TESTING, ANALYSIS AND CLASSIFICATION OF NO-DIG MANHOLE

REHABILITATION MATERIALS

by

ALIMOHAMMAD ENTEZARMAHDI

Presented to the Faculty of the Graduate School of

The University of Texas at Arlington in Partial Fulfillment

of the Requirements

for the Degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

THE UNIVERSITY OF TEXAS AT ARLINGTON

MAY 2015

Acknowledgements

Abstract

TESTING, ANALYSIS AND CLASSIFICATION OF NO-DIG

MANHOLE REHABILITATION MATERIALS

Alimohammad Entezarmahdi, M.S.

The University of Texas at Arlington, 2015

Supervising Professor: Mohammad Najafi

Gravity flow wastewater collection systems are comprised of sewer pipes and manholes. Failure of a manhole may have catastrophic consequences such as developing a sinkhole in the street and roadway, and at a minimum, wastewater flow will be blocked and stream of the manhole will backup causing a sanitary sewer overflow (SSO). Improving structural conditions of a manhole is critical to minimize these types of failures. This thesis considers the impact of several lining materials including cement mortar, epoxy, polyurethane, cured-in-place composites, and a multi- layer structure material on increasing the structural capabilities of deteriorated manholes. The tasks included in this thesis consist literature search and, preliminary laboratory and main testing of select manhole rehabilitation materials. A finite element analysis is included to complement the experiments. Several preliminary tests according to ASTM C-39 on coated concrete cylinders, and ASTM C-293 on lined concrete beams, were performed at UT Arlington's Center for Underground Infrastructure Research and Education (CUIRE) Laboratory. The test results showed significant increase in the performance of concrete samples under compression and flexure. A second round of testing was performed on 4-ft long, 24-in. diameter concrete pipe sections with 3-in. wall thickness manufactured according to ASTM C-76. These pipe sections were lined internally with the same materials as the preliminary

iv

tests, and tested according to ASTM C-497 under Three-Edge-Bearing testing. Using computer data acquisition system, strain gages and displacement extensometers, stress/strain data were measured. The results showed that tested No-Dig manhole rehabilitation materials can significantly improve structural performance of deteriorated manholes.

TABLE OF CONTENTS

List of Illustrations

List of Tables

Chapter 1

Introduction

1.1. Background

Manholes are called "windows" to the sewer system as they are the most visible points in identifying the condition of underground infrastructure (Najafi, 2005). In the USA alone, the total number of manholes is estimated to be around 20 million. Of those, it is estimated that 4 million manholes are at least 50 years old and another 5 million are 30 to 50 years old (Najafi, 2005). Experience to date suggests that several million manholes in the North America have deterioration problems due to hydrogen sulfide corrosion and structural loadings.

While utilities might benefit from many options and the competition among the manufacturers, determining the most feasible and economical rehabilitation material and method often imposes a challenge for design engineers and decision makers (Table 1.1).

Each of materials and methods listed in Table 1.1 has its pros and cons, with unknown results in enhancing structural capacity of a deteriorated manhole. The following parameters play a role in the overall structural durability and life cycle of a manhole:

- Residual strength of the manhole

- Mechanical properties of the lining material

- Adhesion between the lining and substrate (manhole component)

- Magnitude and type of loads exerted on the manhole

- Durability of manhole material against environmental effects (particularly to hydrogen sulfide induced corrosion).

Table 1.1. Common manhole problems (Najafi, 2005)

Defect	Description	Example Rehabilitation Material/Method
Inflow	Rain water entry into manholes through loose covers, and gaps on the frame and chimney. Manholes are the main source of inflow into wastewater collection systems.	Chimney restoration/sealing. Lid sealing or replacement.
Infiltration	Groundwater entry into manholes through cracks, fractures, and loose joints. Exfiltration of wastewater may occur if the groundwater table is below the manhole invert or bottom elevation.	Relining with structural or non-structural methods (discussed below)
Corrosion	More pronounced for concrete manholes those are subject to sulfuric acid attack. Sulfuric acid in manholes forms due to oxidation of hydrogen sulfide by sulfur oxidizing bacteria. Extensive corrosion may result in thinning of manhole wall and thereby trigger a structural failure.	Relining with structural or non-structural methods. Manhole inserts.
Cracks/ Fractures	Cracks and fractures typically occur as the result of poor construction, soil movements, inferior materials and external loads. They result in leaks. Depending on their extent and location, cracks and fractures will reduce manhole strength and impair function.	Relining with structural or non-structural methods. Manhole inserts.
Loose Joints	Displaced or open joints occur for the same reason as cracks and fractures. Manhole strength remains unaffected, but leaks result.	Relining of the interior or grouting (with cementations or polymeric grouts)

1.2. Existing Conditions of Manholes

Based on 2013 ASCE Report Card for America's infrastructure, the average grade for America's wastewater system is D which represents ''Poor'' condition while the average national GPA is D+ for all infrastructure sections combined (Figure 1.1). ASCE also states that the estimated capital investment needs for nation's wastewater and storm water systems to be $298 billion over the next 20 years.
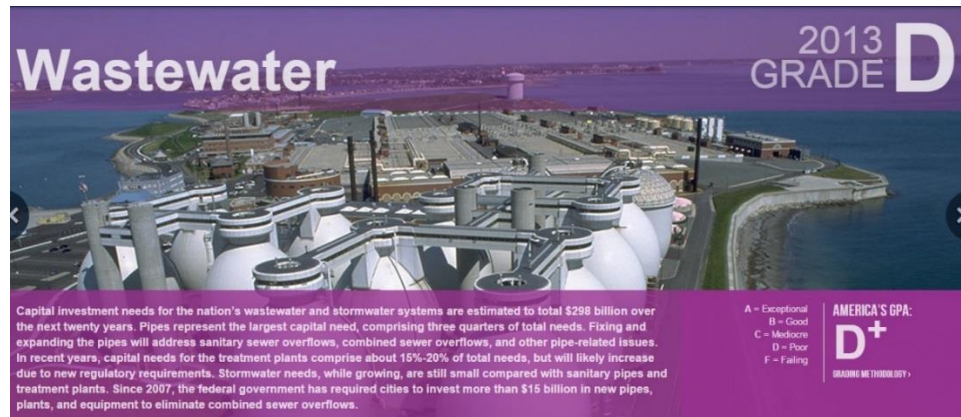
Figure 1.1. 2013 report card (Source: www.ASCE.org)

ASCE Texas Report Card 2012 (latest year available) for Texas Infrastructure grades wastewater systems as C− while this grade was C in 2008.

The reports mentioned above represent the overall condition of wastewater system in the US and state of Texas specifically. Therefore, it can be concluded that manholes, as a part of the wastewater systems require a considerable amount of investment in the next 20 years.

1.3. Advantages of Trenchless Rehabilitation Methods

Some advantages of using trenchless technology include minimum social costs and fewer disturbances to adjacent utilities and structures as well as minimum surface and subsurface excavations. Conventional open-cut construction methods involve the need to restore surfaces such as sidewalks, pavement, landscaping, and so on, which greatly increase the project costs. In addition to the increased costs, there are social and environmental factors associated with open-cut method, i.e., its adverse impacts on the community, businesses, and commuters due to air pollution, noise and dust, safety hazards and traffic disruptions (Najafi and Gokhale, 2005). No-Dig manhole rehabilitation methods can significantly reduce the social costs.

3

1.4. Need Statement

Inadequate structural capacity in deteriorated manholes is among the common issues that affect sewer systems and since manholes are one of the key elements of a sanitary sewer system, maintaining manholes in an acceptable condition can improve the quality of the whole system. On the other hand, deteriorated manholes with I&I issues may allow fines into the manhole and cause catastrophic consequences such as a sinkhole. Broken and separated parts of deteriorated manholes. Although, there have been a number of research studies on pipeline rehabilitation, manholes are often overlooked, though they can be the main source of inflow. On the other hand, considering the multi-billion dollar market (at least potentially), it is not surprising that there are already numerous materials and methods available for manhole rehabilitation. This wide variety in manhole rehabilitation has its pros and cons. Therefore, there are very limited studies, but numerous options to rehabilitate manholes.

1.5. Goals and Objectives

Based on the foregoing discussions, the objective of this thesis is to evaluate structural capabilities of available manhole rehabilitation material by conducting through conducting Preliminary and Main Tests, as are described in this thesis.

1.6. Research Methodology and Scope of Work

As stated previously, this research focuses on structural capabilities of No-Dig manhole rehabilitation materials. The required background information was collected through literature review. The following tests were conducted to measure the structural capabilities of the lining materials.

- Compression test per ASTM C-39 on lined concrete cylinders
- Flexural test per ASTM C-293 on lined concrete beams

4

- Three-edge bearing test on internally lined 24 in. diameter concrete pipe per ASTM C-497

Figure 1.2 outlines the process flow for the research tasks.

Literature Review
- Manhole Rehabilitation Materials
- Previous Studies
- Summary of Findings

Experimental and Analytical Evaluations
- Testing Standards
- Preliminary Testing on Manhole Rehabilitation Materials
- Preliminary Testing Results
- Main Testing on Manhole Rehabilitation Materials
- Main Testing Results
- Summary of Findings

Develop Conclusions and Recommendations
- Conclusions
- Limitations
- Recommendation for Future Researches

Figure 1.2. Research methodology flowchart

1.7. Expected Outcome

From test results conducted with this thesis, it is expected that all the manhole rehabilitation materials in this study, with some degree, enhance the structural capability of the host manhole. However factors such as the thickness, mechanical properties and quality of workmanship can affect the magnitude of increase in the structural capabilities of the lined manholes.

1.8. Chapter Summary

This chapter presented the existing condition of wastewater systems and specifically manholes in the U.S. Advantages of manhole rehabilitation were outlined. Goals and objectives, research methodology and expected outcome of this thesis were presented.

Chapter 2

Literature Review

2.1. Introduction

Sanitary sewer manholes are in a variety of materials and sizes. They are commonly made of brick-mortar, precast concrete block, and cast-in-place concrete. There are currently more than 20 million manholes in the united states, most of which are installed prior to 1960 (Najafi, 2005). A large number of the old manholes suffer from a variety of problems such as corrosion caused by sewage and gases, wearing due to traffic dynamic loads and erosion which may result in serious structural problems. Freeze and thaw, soil movements and hydrostatic loads are also among the factors that influence the deterioration of manholes. Considering the fact that a large number of manholes are located in the streets and open cut manhole rehabilitation methods may result in traffic disruptions on the streets, trenchless renewal methods can widely be used in rehabilitation of these manholes. Cementitious materials, polyurethane, epoxy, and cured-in-place are among the materials and methods that are used in No-Dig manhole rehabilitation and are discussed in this thesis.

2.2. Existing Guidelines on Manhole Rehabilitation

2.2.1. Manhole Inspection and Rehabilitation, ASCE Manual of Practice

Although, there is not a consensus on the capabilities of manhole rehabilitation and methods, there are a few publications that are intended to provide guidelines. A commonly recognized one was compiled by a rehabilitation committee formed under the ASCE. Entitled as Manhole Inspection and Rehabilitation, ASCE Manual of Practice (MOP) No. 92 (Hughes, 2009), provides basic information about inspection and trenchless or conventional rehabilitation of sanitary sewer manholes. A useful tool included in ASCE/MOP No. 92 is infiltration and inflow (I&I) rating based on visual inspection. This is

rather a qualitative method of rating than quantitative as the latter requires costly measurements that may not be justified, especially for small projects. In addition to any active I&I at the time of manhole inspection, the ASCE/MOP No. 92 rates the severity of I&I based on physical evidence such as water marks, corrosion on the metal components (i.e. frame and cover), mineral deposits, and soil intrusion. Similar tools are suggested for a structural rating. Visual observations that are indicative of the structural condition of a manhole include corrosion, cracks/fractures, missing parts pieces, and chipping/spalling.

ASCE/MOP No. 92 provides a basic classification for manhole rehabilitation materials and methods. This classification is comprised of chemical grouting, coating systems, structural linings, corrosion protection, and frame/cover/chimney rehabilitation.

Another tool provided in the ASCE/MOP No. 92 is a present worth analysis for each manhole rehabilitation method based on their market price and expected life cycle. Present worth analysis provided in the manual assumes structural rehabilitation will provide as long as a service life (50 years) as that of a new manhole.

Overall, ASCE/MOP No. 92 is a concise and basic manual that is intended to be an educational tool for manhole rehabilitation. It is useful in terms of learning manhole components, common defects and rehabilitation methods/materials, and the approximate cost of each rehabilitation method.

2.2.2. National Association of Sewer Service Companies (NASSCO)

A more comprehensive guidance on manhole rehabilitation was recently compiled by the National Association of Sewer Service Companies (NASSCO) as part of the organization's Inspector Training and Certification Program (ITCP) for manholes. The NASSCO[1] ITCP program is geared towards educating the field crew (i.e., engineers,

---

[1] http://www.nassco.org/

8

technicians, etc.), thereby certifying them as manhole inspectors .NASSCO ITCP provides a thorough review of the available manhole rehabilitation materials and methods in addition to guidelines on manhole inspection, quality assurance/control practices, and contracting for manhole rehabilitation projects. NASSCO guidelines do not provide an analysis on the structural capabilities of the materials and methods used for manhole rehabilitation; as such, this project could help enhance NASSCO's training program with the manhole rehabilitation material classification. Additionally, NASSCO ITCP does not include a thorough Decision Support System that factors in manhole, soil, groundwater, traffic, and surrounding environment of a manhole. This study and NASSCO ITCP essentially complement each other they could provide a complete set of tools for manhole inspection, condition assessment, decision making, rehabilitation, and testing for quality assurance and quality control.

2.2.3. Automotive System for Selecting Wastewater and Water Rehabilitation Material (TTC)

In a recent study, Matthews and Allouche (2012) developed an automated decision support system for assessing the suitability of trenchless technologies as decisions related to the rehabilitation of wastewater and water infrastructure as they are becoming increasingly more complicated with respect to the number and complexity of technologies in the marketplace. Established methods, such as cured-in-place pipe (CIPP), are constantly evolving, and new techniques continue to be developed in North America and around the globe.

To address the need, the Trenchless Technology Center (TTC)[2], in collaboration with the National Utility Contractors Association (NUCA), Australian Society of Trenchless

---

[2] http://ttc.latech.edu

Technology (ASTT), and NASSCO developed an interactive software for the evaluation of more than 70 technologies that can be employed in the installation, replacement, and rehabilitation of buried water and wastewater pipes (i.e., gravity driven and pressurized). Manholes were also included in the automated decision support system (DSS), which could be accessed through a Web-portal, "the Trenchless Assessment Guide for Rehabilitation (TAG-R)."

The authors describe "TAG-R" as a practical, easy-to-use, comprehensive DSS for the rehabilitation for potable water pipes and gravity sewer pipes. Manholes that provide access to sewer and drainage pipes for maintenance and inspection are also covered. There are two tables that list 14 methods that can be used for the maintenance and restoration of manhole structures or some of their components. The three primary conditions for renewal of manholes are:

- General maintenance for controlling infiltration/inflow,
- Applying a corrosion resistant barrier for wall corrosion, or
- Renewing the manhole structural integrity.

Condition 1: If the manhole is considered structurally sound with little indication of settlement, and/or was determined to have signs of structural fatigue (e.g., minor corrosion, infiltration/inflow through precast joints, mortar joints or around the pipe connections), then only general maintenance is required, which might include chemical grouting or cementitious repair. The corrosion level of a manhole can be minimal, light wall, or heavy wall. Light wall corrosion refers to a condition where the brick mortar is deteriorated and missing, or concrete surfaces are soft and flaking in spots. Heavy wall corrosion is evident

when bricks or mortar are missing in a number of areas, several inches of soft concrete exposed or sections of the wall surface are missing.

Condition 2: When the manhole is exhibiting signs of moderate structural distress [e.g., minor cracks, loss of mortar or bricks, concrete corrosion less than 0.5 in. (12.5 mm) in depth, or minor cross-sectional distortion less than 10%], but is still supporting the soil and live loads, a partially structural coating/corrosion barrier is recommended.

Condition 3: If the manhole is exhibiting signs of severe structural distress and/or collapse is imminent, a fully structural renewal is recommended. Conditions that indicate this degree of deterioration would be distortion greater than 10% of the manhole diameter, severe corrosion exposing the reinforcement steel or large sections of the structure being collapsed or missing altogether. Brick manholes lacking structural integrity have bricks missing in a number of areas with distortion in the wall.

Renewals beyond those mentioned above include bench repairs required when the bench is cracked and/or sections are missing, no bench currently exists, or groundwater infiltrating at the bench. Invert repairs are recommended if the invert is missing or eroded, the pipe running through the invert is fractured or dislodged, or the elevation does not match the elevations of the incoming and/or outgoing pipes.

The authors do not provide an in-depth evaluation of manhole rehabilitation products and methods. Also lacking is the testing results, properties of materials, and a DSS specifically provided for No-Dig manhole rehabilitation.

2.3. Previous Research on Manhole Rehabilitation

2.3.1. Loading and Deformation Conditions on Precast Concrete Manhole

Sabouni conducted doctoral research on loading/deformation conditions on precast concrete manholes (2008). The Sabouni study included full scale laboratory tests on three manholes. A total of 27 tests were run for different loading conditions; i.e., point

11

and distributed loads at different locations. The loadings were based on the truck loads specified in the Canadian Highway Bridge Code (CSA, 2006). In addition to the full scale tests, the study included numerical modeling with the finite element method (FEM) with 3-D elements to simulate the experimental setup as well as other simulations that represent in-situ loads on precast concrete manholes.

The experimental setup was built at a geotechnical testing facility at the University of Western Ontario. Three precast manholes were installed in a testing chamber that was filled with soil which was laid in the chamber in compacted layers. The depth of the manholes were 25 ft. Hydraulic jacks with 202 kip capacity were used on top of the manhole specimen to simulate truck loads for various loading conditions per the Canadian Highway Bridge Code. Strain gages were attached on throughout the manhole as well as on the steel reinforcements, where they were used (the experiments included steel reinforced and unreinforced concrete manholes). Additionally, stresses in the surrounding soil were measured using pressure cells with 102 psi capacity.

Results of Sabouni's experimental and numerical analyses suggest there is minimal, if any, tensile stress/strain along a wall of a manhole. The pattern of compressive strains is somewhat complex and dependent on load type/magnitude and depth. Figure 2.1. indicates compressive strains in the walls of the manhole test specimens.

Hoop strains are another type of deformation that occur on a manhole wall. This is mostly a compressive strain due to lateral soil pressure and analogous to the hoop strains that are encountered in pressurized pipelines, except the stress/strain tensor is in the opposite direction. The maximum strains measured due to hoop stress was between 0.002% and 0.003%, which is significantly below the cracking strain (0.008%).

Bending moments at the base of a manhole resulted in significant tensile strains. The maximum tensile strain measured on a 4-ft diameter manhole base was 0.0018%, which is 24% of the cracking strain for concrete.

Shear strains were not included in the experimental procedure. Shear forces on a manhole can be significant along non-circular wall/chimney parts and around the perimeter of the manhole on the base. Another condition of significant shear stress/strain is a lateral movement of manhole components, which could be detrimental for a liner installed on the manhole.

Sabouni study concludes that the building codes used for manhole design in North America are too conservative as the majority of the strains measured per the loadings, applied based on the CSA 2006, were substantially lower than cracking strains.

Sabouni's work is one of its kind and it is the only full-scale test applied on buried manholes with respect to their structural properties. The findings of the Sabouni study were utilized in designing and executing the tests and computational modeling conducted as a part of this thesis.

The following figure shows the axial vertical strain profile in the manhole structure under the following loading conditions:

a) Four concentrated loads of 15.7 kip each applied around the manhole cover by slow loading

b) A 63 kip distributed load applied on the manhole by slow loading

c) A 63 kip distributed load applied on the manhole by fast loading
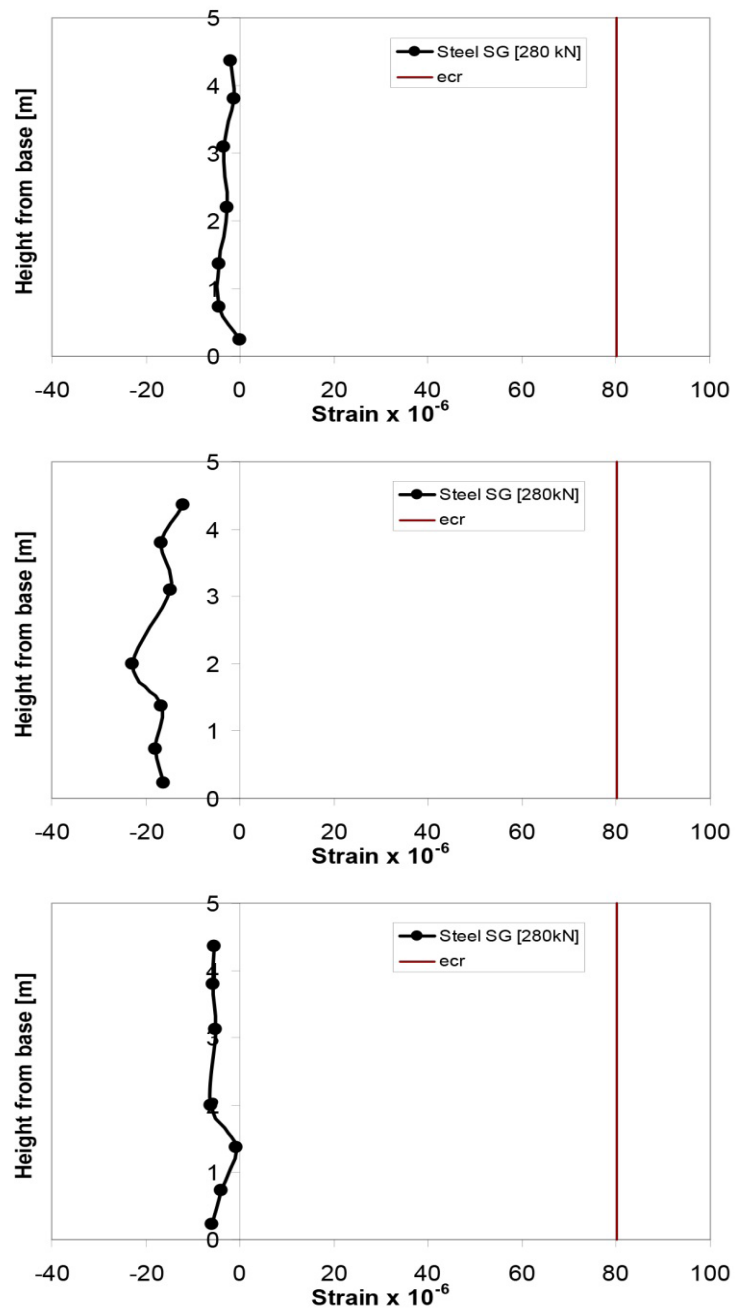
Figure 2.1. Axial vertical strain profile in a reinforced manhole wall

14

2.3.2. The full-scale Laboratory Tests on Manhole Rehabilitation Materials

In a recent experimental study conducted in Germany by IKT (Institute for Underground Infrastructure, 2012), performance of a select group of manhole rehabilitation materials and methods were investigated by implementing full-scale laboratory experiments and "in-situ analysis". The full-scale laboratory tests included 20 precast concrete manholes with an average height of 18 ft. The test manholes were connected to each other with PVC and stoneware pipes. Upon application of cementitious and polymeric coatings, the manholes were subjected to external hydrostatic loads to simulate site conditions with groundwater. Defects were formed (such as holes) on the sample manholes to represent a deteriorated manhole. Then these holes were initially sealed with cementitious and polymeric grouts and lined with cementitious and polymeric coatings/linings. The lined manholes were subjected up to 17 ft of hydrostatic pressure (external) for an extended period of time (five months). The results of the lab tests generally indicated a satisfactory performance for both cementitious and polymeric coatings. Cracking and staining was common on cement mortar linings, whereas the fundamental issue with the polymeric coatings was adhesion to the substrate. A better performance of polymeric grouts was observed in comparison with the cementitious grouts.

The second phase of the IKT study included in-situ inspection of 20 manholes that were coated with cementitious and polymeric linings that have been in service from 3 to 14 years. The 13 cementitious linings generally performed well; the fundamental issue pointed out with respect to cementitious linings was application of this type of linings without stopping infiltration into the manhole completely. This results in premature curing, thereby causing disintegration of the lining applied. The seven polymeric coatings inspected in field as a part of the IKT study underperformed with a number of defects that had formed within a fraction of design life. These defects noted on polymeric coatings inspected in field were

15

attributed to imperfections, such as cavities, on the substrate surface (manhole wall) that resulted in a rupture of polymeric coatings over the voids.

Based on the full-scale tests and field investigations, the IKT study recommended using specific materials and methods based on the condition of the manhole to be rehabilitated, i.e., if there is significant corrosion, cementitious linings were, at least as a substrate, recommended prior to applying a polymeric coating to compensate for the lost wall thickness and provide a better bond between the lining and substrate. The study also pointed out the importance of stopping leaks and creating a completely dry surface prior to application of any type of linings. In addition, surface preparation, by abrasive blasting was recommended, where polymeric coatings are applied to provide a stronger bond between the substrate and lining. The IKT study results also suggest that a manhole coating is "as strong as its weakest link" with respect to adhesion; i.e., uniform adhesive strength is needed to prevent detachment of the lining from the substrate (manhole).

2.3.3. Manhole Uplift Displacement and Trench-Backfill Settlement

Tobita et al. (2012) proposed a simple method to predict the uplift displacement of a manhole and trench-backfill settlement due to liquefaction. The authors proposed that conventional equilibrium of vertical forces acting on a manhole is solely a function of such forces acting and is incapable of predicting the uplift displacement.

The Tobita et al. method adds variables including the uplift displacement, Δf, and settlements of backfill, Δs, under the condition that the volume of an uplifted portion of a manhole is equal to a settled volume of a trench-backfill. To date, the method is verified through comparison with the results of 1-G and centrifuge model tests. To derive equations for estimation of displacement of a manhole uplift and backfill settlements attributable to liquefaction, the following assumptions are made:

16

- The volume of backfill is constant before and after the uplift; i.e., the uplifted portion of a manhole is equal to the settled volume of backfill.

- The groundwater depth in backfill is kept constant before and after the uplift because the duration of uplifting may be short enough for the groundwater to permeate into the ground above the water table.

- Pipes attached to the manhole are neglected for simplicity.

  The following considerations are provided for the analyses:

- Consideration of trench-backfill

- Weight of manhole and buoyant force

- Frictional force between backfill and side-wall of manhole

- Uplifting force from liquefaction

- Maximum manhole uplifts and backfill settlements

- Effects of groundwater depth and side-wall friction

- Effects of excess pore water pressure ratio

The uplift displacement and backfill settlements are derived as a function of thickness of the non-liquefied layer above the groundwater table, unit weight of backfill, width of the trench, and excess pore water pressure ratio. This method was verified through comparison with results of a shaking table test, boiling tests, and dynamic centrifuge model tests. Overall performance of the method was found to be acceptable. A new safety factor, which takes into account the amount of manhole uplift and backfill settlement, was proposed by authors and its performance was compared with that of the conventional one in which only the excess pore water pressure ratio is considered as a variable.

Dynamic effects on the manhole's uplift behavior, which is not considered in this study, may have to be investigated in detail for better estimation of manhole uplift. The predicted amount of backfill settlement by authors might be underestimated because

17

settlements attributable to consolidation after liquefaction are assumed to be zero for simplicity.

2.3.4. Rehabilitation of Polymer Concrete Manhole Rehabilitation

Another study was performed by Ahn et al. (2009) to check the feasibility of concrete polymer manhole rehabilitation through a development test of high strength polymer concrete and prepare fundamental data for design to solve the problems of an existing cement concrete manhole. The lower absorption capacity (0.39%) of polymer concrete is deemed more advantageous in installing manholes in areas with high groundwater table. Also long working-life (63 minutes) of polymer concrete would be adequate for a manhole application.

A testing program was developed, which includes the following parameters:

- Fillers

- Aggregate

- Shrinkage reducing agent

- Releasing agent

- Mixture proportioning

The following specific parameters were measured as a part of the testing procedure:

- Working-life

- Workability

- Ultimate mechanical strength

- Modulus of elasticity, and

- Poisson's ratio

Results of the Ahn et al. study indicated a specific gravity of polymer concrete as 2.30 (on average), its absorption capacity was 0.39% and its unit weight is not much

18

different from that of cement concrete. Nevertheless, its lower absorption capacity would be more advantageous in manholes exposed to groundwater.

The compressive and flexural strengths of polymer concrete were measured 18,400 psi and 3,200 psi, respectively. Such mechanical strength figures suggest polymer concrete has enough stiffness to build a new manhole with this material. 127

The Ahn et al. study is a useful reference with respect to considering polymer concrete as an alternative manhole rehabilitation material.

2.4. Chapter Summary

This chapter discussed the existing guidelines and published researches on manhole rehabilitation. Existing guidelines mainly focus on assessing the condition of the manholes and providing a quantitative or qualitative method of condition assessment.

Sabouni's research concluded that building codes used for manhole design in North America are too conservative. Tobita concluded that the uplift displacement and backfill settlements are derived as a function of thickness of the non-liquefied layer above the groundwater table, unit weight of backfill, width of the trench, and excess pore water pressure ratio.

Chapter 3

Experiment Details

3.1. Introduction

The objective of the experimental work was to evaluate the effects of linings on manholes with respect to their structural properties. A lined manhole is a composite system with two components. The contribution of the lining to the structural capabilities of a host manhole can be best determined by testing the composite (lined) manhole. Although ASTM Standards are used as a reference, the procedure developed herein is to fulfill objectives of the study using a two-step approach. First, a set of preliminary laboratory tests were carried out on small specimens to obtain a general idea on the structural capabilities of the linings. Second, a more elaborate main test procedure was developed based on the preliminary testing experience. These tests are discussed in the following sections.

3.2. Preliminary Tests

The materials included in the preliminary tests were two epoxy liners (EPX1 and EPX2), one polyurethane (PU), one corrosion resistant cement liner (CMT), one multi structural composite lining systems with modified pleurae and foam (MULT), and one resin impregnated cured-in-place lining system (CIP). Two types of tests (compression and flexural) were conducted on the lined (representative of manhole wall-lining system) and unlined (control) specimens regarding loads and stresses on manholes. Structural tests were conducted based on the ASTM standards indicated in Table 3.1 Forty two concrete cylinders (4 × 8 in.) and 41 concrete beam (3 × 3 × 11 in.) samples were prepared for the preliminary tests. The design strength of concrete was approximately 5,000 psi. The unlined (control) specimens prepared for the preliminary experiments are shown in Figure 3.4.

Table 3.1. Applicable ASTM standards

Type of Experiment	Applicable Standard
Compression	ASTM C39 - Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
Flexural	ASTM C293-Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading)

3.2.1. Flexural Strength Tests

Flexural strength of a solid is defined as its ability to withstand failure from bending. In concrete, it is generally measured by loading 6 in. × 6 in. × 20 in. concrete beams. Flexural tests are sensitive to specimen preparation, handling, and curing procedure. Standard specimens are heavy, and insufficient curing of specimen will yield lower strengths; therefore, in this experiment due to the need for sample shipment for lining purposes, a smaller specimen size, 3 in. × 3 in. × 8 in. was used. The 3 in. × 8 in. bottom surface was lined with a manhole rehabilitation material to compare the flexural behavior of lined concrete with that of the control specimen (unlined concrete beam).

Two of the widely used standards for testing concrete beams for flexural strength are: ASTM C78 and ASTM C293. In ASTM C78 a simple concrete beam was tested by three-point loading. In ASTM C293 a simple concrete beam was tested by center-point loading (Figure 3.1). ASTM C293 is followed for the preliminary experiments. The results of this test method may be used to determine compliance with specifications or as a basis for proportioning, mixing and placement operations. This test method produces values of flexural strength significantly higher than Test Method C78. The flexural strength found is expressed as the "Modulus of Rupture" (MR) in MPa or psi. A 60 KIP Baldwin flexure testing machine (see Figure 3.2) is used for the flexural strength tests.
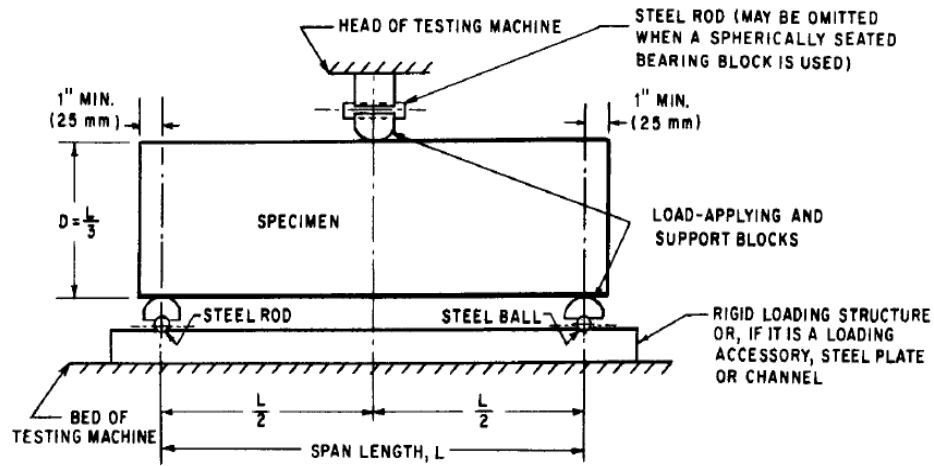
Figure 3.1. Flexural strength test setup per ASTM C293

Figure 3.2. Baldwin 60 KIP flexural strength testing machine

3.2.2. Compressive Strength Tests

Of the many tests applied on concrete, this is perhaps the most important test, which gives an insight about several characteristics of concrete. The purpose of preliminary compressive strength tests was to determine whether there is a significant strength addition by the lining materials to the substrate (standard concrete specimen per ASTM C39). This

preliminary step was used as a basis to move forward with a more elaborate experimental design to better understand at what degree No-Dig Rehabilitation materials can enhance the compressive strength of an actual manhole in the field (see the Main Tests section below). This is an important property in determining the structural class of the lining material as the majority of the stresses/strains observed on a typical circular cross-section manhole are compressive. Compressive strength tests are applied by using an Admet (500 kip) testing machine available at CUIRE(Figure 3.3)

Figure 3.3. The testing instrument (ADMET 500 KIP) for compressive strength tests

Two ASTM Standards considered in developing a compressive strength testing procedure are ASTM C39 and C109. ASTM C39 covers determination of compressive strength of cylindrical concrete specimens such as molded cylinders and drilled cores. ASTM C109 test method provides a means of determining the compressive strength of hydraulic cement and other mortars. It involves compressing 2-inch (50 mm) cube specimens to failure and results may be used to determine compliance with specifications.

3.2.3. Unlined (Control) Specimens

Forty two concrete cylinders (4 × 8 in.) and 41 concrete beams (3 × 3 × 11 in.) samples were prepared for the preliminary tests. The design strength of concrete was approximately 5,000 psi. The unlined (control) specimens prepared for the preliminary experiments are shown in Figure 3.4.
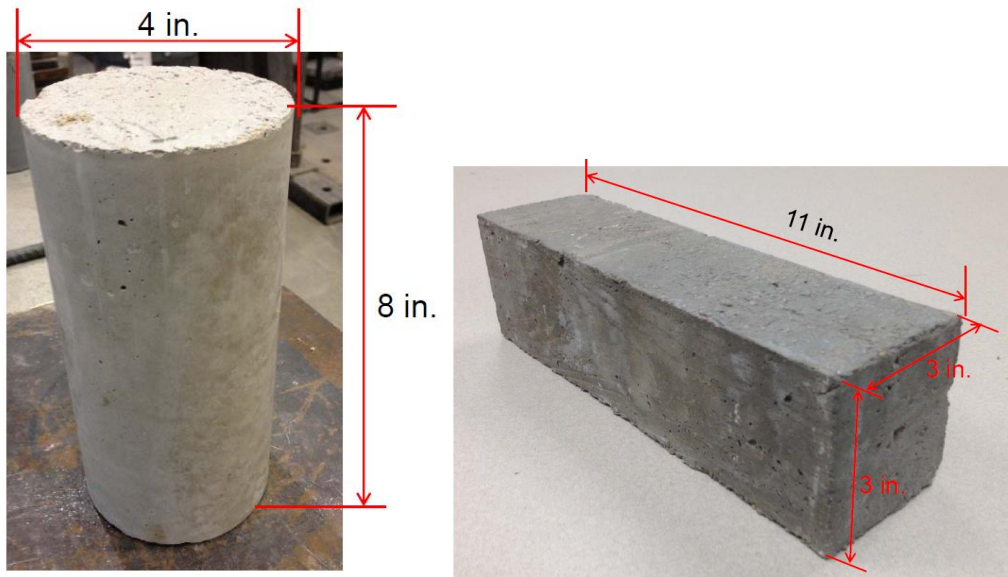
Figure 3.4. Typical concrete cylinder and beams used for the preliminary experiments

3.2.4. Lined Specimens

A total of 70 concrete specimens were sent to seven participating companies (10 each - five concrete cylinder and five beam specimens) for lining with their manhole rehabilitation product, using their standard procedure. Following instructions were given to manufacturers for lining:

1. Line the whole circumferential surface of cylinders evenly using your standard procedure.

2. Line one large surface of the beam. Smaller end surfaces need not be lined.

3. Thickness shall be minimum 100 mils (2.5 mm) for epoxy and polyurethane.

(a)   (b)

Figure 3.5. Concrete cylinder (a) and beam (b) samples lined with high-build

polyurethane

(a)   (b)

Figure 3.6. Concrete cylinder (a) and beam (b) samples lined with cured-in-place liner

3.2.5.   Test Methodology

Before the start of each test all the dimensions were measured using a calibrated

digital caliper (see Figure 3.7), both in the SI and US customary units, also thickness of

each lining was measured.

For the cylindrical samples, the diameter was measured vertically and horizontally

(see Figure 3.8). The average was taken to obtain a more accurate specimen diameter.

The height of the cylinder was measured at two locations and the average was used in the

calculations. Each cylinder was capped with sulfur capping material per ASTM C39. Once the specimen was capped, it was left idle for 2 to 24 hours before being tested.

For the beam samples, each surface of the specimen was measured in three different locations, and then the average was taken for calculations.

Figure 3.7. Digital caliper used for measuring specimen dimensions
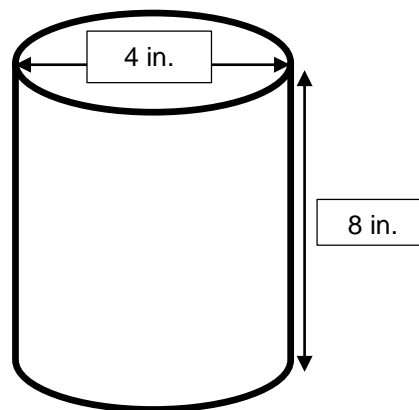
Figure 3.8. Cylinder dimension locations

3.2.6. Preliminary Test Results

The test specimens used for the preliminary tests were coded based on their material composition and the type of test applied upon. Table 3.2 indicates brief descriptions of the Linings tested and their respective codes. Five identical beam and cylinder specimens were lined by the manufacturer using their standard procedure (including surface prep) and tested at CUIRE by two-point bending and compressive loading until failure (Figure 3.9).

26

Table 3.2. List of specimen codes and their descriptions

Abbreviations	Description
UNL	Unlined concrete specimen
CIP	Concrete specimen with cured-in-place liner
EPX	Concrete specimen with Epoxy liner
PU	Concrete specimen with Polyurethane liner
CMT	Concrete specimen with Cementitious liner
MUL	Concrete specimen with Multi- Layer structural liner

(a)   (b)

Figure 3.9. Beam (a) and cylindrical (b) specimens at failure while loaded for flexural and

compressive strength, respectively

3.2.7. Unlined (Control) Specimens

The tests were performed on three unlined concrete beam and cylinder specimens. The test procedure followed was as per the ASTM standard indicated in Table 3.1. The testing was recorded with a camcorder, and once it was complete, pictures of the tested specimens were taken for records. Figures 3.10 and 3.11 show the views of the concrete cylinder before and after compressive loading, whereas Figures 3.12 and 3.13 show the concrete beam views before and after flexural loading.

Figure 3.10. Unlined (control) cylinder prior to compression testing

Figure 3.11. Unlined (control) cylinder after testing

Figure 3.12. Unlined (control) beam prior to flexural testing

Figure 3.13. Unlined (control) beam after testing

A summary of the results for the preliminary tests on control specimens is provided

in Tables 3.3 and Table 3.4.

Table 3.3. Summary of results for unlined (control) specimens

Sample ID	D×H* (in.)	Test Type	Test Date	Cross Section Area (in.²)	Loading Rate (lb/sec)	Peak Load (lb)	Peak Stress (lb/in.²)
UNL#1	3.9×8	ASTM C39	02/15/2013	11.95	<100	62,390	5,220.92
UNL#2	3.8×7.9	ASTM C39	02/15/2013	11.34	<100	60,783	5,360.05
UNL#3	4.1×8.1	ASTM C39	02/1 9/2013	13.21	<100	61,740	4673.73

*D: Diameter of cylinder, H: Height of cylinder

Table 3.4. Summary of results for the unlined (control) beams

Sample ID	Sample Dimensions W×H×L* (in.)	Test Type	Test Date	Cross Section Area (in.²)	Loading Rate	Peak Load (lb)	Peak Stress (lb/in.²)
UNL#1	2.9×2.8×10.9	ASTM C293	02/15/2013	8.12	4.1	1,924	962
UNL#2	2.7×2.8×11.1	ASTM C293	02/15/2013	7.56	4.1	2,013	1,007
UNL#3	2.7×2.9×10.8	ASTM C293	02/19/2013	7.83	4.1	1,564	782

*W: Width of beam, H: Height of beam, L: Length of beam

3.3.8. Lined Specimens

The following samples were tested in the preliminary tests. One epoxy lining materials (EPX1) one cured-in-place applied lining material (CIP), one polyurethane spray applied lining material (PU), one cementitious (CMT), and one multilayer polymer composite (MULT) for flexural strength and compressive strength. Five identical beam and cylinder specimens were lined by the manufacturer using their standard procedure (including surface preparation) and tested at CUIRE by three-point bending and compressive loading until failure.

(a)   (b)

Figure 3.14. Beam (a) and cylindrical (b) specimens at failure while loaded for flexural

and compressive strength, respectively

A summary of the preliminary flexural tests results is given in Figure 3.15.

Additionally, basic statistical analyses for the flexural strength test results are indicated in Tables 3.5 through 3.11.
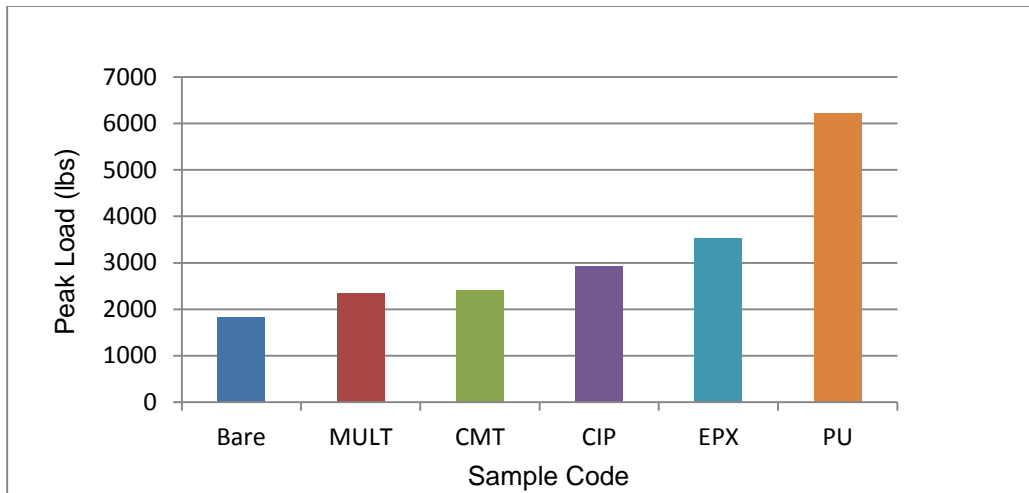
Figure 3.15. Ultimate flexural strength of unlined (bare) and lined specimens

Table 3.5. Epoxy flexural test result

Sample ID	Liner Thickness (mm)	Peak Load (lb)	% Difference Load
EPX1#1	7.7	4,251	(+)131.8
EPX1#2 (Minimum)	5.6	2,109	(+)15.0
EPX1#3 (Maximum)	5.7	4,500	(+)145.4
EPX1#4	4.9	2,318	(+)26.4
EPX1#5	5.5	3,590	(+)95.8
Average (A)	5.9	3,354	(+)82.9
Standard Deviation (S)	NA	1,095	NA
Upper Quartile (UQ)	NA	4,251	NA
Lower Quartile (LQ)	NA	2,318	NA
Interquartile Range (IQR)	NA	1,933	NA
Non-outlier Range	NA	-582 to 7,151	NA
Number of Outliers	NA	0	NA
Average Bare Sample Load: 1,834 lb			

Table 3.6. Cured-in-place flexural test result

Sample ID	Liner Thickness (mm)	Peak Load (lb)	% Difference Load
CIP#1	4.6	2,416	(+)31.7
CIP#2 (Minimum)	4.1	2,201	(+)20.0
CIP#3	4.9	2,607	(+)42.2
CIP#4	4.5	2,503	(+)36.5
CIP#5 (Maximum)	7.0	4,865	(+)165.3
Average (A)	5.0	2,918	(+)59.2
Standard Deviation (S)	NA	1,098	NA
Upper Quartile (UQ)	NA	2,607	NA
Lower Quartile (LQ)	NA	2,416	NA
Interquartile Range (IQR)	NA	191	NA
Non outlier Range	NA	2,130 to 2,894	NA
Number of Outliers	NA	1	NA
Average Bare Sample Load: 1,834 lb			

Table 3.7. Polyurethane flexural test result

Sample ID	Liner Thickness (mm)	Peak Load (lb)	% Difference Load
PU#1 (Minimum)	7.4	5,213	(+)184.3
PU#3 (Maximum)	7.4	7,382	(+)302.6
PU#2	7.9	6,989	(+)281.2
PU#4	8.7	6,728	(+)266.9
PU#5	8.9	5,514	(+)200.7
Average (A)	8.1	6,365	(+)247.1
Standard Deviation (S)	NA	949	NA
Upper Quartile (UQ)	NA	6,989	NA
Lower Quartile (LQ)	NA	5,514	NA
Interquartile Range (IQR)	NA	1,475	NA
Non outlier Range	NA	3,301 to 9,202	NA
Number of Outliers	NA	0	NA
Average Bare Sample Load: 1,834 lb			

Likewise, a summary of the compressive strength tests results are indicated in Figure 3.16. A basic statistical analysis for each lining test results is also indicated in Tables 3.9 through 3.11.

Table 3.8. Cementitious flexural test results

Sample ID	Liner Thickness (mm)	Peak Load (lb)	% Difference Load
CMT #1 (minimum)	3.6	2,205	(+)20.2
CMT #2 (maximum)	3.3	2,551	(+)39.1
CMT #3	3.5	2,390	(+)30.3
CMT #4	3.4	2,250	(+)22.7
Average (A)	3.45	2,349	(+)28.1
Standard Deviation (S)	N/A	156	N/A
Upper Quartile (UQ)	N/A	2,430	N/A
Lower Quartile (LQ)	N/A	2,239	N/A
Interquartile Range (IQR)	N/A	192	N/A
Non outlier Range	N/A	1,951 to 2, 718	N/A
Number of Outliers	N/A	0	N/A
Average Bare Sample Load: 1,834 lb			
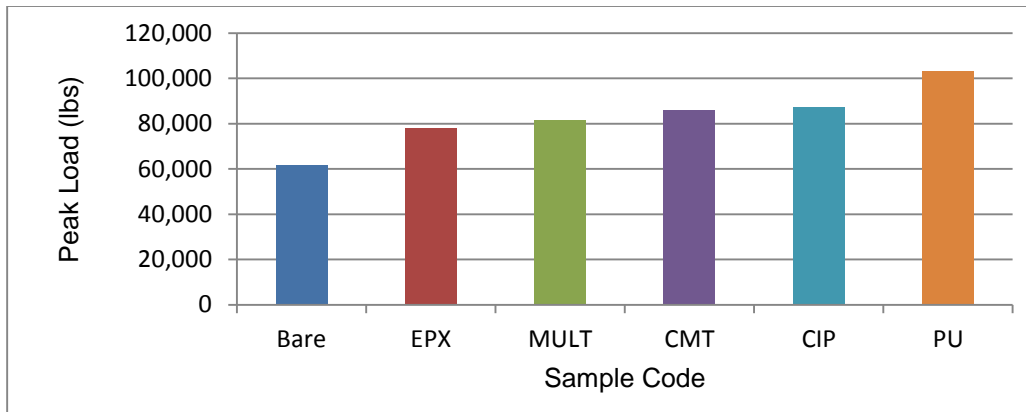
Figure 3.16. Ultimate compression strength of unlined (bare) and lined specimens

Table 3.9. Epoxy 1 compression test result

Sample Name	Liner Thickness (mm)	Peak Load (lb)	% Difference Load
EPX1#1 (Maximum)	3.1	85,520	(+)38.8
EPX1#2	3.2	80,730	(+)31.0
EPX1#3	2.8	75,800	(+)23.0
EPX1#4	3.0	82,510	(+)33.9
EPX1#5 (Minimum)	4.0	64,940	(+)5.4
Average (A)	3.2	77,900	(+)26.4
Standard Deviation (S)	NA	8,058	NA
Upper Quartile (UQ)	NA	84,015	NA
Lower Quartile (LQ)	NA	70,370	NA
Interquartile Range (IQR)	NA	13,645	NA
Non outlier Range	NA	49,903 to 104,483	NA
Number of Outliers	NA	0	NA
Average Bare Sample Load: 61,637 lb			

Table 3.10. Cured-in-place compression test result

Sample Name	Liner Thickness (mm)	Peak Load (lb)	% Difference Load
CIP1 (Minimum)	6.3	76,470	(+)24.1
CIP2	6.4	85,940	(+)39.4
CIP3	6.2	81,180	(+)31.7
CIP4	7.0	80,700	(+)30.9
CIP5 (Maximum)	6.6	111,330	(+)80.6
Average (A)	6.5	87,124	(+)41.4
Standard Deviation (S)	NA	13,941	NA
Upper Quartile (UQ)	NA	98,635	NA
Lower Quartile (LQ)	NA	78,585	NA
Interquartile Range (IQR)	NA	20,050	NA
Non outlier Range	NA	48,510 to 128,710	NA
Number of Outliers	NA	0	NA
Average Bare Sample Load: 61,637 lb			

Table 3.11. Polyurethane compression test results

Sample Name	Liner Thickness (mm)	Peak Load (lb)	% Difference Load
PU#1	4.4	115,020	(+)86.6
PU#2 (Minimum)	3.6	87,380	(+)41.8
PU#3	4.6	101,340	(+)64.4
PU#4 (Maximum)	4.3	116,170	(+)88.5
PU#5	4.4	96,960	(+)57.3
Average (A)	4.3	103,374	(+)67.7
Standard Deviation (S)	N/A	12,252	N/A
Upper Quartile (UQ)	NA	115,595	NA
Lower Quartile (LQ)	NA	92,170	NA
Interquartile Range (IQR)	NA	23,425	NA
Non outlier Range	NA	57,033 to 150,733	NA
Number of Outliers	NA	0	NA
Average Bare Sample Load: 61,637 lb			

3.3. Main Tests

The preliminary tests helped to gain an overall understanding of the capabilities of spray applied and cured-in-place linings with respect to adding strength to concrete specimens per the ASTM Standards C39 and C293. Nevertheless, two questions remained upon completion of the preliminary test:

1. How would an internally lined concrete cylinder, that is more representative of an actual manhole, behave under compressive and tensile stresses?

2. Is there a practical way to analyze the lined system under compressive strains?

To answer these questions, it was decided to use 24-inch concrete cylinders as substrates for the main tests. The concrete cylinders are standard reinforced concrete pipes manufactured to ASTM C76 and typically used for storm sewer applications. The main test samples were loaded by applying the standard Three-edge-bearing test per ASTM C497. The ASTM C497 procedure was slightly modified to install strain gages at four positions (i.e., 12:00, 3:00, 6:00, and 9: 00 o'clock) to measure the tensile and compressive stresses along the internal perimeter of the host pipe (see Figure 3.17). This allowed a comparison of lined specimens strains to that of unlined (bare) specimens during loading.

3.3.1. Testing Procedure

The main test procedure was comprised of five steps:

1. Surface preparation by the manufacturer certified contractor – Each contractor applied the surface preparation procedure per the manufacturer's instructions. This process varies depending on the lining type.

2. Strain gage installation– Upon completion of surface preparation, the strain gages were installed to measure strains during loading at the locations shown in Figure 3.17.

3. Liner installation by the manufacturer certified contractor – Each contractor installed the liner per the manufacturer's instructions. This process varies depending on the lining type.

4. Three-edge-bearing Test per ASTM C497 – This test was applied on control (unlined) and lined specimens. The standard ASTM procedure was slightly modified to measure strains during loading.

5. Reporting of the results – Load (lb) and strain (in./in.) were measured during the test process and the data were transferred to a processing unit incorporated into the test setup. The data processing unit included a PC, wiring, and pertaining software.
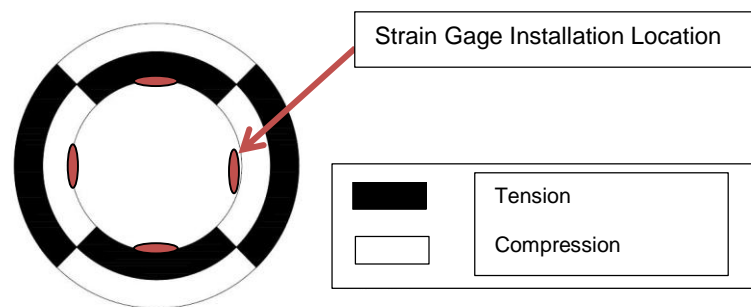

Figure 3.17. Stress/strain distribution along the pipe section loaded per ASTM C497 (D-Load) test and strain gages installation location

Chapter 4

Discussion of Results

4.1. Discussion of Preliminary Test Results

The preliminary tests results suggested that spray applied and cured-in-place linings could significantly, if not substantially, add to the ultimate flexural and compression strengths of concrete substrates. These results were at comparable thicknesses to actual manholes. The added flexural strength to the concrete substrate varied with thickness to a level that the difference in the ultimate strength was affected more by the lining thickness than the material itself for several cases. Nevertheless, the ultimate flexural strength versus liner thickness plot indicated in Figure 4.1 suggests the liner thickness does not have a significant effect on the flexural strength of the lined specimens for thicknesses greater than 7.0 mm (275 mils).

For instance, for CIP the ultimate flexural strength (or peak load at failure) increases almost linearly as the liner thickness increases up to 7.0 mm, which was the thickest in terms of the application of this product. Whereas ultimate flexural strength of EPX1 appears to increase drastically for the thicknesses greater than 5.6 mm, and there seems to be no effect of thickness on the ultimate strength for this material with respect to the thickness range from 5.7 to 7.7 mm. Flexural test on polyurethane specimens was applied at a minimum thickness of 7.4 mm. The thickness within the range of application of this material for the preliminary tests did not have any significant effect on the ultimate flexural strength of the lined concrete beam as lower peak loads were observed on thicker lined samples.

The PU specimens failed in a different pattern than the other spray-on linings did as the cracking started at the center of the concrete substrate, but then advanced towards supports with substantially higher deviations from the center. This can be attributed to the

42

increased effects of the flexural stiffness added to the substrate towards the edge, and this became more apparent as the cracking got closer to the bottom. It is more than likely that this has caused the concrete to crack due to the shear stress along the longer dimension resulting in failures near the support or fracture of the concrete without a rupture of the liner as was the case for one specimen (PU#5). The failure pattern observed on PU#5 suggests the adhesive strength between the polyurethane linings fell short of the shear strength along the long side of the specimen and flexural strength of the lined system.

Vertical fracturing with a clear gap (up to 5 mm or 200 mils) was observed on the fifth CIP specimen at failure; whereas, the other four specimens failed with minor circumferential and vertical cracks in hairline patterns.
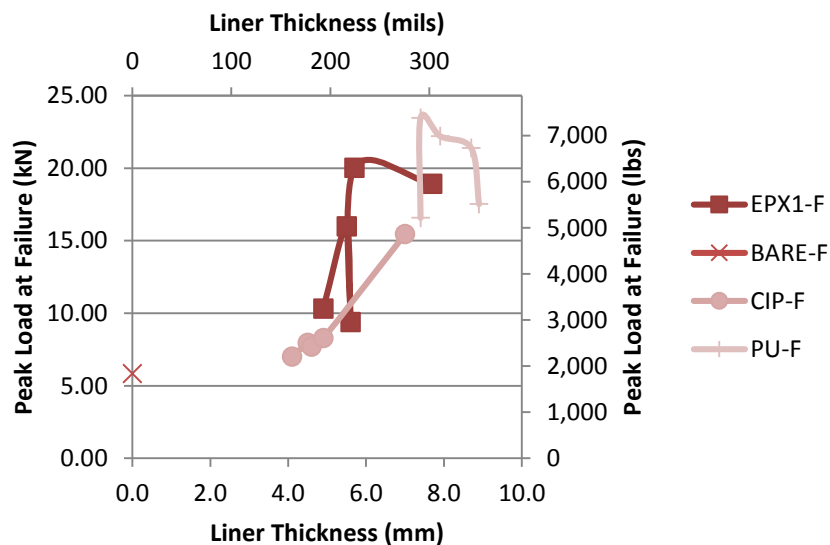
Figure 4.1. Peak load at failure (ultimate flexural strength) versus liner thickness based on the preliminary tests

Compressive strength of the lined cylinders did not appear to be affected by the thickness of the liner significantly for the thickness range applied on the samples for the preliminary tests. Part of the reason for the insignificant effect of thickness on the

compressive strength is the narrow range in which CIP, EPX1, CMT, and PU were applied (within 1 mm or 40 mils).

On the other hand, one can argue that due to possible expansion-contraction of the lining around the concrete cylinder during compressive loading, the added ultimate strength to the concrete cylinders was more of a result of the "confining effect" of the liner on the substrate. This may as well be the case for test specimens as they were lined on the exterior under compression, which is not the case for actual manholes in the field. The stress/strain distribution over the lined cylinders were not investigated as part of the preliminary tests; and therefore, the exact behavior of the lined cylinders under the given loading condition is unknown.

The experience on testing lined specimens gained through the preliminary tests was utilized to design the main tests on concrete cylinders. The main test procedure and results are discussed in Section 4.2 below.

Figure 4.2. Peak load at failure (ultimate compressive strength) versus liner thickness based on the preliminary tests
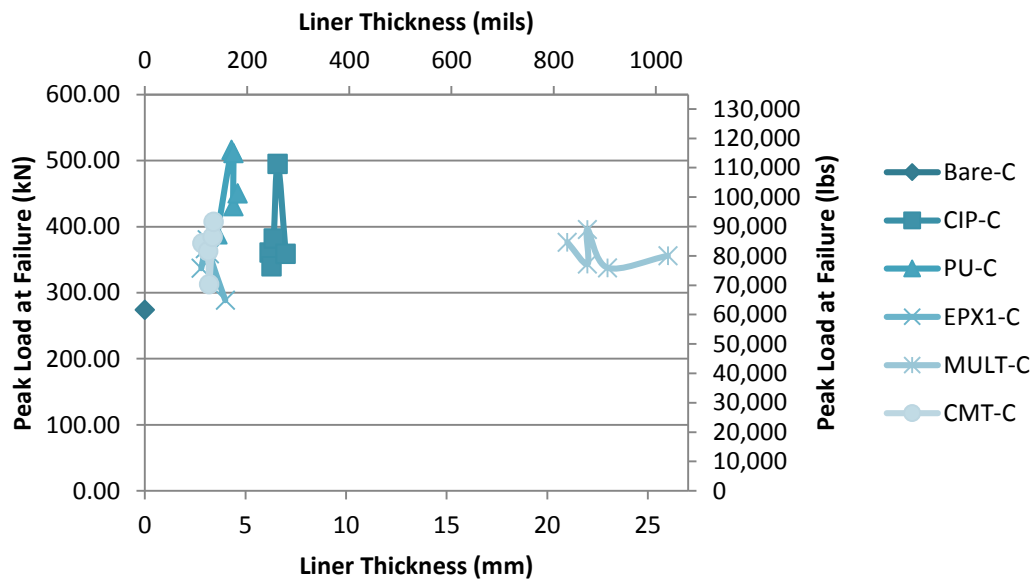
4.2. Discussion of Main Test Results

The results for mainl tests show significant increase in flexural and compression strength of concrete pipes. Figure 4.4 illustrates how the average peak load is increased as a result of applying lining materials on the inner surface of concrete pipes. Similar to the preliminary testing, thickness of the liner and the type of liner were among the key factors that affected the structural capabilities of concrete pipes. The other factors are quality of installation, substrate preparation, weather conditions (temperature, humidity, wind, etc.), and curing time.

Figure 4.3. Three-edge-bearing test setup per ASTM C497

The results of 250-mil thickness specimens for Epoxy, Polyurethane and Multi-Structural liners show that material property has a major impact on ultimate strength of manholes with epoxy having more impact than other two materials. Although thickness is a key factor in increasing the strength, it does not necessary mean that higher thickness of the liner will result in significant increase in the failure load. Test results of the samples with 125, 250 and 500 mils of polyurethane liner illustrated that liner thickness did not increase strength as expected when the liner thickness was increased from 250 to 500 mils.
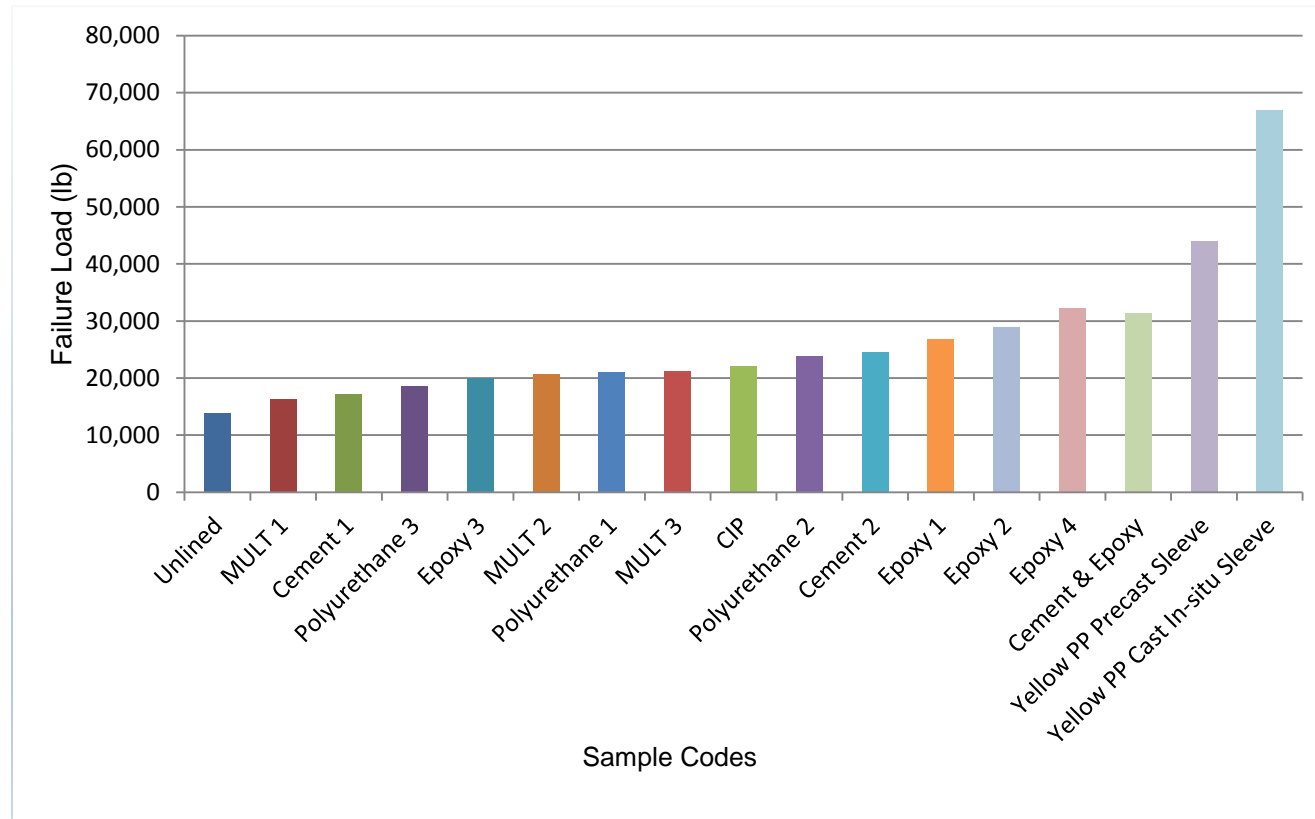
Figure 4.4. Main test results

Figure 4.5. Example of simultaneous cracking in the concrete substrate and liner

Figure 4.6. Example of failure by cracking in the concrete substrate only

4.3. Summary of Experimental Test Results

The preliminary and main test results indicated that the lining materials have significantly increased the structural capabilities of the concrete samples. For instance, epoxy lining material increased the peak load between 45 to 133 percent in main testing based on the thickness of the liner. Similarly, cementitious lining material increased the

47

peak load between 25 to 127 percent, polyurethane between 34 to 52 percent, and cured-in-place lining materials increased the peak load by 54 percent (see Table 4.4).

Table 4.1. Comparison of test results for specimens with a liner thickness smaller than 250 mils with the bare pipe

Type and Sample Code	Liner Thickness (mils)	Average Peak Load (lb)	Peak Strain in Compression (in./in.)* 10^{-6}	Peak Strain in Tension (in./in.)* 10^{-6}	Peak Stress in Compression (psi)	Peak Stress in Tension (psi)	% Failure Load Diff.
Bare Pipe	N/A	13,826	-250	105	1,250	525	N/A
Epoxy 1	150	26,846	-210	465	N/A	N/A	94
Epoxy 2	250	28,846	-204	76	N/A	N/A	108
Epoxy 3	250	20,013	-78	N/A	N/A	N/A	45
Epoxy 4	250	32,188	-164	430	834	2,187	133
MULT 2	250	20,663	-126	207	N/A	N/A	49
MULT 3	250	21,134	-160	3,685	N/A	N/A	53
Polyurethane 1	125	20,961	-231	180	1,300	6,642	52
Polyurethane 2	250	23,824	-54	121	435	968	72
CIP	250	22,050	-1,444	406	N/A	N/A	59
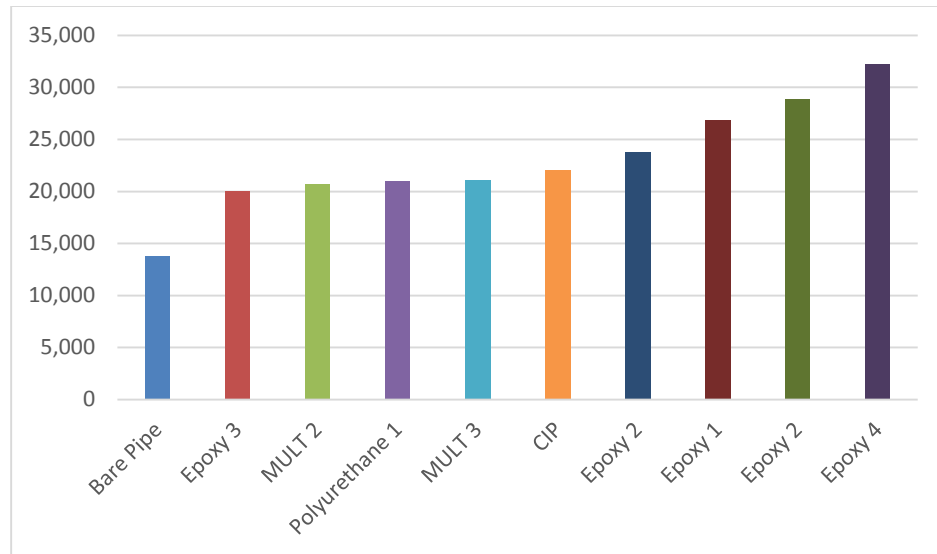
Figure 4.7. Main test results for specimens with a liner thickness smaller than 250 mils

with the bare pipe

Table 4.2. Comparison of main test results for specimens with liner thickness between

250 mils and 1000 mils with bare pipe

Type and Sample Code	Liner Thickness (mils)	Average Peak Load (lb)	Peak Strain in Compression (in./in.)* 10^{-6}	Peak Strain in Tension (in./in.)* 10^{-6}	Peak Stress in Compression (psi)	Peak Stress in Tension (psi)	% Failure Load Diff.
Bare Pipe	N/A	13,826	-250	105	1,250	525	N/A
Cement 1	500	17,261	-148	60	N/A	N/A	25
Cement 2	1,000	24,485	-147	384	1,038	2,712	77
MULT 1	1,000	16,332	-146	87	N/A	N/A	18
Polyurethane 3	500	18,578	-248	102	1,751	720	34

49
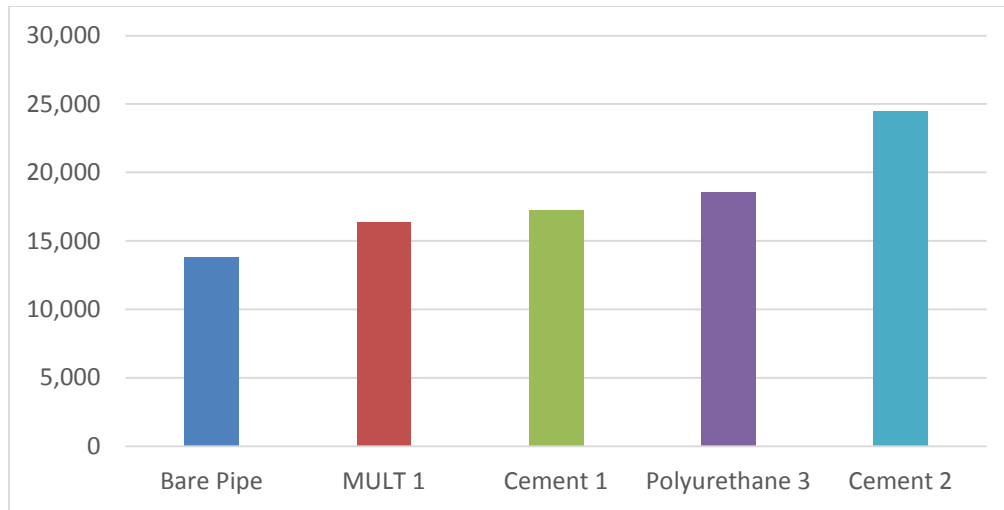
Figure 4.8. Main test results for specimens with liner thickness between 250 mils and

1000 mils with bare pipe

Table 4.3. Comparison of main test results for specimens with liner thickness over 1000

mils with bare pipe

Type and Sample Code	Liner Thickness (mils)	Average Peak Load (lb)	Peak Strain in Compression (in. /in.)* 10^{-6}	Peak Strain in Tension (in./in.)* 10^{-6}	Peak Stress in Compression (psi)	Peak Stress in Tension (psi)	% Failure Load Diff.
Bare Pipe	N/A	13,826	-250	105	1,250	525	N/A
Cement 2	1,000	24,485	-147	384	1,038	2,712	77
Cement & Epoxy	1,125	31,459	-134	69	955	491	127
MULT 1	1,000	16,332	-146	87	N/A	N/A	18
Yellow PP Precast Sleeve	3,000	44,087	N/A	297	N/A	N/A	219
Yellow PP Cast In-situ Sleeve	3,000	66,910	-473	759	N/A	N/A	384
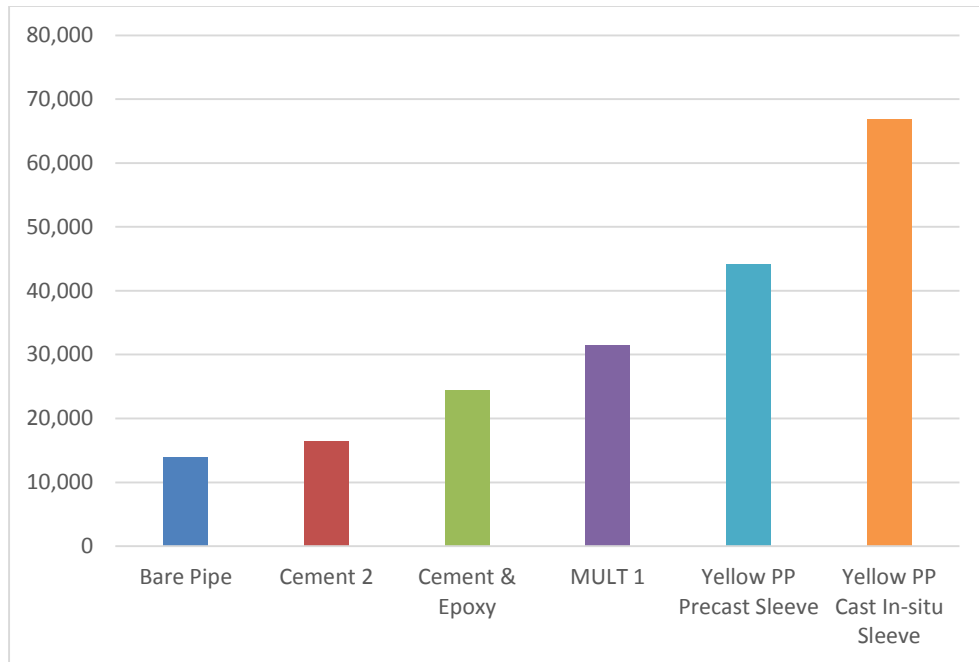
Figure 4.9. Main test results for specimens with liner thickness over 1000 mils with bare
pipe

Table 4.4. Summary of test results

Type and Sample Code	Liner Thickness (mils)	Ave. Peak Load (lb)	Peak Strain in Comp. (in./in.)* 10⁻⁶	Peak Strain in Tension (in./in.)* 10⁻⁶	Peak Stress in Comp. (psi)	Peak Stress in Tension (psi)	Max. Defl. (in.)	% Failure Load Diff.	Failure Mode
Bare Pipe									
Bare Pipe	N/A	13,826	-250	105	1,250	525	0.0621	N/A	N/A
Epoxy									
Epoxy 1	150	26,846	-210	465	N/A[3]	N/A	0.045	+94	C[4]
Epoxy 2	250	28,846	-204	76	N/A	N/A	0.019	+108	C

---

[3] This value could not be calculated because the mechanical properties of the liner were not available.
[4] Both Liner and Concrete pipe failed with longitudinal cracks on the outside at 3 and 9 o'clock and internal longitudinal cracks at 12 and 6 o'clock.

51

Table 4.4 Continued

Epoxy 3	250	20,013	-78	N/A[5]	N/A	N/A	0.2618	+45	C
Epoxy 4	250	32,188	-164	430	834	2,187	0.1614	+133	C
Cement									
Cement 1	500	17,261	-148	60	N/A	N/A	0.1693	+25	C
Cement 2	1,000	24,485	-147	384	1,038	2,712	0.0098	+77	C
Cement & Epoxy									
Cement & Epoxy	1,250	31,459	-134	69	955	491	0.0076	+127	C
MULT									
MULT 1	1,000	16,332	-146	87	N/A	N/A	0.0551	+18	N.C.
MULT 2	250	20,663	-126	207	N/A	N/A	0.1181	+49	N.C.
MULT 3	250	21,134	-160	3,685	N/A	N/A	0.1231	+53	N.C.
Polyurethane									
Polyurethane 1	125	20,961	-231	180	1,300	6,642	0.2283	+52	C
Polyurethane 2	250	23,824	-54	121	435	968	0.1102	+72	C
Polyurethane 3	500	18,578	-248	102	1,751	720	0.1181	+34	N.C.[4]
Thermoplastic									
Yellow PP Precast Sleeve	3,000	44,087	N/A	297	N/A	N/A	0.0164	+219	N.C.
Yellow PP Cast In-situ Sleeve	3,000	66,910	-473	759	N/A	N/A	0.0093	+384	N.C.
CIP									
CIP	250	22,050	-1,444	406	N/A	N/A	0.9382	+59	N.C.

---

[5] This value could not be calculated because the strain gage was damaged during lining process.
[4] Concrete pipe failed with longitudinal cracks on the outside at 3 and 9 o'clock but the liner did not fail.

4.4. Finite Element Modeling

Chapter 3 discussed the preliminary and main experimental tests designed and conducted on concrete specimens to measure the structural capabilities of No-Dig manhole rehabilitation. As mentioned in the previous chapters, the main test specimens were instrumented using strain gages and displacement transducers which were connected to a data acquisition system to record the readings. The test data was used to develop a Finite Element Model. This section presents the results of the FEM analysis on epoxy lining material as a part of the project titled Structural Capabilities of No-Dig-Manhole Rehabilitation Material sponsored by the Water Environment Research Foundation (WERF). Although detailed discussion of the FEM design procedure is not in the scope of this thesis, FEM results and conclusions are presented below.

4.4.1 Methodology

A multipurpose finite element software (ABAQUS) was used to study the structural capability of the epoxy liners. Simulations were performed on three cases: flexural beam test, pipe crushing test, and manhole structure under uniform soil and hydrostatic pressure (Figure 4.10). The experimental test results were used to calibrate the accuracy of the FEM.
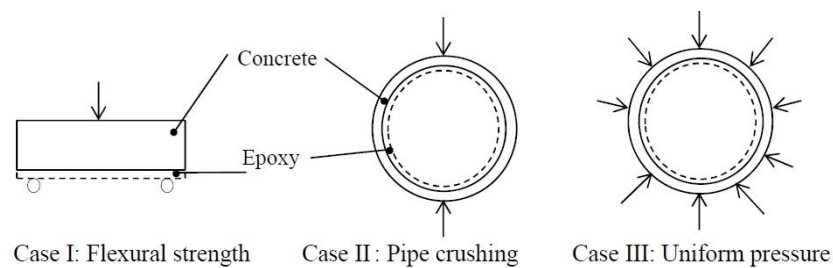
Figure 4.10. Schematic diagram of three simulated cases

4.4.2. Finite Element Analysis Results

4.4.2.1 Flexural Strength

Figure 4.11. shows that the failure load obtained from simulation was 2,165 lb with a deflection of 0.0019 in. which is in good agreement with the average peak load from laboratory tests on bare concrete (1,834 lb). The failure load and deflection for lined specimen were 2,922 lb and 0.0051 in. respectively (Figure 4.12).

Figure 4.11. Crack propagation in the bare concrete beam
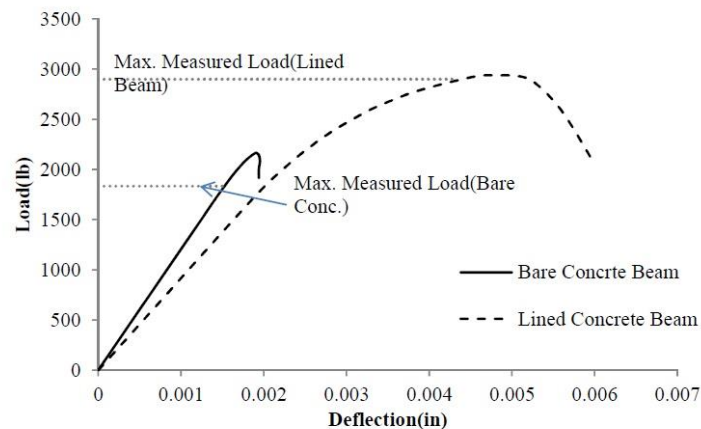
Figure 4.12. Load deflection curve from ABAQUS simulation for bare concrete

(Riahi, 2014)

4.4.2.2. Pipe Crushing
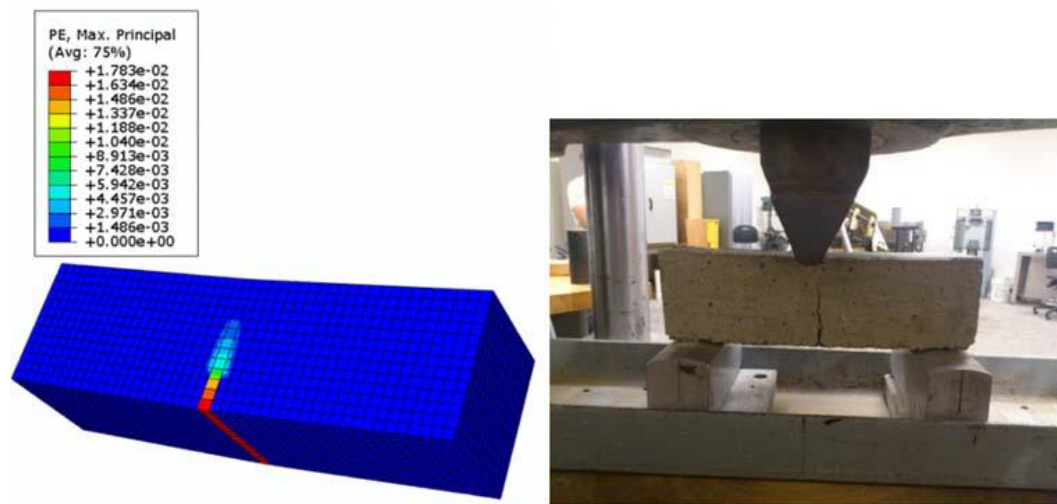
As it is illustrated in Figure 4.13, the bright areas in the plastic strain contour which represent the predicted cracks match with the cracks occurred in the Three- edge-bearing tests. Pressure-deformation curves obtained from the simulations in this case are shown in Figure 4.14 which indicates a peak pressure of 17.8 psi at 0.021 in. deformation for bare concrete pipe.

Figure 4.15 illustrates that using epoxy liner with a cohesive interaction behavior increases the pressure capacity of the pipe.

Figure 4.13. Crack propagation in bare concrete pipe

Figure 4.14. Pressure deformation curve of bare and lined concrete pipes

(Riahi, 2014)

4.4.2.3. Uniform Pressure

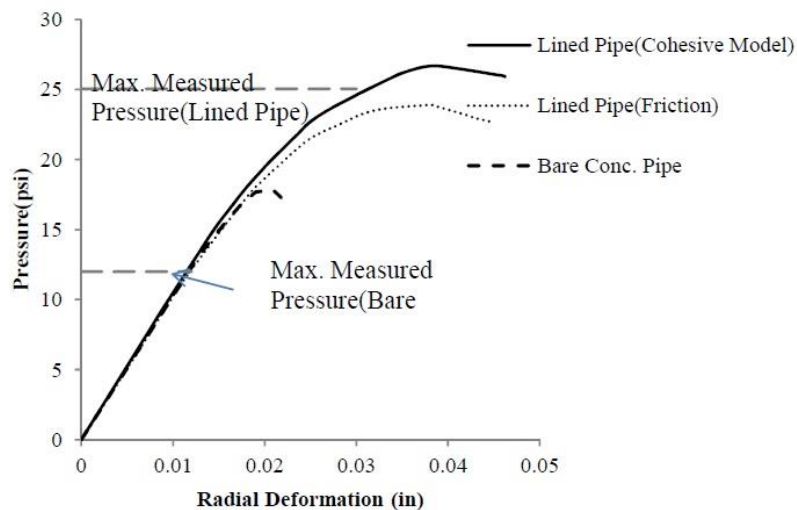Figure 4.15 presents pressure-deformation curve obtained from the lined and unlined pipe simulations under uniform peripheral pressure. The pressure shown in the plot is the concrete radial stress at the outside boundary. The simulations were performed for four models: bare concrete pipe, lined concrete pipe with friction contact, lined pipe with cohesive contact, and bare concrete pipe with increased thickness equal to the total thickness of lined concrete pipe. The curves show that bare concrete with larger thickness can carry the largest pressure as expected. However, the pipe with lining does not show increase of the pressure.
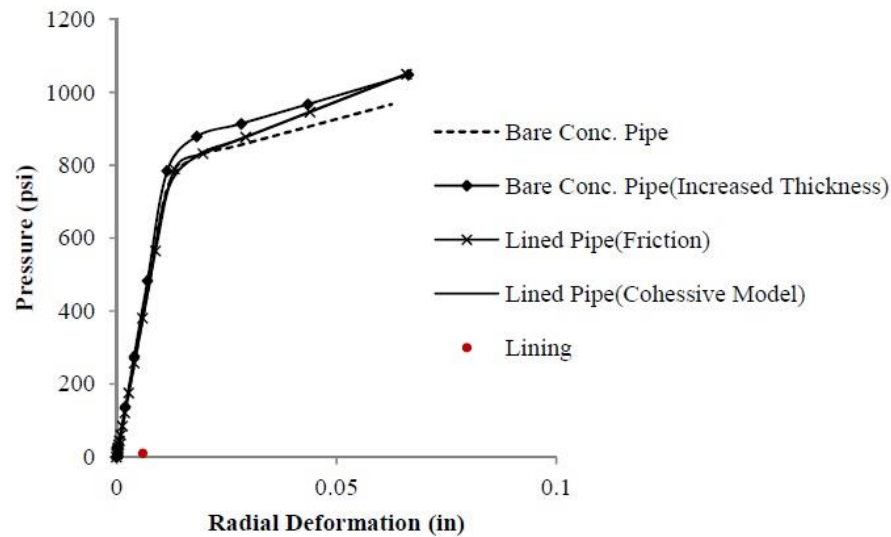

Figure 4.4.15. Pressure deformation curve for bare and lined concrete pipes under

uniform pressure

(Riahi, 2014)

4.5. Chapter Summary

The preliminary and main test results indicated that the lining materials have significantly increased the structural capabilities of the concrete samples. For instance,

epoxy lining material increased the peak load between 45 to 133 percent in main testing based on the thickness of the liner. Similarly, cementitious lining material increased the peak load between 25 to 127 percent, polyurethane between 34 to 52 percent, and cured-in-place lining materials increased the peak load by 54 percent.

A concrete beam lined with epoxy was simulated to be used as a base for finite element modeling (FEM). This model was then calibrated and modified to be used as basis for three-edge-bearing tests. The simulations proved that epoxy can significantly improve the structural capacity of the host pipe. This increase is affected by adhesion and tension reinforcement of the pipes, although loading conditions of the manholes also affect the structural capacity

Chapter 5

Conclusions, Limitations, and Recommendations

for Future Research

5.1. Conclusions

The conclusions of this study can be summarized as follow:

1- The preliminary and main test results proved that all of the selected lining materials in this study added to the structural capacity of the specimens. Therefore it can be concluded that No-Dig manhole rehabilitation material will add to the structural capabilities of the host manholes.

2- Mechanical properties, thickness of the liner, quality of installation, substrate preparation, weather conditions and curing time are the key factors in manholes liner installation.

3- Multi Structural Lining Materials like Epoxy and Glass Fiber or Modified Polyuria and Specialty Foam or CIP can address I&I issues because they do not crack when the concrete substrate cracks at failure.

4- Cementitious Liner or epoxy can be used to exceed structural capacity of the manholes.

5.2. Limitations

The limitations of this study can be summarized as follow:

1- The main lining and testing were done outdoor in cold winter weather which might have slightly impacted the test results.

2- In this research, due to resource limitations, 24 in. concrete pipes were used to simulate manholes.

3- Long-term mechanical properties of the liners may change over time.

5.3. Recommendation for Future Research

The recommendations for future research can be summarized as follow:

1- Conduct experiments on full size manholes with loading conditions similar to actual field conditions.

2- Perform experimental tests on concrete manhole sections.

3- Conduct experiments to measure the long-term structural capabilities of the lining materials.

4- Perform instrumented flexural and compression tests with strain gages.

Appendix A

Preliminary Test Photos

Lined Concrete Beam

Lined Concrete Cylinder

Concrete Beams and Cylinders

Testing Machine

Liner Thickness Measurement

Lined Concrete Beam at Failure

Lined Concrete Cylinder Mounted on the Testing Machine

Lined Concrete Cylinder at Failure

Lined Concrete Cylinder at Failure
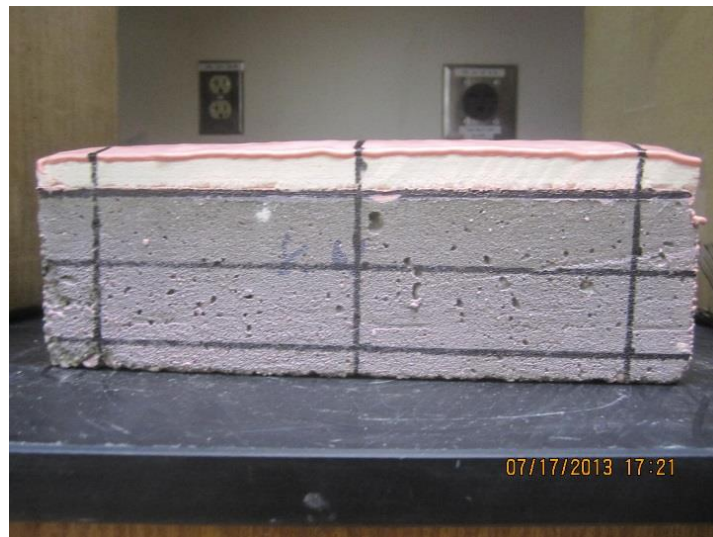
Lined Concrete Beam
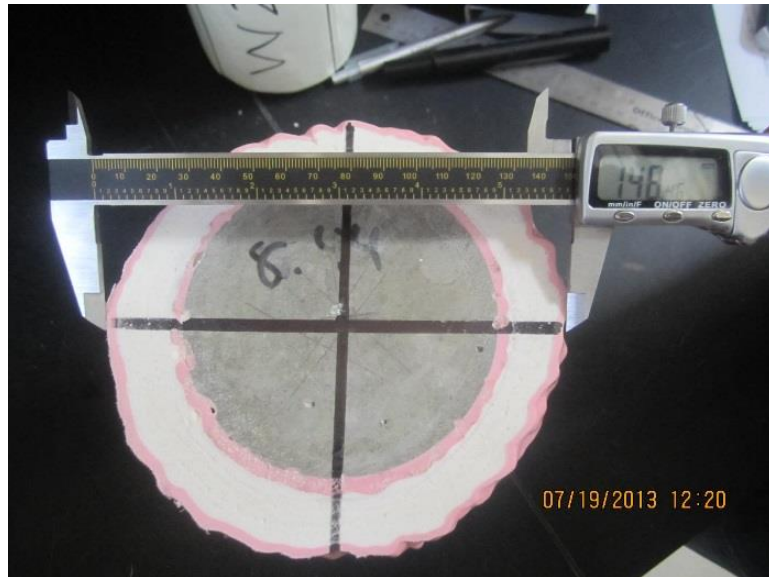
Liner Thickness Measurement

Lined Concrete Beam at Failure

Flexural Test Setup

Lined Concrete Cylinder at Failure

Appendix B

Main Test Photos

Surface Preparation Prior to Lining

Surface Preparation Prior to Strain Gage Installation

Surface Preparation Prior to Strain Gage Installation

Installed Strain Gage

Surface Preparation Prior to Strain Gage Installation

Surface Preparation Prior to Strain Gage Installation

Surface Preparation Prior to Strain Gage Installation

Surface Preparation Prior to Lining

Temperature Measurement Prior to Lining

Lining

Lining

Liner Thickness Measurement

Lining

Lining

Bare Concrete Pipe at Failure

Lined Concrete Pipe at Failure

Lined Concrete Pipe at Failure

Lined Concrete Pipe at Failure

Lined Concrete Pipe at Failure

Lined Concrete Pipe at Failure

Lined Concrete Pipe at Failure

Lined Concrete Pipe at Failure

Appendix C

Sample Daily Report for the Main Tests

Day	Date	Weather	Temp (∘F)	Wind	Holiday?	Prepared by

Personnel Name	Equipment	Own/Rental

Description of Work Performed

Description	Quantity

Issues/Problems/Concerns

Visits/Meetings	
Names	Organization
Description	Results
Task Performed	Task Performed By

Description & Result

Appendix D

List of Acronyms and Abbreviations

List of Acronyms and Abbreviations

Abbreviations	Description
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
ASTT	Australian Society of Trenchless Technology
AWWA	American Water Works Associations
CIP	Concrete specimen with cured-in-place liner
CMT	Concrete specimen with Cementitious liner
CUIRE	Center for Underground Infrastructure Research and Education
EPX	Concrete specimen with Epoxy liner
I&I	Inflow and Infiltration
IIRC	Innovative Infrastructure Research Council
ITCP	Inspector Training and Certification Program
IKT	Institute for Underground Infrastructure
MULT	Concrete specimen with Multi- Layer structural liner
NASSCO	National Association of Sewer Service Companies
NUCA	National Utility Contractors Association
PU	Concrete specimen with Polyurethane liner
SSO	sanitary sewer overflow
TAG-R	Trenchless Assessment Guide for Rehabilitation
UNL	Unlined concrete specimen
WERF	Water Environment Research Foundation

References

American Society of Civil Engineers (ASCE). (2009). "Manhole inspection and rehabilitation." American Society of Civil Engineers, Reston, VA.

American Society for Testing and Materials (ASTM). (2013). "ASTM C-76 Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe."

American Society for Testing and Materials (ASTM). (2013). 'ASTM C 497 Standard Test Methods for Concrete Pipe, Manhole Sections (External Load Crushing Strength).

American Society for Testing and Materials (ASTM). (2013). "ASTM C-39 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.

American Society for Testing and Materials (ASTM). (2013). "ASTM C-293 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center Point Loading)."

Ahn, N., Park, D.K., Lee, J., and Lee, M.K. (2009). "Structural Test of Precast Polymer Concrete." *Wiley Periodicals, Inc., Journal of Applied Polymer Science,* Vol. 114, 1370 - 1376.

Entezarmahdi, A., Najafi, M., and Sever, V.F. (2014). Testing and Analysis of No-Dig Structural Manhole Rehabilitation Materials*," Proceedings of Pipelines 2014 Conference*, ASCE, Portland, Oregon.

Hughes, j. (2009)." Manhole inspection and rehabilitation," Second edition, ASCE, Reston, VA.

Institute for Underground Infrastructure (IKT). (2012). "Rehabilitation of wastewater manholes: Large scale tests and in-situ studies." Available at: http://www.ikt.de/down/english/2012_04.pdf (Accessed: January 1, 2013).

Mathews, J., and Allouche, E. (2012). "Fully Automated Decision Support System for Assessing the Suitability of Trenchless Technologies." *American Society of Civil*

Engineers (ASCE), Journal of Pipeline Systems – Engineering and Practice, 3 (2), 55-64.

Najafi, M. (2005). "Trenchless Technology: Pipeline and Utility Design, Construction, and Renewal." McGraw-Hill, New York.

Najafi, M., Sever, F., (2015) "Structural Capabilities of No-Dig Manhole Rehabilitation Materials". *Water Environmental Research Foundation*. Available at: https://www.hightail.com/download/bXBiRFFYTkE4Q1J2Zk1UQw(Accessed: May, 2015)

Riahi, E., Najafi, M., and Sever, F. (2014) Evaluation of Structural Performance of Epoxy Linings for Manhole Rehabilitation using Laboratory Testing and FEM Simulations. *Proceedings of Pipelines 2014, American Society of Civil Engineers (ASCE), 2014.*

Sabouni, R., and El Naggar, M.H. (2008). "Full Scale Testing of 1200 mm Diameter Precast Concrete Manhole." *Proc. Second Canadian Conference on Effective Design of Structures*, Hamilton, Ontario, Canada, May 20 – 23, 2008.

Tobita, T., and Kang, G., and Lai, S. (2012). "Estimation of Liquefaction-Induced Manhole Uplift Displacements and Trench-Backfill Settlements." *American Society of Civil Engineers (ASCE), Journal of Geotechnical and Geoenvironmental Engineering,* 138(4), 491 - 499.

Biographical Information

Alimohammad Entezarmahdi graduated in 2012 with a bachelor's degree in Urban Infrastructure Engineering from Shiraz University, in Iran. After graduation, he worked as a project engineer on public projects for major cities as well as residential and commercial projects for one year. In January 2013, he entered the University of Texas at Arlington to pursue a Master's Degree in Civil Engineering with a focus in Construction Engineering and Management. He worked as a graduate research/teaching assistant under supervision of Dr. Mohammad Najafi at the Center for Underground Infrastructure Research and Education (CUIRE) on a research project on Structural Capabilities of No-Dig Manhole Rehabilitation. He has presented at the annual ASCE International Pipeline 2014 Conference in Portland, Oregon, 2014 Transportation Research Board Conference in Washington DC, 2015 No-Dig show at Denver, Colorado and 2015 Texas Water conference in Corpus Christi, Texas.

He is currently working as a project manager at No-DigTec which is a trenchless pipeline construction company.