CFRP Renewal of Prestressed Concrete Cylinder Pipe

Web Report #4352

Subject Area: Infrastructure
CFRP Renewal of Prestressed Concrete Cylinder Pipe
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CFRP Renewal of Prestressed Concrete Cylinder Pipe

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Jointly sponsored by:
Water Research Foundation
6666 West Quincy Avenue, Denver, CO 80235

U.S. Environmental Protection Agency
Washington, DC 20460

and

Water Environment Research Foundation
635 Slaters Lane, Suite G-110, Alexandria, VA 22314

Published by:

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FOREWORD

The Water Research Foundation (Foundation) is a nonprofit corporation dedicated to the development and implementation of scientifically sound research designed to help drinking water utilities respond to regulatory requirements and address high-priority concerns. The Foundation’s research agenda is developed through a process of consultation with Foundation subscribers and other drinking water professionals. The Foundation’s Board of Trustees and other professional volunteers help prioritize and select research projects for funding based upon current and future industry needs, applicability, and past work. The Foundation sponsors research projects through the Focus Area, Emerging Opportunities, and Tailored Collaboration programs, as well as various joint research efforts with organizations such as the U.S. Environmental Protection Agency and the U.S. Bureau of Reclamation.

This publication is a result of a research project fully funded or funded in part by Foundation subscribers. The Foundation’s subscription program provides a cost-effective and collaborative method for funding research in the public interest. The research investment that underpins this report will intrinsically increase in value as the findings are applied in communities throughout the world. Foundation research projects are managed closely from their inception to the final report by the staff and a large cadre of volunteers who willingly contribute their time and expertise. The Foundation provides planning, management, and technical oversight and awards contracts to other institutions such as water utilities, universities, and engineering firms to conduct the research.

A broad spectrum of water supply issues is addressed by the Foundation's research agenda, including resources, treatment and operations, distribution and storage, water quality and analysis, toxicology, economics, and management. The ultimate purpose of the coordinated effort is to assist water suppliers to provide a reliable supply of safe and affordable drinking water to consumers. The true benefits of the Foundation’s research are realized when the results are implemented at the utility level. The Foundation's staff and Board of Trustees are pleased to offer this publication as a contribution toward that end.

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ACKNOWLEDGMENTS

The authors of this report would like to thank the Water Research Foundation research manager Dr. Jian Zhang and the Project Advisory Committee members Mr. Michael P. Gibsov of Washington Suburban Sanitary Commission, Mr. Jason DeLaet of Greater Cincinnati Water Works, Ms. Ari Selvakumar of U.S. Environmental Protection Agency, and Stephanie Hooker of National Institute of Standards and Technology.

In addition, the authors would like to thank the following participating organizations: American Concrete Pressure Pipe Association (Reston, VA), Fibrwrap Construction LLC (Ontario, CA), Metropolitan Water District of Southern California (Los Angeles, CA), Howard County Department of Public Works (Columbia, MD), Tarrant Regional Water District (Fort Worth, TX), and Structural Group Inc. (Hanover, MD).

Special thanks go to Mr. Sam Arnaout of Hanson Pressure Pipe for providing the full-scale test specimens, facilities, and skilled personnel on behalf of the American Concrete Pressure Pipe Association, and the Quality Assurance Manager Mr. Jack Williams of the Hanson plant in Palatka, FL and his team, who made the experimental phase of this project possible.

Special thanks also go to Fyfe Company for providing all materials for the CFRP repair system for all test specimens and Fibrwrap Construction for installing the CFRP systems.

Finally, the authors would like to thank the following Simpson Gumpertz & Heger Inc. staff for their help in this project: Mr. Albert Saul and Ms. Joan Cunningham for assistance with the literature search, Dr. Timothy McGrath for providing insight on the issue of constrained soil modulus, Dr. Omer Erbay and Dr. Andrew Sarawit for their assistance in Finite Element Analysis, Mr. Daniel Clark for his assistance during the experimental phase, and Ms. Elizabeth Carroll for formatting and preparing this report.
EXECUTIVE SUMMARY

OBJECTIVES

The purpose of the study reported herein is to develop the requirements on materials, design, and installation of carbon fiber reinforced polymer (CFRP) renewal and strengthening of distressed prestressed concrete cylinder pipe (PCCP); so that the pipe, when subjected to the exposure environment, design working and transient pressures, gravity loads, and live loads, shall have the necessary strength, reliability, and durability to resist the loads throughout the design life as it continues to degrade in the exposure environment.

BACKGROUND

CFRP liners adhered to the inner core can provide an effective means of internal repair and strengthening of PCCP. Water utilities have used this technology since the 1980s, and increasingly in recent years, when other methods such as post-tensioning or pipe replacement were not be feasible or economical. However, a standard that can be used by the professional engineers and utilities for the design of CFRP liners to ensure adequate strength, durability, and reliability of the repaired pipe throughout its service life has been lacking. To develop a design standard, the Concrete Pressure Pipe Committee of the American Water Works Association (AWWA) has formed a subcommittee, chaired by Dr. Mehdi Zarghamee, to draft a standard for CFRP Renewal and Strengthening of PCCP.

Although the AWWA draft Standard produces a consensus document for the design of CFRP liners, the value of the standard is greatly increased if it is based on fundamental analytical data and experimental data. This study is aimed at producing such data. The data collected from executing this standard is expected to form the basis of the provisions of the AWWA draft Standard for CFRP Renewal and Strengthening of PCCP that is under development.

The challenge in the design of the CFRP repair is to ensure that the CFRP liner will continue to perform as the host pipe continues to degrade. CFRP repair may be made at a time when there are a limited number of broken wires and a limited reduction in prestress. The exterior of the CFRP-lined PCCP remains exposed to the corrosive elements of the environment. CFRP-lined PCCP will continue to corrode externally and will experience more wire breaks and a reduction or loss of prestress, cracking of outer concrete core, and the resulting exposure of the steel cylinder to the corrosive elements of the environment, causing it to corrode and perforate.

As the outer concrete core of the CFRP-lined PCCP cracks with time, the pipe stiffness is reduced and, similar to a flexible pipe, it will rely on the stiffness of the surrounding soil to resist external load. Depending on the bond between the CFRP liner and the inner concrete core of the pipe, the following final states of the PCCP with CFRP liner could exist:

- CFRP liner, delaminates from the concrete inner core, resists the internal pressures and external loads as a standalone flexible pipe.
- CFRP liner remains bonded to the concrete inner core, and the composite pipe made of CFRP liner and concrete inner core resist the internal pressures and external loads as a semi rigid or flexible pipe.
CFRP liner is ineffective and the PCCP is subjected to internal pressures and external loads, causing failure of PCCP with time as if it was not repaired. This condition occurs when CFRP laminate or its termination fails.

For CFRP-lined PCCP to remain effective and to continue to deliver water throughout its service life while it is subjected to design loads (e.g., working and transient pressures, gravity load, live load, external groundwater pressure, internal negative pressure), it must have the necessary strength (ability to support the design loads as the host PCCP continues to degrade), durability (have adequate retained strength after exposure to the chemical and thermal environments and sustained loads), and reliability (possess adequate safety margin to accommodate uncertainties in loads and resistance through appropriate load and resistance factors). It is the focus of this project to provide the technical data to form the basis of provisions that are needed to achieve these objectives.

APPROACH

The method of approach included the following:

- Review literature, collect and analyze data on degradation of CFRP exposed to different chemical and thermal environments and sustained loads, and determine the material adjustment and time effect factors that should be used in design of CFRP liners for design lives of five and fifty years. Review literature, buckling resistance of CRFP liners in degrading PCCP, and published data on past full-scale tests of CFRP liners applied to degrade PCCP.
- Perform two full-scale hydrostatic pressure tests and eight three-edge bearing tests to study the behavior of CFRP-lined distressed PCCP with regions of broken wires and cracked outer core subjected to internal hydrostatic pressure and vertical bending loads. In particular, determine the CFRP contribution that can be mobilized to resist bending loads without debonding and to resist internal pressure without allowing the water to get behind the liner.
- Perform detailed nonlinear finite element analysis (FEA) of ten representative CFRP-lined buried PCCP, simulating the degradation process of PCCP after CFRP installation; and verify critical design parameters calculated using the provisions of the AWWA draft Standard including soil load on pipe, pipe deflections, CFRP strains, shape factor, and rerounding.
- Determine the buckling equation that should be used in design of the CFRP liner based on comparison of buckling resistances obtained from FEA of degrading CFRP-lined PCCP and from closed-form solutions with or without effect of soil stiffness.
- Perform reliability analysis and determine load and resistance factors for all design limit states of CFRP-lined PCCP that ensure target reliability indices that are consistent with those used for other civil infrastructure.
- Present all produced data in a format that allows incorporation into the AWWA Draft Standard for CFRP Renewal and Strengthening of PCCP and identification of areas where further research is needed.
RESULTS/CONCLUSIONS

The results of this study are as follows:

- Based on analysis of published durability test results for CFRP exposed to different environments, values of material adjustment factor are recommended for design as a function of service life of the CFRP liner.

- Based on analysis of published creep and creep rupture tests results for CFRP, values of time effect factor are recommended for design strength and modulus as a function of service life of the CFRP liner.

- Characteristic value of material properties are defined from the results of laboratory tests as the 5 percentile value with 80% confidence (to account for sample size). The characteristic value will account for variability of material strength and modulus from different manufacturers and installers, and will allow the resistance factor to be independent of manufacturer and installation workmanship.

- Based on reliability analysis of parameters that govern the design and the objective of having CFRP-lined PCCP to have the same reliability as other materials used in infrastructure, loads and resistance factors were determined. Load factors account for the variability of the loads, and resistance factors account for the variability of material resistance, independent of the manufacturing and installation workmanship.

- A set of simplified design equations are proposed for design of CFRP liner for internal pressures, gravity loads, combined gravity loads and internal pressures, and buckling from external loads and pressures. The simplified approach includes a new equation for rerounding of the flexible liner subjected to internal pressure.

- A nonlinear finite-element model of the concrete pipe wall and the surrounding soil was developed. The results obtained from the finite-element model of the test pipe for a three-edge bearing test were compared to the observations during the test and determine the validity and limitations of the finite-element model. Then the finite-element model of pipe/soil system was subjected to the design loads and the results were compared with the simplified design equations. The comparison showed that the simplified design equations are adequate and conservative.

- The nonlinear finite-element model was extended to determine the buckling strength of the CFRP liner. The results of analysis were used to determine a bias factor that account for soil flexibility to be applied to the Block equation for buckling of a flexible liner inside rigid pipe.

- Three-edge bearing and hydrostatic pressure tests were conducted to determine all likely failure modes. Two failure limit states that were not initially included in the simplified design approach were determined: one involved longitudinal bending of the CFRP liner resulting from expansion of CFRP liner in areas where prestressing is lost due to wire breakage and the outer core is cracked; and the second one involved leakage resulting from poor bond between CFRP liner and steel joint rings and steel cylinder. These failure modes were included in the design process.
APPLICATIONS/RECOMMENDATIONS

The results of this research provide theoretical and experimental foundation for the development of an AWWA Standard on Renewal and Strengthening of PCCP, and are shared with the Standard Committee for this purpose. The Standard, once published, will assist professional engineers and utilities in design, material selection, and installation of CFRP liners in degraded PCCP lines. The results of this research will help ensure that the CFRP renewed PCCP, when properly designed using qualified materials and installed properly will have the necessary strength, durability, and reliability against all failure modes. Some of these failure modes were discovered during this research. As an example, hydrostatic pressure tests performed as a part of this research program to validate the designs from a strength point of view revealed new failure modes related to water tightness that had not been known or considered in the draft Standard. For the new water tightness failure modes, new provisions -- on design, material qualifications, and installation of the CFRP liner -- were included in the draft Standard, which will be presented to the Standard Committee for approval. In absence of such provisions in the Standard, water tightness would not be ensured and the pressure in the CFRP renewed PCCP could not reach the ultimate capacity of the pipe.
INTRODUCTION

BACKGROUND DATA

Since the 1940s, water and sewer transmission lines have been constructed using concrete pressure pipe, and in particular, PCCP. The American Concrete Pressure Pipe Association (ACPPA) reports that 90 of the 100 largest North American metropolitan areas use concrete pressure pipe in their water and/or wastewater systems (ACPPA, 2011).

There are two types of PCCP: embedded-cylinder type with a concrete core cast vertically with an embedded steel cylinder (Figure 1.1), and lined cylinder type with a concrete core centrifugally cast against the steel cylinder. The cores in both types are then prestressed with high-strength steel wires spirally wrapped around the core, and the prestressed cores are coated with mortar for corrosion protection. In corrosive environments, corrosion and hydrogen embrittlement of prestressing wires may occur, and the resulting wire breaks and loss of pressure may cause failure of the pipe.

CFRP liners adhered to the inner core can provide an effective means of internal repair and strengthening of PCCP. Water utilities have used this technology since the 1980s and increasingly in the recent years when other methods such as post-tensioning or pipe replacement were not be feasible or economical. However, a standard that can be used by professional engineers and utilities for the design of CFRP liners to ensure adequate strength, durability, and reliability of the repaired pipe throughout its service life has been lacking. To develop a design standard, the Concrete Pressure Pipe Committee of the American Water Works Association (AWWA) has formed a subcommittee, named CFRP Renewal and Strengthening of PCCP. This subcommittee has drafted a new standard.
Although the AWWA draft Standard produces a consensus document for the design of CFRP liners, absence of fundamental analytical and experimental data remains as a major shortcoming of the document. This project is aimed at eliminating this shortcoming. The data collected from executing this project is expected to form the basis of the provisions of the AWWA draft Standard for CFRP Renewal and Strengthening of PCCP that is under development.

The challenge in the design of the CFRP repair is to ensure that the CFRP liner will continue to perform as the host pipe continues to degrade. CFRP repair may be made at a time where there are a limited number of broken wires and a limited reduction in prestress. The exterior of the CFRP-lined PCCP remains exposed to the corrosive elements of the environments. CFRP-lined PCCP will continue to corrode externally and will experience more wire breaks and a reduction or loss of prestress, cracking of outer concrete core, and delamination between the inner core and steel cylinder. The cracking of the outer core will expose the steel cylinder to the corrosive elements of the environment, causing it to corrode and perforate.

As the outer concrete core of the CFRP-lined PCCP cracks with time, the pipe reduces to a flexible pipe that relies on the stiffness of the surrounding soil to resist external load. Depending on the bond between the CFRP liner and the inner concrete core of the pipe, the following final states of the PCCP with CFRP liner could exist:

1. CFRP liner, delaminated from the concrete inner core, resists the internal pressures and external loads as a standalone flexible pipe.
2. CFRP liner remains bonded to the concrete inner core, and the composite pipe made of CFRP liner and concrete inner core resist the internal pressures and external loads as a semi rigid or flexible pipe.
3. CFRP liner is ineffective and the PCCP is subjected to internal pressures and external loads, causing failure of PCCP with time as if it was not repaired. This condition occurs when failure of CFRP laminate or its end attachments occur.

For CFRP-lined PCCP to remain effective and to continue to deliver water throughout its service life while it is subjected to design loads (e.g., working and transient pressures, gravity loads, live loads, external groundwater pressure, internal negative pressure), it must have the necessary strength (ability to support the design loads as the host PCCP continues to degrade), durability (ability to retain strength after exposure to the chemical and thermal environments and sustained loads), and reliability (ability to accommodate uncertainties in loads and resistance through appropriate load and resistance factors). It is the focus of this project to provide the technical data to form the basis of provisions that are needed to achieve these design objectives.

PURPOSE OF THE PROJECT

The purpose of this project is to develop the requirements on materials, design, and installation of CFRP liners used for renewal or strengthening of distressed PCCP so that the repaired pipe, when subjected to the exposure environment, design working and transient pressures, gravity loads, and live loads, shall have the necessary strength, reliability, and durability to resist the loads throughout the design life as it continues to degrade in the exposure environment.
SCOPE

The scope of work for this project consisted of the following:

1. Literature review and collection of data on degradation of CFRP exposed to different chemical and thermal environments and sustained loads; buckling strength of CFRP-lined PCCP; and tests on CFRP-lined PCCP conducted by others.
2. Experimental investigation in which CFRP-lined PCCP of different type and diameter with simulated distress were subjected to internal pressure and external three-edge bearing load to study modes, sequences, and loads at failure.
3. Analytical investigation through finite element analysis (FEA) of ten different representative models of CFRP-lined buried PCCP subjected to the combined effects of gravity loads and pressures to study the complex interaction of the CFRP liner, host pipe, and surrounding soil as the host pipe degrades during its service life; and determination of buckling strength of CFRP liner in buried PCCP using these models.
4. Determination of the constrained modulus of soil surrounding the pipe that should be used in the design of CFRP liners.
5. Reliability analysis of all design limit states accounting for uncertainties in both loads (i.e., demands) and resistance (i.e., capacities in all possible failure modes) and determination of load and resistance factors for use in design of CFRP liners.
6. Presentation of all results in a format ready to be incorporated into the AWWA Draft Standard for CFRP Renewal and Strengthening of PCCP.

METHOD OF APPROACH OF INVESTIGATION

The method of approach included the following:

- Review literature, collect and analyze data on degradation of CFRP exposed to different chemical and thermal environments and sustained loads, and determine the material adjustment and time effect factors that should be used in design of CFRP liners for design service lives fifty years, or less as selected by the owner.
- Perform full-scale hydrostatic pressure tests on two CFRP-lined PCCP including one embedded Cylinder pipe (ECP) and one lined cylinder pipe (LCP); and full-scale three-edge bearing tests on eight specimens including four CFRP-lined LCP, three CFRP-lined ECP, and one control LCP specimen; and study behavior of CFRP-lined distressed PCCP with regions of broken wires and cracked outer core subjected to internal hydrostatic pressure and vertical bending loads. In particular,
- Determine the CFRP contribution that can be mobilized to resist internal pressure and bending loads.
- Determine the conditions that result in debonding of CFRP liner from the concrete inner core of a distressed PCCP.
- Determine the effectiveness of CFRP termination details that are developed to maintain the watertightness of repaired PCCP.
• Identify further tests that are needed to verify the provisions of the AWWA Draft Standard.

• Develop detailed nonlinear FE models of ten representative CFRP-lined buried PCCP with varying design parameters (i.e., diameter, design loads, number of CFRP layers, soil modulus); simulate degradation process of PCCP after CFRP installation; compare results with those obtained from full-scale three-edge bearing tests; interpret FEA results to determine parameters including soil load on pipe, pipe deflections, and CFRP strains for the combined gravity loads and internal pressure; and compare results with those obtained from the design requirements of AWWA Draft Standard.

• Determine the buckling resistance of CFRP liner in buried PCCP by extending some of the FE models developed for validation of design requirements of AWWA draft Standard cases to include an additional load step representing nonlinear buckling instability resulting from internal negative pressure; compare the buckling resistances obtained from FEA and closed-form solutions available in the literature with and without effect of soil stiffness; select the buckling equation that should be used in design of CFRP liner.

• Perform reliability analysis of all design limit states using Monte Carlo Simulation by accounting for uncertainties in material properties, geometric properties, and loads, and bias in design equations; and determine load and resistance factors that ensure target reliability indices that are consistent with those used for other civil infrastructure.

• Combine and present all produced data in a format that allows incorporation into the AWWA draft Standard for CFRP Renewal and Strengthening of PCCP and identification of areas where further research is needed.
CHAPTER 2
LITERATURE REVIEW

INTRODUCTION

CFRP liners are applied to prestressed concrete cylinder pipes (PCCP) to strengthen or extend their service life. To achieve the service life of the CFRP repaired or strengthened PCCP, the design must be based on strength (ability to support the loads and pressures as the host PCCP continues to degrade), durability (ability to retain strength after exposure to the chemical and thermal environments), and time effects (creep and creep rupture strength under sustained loads). Furthermore, the CFRP liner design must account for the change in loading as the host PCCP continues to degrade with time. CFRP has been successfully used in strengthening of concrete structures and in aerospace, automotive, marine structures, buried structures, and other corrosion-sensitive applications with different exposure and load environments; nevertheless, the CFRP liner design for PCCP is unique because of the change in the actual loads and pressures acting on the liner during its service life until the PCCP becomes fully degraded. We performed a literature review on strength and durability of CFRP as it relates to renewal and strengthening of buried PCCP, as well as on various tests conducted on PCCP with CFRP liner.

Purpose

The purpose of this chapter is to study published literature related to strength, durability, and time effects of CFRP liner during its service life, and related to tests conducted on distressed PCCP with CFRP liner.

Scope

The scope of this chapter is limited to CFRP with carbon fibers and epoxy resins, and includes the following topics:

- Degradation of CFRP liner exposed to different chemical and thermal environments with the objective of determining material adjustment factor for the design of CFRP liner.
- Time effects on CFRP material properties subjected to sustained loading with the objective of determining time-effect factor for the design of CFRP liner.
- Buckling strength of CFRP liners in degrading PCCP.
- Tests performed on PCCP with CFRP liner.

DEGRADATION OF CFRP EXPOSED TO CHEMICAL AND THERMAL ENVIRONMENTS

Moisture diffusion into organic polymers could lead to changes in thermo-physical, mechanical, and chemical characteristics of CFRP material. The primary effect of the absorption is generally on the resin itself, which causes both reversible and irreversible changes in the polymer structure. Effects of moisture on fiber-dominated strengths/moduli, such as tensile
strength and modulus, in most cases, are of the order of 15% over a period of ten to fifteen years (Mazor et al., 1978; Sciolti et al., 2010). Mechanical properties of composites controlled by the matrix or the matrix-fiber interface, such as shear, flexural, compression, and tension in directions that do not coincide with the fiber direction are usually affected more significantly by moisture absorption (Joshi, 1983; Selzer and Friedrich, 1997). CFRP tensile strength is generally considered as the critical property, but it must be noted that other properties also degrade and affect the performance of the CFRP liner.

**Tensile Strength and Modulus Degradation**

The tensile properties of unidirectional single-ply CFRP composites measured in the fiber direction can be considered fiber dominated, and they are not significantly affected by moisture absorption (Selzer & Friedrich 1997, Sciolti et al. 2010, Frigione et al. 2004). Transverse tensile strength (fiber direction perpendicular to the load direction), however, decreases significantly as a result of softening of the epoxy matrices due to moisture absorption. Selzer and Friedrich (1997) reported that, compared to dry specimen, the transverse tensile strength and modulus reduced 52% and 18%, respectively, for saturated carbon fiber/epoxy specimen.

The wet-layup process, as usually used for CFRP renewal and strengthening of PCCP, intrinsically results in the formation of a relatively high percentage of voids as well as significantly greater levels of nonuniformity than prepreg-based autoclave composites. Abanilla et al. (2006a) indicated that the number of the lamina layers used to form the composite has significant effects on the curing process of composites in field conditions and hence on tensile strength. This observation suggests that as far as the long-term durability is concerned, the number of laminae used in strengthening must be kept at a minimum to reduce the number of interfaces with voids and resin-rich zones (Abanilla et al., 2006a).

Data presented in Figure 2.1 and Figure 2.2 are provided by Fyfe Company LLC (Fyfe), a manufacturer of CFRP composites, for the unidirectional carbon fabric Tyfo SCH-41S. The fabric was impregnated with a two-part epoxy Tyfo S. Each composite panel used in experiment consists of two layers of fabric. Panels were made using the wet-layup process and allowed to cure under ambient conditions, after which they were cut to dimensions required for test coupons. Experiments were conducted in durations of two years. The extrapolated retained tensile strength after fifty years of immersion in water, alkali, and salt solutions at ambient temperature are 88%, 87%, and 84%, respectively, as shown is Figure 2.1. The retained tensile modulus extrapolated to fifty years of immersion is reduced to 92% for alkali exposure and is essentially unchanged for water and salt-solution exposures, as indicated in Figure 2.2.
Figure 2.1 Retained tensile strength vs. exposure time for immersion in different solutions at 73°F

Figure 2.2 Retained tensile modulus vs. exposure time for immersion in different solutions at 73°F
Flexural Strength Degradation

Flexural properties are usually measured through three-point bending test per ASTM D790. Under flexural loading, the resin-rich interlamina areas in FRP laminates that often are zones of void concentration and osmotic pressure build-up are placed under stress. Many tests have reported substantial lower values of flexural modulus than tensile modulus, suggesting that defects at the interface between laminae can significantly reduce the flexural properties of the laminate. This emphasizes the need to select realistic material adjustment factors when wet layup is used in laminate construction to carefully assess the effects of exposure environment on the long-term mechanical properties that are mainly controlled by the matrix or matrix-fiber interface (Abanilla, et al., 2006a).

Abanilla et al. (2006b) studied the effect of exposure to different solutions on flexural strength retention of CFRP laminate and reported that thirty-two weeks of immersion in both deionized water and alkali solution results in a higher level of degradation of flexural strength than that in salt water. Abanilla et al. reported that the reduction in flexural strength after thirty-two weeks of exposure are 30.9%, 25.3%, and 32.6% for the six-layer-thick CFRP laminate specimens immersed in deionized water, salt water, and alkali solution, respectively.

Interlaminar Shear Strength Degradation

Without through-thickness reinforcement, the interlaminar properties of unidirectional composites depend on the resin. In cases of wet layup under ambient conditions, the interlaminar regions can have varying thickness and void content and are more susceptible to moisture and cyclic condition induced degradation. The short-beam-shear test per ASTM D2344 is a test method for the assessment of the effect of different interface anomalies on moisture-associated durability and allows for replication of stress-state at the interlaminar level that may be more representative of actual field condition (Abanilla et al., 2006a).

Immersion in deionized water usually results in a reduction of short-beam-shear strength with increasing temperature, immersion time, and specimen thickness. Experiments by Abanilla et al. (2006b) indicated that the retained short-beam-shear strength decreases rapidly in the first few weeks, and the rate decreases monotonically with time. Abanilla et al. (2006b) also observed that immersion in the three solutions of deionized water, salt, and alkali results in almost indistinguishable behavior with time of exposure and all three exposures cause the same 29% to 30% reduction in the shear strength after 100-week immersion phase, suggesting that the interlaminar shear degradation is primarily driven by moisture uptake.

In-Plane Shear Strength Degradation

Abanilla et al. (2006b) conducted in-plane shear testing on fully immersed two-layer and six-layer CFRP/epoxy specimens at different temperatures. At the end of the 100-week period of immersion, the loss of the in-plane shear strength for the two-layer specimens was 30.8%, 41.3%, and 49.8% when immersed in water at 23°C, 37.8°C, and 60°C, respectively. For the six-layer specimens, the degradation of in-plane shear strength was slightly less, with 17.6%, 31.3%, and 48.4% reduction for immersion in water at 23°C, 37.8°C, and 60°C, respectively. It was observed that, unlike the results of short-beam-shear strength, the in-plane-shear-strength degradation did not appear to reach an asymptotic level in any of the exposure environments;
instead, the reduction of in-plane shear strength appeared to continue either at a constant or even an increasing rate at the end of the test period.

**Pull-Off Strength Degradation**

Data presented in Table 2.1 are provided by Fyfe and the tests are conducted using the previously introduced Tyfo SCH-41S. The pull-off tests were conducted by bonding 50 mm (2 in.) diameter aluminum disks to samples of the composite bonded to concrete of twenty-eight-day compressive strength not less than 27.6 MPa (4500 psi) and a twenty-eight-day permeability (AASHTO T277) higher than 4,000 coulombs. Prior to conducting the pull-off, the specimens (concrete with composites bonded to it) were exposed to different exposure conditions, as tabulated in Table 2.1. Experiments were conducted for a duration of two years. The linear extrapolation of the test data presented in Table 2.1 to fifty years of immersion in water, alkali, and salt solutions at ambient temperature are 78%, 81%, and 61%, respectively.

<table>
<thead>
<tr>
<th>Time (Months)</th>
<th>DI Water at 73°F</th>
<th>Alkali Solution at 73°F</th>
<th>Salt Solution at 73°F</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>6</td>
<td>94</td>
<td>94</td>
<td>100</td>
</tr>
<tr>
<td>12</td>
<td>82.6</td>
<td>89.9</td>
<td>81.6</td>
</tr>
<tr>
<td>18</td>
<td>80.6</td>
<td>84.7</td>
<td>77.5</td>
</tr>
<tr>
<td>24</td>
<td>74.4</td>
<td>80.6</td>
<td>40.3</td>
</tr>
</tbody>
</table>

**CREEP AND CREEP RUPTURE OF CFRP**

**Creep**

Creep is the time-dependent deformation of a material under constant load. Since the desired lifetime of a structure that is renewed by CFRP is often measured in tens of years, it is impractical in most cases to conduct long-term creep testing for the entire design life of the structure. Thus, much research has been conducted and published on accelerated characterization of creep in composite materials. Accelerated creep models use the results of tests performed on shorter creep tests at elevated temperatures to predict the long-term behavior of the material in question. Examples of these models include the Findley model and the time-temperature superposition (TTSP).

**Findley’s Power Law**

The Power Law model developed by Findley (1944) has been recommended by ASCE Structural Plastics Design Manual (1984) for use in the long-term analysis and design of FRP composite. The model takes the form

\[ \varepsilon(t) = \varepsilon_0 + m(t/t_0)^n \]  

(2-1)
where $\varepsilon(t)$ = total time-dependent creep strain
$\varepsilon_0$ = stress- and temperature-dependent initial elastic strain
$m$ = stress- and temperature-dependent coefficient
$n$ = material constant independent of stress and temperature
$t$ = load duration
$t_0$ = one unit of time (e.g., 1 hr).

Yen and Morris (1989) introduced the effect of temperature into Findley’s power law and defined the creep strain as follow:

$$
\varepsilon(\sigma, T, t) = \sigma \left[ D_0(\sigma, T) + D_T(\sigma, T) t^n \right]
$$

(2-2)

where
$T$ = temperature
$\sigma$ = stress
$D_0$ = instantaneous or initial creep compliance ($\varepsilon_0/\sigma$)
$D_T$ = transient creep compliance stress per unit stress

Schapery (1969) developed a single integral equation to characterize the nonlinear viscoelastic behavior of solids subjected to varying stress. For linear viscoelastic material under a fixed environmental condition with a constant stress applied, the strain may be expressed as

$$
\varepsilon(t) = D_0 \sigma + D_t \sigma
$$

(2-3)

If the stress is not constant, the strain response is

$$
\varepsilon(t) = D_0 \sigma + \int_0^t D_t (t - \tau) \frac{d\sigma}{d\tau} d\tau
$$

(2-4)

**Time-Temperature Superposition Principle**

The time-temperature superposition principle (TTSP) is widely used in creep testing of composites to determine the effect of temperature on the creep of FRPs based on Arrhenius Equation to relate the time and elevated temperature, as follows:

$$
\frac{dP}{dt} = A \exp\left( \frac{E_a}{RT} \right)
$$

(2-5)

where
$\frac{dP}{dt}$ = variable representing the rate of bond scission (which may be a measure of degradation, or creep in this case)
$E_a$ = activation energy
$R$ = universal gas constant
$T$ = exposure temperature ($^\circ K$).
The above formula shows that the logarithm of cumulative bond scission, $\Delta P$, in a time interval $\Delta t$ is a linear function of $T$ and $\log(\Delta t)$, suggesting that temperature $T$ and $\log(\Delta t)$ can be interchangeable.

By the principle of TTSP, the effect of elevated temperature on creep is equivalent to stretching the real time of the creep response by a certain shift factor. Through this method, shorter-term creep tests at different temperatures can be combined to generate a master creep curve. The length of time of the master creep curve is significantly longer than the shorter-term curves.

In this method, short-term creep isotherm curves (creep strain versus logarithm of time $t$) are first plotted on a log scale. The time scale of an isotherm curve corresponding to a temperature other than the reference temperature is then changed by introducing a shift factor $a_T$ from the isotherm for temperature $T$, $t_r = a_T \cdot t$, or

$$\log(a_T) = \log(t_r) - \log(t)$$

(2-6)

where $t = \text{the actual test time in temperature } T$

$t_r = \text{the equivalent time in reference temperature } T_r \text{ for the same creep strain}$

Superposition of isotherms results in a master creep curve at a reference temperature $T_r$.

**Creep Rupture**

Creep rupture is degradation in strength under sustained loading. Creep-rupture models proposed in the past can be broadly classified as empirical models, statistical models, and theoretical models. The relatively-more-accurate theoretical models are based on either a molecular approach or a continuum approach (fracture mechanics based theories).

In the fracture-mechanics-based creep-rupture theories, an initial flaw is idealized as a central crack condition, and the far field loading activates the crack to grow in a quasi-static manner. The crack rate of growth is taken to have a power-law dependence upon the stress intensity factor. The crack continues to grow until such time as it reaches the size which in relation to the far field loading causes the crack to become unstable and thereby to propagate in an uncontrolled manner. The corresponding time at which this occurs is taken to be the lifetime of the material. The derivation of creep-rupture model presented below follows this kinetic form of fracture mechanics. The key condition in the kinetic aspect of the problem is the assumption of the power-law form for the crack growth rate.

Christensen and Miyano (2006) assumed that the rate of crack growth is controlled by the power law form

$$\dot{a} = \lambda (\sigma \sqrt{a})^r$$

(2-7)

where $\dot{a} = \text{the rate of crack growth}$

$\lambda = \text{material parameter}$

$r = \text{power law exponent}$

$\sigma = \text{stress}$

$a = \text{crack length}$
Equation (2-7) is integrated to obtain

\[
\left( \frac{a}{a_0} \right)^{r+1} - 1 = \lambda \left( 1 - \frac{r}{2} \right) a_0^{r-1} \int_0^t \sigma' (\tau) d\tau
\]

(2-8)

where \( a_0 \) is the initial crack size.

Then, the stress intensity factor at the fracture time \( t_f \) is taken as

\[
\sqrt{a(t)} \sigma(t) \bigg|_{t=t_f} = \sqrt{a_0} \sigma_i
\]

(2-9)

where \( \sigma_i \) is the instantaneous static strength.

Combining Equations (2-7) and (2-8) gives the strength under sustained loading at failure in the form of

\[
1 - \tilde{\sigma}^{-r-2} (\tilde{t}_f) = \int_0^{\tilde{t}_f} \tilde{\sigma}' (\tau) d\tau
\]

(2-10)

where the strength and time are normalized as follows:

\[
\begin{align*}
\tilde{\sigma} &= \frac{\sigma}{\sigma_i} \\
\tilde{t}_f &= \frac{t}{t_i}
\end{align*}
\]

(2-11)

with \( t_i \) = time constant involving \( \lambda, r, \sigma_i, \) and \( a_0 \).

For the case of creep rupture under constant stress:

\[
\tilde{t}_f = \frac{1 - \tilde{\sigma}^{-r-2}}{\tilde{\sigma}'} = \frac{1}{\tilde{\sigma}'} - \frac{1}{\tilde{\sigma}^3}
\]

(2-12)

Experimental data were reported in Miyano et al. (2005) for a CFRP laminate made of T300 woven carbon fiber and vinylester resin with a fiber volume ratio of approximately 52\%, in a beam bending configuration. The author constructed a master creep-rupture strength curve versus logarithm of time based on the three-point bending creep tests performed at different temperatures as shown in Figure 2.3.
Although the analytical derivation of the creep-rupture model is credible, the experimental data used to validate the model uses the time-temperature superposition principle, the validity of which is not established for creep rupture. Furthermore, the elevated temperatures of 80°C (176°F) and 100°C (212°F) used for the second and third set of data are far more than the operational temperatures for epoxy resin used for CFRP lining of PCCP.

Audenaert et al. (2001) recommended an approximation in form linear relationship between the applied stress normalized by the short-term tensile strength and the logarithmic of time as shown below:

$$\log \tilde{\sigma}_f = \alpha \log \tilde{t}_f$$  \hspace{1cm} (2-13)

where \( \tilde{\sigma}_f = \frac{\sigma}{\sigma_i} \) and \( \tilde{t}_f = \frac{t_f}{t_i} \). \( \tilde{\sigma}_f \) and \( \tilde{t}_f \) = stress and time at failure, and \( \sigma_i \) and \( t_i \) are the short-term strength and time to failure.

This relationship can be used for extrapolation to determine the admissible stress for the prescribed target service life. The authors recommended that the ratio of the applied stress to the short term tensile strength of CFRP for fifty years of service life is approximately 80%.

Comprehensive experiment results carried out on 6 mm thick CFRP-epoxy reinforcing bars are reported in Yamaguchi et al. (1998). These results indicate a linear relationship between
creep rupture strength and the logarithmic of time. By extrapolating the test data to fifty-seven years, the ratio of stress level at creep rupture to the short-term strength of CFRP reinforcing bars is about 0.93. For commercially twisted CFRP bars with epoxy resin tested at room temperature in Ando et al. (1997), the estimated retained strength as a percent of short term tensile strength after fifty years was found to be 0.79, which is consistent with Audenaert et al. (2001).

BUCKLING OF CFRP LINER

Introduction

The majority of the CFRP liners for PCCP are designed as a standalone system to avoid reliance on the bond strength of CFRP and reserve capacity of degraded PCCP. A standalone liner is very flexible; therefore, buckling strength of the liner under external loads and pressures or internal negative pressure resulting from transient pressure conditions often controls the design. Design of flexible liners for buckling, however, is not straightforward as evidenced by research since the 1960s.

Two types of buckling have been studied:

- Unconstrained buckling: The liner is supported on the outside by a non-rigid medium (i.e., soil) that allows deformation of the liner prior to buckling depending on the relative stiffness of the liner and surrounding soil (Figure 2.4a). Multiple waves form in the buckling process, and the number of such waves depends on the soil stiffness.
- Constrained buckling: The liner is supported by a rigid cavity that prevents outward deformation of the liner. Buckling is preceded by a local separation of the liner from the cavity wall, followed by snap-through instability of the CFRP liner at one location (Figure 2.4b).

A CFRP liner installed in a degraded PCCP is expected to be partially constrained against outward deformation. The effect of soil stiffness and degraded stiffness of PCCP on constrained buckling pressure of the CFRP liner has not been addressed in the literature to date.
In this section we review the main approaches for buckling of flexible liners in different modes (e.g., constrained, unconstrained) that have been studied in the literature, understand the assumptions, limitations, and uncertainties of each approach.

**Buckling of Unconstrained Pipe**

Bresse (1866) expressed the buckling strength of a thin freestanding (no outside constraint) circular ring under external hydrostatic pressure using small deflection theory:

\[
P_{cr} = \frac{3EI}{R^3}
\]

where \( P_{cr} \) = critical buckling pressure  
\( E \) = modulus of elasticity  
\( I \) = moment of inertia of the ring’s cross section  
\( R \) = mean radius of the ring

Bryan (1888) used minimum potential energy criterion to derive a similar expression for an infinitely long freestanding pipe under external hydrostatic pressure:

\[
P_{cr} = \frac{3EI}{(1-v^2)R^3} = \frac{2E}{1-v^2} \left( \frac{t}{D} \right)^3
\]

where \( D \) = mean diameter of pipe  
\( t \) = mean pipe wall thickness  
\( v \) = Poisson’s ratio.
This equation is alternatively expressed as

\[
P_{cr} = \frac{2E}{1 - \nu^2} \frac{1}{(SDR - 1)^3}
\]  
(2-16)

where the standard dimension ratio SDR is equal to outside diameter of pipe divided by mean pipe wall thickness (i.e., SDR=D/t+1).

Buckling of Buried Pipe

Luscher (1966) considered the equation for elastic buckling strength of a cylinder that is elastically supported by a surrounding thick cylinder of Winkler soil (Figure 2.5a):

\[
p^* = 2 \sqrt{\frac{K_s EI}{R^3}}
\]

\[
K_s = BE_s = 0.75BM
\]  
(2-17)

where \(p^*\) = elastic circular buckling in the circular-symmetric case (Figure 2.5b)
- \(Ei\) = bending stiffness of cylinder wall
- \(R\) = cylinder radius
- \(K_s\) = coefficient of elastic soil reaction
- \(E_s\) = Young’s modulus of soil
- \(M\) = constrained modulus of soil
- \(B\) = dimensionless coefficient dependent on geometry and Poisson’s ratio of soil cylinder plotted in Figure 2.6.

For soil with infinite thickness around the elastic cylinder (Figure 2.5c), \(K_s = 0.77E_s\) for \(\nu_s = 0.3\) (typical of granular soils), and \(K_s = 0.67E_s\) for \(\nu_s = 0.5\) (typical of clayey soils).
Figure 2.5 Flexible tube with (a) Winkler soil (b) thick soil cylinder, and (c) infinitely thick soil. Source: Moore, I.D, E.T Selig, and A. Haggag. 1988. Elastic Buckling Strength of Buried Flexible Culverts. Transportation Research Record 1191:57-64. Reproduced with permission from the Transportation Research Board.

Figure 2.6 Dimensionless buckling coefficient $B$ for case shown in Figure 2.5b (Source: Luscher 1966, with permission from ASCE)
Equation (2-17) is based on Winkler’s linear multi-wave theory where the soil is modeled as a series of independent springs resisting the radial deformation of the pipe. The critical number of buckling waves \( n_{cr} \) is given by

\[
n_{cr} = \sqrt[4]{\frac{K_s R^3}{EI}}
\]  

(2-18)

Note that \( n = 2 \) corresponds to an elliptical buckling shape. In Luscher’s example of a steel pipe with \( D/t = 280 \) that corresponds to \( EI/R^3 = 1.0 \), Equation (2-18) results in \( n = 3.1 \) for \( K_s = 100 \) psi, \( n = 5.6 \) for \( K_s = 1000 \) psi, and \( n = 7.4 \) for \( K_s = 3000 \) psi.

Luscher modified this equation with a factor \( \beta \) to account for arching action and pressure redistribution in the case of a buried flexible cylinder (Figure 2.5c):

\[
p_o^* = 2\beta \sqrt{\frac{K_s EI}{R^3}}
\]

(2-19)

where \( p_o^* \) = surface pressure causing elastic buckling

\( \beta \) = arching factor defined as 1.5 for dense soil and 1.0 for loose soil based on experimental data from Luscher and Hoeg (1964).

In calculating the coefficient \( B \) in this case, Luscher conservatively used the cover over the crown of the cylinder as the soil cylinder thickness.

To account for the reduction in buckling strength due to the deformation of buried cylinder, Luscher developed the following equation:

\[
p_e^* = p_o^* \left( 1 - \frac{3.5}{\beta} \frac{p_e^*}{E^*} \right)
\]

\[
E^* = 0.45M
\]

(2-20)

where \( p_e^* \) = effective buckling strength of the deformed cylinder

\( E^* \) = modified modulus of passive soil resistance

Luscher concluded that the reduction in buckling strength is insignificant for dense soils (\( E^* = 3000 \) psi) and may be significant for loose soils (e.g., 22% reduction at 5% deflection in loose soil), but recommended that this reduction not be considered because of the uncertainty in selection of the modulus of soil rigidity \( K_s \) and in the amount of arching assumed.

Luscher also established a reasonable range of possible values of \( K_s \) and proposed the following buckling equation:

\[
p_o^* = A\beta \left( \frac{EIB}{R^3} \right)^{5/6}
\]

(2-21)
where $A = 780$ for “reasonable upper limit” and 78 for “reasonable lower limit”. Typical design value is $A = 195$.

AWWA C950 Standards Committee for FRP Pipe – Task Group on Buckling in 1980 and 1982 recommended using the Luscher’s formula format, Equation (2-17) above, but modifying the coefficient $B$ empirically based on certain test results on 12 in., 18 in., and 30 in. diameter pipes and adding a water buoyancy factor $R_w$ (Glascock 1982). The 1980 recommendation was:

$$P_{ML} = \sqrt{32 R_w B' E'(EI/D^3)}$$

$$R_w = 1 - 0.33(h_w/h)$$

$$B' = \begin{cases} 
0.015 + 0.041(h/D) & 0 \leq h/D \leq 5 \\
0.150 + 0.014(h/D) & 5 \leq h/D \leq 80
\end{cases}$$

Equation (2-22)

where $R_w$ = water buoyancy factor for typical soil
$h_w$ = height of water above top of pipe
$h$ = height of soil above top of pipe
$B'$=empirically determined coefficient of elastic support, and $E'$ = Spangler’s modulus of soil reaction

In 1982, the Task Group evaluated all buckling test data on pipes and tanks with radii ranging from 12 in. to 144 in. and soil cover heights ranging from 0 ft to 120 ft and reevaluated the applicability of Equation (2-22) above (Glascock 1982, Glascock and Cagle 1984). The results indicated that at cover heights below 4 ft, the buckling strength became less dependent on soil modulus $E'$ and more dependent on $EI/D^3$, resulting in two-lobe buckling inward at the sides and upward movement of the top of the pipe and heaving off the shallow soil cover. Therefore, for pipes with soil covers less than 4 ft and subjected to vacuum, the Task Group recommended the use of alternative design methods that take into account the minimum buckling strength for above-ground design (i.e., Equation (2-14)) and “some amount of additional resistance” due to cohesion and weight of overlying soil. For soil covers of 4 ft or more for pipes having internal vacuum (or for any soil cover without internal vacuum), the Task Group recommended the use of the modified Luscher equation (Equation (2-22) above) with the following modified $B'$ coefficient that reflects dependence of $B'$ on the soil cover height ($H$) alone rather than the soil cover-to-diameter ratio based on linear regression analysis of $B'$ values that were back-calculated from a selected set of buckling tests that resulted in “normal” buckling (one-lobe buckling usually downward at the top of pipe):

$$B' = \frac{1}{1 + 4e^{-0.065H}}$$

Equation (2-23)

where $H$ = soil cover height in ft.

Note that this approach was later abandoned, and the buckling strength equation in the current AWWA M45 *Fiberglass Pipe Design Manual* (2005) is:
where $q_a$ = allowable buckling pressure  
$EI$ = bending stiffness of the pipe wall  
$M_s$ = constrained soil modulus  
$FS = 2.5$ = design factor  
$C_n = 0.55$ = scalar calibration factor to account for some nonlinear effects  
$\varphi_s$ = factor to account for variability in stiffness of compacted soil (suggested value is 0.9)  
$k_v = (1 + \nu)(1 - 2\nu)/(1 - \nu)$ = modulus correction factor for Poisson’s ratio of the soil  
$R_h = 11.4/(11 + D/h)$ = correction factor for depth of fill  
$h$ = height of ground surface above top of pipe.

Equation (2-24) that is used in the current AWWA M45 is a modified version of the equation proposed by Moore et al. (1988), Moore (1989), and Moore and Selig (1990).

Moore et al. (1988) argued that linear continuum theory predicts the buckling strength of buried flexible culverts better than the approaches that are based on Winkler’s model (Figure 2.5a) such as those proposed by Luscher (1966) or AWWA C950 Task Group reviewed above. Moore et al.’s preliminary comparison of the continuum theory prediction to experimental data from Allgood and Ciani (1968), Howard (1972), Gumbel (1983) and Crabb and Carder (1985) was as shown in Figure 2.7.

![Figure 2.7 Buckling predictions by continuum theory versus Winkler theory compared to experimental data](image)

Moore et al. (1988) then calibrated the continuum theory prediction so that it represents the lower bound to the experimental data rather than upper bound, as follows:

$$N_c = \phi N_{ch} R_n R_s$$  \hspace{1cm} (2-25)
where \( \phi \) = calibration factor to account for experimental variation and soil nonlinearity, 0.55 for granular soil, “probably less” for clay

\[ N_{ch} = \text{buckling strength of a uniformly stressed, deeply buried circular culvert in homogeneous soil} \]

\( R_h = \) correction factors for shallow burial and the geometry of the backfill zone (Figure 2.8)

\( R_s = \) correction factor for the culvert shape.

For a smooth culvert/soil interface,

\[ N_{ch} = \frac{(n^2 - 1)EI}{R^2} + \frac{E_s^*R}{2n + (1 - 2\nu_s)/(1 - \nu_s)^3} \]

(2-26)

which, when minimized with respect to the harmonic number \( n \), results in a critical buckling mode of:

\[ n_{cr} = \left( \frac{E_s}{4(1 - \nu_s^2)EI} \right)^{1/3} R^3 \]

\[ N_{ch} \approx 1.2 \left( \frac{E_s}{1 - \nu_s^2} \right)^{1/3} \left( E_s^* \right)^{2/3} \]

\[ E_s^* = \frac{E_s}{1 - \nu_s^2} \]

(2-27)

for typical flexible metal culverts with \( EI/E_s^*R^3 \leq 0.01 \).

Moore et al.'s (1988) major conclusion is that for typical deeply buried pipe, buckling strength is a function of the flexural stiffness of the structure and soil modulus rather than its size. This means that the buckling strength of circular pipes with the same wall thickness buried in the same soil would be identical regardless of their diameter. For shallow burial or poor backfill conditions, however, Moore et al. (1988) concluded that continuum theory indicates dramatic reductions in the buckling strength and may be more conservative than approaches that are based on Winkler theory.
Moore (1989) and Moore and Selig (1990) reviewed various theories for predicting the buckling strength of buried flexible tubes, including multi-wave theories such as Winkler model and elastic continuum model, and single-wave theories such as ring in a rigid cavity and ring in a Winkler medium.

They explained that an elastically supported buried pipe first forms wave-like flexural deformations at locations of maximum hoop thrust (Figure 2.9a). Then, local bending, induced by the buckling deformations, provides eccentricity for the thrust, and nonuniform earth pressures develop as the soil is deformed. When the sum of the interface pressure due to soil weight and the nonuniform earth pressure due to local bending drops to zero and tension develops at the pipe-soil interface at some location, the pipe separates from the soil. The pipe changes from a short wavelength deformation pattern to a long wavelength pattern and snaps into

Note: This chart is for backfill width-to-pipe radius ratio of $w/R = 0.1$. Charts for $w/R = 0.2-1.5$ are available in Selig et al. (1988).

Figure 2.8 Correction factors for shallow burial ($R_{hs}$) and dual zone ($R_{hd}$) proposed by Moore et al. (1988); soil stiffness $E_s^* = E_s/(1-\nu_s^2)$; stiffness of soil surrounding select backfill $E_o^* = E_o/(1-\nu_o^2)$ Source: Moore, L.D, E.T Selig, and A. Haggag. 1988. Elastic Buckling Strength of Buried Flexible Culverts. Transportation Research Record 1191:57-64. Reproduced with permission from the Transportation Research Board.
the cavity (Figure 2.9b). Moore and Selig (1990) state that this behavior is analogous to buckling of a column under compression with multiple supports along its length, which buckles when one of the intermediate supports is removed (Figure 2.9b).

Moore (1989) selected the linear multi-wave buckling solution based on elastic continuum representation of the soil as the most suitable method with the following major arguments:

- The elastic continuum theory provides better prediction of the critical harmonic number, $n_{cr}$, and reliable test data supports the “two-thirds power law” relationship of Equation (2-27) rather than the square-root relationship of Equation (2-17). The continuum theory tends to over-predict the experimental buckling strengths; however, it can be calibrated with the experimental results using a bias factor that is incorporated in the resistance factor $\phi$.
- Initial imperfections or prebuckling deformations are needed before the single-wave buckling mode develops. The multi-wave instability can occur for an undeformed structure.
- Soil resistance to structural movement into the cavity is of considerable importance. If this resistance is negligible compared to the resistance to outward movement, the single-wave mode will control the buckling load. If the resistances of the soil to inward and outward movements of pipe are equal, then multi-wave buckling will initiate instability, even if single-wave buckling follows.
- Multi-wave buckling has been observed in experiments at the initial stages of failure.
- Geometrically nonlinear FEA shows that compressive stresses in the soil induce a prestress between soil and structure so that separation or breakaway of the structure from the soil (the zero stiffness condition for inward movement) is delayed until linear buckling load is approached.

Moore (1989) emphasized the importance of the nonuniformity of the hoop force and the unconservative nature of design procedures based on the use of average hoop force around the pipe circumference with buckling theory based on conditions of uniform thrust. Moore (1989) recommended that the reduction factor $\phi$ in Equation (2-25), used to shift the theoretical continuum prediction to fit the experimental data, be taken as 0.75 for mean buckling strength in nongranular soil and 0.69 for mean buckling strength in granular soil. The characteristic value of this reduction factor computed from the experimental data, is $\phi = 0.55$, corresponding to 80% confidence.
Figure 2.9 Buckling process of buried flexible pipe described by Moore and Selig (1990)

**CONSTRAINED BUCKLING OF FLEXIBLE LINER IN RIGID HOST PIPE**

Constrained buckling is generally preceded by separation of the liner from the host pipe and snap-through at critical pressure. Constrained buckling strength has been generally expressed in one of the two forms presented in the following two sections.

**Enhancement Factor Approach**
Aggarwal and Cooper (1984) conducted forty-nine external pressure tests on Insituform liners inserted in small diameter (unknown) steel pipes where the pressure between the liner and the host steel pipe was increased until buckling. SDR of liners ranged from 30 to 90 in these tests. They defined an enhancement factor $K$ as the ratio of the experimental constrained buckling strength to the unconstrained buckling strength obtained from Equation (2-15).

$$P_{cr} = \frac{2KE}{1-\nu^2} \left( \frac{t}{D} \right)^3$$

Aggarwal and Cooper (1984) reported that $K$ ranged between 6.5 and 25.8 with forty-six of forty-nine tests indicating a $K$ value of greater than 7. This approach along with the lower bound value of $K = 7$ forms the basis of the buckling equation for partially degraded gravity pipe in ASTM F1216 today (Gumbel 2001).

Lo and Zhang (1994) studied the effect of a radial gap between the liner and host pipe on the constrained buckling strength for both unsymmetrical and symmetrical buckle shapes. For each case, they considered the buckled portion of the liner as an arch with clamped ends, and defined an enhancement factor, $H$, with respect to the buckling strength of an unconstrained pipe that could be determined by solving a set of four equations for a given gap-to-radius ($\Delta/R$) ratio. Numerical simulations showed that while the symmetrical buckling strength is significantly greater than that for unsymmetrical buckling for small gap-to-radius ratios, the difference diminishes as the $\Delta/R$ ratio increases, converging to an enhancement factor between 4 and 5 for large $\Delta/R$ (Figure 2.10). The results of earlier tests by Lo et al. (1993) on thirteen pipes made of epoxy that were inserted into 12 in. diameter steel pipes (SDR=55) with a gap ratio of 1% indicated unsymmetrical buckling in most tests and a tendency for symmetrical buckling in some tests with enhancement factors $H = 9.7$ to 15.1 (Figure 2.10). Considering that all thirteen tests resulted in buckling strengths between numerical predictions for symmetrical and unsymmetrical buckling, Lo and Zhang (1994) concluded that the actual buckling strength may be bound by the symmetrical and unsymmetrical buckling modes.
Moore and El Sawy (1996) argued that the assumption made by researchers such as Lo and Zhang (1994) and Pian and Bucciarelli (1967; see below) that the liner is inextensional (zero change in circumferential length) is not reasonable for certain pipeline applications. They performed nonlinear finite element analysis of liners with various D/t ratios and initial gap imperfections and determined the enhancement factor K for unsymmetrical buckling. The models used a numerical procedure for the slip and separation between the liner and the host pipe such that slip occurred when the interface shear exceeded a permissible value given by normal force and friction angle, and the liner was free to move inward when the normal force was not compressive. The results showed that the enhancement factor decreased rapidly up to a gap ratio of $\Delta_0/R=4\%$, where $\Delta_0$ is the radial gap between the liner and the host pipe, with the decrease being more rapid for larger D/t, and that the enhancement factor became almost independent of the D/t ratio for gap ratios $\Delta_0/R>4\%$ (Figure 2.11b). (Note that for Lo et al.’s [1993] experiments with SDR=55, Moore and El Sawy’s (1996) results predict an enhancement factor of about K=10, which is almost equal to Lo and Zhang’s [1994] prediction of K=9.7 for unsymmetrical buckling.) Based on the FEA results indicating that the enhancement factor
converges to $K=4$ for large gaps up to $\Delta_o/R=10\%$ independent of $D/t$ ratio, Moore and El Sawy (1996) recommended that $K=4$ be used.

![Figure 2.1](image)

Figure 2.11 (a) Finite element model, and (b) Enhancement factor for various $t/R$ and $D/R$ ratios used by Moore and El Sawy (1996) Moore, I.D. and K. El Sawy. 1996. Buckling Strength of Polymer Liners Used in Sewer Rehabilitation. *Transportation Research Record* 1542:127-132.

**Analytical Models Based on Radial Constraint**

Several analytical models have been proposed to account for the effect of radial constraint on the buckling strength with a basic form of:

$$P_{cr} = \frac{\eta E}{1-\nu^2} \left( \frac{t}{D} \right)^\beta$$

(2-29)

The coefficient $\eta$ and exponent $\beta$ proposed by different researchers varied, sometimes significantly, depending on the theories and assumptions used. Areas of differences in approach include the following:

- linear versus nonlinear deformation theory
- small versus large deflection theory
- unsymmetrical versus symmetrical buckling of liner
friction between the liner and host rigid pipe
whether host pipe wall moves inward with the buckled liner
whether the circumferential length of the liner is fixed or variable

Pian and Bucciarelli (1967) studied buckling of an elastic thin ring constrained by, but not bonded to, a rigid circular boundary under vertical distributed loading (Figure 2.12). The load was considered to be follow-through, and no friction was considered between the ring and the circular boundary. They first developed an approximate solution considering small deflections by solving Marguerre nonlinear shallow shell equations and arrived at:

\[
P_{cr} = 0.73E \frac{I^{3/5}A^{2/5}}{R^{1/5}} = 0.755E \left( \frac{t}{D} \right)^{2.2}
\]  

(2-30)

where

\( I = \) moment of inertia of the pipe wall
\( A = \) area of the pipe wall
\( R = \) mean radius of the pipe

They also developed an exact solution for large ring deflection which consists of a system of differential equations, and solved it numerically. For \( D/t = 200 \), the approximate solution predicted the buckling strength to be 3% less than the exact solution, and it was stated that the accuracy of the approximate solution would decrease for thicker rings.

Cheney (1971) used small-deflection linear theory to analyze the buckling of a rigidly encased thin ring that was envisioned to be made up of two parts: an upper part that buckles inward, away from the rigid circular wall, and a lower part that bears tightly against the rigid circular ring wall. Cheney assumed that the walls of the cavity move inward in the upper part with the liner but resist outward movement of the liner in the lower part. For relatively thin
For infinitely long pipes, Cheney’s derivation corresponds to the following expression (Omara et al. 1997):

\[ P_{cr} = \frac{2.55E \left( \frac{t}{D} \right)^{2.2}}{1 - \nu^2} \]

(2-31)

Glock (1977) used nonlinear deformation theory and energy principles to analyze the stability of rigidly encased thin ring under external hydrostatic pressure and thermal load with the assumptions that buckling occurs unsymmetrically, there is no friction between the liner and the cavity wall, and that the cavity wall does not move inward with the ring. For a rigidly encased thin ring, Glock derived:

\[ P_{cr} = 0.969 \frac{E \frac{t^{3/5}}{A^{2/5}}}{R^{11/5}} = 1.002 \frac{E \left( \frac{t}{D} \right)^{2.2}}{1 - \nu^2} \]

(2-32)

For the plane strain condition of long pipes, this expression can be modified for the case of thin pipes encased in rigid cavity as follows (Omara et al. 1997)

\[ P_{cr} = \frac{E \left( \frac{t}{D} \right)^{2.2}}{1 - \nu^2} \]

(2-33)

It is also to be noted that, Glock’s (1977) equation predicts a buckling strength for external hydrostatic pressure that is 39% of the buckling load predicted by Cheney (1971), but is 133% of the buckling load obtained by Pian and Bucciarelli (1967) for distributed vertical loads.

Omara et al. (1997) used experimental data from Aggarwal and Cooper (1984), Lo et al. (1993) and Guice et al. (1994) to compare the accuracy of predictions using Aggarwal and Cooper’s enhancement factor of K=7, Cheney’s (1971) equation, and Glock’s (1977) equation. Omara et al. used linear regression analysis and determined the best fit to the experimental data as:

\[ P_{cr} = 1.07 \frac{E \left( \frac{t}{D} \right)^{2.17}}{1 - \nu^2} \]

(2-34)

which is very close to Glock’s equation. Omara et al. concluded that not only is the enhancement factor K = 7 conservative, but the enhancement factor approach should not be used due to variability in adjusting the theoretical unconstrained pipe buckling strength to actual experimental results. Omara et al. recommended that Glock’s model be used with a coefficient \( \eta \) (i.e., the coefficient 1.07 in the above equation) that is calibrated with additional test data encompassing a greater range of SDR values.

Boot (1998) extended Glock’s (1977) theory to reflect symmetrical buckling and to account for initial gap imperfections (as reported by Gumbel 2001). For symmetrical buckling with zero imperfection, Boot found the coefficient \( \eta \) in Equation (2-29) above to be 1.323, a 32% increase over the coefficient of 1.002 for unsymmetrical buckling (as in Glock’s equation).
Boot showed that the sensitivity to gap imperfection increased with increasing D/t ratio but the resulting reduction in buckling strength hardly varied with D/t when the gap size was expressed as gap/thickness ratio (Figure 2.13). Boot confirmed that for large gap ratios, the symmetric buckling strength is equal to the unconstrained buckling strength in Equation (2-15) above.

![Figure 2.13 Symmetric buckling strength reduction factor due to symmetrical gap imperfection expressed as a function of gap/thickness ratio (Source: Gumbel, J. 2001. New Approach to Design of Circular Liner Pipe to Resist External Hydrostatic Pressure. Pipelines 2001: Advances in Pipeline Engineering and Construction, pp. 1-18. doi: 10.1061/40574(2001)77, with permission from ASCE.)](image)

Gumbel (2001) compared Boot’s (1998) symmetrical buckling strength with zero imperfection to the experimental data with an SDR range of 30 to 100 from Aggarwal and Cooper (1984), Guice et al. (1994), Lo and Zhang (1994), Boot and Javadi (1998), and Seeman et al. (2001) after correcting the experimental results for gap. Gumbel found that the symmetrical buckling model fit the experimental data better than the unsymmetrical buckling model for close-fitting, perfectly circular liners (Figure 2.14).
PREVIOUS TESTS ON CFRP-LINED PCCP

We performed a literature review to identify hydrostatic pressure tests, three-edge bearing tests, and other tests that have been performed by others on CFRP-lined PCCP. The results of literature review are summarized below.

Hydrostatic Pressure Tests

We identified the following hydrostatic pressure tests performed on PCCP in diameters ranging from 96 in. to 102 in. by three different authors:

1. Alkhrdaji and Thomas (2003) of Structural Preservation Systems (SPS) performed two tests on 102 in. diameter PCCP. The first test was performed on a single pipe section with no special CFRP termination detail at pipe ends, and the second test was performed on a three-pipe test specimen with the middle pipe having a CFRP liner with end termination details. The end-termination detail included bonding the CFRP directly to the joint rings and radially expanding steel rings in the joint recesses against the CFRP liner (Alkhrdaji and Thomas 2003). For the specimen in Test 1 and for the middle section of the three-pipe specimen in Test 2, pipe degradation was simulated by cutting the prestressing wires, while the outer core and steel cylinder were left in place. The first specimen failed at a pressure equal to 12% of the ultimate strength of the pipe, corresponding to the yield capacity of the steel cylinder only and without rupture of CFRP liner. The
test demonstrates that water must-have bypassed the CFRP liner, and hence CFRP liner was not carrying any stress. In the second test, rupture of the steel cylinder along with damage of CFRP liner (reported as rupture) were observed. The test demonstrates that either termination detail was not watertight, causing cylinder rupture that promoted CFRP liner damage, or some other failure mode, unrecognized by the author contributed to premature CFRP liner damage and resulted in leakage of water behind the steel cylinder and cylinder rupture (see Chapter 7 on longitudinal bending failure of CFRP liner at the boundary of loss of prestress band). (No measurement, other than pressure, was reported for these tests.) The tests highlighted the need for appropriate CFRP termination details at bell and spigot joints.

2. Lee and Karbhari (2005), supported by California Department of Water Resources, tested one CFRP-lined 96 in. diameter PCCP in an upright position where pipe degradation was simulated by cutting the prestressing wires over a 25% of the pipe length (exact location not reported) without removing the outer core or the steel cylinder. The CFRP liner in the circumferential direction consisted of seven layers CFRP, each consisting of 20 oz/yd² unidirectional carbon fabric, 52 mil thick, with a modulus of 10,730 ksi and strength of 55 ksi. (Note that the reported strength and modulus results in a low value of strain at failure of 0.51 %.) The CFRP termination detail is not reported in the paper.

The pipe was tested up to a pressure of 300 psi at which point the test was terminated due to leakage through seals between the PCCP ends and the reinforced concrete end caps. The maximum pressure of 300 psi should be compared to the as-built strength of PCCP prior to damage of 430 psi and the similar strength of CFRP-repaired pipe. (Note that the steel cylinder has a yield strength of $f_{yy} = 45$ ksi and an ultimate strength of $f_{yu} = 58$ ksi.) The authors claimed that at 300 psi, the strain in CFRP, as measured by a strain gage mounted on liner surface, was 25% of the ultimate strain capacity of the CFRP. This shows that the failure of CFRP was most likely not related to its strength, but was due to loss of watertightness, either because of cracks forming in the CFRP, failure of CFRP termination details, or simply because of inability of the bulkheads to retain water.

3. Foundation Technologies Inc. and HJ3 Composites (2000) refer to testing of a 96 in. diameter PCCP (ECP) with internally bonded CFRP in a marketing brochure. HJ3 did not present whether degradation of PCCP was simulated. The specimen was tested up to a pressure of 150 psi with no failure. The pressure of 150 psi corresponds to approximately 3.8 times the nominal yield strength of the steel cylinder (based on calculation using a minimum yield strength of 33 ksi and the reported cylinder thickness). We do not know how much of the CFRP strength was developed since the details of the pipe condition and CFRP liner were not presented. We contacted HJ3 for additional data; although HJ3 expressed interest in sharing the data, we have not received any additional information to date.
We also contacted MWD, Fyfe Co., and Fibrwrap Construction for availability of test data in this area, but learned that they do not have hydrostatic pressure test data.

Three-Edge Bearing Tests

Lee and Karbhari (2005) also performed three-edge bearing test on a 12 in. long section of a 96 in. diameter PCCP (ECP) according to ASTM C497 but in a horizontal position. The concrete core and steel cylinder were in place but all prestressing wires were removed at the time of test. Vertical and horizontal deflections were monitored, but strains were not. The progression of failure was reported as follows:

- Concrete cracking (location not reported) at 0.41% vertical deflection and negligible horizontal deflection, and at 38% of the maximum load,
- CFRP debonding (location and extent not reported) at 0.49% vertical deflection and negligible horizontal deflection, and at 55% of the maximum load,
- Steel cylinder yielding (location not reported) at 0.78% vertical deflection and 0.16% horizontal deflection, and at 72% of the maximum load,
- Maximum load occurring at 3.1% vertical deflection and 1.2% horizontal deflection,
- “Complete failure of the repair scheme” at 3.5% vertical deflection and 2.1% horizontal deflection.

The above results indicate a significant difference between horizontal and vertical deflection of pipe, which would not be expected from a typical three-edge bearing test of a linearly elastic pipe. A reason for this behavior is local flattening of the pipe at invert and crown, while the left and right halves of the pipe are intact.

We are also aware of a three-edge bearing tests conducted by MWD on a PCCP lined with Fyfe materials; however, the data from that study could not be located by MWD.

Mechanical Simulation of Pressure Test

Lee and Karbhari (2005) conducted eight additional tests on 12 in. long sections of a 96 in. diameter PCCP (ECP) with CFRP, GFRP, or no liner, where internal pressure was simulated by four hydraulic jacks mounted inside the pipe section 90° apart. One test was conducted on a control specimen with prestressing wires and without CFRP liner, representing a “good pipe,” without distress. Of the remaining seven specimens, three specimens without prestress were tested with six and seven layers of CFRP installed using wet layup technique. Radial deflections and strains in the outer concrete core, steel cylinder, and FRP liner were monitored. The actual data presented in this publication is limited, but they show the following:

- Full strength of two specimens with no prestress, strengthened with a seven-layer CFRP liner using wet-layup, was developed with 7% to 15% increase over the capacity of the design good pipe and higher than the calculated capacity of the repaired pipe.
• One distressed pipe that had all wires removed and repaired using wet-layup CFRP liner failed due to debonding of CFRP liner at 83% of the capacity of the good pipe.

PREVIOUS TESTS ON CFRP-REPAIRED CONCRETE STRUCTURES RELEVANT TO PCCP REPAIR

We reviewed the literature to identify previous tests on other CFRP-repaired concrete structures that are relevant to PCCP repair and identified a study by Yuan and Li (2010) that is relevant to the bond strength test to be developed and conducted as part of the current project.

Yuan and Li (2010) conducted “peeling tests” on twenty-eight beam specimens with one or two layers of CFRP sheets bonded to their soffit where twenty-one of the specimens had concave curved soffits to simulate the interior surface of pipes, culverts, tunnels, and other similar structures with CFRP lining, and had radii of curvature ranging between 157 in. and 236 in. A “peeling load” was applied directly on the CFRP, perpendicular to the surface of the CFRP, and the load-deflection and debonding behavior of the specimens was monitored. While the results were variable due to variation in concrete properties, Yuan and Li concluded based on comparison of load-deflection curves for four specimens with different radii of curvature that debonding load reduces as the radius of curvature to which CFRP is bonded decreases, and is expressible as a linear function of curvature (a + b/R).

CONSTRAINED MODULUS OF SOIL SURROUNDING HOST PIPE

The CFRP-lined PCCP continues to degrade due to exposure to corrosive elements on its exterior. The degradation can be in the form of corrosion or embrittlement of wires resulting in loss of prestress and cracking of the outer core exposing the steel cylinder to corrosion and possible perforation. The PCCP with lost prestress and cracked outer core will become flexible, and its deflection and strain distribution will be controlled by the stiffness of the soil surrounding the pipe as with any other flexible pipe. For fiberglass pipe, also a flexible pipe, AWWA Manual M45 Fiberglass Pipe Design Manual includes a method of determining soil modulus to be used in design. In this section, we summarize the AWWA M45 approach and review the literature that forms the basis of the values recommended therein. Selection of soil modulus to be used in design of CFRP liner for PCCP is discussed in Chapter 7.

AWWA M45 Approach

AWWA M45 provides values of constrained modulus for both in-situ soil (M_{sn}) and backfill (constructed soil) (M_{sb}). For flexible pipe installations, M_{sb} and M_{sn} are combined in a single value, referred to as the constrained modulus of soil (M_s), defines as

\[ M_s = S_c \cdot M_{sb} \]  \hspace{1cm} (2-35)

where \( S_c \) = soil support combining factor obtained as a function of the \( M_{sn}/M_{sb} \) ratio and the ratio of the trench width to pipe diameter.
AWWA M45 presents estimated values of $M_{sn}$ for in-situ soils based on the standard correlations between the results of the standard penetration test ($N$) and/or the unconfined compression strength ($q_u$). These values are based on the work of Bowles (*Foundation Analysis and Design, 3rd Edition, 1982*). Bowles recommends Young’s modulus ($E_s$) for in-situ soils as shown in Table 2.2. Constrained modulus $M_{sn}$ is related to $E_s$ by:

\[ M_{sn} = E_s \cdot \frac{1-\nu}{(1+\nu)(1-2\nu)} \]  

(2-36)

Constrained soil modulus values obtained from Equation 2-36 using $\nu = 0.3$ are included in Table 2.2.
Table 2.2
Typical range of Young’s modulus ($E_s$) and constrained modulus ($M_{sn}$) for selected soils

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Young’s Modulus, $E_s$</th>
<th>Constrained Soil Modulus, $M_{sn}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ksf</td>
<td>psi</td>
</tr>
<tr>
<td>Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very soft</td>
<td>50-250</td>
<td>350-1,700</td>
</tr>
<tr>
<td>Soft</td>
<td>100-500</td>
<td>700-3,500</td>
</tr>
<tr>
<td>Medium</td>
<td>300-1,000</td>
<td>2,100-7,000</td>
</tr>
<tr>
<td>Hard</td>
<td>1,000-2,000</td>
<td>7,000-14,000</td>
</tr>
<tr>
<td>Sandy</td>
<td>500-5,000</td>
<td>3,500-35,000</td>
</tr>
<tr>
<td>Glacial Till</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>200-3,200</td>
<td>1,400-20,000</td>
</tr>
<tr>
<td>Dense</td>
<td>3,000-15,000</td>
<td>20,000-100,000</td>
</tr>
<tr>
<td>Very dense</td>
<td>10,000-30,000</td>
<td>70,000-200,000</td>
</tr>
<tr>
<td>Loess</td>
<td>300-1,200</td>
<td>2,100-8,500</td>
</tr>
<tr>
<td>Sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty</td>
<td>150-450</td>
<td>1,000-3,000</td>
</tr>
<tr>
<td>Loose</td>
<td>200-500</td>
<td>1,400-3,500</td>
</tr>
<tr>
<td>Dense</td>
<td>1,000-1,700</td>
<td>7,000-12,000</td>
</tr>
<tr>
<td>Sand and Gravel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>1,000-3,000</td>
<td>7,000-21,000</td>
</tr>
<tr>
<td>Dense</td>
<td>2,000-4,000</td>
<td>14,000-28,000</td>
</tr>
<tr>
<td>Shale</td>
<td>3,000-300,000</td>
<td>20,000-2,000,000</td>
</tr>
<tr>
<td>Silt</td>
<td>40-400</td>
<td>300-2,800</td>
</tr>
</tbody>
</table>
CHAPTER 3
FULL-SCALE EXPERIMENTAL TESTING OF CFRP-LINED PCCP

INTRODUCTION

The experimental testing in this project was initially envisioned to consist of three types of tests on CFRP-lined PCCP: CFRP-to-concrete inner core bond strength tests, hydrostatic pressure tests, and three-edge bearing (3EB) tests. Hydrostatic pressure and 3EB tests can be performed according to the standard established procedures used in quality control testing of newly manufactured PCCP; however, a standard test for CFRP-to-concrete bond strength in a CFRP-lined PCCP that can capture all likely modes of CFRP debonding (i.e., radial tension, shear, concrete crushing) is not readily available. We considered several potential test setups to study the debonding behavior of CFRP from the concrete core but decided that none had the advantages of the 3EB test from the standpoint of ease of specimen preparation, instrumentation, testing, and interpretation of test results. Since the 3EB specimens are likely to exhibit debonding of the CFRP liner from the inner concrete core at the invert or crown, we decided to increase the number of 3EB tests and to instrument the specimens to capture debonding of CFRP.

Purpose

The purpose of full-scale testing of CFRP renewed PCCP with broken wires and cracked outer concrete core presented in this chapter is to:

- Conduct full scale tests on both embedded- and lined-cylinder pipe (ECP and LCP).
- Identify the failure modes that have not been included in the CFRP liner design approach.
- Determine the watertightness of CFRP-lined distressed PCCP over the full range of design pressures.
- Study the behavior of CFRP-lined distressed PCCP subjected to internal pressure.
- Study the behavior of CFRP-lined distressed PCCP subjected to pipe wall bending resulting from 3EB load.
- Demonstrate that the calculated capacity of the CFRP liner in a distressed PCCP can be reached in a hydrostatic pressure test.
- Determine the effectiveness of CFRP termination details developed to maintain watertightness of CFRP-renewed PCCP as the pipe continues to degrade.
- Provide test data for calibration of Finite Element Analysis models that are developed for pipe-soil interaction analysis (Chapter 5).

Scope

The initial test program included the following:

- Three 3EB tests on specimens cut from 54 in. diameter ECP type PCCP.
• Five 3EB tests on specimens cut from 48 in. diameter LCP type PCCP.
• One hydrostatic pressure test on 54 in. diameter ECP type PCCP.
• One hydrostatic pressure test on 48 in. diameter LCP type PCCP.

The specimens had varying number of CFRP layers and different CFRP terminations, as described in the following sections.

THREE-EDGE BEARING TESTS

Test Specimens

The 3EB tests were conducted on 4 ft long sections cut from one 54 in. diameter ECP type PCCP and one 48 in. diameter LCP type PCCP, provided by Hanson. The pipes were not known to be distressed or otherwise deficient before the preparation of the test specimens. The design loads of the original pipes are included in Table 3.1, and the complete pipe design sheets are included in Appendix A. Preparation of 3EB test specimens from both pipes included the following typical steps:

• Remove all prestressing wires along the entire pipe length by scoring the mortar coating to expose the wires, and then by cutting the wires by using either circular saw or torch.
• Remove the outer concrete core (in the case of ECP only) by making longitudinal cuts in the outer core along the pipe length at both springlines, and then by prying off the outer core from the steel cylinder. At the end of this step, both the ECP and LCP pipes were left with exposed steel cylinder and inner concrete core along their entire length.
• Cut three 4 ft wide sections from the 54 in. diameter ECP and four 4 ft wide sections from the 48 in. diameter LCP without any joint rings included in the cut sections.
• Install CFRP liner in all seven specimens consisting of the number of CFRP layers listed in Table 3.1.

The number of CFRP layers was varied between the specimens so that both the effect of pipe type (ECP versus LCP) and the amount of CFRP on the debonding behavior can be studied. All CFRP installations were performed by Fibrwrap Construction using materials manufactured by Fyfe Co. The CFRP-lined specimens were protected from rain until sufficient cure level and a tack-free surface has been achieved, and then stored outside until testing 60 days later. In addition to the CFRP-lined specimens, an eight 3EB test was performed as a control specimen with no CFRP liner. This control test was conducted on a 3.4 ft long section of the 48 in. diameter LCP that was left over after the first four sections were cut. The final 3EB test program was as shown in Table 3.1.

During preparation of the specimens prior to CFRP installation, Specimen 54 ECP-1 developed a crack in the inner concrete core along both invert and crown, and Specimen 54 ECP-2 developed a crack along the invert (in the storage position). To investigate debonding behavior of CFRP from uncracked concrete core effectively, these two specimens were tested in rotated
position such that the pre-cracked locations corresponded to the springlines where CFRP is in compression and not expected to debond.

Test Setup

All 3EB tests were conducted at the Hanson plant in Palatka, Florida. The test setup is shown in Figure 3.1. The test frame has a stationary bottom horizontal beam and a moveable upper horizontal beam, both attached to vertical columns on each side. Each beam consists of a steel section with a wood log bolted to it on its side facing the specimen. Two neoprene strips, spaced according to ASTM C497, are attached to the lower beam. The load is applied through two hydraulic jacks that are mounted vertically on the upper beam such that the upper beam moves down to load the specimen as the hydraulic pump is operated. The rate of loading is adjusted manually by controlling a valve.

Table 3.1
3EB test specimens

<table>
<thead>
<tr>
<th>Test Pipe Type</th>
<th>Mark No.</th>
<th>Design Loads of Original PCCP</th>
<th>Specimens Cut from Pipe</th>
<th>Specimen Length (ft)</th>
<th>No. of CFRP Layers(1)</th>
<th>CFRP Termination Detail</th>
</tr>
</thead>
<tbody>
<tr>
<td>54 in. ECP</td>
<td>280</td>
<td>Pw = 250 psi, Pt = 100 psi, H = 6 ft (We = 4,785 lbf/ft), Wt = 720 lbf/ft</td>
<td>1</td>
<td>4</td>
<td>1L + 1H</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>4</td>
<td>1L + 1H</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>4</td>
<td>1L + 4H</td>
<td>-</td>
</tr>
<tr>
<td>48 in. LCP</td>
<td>211</td>
<td>Pw = 200 psi, Pt = 100 psi, H = 8 ft (We = 6,316 lbf/ft), Wt = 462 lbf/ft</td>
<td>1</td>
<td>4</td>
<td>1L + 3H</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>4</td>
<td>1L + 3H</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>4</td>
<td>1L + 1H</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>4</td>
<td>1L + 1H</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>5</td>
<td>3.4</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

(1) Tyfo SCH-41-2X; L = Longitudinal; H = Hoop

Figure 3.1 Typical view of three-edge bearing test setup
Instrumentation

The instrumentation used for all CFRP-lined 3EB specimens is summarized below and shown schematically in Figure 3.2. Measurements from all instruments were recorded continuously using a National Instruments data acquisition system with LabVIEW software.

- Load was measured using a voltage-output pressure transducer that was installed in the hydraulic hoses between the hydraulic pump and the two jacks on the upper beam of the test frame. The pressure transducer was pre-calibrated in SGH laboratory prior to use in the field. The correlation between pressure and 3EB load was established prior to the actual 3EB tests by loading wooden logs up to 45 kip and correlating the pressure readings to the load readings obtained from the dial of the test machine. The slope of the best line fit to the recorded calibration data was entered into the data acquisition system as the calibration factor to be used during the actual tests.
- Pipe deflection was measured using two string potentiometers installed vertically and horizontally in the pipe at mid-length.
- Crosshead displacement was measured using two string potentiometers attached vertically between the lower and upper beams of the test frame.
- Strain in the CFRP liner in the circumferential direction was measured using six electrical resistance strain gages: two installed at the invert at one quarter and three quarters of the specimen length, two installed at the crown at the same locations as the invert, and one at each springline at mid-length of the specimen.
- Strain in the steel cylinder in the circumferential direction was measured using two electrical resistance strain gages, one installed at each springline.

In testing the 48 in. diameter LCP control specimen with no CFRP liner, the load, pipe deflections, and crosshead deflections were monitored, and no strain gages were used.

Test Procedure

The tests were performed according to ASTM C497 – Standard Test Methods for Concrete Pipe, Manhole Sections, or Tile. For each test, the specimen was placed on the lower beam of the test frame and lined up parallel to the upper and lower beams, a neoprene pad was placed on top of the specimen; the upper beam was lowered until it reaches contact with the specimen without applying load; and the test frame was locked. The string potentiometers for measuring crosshead displacement were installed (all other instruments were placed beforehand), and all instruments were zeroed. The load was applied by manually operating the hydraulic pump to achieve a loading rate of not more than 7,500 lbf/linear foot or 30,000 lbf /min for the 4 ft specimen. The loading was paused at approximately 10,000 lbf intervals at the beginning of the test and 5,000 lbf intervals as the test progressed, or when the specimen exhibited a significant event (e.g., cracking, debonding), to visually investigate and document the progression of failure. The tests were terminated when the specimen exhibited a significant loss of load carrying capacity, and further deflection of the specimen resulted in reduction in the measured load.
Figure 3.2 Instrumentation used in 3EB tests on CFRP-lined PCCP

Results

Control Specimen

The control specimen with no CFRP liner (48LCP-5) carried a maximum load of $P_{\text{max}} = 30,500$ lbf (8,970 lbf/ft) and failed by concrete crushing at the inside of the springlines. The failure progressed as follows:

- The first cracks formed along the invert at approximately 26% of $P_{\text{max}}$ (Figure 3.3a,b) which was followed by cracks at the crown (Figure 3.3c) and outside of the springlines at approximately 39% of $P_{\text{max}}$. These cracks were vertical with respect to the concrete core and through the thickness.
- As loading continued, cracks widened, and additional cracks formed in the vicinity of the springlines, spaced at 3 in. to 4 in. (Figure 3.3d).
- As the maximum load was approached, crack widths had reached 0.5 in. at the invert and crown, and 20 - 60 mils around the springlines.
- At the maximum load, concrete crushed along both springlines on the inside surface of the specimen (Figure 3.3e), and the previously formed crack at the invert progressed horizontally at the concrete-steel cylinder interface (Figure 3.3f).
(a) Invert crack at 0.26P_{\text{max}}

(b) Invert crack at 0.26P_{\text{max}}

(c) Crown crack at 0.39P_{\text{max}}

(d) Springline cracks at 0.39P_{\text{max}}

(e) Crushing of core at springline at P_{\text{max}}

(f) Invert crack at P_{\text{max}}

**Figure 3.3** Progression of failure of 3EB control specimen 48LCP-5 with no CFRP liner
**CFRP-Lined Specimens**

All CFRP-lined 3EB specimens exhibited debonding of CFRP at invert and/or crown, allowing investigation of both the bending behavior of a CFRP-lined degraded PCCP and debonding behavior of CFRP liner. Therefore, no additional bond strength tests were needed to capture CFRP bonding failure mode. Typical progression of failure in testing of both 48 in. and 54 in. diameter specimens is summarized below and shown photographically shown in Figure 3.4 and Figure 3.5, respectively:

- The first cracks formed at the springlines. These cracks widened and additional cracks formed in the vicinity of springlines as the load increased, but no cracks formed at invert or crown until CFRP debonding occurred.

- At the maximum recorded load, CFRP debonded at invert or crown over a width of approximately 12 in., accompanied by a vertical or inclined (diagonal tension) crack in the concrete core within the debonded area, and a significant drop in load (Figure 3.4a,b,c). Debonding occurred with cohesive concrete failure (Figure 5.4.d) and maximum CFRP hoop strains at the debonding locations between 1,100 \( \mu \varepsilon \) and 3,000 \( \mu \varepsilon \) (Table 3.2).

- As loading continued after the first debonding, the 3EB load started to increase again, more cracks formed in the concrete core at the springlines (Figure 3.4.e), the debonded areas became wider, and then CFRP debonded at the location opposite to the first debonding location (e.g., first invert then crown, or vice versa). The load at which the second debonding occurred was generally lower than the first debonding load except for Specimen 54ECP-2 which exhibited a second debonding load higher than the first, and 48LCP-2 which exhibited a second debonding load that is almost equal to the first.

- As the loading continued after the second debonding, debonding at invert and crown progressed toward the springlines, cracks in the concrete core at invert and crown became very wide, the specimen did not pick up significant load, and failed.

- At the conclusion of the test, the original circular shape of the specimen had become a non-circular, non-oval shape, but rather a shape with kinks at invert or crown (Figure 3.4.f). CFRP ruptured in these kinked locations in some specimens. The cracking in the concrete core was distributed within 45 degrees on each side of both springlines with 3 in. to 5 in. spacing. In these areas of distributed cracking around the springlines, CFRP remained bonded to the concrete core, and concrete had not crushed.

A summary of the maximum loads, corresponding pipe deflections, and failure modes is presented in Table 3.2. Load versus pipe vertical deflections are compared in Figure 3.5 for the 54 in. diameter ECP specimens and in Figure 3.6 for the 48 in. diameter LCP specimens. Compared to the control specimen, the 48 in. diameter CFRP-lined specimens exhibited 51% and 65% increase in the normalized load capacity (lbf/ft) when one hoop layer of CFRP was used (48LCP-3, 48LCP-4), and 71% and 73% increase in normalized load capacity when three hoop layers of CFRP were used (48LCP-1, 48LCP-2). Such a comparison was not available for the 54 in. diameter specimens due to the absence of a 54 in. diameter control specimen; however, the
specimen with four hoop layers (54ECP-3) exhibited 53% and 78% increase in load capacity compared to the specimens with one hoop layer of CFRP (54ECP-1, 54ECP-2).
Figure 3.4 Typical failure modes observed in 48 in. diameter 3EB tests
(a) Flattening of invert  
(b) Debonding at invert with diagonal tension crack  
(c) Flattening of crown  
(d) Debonding at crown with diagonal tension crack  
(e) Cracks at springline  
(f) Failed specimen

Figure 3.5 Typical failure modes observed in 54 in. diameter 3EB tests
Table 3.2
Summary of 3EB test results

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Section Length (ft)</th>
<th>No. of CFRP Layers(1)</th>
<th>Max. Load</th>
<th>Pipe Deflection at Max. Load (in.)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>48LCP-5</td>
<td>3.4</td>
<td>-</td>
<td>30,497</td>
<td>Not available at max. load (Removed at 8,596 lbf total load when Δv=0.52 in., Δh=0.43 in.)</td>
<td>Horizontal crack in concrete at invert followed by concrete crushing</td>
</tr>
<tr>
<td>(Control)</td>
<td></td>
<td></td>
<td>8,970</td>
<td></td>
<td></td>
</tr>
<tr>
<td>48LCP-1</td>
<td>4</td>
<td>1L + 3H</td>
<td>61,470</td>
<td>0.28</td>
<td>CFRP debonding at crown followed by invert</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>15,368</td>
<td>0.22</td>
<td></td>
</tr>
<tr>
<td>48LCP-2</td>
<td>4</td>
<td>1L + 3H</td>
<td>62,145</td>
<td>0.27</td>
<td>CFRP debonding at invert followed by crown</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>15,536</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>48LCP-3</td>
<td>4</td>
<td>1L + 1H</td>
<td>54,011</td>
<td>0.41</td>
<td>CFRP debonding at invert followed by crown</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>13,503</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>48LCP-4</td>
<td>4</td>
<td>1L + 1H</td>
<td>59,314</td>
<td>0.43</td>
<td>CFRP debonding at crown followed by invert</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>14,829</td>
<td>0.49</td>
<td></td>
</tr>
<tr>
<td>54ECP-1</td>
<td>4</td>
<td>1L + 1H</td>
<td>19,700</td>
<td>1.25</td>
<td>CFRP debonding at crown followed by invert</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4,925</td>
<td>1.67</td>
<td></td>
</tr>
<tr>
<td>54ECP-2</td>
<td>4</td>
<td>1L + 1H</td>
<td>16,873</td>
<td>3.35(2)</td>
<td>CFRP debonding at crown, later at invert</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4,218</td>
<td>2.15</td>
<td></td>
</tr>
<tr>
<td>54ECP-3</td>
<td>4</td>
<td>1L + 4H</td>
<td>30,045</td>
<td>0.81</td>
<td>CFRP debonding at invert followed by crown</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7,511</td>
<td>0.91</td>
<td></td>
</tr>
</tbody>
</table>

{(1) Tyfo SCH-41-2X; L = Longitudinal; H = Hoop}
{(2) In 54ECP-2, the second debonding (invert) occurred at a higher load level than the first (crown); therefore, the reported pipe vertical deflection at maximum load may be greater than actual since it may have been affected by the separation of CFRP at the first debonding.}
Figure 3.6 Load versus crosshead displacement curves from 3EB tests
Figure 3.7 Load versus pipe vertical deflection curves from 3EB tests

(a) 48 in. diameter specimens

(b) 54 in. diameter specimens

Note: After the 1st debonding, pipe vertical deflection readings become inaccurate due to separation of CFRP, and therefore, the second debonding is not shown on this plot.

Actual $P_{\text{max}}$ occurred at 2nd debonding at 16,873 lbf which is not plotted here (see note above).
Figure 3.8 Typical horizontal and pipe vertical deflections measured with string potentiometers in 3EB tests

(a) 48 in. diameter specimens

(b) 54 in. diameter specimens

Note: Pipe deflection readings may not be accurate after the 1st debonding due to CFRP separation.
Figure 3.9 Typical vertical deflection of pipe versus CFRP hoop strain from 48 in. diameter 3EB tests
Figure 3.10 Typical vertical deflection of pipe versus CFRP hoop strain obtained from 54 in. diameter 3EB tests

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Analytical Prediction of Debonding Shear Strain

ACI Committee 440 (440.2R-08) recommends calculation of the CFRP debonding shear strain for flexural strengthening of concrete structures as:

$$\varepsilon_{fd} = 0.083 \frac{f'_c}{\sqrt{E_{frp}t_{frp}}} \tag{3-1}$$

where $f'_c$ = concrete compressive strength (psi)

$E_{frp}$ = CFRP modulus (psi)

$t_{frp}$ = total thickness of CFRP (in.)

The committee states that Equation 3-1 is a modified form of the debonding shear strain equation originally proposed by Teng et al. (2003) who experimentally studied intermediate crack-induced debonding in reinforced concrete beams and slabs.

The original equation proposed by Teng et al. was as follows:

$$\sigma_{dbic} = 0.48 \cdot \beta_p \cdot \beta_L \frac{E_{frp} \sqrt{f'_c}}{t_{frp}} \tag{3-2}$$

where $\sigma_{dbic}$ = stress in CFRP at intermediate crack-induced debonding (MPa)

$\beta_p$ is a function of the ratio of width of concrete section being strengthened, and is equal to 0.71 when the entire concrete width is strengthened

$\beta_L$ is a function of the bond length and is equal to 1.0 if the bond length is greater than the effective bond length

$E_{frp}$ = CFRP modulus (MPa)

$t_{frp}$ = total thickness of CFRP (mm)

Teng et al. (2003) obtained this equation by first developing a concrete-to-CFRP bond strength model based on shear bond tests, and then adjusting the equation with a factor (0.48 in Equation 3-2) based on the results of flexure tests on CFRP-strengthened concrete beams and slabs that exhibited intermediate crack-induced debonding. Inserting $\beta_p = 0.71$, $\beta_L = 1.0$, converting debonding stress to debonding strain, and converting SI units to English units, Equation 3-2 becomes:

$$\varepsilon_{dbic} = 0.234 \frac{f'_c}{\sqrt{E_{frp}t_{frp}}} \tag{3-3}$$

where $f'_c$ and $E_{frp}$ are in psi, and $t_{frp}$ is in inches. Note that Equation 3-3 proposed by Teng et al. results in a debonding shear strain that is 0.31 to 0.34 times that calculated from Equation 3-1 recommended by ACI 440 for $f'_c$ of 5,000 psi to 7,000 psi.
We compared the debonding shear strains that we measured in our 3EB tests to the analytical prediction of Teng et al. (2003) in Equation 3-3. The results are presented in Table 3.3. In this comparison, we used the minimum debonding strain obtained from each test (values underlined in Table 3.3), and considered $f'_c$ as the 28-day concrete compressive strength provided by Hanson for the test specimens, $E_{frp}$ as the mean CFRP modulus that we obtained from tension testing of witness panels that Fibrwrap prepared at the time of CFRP installation in the specimens, and $t_{frp}$ as the nominal thickness of the CFRP hoop layers. The results in Table 3.3 indicate that Teng et al.’s equation predicts the debonding shear strain for CFRP-lined PCCP reasonably well with an average measured-to-predicted debonding strain ratio of 1.04 and a coefficient of variation of 27%.

As a result, we modified the debonding shear strain equation used in the design process to incorporate Teng et al.’s (2003) equation, and used the results presented in Table 3.3 to develop the corresponding strength reduction factor $\phi$ (Chapter 6).

**Analytical Prediction of Radial Tensile Stress**

Calculations performed and reported for 3EB test in Chapter 5 showed that radial tension was not the mode of debonding of the CFRP from the concrete inner core.
Table 3.3
Analytical prediction of debonding strain using Eq. 3.3

<table>
<thead>
<tr>
<th>Test No.</th>
<th>No. of CFRP Layers(^{(1)})</th>
<th>Failure Mode</th>
<th>Tensile CFRP Hoop Strain at Debonding (µε)(^{(2)})</th>
<th>Thickness of CFRP Hoop Layers (in.)</th>
<th>CFRP Modulus (psi)</th>
<th>Concrete Comp. Strength (f'_{c}) (psi)(^{(3)})</th>
<th>Debonding strain Predicted by Teng et al. 2003 (µε)</th>
<th>Min. Debonding Strain from Test / Prediction by Teng et al.(^{(2)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>48LCP-1</td>
<td>1L + 3H</td>
<td>CFRP debonding at crown followed by invert CFRP debonding at invert</td>
<td>2,000</td>
<td>1,200</td>
<td>0.24</td>
<td>13.4x10(^{6})</td>
<td>6,920</td>
<td>1,190</td>
</tr>
<tr>
<td>48LCP-2</td>
<td>1L + 3H</td>
<td>CFRP debonding at crown followed by invert CFRP debonding at invert</td>
<td>1,100</td>
<td>1,400</td>
<td>0.24</td>
<td>13.4x10(^{6})</td>
<td>6,920</td>
<td>1,190</td>
</tr>
<tr>
<td>48LCP-3</td>
<td>1L + 1H</td>
<td>CFRP debonding at invert followed by crown CFRP debonding at crown</td>
<td>2,300</td>
<td>3,000</td>
<td>0.08</td>
<td>13.4x10(^{6})</td>
<td>6,920</td>
<td>2,061</td>
</tr>
<tr>
<td>48LCP-4</td>
<td>1L + 1H</td>
<td>CFRP debonding at invert followed by crown CFRP debonding at crown</td>
<td>2,500</td>
<td>3,000</td>
<td>0.08</td>
<td>13.4x10(^{6})</td>
<td>6,920</td>
<td>2,061</td>
</tr>
<tr>
<td>54ECP-1</td>
<td>1L + 1H</td>
<td>CFRP debonding at crown followed by invert CFRP debonding at crown</td>
<td>1,600</td>
<td>2,200</td>
<td>0.08</td>
<td>13.4x10(^{6})</td>
<td>5,820</td>
<td>1,974</td>
</tr>
<tr>
<td>54ECP-2</td>
<td>1L + 1H</td>
<td>CFRP debonding at crown, later at invert CFRP debonding at crown</td>
<td>1,500</td>
<td>1,500</td>
<td>0.08</td>
<td>13.4x10(^{6})</td>
<td>5,820</td>
<td>1,974</td>
</tr>
<tr>
<td>54ECP-3</td>
<td>1L + 4H</td>
<td>CFRP debonding at invert followed by crown CFRP debonding at crown</td>
<td>1,500</td>
<td>1,500</td>
<td>0.32</td>
<td>13.4x10(^{6})</td>
<td>5,820</td>
<td>987</td>
</tr>
</tbody>
</table>

\(^{(1)}\) Tyfo SCH-41-2X; L = Longitudinal; H = Hoop
\(^{(2)}\) The debonding strains obtained from testing that are compared to analytical prediction are the underlined minimum values in columns 4 and 5.
\(^{(3)}\) \(f'_{c}\) values are the 28-day values provided by Hanson. For the 54 in. 3EB test specimens, \(f'_{c}\) was not available, and we used \(f'_{c}\) provided for the 54 in. hydrostatic test specimen.

Average 1.04
COV 27%
HYDROSTATIC PRESSURE TESTS

Hydrostatic pressure tests were performed in two phases at the Hanson plant in Palatka, Florida. Phase 1 represents the first test attempts on two specimens, one 20 ft long, 54 in. diameter ECP type PCCP and one 20 ft long, 48 in. diameter LCP type PCCP, provided by Hanson. This phase was undertaken in January-February 2012 but was not completed as planned either due to damages to the specimen during preparation or due to new failure modes that were not accounted for in the design. Phase 2 represents re-testing of the same two specimens after additional repairs, and these tests were completed in August 2012 at the same facility. Phase 1 and Phase 2 tests are presented separately in the following sections.

Phase 1 Testing

Test Specimens

The original ECP and LCP type PCCP provided by Hanson for testing were not known to be distressed or otherwise deficient before the preparation of the specimens to simulate distress. The original design pressures of the pipes are included in Table 5.4, and the complete pipe design sheets are included in Appendix A. Both specimens were lined with CFRP consisting of one longitudinal layer followed by one hoop layer of Tyfo SCH-41-2X manufactured by Fyfe Co. The number of CFRP hoop layers was limited to one, resulting in an expected ultimate pressure capacity for CFRP alone of 424 psi for the 54 in. diameter ECP and 477 psi for the 48 in. diameter LCP, so that the pressure capacity of the testing machine (approximately 873 psi for 54 in. diameter pipe; 1,100 psi for 48 in. diameter pipe) was not exceeded and the CFRP rupture failure mode could potentially occur. To study the effectiveness of CFRP termination details, the specimens had different end details. In the 54 in. diameter ECP specimen, CFRP was terminated on steel substrate (i.e., joint ring and steel cylinder) at both ends, pressed against the steel substrate using stainless steel expansion rings expanded to achieve a 100 psi radial pressure, and covered with epoxy mortar (Figure 3.11.a). In the 48 in. diameter LCP specimen, CFRP was terminated on steel substrate but no additional anchorage was provided (Figure 3.11.b). All CFRP installations were performed by Fibrwrap Construction.

Table 3.4 Hydrostatic pressure test matrix.

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Mark No.</th>
<th>Pipe Length (ft)</th>
<th>Design Pressure of Original PCCP</th>
<th>Test</th>
<th>No. of CFRP Layers$^{(1)}$</th>
<th>CFRP Termination Detail</th>
<th>Pressure Capacity of CFRP-Lined Pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>54 in. ECP</td>
<td>77</td>
<td>20</td>
<td>Pw = 25 psi, Pt = 40 psi</td>
<td>Phase 1</td>
<td>1L + 1H</td>
<td>Terminated on steel joint ring with stainless steel expansion ring</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Phase 2 (retesting) 1L + 1H + 2L</td>
<td>Terminated on steel joint ring without stainless steel expansion ring</td>
<td>151 psi</td>
<td></td>
</tr>
<tr>
<td>48 in. LCP</td>
<td>80</td>
<td>20</td>
<td>Pw = 15 psi, Pt = 30 psi</td>
<td>Phase 1</td>
<td>1L + 1H</td>
<td>Terminated on steel joint ring with stainless steel expansion ring</td>
<td>253 psi</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Phase 2 (retesting) 1L + 1H + 2L</td>
<td>Terminated on steel joint ring without stainless steel expansion ring</td>
<td>172 psi</td>
<td></td>
</tr>
</tbody>
</table>

$^{(1)}$ Tyfo SCH-41-2X; L = Longitudinal; H = Hoop
After CFRP installation, the specimens were stored outside for 60 days until the test day. On the test day, each specimen was first placed in the test setup, and then, prior to testing, they were prepared to simulate pipes with broken wire zones, and the mortar coating and the outer core (in the case of ECP) were severely cracked. For this purpose, the mortar coating, prestressing wires, and the outer core were removed completely using the same procedures described above for the 3EB specimens, in certain zones of both hydrostatic pressure test specimens, exposing the steel cylinder, while in other areas, the original pipe with prestress was left in place. The simulated degradation in the two specimens consisted of the following, staring from the bell end:

- 54 in. diameter ECP: 4 ft long exposed steel cylinder, 4 ft long original pipe, 8 ft long exposed cylinder, 4 ft long original pipe (Figure 3.12.a).
- 48 in. diameter LCP: 5 ft long original pipe, 10 ft long exposed steel cylinder, 5 ft long original pipe (Figure 3.12.b).
During this process, the steel cylinder in the 54 in. diameter ECP specimen was damaged near the bell end. A steel patch was welded externally on the steel cylinder to repair the damaged area. The 48 in. diameter LCP specimen did not have any visible sign of damage to the steel cylinder after the specimen preparation.

Figure 3.12 Hydrostatic pressure test setup
Test Setup

All hydrostatic pressure tests were conducted in the steel test frame shown in Figure 3.12. The test frame consists of two steel pillars at both ends and longitudinal steel beams that are welded to the pillars to provide longitudinal support to the specimen. The specimen, with steel bulkheads installed prior to placement into the test frame, is tightly fit into the test frame through movement of a hydraulic head at the spigot end. The specimen is loaded by first filling the pipe with water, and then by applying pressure using a pump connected to the specimen at the spigot end. The rate of loading is adjusted manually by controlling a valve on the pump.

Instrumentation

The instrumentation used in Phase 1 tests for both specimens is summarized below and shown schematically in Figure 3.13. Measurements from all instruments were recorded continuously using a National Instruments data acquisition system with LabVIEW software.

- Pressure was monitored visually using a dial gage and recorded continuously using a pressure transducer that was installed in the pipework between the pump and the test specimen (Figure 3.12.c). The dial gage had a current calibration, and the pressure transducer was pre-calibrated in the SGH laboratory prior to use in the field.
- Strain in the steel cylinder in the circumferential direction was measured using electrical resistance strain gages. For the 54 in. diameter ECP pipe, six strain gages were used: one at each springline and one at the crown of each exposed steel cylinder zone, at mid-length of each zone. For the 48 in. diameter LCP specimen, four strain gages were used: two at each springline at approximately one-quarter and three-quarters of the length of the exposed steel cylinder zone.
- Expansion of pipe at three locations along the pipe length was measured using fixtures, each made of a string potentiometer attached to a non-stretchable string wrapped around the pipe on a slippery tape.

Test Procedure

All instruments were balanced and data recording was started before the specimen was filled with water. After filling, internal pressure was increased gradually until the watertightness of the specimen was lost, and further pumping of water into the specimen did not increase the pressure, at which point the test was terminated.
Figure 3.13 Instrumentation used in hydrostatic pressure tests on CFRP-lined PCCP

(a) 54 in. diameter ECP
(b) 48 in. diameter LCP

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Results

54 in. Diameter ECP Specimen. The test had to be terminated prematurely since internal pressure could not be developed from the beginning of the test. We inspected the specimen after removal from the test setup and found that the integrity of the CFRP termination detail at the bell of the pipe had been compromised because of the attempted external weld repairs on the steel cylinder mentioned above. Since the CFRP liner was not damaged elsewhere, the specimen was scheduled for repair and retesting in Phase 2. It was later discovered during preparation for Phase 2 tests that the bulkhead had also been damaged with a large hole and that the inability to develop pressure in the Phase 1 test was partly due to the damage in the bulkhead.

48 in. Diameter LCP Specimen. The internal pressure was increased to above 200 psi without significant increase in the measured hoop strains in the steel cylinder or pipe radial expansion (Figure 3.14). At approximately 230 psi, the hoop strains and pipe radial expansion started to increase rapidly, and the steel cylinder started to exhibit bulging gradually. Further attempts to increase the pressure resulted in further bulging and significant yielding of the steel cylinder, and the test was terminated. The maximum pressure achieved was 253 psi, 5.6 times the design working plus transient pressure ($P_w + P_t = 45$ psi) of the host pipe at the time of its manufacture. The variation of hoop strains with internal pressure throughout the test is presented in Figure 3.14, and a view of the specimen at the termination of the test is shown in Figure 3.15.a.

Rapid increase in hoop strains and bulging of the steel cylinder at increased pressure was indicative of water getting behind the CFRP liner. We verified this by cutting a small hole in the middle of the exposed steel cylinder zone at the springline, and observing water discharge at the cut location (Figure 3.15,b,c). To further investigate how water bypassed the CFRP liner, we removed the specimen from the setup, and visually inspected both the interior surface of the CFRP and the CFRP terminations at the ends of the pipe. We found that transverse gaps had formed between the weaves of the CFRP hoop layer at approximately 5:00 and 7:00 o’clock positions at several locations along the length of the pipe, and water had gone through these gaps and gotten behind the CFRP liner (Figure 3.15,d,e). This was evident because water was oozing back into the pipe through these gaps during our inspection right after the test, especially in the middle 10 ft long portion of the pipe corresponding to the simulated degradation zone with exposed steel cylinder. Our inspection of the CFRP terminations at pipe ends did not indicate any signs of cracks that would provide a path for water to get behind the CFRP liner.

Our calculations after the tests showed that the transverse gaps between weaves of CFRP likely formed because of the longitudinal demand in the simulated degradation zone that was not accounted for in the design of the CFRP liner. The differential radial expansion of CFRP in the simulated degradation zone with no prestressing relative to the limited radial expansion in the adjacent zones where prestressing exists created demand in the CFRP in the longitudinal direction due to bending. As a result, we introduced a new limit state in the design process to check CFRP strains due to longitudinal bending in broken wire zones of degraded PCCP (see Chapters 7 and 8). Since the CFRP liner remained undamaged after this test, we decided to add longitudinal CFRP layers, and retest both this 48 in. diameter LCP specimen and the 54 in. diameter ECP specimen in Phase 2.
(a) Strains measured with strain gages

(b) Strains calculated from pipe radial expansion measured with potentiometers

Figure 3.14 Hoop strain in steel cylinder: 48 in. LCP Specimen, Phase 1 hydrostatic pressure testing
(a) Bulging of steel cylinder

(b) Cut made in the cylinder after test

(c) Water between cylinder and CFRP

(d) Typical locations of gaps in weaves of CFRP. Note that water is leaking back into the pipe at the indicated locations.

(e) Close-up of gaps in CFRP weaves

**Figure 3.15** Views of 48 in. LCP hydrostatic pressure test specimen after Phase 1 testing
Phase 2 Testing

Preparation of 54 in. Diameter ECP Specimen

Considering that this pipe was not pressurized in the Phase 1 test and that the CFRP liner, concrete inner core, and steel cylinder were still in contact, we performed the Phase 2 test with the objective of studying the composite action of CFRP and concrete inner core. The following repairs were performed prior to the test:

- The entire termination detail at the bell end was removed. This included removal of all CFRP and part of the concrete inner core to expose the steel substrate, repairing the puncture in the steel cylinder by patch welding on both inside and outside surfaces. The existing termination detail at the spigot end was not removed.
- Three other locations where the steel cylinder was gauged externally were repaired by patch welding.
- The surface of the existing CFRP was prepared by scuff sanding.
- In terms of CFRP installation, first the CFRP layers removed at the bell end were replaced (one longitudinal and one hoop layer) by overlapping onto the existing liner by about 12 in. Then, to increase strength against longitudinal bending in the simulated degradation zone, two additional layers of Tyfo SCH-41-2X laminate were installed in the longitudinal direction. No additional layers were installed in the hoop direction to avoid an increase in the hoop capacity of the specimen and to allow the test machine to reach high hoop strains in the CFRP liner and potentially rupture it.
- All newly installed CFRP was covered with a top coat of thickened epoxy.
- A new steel expansion ring was installed at the bell end, and the joint recess was filled with epoxy mortar.

The finished views of the CFRP surface and the detail at the bell end is shown in Figure 3.16.

The specimen was re-instrumented with new gages as in Phase 1 tests (Figure 3.13.a) except that the strain gage labeled as “Bell Left” was removed, and two additional strain gages were mounted on the steel cylinder in the longitudinal direction, 4 in. away from each end of the exposed steel cylinder (simulated degradation) zone.
Test Results for 54 in. Diameter ECP Specimen

The specimen did not exhibit any leakage up to 150 psi internal pressure, at which point an approximately 4 in. long longitudinal tear formed in the steel cylinder at the bell end above the area previously repaired by patch welding, and water started coming out (Figure 3.17.a). Further attempts to increase the pressure resulted in further leakage, and the test was terminated. The location of this tear corresponded to the location of one of the longitudinal cuts at the CFRP termination that were intentionally made on the inside of the pipe during our investigation after the Phase 1 tests, meaning that the steel cylinder had been gouged, which led to the tear in the Phase 2 test. Nevertheless, this leak indicated that water was able to get behind the CFRP liner. The maximum pressure achieved was 151 psi, 2.3 times the design working plus transient pressure ($P_w + P_t = 65$ psi) of the host pipe at the time of its manufacture, and 36% of the capacity of the CFRP liner of 424 psi.

To investigate how water bypassed the CFRP liner, we removed the specimen from the setup, and visually inspected both the interior surface of the CFRP and the CFRP terminations at the ends of the pipe. We found no cracks or gaps in the CFRP within the body of the pipe, indicating that the addition of the longitudinal layers due to the newly considered longitudinal bending limit state in Phase 2 prevented such failure. We, however, found a hairline circumferential crack debonding at the interface of the CFRP liner with the bell ring through which water was seeping during our inspection (Figure 3.17.b). We removed a portion of the termination detail by cutting through the epoxy mortar and the stainless steel expansion ring and exposed the steel cylinder (Figure 3.17.c). The surface of the steel cylinder was smooth and without any cohesive epoxy residue, indicating a lack of proper bond between the CFRP liner and the steel cylinder. We did not find a circumferential crack at the interface of the CFRP liner and the spigot ring and therefore did not perform a destructive investigation at the spigot end.

We also removed the steel cylinder in a 40 in. wide band around most of the circumference within the 8 ft long exposed steel cylinder area and inspected the surface of the
concrete inner core to evaluate the demand created in the CFRP liner and the concrete core in the hoop direction during the test (Figure 3.17.d). We did not find any longitudinal cracks, and found only one minor circumferential crack, indicating that the water bypassed the CFRP liner early in the test and rendered it ineffective due to the loss of bond at the CFRP termination at the bell end. This is also evident from the variation of hoop strains in the steel cylinder with internal pressure presented in Figure 3.18, which shows that the steel cylinder was subjected to pressure and exhibited hoop strains early in the test. The string potentiometer assemblies that were installed at three locations along the pipe did not detect any radial expansion of pipe.

![Figure 3.17 Views of 54 in. ECP hydrostatic pressure test specimen after Phase 2 testing](image)

(a) Tear in steel cylinder at bell end and resulting water leakage

(b) Water seepage through circumferential hairline crack at the CFRP/joint ring interface

(c) Smooth steel surface with no epoxy residue

(d) Cut in steel cylinder and exposed concrete inner core

Figure 3.17 Views of 54 in. ECP hydrostatic pressure test specimen after Phase 2 testing
Considering that this pipe was pressurized up to 253 psi in Phase 1 which resulted in significant radial expansion of the pipe, separation of the steel cylinder from the concrete inner core, and cracking of the concrete inner core, we performed the Phase 2 test on this specimen with the objective of studying the standalone performance of the CFRP liner. To allow the CFRP liner to behave as a standalone system, the existing hole in the steel cylinder zone that was intentionally cut after the first test attempt (Figure 3.15.b,c) was not repaired and left as-is which would exhibit leakage as soon as the water gets behind the CFRP liner. Preparation of this pipe for Phase 2 testing consisted of scuff sanding the surface of the existing CFRP, adding two layers of Tyfo SCH-41-2X laminate in the longitudinal direction, and applying a top coat of thickened epoxy. No additional layers were installed in the hoop direction to avoid an increase in the hoop capacity of the specimen and to allow the test machine to reach high hoop strains in the CFRP liner and potentially rupture it. The finished views of the CFRP surface are shown in Figure 3.19.
**Test Results for 48 in. Diameter LCP Specimen**

The specimen did not exhibit any leakage up to 150 psi internal pressure, at which point water started dripping from the existing hole in the steel cylinder. As pressure was increased, the water leakage through this hole became a steady stream, the steel cylinder exhibited sweating at more than twenty locations, further attempts to increase the pressure resulted in further leakage, and the test was terminated (Figure 3.20.a). The maximum pressure achieved was 172 psi, 3.8 times the design working plus transient pressure ($P_w + P_t = 45$ psi) of the host pipe at the time of its manufacture, and 36% of the capacity of the CFRP liner of 477 psi.

To investigate how water bypassed the CFRP liner, we removed the specimen from the setup, and visually inspected both the interior surface of the CFRP and the CFRP terminations at the ends of the pipe. We found no cracks or gaps in the CFRP within the body of the pipe, indicating that the addition of the longitudinal layers due to the newly considered longitudinal bending limit state in Phase 2 prevented such failure. We, however, found several circumferential cracks debonding around the circumference at the interface of the CFRP liner with the bell ring (Figure 3.20.b,c), similar to those found after the Phase 2 testing of the 54 in. ECP specimen. The width of the circumferential cracks ranged from 2 mils to 6 mils. We did not find such a circumferential crack at the spigot end of the pipe, which is also consistent with our observations after the Phase 2 testing of the 54 in. ECP specimen.

We removed the steel cylinder in a 30 in. x 40 in. area in the middle of the 10 ft long exposed steel cylinder area and inspected the surface of the concrete inner core to evaluate the demand created in the CFRP liner and the concrete core in the hoop direction during the test (Figure 3.20.c,d). In this exposed concrete area, we found three longitudinal cracks spaced at 10 in. and 16 in., indicating that the CFRP liner and concrete inner core expanded together up to a certain pressure, and the water bypassed the CFRP liner later in the test due to the loss of bond at the CFRP termination at the bell end. This is also evident from the variation of hoop strains in the steel cylinder with internal pressure presented in Figure 3.21, which shows that the steel cylinder did not expand significantly until 100 psi to 150 psi, which is likely the pressure range in which water bypassed the CFRP liner. Hoop strains calculated from the pipe radial expansion data obtained from the string potentiometer assemblies indicated that the prestressed zone at the
spigot end of the pipe underwent a maximum hoop strain of approximately 570 με as longitudinal cracks formed in this zone (Figure 3.21.b).

Note that the steel substrates at the CFRP terminations in these specimens were prepared by simply cleaning of all dust and debris but not ground to expose white metal prior to CFRP installation.

Summary and Conclusions of Hydrostatic Pressure Tests

The following is a summary of our Phase 1 test on the 48 in. LCP specimen and Phase 2 tests on both 48 in. LCP and 54 in. ECP specimens, and our conclusions based on these tests:

- We conducted full scale hydrostatic pressure tests on both ECP and LCP type PCCP.
- We identified failure modes that had not been included in the CFRP liner design approach.
- We determined issues with watertightness of CFRP-lined distressed PCCP and how the CFRP termination details can be made effective to prevent pressure build-up behind the CFRP liner and provide watertightness.
- The differential radial expansion of the CFRP liner in the broken wire zones and in prestressed areas creates longitudinal bending stresses in the CFRP liner. Such a failure mode had not been considered as possible design criterion prior to this testing. At high pressures, this differential radial expansion causes circumferential cracks in the CFRP liner, allowing the water to pass through the laminate. As a result of these tests, we have introduced a new limit state in our design approach regarding longitudinal bending of the CFRP liner resulting from differential radial expansion of the CFRP liner.
- Even when the failure modes related to strength are prevented, the success of the CFRP lining of distressed PCCP is governed by the effectiveness of the CFRP termination detail in providing watertightness. The bond between the CFRP liner and the steel substrate at the CFRP termination at pipe ends is critical, and this bond should not be compromised.
- To demonstrate that the calculated capacity of the CFRP liner in a distressed PCCP can be reached in a hydrostatic pressure test, we recommend that confirmation tests of CFRP liner repair design of distressed PCCP be performed with appropriate surface preparation of steel substrate to near white surface condition. The pressure in the repaired pipe should be increased to the calculated capacity of the CFRP liner.
Leakage through pre-existing hole in the steel cylinder

Feeler gage inserted into the circumferential crack at the CFRP liner/bell ring interface

Cut in steel cylinder and exposed concrete inner core with longitudinal cracks

Spacing of longitudinal cracks in the concrete inner core

Figure 3.20 Views of 48 in. LCP hydrostatic pressure test specimen after Phase 2 testing
(a) Steel cylinder strain measured with strain gage

(b) Strain calculated from the measured pipe radial expansion in the prestressed zone at spigot end

Figure 3.21 Hoop strains measured in 48 in. LCP specimen, Phase 2 hydrostatic pressure testing
CHAPTER 4  
BUCKLING OF CFRP LINER IN PCCP AFTER LOSS OF PRESTRESS AND CORE CRACKING

ANALYSIS APPROACH

To determine the buckling resistance of a CFRP liner in PCCP that has been subjected to gravity loads and internal pressure, and then dewatered, we added a load step, Step 12, to the finite element analysis (FEA) described in Chapter 3 (Figure 3.6). In this additional load step, the pipe in its deformed state after application of gravity and internal pressures and after removal of internal pressure was subjected to a gradually increasing in magnitude negative internal pressure. With increasing negative pressure, the deformed pipe developed a buckle at the crown, as shown in Figure 4.1. The magnitude of the buckle increased with the magnitude of negative pressure until it reached a plateau, as shown in Figure 4.2. The buckling strength is defined as the negative pressure that causes unstable buckling of the CFRP liner.

Figure 4.1 Buckling mode shape (true scale) and displacement contour (in.) for Case 7
Figure 4.2 Load deflection curves during the application of negative pressure to determine the buckling strength of the CFRP liner.

The applied internal negative pressure in the FEA model represents the sum of internal negative pressure (i.e., vacuum from a transient pressure condition) and external pressure from ground water. It does not include the effect of external pressure from the earth load, because the CFRP liner was in the deformed and stressed state from gravity loads and the CFRP liner wall was under compression prior to the application of internal negative pressure.

This analysis was performed on five of the ten FEA models presented in Chapter 5 (Table 5.3), namely Cases 4, 6, 7, 8, and 10, to account for varying pipe diameters, number of CFRP layers, and constrained soil moduli as presented in Table 4.1.

RESULTS

The typical buckling mode of the CFRP liner for the five analysis cases considered is shown in Figure 4.1. The buckling resistance obtained from FEA is compared to those obtained from closed-form solutions of Glock (1977) and Moore and Selig (1990) in Table 4.1. Glock’s formulation represents buckling of a flexible liner in a rigid cavity (i.e., constrained buckling with soil modulus assumed to be rigid, and Moore and Selig’s formulation represents buckling of a buried flexible liner in an elastic soil of known modulus. The basis of Glock (1977) and Moore and Selig (1990) formulations are discussed in Chapter 2, and closed-form solutions are shown in Equations 2-32 and 2-27, respectively.
The results show that the Moore and Selig (1990) buckling equation significantly overestimates the true buckling strength of the CFRP liner and should not be used.

Direct comparison of buckling strength obtained from FEA with the results obtained from Glock’s (1977) equation cannot be made meaningfully because the CFRP liner is stressed and deformed from the earth load prior to application of internal negative pressure. Using the procedure in the draft AWWA Standard for CFRP Renewal and Strengthening of CFRP, we calculated the external pressure corresponding to the earth load without load factor (i.e., 7 psi for Case 4; 14.5 psi for Case 6 and 7; 8.3 psi for Case 8; and 11.6 psi for Case 10), and added this pressure to the calculated buckling pressure obtained from FEA. We then calculated the ratio of the sum of earth load pressure and buckling pressure obtained from FEA to the buckling strength obtained from Glock’s equation (see last column of Table 4.1). The results indicate that the buckling of the CFRP liner may be predicted accurately using Glock’s formula with a bias factor of 0.75 that account for the flexibility of soil compared to the rigid constraint assumed by Glock. The bias appears to be independent of the constrained soil modulus and pipe diameter in the range of 1,500 psi to 5,000 psi.

Moore and Selig’s (1990) formula over-predicts the buckling resistance significantly and should not be used for design of CFRP liners for PCCP.

Table 4.1
Comparison of buckling resistance obtained from FEA with closed-form solutions of Glock (1977) and Moore and Selig (1990)

<table>
<thead>
<tr>
<th>Analysis Case (See Table 3.3)</th>
<th>Pipe Diam. and Type</th>
<th>Cover Height (ft)</th>
<th>CFRP Liner Cons. Soil Modulus (psi)</th>
<th>Buckling Resistance (psi)</th>
<th>Glock/FEA Ratio</th>
<th>FEA + Earth Load/Glock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 4</td>
<td>60 in. ECP</td>
<td>6</td>
<td>1L + 3H</td>
<td>3,000</td>
<td>36</td>
<td>56</td>
</tr>
<tr>
<td>Case 6</td>
<td>60 in. ECP</td>
<td>15</td>
<td>1L + 4H</td>
<td>1,500</td>
<td>57</td>
<td>102</td>
</tr>
<tr>
<td>Case 7</td>
<td>60 in. ECP</td>
<td>15</td>
<td>1L + 4H</td>
<td>5,000</td>
<td>66</td>
<td>107</td>
</tr>
<tr>
<td>Case 8</td>
<td>120 in. ECP</td>
<td>6</td>
<td>2L + 6H</td>
<td>3,000</td>
<td>40</td>
<td>59</td>
</tr>
<tr>
<td>Case 10</td>
<td>120 in. ECP</td>
<td>10</td>
<td>2L + 7H</td>
<td>5,000</td>
<td>48</td>
<td>83</td>
</tr>
</tbody>
</table>

L = Longitudinal; H = Hoop

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CHAPTER 5
FINITE ELEMENT MODELING OF FAILURE PROCESS OF CFRP-LINED PCCP

INTRODUCTION

In this chapter we will present the results of finite-element model of the pipe-soil system subjected to gravity and internal pressure. The results of these finite-element models are used to validate the design equations of Chapter 7.

Purpose

The purpose of this chapter is to perform finite element analyses (FEA) of typical CFRP lined PCCP as it continues to degrade with time after CFRP liner installation, and to use the results of FEA on loads, deformations and strains to validate the design approach proposed in the draft AWWA Standard for Renewal and Strengthening of PCCP.

Scope

The scope of this chapter is to present the following:

• Details of the FE modeling of the buried PCCP with CFRP line.
• Simulation of the degradation process of the PCCP with time after the installation of CFRP liner.
• Calculation of state of deformation and stress/strain in CFRP when the repaired pipe is subjected to the design gravity loads and working and transient pressures after degradation of the PCCP for a sample of ten pipes selected to represent different PCCP types, and a range of PCCP pipe diameters, design cover height, design pressures, and soil moduli.
• Comparison of the results obtained from FE modeling with the results from the design requirements of draft AWWA Standard for Renewal and strengthening of PCCP, as shown in Table 5.1.

ANALYSIS PLAN

Variable Input Parameters

Considering the number of design variables that are involved in the design of a CFRP liner for a PCCP and covers the entire range of geometries and properties of the PCCP, CFRP, soil, and load combination, it is neither feasible nor practical to perform analysis of all possible sets of design conditions. Therefore, we have selected ten PCCPs for analysis and for each PCCP we have developed a CFRP design based on the draft AWWA standard approach as discussed in Chapter 7. The range of design parameters that are varied in the analyses are made wide enough to verify the use of the developed design approach for both LCP- and ECP-type PCCP. The important parameters we considered and attempted to vary in our analyses are listed in Table 5.2.
Based on these variables, we developed an analysis plan that consists of ten cases as listed in Table 5.3. In each case, the host pipe is first designed using the UDP software to ensure consistency of the host-pipe properties (e.g., amount of prestress, concrete and steel stresses) for the design loads. The CFRP liner is then designed using the current design approach of the draft AWWA Standard. Range of properties of the PCCP design selected for FEA are shown in Table 5.2. With the range shown in Table 5.2, we selected ten cases for FEA. The concrete compressive strength is assumed at 6,000 psi for all LCP and 5,000 psi. The depth of soil below the pipe is assumed to be one pipe diameter in all cases.

To automatize the generation of analysis models for each case and to allow rapid variation of selected parameters throughout the project as needed, we have developed a computer script that creates an ABAQUS input file based on the parameters defined in Table 5.3. The script also allows rapid addition of other analysis variables should we desire to do so. The FEA analysis steps and output are shown in Table 3.4.

Table 5.1
FEA objectives

<table>
<thead>
<tr>
<th>Item</th>
<th>Relevant Equation in Reference Standard</th>
<th>Question</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP Strain due to Internal Pressure</td>
<td>$\varepsilon_a = \frac{P_r a}{E_f t_f}$</td>
<td>How accurate or conservative is this equation for a pipe that continues to degrade after CFRP installation?</td>
</tr>
<tr>
<td>Earth Load on CFRP Liner</td>
<td>Weight of columns of soil above the pipe</td>
<td>Is the earth load determined from the reference standard accurate/conservative?</td>
</tr>
<tr>
<td>Pipe Deflection</td>
<td>$\Delta = \frac{W_c K_b}{0.061 M_s + \frac{EI}{r^3}}$</td>
<td>How accurate or conservative are these equations and how accurately we can define the shape factor, $\eta$?</td>
</tr>
<tr>
<td>Bending Strain due to Pipe Deflection</td>
<td>$\varepsilon_b = \eta \left( \frac{\Delta}{D} \right) \left( \frac{t_f}{D} \right)$</td>
<td>How accurate or conservative is this equation and how accurately we can define the rerounding factor, $R_c$?</td>
</tr>
<tr>
<td>Strain due to Combined Pressure and Bending</td>
<td>$\varepsilon = \varepsilon_a + R_c \varepsilon_b$</td>
<td></td>
</tr>
</tbody>
</table>
Table 5.2
Range of variables used in FEA

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe Diameter, $D_i$</td>
<td>36 in., 54 in.</td>
</tr>
<tr>
<td>Steel Cylinder Outside Diameter, $D_y$</td>
<td>40.5 in., 60.75 in.</td>
</tr>
<tr>
<td>Pipe-Core Thickness, $h_c$</td>
<td>$D_i/16$</td>
</tr>
<tr>
<td>Steel Cylinder Thickness, $t_y$</td>
<td>16 ga (0.0598 in.)</td>
</tr>
<tr>
<td>Working Pressure, $P_w$</td>
<td>100 psi, 150 psi</td>
</tr>
<tr>
<td>Transient Pressure, $P_t$</td>
<td>$0.5P_w$</td>
</tr>
<tr>
<td>Soil-Cover Height, $H$</td>
<td>6 ft, 15 ft</td>
</tr>
<tr>
<td>Height of Groundwater above Top of Pipe, $H$</td>
<td>Up to ground surface</td>
</tr>
<tr>
<td>Constrained Soil Modulus, $M_s$</td>
<td>3,000 psi</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>LCP</td>
</tr>
<tr>
<td></td>
<td>60 in., 120 in.</td>
</tr>
<tr>
<td></td>
<td>63 in., 126 in.</td>
</tr>
<tr>
<td></td>
<td>$D_i/16$</td>
</tr>
<tr>
<td></td>
<td>14 ga (0.0747 in.) for 120 in. diameter,</td>
</tr>
<tr>
<td></td>
<td>16 ga (0.0598 in.) otherwise</td>
</tr>
<tr>
<td></td>
<td>100 psi, 180 psi</td>
</tr>
<tr>
<td></td>
<td>max(0.4$P_w$, 40 psi)</td>
</tr>
<tr>
<td></td>
<td>6 ft, 15 ft</td>
</tr>
<tr>
<td></td>
<td>Up to ground surface</td>
</tr>
<tr>
<td></td>
<td>1,500 psi, 3,000 psi, and 5,000 psi</td>
</tr>
<tr>
<td>Parameters</td>
<td>Range</td>
</tr>
<tr>
<td>------------------------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>Lower</td>
<td>Upper</td>
</tr>
<tr>
<td>Pipe Type</td>
<td>LCP</td>
</tr>
<tr>
<td>Pipe Diameter, D, (in.)</td>
<td>16</td>
</tr>
<tr>
<td>Concrete Core Thickness, h, (in.)</td>
<td>1.000</td>
</tr>
<tr>
<td>Concrete Strength, f, (ksi)</td>
<td>5.000</td>
</tr>
<tr>
<td>Steel Cylinder Outside Diameter, D, (in.)</td>
<td>18</td>
</tr>
<tr>
<td>Steel Cylinder Thickness, t, (in.)</td>
<td>0.0598</td>
</tr>
<tr>
<td>Steel Cylinder Modulus, E, (ksi)</td>
<td>30,000</td>
</tr>
<tr>
<td>Steel Cylinder Strength, f, (ksi)</td>
<td>33,000</td>
</tr>
<tr>
<td>Wire Diameter (in.)</td>
<td>0.192</td>
</tr>
<tr>
<td>Prestress in Wires, f, (psi)</td>
<td>UDP</td>
</tr>
<tr>
<td>Prestress in Concrete Core, f, (ksi)</td>
<td>UDP</td>
</tr>
<tr>
<td>Prestress in Steel Cylinder, f, (psi)</td>
<td>UDP</td>
</tr>
<tr>
<td>No. of CFRP Hoop Layers</td>
<td>2</td>
</tr>
<tr>
<td>CFRP Thickness- HOOP, tCFRP, (in.)</td>
<td>CFRP</td>
</tr>
<tr>
<td>No. of CFRP Longitudinal Layers</td>
<td>1</td>
</tr>
<tr>
<td>CFRP Thickness-LONGITUDINAL, tCFRP, (in.)</td>
<td>0.08</td>
</tr>
<tr>
<td>CFRP Modulus, Circumferential, ECFRP, (ksi)</td>
<td>10,000</td>
</tr>
<tr>
<td>CFRP Strength, Circumferential, fCFRP, (ksi)</td>
<td>90</td>
</tr>
<tr>
<td>Soil Cover Depth, H, (ft)</td>
<td>3</td>
</tr>
<tr>
<td>Soil Modulus (Constrained), M, (psi)</td>
<td>1,500</td>
</tr>
<tr>
<td>Soil Density, g, (pcf)</td>
<td>100</td>
</tr>
<tr>
<td>Height of Ground Water, Hw, (ft)</td>
<td>3</td>
</tr>
<tr>
<td>Minimum Radial Bond Strength, f, (psi)</td>
<td>50</td>
</tr>
<tr>
<td>Working Pressure, P, (psi)</td>
<td>30</td>
</tr>
<tr>
<td>Transient Pressure, P, (psi)</td>
<td>12</td>
</tr>
<tr>
<td>Traffic Load (HS20), Wt(lbf/ft)</td>
<td>532</td>
</tr>
</tbody>
</table>
### Table 5.4
FEA output analyzed for cases of CFRP-renewed PCCP

<table>
<thead>
<tr>
<th>Analysis Steps</th>
<th>Relevant Question</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Install pipe and soil; apply gravity loads.</td>
<td>a) Load on pipe</td>
</tr>
<tr>
<td>2. Install CFRP.</td>
<td>–</td>
</tr>
</tbody>
</table>
| 3. Degrade pipe by removing prestress, outer concrete core, and steel cylinder. | a) Load on pipe  
b) Vertical and horizontal pipe deflections  
c) Circumferential strain in CFRP  
d) Radial tension at CFRP-core interface  
e) Compressive stress in inner core  
f) Shape factor, $\eta$  
g) Deflection lag factor, $D_L$  
h) Circumferential shear at CFRP-core interface |
| 4. Apply water weight and working pressure. | (a) through (g) above plus rerounding coefficient $R_c$ |
| 5. Apply transient pressure. | (a) through (g) above plus rerounding coefficient $R_c$ |
| 6. Remove transient pressure. | – |
| 7. Apply transient live load. | (a) through (g) above plus rerounding coefficient $R_c$ |
| 8. Remove transient live load. | – |
| 9. Remove water weight and working pressure. | (a) through (h) above |

**FINITE ELEMENT MODEL DETAILS**

We performed a set of nonlinear finite element analyses (NL-FEA) to investigate the failure mode of the sample pipes tabulated in Table 5.3. The results of the NL-FEA are used to verify the design procedure recommended in the draft AWWA Standard for PCCP renewal using CFRP. The FE models were developed using ABAQUS, version 6.11 (Dassault Systèmes Simulia Corp., 2006).

We modeled the buried pipe considering a plane strain condition using a two-dimensional finite element model. Due to the symmetric nature of the pipe cross section and loading conditions, only half of the pipe was modeled as shown in Figure 5.1. Symmetrical boundary conditions are applied at the axis of symmetry. To track the pipe behavior over the entire loading range and to replicate pipe performance under service and extreme load conditions, the FEA must have the nonlinear material constitutive properties and must account for geometric nonlinearity due to large deflections. Details of the material models, element choices, and boundary conditions are discussed below.
Material Models and Element Choices

The buried pipe model comprise of following components that are individually modeled: the PCCP components include concrete inner core, steel cylinder and concrete outer core; the CFRP liner is applied to the pipe inside surface; and the soil is surrounds the pipe. Specifically,

- Steel cylinder is represented with beam elements. Steel is assumed to have elastic-plastic stress-strain behavior with von Mises yield criterion.
- Concrete inner core and outer core are represented with plain strain elements. The failure mechanism in the concrete was assumed to be either cracking in tension or crushing in compression. Damage plasticity with smeared cracking model is used to account for tensile softening and compression failure in concrete.
- CFRP liner is also represented with beam elements. CFRP is assumed to be linearly elastic.
- Soil is modeled as elastic with Mohr-coulomb failure criteria.

Modeling of CFRP and Steel Cylinder Bond With Concrete

Modeling of the bond between the concrete inner core and steel cylinder and that between the concrete inner core and CFRP liner require particular attention. We designed a bundle of four connector elements, C1, C2, C3, and C4, as shown in Figure 5.2 to model the bond at these interfaces.

C1 and C2 are parallel to each other and are connected to C3 and C4 in series. The connector C1 defines the debonding against radial tension and shear. The connector C2 is a gap connector which allows gaps to form and surfaces to separate and prevents negative gap from
forming. The connector C3 is an elastic spring and represents the transverse stiffness of the interface. The connectors C1, C2, and C3 are placed at the concrete face of the interface, and together are connected to cylinder or CFRP liner with a rigid link shown as connector C4. Three hundred and sixty bundles of four connector elements are equally spaced from the crown to the invert at each of the two interfaces.

*Figure 5.2 Illustration of CFRP-to-concrete and steel cylinder-to-concrete interface modeling.*

**MODELING OF THREE-EDGE BEARING TESTS**

We developed NL-FE models of the 54 in. diameter ECP and 48 in. diameter LCP test pipe specimens as describe in the previous part of this chapter and subjected the pipe model to three-edge bearing loads, simulating the test that was conducted in January-February 2012. The details of all test specimens along with the properties of the PCCP constituents and the CFRP liners installed in each pipe are presented in details in Chapter 3. The CFRP liners in all specimens consisted of one longitudinal layer, and one or three hoop layers in the 48 in. diameter LCP specimens and one or four hoop layers in the 54 in. ECP specimens.

After the tests, the results of the FE models were compared with the observed behavior of the test specimens. The discrepancies between the FE model results and test results were analyzed and were either corrected or were taken into consideration in the interpretation of test results. Then the model was used in the analysis of the buried pipes listed in Table 5.3, using a finite-element model of soil-pipe system. In these models the pipe and CFRP were modeled in the same manner as the FE model used for the analysis of the test pipes, though with the geometries and properties of the pipes as listed in Table 5.3.

**Material Models and Element Choices**

Each test specimen consists of three structural components: (1) steel cylinder located outside of the test specimen, (2) concrete inner core located inside of the steel cylinder, and (3) the CFRP liner bonded to the inside surface of the inner core, as illustrated in Figure 5.3. The mortar coating, prestressing wires, and the concrete outer core were removed from all specimens prior to the installation of CFRP to simulate severe degradation with complete loss of prestress.
The finite-element model of test pipe is identical to that used for modeling of the ten cases discussed above with the exception that soil elements are not included and the pipe is subjected to gravity load and a three-edge bearing load applied as concentrated load at the top of the pipe. The pipe is supported by a simple support located at 0.5 in. per foot of diameter from the bottom of the pipe, in accordance with the requirements of ASTM C497-05 (2005) Standard Test Methods for Concrete Pipe, Manhole Sections, or Tile. We modeled only one-half of each test pipe and imposed symmetry boundary conditions about the vertical axis at invert and crown of the steel cylinder, concrete inner core, and the CFRP liner.

Simulation of Test Procedure

To simulate the test procedure, each pipe specimen is subjected to two load steps. In the first load step, gravity is applied. In the second load step, an additional concentrated test load is applied at the top of the pipe, outside of the steel cylinder. The three-edge bearing test load is then increased gradually in small increments until the pipe fails and cannot take any additional load.

This analysis was performed for four test cases:

1. 54 in. ECP with one hoop layer of CFRP
2. 54 in. ECP with four hoop layers of CFRP
3. 48 in. LCP with one hoop layer of CFRP
4. 48 in. LCP with three hoop layers of CFRP
Comparison of FE Model Results With Test Results

After the full scale three-edge bearing tests, we revised our FE models based on the concrete material properties of test specimens provided by Hanson and debonding strength obtained from direct pull-off test per ASTM D4541.

We compared the results of our FEA with test results and with hand calculated radial tension between the CFRP and concrete core. The results of comparison of FEA with test results show essentially two major discrepancies: (1) the extent of visible cracking in the concrete core predicted by FEA was not observed in the test, and (2) the radial tension delamination of CFRP did not occur gradually as predicted by the FE model, but explosively.

To examine the sensitivity of the predicted radial tension failure between CFRP and concrete core by the FE model with the tensile bond strength and concrete strength, we performed a parametric study in which we changed these two parameters. Figure 5.5 plots the results of this sensitivity study for the 48 in. diameter specimen with three CFRP hoop layers. The legend FEM_XX_YYYY_ZZZ in the figure designates the result of FEM with a run ID of XX, a concrete compressive strength of YYYY and radial tension bonding strength between CFRP and concrete core of ZZZ. The analysis indicates that pipe behavior is very sensitive to both the concrete strength and the radial tension bond strength. FEM_10_6500_600 agree reasonably well with the test results.

We also hand calculated the radial tension between CFRP liner and concrete core from the results of three-edge bearing tests. For the 48 in. LCP with three layers of CFRP in the hoop direction, the debonding load was at 60,000 lbf (Figure 5.5). The bending moment at the crown is \( M = 0.318 \ P_{3EB} \ R \) where \( P_{3EB} \) is the three-edge bearing load in units of LBF and \( R \) is the radius in inches to the neutral axis of the composite CFRP and concrete core and is approximately 25.14 in., resulting in a moment of 480,000 lbf-in., or for a 48 in. long test specimen, 10,000 lbf-in./in. Noting that the radial tensile failure at CFRP liner interface with concrete core was cohesive, it is reasonable to expect that the radial tensile strength was close to the tensile strength of concrete, or for the LCP with compressive strength of about 7,000 psi, not much less than 600 psi. With a radial tensile stress of \( \sigma_{RT} = 600 \) psi, radius \( R = 24 \) in., three CFRP laminae with n.t = 0.24, the strain in CFRP must be \( \varepsilon_{CFRP} = \sigma_{RT} * R/(E_{CFRP} \cdot nt) = 0.004 \). Measured strains in the CFRP never reached such a high value, therefore, debonding was not due to radial tension, but shear (see Table 3.2).
Figure 5.4 FEM capability to simulate concrete cracking and CFRP debonding

Figure 5.5 FEA-based sensitivity study
MODELING DEGRADATION PROCESS OF CFRP-LINED BURIED PCCP

Finite element analyses were performed to simulate the structural response of an undistressed PCCP lined with CFRP, buried in soil and subjected to all gravity loads, as the PCCP degrades with time. The degradation is simulated by loss of prestress, cracking of the outer core, and corrosion of steel cylinder. The flexibility of the distressed PCCP results in redistribution of gravity loads. The distressed pipe is then subjected to working and transient pressures.

Analysis Sequence

The steps used in each pipe analysis are summarized in Figure 5.6. It is designed to simulate the effect of degradation processes that are likely to be experienced by a buried PCCP with CFRP liner applied early in the degradation process during its service life. As shown in Figure 5.6, pipes are subjected to 12 load steps: steps (1) through (5) simulate pipe installation and CFRP liner application; steps (6) through (8) characterize the aging process of PCCP through complete of loss of the prestressing, cracking and loss of stiffness of the outer core (for ECP) and corrosion of steel cylinder, while gravity load is maintained on the pipe. Step (9) through (12) simulate the effect of application and removal of working pressure and working plus transient pressures.

![Figure 5.6 Illustration of load steps.](image-url)
Interpretation of FEA Results

In this section we will present representative results of our FE analysis of the buried PCCP with CFRP liner as the PCCP continues to degrade after installation of the liner under the action of gravity and design working and transient pressure conditions.

Figure 5.7 plots the thrust in pipe for Case 4 at the springline and the portion of load carried by soil. The results shows the thrust from prestress in steps 1 and 2, addition of earth load in Step 3, addition of CFRP in Step 4 and 5 and removal of prestress, cracking of the outer core, and corrosion of steel cylinder in Steps 6, 7, and 8. At the end of Step 8, the degraded PCCP with CFRP liner is subjected to gravity loads and the thrust at springline from earth load is estimated at 180 lbf/in. The thrust at springline becomes tensile as working and transient pressures are introduced in Steps 9 and 10, which causes severe cracking of the core. These pressures are removed from the pipe in Steps 11 and 12.

![Graph showing thrust and load distribution](image)

Figure 5.7 Thrust at springline of 60 in. diameter PCCP lined with CFRP, equal to one-half of the gravity load on the pipe (Case 4: 60 in. ECP); also shown is the load carried by the soil

The change in pipe diameter in the horizontal and vertical directions is shown in Figure 5.8. The deflection of the pipe after degradation of the PCCP is 0.10 in. horizontally and -0.08 in. vertically. This calculated horizontal deflection is noticeably less than the horizontal deflection of 0.17 in. calculated from the draft reference AWWA standard. This comparison shows that the CFRP liner adhered to the inner core is only subjected to the 60 percent of the load applied to PCCP. A part of the load on the PCCP is transferred to the surrounding soil. As pressure is increased to $P_w$ and later to $P_w + P_t$, the horizontal deflection increases (not expected compared to the behavior of flexible buried pipe) and the vertical deflection reduces, i.e., the pipe tends to reround near invert and crown.
Figure 5.8 Pipe diameter change at each load step (Case 4: 60 in. ECP)

Figure 5.9 shows the change in the shape of the liner (case 4) under different loading. It is clear that the CFRP liner, fully or partially bonded to the inner concrete core, does not maintain a perfect elliptical shape throughout the analysis. The shape of the liner is largely dependent on the condition of the concrete core and the bonding strength between the CFRP and the concrete.

Figures 5.10 (a)-(d) plot circumferential stress in CFRP liner for Case 4 along pipe circumference at load steps 8, 9, 10, and 12. The horizontal axis is the circumferential location of the stress in liner expressed in terms of the circumferential angle from the invert. Figure 5.10(a) plots CFRP liner stress under pure bending due to gravity load alone at the end of Step 8. Figure 5.10(b) and (c) show the stresses after the application of working and working plus transient pressure conditions at the ends of Step 9 and 10. The spikes in the CFRP stress are caused by local cracks in the concrete inner core. Comparison of the results of FEA model with three-edge bearing test results showed that cracking of the concrete core did not occur to the extent predicted by the FE model, although concrete may have extensively softened.\(^1\) The absence of

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\(^1\) The extent of cracking observed in FEA is related to the modeling of concrete elements. The concrete elements used in this analysis is ABAQUS Concrete Damage Plasticity with tension softening defined by a bilinear stress-strain diagram consisting of an initial concrete stiffness portion followed by a descending branch representing softening of cracked concrete. This model results in strain localization at cracks and ignores the fracture energy release rate. Using ABAQUS fracture energy cracking criterion in which fracture energy is inversely proportional to the element length reduced the number of cracks in three-edge bearing test model significantly but did not improve the results for the test runs of buried CFRP lined PCCP subjected to combined loads.
cracks in the tests and softening of concrete results in the CFRP liner to behave more as a standalone pipe, without the effect of additional strain caused by cracking of the inner concrete core. The stresses in the CFRP liner after removal of pressure at the end of step 12 are shown in Figure 5.10(d). This figure shows that the CFRP liner is deboned from the inner core. Note that the stresses at the end of Step 12 are significantly different from those computed at the end of step 8. This change is a result of damages imparted to the pipe, i.e., concrete cracking and CFRP debonding, as pressure is increased.

![Diagram showing pipe shape change](image)

**Figure 5.9** Pipe shape change at the end of Steps 8, 9, 10, 11 and 12 (Case 4: 60 in ECP)
(a) Stress in CFRP liner under gravity loads (Step 8)

(b) Stress in CFRP liner under gravity loads and 100 psi working pressure (Step 9)

(c) Stress in CFRP liner under gravity loads, 100 psi working and 40 psi transient pressure (Step 10)
To address the objectives tabulated in Table 5.1, for each case, we calculated: the earth load acting on the CFRP liner after degradation of PCCP (end of Step 8); pipe deflection, and the stress/strain generated in the pipe wall at the ends of steps 8, 9 and 10; and the rerounding effect of internal pressure, i.e., the reduction in bending stress/strain in the CFRP liner as pressure is applied to the deformed liner. These results are summarized in Figures 5.11(a)-(j). We can make the following observations for our review of the results for the 10 cases analyzed.

Comparison of FEA Results With AWWA Draft Standard

Tables 5.5.1 through 5.5.10 show the results of the comparison of the FEA results with the design equations and assumption of AWWA draft standard. The following observations are made:

1. **Earth load** on pipe calculated from FEA as the sum total of all stresses at the springline of the CFRP renewed PCCP is basically consistent with the prism soil column method used in the design equations.

2. **Pipe deflection under gravity loads.** FEA shows consistently smaller deflections than those calculated using the design equation. The reason for this is the higher stiffness of the CFRP renewed PCCP as concrete in the inner core carries some of gravity load, and CFRP liner is not subjected to the entire earth load and hence demonstrates reduced deflection. Moreover, when CFRP liner is installed, earth load is completely resisted by the PCCP and the newly applied liner is at stress-free. During the subsequent degradation process when pipe is losing its steel cylinder and concrete outer core, portion of the earth load is redistributed onto the CFRP liner. The load redistribution process is simulated in FEA, but the gravity load carried by the liner remains less than the earth load predicted from the design equations. Consequently, the design equations for standalone CFRP liner that are based on it carrying the entire earth load provide conservative results.
The FE model shows significant axial tensile stress developing in the liner resulting from the loss of prestress of PCCP, and the resulting expansion of the inner core and CFRP adhered to it. This tensile stress is not currently included in the design of CFRP liner. The bending strains resulting from pipe deflection in the FE model are proportionally smaller than the strain calculated from design equations, because as discussed above, the CFRP liner will not be subjected to the entire earth load.

3. **Strain in CFRP liner under combined earth load and internal pressure.** The FEA calculated strains have two components: (1) the basic strain in the CFRP laminate, resulting from bending and axial force in the CFRP laminate, and (2) a strain perturbation that occurs only at cracks in the concrete core and peaks over a one or two elements with a total length much smaller than the aggregate size or the size of fracture process zone associated with visible cracks. The second component is to a large extent a by-product of the FE modeling and the stiffness of concrete adjacent to core cracks. To get a better estimate of strains near the cracks, we have calculated the running average over a distance of 1.5 in., believed to be approximately equal to the size of the fracture process zone for concrete with ¾ in. maximum aggregate size. The maximum values of the running averages thus computed along with the maximum value of the basic strains are reported in Tables 5.5.1 through 5.5.10. The results show the following:

1. The maximum basic strain is on the average 72% (ranging from 49% to 102%) of the maximum strain computed from the design equations for the combined gravity load and design working pressure and 86% (ranging from 68% to 106%) of the maximum strain computed from the design equations for the combined gravity loads and design working plus transient pressures.

2. The maximum value of the running average strain is on the average 108% (ranging from 80% to 134%) of the maximum strain computed from the design equations for the combined gravity load and design working pressure and 111% (ranging from 90% to 128%) of the maximum strain computed from the design equations for the combined gravity loads and design working plus transient pressures.

With extensive cracking of the inner core as predicted by the FEA, the concrete is expected to be softened to an extent that strain/stress perturbations that increase the stress in the CFRP liner significantly cannot exist. Furthermore, the extent of cracking predicted by the FEA was not observed in the three-edge bearing tests. Therefore, the basis of comparison of FEA strain under combined loads should be the basic strains values. Furthermore, even if the maximum stresses/strains calculated from the FE model were to occur, the load factors are sufficiently high to prevent rupture of CFRP liner.

One should note that the results obtained from FEA show that the axial strains from pressure are by far greater than those produced by bending. A by-product of small bending stress is that rerounding equations do not play a major role in the design. Furthermore, large bending generated at crack widths do not allow the FE model to independently validate the design equation for rerounding and reduction of bending stresses from pressure application.
Table 5.5.1
Analysis summary for Case 1 – 36 in. LCP
\( (H = 6 \text{ ft}, P_w = 100 \text{ psi}, P_t = 50 \text{ psi}, M_s = 3,000 \text{ psi}) \)

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-124</td>
<td>-131</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td></td>
<td>Earth load and pipe weight</td>
<td>0.03</td>
<td>0.05</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>199</td>
<td>408</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and</td>
<td>920 (1,544)</td>
<td>1,243</td>
<td></td>
</tr>
<tr>
<td></td>
<td>working pressure</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and</td>
<td>1450 (2,125)</td>
<td>1,667</td>
<td></td>
</tr>
<tr>
<td></td>
<td>working and transient pressure</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.5.2
Analysis summary for Case 2 - 54 in. LCP
\( (H = 6 \text{ ft}, P_w = 100 \text{ psi}, P_t = 50 \text{ psi}, M_s = 3,000 \text{ psi}) \)

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-200</td>
<td>-208</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td></td>
<td>Earth load and pipe weight</td>
<td>0.08</td>
<td>0.08</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>213</td>
<td>428</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and</td>
<td>920 (1,392)</td>
<td>1,260</td>
<td></td>
</tr>
<tr>
<td></td>
<td>working pressure</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and</td>
<td>1,450 (2,006)</td>
<td>1,683</td>
<td></td>
</tr>
<tr>
<td></td>
<td>working and transient pressure</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 5.5.3
Analysis summary for Case 3 - 54 in. LCP
(H = 15 ft, \(P_w = 145\) psi, \(P_t = 58\) psi, \(M_s = 3,000\) psi)

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-392</td>
<td>-444</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>0.05</td>
<td>0.16</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Strain in CFRP at invert (με)</td>
<td>Earth load and pipe weight</td>
<td>384</td>
<td>926</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working pressure</td>
<td>1,410 (1,701)</td>
<td>2,033</td>
<td>AWWA is conservative (conservative)</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working and transient pressure</td>
<td>2,035 (2,271)</td>
<td>2,498</td>
<td>AWWA is conservative (conservative)</td>
</tr>
</tbody>
</table>

Table 5.5.4
Analysis summary for Case 4 – 60 in. ECP
(H = 6 ft, \(P_w = 100\) psi, \(P_t = 40\) psi, and \(M_s = 3,000\) psi)

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-242</td>
<td>-247</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>0.10</td>
<td>0.17</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Strain in CFRP at invert (με)</td>
<td>Earth load and pipe weight</td>
<td>27</td>
<td>331</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working pressure</td>
<td>910 (1,252)</td>
<td>1,278</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working and transient pressure</td>
<td>1,350 (1,614)</td>
<td>1,660</td>
<td></td>
</tr>
</tbody>
</table>
Table 5.5.5 Analysis summary for Case 5 – 60 in. ECP
(H = 6 ft, \(P_w = 180\) psi, \(P_t = 72\) psi, and \(M_s = 3,000\) psi).

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-246</td>
<td>-247</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>0.11</td>
<td>0.17</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Strain in CFRP at invert (με)</td>
<td>Earth load and pipe weight</td>
<td>24</td>
<td>331</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working pressure</td>
<td>2,080 (2,283)</td>
<td>2,045</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working and transient pressure</td>
<td>2,907 (3,058)</td>
<td>2,730</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.5.6 Analysis summary for Case 6 – 60 in. ECP
(H = 15 ft, \(P_w = 100\) psi, \(P_t = 40\) psi, and \(M_s = 1,500\) psi).

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-507</td>
<td>-513</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>0.35</td>
<td>0.64</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Strain in CFRP at invert (με)</td>
<td>Earth load and pipe weight</td>
<td>210</td>
<td>1,191</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working pressure</td>
<td>790 (1,301)</td>
<td>1,619</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working and transient pressure</td>
<td>1,200 (1,581)</td>
<td>1,764</td>
<td></td>
</tr>
</tbody>
</table>
Table 5.5.7
Analysis summary for Case 7 – 60 in. ECP
(H = 15 ft, \( P_w = 100 \) psi, \( P_t = 40 \) psi, and \( M_s = 5,000 \) psi)

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-506</td>
<td>-513</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>0.16</td>
<td>0.22</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Strain in CFRP at invert (με)</td>
<td>Earth load and pipe weight</td>
<td>34</td>
<td>415</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working pressure</td>
<td>715 (1,148)</td>
<td>1,122</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working and transient pressure</td>
<td>1,170 (1,504)</td>
<td>1,337</td>
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</tr>
</tbody>
</table>

Table 5.5.8
Analysis summary for Case 8 – 120 in. ECP
(H = 6 ft, \( P_w = 100 \) psi, \( P_t = 40 \) psi, and \( M_s = 3,000 \) psi)

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-614</td>
<td>-567</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>0.23</td>
<td>0.39</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Strain in CFRP at invert (με)</td>
<td>Earth load and pipe weight</td>
<td>5</td>
<td>381</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working pressure</td>
<td>1,085 (1,736)</td>
<td>1,320</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working and transient pressure</td>
<td>1,610 (2,128)</td>
<td>1,700</td>
<td></td>
</tr>
</tbody>
</table>
### Table 5.5.9
Analysis summary for Case 9 – 120 in. ECP  
\((H = 10 \text{ ft}, P_w = 100 \text{ psi}, P_t = 40 \text{ psi}, \text{ and } M_s = 1,500 \text{ psi})\)

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-872</td>
<td>-837</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>0.55</td>
<td>0.99</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Strain in CFRP at invert (με)</td>
<td>Earth load and pipe weight</td>
<td>138</td>
<td>951</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working pressure</td>
<td>943</td>
<td>1,550</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working and transient pressure</td>
<td>1,380 (1,966)</td>
<td>1,826</td>
<td></td>
</tr>
</tbody>
</table>

### Table 5.5.10
Analysis summary for Case 10 – 120 in. ECP  
\((H = 10 \text{ ft}, P_w = 100 \text{ psi}, P_t = 40 \text{ psi}, \text{ and } M_s = 5,000 \text{ psi})\)

<table>
<thead>
<tr>
<th>Item</th>
<th>Load</th>
<th>FEA at Invert (max. around circumference)</th>
<th>AWWA Draft Standard</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust at springline (lb./in.)</td>
<td>Earth load and pipe weight</td>
<td>-876</td>
<td>-795</td>
<td>AWWA is in agreement</td>
</tr>
<tr>
<td>Horizontal deflection of pipe (in.)</td>
<td>Earth load and pipe weight</td>
<td>0.26</td>
<td>0.34</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td>Strain in CFRP at invert (με)</td>
<td>Earth load and pipe weight</td>
<td>11</td>
<td>329</td>
<td>AWWA is conservative</td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working pressure</td>
<td>885</td>
<td>1,152</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth load, pipe weight and working and transient pressure</td>
<td>1,375 (1,900)</td>
<td>1,483</td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 6
RELIABILITY ANALYSIS

INTRODUCTION

Modern codes and standards for structural design of civil infrastructure are based on load and resistance factor design (LRFD) and its underlying probability-based limit states design (PBLSD). The design for CFRP renewal and strengthening of PCCP must address the uncertainties in loads (i.e., demands) and resistance (i.e., capacities in all possible failure modes or limit states) through appropriate load and resistance factors (Plevris, et al, 1995; Monti and Santini, 2002; Okeil et al. 2002; Ellingwood, 2003; Atadero and Karbahi, 2005; Zureick, Bennett and Ellingwood, 2006; Wang et al, 2010).

Purpose

The purpose of this chapter is to present the reliability basis for developing the limit states (strength) design procedure for CFRP renewal and strengthening of PCCP and to select the load and resistance factors that ensure acceptable reliability level for the repaired or strengthened buried pipe consistent with the reliability of more conventional construction materials used in infrastructure.

Scope

The remainder of this report consists of five sections:

- Reliability basis of LRFD approach
- Target reliabilities for PCCP renewal
- Load combinations for buried PCCP
- Characteristic value of CFRP strength for single-ply and multi-ply laminates.
- Resistance factors for design of CFRP liners for different failure modes.

Method of Approach

The general representation of safety criteria in probability-based limit states design codes is

\[
\text{Required Strength} \leq \text{Time Effect Factor} \times \text{Design Strength} \times \text{Resistance Factor} \quad (6-1)
\]

in which the required strength is determined from structural analysis using factored loads, the design strength is determined from the results of strength tests adjusted for the end-use condition, the time effect factor accounts for creep and creep rupture of material when subjected to sustained loads, and the resistance factor \( \phi \) accounts for the variability of strength and bias of test results. The design strength in the end-use condition is the product of strength in specific failure modes determined from the results of laboratory strength tests after statistical processing of the data, and material adjustment factor accounts for the degradation of the material during its service life and is established from durability tests conducted on the material. The statistical
processing of the test results is accomplished first by calculating the characteristic value of laboratory strength test results corresponding to 5th-percentile strength with 80% confidence to account for sample size.

The load and resistance factors are intended to account for uncertainties associated with the inherent randomness of the load and resistance, as well as the bias and the uncertainties inherent in the use of approximate models used to represent the mechanics of failure modes. These load and resistance factors are adjusted so that structures designed by the code achieve a target performance goal, which is expressed in terms of a structural reliability index, as discussed in more detail below.

RELIABILITY BASIS OF LRFD

The starting point of probability-based design is a description of the limit states of concern (tension failure, flexural failure, instability, etc.) by mathematical models derived from principles of structural mechanics, supported by experimental data. Each model, denoted the limit state function, is given by

\[ G(X) = G(X_1, X_2, \cdots X_m) = 0 \]  

where \( X = (X_1, X_2, \cdots X_m) \) = vector of random resistance and load variables. The “failure” event is defined, by convention, such that the limit state is exceeded when \( G(X) < 0 \). Thus, the limit state probability becomes

\[ P_f = \int_{\Omega} f_X(x_1, x_2, \cdots x_m) \, dx_1 \, dx_2 \cdots dx_m \]  

(6-3)

where \( f_X(x) \) = joint probability density function of \( X \) and the domain of integration \( \Omega \), is that region of \( X \) where \( G(X) < 0 \).

The first-order reliability method (Melchers, 1999), in which \( P_f \) is replaced by the reliability index, \( \beta \), was developed to address the difficulties in utilizing Equation (6-3) in practical structural engineering situations (Ellingwood, 1994). For well-behaved limit state functions (i.e., those not involving bifurcation of equilibrium or large material nonlinearities), the limit state probability, \( P_f \), can be approximated by \( P_f = \Phi(-\beta) \), in which \( \Phi(\cdot) = \) standard normal cumulative distribution function. The first-order reliability method provides the basis for the load-combination requirements in ASCE Standard 7-10, the resistance criteria for steel in the AISC Specification (2010), and other modern standards based on probability-based limit states design. Monte Carlo simulation is used to determine \( P_f \) (or \( \beta \)) in situations where Equation (6-3) is difficult to evaluate in closed form. The reliability assessments in subsequent sections of this report are performed using Monte Carlo simulation.

TARGET RELIABILITY

The selection of the target reliability should consider the estimated risk associated with component failures (Ellingwood, 1994; Melchers, 1999). Typically, target reliability index values in the range of 2.5 to 4.5 have been used in the United States in formulating LRFD design...
criteria for building and bridge structures with a service life of fifty to seventy-five years (e.g., in ACI 2005, AASHTO 2007, and ASCE 7-2010). These values were obtained, for the most part, by calibration to the existing practices with satisfactory service performance. The validity of these reliability benchmarks used previously for steel and reinforced concrete construction stems from the fact that existing design standards for these construction materials had been used successfully in engineering practice since the early years of the twentieth century (Galambos et al, 1982). The target reliabilities should reflect the mode and consequences of exceeding various limit states (Ellingwood, 1994). Design criteria for limit states that occur with little warning (tensile rupture, instability) or have severe consequences (life safety) lie at the higher end of the above range.

In renewals and strengthening of prestressed concrete cylinder pipe, the CFRP liner provides strength, but offers little ductility in comparison with ordinary reinforced/prestressed concrete and steel structures. The consequences of pipe failure include economic, social, and political losses and on occasions may include life safety. Pipe failure in circumferential direction is more catastrophic than other possible failure modes; therefore, a higher target reliability is assigned to this limit state accordingly. Thus, a target reliability $\beta$ equal to 3.5 for circumferential tension and 3.0 for other limit states (discussed subsequently in Section 6) are used for an intended service life of about fifty years in this study.

LOAD FACTORS AND LOAD COMBINATIONS FOR BURIED PCCP

The required strength of the CFRP liner is determined by structural analysis for appropriate load combinations. The load combinations in ASCE 7-10 (ASCE, 2010) were developed using probability-based load-modeling concepts (Ellingwood, et al, 1982) and have been adopted by both the International Building Code 2012 (ICC IBC 2012) and NFPA 5000 (2012) for all common building constructions and materials. However, due to the unique load situations applicable to buried PCCP, the ASCE 7 load combinations are not directly applicable to CFRP renewal and strengthening of PCCP.

Buried pipes are subjected to the following loads:

- $W_p$ = pipe weight
- $W_w$ = water weight
- $W_e$ = earth load
- $P_w$ = working pressure
- $P_t$ = transient pressure
- $P_{gw}$ = groundwater pressure
- $P_v$ = vacuum pressure
- $W_t$ = live load from ground surface traffic

Combinations of these loads for structural design should be based on both the physical and the statistical characteristics of each of the loads. Once backfill has been placed on a pipe, gravity loads are always acting, regardless of whether the pipe is in operation. Accordingly, gravity loads should appear in all load combinations, albeit with different load factors, depending on which loads are combined with them. Once the pipe is in operation, water weight, $W_w$, and internal working pressure, $P_w$, will also be acting during the entire service life. Thus, $W_w$ and $P_w$
should appear in all combinations related to the operation of the piping system under both normal and extreme operating conditions. Finally, the short-term transient loads do not combine with other transients (or does so with negligible probability), so live loads $W_t$, and transient pressures, $P_t$ and $P_v$, need their own load combination. Furthermore, $P_t$ and $P_v$ do not occur at the same location along a pipeline at the same time; rather, they both occur infrequently when flow velocity changes rapidly, e.g., valve operation, power blackout, or when pump startup and shutdown.

Based on the above rationale, load factors and load combinations (1) through (6) below are proposed. Note that these load combinations are independent of the design limit states.

For permanent loads when pipe is empty:

(1) $1.4 \ (W_e + W_p)$
(2) $1.2 \ (W_e + W_p) + 1.6 \ W_t$

For normal operation:

(3) $1.4 \ (P_w + W_e + W_p) + 1.0 \ W_w$
(4) $1.2 \ (P_w + W_e + W_p) + 1.0 \ W_w + 1.6 \ W_t$

For combinations involving transient or vacuum pressure:

(5) $1.2 \ (W_e + W_p) + 1.0 \ W_w + 1.2 \ P_w + 1.2 \ P_t$
(6) $1.2 \ (W_e + W_p) + 1.2 \ (P_{gw} + P_v)$ with $1.2 \ P_v \geq -14.7 \ psi$

The required strength resulting from these load combinations shall be determined from the structural actions in both circumferential and longitudinal directions of the pipe. The structural effects of temperature change $\Delta T$ shall be considered in load combinations (5) and (6) with a load factor of 1.2 on $\Delta T$.

The load factors in the above combinations are guided by ASCE Standard 7-10 (2010) and engineering judgment as to the means and coefficients of variation (uncertainties) in the loads. In particular:

- A load factor of 1.4 is applied to the sum of the “permanent loads” in load combinations (1) and (3). This factor is reduced to 1.2 when combined with other “short-term” loads in other load combinations.
- Load combination (3) and (4) are the basic “operation condition.” A load factor of 1.4 is applied to $P_w$, consistent with the load factor on gravity loads. Since $W_w$ is essentially deterministic, a load factor of 1.0 can be assigned to it. The live load $W_t$ is combined with the normal operating pipe loads with a live load factor of 1.6, based on ASCE 7-10 for a 50 yr service live and a target $\beta$-value of 3.
- Combinations (5) and (6) are infrequent service conditions. Vacuum conditions occur under transient load and we have conservatively assumed that $W_w = 0$ under this condition. If $P_t$ or $P_v$ are specified as extreme events, the load factor can be set equal to one and the loads can be treated the way wind, earthquake, and fire loads are treated in ASCE Standard 7-10 with load factors equal to 1.0 and the nominal values specified as the maximum possible. However, if they have been
obtained by analysis of certain scenarios that may not be completely exhaustive of possible events that can occur, a larger load factor may be used. The engineer should select a load factor that is consistent with the method of analysis for determination of transient pressures. Since the probability of the coincidence of $P_t$ and $P_v$ at a particular location is zero, they need not appear in the same load combination.

- Temperature changes result in self-straining actions, the effect of which depend on whether there is any reserve strength or ductility beyond the elastic limit. Since temperature effects develop and abate slowly, they must be treated along with the permanent and service loads. A load factor equal to 1.2 on self-straining actions is sufficient.

CHARACTERISTIC VALUE OF CFRP TENSILE STRENGTH

The resistance factors $\phi$ necessary to achieve a prescribed value of $\beta$ depend on the way that characteristic values are specified. Therefore, the characteristic value of strength and stiffness must be selected from the actual tests performed on a material property of a manufacturer using the procedure set forth in ASTM D7290. The $\phi$-factors allow for the variability of material properties as affected by the level of quality control expected in manufacturing and installation of CFRP, but their values must be independent of the supplier and installer of the CFRP product. To render the $\phi$-factors independent of the supplier and installer, the reference strength is expressed as the “characteristic value”.

The characteristic value of CFRP tensile strength (obtained from tests per ASTM D3039), expressed in terms of force per unit width, $T_{CFRP}$, is the 5th-percentile value (with 80% confidence) of a two-parameter Weibull distribution that best fits the test data (ASTM D7290, 2009; Zureick, Bennett and Ellingwood, 2006). The basic concept for determining the characteristic value is illustrated in Figure 6.1. The characteristic value of the CFRP modulus, $E_{CFRP}$, is calculated in the same fashion and used in limit states involving pipe buckling. For analysis, $E_{CFRP}$ used in any other limit state should be taken as the mean value of the governing Weibull distribution.

In the renewal of a buried pipe, multiple layers of laminate are often necessary to satisfy the strength demands. Ideally, to determine the characteristic value of a CFRP laminate with, say, five laminae, one must first fabricate a laminate with five layers, cut N specimens from that laminate (with N ≥ 10), and test them in tension in accordance with ASTM D3039. The results of the tests should then be analyzed according to ASTM D7290, and the 80% confidence on the 5th percentile is determined as the characteristic value of the five-laminae liner in tension (Figure 6.1). However, since the test load that is required to fail a multilayer specimen increases as the number of layer increases, the test may become impractical due to the capacity of the available test equipment. A coupon test is conducted relatively easily to determine the characteristic value of a single-ply laminate; moreover, the characteristic strengths of multi-ply laminate for design can be established from the characteristic values of single-ply as described below.

Characteristic Value of Tensile Strength and Modulus of a Single-Ply Laminate

We used two set of test data on single-ply laminate, as plotted in Figure 6.2, as a starting point. These tests were conducted on Tyfo SCH-41-2X manufactured by Fyfe Company. Data
Sets 1 and 2 shown in the Figure 6.2 consist of 89 and 108 data points, respectively. Considering the data sets together, the estimated mean strength and modulus are 136 ksi and 12,756 ksi, respectively, and the estimated characteristic value of strength and modulus of the governing Weibull distributions for a single lamina are 103 ksi and 10.4 Msi, respectively. It is also clear from the figure, though somewhat surprising, that the CFRP strength and modulus are only weakly correlated with a correlation coefficient approximately 0.15.

Characteristic Value of Tensile Strength of a Multi-Ply Laminate

Let us assume that the CFRP laminate consists of n laminae. Further, assume that the tensile strength, $T_{CFRP}$, and elastic modulus, $E_{CFRP}$, are distributed (as shown in Figure 6.2) for each lamina and that each lamina behaves elastically to failure. We first assign n pairs of $T_{CFRP}$ and $E_{CFRP}$ to the n-lamina liner. Assuming that $T_{CFRP}$ and $E_{CFRP}$ are correlated with a correlation coefficient equal to $\rho$, we then subject the n-layer laminate to increasing deformations and maintain compatibility of lamina deformations. We performed the calculation for different values of $\rho$. The objective is to determine the load-deformation relationship for this sample. At each deformation, the forces developed in each lamina will be different because the moduli are different, and the total load is simply the sum of the forces developed in each lamina. We then increase the deformation until the first lamina fails. At this point, one of two things will happen:

- If n is small, the loss of one lamina will cause the entire system to fail immediately because the load cannot be redistributed.
- If n is large, there will be an initial drop in load, but the load can be increased further due to load redistribution. However, the stiffness of the assembly will drop following the initial lamina failure.

The ultimate load is reached when enough laminae have failed that increasing the deformation leads to a drop in the load. The ultimate tensile strength of the laminate, $T_{CFRP}(n)$, equals the ultimate load divided by the initial area of the laminate. Repeating the process a sufficient number of times, we obtain the PDF of $T_{CFRP}(n)$.

Figure 6.3(a) illustrates the Probability Density Function of the applied load at failure for n = 2,3,4,5… under the assumption that $T_{CFRP}$ and $E_{CFRP}$ are jointly normal and Figure 6.3(b) shows the mean and 5th-percentile value of these PDFs as a function of n. Naturally, the larger the n, the larger the failure load causing rupture in the liner, as shown in Figure 6.3(a), and as expected, both the mean and the 5th-percentile value of the failure load increase as n gets larger, as illustrated in Figure 6.3(b). But the mean failure load does not increase as fast as n x (mean failure load of one lamina).

The 5th-percentile value – the characteristic value of tensile strength for n laminae expressed in ksi unit – is illustrated in Figure 6.4(a) – (c) with the assumption that $T_{CFRP}$ and $E_{CFRP}$ are (a) uncorrelated Weibull distributions, (b) uncorrelated normal distributions and (c) jointly normal with the coefficient of correlation $\rho = 0.5$ (a conservative value), respectively. As shown in Figure 6.4, the mean and the 5th-percentile value of the CFRP strength, in ksi, decreases in all the three cases due to the size effect as n increases. The 5th-percentile value of strength becomes essentially constant for $n \geq 2$, while the mean value of strength decrease is more gradual.
The predicted decrease in tensile strength is consistent with the laboratory data available to SGH on Tyfo SCH-41S-1 manufactured by Fyfe Company summarized in Table 6.1. As shown in Table 6.1, the characteristic value of the four-layer laminate is about 7% smaller than that of the three-layer laminate. This decrease in the mean is a direct result of the load redistribution in the laminate after rupture occurred in the “weakest” lamina enforced by strain compatibility between the laminae.

It is clear from above analysis results that, while the correlation coefficient is in the range of 0.0 to 0.5, the characteristic strength decreases initially as the number of laminae in a laminate increases, but becomes independent of \( n \) for \( n \geq 4 \). The characteristic value of strength (in ksi) of an \( n \)-lamina liner may be expressed by

\[
\sigma_{\text{chrac}} (n) = \begin{cases} 
(1 - 0.03(n - 1))\sigma_1 , & n < 4 \\
0.92\sigma_1 , & n \geq 4 
\end{cases} \quad (6-5)
\]

where \( \sigma_{\text{chrac}}(n) \) = characteristic value of strength of \( n \)-layer laminate (in ksi)

\( \sigma_1 \) = characteristic value of strength of a single lamina

Figure 6.1 Characteristic strength of CFRP is the lower 5 percentile value determined from CFRP strength test results with 80% confidence
Figure 6.2 Correlation between CFRP Strength and CFRP modulus obtained from data collected from coupon tests conducted at SGH
(a) Probability Density of CFRP Laminate Strength for Different Number of Laminae \( n \)

(b) Mean and 5th-Percentile Value of CFRP Strength as a Function of Number of laminae \( n \)

Figure 6.3 Characteristic values of CFRP as a function of the number of laminae \( n \)
(a) Lamina Strength and Modulus Uncorrelated with Weibull Distributions

(b) Lamina Strength and Modulus Uncorrelated Normal Distributions
Figure 6.4 Characteristic value of CFRP strength (ksi) for different numbers of laminae $n$, for CFRP strength and modulus (a) uncorrelated with Weibull distributions, (b) uncorrelated with Normal distributions, and (c) correlated with a joint coefficient of correlation of $\rho = 0.5$
## Table 6.1
Statistical characteristics of strength of multilayer Tyfo SCH-41S-1 laminate (source: Fyfe)

<table>
<thead>
<tr>
<th>Sample</th>
<th>3-Layer Specimen</th>
<th>4-Layer Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>133.2</td>
<td>89.3</td>
</tr>
<tr>
<td>2</td>
<td>135.5</td>
<td>109.0</td>
</tr>
<tr>
<td>3</td>
<td>131.6</td>
<td>109.6</td>
</tr>
<tr>
<td>4</td>
<td>127.3</td>
<td>111.9</td>
</tr>
<tr>
<td>5</td>
<td>132.3</td>
<td>122.4</td>
</tr>
<tr>
<td>6</td>
<td>120.2</td>
<td>122.5</td>
</tr>
<tr>
<td>7</td>
<td>114.0</td>
<td>108.8</td>
</tr>
<tr>
<td>8</td>
<td>119.7</td>
<td>123.2</td>
</tr>
<tr>
<td>9</td>
<td>123.1</td>
<td>99.4</td>
</tr>
<tr>
<td>10</td>
<td>105.7</td>
<td>112.6</td>
</tr>
<tr>
<td>Average</td>
<td>124.3</td>
<td>113.3</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>9.6</td>
<td>8.0</td>
</tr>
<tr>
<td>COV (%)</td>
<td>7.7</td>
<td>7.1</td>
</tr>
<tr>
<td>Characteristic Value</td>
<td>106.3</td>
<td>98.2</td>
</tr>
</tbody>
</table>

## RESISTANCE FACTORS

### Design Limit States

CFRP liner should be designed either as a standalone structure or as a composite fashion with concrete inner core of the host pipe. Design of the standalone CFRP liner should consider limit states of buckling and rupture of the liner due to circumferential tension from the combined effect of internal pressure and bending under gravity loads.

For the design of CFRP liner acting in a composite fashion with the concrete inner core, in addition to all the above mentioned limit states, the design should also consider debonding of CFRP from inner concrete cores, resulting from concrete crushing in compression at springline and CFRP liner debonding from concrete inner core due to radial tension or shear strain.

### Monte Carlo Simulation

The statistical evaluation of a design formula in terms of the characteristics of the random variables that affect the design is performed by Monte Carlo simulation. In this approach, design uncertainty factors are introduced.

Probabilistic parameters of the random variables that represent loads and contribute to the resistance are summarized in Table 6.2. With the probabilistic models specified for all critical design variables, the resistance factor calibration process and the results for each of the limit states discussed in Section 6.1 are presented in the following Sections 6.3 through 6.8.
Table 6.2  
Probabilistic models of loads and pipe resistance

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Unit</th>
<th>Bias Mean/Design Value</th>
<th>COV</th>
<th>CDF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Working Pressure</td>
<td>$P_w$</td>
<td>psi</td>
<td>1.00</td>
<td>0.10</td>
<td>Normal</td>
</tr>
<tr>
<td>Transient Pressure</td>
<td>$P_t$</td>
<td>psi</td>
<td>1.00</td>
<td>0.15</td>
<td>Normal</td>
</tr>
<tr>
<td>Host PCCP Inside Diameter</td>
<td>$D$</td>
<td>in.</td>
<td>1.00</td>
<td>0.02</td>
<td>Normal</td>
</tr>
<tr>
<td>Uncertainty Factor for Pressure Effect</td>
<td>$B_p$</td>
<td></td>
<td>0.90</td>
<td>0.05</td>
<td>Normal</td>
</tr>
<tr>
<td>Uncertainty Factor for Analytical Modeling of Radial Tension</td>
<td>$B_r$</td>
<td></td>
<td>1.00</td>
<td>0.10</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Uncertainty Factor for Gravity-Load Effect</td>
<td>$B_w$</td>
<td></td>
<td>1.00</td>
<td>0.10</td>
<td>Normal</td>
</tr>
<tr>
<td>Analytical Modeling Error for Buckling</td>
<td>$B_b$</td>
<td></td>
<td>0.75</td>
<td>0.075</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Analytical Modeling Error for Shear at CFRP/Concrete Interface</td>
<td>$B_d$</td>
<td></td>
<td>1.00</td>
<td>0.10</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Earth Load</td>
<td>$W_e$</td>
<td>lbf/in.</td>
<td>1.00</td>
<td>0.15</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$W_p$</td>
<td>lbf/in.</td>
<td>1.00</td>
<td>0.08</td>
<td>Normal</td>
</tr>
<tr>
<td>Live Load</td>
<td>$W_t$</td>
<td>lbf/in.</td>
<td>1.15</td>
<td>0.18</td>
<td>Normal</td>
</tr>
<tr>
<td>Soil Modulus</td>
<td>$M_s$</td>
<td>psi</td>
<td>1.15</td>
<td>0.30</td>
<td>Normal</td>
</tr>
<tr>
<td>Bedding Coefficient</td>
<td>$K_b$</td>
<td></td>
<td>1.10</td>
<td>0.10</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Concrete Modulus</td>
<td>$E_c$</td>
<td>psi</td>
<td>1.10</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Concrete Compression Strength</td>
<td>$f'_c$</td>
<td>psi</td>
<td>1.18</td>
<td>0.18</td>
<td>Normal</td>
</tr>
<tr>
<td>CFRP Modulus</td>
<td>$E_{CFRP}$</td>
<td>psi</td>
<td>1.00</td>
<td>0.0075</td>
<td>Weibull</td>
</tr>
</tbody>
</table>

Effect of Workmanship

The characteristic values of CFRP strength used in CFRP renewal and strengthening of PCCP depend on material properties, and construction workmanship (Zureick and Kahn, 2001). These factors affect the nominal strength of the composite, $T_{CFRP}$, and the resistance factor, $\phi$, used in the renewal design.

We obtained more than thirty sets of tensile strength test results. Each set represented the results of tests conducted on specimens cut from test panels constructed during installation of CFRP in repair pipes. The quality of test panels is very close approximations of field-installed laminates. We selected three sets of tensile strength tests on Tyfo SCH-41-2X CFRP coupons from all the data sets that were available to us to reflect three different levels of construction workmanship, designated as CFRP I for Workmanship Level I, CFRPII for Workmanship Level II, and CFRP III for Workmanship Level III, respectively. The means, coefficients of variation (COV), and characteristic values of the tensile strengths for these three levels of workmanship are tabulated in Table 6.3, with their COV rounded to 15% for CFRP I, 10% for CFRP II, and 5% for CFRP III. Similar statistics for the CFRP modulus are presented in Table 6.3. Note that even with the COV of CFRP strength ranging from 5% to 15% as in Table 6.3, the mean value of the modulus does not change appreciably with different workmanship levels, and the COV of modulus ranges between 5% and 8% as shown in Table 6.4.
Table 6.3
Weibull parameters and characteristic value of CFRP strength (lbf/in.) for three test sets with different workmanship

<table>
<thead>
<tr>
<th>Material</th>
<th>Sample Size</th>
<th>Mean (lbf/in.)</th>
<th>COV</th>
<th>Characteristic Value (lbf/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP I</td>
<td>11</td>
<td>9,667</td>
<td>0.15</td>
<td>7,316</td>
</tr>
<tr>
<td>CFRP II</td>
<td>10</td>
<td>12,204</td>
<td>0.10</td>
<td>10,021</td>
</tr>
<tr>
<td>CFRP III</td>
<td>10</td>
<td>11,412</td>
<td>0.05</td>
<td>10,429</td>
</tr>
</tbody>
</table>

Table 6.4
Weibull parameters and characteristic value of CFRP Modulus (ksi) for three test sets with different workmanship

<table>
<thead>
<tr>
<th>Material</th>
<th>Sample Size</th>
<th>Mean (ksi)</th>
<th>COV</th>
<th>Characteristic Value (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP I</td>
<td>11</td>
<td>12,384</td>
<td>0.08</td>
<td>10,413</td>
</tr>
<tr>
<td>CFRP II</td>
<td>10</td>
<td>12,807</td>
<td>0.08</td>
<td>10,976</td>
</tr>
<tr>
<td>CFRP III</td>
<td>10</td>
<td>12,604</td>
<td>0.05</td>
<td>11,614</td>
</tr>
</tbody>
</table>

Reliability Analysis of CFRP Liner for Circumferential Tension

For PCCP with an inside diameter $D$, the CFRP liner is designed to resist the entire pressure load, i.e.,

$$\phi_a \frac{T_{CFRP}}{E_{CFRP} \cdot t_1} \geq \varepsilon_a (\Sigma \gamma P_i P_i)$$  (6-6)

where $T_{CFRP}$ = characteristic value of strength of a single lamina in an n-lamina CFRP liner in lbf/in. (Figure 6.2)

$E_{CFRP}$ = mean value of n-lamina CFRP liner modulus

$t_1$ = thickness of a single lamina

$\varepsilon_a$ = the calculated circumferential strain resulting from the pressure load combinations ($\Sigma \gamma P$)

$\gamma P$ = pressure load factor. The design process yields the number of lamina layers, $n$, necessary to resist pressure demand.

The governing limit state (failure mode) of a standalone n-lamina CFRP liner subjected to internal pressure is rupture of the CFRP liner in tension. Failure occurs when the random variable defining the required strength, i.e., the circumferential tensile strain capacity, $\varepsilon_{an}$, calculated from the combined effects of working pressure, $P_w$, and transient pressure, $P_t$, exceeds the design strength, or

$$\varepsilon_{an} = B_p \cdot \frac{T_{CFRP}}{E_{CFRP} \cdot t_1}$$  (6-7)

Modeling uncertainty variable, $B_p$, represents the variability in the ratio of test-to-calculated values that remains when all parameters in the strength equations are known. Statistical
descriptions of all variables in Equation (6-7) are defined in Table 6.2 through 6.4. The required number of lamina in the CFRP liner, \( n \), is obtained from the design calculation using Equation (6-6). Since \( n \) is clearly affected by the value of \( \phi_a \) used in design, the reliability of the CFRP liner designed by Equation (6-6) will be dependent on \( \phi_a \).

Reliability analyses are performed for the limit state described by Equation (6-7) for a series of typical design cases and feasible values of \( \phi_a \). Design cases considered include pipes with \( D \) ranging from 30 to 120 in. and working pressure from 30 to 300 psi. The reliability of the CFRP liner is dependent on the workmanship in CFRP liner installation, which is reflected in the statistics of the CFRP strength in Table 6.3.

Figure 6.5 illustrates the variation in reliability index, \( \beta \), as a function of \( \phi_a \) for different quality control levels and shows that \( \beta \) decreases as \( \phi_a \) increases from 0.55 to 1.0. Clearly, smaller values of \( \phi_a \) result in more-conservative designs and higher reliabilities. To determine the best value of \( \phi_a \) to achieve an overall target \( \beta \) of 3.5 for the trial design cases, a least-squares average \( (LSA) = \frac{1}{N}\sum (\beta_i - 3.5)^2 \) of all design cases considered in the calibration is plotted as a function of \( \phi_a \), as shown in Figure 6.6. The results suggest that to achieve the same reliability, slightly different \( \phi_a \) values may be used in the design according to the quality of workmanship. As might be expected, the optimal \( \phi_a \) to achieve the target reliability decreases with increase in the uncertainty of the strength of the CFRP liner material resulting from poor workmanship. However, it would be preferable to specify one value of \( \phi_a \) for this limit state and to allow the workmanship to be reflected in the characteristic value of \( T_{CFRP} \). Figure 6.6 suggests that this one value of \( \phi_a \) for design of CFRP liners for circumferential tension should be 0.75.

Figure 6.5 Variation in \( \beta \) as a Function of \( \phi_a \) for CFRP Liner in Tension with Different Workmanship (\( D = 30-120 \text{ in.}; P_w = 30-300 \text{ psi} \))
Figure 6.6 Least-Squares Analysis to Determine $\phi_a$ for CFRP Liner in Tension with Different Workmanship. ($D = 30$-120 in.; $P_w = 30$-300 psi)

Reliability Analysis of CFRP Liner for Bending

To design a CFRP liner to resist the demand from factored gravity-load combinations, expressed in terms of bending strain, $\varepsilon_{b}$, resulting from load combination in Section 4 ($\sum \gamma_w W$), the following design equation should be satisfied:

$$\phi_a \frac{T_{CFRP}}{E_{CFRP} \cdot t_1} \geq \varepsilon_{b} \left( \sum \gamma_w W_i \right).$$  \hspace{1cm} (6-8)

where $T_{CFRP}$ = the characteristic value of the strength of the lamina in an n-lamina CFRP (Table 6.3);

$E_{CFRP}$ = mean value of the CFRP modulus (Table 6.4)

$t_1$ = thickness of a single lamina used in design of the liner

The right side of the Equation (6-8) is the required strength from gravity load combinations expressed in terms of strain by Equations (6-9a) for a standalone liner and (6-9b) for a liner that acts together with concrete inner core in a composite fashion:
where \(D\) = inside diameter of pipe
\(\eta\) = shape factor as defined in AWWA 45
\(\Delta\) = pipe vertical deflection under factored gravity load calculated using the Modified Iowa Formula
\(y\) = distance of extreme CFRP fiber from neutral axis of the composite pipe wall. For a standalone CFRP liner,

\[
\varepsilon_d = \eta \left( \frac{\Delta}{D - n \cdot t_1} \right) \left( \frac{n \cdot t_1}{D - n \cdot t_1} \right) \quad (6-9a)
\]

\[
\varepsilon_d = \eta \left( \frac{\Delta}{D - n \cdot t_1} \right) \left( \frac{2y}{D - n \cdot t_1} \right) \quad (6-9b)
\]

and for the CFRP liner-concrete composite section:

\[
\Delta = \frac{\sum D_L \left( \sum \gamma_w W \right) K_b}{0.061 M_s + \frac{E_{CFRP} (n \cdot t_1)^3}{12} \left( \frac{D - n \cdot t_1}{2} \right)^3} \quad (6-10a)
\]

\[
\Delta = \frac{\sum D_L \left( \sum \gamma_w W \right) K_b}{0.061 M_s + \frac{E_c \cdot I_{composite}}{R_{composite}^3} \left( \frac{D - n \cdot t_1}{2} \right)^3} \quad (6-10b)
\]

where \(\sum \gamma_w W_i\) = factored gravity loads
\(D_L\) = deflection lag factor
\(K_b\) = bedding coefficient
\(M_s\) = constrained soil modulus

\(I_{composite}\) = moment of inertia of composite CFRP and inner core
\(R_{composite}\) = radius of composite CFRP and inner core; and \(E_c\) = effective modulus for the liner-concrete composite section.

Substituting Equations (6-9) and (6-10) into Equation (6-8), the resulting equation can yield the number of laminae, \(n\), necessary for the liner to resist gravity-load demand.

The limit state (failure mode) of the CFRP liner subjected to gravity load is rupture due to bending. The limit state can be written as

\[
\varepsilon_d = B_w \cdot \frac{T_{CFRP}}{E_{CFRP} \cdot t_1} \quad (6-11)
\]
The random variable $B_w$ models the error embedded in the analytical method presented above including the bias and the uncertainty in shape factor, $\eta$. In other words, $\eta$ is considered as a random variable and its bias and uncertainty are lumped in $B_w$.

Reliability analyses are performed using Monte Carlo simulation for the limit state described by Equation (6-11) and for feasible values of $\phi_b$. Design cases considered in the analysis include pipes with inside diameters, $D$, ranging from 30 to 144 in.; soil cover depths, $H$, ranging from 6 to 40 ft; soil modulus, $M_s$, ranging from 500 to 7,000 psi; and three level of workmanship in the liner installation that are reflected in the coefficients of variation of the CFRP strength. The random variables are those listed in Tables 6.2 through 6.4. A total of 192 design cases were considered in this calibration. Figures 7.7 through 7.9 illustrate the variation in reliability index as a function of $\phi_b$ for different levels of workmanship, different soil modulus, and different depths of soil cover, respectively. The soil cover depths, $H$, which affects both the soil weight and live load, has the most significant influence on pipe reliability compared to the other design variables.

Figure 6.10 shows the results of a LSA aimed at determining the optimal resistance factor, $\phi_b$, for both the standalone CFRP liner and the liner-concrete inner core composite section. As shown in Figure 6.10, the optimal $\phi_b$ of 0.65 for the composite section is slightly lower than the optimal $\phi_b$ of approximately 0.72 for the standalone liner because of the extra uncertainty introduced by the concrete inner core thickness and concrete modulus presented in Equation (6-10b). A value of $\phi_b = 0.70$ may be used for both CFRP standalone and, somewhat conservatively, for composite designs.
Figure 6.7 Variation in $\beta$ as a function of $\phi_b$ for CFRP liner in bending with different quality of workmanship for (a) standalone liner and (b) liner acting composite with concrete inner core (D = 30-120 in.; $M_s$ = 500-7,000 psi; H = 6-40 ft)
Figure 6.8 Variation in $\beta$ as a function of $\phi_b$ for CFRP liner in bending for soils with different constrained modulus (CFRP I, II, and III; $D = 30\text{-}120$ in.; $H = 6\text{-}40$ ft)

Figure 6.9 Variation in $\beta$ as a function of $\phi_b$ for CFRP liner in bending for pipes with different soil cover depths (CFRP I, II, and III; $D = 30\text{-}120$ in.; $M_s = 500\text{-}7,000$ psi)
Figure 6.10 – Least-squares analysis to determine optimal $\phi_b$ for CFRP liner in bending for standalone liner as well as for the CFRP-concrete inner core composite liner (CFRP I, II and III; D = 30-120 in.; $M_s = 500$-$7,000$ psi; $H = 6$-$40$ ft)

Reliability Analysis of CFRP Liner for Combined Tension and Bending

As discussed in section 6.3 and 6.4, we use two resistance factors, $\phi_a$ for circumferential tension and $\phi_b$ for bending, to ensure reliability of the two individual limit states separately. When the CFRP renewed PCCP is subjected to the combined effect of circumferential tension and bending, we require that the sum of ratios of factored load effect to the factored resistance of the circumferential tension and gravity loads should not exceed one, i.e.,

$$\frac{\varepsilon_a (\Sigma \gamma_p P_i)}{\phi_a \varepsilon_{an}} + \frac{R_c \varepsilon_b (\Sigma \gamma_w W_i)}{\phi_b \varepsilon_{bn}} \leq 1$$  (6-12a)

where the numerators $\varepsilon_a$ and $\varepsilon_b$ are the strains from the factored design pressures and gravity loads, respectively, the denominators are the factored resistance for circumferential tension and bending expressed in terms of strain, and $R_c$ is rerounding factor. $R_c$ is defined as:
\[ R_c = \frac{1}{1 + \frac{P}{3 \left( 0.061 M_s + \frac{E_{CFRP} \cdot I_H}{R_f^3} \right)}}. \]  \hspace{1cm} (6-12b)

where \( P \) = pressure  
\( M_s \) = constrained soil modulus  
\( E_{CFRP} \) = modulus of CFRP  
\( I_H \) = moment of inertia of CFRP for standalone and CFRP and inner core for composite pipe wall  
\( R_f \) = radius of CFRP for standalone and CFRP and inner core for composite pipe wall.

### Reliability of CFRP Liner for Debonding Due to Radial Tension

To prevent the CFRP liner from debonding from the concrete inner core of the host pipe due to radial tension at the CFRP and concrete interface, the following criterion must be satisfied:

\[ \sigma_{rtn} \leq \phi_{rt} \cdot f_t' \]  \hspace{1cm} (6-13)

where \( f_t' = 5.5 \sqrt{f_c} \) = tensile strength of concrete in psi. The value of the radial tensile stress due to the load effect can be calculated as

\[ \sigma_{rtn} = \frac{2 \cdot \varepsilon_b \cdot E_{CFRP} \cdot n \cdot t_s}{D} \]  \hspace{1cm} (6-14)

where \( E_{CFRP} \) = modulus of CFRP in the hoop direction  
\( \varepsilon_b \) = tensile strain in CFRP, calculated from load combinations of Equation (6-4) and strain from Equation (6-9b).

The limit state for debonding due to radial tension is

\[ B_{r_t} \cdot \sigma_{r_t} \leq f_t' \]  \hspace{1cm} (6-15)

The random variable \( B_{r_t} \) represents the uncertainty in the analytical model represented by Equations (6-9b) and (6-10b). We performed Monte Carlo simulation by replacing the nominal values of the design variables in Equations (6-9b) and (6-10b) with the corresponding random variables for these quantities, as listed in Tables 6.2 through 6.4. Reliability analyses were performed for the limit state described by Equation (6-15) and for feasible values of \( \phi_{rt} \). Design cases considered in the analysis include pipes with inside diameters, \( D \), ranging from 30 to 120 in.; soil cover heights, \( H \), ranging from 6 to 40 ft; soil modulus, \( M_s \), ranging from 500 to 7,000
psi; and three level of workmanship in the liner installation reflected in the coefficients of variation of the CFRP strength.

Figures 6.11 through 6.14 illustrate the variation in reliability index as a function of $\phi_{rt}$ for different levels of workmanship, different pipe diameter, different soil modulus and different soil cover height, respectively. The coefficients of variation of CFRP strength and soil cover height have a negligible influence on the required $\phi_{rt}$, as indicated in Figures 6.11 and 6.14; note that CFRP strength is not a part of the limit state function for debonding under radial tension expressed in Equation (6-15). The pipe diameter and the constrained soil modulus have the most effect on the reliability of the pipe against radial tension comparing to other design variables, as shown in Figures 6.12 and 6.13, respectively. Figure 6.15 shows the mean-square residual error with respect to the target reliability of 3.0 as $\phi_{rt}$ increases from 0.40 to 0.85, suggesting that in order to achieve the overall target reliability, $\phi_{rt}$ should be equal to 0.50 in the CFRP renewal of PCCP.

![Figure 6.11 Variation in β as a function of $\phi_{RT}$ for radial tension debonding of CFRP liner for different quality of workmanship in liner installation (D = 30-120 in.; $M_s$ = 500-7,000 psi; $H$ = 6-40 ft)](image)
Figure 6.12 – Variation in $\beta$ as a function of $\phi_{RT}$ for radial tension debonding of CFRP liner for different pipe diameter, D (CFRP I, II, and III; $M_s = 500$-$7,000$ psi; $H_s = 6$-$40$ ft)

Figure 6.13 – Variation in $\beta$ as a function of $\phi_{RT}$ for radial tension debonding of CFRP liner for different soil modulus, $M_s$ (CFRP I, II, and III; $D = 30$-$120$ in.; $H = 6$-$40$ ft)
Figure 6.14 – Variation in $\beta$ as a function of $\phi_{RT}$ for radial tension debonding of CFRP liner for different soil cover heights, $H$ (CFRP I, II, and III; $D = 30\text{-}120$ in.; $M_s = 500\text{-}7,000$ psi)

Figure 6.15 – LSA analysis of $\beta$ as a function of $\phi_{RT}$ for radial tension debonding of CFRP liner for different soil cover heights, $H$ (CFRP I, II, and III; $D = 30\text{-}120$ in.; $M_s = 500\text{-}7,000$ psi)
Reliability Analysis of CFRP Liner for Debonding Due to Shear

To prevent the CFRP liner from debonding in shear from the concrete inner core of the host pipe, the following criterion must be satisfied:

\[ \varepsilon_u \leq \varphi_{\text{debonding}} \cdot \varepsilon_{dn} \]  

(6-16)

where \( \varepsilon_u \) is the maximum strain demand in the CFRP liner from the load combinations of Equation (6-4) calculated using Equations (6-9b) and (6-10b) and \( \varepsilon_{dn} \) is the debonding resistance, defined by

\[ \varepsilon_d = 0.234 \sqrt{\frac{f'_c}{E_{\text{CFRP}} \cdot n \cdot t_1}} \]  

(6-17)

where \( f'_c \) = nominal concrete compression strength  
\( E_{\text{CFRP}} \) = CFRP modulus in the hoop direction  
\( n \) = number of hoop layers  
\( t_1 \) = thickness of a single lamina in hoop direction.

The limit state for debonding due to shear can be written as

\[ \varepsilon_u = B_d \cdot \varepsilon_d \]  

(6-18)

where \( \varepsilon_u \) = required strength and the random variable \( B_d \) = the error embedded in the analysis method presented by Equation (6-17).

The resistance factors can be approximated from the following equation from ASCE 7-10:

\[ \phi = \left( \mu_R / R_n \right) \exp \left[ - \alpha_R \beta V_R \right] \]  

(6-19)

where \( \mu_R \) = mean strength  
\( R_n \) = nominal strength  
\( V_R \) = coefficient of variation in strength  
\( \alpha_R \) = a coefficient typically accepted as 0.7.

Using the bias and the coefficient of variation of three-edge bearing test results (Table 5.3), and the bias and coefficient of variations for \( f'_c \) and \( E_{\text{CFRP}} \) given in Table 6.2, we obtain a value for \( \phi_{\text{debonding}} = 0.6 \).
Reliability Analysis of CFRP Liner for Buckling

To prevent the CFRP liner from debonding due to buckling, the following design criterion must be satisfied:

\[ P_u \leq \phi_{\text{buckling}} \cdot P_{cr} \quad (6-20) \]

where \( P_u \), in psi, is the pressure acting on the pipe from load combination of Equations (6-4) and \( P_{cr} \), in psi, is the critical buckling load, which can be calculated from

\[ P_{cr} = b \cdot \frac{E_{\text{CFRP}}}{1 - \nu_{LT} \nu_{TL}} \left( \frac{n \cdot t_1}{D - n \cdot t_1} \right)^{2.2} \quad (6-21) \]

where \( E_{\text{CFRP}} \) = characteristic value of the governing Weibull distribution of CFRP modulus, \( \nu_{LT} \) = CFRP Poisson’s ratio in transverse direction due to deformation in longitudinal direction, and \( t_1 \) = thickness of a single lamina.

The limit state of buckling can be written as the following:

\[ P_u = B_b \cdot P_{cr} \quad (6-22) \]

where random variable \( B_b \) represents the error embedded in the analysis model represented by Equation (6-21) consisting of a bias of 0.75 and uncertainty with a coefficient of variation of 0.075. \( P_u \) is the random variable that results in buckling failure. \( P_{cr} \) is the random variable representing the critical buckling load for a given pipe, which can be obtained through Monte Carlo simulation by replacing the nominal values of the design variables in Equation (6-22) with the corresponding random variables, with probability distributions listed in Tables 6.2 through 6.4.

A series of PCCP design cases were analyzed for possible values of \( \phi_{\text{buckling}} \). Figure 6.16 illustrates the variation in reliability index as a function of \( \phi_{\text{buckling}} \) for different levels of workmanship. The LSA for a target reliability index of 3.0 in Figure 6.17 indicates that the optimal \( \phi_{\text{buckling}} \) for design is 0.55.
Figure 6.16 – Variation in $\beta$ as a Function of $\phi_{\text{buckling}}$ for CFRP liner buckling for different quality of workmanship ($D = 30$-120 in.; $H = 6$-40 ft; $P_w = 30$-300 psi)

Figure 6.17 Variation in $\beta$ as a Function of $\phi_{\text{buckling}}$ for CFRP liner buckling for different quality of workmanship ($D = 30$-120 in.; $H = 6$-40 ft; $P_w = 30$-300 psi)
Reliability Analysis of CFRP Liner for Pressure-Induced Longitudinal Bending at Edges of Broken Wire Zones

To prevent CFRP rupture in the longitudinal direction at the edges of broken wire zones due to radial expansion of the CFRP in the broken wire zone caused by internal pressure, the following design criterion must be satisfied:

\[ \varepsilon_{lb,u} \leq \phi_b \varepsilon_{lb,n} \] (6-23)

where \( \varepsilon_{lb,u} \) = maximum bending strain in the longitudinal direction resulting from internal pressure
\( \varepsilon_{lb,n} \) = tensile or compressive bending strain capacity of the CFRP liner in the longitudinal direction from internal pressure.

The maximum bending strain in the longitudinal direction can be computed from the following formula:

\[ \varepsilon_{lb,u} = \frac{1}{(n \cdot t_1) I_L E_{CFRP}} y R_f \left( \sum y_i P_i \right) \] (6-24)

where \( t_1 \) = thickness of a single lamina
\( I_L \) = is the moment of inertia of the liner in the longitudinal direction
\( E_{CFRP} \) is the modulus of the liner
\( R_f \) = radius of the liner.

Note the statistical similarity between Eq. (6-24) and Eq. (6-6) for circumferential tension from factored pressure. From this comparison, one can conclude that the resistance factor in Eq. (6-23) must be the same as that used for circumferential tension from pressure, i.e., \( \phi = 0.75 \).

Reliability Analysis of CFRP Liner for Debonding From Steel Substrate at Pipe Ends Due to Shear

To prevent CFRP debonding from steel substrate (i.e., steel cylinder, joint rings) at CFRP terminations at pipe ends due to shear induced by longitudinal forces, the shear stress at the interface of CFRP and steel must be limited. However, the shear strength at this interface is also limited by the tensile strength of CFRP. Therefore, a combination of the following two limit states should be satisfied:

\[ \frac{V}{bl} \leq \phi_{tb} f_{tn} \] (6-25)
\[ \frac{V}{bt} \leq \phi_t f_{tn} \] (6-26)

where \( V \) = CFRP-steel substrate shear in lb./in.
\( b \) = sample width equal to 1 in. for \( V \) in lbf/in.
\[ l = \text{length of CFRP-to-steel bond} \]
\[ t = \text{CFRP thickness} \]
\[ f_{sn} = \text{nominal shear strength} \]
\[ f_{tn} = \text{nominal tensile strength of laminate in the longitudinal direction at termination} \]

Tests were conducted by Fyfe Co. on shear bond strength of CFRP and steel. Two failure modes were noted, a shear bond failure mode (see Table 6.5 for test results) and a CFRP tension failure mode (see Table 6.6 for test results). For a nominal shear strength of \( f_{sn} = 1,500 \text{ psi} \) and for a nominal tensile strength of laminate at termination of \( f_{tn} = 16,000 \text{ psi} \), which are based on the 5\(^{th}\) percentile value of Weibull distributions of the test data shown.

### Table 6.5

Shear test results with shear bond failure between CFRP and steel substrate
(Data source: Fyfe Co.)

<table>
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<tr>
<th>Sample ID</th>
<th>Overlap Length (in.)</th>
<th>Applied Load (lbf)</th>
<th>CFRP Thickness (in.)</th>
<th>Sample Width (in.)</th>
<th>Shear Area (in(^2))</th>
<th>Shear Strength (psi)</th>
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<th>CFRP Thickness (in.)</th>
<th>Sample Width (in.)</th>
<th>Shear Area (in(^2))</th>
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<th>CFRP Thickness (in.)</th>
<th>Sample Width (in.)</th>
<th>Shear Area (in(^2))</th>
<th>CFRP Tensile Strength Limit on Shear Capacity (psi)</th>
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Using Equation (6-19), the corresponding \( \phi \) values in Equation (6-25) and (6-26) can be calculated as follows:

\[
\begin{align*}
\phi_1 &= \left(\frac{1.874}{1.500}\right) \exp \left[-0.7 \times 3.5 \times \left(\frac{386}{1874}\right)\right] = 0.75 \\
\phi_2 &= \left(\frac{25,016}{16,000}\right) \exp \left[-0.7 \times 3.5 \times \left(\frac{5373}{25016}\right)\right] = 0.92
\end{align*}
\] (6-27)

The resistance factors, \( \phi_1 = 0.75 \) and \( \phi_2 = 0.90 \), should be used in Equations (6-25) and (6-26), respectively, to ensure a reliability index of about 3.5.
CHAPTER 7
DESIGN REQUIREMENTS

GENERAL DESIGN APPROACH

The design of the CFRP liner for renewal and strengthening of PCCP is based on the assumption that the host PCCP continues to degrade until it is fully degraded from external corrosion, i.e., the prestressing wires are broken, the outer core is cracked, and steel cylinder is corroded and is perforated. The design of CFRP liner is based on the following requirements:

\[ R_u \leq \lambda \phi R_n = \lambda \phi (R_{o}) \]  

(7-1)

where \( R_u \) = required strength  
\( \lambda \) = time effect factor that accounts for the creep and creep rupture of material under sustained loads  
\( \phi \) = resistance factor that accounts for the variability of material properties and the uncertainty in the predictive formulas  
\( R_n \) = nominal strength in the end-use condition  
\( C \) = material adjustment factor that accounts for durability material exposed to exposure environment for the duration of its service life  
\( R_o \) = test strength of the materials

The design procedures for CFRP liner are presented in the AWWA Draft Standard (2012) for CFRP Renewal and Strengthening of PCCP. The provisions of the draft AWWA Standard are divided into three major chapters that deal with materials, design, and installation and quality control. Note that the provisions of this draft standard are subject to change in the process of its approval by the Concrete Pressure Pipe Committee and Standards Council of AWWA.

MATERIAL PROVISIONS

The provisions of the draft standard related to materials include provisions on durability and time effects factor, based on long-term testing of the CFRP system in the chemical environment (e.g., water resistance, salt water resistance, alkali resistance and dry heat resistance) and thermal environment (as defined by the exposure temperature) of the actual or more severe environments. The draft standard provides requirements for physical properties and expresses all mechanical strength properties of interest in terms of their characteristic value based on 5% likelihood of exceedance with a 80% confidence. For each mechanical property, the draft standard provides ASTM test procedure, minimum number of test specimens, as well as the minimum required properties. Material adjustment factors, C, are specified for 5 years and 50 years exposure to water, salt, and alkali solutions at room temperature, 100°F and 140°F. The time-effect factors that account for creep and creep rupture of CFRP under sustained loads are provided for strength in tension for 5 and 50 years of exposure.
DESIGN PROVISIONS

The provisions of the draft standard related to design include provisions on design loads (gravity, pressure, and thermal) and load combinations, including load factors that account for the variability of loads. The draft standard identifies the design sustained loads and short term loads and provides procedures for combining the effects of sustained and short term loads. For the design of CFRP liner, two conditions are considered: one in which the CFRP acts as a standalone liner well-bonded at the ends to the steel joint ring and if needed to the steel cylinder for watertightness, and the other in which the CFRP liner and inner concrete core act as a composite with CFRP liner, again well-bonded at its termination to the steel joint rings and if needed, to the steel cylinder for watertightness. The design process assumes initially that the CFRP is acting as a composite with the concrete inner core. The process checks the bond between the CFRP liner and the inner concrete core. If the bond fails due to radial tension or strain in the liner that would result in debonding of the CFRP liner from the concrete core, then the CFRP liner is designed as a standalone liner. This process ensures that the performance of the CFRP renewed PCCP regardless of whether it acts as a composite with the inner concrete core or as a standalone liner.

The provisions of the draft standard provide design formulas for different failure modes and limit states. These limit states consist of the following:

- Internal pressure: The entire pressure in the pipe is assumed to be supported by CFRP. This is a conservative assumption as the effect of resistance to internal pressure of PCCP and soil surrounding the PCCP is neglected.
- Bending of Empty Pipe Wall: When the CFRP-lined PCCP is empty and after it has undergone loss of prestress, cracking of the outer core, and corrosion of steel cylinder, the CFRP liner deflects under gravity loads either as a standalone system or as a composite system acting together with the microcracked inner concrete core. In both forms, the pipe acts as a flexible pipe and resists the gravity loads in conjunction with the surrounding soil. For a pipe that has been in the ground for many years, the soil surrounding the pipe (backfill) is not expected to have a stiffness that is different from the in-situ soil if the backfill used during construction was processed natural soil from the site. Therefore, constrained soil modulus, $M_s$, that resists pipe deflection may be assumed to be equal to that of the in-situ soil. The AWWA Manual M45 Fiberglass Pipe Design Manual provides values of $M_s$ for both backfill (constructed soil) and in-situ soil, and the values for the in-situ soil should be used in design of CFRP liner for PCCP.
- Combined Pressure and Bending: The design of CFRP-lined PCCP is based on the sum of strain from internal pressure and bending due to gravity loads, reduced by a rerounding factor that occurs when the CFRP liner is subjected to internal pressure. A new formula is derived for the rerounding factor based on theory of elasticity.
- Buckling of CFRP Liner: The CFRP liner, either in the standalone condition or in composite with the inner concrete core is subject to buckling instability from external loads, groundwater pressure, and internal vacuum from transient pressure in the pipe. The draft standard presents design formula based on Glock equation with an adjustment factor that accounts for the flexibility of surrounding soil.
The design of CFRP-lined PCCP also accounts for longitudinal stresses in the pipe resulting from thermal tensile stresses, Poisson effect of circumferential tensile strains, thrust-induced tensile stresses, and longitudinal bending of the CFRP liner at the edge of an area where prestressing is lost. In such an area, the CFRP liner is subjected to longitudinal bending resulting from differential radial stiffness of PCCP between prestressed area and the area where prestressing is lost.

The CFRP-lined PCCP must be watertight by providing full bond between the CFRP and steel substrate at pipe ends. Furthermore the termination detail is of utmost importance to prevent water pressure build-up outside of the CFRP liner, thus rendering the CFRP ineffective. Prescriptive requirements are provided to ensure performance of the CFRP-lined PCCP subjected to design pressures and gravity loads as the host pipe continues to degrade during its service life.

INSTALLATION AND QUALITY CONTROL PROVISIONS

The installation and quality assurance provisions are presented in detail in the AWWA Draft Standard for CFRP Renewal and Strengthening of CFRP. These provisions represent the consensus of the committee on the relevant issues of installation and quality control.
CONCLUSIONS

The following conclusions are derived from the results of this study:

- Based on analysis of published durability test results for CFRP exposed to different environments, values of material adjustment factor are recommended for design as a function of service life of the CFRP liner.
- Based on analysis of published creep and creep-rupture tests results for CFRP, values of time effect factor are recommended for design strength and modulus as a function of service life of the CFRP liner.
- Characteristic value of material properties are defined from the results of laboratory tests of single-ply CFRP lamina as the 5th percentile value with 80% confidence (to account for sample size). The characteristic value will account for variability of material strength and modulus from different manufacturers and installers, and will allow the resistance factor to be independent of manufacturer and installation workmanship.
- Monte Carlo simulation shows that the strength of a laminate consisting of $n$ layers of CFRP lamina is less than the $n$ times the strength of a single-ply lamina and predictive formulas are presented for calculating the characteristic value of the strength of a laminate from that of a single-ply lamina.
- Based on reliability analysis of parameters that govern the design and the objective of having CFRP-lined PCCP to have the same reliability as other materials used in infrastructure, loads and resistance factors were determined. Load factors account for the variability of the loads, and resistance factors account for the variability of material resistance, independent of the manufacturing and installation workmanship.
- A set of simplified design equations are proposed for design of CFRP liner for internal pressures, gravity loads, combined gravity loads and internal pressures, and buckling from external loads and pressures. The simplified approach includes a new equation for rerounding of the flexible liner subjected to internal pressure.
- A nonlinear finite-element model of the concrete pipe wall and the surrounding soil was developed. The results obtained from the finite-element model of the test pipe for a three-edge bearing test were compared to the observations during the test and used to determine the validity and limitations of the finite-element model. Then, the finite-element model of pipe-soil system was subjected to the design loads, and the results were compared with the simplified design equations. The comparison showed that the simplified design equations are adequate and conservative.
- The nonlinear finite-element model was extended to determine the buckling strength of the CFRP liner. The results of analysis were used to determine a bias factor that accounts for soil flexibility to be applied to Glock’s equation for constrained buckling of flexible liners installed in rigid pipe.
Three-edge bearing and hydrostatic pressure tests were conducted to determine all likely failure modes. The results of three-edge bearing tests performed showed that CFRP debonding from concrete substrate is more governed by shear bond failure as a critical strain in CFRP than radial tension between CFRP and concrete inner core. Two failure limit states that were not initially included in the simplified design approach were determined: one involved longitudinal bending of the CFRP liner resulting from expansion of CFRP liner in areas where prestressing is lost due to wire breakage and the outer core is cracked; and the second one involved leakage resulting from poor bond between CFRP liner and steel joint rings and steel cylinder. These failure modes were included in the design process.

APPLICATIONS / RECOMMENDATIONS

The results of this research provide theoretical and experimental foundation for the development of an AWWA Standard on Renewal and Strengthening of PCCP, and are shared with the Standard Committee for this purpose. The Standard, once published, will assist professional engineers and utilities in design, material selection, and installation of CFRP liners in degraded PCCP lines. The results of this research will help ensure that the CFRP renewed PCCP, when properly designed using qualified materials and installed properly will have the necessary strength, durability, and reliability against all failure modes. Some of these failure modes were discovered during this research. As an example, hydrostatic pressure tests performed as a part of this research program to validate the designs from a strength point of view revealed new failure modes related to water tightness that had not been known or considered in the draft Standard. For the new water tightness failure modes, new provisions -- on design, material qualifications, and installation of the CFRP liner -- were included in the draft Standard, which will be presented to the Standard Committee for approval. In absence of such provisions in the Standard, water tightness would not be ensured and the pressure in the CFRP renewed PCCP could not reach the ultimate capacity of the pipe.
APPENDIX A
DETAILS OF FULL-SCALE TEST SPECIMENS
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tr>
<td>Mark Number (MX#)</td>
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<td>Pipe Size &amp; Type</td>
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<tr>
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</tr>
<tr>
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<td>11</td>
</tr>
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<td>Soil Unit Weight (w) (lbs/cu.ft.)</td>
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</tr>
<tr>
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<td>Internal Working Pressure (Pw) (psi)</td>
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</tr>
<tr>
<td>Internal Transient Pressure (Pt) (psi)</td>
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<td>Field Test Pressure (PT) (psi)</td>
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<td>External Live Load (Wt) (#/lf)</td>
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<td>Wire Pitch (ctr. to ctr.) (in)</td>
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<td>Reinforcing Band Thickness (in)</td>
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<tr>
<td>Fitting Cylinder Thickness (in)</td>
<td>5/16</td>
</tr>
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<td>5/16</td>
</tr>
<tr>
<td>Rodded Conc. Cyl. At Wrapping (psi)</td>
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<tr>
<td>Rodded Conc. Cyl. At 28 days (psi)</td>
<td>4500</td>
</tr>
<tr>
<td>Admixtures: W.R. Grace WRDA 79 may be used</td>
<td></td>
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</table>

General Notes:
1. All prestressed pipe sections will have the MARK NO. identification to indicate pipe strength.
2. Where a heavier cylinder gauge is required, it will be shown on the pipe laying schedule.
3. Designs include AASHTO HS-20 live load unless otherwise stated above.
4. Designs are suitable for a full internal vacuum condition (-14.7 psi).

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54 in. DIAMETER ECP FOR THREE-EDGE BEARING AND BOND STRENGTH TESTS  
(Pipe Length = 13 ft)

<table>
<thead>
<tr>
<th>Mark Number (MK#)</th>
<th>Pipe Size &amp; Type</th>
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<td>120</td>
<td>120</td>
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<td>(degrees)</td>
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<td>60</td>
<td>60</td>
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<td>250</td>
<td>250</td>
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<td>Internal Transient Pressure (Pt)</td>
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<td>(#/lf)</td>
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<td>774</td>
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<td>Coating Thk. (Over Wire)</td>
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<td>1/4 / 3/4</td>
<td>3/4</td>
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<td>Coating Thickness (Nominal)</td>
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<td>.42 / .42</td>
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<td>.83 / .83</td>
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</table>

**General Notes:**
- All prestressed pipe sections will have the MARK NO. identification to indicate pipe strength.
- Where a heavier cylinder gauge is required, it will be shown on the pipe laying schedule.

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## 48 in. DIAMETER LCP FOR HYDROSTATIC TEST
(Pipe Length = 20 ft)

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</tr>
<tr>
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<td>(ft)</td>
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<td>14</td>
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<tr>
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<td>(lbs/cu. ft.)</td>
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<td>120</td>
<td>120</td>
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<td>(psi)</td>
<td>30</td>
<td>30</td>
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<td>1</td>
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<td>#6 CL-3</td>
<td>#6 CL-3</td>
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<td>189000</td>
<td>189000</td>
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<tr>
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<tr>
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<tr>
<td>Rodded Conc. Cyl. At 28 days</td>
<td>(psi)</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
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</tbody>
</table>

**General Notes:**
1. All prestressed pipe sections will have the MARK NO. identification to indicate pipe strength.
2. Where a heavier cylinder gauge is required, it will be shown on the pipe laying schedule.
3. Designs include AASHTO HS-20 live load unless otherwise stated above.
4. Designs are suitable for a full internal vacuum condition (-1.4 psi).

**HANSON PRESSURE PIPE**
**HATTIESBURG, MS**
**DESIGN SHEET**

**Manhattan, Kansas**
**Water Treatment Plant Improvements**

Des'd & App'd:

RW/Nicky Wu
Date: 07-27-10
Rev.

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# 48 in. DIAMETER LCP FOR THREE-EDGE BEARING AND BOND STRENGTH TESTS

(Pipe Length = 20 ft)

<table>
<thead>
<tr>
<th>Pipe Design per AWWA C301 &amp; C304</th>
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<td>Mark Number (MK#)</td>
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<td>Pipe Size &amp; Type</td>
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<tr>
<td>Earth Cover (H)</td>
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<tr>
<td>Soil Unit Weight (w) (lbs/cu. ft.)</td>
</tr>
<tr>
<td>Bedding Angle (degrees)</td>
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<tr>
<td>Internal Working Pressure (Pw) (psi)</td>
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<td>Internal Transient Pressure (Pt) (psi)</td>
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<tr>
<td>Field Test Pressure (Pft) (psi)</td>
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<td>External Live Load (Wl) (#/12)</td>
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<tr>
<td>Resultant Core Prestress (for) (psi)</td>
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<td>Core Thickness Incl. Cyl. (hc) (in)</td>
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<tr>
<td>Coating Thk. (Over Wire) (in)</td>
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<tr>
<td>Coating Thickness (Nominal) (in)</td>
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<tr>
<td>Wire Size &amp; Class</td>
</tr>
<tr>
<td>Wire Area (As) (sq.in./lf)</td>
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<tr>
<td>Min. Tensile Strength of Wire (fsu) (psi)</td>
</tr>
<tr>
<td>Gross Wrapping Stress of Wire (fsg) (psi)</td>
</tr>
<tr>
<td>Wire Pitch (ctr. to ctr.) (in)</td>
</tr>
<tr>
<td>Pipe Cylinder Gauge (See Notes)</td>
</tr>
<tr>
<td>Reinforcing Band Thickness (in)</td>
</tr>
<tr>
<td>Pitting Cylinder Thickness (in)</td>
</tr>
<tr>
<td>Elbow Cylinder Thickness (in)</td>
</tr>
<tr>
<td>Rodded Conc. Cyl. At Wrapping (psi)</td>
</tr>
<tr>
<td>Rodded Conc. Cyl. At 28 days (psi)</td>
</tr>
</tbody>
</table>

**Note:**

Designs are based on the earth dead load calculated using the Marston trench equations with the maximum trench width at the top of the pipe not to exceed Pipe outside diameter plus 42" and type R3 bedding.

**General Notes:**

1. All prestressed pipe sections will have the MARK NO. identification to indicate pipe strength.
2. Where a heavier cylinder gauge is required, it will be shown on the pipe laying schedule.
3. Designs include AASHTO HS-20 live load unless otherwise stated above.
4. Designs are suitable for a full internal vacuum condition (-14.7psi).

**Price Brothers Company**

Hattiesburg, MS

**Design Sheet**

Bentonville, Arkansas

Contract Section 1

Des'd: 050403-DS02

App'd: 050403-DS02

Date: 01-30-06

Rev. 050403-DS02
REFERENCES


American Water Works Association 2012 Draft AWWA Standard for CFRP Renewal and Strengthening of PCCP.


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Fyfe Experiment data and Reports, provided by Fyfe Co. LLC.


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ABBREVIATIONS

ACI          American Concrete Institute
ASCE         American Society of Civil Engineers
ASTM        ASTM International (formerly American Society for Testing and Materials)
AWWA        American Water Works Association
BWZ          Broken wire zone
CFRP        Carbon fiber reinforced polymer
ECP          Embedded cylinder pipe
FEA          Finite element analysis
GFRP        Glass fiber reinforced polymer
LCP          Lined cylinder pipe
PCCP        Prestressed concrete cylinder pipe
SGH          Simpson Gumpertz & Heger Inc.
WaterRF     Water Research Foundation
3EB          Three-edge bearing test