigated the seismic performance of circular hollow reinforced concrete columns under constant axial and cyclic lateral load. The 21 test specimens included
lengths of 13.1 ft or 17.7 ft, outside diameters of 39.4” and 55.1”, and inside diameters of 19.7”, 29.5”, and 38.6”. The columns had transverse reinforcement either near the outside wall, or near both the outside and inside walls. The test specimens were divided into flexure-controlled and compression-controlled specimens. Flexure-controlled specimens were defined as having a neutral axis that fell within the wall thickness at failure, whereas compression-controlled specimens had a neutral axis that fell in the column void at failure.

For the flexure-controlled specimens, the inside face of the wall did not show any damage and the columns showed very ductile behavior. The flexure-controlled specimens behaved similarly to solid circular columns. For the compression-controlled specimens, relatively brittle failure was observed due to concrete crushing and spalling of the inside face of the circular wall. For these specimens, the damaged area of the inside wall decreased as the amount of transverse reinforcement near the outside face increased, which implied that the outside transverse steel confined the concrete effectively, decreased the neutral axis at failure, and increased ductility. The damaged area on the inside wall could also be reduced by the existence of inside transverse steel, even though a significant confinement effect was not achieved. Once again, these observations of ductile or brittle behavior based on the location of the neutral axis agree with conclusions by Zahn et al.

A study by Muguruma et al. (1987) specifically investigated the flexural ductility of high-strength spun cast concrete piles. The pretensioned high-strength spun cast concrete piles, which had 15.7” diameters, 2.95” thick walls, and were 16.4 ft in length, were tested under flexural loading. The study found that one layer of transverse reinforcement improved the flexural ductility of the piles. However, it was discovered that flexural failure occurred due to fracturing of the prestressing tendons before the full confinement capability of the concrete could be developed,
and it was determined that the flexural ductility of the piles studied would have been greatly improved by using prestressing steel with a greater strain capacity.

In addition to the experimental studies outlined above, a few analytical studies have also been performed to investigate the confinement of hollow concrete members. Two companion papers by Papanikolaou and Kappos (2009a, 2009b) describe a large parametric numerical analysis of solid and hollow reinforced concrete piers. Approximately 180 circular and square, solid and hollow, members were studied. The objective of the three-dimensional nonlinear finite element analysis was to identify the most convenient confinement configuration in terms of enhanced strength, ductility, ease of construction, and cost effectiveness. Various transverse reinforcement arrangements and spacings were analyzed. The study concluded that the difference in confinement effectiveness between spirals and circular hoops was marginal, with a minor advantage being noted for hoops due to their horizontal arrangement which coincides with the concrete expansion plane. The study further concluded that in circular hollow sections, the presence of an inner layer of transverse reinforcement without cross-ties to an outer layer of transverse reinforcement actually created a “negative confinement” effect with the inner transverse layer of reinforcement acting to pull the inner concrete wall away from the outer wall. However, when ties between the inner and outer layers of transverse reinforcement were used, both strength and ductility were significantly increased. It was also noted that, for the same transverse reinforcement volumetric ratio, hollow circular sections with thicker walls were more effectively confined. Additionally, a smaller transverse reinforcement spacing increased confinement effectiveness.

Liang, Beck, and Ling et al. (2014; 2015; 2015) conducted both analytical and experimental investigations of the confined concrete behavior of circular and rectangular hollow concrete columns. Their study investigated hollow columns with one or two layers of transverse
reinforcement. Two layers of transverse reinforcement with cross ties were found to be more effective than a single layer of transverse reinforcement because the cross ties transferred the tensile demand on the inner layer to the outer layer. However, detailing and constructing a hollow column with two layers of transverse reinforcement is cumbersome, and as a result, the in-depth investigation focused primarily on hollow columns with a single layer of transverse reinforcement.

For the analytical portion of the study, the finite element method was used to model hollow concrete columns under axial and flexural loads. The variables investigated included the amount of transverse reinforcement, confinement configuration, and wall thickness. Hollow sections were found to have a reduced confinement effect compared to solid sections. For circular hollow cross-sections with a single layer of transverse reinforcement, it was determined that the circumferential stress was significantly higher for hollow sections as compared to solid sections, with the amount of circumferential stress increasing as the wall thickness decreased. The radial stress was found to be reduced for a hollow as compared to a solid cross-section. The lower radial stress was attributed to the deformability of the hollow cross-section, which, unlike solid cross-sections, has the ability to dilate in toward its void. It was concluded that the concrete was in a triaxially confined state near the transverse reinforcement, which was located near the outer concrete face. From the outside face, the radial stress decreased to zero at the void of the cross-section, where the concrete was in a biaxially confined state. Furthermore, hollow columns experienced lower transverse reinforcement stresses than solid columns, and consequently, transverse reinforcement failure appeared to be less of a concern. This result was also attributed to the deformability of the hollow columns. Based on the analysis results, a confinement effectiveness factor was introduced to adjust Mander et al.’s confinement model to account for circular cross-sections. The factor accounts for
the reduction in radial stress in hollow columns and depends on the wall thickness of the circular hollow column.

The experimental investigation included eight small-scale hollow circular columns which were subject to a combination of pure bending and axial loads. Wall thickness and axial load ratio were the main variables investigated. The specimens contained one layer of reinforcement and were 48” in height with an outside diameter of 12”. Two of the specimens were solid, and the remaining six had either a 1” or 2” wall thickness with 0.35” concrete cover. Spiral diameter was 0.208”, and spiral pitch was set at 1.2” in the critical region and 1” elsewhere. The results demonstrated that providing one layer of reinforcing steel can provide limited ductility as long as the neutral axis is located in the wall, limiting the concrete compression strains at the inside wall face and avoiding brittle failure. Low axial load and a low amount of longitudinal reinforcement are required to keep the neutral axis away from the void. Comparison of the experimental results with the finite element analysis indicated satisfactory agreement.

2.4.3 Confinement of FRP Reinforced Circular Concrete Members

Confinement models for steel-reinforced concrete members are generally based on the assumption that the steel transverse reinforcement will yield and provide a constant lateral confining pressure as the concrete core expands. This assumption is inappropriate for FRP because FRP exhibits linear-elastic behavior and provides an increasing confining pressure as the concrete expands until the point of rupture. With this consideration, many confinement models have been proposed for concrete members externally confined with FRP. This research focuses on internal FRP reinforcement, and concrete models for sections externally confined with FRP are outside of the program scope. More limited research has been performed on concrete members confined
A summary of research on the confinement of FRP-reinforced circular concrete members is presented below.

A few studies have investigated the use of carbon FRP (CFRP) grid as internal confinement reinforcement for piles. A study by Michael et al. (2005) looked at improving the flexural ductility of prestressed piling by using CFRP grid to provide confinement to the concrete core. Nine 6” x 12” concrete cylinders were cast and tested in compression to determine concrete strength and ductility. Three of the cylinders had embedded CFRP grid which was formed into 5.25” diameter tubes, three of the cylinders had embedded CFRP grid which was formed into 5.5” diameter tubes, and three cylinders were cast without reinforcement. The control cylinders did not exhibit any significant post peak behavior and crushed after reaching their peak load. The CFRP grid cylinders reached higher peak loads and were found to exhibit significantly improved ductility compared to the control cylinders. Several existing confinement models were evaluated to assess their accuracy in modeling the CFRP grid confined concrete. An equivalent grid thickness which accounted for the non-continuously thick CFRP grid was incorporated into these models. A modified Hognestad stress-strain curve for the CFRP grid cylinders was also derived using the equivalent grid thickness and pressure vessel mechanics.

Another study investigating CFRP grid as internal reinforcement was conducted by Abalo et al. (2010). For this study, two 24” square prestressed concrete piles were tested to evaluate the use of CFRP grid in lieu of spiral ties or conventional reinforcement stirrups for a 24” square prestressed concrete pile. The piles were tested in flexure. The first pile contained twenty 0.52” diameter prestressing strands in a circular pattern wrapped with CFRP grid with an 8” overlap, and the second pile contained sixteen 0.6” diameter prestressing strands in a square pattern with W3.4 spiral ties. The pile cross-sections are shown in Figure 2-9. Both piles were 40 ft in length.
Theoretical moment capacities were compared to the experimental results. It was concluded that the use of the CFRP grid in place of the spiral ties did not reduce the moment capacity of the pile. Furthermore, inspection of a cut cross-section after casting revealed that only minor segregation occurred due to openings within the CFRP grid.

![Figure 2-9: Control Pile and CFRP Pile Cross-Sections (Abalo et al., 2010)](image)

Seliem et al. (2016) conducted three studies to investigate the replacement of steel spirals with CFRP grid as an alternative for confining precast, prestressed concrete piles in harsh marine environments. To determine the CFRP grid development length, the first study tested eight specially-designed pull-out specimens. The CFRP grid embedment length was varied. Failure occurred due to strand rupture. Results suggested an embedment length of two times the grid spacing could develop the full tensile strength of the grids.

The second study tested seven short 14” square concrete columns subjected to concentric axial load. The columns were designed to represent the top 36” of a typical precast square concrete pile. The reinforced columns contained either CFRP grid, which was formed into a 12.5” diameter and had 0.75” of concrete cover, or W3.5 steel spiral which had 3” of concrete cover. Results indicated that the CFRP grid could achieve the confinement currently provided by steel spirals for
the end zones of typical precast concrete square piles. The confinement reinforcement increased
the overall ductility of the columns, however failure was sudden and brittle. Increasing the CFRP
grid overlap did not have any effect on the load-carrying capacity, and the use of a second CFRP
grid layer did not increase the axial capacity of the column. The experimental results from this
study were used by Ding et al. (2011) to propose a confinement model which adopted the form of
the Richart et al. model and included a confining coefficient which accounted for the spacing of
the CFRP grid fibers.

The third study tested full-scale precast, prestressed concrete piles in four-point bending,
loaded to failure. The piles were 24” square and 40 ft long. One pile contained twenty 0.52”
diameter prestressing strands in a circular pattern, wrapped with the CFRP grid as transverse
reinforcement. This pile prior to concrete casting is shown in Figure 2-10. The other pile contained
sixteen 0.6” diameter prestressing strands in a square pattern with W3.5 spiral ties. (These
reinforcement patterns are similar to reinforcement of the piles tested by Abalo et al.) The two
piles exhibited the same general behavior up to failure. The CFRP reinforced pile had a slightly
higher moment capacity, which was attributed to the enhanced confinement provided by the CFRP
grid as well as the contribution of the additional longitudinal strands. Both piles failed due to
crushing in the compression zone at a concrete strain level of 0.003.
A few studies have also investigated the axial load behavior and confinement effect from using hoop or spiral FRP reinforcement, as opposed to FRP grid reinforcement. Pantelides et al. (2013) tested ten 10” diameter and 28” long concrete columns to failure under axial load. The specimens contained either steel or GFRP longitudinal bars and spirals, to create all steel, all GFRP, and steel-GFRP hybrid specimens. The GFRP spirals were size No. 3 and were placed with a 3.0” pitch. Some of the steel and hybrid specimens were subjected to accelerated corrosion before testing. The researchers modified the ACI 440.2R-08 model for externally bonded FRP jackets to predict the axial load capacity of the columns containing GFRP spiral. These analytical results were found to be conservative when compared to the experimental results. The study further concluded that for the specimens subjected to accelerated corrosion, the hybrid columns exhibited
a smaller corrosion rate, similar axial load capacity, and equal to higher ductility when compared to the steel columns.

The majority of research into the confinement of circular columns with internal FRP reinforcement has been undertaken at the University of Sherbrooke (Afifi, Mohamed, & Benmokrane, 2014a, 2014b, 2015; Afifi, Mohamed, Chaallal, & Benmokrane, 2015; Afifi, 2013; Mohamed, Afifi, & Benmokrane, 2014). The research conducted has investigated the axial performance of circular concrete columns reinforced with GFRP and CFRP longitudinal bars and transverse reinforcement. To achieve this, 27 full-scale circular concrete columns, 11.8” in diameter and 59.1” in height, were tested under concentric axial load. Transverse reinforcement included GFRP and CFRP No. 3 hoops, and GFRP and CFRP No. 2, No. 3, and No. 4 spirals. Hoop spacing and spiral pitch varied from 1.6” to 5.7”. Details of the columns are shown in Figures 2-11 and 2-12. Test parameters included the type and size of reinforcement, longitudinal reinforcement ratio, volumetric ratio, spacing or pitch, confinement configuration (spirals or hoops), and lap length of hoops.

Figure 2-11: Configuration, Reinforcement Details, and Dimensions of Circular Columns (Mohamed et al., 2014)
Results indicated that the FRP and steel-reinforced columns behaved in a similar manner up to their peak loads. Specimens with large spiral spacing experienced brittle behavior. Specimens with small to moderate spiral spacing, on the other hand, failed due to crushing of the concrete core and FRP rupture and exhibited ductile behavior. It was concluded that strength and ductility increased with increasing volumetric ratio. Furthermore, ductility and confinement efficiency were improved when using small diameter transverse reinforcement with smaller spacing as opposed to large-diameter transverse reinforcement with larger spacing. FRP circular hoops were found to be as efficient as spirals in confining the concrete.

The experimental test results were used to propose several confinement models and stress-strain relationships to describe the behavior of the CFRP and GFRP reinforced columns. Regression analysis of the test results was used to develop modified versions of Mander et al.’s model for both the GFRP and CFRP reinforced columns. Equations were proposed for the confined concrete compressive strength, and the maximum confined concrete strain. Additionally, confinement models that adopted a nonlinear form of Richart et al.’s confinement model were also
proposed for both the GFRP and CFRP reinforced columns. These proposed models were also calibrated using regression analysis of the test results. Comparison of the experimental results with the results predicted using the proposed confinement models indicated that the proposed equations provided accurate and conservative results for the range of parameters investigated. Comparison of the experimental results with Mander et al.’s original model did not accurately represent the behavior of the FRP columns. This is likely because Mander et al.’s original model assumes a constant confining pressure, whereas FRP reinforcement exhibits linearly elastic behavior until failure.

An additional study at the University of Sherbrooke considered the effects of eccentric loading. Hadhood et al. (2017) investigated the structural performance of ten full-scale circular concrete columns reinforced with CFRP bars and spirals which were subjected to combined axial compression loads and bending moments. Test parameters included eccentricity-to-diameter ratios and type of reinforcement. Two series of columns were constructed, one reinforced with steel and one reinforced with CFRP. Each series tested a control specimen under pure concentric loading and remaining specimens were tested under monotonically increasing eccentric-compression loading. Columns were 12.0” in diameter and 59.1” in height. Transverse reinforcement for the CFRP columns consisted of No. 3 CFRP spirals with an 3.1” pitch. A parametric study was also conducted to generate interaction diagrams. Although the study mostly focused on the behavior of CFRP bars in compression, it was concluded that the CFRP spirals were able to satisfactorily confine the concrete core.
2.5 Previous Research on Shear in Circular Concrete Members

2.5.1 Introduction

Limited research has been conducted on the shear strength of circular reinforced concrete members, particularly for hollow circular members. The shear design equations used in practice for steel reinforced concrete members have been developed for rectangular beams but are applied to circular members. This is generally done by defining an effective shear area for a circular member as an equivalent rectangular area. AASHTO LRFD uses this procedure by modifying the effective web width and effective shear depth used to calculate the shear strength of circular members, as described in Section 2.2.3.2. Furthermore, although there is ongoing research investigating the shear behavior of rectangular FRP reinforced beams, little research has investigated the shear behavior of FRP reinforced circular members (Ali, Mohamed, & Benmokrane, 2016). The following sections review existing research that has been conducted on hollow steel reinforced circular concrete members and FRP reinforced circular concrete members.

2.5.2 Shear Strength of Hollow Steel Reinforced Circular Concrete Members

As follow up work to the study conducted by Hoshikuma and Priestley in 2000, Ranzo and Priestley (2001) investigated the shear strength of thin-walled circular hollow columns with one layer of transverse reinforcement. Three 12.7 ft tall specimens were tested under a constant compressive axial load and cyclically varying lateral load. The first specimen was 61.4" in diameter with 6" thick walls, and was designed to fail in flexure. The remaining two specimens had 60" diameters and 5.5" thick walls, and were designed to fail in shear. All specimens experienced concrete spalling of the inside face, which caused rapid strength degradation. Experimental results were compared to existing shear models (UCSD model, ATC 32 model, and Caltrans Memo 20-4 model), which showed good agreement. The study concluded that ductile
performance could be obtained with relatively low amounts of longitudinal reinforcement and axial load, and that the enhancement of shear strength due to axial load appeared to be less significant in hollow as compared to solid members.

A paper by Turmo et al. (2009) presented an analytical model for evaluating the shear strength contribution of transverse reinforcement, $V_s$, in solid and hollow circular cross-sections. An examination of the shear stress distribution in uncracked and cracked, solid and hollow circular cross-sections demonstrated that the crack pattern for a solid circular cross-section is contained in a plane, whereas the crack pattern for a hollow circular cross-section forms along a helix. These observations were used to develop formulas for the shear strength contribution due to transverse reinforcement in solid and hollow circular cross-sections with either circular stirrups or spirals. The formulas incorporate efficiency factors which account for the inclination of the spiral with respect to the transverse axis and the inclination of the spiral with respect to the longitudinal axis. The paper concluded that spiral transverse reinforcement is surprisingly more effective in hollow cross-sections as compared to solid cross-sections because the geometry of the reinforcement follows the orientation of the shear stresses. The distribution of shear stresses in uncracked and cracked, solid and hollow circular cross-sections is shown in Figure 2-13. Circular stirrups were found to be more efficient than spiral reinforcement. Furthermore, the study found that the lower the ratio of pitch to spiral diameter, the higher the efficiency of the spiral reinforcement. Additionally, smaller reinforcement diameters with low pitch to spiral diameter ratios were preferable over larger reinforcement diameters with higher pitch to spiral diameter ratios.
In order to experimentally verify the theoretical formulations, shear tests on four circular hollow concrete specimens with stirrups were conducted. The specimens were 9.84 ft long, with 23.6” diameters and 3.94” thick walls. The specimens were simply supported over a 9.2 ft span.
and a point load was applied at midspan. All specimens failed after the transverse reinforcement yielded. The formation of a large crack in the shape of a helix of constant pitch on each specimen agreed with the theoretical formulations used to calculate the shear strength contribution of the transverse reinforcement.

Jensen and Hoang (2010) used a plasticity approach to propose a model to calculate the shear strength of hollow circular steel reinforced concrete members experiencing shear and axial compressive forces. The model assumes that the shear strength is governed by shear failure in cracked concrete or uncracked concrete, depending on the degree of axial compression. Failure in cracked concrete, in the form of a crack sliding failure, controls when the axial force is relatively small. Model predictions were compared with the limited test results available in the literature. Good agreement was found for the available tests, however, test results from specimens subject to very high axial compression could not be found. The paper indicates that flexural failure rather than shear failure would likely control for members with very high compressive forces.

Two companion papers by Völgyi et al. (2014) and Völgyi and Windisch (2014) presented an experimental investigation and subsequent model to calculate the shear resistance of hollow circular reinforced concrete members subjected to combined bending and shear. For the experimental investigation, 45 specimens with an outside diameter of 11.8” and wall thickness of either 2.2” or 3.5” were tested as simply supported members with a concentrated load applied at variable distances from one of the supports. Test variables included the wall thickness, the amount of longitudinal and transverse reinforcement, the load-to-support distance, and the axial force. The propagation of a characteristic crack pattern was analyzed. All specimens exhibited a complex failure mode attributed to the combined bending and shear, where a crack would develop in the tensile zone and a sliding surface would develop across the compression zone. It was concluded
that the shear resistance of the hollow circular cross-section increased with greater wall thickness, with the amount of longitudinal and transverse reinforcement, with the degree of prestress, and with a reduction in the shear span.

A mechanical model for hollow circular reinforced concrete members under combined bending and shear was proposed based on the results of the experimental investigation. Unlike conventional design models that consider bending and shear design procedures independently, shear and bending resistances are interrelated in the proposed model. The shear resistance for the proposed model is equal to the sum of a contribution from the concrete compression zone and a contribution from the transverse reinforcement, with the failure of the concrete compression zone being analogous to the failure of soil along a sliding surface. The proposed model considers several potential polyline-shaped failure sections. The proposed model showed very good agreement with the experimental test results.

Queiroz Junior and Horowitz (2016) presented a study which compared the experimental shear strength of hollow circular reinforced concrete cross-sections with the values calculated using the Canadian Code CSA A23.3, which is based on MCFT, and the Brazilian standard NBR 6118. The study was motivated by a lack of research on hollow circular cross-sections. The few existing models which have been developed are limited or overly complex for routine design work. To calculate shear strength using the Canadian and Brazilian codes, the study set the effective web width, \( b_w \), as equal to twice the wall thickness. The depth, \( d \), was set equal to \( 0.8D \) where \( D \) is the cross-section diameter. Results of 79 experimental tests were collected, and the calculated shear strengths were found to be conservative when compared to the experimental results. Results indicated that as the axial compressive forces increased, so did the shear strength. Additionally, the shear strength increased as the wall thickness increased and as the shear span ratio decreased.
A simple procedure which employs the modifications for $b_w$ and $d$ was proposed to compute a safe shear strength for hollow circular cross-sections using the Brazilian code NBR 6118.

### 2.5.3 Shear Strength of FRP Reinforced Circular Concrete Members

The American Concrete Institute (ACI) Committee 440’s “Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced (FRP) Bars” (ACI 440.1R-15), provides design equations to calculate the shear capacity of FRP-reinforced beams (ACI Committee 440, 2015). The formulas presented in this design guide are based on the truss analogy concept used for steel-reinforced members, $V_n = V_c + V_f$, where the total nominal shear resistance, $V_n$, is equal to the sum of the concrete shear resistance, $V_c$, and transverse reinforcement resistance, $V_f$. The adoption of this same basic concept implies that the shear resistance mechanisms for steel and FRP reinforcements are similar. Specifically, the truss analogy relies on the assumption that the reinforcement yields and exhibits plastic behavior, which is the case for steel reinforcement, but not for FRP reinforcement which exhibits a linear-elastic behavior. To account for this difference, ACI 440.1R-15 limits the effective strain in the FRP shear reinforcement to 0.004, which is justified as the strain that prevents degradation of aggregate interlock and the corresponding concrete shear resistance. ACI 440.1R-15 also includes a reduction for the bend in stirrup bars, accounting for reduced bar tensile strength due to the kinking of the innermost fibers at the bend. The strength capacity at the bend in rectangular FRP stirrups can be determined through the use of a design formula or by the ACI 440.3R-12 B.5 test method. The equations for the determination of concrete shear resistance and transverse reinforcement resistance are presented below:

$$V_c = \left(\frac{n}{2} k\right)2\sqrt{f_c' b_w d}$$  \hspace{1cm} (Eq. 8.2b, ACI 440.1R-15)
Where $b_w$ is the cross-section width, $d$ is the distance from the extreme compression fiber to the centroid of the tension reinforcement. The factor $k$ is calculated as:

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2 - \rho_f n_f}$$  \hspace{1cm} \text{(Eq. 7.3.2.2b, ACI 440.1R-15)}

Where $\rho_f$ is the reinforcement ratio and $n_f = \frac{E_f}{E_c}$. $E_f$ and $E_c$ are the modulus of elasticity of the FRP and concrete, respectively.

For FRP stirrups that are perpendicular to the axis of the member,

$$V_f = \frac{A_{fsv} f_{sv} d}{s}$$  \hspace{1cm} \text{(Eq. 8.2c, ACI 440.1R-15)}

And for continuous FRP rectangular spirals,

$$V_f = \frac{A_{fsv} f_{sv} d}{s} \sin \theta$$  \hspace{1cm} \text{(Eq. 8.2f, ACI 440.1R-15)}

Where $A_{fsv}$ is the area of the FRP shear reinforcement, $s$ is the stirrup spacing or pitch of the spiral, $\theta$ is the angle of inclination of the spiral, and $f_{sv}$ is the tensile shear strength of the FRP. The tensile shear strength of the FRP is limited by:

$$f_{sv} = 0.004 E_f \leq f_{fb}$$  \hspace{1cm} \text{(Eq. 8.2d, ACI 440.1R-15)}

$$f_{fb} = \left(0.05 \frac{r_b}{d_b} + 0.3\right) f_{fu} \leq f_{fu}$$  \hspace{1cm} \text{(Eq. 6.2.1, ACI 440.1R-15)}

Where $f_{fb}$ is the strength of a bent portion of FRP reinforcement bar, $r_b$ is the internal bend radius of the FRP reinforcement, $d_b$ is the diameter of the reinforcing bar, and $f_{fu}$ is the design tensile strength of the FRP reinforcement.

The equations presented in ACI 440.1R-15 apply to rectangular beams, and the evaluation of circular and hollow members is not specifically addressed. Additionally, there is no standard test method to determine the strength capacity of fabricated circular FRP stirrups, which lack a discrete bend. The studies reviewed below incorporated the simplified effective depth methods
currently used in practice for steel reinforced concrete members to determine the shear capacity of the circular FRP reinforced members which were investigated.

An initial study by Ali et al. (2013) at the University of Sherbrooke analyzed the shear strength of circular concrete sections reinforced with longitudinal GFRP bars and no transverse reinforcement. Shear design methods from ACI 440.1R-06 were reviewed and compared to the experimental results of six beams tested in previous studies. In order to apply the ACI 440.1R-06 equations, $b_w$ was set equal to the cross-section diameter. The ACI 440.1R-06 predictions were found to be overly-conservative. This result was attributed to a lower neutral axis depth for the circular beams as compared to rectangular beams with an equivalent cross-section. A finite element model of concrete beams reinforced longitudinally with GFRP bars and without transverse reinforcement was also presented. It was concluded that more research is needed to evaluate design equations for FRP reinforced circular beams.

Two subsequent studies at the University of Sherbrooke (Ali, Mohamed, & Benmokrane, 2017; Mohamed, Ali, & Benmokrane, 2017) reported experimental data on the shear strength of circular concrete beams reinforced with GFRP or CFRP longitudinal bars and spirals. This is believed to be the first and only research conducted on circular concrete cross-sections reinforced with FRP spirals. Test specimens were 20” in diameter and 118” in length. Transverse reinforcement consisted of No. 4 or No. 5 CFRP or GFRP spirals spaced at 3.9”, 5.9”, or 7.9”. Dimensions and reinforcement details of the test specimens are shown in Figures 2-14 and 2-15. The beams were tested in four-point bending and failed in shear due to FRP spiral rupture, as designed.
Prior to flexural cracking, the test specimens all exhibited similar approximately linear behavior. After cracking, the FRP reinforced specimens behaved nearly linearly with reduced stiffness up to failure (attributed to linear-elastic characteristics of FRP), whereas after cracking, the steel reinforced control specimens exhibited elastic-plastic behavior and had large deflections at their eventual failure. Test results indicated that the ultimate shear strength increased as the
spiral reinforcement ratio increased. Similarly, a large spiral spacing corresponded to a higher spiral strain, which indicated that the spiral reinforcement spacing controlled the widening of shear cracks. Additionally, the modulus of elasticity was found to insignificantly influence shear strength. The CFRP-reinforced beams were found to exhibit higher shear resistance and comparable performance to the control beam, and the GFRP-reinforced beams were found to exhibit a similar shear strength and cracking behavior compared to the control.

The experimental results were compared to predictions calculated using ACI 440.1R-15 design equations. Because ACI 440.1R-15 does not specifically address circular concrete beams, the study adopted the effective depth formulas presented in the AASHTO LRFD specifications, which are described in Section 2.2.3.2. Additionally, the width of the member was taken equal to the diameter and the area of flexural reinforcement was taken equal to the area of the longitudinal FRP reinforcement below the mid-depth of the section. The analysis results indicated that the ACI 440.1R-15 design equations significantly overestimated the spiral stresses at failure and did not appropriately represent the strength of the FRP spirals.

Finally, the studies proposed a modified formula based on the conventional truss model for the shear contribution of the FRP spirals. The proposed formula adopted the two efficiency factors originally presented in the study by Turmo et al. which account for the spiral inclination with respect to the transverse axis and the spiral inclination with respect to the longitudinal axis. A third efficiency factor which accounts for a strength reduction due to the stretching process of the FRP spirals (during fabrication), was also introduced. The study further recommended that the strength of the FRP spirals be limited to the lesser of 40% of the tensile strength of the straight portion of the spirals and the bend strength based on the ACI 440.3R-04 B.5 test method for the same stirrup diameter.
CHAPTER 3: Scaled Compression Tests

3.1 Introduction

This chapter describes the experimental scaled compression tests undertaken as a part of the current research to investigate the feasibility of replacing steel transverse reinforcement in precast concrete cylinder piles with FRP transverse reinforcement. The experimental program, including the fabrication of the test specimens, the test setup and instrumentation, and material properties are presented in this chapter. Analysis and discussion of the test results is also provided in this chapter.

3.2 Description and Fabrication of Test Specimens

The scaled compression tests portion of the experimental program included six 40” tall hollow concrete cylinder pile specimens which were fabricated with nominal 24” outer diameters and a 3.75” wall thickness. The specimens were intended to represent a reduced-scale hollow cylinder pile segment. The type of transverse reinforcement was varied for each specimen. A plain concrete (unreinforced) specimen and a mild steel spiral-reinforced specimen served as negative and positive controls. Two specimens contained different GFRP spirals and one specimen contained a CFRP spiral. The remaining specimen contained a carbon grid. Each type of FRP transverse reinforcement was sourced from a different manufacturer to represent a variety of the FRP reinforcement products currently available in industry. The test specimens also contained six longitudinal PVC pipes to create the voids that would normally be cast into precast pile segments for use as post-tensioning ducts. Sketches of the specimen geometry, a table summarizing the mechanical properties for the transverse reinforcement types, and photographs of the transverse reinforcement types are provided below.
### Table 3-1: Mechanical Properties of Transverse Reinforcement

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Size</th>
<th>Effective Cross-Sectional Area</th>
<th>Modulus of Elasticity</th>
<th>Tensile Strength of Straight Portion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>No Reinforcement Provided</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Spiral</td>
<td>No. 3</td>
<td>0.11 in²</td>
<td>29,000 ksi</td>
<td>Yield strength, $f_y = 63.9$ ksi Ultimate strength, $f_u = 93.6$ ksi</td>
</tr>
<tr>
<td>Carbon Grid</td>
<td>1.8&quot; x 1.6&quot; grid (longitudinal x transverse spacing)</td>
<td>0.0191 in²/ft (longitudinal direction); 0.0215 in²/ft (transverse direction)</td>
<td>34,000 ksi</td>
<td>5530 lb/ft (longitudinal direction); 5480 lb/ft (transverse direction)</td>
</tr>
<tr>
<td>CFRP Spiral</td>
<td>0.20&quot; diameter</td>
<td>0.0236 in²</td>
<td>24,220 ksi</td>
<td>8.5 kips</td>
</tr>
<tr>
<td>GFRP Spiral Type 1</td>
<td>No. 2</td>
<td>0.049 in²</td>
<td>7250 ksi</td>
<td>145 ksi</td>
</tr>
<tr>
<td>GFRP Spiral Type 2</td>
<td>No. 4</td>
<td>0.20 in²</td>
<td>7690 ksi</td>
<td>110 ksi</td>
</tr>
</tbody>
</table>
The reported properties for the FRP transverse reinforcements are those provided by the manufacturers. Guaranteed or minimum required values, not average test values, are typically reported for FRP reinforcements. The tensile strengths presented are either in force or stress units, consistent with the values reported by each manufacturer for their product.

Figure 3-2: Transverse Reinforcement: (a) Carbon Grid, (b) CFRP Spiral, (c) GFRP Spiral Type 1, (d) Steel Spiral, and (e) GFRP Spiral Type 2

The test specimens were designed to represent scaled models of cylinder pile segments, which typically range from 36” to 66” in diameter. The chosen 24” diameter of the test specimens is not much smaller than the low end of this range; a 24” diameter and 40” height were chosen for the test specimens to maximize specimen size while meeting the geometrical constraints of the compression machine available for testing. Additionally, these selected dimensions provided an
aspect ratio of 1.7, and the test specimen resembled a “squat column”. This low aspect ratio was intended to eliminate slenderness effects and achieve a true compression failure of each cross-section. A 4” nominal wall thickness was selected because this provided a thinner wall than is generally dictated by traditional cover requirements while still being thick enough to minimize casting issues that could result from the static wet casting of thinner walls. Furthermore, a 4” wall thickness could be achieved in the laboratory using readily-available cardboard concrete form tubes and did not require special formwork.

The spiral reinforcement was spaced at 6” to match the requirements specified in AASHTO LRFD Article 5.13.4.4.3 for the middle portion of piles 24” or less in diameter, as described in Section 2.2.3.3. Although state DOTs and common industry practice may require a tighter spiral pitch, as small as 2”, the 6” pitch was chosen because it provided the least amount of confinement reinforcement and was the “worst-case” scenario that still met the requirements in AASHTO LRFD. The pitch was kept constant while the reinforcement type was varied, instead of varying the pitch for each type of reinforcement to provide a constant hoop strength for each specimen. The motivation behind this approach was to investigate the feasibility of directly replacing steel reinforcement with FRP to fit the requirements that already exist for steel reinforcement; the requirements already in place for steel reinforcement are prescriptive and specify pitch, not strength. A No. 3 steel spiral was chosen to match the reinforcement size commonly used in industry, and to match the size of the steel spiral that would be used in subsequent full-scale tests, described in Chapter 4.

The formwork for each specimen consisted of inner 16” nominal and outer 24” nominal diameter stiff cardboard formwork tubes, held in place by circular ¾” thick plywood bases. For each specimen, an inner plywood base was cut to fit inside the inner form tube and a second
plywood base was cut to fit inside the outer form tube. Holes were also drilled in the second plywood base to fit the 3/4” diameter longitudinal PVC pipes. The plywood bases were aligned and screwed together, and the 16” inner formwork tube was placed on the bases. The PVC pipes were fit into the plywood bases, and the transverse reinforcement was wrapped around and secured to the PVC ducts with plastic or steel ties. Small holes were drilled through the PVC pipes at a 6” spacing to serve as tie-points on a 6” pitch for the spiral FRP. The PVC pipes were located within the cross-section to approximately center the transverse reinforcement within the wall thickness. A few extra turns of spiral were provided at the top and bottom of each specimen to try to force failure to occur away from the specimen ends. The carbon grid-reinforced specimen included a short second layer of grid at the top and bottom of the specimen for the same reason. Additionally, the carbon grid reinforcement was overlapped four full squares in the hoop direction to develop the reinforcement. After the transverse reinforcement was secured, the 24” outer form tube was placed on the bases, caulk was applied at the joint, and the specimens were ready for casting. Figure 3-3 shows the details of the fabrication process. It should be noted that the inner formwork tube was intentionally projected above the outer formwork tube to facilitate casting.
Figure 3-3: Fabrication of Test Specimen Formwork: (a) Assembled Plywood Bases, (b) Typical 6” Spacing, (c) Carbon Grid Overlap, (d) Transverse Reinforcement Tied to Formwork, (e) Completed Formwork, and (f) Caulked Joints

The test specimens were cast with normal weight concrete having small aggregate (3/8”) that was batched and delivered by a local ready-mix concrete supplier. Small aggregate was used to ensure the concrete would flow through the relatively thin walls. The concrete was shoveled from the concrete truck into buckets which were poured into the formwork, with one form being
filled at a time. Care was taken to ensure the concrete was evenly distributed around the form as the forms were filled. The concrete was vibrated using a thin pencil vibrator. Twenty 4”x8” concrete cylinders were molded at the time of casting. The target concrete compressive strength was 3 ksi after 28 days. This lower concrete compressive strength was specified to ensure that failure of the specimens could be achieved during testing. The outer form tubes were stripped after 11 days, and the inner form tubes were stripped after 13 days. The ends of the PVC pipes were cut off and the ends of the specimens were ground smooth and flat where necessary. Figure 3-4 shows casting of the test specimens.

Figure 3-4: Casting of Test Specimens: (a) Shoveling from Truck, (b) Pouring Buckets, (c) Cast Specimens, (d) and Test Cylinders
3.3 Test Setup

The pile specimens were tested to failure under concentric axial loading using a 2,000 kip capacity Baldwin closed-loop compression machine. The loading rate was approximately 3 kip/sec, and data from the compression machine and instrumentation were collected using an electronic data acquisition system. The test setup is shown in Figure 3-5.

Figure 3-5: Test Setup
To prepare the specimens for testing in the compression machine, a thin layer of gypsum was poured onto a 4” steel plate, and then a pile specimen was placed into the gypsum. Aluminum tape and silicone were used to form “dams” at the outer and inner top edges of the specimen, and another layer of gypsum was poured onto the top of the specimen. These steps are shown in Figure 3-6. A forklift was then used to place the steel plate and specimen into the compression machine, and a ¾” thick piece of plywood “cushion” was placed on top of the pile specimen. The layers of gypsum and plywood were intended to reduce stress concentrations and to provide flat bearing surfaces, thus minimizing any unintended load eccentricity. For safety, plastic and plywood guards were placed on the sides and back of the machine, and a wire screen was secured to the front of the machine.

After the first three pile specimens were tested, there was concern that the plywood cushion contributed to edge failure location at the top of the tested specimens. To alleviate these concerns, an external carbon FRP wrap was used to confine the top and bottom ends of the specimens in an attempt to force failure into the middle section for the remaining specimens. The FRP wrap was applied continuously for two layers and covered approximately the top and bottom 6 inches of the specimens, as can be seen in Figure 3-6. More detail on the rationale for the application of the external FRP wrap is provided in Section 3.6.1.

Figure 3-6: FRP Wrap to Avoid End Failure: (a) Application and (b) Test Specimens with Wraps
3.4 Instrumentation

Electrical resistance strain gauges were used to collect and record strain data during testing. Strain gauges with a gauge length of 60 mm were applied to both the outer and inner concrete surfaces to measure longitudinal and hoop strains at the mid-height of each specimen. The strain gauges were spaced in 60 degree increments, alternating between longitudinal and hoop orientations. The labelling system for the gauges was “orientation-surface-number”, where the orientation was either “L” or “H” for longitudinal or hoop, the surface was either “O” or “I” for outer or inner, and the number was “1”, “2”, or “3”. For example, the gauge “L-O-1” was the first longitudinal gauge on the outer surface of the specimen. Figures 3-7 and 3-8 show the orientation and application of the strain gauges.

![Strain Gauge Location and Orientation](image)

Figure 3-7: Strain Gauge Location and Orientation
3.5 Material Properties

Material tests were performed on the concrete to determine the relevant engineering properties. These tests are summarized below.

Three 4”x8” concrete cylinders molded at the time of specimen casting were tested in accordance with ASTM C39 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens to determine the concrete compressive strength. Six electrical resistance strain gauges were applied to the outer surface each concrete cylinder to collect strain data. Three of the gauges were oriented longitudinally and were spaced equally around the perimeter of the concrete cylinder at mid-height. The remaining three gauges were oriented horizontally and were spaced between the longitudinal gauges at mid-height.
The strain data from the concrete cylinders was used to generate average stress-strain plots for each cylinder, which are presented in Figure 3-10. One plot for axial strain and one plot for transverse strain were generated for each cylinder.
Table 3-2 presents the concrete compressive strengths and elastic moduli, calculated as the slope of the longitudinal portion of the stress-strain curve in the linear range from 10% to 35% of the ultimate axial stress. The ratio of the experimental compressive moduli compared to the ACI 318-11 recommended value of $57,000 \sqrt{f'_c}$ is also presented. It should be noted that a relatively low compression strength of 3,000 psi was specified to ensure that the tested specimens did not exceed the compression capacity of the available machine.

<table>
<thead>
<tr>
<th>Test Cylinder</th>
<th>Test Cylinder Compressive Strength, $f'_c$ (psi)</th>
<th>Compressive Modulus, $E_c$ (ksi)</th>
<th>Result / ACI [ACI=$E_c / 57,000 \sqrt{f'_c}$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2,840</td>
<td>2,900</td>
<td>95%</td>
</tr>
<tr>
<td>2</td>
<td>2,850</td>
<td>3,000</td>
<td>99%</td>
</tr>
<tr>
<td>3</td>
<td>2,850</td>
<td>2,600</td>
<td>85%</td>
</tr>
</tbody>
</table>

Three additional 4”x8” concrete cylinders were tested in accordance with ASTM C496 *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens* to determine the splitting tensile strength. The experimental splitting tensile strengths and the ratio of the experimental strengths to the ACI 318-11 recommended value of $6.7 \sqrt{f'_c}$ are presented in the table below.

Figure 3-11: Splitting Tensile Strength Test
Table 3-3: Splitting Tensile Strength Test Results

<table>
<thead>
<tr>
<th>Test Cylinder</th>
<th>Test Cylinder Splitting Tensile Strength, $f_t$ (psi)</th>
<th>$f_t / ACI$ [ACI=$f_t/6.7\sqrt{f'_c}$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>483</td>
<td>1.35</td>
</tr>
<tr>
<td>2</td>
<td>359</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>432</td>
<td>1.21</td>
</tr>
</tbody>
</table>

3.6 Test Results, Analysis, and Discussion

3.6.1 Strength and Failure Modes

The six pile specimens were loaded to failure under concentric axial force, as described in Section 3.3. For all specimens, there was no apparent crack formation or signs of failure until the specimens were close to their peak load, and the specimens did not continue to carry load after the peak load capacity was reached. Additionally, the reinforcement did not appear ruptured for any of the specimens after testing. As shown in Table 3-4, four of the specimens exhibited a similar compressive capacity with peak loads ranging from 788 kips to 798 kips. The remaining two specimens exhibited lower and higher capacities of 700 kips and 950 kips, which are within 20% of the average peak load. The performance of these two specimens can partly be attributed to the scatter and variability inherent in concrete.

An external carbon FRP wrap which was applied to the last three specimens to confine their ends and force failure at mid-height of the specimens did not appear to contribute to higher peak loads. It should be noted that the specimen which obtained the lowest peak load was confined by the FRP wrap, as was the specimen that achieved the highest peak load. The locations of failure varied among the specimens, with failures occurring in the top, middle, and lower regions of the specimens. The following paragraphs provide descriptions of the failure mode for each specimen. The specimens are presented in the order in which they were tested.
Table 3-4: Summary of Peak Loads

<table>
<thead>
<tr>
<th>Pile Specimen</th>
<th>Peak Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>794</td>
</tr>
<tr>
<td>Steel Spiral</td>
<td>798</td>
</tr>
<tr>
<td>Carbon Grid</td>
<td>798</td>
</tr>
<tr>
<td>CFRP Spiral</td>
<td>700</td>
</tr>
<tr>
<td>GFRP Spiral Type 1</td>
<td>950</td>
</tr>
<tr>
<td>GFRP Spiral Type 2</td>
<td>788</td>
</tr>
</tbody>
</table>

Figure 3-12: Test Specimens after Failure: (a) Unreinforced, (b) Steel Spiral, (c) Carbon Grid, (d) CFRP Spiral, (e) GFRP Spiral Type 1, and (f) GFRP Spiral Type 2
3.6.1.1 Unreinforced Specimen

The unreinforced pile specimen was initially loaded up to a small percentage of the ultimate load with only a layer of grout on the top of the specimen. A wide variation in the initial strain gauge readings indicated that there were uneven bearing stresses at the top of the specimen. To correct this, the specimen was unloaded, and a ¾” plywood “cushion” was inserted on top of the specimen to help distribute the compressive stresses. The specimen was then reloaded, and after reaching a seating force of approximately 100 kips, the strain gauge readings became more uniform, which indicated that the bearing stresses were more evenly distributed and concentrically applied. The remaining specimens were all loaded in a similar fashion with both a layer of grout and a fresh plywood cushion for each test. A seating force of approximately 100 kips was observed for each of these specimens as well.

After reaching the peak load, the unreinforced pile specimen split longitudinally with six longitudinal cracks forming and extending along the full height of the specimen and through the entire wall thickness. The cracks were approximately evenly spaced along the perimeter of the specimen, and their locations coincided with the location of the PVC pipes. The cracks likely formed at these locations since the presence of each PVC pipe created a local weakness in the cross-section. The weakness created by the PVC pipe is not unlike the condition that might be generated by a post-tensioning duct.
3.6.1.2 Steel Spiral Specimen

The steel-spiral-reinforced specimen failed near the top of the specimen. An inward slanting, cone-shaped failure surface formed uniformly around the perimeter. The failure plane could be best seen on the inside surface of the specimen, as shown in Figure 3-14. The outer cracked layers of concrete were removed after testing, revealing the depth of the failure and exposing the steel spiral reinforcement. It could be seen that the section failed through the entire wall thickness. The failure extended from the top of the specimen down to the mid-height of the section at one location along the perimeter, as shown in Figure 3-15. The uniform failure around the specimen’s perimeter and the extent of the failure into the mid-height of the section indicate that the specimen strength was controlled by the compressive strength of the cross-section itself. However, the use of the plywood cushion raised concerns that an edge effect may have contributed to much of the failure being located at the top of the specimen. To investigate this concern, the third pile tested was inverted prior to testing (with respect to the as-cast orientation) to investigate
whether casting effects contributed to the location of failure, as further described in the next section.

Figure 3-14: Steel Spiral Specimen, Inside Failure Surface

Figure 3-15: Steel Spiral Specimen after Failure
### 3.6.1.3 Carbon Grid Specimen

The first and second scaled pile specimens were oriented during testing such that the form side was the lower portion of the specimen and the cast side was the upper portion of the specimen, on top of which the plywood cushion was placed. After the steel spiral pile specimen failed in its top portion, the orientation of the third pile was inverted so that the top surface as-cast was the bottom surface as-tested. This was done to investigate whether the vertical casting of the specimens caused an uneven strength distribution due to aggregate separation, known as a “top cast effect”. After reaching its peak load, however, the carbon grid specimen failure still occurred in the top portion, indicating that casting orientation was not likely a meaningful parameter.

After failure, cracks were observed to extend from the top of the pile segment through the upper third of the specimen along approximately a quarter of the perimeter. The carbon grid was exposed at the top of the specimen on the side opposite of the cracks. Much of the lower half surface of the specimen did not have any visual cracks. A crowbar was used to remove the outer cracked layers of concrete. This revealed that the failure plane extended several inches into the wall thickness, to the level of the carbon grid. Cracking did not extend through the wall, unlike for the steel spiral specimen.

![Figure 3-16: Carbon Grid Specimen, Inside Failure Surface](image-url)
After the second and third tested piles failed in their top portions, a decision was made to apply an external carbon FRP wrap to confine the top and bottom portions of the remaining specimens in an attempt to see if failure could be forced into the middle section. The FRP wrap application process was described in Section 3.3.

### 3.6.1.4 CFRP Spiral Specimen

The CFRP spiral specimen failed from beneath the upper FRP wrap into its middle section. Approximately four hairline cracks extended from the top of the specimen, from underneath the FRP wrap, to about halfway down the section formed along one half of the cross-section perimeter. A large cracked piece of concrete which formed on the opposite half of the cross-section was removed with a crowbar, exposing the CFRP spiral, as shown in the figure below. The depth of the failure was approximately 3”, which was not deep enough to extend through the entire wall thickness. On the inside surface, approximately one quarter of the height down from the top of the specimen, cracking was approximately a half inch deep and uniform along almost the entire perimeter.
Figure 3-18: CFRP Spiral Specimen, Inside Failure Surface

Figure 3-19: CFRP Spiral Specimen, after Failure
3.6.1.5 GFRP Spiral Type 1 Specimen

The GFRP spiral type 1 pile specimen failed in the middle and bottom portions of the specimen, above the lower FRP wrap. Hairline cracks formed along one third of the cross-section perimeter, with approximately $\frac{1}{2}$” to 2” deep spalling occurring along the remainder of the perimeter. The GFRP spiral was exposed in the spalled regions, and the failure did not extend through the wall thickness. There was cracking along the entire perimeter of the inside surface with a depth of up to approximately 1” at a height just above the lower FRP wrap.

Figure 3-20: GFRP Spiral Type 1 Specimen, Inside Failure Surface
3.6.1.6 GFRP Spiral Type 2 Specimen

The GFRP spiral type 2 pile specimen failed underneath the upper FRP wrap, with failure extending into the mid-height of the specimen. Cracking at the location of failure extended around the entirety of the perimeter. After a crowbar was used to remove a spalled portion of the concrete, the GFRP spiral was exposed and the depth of failure was found to be approximately 4” deep. The failure did not extend through the wall thickness. On the inside surface, the specimen failed at its very top, also around the entire perimeter.
Figure 3-22: GFRP Spiral Type 2 Specimen, Inside Failure Surface

Figure 3-23: GFRP Spiral Type 2 Specimen, after Failure
3.6.2 Stress-Strain Behavior

The applied load, longitudinal strain, and hoop strain values were recorded during testing. This data were used to generate an axial and a transverse stress-strain curve for each specimen. The curves for all specimens have been overlaid and are presented in Figure 3-24. For each specimen, stress-strain behaviors from the inner three longitudinal strain gauges were compared with the behaviors measured by the outer longitudinal strain gauges. Additionally, the hoop strain behaviors from the inner hoop gauges were compared to the behaviors measured by the outer gauges. No significant difference between the inner and outer surface strain measurements were observed for any specimen in either the longitudinal or hoop direction. Therefore, for each specimen, the average longitudinal strain from the six longitudinal strain gauges and the average hoop strain from the six hoop strain gauges were used to generate the stress-strain curves.

![Figure 3-24: Longitudinal and Transverse Stress-Strain Behaviors for Scaled Compression Tests](image-url)
The compressive modulus for each specimen was calculated as the slope of the longitudinal portion of the stress-strain curve in the linear range from 10% to 35% of the peak load for that specimen. As seen in Table 3-5, the magnitudes of the compressive moduli vary from 3100 ksi to 3400 ksi and are close in range. The levels of peak compressive stress developed in the piles were slightly in excess of the 2.9 ksi average compressive stress developed for the companion cylinders, a result likely due to improved concrete curing in the larger specimens as opposed to the smaller cylinders.

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Peak Load (kips)</th>
<th>Peak Compressive Stress [Load / Area] (ksi)</th>
<th>Compressive Modulus, $E_c$ (ksi)</th>
<th>Average Longitudinal Strain at Peak Load ($\mu \varepsilon$)</th>
<th>Average Hoop Strain at Peak Load ($\mu \varepsilon$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>794</td>
<td>3.4</td>
<td>3400</td>
<td>-1266</td>
<td>239</td>
</tr>
<tr>
<td>Steel Spiral</td>
<td>798</td>
<td>3.4</td>
<td>3300</td>
<td>-1392</td>
<td>327</td>
</tr>
<tr>
<td>Carbon Grid</td>
<td>798</td>
<td>3.4</td>
<td>3400</td>
<td>-1269</td>
<td>291</td>
</tr>
<tr>
<td>CFRP Spiral</td>
<td>700</td>
<td>3.0</td>
<td>3200</td>
<td>-1199</td>
<td>316</td>
</tr>
<tr>
<td>GFRP Spiral Type 1</td>
<td>950</td>
<td>4.1</td>
<td>3100</td>
<td>-2133</td>
<td>814</td>
</tr>
<tr>
<td>GFRP Spiral Type 2</td>
<td>788</td>
<td>3.4</td>
<td>3100</td>
<td>-1464</td>
<td>471</td>
</tr>
</tbody>
</table>

The average longitudinal and hoop strains at peak load are also presented in Table 3-5. Except for the GFRP spiral type 1 specimen, the longitudinal strains at peak load have a close range of -1199 $\mu \varepsilon$ to -1464 $\mu \varepsilon$ and the hoop strains range from 239 $\mu \varepsilon$ to 471 $\mu \varepsilon$. The larger longitudinal strains recorded for the GFRP spiral type 1 specimen are attributed to the larger peak load this specimen was able to achieve before failure. The fact that all reinforced specimens developed larger hoop strains at failure than did the unreinforced specimen indicates that the reinforcement was activated and was acting to reduce the tendency for splitting cracks to develop.
as they did in the unreinforced control. However, the higher strains in the hoop direction for the reinforced piles did not generally translate into higher axial load capacities as compared to the unreinforced control. This result indicates that transverse confinement did not reliably develop in the hollow cross-section, and confinement of concrete should not be relied upon to enhance axial strength in a hollow pile, even if substantial transverse reinforcement is provided.

Values of Poisson’s ratio for the six specimens were plotted and overlaid in Figure 3-25. Poisson’s ratio was calculated as the average longitudinal strain divided by the average hoop strain and was plotted for the stresses corresponding to a range between 10% and 35% of the peak load for each specimen. The Poisson’s ratio values are similar among five of the specimens, with values for the GFRP spiral type 1 specimen being somewhat higher. The range of values is from 0.11 to 0.21, which is typical for concrete. This result indicates that a confinement effect was likely observed for the GFRP spiral type 1 specimen, however, this confinement effect could not be reliably developed.

![Figure 3-25: Poisson’s Ratio for Scaled Compression Tests](image)
3.6.3 Hoop Force

As stated above, no significant difference was observed between the magnitudes of the inner and the outer hoop strain gauges. All hoop strain gauge readings were positive, which indicates that both the inner and outer surfaces of all specimens were stressed in tension as axial compression loads were increased. Tensile strain gauge readings on the inner surface indicate that the inner surface was dilating outward as opposed to inward. A review of the confinement of hollow reinforced concrete sections has been presented in Chapter 2. Most of the previous research was concerned with flexural behavior and ductility of hollow columns under cyclic loading, with confinement effects have been observed in relation to the location of the neutral axis within the cross-section. For the hollow pile specimens tested here under pure compression, without flexural or lateral loading, no reliable, consistent confinement effect was observed, as evidenced by the outward dilation of the inner wall surface and the inability of most tested transverse reinforcement to enhance axial load carrying capacity.

Based on the strain gauge measurements, the walls of each specimen appear to have been in a relatively uniform state of hoop strain. The average hoop strain at peak load and the mechanical properties for each type of transverse reinforcement, provided in Section 3.2, have been used to determine the force in the transverse reinforcement at failure for each specimen. This hoop force has been calculated using the area of transverse reinforcement in one wall for a 6” length of half the cross-section, as indicated in Figure 3-26. A 6” calculation length was chosen to match the spacing of the spiral reinforcement.
\[ F_{\text{hoop}} = \varepsilon_{\text{hoop}} E \cdot A_{\text{retnf}} \]

**Figure 3-26: Hoop Force in a Half Cross-Section**

The maximum theoretical hoop force for each type of transverse reinforcement has also been calculated. For the steel spiral, the maximum theoretical hoop force has been calculated as the product of the yield stress and spiral area. For the FRP transverse reinforcement, the maximum theoretical hoop force has been calculated using the tensile strength of the straight portion properties reported by the manufacturer. As discussed in Section 2.5.3, ACI 440.1R-15 includes a reduction in strength for the bend portion of stirrups, which may be determined using a design formula or with the ACI 440.3R-12 B.5 test method. However, these methods pertain to stirrups, and spirals are not specifically addressed in ACI 440.1R-15. Since the specimens did not fail due to rupture of the reinforcement, and to be consistent when comparing values reported from different manufacturers using different fabrication processes, the straight portion tensile strengths have been used here. The hoop force values are presented and compared in Table 3-6.
<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Hoop Force $(F_{\text{Hoop}})$ at Failure (kips per 6&quot; of pile length)</th>
<th>Maximum Theoretical Hoop Force $(F_{\text{Hoop}})$, (kips per 6&quot; of pile length)</th>
<th>Percentage of Maximum Theoretical Hoop Force Engaged</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Spiral</td>
<td>1.04</td>
<td>7.0</td>
<td>14.8%</td>
</tr>
<tr>
<td>Carbon Grid</td>
<td>0.11</td>
<td>2.7</td>
<td>3.9%</td>
</tr>
<tr>
<td>CFRP Spiral</td>
<td>0.18</td>
<td>8.5</td>
<td>2.1%</td>
</tr>
<tr>
<td>GFRP Spiral Type 1</td>
<td>0.29</td>
<td>7.1</td>
<td>4.1%</td>
</tr>
<tr>
<td>GFRP Spiral Type 2</td>
<td>0.72</td>
<td>22.0</td>
<td>3.3%</td>
</tr>
</tbody>
</table>

A small percentage of the theoretical maximum hoop force was engaged at failure for the FRP-reinforced pile specimens as compared to the steel-reinforced section. This result, combined with the fact that the FRP-reinforced specimen developed higher hoop strains at failure, demonstrates the effect that a reduced transverse reinforcement area has on hoop force (in the case of the CFRP reinforcement), or the effect that a reduced transverse reinforcement modulus has on hoop force (in the case of the GFRP reinforcement).

This result also indicates that the provided levels of transverse reinforcement are not being mobilized under the applied axial loads and are not likely contributing significantly to the axial load capacity. Stated otherwise, it is reasonable to conclude that failure of the scaled compression pile specimens was due to concrete crushing, and the relatively small hoop force provided by the transverse reinforcement was not significant for axial load capacity, regardless of the reinforcement type. The magnitude of the hoop force provided at failure by the Type 1 GFRP spirals was substantially lower than the hoop force provided at failure by the steel spirals (and by the Type 2 GFRP spirals), so enhanced concrete confinement is not a reliable explanation for the higher axial load carrying capacity observed for this specimen.
It should be noted that the stiffness of the transverse reinforcement is just as important of a consideration as its strength. In order for the ultimate strength of the transverse reinforcement to be reached, the strain in the transverse reinforcement at its ultimate strength must be small enough to limit crack widths in the concrete and to maintain composite action of the cross-section. This concept is illustrated by the effective strain limit of 4000 με that ACI 440.1R-15 requires for shear reinforcement to prevent degradation of aggregate interlock, as discussed in Section 2.5.

The modulus of elasticity is a measure of the stiffness of the reinforcement. As shown in the material properties table in Section 3.2, the moduli of elasticity of the CFRP transverse reinforcement types are similar to the modulus of elasticity for the steel spiral, and the moduli of elasticity of the GFRP transverse reinforcement types are equal to approximately one quarter of the modulus of elasticity for the steel spiral. FRP reinforcement that has a lower modulus of elasticity than does steel, so FRP bars must stretch more to develop the same hoop force provided by steel reinforcement of equal area. The strain required to achieve a given hoop force can be reduced by increasing the area of reinforcement. This relationship is shown by the equation,

$$F_{\text{hoop}} = \varepsilon_{\text{hoop}} EA_{\text{reinf}}.$$

The maximum theoretical hoop force capacity that can be obtained by each of the transverse reinforcement types when the hoop strain is limited to 4000 με is presented in Table 3-7. The hoop force capacities for the CFRP transverse reinforcement types, which have similar moduli of elasticity as compared to the steel spiral, are much smaller than that of the steel spiral due to their smaller cross-sectional areas. The ratio of the area of FRP transverse reinforcement to the area of steel spiral required for a given hoop force is also provided in Table 3-7. The ratios for the GFRP reinforcement are larger since their moduli of elasticity are approximately one quarter that of steel.
Table 3-7: Summary of Hoop Forces at 4000 με Strain Limit

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Maximum Hoop Force ($F_{\text{Hoop}}$), Strain Limited to 4000 με (kips per 6” of pile length)</th>
<th>$\frac{A_{\text{FRP}}}{A_{\text{steel}}}$ Required for a Given Hoop Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Spiral</td>
<td>7.0</td>
<td>1.00</td>
</tr>
<tr>
<td>Carbon Grid</td>
<td>1.5</td>
<td>0.85</td>
</tr>
<tr>
<td>CFRP Spiral</td>
<td>2.3</td>
<td>1.20</td>
</tr>
<tr>
<td>GFRP Spiral Type 1</td>
<td>1.4</td>
<td>4.00</td>
</tr>
<tr>
<td>GFRP Spiral Type 2</td>
<td>6.2</td>
<td>3.77</td>
</tr>
</tbody>
</table>

3.7 Summary of Scaled Compression Tests

The intent of the scaled compression tests was to investigate the feasibility of replacing steel transverse reinforcement in precast concrete cylinder piles with FRP transverse reinforcement. To achieve this, six scaled hollow concrete cylinder pile specimens with varied types of transverse reinforcement were tested under concentric axial load. Analysis of the test data indicates that replacing steel transverse reinforcement with FRP transverse reinforcement neither degraded nor enhanced the performance of the test specimens. In general, no significant difference in behavior was observed among the pile specimens, as indicated by the small range in peak loads, the compressive moduli, strains, and Poisson’s ratios. No reliable confinement effect was observed in the pile specimens, and calculation of the hoop force at failure indicated that the transverse reinforcement did not significantly contribute to the axial load capacity. Results from the scaled compression tests indicate that replacing steel transverse reinforcement with FRP transverse reinforcement should not be a concern from the perspective of the axial load capacity of the reinforced cross-section, because it appears that this axial load capacity is not significantly influenced by transverse reinforcement, regardless of the type.
CHAPTER 4: Full-Scale Tests

4.1 Introduction

This chapter describes a series of full-scale tests undertaken as a part of the current research to investigate the feasibility of replacing steel transverse reinforcement in precast concrete cylinder piles with FRP transverse reinforcement. The experimental program, including fabrication of the test specimens, test setup, and instrumentation are presented in this chapter. Analysis and discussion of the test results are also provided in this chapter.

4.2 Description and Fabrication of Test Specimens

The full-scale experimental program included four cylinder pile segments, each 16’ long and 54” in diameter, all fabricated at a precast plant using a spin-casting process. As described in Section 1.1, spin-casting is a specialized technique that is often used to manufacture cylinder pile segments which are subsequently joined together to form a full-length pile using a segmental construction process. For each pile segment, an assembled reinforcing cage was placed into a cylindrical steel form. Small-diameter, rubber-coated mandrels stretched along the length of the form to leave voids in the pile segment wall to be used as post-tensioning ducts. A stinger was then used to distribute zero-slump concrete into the steel form as it was rapidly rotated, compacting the concrete against the form by centrifugal force. The rate of rotation was then increased further, and the concrete was consolidated for approximately five minutes, after which the inner pile segment wall was vibrated and roller-compacted. Still in the mold, the pile segment was then removed from the spinning apparatus to be steam cured, and then ultimately stripped from the steel form, stored, and inspected. The prestressing tendons were then installed, stressed, grouted, and the excess flame cut. It should be noted that the segments tested in this experimental program
were post-tensioned individually, however, the post-tensioning is typically used to join many segments together end-to-end to form long piles.

Two of the four pile segments contained steel spiral transverse reinforcement. An automatic welding machine was used to fabricate the reinforcing cage for the steel-spiral-reinforced pile segments. This machine welded the steel spiral to longitudinal “keeper” rods at a constant 2” pitch. The remaining two pile segments contained transverse FRP reinforcement in the form of a carbon grid. To assemble this reinforcing cage, five separate sheets of carbon grid were overlapped approximately 10” in the longitudinal direction and were then wrapped around a wooden form into the shape of a cylinder. Overlap of the assembled sheet of carbon grid in the circumferential direction was provided at a minimum of 8”. The carbon grid was secured to itself and to the rubber-coated mandrels with plastic ties, and the wooden form was finally removed from the inside of the assembled carbon grid cylinder. The carbon grid cylinder was then inserted into the cylindrical steel form, with 1 ½” diameter by 1 ½” long half PVC pipe spacers used between the carbon grid and rubber mandrels at the end of each carbon grid sheet. Fabrication of the reinforcing cages and the spin-cast process are shown Figures 4-1, 4-2, and 4-3.

Figure 4-1: Assembly of Steel Spiral Reinforcing Cage: (a) Automatic Welding Machine and (b) Completed Steel Spiral Reinforcing Cage
Figure 4-2: Assembly of Carbon Grid Reinforcing Cage: (a) Wrapping of Carbon Grid around Wooden Form, (b) Secured Carbon Grid Cylinder, (c) Removal of Wooden Form, and (d) Carbon Grid Reinforcement Inside Steel Form
The two steel-spiral-reinforced pile segments each had a 6” nominal wall thickness. One of the carbon-grid-reinforced pile segments had a 6” nominal wall thickness, and the remaining carbon-grid-reinforced pile segment had a 5” nominal wall thickness. The wall thickness of the second carbon-grid-reinforced pile segment was reduced to investigate the behavior of a pile segment with a reduced wall thickness, as would be potentially possible with the use of transverse FRP reinforcement due to reduced concrete cover requirements. Each pile segment had twenty-four 1-3/8” diameter post-tensioning ducts that were evenly spaced. A typical pile segment cross-section is shown in Figure 4-4.
The mechanical properties for the steel spiral and carbon grid are summarized in Table 4-1. The carbon grid used in the full-scale pile segments is identical to the carbon grid used for the scaled compression test specimen. The longitudinal and transverse directions reported for the carbon grid are with reference to the orientation of the pile segment, which is opposite of the orientation reported by the manufacturer.

**Table 4-1: Mechanical Properties of Transverse Reinforcement**

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Size</th>
<th>Effective Cross-Sectional Area</th>
<th>Modulus of Elasticity</th>
<th>Tensile Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Spiral</td>
<td>W11 spiral wire</td>
<td>0.11 in²</td>
<td>29,000 ksi</td>
<td>Yield strength (min) $f_y = 70$ ksi; Ultimate strength (min) $f_u = 80$ ksi</td>
</tr>
<tr>
<td>Carbon Grid</td>
<td>1.8&quot; x 1.6&quot; grid (transverse x longitudinal spacing)</td>
<td>0.0191 in²/ft (transverse direction); 0.0215 in²/ft (longitudinal direction)</td>
<td>34,000 ksi</td>
<td>5530 lb/ft (transverse direction); 5480 lb/ft (longitudinal direction)</td>
</tr>
</tbody>
</table>
Longitudinal prestressing was applied to each pile segment with tendon pairs installed in 16 of the 24 post-tensioning ducts. Each tendon pair consisted of two ½” diameter, 270 ksi low relaxation strands. The prestressing patterns used are shown in Figure 4-5. The eight post-tensioning ducts which did not contain tendons were filled with grout for the steel-spiral-reinforced pile segments and were left unfilled for the carbon-grid-reinforced pile segments. As the FRP reinforcement does not corrode, there would be no need to grout any voids not utilized by prestressing tendons. All tendons were stressed to 80% of their ultimate capacity because the wedge seating loss was expected to be about ¼”. This loss is substantial on a short 16’ pile length, and the effective prestress after anchorage losses alone was estimated to be close to 65% of the ultimate capacity, which would be further reduced by the normal creep, shrinkage, and relaxation losses. This level is within the range of the effective prestress expected for a normal long pile constructed of many segments with strands pulled to an initial tension equal to 70% of their ultimate strength.

![Figure 4-5: Prestressing Pattern: (a) Pile Segment with 6” Wall Thickness and (b) Pile Segment with 5” Wall Thickness](image)
The concrete compressive strengths for the pile segments were not known with certainty, as concrete cylinders were not available with the pile segments. With the spin casting process, test cylinders have to be specially pressed during casting to achieve the similar high-density concrete obtained in the spin casting process. The manufacturer’s estimated strength range was between 10 and 12 ksi for all tested segments. The steel-spiral-reinforced pile segments tested were taken from a stock of segments cast approximately fifteen years earlier than the carbon-grid-reinforced pile segments. The concrete compressive strength for the steel-spiral-reinforced pile segments was expected to be greater for the carbon-grid-reinforced pile segments due to strength gain that would have resulted from their extended curing time. For all tested pile segments, the specified 28 day concrete compressive strengths (design strengths) are lower than are the actual compressive strengths predicted by the manufacturer. The specified 28 day concrete compressive strength for the carbon-grid-reinforced pile segments was 7 ksi. The specified strength for the steel-spiral-reinforced pile segments was 6 ksi.

It should be noted that the full-scale pile segments were not specifically designed for the research, but were taken from available existing stock. The steel-reinforced segments were available extra segments from a previous bridge project, and the CFRP-reinforced segments were available from a previous trial that investigated whether the spin casting process would work with FRP grid reinforcement. In other words, while the area of steel reinforcement was proportioned to meet industry standards, the provided area of carbon grid reinforcement was simply a function of the single layer of carbon grid that was used to determine whether the spin-casting process could be successfully implemented with an FRP grid reinforcement. Spin casting with the carbon-grid reinforcement was successful, and the completed CFRP-reinforced pile segments appeared
visually identical to traditional steel-reinforced segments. At this point, the completed CFRP-reinforced segments were made available for testing in the current research program.

4.3 Test Setups

4.3.1 Overview of Test Setups

Each pile segment was subjected to a sequence of multiple axial compression tests and transverse splitting tests. The axial compression tests included both concentric and eccentric loading, and these tests were representative of service loading conditions. The transverse splitting tests subjected the pile segments to direct internal loading at one end; these tests were intended to simulate damage that can occur during the pile driving process. Each test setup and the sequence of testing for each of the pile segments are described in the sections that follow.

4.3.2 Axial Compression Test Setup

For the axial compression tests, reaction blocks were placed at both ends of a horizontally-supported pile segment. Hydraulic jacks and threaded steel bars were used to apply axial load, as shown in Figures 4-6 and 4-8. Two different loading patterns were used to apply either concentric or eccentric axial load, as shown in Figure 4-7. The eccentricity of the eccentric configuration was approximately one foot.

Figure 4-6: Axial Compression Test Setup Sketch
Figure 4-7: Hydraulic Jack Configuration Sketches: (a) Concentric Axial Compression and (b) Eccentric Axial Compression

Figure 4-8: Axial Compression Test Setup

The supports under the pile segment were located so that there would be no moment from the self-weight of the pile at midspan. Hilman rollers were placed under one reaction block and the support closest to it to allow for lateral translation of the test setup during loading. The opposite support and reaction block were placed on fixed supports.
Figure 4-9: Roller-Supported End of Axial Compression Test Setup

Figure 4-10: Fixed End of Axial Compression Test Setup
The reaction blocks were specifically designed and fabricated to resist the large axial loads, and to enable application of a concentric load pattern and an eccentric load pattern using the same blocks. During block fabrication, PVC pipes were secured to the reaction block reinforcement cages to create voids, as shown in Figure 4-11.

![Fabrication of Reaction Blocks](image)

*Figure 4-11: Fabrication of Reaction Blocks: (a) Reinforcement Cage with PVC Pipes and (b) Reaction Blocks after Casting*

In the axial load test setup, high-strength steel bars extended through the PVC pipes in a reaction block on one end of the pile segment, passed through the hollow core of the pile segment, and extended through the PVC pipes in the reaction block on the opposite end of the pile segment. Hydraulic jacks and a load cell were placed on the high-strength steel bars, and secured in place with nuts and plates. The jacks were extended against this hardware to apply the load. All four hydraulic jacks shared the same pressure source, ensuring that each jack applied the same load. The jacks were operated manually with an electric pump. Data were collected during testing using a data acquisition system.
Figure 4-12: Hydraulic Jack Configuration for Concentric Axial Compression Test Setup

Figure 4-13: Hydraulic Jack Configuration for Eccentric Axial Compression Test Setup
Oriented strain board was placed between the ends of the pile segments and the reaction blocks to help distribute the loads and reduce stress concentrations between the concrete surfaces. A combination of oriented strand board and rigid steel plates were used to distribute the applied load from each jack uniformly onto the reaction block.

**4.3.3 Transverse Splitting Test Setup**

The transverse splitting test setup is shown in Figures 4-14 and 4-15. For this test, moveable concrete saddles were constructed to provide level loading surfaces that fit inside of the cylindrical pile segment. The concrete saddles were placed inside one end of the pile segment, and hydraulic jacks were used to apply load to induce cracking at the end of the pile. The transverse splitting test was conducted at one end of the pile only, representing bursting stresses that could occur at either end of the pile during driving.

![Figure 4-14: Transverse Splitting Test Setup Sketch](image_url)
Expansion board was inserted along the mid-width of each saddle during casting to create a weak plane, allowing the saddle to separate into two pieces during loading. This was done to provide for a more even load distribution along the inner pile wall, preventing stress concentration at the top and bottom of the pile segment cross-section.

The hydraulic jacks were placed on rigid steel plates to bring them to the correct height, to uniformly distribute the applied load, and to reduce stress concentrations. Neoprene pads were also placed between the concrete saddles and the inside surface of the pile segment to reduce stress concentrations. Tilt saddles were placed on top of all hydraulic jacks to allow for adjustment between the saddle and the inside surface of the pile segment. The hydraulic jacks shared the same pressure source to ensure that each jack applied the same load, and they were operated manually.
with an electric pump. A load cell was used to measure the applied load. Data were collected using an electronic data acquisition system.

### 4.3.4 Test Procedure

The test procedure for each pile segment included an initial concentric axial compression test, several eccentric axial compression tests, and several transverse splitting tests.

As stated previously, the axial compression tests were representative of service loading conditions and the transverse splitting tests were intended to simulate damage that occurs during the pile driving process. An initial concentric axial compression test was conducted on each pile to verify pile performance at service load. Eccentric axial compression tests were then conducted on each pile segment before and after a transverse splitting test to investigate the influence of cracking on the flexural capacity of the pile segment, and to observe the service behavior of a pile segment that may be damaged from pile driving. For some piles, a series of alternating eccentric compression tests and splitting tests were performed.

Table 4-2 summarizes the test sequence followed for each of the pile segments. For the transverse splitting tests, load was applied until a desired level of damage (cracking) had been achieved. The initial desired level of damage was the development of a definitive longitudinal crack, with subsequent increments being defined by substantial cracking events under continued loading. The increments reported for a given transverse splitting test each include a pause during testing to document crack propagation. More details on the extent of cracking and the level of damage at each increment are provided in Section 4.5.3. For all transverse splitting tests, the jacks were held at a fixed position at the end of each increment for approximately five minutes before the pile segment was unloaded.
<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Pile Type</th>
<th>Sequence of Tests Performed</th>
<th>Maximum Applied Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile 1</td>
<td>Carbon-Grid-Reinforced, 6&quot; Wall</td>
<td>#1) Concentric Axial Compression</td>
<td>600 kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#2) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#3) Transverse Splitting</td>
<td>49 kips (1st increment); 45 kips (2nd increment, loads did not reach the 1st increment peak)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#4) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#5) Transverse Splitting</td>
<td>28 kips (to failure, failure load was less than previous increment maximum)</td>
</tr>
<tr>
<td>Pile 2</td>
<td>Steel-Spiral-Reinforced, 6&quot; Wall</td>
<td>#1) Concentric Axial Compression</td>
<td>600 kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#2) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#3) Transverse Splitting</td>
<td>142 kips (1st increment); 219 kips (2nd increment); 238 kips (3rd increment)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#4) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#5) Transverse Splitting</td>
<td>213 kips (to failure, failure load was less than previous increment maximum)</td>
</tr>
<tr>
<td>Pile 3</td>
<td>Steel-Spiral-Reinforced, 6&quot; Wall</td>
<td>#1) Concentric Axial Compression</td>
<td>600 kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#2) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#3) Transverse Splitting</td>
<td>50 kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#4) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#5) Transverse Splitting</td>
<td>100 kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#6) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#7) Transverse Splitting</td>
<td>150 kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#8) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#9) Transverse Splitting</td>
<td>196 kips</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#10) Eccentric Axial Compression</td>
<td>600 kips with 1' eccentricity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>#11) Transverse Splitting</td>
<td>197 kips (to failure)</td>
</tr>
</tbody>
</table>
Table 4-2 (continued)

<table>
<thead>
<tr>
<th>Pile 4</th>
<th>Carbon-Grid-Reinforced, 5&quot; Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>#1) Concentric Axial Compression</td>
</tr>
<tr>
<td></td>
<td>#2) Eccentric Axial Compression</td>
</tr>
<tr>
<td></td>
<td>#3) Transverse Splitting</td>
</tr>
<tr>
<td></td>
<td>#4) Eccentric Axial Compression</td>
</tr>
<tr>
<td></td>
<td>#5) Transverse Splitting</td>
</tr>
</tbody>
</table>

The applied axial service loadings chosen for the concentric and eccentric axial compression tests are plotted on the service interaction diagram shown in Figure 4-16. This service interaction diagram was developed based on an assumed design concrete compressive strength of 6 ksi and the AASHTO LRFD allowable stress limits for service loading in corrosive conditions, which are listed in Table 2-1. The service interaction diagram was generated using a spreadsheet program developed by PCI (discussed in Section 2.2.3.2). A service limit state was chosen for the maximum applied loads instead of a strength limit state because the performance of the pile segments in normal operational use, rather than performance near their ultimate capacity, was of interest for this project. Practical test setup limitations also controlled the maximum load that could be applied to the pile segments.
4.4 Instrumentation

Electrical resistance strain gauges with a gauge length of 60 mm were applied at midspan on the outer concrete surfaces to collect and record concrete surface strains during testing, as shown in Figure 4-17.
The strain gauges were applied in both longitudinal and hoop orientations. The labelling system for the gauges was “orientation-number” where the orientation was either “L” or “H” for longitudinal or hoop, and the number was incremented along the circumference of the pile segment. Figures 4-18 and 4-19 show the orientation of the strain gauges. A load cell was also used in each test setup to measure the applied load from the hydraulic jacks, as shown in the test setup figures above.
4.5 Full-Scale Test Results

4.5.1 Concentric Axial Compression Tests

The applied load, longitudinal strain, and hoop strain values recorded during the concentric axial compression test were used to generate stress-strain curves for each pile segment. A typical set of complete stress-strain data is presented in Figure 4-20. This data set is representative of the typical complete set of strain data collected for each of the pile segments during the concentric...
axial compression tests. Note that negative strains are compression (longitudinal) and positive strains are tension (hoop).

![Figure 4-20: Typical Complete Strain Data Set from Concentric Axial Compression (from Pile 3)](image)

In theory, under a perfectly-concentric axial load with perfectly-uniform bearing at the end supports, the measured strain at all longitudinal gauge locations should be the same. Likewise, the measured strain at all hoop gauge locations should theoretically be the same. At any given applied load, the range in magnitudes of the measured strains in the full raw data set for each pile indicates that there was likely some unintended eccentricity and some minor non-uniformity in the support bearings caused by the practical difficulties of applying a perfectly concentric axial force in the laboratory. For all pile segments in their as-tested horizontal position, the bottom of the pile segment (strain gauge L-1) was slightly more compressed under the concentric axial load than was the top of the pile segment (strain gauge L-9), however, the magnitude of this difference was
minor. The observed offset in hoop strains was consistent with the offset in longitudinal strains –
that is, measured hoop strains near the bottom of the section were slightly higher (more tension)
than hoop strains near the top of the section. An effective accidental eccentricity due to these
offsets was back-calculated from the measured concentric strain distributions and was determined
to be less than 2” in all cases (less than 4% of the 54” pile diameter). This magnitude is not
significant, and therefore, the average strain values were used for the analysis of the concentric
axial compression tests. The average of all longitudinal gauges for a given pile are reported from
this point forward as the longitudinal strain for that pile. Likewise, the average of all hoop strain
measurements for a given pile are used as the hoop strain for that pile.

The stress-strain curves generated using the average strain values from the concentric axial
compression tests for all four pile segments are overlaid and presented in Figure 4-21. The actual
measured wall thickness for each pile, which was slightly greater than the 5” or 6” nominal wall
thickness specified, was used to calculate the stress for each of the pile segments from the
measured applied load. Differences in cross-sectional area between the four pile segments explain
the variation in the maximum stress shown in Figure 4-21. Recall that Pile 4 had a 5” wall thickness
(and thus, a much smaller cross-sectional area, resulting in larger stresses under the same applied
load). Un-grouted post-tensioning ducts were also considered as holes in the cross-section when
they were present.
The compressive modulus for each pile segment was calculated as the slope of the longitudinal portion of the stress-strain curve. Compressive moduli values are provided in Table 4-3. The steel-spiral-reinforced pile segments had higher moduli and were stiffer than were the carbon-grid-reinforced pile segments. This result likely occurred because the steel-spiral-reinforced pile segments were taken from a stocked supply that was cast approximately fifteen years earlier than the carbon-grid-reinforced pile segments.

**Table 4-3: Compressive Moduli and Concrete Compressive Strengths**

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Compressive Modulus Determined from Measured Load, Area, and Strain Data, $E_c$ (ksi)</th>
<th>Concrete Compressive Strength Determined from Elastic Modulus, $f'_c$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile 1</td>
<td>6300</td>
<td>10</td>
</tr>
<tr>
<td>Pile 2</td>
<td>8500</td>
<td>16</td>
</tr>
<tr>
<td>Pile 3</td>
<td>8400</td>
<td>16</td>
</tr>
<tr>
<td>Pile 4</td>
<td>6100</td>
<td>9</td>
</tr>
</tbody>
</table>
The concrete compressive strengths for each pile segment were back-calculated from the actual measured compressive moduli using the ACI 318 recommended value for elastic modulus of $w_c^{1.5}33\sqrt{f'_c}$ where $w_c$ is the concrete density. The manufacturer-specified concrete density of 155 pcf was assumed. The calculated concrete compressive strengths compare well to the expected strengths specified by the manufacturer, factoring in the long additional curing time for the steel-spiral-reinforced pile segments.

Plots of Poisson’s ratio across the range of applied axial loads for all four pile segments are overlaid in Figure 4-22. Poisson’s ratio was calculated for each pile segment as the average longitudinal strain divided by the average hoop strain for all stresses within the range of 20% to 100% of the maximum applied stress. Poisson’s ratio was not calculated for stress values below 20% of the maximum, as noise in the strain data and seating effects were observed at these low levels. The range of values calculated for Poisson’s ratio varied from 0.15 to 0.21, which is typical for concrete.

![Figure 4-22: Poisson’s Ratio for Concentric Axial Compression Tests](image)
4.5.2 Eccentric Axial Compression Prior to Transverse Splitting

4.5.2.1 Longitudinal Strain Data and Strain Distributions

Figures 4-23 to 4-26 present the raw longitudinal strain data collected for each of the pile segments during the initial eccentric axial compression tests, before the transverse splitting tests were conducted.

![Figure 4-23: Complete Longitudinal Strain Data from Eccentric Axial Compression Test before Transverse Splitting Tests for Pile 1](image-url)
Figure 4-24: Complete Longitudinal Strain Data from Eccentric Axial Compression Test before Transverse Splitting Tests for Pile 2

Figure 4-25: Complete Longitudinal Strain Data from Eccentric Axial Compression Test before Transverse Splitting Tests for Pile 3
Figure 4-26: Complete Longitudinal Strain Data from Eccentric Axial Compression Test before Transverse Splitting Tests for Pile 4

Under eccentric axial loading, the measured strain values varied along the height of the cross-section in proportion to the distance between the strain gauge and the mid-height of the cross-section (assumed neutral axis). The concrete located closer to the bottom of the cross-section (strain gauge L-1) is more heavily compressed than the concrete located near the top of the cross-section (strain gauge L-9) because the applied load was located below the mid-height. The difference in magnitude between the strains recorded by strain gauges L-1 and L-2 and by strain gauges L-8 and L-9 is small since these strain gauges were only slightly offset vertically from each other due to the equal radial spacing between gauges. The values recorded by strain gauges L-5 and L-10 are similar because these gauges were placed at the same height on the cross-section on opposite sides of the pile.

The theoretical relationship between the experimentally-recorded strain, $\varepsilon_{exp}$, and the strain gauge location on the cross-section is provided by the following equation:
\[ \varepsilon_{\text{exp}} = -\frac{P}{A_c E_c} \pm \frac{P e y}{I E_c} \]

Where \( P \) is the applied force, \( e \) is the eccentricity of the applied force, \( y \) is the distance from the mid-height of the cross-section to the location of the strain gauge, \( A_c \) is the area of the pile segment’s cross-section, \( E_c \) is the compressive modulus, and \( I \) is the pile segment’s moment of inertia.

It should be noted that the strain gauges were applied after the pile segments were prestressed. For this reason, the strain gauges captured the strain due to the applied load only. An initial strain due to the prestressing force must also be considered in order to calculate the total strain in the pile segments. The initial strain due to the prestressing has been back-calculated using the actual measured compressive modulus for each pile segment and an assumed effective prestressing force equal to 65% of the ultimate tensile strength of the prestressing tendons (as described in Section 4.2). The total strain in the pile segment is given by the following equations:

\[ \varepsilon_{\text{total}} = -\frac{P}{A_c E_c} \pm \frac{P e y}{I E_c} - \varepsilon_{i,ps} \]

\[ \varepsilon_{i,ps} = \frac{f_{pc}}{E_c} \]

\[ f_{pc} = \frac{P_e}{A_c} \]

Where \( \varepsilon_{i,ps} \) is the initial strain due to the prestressing, \( f_{pc} \) is the compressive stress in the concrete due to the effective prestressing force, and \( P_e \) is the effective prestressing force.

Figures 4-27 to 4-30 present the longitudinal strain data collected for each of the pile segments including the addition of the initial compressive strain due to prestressing. As can be seen, this adjustment had the effect of shifting the measured strain data to the left along the x-axis. For all pile segments, the total net strain in the cross-section is compressive because the initial
prestressing strain is greater than any of the measured tensile strains. This result is expected for a prestressed pile loaded to the service level limit state.

Figure 4-27: Total Longitudinal Strain Data from Eccentric Axial Compression Test Including Prestressing for Pile 1

Figure 4-28: Total Longitudinal Strain Data from Eccentric Axial Compression Test Including Prestressing for Pile 2
Figure 4-29: Total Longitudinal Strain Data from Eccentric Axial Compression Test Including Prestressing for Pile 3

Figure 4-30: Total Longitudinal Strain Data from Eccentric Axial Compression Test Including Prestressing for Pile 4
A summary of the total strain values at the maximum applied load are provided in Table 4-4.

<table>
<thead>
<tr>
<th>Strain Gauge Label</th>
<th>L-1</th>
<th>L-2</th>
<th>L-3</th>
<th>L-4</th>
<th>L-5</th>
<th>L-6</th>
<th>L-7</th>
<th>L-8</th>
<th>L-9</th>
<th>L-10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain Gauge</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Distance from</td>
<td>-27.0</td>
<td>-24.9</td>
<td>-19.1</td>
<td>-10.3</td>
<td>0.0</td>
<td>10.3</td>
<td>19.1</td>
<td>24.9</td>
<td>27.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Mid-height (in)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The measured longitudinal strains can be plotted along the height of the pile segment’s cross-section to generate a strain distribution. Strain distributions for the longitudinal strains that were recorded at the maximum applied load are shown in Figures 4-31 to 4-34. These strain distributions have been adjusted to include the initial compressive strain due to prestressing.

![Figure 4-31: Total Longitudinal Strain Distribution from Eccentric Axial Compression Test before Transverse Splitting Tests for Pile 1](image)
Figure 4-32: Total Longitudinal Strain Distribution from Eccentric Axial Compression Test before Transverse Splitting Tests for Pile 2

Figure 4-33: Total Longitudinal Strain Distribution from Eccentric Axial Compression Test before Transverse Splitting Tests for Pile 3
For all the pile specimens, the strain distribution at maximum load from the initial eccentric loading test was generally linear, as would be expected for a pile subjected to service level loading. For the steel-spiral-reinforced pile segments (Piles 2 and 3), a portion of the steel spiral passed through the cross-section at the midspan where the strain gauges were located. The presence of this steel spiral as a discontinuity in the cross-section may explain the slight nonlinearity observed for the steel-spiral-reinforced pile segments.

4.5.2.2 Hoop Strain Data

Figures 4-35 to 4-38 and Table 4-5 present the complete hoop strain data collected for each of the pile segments during the eccentric axial compression tests before the transverse splitting tests were conducted.
Figure 4-35: Complete Hoop Strain Data from Initial Eccentric Axial Compression Test for Pile 1

Figure 4-36: Complete Hoop Strain Data from Initial Eccentric Axial Compression Test for Pile 2
Figure 4-37: Complete Hoop Strain Data from Initial Eccentric Axial Compression Test for Pile 3

Figure 4-38: Complete Hoop Strain Data from Initial Eccentric Axial Compression Test for Pile 4
Table 4-5: Raw Hoop Strain Data from Eccentric Axial Compression Tests

<table>
<thead>
<tr>
<th>Strain Gauge Label</th>
<th>H-1</th>
<th>H-2</th>
<th>H-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain Gauge Distance from Mid-height (in)</td>
<td>-3.52 (Piles 1-3)</td>
<td>24.94 (Piles 1-3)</td>
<td>-21.42 (Piles 1-3)</td>
</tr>
<tr>
<td></td>
<td>-6.99 (Pile 4)</td>
<td>26.08 (Pile 4)</td>
<td>-19.09 (Pile 4)</td>
</tr>
<tr>
<td>Measured Strain (με)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile 1</td>
<td>21</td>
<td>-4</td>
<td>36</td>
</tr>
<tr>
<td>Pile 2</td>
<td>17</td>
<td>-2</td>
<td>31</td>
</tr>
<tr>
<td>Pile 3</td>
<td>19</td>
<td>-1</td>
<td>28</td>
</tr>
<tr>
<td>Pile 4</td>
<td>28</td>
<td>-3</td>
<td>42</td>
</tr>
</tbody>
</table>

Figures 4-35 to 4-38 and Table 4-5 indicate that under eccentric loading, the hoop strain is non-uniform along the cross-section and varies in proportion to the distance that the strain gauge is located from the pile segment’s mid-height. The strain gauges located below the mid-height of the cross-section (H-1 and H-3) were in tension because at these locations the outer surface of the pile segment expanded in the hoop direction as the longitudinal direction was compressed. The strain gauge located above the mid-height of the cross-section (H-2) was in compression because at this location, the outer surface of the pile segment contracted in the hoop direction as the longitudinal direction was stretched in tension. The observed variation in hoop strains demonstrates a complex state of stress for the transverse reinforcement in the pile segments.

4.5.3 Alternating Transverse Splitting and Eccentric Axial Compression Tests and Failure Modes

4.5.3.1 Crack Propagation and Failure Mode for Pile 1 (CFRP, 6” Wall)

As described in Section 4.3, a series of alternating transverse splitting tests and eccentric axial compression tests were conducted on each pile segment to simulate damage that occurs during the pile driving process, and to investigate the influence of cracking on the flexural behavior of the pile segment. The test sequence which explains the alternating order between the transverse splitting tests and the eccentric axial compression tests for each pile segment is described in Table
4-2. All pile segments were loaded to failure during the final transverse splitting test. Failure was considered to have occurred once there was a significant drop in the load-carrying capacity of the pile segment.

Table 4-6 illustrates the extent of cracking and level of damage for Pile 1 during the series of transverse splitting tests. Each increment listed included a pause in testing to document crack propagation. All piles were kept loaded with the transverse splitting force while cracks were marked.

<table>
<thead>
<tr>
<th>Transverse Splitting Force</th>
<th>Photo of Left Side after Transverse Splitting Force Applied</th>
<th>Photo of Right Side after Transverse Splitting Force Applied</th>
</tr>
</thead>
<tbody>
<tr>
<td>49 kips (1st incr.)</td>
<td>![Photo of Left Side]</td>
<td>![Photo of Right Side]</td>
</tr>
<tr>
<td>45 kips (2nd incr.)</td>
<td>![Photo of Left Side]</td>
<td>![Photo of Right Side]</td>
</tr>
</tbody>
</table>

Pile Re-Tested under Eccentric Axial Compression to 600 kips with 1’ Eccentricity

| Pile Reloaded in Splitting to 28 kips (failure) | ![Photo of Left Side] | ![Photo of Right Side] |
For Pile 1, several longitudinal cracks quickly formed during application of the initial splitting load at the 49 kip load level. The test was paused when these cracks were observed, and 49 kips was considered to be the first splitting load increment. On the left side of the pile, a 3’-2” long crack formed at mid-height. This crack was 0.030” wide at the loaded end of the pile segment and tapered down to 0.005” near its tip. On the upper right side of the pile, a 5’-10” long crack formed which tapered from 0.015” down to 0.005”. A second 4’-8” long crack formed slightly above mid-height on the right side. This crack tapered from 0.10” at the loaded end to 0.005” near the crack tip.

After initial cracking was documented, loading was continued in an attempt to propagate the crack to the midspan of the segment. Cracks propagated to near midspan with continued loading at levels up to 45 kips, however, the maximum force obtained during the first increment was not reached again. The second loading increment was paused when the crack on the left side extended to 7’-2” in length. At the loaded end, this crack had widened to 0.30”, tapering down to 0.005” with a few branches developing between 4’-0” and 6’-0” from the loaded end. On the right side, the lower crack extended to 8’-4” in length and the upper crack extended to 7’-0” in length. At the loaded end, the lower crack had widened to 0.40”, and the upper crack had widened to 0.015”. Two vertical cracks formed that connected these longitudinal cracks together.

After the longitudinal cracks had propagated to the midspan during the second transverse splitting loading increment, the transverse splitting test setup was removed, and the pile was re-tested under eccentric axial load to determine whether the presence of longitudinal cracks negatively impacted moment capacity. Results of the post-cracking eccentric axial test are presented in Section 4.5.3.5.
After re-testing the cracked pile under eccentric axial load, the splitting apparatus was re-installed in the end of Pile 1, and the pile segment was loaded to failure in splitting. At failure, the pre-existing longitudinal cracks propagated down the entire segment length, rupturing the carbon grid. The pile segment failed globally, essentially splitting down its length into two “half-cylinder” shapes, as shown in Figure 4-39. The maximum splitting load during the failure test of 28 kips was less than the peak load that initially caused cracking.

![Figure 4-39: Overall Failure Mode of Pile 1 (CFRP, 6” Wall)](image)

On both sides of the pile, the primary longitudinal cracks briefly angled diagonally upward at a location approximately one-third of the pile segment’s length from the loaded end before again becoming longitudinal. At these angled locations, it was observed that the carbon grid ruptured in both its longitudinal and transverse directions. It is noted that the longitudinal cracks did not appear to develop at locations where the carbon grid was overlaps in the circumferential direction.
For Pile 1, the initial longitudinal cracks formed suddenly and propagated rapidly. The three cracks that developed during the first transverse splitting test all originated at PT ducts that contained prestressing tendons (as opposed to un-grouted adjacent ducts). These crack locations are shown in Figures 4-41 and 4-42. Crack initiation at the location of bonded prestressed tendons is explained by the reduced cross-section in this area (due to the PT duct), and by the local bursting stresses created along the transfer length of the tendon.
It was observed that the crack widths reduced in size after the transverse splitting force was removed as the hydraulic jacks used in the test were retracted. This behavior was observed for all pile segments. The crack widths did not re-open to the full extent recorded during the transverse
splitting test when the eccentric axial compression test was conducted on the cracked pile segment. Additionally, the cracks did not propagate during the eccentric axial compression test. This result suggests that the carbon grid hoop reinforcement was transferring some level of force across the cracks, and this behavior was also observed for all pile segments. Figure 4-43 provides a visual comparison of the crack width size on the left side of Pile 1 during and after the first transverse splitting test.

Figure 4-43: Crack Width Comparison for Pile 1: (a) Transverse Splitting Load Applied and (b) Transverse Splitting Load Removed
4.5.3.2 Crack Propagation and Failure Mode for Pile 2 (Steel Spiral, 6” Wall)

Table 4-7 presents the extent of cracking and the level of damage observed for Pile 2 during the series of transverse splitting tests. Obvious initial cracking was observed at a load level of 142 kips, so loading was paused at this increment. Loading was continued in a second and a third increment with a goal of propagating the longitudinal cracks to a point near midspan. With the longitudinal cracks near the midspan location at the end of the third loading increment, the splitting test was removed, and the pile was re-loaded in eccentric axial compression to investigate the impact of the observed cracking on the moment capacity. After the eccentric axial compression test, the splitting apparatus was re-installed and Pile 2 was loaded in splitting to failure.

<table>
<thead>
<tr>
<th>Transverse Splitting Force</th>
<th>Photo of Left Side after Transverse Splitting Force Applied</th>
<th>Photo of Right Side after Transverse Splitting Force Applied</th>
</tr>
</thead>
<tbody>
<tr>
<td>142 kips (1st incr.)</td>
<td><img src="image1.png" alt="Photo of Left Side after 142 kips" /></td>
<td><img src="image2.png" alt="Photo of Right Side after 142 kips" /></td>
</tr>
<tr>
<td>219 kips (2nd incr.)</td>
<td><img src="image3.png" alt="Photo of Left Side after 219 kips" /></td>
<td><img src="image4.png" alt="Photo of Right Side after 219 kips" /></td>
</tr>
</tbody>
</table>
Several longitudinal and longitudinal/diagonal cracks formed after the 142 kip first splitting increment for Pile 2. On the left side, two longitudinal cracks formed near the top of the pile segment. The upper crack extended 5’-9” from the loaded end and tapered from 0.005” to hairline width. The lower crack extended 5’-2” from the loaded end, and a 1’-6” hairline-wide diagonal branch extended from this crack towards the middle of the section. The lower crack tapered from 0.015” at the loaded end down to hairline width. On the right side, two major cracks formed near the top of the pile segment. The upper of these cracks extended a length of 6’-2”. This crack tapered from 0.025” at the loaded end to 0.005”. At approximately 2’-0” from the loaded end of the pile segment, the lower crack split into two cracks which turned diagonally towards the middle of the section. The upper branch of this crack tapered from 0.020” to hairline width, and the lower branch tapered from 0.005” to hairline width. Crack propagation occurred more slowly for Pile 2 than for Pile 1.
After the 219 kip splitting load increment, the existing longitudinal cracks had not propagated much further in the longitudinal direction. Instead, these cracks began to turn vertical, seemingly following the direction of the steel spiral reinforcement. Additional longitudinal cracks formed at the bottom of the cross-section at this load level. At the loaded end of the pile segment, the longitudinal crack widths ranged from 0.005” to 0.20”. On the left side, a major vertical crack was located at 3’-6” from the end of the pile segment. The width of this vertical crack ranged from 0.025” to 0.050”. On the right side, two major vertical cracks were located at 3’-0” and 4’-8” from the end of the pile segment. The width of the vertical crack closest to the end of the pile segment varied from 0.015” to 0.025”. The width of the second vertical crack varied from 0.005” to 0.010”.

At the end of the 238 kip splitting increment, the cracking had continued to fan out. The cracks at the end of the pile segment widened up to 0.25”. The vertical crack on the left side widened to 0.10”. The vertical crack nearest the pile segment’s end on the right side widened from 0.050” to 0.075”. The second vertical crack on the right side widened to 0.020” to 0.030”. Spalling was also observed at mid-height on both the left and right sides of the pile segment.

After an eccentric axial compression test was conducted, the pile segment was loaded to failure during the final transverse splitting test. At failure, the crack pattern fanned out considerably, more vertical cracks developed, and the previous vertical cracks widened. The spalling at mid-height at the loaded end was severe at failure. The failure observed for Pile 2 was relatively localized at the loaded end of the pile segment. The overall failure mode is shown in Figure 4-44.
At failure, the internal steel spiral fractured at the top, left side of the pile segment. The fracture occurred near where the steel spiral had been welded to a longitudinal keeper rod, and the steel spiral was fractured for a distance of at least 28” from the loaded end of the pile segment. Beyond this distance, the crack width had closed, and it could not be determined precisely where the steel spiral fracture ended. The fractured steel spiral is shown in Figure 4-45.
As mentioned above, severe spalling at the mid-height of the cross-section occurred on both the left and right sides of the pile segment. The spalled portion of concrete extended to the depth of the steel spiral. This spalling is shown in Figure 4-46.

Figure 4-46: Spalled Concrete Cover for Pile 2

As with the prior test of Pile 1, all of the initial longitudinal cracks observed in Pile 2 originated at PT ducts that contained prestressing tendons. These crack locations can be seen in the cross-section view of the pile segment after failure, as shown in Figure 4-47.
On the inside surface of Pile 2, longitudinal cracks formed along the left and right sides of the pile segment. At a distance of approximately 3’-0” from the end of the pile segment, these cracks turned inward and connected in the hoop direction along the top and bottom inside surfaces. The 3’-0” length where the cracks converged matched the 3’-0” length of the concrete saddle used as a loading surface in the transverse splitting test setup. Figure 4-48 shows the internal cracking pattern for the bottom of the pile segment.
4.5.3.3 Crack Propagation and Failure Mode for Pile 3 (Steel Spiral, 6” Wall)

Table 4-8 presents the extent of cracking and level of damage for Pile 3 during the series of transverse splitting tests. Due to non-linearity observed in the strain distribution of Pile 2 after the post-cracking eccentric loading test, it was decided to subject Pile 3 to an intensive series of splitting and eccentric loading. The splitting load increments for Pile 3 were selected as 50 kip increments, and the pile was subjected to an eccentric load test after every splitting increment.
Table 4-8: Crack Propagation during Transverse Splitting Tests for Pile 3

<table>
<thead>
<tr>
<th>Transverse Splitting Force</th>
<th>Photo of Left Side after Transverse Splitting Force Applied</th>
<th>Photo of Right Side after Transverse Splitting Force Applied</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 kips</td>
<td>Uncracked</td>
<td>Uncracked</td>
</tr>
</tbody>
</table>

**Pile Re-Tested under Eccentric Axial Compression to 600 kips with 1’ Eccentricity**

100 kips

150 kips

196 kips (increment stopped short of 200 kips due to signs of impending failure)

**Pile Re-Tested under Eccentric Axial Compression to 600 kips with 1’ Eccentricity**

Pile was reloaded in an effort to reach 250 kips, but failed at 197 kips (failure)
No visible signs of cracking were observed after the first 50 kip transverse splitting test. After the 100 kip transverse splitting test, one longitudinal crack formed on the top, left side of the pile segment, and two longitudinal cracks formed at the bottom of the pile segment (bottom cracks not shown in Table 4-8). The upper crack was 4’-0” long and tapered from 0.015” at the end of the pile segment to hairline width. The bottom cracks varied in width from 0.005” to hairline. The right side appeared uncracked at this load level. One longitudinal crack appeared on each of the left and right inside surfaces of the pile segment near mid-height. These cracks varied in width from 0.015” to 0.010” at the end of the pile segment. Crack propagation for Pile 3 was gradual, similar to the behavior exhibited by Pile 2.

After the 150 kip transverse splitting test, a 7’-2” long longitudinal crack developed on the top, right side, and the existing longitudinal crack on the top, left side propagated to be 5’-2” in length. At the end of the pile segment these cracks ranged in width from 0.015” to 0.040”. A few vertical and diagonal cracks began to branch off of the longitudinal cracks and appeared to follow the direction of the steel spiral reinforcement. A major vertical crack formed 3’-6” from the end of the pile segment on the left side. This vertical crack width ranged from 0.010” to 0.020”. A major vertical crack also formed on the right side at 4’-0” from the end of the pile segment. This vertical crack width ranged from 0.010” to 0.030”. Two longitudinal cracks had now formed on each of the left and right inside surfaces of the pile segment. These cracks varied in width from 0.050” to 0.075” at the end of the pile segment.

After the 196 kip increment, the existing cracks continued to propagate in the vertical direction, following the orientation of the steel spiral. At this load level, the cracks at the end of the pile segment had widened up to 0.30”. The existing vertical cracks from the 150 kip transverse
splitting test widened up to 0.20”. The longitudinal cracks on the inside surface of the pile segment had widened up to 0.40”. Spalling was observed at the mid-height of the pile segment.

Near failure, the crack pattern fanned out considerably, and more vertical cracks developed during the final transverse splitting test to failure. The previous vertical cracks widened, and spalling at mid-height became severe. The failure for Pile 3 was relatively localized at the loaded end of the pile segment. In general, the crack development and the failure modes for Piles 2 and 3 were similar. The overall failure mode for Pile 3 is shown in Figure 4-49.

At failure, the steel spiral of Pile 3 fractured at the top, left side of the pile segment. The fracture did not appear to occur near a welded longitudinal keeper rod. The steel spirals were observed to be fractured for a distance of at least 17” from the loaded end of the pile segment. Beyond this distance, the crack width had closed, and it could not be determined precisely where the steel spiral fracture ended. The fractured steel spiral is shown in Figure 4-50.
As with Piles 1 and 2, all of the longitudinal cracks observed during the first four transverse splitting tests originated at PT ducts that contained prestressing tendons. These crack locations are shown in the cross-section view in Figure 4-15.

Figure 4-50: Fractured Steel Spiral for Pile 3: (a) Front View and (b) Top View

Figure 4-51: Cross-Section View of Pile 3
The internal cracking pattern observed for Pile 3 was similar to that described in the previous section for Pile 2.

4.5.3.4 Crack Propagation and Failure Mode for Pile 4 (CFRP, 5” Wall)

Table 4-9 presents the extent of cracking and level of damage for Pile 4 during the series of transverse splitting tests. Pile 4 was loaded in splitting until obvious longitudinal cracks were observed at a load level of 41 kips. The pile was then tested under eccentric axial load before being loaded to failure in splitting.

<table>
<thead>
<tr>
<th>Transverse Splitting Force</th>
<th>Photo of Left Side after Transverse Splitting Force Applied</th>
<th>Photo of Right Side after Transverse Splitting Force Applied</th>
</tr>
</thead>
<tbody>
<tr>
<td>41 kips</td>
<td><img src="image" alt="Photo of Left Side after 41 kips" /></td>
<td><img src="image" alt="Photo of Right Side after 41 kips" /></td>
</tr>
</tbody>
</table>

Pile Re-Tested under Eccentric Axial Compression to 600 kips with 1’ Eccentricity

| Pile Reloaded in Splitting to 23 kips (failure) | ![Photo of Left Side after 23 kips](image)                 | ![Photo of Right Side after 23 kips](image)                |

Several longitudinal cracks formed after the first transverse splitting test. A large crack formed suddenly underneath at the bottom of the cross-section (bottom crack not shown in Table 4-9). This crack was approximately 0.40” wide at the loaded end of pile, and it tapered down to
0.20” at a location approximately 1’-0” in from the end before splitting into two cracks. The carbon grid reinforcement ruptured at this crack location. Smaller longitudinal cracks also formed along the perimeter of the cross-section. On the top, left side a 7’-2” long crack formed which tapered from 0.010” at the loaded end down to hairline width. A lower crack on the left side formed which was 1’-10” long and tapered from 0.005” down to hairline width. On the top (not shown in Table 4-9), a 3’-6” long crack formed which tapered from 0.010” to hairline width. On the upper right side, a 2’-6” hairline width crack formed. At mid-height on the right side, a 2’-0” long crack formed which tapered from 0.010” to hairline width. Lower on the right side a wider, 1-4” long crack formed which tapered from 0.025” to 0.005”.

During the final transverse splitting test, the cracks on the bottom and lower left and right sides widened substantially and the carbon grid ruptured at the bottom of the cross-section extended to the support. The carbon grid also ruptured at the location of the lower crack on the right side. The failure for Pile 4 was relatively localized at the loaded end of the pile segment. The failure mode is shown in the Figures 4-52 and 4-53.

Figure 4-52: Overall Failure Mode of Pile 4
Unlike for Piles 1 through 3, not all of the longitudinal cracks in Pile 4 originated at PT ducts that contained prestressing tendons. It should be noted that the prestressing pattern used for Pile 4 differed slightly from the prestressing pattern used for Piles 1 through 3, as described in Section 4.2. The pattern for Pile 4 consisted of two consecutive PT ducts without prestressing tendons, unlike the pattern for the other piles, which contained prestressing tendons in at least every other PT duct. The cross-section view in Figure 4-54 shows the longitudinal crack locations.
4.5.3.5 Strain Distributions from Post-Splitting Eccentric Load Tests

The methodology described above in Section 4.5.2.1 was used to develop the strain distributions for the eccentric axial compression tests conducted after the pile segments were subjected to transverse splitting. For each pile segment, the strain distributions for the eccentric axial compression tests have been overlaid on the pile cross-section, as shown in Figures 4-55 to 4-58.
Figure 4-55: Total Strain Distributions Before and After Cracking for Pile 1

Figure 4-56: Total Strain Distributions Before and After Cracking for Pile 2
Figure 4-57: Total Strain Distributions Before and After Cracking for Pile 3

Figure 4-58: Total Strain Distributions Before and After Cracking for Pile 4
A linear strain distribution indicates that the pile segment has exhibited elastic behavior, and that all parts of the cross-section are behaving as a composite unit. This means that the linear region of the pile segment's stress-strain curve was not exceeded when the pile was loaded. When a pile is designed for a service loading condition, it is designed to behave elastically. A linear strain distribution after the transverse splitting test has been conducted on a given pile segment indicates that the design assumption for elastic behavior has not been violated for that pile segment.

In contrast, a nonlinear strain distribution after the transverse splitting test has been conducted on a given pile segment indicates that some portion of the cross-section is behaving in a non-elastic manner. In the case of a non-linear strain distribution, molecules have displaced longitudinally in shear, or in other words, elements within the pile segment's cross-section are slipping with respect to each other along the pile axis. The assumption that "plane sections remain plane", which is the basis for elastic design, has been violated, and the pile segment has exhibited some inelastic behavior.

After the pile segment exceeds its elastic capacity, localized concrete crushing and microcracking will occur. As a result, the elastic limit for the pile segment will be reduced, and the actual service loads imposed on the pile segment may begin to exceed the elastic stress limits calculated during design. Damage will accumulate in the pile segment as it is subjected to long term repeated loading, even to the service level, and the service life of the pile will be reduced.

The strain distributions for Piles 1 and 4 were linear after the first transverse splitting tests were performed. This result indicates that the axial service capacities of the carbon-grid-reinforced pile segments were not reduced after these piles were cracked to a selected level of damage (longitudinal cracks extending for half the length of the segment). However, for Pile 1, this level of damage developed after a splitting force of only 49 kips was applied, and for Pile 4, this level
of damage developed after only 41 kips was applied in splitting. Both of these load levels appeared to roughly correspond to the first cracking load of the pile, indicating that the provided CFRP hoop reinforcement was not effective at limiting longitudinal crack propagation after initial cracking. The linear strain distributions from the eccentric loading tests of Piles 1 and 4 indicate that the elastic carbon grid reinforcement was effective at clamping the opposing crack faces back together once the applied splitting loads were removed.

In contrast, the strain distribution for Pile 2 became nonlinear under eccentric loads after the splitting test forced longitudinal cracks to propagate to the midspan. However, for Pile 2, a maximum splitting force of 238 kips was needed to create the selected extent of cracking. For Pile 3, the strain distribution became somewhat non-linear after a transverse splitting force of 150 kips was applied, and it became highly nonlinear after a transverse splitting force of 200 kips was applied. This change in strain distribution linearity indicates that, after sustaining a splitting load of 150 kips, the original elastic moment capacity of the steel-spiral-reinforced pile segment was reduced, and the ability of the cross-section to sustain repeated loadings to the full service level may also be reduced. It is not possible to determine from the available data whether the observed shifts in strain distribution would impact the ultimate moment capacity, but an effect on ultimate capacity is a distinct possibility. Ultimate capacity could be impacted due to the potential loss of longitudinal shear capacity along the cracks. Effectively, the cracked cross-section would behave as a partially-composite system to the extent that aggregate interlock and dowel action of the reinforcement could not provide full longitudinal shear transfer. Moment capacity could also potentially be reduced due to the negative effects that longitudinal cracking could have on strand bond. The level of damage and extent of cracking that correspond to the transverse splitting forces are described in Sections 4.5.3.2 and 4.5.3.3.
It should be noted that the strain distributions were determined from strain gauges located at the midspan of pile segments. In the steel-reinforced piles, the damage simulated by the transverse splitting tests was concentrated at the ends of pile segments. This observation indicates that the localized heavy cracking at the end of the pile segment impacted the moment capacity of the pile segment at its midspan. This result suggests that the impact of cracking due to internal splitting forces or pressures on the bonded prestressed tendons is likely a significant mechanism. Cracking at any location along a steel-reinforced pile would also present a concern for durability, as chlorides could more easily reach the internal steel via the cracks.

In comparing the post-cracking strain distributions of the steel-reinforced and carbon-reinforced piles, it is important to recognize that the maximum transverse splitting forces required to generate the selected level of simulated damage differed greatly in magnitude between the two types of reinforcement. The maximum transverse splitting forces for the two carbon-grid-reinforced pile segments were 49 kips and 41 kips, whereas the maximum transverse splitting forces for the steel-spiral-reinforced pile segments were 238 kips and 197 kips. Thus, the steel-spiral-reinforced pile segments sustained approximately four to five times the transverse splitting force as compared to the carbon-grid-reinforced pile segments. As such, it is obvious that the provided level of carbon grid reinforcement was not sufficient to offer comparable structural performance to the standard steel-reinforced pile.

4.6 Analysis

4.6.1 Hoop Force

The maximum theoretical hoop force provided by the reinforcement types and configurations tested have been calculated and are provided in the table below. These forces have been calculated using the methodology outlined in Section 3.6.3. The maximum theoretical hoop
force for the steel spiral has been calculated as the product of the yield stress and the spiral wire area. For the carbon grid, the maximum theoretical hoop force has been determined using the mechanical properties provided by the manufacturer. The forces have been calculated for a linear foot of pile length, and neglect any contribution provided by the concrete in tension.

<table>
<thead>
<tr>
<th>Transverse Reinforcement Type</th>
<th>Area (in² per 1’ of pile length)</th>
<th>Tensile Strength (ksi)</th>
<th>Maximum Theoretical Hoop Force (kips per 1’ of pile length)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Spiral</td>
<td>0.66</td>
<td>$f_y = 70 \text{ ksi}$</td>
<td>46.2</td>
</tr>
<tr>
<td>Carbon Grid</td>
<td>0.019</td>
<td>$f_u = 290 \text{ ksi}$</td>
<td>5.5</td>
</tr>
</tbody>
</table>

Table 4-10: Maximum Theoretical Hoop Forces for Full-Scale Tests

The provided steel spiral reinforcement offers over eight times the maximum theoretical hoop force as compared to the provided carbon grid reinforcement. However, almost 35 times as much reinforcement area is provided by the steel spiral as compared to the carbon grid. This comparison implies that for a given amount of steel spiral reinforcement, approximately one-fourth of that amount in carbon grid reinforcement would provide the same theoretical hoop force, but would offer substantially less stiffness in tension due to the reduced reinforcement area. This relationship is explained with the equations presented below which equates the maximum theoretical hoop force provided by the steel spiral and the carbon grid. Note that the ratio of $A_{FRP}/A_{stee} = 0.24$ for the configurations and material properties tested:

$$A_{FRP} f_u = A_{steel} f_y$$

$$A_{FRP}/A_{steel} = f_y/f_u$$

As discussed in Chapter 2, current design requirements for the minimum transverse reinforcement in cylinder piles are prescriptive, so it is logical that any provided FRP
reinforcement must deliver the same performance as the currently-prescribed steel reinforcement. One concern with the as-provided level of carbon reinforcement was that it was not sufficient to develop forces substantially above those required to crack the concrete. Thus, while it is noted that no rational basis currently exists for proportioning hoop reinforcement in cylinder piles, it is proposed that transverse reinforcement of any type always provide a level of strength that offers a hoop tension capacity of at least $1.2T_{cr}$, where $T_{cr}$ is the cracking load of the concrete wall in tension. Based on ACI 318 Commentary Section R8.6.1, it is suggested that $T_{cr}$ be calculated using a concrete tension strength equal to $6.7\sqrt{f_c}$. The value $1.2T_{cr}$ is adopted from the requirement commonly specified in design codes that flexural reinforcement provide a minimum capacity of $1.2M_{cr}$, where $M_{cr}$ is the cracking moment of the section. The intent of this requirement is to provide sufficient strength to prevent a brittle failure at or immediately after cracking.

Assuming a design concrete compressive strength of 7 ksi, a 6” wall thickness, and subtracting the area lost due to the presence of a PT duct, any provided transverse reinforcement should provide a strength of at least 46.6 kips/ft to meet the $1.2T_{cr}$ criteria:

$$1.2T_{cr}A_c = 1.2 \times 6.7\sqrt{7000 \text{ psi}} \times (6-1.375")^{12} \frac{\text{in}}{\text{ft}} \times \frac{1 \text{ kip}}{1000 \text{ lb}} = 37.3 \frac{k}{ft}$$

The steel spiral reinforcement as provided effectively met this level of strength at 46.2 kips of capacity per foot. The tested level of carbon grid reinforcement provided only 15% of $1.2T_{cr}$.

**4.6.2 Proposed Failure Mechanism: Carbon Grid**

**4.6.2.1 Strain Distribution and Longitudinal Splitting Mechanism**

Strain primarily develops in the pile segment’s transverse reinforcement when cracks form. The magnitude of the strain in the transverse reinforcement is closely related to the crack width and the distance over which the reinforcement is able to distribute the change in length due to concrete cracking. The following equation defines this relationship:
\[ \varepsilon = \frac{w}{L} \]

Where \( \varepsilon \) is the strain in the transverse reinforcement, \( w \) is the crack width (or sum of crack widths, \( \sum w \), if multiple cracks), and \( L \) is the length of reinforcement over which the strain caused by cracking is distributed. Note that if a reinforcement were to have perfect bond with the concrete, which is not possible, \( L \) would be zero and the strain generated by any crack would be infinite. As such, the relative slip between the reinforcement and the concrete at cracking is of great importance to reinforcement strain.

The anchorage of the transverse reinforcement defines the length that is available to distribute the tension displacement caused by the crack width. The provided carbon grid reinforcement is rigidly connected at each crosstie, potentially limiting the length over which the tension displacement caused by the crack may be distributed, and creates a concentration of strain in the reinforcement. Thus, the spacing of the carbon grid crossties effectively dictates the length over which the displacement caused by a crack can be distributed.

If it is assumed that a crack width is distributed over a single 2” grid space, the crack width that exceeds the rupture strain in the carbon grid reinforcement is calculated as:

\[ w = \varepsilon_{FRP}L = 0.0085(2") = 0.017" \]

Where \( \varepsilon_{FRP} \) is the specified rupture strain of the carbon grid.

The crack width that exceeds the steel yield strain may be calculated in a similar manner. The steel spiral is rigidly restrained where it is welded to longitudinal keeper rods, and the spiral length between the keeper rods is the distance over which the crack may be distributed. For the steel-spiral-reinforced pile segments, this distance is approximately 26”. The corresponding crack width is calculated as:

\[ w = \varepsilon_yL = 0.0024(26") = 0.063" \]
Where $\varepsilon_y$ is the specified yield strain of the steel spiral. The crack width that exceeds the steel yield strain is over three times greater than the crack width that exceeds the rupture strain of the carbon grid.

It may be unrealistic to assume that the provided carbon grid can be fully-developed by a single crosstie. If it is assumed that two crossties are needed to develop the grid in tension, then a crack width would be distributed a longer length of grid, and crack width causing rupture would increase. Table 4-11 summarizes the crack width calculation, demonstrating the crack width that would be required to rupture the grid given various assumed distribution lengths, $L$.

**Table 4-11: Summary of Crack Widths Corresponding to Rupture in Carbon Grid**

<table>
<thead>
<tr>
<th>No. of 2” Grid Spaces</th>
<th>Crack Distribution Length, $L$ (in)</th>
<th>Crack Width to Cause Rupture, $w$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>0.017</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>0.034</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>0.051</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>0.068</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>0.085</td>
</tr>
</tbody>
</table>

A distribution length of 8” (4” on either side of the crack) would be required for a carbon grid with the specified rupture strain to handle a crack width equal to that which would theoretically cause the steel spiral to yield. This amount of distribution is probably not realistic for the 2” grid spacing provided because it implies that the development length of the grid exceeds the spacing of two cross-ties. For plain welded-wire reinforcement, ACI 318 Section 12.8 specifies that a wire with welded cross-ties is fully developed by two cross-ties.

The carbon grid reinforcement also differs from the steel spiral reinforcement because it is a material that exhibits a brittle linear-elastic behavior. Unlike the steel spiral which is ductile and
has reserve capacity after yield, the carbon grid can no longer carry load once the rupture strain has been reached. Once a carbon strand has ruptured, the load that it was carrying is transferred to neighboring strands. This behavior explains the relatively rapid formation of the longitudinal cracks that developed when the carbon-grid-reinforced pile segments were loaded with a transverse splitting force. It is likely that a carbon strand located near the loaded end of the pile segment ruptured initially. The force in this strand was then transferred to a neighboring strand, and the strength of this neighboring strand was rapidly exceeded, it ruptured, and the force was again transferred to the next strand down the length of the pile segment. This cycle of strand rupture and load transfer progressed along the length of the pile segment, resulting in a rapidly-progressing longitudinal crack. The development of extensive longitudinal cracks observed during the transverse splitting tests of Pile 1 agrees with this longitudinal splitting mechanism.

A carbon-grid-reinforced pile segment that is split longitudinally will begin behave as two “half-cylinders” instead of behaving as a composite cross-section. As the carbon grid stretches and then ruptures along the crack interface, no reinforcement remains to hold the two pile segment halves together. Prior to grid rupture, aggregate interlock between the two half-cylinders will break down as crack widths widen. It is also postulated that FRP reinforcement will not contribute to dowel action in the same way that steel reinforcement will due to the inherently is weak shear strength of the FRP matrix. As a result, a carbon-grid-reinforced pile segment loses structural

Figure 4-59: Longitudinal Splitting Mechanism

150
integrity if the longitudinal splitting mechanism is allowed to develop and propagate down its length.

4.6.2.2 Effect of Wall Thickness

The failure mode for Pile 1 was a global splitting failure, whereas the failure mode for Pile 4 was a local failure. The difference in behavior may be attributed to the difference in wall thickness between Pile 1 and Pile 4. Pile 1 had a 6” wall thickness, whereas Pile 4 had a 5” wall thickness.

The longitudinal cracks which formed during the transverse splitting tests for Pile 1 propagated down the entire length of the pile segment, splitting it longitudinally into two “half-cylinders”, as described in Section 4.5.3.1. The longitudinal cracks which formed for Pile 4, however, did not propagate beyond midspan, and the large cracks which formed at the bottom of the cross-section near the loaded end of the pile segment resulted in a local failure. These failure modes are described in more detail in Sections 4.5.3.1 and 4.5.3.4.

It is proposed that the reduced wall thickness resulted in the change from global to local failure. The half-cylinders which formed for Pile 1 can be idealized as fixed cantilevers which resist the transverse splitting force, as shown in Figure 4-60. The flexural stiffness of these cantilevers depends largely on the wall thickness. If the pile segment’s wall is sufficiently thick, half-cylinder cantilevers will form as the longitudinal crack propagates down the length of the pile segment. As the wall thickness is reduced, the concrete will begin to crush and break locally, and the cross-section will not be stiff enough to form long half-cylinder cantilevers. The stiffness provided by the cross-section geometry of the pile must be sufficient to enable the longitudinal splitting mechanism described in the previous section.
The difference in failure modes for Piles 1 and 4 may also be explained by the location of the voids due to ungrouted PT ducts within the two prestressing patterns. The pattern of ungrouted PT ducts for Pile 1 was different than for Pile 4, as shown in Figure 4-5. For Pile 1, each ungrouted PT duct had PT ducts on either side that were grouted and contained prestressing tendons. For Pile 4, each ungrouted PT duct had a grouted PT duct on only one side. The multiple consecutive voids in Pile 4 created locally weak areas in the cross-section, rendering the concrete less able to resist local shear. This proposal is supported by the observation that only for Pile 4 did cracks originate during the transverse splitting tests at empty PT ducts.

The contribution of the concrete tensile strength to the pile segment’s capacity to resist longitudinal cracking also affects the failure mode. The tensile force provided by the concrete depends on the cross-sectional area of the wall and the tension rupture stress of the concrete. As the wall dimension is decreased, the gross cross-sectional area is reduced in proportion. However, the reduction in net cross-sectional area is not proportional to the decrease in wall thickness due to the voids created by the PT ducts, particularly any ungrouted PT ducts. This non-proportional relationship between wall thickness and net area is demonstrated in Figure 4-61 and Table 4-12.
Table 4-12: Summary of Reduction in Wall Cross-Sectional Area due to PT Duct Void

<table>
<thead>
<tr>
<th>Wall Thickness (in)</th>
<th>Gross Wall Area, $A_{\text{gross}}$ (in$^2$/ft)</th>
<th>Area of Void, $A_{PT \text{ void}}$ (in$^2$/ft)</th>
<th>Gross Wall Area, $A_{\text{net}}$ (in$^2$/ft)</th>
<th>$A_{PT \text{ void}}/A_{\text{gross}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>60</td>
<td>16.5</td>
<td>43.5</td>
<td>28%</td>
</tr>
<tr>
<td>6</td>
<td>72</td>
<td>16.5</td>
<td>55.5</td>
<td>23%</td>
</tr>
<tr>
<td>7</td>
<td>84</td>
<td>16.5</td>
<td>67.5</td>
<td>20%</td>
</tr>
<tr>
<td>8</td>
<td>96</td>
<td>16.5</td>
<td>79.5</td>
<td>17%</td>
</tr>
</tbody>
</table>

The percentage of the gross cross-sectional area taken up by the void becomes more significant as the wall thickness is reduced. The difference in the net cross-sectional area for a 5” versus a 6” wall thickness is significant (22% less net area for a 5” thick wall), and helps to explain the lower capacity observed for Pile 4 (5” wall) as compared to Pile 1 (6” wall). The difference becomes even more significant when comparing a 5” wall thickness to an 8” wall thickness (45% reduction). Consequently, hoop reinforcement becomes a more important consideration as the wall thickness is reduced because the thinner wall is more likely to crack under applied loads, meaning that the internal hoop reinforcement is more likely to be mobilized.

4.6.3 Proposed Failure Mechanism: Steel Spiral

The cracking pattern for the steel spiral-reinforced-pile segments was much more distributed as compared to the carbon-grid-reinforced pile segments. The cracks fanned out as the
transverse splitting force was increased, and longitudinal, vertical, and diagonal cracks developed. This pattern formed as the steel spiral yielded and distributed the load throughout the end region of the pile segment. Crack propagation was gradual for this reason.

The local failure mechanism for the steel-spiral-reinforced pile segments under the applied concentrated loading can be explained by idealizing the pile segment as a series of composite concrete and steel “rings”. These rings dilate inward at their sides as they are stretched vertically by the transverse splitting force, deforming the cross-section into an “egg-shape”, as shown in Figure 4-62.
Figure 4-63 shows how three vertical cracks divide the end of the pile segment into sections that can be conceptualized as three rings. The arrows indicate the direction of tensile (blue) and compressive (red) forces internal to the pile segment. The vertical cracks can be explained by the fact that the hoop direction of the pile is placed into tension causing the width of each ring to contract along the longitudinal axis of the pile.

As the transverse splitting force was applied, an initial vertical crack formed near the end of the pile segment. The left and right sides of the pile segment began to straighten, and longitudinal cracks formed at these locations on the inside surface. As the sides straightened, the steel spiral also straightened, pushing an inner piece of the concrete wall into the pile segment’s
void. The concrete cover outside of this straightening hoop reinforcement was compressed and started to spall off. At the top and bottom of the pile segment, cracks formed which tapered down from the outside surface towards the inside surface. This behavior is shown in Figures 4-64 and 4-65.

Figure 4-64: Cross-Section View of Steel-Spiral-Reinforced Pile Segment Failure Mechanism
As the transverse splitting force was increased, damage incrementally progressed along the length of the pile segment. Successive vertical cracks divided the pile segment into composite concrete and steel “rings”. The steel spiral continued to yield and distribute load. This was accompanied by additional spalling of the concrete cover on the outside surface and crushing of the concrete on the inside surface at mid-height. The tapered cracks located at the top and bottom of the cross-section widened and increased in length.

The failure mechanism described above is partly attributed to the design of the test setup. The inside of the pile segment was loaded vertically and not with a uniform radial pressure. As a result, the walls of the pile segment were in a non-uniform state of stress instead of a more uniform state of tension. This distribution of concentrated internal loading created an axial effect and a moment effect in the plane of the cross-section. The forces on the cross-section are shown in Figure 4-66. This figure has been adopted from *Advanced Mechanics of Materials* by Boresi and Schmidt, page 349, which presents ultimate stress calculations based on elastic theory for a closed ring that is subjected to a concentrated load. Although the pile segments tested were deformed well beyond their elastic states, the concept of additive axial and moment effects within elements of the cross-section is still applicable.
Figure 4-66: Forces on Cross-Section due to Vertical Loading

Under the concentrated internal loading applied, the outside surface of the pile segment was in tension at the top and the bottom of its cross-section. This tension decreased across the wall thickness, in toward the void of the cross-section. At mid-height, the net stress in the pile wall was the summation of stresses from the axial and moment effects. At this location, the inside surface experienced greater tension in this vertical loading condition than it would have under an equivalent uniform radial pressure due to the moment effect. The state of stress on the outside surface at mid-height depended on the magnitude of the local moments that developed in the plane of the cross-section. To a large degree, the magnitude of those local moments depends on the extent of deformation of the cross-section. As the cross-section is loaded, moments develop in the plane of the cross-section; those moments cause the cross-section to locally straighten, which in turn reduces the moment demand; however, the applied load is likely increasing to cause continued deformation which is acting to increase the moment demand. Thus, the problem is highly non-linear.

For a given pile segment cross-section diameter, a thinner wall would be relatively more flexible and would straighten more easily than would a thicker wall. Additionally, the internal
tension added to the hoop reinforcement due to the moment effect would be less severe for a section with a thinner wall because the lever arm within the wall thickness would be smaller. As a result, the hoop reinforcement tension demand for a concentrated internal loading condition should theoretically reduce as the wall becomes thinner. In practice, this effect may not be significant, however, it is an interesting mechanism.

The concentrated vertical loading condition used in the transverse splitting test setup was intended to approximate forces that may develop during pile driving. In actual driving conditions, a pile will likely be subjected to a more uniform loading due to internal pressure such as water hammer or the formation of a mud plug. In this case, the pile segment would deform into an “elephant-foot” shape instead of an “egg-shape”, as shown in Figure 4-67.

It is possible, however, for non-uniform internal stress conditions to develop during pile driving. This may occur if the pile encounters uneven soil conditions, obstacles (boulders) in the soil, or other resistance during driving. As shown above, non-uniform loading generally magnifies
the tension demands on the hoop reinforcement. This loading condition is one justification for a conservative amount of transverse reinforcement to be included in a pile segment.
Chapter 5: Summary, Recommendations, and Conclusions

5.1 Summary of Research Findings

The research presented in this thesis summarizes the results of an experimental program undertaken to investigate the feasibility of replacing steel transverse reinforcement in precast concrete cylinder piles with FRP transverse reinforcement. The potential use of FRP transverse reinforcement would allow for more durable piles, due to the fact that FRP does not corrode. The use of FRP may also allow for thinner pile walls, as requirements for concrete cover over the transverse reinforcement could be reduced. Initially, six scaled concrete hollow cylinder pile segments were fabricated using varied types of transverse reinforcement. These specimens were tested in concentric axial compression and were analyzed to evaluate the potential for using FRP transverse reinforcement. Four full-scale spun-cast pile segments were also experimentally tested as part of this research. Two full-scale pile segments contained transverse FRP reinforcement, and one of these was produced with a reduced wall thickness. The remaining two pile segments contained transverse steel reinforcement. These pile segments were tested in the laboratory under concentric axial load, eccentric axial load, and transverse splitting. The axial compression tests were representative of service loading conditions, and the transverse splitting test simulated damage that could occur during pile driving. The pile segments were loaded under eccentric axial load both before and after splitting to investigate the effects of longitudinal cracking on their ability to resist an applied moment. In service, applied moments could result from eccentric axial load, lateral load, or both. Findings of this research are summarized as follows:

1. A review of many state DOT design standards revealed that a variety of standard hollow precast pile cross-sections are used. Design details such as concrete cover thickness vary
from agency to agency, however, all of the hollow pile design standards located were prescriptive for transverse reinforcement. Prescriptive transverse reinforcement specifications proportion the reinforcement from experience, not from any stated rational approach. Internal transverse steel spiral reinforcement is typically specified at a 6” or 2” pitch (depending on pile diameter) with a wire diameter of approximately 3/8”. Hollow cylinder piles are typically post-tensioned longitudinally with steel tendons, and flexural design for this longitudinal reinforcement follows the rational methods of conventional practice.

2. Transverse reinforcement in cylinder piles is primarily necessary to arrest the propagation of cracks which may occur during driving. The transverse reinforcement serves to maintain the integrity of the cross-section, to prevent splitting, and to restrain any developed cracks to reasonable widths so that longitudinal shear transfer across the crack is not compromised. Loss of longitudinal shear capacity along a crack could mean that strain distributions on the pile cross-section become inelastic (plane sections would no longer remain plane), potentially compromising the ability of the pile to resist applied moments. Maintaining small crack widths is also important for protecting the strand-to-concrete bond needed to ensure effectiveness of the longitudinal reinforcement, and for maintaining corrosion resistance in steel-reinforced piles.

3. In general, the transverse reinforcement in the hollow sections tested did not provide a reliable concrete confinement mechanism. With no concrete core to confine, transverse
reinforcement in hollow sections should not be relied upon to enhance axial strength through confinement, even if substantial transverse reinforcement is provided.

4. Hollow cross-sections should generally be considered to have a limited shear capacity. A review of the available literature indicates that substantial levels of transverse reinforcement do not dramatically enhance the ability of a hollow cross-section to resist shear. As such, hollow concrete cylinder piles are most appropriate for applications with high axial and possibly high flexural demands. This shear limitation is likely why uses of hollow cylinder piles in the United States tend to be concentrated in the eastern portion of the country, where shear-critical seismic applications are uncommon.

5. Replacement of traditional steel transverse reinforcement with FRP transverse reinforcement in the scaled compression tests neither degraded nor enhanced the axial load capacity of the scaled hollow cylinder pile segments. The reduced transverse stiffness of the FRP reinforcement compared to the steel reinforcement did not negatively impact the performance of these specimens. One FRP-reinforced specimen did substantially out-perform all others (including the steel-reinforced specimen), possibly indicating that some confinement effect did develop in this specimen. Alternatively, it is possible that other specimens suffered a loss of performance due to localized stresses at their ends, however, the observed failure modes do not support this conclusion. With the limited number of tests, it is not possible to conclude that any reliable confinement effect was observed, particularly given that the unreinforced specimen and the steel-reinforced specimen exhibited similar behaviors.
6. For the full-scale tests, the steel-spiral-reinforced pile segments exhibited gradual crack propagation as the steel spiral yielded and transferred load. These specimens exhibited relatively localized failures and retained substantial integrity after failure. One carbon-grid-reinforced pile segment exhibited a global longitudinal splitting mechanism, highlighting the brittle linear elastic nature of the carbon grid reinforcement. The second CFRP-reinforced specimen exhibited a more local failure mode, also with substantial cracking. Crack propagation for both FRP-reinforced specimens was sudden and did not allow for substantial load redistribution. The quantity of FRP transverse reinforcement in the tested specimens was minimal compared to the quantity of steel reinforcement, and the levels of FRP provided were not sufficient to prevent splitting failure. It is anticipated that full-scale pile segments reinforced with more substantial quantities of FRP could exhibit failure modes more similar to the modes observed for the steel reinforced piles, however, additional tests would be needed.

7. The ultimate splitting capacities of the steel-reinforced pile segments were substantially larger than were the capacities for the FRP-reinforced pile segments. This result is unsurprising given the substantial difference in reinforcement quantity provided in the steel specimens as compared to the FRP specimens. The steel-reinforced piles sustained splitting loads of 238 and 197 kips while the FRP-reinforced piles sustained loads of 49 and 41 kips. The peak splitting loads for the FRP-reinforced piles occurred early in the loading process, as cracking first developed, indicating that the tensile strength of the concrete was a major contributor to splitting strength in these specimens. For the steel-reinforced piles, peak loads were reached after substantial cracking had developed, indicating the ability of the
steel reinforcement to take substantial additional load after cracking. It is anticipated that piles with increased quantities of FRP transverse reinforcement could develop similar splitting loads to those exhibited by the steel piles, but again, additional tests would be needed to verify this hypothesis.

8. An alternative distribution of FRP reinforcement may help to mitigate the brittle failure observed with the FRP reinforcement tested. The transverse strands of the carbon grid used in the full-scale test specimens were rigidly anchored at close intervals by the grid strands running in the longitudinal direction. This arrangement of a relatively tight, square grid spacing likely concentrated strains in the transverse reinforcement over a relatively short distance at each crack in the concrete. Selecting a grid with a larger distance between transverse strand anchors (or a grid with less rigid connections at each node) would likely enhance the ability of the grid to better distribute strain near cracks. Such a change would also likely improve the ability of a given transverse strand of grid to share load between adjacent transverse strands of grid prior to rupture. It is noted that steel reinforcement can also be sensitive to localized strain concentrations due to cracking if it is anchored over a very short length. The steel spirals tested were anchored by welded keeper rods spaced approximately 26” apart along the circumference of the spiral. An ideal configuration may be transverse reinforcement (steel or FRP) with no discrete points of anchorage, however, this may not be practical from a constructability standpoint.

9. Reducing the wall thickness in the full-scale pile segments seemed to trigger a change in failure mode from a global failure to a local failure in the splitting test performed. Factors
influencing this failure mechanism include the pile cross-section geometry, the prestressing pattern, and the strength and stiffness of the provided transverse reinforcement. The failure mode of hollow piles subjected to internally-applied splitting loads is governed by a variety of factors, not only transverse reinforcement type and quantity.

10. The service level moment capacities of the carbon-grid-reinforced pile segments, as tested under eccentric axial load, were not reduced after substantial longitudinal cracking was developed. This conclusion is based on the linear strain distributions observed during eccentric axial load testing after substantial splitting damage was induced. Substantial splitting damage included crack widths of up to 3/8” with the load applied (and up to 0.150” with the splitting load removed) at splitting loads up to 49 kips. The result demonstrates a potential benefit of the linear elastic behavior of the FRP reinforcement, as the carbon grid reinforcement (even in the small quantities provided) was very effective at closing cracks after applied loads were removed. This finding may be particularly relevant to the pile driving process, as loads that often crack a pile during driving are temporary. Prior to yielding, steel reinforcement would also act to close cracks after applied loads are removed, however, the elastic strain limit of steel reinforcement is much less than the elastic strain limit of typical FRP reinforcement.

11. The steel-spiral-reinforced pile segments also retained their service moment capacities after substantial cracking had developed, as evidenced by linear strain distributions for Pile 3 under eccentric axial load tests conducted after splitting tests up to 100 kips. However, nonlinearity in the strain distributions for the steel-reinforced piles developed during eccentric axial loading after higher levels of splitting loads were applied (238 kips for Pile
2 and 150 kips for Pile 3). These nonlinear strain distributions at midspan demonstrated that relatively localized damage observed at the end of a pile segment affected its capacity at midspan, probably by negatively influencing strand bond. This result implies that the levels of damage observed in the steel-reinforced piles after the 150 kip splitting load level may not be acceptable from a structural perspective. Damage beyond the levels observed at the 150 kip splitting load levels could potentially degrade the elastic moment capacity of a pile, or at least of the segment containing the damage. Observed crack widths at the 150 kip splitting level for the steel-reinforced piles were generally less than 0.075” with the load applied and less than 0.015” after the load was removed. Note that concerns for environmental durability may require smaller acceptable crack width limits, particularly for piles with steel reinforcement.

12. The carbon grid reinforcement provided in the full-scale pile segments was dramatically undersized in terms of providing the same hoop strength as the steel spiral reinforcement. The steel spiral reinforcement in the configuration tested provided over eight times the maximum theoretical hoop force as did the provided carbon grid reinforcement in the configuration tested. This difference was highlighted by the ability of the steel-spiral-reinforced pile segments to sustain much greater transverse splitting forces than the carbon-grid-reinforced pile segments. For FRP-reinforced pile segments to offer the same level of performance as steel-reinforced segments, a substantially larger quantity of FRP transverse reinforcement would be required. Recall that the carbon-grid-reinforced pile segments were not designed specifically for this research program, and were not expected to match the performance of the steel-reinforced specimens. Rather, the CFRP-reinforced
segments were originally fabricated to examine whether the spin-casting process could be successfully implemented with a grid reinforcement and were then made available to the current research.

13. Transverse reinforcement of any type should be approximately centered in the wall thickness, as longitudinal cracking under the applied concentrated splitting load was observed to initiate on both the outside and inside surfaces of the pile segments. While a more uniform internal load is likely for typical driving conditions, concentrated splitting forces are possible due to non-uniform hammer strike, contact of the pile tip with objects, or uneven driving resistance. Typical practice of centering the strands in the wall thickness and placing the transverse reinforcement outboard of the strands is reasonable. Configurations that place the transverse reinforcement very close to the extreme outer location of the section, such as a very thin outer concrete cover over FRP transverse reinforcement, should be avoided, as this placement would render the transverse reinforcement ineffective at arresting cracks that may develop on the inner surface of the pile.

14. The prestressing pattern and the wall thickness may affect the location of longitudinal crack formation. For piles which had the same prestressing pattern and the same wall thickness, longitudinal cracks always originated at grouted PT ducts that contained prestressing tendons. For the pile with a reduced wall thickness and a different prestressing pattern, longitudinal cracks originated at PT ducts regardless of whether those ducts contained prestressing tendons. Unused ducts should always be grouted for steel-reinforced piles (for
corrosion prevention) but could theoretically be left un-grouted for FRP-reinforced piles since corrosion of the FRP transverse reinforcement would not be a concern.

15. Non-uniform radial loads can magnify tensile demand on the transverse reinforcement beyond the demand that would commonly be assumed for uniform radial loading. The potential to have uneven radial loading justifies providing a conservative amount of transverse reinforcement.

16. The tensile demand on the transverse reinforcement under non-uniform internal radial loads reduces as the wall becomes thinner because the lever arm for the moment effect is reduced. Stated otherwise, the flexural rigidity of a thin wall is less than that of a thick wall. In practice, this effect is likely not relevant for design.

17. The full-scale test piles fabricated with carbon grid did not show any indications of the grid interfering with the pile fabrication. Concrete was well-consolidated in all areas, and there were no signs of aggregate segregation at the grid interface. This result is especially important given the relatively small dimensions of the grid openings for the FRP used. Overall, it will be important to optimize the fabrication process for FRP-reinforced cylinder piles if FRP reinforcement will compete with the highly-automated production of steel-spiral reinforcement for cylinder piles. Considerations include the use of prefabricated reinforcement cages to reduce time and labor, the use of non-corrosive reinforcement accessories (such as plastic ties and longitudinal FRP keeper rods), and a reinforcement arrangement that permits suitable consolidation of concrete without unintentionally displacing or damaging that reinforcement.
5.2 Design Recommendations

The research findings have been used to develop design recommendations for hollow precast concrete cylinder piles with steel or FRP transverse reinforcement. The design recommendations derived from this research are presented as follows:

1. Current transverse reinforcement requirements for hollow piles are prescriptive. It is proposed that a rational basis for proportioning transverse reinforcement would specify that a minimum amount of transverse reinforcement be provided so that the strength from the reinforcement at least equals \(1.2T_{cr}\) where \(T_{cr}\) is the cracking load of the concrete wall in tension. It is suggested that \(T_{cr}\) be calculated using the net wall area (PT ducts subtracted) and a concrete tension strength equal to \(6.7\sqrt{f'_c}\) (based on ACI 318 Commentary Section R8.6.1). The intent of this requirement is to provide transverse reinforcement strength sufficient to prevent a brittle failure immediately after cracking. Such a requirement should apply to all types of transverse reinforcement for the types of large-diameter hollow piles tested.

2. It is proposed that for FRP transverse reinforcement to be used in large-diameter hollow cylinder piles it should, at a minimum, provide the same theoretical hoop force as the steel transverse reinforcement it replaces. It is also recognized that the stiffness of the provided transverse reinforcement is an equally important consideration. Therefore, it is further proposed that the maximum strain used to calculate the hoop force in any transverse reinforcement for hollow piles (but particularly for FRP transverse reinforcement) be limited to 4000 \(\mu\varepsilon\). The proposed minimum required area of FRP transverse reinforcement...
required to replace prescribed steel reinforcement in the types of piles tested, $A_{FRP, min}$, is given by the following equations:

$$A_{FRP, min} = \left( \frac{f_y}{f_{frp}} \right) A_{steel}$$

$$f_{frp} = 0.004E_{frp} \leq f_{fu}$$

Where $f_y$ is the yield strength of the steel reinforcement, $A_{steel}$ is the area of the specified steel reinforcement being replaced, $E_{frp}$ is the modulus of elasticity of the FRP reinforcement, and $f_{fu}$ is the design tensile strength of the FRP reinforcement.

The effective strain limit of 4000 $\mu$e is adopted from ACI 440.1R-15 as the maximum strain that prevents degradation of aggregate interlock in a resistance mechanism for beam shear. This limit is adopted here for the less severe condition of reduced longitudinal shear capacity along a longitudinal crack. Such a limit will also help to mitigate the loss of strand bond due to the development of excessively-wide cracks.

It is recognized that in lieu of the proposed 4000 $\mu$e effective strain limit, stiffness requirements could be imposed on the FRP reinforcement to equate the axial rigidity, $EA$, of the FRP transverse reinforcement to that of the steel reinforcement ($EA_{FRP} = (EA)_{steel}$). This “equal EA” approach is viewed as overly-restrictive and is not recommended because it has the potential to greatly increase the minimum required area of FRP, especially for GFRP, which showed promise in the scaled compression tests. The “equal EA” approach is also not justified in light of the linear strain distributions observed for the damaged CFRP-reinforced piles in the full-scale pile segment tests. Additional research is needed to investigate the validity of these proposed conditions.
3. It is proposed that sustained stress in the transverse reinforcement of hollow cylinder piles be limited. In practice, the level of sustained stress in the transverse reinforcement would be evidenced and evaluated by the presence of cracks after driving that were not fully closed. For steel transverse reinforcement, the yield stress is a convenient and appropriate limit to ensure the stability of any existing cracks. For FRP transverse reinforcement, stress in the FRP should be limited so that any sustained stress does not exceed the creep rupture stress of the FRP. The proposed limitation is based on allowable creep rupture stress because any stress remaining in the transverse reinforcement after driving would likely be permanent (as evidenced by cracks of measurable width).

For steel transverse reinforcement, the proposed limitation on yield stress could be evaluated through observed crack width where the sum of observed longitudinal crack widths, $\sum w$, may be calculated as:

$$\sum w = \varepsilon_y L$$

Where $\varepsilon_y$ is strain in the steel at its yield stress and $L$ is the length of transverse reinforcement along the pile circumference over which the cracks are distributed. For steel spiral transverse reinforcement welded to longitudinal keeper rods, it is proposed that $L$ can be taken as equal to the length of the steel spiral between welds, with a maximum value equal to one-quarter of the pile’s circumference. For steel transverse reinforcement that does not have rigid connections, it is proposed that $L$ be taken as one-quarter of the pile’s circumference.

For FRP transverse reinforcement, it is proposed that the limit on FRP stress be evaluated through the sum of observed longitudinal crack widths, $\sum w$, as follows:

$$\sum w = \frac{f_{fs,sus}}{E_{FRP}} L$$
Where \( f_{S, SUS} \) is the creep rupture stress of the FRP reinforcement, \( E_{FRP} \) is the modulus of rupture of the FRP, and \( L \) is the length of transverse reinforcement over which the observed cracks are distributed. For grid arrangements with rigid cross-ties, \( L \) is taken as three times the grid spacing, assuming that a given transverse strand of the grid can be fully developed by two cross-ties in either direction. For other arrangements (i.e., spiral or hoop), it is proposed that \( L \) may be taken as the distance between rigid connections, or one-quarter of the pile’s circumference, whichever is less. The creep rupture stress of the FRP reinforcement may be based on the creep rupture stress limits defined in ACI 440.1R-15. The stress limits are \( 0.55 f_{fu} \) for CFRP, \( 0.20 f_{fu} \) for GFRP, and \( 0.30 f_{fu} \) for AFRP where \( f_{fu} \) is the design tensile strength of the FRP.

The proposed methodology can be used by engineers, designers, and future researchers to develop suitable acceptance criteria for hollow piles under specific conditions. Engineers are often asked to evaluate cracking observed in piles after driving to determine whether a particular pile remains suitable for the intended use.

### 5.3 Conclusions

The research findings and design recommendations are distilled into the following conclusions.

- The use of FRP as transverse reinforcement in hollow cylinder piles does not appear to degrade or enhance the axial load carrying capacity. The primary job of transverse reinforcement in these types of piles is to prevent longitudinal cracking during driving.
- Transverse reinforcement in hollow sections does not offer a reliable confinement mechanism for a hollow section as there is no core of concrete to confine. As such,
transverse reinforcement should not be used to enhance axial load capacity (via confinement) of hollow piles.

- Transverse reinforcement in hollow pile sections is currently provided by prescriptive specification. No rational design methods exist in the available literature. Traditionally, heavy levels of transverse reinforcement are provided. The heavy reinforcement appears justified based on the potential for highly-variable (and potentially non-uniform) splitting forces to develop during driving.

- FRP transverse reinforcement has the potential to replace steel transverse reinforcement in hollow cylinder piles, however, the provided levels of FRP need to offer strengths equivalent to those offered by the prescribed steel reinforcement. Alternatively, the overall performance of an FRP-reinforced pile would have to be tested through full-scale driving and testing and shown to be equivalent to a traditional steel-reinforced pile.

- The full-scale FRP-reinforced piles did not have a sufficient level of transverse reinforcement as tested. As such, they failed due to splitting in a manner that would be undesirable in the field. Future tests of FRP-reinforced piles should examine specimens with greater levels of transverse reinforcement.

- The linear elastic nature of the FRP transverse reinforcement could be a benefit in hollow cylinder piles from the perspective of closing cracks that develop during driving once the temporary driving loads are removed. Tests of full-scale piles with FRP transverse reinforcement indicated that even light levels of FRP transverse reinforcement could hold cracks together after substantial simulated driving damage.
• The basis for a rational design approach for proportioning steel or FRP transverse reinforcement in hollow cylinder piles was proposed. The proposed method equates the force provided by FRP reinforcement at a strain of 4000 microstrain to the force provided by steel reinforcement at the yield strain. Additional testing and research will be needed to validate and complete the proposed methodology.

5.4 Future Work

The proposed requirements for proportioning FRP transverse reinforcement in hollow cylinder piles have been developed based on matching the strength currently provided by the prescribed quantity of traditional steel transverse reinforcement, and by enforcing an effective transverse strain limit. The strain limit has been adopted from ACI 440.1R-15 for the case of beam shear, and it may be overly-conservative for the application of longitudinal shear. As a result, the proposed minimum area requirements may be restrictive for the use of FRP reinforcement in such piles. However, these proposed requirements are thought to be safe given the absence of additional experimental test data. Future studies are needed to investigate pile segments reinforced with appropriate levels of FRP reinforcement. Both grids and spirals should be studied, and both glass and carbon should be studied, as these parameters could influence behavior. Future studies should verify the validity of the proposed design requirement for transverse FRP reinforcement, and they should also investigate the possibility of increasing the 4000 με limit imposed on the transverse reinforcement. An increase in this effective strain limit would proportionally reduce the minimum required area of FRP reinforcement (ie: doubling the strain limit would half the required area), up to the point where another governing requirement (such as $1.2T_{cr}$) or the ultimate strength of the FRP were reached. A greater effective strain limit would make FRP transverse reinforcement a more competitive alternative to steel transverse reinforcement.
Future studies should also consider full-scale tests on pile segments reinforced with other types of FRP, such as GFRP and BFRP. From a constructability perspective, alternative arrangements of FRP reinforcement including spiral reinforcement and modified grid spacing should be examined. It is further recommended that an FRP-reinforced pile ultimately be driven and then load tested in-situ to fully vet this material alternative for acceptance by DOTs, pile producers, and pile driving contractors. Ultimately, the steel-reinforced and FRP-reinforced piles should offer equivalent structural performance, and the FRP-reinforced piles should offer superior corrosion resistance. The potential remains to gain efficiency through reduced wall thickness if FRP reinforcement is used, however, appropriate quantities of FRP transverse reinforcement are required.
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