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FINAL REPORT

Structural Capabilities of No-Dig Manhole Rehabilitation

INFR1R12

STRUCTURAL CAPABILITIES OF NO-DIG MANHOLE REHABILITATION

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2015

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Library of Congress Catalog Card Number: 2014936267

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The research on which this report is based was developed, in part, by the United States Environmental Protection Agency (EPA) through Cooperative Agreement No. CR-83419201-0 with the Water Environment Research Foundation (WERF). However, the views expressed in this document are not necessarily those of the EPA and EPA does not endorse any products or commercial services mentioned in this publication. This report is a publication of WERF, not EPA. Funds awarded under the Cooperative Agreement cited above were not used for editorial services, reproduction, printing, or distribution.

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ABSTRACT AND BENEFITS

Abstract:

Failure of a manhole may have catastrophic consequences such as a sinkhole. At a minimum, wastewater flow will be blocked and flow upstream of the manhole will backup, causing a sanitary sewer overflow (SSO). Accordingly, the structural condition of a manhole is an important performance indicator and risk of failure should be minimized. Mechanical strength of manhole rehabilitation materials vary substantially and little is known about their ability to withstand the dead and live loads exerted on the manhole. As such, this project investigates the structural capabilities of commonly used manhole rehabilitation materials and methods via literature review, and case study compilation, lab tests on mechanical strength, and computational modeling. A classification for manhole rehabilitation techniques is provided based on their structural capabilities (i.e., fully, semi-, or non-structural). The results of this project suggest that any type of manhole rehabilitation material can be applied as fully structural; nevertheless, it may be difficult to achieve the thickness required to qualify as fully structural for the spray-applied, cured-in-place type liners. A user-friendly decision support tool is provided with this report as a practical tool to help choose structural class and construction methods appropriate for a manhole considered for rehabilitation.

Benefits:

- Creates awareness on the condition and importance of the sanitary sewer manholes.
- Describes the effects of a manhole failure with respect to utility infrastructure asset management as well as environmental, social, and economic consequences.
- Provides insight on the available manhole rehabilitation techniques to help wastewater community understand the capabilities of them.
- Provides cost information and tools to rehabilitate a manhole in the most cost efficient manner.
- Guides the wastewater (and stormwater) utilities and their consulting engineers with making decisions on material and method selection.
- Establishes sound wastewater rehabilitation programs, thereby reducing sanitary sewer overflows (SSOs) and their effects on the environment.

Keywords: Manhole, rehabilitation, asset management, sanitary sewer overflow SSO, case studies.

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ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials (now ASTM International)
ASTT	Australian Society of Trenchless Technology
AWWA	American Water Works Association
AWWU	Anchorage Water and Wastewater Utility
BOD	Biochemical Oxygen Demand
CIP	Cured in Place
CIPP	Cured-in-Place Pipe
CSA	Canadian Highway Bridge Design Code
CSES	Collection System Evaluation Studies
CUIRE	Center for Underground Infrastructure Research and Education
DSS	Decision Support System
DST	Decision Support Tool
EI	Existing Infrastructure
EPA	Environmental Protection Agency
FEM	The Finite Element Method
FRCL	Fiber Reinforced Cementitious Lining
GPS	Global Positioning System
GWRS	Ground Water Replenishment System
GWWTF	Girdwood Wastewater Treatment Facility
HDPE	High-Density Polyethylene
IIRC	Innovative Infrastructure Research Committee
I/I	Infiltration and Inflow
IKT	Institute for Underground Infrastructure (Germany)
ISCORD	International Symposium on Cold Regions Development
ITCP	Inspector Training and Certification Program
JCW	Johnson County Wastewater
KIP	Kilo Pounds
kN	Kilo Newton
kPa	Kilo Pascal

lbs	Pounds
MACP	Manhole Assessment and Certification Program (by NASSCO)
MDPE	Medium-Density Polyethylene
MOP	Manual of Practice
MR	Modulus of Rupture
MRDSS	Manhole Rehabilitation Decision Support System
MPa	Mega Pascal
MWRDGC	Metropolitan Water Reclamation District of Greater Chicago
NASSCO	National Association of Sewer Service Companies
NPDES	National Pollutant Discharge Elimination System
NUCA	National Utility Contractors Association
OCSD	Orange County Sanitation District
O&M	Operations and Maintenance
pcf	Pounds per Cubic Feet
psf	Pounds per Square Feet
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
QA/QC	Quality Assurance/Quality Control
SSERP	Sewer System Evaluation and Rehabilitation Program
SSO	Sanitary Sewer Overflow
STP	Sewage Treatment Plant
SWBNO	Sewerage and Water Board of New Orleans
TAG-R	Trenchless Assessment Guide for Rehabilitation
TTC	Trenchless Technology Center
UI	User Interface
UT	University of Texas
VCP	Vitrified Clay Pipe
WEF	Water Environment Federation
WERF	Water Environment Research Foundation
WRc	Water Research (United Kingdom)

EXECUTIVE SUMMARY

Experience to date suggests that several million manholes in North America are suffering from serious deterioration and structural degradation due to hydrogen sulfide induced corrosion and other reasons. Because it is not economical or feasible to replace all of the deteriorated manholes, there is, at least potentially, a multi-billion dollar market in no-dig manhole rehabilitation; hence, there are numerous materials and methods available on the market. As such, determining the most feasible and economical rehabilitation material and method often imposes a challenge for wastewater utilities and consulting engineers.

Manhole rehabilitation materials include Portland-based cement mortar, calcium aluminates, geopolymers, polymeric (polyurethanes, epoxies, polyureas, and their mixtures), fiberglass, epoxy-coated cement, coal tar epoxy, polymeric sheets (e.g., polyvinyl chloride (PVC)), steel inserts, rubber inserts (used for chimney and joint rehabilitation), elastic polyurethane (for chimney restoration), cured-in-place composites (similar to cured-in-place pipe (CIPP) used for pipeline rehabilitation), polymer concrete inserts, and fiber- reinforced cement linings. Each of these materials and methods has its pros and cons, with a number of unknowns in terms of the structural support they can provide to a deteriorated manhole. The structural condition of a manhole is an important performance indicator and risk of failure should be minimized. However, the mechanical strength of rehabilitation materials vary substantially, and to what extent manholes should be structurally supported is a matter of condition of the manhole to be rehabilitated and loads exerted on it.

This study evaluates structural capabilities of available and emerging manhole rehabilitation materials and methods. The researchers conducted mechanical strength tests, performed computational modeling, and evaluated case histories throughout North America, thereby providing guidelines and a decision support tool (software program) for manhole rehabilitation.

The tasks implemented to accomplish the project objectives were as follows:

Literature Review: A comprehensive literature review was carried out at the initial phase of the project. The literature review enabled the team to identify the work done on manhole rehabilitation by others. It enhanced the scope of work and experimental procedure. The literature cited includes existing guidelines, experimental work by others, published case histories on manhole rehabilitation, and studies conducted on hydrogen sulfide induced corrosion.

Expert Workshop: A workshop for wastewater professionals was held at the WEFTEC in New Orleans, Louisiana on October 1, 2012. The 31 participants included wastewater utility representatives from cities (mostly large urbanized settings), manufacturers, contractors, consultants, one representative from the IIRC (Innovative Infrastructure Research Committee), and the project team members. The workshop met its objective by providing a forum for the participants to enhance the project scope and objectives.

Case Studies: Eleven case studies were compiled from 10 utilities throughout the Unites States regarding their experiences with no-dig manhole rehabilitation. The case studies gathered represent a wide variety of geographies (from Florida to Alaska) and utility size (from small towns to Metropolitan Reclamation District of Greater Chicago). These case studies helped the project team use past experience in manhole rehabilitation to the maximum extent in developing the experimental procedure, rehabilitation guidelines, and the decision support tool.

Lab Tests: Commonly used manhole rehabilitation materials were tested via a two-phase approach (i.e., preliminary and main tests). The preliminary tests were comprised of testing lined concrete beams (ASTM C293) and cylinders (ASTM C39) for their flexural and compressive strengths. The preliminary test results suggest manhole linings could significantly, if not substantially, add to the ultimate flexural and tensile strengths of concrete substrates, which were at comparable thicknesses to actual manholes. The results also indicated an increase on the compressive strength of the samples by lining; nevertheless, the improvement on compressive strength was inconclusive with respect to actual lined manhole condition due to the "confining" effect of the liner on the cylindrical substrate (lined externally).

The main tests were conducted on 24-in. concrete pipes (manufactured to C76), which were internally lined with different type of lining materials and methods. Then these internally lined cylinders were loaded in accordance with ASTM C-497 (D-load test). The main test results suggest that most of the tested rehabilitation materials would significantly improve the structural integrity of a deteriorated manhole, and some of them qualify to be fully structural.

Computational Modeling: Computational modeling with the finite element method (FEM) using the Abaqus software was performed to cross-check test results and expand the investigation to full-scale, lined manholes for different manhole condition and loading scenarios. There was a good match between the model results and experimental data. The FEM also helped the team to project the lining material performance on a full-scale manhole with respect to various site conditions.

Decision Support Tool: A user-friendly decision support tool for manhole rehabilitation was created using Java™ Swing in conjunction with Microsoft® Access. The decision support tool uses the input data on manhole condition, soil conditions, groundwater table and characteristics, site specific loads, and location properties (i.e., high, low, or no traffic); then processes these data/information to recommend the type of lining (structural class) and method of installation (no-dig, low-dig, or open-cut).

The two main objectives of this project, classifying manhole rehabilitation materials and methods (based on their structural capabilities) and providing utilities and engineers a practical decision support tool for manhole rehabilitation, were accomplished by implementing the tasks indicated above. The structural classification of manhole rehabilitation materials (as fully, semi, and non-structural) is indicated in Chapter 8.0, and the decision support tool is available at from WERF's website. The findings of this research project can be summarized as follows:

- Most manhole rehabilitation methods applied today are semi-structural.
- Fully structural (standalone) methods are not needed for the majority of the manholes.
- Semi- or non-structural rehabilitation could be efficient for infiltration and inflow (I/I) removal at lower cost.

- Application/surface preparation is of utmost importance. Same type of material (e.g., epoxy) can be classified as structural, semi-structural or non-structural depending on the thickness and application quality.
- Each manhole is different and there is no “silver bullet” solution; and therefore; use of the decision support tool is recommended.
- Sound engineering and thorough technical specifications are crucial in implementing a successful project.

CHAPTER 1.0

INTRODUCTION/BACKGROUND

Gravity flow wastewater and stormwater collection systems are essentially comprised of sewer pipes, manholes and transmission components consisting of lift stations and force mains for sanitary sewers. Manholes are called "windows" to the sewer system as they are the most visible points in identifying the condition of underground infrastructure. In the U.S. alone, the number of manholes is estimated to be around 20 million. Of those, it is estimated that four million are at least 50 years old and another five million are 30-50 years old (Trenchless Technology, 2010)[1]. Experience to date suggests several million manholes in North America are suffering from serious deterioration and structural degradation due to hydrogen sulfide induced corrosion and other reasons. 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 materials and methods has its pros and cons. While utilities might benefit from many options and the fierce competition among the manufacturers, determining the most feasible and economical rehabilitation material and method often imposes a challenge for design engineers and decision makers (see Table 1-1).

To date, there has been a number of research studies on pipeline rehabilitation. These research studies predominantly focused on the materials and methods used for pipeline rehabilitation from structural, hydraulic, and economic perspectives. Thereby, there is a wealth of experience and tools for engineers to evaluate their options while selecting the material and technology to rehabilitate sanitary sewer pipes. On the other hand, manholes, another vital part of sanitary sewer systems, are often overlooked, though they can be the main source of inflow (rainwater entry into wastewater collection systems). There are very limited studies, but numerous options to rehabilitate manholes.

Manhole rehabilitation materials include:

- Portland based cement mortar (with added chemicals to prevent hydrogen sulfide induced corrosion, see Figure 1-1).
- Calcium aluminates.
- Geopolymers.
- Polymeric (polyurethanes, epoxies, polyureas and their mixtures, see Figure 1-2).
- Fiberglass, epoxy-coated cement, coal tar epoxy, polymeric sheets (e.g., PVC).
- Steel inserts, rubber inserts (used for chimney and joint rehabilitation).
- Elastic polyurethane (for chimney restoration).
- Cured-in-place composites (similar to CIPP used for pipeline rehabilitation).
- Polymer concrete inserts.
- Fiber reinforced cement linings.

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[1] *These numbers do not go beyond an estimate as to the principal investigator's knowledge; there has not been a thorough study on quantifying or evaluating the condition of the manholes in the U.S.*

Figure 1-1. Manhole Rehabilitated with Cementitious Lining. **Figure 1-2. Manhole Rehabilitated with Polymeric Lining.**

Each of these materials and methods has its pros and cons, with a number of unknowns in terms of the structural support they can provide to a deteriorated manhole. A rehabilitated manhole is a system with the following parameters that play a role in the overall durability and life cycle:

- 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 that 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, thereby triggering 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 cementitious or polymeric grouts), joint seals.

Manholes that have deteriorated and allowed to breach may lead to I/I that can result in catastrophic consequences such as a sinkhole. Manholes that have deteriorated by corrosion may allow material to fall into the wastewater flow causing a backup that can result in a sanitary sewer overflow (SSO) and/or basement backups.

Accordingly, structural condition of a manhole is an important performance indicator and risk of failure should be minimized. However, as discussed, mechanical strength of rehabilitation materials vary substantially, and to which extent manholes should be structurally supported is a matter of condition of the manhole to be rehabilitated and loads exerted on it. Little is known about the ability of rehabilitation materials in withstanding the dead and live loads that are exerted on them.

Based on the foregoing discussion, the objective of this study is stated as follows:

Evaluate structural capabilities of available and emerging manhole rehabilitation materials and methods by conducting mechanical strength tests, computational modeling, and evaluating case histories throughout North America and provide a tool for developing a decision support system for manhole rehabilitation.

To fulfill this objective, the project classified manhole rehabilitation materials and methods based on the experimental work on manhole rehabilitation materials, computational analysis, and case histories. This classification is similar to that of AWWA for water main rehabilitation linings (AWWA M28):

Class A Rehabilitation Materials: Manhole relining methods that provide fully structural solutions in addition to stopping I/I. Some of the rehabilitation solutions are fully structural (e.g., polymer concrete manholes and some of the fiberglass inserts) based on the third-party test data. Structural capacity of other types of linings were investigated as a part of this study.

Class B Rehabilitation Materials: Class B manhole rehabilitation methods will be referred to as semi-structural; meaning that these methods can add to the residual strength of a deteriorated manhole, but they are not capable of withstanding the dead and live loads exerted on a manhole by themselves.

Class C Rehabilitation Materials: Class C manhole rehabilitation methods and materials were deemed non-structural. The purpose of using Class C rehabilitation materials was to provide preemptive protection for manholes that are in relatively good condition in addition to stop minor leaks into the manhole.

As durability and stiffness of manhole lining material increases, so does the cost of it; therefore, selection of the optimum class of manhole rehabilitation material should be based on a life cycle analysis that takes the residual strength of the deteriorated manhole into account, to achieve the most economical solution that will meet the expected service life (minimum 50 years) of the rehabilitated system.

The project tasks are comprised of the following:

- ♦ Data Collection.
- ♦ Expert Workshop.
- ♦ Mechanical Tests on Manhole Linings.
- ♦ Case History Compilation.
- ♦ Computational Analysis with the FEM.
- ♦ Decision Support Tool (DST) Development.

The pyramid chart below outlines the process flow for the project tasks.

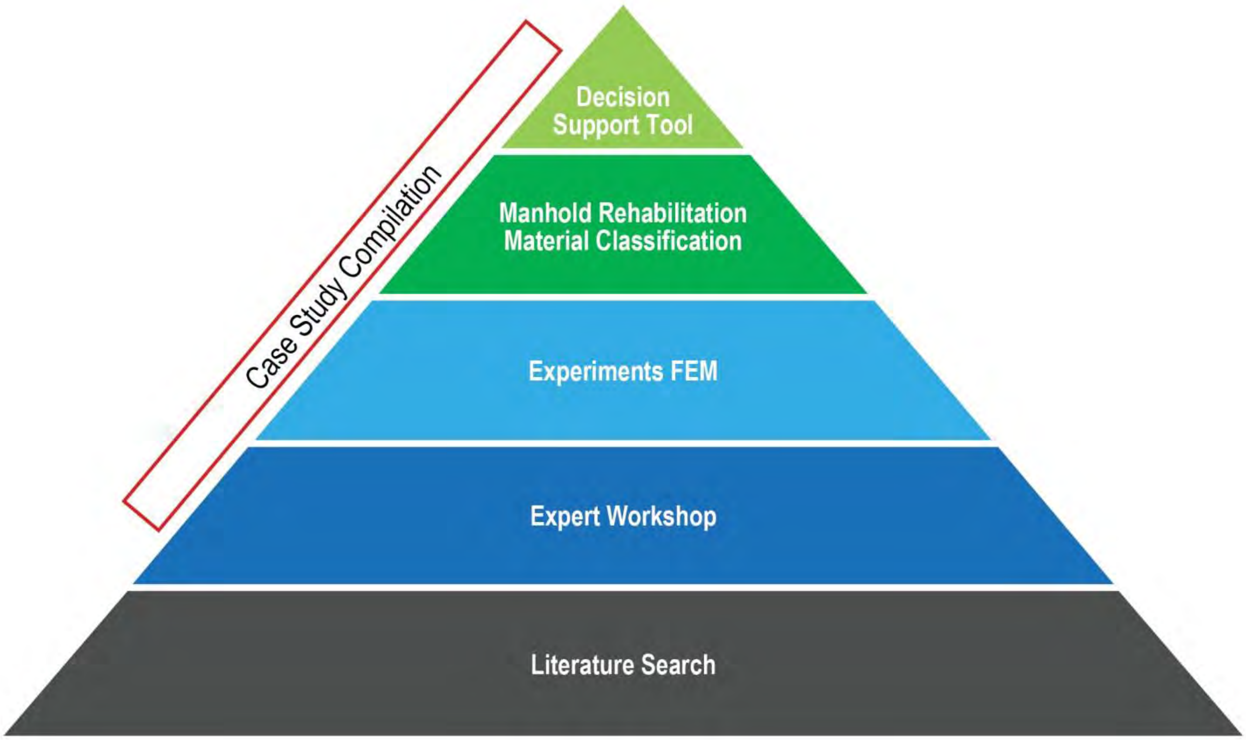

Figure 1-3. Process Flow Chart for the Project Tasks.

CHAPTER 2.0

DATA COLLECTION/LITERATURE SEARCH

The University of Texas Library System (along with Interlibrary Loan System at the UT Arlington), WEF and WERF databases, American Society of Civil Engineers (ASCE) database, and the Engineering Village database were used for the literature search. There is a significant amount of published material on manhole rehabilitation. The majority of the articles cover manhole rehabilitation as a part of a sanitary sewer rehabilitation program, most of which is comprised of pipeline rehabilitation. Nevertheless, a number of studies that are specifically on no-dig manhole rehabilitation were identified and cited as a part of this project.

2.1 Existing Guidelines

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. One commonly recognized was compiled by a rehabilitation committee formed under the ASCE. Entitled as Manhole Inspection and Rehabilitation, ASCE Manual of Practice (MOP) No. 92 provides basic information about inspection and trenchless or conventional rehabilitation of sanitary sewer manholes. A useful tool included in ASCE/MOP No. 92 is 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.

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 ITCP program is geared towards educating the field crew (engineers, technicians, etc.), thereby certifying them as manhole inspectors[2]. 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 include 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 DST 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.

In a recent study, Matthews and Allouche (2012) developed a fully automated decision support system (DSS) for assessing the suitability of trenchless technologies as decisions related to the rehabilitation of wastewater and water infrastructure 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), in collaboration with the National Utility Contractors Association (NUCA), Australian Society of Trenchless Technology (ASTT), and NASSCO developed a comprehensive and 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 DSS, which could be accessed through the 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.
- 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

[2] The PI is of this project is certified by NASSCO for manhole inspection and rehabilitation.

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.

A study by National Research Council, Canada presented an approach to assess the condition and planning of rehabilitation for manholes (McDonald et al., 2002). A simple decision-making process was presented to determine the type of rehabilitation (as needed) or determine the time for the next condition assessment. The decision making process includes structural condition assessment, service condition assessment, and impact assessment. The method developed in the study was tested on two actual manholes in the city of Ottawa (Ontario, Canada) sewer system.

A process flow diagram is provided in the McDonald et al. study for the basic approach to condition assessment and rehabilitation of manholes. Items in the diagram include: inventory database, impact assessment, prioritization, inspection, condition assessment, decision-making on rehabilitation actions, rehabilitation, and frequency of inspection. The inventory database is covered by the municipality to provide retrieval of historic data, storing subsequent inspection and rehabilitation data. Impact assessment addresses the degree of impact of sewer pipe failure.

The "impact factor" is determined by applying the following equation, which calculates a weighted total score:

$$I_w = .2f_1 + .16f_s + .16f_z + .16f_d + .16f_f + .16f_q \qquad \text{(Equation 2-1)}$$

Where, f_l, f_s, f_z, f_d, f_f, and f_q are the factors for location, surrounding soil, pipe size, burial depth, sewer function (i.e., main, trunk, or interceptor), and seismic activity, respectively.

Prioritization is recommended as the first step in scheduling inspection and rehabilitation. Then inspection is completed to provide an insight to the condition of the structure. The defects noted (structural and service) are documented and a code is assigned for each defect, which is a simplified version of the defect coding provided by WRc/NASSCO.

Table 2-1. Manhole Service Condition Coding and Scoring.
McDonald et al. 2002.

Defect Type	Code	Weight
Roots (R)		
Light	RL	0
Moderate	RM	2
Severe	RS	5
Unsafe Ladder (LU)		
Light - slight corrosion, slightly bent	LUL	2
Moderate - heavy rust, deformed bars and rungs, anchor loose, but still in place	LUM	5
Severe - dislodged or missing anchors, bars or rungs, >50% rust on metal elements	LUS	10
Unsafe Landing (DU)		
Light - slight corrosion, slightly bent	DUL	2
Moderate - heavy rust, deformed railing or grating, anchor loose, but still in place	DUM	5
Severe - dislodged or missing anchors, bars or rungs, >50% rust on metal elements	DUS	10
Connection Faults (CF)		
Light - gaps<10 mm (0.4 inches), evidence of infiltration	CFL	2
Moderate - gaps from 10 to 50 mm (0.4 to 2.0 inches) wide, leakage, cracked drop pipe	CFM	5
Severe – fractured drop pipe, gaps > 50 mm (2.0 inches)	CFS	10
Infiltration (I)		
Light - seeping, dripping	IL	2
Moderate – running, trickling	IM	5
Severe - gushing	IS	10

Condition assessment is dependent on the inspection findings and an evaluation table based on the summation of weighted defects is included. The rehabilitation action is assigning priority for rehabilitation through the condition assessment findings; a table is provided to assist with this decision making. Frequency of recommended future inspection is based on the condition assessment (when rehabilitation is not done).

The McDonald et al. study provides a good manhole assessment protocol. The condition assessment procedure is efficient; particularly, the service factors used, as a criterion for decision making, are elaborate. Nevertheless, the study lacks a sound approach in defining what is structural with respect to the rehabilitation methods as there was no clear definition of structural rehabilitation at that time. The cost of replacing a manhole provided in the study (from $1,915 to $3,830) is unrealistically low. The McDonald et al. study was used in developing the decision support tool presented in this project (discussed in detail in Chapter 7.0).

2.2 Experimental Work by Others

Sabouni (2008) conducted her doctoral research on loading/deformation conditions on precast concrete manholes. 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 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 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 compacted layers. The depth of the manholes was 7.62 m (25 ft). Hydraulic jacks with 900 kN (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 700 kPa (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 manhole walls. 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 strain 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 four-foot-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.

The 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 to the research team's knowledge, is the only full-scale test applied on buried manholes with respect to their structural properties. The findings of the Sabouni study were used in designing and executing the tests and computational modeling conducted as a part of this research project.

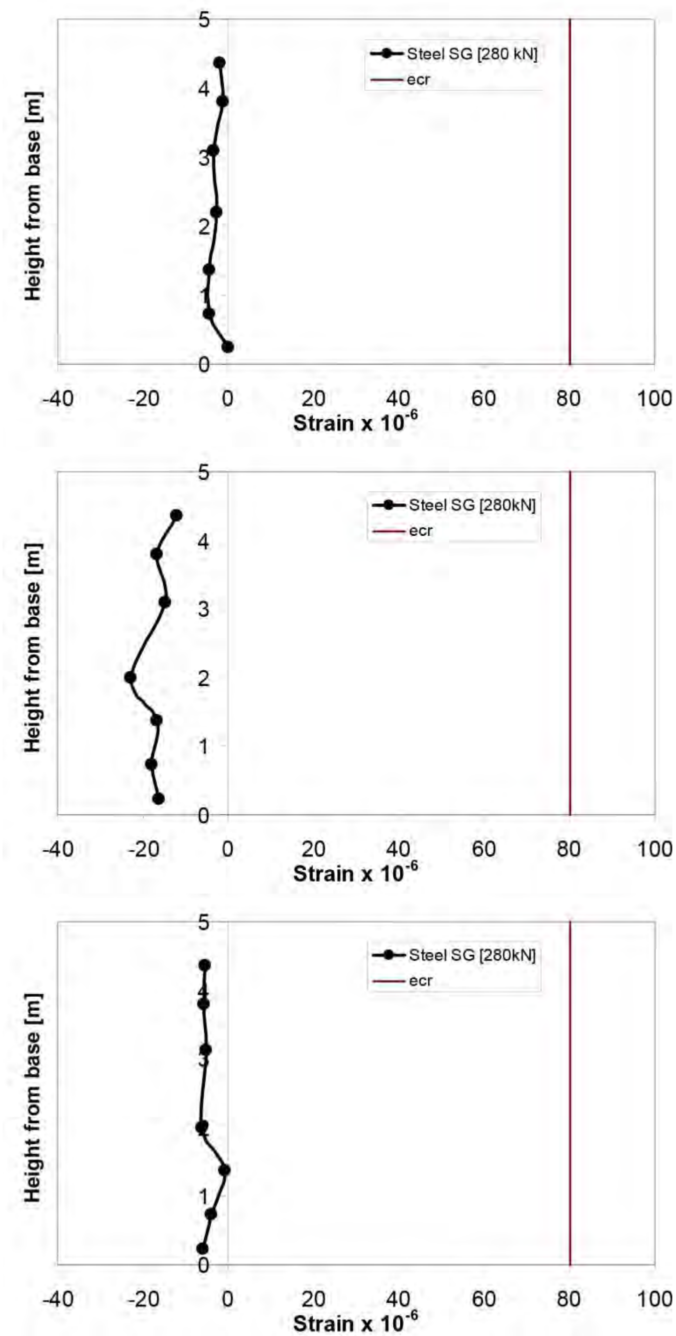

Figure 2-1. Axial Vertical Strain Profile in a Reinforced Manhole Wall Included in the Experiments for Three Different Load Patterns per the Canadian Highway and Bridge Code.

Source: Sabouni, 2008.

a) Four concentrated loads of 70 kN each applied around the manhole cover by slow loading
b) 280 kN distributed load applied on the manhole by slow loading
c) Same condition as (b) but performed by fast loading diameter = 1,200 mm (4 ft).
Note 1 kN = 225 lbs. ecr = concrete cracking strain.

A comprehensive, long-term evaluation of protective coatings used on concrete was carried out by Redner et al. at the Sanitation District of Los Angeles County. The study essentially evaluated resistance of various polymeric and cementitious linings versus highly concentrated (10% by weight) sulfuric acid solution. The tests started in 1983 and ended in 2004 with some of the materials continuously tested for several years. The study included 96 different linings, which were applied on concrete cylinders that are 3 ft (0.9 m) in diameter and 2.5 ft (0.8 m) deep.

The concrete cylinders on which the linings were applied were surrounded by an outer cylinder with 4 ft (1.2 m) diameter. The annular space between the inner and outer cylinders was filled with water to simulate moist soil effects on a manhole (or a buried concrete structure). A 10% sulfuric acid solution was added to lower half of the inner cylinders to create corrosion on the concrete walls. Then the sulfuric acid at the bottom half of the test cylinders were drained and the cylinders were lined with protective coatings (or linings) by the manufacturers' representatives following their standard procedures including surface preparation. Once a full cure for the liner was reached, the lined cylinders were filled with 10% sulfuric acid solution. The solutions were replenished periodically to monitor any damage to the liners that did not demonstrate an apparent failure. The lining materials included coal tar, coal tar epoxy, coal tar epoxy mortar, coal tar urethane, concrete sealers, epoxy, epoxy mortars, phenolic coating, polyester, polyester mortars, polyurea, silicone, specialty concrete, urethane, vinyl ester, and vinyl ester mortars.

Each lining material was evaluated based on ease of application, bonding to concrete, and resistance to acid solution. A scoring scale of 1-3 was used for each category with 1 and 3 being the best and worst rating, respectively. Then the three ratings were summed up for an overall score for each liner. If the total score exceeded 5 or the liner received a 3 for any of the three categories, the result was recorded as a failure.

Although the Redner et al. study tested only one aspect (i.e., durability against sulfuric acid corrosion) of lining performance, it is a long-term and widely recognized study in the industry. While certain type of liners, on average, demonstrated better durability against sulfuric acid exposure, an interesting finding of the study is the highly varying results among the same type of base lining materials.

A summary of the test results are given in Table 2-2. The left-most column indicates the base material of the lining, then subsequently, the total number of liner brands, total number that received an overall score of 3, 4, 5, and total number of failures for each base lining material are indicated.

Another noteworthy finding of the Redner et al. study is the significant, if not substantial increase in the durability of the newer liners to sulfuric acid solution. Figure 2-2 shows the pass/fail ratios of the tested materials over a 20-year period in four- to five-year increments. While the type and quantity of the materials tested over the years were not evenly distributed[3], it is fair to conclude that the liners that have been manufactured since the early 1990s are superior to the older ones with respect to ease of application, bonding to concrete substrate, and durability against high concentration hydrogen sulfide concentration.

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[3] *For instance, 43 tests were done during the 1983-1987 time bracket, whereas the number of tests conducted from 1993 through 1998 was only 12.*

Table 2-2. Type of Materials Tested by Redner et al. and Their Ratings
Prepared Based on the Results Obtained from the Reference Study.

Base Material	# of Brands Tested	Total # Rated 3 or less[4]	Total # Rated 4	Total # Rated 5	Total # Failed
Coal Tar	1	–	–	–	1
Coal Tar Epoxy	1	–	–	–	1
Coal Tar Urethane	1	–	–	–	1
Coal Tar Epoxy Mortar	1	1	–	–	–
Concrete Sealer	2	–	–	–	2
Epoxy	11	2	2	2	5
Epoxy Mortar	16	2	6	1	7
PVC Liner [5]	14	6	3	1	4
Polyethylene Liner[6]	5	2	1	–	2
GRP	1	1	–	–	–
Phenolic Coating	1	–	–	–	1
Polyester	1	–	–	–	1
Polyester Mortar	1	1	1	–	–
Polyurea	3	1	1	–	1
Silicone	1	–	–	–	1
Specialty Concrete/Mortar	12	3	1	–	8
Polyurethane	18	–	1	2	15
Vinyl Ester	1	–	–	–	1
Vinyl Ester Mortar	1	1	–	–	–

[4] *Some of the liners were not tested for all three categories, and therefore could receive a score of less than 3.*
[5] *Includes extruded (factory manufactured) PVC sheets, PVC and urethane foam and PVC and fiberglass composites. Two of the PVC linings included are used for new construction (must be applied on curing concrete).*
[6] *Includes extruded HDPE linings (sheets) and one corrugated steel lining with polyethylene coating.*

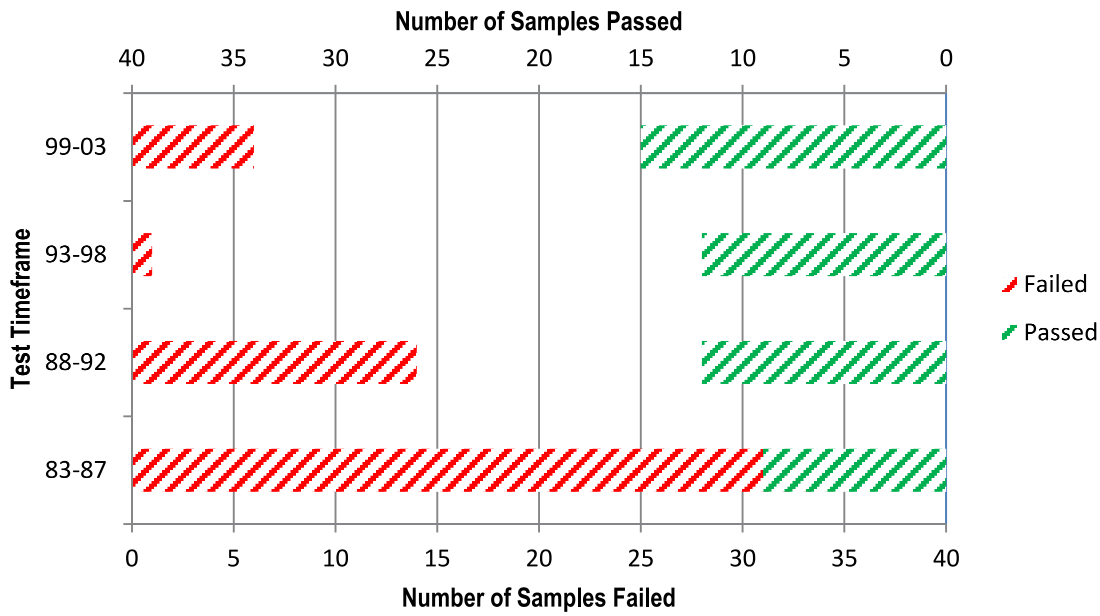

Figure 2-2. Pass/Fail Rate of the Tested Lining Materials over the 20-Year Testing Period.
Prepared from the Results of the Redner et al. Study.

The Redner et al. study concludes that the primary reason for failure of cured-in-place linings/coatings was the formation of pinholes and blowholes; whereas, extruded sheets (PVC and High-Density Polyethylene (HDPE)) and specialty concrete/mortar linings were believed to fail due to poor bonding at the seams and inadequate durability against sulfuric acid corrosion, respectively.

The previously referenced study is well respected and it might have even influenced improvement of the coating/lining materials over the years. (New generation liners performed better.) Nevertheless, the procedure and henceforth the conclusions drawn are debatable due to the following reasons:

- ♦ The testing protocol was limited to measuring application, bonding, and resistance to hydrogen sulfide corrosion. One of the main parameters of manhole lining durability is the ability to withstand loads exerted on them (e.g., hydrostatic pressure, soil pressure, and traffic).
- ♦ The test condition is analogous to a manhole that is submerged with 10% hydrogen sulfide solution. This will be an extremely rare field condition, if exists at all. Sulfuric acid typically forms on manhole walls due to oxidation of hydrogen sulfide gas *by thiobacillus* bacteria; as such, the concentration of sulfuric acid is limited to presence of this bacteria and hydrogen sulfide concentration.
- ♦ There is no statistical confidence in the results, as there is only one test setup for each material included in the study.
- ♦ Total number of linings tested for each material is skewed. For instance, a total of 18 different types of urethane linings were tested vs. just one sample of coal tar epoxy mortar; therefore, the study conclusions on rating the linings per their base material overreaches.

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 5.6 m (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 5.2 m (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 three 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 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 recommends 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).

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:

- 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 the thickness of the vadose zone, 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. 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 settlements attributable to consolidation after liquefaction are assumed to be zero for simplicity.

Another study was performed by Ahn et al. (2009) to check the feasibility of concrete polymer manhole through a development test of high-strength polymer concrete and to 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.
- 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 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 127 Mega Pascal (MPa) (18,400 psi) and 22 MPa (3,200 psi), respectively. Such mechanical strength figures suggest polymer concrete has enough stiffness to build a new manhole with this material.

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

Cady and Weyers (1990) carried out a forensic investigation to establish the causes for severe, early deterioration of precast concrete sewer manhole sections (Figure 2-3). The study suggests that petrographic[7] examinations play a major role in determining the causes of the deterioration. Early on, it was established that the entrained air-void systems were inadequate in the affected sections, which had high water/cement ratio. In addition, crack patterns characteristic of expansive reactions (map cracking) were present along with the typical fracture planes parallel to exposed surfaces commonly associated with freeze- thaw cycles (Figure 2-4).

Half of the 50 samples tested had cracks well below the frost line. Petrographic examinations included in the Cady and Weyers (1990) study revealed that the magnesium oxide (MgO) content of the Portland cement in those sections was 3.5 times that of the unaffected sections and, at 9.1%, was more than 50% higher than permitted in ASTM C-150.

Figure 2-3. Example of Severely Deteriorated Manhole Cone Investigated in the Cady and Weyers Study.

[7] *The term here entails examination of hardened concrete with respect to aggregate size, content, and concrete cracking patterns.*

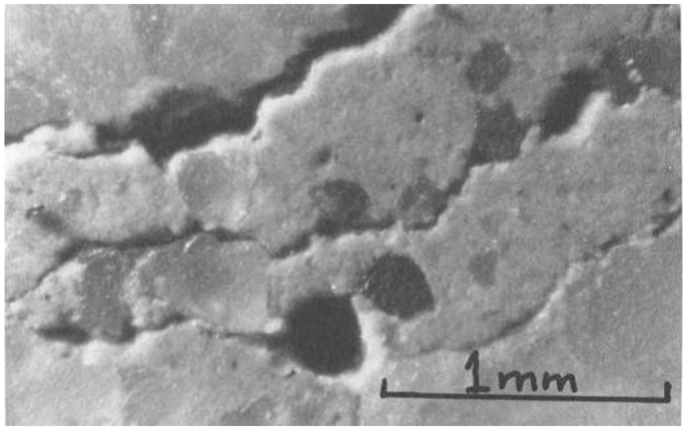

Figure 2-4. Freeze-Thaw Cracks Observed on Manhole Wall after Only One Year of Service.

The Cady and Weyers study suggests hydrogen sulfide induced corrosion or even freeze-thaw cycle may not be the primary driver for structural deterioration and premature failure for precast manholes. A major factor to be considered here is the quality of the concrete used for the manholes as the samples used in this reference work were from actual manholes, which had been in service for merely one year.

In an earlier study, Nakano et al. (1989) investigated failure mechanism of manholes constructed in soils that are susceptible to liquefaction during an earthquake.

A series of model shaking tests were conducted to investigate the failure phenomena due to liquefaction, and test the effectiveness of four types of manhole stabilization techniques against liquefaction, such as surrounding with crushed grain and sheet pile or dewatering to the bottom level of the manhole. The results of the tests were compared with a manhole employing no stabilization technique (control specimen).

Based on measured accelerations, excessive pore pressures, and observed floating up and settlement; the study described the effectiveness of stabilizing techniques.

The Nakano et al. study concluded that surrounding a manhole with crushed grain and sheet pile, dewatering groundwater to the bottom of the manhole were effective methods of mitigating soil liquefaction on manholes.

Griffith and Ball (2000) conducted a study to investigate the mechanical properties of glass-fiber-reinforced polyester polymer concrete. While glass-fiber-reinforced polyester polymer concrete is becoming popular in new manholes construction and existing manhole rehabilitation, little is known about this material's long-term durability.

In this reference study, the modulus of rupture (MR) and the fracture toughness of polyester polymer concrete were determined by using three-point flexural strength testing. The strengthening effects of glass-fiber reinforcement, woven roving and silane coupling agent additions were investigated. In addition, the effects of exposure to various aggressive environments were investigated.

It was found that fiber reinforcement of polymer concrete significantly improves the strength and toughness of the material. However, deterioration in strength could occur due to exposure to UV radiation, acids, alkalis, and saline solutions.

A thesis written at the Queen's University by Bandler (2007) outlines the lab-created and lab-tested masonry and unreinforced concrete manholes to simulate existing manholes and test the structural aspects of rehabilitation methods. The methods tested: plastic polyurea on brick manhole, HDPE slipliner applied to brick and concrete manholes, and calcium aluminate grout applied to brick and concrete manholes. Samples were tested under axisymmetric pressure to simulate horizontal effective stresses and under diametrically opposed two-point loading to test in bending to simulate applied loads from surface activity or excavation.

The axisymmetric manhole loading was applied after the linings had been applied to the samples, but in the real-world application, these loads are already supported by the existing structure. There is a question whether the liner of a repaired manhole experiences any of these loads unless further deformations of the existing manhole occur (further deterioration of the structure, soil disturbance, surcharge loading, etc.)

Through investigating the physical response, deformations, and local bending of the lined structures, the samples responded as follows:

- The plastic spray-on liner behaved in a ductile manner with a low strength.
- The calcium aluminate grout liner exhibited high peak strength and yielded catastrophically at small deformations.
- The HDPE slip liner (grouted in place) produced high peak strengths, yielding in brittle failure, but residual strength in the HDPE liner prevented total collapse.

The thesis concludes that the HDPE liner is best for moderately damaged deep manholes due to the strength and ductility; with all methods acceptable for lightly damaged manholes.

While Bandler's study is directly related to this project and serves as a useful reference, it does not include a control (unlined) manhole to compare performance. Without this control, it cannot exactly be known how much strength the linings contribute. In addition, the study tests each sample in axisymmetric loads and then tests the same sample in two-point bending. The bending results may not be valid due to deformations from the axisymmetric testing. Another shortcoming of this study is that there is no statistical confidence in the results, as there is only one test setup for each material included in the study.

Liu and Vipulanandan (2004) carried out an experimental study on bonding properties of four different commercially available epoxy coatings. In a two-year experimental period, they tested bonding strength of these epoxies to concrete substrate by applying two custom design experimental procedures that are based on ASTM D4541 and ASTM C321. ASTM D4541 is geared more towards polymeric coatings, whereas ASTM C321 is designed for testing bond strength of chemical resistance mortars to brick.

Liu and Vipulanandan identified five modes of failure based on the tests they implemented:

1. Concrete (substrate) failure – This happens when the bonding strength between the coating and substrate exceed the ultimate tensile strength ("z-direction") of the substrate (concrete), and indicates excellent bonding between the substrate and coating.[8]
2. Coating failure – Indicates poor cohesive strength among the coating molecules or low "z-direction" tensile strength (lamellar tearing).

[8] *This may not always be a desired condition, because if the bonding strength is on the extreme and the substrate (concrete) fractures, so will the coating/lining.*

3. Bonding interface failure – Is a result of failure between the substrate and coating.[9]
4. Bonding and concrete failure – Is a combined failure mode of concrete (substrate) and bonding strength between the coating and substrate. This failure mode occurs when the bonding strength is close to the tensile (or flexural) strength of the substrate and less than the tensile strength of the coating.
5. Bonding and coating failure – Is a combined failure mode of coating/lining and bonding strength between the coating and substrate. This failure mode occurs when the bonding strength is close to the tensile (or flexural) strength of the coating and less than the tensile strength of the substrate.

Liu and Vipulanandan used wet and dry substrates[10]and applied the same experimental procedure in-situ on epoxy coated concrete pipe segments. They observed somewhat significant, but not substantial difference between results of the laboratory tests and that of in-situ.

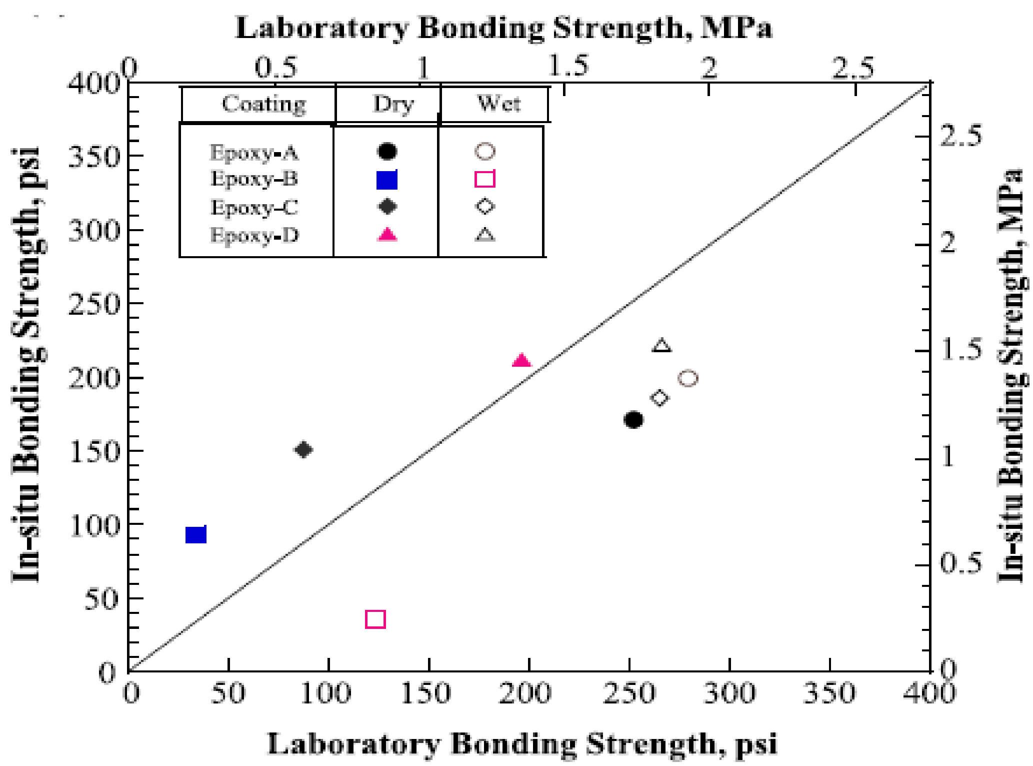

Figure 2-5. Comparison of Lab and In-Situ Bonding Strength (between epoxy coating and concrete) Test Results.
Liu and Vipulanandan, 2004.

Other findings of the Liu and Vipulanandan study include the substantial difference among the four types of epoxy coatings with respect to their bonding strength. This can be attributed to the proprietary formulae of manufacturers with various types of filler and additives that significantly affect the material properties. The study also concluded that the difference between lab and *in-situ* test results diminishes as the bond strength of the tested material increases.

The most notable findings of the reference study from the project team's perspective were:

[9] *The pulling force at which the failure occurs is important as this type of failure does not necessarily mean poor adhesion between the substrate and coating.*
[10] *Concrete specimens were submerged in water for seven days for the wet substrate tests.*

♦ Relatively low difference between the lab and in-situ performance of the tested epoxy coatings. This is particularly true for epoxies with high bond strength.

♦ Insignificant difference between the bond strength of epoxy to wet and dry concrete substrate.

2.3 Published Case Histories

A number of case histories of no-dig manhole rehabilitation were published in the literature. For instance, in an article published by *Public Works,* W. Ries (1991) discusses a case study of lining manholes with a fiberglass system in Pinellas County, Florida. Ries outlines nothing but benefits of no-dig manhole rehabilitation with a cured-in-place fiberglass liner with epoxy resin. Among those benefits are:

♦ Convenience of no-dig rehabilitation without any interruption to the traffic.

♦ Lower cost in comparison with replacement.

♦ Structural support to the deteriorated manhole without a significant reduction in the manhole cross-sectional area (thickness of the lining was from 0.25 in. to 0.50 in. or from 6.5 mm to 13.0 mm).

Another noteworthy experience from Pinellas County is that a 42-in. (1 m) diameter interceptor that was in good condition had several deteriorated manholes on it. These manholes were scheduled to be lined with the fiberglass lining system at the time the article was published.

The installation of each fiberglass lining system took from four to six hours. The County had to train the crews on installing the lining after a couple of successful applications by a contractor. Hence, upon completion of a training program and acquiring the necessary equipment, a wastewater utility can install the referenced fiberglass lining in-house.

Figure 2-6 shows an example of a cured-in-place, fiberglass lining system, which includes an impervious "membrane" layer between two fiberglass layers.

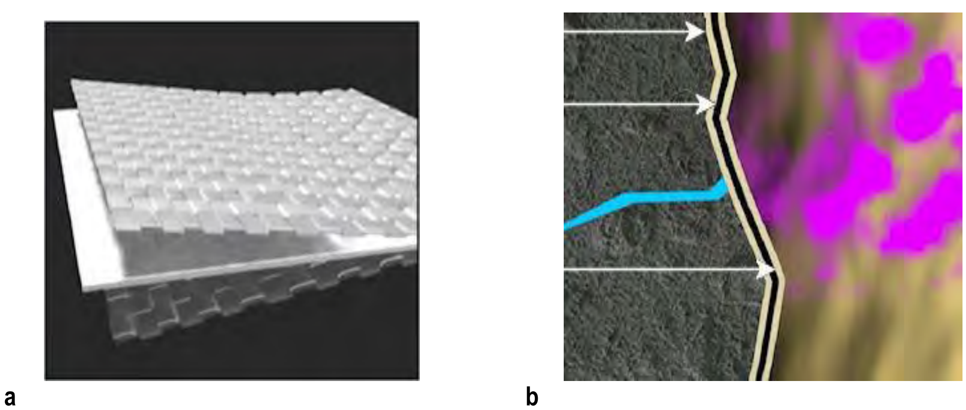

Figure 2-6. Three Layer Cured-in-Place Fiberglass Lining System with an Impervious "Membrane" Between the Resin (Epoxy) Impregnated Fiberglass Layers (a) and a Cross-Section of the Same System Installed on a Manhole Interior Wall (b).
Source: Poly-triplex Technologies

Another case history on no-dig manhole rehabilitation was published by the Water Environment Federation (WEF) periodical *Water Environment & Technology* (1998). Authored by Holmberg and Rowe, the article on this case history discusses a successful manhole rehabilitation project, in Montgomery, Alabama, using a cement mortar (substrate) and epoxy lining system. The Water Works and Sanitary Sewer Board of the City of Montgomery considered four criteria while choosing a material and method for manhole rehabilitation, i.e., tensile strength, compressive strength, adhesion, and environmental effects (hydrogen sulfide induced corrosion).

Compressive strength of the liner was used as a design criterion with respect to hydrostatic pressure on the manhole. For a "no bond" condition between the liner and manhole wall, the required liner thickness with respect to compressive strength was given with the following equation:

$$t = \frac{PD}{144(2C)} \qquad \text{(Equation 2-2)}$$

Where *t* is the minimum required lining thickness with respect to compression, *P* is the hydrostatic pressure, *D* is the liner diameter, and *C* is the compressive strength of the liner.

Accordingly, for a lining with 6,000 psi (4,138 N/cm^2) under 15 ft (4.6 m) water column of hydrostatic pressure, a minimum thickness of 0.025 in. (0.6 mm) was calculated. This value is substantially below the recommended minimum (13 mm or 0.5 in.) for cementitious linings by the manufacturers. Hence, compression did not determine the thickness of the liner material for the foregoing conditions in the Montgomery, AL project.

Similarly, tension exerted on manhole wall and lining was not deemed as a parameter in determining minimum thickness; nevertheless a minimum of 200 psi (1,379 kPa) bond (adhesion) strength and 0.06 in. (1.5 mm) thickness were required for the lining.

The Los Angeles County study (Redner et al., 2004) on hydrogen sulfide induced corrosion and other environmental effects on various lining materials, was used as a reference for acceptance of the lining material. The work by Redner et al. is widely recognized with respect to environmental effects on protective coatings applied on concrete and discussed in detail previously. A two-part system, with cement mortar lining serving as the substrate and coal tar epoxy coating as the outer lining, was used where it was believed that hydrogen sulfide induced corrosion was prevalent. Otherwise, cement mortar lining only was used for manhole rehabilitation.

Holmberg and Rowe also emphasized the importance of surface preparation as a part of the Montgomery manhole rehabilitation project at the Towasa Basin. The lining system was applied using the following procedure:

- Adjust manhole grade, frame, and replace the lid (as required).
- Prepare the manhole wall and bench surfaces by pressure washing.
- Plug active leaks.
- Batch premixed bags of cementitious mortar liner material.
- Apply the lining with low pressure application (shotcrete) using a progressive cavity pump for uniformity.
- Apply the finish (by trowel) to cementitious lining and apply coal tar epoxy as required.

The interior of the manhole was washed with high pressure to remove loose debris as a part of the surface preparation procedure. Sand blasting was considered for surface preparation, but not applied due to concerns regarding sand accumulation in the collection system.

Adhesion tests were applied on lined manholes using the American Concrete Institute Test Method 503 (ACI 503R, 1993 – reapproved 1998). The sewage basin included four types of manholes; i.e., brick, mortar lined brick, cast-in-place concrete, and precast concrete. The results of the adhesion tests varied remarkably depending on type of the substrate, i.e., cement mortar lining had better adherence on brick in comparison with cement mortar or concrete surface (see Figures 2-7 and 2-8).

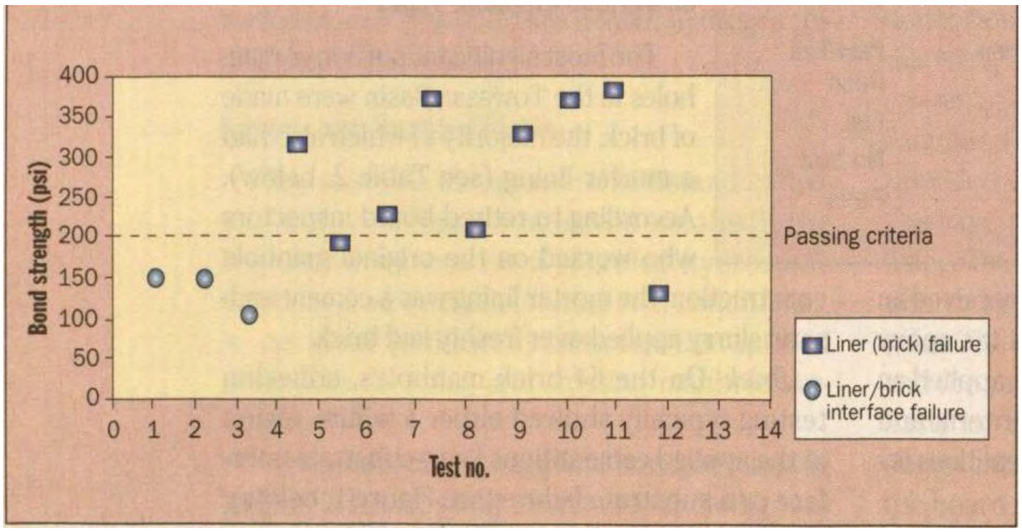

Figure 2-7. Results of Adhesion Testing of Cementitious Lining Over Brick.
Holmberg and Rowe, 1998.

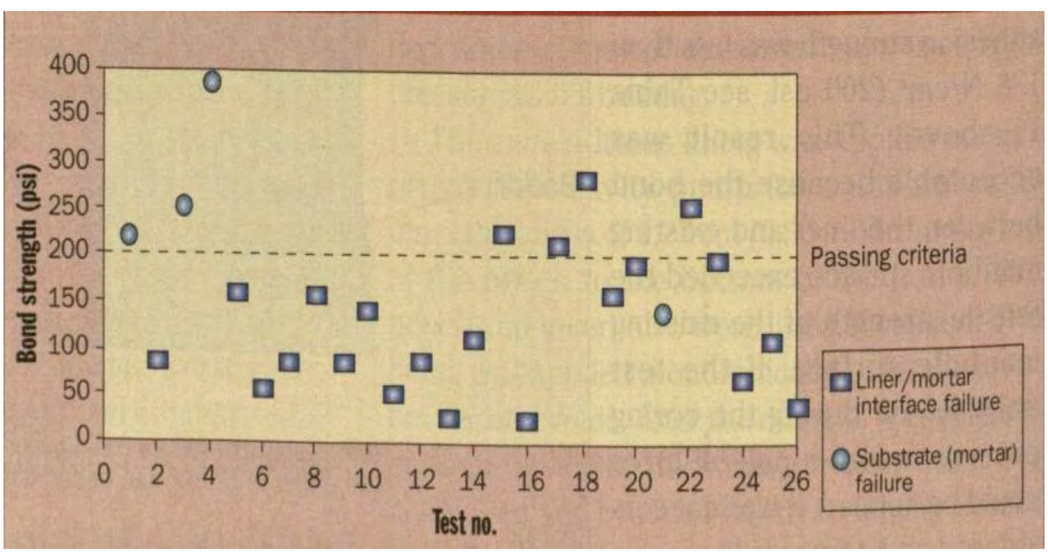

Figure 2-8. Results of Adhesion Testing of Cementitious Lining over Cement Mortar.
Holmberg and Rowe, 1998.

Varghese and Nelson (2010) provided a case study of a systematic manhole inspection program. The study area consisted of three basins. Manholes were inspected for both structural defects and I/I defects. This study also provided an in-depth look into what to do with the data and how to prioritize rehabilitation of the manholes. The guidance to conduct a comprehensive manhole inspection program was obtained from ASCE's Manhole Inspection and Rehabilitation Manual No. 92. Inspection includes data recording for asset management and evaluating the structure for structural condition, flow condition, and maintenance conditions. From the defect rating, the ASCE MOP 92 was used as a guide to develop flow rates resulting from these defects. The defects were prioritized and ranked to understand the overall system integrity. These rankings were used to prioritize manholes for rehabilitation. Cost effectiveness analysis was performed to compare the cost of rehabilitation to the cost of transporting and treating I/I contributed by the defects. This study also included recommendations on how to translate the manhole defect schedule into a manhole rehabilitation schedule.

The following processes were used in the Varghese and Nelson study:

- Inspection procedure.
- Inspection status.
- Condition assessment.
- Prioritizing manholes for rehabilitation.
- Preliminary rehabilitation schedule.

A notable finding of this study was the discovery of significant amount of I/I (almost 20%) into the three basins included. The cost for fixing these defects only accounted for 7% of the total rehabilitation cost. The study also showed manhole rehabilitation was cost effective when compared to transporting and treating the I/I.

Manholes installed in asphalt pavement represent a structural discontinuity, because of the different characteristics of the pavement and the manhole chamber construction. This partly explains the high incidence of premature failure in the asphalt surfacing around these installations. It is considered to be caused by the larger vertical stiffness of the manhole, which results in a potentially damaging combination of stresses in the asphalt surfacing under wheel loading as it bridges the interface between the pavement and the manhole construction.

Careful field observations by Brown and Brown (1999) revealed that the pavement material around a manhole could fail as early as within a few months of installation. Field experiments were carried out to study the *in situ* behavior of manholes and the immediately surrounding pavement. These experiments involved the use of a falling-weight deflectometer to test several in-service manholes. In addition, theoretical analysis was used to provide some preliminary guidance as to how to improve design. This study is useful in understanding the effects of live loadings on manhole and pavement structure.

Saber and Cerda (2003) provided an evaluation of the spray-on lining materials used for manhole rehabilitation. They included cementitious (calcium aluminate cement) and polymeric materials (epoxy, polyurethane, and polyurea). Saber and Cerda emphasized the importance of following manufacturer recommendations in application of these linings. Accordingly, the following measures are included with respect to spray-on lining application in manholes:

- Stop active infiltration using chemical grout injections or hydraulic cement prior to product application.

- Apply surface treatments to remove contamination.
- Make sure there is a good bond between the spray-on material and substrate with weak layers of contamination removed.
- Apply high-pressure cleaning before application.
- Apply surface abrasion on the substrate for better bonding between the lining and manhole
- Follow the manufacturer's recommendations on coat thickness.
- Allow adequate curing and hardening time according to the manufacturer's recommendations.

The Saber and Cerda study recommended testing spray-on coatings for chemical resistance, permeability, tensile strength, compressive strength, flexural strength, adhesion, and bond strength. Saber and Cerda suggest the designer of a rehabilitation project choose the test method that they see fit for a specific application and material.

2.4 Hydrogen Sulfide Induced Corrosion

Hydrogen sulfide induced corrosion is one of the primary reasons for a wastewater collection system component failure and manholes are no exception. In fact, many manholes suffer from severe corrosion where conditions favor excessive hydrogen sulfide generation. These conditions include long detention times within the sewer system and relatively high-temperature wastewater with high biochemical oxygen demand (BOD) concentration. Another cause for hydrogen sulfide induced corrosion specifically at manholes is force main discharge into a manhole, which results in high levels of hydrogen sulfide release from the sewage. This topic has been studied elsewhere in a number of studies and the exact mechanism of hydrogen sulfide induced corrosion and hydrogen sulfide reduction in collection systems is not within the scope of this project. Nevertheless, corrosion directly relates to the residual structural strength of a manhole; i.e., as the wall thickness decreases need for a more structural repair on a manhole increases.

U.S. EPA released a design manual to prevent odor and corrosion control for sanitary sewers back in 1985 (*Odor and Corrosion Control in Sanitary Sewerage Systems and Treatment Plants/EPA/625/1-85 018*). The U.S. EPA manual emphasizes the effects of slope/velocity and temperature on sulfide generation in sanitary sewer systems (Figure 2-9).

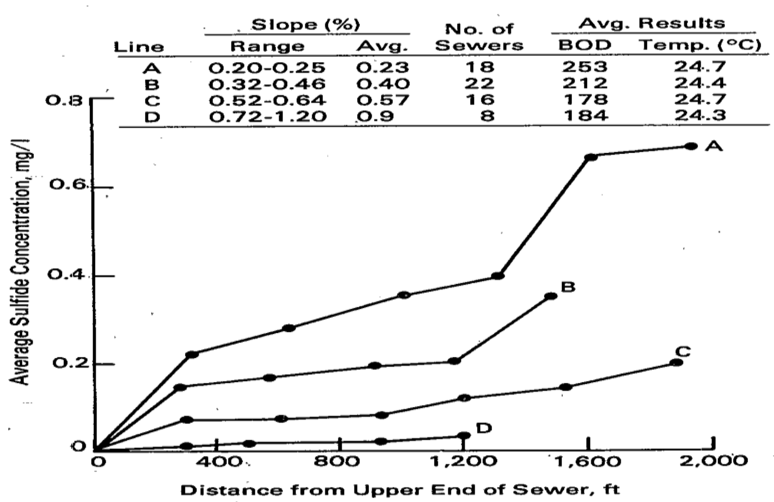

Figure 2-9. Sulfide Occurrence in Small Sewers.
U.S. EPA, 1985.

Boon (1995) performed a comprehensive review of septicity in sanitary sewers. Boon's work cited 57 articles published on this topic, and investigated the factors that cause septicity or hydrogen sulfide (H_2S) formation in collection systems and remedial measures that are available to reduce septicity and excessive H_2S generation. Boon's paper indicates a correlation among H_2S concentration, pipe diameter (full-section flow), and chemical oxygen demand (COD) concentration in wastewater:

$$C^s = K_4 L_{COD} t_s \left(\frac{1+K_5 d_1}{d_1}\right) \qquad \text{(Equation 2-3)}$$

Where C^s is the hydrogen sulfide concentration, t_s is the retention time of the sewage, K_4 and K_5 are empirical constants, and d_1 is the internal diameter of the pipe (assuming full section flow). Boon and Lister (1975) calculated K_4 and K_5 for domestic sewage to be 0.00152 and 0.004, respectively.

Boon's review outlined two major consequences of septicity/hydrogen sulfide in wastewater collection systems:

- Toxicity/odor: This is due to the release of hydrogen sulfide from sewage, and can be lethal.
- Corrosion: As discussed, this is because of formation of sulfuric acid in the collection system upon conversion of H_2S by *thiobaccilus* bacteria.

Boon listed six factors with respect to prevention and containment of septicity in sewers:

1. Oxidation of H_2S before it can be emitted to the atmosphere.
2. Conversion of H_2S to HS^- and S^{2-} ions.
3. Avoiding turbulent flow of septic sewage to prevent excessive loss of H_2S to the atmosphere.
4. Scrubbing vented gases to remove odors.
5. Using corrosion resistant materials in construction.
6. Cleaning of sewers to prevent accumulation of silt and slimes.

While it is better to prevent hydrogen sulfide formation than to remedy the consequences, for most cases, it may not be cost effective or feasible to eliminate H_2S; and therefore, it is present in the majority of the sewers around the globe. Item 5 listed above is directly related to the scope of this study from the rehabilitation standpoint of manholes. Polymeric materials (such as epoxies, polyurethanes, polyureas) are known to be more corrosion resistant than Portland based cements, but on the other hand, there is evidence that corrosion resistance of cement/concrete can be significantly, if not substantially increased by using micro silica admixtures, geopolymers and replacing Portland cement with calcium aluminates (Ehrich et al., 1999). Boon sets forth an argument against thin applied polymeric coatings, and states that pin holes that may form on these coatings will result in hydrogen sulfide intrusion and corrosion of the concrete substrate, which might go unnoticed until the structural integrity is compromised.

Nielsen et al. (2008) investigated hydrogen sulfide oxidation activity by the *thiobaccilus* bacteria with respect to corrosion in concrete manholes. The bacteria were exposed to hydrogen sulfide starvation for up to 18 months, upon which their hydrogen sulfide oxidizing activity was measured. It tested whether the observed reduction in biological activity was caused by a biological lag phase or by decay of the bacteria.

The results showed that the bacterial activity declined approximately 40% per month during the first two months of hydrogen sulfide starvation. After two to three months of starvation, the activity stabilized. After six months of starvation, exposure to hydrogen sulfide for six hours a day on three successive days could restore the bacteriological activity to about 80% of the initial activity. After 12 months of starvation, the activity could, however, not be restored, and after 18 months the biological activity approached zero. The long-term survival aspect of concrete corroding bacteria has implications for predicting hydrogen sulfide corrosion in sewer systems subject to irregular hydrogen sulfide loadings, e.g., as they occur in temperate climates where hydrogen sulfide often is a summer-problem only.

2.5 Auxiliary Manhole Components and Other Types of Rehabilitation

Condition of auxiliary manhole components such as the cover, frame, and steps can play an important role on the overall structural condition of a manhole. For instance, the way a manhole cover reacts to traffic loads and other external impacts has a direct impact on the manhole chimney and wall. Likewise, corroded iron steps may result in holes on the manhole wall and affect overall structural integrity. In addition, deteriorated steps are also a safety issue as they could easily break when stepped on[11].

The top manhole parts, riser rings, frame, and cover are also important from the I/I prevention standpoint as a good deal of I/I (particularly inflow) enter into manholes through holes and gaps along the perimeter of these parts. The good news is that as long as they are structurally sound, the top and other components of a manhole (including the chimney) can be sealed using inexpensive methods including the following:

- Chimney/Frame Seals: "Mechanical" rubber seals, external wraps (composed of HDPE external sheet, polypropylene reinforcement, and rubberized mastic sealer), elastomeric polyurethane (applied internally). Mechanical rubber seals can be applied internally (see Chapter 6.0, Figure 6-13) or externally (Figure 2-10).

Figure 2-10. "Mechanical" Rubber Seal Applied Externally
Source: Cretex Specialty Products.

[11] *This is the primary reason why many municipalities use portable ladders vs. fixed steps to get access to manholes.*

- Medium-Density Polyethylene (MDPE)/Stainless Steel Frame Inserts ("Inflow Dish"): This is an inexpensive method to prevent inflow entering the manhole cover. A stainless steel or MDPE bowl is simply inserted between the frame and cover (see Chapter 6.0, Figure 6-20).
- Internal Joint Seals: This is a rubber strip and stainless steel expansion system designed to seal the joints in a circular manhole (see Figure 6-15).
- Chemical or Cement Mortar Grout: Chemical or cement mortar grout injection is another inexpensive method to seal leaks that enter into the manhole through cracks and joints. Grout sealants are suitable for structurally sound manholes or could be applied prior to lining a manhole to stop leaks. They can be applied directly on the crack/gap or injected through a hole drilled on the manhole wall from the inside (Figure 2-11). The latter method works by filling in the voids around the manhole, thereby sealing it externally.

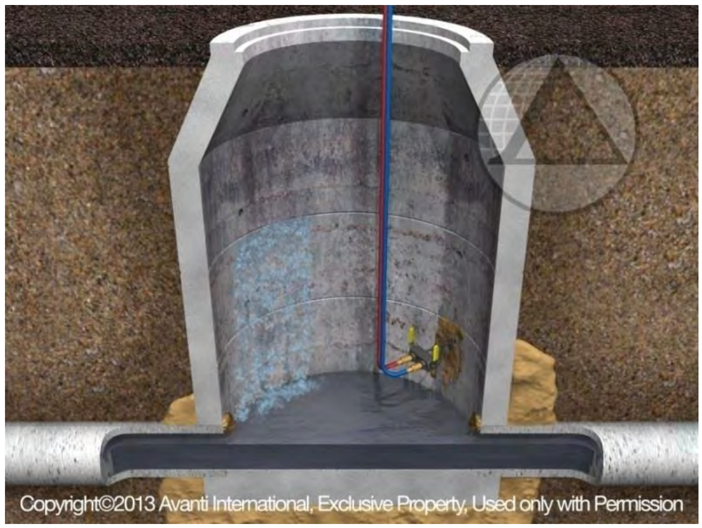

Figure 2-11. Chemical (Acrylamide) Grout Injection from the Inside of the Manhole to Stop Leaks.
Source: Avanti International.

- Flood Grouting: This is a relatively new method (at least in the U.S.) for trenchless rehabilitation of sanitary and storm sewer components (including manholes). Flood grouting is essentially filling an entire sewer segment (including the laterals) between two consecutive manholes with a two-part liquid chemical fluid, thereby sealing the holes and gaps upon curing of the engineered fluid (Burke et al., 2013). This is another non-structural rehabilitation method used for I/I removal. It has been applied in a number of projects in Europe, particularly in Germany.

The rehabilitation methods that pertain to auxiliary parts of manholes or designed to just seal leaks are not included in the testing program of this project. These products are not linings or prefabricated inserts; and therefore, structural classification does not apply to them. Nevertheless, some basic information was included herein to give a more complete picture to the reader on manhole rehabilitation materials and methods. Additionally, these methods were included in the cost table presented in Table 7-2 and added to the sample technical specification for manhole rehabilitation prepared as a part of this project (Appendix A).The partial rehabilitation solutions presented herein can be a very efficient solution in reducing (or even eliminating at least at manholes) I/I. However, they will not protect a manhole against corrosion; and therefore, although they cost less, they could not be a complete substitute for Class C linings.

CHAPTER 3.0

EXPERT WORKSHOP

Utility and manufacturer participation for the project consisted of sharing case histories and participating in the Expert Workshop, which was held on October 1, 2012 at WEFTEC 2012 in New Orleans, Louisiana. In addition to these tasks, the participating manufacturers provided lined samples for the preliminary tests and installed their linings into the concrete cylinders for the main tests (discussed in Chapter 4.0).

The objective of the Expert Workshop was to obtain as much contribution as possible from the participating industry professionals from wastewater utilities and rehabilitation material manufacturers/vendors by implementing small and large group discussions (see Table 3-1, workshop schedule). The workshop met its objective with 31 attendees including manhole rehabilitation material manufacturers, wastewater utility representatives from larger urbanized settings, key project team members, and one attendee from the U.S. EPA/IIRC (see Appendix B for the complete list of workshop attendees).The key findings of the Expert Workshop included:

- There is a need for manhole rehabilitation material classification based on their structural capabilities.
- There is a need for a standard (e.g., ASTM) specifically written for manhole rehabilitation.
- It is important to determine whether a structural rehabilitation is needed to stop I/I as for many cases, a manhole can be structurally intact with significant leaks.
- The decision support tool should recommend the most economical solution that can meet the objective of rehabilitation.
- Fully structural does not necessarily mean better, as this may drive up the cost and often times it is not needed to reduce or eliminate I/I.
- Adhesive properties of the lining material may be more important than its structural properties, especially if the host manhole is intact.
- Surface preparation and application quality is at least as important as the physical and mechanical properties of the rehabilitation material/system.

Table 3-1. Industry Expert Workshop Schedule.

Time	Activity
9:30 - 9:45	Introductions
9:45 - 10:15	Presentation by PI and Co-PI
10:15 - 11:00	Small Group Discussions
11:00 – 11:30	Presentation from Each Small Group
11:30 – 12:00	Conclusions, Remarks & Future Activities

3.1 Introductory Session

After a brief self-introduction of each project participant, the Principal Investigator introduced the project scope and objectives. The introductory session immediately created a forum for the participants to raise their concerns and expectations regarding the project.

For instance, a few participants pointed out the importance of standardization for manhole rehabilitation and a need for an ASTM Standard. This was a well-received comment as an ASTM International Committee (F17) is currently developing a standard for cured-in-place manhole rehabilitation - *New Practice for The Standard Practice For The Installation Of A Single-Sized Cured-In-Place Liner For Manholes Having Various Sizes (ASTM WK36573).* This standard is however, limited to resin impregnated felt tube linings, which are analogous to cured-in-place-pipe linings used for pipeline rehabilitation.

Another issue that was emphasized by the participants was the importance of setting boundaries to the type of materials and tests that will be applied on them due to the excessive number of materials and tests that could be used to characterize these materials. In other words, the majority of the project participants and the project team agreed that it is important to develop a testing protocol that will get straight to the point by evaluating the crucial properties of manhole rehabilitation systems in order to accomplish the project goals.

One question was raised whether the project would result in a design formula for structural manhole rehabilitation. Such a formula was incorporated into the decision support tool (DST) (Chapter 7.0). The decision support tool developed as a part of this study, enables the user to enter information about the site and manhole conditions, and returns a recommendation on which class rehabilitation system to use.

3.2 Small Group Discussions

Introduction of the workshop attendees and PI/Co-PI presentations were followed by small group discussions. The participants were divided into three groups. A project team member was present in each group (rather as a facilitator than leader) as the intent was to have other participants talk to the maximum extent.

3.2.1 Questions Asked

Participants were asked the following questions to help them address the issues that were of particular interest to this project.

3.2.1.1 What is the Primary Reason for Manhole Rehabilitation?

The answers from the participants varied. The most common answers were to stop I/I and restore structural integrity. A number of participants indicated the importance of preventing hydrogen sulfide induced corrosion and filling cracks and gaps. The follow-up question was whether I/I reduction and structural integrity restoration were related. This topic was further discussed in the large group discussion (see below).

3.2.1.2 How Do you Define "Structural Lining"?

There was no consensus among the participants regarding the definition of a structural liner. The need for classification of manhole rehabilitation materials and methods with respect to their structural capabilities was evident by the wide variety of answers to Question 2. There are several reasons for this. First, many of the manhole rehabilitation materials available today are neither structural nor non-structural, hence the term semi-structural is necessary to define structural capabilities of manhole rehabilitation materials.

Secondly, the definition of structural rehabilitation varied widely depending on the profession and background of the workshop participant. For instance, most manufacturers were of the opinion that if a liner system can add to the residual strength of an existing manhole, it should be considered structural, whereas most wastewater utility representatives and consulting engineers thought that a liner can be only deemed as structural if it can, at least after installation, withstand all of the exerted loads by itself.

Finally, definition of fully structural was an ongoing debate for pipeline rehabilitation as well. Many considered that a liner should be considered structural only if it can withstand the loads without the host structure (manhole) by itself (standalone). Should the rehabilitation material be "standalone" without any support from the host structure, then none of the cured-in-place linings, cementitious or polymeric, would qualify to be structural as they will require the host structure to form and cure. Another opinion was that the standalone condition should apply after the liner has cured fully. In other words, a liner should be considered structural if it can withstand the loads exerted on it without any support from the host structure (manhole) after application. An actual field condition for this would be a manhole losing its residual strength completely in long-term upon installation of the liner (for instance, due to extensive external corrosion). In contrast, a number of participants thought that a liner should be called structural as long as it adds to the strength of a manhole. This point of view claims a lined manhole should always be regarded as a system and a standalone condition for a liner will rarely be required, if ever, during the course of the service life.

3.2.1.3 What is the Most Important Property of a Manhole Rehabilitation Material with Respect to Performance and Longevity (rank the items in the order of importance and briefly explain why).

__ Resistance to hydrogen sulfide induced corrosion
__ Stiffness (ability to withstand loads exerted on the manhole)
__ Adhesion to manhole wall
__ Resistance to water exposure
__ Impact resistance
__ Other: ______________________

Likewise, the answers to this question varied in a wide range. Nevertheless, hydrogen sulfide induced corrosion, adhesion, and stiffness were ranked high by most of the participants. One participant emphasized the importance of blocking permeation through manhole walls. Another participant indicated that stopping I/I is the most and only important property of a manhole rehabilitation material. While the majority of the project participants and the project team agreed that I/I is the primary reason for manhole rehabilitation, it is rather the goal of manhole rehabilitation than a property of a manhole rehabilitation material. A few other participants stated that properties sought on a rehabilitation system completely depends on the site and host manhole conditions and avoided a ranking of the properties listed above.

Overall, the answers to Question 3 emphasized the importance of adhesion, resistance to hydrogen sulfide induced corrosion, and stiffness. Resistance to water exposure (during cure) and impact resistance were also included as important parameters of a manhole rehabilitation material by the majority of the participants.

Hydrogen sulfide induced corrosion has been addressed elsewhere in a number of studies (e.g., EPA Report 625/1-85 018, 1985, Boon, 1995, Nielsen et al., 2008). Determining structural capabilities (stiffness) of rehabilitation materials and methods is the primary objective of this study and adhesion testing is also included in the initial testing protocol.

The small group discussion on Question 3 bolstered the objectives and methods of the project. Additionally, it was useful in determining the parameters that would be tested on manhole rehabilitation systems as a part of this study.

3.2.1.4 Tell us About a Manhole Rehabilitation Project You Were Involved in. Was the Project Successful? What Lessons Were Learned?

Most of the examples given to question were the "successful" ones, and case histories were cited from different places including Massachusetts, Ohio, Wisconsin, and Louisiana. Careful planning and a thorough inspection/evaluation of the manholes prior to selecting and applying a manhole rehabilitation material were deemed as key factors for success. Nevertheless, a few participants did share the failures they had experienced and these failures were attributed to poor adhesion to the substrate (i.e., manhole wall). One failure was attributed to oil and grease, which was not completely removed off the surface of the manhole prior to installation of the liner.

Some of the participants indicated that they had "too many" experiences to choose from and did not provide a specific answer. Whereas, a couple of participants did not have direct experience with manhole rehabilitation, but they were participating because of plans to enter the manhole rehabilitation market or were interested in the subject from the regulatory standpoint.

Among the other topics discussed in the small group discussions was a comparison between conventional (open-trench) and no-dig technologies for manhole rehabilitation. The participants listed the following as advantages of using no-dig methods:

- Depending on the location, no-dig rehabilitation may substantially reduce expenditure of rehabilitation.
- Social cost saving.
- Rehabilitation can be performed in a crowded urban city without disruption to traffic.
- Excavation may weaken the whole system (i.e., pavement, soil around the manhole, etc.).

These comments were well in line with the literature published to date (see Chapter 2.0) and the project team experience with trenchless technologies (or no-dig for the case of manhole rehabilitation). Another advantage of no-dig manhole rehabilitation is reduced greenhouse gas emissions associated with the open-cut method.

Importance of surface preparation prior to application of a liner on the manhole interior was emphasized by a number of participants. The general opinion on this is "a liner is as good as the substrate." It was also suggested that a surface preparation be discussed in this report, which is intended to serve as a guideline for manhole rehabilitation.

Another topic of discussion was the applicability of a rehabilitation material while the manhole is in service. In other words, what would be the downtime for a manhole during rehabilitation? Will there be a need for bypassing the flow to apply rehabilitation. The answer to this question is "yes" for essentially all of the cured-in-place manhole linings as they require a certain cure time that may result in overflow if the flow into the manhole is simply plugged in the upstream pipe(s). Nevertheless, some prefabricated manhole inserts could be applied with minimal downtime without the need for a bypass.

A particular concern raised by the manufacturers was the lack of an ASTM standard specifically developed for manhole rehabilitation. Perhaps quite rightfully, a number of manufacturers stated that using ASTM F1216[12] as the reference specification for manhole rehabilitation is misleading to the wastewater utilities. ASTM F1216 is designed for cured-in-place-pipe rehabilitation with the inversion method, and for the most part, it is not applicable to manholes. Upon this discussion, the project team discovered that there is indeed an ASTM standard currently being developed for manhole rehabilitation (ASTM WK36573). However, the standard-in-progress is geared towards one kind of manhole liner, i.e., resin impregnated cured-in-place lining system. As another outcome of the small group discussions, the project team discovered the necessity of establishing a task force to develop a new ASTM Standard for other kinds of cured-in-place manhole rehabilitation materials and methods (e.g., epoxy/polyurethane or corrosion resistant cement mortar linings).

"Manholes can be stable with defects" – stated one of the participants in the small group discussions, and this is true. It was emphasized by the project participants that although most of the manholes that have been in service beyond their design life will have defects of some sort, this does not mean that they are structurally deficient and need structural repair. Though, the views on "for what percentage of manholes that are in need of rehabilitation require structural rehabilitation" depends on the background of the participants. Hence, NASSCO's Manhole Assessment and Certification Program (MACP) provides codes defining the type and magnitude of a defect. MACP is gaining more acceptance in sanitary sewer evaluation, and is a useful tool in condition assessment of a manhole. Although, using NASSCO's comprehensive defect coding system can give a good insight to a manhole's condition, sometimes visual inspection may not be adequate to determine the structural condition (i.e., residual strength) of a manhole if there is degradation in the manhole material without a clear sign (though often times structural deficiencies are apparent with wall thinning due to corrosion or cracking/fracturing on the manhole wall or other components).

3.2.1.5 Other Small Group Discussions Items

"Integrity of liners" was another discussion item in the small group discussions. Some of the participants raised the issue of overall integrity of a lining material in addition to material properties. To illustrate, a manhole rehabilitation material can have a high tensile strength , for instance, but a lined manhole is a system that is subjected to soil movements, groundwater pressure (where it applies), and other loads. One or two physical properties that indicate high strength do not necessarily mean that the material will perform well. After all, what is found in the field is a host-structure liner system that may be tens of feet (or meters) deep, not a rectangular coupon that is per say 1 in. (25 mm) wide and 3 in. (75 mm) long. This is a valid concern and the primary reason why this study included a minimum of three types of testing (discussed in Chapter 4.0) in addition to computational modeling using the FEM. Unlike the standard tests carried out by the manufacturers (this is because ASTM tests on flexure/tension and compression, for example, do not call for using substrates), the material tests were conducted on lined specimens, which enabled the project team to evaluate the substrate-lining system with respect to its mechanical properties. Then using the finite element analysis with the actual material properties will provide performance assessment of these lined systems as integral systems with simulated varieties of site conditions representative of actual conditions that can be

[12] *Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube.*

encountered in different soil types and other parameters (such as groundwater table, traffic load and other dynamic loads such as earthquakes). All of this plays a role on the overall performance of a lined manhole as a system.

Another discussion was on the importance of bonding strength of a liner to the substrate. Unlike other mechanical properties, bonding strength of a liner can be a double-edged sword. High bonding strength is desired to eliminate any annular space between the lining and manhole wall, so that the two can work as a system utilizing the residual strength of the manhole. On the other hand, if the bonding strength is too strong, i.e., stronger than the tensile strength of the liner, then any crack or fracture in the manhole wall will be transmitted to the liner, and will result in a "system" failure. This is described as adhesion (between lining and the substrate) vs. cohesion (intermolecular forces within the lining). Accordingly, ideal liners would have adequate adhesive strength to work as a lining-substrate system, but not stronger than the ultimate tensile strength of the lining material that would make it break along with the substrate (manhole wall). This condition applies to cured-in-place linings, particularly polymeric ones as bonding is not an issue for cement based linings, and structural manhole inserts are stand-alone and do not need to adhere to the manhole wall for structural purposes. Nevertheless, good bonding will be required for any grout that is injected between the manhole interior wall and manhole insert to fill the annular space. Some of the participants raised concern on failure of liner due to poor bonding when it is applied on wet surface. This type of failure is in fact claimed as an advantage of cement liners over polymers by the cement mortar lining manufacturers.

Small group discussions also served as a forum to discuss any other issues pertaining to manholes not covered in the questionnaire handed out by the project team. For instance, among these discussions was the lack of understanding of the importance of manholes by the public and many wastewater utilities. This is not surprising as the focus of the rehabilitation industry has been on pipelines. Pipelines are more valuable assets than manholes due to their quantity, nevertheless, the estimates on the number of manholes in the U.S. alone are as many as 20 million, making manholes a very important asset class for the municipalities around North America and around the globe. In addition, manholes are the primary source for rain water entry into the collection systems (inflow), which is one of the main reasons for underground infrastructure rehabilitation programs. Without addressing problems with manholes, a sanitary sewer rehabilitation project cannot be complete. Fortunately, there is a growing awareness on the importance of manholes (hence this project), and the participants have emphasized the importance of "getting the word out" to the utilities as a side benefit of this project.

Some of the participants pointed out the usefulness of spot repairs by chemical grout injection in stopping infiltration (and perhaps some of the inflow). One of the participants claimed that structural repair of a manhole would not be needed on a manhole as long as I/I is stopped and the surrounding soil is stabilized by chemical grout injection. While chemical grout injection is proven to be an efficient method of filling voids around a manhole, and thereby stopping leaks, there is yet no evidence that a manhole can be securely stabilized by chemical grout injection from the inside. Even if this is achieved, if the host structure is structurally deficient due to corrosion and other effects, soil stabilization will not offset the need for structural rehabilitation. Moreover, it is hard to imagine a chemical grout that can prevent vertical movements of a manhole or the surrounding soil.

Installation/cure time of manhole rehabilitation materials was mentioned as an important design and material selection parameter by the participants. This is important as fast curing materials or manhole inserts might enable installation without bypassing the flow. Nevertheless, manhole rehabilitation without bypassing flow should be carefully planned and coordinated with all the involved parties. This is usually done by simply plugging the upstream pipes. Overflows may occur due to exceeding the storage capacity upstream and unfavorable topographic conditions (that is pipes laid close to the surface). Returning a rehabilitated manhole back to service before the lining cures completely may result in a failure. Therefore, although there seems to be a consensus on considering the cure time as a selection parameter, if the wastewater flow is bypassed, a few hours of difference is not a detrimental factor.

Participants of a small group discussion also emphasized the need for more certified applicators of manhole rehabilitation materials. There are fundamental differences among manufacturers with respect to certification of applicators. Some of them have more vigorous training requirements, and they strictly limit the application of their materials by the certified contractors to issue a warranty on the product. Others favor the do-it-yourself approach, and they encourage wastewater utilities' personnel to get a rather short training, and apply the material to their system using in-house resources. This is also obviously driven by the type of rehabilitation material. For instance, trowel applied systems are easier to apply than spin-cast ones, which would require more equipment and training.

Another important aspect of manhole rehabilitation pointed in one of the small group discussions was the importance of "complete" manhole rehabilitation. This remark specifically refers to manhole rehabilitation that solely focuses on the top part of a manhole (top cones/wall, chimney, and cover). This type of rehabilitation is inadequate for particularly deep manholes that receive infiltration close to the base (bench) and suffer from structural defects due to increased soil/hydrostatic pressure. Another reason that might dictate a "bottom up" rehabilitation vs. "top down" is poorly constructed, damaged, or missing benches and inverts at the bottom of manholes. These defects are quite common, and could significantly affect the flow through a manhole in addition to causing structural instability if the defects are severe.

Figure 3-1. Snapshot from the Introductory Session of the Workshop Held at WEFTEC 2012 in New Orleans, Louisiana.

3.3 Large Group Discussion

Upon completion of the small group of discussions, each group assigned a representative to present the key points addressed by his/her group. It was interesting to note the difference among the groups with respect to the subjects that received more emphasis in comparison with the others. For instance, one group had a long discussion on case histories, whereas, another group did not discuss Question 4 at all due to the time limitations. There was however, a consensus on addressing capabilities of rehabilitation systems with respect to prevention of I/I and providing structural support where needed.

A large group discussion followed the small group presentations, and most of the comments by the participants were on the following four topics.

3.3.1 Need to Define What is Structural and Non-Structural

There were three different opinions on this critical question. The first group believed that a liner should be labeled structural as long as it provides "some" support to the manhole and adds to its stiffness. Unsurprisingly, this opinion typically belonged to the manufacturers that sell linings that are rather thinly applied with low stiffness. The second group advocated that the lining should be deemed fully structural after it cures (if it is a cured-in-place liner). That means it is okay to rely on the host structure (manhole) during application, but the lining should withstand all the loads exerted on it even if the manhole completely loses its strength. This would be an exceptional, but possible case if the manhole is relatively intact during application, but the exterior corrodes to a point that its residual strength becomes negligible. The third group thought that a rehabilitation system can be only fully structural if it is standalone, i.e., the liner or manhole insert should be completely independent of the manhole during application or withstanding the loads throughout the course of service life. If the third definition is accepted, then none of the cured-in-place lining, no matter how strong they are after application, would be regarded as structural. The project team, as well as the majority of the participants, favored the second definition stated above as the most appropriate definition for "fully structural" liner. As proposed by the project team from the beginning of the project, this discussion emphasized the need for a definition of the third group: Semi-structural liners.

3.3.2 Prior Studies

The workshop participants pointed out previous studies that were done on manhole rehabilitation and were not yet discovered by the project team. One example was the experimental work done at the University of Houston, where Dr. Vipulanandan et al. conducted several projects on bonding strength of cured-in-place polymeric linings/coatings (epoxy) and their durability against hydrogen sulfide induced corrosion (see Chapter 2.0).

3.3.3 Sample Collection for Testing

A few participants argued that testing factory manufactured liner samples would not provide much useful data as the manufacturers have already done the tests included in this study, for the most part, by a third party laboratory. These participants thought it would be more useful to obtain liner samples from the field that have been in service for an extended period. The project team on the other hand, was of the opinion that testing lined specimens vs. field samples from actual applications would be more beneficial for the following reasons:

1. Even if it is done by an independent lab, the test on lining materials and other systems, are from experiments conducted on the rehabilitation material only (except adhesion tests), excluding the substrate as ASTM procedures do not require testing a lined system. The testing protocol of this study includes lined specimens as opposed to just testing the liner. The project team (as well as a good number of the workshop participants) think that it is the performance of the lined "system" that matters, not the liner or another rehabilitation material of its own.
2. It would be difficult to develop a method to obtain identical samples without causing structural damage to the manholes that are in service. Time and budget limitations may not allow it.
3. Even if the project team successfully obtained lined manhole specimens, the results of the tests done on these samples would only represent the condition of the specific site where the specimens are taken. Another question was which part of the manhole, should the specimens be cut out? While it was more cumbersome (and risky) to take specimens from the bottom part of the manhole, this is where the manholes are typically subjected to the highest groundwater and soil pressure.
4. Statistical confidence of any testing done with field samples may be poor due to the difficulty of obtaining identical specimens in very limited numbers.
5. It would be difficult to convince wastewater utilities to allow taking field samples from their manholes that are in service, even if the manholes were restored to the original condition. Nevertheless, filling the voids (where specimens are taken) to the prior or better condition was yet another challenge to be met if field samples were tested as a part of the experimental work.

3.3.4 Decision Support Tool (DST) Content

Some of the participants raised concerns about the parameters, that would be included in the decision support tool, as well as the format of the results returned by the software. For instance, referring to the title of the project, some of the participants stated that a decision support tool that makes recommendations on only structural capabilities of the manhole rehabilitation systems would be misleading as for many manholes structural support may not be even needed to stop I/I (discussed further below). Another concern was whether the DST would make recommendations on using specific brands. The project team understood and agreed with these concerns; as such, all of the known parameters of manhole rehabilitation performance were included in the DST developed as a part of this project (discussed in Chapter 7.0). These parameters included, but were not limited to, manhole condition, site condition (soil, groundwater, traffic and other loads), cost of the rehabilitation system, and overall objective of the rehabilitation project. Thus, the DST is not designed around just the structural capabilities of manhole rehabilitation systems. The PI also emphasized that the DST would avoid using brand names and recommend manhole rehabilitation systems based on the classification developed as a part of the project.

3.3.5 Need for Structural Rehabilitation vs. Stopping I/I

Another group of participants claimed that the primary reason for manhole rehabilitation is reducing/stopping I/I and structural rehabilitation is not needed for most of the manholes. The project team believed that this was a biased argument, because at a minimum, structural rehabilitation is needed. For the manholes that are in poor condition for most cases, it will cost less than replacement (economically and socially). Another reason for optimum structural strength that would be desired on a rehabilitation system is the correlation between preventing I/I and structural capability of the rehabilitation system. For instance, in particularly a deep manhole, if the rehabilitation system is non-structural, any failure due to soil pressure or hydrostatic pressure will result in infiltration. In other words, structural failure of a manhole is directly related to I/I (particularly to infiltration as most of the inflow enter into the manholes through the gaps around the perimeter of the cover).

The expert workshop was useful in improving the project scope and experimental approach. While it was almost impossible to reach a consensus within a group of participants with different backgrounds and motives, this meeting enabled the project team to explore these different perspectives and discover some other studies and experiences brought up by the project participants.

Most importantly, it was understood that there is an absolute need to classify manhole rehabilitation materials and provide rehabilitation guidelines that are specifically designed for manhole rehabilitation.

CHAPTER 4.0

EXPERIMENTAL: STRUCTURAL TESTS ON MANHOLE LININGS

This chapter covers the experimental work carried out to evaluate the effect of linings on manholes with respect to their structural properties. The majority of the manhole rehabilitation materials were tested per the applicable ASTM Standards by the manufacturers, and a good deal of the data came from third party testing. Nevertheless, these tests were run on the lining material itself, except adhesion tests for which a concrete substrate is used per ASTM D4541. A lined manhole is a system with two components; and the contribution of the lining to the structural capabilities of a host structure can be best determined by testing a lined system. Accordingly, the experimental method used in this study is based on testing a lined system. Although, ASTM standards are used as a reference, the procedure developed herein is customized to fulfill the objective. A two-step approach was implemented to investigate the structural capabilities of manhole rehabilitation materials. A set of preliminary tests were carried out on small specimens to obtain a general idea on the structural capabilities of the linings from the participating manufacturers; and then a more elaborate test procedure was developed based on the preliminary testing experience. These tests are discussed in detail below.

4.1 Preliminary Tests

The project team developed a preliminary procedure for the preliminary experiments; and then, held a conference call with a select group of manufacturers to discuss the preliminary tests. The materials included in the preliminary tests were epoxy liner (EPX), one polyurethane (PU), one corrosion resistant cement liner (CMT), two composite lining systems with cement liner and polymeric coating (CMP1 and CMP2), 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 strength tests are conducted based on the ASTM standards indicated in Table 4-1.

Table 4-1. Reference ASTM Standards Used for the Preliminary Tests.

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)

4.1.1 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. (152 mm × 152 mm × 508 mm) 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. (76 mm × 76 mm × 203 mm) was used. The 3 in. × 8 in. (76 mm × 203 mm) 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).

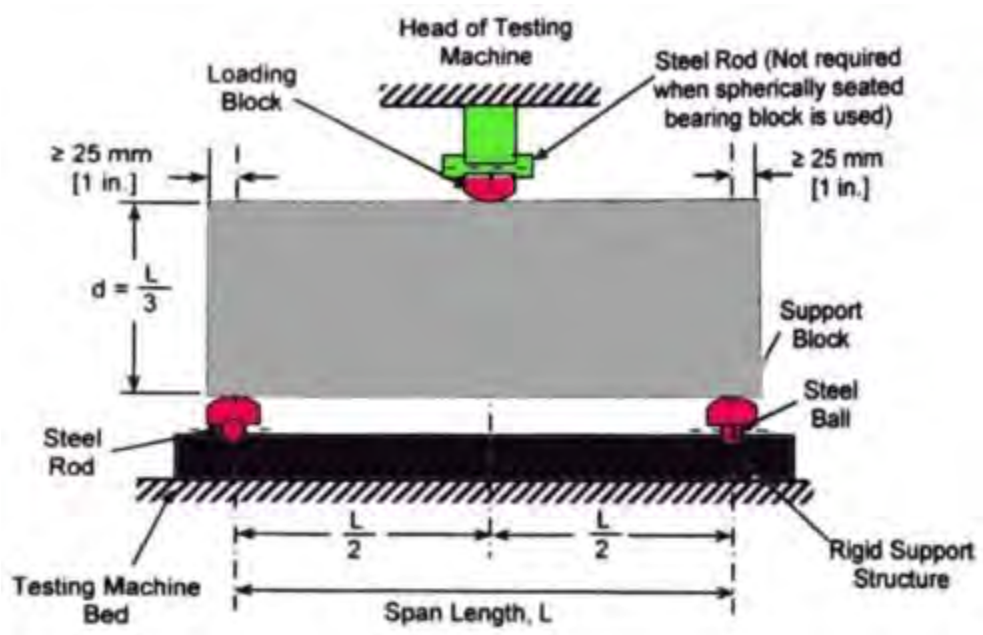

Figure 4-1. Flexural Strength Test Setup per ASTM C293.

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 is tested by third-point loading. In ASTM C293 a simple concrete beam is tested by center-point loading (Figure 4-1). 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 MR in MPa or psi. A 60 KIP Baldwin flexure testing machine (see Figure 4-2) was used for the flexural strength tests.

Figure 4-2. Baldwin 60 KIP Flexural Strength Testing Machine.

4.1.2 Compressive Strength Tests

Of the many tests applied on concrete, this is perhaps the most important test, because it gives an insight about several characteristics of concrete. The purpose of preliminary compressive strength tests is 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 lining 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 4-2).

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-in. (50 mm) cube specimens to failure and results may be used to determine compliance with specifications.

The small cube test specimens (ASTM C109) were easy to prepare and ship to the lab from various parts of the country, where the substrates are to be lined by the manufacturers for testing. Nevertheless, some of the lining materials included in this study cannot be applied on the standard small cube substrate; and the project team was intended to keep the substrate identical for all of the lining materials included in this study to compare the results; so that they could be classified with respect to their structural capabilities. The concrete cylinder substrates were lined externally by the manufacturers; and shipped to CUIRE for a compressive strength test that is parallel to ASTM C39.

Figure 4-3. The Testing Instrument (ADMET 500 KIP) Used for the Compressive Strength Tests.

4.1.3 Unlined (Control) Specimens

Forty-two concrete cylindrical [4 × 8 in. (100 × 200 mm)] and 41 concrete beam [3 × 3 × 11 in. (75 × 75 × 280 mm)] samples were prepared for the preliminary tests. The design strength of concrete was approximately 5,000 psi (34,500 kPa). The unlined (control) specimens prepared for the preliminary experiments are shown in Figure 4-4.

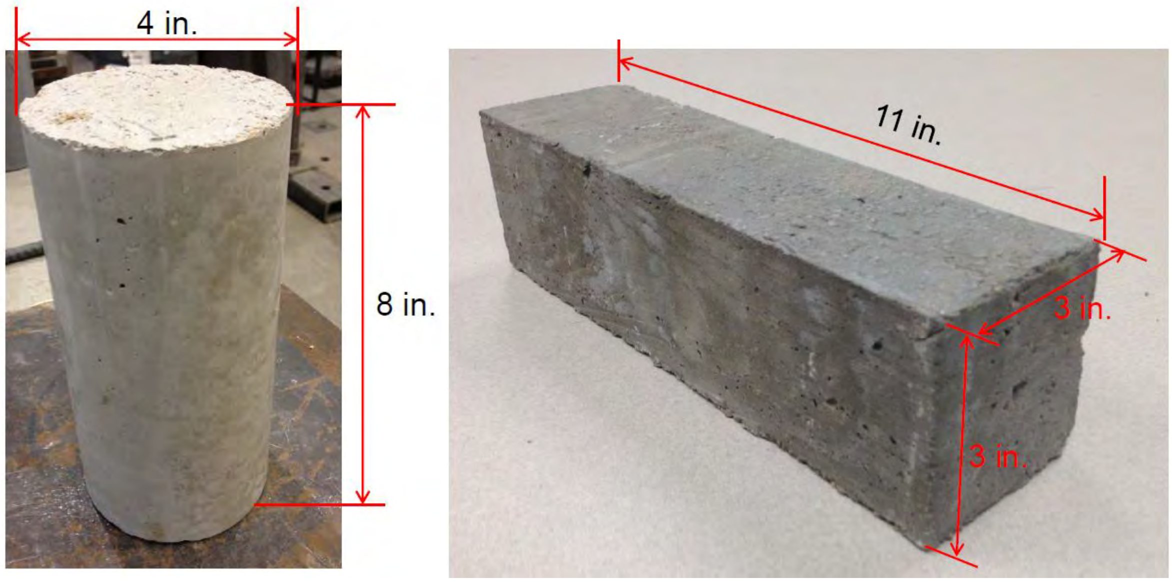

Figure 4-4. Typical Concrete Cylinder and Beams Used for the Preliminary Experiments.

4.1.4 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. The following instructions were given to manufacturers:

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.

Six of the seven manufacturers sent the lined specimens back to CUIRE for testing. Figures 4-5 and 4-6 show the lined concrete beam and cylinder samples for polyurethane and cured-in-place (CIP) lined specimens, respectively.

a b

Figure 4-5. Concrete Cylinder (a) and Beam (b) Samples Lined with High-Build Polyurethane.

a b

Figure 4-6. Concrete Cylinder (a) and Beam (b) Samples Lined with Cured-in-Place Composite (Nonwoven Textile and Thermoset Polymer) Liner.

4.1.5 Test Methodology

Before the start of each test, all the dimensions were measured using a calibrated digital caliper (see Figure 4-7), both in the SI and U.S. customary units. The thickness of each lining was also measured.

For the cylindrical samples, the diameter was measured vertically and horizontally (see Figure 4-8). 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 two 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 4-7. Digital Caliper Used for Measuring Specimen Dimensions.

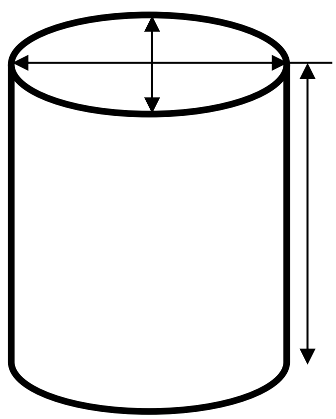

Figure 4-8. Cylinder Dimension Locations.

4.1.6 Summary of Preliminary Test Results

A summary of the preliminary test results along with a discussion of the results are provided below. Detailed information on the preliminary tests such as specimen condition before and after testing and pictures along with the complete test data are provided in Appendix C.

The test specimens used for the preliminary tests were coded based on their material composition and the type of test applied upon. Table 4-2 indicates brief descriptions of the linings tested and their respective codes. The final letter used in the test specimen code indicates the type of test applied on the specimen as F and C for flexure and compression, respectively.

Table 4-2. Specimen Code Used for the Preliminary Tests.

Specimen Code	Material Composition	Type of Test Applied
UNL-F	Unlined, bare concrete specimen	Flexure
EPX-F	High-build epoxy	Flexure
CIP-F	Cured-in-place thermoset resin and non-woven textile composite	Flexure
PU-F	High-build polyurethane	Flexure
CMP-F	Cement mortar plus epoxy on the top	Flexure
MULT-F	Three-layer polymer composite (polyurethane foam and proprietary thermoset)	Flexure
UNL-C	Unlined, bare concrete specimen	Compression
EPX-C	High-build epoxy	Compression
CIP-C	Cured-in-place thermoset resin and non-woven textile composite	Compression
PU-C	High-build polyurethane	Compression
CMP-C	Cement mortar plus epoxy on the top	Compression
MULT-C	Three-layer polymer composite (polyurethane foam and proprietary thermoset)	Compression

4.1.6.1 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 4-2. The testing was recorded with a camcorder, and once it was complete, pictures of the tested specimens were taken for records. Figures 4-9 and 4-10 show the views of the concrete cylinder before and after compressive loading, whereas Figures 4-11 and 4-12 show the concrete beam views before and after flexural loading.

Figure 4-9. Unlined (Control) Cylinder Prior to Testing.

Figure 4-10. Unlined (Control) Cylinder After Testing.

Figure 4-11. Unlined (Control) Beam Prior to Testing.

Figure 4-12. Unlined (Control) Beam After Testing.

Summaries of the results for the preliminary tests on control specimens are provided in Tables 4-3 and 4-4.

Table 4-3. Summary of Results for Unlined (Control) Cylinders.

Sample ID	D×H*		Test Type	Test Date	Cross Section Area		Loading Rate		Peak Load		Peak Stress	
	mm	in.			mm²	in.²	N/s	lbs./sec	kN	lbs.	MPa	lbs./in.²
UNL-C#1	99.06×203.20	3.9×8	ASTM C39	02/15/2013	7,709.66	11.95	<445	<100	278	62,390	35.99	5,220.92
UNL-C#2	96.52×200.66	3.8×7.9	ASTM C39	02/15/2013	7,316.11	11.34	<445	<100	270	60,783	36.95	5,360.05
UNL-C#3	104.14×205.74	4.1×8.1	ASTM C39	02/19/2013	8,522.56	13.21	<445	<100	275	61,740	32.22	4673.73

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

Table 4-4. Summary of Results for the Unlined (Control) Beams.

Sample ID	Sample Dimensions W×H×L*		Test Type	Test Date	Cross Section Area		Loading Rate	Peak Load		Peak Stress	
	mm	in.			mm²	in.²		kN	lbs.	MPa	lbs./in.²
UNL-F#1	73.66×71.12×276.86	2.9×2.8×10.9	ASTM C293	02/15/2013	5,238.69	8.12	4.1	8.55	1,924	6.63	962
UNL-F#2	68.58×71.12×281.94	2.7×2.8×11.1	ASTM C293	02/15/2013	4,877.40	7.56	4.1	8.95	2,013	6.94	1,007
UNL-F#3	68.58×73.66×274.32	2.7×2.9×10.8	ASTM C293	02/19/2013	5,051.60	7.83	4.1	6.95	1,564	5.39	782

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

4.1.6.2 Lined Specimens

The project team tested one epoxy spray applied lining materials (EPX1)[13] one cured-in-place applied lining material (CIP), one polyurea spray applied lining material (PU), one cement-polymer composite (CMP), and one multi-layer 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 prep) and tested at CUIRE by three-point bending and compressive loading until failure (Figure 4-13).

a b

Figure 4-13. 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 4-14. Additionally, basic statistical analyses for the flexural strength test results are indicated in Tables 4-5 through 4-11. Details of the flexural strength tests (preliminary) results on lined specimens are included in Appendix C.

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[13] Another set of epoxy lined specimens from a participating manufacturer were not tested, because, due to miscommunication, these samples were not prepared per the procedure developed for the preliminary tests.

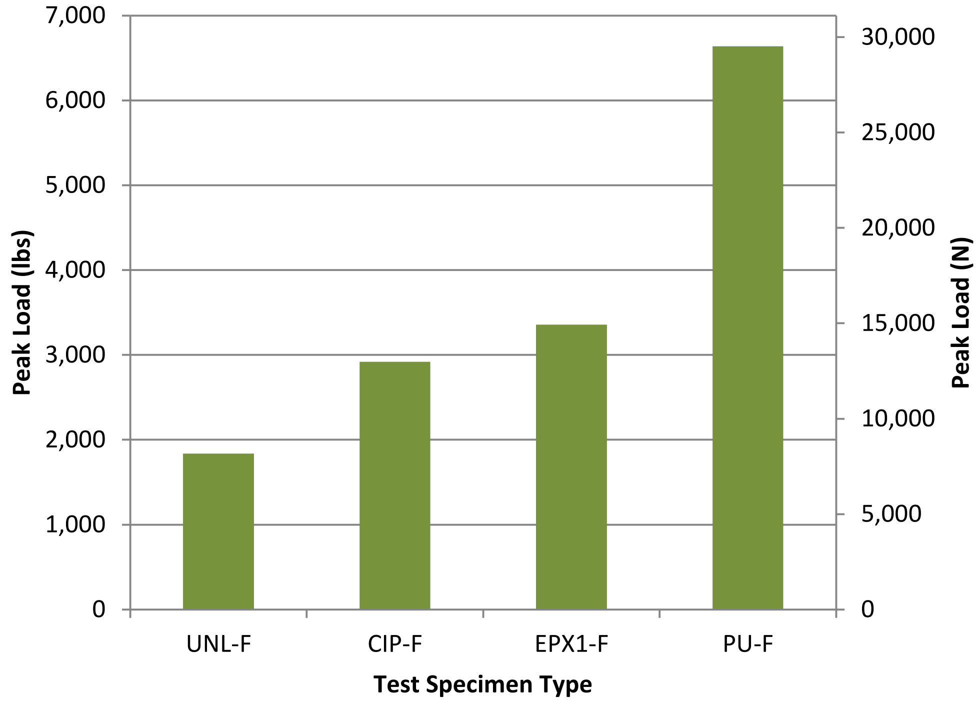


Figure 4-14. Ultimate Flexural Strength of Unlined (Bare) and Lined Specimens.

Table 4-5. EPX1-F Test Result Statistics.

Average Bare Sample Load: 1,834 lbs.

Sample ID	Liner Thickness mm	Peak Load kN	Peak Load lbs	% Difference Load
EPX1#1	7.7	18.91	4,251	(+)131.8
EPX1#2 (Minimum)	5.6	9.38	2,109	(+)15.0
EPX1#3 (Maximum)	5.7	20.02	4,500	(+)145.4
EPX1#4	4.9	10.31	2,318	(+)26.4
EPX1#5	5.5	15.97	3,590	(+)95.8
Average (A)	5.9	14.91	3,354	(+)82.9
Standard Deviation (S)	NA	4.85	1,095	NA
Upper Quartile (UQ)		18.91	4,251	
Lower Quartile (LQ)	NA	10.31	2,318	NA
Interquartile Range (IQR)	NA	8.60	1,933	NA
Non-Outlier Range	NA	-2.59 to 31.81	-582 to 7,151	NA
Number of Outliers	NA	0	0	NA

Table 4-6. CIP-F Test Result Statistics.
Average Bare Sample Load: 1,834 lbs.

Sample ID	Liner Thickness mm	Peak Load kN	lbs	% Difference Load
CIP#1	4.6	10.75	2,416	(+)31.7
CIP#2 (Minimum)	4.1	9.79	2,201	(+)20.0
CIP#3	4.9	11.59	2,607	(+)42.2
CIP#4	4.5	11.13	2,503	(+)36.5
CIP#5 (Maximum)	7.0	21.64	4,865	(+)165.3
Average (A)	5.0	12.98	2,918	(+)59.2
Standard Deviation (S)	NA	4.88	1,098	NA
Upper Quartile (UQ)	NA	11.59	2,607	NA
Lower Quartile (LQ)	NA	10.75	2,416	NA
Interquartile Range (IQR)	NA	0.84	191	NA
Non Outlier Range	NA	9.49 to 12.85	2,130 to 2,894	NA
Number of Outliers	NA	1	1	NA

Table 4-7. PU-F Test Result Statistics.
Average Bare Sample Load: 1,834 lbs.

Sample ID	Liner Thickness mm	Peak Load kN	lbs	% Difference Load
PU-F#1 (minimum)	7.4	23.19	5,213	(+)184.3
PU-F#3 (maximum)	7.4	32.83	7,382	(+)302.6
PU-F#5	7.9	31.08	6,989	(+)281.2
PU-F#6	8.7	29.92	6,728	(+)266.9
PU-F#7	8.9	24.52	5,514	(+)200.7
Average (A)	8.1	28.31	6,365	(+)247.1
Standard Deviation (S)	NA	4.22	949	NA
Upper Quartile (UQ)	NA	31.08	6,989	NA
Lower Quartile (LQ)	NA	24.52	5,514	NA
Interquartile Range (IQR)	NA	6.56	1,475	NA
Non Outlier Range	NA	14.68 to 40.92	3,301 to 9,202	NA
Number of Outliers	NA	0	0	NA

Likewise, a summary of the compressive strength tests results is indicated in Figure 4-15. A basic statistical analysis for each lining test results is also indicated in Tables 4-9 through 4-11. Details of the compressive strength test (preliminary) results on lined specimens are included in Appendix C.

Table 4-8. CMP-F Test Result Statistics.
Average Bare Sample Load: 1,834 lbs.

Sample ID	Liner Thickness mm	Peak Load kN	lbs	% Difference Load
CMP-F #1	3.6	9.80	2,205	(+)20.2
CMP-F #2 (maximum)	3.3	11.34	2,551	(+)39.1
CMP-F#3	3.5	10.63	2,390	(+)30.3
CMP-F #4	3.4	10.01	2,250	(+)22.7
Average (A)	3.45	10.45	2,349	(+)28.1
Standard Deviation (S)	N/A	0.60	156	N/A
Upper Quartile (UQ)	N/A	10.63	2,430	N/A
Lower Quartile (LQ)	N/A	10.01	2,239	N/A
Interquartile Range (IQR)	N/A	0.62	192	N/A
Non Outlier Range	N/A	9.08 to 11.56	1,951 to 2,718	N/A
Number of Outliers	N/A	0	0	N/A

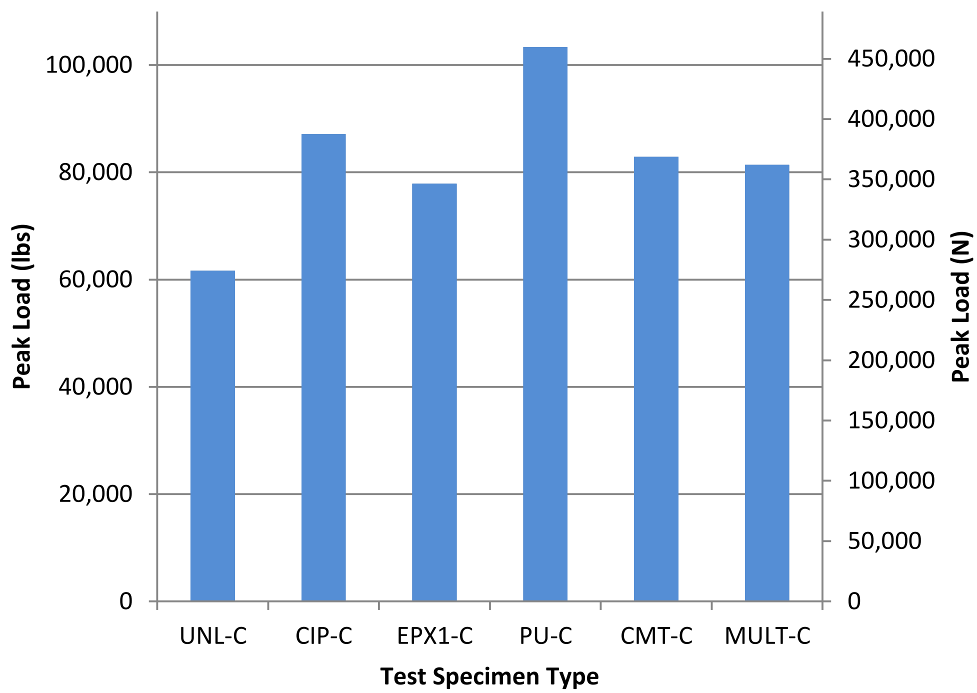


Figure 4-15. Ultimate Compressive Strength of Unlined (Bare) and Lined Specimens.

Table 4-9. EPX1-C Test Result Statistics.

Average Bare Sample Load: 61,637 lbs.

Sample Name	Liner Thickness mm	Peak Load kN	lbs	% Difference Load
EPX1#1 (Maximum)	3.1	380.42	85,520	(+)38.8
EPX1#2	3.2	359.1	80,730	(+)31.0
EPX1#3	2.8	337.18	75,800	(+)23.0
EPX1#4	3.0	367.02	82,510	(+)33.9
EPX1#5 (Minimum)	4.0	288.87	64,940	(+)5.4
Average (A)	3.2	346.52	77,900	(+)26.4
Standard Deviation (S)	NA	35.84	8,058	NA
Upper Quartile (UQ)	NA	373.72	84,015	NA
Lower Quartile (LQ)	NA	313.03	70,370	NA
Interquartile Range (IQR)	NA	60.69	13,645	NA
Non Outlier Range	NA	221.99 to 464.76	49,903 to 104,483	NA
Number of Outliers	NA	0	0	NA

Table 4-10. CIP Test Result Statistics.

Average Bare Sample Load: 61,637 lbs.

Sample Name	Liner Thickness mm	Peak Load kN	lbs.	% Difference Load
CIP1 (Minimum)	6.3	340.16	76,470	(+)24.1
CIP2	6.4	382.28	85,940	(+)39.4
CIP3	6.2	361.11	81,180	(+)31.7
CIP4	7.0	358.97	80,700	(+)30.9
CIP5 (Maximum)	6.6	495.22	111,330	(+)80.6
Average (A)	6.5	387.54	87,124	(+)41.4
Standard Deviation (S)	NA	62.01	13,941	NA
Upper Quartile (UQ)	NA	438.75	98,635	NA
Lower Quartile (LQ)	NA	373.72	78,585	NA
Interquartile Range (IQR)	NA	89.18	20,050	NA
Non Outlier Range	NA	239.95 to 572.52	48,510 to 128,710	NA
Number of Outliers	NA	0	0	NA

Table 4-11. PU-C Test Result Statistics.

Bare Sample Average Ultimate Compressive Strength: 61,637 lbs.

Sample Name	Liner Thickness mm	Peak Load kN	lbs	% Difference Load
PU#1	4.4	511.63	115,020	(+)86.6
PU#2 (Minimum)	3.6	388.68	87,380	(+)41.8
PU#3	4.6	450.78	101,340	(+)64.4
PU#4 (Maximum)	4.3	516.74	116,170	(+)88.5
PU#5	4.4	431.29	96,960	(+)57.3
Average (A)	4.3	459.82	103,374	(+)67.7
Standard Deviation (S)	NA	54.50	12,252	NA
Upper Quartile (UQ)		514.19	115,595	
Lower Quartile (LQ)		409.99	92,170	
Interquartile Range (IQR)		104.20	23,425	
Non Outlier Range		253.69 to 670.47	57,033 to 150,733	
Number of Outliers		0	0	

4.1.6.3 Discussion of Preliminary Test Results

The preliminary tests results suggest spray-applied and cured-in-place linings could significantly, if not substantially, add to the ultimate flexural and tensile strengths of concrete substrates, which 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 vs. liner thickness plot indicated in Figure 4-16 suggests the liner thickness does not appear to 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-F 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-F 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. PU-F was applied at a minimum thickness of 7.4 mm, and the thickness within the range of application of this material for the preliminary tests does not seem to 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 toward support with substantially higher deviations from the center. This can be attibuted to the increased effect 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-F#7). The failure pattern observed on PU-F#7 suggests the adhesive strenght between the polyurethane lining 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-C specimen at failure; whereas, the other four specimens failed with minor circumferential and vertical cracks in a hairline pattern.

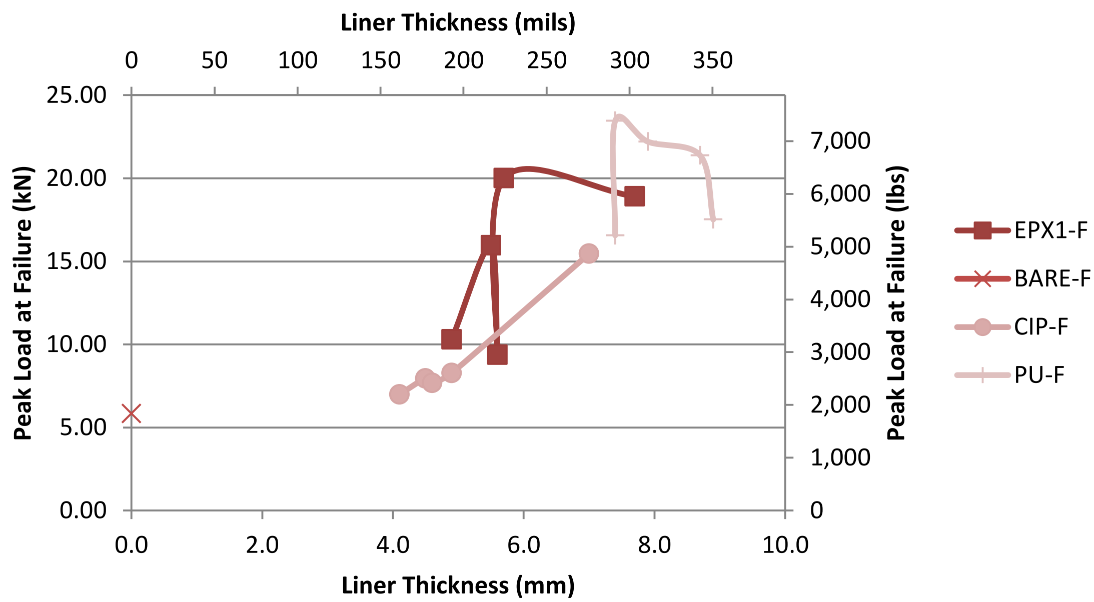


Figure 4-16. Peak Load at Failure (Ultimate Flexural Strength) vs. Liner Thickness Based on the Preliminary Tests.

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-C, EPX1-C, CMT-C, and PU-C were applied (within 1 mm or 40 mils).

On the other hand, one could 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 (typically lined internally except a few low-dig chimney repair methods discussed in Chapter 2.0). 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 used to design the main tests on concrete cylinders. The main test procedure and results are discussed in the Section 4.2.

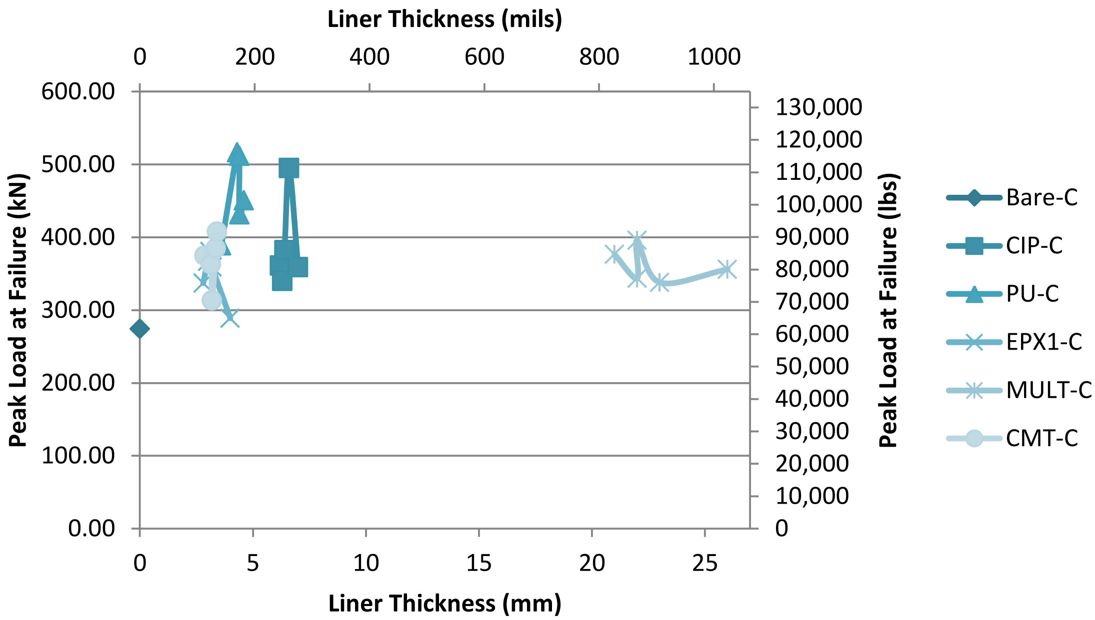


Figure 4-17. Peak Load at Failure (Ultimate Compressive Strength) vs. Liner Thickness Based on the Preliminary Tests.

4.2 Main Tests

The preliminary tests helped the project team 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 tests:

- How would an internally lined concrete cylinder, that is more representative of an actual manhole, behave under compressive and tensile stresses?
- Is there a practical way to analyze the lined system under compressive strains?

To answer these questions, the project team decided to use 24-in. (610 mm) 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 and roadway drainage applications. The main test samples were loaded by applying the standard concrete crushing test per ASTM C497 (Figure 4-18). 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 4-19). This allowed the project team to compare the strains of lined specimens to that of unlined (bare) specimens during loading.

Figure 4-18. Pipe Crushing Test Setup per ASTM C497.

4.2.1 Test 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 by the project team – Upon completion of surface preparation, the project team installed strain gages to measure strains during loading at the locations shown in Figure 4-19.
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. Testing by the project team – Crushing test per ASTM C497 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 by the project team – Load (lbs.) and strain (%) were measured during the test process and the data were transferred to a processing unit incorporated into the test setup by the project team. The data processing unit included a PC, wiring, and pertaining software. The results were reported in a standard form prepared specifically for this project (see Appendix D).

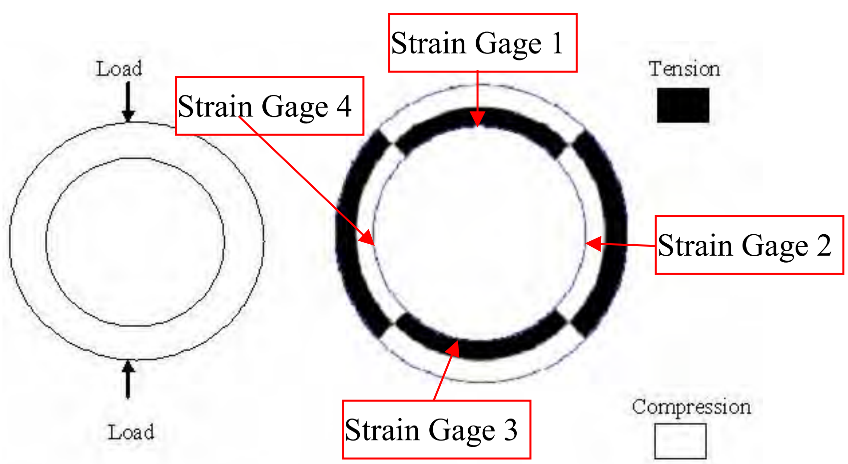


Figure 4-19. Stress/Strain Distribution Along the Pipe Section Loaded per ASTM C497 (D-Load) Test.

Stress/strain distribution throughout the cross-section of a 24- in.(610 mm) concrete pipe was also simulated using the FEM prior to commencing the main tests. The FEM simulation verified the known types of stresses shown in Figure 4-19 in addition to mapping out a detailed stress/strain distribution on the pipe section. The FEM model on the unlined and lined test pipe segments is discussed in detail in Chapter 5.0.

Table 4-12. Summary of Main Test Results.

Sample Code	Lining Material	Liner Thickness (mils)	Average Peak Load (lb.)	Maximum Deflection (in.)	% Difference Load	Failure Mode
UNL	N/A	N/A	13,826	0.0621	N/A	N/A
EPX1	Epoxy	150	26,846	0.0453	+94	Crack in lining
EPX2	Epoxy	250	28,746	0.0193	+108	Crack in lining
EPX3	Epoxy	250	20,013	0.2618	+45	Crack in lining
EPX4	Epoxy	250	32,188	0.1614	+133	Crack in lining
CMT1	Cementitious	500	17,261	0.1693	+25	Crack in lining
CMT2	Cementitious	1,000	24,485	0.0098	+77	Crack in lining
CMT-EPX	Cementitious base and epoxy top layer	1,250	31,459	0.0076	+127	Crack in lining
MULT 1	Polyurethane foam and proprietary thermoset	1,000	16,332	0.0551	+18	No crack in lining
MULT 2	Mesh reinforced epoxy	250	20,663	0.1181	+49	No crack in lining
MULT 3	Glass fiber reinforced epoxy	250	21,134	0.1231	+53	No crack in lining
MULT 4	1- in. cement & two layers of glass fiber	1,250	20,912	0.1460	+53	No crack in lining
MULT 5	.5-in. cement & one layer of glass fiber	625	19,707	0.162	+43	No Crack in Lining
PU1	Polyurethane	125	20,961	0.2283	+53	Crack in lining
PU2	Polyurethane	250	23,824	0.1102	+72	No crack in lining
PU3	Polyurethane	500	18,578	0.1181	+34	No crack in lining
TPL	Cement base and thermoplastic (polypropylene) top layer	3,000	44,087	0.0164	+219	No crack in lining
MULT-ST	Steel reinforced cement base and polypropylene top layer	3,000	66,910	0.0093	+384	No crack in lining
CF	Carbon fiber	80	24,370	0.0094	+76	Liner chipped off
CIP	Cured-in-place composite (analogous to CIPP used for pipes)	250	22,050	0.9382	+59	No crack in lining

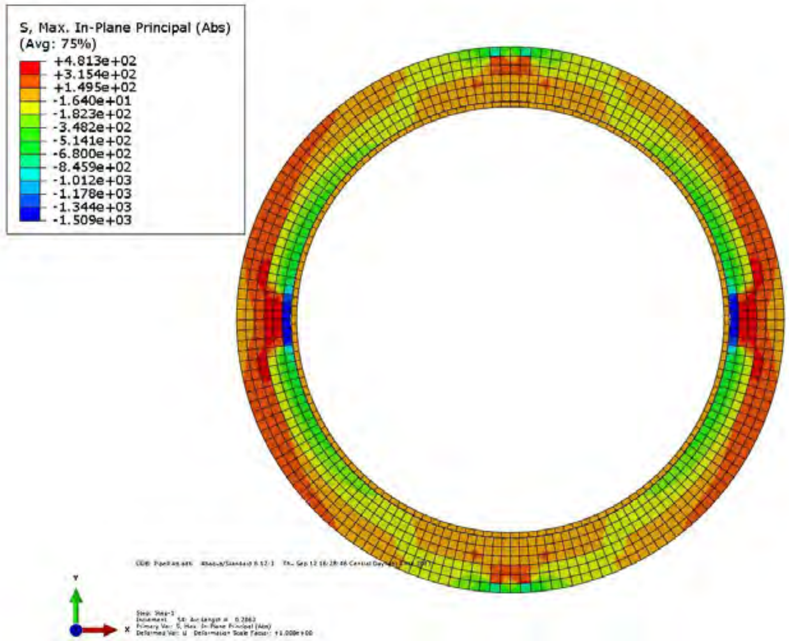


Figure 4-20. Stress Contours from the FEM Based on the D-Load (ASTM C497) Test on High-Build Epoxy Lined 24-in. (610 mm) Reinforced Concrete Cylinder.

4.2.2 Main Tests Results and Discussion

Table 4-12 shows a summary of the main test results. All of the lining materials significantly (or substantially for some) improved the crushing strength of the 24-in. concrete pipe. The main test results suggest that even spray applied epoxies and cast-in-place cement linings can substantially enhance the structural properties of a reinforced concrete cylinder if they are applied firmly at a certain thickness. On the other hand, some of the liners (e.g., MULT, PU) survived the crushing test without an apparent failure when the concrete substrate failed by tensile stress cracking at 3:00 and 9:00 o'clock positions. This is essentially due to lower adhesion and/or high flexibility, and does not necessarily translate to higher peak loads at failure. Figures 4-21 and 4-22 indicate examples of liner failures due to cracking and concrete substrate failure at the peak load while the lining remained intact.

Figure 4-21. Example of Simultaneous Cracking in the Concrete Substrate and Liner.

Figure 4-22. Example of Failure by Cracking in the Concrete Substrate Only.

Another way of examining the added strength by the lining materials to the sample pipes is looking at the modulus of elasticity of the lined system, which is the slope of stress vs. strain plot for the elastic region. This was approximated by calculating the overall stiffness and modulus of elasticity of the lined samples on which strains were measured during loading and material properties of each component of the lining system were known. Then these moduli of elasticity values were multiplied by the strains to achieve stresses in tension and compression (depending on where the strain gage is positioned). Stress vs. strain graphs for the tension and compression zones are shown in Figures 4-23 and 4-24.The results shown in these graphs suggest a significant increase in the lined system modulus of elasticity for both in tension and compression. This significant increase in the modulus of elasticity applies to all of the lined specimens except one epoxy lined specimen (EPX-4).

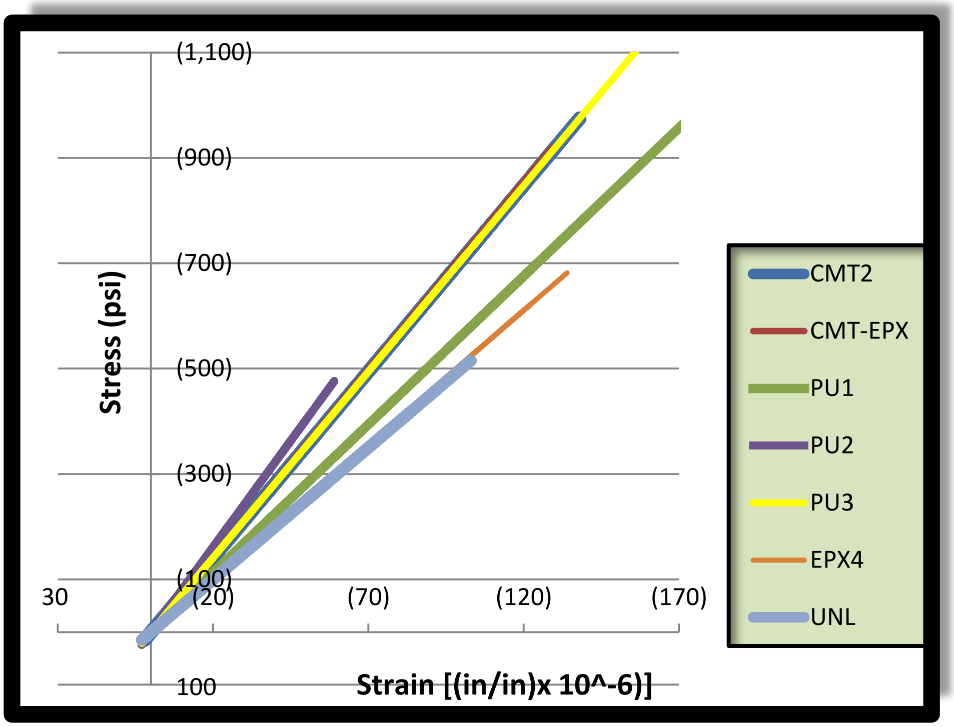


Figure 4-23. Stress vs. Compressive Strain Graph for the Unlined and Lined Samples.

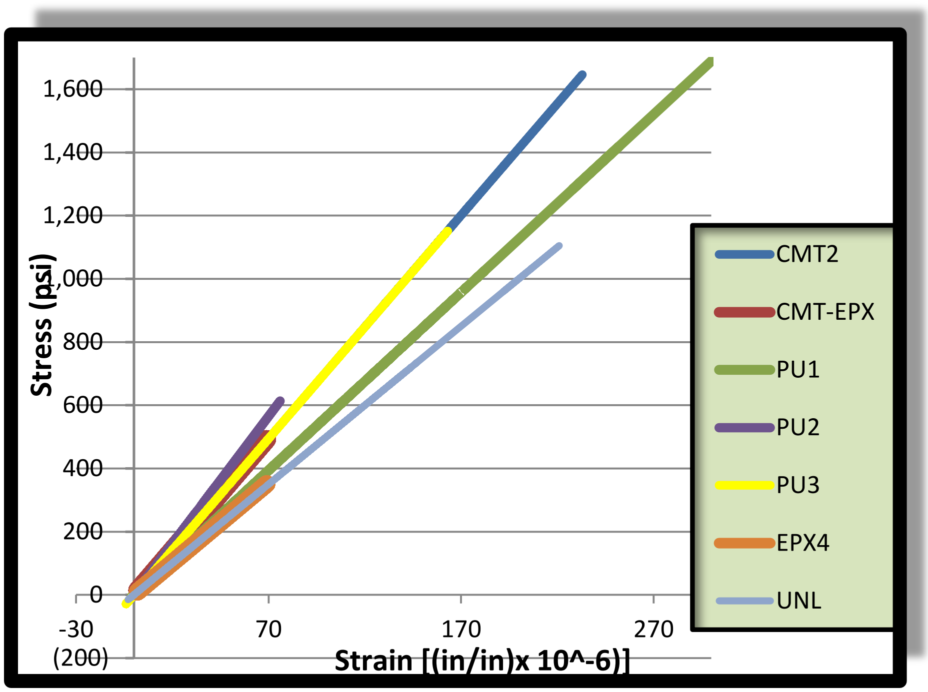


Figure 4-24. Stress vs. Tensile Strain Graph for the Unlined and Lined Samples.

CHAPTER 5.0

COMPUTATIONAL ANALYSIS WITH THE FINITE ELEMENT METHOD (FEM)

The general purpose module of Abaqus[14] finite element (FE) program was selected to perform analyses of manholes. Abaqus is a widely used FE program suitable for linear and nonlinear analysis of structural and mechanical systems subjected to static and/or dynamic loading conditions. The program was chosen for the high reliability of its results, its versatility, and rich library of FEs, material constitutive models, and solution strategies.

5.1 Simulation of Concrete Beam Test for Flexural Strength

As discussed in Chapter 4.0, concrete beam tests were performed according to ASTM C293 to determine flexural strength of the specimens. These tests were simulated using the FEM with Abaqus. The purpose of this simulation was to establish a cross-check between the actual experiments and computational modeling and to create the input for materials regarding the full-scale simulations with the FEM. These material properties are mechanical properties of the concrete substrate, liner, and contact mechanism (adhesion) between the substrate and liner.

5.1.1 Flexural Strength Test Simulation with the FEM

A concrete beam with a length of 11 in. (280 mm) and a cross-section of 3 in. × 3 in. (75 mm) was modeled in Abaqus. Material properties of the concrete beam are presented in Table 5-1.

Table 5-1. Material Properties Used for Beam Test Simulation.

Modulus of Elasticity		**4,000,000 psi**
Poisson Ratio		0.2
Density		0.081 lbs./in^3
Plasticity	Dilation Angle	20
	Eccentricity	0.1
	f_{b0}/f_{c0}	1.16
	K	0.667
	Viscosity Parameter	10^{-7}

To model the concrete beam, damaged plasticity model was used for concrete material. This model was used to capture the effects of irreversible damage associated with the failure mechanisms that occur in concrete and other quasi-brittle materials under fairly low confining pressures. As shown in Table 5-1, several parameters are needed to define this model. These model parameters can be obtained through calibrating the simulation results with lab test measurements.

After defining the material properties, the load was applied to the beam at the mid span and restraints were defined at 1 in. (25 mm) from two ends of the beam. The supports restricted

[14] *http://www.3ds.com/products/simulia/overview/*

the beam to move in vertical direction. A ramped load of 3,000 lbs. (13.3 kN) was applied to the beam. To avoid the effects of load concentration on the beam, a strip area with a width of 0.25 in. (6 mm) and length equal to the width of the beam was created. A pressure load was applied on the strip area. Variation of the load vs. time is shown in Figure 5-1.

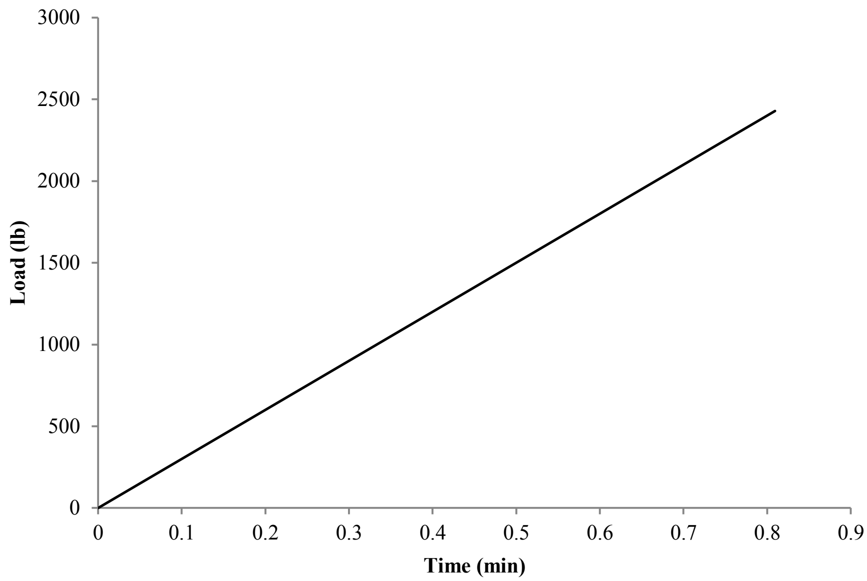


Figure 5-1. Loading Rate (Loading Time).

5.1.2 Results

The load vs. the deflection data from the experiments were applied to the Abaqus model in pursuit of the best match between simulation and laboratory tests. When the best match was obtained (minimum 85%), the parameters used for the concrete damage model were the calibrated parameters and considered as the true parameters for manhole simulation. Figure 5-2 shows the contours of the plastic strain of the concrete beam under 3,000 lbs. (13.3 kN) of load. It clearly demonstrates that the center bottom develops the largest plastic strain. This is not surprising as this is where failure occurred on the beam specimens (discussed in Chapter 4.0).

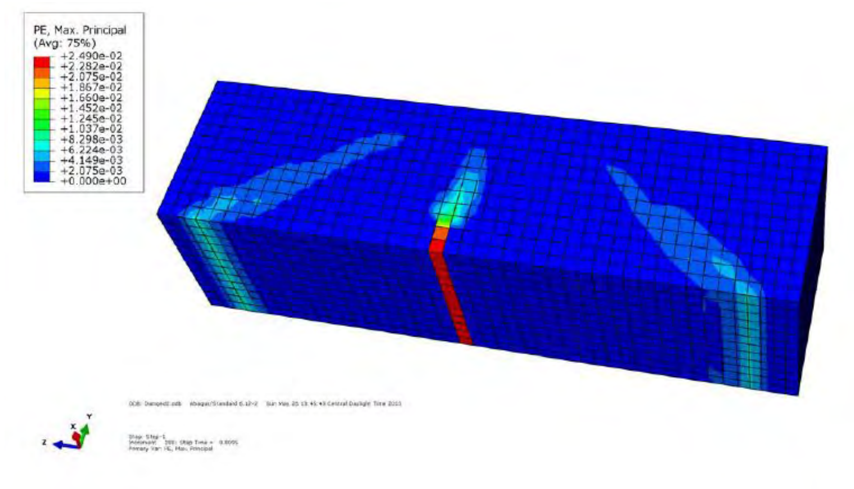


Figure 5-2. Plastic Strain in the Concrete Beam.

Figure 5-3 shows stress distribution in the direction of the beam length. The beam top is under compression and the beam bottom is under tension. Figure 5-4 shows the magnitude of the beam deformation. The center of the beam experiences the largest deformation. Figure 5-5 presents load-deflection curve for the concrete beam obtained from the FEM simulation that was calibrated with the laboratory tests.

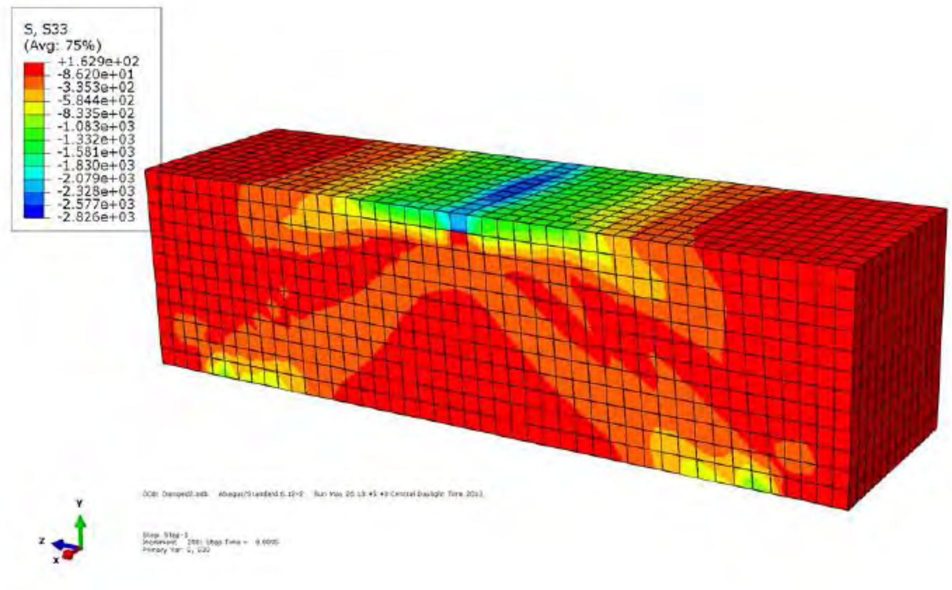


Figure 5-3. Stress Distribution in Concrete Beam.

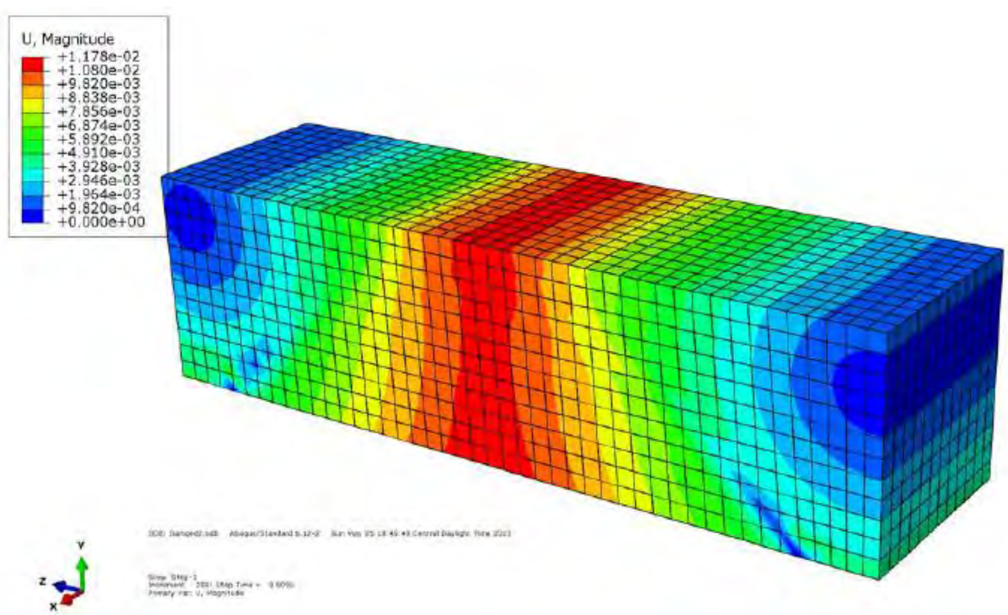


Figure 5-4. Deformation of the Concrete Beam.

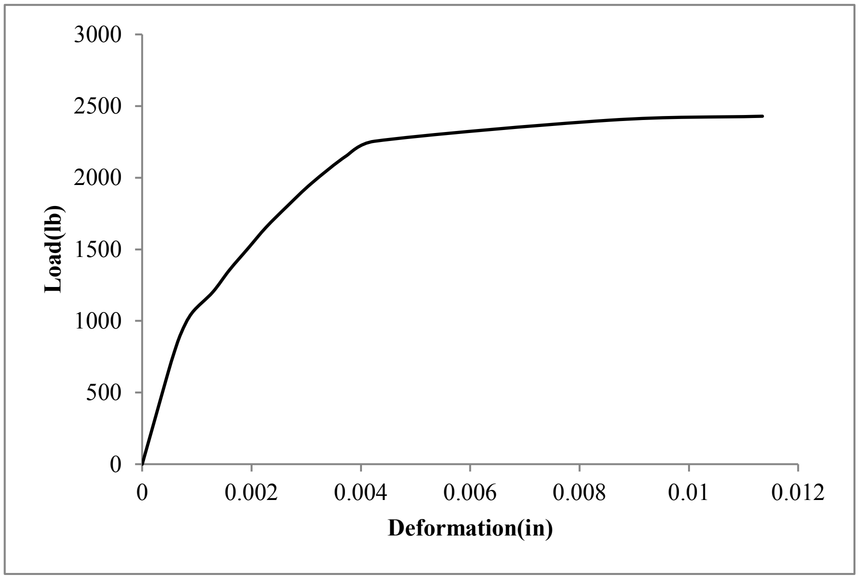


Figure 5-5. Load-Deflection Curve of the Concrete Beam.

5.1.2.1 Materials

An elasto-plastic model with a Mohr-Coulomb yield surface was selected for the soil materials. The Mohr-Coulomb criterion assumes that yield occurs when the shear stress on any point in a material reaches a value that depends linearly on the normal stress in the same plane. The Mohr-Coulomb plasticity model is widely used for design applications in the geotechnical and pipeline engineering. The smeared crack concrete model in Abaqus/Standard is intended for applications in which the concrete is subjected to essentially monotonic straining and when material exhibits either tensile cracking or compressive crushing. Considering the major static loading conditions of manhole, the smeared crack concrete model was selected.

5.1.2.2 Contact Model

Linings are used frequently to repair deteriorated manholes. The added linings and existing old manhole structure, when bonded together, form a composite structure. The interaction between the newly constructed lining and the existing manhole has significant impact on the repair performance. The adhesive properties at the interface were modeled by hard surface contact model in Abaqus. Laboratory testing of composite samples consisted of lining. Existing manhole materials can be used to calibrate the contact model and improve the simulation of the overall behavior of the repairing.

5.2 Full-Scale Manhole-Soil Model

The purpose of a full-scale manhole FEM with soil interaction is to apply the findings of the lab tests and small scale models to a full site condition, thereby analyzing the effectiveness of using liners on the inside of manholes under selected soil pressure, hydrostatic water pressure, and traffic loads.

To simulate the manhole structure in soil, an axisymmetric model of a manhole was generated in Abaqus. An axisymmetric model can model the field loading condition with high computation efficiency as compared to a 3D model. The generated part and mesh of the model is shown in Figure 5-6. The three horizontal and one vertical lines are auxiliary lines used for model partition and mesh generation. The total height of the manhole was considered to be 15 ft. (4.6 m) with a conical height of 3 ft. (0.9 m). The inside diameter of the manhole was 4 ft. (1.2 m) at the bottom, which was reduced to 2 ft. (0.6 m) at the top (conical). The wall thickness was 4 in. (102 mm) and the thickness of the base (bench) was assumed 6 in. (150 mm). This model was created to generate the desired lateral soil pressure in the soil. After obtaining the desired horizontal soil pressure, the soil element inside the manhole was removed and the manhole structure element was activated. The distance from right boundary to the symmetric axis was 23 ft (7.0 m), which was chosen to eliminate the boundary effect. The properties of the soil and the concrete material used to do the simulations are shown in Table 5-2. The concrete material properties used for the FEM simulation are presented in Table 5-3.

Table 5-2. Material Properties Used for the FEM.

Material	**Modulus of Elasticity E (psf/kPa)**	**Density ρ(pcf/g/cm³)**	**Poisson Ratio ν**	**Cohesion C (psf)**	**Friction Angle φ(°)**	**Yield Stress (psf/kPa)**	**Plastic Strain**
Concrete	$5.76 \times 10^8 / 2.8 \times 10^7$	140/2.2	0.2	–	–	*	*
Soil	$5 \times 10^5 / 2.4 \times 10^4$	120/1.92	0.3	10	30	–	–
Epoxy	$1.12 \times 10^8 / 5.37 \times 10^6$	70.8/1.13	0.45	–	–	$2.6 \times 10^6 / 1.29 \times 10^5$	0.025

**Damaged plasticity behavior of the concrete material is presented in Table 5-3.*

Table 5-3. Concrete Material Properties Used for the FEM Simulation.

Modulus of Elasticity (psi/MPa)			4.35×10^6/29,992	
Poisson Ratio			0.2	
Density (pcf / g/cm³)			140/2.2	
Plasticity		Dilation Angle	38	
		Eccentricity	0.1	
		f_{b0}/f_{c0}	1.16	
		K	0.667	
		Viscosity Parameter	10^{-7}	
Compressive Behavior	Yield stress (psf/kPa)	563,904/27,000	835,200/39,990	
	Inelastic Strain	0	0.01	
Tensile Behavior	Yield stress (psf/kPa)	104,427/5000	45,936/2200	1,044/50
	Cracking strain	0	0.006	0.015

For the manhole structure and the lining, a four-node bilinear axisymmetric quadrilateral, reduced integration, hourglass control (CAX4R) was used and the soil was modeled by using a four-node axisymmetric quadrilateral, bilinear displacement, bilinear pore pressure (CAX4P).

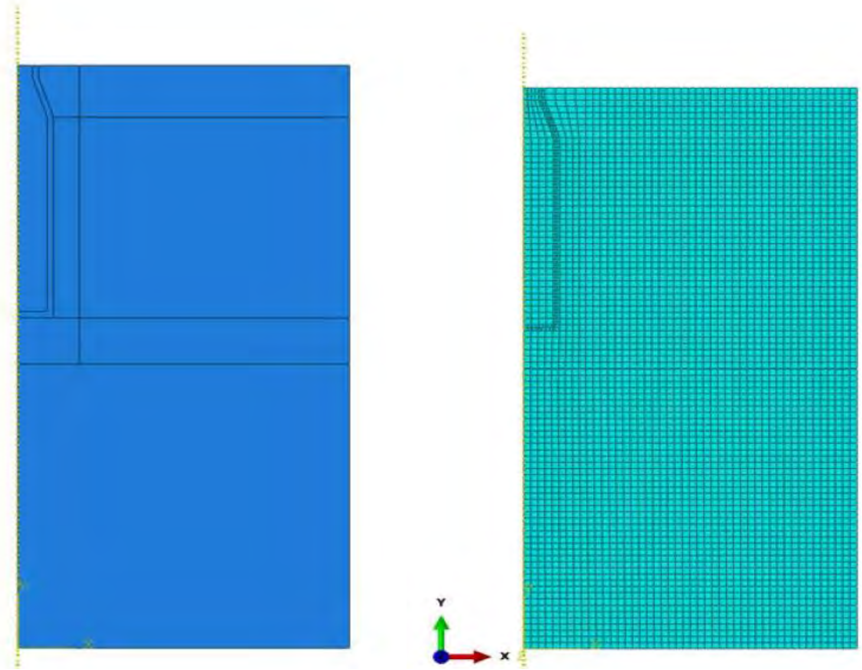


Figure 5-6. Geometry of the Manhole Model.

5.2.1 Materials

An elasto-plastic model with a Mohr-Coulomb yield surface was selected for the soil material. The Mohr-Coulomb criterion assumes that yield occurs when the shear stress on any point in a material reaches a value that depends linearly on the normal stress in the same plane. The Mohr-Coulomb plasticity model is widely used for design applications in geotechnical engineering. An elastic material model with concrete damaged plasticity was selected for concrete materials. For the lining material an epoxy layer was considered with an elasto-plastic property.

5.2.2 Boundary Conditions

An axisymmetric boundary condition was applied to the left side of the soil mass and on the right it was restrained from moving in the horizontal direction. The soil mass was constrained vertically and horizontally at the bottom.

5.2.3 Simulation Steps

For soil modeling in Abaqus the first step is obtaining geostatic stress in the soil. In this simulation the geostatic stress was obtained by considering the whole simulation domain as soil mass only in the initial step. The vertical soil stress was applied as predefined field data, which was calculated by multiplying unit weight of the soil to the height of the soil mass. In the first step the gravity load was applied and geostatic stress was obtained through geostatic analyses. As shown in Figure 5-7, at the end of the first step, the stress distribution was achieved according to the unit weight of the soil without observing any considerable deformation in the soil.

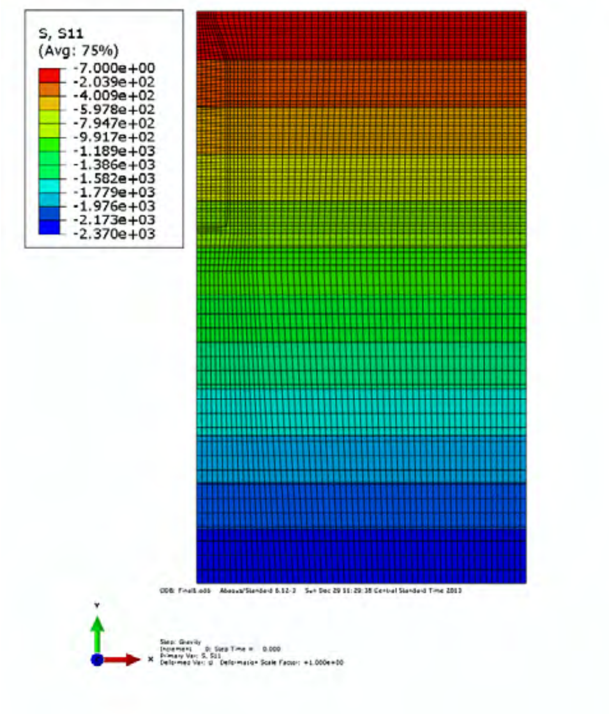


Figure 5-7. Geostatic Stress in Soil Mass.

In step 2, the soil mass inside of manhole was removed and a manhole with lining created. At this stage the interaction between the soil and the concrete was activated. A hard contact was assumed as normal behavior of the general contact between the soil and the manhole structure and the tangential contact behavior was considered as "penalty" with a friction coefficient of 0.35. The interaction between the concrete and lining was modeled by the means of cohesive behavior; a damage property was also assigned to the interaction.

Surface-based cohesive behavior provides a simplified way to model cohesive connections with negligibly small interface thicknesses using the traction-separation constitutive model. The formulae and laws that govern surface-based cohesive behavior are: linear elastic traction-separation, damage initiation criteria, and damage evolution laws.

Linear elastic traction-separation behavior relates normal and shear stresses to the normal and shear separations across the interface before the initiation of damage. By default, elastic properties are based on underlying element stiffness, which can optionally specify the properties. The traction-separation behavior can be uncoupled (default) or coupled. In this simulation the stiffness of the underlying material was considered 10,000 lbs./in. (1.13 kN/m) in all directions and a set of bonded nodes were defined on the interior edge of the epoxy layer to which the cohesive behavior was applied.

To restrict the cohesive constraint to act throughout the contact surface in the normal direction only, the uncoupled cohesive behavior was defined and zero values for the shear stiffness components, K_{tt} and K_{ss}, were specified. Alternatively, if only tangential cohesive constraints are to be enforced, the normal stiffness term, K_{nn}, could be set to zero, in which case the normal “separations” would not be constrained, and the normal compressive forces would be resisted as per the usual contact behavior.

Damage modeling allows one to simulate the degradation and eventual failure of the bond between two cohesive surfaces. The failure mechanism consists of two components: a damage initiation criterion and a damage evolution law. The initial response is assumed to be linear as discussed above. However, once a damage initiation criterion is met, damage can occur according to a user-defined damage evolution law. Figure 5-8 shows a typical traction-separation response with a failure mechanism.

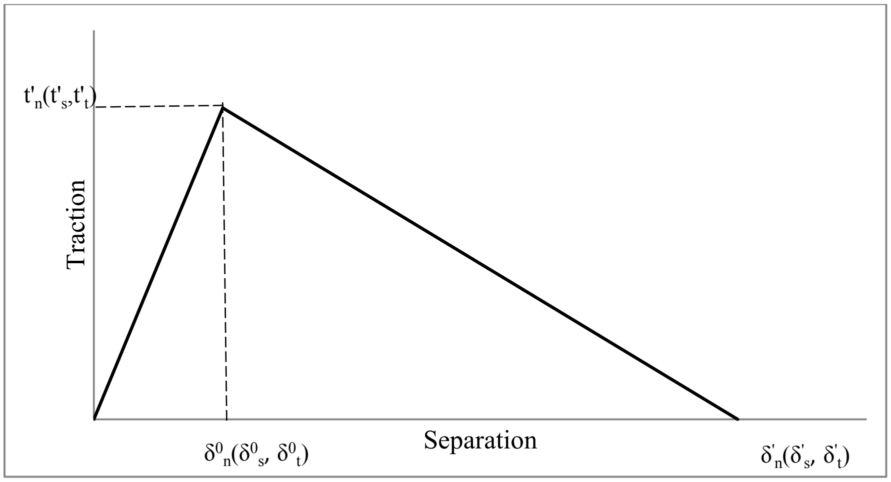


Figure 5-8. Typical Traction Separation Response.

Damage initiation refers to the beginning of degradation of the cohesive response at a contact point. The process of degradation begins when the contact stresses and/or contact separations satisfy certain damage initiation criteria. (Several damage initiation criteria are available.) In the current simulation, maximum stress criterion was used and set to 300 psi (2,070 kPa) in all directions. In this criterion, damage is assumed to initiate when the maximum contact stress ratio reaches a value of one.

The damage evolution law describes the rate at which the cohesive stiffness is degraded once the corresponding initiation criterion is reached. There are two components to the definition of damage evolution. The first component involves specifying either the effective separation at complete failure, δ^f_m, relative to the effective separation at the initiation of damage, δ^0_m; or the energy dissipated due to failure, G^c.

The second component to the definition of damage evolution is the specification of the nature of the evolution of the damage variable, *D*, between initiation of damage and final failure. This can be done by either defining linear or exponential softening laws or specifying *D* directly as a tabular function of the effective separation relative to the effective separation at damage initiation[15]. The ratio of the total displacement to the plastic displacement in the simulation of the lined manhole was considered 2 with a linear softening law.

[15] ABAQUS 6-12 User’s Manual

5.2.3.1 Lateral Soil Pressure

The lateral earth pressure acting on the manhole structure can be calculated by the means of Rankin earth pressure theory considering at rest lateral earth pressure coefficient of 0.5. Calculated lateral earth pressure and the earth pressure obtained from the simulation results are compared in Figure 5-9, as shown in the figure the results from simulation and calculation are in good agreement.

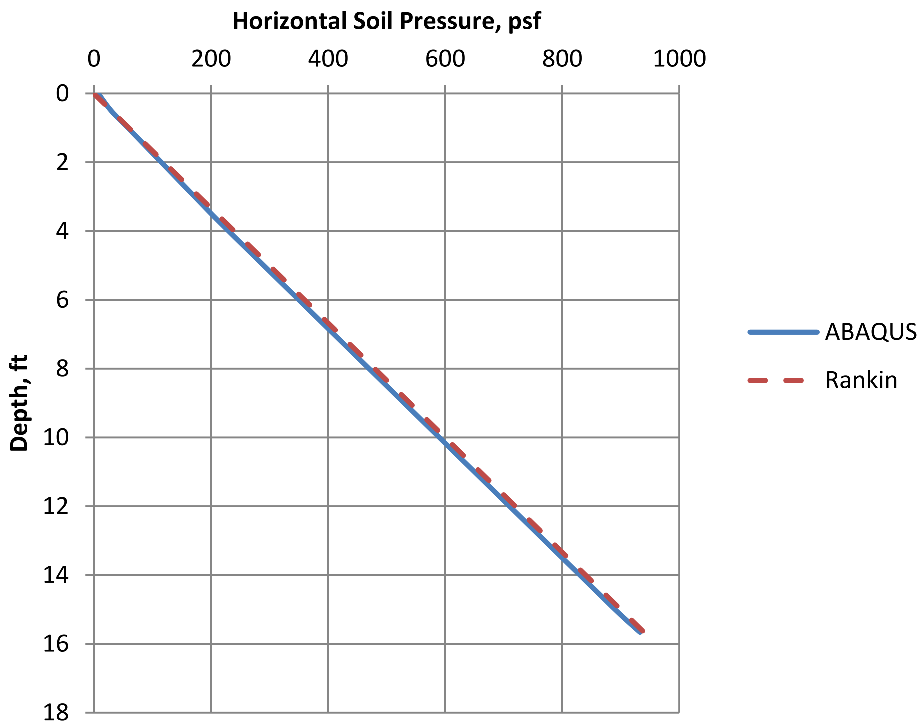


Figure 5-9. At Rest Horizontal Soil Pressure.

5.2.3.2 Pore Water Pressure

The other lateral pressure acting on manhole structure during operation was groundwater pressure. The simulation of the water pressure was conducted by assigning permeability and void ratio of the soil as 1.0 ft/day (0.3 m/d) and 0.6 respectively in material property of the soil. The void ratio of the soil was also assigned as predefined field parameter as a constant value of 0.6 to the soil mass in the initial step. Pore water pressure was assigned by using a boundary condition of pore pressure type, equal to zero at the surface of the ground in the initial step. By assigning this boundary condition the program considers a linear distribution of water pressure in depth. After achieving geostatic stress in the first step the second step was defined as “soil” procedure with transient consolidation state in order to consider presence of water in the porous material. Distribution of water pressure acting on manhole structure obtained from the FEM simulation and manual calculation are shown in Figure 5-10.

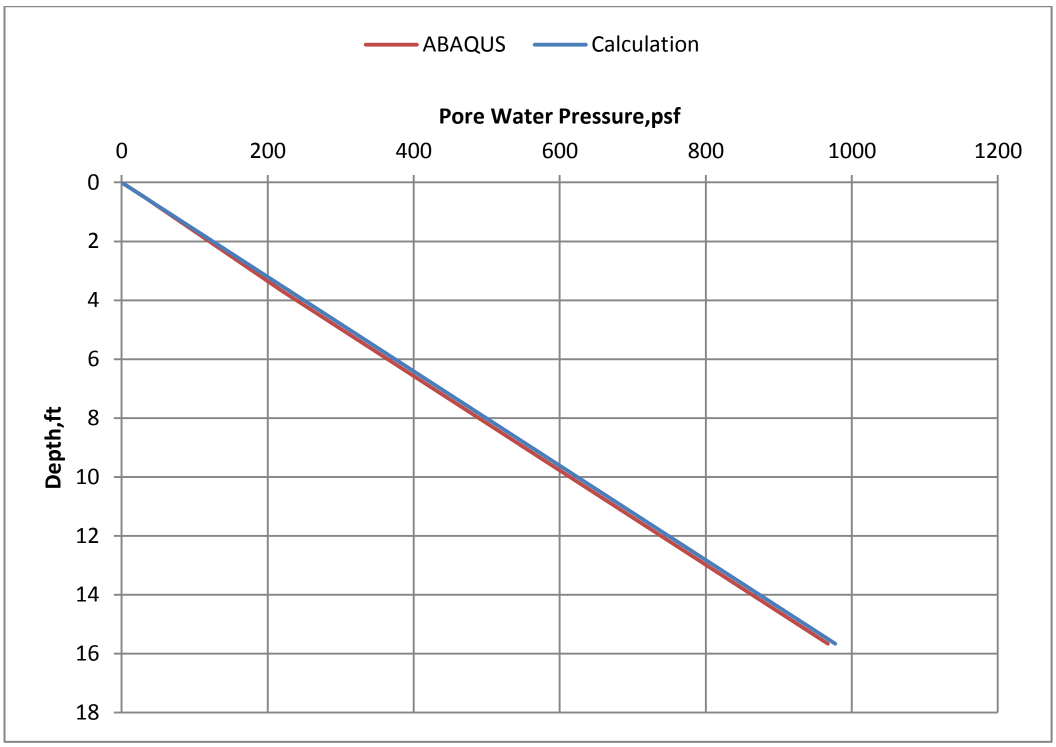


Figure 5-10. Pore Water Pressure vs. Depth.

The results of the manual hydrostatic pressure calculation and ABAQUS simulation are essentially the same, which indicates that water pressure simulation is assigned accurately.

5.2.3.3 Traffic Load

The other load considered in simulation of the full-scale manhole was traffic load. To apply the traffic load to the manhole structure ASTM C890 was used. In this standard the vehicle and pedestrian load designation are as presented in Table 5-4.

Table 5-4. Vehicle and Pedestrian Load Designations.

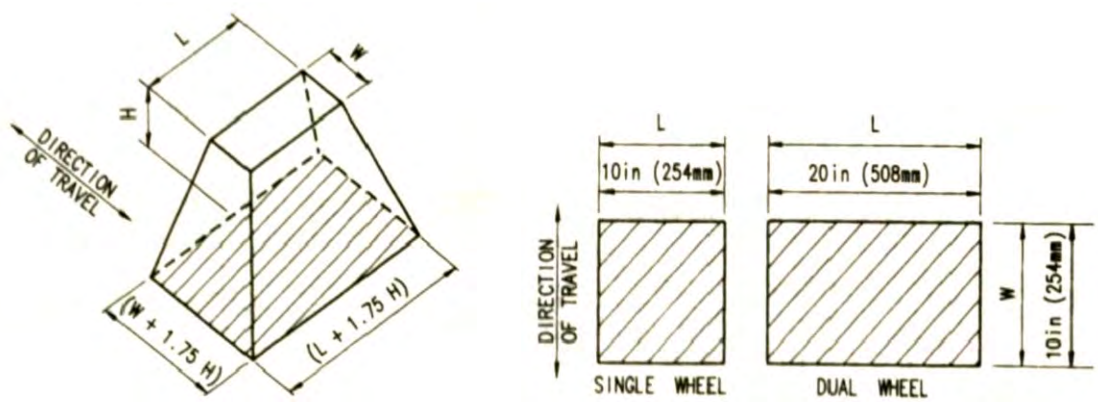
Designation	Load, Max	Uses
A-16	16,000 lbf per wheel	Heavy traffic
A-12	12,000 lbf per wheel	Medium traffic
A-8	8,000 lbf per wheel	Light traffic
A-03	300 lbf/ft^2	walkways

ASTM C890 also indicates a distribution of wheel loads through earth fills considering below ground structures, where backfill separates the vehicle wheels and the top surface of the structure, the load distribution is in the shape of a truncated pyramid as shown in Figure 5-11. As such, the loaded area can be calculated as:

$A = (W + 1.75H) \times (L + 1.75H)$ (Equation 5-1)

Where:

A = wheel load area
W = wheel width
L = wheel length
H = height of backfill between wheels and structure



a b

Figure 5-11. Traffic Load Distribution in 3D (a) and 2D (b) per ASTM C890.

As a starting point for the traffic load simulation, a dual wheel load area with a backfill height of 1.5 ft. (0.5 m) was considered. Using the above equations and assuming a medium traffic with a maximum load of 12,000 lbs. (53.4 kN) per wheel the total pressure on the soil surface due to traffic will be:

$P = \frac{12,000}{(\frac{10}{12}+1.75\times1.5)(\frac{20}{12}+1.75\times1.5)} = 808.5\, psf$ (Equation 5-2)

According to ASTM-C890, to consider the dynamic effect, traffic load should be increased by 20% which results in using a distributed load of 1,000 psf (47.8 kPa) applied in a width of 4.29 ft (1.3 m) representing dual wheel load traffic on the surface.

In this simulation the distance of the traffic load from manhole structure was 5 ft. (1.5 m) distance from center of the distributed load to the edge of the manhole). Abaqus simulations of traffic load acting at various horizontal distances to a manhole shows the resulted horizontal stress on the manhole wall is maximum when the horizontal distance is 5 ft. Figure 5-12 presents the lateral stress on the manhole wall due to traffic obtained from simulation. The induced lateral stress is limited to 8 ft. (2.4 m) depth.

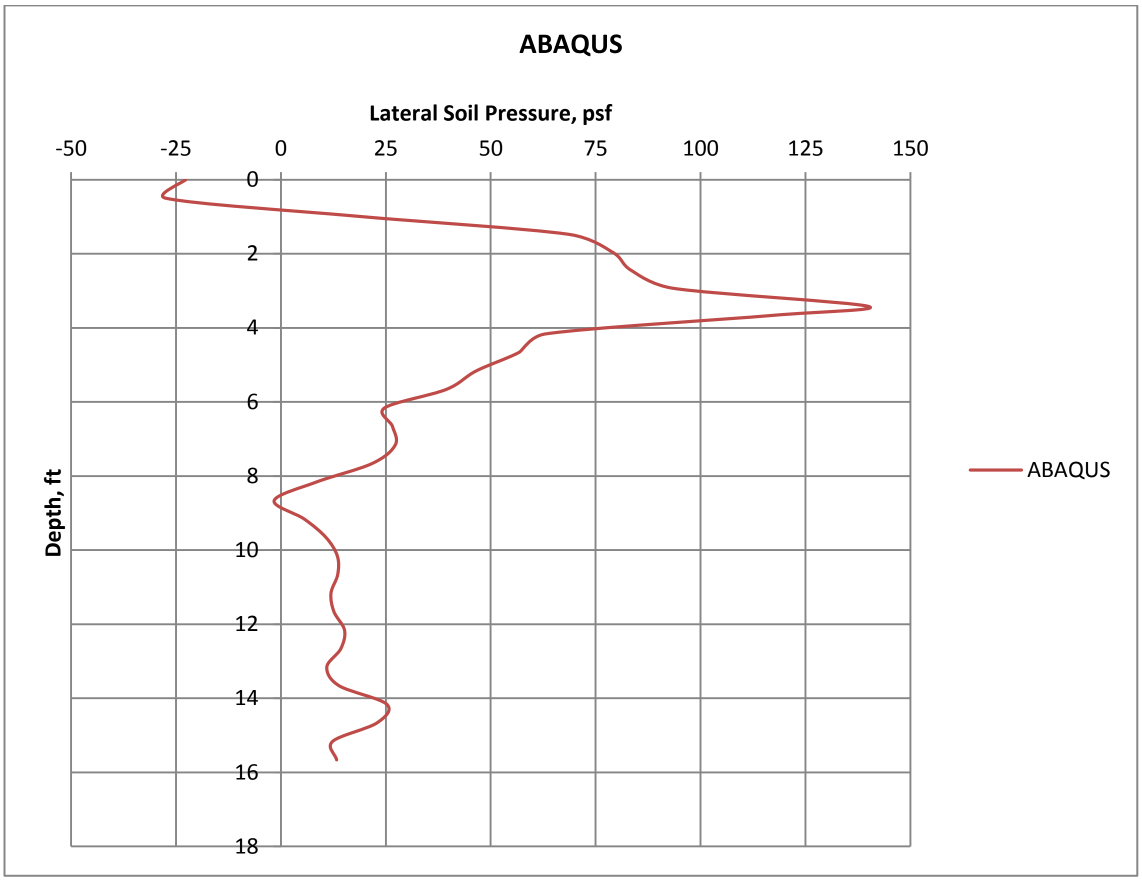
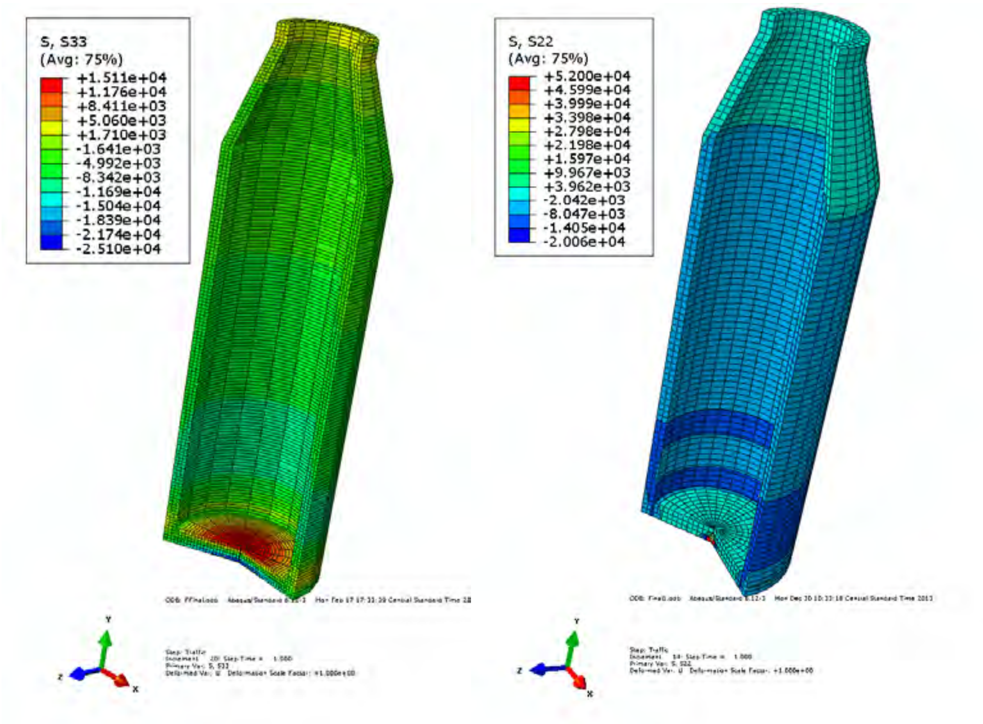


Figure 5-12. Lateral Soil Pressure Distribution Due to Traffic Load.

5.3 Full-Scale Manhole Simulation Results

Selected results from the simulation are shown in Figures 5-13, 5-14, and 5-15. Figure 5-13 shows the contour distribution of hoop and vertical stress, which increases with the depth. Maximum stresses occur at the joint of wall and base and also at the center of the base.



a b

Figure 5-13. Hoop (a) and Vertical (b) Stress Distribution in the Manhole Structure.

Figure 5-13 shows the vertical and horizontal deformations on manhole, and Figure 5-14 presents the vertical deformation of the soil mass. The manhole structure is settling due to its weight and the soil surrounding the manhole shows similar movement as the manhole. The magnitude of soil deformation decreases as the distance to the manhole increases.

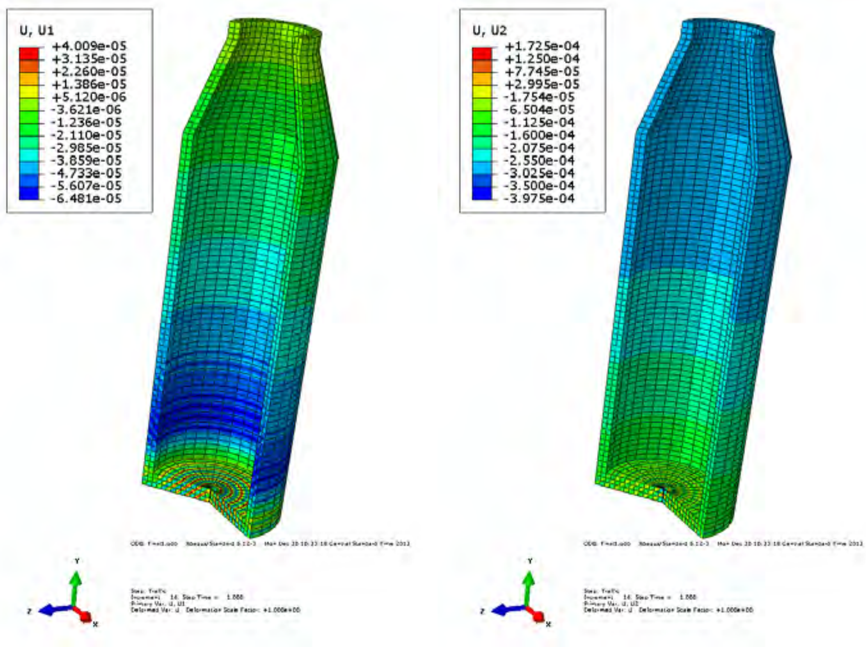


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Figure 5-14. Horizontal and Vertical Deformation of the Manhole Structure.

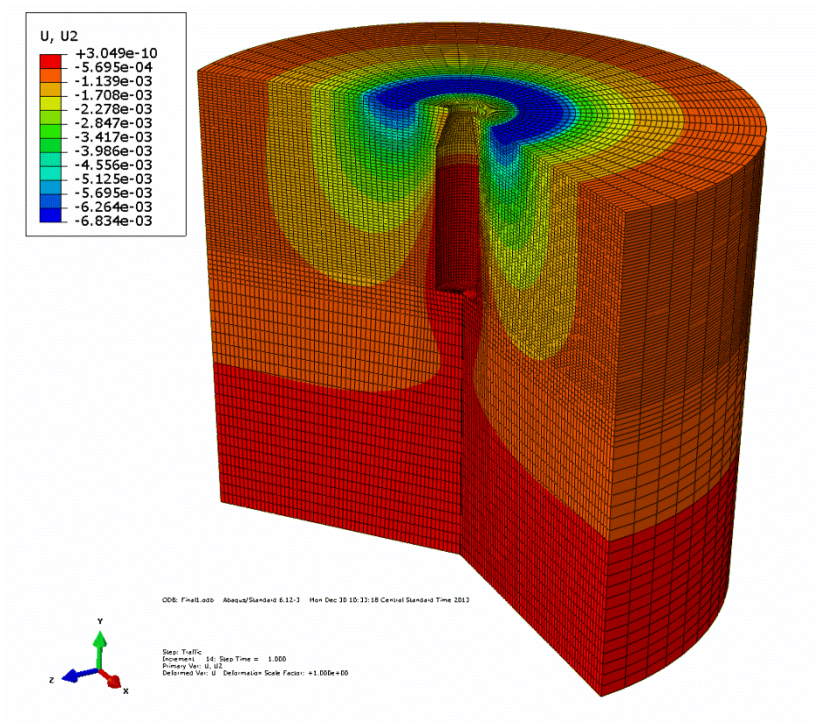

Figure 5-15. Vertical Deformation of the Soil Mass.

The lateral pressure acting on the manhole structure considering two different loading cases are shown in Figure 5-16. One case refers to the condition in which soil pressure, pore water pressure and traffic load are applied and the other one is obtained from the situation without applying the traffic load. As shown in Figure 5-16, presence of traffic load has more effect for depths up to 3.5 ft. (1.1 m), which is the approximate depth of a cone (without a chimney) on top of the manhole wall. The lateral pressure illustrated in this figure refers to the active soil pressure condition (K_a).

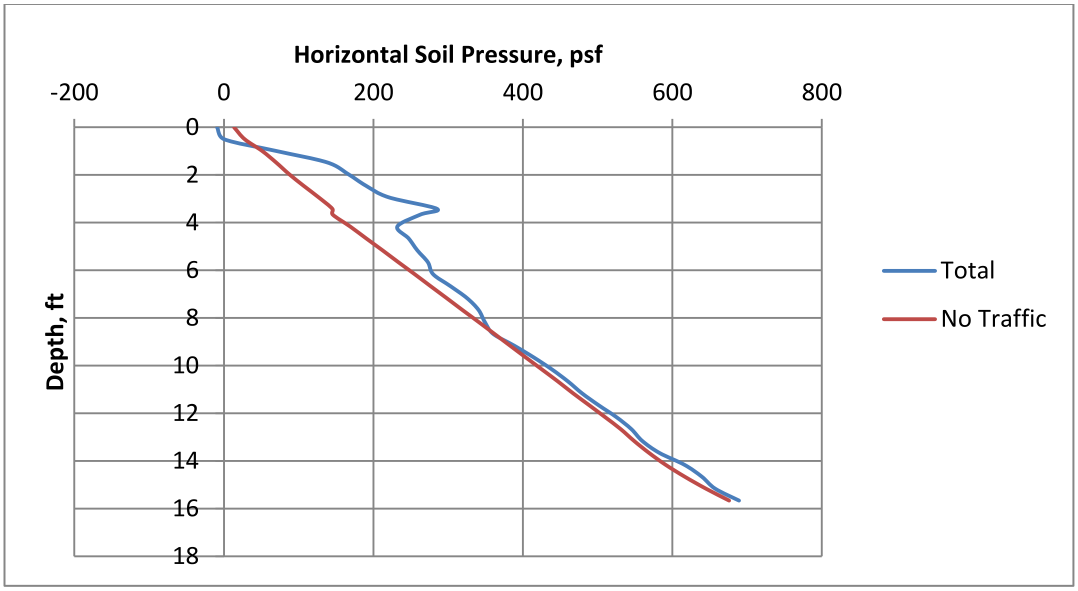

Figure 5-16. Comparison of the Lateral Load Acting on the Manhole Structure Due to Different Loading Cases.

The pressure induced in soil under the manhole structure is shown in Figure 5-17. Since the soil is considered to be a sandy soil, the pressure distribution under the manhole base is the same as the pressure distribution under foundations in sandy soils, which is greater in the middle and decreases towards the edge.

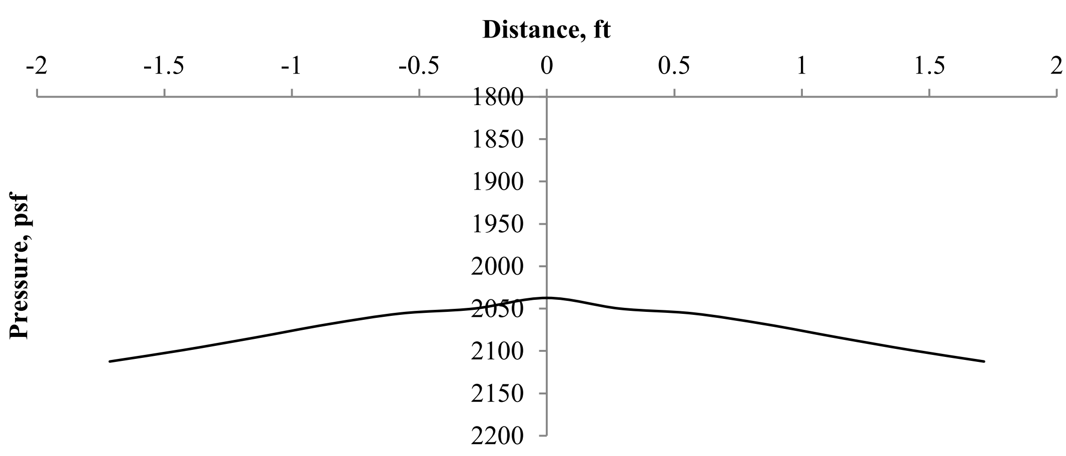

Figure 5-17. Soil Pressure Under Manhole Base.

The moment diagram for case 1 is shown in Figure 5-18. According to the diagram, the maximum moment occurs at the center of the manhole base and it is equal to 736 lb-ft (1,005 N-m). Using the classical plate theory (CPT) for circular slabs with clamped edges under a pressure of 2,000 psf (95.8 kPa) produces a moment of 662 lb-ft (898 N-m). The effective pressure under the manhole base from the simulation is about 1,070 psf (51.3 kPa) adding the water pressure equal to 967 psf (46.3 kPa) at a depth of 15.5 ft., the total pressure acting on the base of the manhole structure is about 2,040 psf (97.7 kPa) as shown in Figure 5-17. Considering other conditions such as interaction between the soil and the base it can be concluded that the moment diagram obtained from the simulation is in good agreement with classical plate theory.

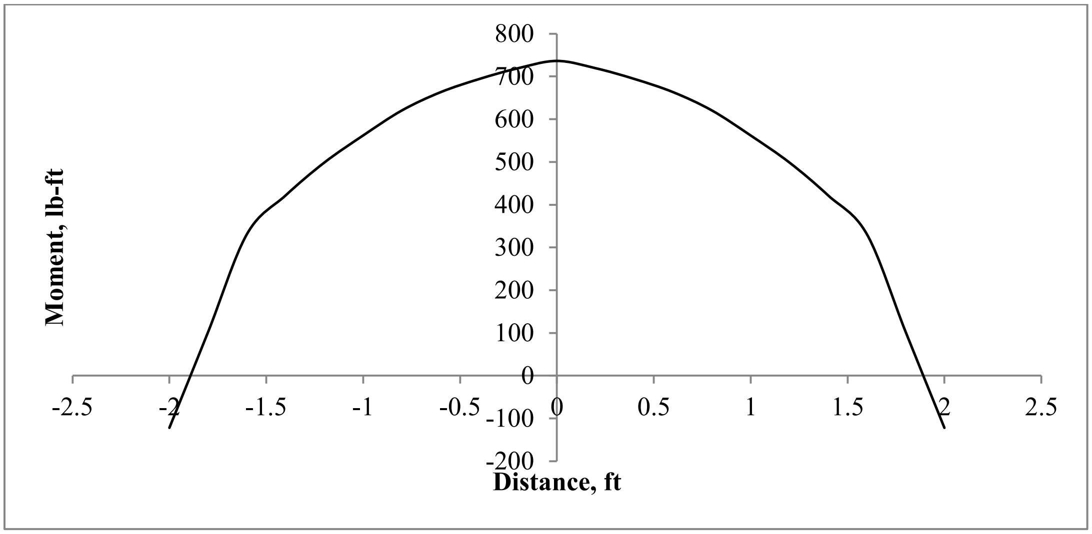

Figure 5-18. Moment Diagram in Manhole Base.

ACI318 limits strain in the concrete to 0.003 while the maximum strain induced in a manhole structure is 0.00019. The maximum principal strain distribution in a manhole structure, based on the FEM presented herein, is shown in Figure 5-19.
According to ACI318 the cracking moment of the concrete is calculated as follows:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \text{(Equation 5-3)}$$

Where for normal weight of concrete,

$$f_r = 7.5\sqrt{f_c'}\ psi \qquad \text{(Equation 5-4)}$$

and I_g is the gross moment of inertia of the circular slab. Substituting in the M_{cr} formulation, the cracking moment for base of the manhole is

$$M_{cr} = \frac{bh^2 f_{cr}}{6} = \frac{4\times0.5^2\times7.5\times\sqrt{4{,}000}\times144}{6} = 11{,}384.2\ lb - ft \qquad \text{(Equation 5-5)}$$

This is much greater than the maximum moment induced in the base of the manhole structure. Figure 5-20 shows the deformation of the manhole base due to the applied loads. As expected the maximum deformation occurs in the center of the base and is equal to 0.00543 in. (0.14 mm).

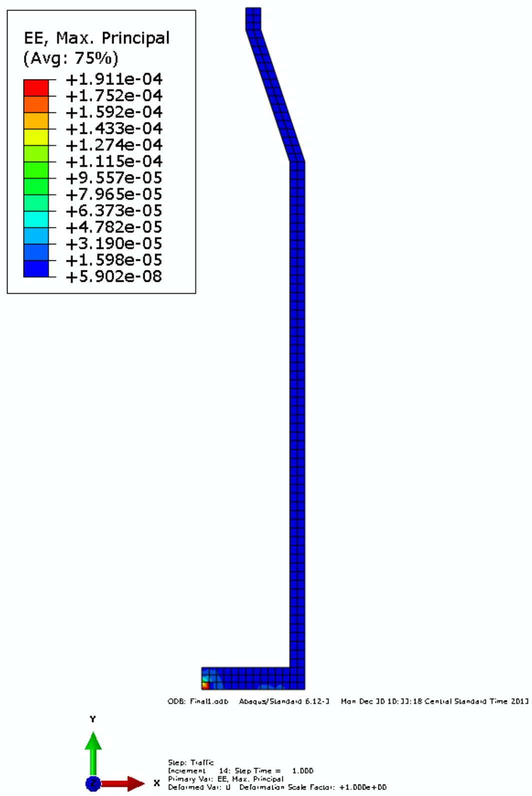

Figure 5-19. Maximum Principal Strain Distribution in Manhole Structure.

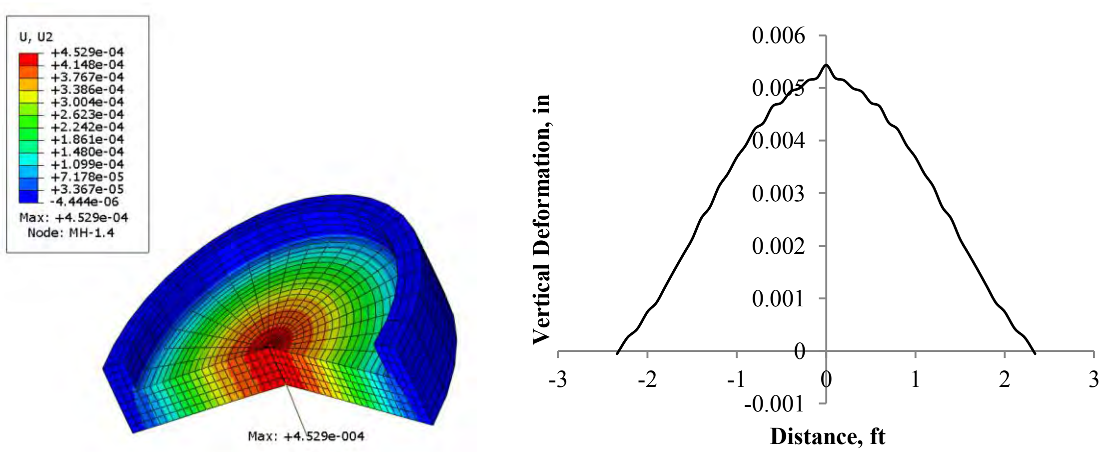

Figure 5-20. Vertical Deformation at the Base.

As a result it can be concluded that the specified loads can be carried by the manhole structure when it is not deteriorated. The rest of the chapter focuses on deteriorated manhole structure in order to study the behavior of liners used for rehabilitation.

5.4 Manhole Simulations – Deteriorated Manholes

The previous manhole simulations are for newly constructed manholes indicated as Case 1(C1) shown in Table 5-5. For a deteriorated manhole structure, different conditions have been studied. The summary and the case numbers are presented in Table 5-5.

Table 5-5. Different Simulation Scenarios on a Rehabilitated Manhole.

Case No.	Concrete Thickness (in/cm)		Lining (Epoxy)		Loads
	Wall	Base	Compressive Strength(psf/kPa)	Modulus of Elasticity (psf/kPa)	
C1	4/10.16	6/15.24	$2.6x10^{6}/1.29x10^{5}$	$1.12x10^{8}/5.37x10^{6}$	Soil/Water/Traffic
C2-1	2/5.08	3/7.62	$2.6x10^{6}/1.29x10^{5}$	$1.12x10^{8}/5.37x10^{6}$	Soil/Water/Traffic
C2-2	2/5.08	3/7.62	$1.3x10^{6}/6.45x10^{4}$	$5.6x10^{7}/2.68x10^{6}$	Soil/Water/Traffic
C3-1	0	0	$1.3x10^{6}/6.45x10^{4}$	$5.6x10^{7}/2.68x10^{6}$	Soil/Water
C3-2	0	0	$1.3x10^{6}/6.45x10^{4}$	$5.6x10^{7}/2.68x10^{6}$	Soil/Water/Traffic

Case 1 refers to a sound manhole, and this condition was discussed previously. In Case 2 it is assumed that the manhole structure is partially deteriorated, and its thickness was assumed half of the initial thickness due to corrosion. Two conditions were considered for this scenario: i) Lining (epoxy) at its full stiffness, and ii) Lining with its stiffness (modulus of elasticity) reduced by half regarding long-term deformation due to creep. Case 3 represents the condition, in which the manhole structure is fully deteriorated, and only the loads are supported by the lining with its stiffness reduced by 50%. Two different loading conditions are considered for this scenario (see Table 5-5). Select results from the foregoing simulation scenarios are presented below.

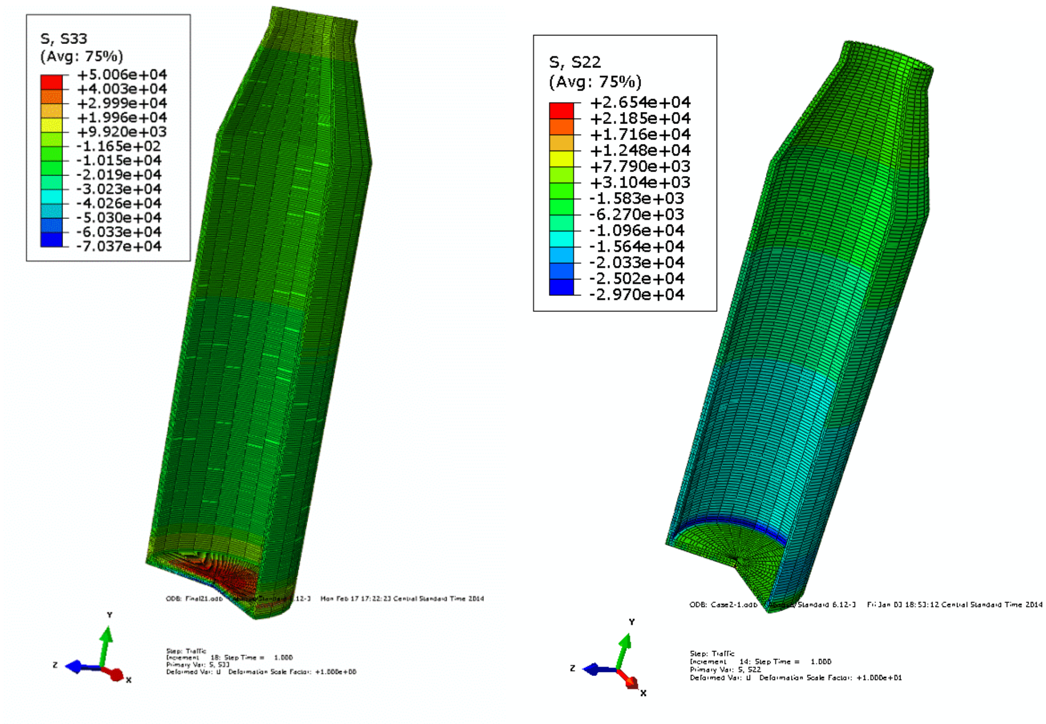

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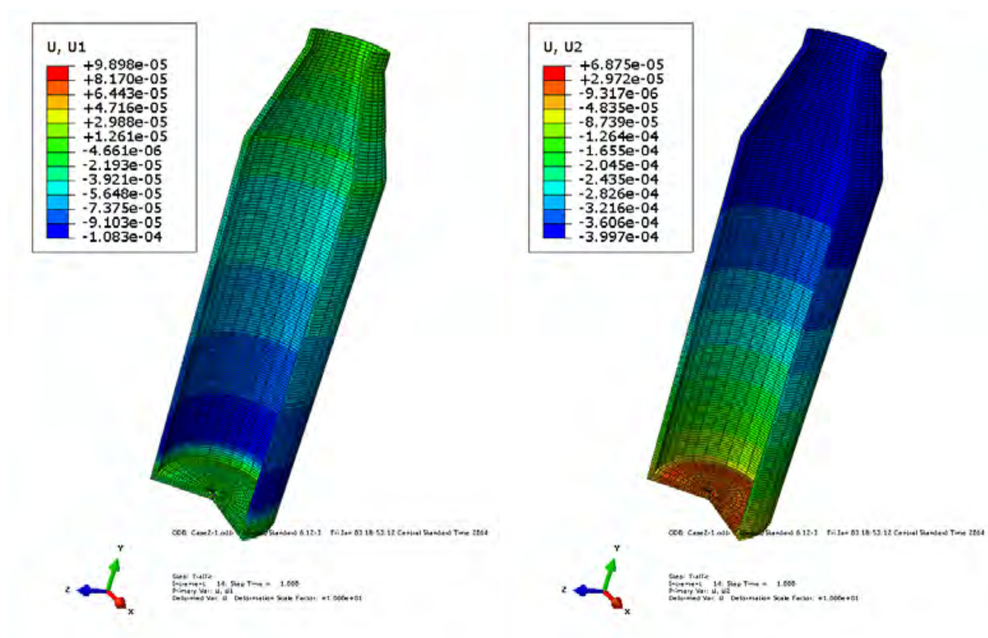
Figure 5-21. Hoop (a) and Vertical (b) Stress (psf) Distribution in Manhole Structure (C2-1).



a b

Figure 5-22. Horizontal (a) and Vertical (b) Deformation (ft) of Manhole Structure (C2-1).

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In spite of decreasing the thickness of the manhole structure, the elastic strain of the concrete is still smaller than 0.003 as shown in Figure 5-23. This result indicates that the manhole structure can carry the applied load without undergoing major deformation.

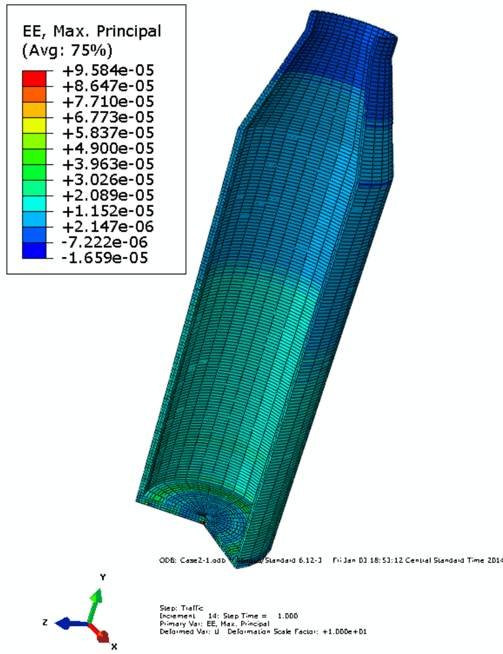

Figure 5-23. Maximum Principal Elastic Strain in Manhole Structure (C2-1).

The moment distribution in the base for C2-1 is shown in Figure 5-24. The cracking moment for this case is 2,846 lb-ft (2,099 N-m), which is 4.5 times greater than the calculated maximum moment 627 lb-ft (462 N-m) in the center of the base.

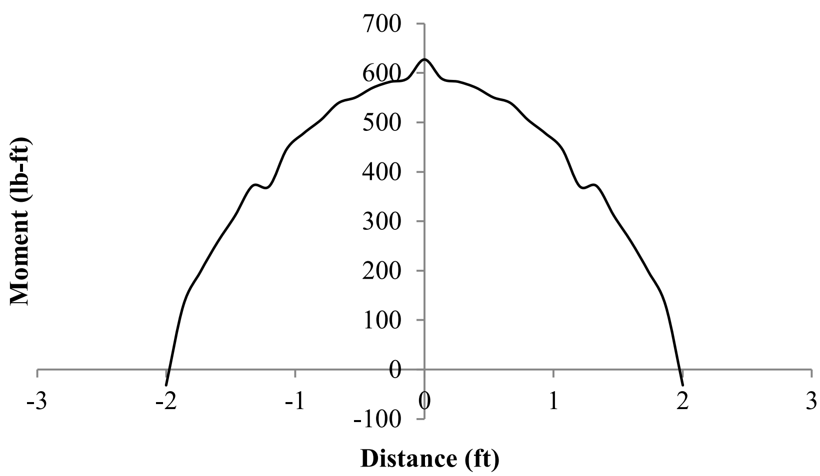

Figure 5-24. Moment Distribution in the Manhole Base (C2-1).

Figure 5-25 presents the vertical deformation of the base for C2-1. The maximum deformation for this case is 0.026 (0.7 mm), which is larger than Case 1 as a result of lesser thickness used for Case 2.

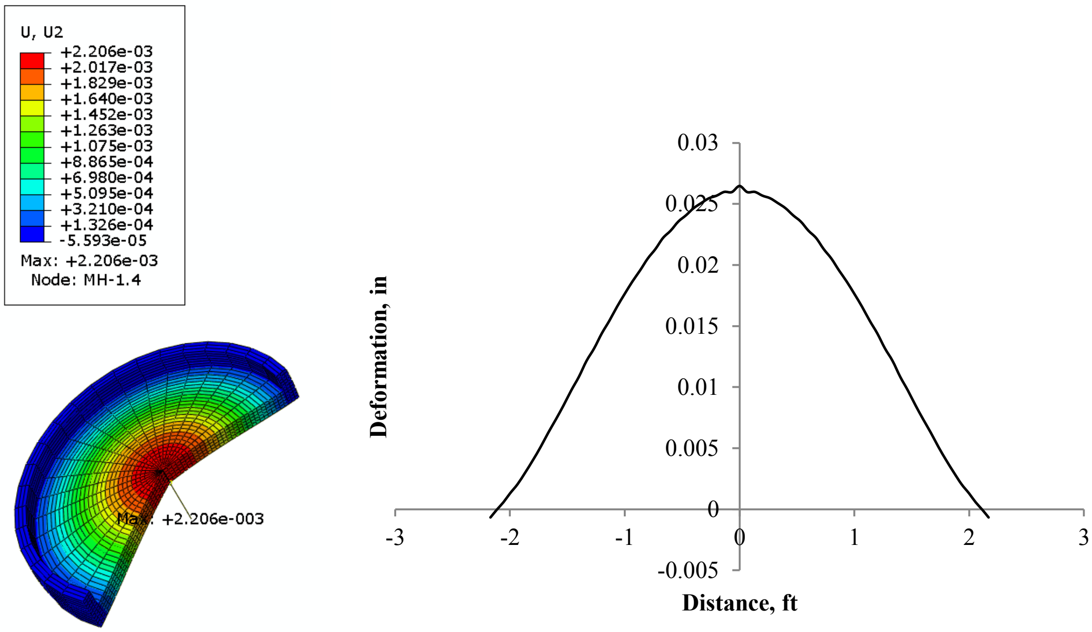

Figure 5-25. Vertical Deformation of the Manhole Base (C2-1).

The results obtained from the C2-2, which is the case with deteriorated manhole structure and lining are shown in Figures 5-26 and 5-27. The results are almost the same as the previous case, which shows that deterioration of the lining does not have a significant effect on behavior of the manhole structure under this scenario.

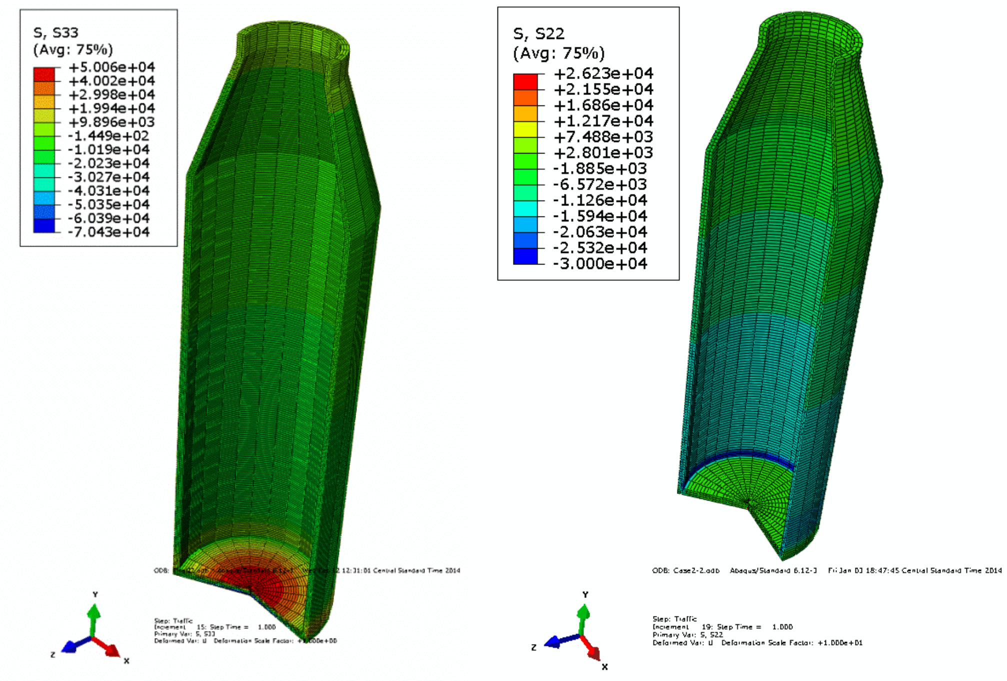

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Figure 5-26. Hoop (a) and Vertical (b) Stress (psf) Distribution in Manhole Structure (C2-2).

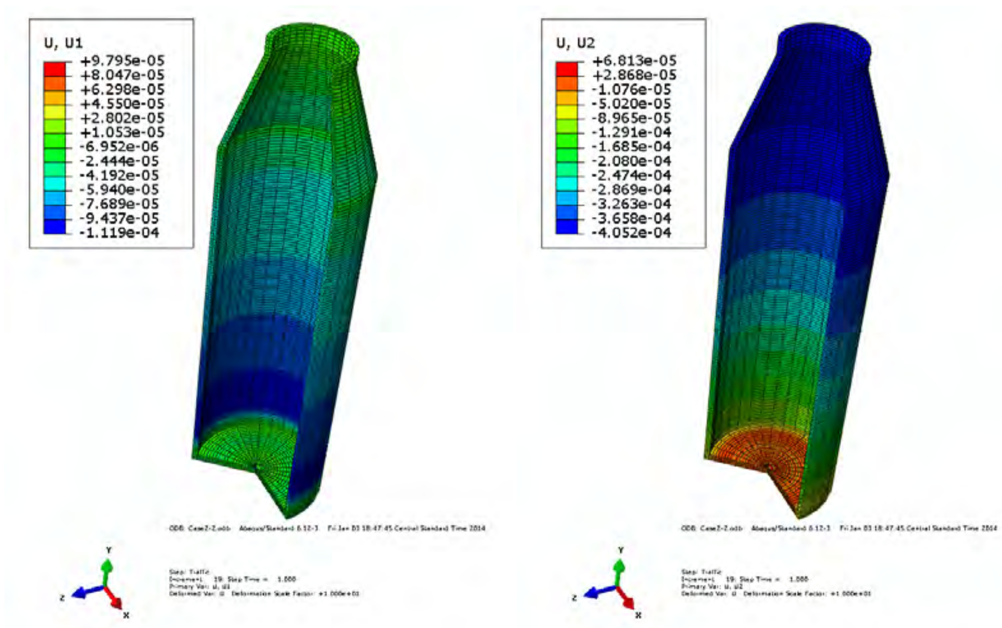

Figure 5-27. Horizontal and Vertical Deformation (ft) of Manhole Structure (C2-2).

Horizontal (hoop) and vertical (longitudinal) stresses and deformations induced in the lining for case C3-2 are presented in Figures 5-28 and 5-29. To simulate the interaction between the epoxy lining and soil in Case 3, a surface to surface interaction with friction coefficient of 0.1 in tangential behavior and hard contact in normal behavior were used.

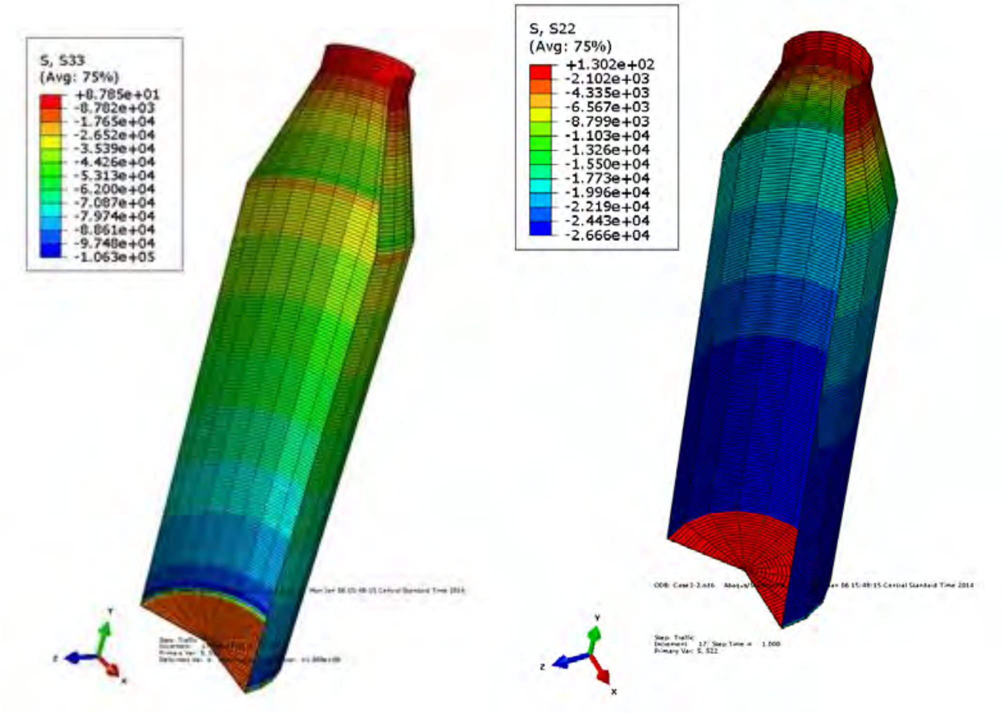

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Figure 5-28. Hoop (a) and Vertical (b) Stress (psf) Distribution in 300-mil (7.6 mm)-Thick Epoxy Lining (C3-2).

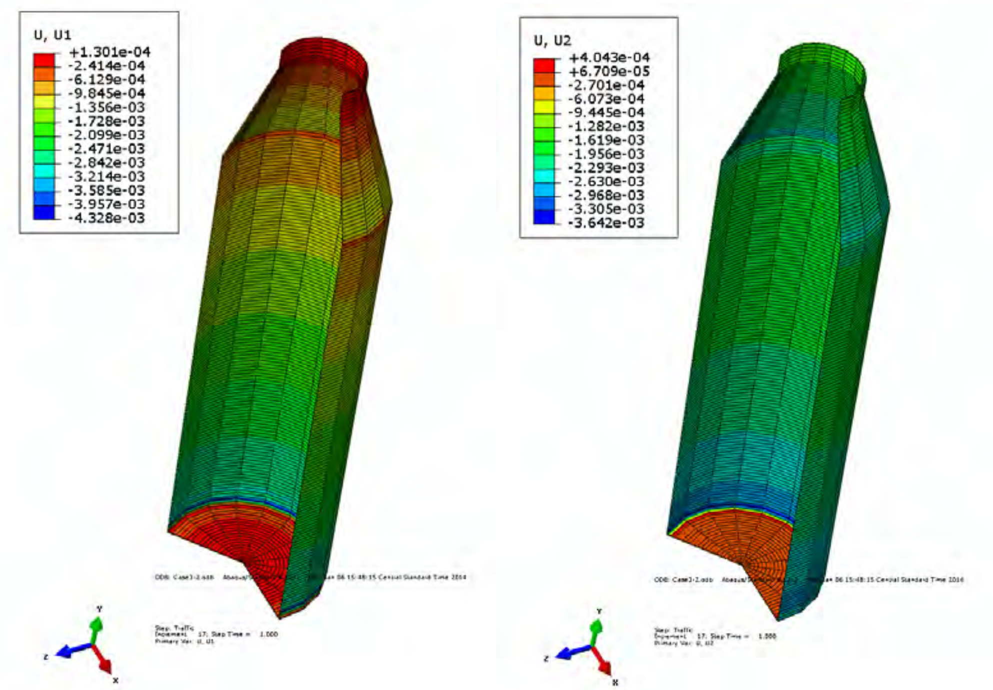

Figure 5-29 Horizontal and Vertical Deformation in the Simulated Epoxy Lining (C3-2).

As stated previously, the third case represents a scenario, in which the manhole structure is fully deteriorated and the loads are supported by the deteriorated lining. In case C3-1, the only existing load besides the soil pressure was pore water pressure and in case C3-2 traffic load was also added. The strains and deformations observed for Case 3-1 and 2 were significantly higher than the Case 1 and 2; nevertheless, an apparent failure of the liner was not observed per the FEM simulation.

Another scenario of concern was concentrated pressures on the liner due to a hole on the host manhole wall and stresses at such openings as pipe penetrations and holes due to deformation. These two supplemental cases were simulated with Abaqus using the FEM and discussed further below.

5.5 Manhole Simulations – Special Considerations

To simulate the presence of a hole in the manhole structure and to observe the behavior of the manhole and lining, a 3D model with an inner diameter of 4 ft (1.2 m), thickness of 5 in. (127 mm) and height of 5 ft (1.5 m) was created in Abaqus. The lining with a thickness of 300 mils (7.6 mm) was applied inside the manhole segment and a 3-in. (76 mm) diameter was located on the manhole structure as shown in Figure 5-30. An eight-node linear brick, reduced integration, hourglass control (C3D8R) mesh type was used for the simulation.

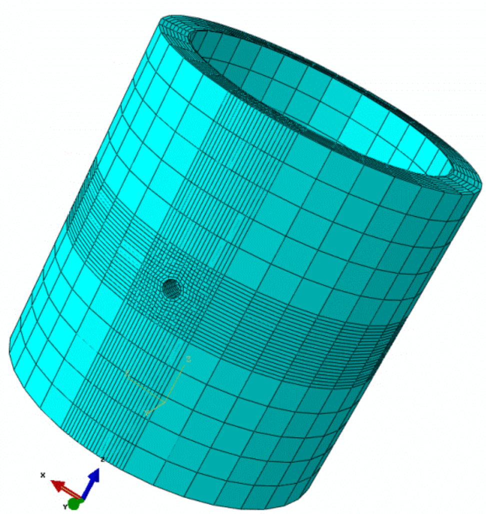

Figure 5-30. Geometry and Mesh of the Manhole Segment with Hole.

The contact model between the concrete and the lining was simulated by means of the cohesive model, the same approach used in the previous simulations. The same material properties (for the concrete and lining) as the ones for the previous simulation were used. A circumferential pressure was applied to the manhole structure representing the pressure from the water and surrounding soil. The simulation was conducted with 2,000 psf (95.8 kPa) pressure for the combined soil and water pressure.

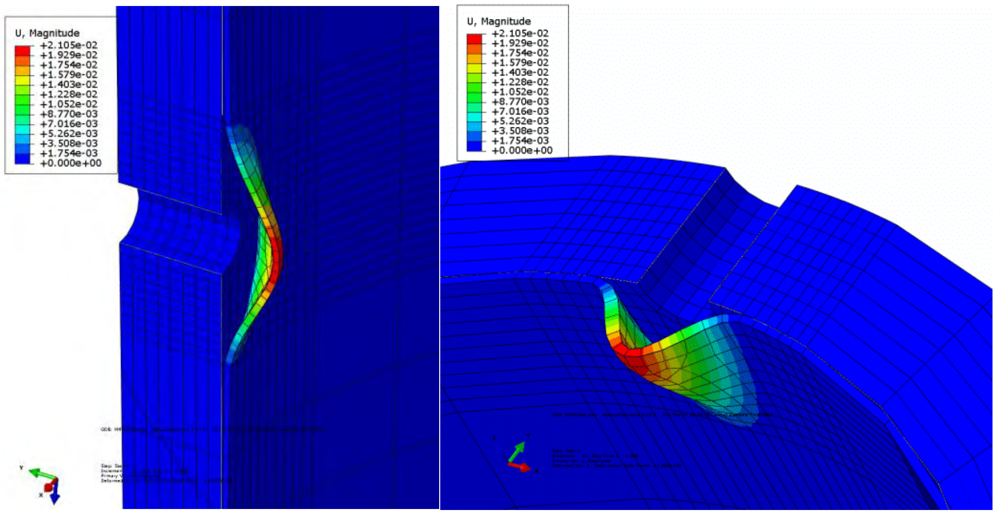

Figure 5-31. Deformation of the Lining at the Manhole Section at the Location of the Hole (ft).
Note the Deformation is Exaggerated in this View for Clarity.

As shown in Figure 5-31, the pressure applied to the lining at the location of the hole, results in the separation of the lining from the concrete around the perimeter. Radial deformation and the hoop stresses induced in the lining due to the applied pressure of 2,000 psf (96 kPa) are shown in Figures 5-32 and 5-33.

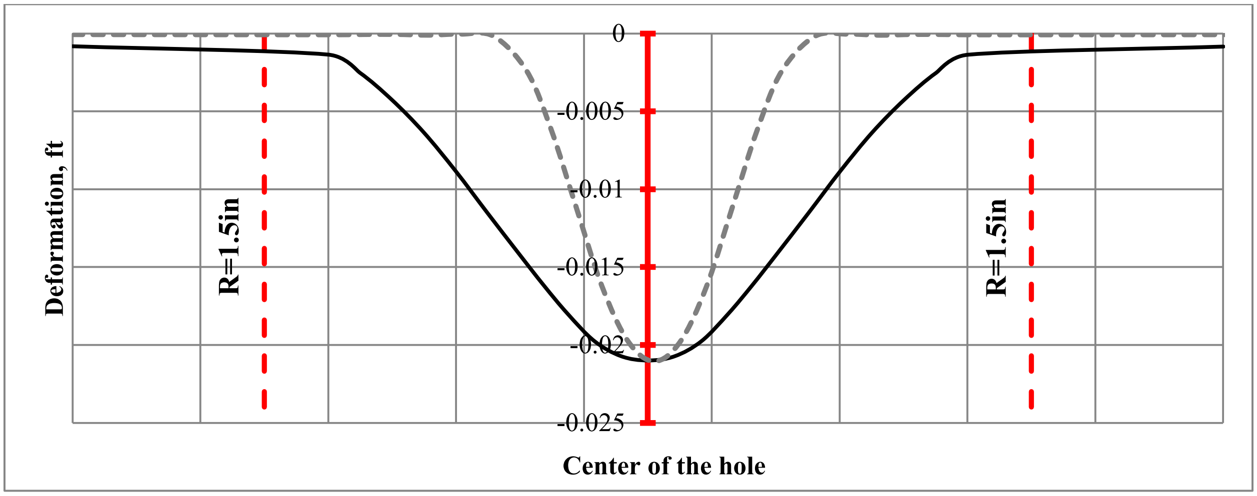

Figure 5-32. Deformation of the Lining at the Location of the Hole in Two Directions

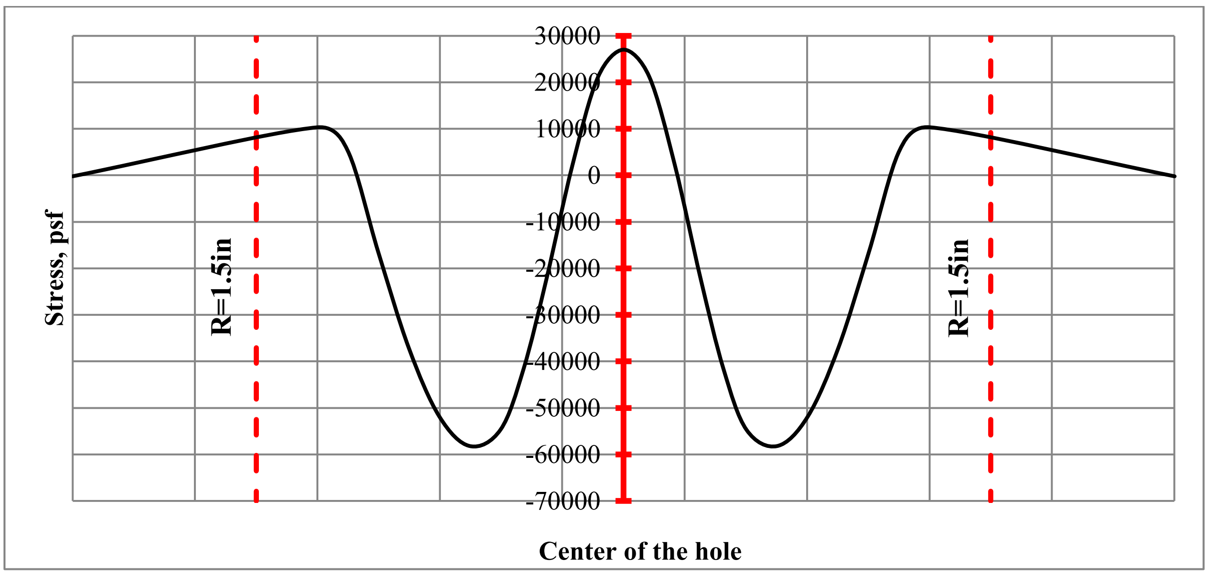

Figure 5-33. Stresses in the Lining at the Location of the Hole.

CHAPTER 6.0

CASE HISTORIES

The purpose of collecting case histories from participating wastewater utilities is to use past experience in manhole rehabilitation to the maximum extent in developing the experimental procedure, rehabilitation guidelines, and the DST. In addition to the write-ups, this section also includes industry expert opinions on how a manhole rehabilitation project should be approached using their experience with numerous projects at different locations.

The 10 participating utilities provided 11 case histories/comments regarding their manhole rehabilitation experience. These utilities are as follows:

- Metropolitan Water Reclamation District of Greater Chicago (MWRDGC), Illinois.
- City of Rowlett, Texas.
- Sewerage and Water Board of New Orleans (SWBNO - via MWH Global).
- Village of Palmyra, Illinois (via Benton & Associates, Inc.).
- Johnson County Wastewater (JCW), Kansas (two case histories).
- Orange County Sanitation District (OCSD), California.
- Anchorage Water and Wastewater Utility (AWWU), Alaska.
- Village of New Lenox, Illinois.
- New Castle County, Delaware (via Arcadis).
- Sarasota County, Florida.

The case stories provided below are directly incorporated from the original narratives provided by each wastewater utility with editorial changes to make it compatible with the report format.

6.1 Metropolitan Water Reclamation District of Greater Chicago (MWRDGC)

During the early 1990s, a number of cave-ins and failures occurred within the Metropolitan Water Reclamation District's (District) system. As a result, the District implemented the Intercepting Inspection & Rehabilitation Program (IIRP) in 1993. The IIRP was implemented to specifically address the inspection and rehabilitation needs of the aging conveyance system.

Visual inspections of manholes and structures are completed on a five-year inspection cycle and video inspections are done on a 10-year inspection cycle. Once it has been determined repairs are required, minor repairs are performed by the Operations and Maintenance (O&M) Department of the District. The Engineering Department of the District handles design and construction management for major repairs of manholes and structures. Major repairs that need to be completed immediately are done under an emergency O&M contract.

In 2003, the Engineering Department began to include manhole rehabilitation in all sewer rehabilitation contracts. Typical manhole rehabilitation procedures specified by the District include: removal of damaged concrete and rebar, infiltration control, grouting, featheredge repairs, reinforcing replacement (if necessary), and installation of manhole rehabilitation materials. A typical manhole rehabilitation consists of 1 in. of high strength cementitious product with a 125 mils (3 mm) top coat of epoxy. If the contractor chooses to use a urethane product, the contract requires the contractor to submit a calculation based on equation X1.1 of ASTM F1216 to determine the thickness. Typically the District uses materials listed on the "The Evaluation of the Protective Coatings for Concrete" (Redner Report, Chapter 2) published by the County Sanitation District of Los Angeles County.

Having performed manhole rehabilitations for approximately 10 years, the District has learned a few lessons:

- Lesson One, which is almost impossible to guarantee with government low-bid procurement requirements, is to use a reputable installer. Doing so provides more assurance that the installation of the material is being done in accordance with manufacturer's guidelines in order to avoid blistering/pitting of the liner (Figure 6-1), as well as other problems.

Figure 6-1. Blistering and Pitting Observed on a Composite Lining System.
(Cement Lining and Polymer Coating on Top)

Figure 6-2. Polymeric Liner Failure Along the Edges Inside the Manhole.

- Lesson Two, which relates to Lesson One, is the difficulty in performing Quality Assurance/Quality Control (QA/QC) on installed materials without the use of destructive methods. The QA/QC work really needs to be done during the installation and not as a part of post installation inspections alone. This requires a District representative to be present at virtually all times to perform wet film thickness testing.
- Lesson Three, which also relates to the Lesson One, is the importance of QA/QC to premature cracking and delamination of the installed liner (Figure 6-2).
- Lesson Four would be to coordinate the manhole rehabilitation with resurfacing work so that manholes either aren't buried or the liner damaged during resurfacing work.

6.2 Sewerage and Water Board of New Orleans (SWBNO)

SWBNO, through its Sewer System Evaluation and Rehabilitation Program (SSERP), has developed a custom process for manhole condition assessment and rehabilitation method selection. This process consists of standardized data collection, computerized decision making and implementation of appropriate rehabilitation methods to quickly and effectively address manhole rehabilitation needs.

6.2.1 Introduction/Background

Rehabilitation of manholes is an important part of most sanitary sewer system rehabilitation programs. Manholes, in many cases, are a primary location of sanitary sewer defects and can be a significant source of I/I into a system. Because of a number of factors such as age, material of construction, soil conditions and strength of wastewater, manholes can become structurally unstable, even to the point of complete structural failure. Routine inspection and ongoing rehabilitation, therefore, are necessary steps that must be taken to prolong service life and reduce the potential liability associated with unsound manhole structures.

The SWBNO's custom process, the Manhole Rehabilitation Decision Support System (MRDSS) consists of standardized inspections and data collection, as well as a computerized decision-making tool to quickly and effectively evaluate and address manhole rehabilitation needs.

6.2.2 Manhole Inspection and Rehabilitation Goals

The primary goals of SWBNO for the manhole inspection and rehabilitation effort are to:

- Accurately inspect sewer manholes and provide consistent interpretation of collected data.
- Efficiently develop rehabilitation recommendations.
- Successfully implement cost-effective rehabilitation methods.

To achieve these goals, several facets of the inspection and rehabilitation recommendation process have been standardized. Because consistent interpretation of data is such an important part of an inspection and rehabilitation project, all parties involved in the manhole inspection and rehabilitation process (e.g., field inspectors, SWBNO staff, design consultants, project management consultants) are required to be educated on the standardization.

6.2.3 System Approach

The SWBNO sewer service area is divided into 10 basins. Manholes and sewer gravity mains were inspected as a part of the Collection System Evaluation Studies (CSES) undertaken in each basin. The CSES were a comprehensive effort to inspect the collection system and identify structural defects and significant I/I sources.

To ensure consistent interpretation of manhole defects, a standardized system was established that includes a manhole defect-coding manual and standard manhole inspection forms. Field inspector training was developed to ensure that the requirements of the defect-coding manual were met. A computerized decision tool that both selects the manholes in need of repair and determines cost-effective methods of rehabilitation was developed to provide consistent interpretation of the inspection data.

There are three major components of the manhole inspection and rehabilitation selection system.

- Training inspection personnel to properly use the defect coding manual and inspection forms.

- Actually completing the manhole inspections, which includes locating and physically inspecting the manholes.
- Data manipulation and interpretation, which includes entering inspection data into the electronic database and evaluating the inspection data to determine cost-effective rehabilitation methods.

The standardization of each of these components has resulted in an efficient and accurate system for determining manhole rehabilitation needs.

6.2.4 Training Inspection Personnel

The training covers each of the items included on the manhole inspection forms. With proper training and monitoring, accurate and objective inspection of manholes can be easily accomplished.

A review of the defect-coding manual and the standard inspection forms is performed in the classroom type setting, and then the inspectors are taken in the field to “practice.” The training session concludes with the inspectors completing inspection forms for the manholes observed during the training.

Establishing objectivity is likely the most important factor in any inspection effort. A defect that is considered minor by one inspector may be considered critical by another inspector. To improve the objectiveness of the inspection effort, the manhole defect-coding manual has clear and concise descriptions of the characteristics of many of the defects. Some examples of the descriptions of the defects are:

Corbel Condition Deteriorated – Multiple cracks; openings in wall are visible but pieces (or brickwork) are still in place (Figure 6-3).

Figure 6-3. Corbel Condition Deteriorated.

Wall Condition Deteriorated (Heavy) – Cementitious "shag" coating missing in some areas; multiple cracks, openings in wall are visible but pieces (or brickwork) are still in place; all mortar between bricks exists (Figure 6-4).

Figure 6-4. Wall Condition Deteriorated (Heavy).

These descriptions and the others included in the manual allow the inspection crew to assess the condition of a manhole by comparing what they see in the manhole with the description of the code in the manual. This reduces the amount of subjectivity that is used by the inspection crews during the inspection effort.

6.2.5 Data Manipulation, Storage and Interpretation

Once the inspection information has been collected and submitted on the standardized form, the data is entered into a Microsoft Access database for storage, quality assurance, decision-making and development of rehabilitation recommendations. Data entry is accomplished by use of a database customized specifically for entry and manipulation of manhole inspection data. The database consists of a form that has been developed so that the fields and entries correspond to the inspection form. Each of the entries into the database can be made using the number pad on the computer keyboard.

Quality assurance is accomplished through a predefined set of queries. The queries examine the condition assessment data to ensure that each of the fields in the database is populated. In the event that there is missing information, the database will create an "exceptions" listing containing the inspection data collected for the manholes that are missing data. These manholes must be re-inspected or the missing information must be re-entered from the inspection report.

There are several items on the inspection form that are critical to developing appropriate rehabilitation recommendations. These items, frame condition, corbel condition, wall condition, and trough condition, are the primary factors that determine the rehabilitation recommendations. The list includes cementitious lining of manholes, adjustment of manhole frames to grade, and replacement of castings (frame and cover).

The specific rehabilitation measures are determined by applying guidelines developed by SWBNO to identified defects. For example, SWBNO determined that two types of defects, classified as major, would be rehabilitated: those that threaten the structural integrity of the manhole and those that could result in a significant amount of I/I entering the manhole. All other defects are classified as minor and are only repaired if present with major defects.

Following is a list of the manhole rehabilitation methods currently used for the SSERP with a brief description of each:

Full-Depth Lining:	Installation of a manhole lining system from the bottom of the casting to the trough.
Partial-Depth Lining:	Installation of a manhole lining system in a portion of the manhole.
Install Inflow Pan:	Installation of a stainless steel dish to prevent inflow through the manhole cover.
Elastomeric Frame Seal:	Installation of a flexible sealant to the joint between the frame and corbel.
Adjust Manhole to Grade:	Adjusting of manhole casting vertically to match the existing grade using bricks or pre-cast rings.
Reset Casting:	Adjust manhole casting horizontally if it has become offset.
Replace Casting:	Remove and replace worn, corroded or broken castings.
Replace Manhole:	Replace entire manhole with brick or pre-cast concrete.
Grouting:	Pressure injection (grouting) to stop infiltration prior to other internal rehabilitation.

Once the data entry and quality assurance steps have been completed, the rehabilitation selection queries can be run. By using the inspection information and applying the rehabilitation guidelines, recommendations are developed for rehabilitation utilizing the inspection database. A series of queries is run in the database to populate a rehabilitation table, which identifies the rehabilitation methods to be used for each manhole. Costs are assigned to each of the recommended methods and an estimated cost of rehabilitation is developed.

The Sewerage and Water Board of New Orleans, through its Sewer System Evaluation and Rehabilitation Program, has proven that there is a way to identify rehabilitation needs for manholes. This process has also provided more efficient and consistent interpretation of manhole condition assessment and rehabilitation method selection, which reduces the likelihood of changes during construction.

6.3 City of Rowlett, Texas

The following narrative is written by Dr. Dennis Abraham (then with the City of Rowlett) based on his experience with sanitary sewer manhole rehabilitation carried out in Rowlett over the years.

Manhole rehabilitation is a combination of judgment calls made by engineers in cities and a majority is actual engineering. I usually breakdown manholes into two categories:

- Manholes on major lines typically force mains or huge gravity lines [18 in. (460 mm) in diameter or greater].
- Manholes on minor lines [< 16 in. (410 mm)].

The critical difference is in the generation of sewer gases that are in the major lines, which are usually higher and therefore cause faster deterioration of the manhole and severe structural failures. The rehabilitation of these manholes requires additional safety measures, because the flows are usually dynamic and hydrogen sulfide is generated continuously. The typical rehabilitation involves the addition of concrete lining, or replacement of mortar in brick manholes. The best case is typically rebuilding the manhole with new polymer compounds, which have captured the markets in recent years. This is where the judgment call becomes crucial, as every vendor says that their polymer compound is the best. There is no clear industry standard to rank the different products, and every municipal engineer is looking for a definite answer with respect to the quality of these liners. They also depend on experience of other cities that have used the technology, or samples provided by the vendor. Usually the sample manholes have the polymer application for less than a year or two. The sample manhole can be placed in the most corrosive atmosphere, which is where the transition of force-main to gravity line takes place. However, the engineer takes a risk if the polymer does not work the way it was intended to, the result being in paying for the rehabilitation again.

Usually the major line manholes are greater than 5 ft. (1.5 m) in diameter and have bolted lids that prevent I/I from the surface. The minor lines are usually residential lines that lead to collectors and then to transmission lines. The deterioration in these manholes are usually slow, and the failures occur usually due to no maintenance. These manholes are also placed on streets and if the compaction while building the manhole is not done to standards, then even minimal traffic action will cause failures of settling, shearing of the ring and cone, and crushing of pipe or separation of the pipe from the manhole.

Elevated levels of groundwater also lead to leaks and penetration of the water through weak points and at the joint where the pipe is fused into the manhole. Manholes in floodplains and swamps need to be designed and constructed with extra vigilance in terms of geotechnical checks, placement above the flood or high water elevations and maintained as failures occur frequently due to the groundwater and flow conditions that the manhole is subjected from the outside.

6.4 Village of Palmyra, Illinois

Palmyra is a small town located in central Illinois. The Village of Palmyra constructed its wastewater collection system and treatment facilities in the 1970s. The wastewater is collected via a gravity system with two lift stations (North and South). The collection system also includes a "manhole station" and five grinder pumps that serve individual connections.

The Palmyra wastewater collection system includes 136 manholes. The majority of the manholes in the collection system are precast concrete, approximately 4 ft., in diameter with the upper 3 ft. tapering to a corbel (cone) section to receive the manhole steel frame and cover. Channels in the manhole floor accommodate the flow that conveys the sewage through the manhole.

The Palmyra sewage treatment plant (STP) receives significant I/I during heavy rainfall events. As such, a sewer system evaluation program was initiated in 1998. As a part of the program, essentially all of the manholes (134 of 136) were visually inspected (without man entry). Additionally, flows through the manholes at critical areas were monitored using portable weirs for an extended period of time (four months).

Upon completion of the inspection, a manhole rehabilitation program was developed and the first tier manholes were rehabilitated in 1999/2000. A three-layer, cured-in-place, polyester-fiberglass system was used for the rehabilitation.

The rehabilitated manholes were inspected in January 2013, after 13 years of service. The lining was still intact in all five manholes that were inspected (Figure 6-5). The Palmyra manhole rehabilitation can be regarded as a success story; nevertheless, it should be noted that even the most defected manholes had not shown any apparent sign of structural failure and they were less than 14 ft. deep. Based on the service life to date, it can be concluded that this three-layer cured-in-place system provided a good solution at least for sealing gaps, thereby stopping I/I in addition to preventing hydrogen sulfide induced corrosion.

Figure 6-5. Manhole That Was Lined (Cured-in-Place) with Reinforced Polymeric Lining.
The Liner Seems Intact after 13 Years of Service with No Apparent Defects.

6.5 Johnson County Wastewater (JCW), Kansas (Case Study 1)

As a part of an extensive sanitary sewer rehabilitation project, JCW used several manhole rehabilitation methods to reduce or eliminate excess I/I. JCW and their engineering consultants prepared specifications for the manhole rehabilitation products along with installation and subsequent testing requirements. Typically, if the manhole rehabilitation products comply with the contract specifications; were observed to be installed in accordance with the specifications and approved shop drawings; and meet all inspection, quality control, and testing requirements, it is assumed that the associated I/I has been eliminated and the rehabilitated manhole's service life has been extended. However, through post-construction inspection activities, JCW realized that defects in manhole rehabilitation work can still occur weeks and months after acceptable installation resulting in continued I/I, project success goals not being met, and unnecessary expenditures.

6.5.1 Background

JCW and its engineering consultants are working to reduce or eliminate excess I/I from entering the sanitary sewer system by evaluating the best methods to accomplish this through a pilot I/I rehabilitation project in areas of the Nelson Wastewater Treatment Plant (WWTP) Complex service area. The entire Nelson Complex service area is approximately 18,000 acres (7,300 ha), with the pilot project area representing approximately 500 acres (200 ha) in two parts of the service area. The long-term objective is to protect the environment and provide more dependable, cost-effective, and environmentally sound wastewater service to their customers.

The project was initiated in 2009 as a result of ongoing efforts by JCW to effectively manage wet weather flows in the Nelson Complex service area. It is part of the most recent National Pollutant Discharge Elimination System (NPDES) Permit renewal process for the Nelson WWTP Complex. As such, the pilot I/I rehabilitation project was designed and constructed to evaluate the effectiveness and costs of several I/I removal strategies to reduce the risk of sewer overflows and basement backups within two smaller pilot areas. The objective of this portion of the project was to evaluate and develop the processes and procedures for future removal of public and private sector I/I sources throughout the larger Nelson Complex Service Area.

To determine the most cost-effective I/I removal strategy, the pilot project area was divided into several smaller rehabilitation strategy areas in which different combinations of public and private sector I/I source removal were designed and constructed, including:

- Rehabilitation of public main sewers and manholes.
- Removal of building and inflow sources/defects.
- Rehabilitation of private service lines.

The construction project included full-time construction observation and administration by one of JCW's engineering consultants. Duties of the construction observers included confirming that manhole rehabilitation work was performed in accordance with the contract documents and approved shop drawings and ensuring that testing required in the contract documents was performed. Due to the pilot project nature of the project, several post-construction manhole inspections were performed several weeks and months after the manhole rehabilitation work and associated testing was completed.

6.5.2 Chemical Grouting

Chemical grouting was used to seal external voids and internal defects in 113 manholes prior to installing a cementitious liner and, in 10 manholes, to stop active leaks prior to other rehabilitation work. Grout was applied at points of visible infiltration and points with indications of leakage (i.e., not actively leaking). Chemical grouting was observed to be installed in accordance with the specifications and approved shop drawings. Installation was performed within recommended environmental conditions and included proper surface preparation and injection. Pressure was monitored during installation and active leaks stopped.

During post-construction inspection activities that occurred during higher groundwater conditions than those existing during product installation, three of the 10 rehabilitated manholes that were not cementitious lined were found to be leaking. Visible leaks were then grouted a second time to stop the leaks. As a result, it is recommended that post-construction inspection of manholes rehabilitated via chemical grouting occur during high groundwater conditions to confirm that the leaks are sealed adequately.

6.5.3 Cementitious Liners

Cementitious lining was spray applied to provide a permanent seal against I/I in 108 manholes. The cementitious lining was observed to be installed in accordance with the specifications and approved shop drawings. Installation was performed within recommended environmental conditions and included proper surface preparation, application, and material curing. Compressive strength testing was subsequently performed.

During post-construction inspection activities, two manholes exhibited cementitious lining installation defects such as groundwater weeping and material sloughing (Figure 6-6). Surface flaking and spider cracking have been witnessed on other projects. Thus, a satisfactory installation based on visual inspection immediately after installation may not be adequate to ensure the long-term effectiveness of the lining. Since these types of defects may not occur until weeks or months after installation, it is recommended that post-construction inspection of manholes rehabilitated via cementitious lining occur several months after installation.

Figure 6-6. Cementitious Liner that Experienced Material Sloughing.

6.5.4 Polyurea Liners

Spray applied polyurea liners were installed in 98 manholes to repair frame seal and grade adjustment leaks in numerous manholes. The liners were observed to be installed in accordance with the specifications and approved shop drawings. Installation was performed within recommended environmental conditions and included proper surface preparation, application, and material curing. A wet film gauge was used to ensure the minimum thickness was maintained during installation. Subsequently, visible pinholes were repaired, and holiday testing was performed (and repairs were performed if necessary).

During post-construction inspection activities, it was determined that 90 of the installed frameseal and grade adjustment liners had failed to some degree. Numerous liners had peeled away from the substrate and many liners had various sized bubbles that, when cut open, were found to be filled with water (Figures 6-7 and 6-8). Similar to cementitious liners, a satisfactory visual inspection immediately after installation may not be adequate. As a result, a post-construction inspection of these liners should be performed several months after installation to better ensure the long-term effectiveness of the liner.

Figure 6-7. Spray Applied Liner that Peeled Away from the Substrate.

Figure 6-8. Spray Applied Liner that Exhibited Bubbles.

6.5.5 Conclusion

Through post-construction inspection activities, JCW fortunately realized that assuming the long-term integrity of manhole rehabilitation based on acceptable installation observation and testing alone may not be justified. During the course of this rehabilitation project, it was determined that installation of several common manhole rehabilitation products are sensitive to numerous environmental and installation conditions that can result in product failure weeks to months after an acceptable installation. Acknowledgement of this gave JCW the opportunity to repair the failed products such that the I/I removal and project success goals were met and it was done without unnecessary expenditures on their part.

6.6 Johnson County Wastewater, Kansas (Case Study 2)

As part of JCW's ongoing asset management efforts, JCW identified manhole condition assessment as an area where data was lacking. JCW's Existing Infrastructure (EI) engineering staff began working with the O&M staff responsible for line cleaning to determine a way for existing JCW O&M crews to perform a manhole condition assessment when manholes are accessed for line cleaning.

6.6.1 O&M Crew Inspections

JCW O&M Line Cleaning Crews clean approximately 30% of the JCW system annually. JCW crews focus their efforts on cleaning the older vitrified clay pipe (VCP) sewer lines on a targeted 3-year rotation. This line cleaning program provides an excellent opportunity to gather condition data on many of the older manholes in JCW's system that are being accessed and viewed at the surface by line cleaning crew members.

In 2012, JCW O&M Line Cleaning Crews began assigning a "quick rating" of 1 through 5 for manholes they accessed during the course of their normal line cleaning activities.

The following are the descriptions given for each quick rating:

- Like New.
- No Deterioration – Damage – I/I.
- Minor Deterioration – Damage – I/I.
- Moderate Deterioration – Damage – I/I.
- Severe Deterioration – Damage – I/I.

The ratings are entered into the JCW CMMS database in a field in the work-order form when the line segment cleaning work-order is completed.

The results of the condition assessment inspections completed in 2012 were evaluated by JCW Existing Infrastructure engineering staff. The manholes rated 4 or 5 are currently being re-inspected by JCW EI engineering staff to confirm the condition of the manhole and determine if a repair is required (surface inspection only). If a repair is required, the type of repair is determined at the time of the re-inspection and EI engineering staff then manages the process of scheduling and completing the repair with JCW manhole repair crews or an outside contractor.

Future plans include the following activities to improve the process:

- JCW EI engineering staff are going to summarize and share the results (with pictures) to O&M Staff and offer feedback to O&M crews to help ensure condition assessment ratings are being assigned appropriately in the future.

- JCW EI engineering staff will evaluate the manhole condition rating descriptions in the database. The descriptions may need to be revised to make them clearer and more descriptive of what action is needed based on the condition of the manhole observed. For example (in JCW's opinion), a description of "Needs Repaired ASAP" or "Needs Inspected by Engineering" or "Good – No Action Needed" may be easier for field crews to assign than an arbitrary rating of "Moderate Deterioration/Damage." This may help improve the accuracy and quality of the ratings and avoid re-inspecting manholes that are in good condition.

As more data are gathered and staff gains experience, the program will be continually improved as new ideas are generated and implemented. This is an area of continuous improvement in JCW's asset management program.

6.7 Orange County Sanitation District (OCSD), California

OCSD was formed as a regional agency in 1948 for its member cities and sewerage agencies to provide wastewater collection, transport and treatment and ocean disposal of treated effluent. It currently has a service area of 471 square miles and serves central, west, and north Orange County. Population today is about 2.5 million. Dry weather flows are about 200 MGD at this time.

6.7.1 Background

After World War II, the county area served by OCSD was transitioning from rural and agricultural to residential and industrial and growing. A backbone system of six regional trunk lines were installed in the early 1950s to drain to the Reclamation Plant 1 in Fountain Valley and five drain to the Treatment Plant 2 in Huntington Beach. OCSD today provides full secondary treatment at its two plants. Treated effluent from Plant 1 drains to Plant 2 then is comingled and pumped five miles offshore for ocean disposal. Several regional pumping plants are needed in the flatter areas. Member cities and sewerage agencies also own, manage, operate, and maintain satellite facilities that drain by gravity or are pumped to OCSD's regional facilities. Treated effluent from Plant 1 is also now provided to the Orange County Water District for their advanced treatment for the Ground Water Replenishment System (GWRS) which recharges the aquifer in central Orange County.

Wastewater collection is managed through a General Waste Discharge Requirements Order issued by the state of California in 2006. Cities and sewerage agencies that own one mile or more of sanitary sewer facilities are mandated to enroll in the state's program. OCSD, with its satellite cities and sewerage agencies, developed a regional model of this statewide program in 2002. This model program helped reduce sewer spills and improve system management and funding.

6.7.2 Manhole Rehabilitation Programs

OCSD specified PVC lining (extruded liners with custom design "locking" seams) in its concrete regional trunk and interceptor piping when VCP could not be used. Some of the early regional manhole structures however were unlined. As heavy metals were removed from industrial discharge streams in the early 1970s as a part of EPA's Source Control Program needs, corrosion increased. By the mid-1980s some structures needed significant structural repairs prior to coatings being applied. Manhole structure or pipe failures have not occurred. Early work with coatings focused on polyurethanes and similar spray on products. OCSD also used the outcome from the work underway at the LA County Sanitation District and test criteria established by John Redner (Chapter 2) and their team. Later regional networking evolved to the LA Public Work's (Greenbook) Pipe and Manhole Rehab Task Force. OCSD's Engineering Standards also evolved. Corrosion Engineers joined OCSD staff members in the early 2000s as OCSD's Advanced Asset Management Program was improving based on the Australian/New Zealand models.

6.7.3 Sewage Conditioning Programs

Caustic soda slug dosing was initiated in the early 1980s to reduce and inhibit formation of the slime layers in the wetted perimeter of the regional sewers. It is still used today in some trunks where continuous chemical dosing at remote sites is not feasible to achieve our sewage conditioning goals. OCSD's current approach to sewage conditioning has been covered in presentations done previously at WEFTEC. This approach helps extend asset life and the by-product is fewer odor complaints.

6.7.4 Some Lessons Learned Thus Far

- Surface preparation of the concrete is the most important step for any coating or glued on PVC liner product to be successful.
- Spark testing of PVC weld strips is important to discover defects which if unrepaired, will allow gases to travel into the concrete.
- Utilize tension or pull tests to make sure proper bonding is achieved by the coating/lining to the substrate.
- Grouting through the structural wall into the soils is now done to minimize exterior groundwater infiltration that damages liners and coatings.
- Improving engineering standards from job to job is a continuous learning process.
- Specialized inspector training and use of corrosion engineers have helped OCSD to ensure successful repairs and improvements are done by contractors to standards and detailed job specifications.
- Metal rungs were removed from manhole structures and surfaces were properly sealed.
- PVC liners can now be successfully applied to the grade ring and mortar areas to the manhole cover frame.
- OCSD is moving towards an improved condition assessment program so manholes can be visually inspected more quickly and frequently to recognize and report potential stressors in the sewage, its hydraulic performance, and manhole structures and coatings.
- Sewage is generally fresher in the satellite owned systems and minimal corrosion is seen except in areas closer to OCSD's regional system or in areas near city pump or lift stations.
- Manhole inspection forms used by field crews to acquire and report data to others should meet the long range needs of the condition assessment program in order to support the overall asset management program.
- Private facilities: septic tanks and grease interceptors and pumping systems can also be subject to corrosion and should be inspected and coated by others as needed to ensure long life.

6.8 Anchorage Water and Wastewater Utility (AWWU)

This case study is based on a technical paper presented at the International Symposium on Cold Regions Development (ISCORD) by Lynda Barber-Wiltse, PE, Project Management Supervisor at AWWU. Additionally, a manhole (junction chamber) in Anchorage rehabilitated with a two-part polymeric system was included in this case study.

Girdwood, Alaska, U.S., consists of a diverse and transient population of outdoor enthusiasts, local businesses, recreational services, and Anchorage commuters. It is home to Alyeska Resort and skiing facilities. The population flactuates from about 1,800 (local residents) to over 5,500 (residents plus visitors and tourists). The average daily flow at the Girdwood Wastewater Treatment Facility (GWWTF) can increase from 0.02 to 0.09 million m^3/s (0.5 to 2.0 mgd) during typical snowmelt and rainfall events. The wastewater characteristics include relatively low organic material loads, low (cold) wastewater temperature, and colloidal material. The 2004 facilities plan update concluded that without significant reductions in inflow and infiltration (I/I), a new $20-25 million wastewater treatment facility would be needed by 2008. Leaky manholes and cleanouts were given high priority in reducing the I/I ino the wastewater system.

The Girdwood wastewater collection system consists of approximately 418 manholes, 40 cleanouts, and 34 km (21 miles) of 5 cm through 61 cm (2 in. through 24 in.) pipes. The majority of the pipes are 8-in. (20 cm) ductile iron pipe; the collection system also includes some HDPE pipes. The elevation of the sewer pipes start at 136 m (446 ft.) above sea level and go down to 1.8 m (6 ft.) above sea level.

6.8.1 Girdwood, Alaska I/I Reduction Program

AWWU resolved to evaluate and repair the wastewater collection system to reduce I/I. For the last few decades, AWWU personnel and contractors tried various repair techniques. Some of the techniques worked initially, but eventually succumbed to frost jacking and other damage. In an attempt to "divide and accomplish" the project, AWWU deployed flow meters throughout the collection system, and see whether I/I could be traced and isolated to a specific basin. This effort provided valuable information, but was inconclusive. Many of the homes in Girdwood are only occupied on weekends and holidays; and therefore, wastewater flows do not follow patterns seen in other AWWU facilities. In some areas, the flow was too low for the meters to work. Another challenge was that many of the manholes in Girdwood are either in submerged locations or buried in streets rights-of-way, and therefore, are not readily accessible.

AWWU has a relatively new gravity flow sewer system essentially comprised of sewer mains, laterals, and manholes. The sanitary sewer pipes are generally in good condition. Data from flow measurement and CCTV inspection pointed to the manholes being the "weakest link" in the collection system. AWWU decided to implement a pilot project in a small area, try new manhole repair techniques, then evaluate how to expand the lessons learned to the larger area.

AWWU is a publicly owned utility that requires competitive bidding for construction services. The nature of this work varied from virtually every other competitively bid type of project AWWU performs, since competitively bid projects are priced upon a defined scope of work, at a known location, with an implied "warranty of fitness" of the plans and specifications, on which a bidder bases the price. Whereas, the scope of work for each manhole varied depending on its individual condition, which cannot be determined without inspection. AWWU determinted that the most cost-effective method of executing a project to repair nearly 400 manholes was to combine condition assessment and repair steps into one process by hiring a contractor to work in tandem with engineers.

A summary of the work done for manhole rehabilitation is provided in Table 6-1.

Table 6-1. Repair Summary as of October 2006.

Total Manholes in the System	# MH	%System
	418	
Manholes Located/Inspected/Cleaned	400	95.7
Total Manholes Repaired	364	87.1
Lids Sealed	213	53.3
Chimneys Repaired	214	53.5
Chimneys Sealed	304	76.0
Barrels/Cones Adjusted	36	9.0
Frames Replaced	52	13.0
Manholes w/ Barrel Joints Sealed		
One joint	108	27.0
Two joints	90	22.5
Three joints	30	7.5
Four joints	6	1.5
Five joints	3	.8
Manholes w/ Leaks Grouted		
1-3 Grout fixes	84	21.0
4-7 Grout fixes	14	3.5
>8 Grout fixes	3	.8

6.8.2 Manholes Located/Inspected/Cleaned

The manholes in Girdwood are within road with road ROWs (state and local) buried in roadbeds, and in easements, vegetated areas, areas with high water tables, wetlands, and in road ditches and slopes. Some manholes are buried and some are exposed either at or above the ground surface. Most of the manholes located in easements and off the roadway have lids and frames that are exposed above ground surface. Some of the vegetation and bush around manholes had not been cut since the facilities were installed.

The contractor used record drawings, metal detectors, survey equipment, and other available information to find the manholes, then annotated the AWWU manhole number on a stake in the ground near the manhole. These reference stakes were useful as a communication tool between the engineer and contractor superintendent, construction crews and subcontractors. Most of the streets in Girdwood are gravel surface and manholes are buried at various depths. With the manholes uncovered and accessible, the engineers performed condition assessment before any further work occurred. AWWU also secured global positioning system (GPS) coordinates for all the manholes, which will enable the field crews to locate them easily in the future.

The engineer inspected the manholes and gave direction to the contractor on what type of repairs were required. The repair work started off with cleaning of each manhole. Infiltration (Figure 6-9) and root intrusion (Figure 6-10) were among the common defects and dysfunctions noted among the manholes inspected.

Figure 6-9. Infiltration (Runner) Along the Joints.

Figure 6-10. Manhole with Heavy Root Intrusion.

6.8.3 Lid Sealing

Lid sealing was simply comprised of inserting standard black rubber plugs in the circular vent and pick holes. The U-shaped holes at the edge of the lid were sealed with one-inch thick neoprene rubber cut in a form to fit firmly into the lid hole. These plugs were only used in manhole lids that were buried.

6.8.4 Chimney Repairs

The chimney or riser is the narrow opening between the surface or lid of the manhole and the cone section consisting of a series of concrete adjusting rings and grade rings.
This portion of the manhole structure is the most vulnerable to seasonal frost jacking and being

hit by snow removal equipment. Both the lids and frames and the snowplow operators can be hurt or jarred when the frames are hit. Figure 6-11 shows a gap between the soil and manhole frame, where the manhole frame has been shifted off the chimney creating a direct access for groundwater and other debris to enter the manhole.

Figure 6-11. Shifted Manhole Frame with a Gap.

The chimney repair work effort included adding or removing grade rings to raise or lower the manhole, removing and replacing damaged grade rings and realigning grade rings so that personnel and equipment could access the manhole. Figure 6-12 shows misaligned grade rings, which makes it difficult to access the manhole.

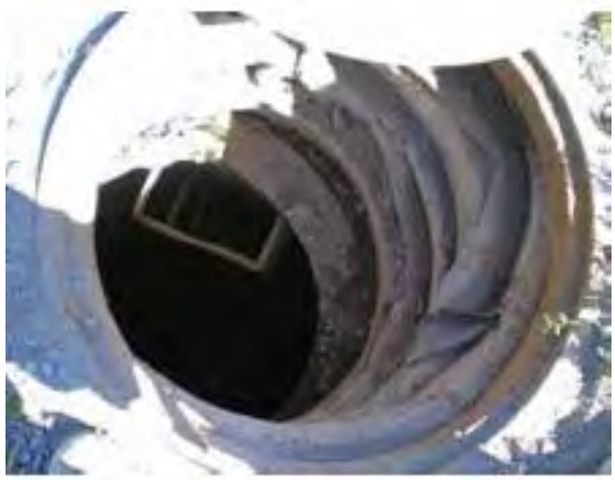

Figure 6-12. Misaligned Grade Rings.

6.8.5 Chimney Sealing

Once the chimneys were repaired, the contractor was directed to install internal chimney seals (Figure 6-13) if the manholes were located in areas where water could enter into the manhole between the grade rings.

Figure 6-13. Example of a Rubber Internal Chimney Seal.
Source: Cretex Specialty Products.

6.8.6 Barrel/Cone Adjusting

The barrels or cones were adjusted if the contractor could not raise or lower the manhole with grade rings or new frames only. AWWU has an established minimum (15 cm or 6 in.) and maximum (46 cm or 18 in.) range for the chimney portion of the manhole. If the adjustment could not be made at the chimney level, then the cone/barrel was excavated and replaced. Barrel to barrel (wall) joints or cone to barrel joints that were uncovered, replaces or installed during this project were also sealed on the outside of the manhole surface with an external joint seal.

A few manholes had structural damage, which were addressed by open cut removal and replacement of the damaged sections with new ones. The manhole shown in Figure 6-14 appeared to have been broken for a long time and a sinkhole formed at the ground surface above the manhole, and either the local residents or the road maintenance staff filled the hole with gravel. When AWWU discovered this manhole, the cone section was sheared and the frame, grade rings and top portion of the broken cone section had slipped suggesting impact damage (possibly during installation of another utility).

Figure 6-14. Manhole with Broken Cone.

6.8.7 Frame Replacement

Some of the “water tight” frames were deemed heavy and bulky by AWWU staff and vulnerable to damage due to diffulties in removing these for maintenance purposes. Hence, the contractor was required to replace these manhole frames/lids with the standard manhole frames specified for the project. Occasionally, frames damaged by snowplow equipment were replaced. Shorter frames were also used for small grade adjustments.

6.8.8 Joint Sealing

Frost jacking also contributed to barrel joint separation. One technique used in earlier attempts to mitigate damage due to frost jacking was bolted metal straps between sections. However, the strap bolts failed in the areas where severe frost jacking occurred. Hence, AWWU used a different solution for the succeeding phases, and directed the contractor to install an elastomeric internal sealing system (Figure 6-15) at manhole joints that showed evidence of leaking. A few of the deeper manholes had up to five joints sealed.

Figure 6-15. Example Manhole with Joints Internally Sealed to Prevent Infiltration.

6.8.9 Chemical Grouting to Stop Leaks

A chemical grout sealant was used to repair leaking defects in the manholes when chimney seals, barrel joint seals or replacement of manhole sections could not be used or the excavation would be very disruptive. The chemical grout sealant was injected from the interior of the manhole to the exterior, so that it was able to perform in the presence of infiltrating water. The majority of these repairs were located in the base section, where the influent and effluent pipes penetrated the manhole walls. A field construction technique for some installations or modifications uses a sledgehammer to knock out the hole in the manhole base to install the pipes. This kind of impact could result in extreme stresses, thereby weakening the structure. Another significant stress on the manholes is differential settlement between the pipe and manhole. Diagonal cracks that started at the pipe penetrations were visible in many of the manholes. Leaks often occurred wherever manhole penetrations were made (e.g., around the bolts used to strap manhole sections together to prevent frost cracking and at manhole steps.) This work was performed by a subcontractor, and is more temperature sensitive than the other repairs.

6.8.10 Junction Chamber Rehabilitation (Anchorage, AK)

AWWU used a composite rehabilitation system, to rehabilitate a severely corroded sewer junction chamber. A junction chamber has essentially the same function as a manhole, but it is custom designed and a larger structure vs. a standard cast circular concrete or brick manhole. Figures 6-16 through 6-18 show the before, during, and after rehabilitation pictures for the junction chamber that was successfully rehabilitated in Anchorage

Figure 6-16. Junction Chamber Prior to Rehabilitation.
Note the Severe H_2S Induced Corrosion.

Figure 6-17. Two-Part (Extruded PVC Plus Cured-in-Place Polymer) Being Applied at the Junction Chamber.

Figure 6-18. Junction Chamber Interior after Rehabilitation with the Two-Part System.

6.8.11 Conclusions

AWWU has been continously rehabilitating its manholes using a number of methods primarily to reduce I/I. Where manholes were deemed structurally deficient, a two-part (PVC sheet plus cured-in-place polymer on concrete substrate) lining system was used for rehabilitation. This system has worked well so far for AWWU.

For the Girdwood project, the average price per manhole was lower with the time and materials approach used for Phases I and II over the competitively bid, fixed unit prices method used for Phase III. AWWU was of the opinion that for an unknown or difficult-to-define scope of work, the time and materials approach would allow for more shared risk between the owner and contractor. On unit price projects, the contractor assumed most of the risk and this was reflected in the higher costs. Table 6-2 below compares the costs of Girdwood project among the three phases.

Table 6-2. Construction Cost for Manhole Rehabilitation for the Girdwood Project (2005-2007).

Phase	Number of Manholes	Construction Cost ($)	Average Cost/Manhole ($)
I	34	67,500	2,000
II	142	387,500	2,800
III	224	902,00	4,100

AWWU is already seeing the benefits of manhole rehabilitation. For instance, during a post rehabilitation heavy rainfall event in fall 2006 that caused damage in south central Alaska, the GWWTF received lower flows than experienced with prior heavy rainfall events.

6.9 Village of New Lenox, Illinois

In 1989 the Village of New Lenox's wastewater treatment plant underwent an expansion designed to handle current needs and included a storm lagoon to handle major rains. The plan was for the storm lagoon to run a few days per year. The plant was permitted for 750,000 gallons per day and the storm lagoon was designed to handle 500 gallons per minute flow. The plant average influent its first year of service was 1,244,000 gallons per day. The plant effluent averaged 896,000 gallons per day and the storm lagoon averaged 225,000 gallons per day and actually discharged 172 days. The collection system serves only 7,500 people, which indicates the I/I was so severe that the expansion could not handle the excessive flows. In coordination with their consulting engineer, the Village decided on a plan to transport and treat wet weather flow was not going to be enough and I/I reduction was needed.

As a first step, the sewer system map was updated to include the additions to the system since the 1980s. Then the Village moved and gave high priority to the manholes as the staff suspected significant I/I was entering into the collection system through manholes. Custom-design forms were created for manhole inspection. The manholes were visually inspected by the Village staff by using these forms. Additionally, the Village conducted wet/dry weather flow monitoring at manholes and measured ammonia (NH_3) concentrations in the samples collected from select manholes (Figure 6-19).[16]

Figure 6-19. Village Employee Collecting Wastewater Sample to Analyze for Ammonia Concentration.

[16] *A lower ammonia concentration indicates diluted sewage as a result of I/I.*

The inspections were then evaluated to develop a repair strategy. Many of the manhole covers had open pick covers, which allowed direct inflow. Most of the manholes with no open pick whole did not have gaskets, and the Village staff concluded much of the inflow entered into the system through these gaps and holes on the manhole top during heavy rainfall events. Rain stopper inserts (also known as inflow dish – Figure 6-20) were installed into the manholes that could have standing water over them.

Figure 6-20. Inflow Dish ("Rain Stopper") Inserted in a Manhole to Prevent Inflow Entering Through the Top Segment.

The next step in the manhole repair program was to address the upper 3 ft. of the manhole. The Village staff found the area between the cone and the frame to be a major source of I/I during wet weather. Hence, the voids were filled with elastomeric polyurethane from the cone to the frame. Then a two-part epoxy was applied from the cone to the manhole frame. Cement mortar did not work due to the difference in contraction/expansion rates of concrete cone and metal frame. To the Village's experience, the two-part epoxy worked well to seal the top parts of manholes if properly applied.

The last step in the manhole repair program was to address the lower sections of the manholes. The Village staff found water leaking between sections and around outside of pipe connections. Some holes had developed in manhole walls as well. If the source of the leak did not have active flow, the Village just packed it with hydraulic caulk cement. The holes and gaps with active infiltration were filled by chemical grout injection.

The Village consulted with an engineering firm to address the problems with few manholes that had severe corrosion or significant structural defects. These manholes were lined with cementitious linings. If the corrosion was hydrogen sulfide induced, then a two-part epoxy coat was applied on top of the cementitious liner

Upon completion of the first phase of the multi-year sewer improvement program (scheduled to be completed by the end of 2015), the Village started monitoring the sewer system for I/I reduction. A minimum of three dry and wet weather samples were collected from each area to compare the turbidity and ammonia concentrations in sewage (Figure 6-21). Additionally, the Village implemented visual inspection, CCTV surveys, and home inspections for direct storm sewer connections.

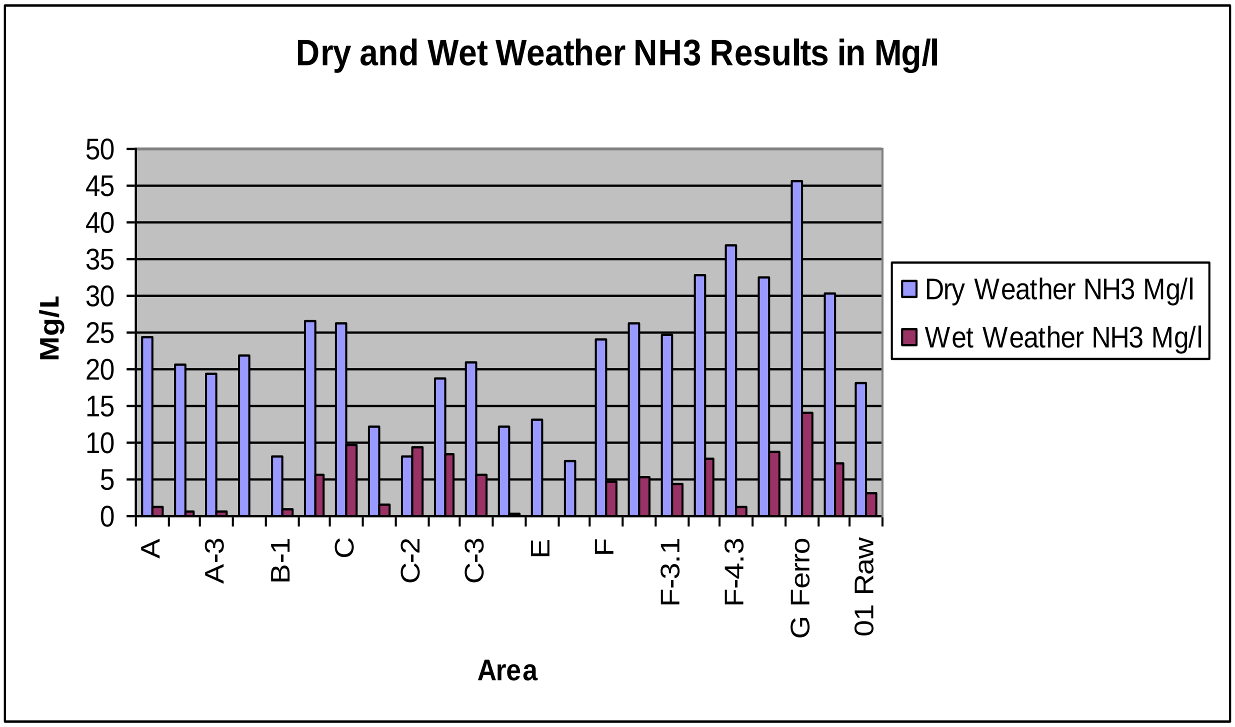

Figure 6-21. Comparison of Dry vs. Wet Weather Ammonia Concentrations in Sewage.

The Village found out that as many as 10% of the homes had sump pumps connected to the sanitary sewer, and an initiative to remove these sump pump connections is underway. Sewage analysis for ammonia concentration and turbidity proved to be an efficient method to monitor the reduction in I/I. As an interesting observation, higher turbidity was detected in some of the samples when the ammonia concentration was lower in wet weather. It turned out this was a result of "gravel wash" through the manhole frame and ring, and was indicative of severe inflow. Visual inspections were limited to monitoring the water level at the pipe connections in manholes.

The Village has already seen significant reduction in I/I upon implementation of the first phase of the rehabilitation program. As a simple and inexpensive method, inflow dish (rain stopper) was deemed to be an efficient method for reducing inflow.

6.10 New Castle County, Delaware

In 2001 New Castle County, DE, embarked on a comprehensive sewer rehabilitation program in the northern portion of its service area referred to as the “Brandywine Hundred.” The Brandywine Hundred, which contains the oldest and leakiest sewers in the system, was selected for rehabilitation due to excessive I/I in portions of the system that contributed to hydraulic overload of the collection system during heavy rains. In order to get the most benefit for the investment, the County targeted the basins, which were believed to receive the highest I/I in the Brandywine Hundred for a holistic rehabilitation program including mains, manholes and laterals.

Although reduction of I/I was the primary driver for all rehabilitation performed as part of the Brandywine Hundred Sewer Rehabilitation Program, structural repairs and reinforcements were included, where needed.

6.10.1 Inspection Methodology

The sewers in each of the basins targeted for rehabilitation were inspected using closed-circuit television (CCTV) and rehabilitated with either cured-in-place pipe lining or test-and-seal grouting. All manholes in each basin were visually inspected by the consulting engineering firm hired by the County as the Program Manager; then the appropriate type of rehabilitation for each manhole was determined. Table 6-3 lists the primary forms of manhole rehabilitation used in the Brandywine Hundred Sewer Rehabilitation Program.

Table 6-3. Manhole Rehabilitation Approach Implemented New Castle County for the Brandywine Hundred Sewer Rehabilitation Program.

Type of Manhole Rehabilitation	Where Used
Fiber-reinforced cementitious lining (FRCL)	Masonry manholes with visual evidence of leakage
Flexible Chimney Seal	Manholes with defects limited to chimney
Injection Grouting	Precast manholes with leaking joints Manholes with leaking pipe penetrations
Replace Frame and Cover	Manholes with old style covers that allowed inflow Damaged frame and cover
Reset Frame and Cover	Grade adjustment needed Severely damaged chimney

FRCL lining of manholes was selected to eliminate infiltration into masonry manholes (i.e., constructed of brick or concrete block) even if the observed leakage was only at one or two discrete locations in the manhole. Injection grouting a masonry manhole to remove leakage was not used due to concern that the groundwater would migrate to another defect in the manhole and start a new leak. Full manhole lining was selected to provide a longer-lasting fix and also allowed the manholes to pass a vacuum test to assure that infiltration had been eliminated. Discussions with various FRCL suppliers had indicated that the fiber reinforcement in the FRCL material provides tensile strength to resist cracking. Alternate manhole lining materials (e.g., epoxy and polyurethane) were permitted, but the selected manhole rehabilitation contractors have used FRCL due to its cost advantage. Corrosion was generally not a concern in the basins selected for rehabilitation; and therefore, liners with high corrosion resistance were not required.

6.10.2 Manhole Repairs Using FRCL

Between 2008 and 2010, the County rehabilitated 230 manholes of varying depths, in four different project areas, using full-depth FRCL. In most cases manhole repairs were performed after the sewer mains and laterals were rehabilitated in order to minimize the chance that a lined manhole may be damaged by the other rehabilitation work. Furthermore, when mainline sewers were lined, the pipe liner segments extended into manholes were removed from manhole channels as the presence of a mainline liner in the channel caused the manhole to fail vacuum testing after manhole lining. Prior to installation of the FRCL liner, all voids were filled and active leaks stopped.

In 2013, 211 of these manholes were inspected as part of a warranty inspection program. Out of the 211 FRCL-lined manholes inspected, 140 manholes had cracking of the FRCL liner within the top 3 ft. of the manhole (i.e., within the frost zone). Table 6-4 provides a summary of the inspection results. Figures 6-22 and 6-23 indicate photos of typical defects observed in the frost zone of FRCL-lined manholes.

Table 6-4. Post Rehabilitation Inspection Results.

Project Name	FRCL Installation Date	Warranty Inspection Date	# MHs Lined with FRCL	# MHs Lined with FRCL Needing CS	% of MHs Needing CS*
NA2	2010	2013	65	46	71%
AH	2008	2013	10	9	90%
NB05	2008	2013	56	40	71%
SP24	2009	2013	80	45	56%
Total			211	140	66%

**Chimney Sealing.*

Figure 6-22. Cracking in the Fiber-Reinforced Cement Liner Within a Few Years of Installation.

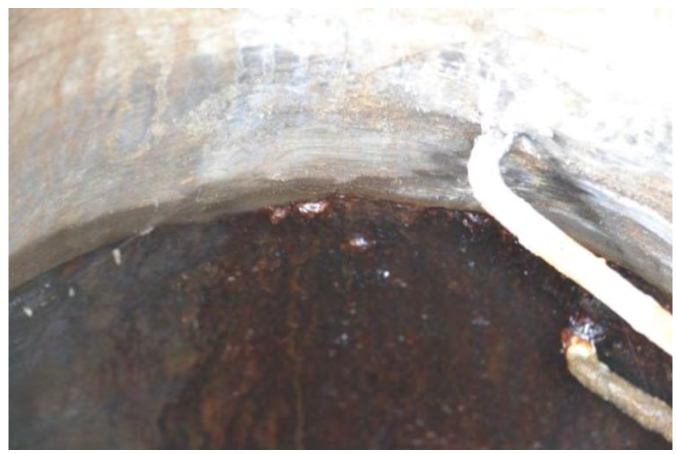

Figure 6-23. Infiltration/Inflow Marks Due to Cracking in the Fiber-Reinforced Cement Liner Within a Few Years of Installation.

6.10.3 Future Actions

Due to the high percentage (66%) of FRCL-lined manholes needing a post-FRCL rehabilitation chimney seal, going forward, the County plans to give two options to the contractors for the next phase of manhole lining:

- Use FRCL with a flexible chimney sealing system (such as elastomeric polyurethane) upon installation of the FRCL. The chimney seal shall cover the top 3 ft. of the manhole
- Use a flexible liner for depth lining (such as polyurethane) as an alternative to FRCL, thereby eliminating the need for chimney sealing.

The County intends to continue to evaluate alternate manhole rehabilitation materials and to monitor the long-term effectiveness of products used for manhole rehabilitation.

6.11 Sarasota County Utilities, Florida

The Sarasota County case study differs from the others because it is based on stormwater collection system rehabilitation. In fact, materials and methods used for sanitary and stormwater rehabilitation are essentially the same and as such, the scope of this work is applicable to stormwater manholes and structures rehabilitation. The only difference is that hydrogen sulfide induced corrosion is less of a concern for stormwater rehabilitation in comparison to sanitary and combined sewer systems.

6.11.1 Introduction

Over the last two decades, Sarasota County Public Utilities has been actively involved with infrastructure and manhole rehabilitation. Several asset management and rehabilitation programs have been instituted over the course of time. Based on the institutional experience, for non-structural applications in sewer systems, the County has been successfully using epoxy and calcium aluminate cement linings (Figure 6-24). Chemical and cementitious grouting has been used to seal the leaks and defects prior to lining. For stormwater systems with less corrosive environments, polyurethane products have been primarily used.

Figure 6-24. Manhole Lined with Calcium Aluminate Cement Liner.

6.11.2 Background

In the last five years, in order to manage its aging stormwater infrastructure, Sarasota County has taken a proactive approach for the management of its stormwater assets with the development of the Stormwater Infrastructure Rehabilitation Program (SWIRP). This program provides a practical approach for the inventory, assessment, design, construction and operation associated with the repair, renewal and rehabilitation of the County’s closed stormwater assets. These assets include stormwater pipes, outfalls, and related structures such as manholes, catch basins and end structures. As part of the program, County has successfully implemented NASSCO’s MACP standards integrated with jurisdictional preferences to evaluate/assess manholes and structures.

The most common problem with manholes (not associated with improper installation) was the failure of the joints connecting the manholes and pipes. Failures resulted from the vibrations of traffic as well as shifts from expansion or contraction, causing the joint to fail and the pipe and manhole to move, thereby risking separation. Common failure modes included: cracking, misalignment and separation.

6.11.3 Currently Used Manhole and Stormwater Structures Rehabilitation Techniques

Grouting

Grouting involved either chemical or cementitious materials, and was used to prevent infiltration and seal the voids within the soil surrounding the exterior of the pipe at the leakage point. This method reduced soil permeability and provided a long-term solution to leakage problems. Grouting was never viewed as a structural solution and only used for structurally sound manholes. Sarasota County's views on chemical grouting are:

Advantages of grouting:

- ♦ Generally inexpensive.
- ♦ Effective elimination of infiltration and exfiltration problems.

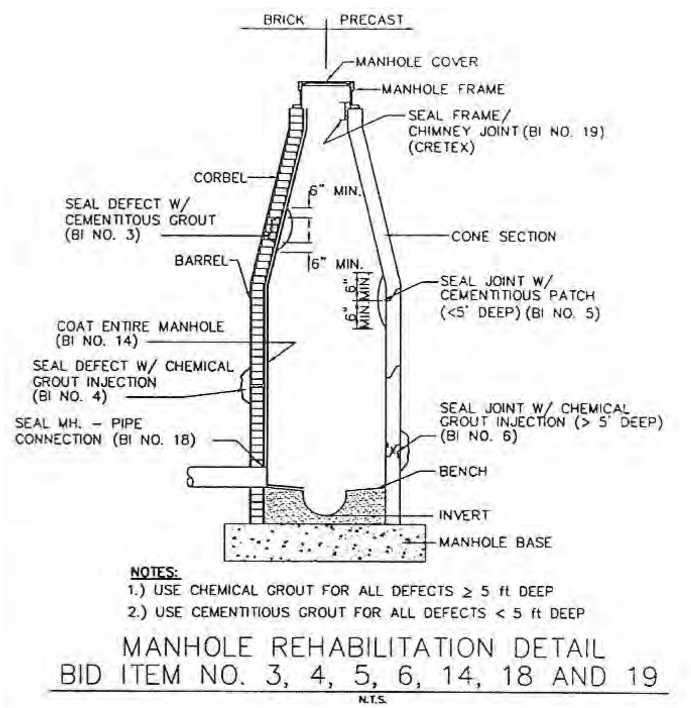

Figure 6-25. Manhole Rehabilitation Detail.
Source: Sarasota County.

Limitations of grouting:

- ♦ Does not enhance the structural integrity.
- ♦ Limited grout durability.

Spray-On Coatings and Linings

The humid environment typically encountered in Florida was the chief concern in application of spray-on coatings and linings as it impeded curing and bonding.

The various types of materials used for spray-on coatings and linings used as part of SWIRP are outlined below:

<u>Epoxy</u>

Epoxies were typically applied to structures in layers from 0.25 to 2 in. thick (Figure 6-26). Epoxy spray liners took approximately 15-16 hours to cure. Epoxies were generally used to offer the best moisture tolerance for rehabilitation and corrosion protection of manholes and other structures. Sarasota County's views on spray-on epoxies are:

Advantages of epoxies:

- Good abrasion resistance.
- Effective corrosion control.
- Shorter cure time than cement.
- No major health concern posed by byproducts.
- Moisture tolerant.
- Can also be used as a topcoat to a cementitious product to provide a chemical barrier.
- Usually improves structural integrity.

a b

Figure 6-26. Manhole with Epoxy Shortly After Application (a) and After One Year of Service (b).

Limitations of epoxies:

- Possess brittle properties.
- May not easily bond to the substrate.
- More expensive than cement.
- Infiltration control is required for proper adhesion.

Polyurethane Liners

Polyurethane linings provide corrosion protection, leak protection and semi-structural rehabilitation in manholes. Additionally thicker applications of polymer coatings offered some degree of structural benefit and leak protection. However, they were more expensive and required careful quality assurance during application. Curing was also required to ensure that the lining is free of defects that could allow corrosion to restart. Sarasota County's views on spray-on polyurethane liners are:

Advantages of polyurethane:

- Resistant to chemicals.
- Resistant to water penetration.
- Tolerant to temperature extremes.

♦ Good adhesion and corrosion protection on metals.
♦ Can improve structural integrity.

Limitations of polyurethane:
♦ Moisture sensitivity
♦ Application complexity
♦ Health implications of isocyanates still need to be taken into consideration

<u>Cementitious Liners</u>

Cement-based coatings have been used in the stormwater structures and manholes to provide structural support (Figures 6-27 and 6-28). Cement mortar (including calcium aluminate cement) was generally the least costly type of coating and the easiest to install, but was thicker and slower to cure. It provided a smooth interior surface layer that repaired damages or corrosion.

a b

Figure 6-27. Manhole Needing Repair with a Failed Liner on (a) and the Same Manhole One Year After Rehabilitation with Calcium Aluminate Cement (b).

The cost of cement mortar lining depended on the type of lining material used, whether structural reinforcement was required, the location and accessibility of the site, and contractor availability. Sarasota County's views on spray-on cement lining are:

Advantages of cement lining:
♦ Minimal service interruption compared to other renewal methods.
♦ Protects against deterioration and corrosion.
♦ Improves flow capacity.
♦ Prevents or slows rust.

Limitations of cement lining:
♦ Infiltration control required.
♦ Specialized equipment and trained personnel needed.
♦ Curing time may be extensive.
♦ May not enhance structural integrity by itself.

- Quality of installation is dependent on adhesion to the internal surface of the existing pipe, which is dependent on the cleanliness of the existing structure.

a b

Figure 6-28. Catch Basin Before (a) and After (b) Rehabilitation with Cement Mortar Liner.

6.11.4 Product/Application Requirements

Through experience it was learned that prior to installing the manhole coating system, active infiltration should be controlled. Infiltration control materials should be rapid-setting. The sealant materials should be non-shrinking, non-metallic, non-corrosive, and compatible with the coating/lining material to be used.

6.11.5 Field Testing and Acceptance

Field acceptance of coating and lining systems was based on the engineer's evaluation of the appropriate installation of the coating per field inspections. If deemed necessary, acceptance at times was also based on the project engineer's evaluation of the curing test data – holiday (spark) testing and adhesion (bond) testing. It was made sure that there was no groundwater infiltration or other leakage through the manhole wall after it was lined. If leakage was found, it was eliminated with an appropriate method as recommended by the liner manufacturer and approved by the project engineer at no additional cost to the owner. Typically, it was required that were no cracks, voids, pinholes, uncured spots, dry spots, lifts, delaminations or other type defects in the coating. If any defective coating was discovered after it was installed, it was repaired or replaced in a satisfactory manner within mutually agreed time period and at no additional cost to the owner. This requirement was applied for the entire guarantee period.

6.11.6 Pilot Projects

Level of service criteria adapted by the County requires using products, which shall provide a minimum service life of 25 years. In the past three years, several pilot stormwater rehabilitation projects have been successfully completed, few of which are listed below. As the County moves forward with the SWIRP program, it is the intent that information obtained by these pilot projects will be used in improving the process.

1. Shade Ave. Rehabilitation – Approximately 30-year old system with 22 total structures spread throughout 25 acres, including 20 catch basins and 1 double manhole. Structural repair for nine catch basins (41%) was recommended using cementitious liner.

2. Country Woods Rehabilitation – Consisted of 14 acres neighborhood having a 25-year old system with 40 total structures. Repair of 22 catch basins (55%) was recommended.
3. Country Place Rehabilitation – A 40-year old system within 83-acre subdivision with 26 total structures including 25 catch basins. Repair of nine (35%) catch basins/structures was recommended.

6.11.7 Conclusion

Sarasota County has had positive success with several of the rehabilitation products identified above, and in several cases saved money by doing rehabilitation work instead of replacing with new structures. Ongoing construction, post-construction and regular inspections with periodic maintenance thereafter are the key factors in extending the useful life of these assets. Corrective actions must be warranted as part of the rehabilitation work contracts. County requests guaranty/warranty offered by the manufacturer and/or contractor for a period of two to five years from the date of final acceptance. Initially engineering consultants at times overlooked the benefits of using no-dig manhole rehabilitation techniques, but this approach has changed given the success of the pilot projects. The County has multi-million dollar programs currently underway to rehabilitate the aging infrastructure.

CHAPTER 7.0

DECISION SUPPORT TOOL

7.1 Background

One of the main objectives of this project was to provide a practical manhole rehabilitation support tool that can be used by wastewater utilities and consulting engineers. The project team completed an extensive search for the available tools/systems used for sanitary sewer rehabilitation.

There are a number of tools developed for sewer pipe rehabilitation since the early 1990s and a detailed discussion of these tools is in a recent EPA study (EPA/600/R-11/077 by Matthews et al., 2011). Some of the formerly developed tools include an extensive field data and available technology evaluation to address the defects (an example of such a decision support tool is TAG-R developed at the Louisiana Tech's Trenchless Technology Center). Another recently developed decision support tool by Halfawy et al. (2009) uses a renewal plan for sanitary and storm sewers based on a genetic algorithm that seeks a solution for minimized risk of failure at the lowest life cycle cost. Most other decision support tools are based on a weighted score that calculates risk based on the consequences of failure and likelihood of failure. In fact, some of the available decision support tools include a section on manhole rehabilitation (e.g., TAG-R). Additionally, a number of the available decision support systems are user friendly and contain a good deal of software development with capabilities such as importing GIS data into the decision support system.

The fundamental shortcoming of the available tools with respect to manhole rehabilitation is that they return recommendations on which type of rehabilitation material/method to use without adequate knowledge of their capabilities. Another point of concern is the way cost factor (which is among the most important) is incorporated into these systems. The way these decision support tools are designed, they do not use cost as a factor at all, and make debatable recommendations. Alternatively they are fully automated and make crude assumptions based on cost data from a specific location and time period. The EPA study (EPA/600/R-11/077) also points out that although there is a fair number of decision support systems out there, only a handful of them are used in "real life" by wastewater utilities. This was attributed to most utilities using in-house capabilities/experience and custom developed decision support tools specific to each project (typically developed by getting assistance from a consultant).

This project team was of the opinion that a primary reason for the reluctance of most utilities and consulting engineers to using the available decision support tools is that they feel these tools are somewhat esoteric and not applicable to their specific sewer rehabilitation programs. A conventional (i.e., design-bid-build) rehabilitation project often requires the wastewater utility, in coordination with a consulting engineer, prepare standard technical specifications for sewer rehabilitation (which typically contain separate sections for manholes and pipelines). They then award the entire project to the lowest bidder. Even if a decision support system uses the most advanced tools and accurate data to evaluate the existing sewer condition, the overall cost of the project will determine the contractor. Hence, the method and material to be used for renewal is also selected as long as it meets the technical specifications. For a design-build project, the owner (wastewater utility) often requires the design-build contractor to implement the rehabilitation part of the project based on the technical specifications provided by the owner.[17]

7.2 Methodology

A main focus of this project was to provide guidelines to the wastewater utilities and consulting engineers on addressing rehabilitation needs of each manhole and preparing technical specifications for manhole rehabilitation. The purpose of the DST is to transform this logic to an electronic and user friendly platform that complies with the existing tools developed by WERF (e.g., SIMPLE).

The DST provided herein is an eight step approach:

1. Enter available data on manhole condition, soil and groundwater properties.
2. Enter static and dynamic loads.
3. DST will return the acceptable structural class and materials.
4. Enter location information (traffic and soil).
5. DST will return the acceptable methods (i.e., no-dig, open-cut, replacement, rehabilitation).
6. DST will return the acceptable methods, materials, and structural class by combining (3) and (5).
7. Prepare technical specifications (a sample specification is attached to this report).
8. Select the option with the lowest price[18].

Figure 7-1 shows the logic diagram for the DST being developed for this project. The blue boxes represent data entry steps, which are processed via a user interface developed using Java Swing (Figure 7-2). The user interface (UI) is designed to be user friendly. It retrieves and stores the data entered in a Microsoft Access database file. These data populate into the algorithms which provide recommendations on whether to rehabilitate/replace a manhole or leave as is. If rehabilitation is the selected remedy, the program recommends which class liner and methodology (no-dig, low-dig, or open-cut) to use. The data fields include soil properties, groundwater properties, static/dynamic loads, site conditions (Figure 7-1).

The results are presented in three groups as acceptable structural class (A, B, or C) and materials (item 3 in the DST process steps indicated above), acceptable renewal methods (item 5), and the combination of the two (item 6). The final orange box – that lists the acceptable structural class, materials, methods – is followed by a purple box that indicates the process of preparing technical specification based on the DST results indicated in the orange boxes. A sample technical specification is included in this report as Appendix A, thereby providing a complete a guideline to the user on selecting among the acceptable options with the lowest price. The terminal point indicated in the DST indicates the final step of soliciting the lowest acceptable manhole renewal material and method.

If the manhole is in such poor condition (e.g., collapsed or "X'ed" per the PACP/MACP nomenclature) then the DST overrides rehabilitation and recommends rebuilding/replacement of

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[17] *The owner might choose to use a third party engineer/consultant to prepare technical specifications and contract documents.*

[18] *This is assuming that manhole rehabilitation is contracted separately due to the scope of this project, in real life this may not be the case.*

the manhole. Additionally, holes of excessive size and quantity, substantial deformation or offset are also considered detrimental defects. The DST categorizes options based on weighted average score for each criteria used to select a material and structural class.

7.3 DST Algorithm

Output generated by the DST is based on a weighted score that is calculated from the input data entered by the user. The data entry user interfaces are based on the logic diagram presented in Figure 7-3. The DST essentially returns two recommendations:

- Acceptable manhole rehabilitation type and class.
- Method of rehabilitation (i.e., open cut, no-dig, or low-dig).

Table 7-3 indicates the score range for manhole rehabilitation material structural class based on the total weighted score for each manhole. The DST algorithm includes certain assumptions for void entry as a good deal of the information may not be available to the utility/engineer or the cost of obtaining the data may be prohibitive. As such, an average score is given for void entries vs. zero points, which would result in a significantly, if not substantially lower score than the manhole should receive, thereby recommending, for instance, a non-structural solution, where structural rehabilitation is needed.

Table 7-1. DST Total Weighted Score Ranges for Structural Class Selection.

Total Weighted Score (TWS)	Structural Class Recommended
TWS ≤ 1,000	A, B, C
1,000 < TWS ≤ 1,400	A, B
1,400 < TWS	A

Details of the data entry UIs and spreadsheets that outlined the scoring system for each DST step are indicated in the DST User Manual prepared as a part of this project.

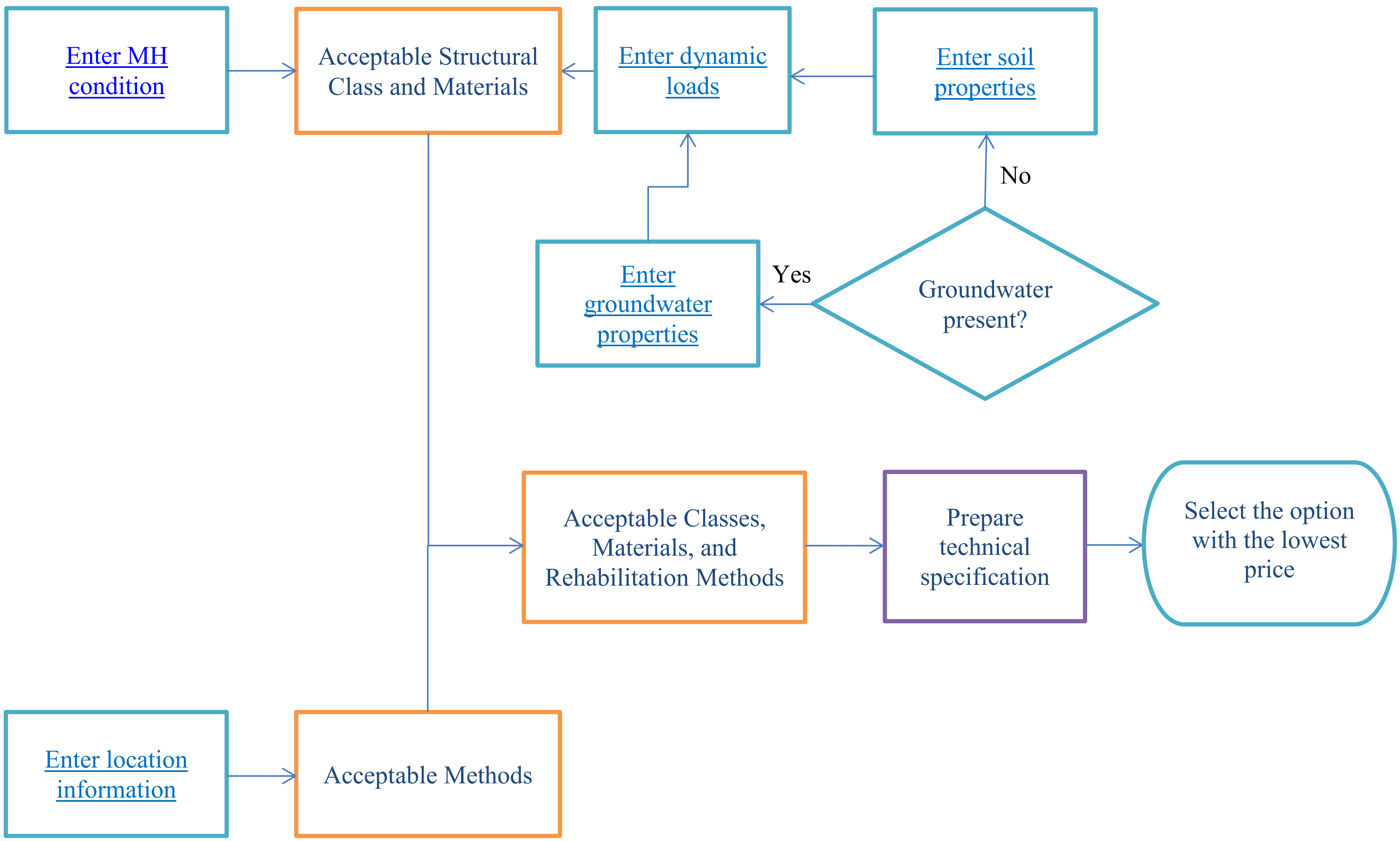


Figure 7-1. Logic Diagram for the Manhole Rehabilitation Decision Support Tool (DST).

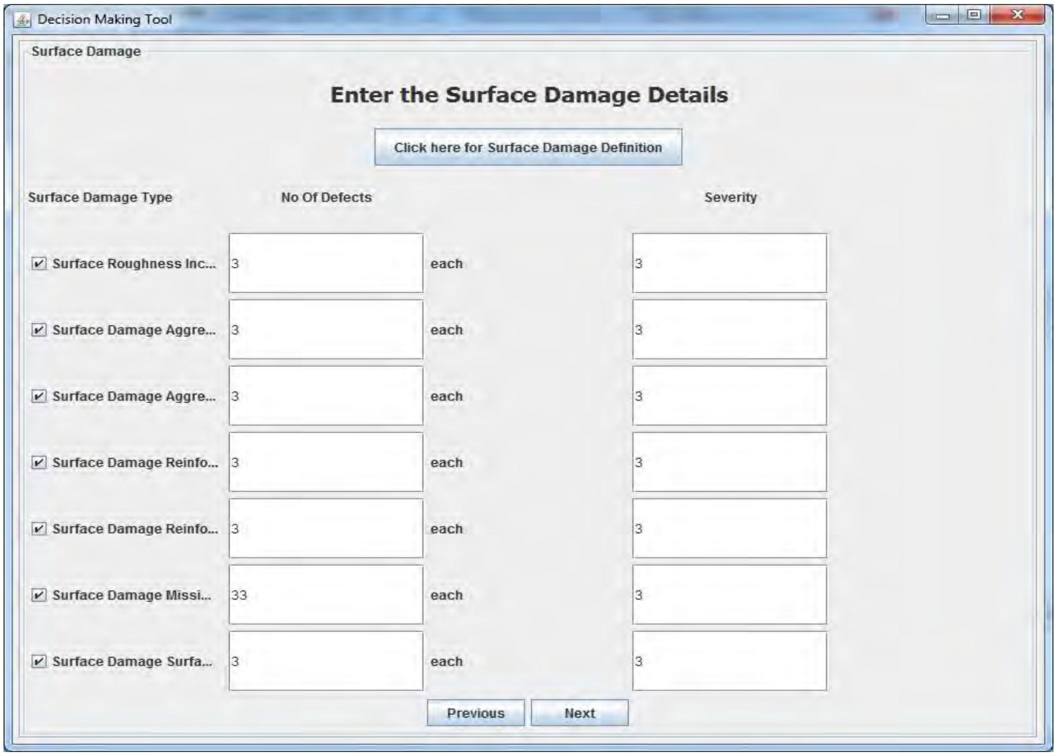


Figure 7-2. Decision Support Tool User Interface for Data Entry (Manhole Condition).

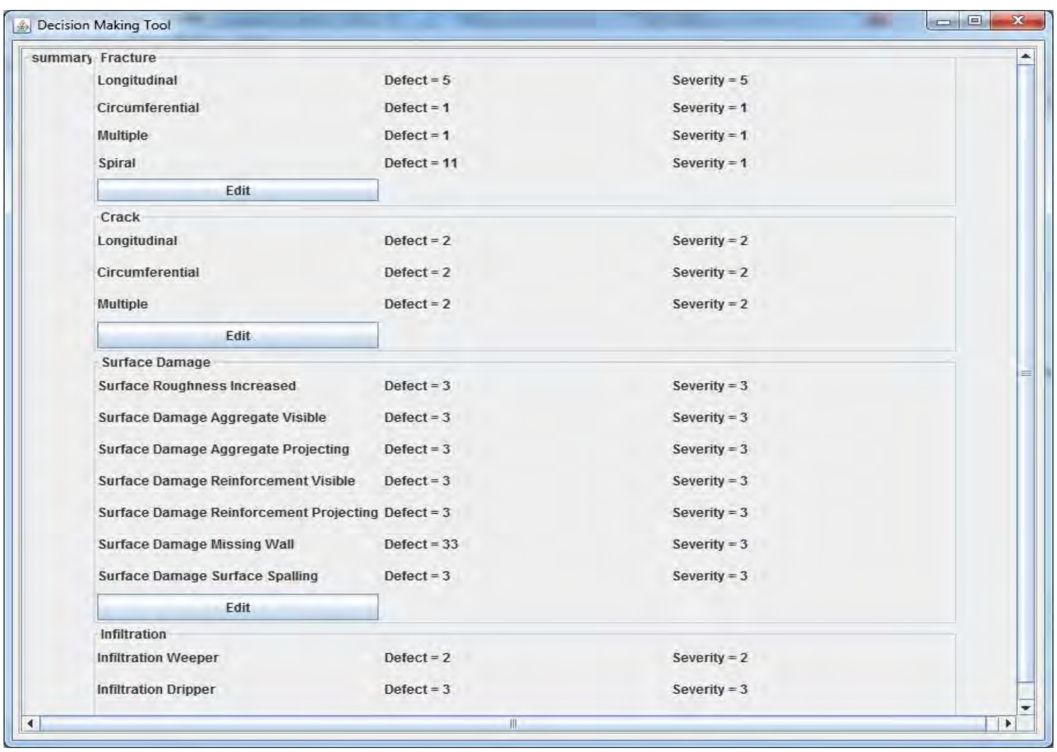


Figure 7-3. Decision Support Tool User Interface Input Data Summary (Manhole Condition).

7.4 Effect of Cost on Decision Making

Cost is one of the primary drivers in making decisions in the construction industry, nevertheless, it depends on several parameters (such as geographic location and site/soil conditions) in addition to the material and labor costs associated with the chosen technique. As such, an approximate range of cost for specific types of rehabilitation materials and methods are presented in Table 7-2. The purpose of providing cost information herein is to give an overall idea of the cost of rehabilitation methods associated with this study. It should be noted that costs vary significantly over time due to inflation and market conditions, and in addition to the parameters indicated above, it is advised that users factor in the economic condition at the time of rehabilitation and extrapolate the cost figures accordingly.

The cost information provided in Table 7-2 was analyzed and converted to vertical linear foot with respect to the unit price. These data were plotted (Figure 7-4) for linings considered Class A, B, C per the findings of this study. The cost range for each structural class suggests that there is a significant, if not substantial, cost difference among structural, semi-structural, and non-structural linings. As such, it is this project team's opinion that fully structural linings should be used only where needed, and the DST presented herein can determine the structural class of the lining type needed for each manhole included in a rehabilitation project.

Table 7-2. **Approximate Cost Range for Some of the Commonly Used Manhole Rehabilitation Techniques.**

Rehabilitation Material	Installation Method	Structural Class	Unit	Cost Range[19]
Polyurethane Lining (e.g., Sprayroq)	No-dig. Spin cast or trowel applied	A or B	VLF	$250-700
Epoxy Lining (e.g., Raven, Warren)	No-dig. Spin cast or trowel applied	A or B	sq. ft.	$18-35
Cement + Epoxy composite (e.g., Mainstay, Permacast)	No-dig. Spin cast or trowel applied	A or B	VLF	$200-300[20]
PVC or HDPE welded sheets (e.g., Arrow-Lock)	No-dig. Applied with cementitious binder.	B	sq. ft.	$30-35
Molded polypropylene liner (e.g., Predl Systems)	No-dig. Lining system is factory molded for each manhole.	A[21] or B	Each[22]	$5,000-7,000
Polymer Concrete (e.g., Geneva Polymer)	Low-dig. Inserted into the manhole after removing the cone.	A or B	Each	$7,400-8,800
Fiber reinforced cement lining (e.g., Quadex/Dynastone)	No-dig. Spin cast or spray applied with low pressure.	A or B	sq. ft.	$12-18
Fiberglass composite inserts (e.g. Sewershield)	Low-dig. Inserted into the manhole after removing the cone.	A or B	Each	$11,000-13,000
Elastomeric Polyurethane Chimney Seal (e.g. Elastaseal)	No-dig. Trowel applied on the interior of manhole chimney.	C	Each	$300-400
Cured-in-place Chimney Seal (e.g., LMK)	No-dig.	C	Each	$750-1,000
Chemical grout injection (e.g., Avanti)	No-dig. Polymeric resin injection to stop leaks	C	Each	$1,500-2,500
Polymeric (epoxy, polyurethane, etc.) grout (e.g., Sealguard, HyroPox)	No-dig. Grout application to stop leaks.	C	LS	$100-1,000
Medium Density Polyethylene Inflow Barrier (e.g., I/I Barrier)	Low-dig. Need to remove frame and adjustment rings for installation	NA	Each	$250-300
Hinged/Watertight Manhole Cover (e.g., Certainteed)	No-dig.	NA	Each	$280-380
Rubber Manhole Cover	No-dig.	NA	Each	
Stainless Steel Manhole Insert ("inflow dish" – e.g., Inflowshield by Inflow Systems)	No-dig.	NA	Each	$120-200
High-Density Polyethylene Manhole Insert ("inflow dish" – e.g., Inflow Defender)	No-dig.	NA	Each	$50-100

[19] *Actual cost may be out of the given range depending on location and market conditions at the time of application.*
[20] *For 80 mils (2 mm) thick epoxy coating.*
[21] *If installed with steel rebars (embedded in the cement mortar poured between the manhole interior wall and polypropylene liner).*
[22] *For a 10-ft. (3.3 m) deep manhole under average site conditions.*

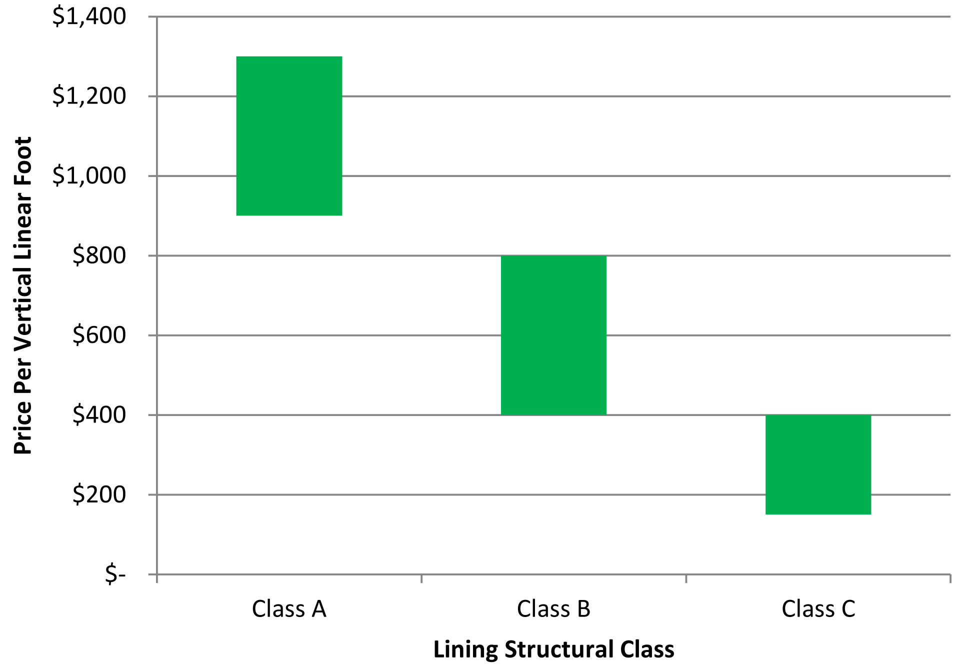


Figure 7-4. Price of Rehabilitation by Lining with Respect to the Structural Class of Rehabilitation Materials.

CHAPTER 8.0

CONCLUSIONS

The two main objectives of this project, classifying manhole rehabilitation materials and methods (based on their structural capabilities) and providing utilities and engineers a practical decision support tool for manhole rehabilitation, were accomplished by implementing the five tasks. The structural classification of manhole rehabilitation materials is indicated in Figure 8-1 and the decision support tool is available from WERF's website.

The literature review enabled the project team to find the published material on manhole rehabilitation. The topic was investigated over past decades in a number of lab tests. Some of the publications were based on actual applications in the field (case studies). The project was designed to not repeat any prior work that was thoroughly investigated by others; as such, adhesive tests were excluded from the test protocol to give more emphasis to the structural capabilities of the rehabilitation materials and methods. Another well-documented property is the inertness of the commonly used manhole rehabilitation materials against hydrogen sulfide induced corrosion; and therefore, this property was excluded from the testing protocol as well. The literature review conducted as a part of this project was a pivotal factor in developing an experimental design and computational modeling with the FEM.

The expert workshop held at the initial phase of the project was very useful in terms of receiving the opinions of the experts from various fields of the industry, on the proposed scope of the project. Essentially all of the participants, regardless of their background, agreed that there was a need for defining what is "structural" with respect to manhole rehabilitation, thereby giving credit to the first objective of the project. A number of experimental studies were also discovered via the expert workshop as these studies were conducted by third parties for the manufacturers, but not published.

The case studies revealed that a rehabilitation material is only as good as it is installed. If there is not thorough surface preparation, then even the strongest liners will not improve the condition of the host manhole. It was interesting to note that the 10 utilities participated in the study, spanning a wide geography from Alaska to Florida, from the smallest utilities that serve 700 people to the major ones (e.g., Metropolitan Water Reclamation District of Greater Chicago) serving millions, encounter essentially the same types of problems with manholes, and have been implementing a variety of materials and methods to address them.

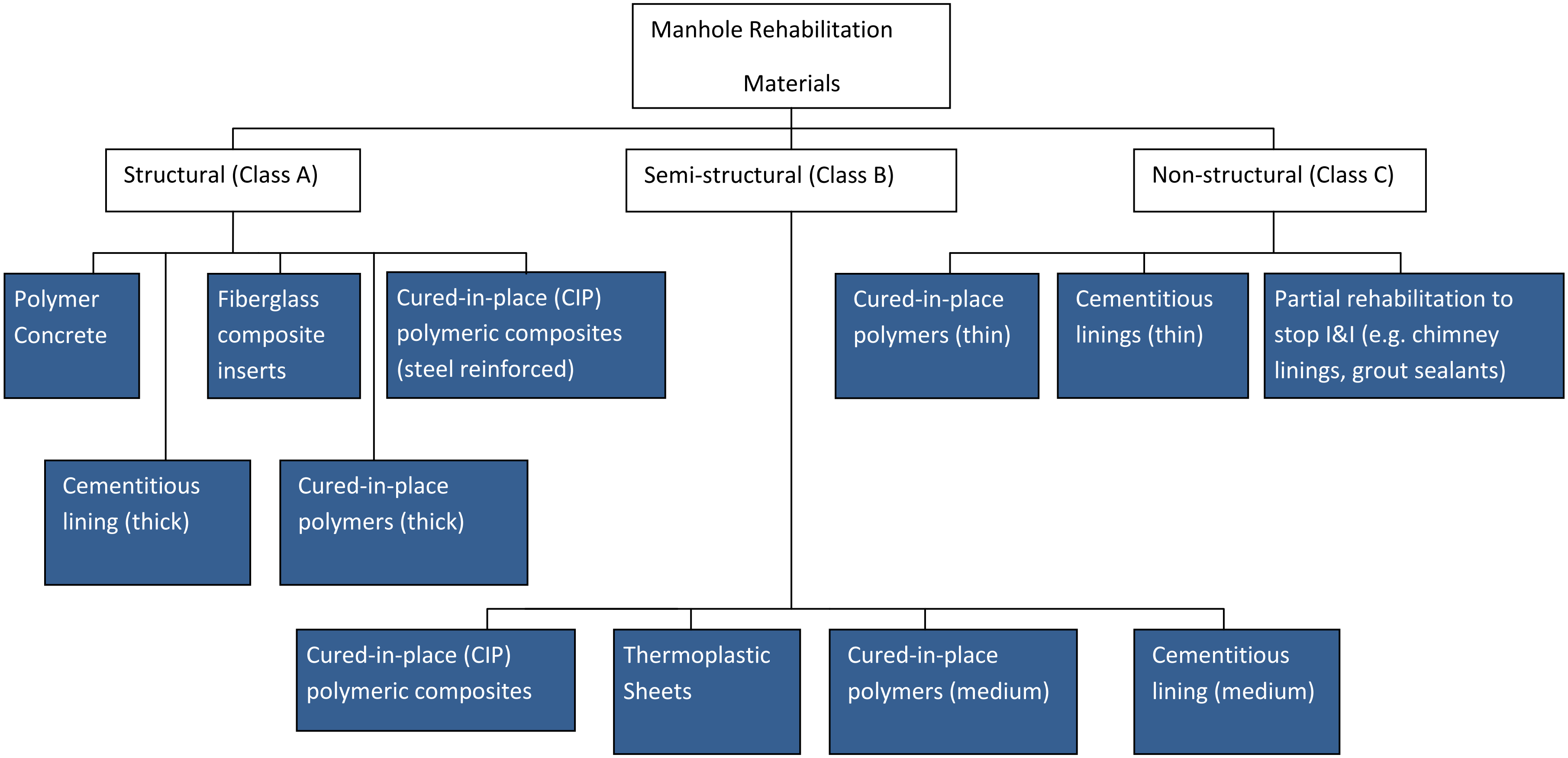


Figure 8-1. Manhole Rehabilitation Material Classification Based on Structural Capabilities.

Another note on the importance of application quality is that for a major structure, the contractor could be given days (if not weeks) to inspect and prepare the surface of the structure before applying any form of rehabilitation product, whereas, this time is very limited for each manhole as typically many of them are included under a single project to increase cost effectiveness. As such, it is imperative for the contractor to have an experienced and skilled crew that can deal with the various conditions from one manhole to the next, thereby having the flexibility to implement the necessary actions for a successful surface preparation and installation utilizing different types of tools (e.g., sand/water blasting machines, cutting tools, chemical and cementitious grouts).

The preliminary tests enabled the research team to determine whether some of the select and commonly used manhole rehabilitation materials would add any considerable strength at all to a concrete substrate. The results indicated that these materials can significantly, if not substantially, improve the flexural strength of concrete. The compressive strength of the concrete cylinders was also significantly increased by applying the select linings on them. Nevertheless, this observation from the preliminary tests was inconclusive due to the “confining effect” of the liners on the compression samples.

The main tests were designed based on the lessons learned from the preliminary tests. Therefore, they addressed the issues associated with applying the linings on a sample that is at least geometrically more representative of an actual manhole and monitoring deformation in compression for the unlined and lined samples so that effect of lining on the compressive strength could be determined. The main tests were implemented successfully, and were one of the major factors in developing the manhole rehabilitation classification presented in Figure 8-1.

Computational modeling with the FEM was used to project the smaller scale tests to full-scale manholes. The model was validated by simulating the preliminary and main tests. There was a good agreement between the test results and FEM simulations, which gave the team the confidence to further the model and simulate full-scale manholes and implement different scenarios, i.e., deteriorated liner condition (this is particularly the case for polymeric linings due to creep over time), direct hydrostatic pressure on the lining due to a gap/hole on the host structure, and reduced wall thickness (due to corrosion) on the host manhole. The full-scale FEM results suggest, if applied thick enough (greater than 300 mils or 8 mm) an epoxy lining can significantly enhance the structural integrity of a deteriorated manhole or even serve as a replacement depending on the site conditions. Nevertheless, the effect of polymeric linings on the compressive strength of a concrete host structure (manhole) is minimal due to the substantial difference between the compressive moduli of elasticity of these two materials (concrete and polymeric liner).

The Decision Support Tool (DST) developed as a part of this study provides a user-friendly interface to make the best decision on what type of material and method to use to rehabilitate each manhole. There is no silver bullet that would apply to all kinds of situations; and therefore, the DST uses more than 100 parameters that take manhole condition, loads, groundwater, soil condition, and construction into account. While this may seem like an onerous process to apply for just one manhole, it takes on average only approximately five minutes to enter the available information into the DST, and there is no minimum input requirement for the DST to return a recommendation.

Accordingly, the findings of this research project can be summarized as follows:

- Most manhole rehabilitation methods applied today are semi-structural.
- Fully structural (standalone) methods are not needed for the majority of the manholes.
- Semi- or non-structural rehabilitation could be efficient for I/I removal at lower cost.
- Application/surface preparation is of utmost importance. Same type of material (e.g., epoxy) can be classified as structural, semi-structural or non-structural depending on the thickness and application quality.
- Each manhole is different and there is no “silver bullet” solution; and therefore use of decision support tool is recommended.
- Sound engineering and thorough technical specifications are crucial in implementing a successful project.

APPENDIX A

SAMPLE TECHNICAL SPECIFICATION

INFR1R12 –STRUCTURAL CAPABILITIES OF NO-DIG MANHOLE REHABILITATION

PART 1: GENERAL

A. Description

1. The work specified in this section includes all labor, materials, accessories, equipment, and tools for performing operations required to rehabilitate sanitary sewer manholes to reduce infiltration and inflow (I&I) and restore structural integrity where needed.

B. Definitions

1. Manhole Rehabilitation: All of the materials and methods associated with improving the condition of a deteriorated manhole. Manhole rehabilitation methods include, but not limited to, cover and frame replacement, chimney repairs, sealing of cracks and fractures, and lining of manhole bench, walls, cone, and chimney.

2. Class A Lining: Fully structural manhole lining solutions suitable for severely deteriorated manholes. Class A linings shall stop infiltration and inflow (I&I) in addition to the capability of withstanding all the static and dynamic loads that are exerted on the manhole. Upon installation, Class A linings shall not rely on the residual strength of the host manhole.

   Table A-1 indicates the minimum mechanical and physical properties required for a Class A lining.

Table A-1. Minimum Mechanical and Physical Properties Required for Class A Lining.

Sample Lining Methods	Adhesive Strength (psi/kPa)	Compressive Strength (psi/kPa)	Flexural Strength (psi/kPa)	Min. Thickness (mils)
Cementitious Spray-on Polymer Polymer Sheet Geopolymer Polymer Concrete Fiberglass Cured-in-place (CIP)	To be completed by Project Engineer			

3. Class B Lining: Semi-structural manhole lining solutions suitable for moderately deteriorated manholes. Class B linings shall stop I&I in addition to providing structural support to the host structure. A manhole lined with a Class B lining shall withstand all the static and dynamic loads that are exerted on the manhole upon rehabilitation.

Table A-2 indicates the minimum mechanical and physical properties required for Class B lining.

Table A-2. Minimum Mechanical and Physical Properties Required for Class B Lining.

Sample Lining Methods	Adhesive Strength (psi/kPa)	Compressive Strength (psi/kPa)	Flexural Strength (psi/kPa)	Min. Thickness (mils)
Cementitious Spray-on Polymer Polymer Sheet Geopolymer Polymer Concrete Fiberglass Cured-in-place (CIP)	To be completed by Project Engineer			

4. Class C Lining/Coating: Non-structural manhole linings and coatings suitable for mildly deteriorated manholes or sound manholes that receive significant I&I. The purpose of using Class C manholes is to stop I&I and coating the interior surface of the manhole as a preventive measure versus corrosion. A manhole lined with Class C lining shall sustain no structural damage and withstand all the static and dynamic loads that are exerted on it without receiving any support from the lining. Class C linings should have a minimum of ___ psi adhesive strength and should be tested for adhesion upon application using ___.

C. Submittals

1. Furnish detailed and complete data pertaining to the interior surface of the structure to be rehabilitated, the rehabilitation product, surface preparation and installation to the Owner's Project Manager for approval. The submission of these data shall be made in a timely manner to prevent project delay. At the request of the Engineer, test for adverse chemical conditions that may hinder overall product performance.
2. Test Reports: Submit manufacturer's test reports of physical/chemical properties of the product.
3. A certificate of "Compliance with Specifications" shall be furnished for all materials supplied.
4. Documentation showing compliance with OSHA VOC emissions regulations.
5. A work plan including by-pass pumping.
6. A safety plan: Comply with OSHA standards and all regulations pertaining to the work including confined space entry.
7. Applicator Qualifications: Submit qualifications of applicator. Certification by the manufacturer stating that the applicator is trained and approved in the application of the specified products.
8. Material Safety Data Sheet (MSDS) for each product used (applies to coatings/linings and chemical grouts).

9. Final installation report on completed manholes including thickness measurements (applies to linings only).

D. Product and Applicator Acceptability

Because sewer products are intended to have a 50-year design life, and in order to minimize the Owner's risk, only proven products with successful long-term track records will be approved. All manhole lining products and installers must be pre-approved prior to receiving bid documents.

1. To be approved, Contractor shall have a minimum of ____ sq. ft. of manhole lining application experience in the state of ____.

PART 2: MATERIALS

E. General

1. The materials to be utilized in the lining of manholes shall be designed and manufactured to withstand the severe effects of hydrogen sulfide in a wastewater environment. Manufacturer of corrosion protection products shall have long proven experience in the production of the lining products utilized and shall have satisfactory installation record.

2. Equipment for installation of lining materials shall be high quality grade and be as recommended by the manufacturer.

3. The finished structure shall be corrosion resistant to: Hydrogen sulfide, 20% sulfuric acid, 17% nitric acid, 5% sodium hydroxide, as well as other common ingredients of the sanitary sewage environment.

F. Spray-Applied Polymeric Lining (e.g., Epoxy, Polyurethane, Polyurea)

<u>Description</u>: Rehabilitation of internally corroded manholes with high-build, polymeric liner. Polymeric linings shall be applied to the chimney in addition to the cone, wall, and bench.

1. The resin based material shall be used to form the sprayed on/structural enhanced monolithic liner covering all interior surfaces of the structure including benches and inverts of manholes. The finished liner will conform the thickness requirements per the selected structural class as indicated in Tables A-1 or A-2. The wall thickness of the resin based liner shall be structurally designed to withstand the hydraulic load generated by the groundwater table and restore structural integrity.

2. Spray-applied polymeric lining can be of the same base polymer or a composite of different polymeric layers.

G. Cementitious Linings

1. All cementitious lining materials shall be specifically designed for rehabilitation of manholes and other related wastewater structures. Liner materials are cement based, poly-fiber reinforced, shrinkage compensated, and enhanced with chemical admixtures and siliceous aggregates. Liner materials are mixed with water per manufacturer's written specifications and applied using equipment specifically designed for low-pressure spray, trowel, or centrifugal spin casting application of cement mortars. All cement liner materials must be capable of a placement thickness of 1/2" to 4" in a one pass monolithic application.

2. The finished liner shall conform the thickness requirements per the selected structural class as indicated in Tables A-1 or A-2. At a minimum, thickness of the cementitious liner shall be structurally designed to withstand the hydraulic load generated by the groundwater table and restore structural integrity.

   a. Portland Cement: Portland cement materials shall be manufactured from Type II Portland cement and enhanced with silica fume and high-density chemically stable aggregates. Materials must resist corrosion when placed in an environment capable of producing a maximum substrate pH level of 3.0.

   b. Calcium Aluminate: Calcium aluminate materials shall be manufactured from 100% pure calcium-aluminate cement and enhanced with silica fume and high-density chemically stable aggregates. Materials must resist corrosion when placed in an environment capable of producing a maximum substrate pH level of 2.0.

   c. Specialty Cementitious Lining Materials: Specialty cementitious lining materials are manufactured from Type II Portland cement with chemically activated fly ash, and enhanced with silica fume and high-density chemically stable aggregates. Materials must resist corrosion when placed in an environment capable of producing a substrate pH level of less than 2.0.

3. The cementitious lining shall have a polymeric top layer (e.g., epoxy) unless it is calcium aluminate or silica cement. Such top layer shall not be less than 80 mils (2 mm) thick.

4. Cementitious lining described herein includes applications with high wall thickness [2.0 inches (5 cm) or greater] to achieve more structural strength for highly deteriorated manholes. Thick cementitious linings shall be applied with using steel forms. Steel reinforcement can be added to achieve Class A rehabilitation per Table A-1.

H. Plastic Sheet Lining Systems

1. The plastic sheet lining sysem shall be made of PVC, polypropylene, polyethylene, or glass fiber reinforced plastic (FRP).

2. PVC sheet lining shall have locking extensions, a two part epoxy mastic intermittent layer and primer. The liner shall be continuous and free of pinholes both across the joints and in the liner itself.

3. Polypropylene and polyethylene linings shall be embedded in cement mortar grout in accordance with manufacturer's specifications.

4. Glass fiber reinforced linings sheets shall be attached to the manhole interior wall with bolts upon drilling holes through the liner and manhole wall per manufacturer's specifications.

5. The polymeric sheet linings shall have good impact resistance, shall be flexible and shall have an elongation sufficient to bridge up to ¼" (6 mm) settling cracks, which may occur in the concrete or in joints after installation without damage to the lining.

6. The liner shall be reparable anytime during the service life of the rehabilitated manhole.

7. To achieve Class A structural classification plastic sheet linings shall be applied along with steel reinforcement and Portland cement.

I. Fiber Reinforced Plastic (FRP) Inserts

1. Glass-Fiber Reinforced Polyester Manholes shall be a one-piece monolithic designed unit constructed of glass-fiber reinforced, supplier certified, unsaturated commercial grade polyester resin containing chemically enhanced silica to improve corrosion resistance, strength and overall performance. FRP manholes manufactured shall comply with ASTM D-3753.

2. To be considered Class A, the FRP manhole insert shall be manufactured in one class of load rating. This class shall be H-20 wheel load [minimum 16,000 pounds (71 kN) dynamic wheel load].

3. Resin: The resins used shall be unsaturated, supplier certified commercial grade polyester resins. Mixing lots of resin from different manufacturers or "odd-lotting" of resins shall not be permitted. Quality-assurance records on the resin shall be maintained. UV Inhibitors shall be added directly to resins to prevent photodegradation.

4. Reinforcing Materials: The reinforcing materials shall be commercial grade "E" type glass in the form of mat, continuous roving, chopped roving, roving fabric, or both, having a coupling agent that will provide a suitable bond between the glass reinforcement and the resin.

5. Surfacing Material: If reinforcing material is used on the surface exposed to the contained substance, it shall be a commercial grade chemical-resistant glass or organic surfacing mat having a coupling agent that will provide a suitable bond with the resin.

6. Fillers and Additives: Fillers, when used, shall be inert to the environment and manhole construction. Additives, such as thixotropic agents, catalysts, promoters, etc., may be added as required by the specific manufacturing process to be used to meet the requirements of this standard. However, calcium carbonate mixed by the fabricator shall not be permitted. The resulting reinforced plastic material must meet the requirement of this specification.

7. Laminate: The laminate shall consist of multiple layers of glass matting and resin. The surface exposed to the sewer/chemical environment shall be resin rich and shall have no exposed fibers.

J. Polymer Concrete Inserts

1. This section defines a resin aggregate manhole insert system such that manholes becomes structurally sounds, impervious to infiltration and inflow, and inert to hydrogen sulfide induced corrosion. Manhole rehabilitation riser sections shall be constructed of supplier-certified isophthalic polyester resin, sand and aggregate.

2. Acid resistant polymer manhole riser, cone sections and related components shall con form to the structural intent of ASTM C – 478. Riser sections and eccentric cones are provided with flush edge configurations assembled with the appropriate alignment guides gaskets and/or butyl mastic to make a continuous and uniform manhole.

3. Polymer manhole riser and cone sections are to be provided in various lengths in combination to provide correct height with the fewest joint.

4. To be considered Class A, the polymer concrete manhole sleeve shall have a minimum thickness of 2 inches (51 mm). Additional thicknessed may be required for larger manholes site specific load conditions.

5. The outside diameter of the polymer concrete sleeve shall be sized to leave 1.5 inches (38 mm) clearance from the manhole wall.

6. Polymer manholes will be designed based upon live and dead load criteria in ASTM C857.

7. Polymer Mixture the mixture shall consist solely of thermosetting resin, sand and aggregate. Resin used shall be unsaturated, certified, isophthalic polyester resins. Mixing lots of resin from different manufacturers is not allowed.

8. Manhole insert joints shall be of a flush edge design assembled per the alignment guides with gaskets and/or butyl mastic to make a continuous and uniform manhole

interior. Joint sealing surfaces shall be free of dents, gouges, and other surface irregularities that would affect joint integrity.

9. Each manhole component shall be free of all defects, including indentations, cracks, foreign inclusions and resin starved areas that, due to their nature and degree of extent, could detrimentally affect the strength and servicability of the component part. Variations in height of two opposite sides of risers and conical tops shall not be more than 5/8 inches (16 mm).

K. Cured-In-Place (CIP) Linings

1. The CIP lining defined herein refers to flexible tube liners (woven or non-woven) impregnated with a thermosetting resin (e.g. epoxy), which cures in the field upon installation, thereby taking the interior shape of the rehabilitated manhole.

2. The liner is vacuum-impregnated (saturated) on-site with the thermo-set resin. The saturatd liner is then lowered into the manhole, and is temporarily held in position. The installation device is then lowered and properly positioned inside of the liner. The bladder on the installation device is then pressurized so that the liner is pressed against the existing structure. Once the resin saturated liner is cured, the installation device is removed. The liner is then trimmed flush with the manhole cover seat.

3. The liner shall be continuous in length and consist of one or more layers of a stretchable absorbent textile material or fiberglass mat. The liner is designed to prevent I&I, withstand hydrostatic pressures, bridge missing mortar or brick segments, withstand freeze/thaw cycles, and conform to the contours of the existing structure. The saturated liner shall have uniform thickness, and have excess resin distribution that, when compressed at installation pressures, it will meet the design thickness upon completion of the curing process.

4. The finished liner shall conform the thickness requirements per the selected structural class as indicated in Tables B-1 or B-2. At a minimum, thickness of the CIP liner shall be structurally designed to withstand the hydraulic load generated by the groundwater table and restore structural integrity.

L. Partial (Chimney) Restoration

1. This section applies to the following conditions:

   a. Manholes that have apparent defects and/or inflow/infiltration through the chimney and require rehabilitation at this part only.

   b. Manholes lined with cementitious linings.

2. Cured-In-Place Manhole Lining (CIPMH) – Chimney

This type of chimney linings are essentially the same as CIP full-depth linings described above. The only difference is that the CIP chimney linings are designed to line the chimney area of the manhole only.

3. Internal Rubber Seal

Design Requirements – The manhole frame seal shall be designed to prevent leakage of water through the portions of the manhole described above throughout a 50-year design life.The seal shall also be designed so that it can be installed in manholes where the diameters of the frame and chimney differ by up to 20%.

Performance Requirements – The frame seal shall be capable of repeated vertical movement of not less than 2 inches and/or repeated horizontal movement of not less than ½ inch after installation and thorughout its design life.

Rubber Sleeve and Extension – The flexible rubber sleeve and extensions shall be extruded or molded from a high grade rubber compound conforming to the applicable material requirements of ASTM C923, with a mminimum of 1,500 psi (10,340 kPa) tensile strength, maximum 18% compression set and hardness (durometer) of 48±5.

The rubber sleeve shall be double, triple or quadruple pleated with a minimum unexpanded vertical height of 8 inches (200 mm), 10 inches (255 mm) or 13 inches (330 mm), respectively and a minimum thickness of 3/16 inches (5 mm). The top and bottom sections of the sleeve that compress against the manhole frame casting and the chimney/cone shall have an integrally formed expansion band recess and a series of sealing fins to facilitate and watertight seal.

The top section of the extension shall have a minimum thickness of 3/32 inches (2 mm) and shall be shaped to fit into the bottom band recess of the sleeve under the bottom chimney seal band and the remainder of the extension shall have a minimum thickness of 3/16 inches (5 mm). The bottom section of the extension shall contain integrally formed expansion band rcess and multiple sealing fins matching that of rubber sleeve. Any splice used to fabricate the sleeve and extension shall be hot vulcanized and have a strength such that the sleeve shall withstand a 180° bend with no visible separation.

Expansion Bands – The expansion bands used to compress the sleeve against the manhole shall be integrally formed from 16-gauge stainless steel conforming to the applicable material requirements of ASTM C923, Type 304, with no welded attachments and shall have a minimum width of 1.75 inches (44 mm).

The bands shall have a minimum adjustment range of 2.5 inches (64 mm) and the mechanism used to expand the band shall have the capacity to develop the pressures necessary to make a watertight seal. The band shall be permanently held in place with a positive locking mechanism, which secures the band in its expanded position after tightening.

4. Polyurethane Based Rubber Seal

Deteriorated manhole chimneys shall be sealed with two-part elastomeric polyurethane based rubber with 200 mils (5 mm) thickness throughout the substrate. Physical properties shall be as provided below.

Table A-3. Top Coat Properties.

Report	Tet Method	Test Results
Weight (as applied)	ASTM E-201	9.07 lbs. gal
Specific Gravity	ASTM D-792	1.09
Solids (by weight)	ASTM D-2369	100%
Solids(by weight as applied)	ASTM D-2369	71%
Hardness, Shore "A"	ASTM D-2240	75$\pm$5
Elongation (Ultimate) Elongation (as applied)	ASTM-D412	850% $\pm$ 50 335% $\pm$ 10
Tensile Strength	ASTM D-412	2000 $\pm$ 50
Adhesive Strength	ASTM D-903	(See Primer)
Tear Resistance, Die C	ASTM D-624	300 $\pm$10
Temperature Service Range	Fed Std. 141 Method 6223	-65 to 200° F
Water Absorption	ASTM D-471	<0.05% by weight
Negative Air Pressure (Vacuum) Test	ASTM C-1244	5minutes@10 inches (254mm)
Weatherability (Weather-Ometer) – 500 hrs	ASTM D-822	Slight Color Change
Flash Point (Pensky-Martens Closed Cup)	ASTM D-93	Non-flammable >212°F (100C)

Table A-4. Primer Properties.

Report	Test Method	Test Results
Weight (as applied)	ASTM E-201	8.72 lbs/gal
Specific Gravity	ASTM D-792	1.045
Solids (by weight)	ASTM D-2369	91.37%
Hardness, Shore "A"	ASTM D-2240	85 ±5
Ultimate Elongation	ASTM D-412	650 ±50
Tensile Strength	ASTM D-412	3200 ±50 psi
Adhesive Strength	Elcometer 109	>700psi(5MPa)on steel >700psi(5Mpa)on concrete
Tear Resistance, Die C	ASTM D-624	325 ± 10
Temperature Service Range	FedStd. 141 Method 6223	-65 to 200° F.
Water Absorption	ASTM D-471	<0.03% by weight
FlashPoint(Pensky-Martens Closed Cup)	ASTM-D93	Non-flammable >212°F (100C)

M. Frame and Cover Retrofitting

Retrofit Manhole frame and cover in lieu of replacement for reparable manholes. Include coating of the surface to prevent further rusting/corrosion and inserting an inflow dish to prevent inflow.

1. Coating materials to be used for manhole frames and covers shall be in compliance with Section ____.

2. The inflow dish body shall be manufactured from polyethylene or stainless steel material. The polyethylene inflow dish shall comply with UL Standard, 94-HB, and meet ASTM D3350. Dish thickness shall be uniform and minimum 1/8-inch. Fabricate inflow dish body with molded ribbing members in bowl area for structural integrity. Inflow dish shall have smooth radius molded edges for additional strength and prevention of cracking. Inflow dish shall have manufacture date (month and year) permanently molded in dish body for future warranty identification.

PART 3: EXECUTION

To be written per the selected material manufacturer's recommended procedures.

APPENDIX B

EXPERT WORKSHOP ATTENDEE LIST

Name	Organization	Type of Organization
Lake Barrett	Sauereisen	Manufacturer
Steve Bass	Sewerage &Water Board of New Orleans	Wastewater Utility
John Calise*	Benton & Associates	Engineering/Consulting
Dan Cook	AW Cook Cement	Contractor
Jason Crain	Sigma Cons. Group	Engineering/Consulting
Sahar Hasan	LMK Technologies	Manufacturer
Larry Kiest	LMK Technologies	Manufacturer
Dorcas Hermes	WBE Dorcas	Manufacturer
Abhay Jain*	Univ. of Texas/CUIRE	University
Mohammad Najafi*	Univ. of Texas/CUIRE	University
Amy Schulze	City of Baton Rouge	Wastewater Utility
Frank Crumb	City of Ft. Worth	Wastewater Utility
Moosa Damerschie	AP/M Permaform	Manufacturer
Eric Dickson	Cretex	Manufacturer
Alexandra Ellis	Southwestern	Contractor
Aaron Hoffman	Raven Lining	Manufacturer
John Manijak	National Power Rodding	Contractor
Gerry Muenchemeyer	National Association of Sewer Service Companies (NASSCO)	Trade Organization
Dan Murray	U.S. EPA	Federal Government
B. Oberti	Terre Hill Composites	Manufacturer
Carmen Scalise	Metropolitan Water Reclamation District of Greater Chicago	Wastewater Utility
Albert Sealy	Stopaq	Manufacturer
Firat Sever*	Benton & Associates (now American Structurepoint)	Engineering/Consulting
Bill Shook	AP/M Permaform	Manufacturer
Bruce Snyder	Sherwin Williams	Manufacturer
Andrew Stone	UMC-Wildcat	Contractor
Dan Warren	Warren Environmental	Manufacturer
Ross Williams	Advantage Manhole	Contractor
Charles Wilmut	Burgess & Niple	Engineering/Consulting
Renn Zhao	DC Water	Wastewater Utility

**Project team member.*

APPENDIX C

PRELIMINARY TEST DETAILS

EPX1-F – Beam Specimen

Before each test, the entire specimen was carefully observed with a magnifier. No visible cracks or holes were observed in the concrete or liner of this specimen. The liner seemed to be perfectly attached to the surface of the specimen. After the appropriate measurements (Table B-1) specimen was setup in the test machine (Figure C-1).

Specimen One (EPX1-F#1)

Load was applied to the specimen with a constant rate of 4 lbs./sec (18 N/s). The first specimen failed at a peak load of 4,251 lbs. (18.9 kN), and peak stress of 2,040 psi (14,065 kPa). Failure crack line was inclined towards the bottom with respect to the center of the specimen.

Figure C-1. EPX1-F#1 Mounted on the Baldwin 60 KIP for the Flexure Test.

Specimen Two (EPX1-F#2)

The liner of this specimen was damaged during the shipping. A small section of the liner was chipped off the concrete substrate (Figure C-2). No visible cracks were seen on the specimen, but some small holes were observed.

Figure C-2. Liner Pieces Chipped off EPX1-F#2 During Shipping.

The specimen was setup in the machine after the appropriate measurements were made (Table C-1). The test was completed by application of a constant load rate around 4 lbs./sec (18 N/s). At the time of failure, the load was 2,109 lbs. (9 kN), with a peak stress of 1,093 psi (7,536 kPa). The failure plane was not as inclined as the first specimen (Figure C-3).

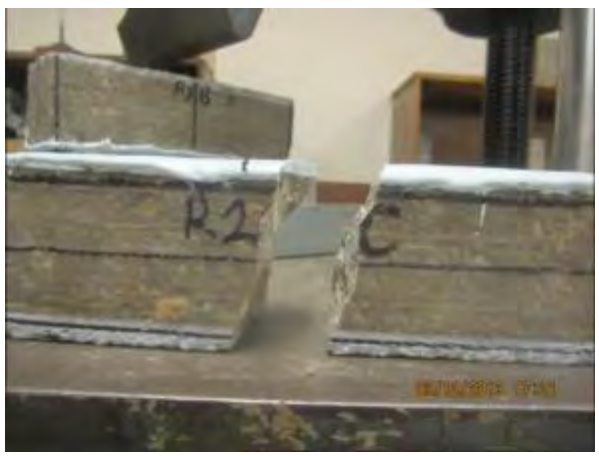


Figure C-3. Failure of Specimen EPX1-F#2.

Specimen 3 (EPX1-F#3)

No cracks or holes were observed in this specimen. The liner thickness varied in some parts of the surface of the specimen (Figure C-4). Specimen was measured and setup in the testing machine. The specimen reached failure when the peak load was 4,500 lbs., and the peak stress was 2,250 psi. Shape of the crack at failure was inclined towards the base with respect to the center of the beam (Figure C-5).

Figure C-4. Cross-Section of EPX1-F#3, Note the Variation in Lining Thickness Across the Specimen.

Figure C-5. Specimen EPX1-F#3 at Failure, Note the Inclination in the Failure Plane.

Specimen Four (EPX1-F#4)

The specimen was observed carefully and pinholes were seen in the concrete surface and lining (Figure C-6). The specimen was measured and setup in the machine. Loading was applied to the specimen with a constant rate until the specimen reached failure at a peak load of 2,318 lbs. (10.3 kN) with a peak stress of 1,151 psi (7,936 kPa). The failure plane was not as inclined as the other specimens (Figure C-7).

a b

Figure C-6. Pinholes in the Concrete Substrate (a) and Liner (b).

Figure C-7. Specimen EPX1-F#4 at Failure.

Specimen Five (EPX1-F#5)

This specimen also had some visible pinholes on the surface of the concrete. After the measurements and setup the load was applied at a constant rate and failure was reached in the peak load of 3,590 lbs and the stress of 1,755 psi. The failure plane was more inclined in comparison with the other specimens (Figure C-8).

Figure C-8. Failure Pattern of EPX1-F#5.

Summary of Results

Tables C-1 and C-2 indicate summaries of the results of the flexural strength tests applied on the high-build epoxy (EPX1-F) lined specimens. The average ultimate strenght for the three control specimens and five lined specimens were 917 psi (6,322 kPa) and 1,658 psi (11,432 kPa), respectively. Nevertheless, the standard deviation for the flexural strength test results on EPX1-F was high (520 psi for ultimate flexural strength).

Table C-1. Summary of the Preliminary Flexure Test Results for the EPX1-F Beam Specimens.

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross-Sectional Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
R1 (EPX1#1)	W_1=80 W_2 =81 W_3=77 W_{ave}=79 H_1=86 H_2=86 H_3=83 H_{ave}=84 L_1=279 L_2=278 L_3=278 L_{ave}=278	W_1=3.1 W_2 =3.2 W_3=3.0 W_{ave}=3.1 H_1=3.4 H_2=3.3 H_3=3.3 H_{ave}=3.2 L_1=10.9 L_2=11.0 L_3=11.0 L_{ave}=10.9	7.7	304	03/12/2013	ASTM C293	6,669	10.1	18	4	18.91	4,251

WERF

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross-Sectional Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
R2 (EPX1#2)	W_1=78 W_2 =81 W_3=75 W_{ave}=78 H_1=82 H_2=81 H_3=80 H_{ave}=81 L_1=278 L_2=276 L_3=277 L_{ave}=277	W_1=3.1 W_2 =3.2 W_3=3.0 W_{ave}=3.1 H_1=3.2 H_2=3.2 H_3=3.2 H_{ave}=3.2 L_1=10.9 L_2=10.8 L_3=10.8 L_{ave}=10.8	5.6	220	03/12/2013	ASTM C293	6,347	9.6	18	4	9.38	2,109
R3 (EPX1#3)	W_1=83 W_2 =82 W_3=82 W_{ave}=82 H_1=81 H_2=82 H_3=81 H_{ave}=81 L_1=276 L_2=277 L_3=277 L_{ave}=277	W_1=3.3 W_2 =3.0 W_3=3.2 W_{ave}= 3.2 H_1=3.2 H_2=3.2 H_3=3.1 H_{ave}=3.2 L_1=11.0 L_2=10.9 L_3=10.9 L_{ave}=10.9	5.7	226	03/11/2013	ASTM C293	6,677.31	9.92	18	4	20.02	4,500

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross-Sectional Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
R5 (EPX1#5)	W_1=82. W_2 =83 W_3=79 W_{ave}=81 H_1=81 H_2=86 H_3=85 H_{ave}=84 L_1=276 L_2=278 L_3=279 L_{ave}=278	W_1=3.2 W_2=3.2 W_3=3.1 W_{ave}=3.0 H_1=3.2 H_2=3.4 H_3=3.3 H_{ave}=3.3 L_1=10.8 L_2=10.8 L_3=10.9 L_{ave}=10.8	5.5	216	03/12/2013	ASTM C293	6,824	10.1	18	4	15.97	3,590

CIP-F – Beam Specimen

Before each test, each specimen was carefully observed with a magnifier. No visible cracks or holes were observed in the concrete or liner specimen. The liner seemed to be perfectly attached to the surface of the specimen. After the appropriate measurements (Table C-2), each specimen was setup in the test machine (Figure C-9). Load was applied to the lined concrete specimen with a constant rate of 4 lbs./sec (17.8 N/s).

Specimen One (CIP-F#1)

The first specimen failed at a peak load of 2,416 lbs. (10.75 kN), and a peak stress of 1,303 psi (8,984 kPa). Failure crack line was inclined with respect to the center of the specimen (but not as much as the EPX1-F specimens (Figure C-9). The top surface of the specimen was not broken at failure (Figure C-10). Also, the liner did not break, but a thin white line along the crack can be seen due to the stretching of fiber (Figure C-10b).

a b

Figure C-9. Specimen Setup (a) and CIP-F#1 at Failure (b).

a b

**Figure C-10. CIP-F#1 Top Surface Without Any Cracks After Failure (a).
Crack Along the Liner on the Bottom Surface of the Specimen (b).**

Specimen Two (CIP-F#2)

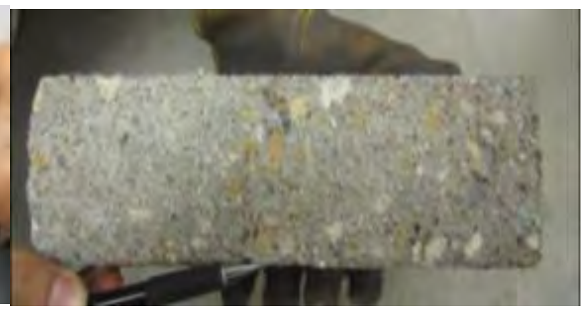
The second specimen failed at a peak load and peak stress of 2,201 lbs. (9.79 kN) and 1,165 psi (8,032 kPa), respectively. Failure crack line was at the center of the specimen. Unlike CIP-F#1, the top surface of the specimen was broken at failure. However, the liner did not crack as thoroughly as the substrate did (Figure C-11).

Figure C-11. CIP-F#2 at Failure.

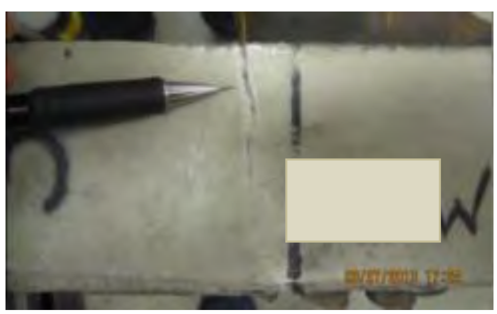
Specimen Three (CIP-F#3)

The third specimen reached failure when the peak load was 2,607 lbs. (11.60 kN) and the peak stress was 1,355 psi (9,342 kN). The crack was located in the center of the beam (Figure C-12a). The top surface of the beam did not crack (Figure C-12b). The liner was broken approximately at 1 in. (25 mm) off the center line (Figure C-12c).

a



b



c

Figure C-12. Specimen CIP-F#3 at Failure (a).
Top View After Failure (No Visible Cracks on the Concrete) (b).
Fracture on the Liner (c).

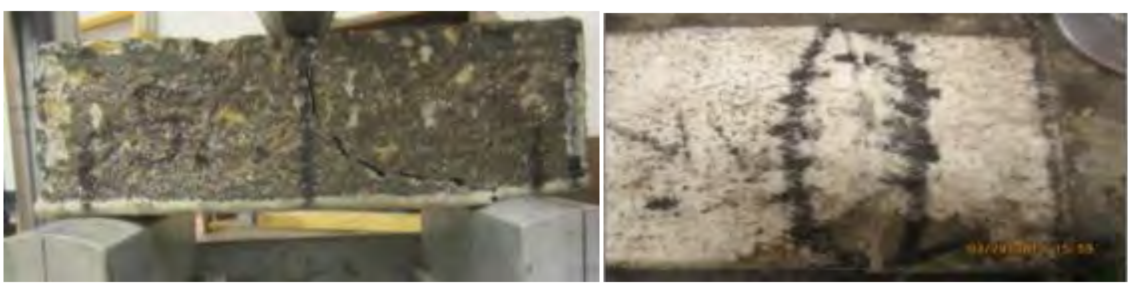
Specimen Four (CIP-F#4)

Loading was applied to the specimen with a constant rate until the specimen reached failure in the peak load of 2,503 lbs. (11.13 kN) with a peak stress of 1,067 psi (7,357 kPa). The failure crack in the concrete was slightly inclined with respect to the centerline. However locations of the cracks in the concrete and liner were different from each other (Figure C-13).

Figure C-13. Specimen CIP-F#4 at Failure.

Specimen Five (CIP-F#5)

After observation of this specimen and measurments, it was setup in the test machine, the load was applied at a constant rate and failure was reached at the peak load and stress of 4,865 lbs (21.64 kN) and 2,068 psi (14,258), respectively. The failure crack was centered at top and it inclined as it moved downward (Figure C-14a). The liner broke at three inches from the end of the specimen (Figure C-14b).



a b

Figure C-14. Failure Plane in CIP-F#5(a) and Fracture Along the Liner (b).

Summary of Results

Tables C-6 and C-7 indicate a summary of the results of the flexural strength tests applied on the cured-in-place (CIP) lined specimens. The average ultimate strength for the three control specimens and five lined specimens were 917 psi (6,322 kPa) and 1,393 psi (9,607 kPa), respectively. Results obtained from the first four specimens were close to each other, however the fifth specimen showed a much higher strength than other specimens.

Table C-2. Summary of the Preliminary Flexure Test Results for the CIP Beam Specimens.

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
CIP#1	W_1=77 W_2 =79 W_3=79 W_{ave}=78 H_1=77 H_2=79 H_3=81 H_{ave}=79 L_1=276 L_2=278 L_3=279 L_{ave}=278	W_1=3.0 W_2 =3.1 W_3=3.1 W_{ave}=3.1 H_1=3.0 H_2=3.1 H_3=3.2 H_{ave}=3.1 L_1=10.6 L_2=10.8 L_3=10.9 L_{ave}=10.8	4.6	182	03/27/2013	ASTM C293	6,203	9.62	18	4	10.75	2,416
CIP#2	W_1=77 W_2 =79 W_3=79 W_{ave}=78 H_1=80 H_2=80 H_3=80 H_{ave}=80 L_1=277 L_2=278 L_3=278 L_{ave}=278	W_1=3.0 W_2 =3.1 W_3=3.1 W_{ave}=3.1 H_1=3.1 H_2=3.2 H_3=3.1 H_{ave}=3.1 L_1=10.8 L_2=10.8 L_3=10.8 L_{ave}=10.8	4.1	161	03/27/2013	ASTM C293	6,278	9.73	18	4	9.79	2,201

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm²	in.²	N/s	lbs./sec	kN	lbs.
CIP#3	W_1=77 W_2 =79 W_3=80 W_{ave}=79 H_1=80 H_2=80 H_3=81 H_{ave}=80 L_1=276 L_2=277 L_3=278 L_{ave}=277	W_1=3.0 W_2 =3.1 W_3=3.2 W_{ave}= 3.1 H_1=3.1 H_2=3.2 H_3=3.2 H_{ave}=3.2 L_1=10.6 L_2=10.8 L_3=10.8 L_{ave}=10.7	4.9	193	03/29/2013	ASTM C293	6,324	9.80	18	4	11.59	2,607
CIP#4	W_1=79 W_2 =79 W_3=78 W_{ave}=79 H_1=83 H_2=83 H_3=78 H_{ave}=81 L_1=278 L_2=279 L_3=279 L_{ave}=279	W_1=3.1 W_2=3.1 W_3=3.1 W_{ave}=3.1 H_1=3.2 H_2=3.3 H_3=4.0 H_{ave}=3.5 L_1=10.8 L_2=10.9 L_3=10.9 L_{ave}=10.9	4.5	179	03/29/2013	ASTM C293	6,375	10.84	18	4	11.13	2,503

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
CIP#5	W_1=80 W_2 =79 W_3=79 W_{ave}=80 H_1=88 H_2=88 H_3=88 H_{ave}=88 L_1=278 L_2=276 L_3=277 L_{ave}=277	W_1=3.2 W_2=3.1 W_3=3.2 W_{ave}=3.1 H_1=3.4 H_2=3.5 H_3=3.4 H_{ave}=3.5 L_1=10.8 L_2=10.6 L_3=10.6 L_{ave}=10.7	7.0	276	03/29/2013	ASTM C293	6,989.72	10.87	18	4	21.64	4,865

PU – Beam Specimen

Prior to each test, the entire surface of each specimen was carefully observed with a magnifier. The liner was adhered perfectly to the surface of the specimen. After the appropriate measurements, (Table C-3) each specimen was set up in the test machine (Figure C-15). The number used in the each specimen label does not coincide with the specimen quantity; because, some of the PU specimens were excluded from testing due to extensive damage during shipping. Load was applied to the lined concrete specimen with a constant rate of 4 lbs./sec (18 N/s) based on ASTM C293 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading). The test procedure and results are discussed below.

Figure C-15. Polyurethane (PU) Beam Specimen Mounted on the Baldwin 60 KIP for the Flexure Test.

Specimen One (PU-F#1)

The specimen failed at a peak load of 5,213 lbs. (23.19 kN) and peak stress of 2,363psi (16,292 kPa). Failure plane in the concrete substrate was inclined with respect to the center of the specimen. Unlike the concrete substrate, the liner cracked and was broken at the support (Figure C-16).

Figure C-16. Specimen PU-F#1 Liner Failure at the Support.

Specimen Two (PU-F#3)

The second specimen failed at a peak load of 7,382 lbs. (32.83 kN) and peak stress of 3,283 psi (22,635 kPa). Failure crack line was inclined with respect to the center of the specimen (Figure C-17a). Failure plane in the concrete substrate was inclined with respect to the center of the specimen. As was the case for PU-F#1, the liner cracked and was broken at a different location (along the support) than the concrete substrate (Figure C-17b).

a b

Figure C-17. Fracture in the Concrete Substrate (a) and PU-F#3 Liner Failure at the Support (b).

Specimen Three (PU-F#5)

The specimen failed at a peak load of 6,989 lbs. (31.08 kN) and peak stress of 3,093 psi (21,325 kPa). Failure crack line in the concrete substrate was inclined with respect to the center of the specimen (Figure C-18a). As was the case for PU-F#1 and PU-F#3, the liner cracked and was broken at a different location (along the support) than the concrete substrate (Figure C-18b).

a b

Figure C-18. Fracture in the Concrete Substrate (a) and PU-F#5 Liner Failure at the Support (b).

Specimen Four (PU-F#6)

The fourth specimen failed at a peak load of 6,728 lbs. (29.92 kN) and peak stress of 2,406 psi. The cracking and fracture advanced toward the support (Figure C-19a) and finally a thin section of concrete substrate broke off with the polyurethane lining on it (Figure C-19b).

a b

Figure C-19. Specimen PU-F#6 at Failure (a) with a Thin Section Separating Near the Support (b).

Specimen Five (PU-F#7)

The fifth specimen failed at a peak load of 5,514 lbs. (24.52 kN) and peak stress of 2,342 psi (16,148 kPa). The failure crack line was inclined with respect to the centerline of the specimen (Figure C-20a). At failure, one of the broken sections of the concrete sheared off the liner and separated (Figure C-20b). Unlike the other four specimens, the liner was detached from the substrate, but it was intact at the failure of the concrete substrate.

a b

Figure C-20. Specimen PU-F#7 at Failure (a) and Detached Liner from the Concrete Substrate upon Failure.

Summary of Results

Table C-3 indicates a summary of the results of the flexural strength tests applied on the polyurea (PU) lined specimens. The average ultimate strenght for the three control specimens and five lined specimens were 917 psi (6,322 kPa) and 2,644 psi (18,230 kPa), respectively.

Table C-3. Summary of the Preliminary Flexure Test Results for the PU Beam Specimens.

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm²	in.²	N/s	lbs./sec	kN	lbs.
PU-F#1	W_1=84 W_2 =89 W_3=85 W_{ave}=86 H_1=72 H_2=89 H_3=89 H_{ave}=83 L_1=279 L_2=279 L_3=278 L_{ave}=279	W_1=3.3 W_2 =3.5 W_3=3.4 W_{ave}=3.4 H_1=2.8 H_2=3.5 H_3=3.5 H_{ave}=3.3 L_1=11.0 L_2=11.0 L_3=10.9 L_{ave}=11.0	7.4	290	04/22/2013	ASTM C293	7,145	11.07	18	4	23.19	5,213
PU-F#5	W_1=78 W_2 =83 W_3=84 W_{ave}=82 H_1=87 H_2=87 H_3=85 H_{ave}=86 L_1=278 L_2=279 L_3=280 L_{ave}=279	W_1=3.1 W_2 =3.3 W_3=3.3 W_{ave}=3.2 H_1=3.4 H_2=3.4 H_3=3.3 H_{ave}=3.4 L_1=11.0 L_2=11.0 L_3=11.0 L_{ave}=11.0	7.9	309	04/22/2013	ASTM C293	7,066	10.95	18	4	31.08	6,989

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm²	in.²	N/s	lbs./sec	kN	lbs.
PU-F#6	$W_1=82$ $W_2=92.4$ $W_3=86.0$ $W_{ave}=86.9$ $H_1=93.2$ $H_2=93.3$ $H_3=92.6$ $H_{ave}=93.0$ $L_1=278$ $L_2=279$ $L_3=278$ $L_{ave}=278$	$W_1=3.2$ $W_2=3.6$ $W_3=3.4$ $W_{ave}=3.4$ $H_1=3.7$ $H_2=3.7$ $H_3=3.6$ $H_{ave}=3.7$ $L_1=10.9$ $L_2=11.0$ $L_3=10.9$ $L_{ave}=10.9$	8.7	344	04/22/2013	ASTM C293	8,081	12.52	18	4	29.92	6,728
PU-F#3	$W_1=80$ $W_2=84$ $W_3=79$ $W_{ave}=81$ $H_1=84$ $H_2=89$ $H_3=86$ $H_{ave}=82$ $L_1=278$ $L_2=277$ $L_3=278$ $L_{ave}=278$	$W_1=3.1$ $W_2=3.3$ $W_3=3.1$ $W_{ave}=3.2$ $H_1=3.3$ $H_2=3.5$ $H_3=3.4$ $H_{ave}=3.4$ $L_1=10.9$ $L_2=10.9$ $L_3=10.9$ $L_{ave}=10.9$	7.4	290	04/22/2013	ASTM C293	6,609	10.81	18	4	32.83	7,382

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
PU-F#7	W_1=79 W_2 =79 W_3=81 W_{ave}=81 H_1=89 H_2=89 H_3=88 H_{ave}=84 L_1=278 L_2=278 L_3=277 L_{ave}=278	W_1=3.1 W_2=3.1 W_3=3.2 W_{ave}=3.1 H_1=3.5 H_2=3.5 H_3=3.5 H_{ave}=3.5 L_1=10.9 L_2=10.9 L_3=10.8 L_{ave}=10.9	8.9	350	04/22/2013	ASTM C293	6,824	10.95	18	4	24.52	5,514

CMP – Beam Specimen

Prior to each test, the entire surface of each specimen was carefully observed with a magnifier. No hole or crack was observed in the samples. Since one of the beam samples was broken in the lining process at the manufacturer's facility, only four beams were available for the flexural testing. Based on the observations prior to testing, the liner was attached perfectly to the surface of the specimen. After the appropriate measurements, each specimen was set up in the test machine. Load was applied to the lined concrete specimen with a constant rate of 4 lbs./sec (18 N/s) based on ASTM C293 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading).

Figure C-21. Specimen CMP-F Mounted on the Test Setup.

Specimen One (CMP-F#1)

The specimen failed at a peak load of 2,205 lbs. (9.80 kN) and peak stress of 1,175 psi (81,101 kPa). The crack line started from the centerline at the top and continued inclined with respect to the centerline of the specimen. At failure, one of the broken sections of the concrete sheared off from the liner and separated.

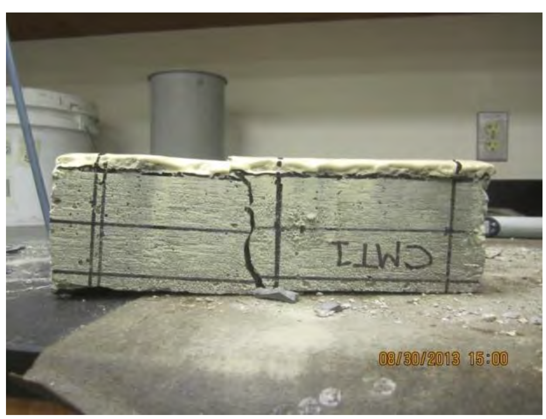


Figure C-22. Specimen CMT1#1 at Failure.

Specimen Two (CMP-F#2)

The specimen failed at a peak load of 2,551 lbs. (11.34 kN) and peak stress of 1,360 psi (9,377 kPa). The failure crack line was located at the center of the beam along the height of the beam. At failure the liner was attached to the surface of the concrete beam (Figure C-23).

a b

Figure C-23. Specimen CMP-F#2 Mounted on the Flexure Test Setup (a) and at Failure (b).

Specimen Three (CMP-F#3)

The specimen reached failure when the peak load was 2,390 lbs. (10.63 kN) and the peak stress was 1,274 psi (8,784). The crack line was located at the center of the beam at the top, but inclined at the very bottom of the specimen with respect to the center line (Figure C-24). The liner was perfectly attached to the concrete part at failure (Figure C-25).

Figure C-24. CMP-F#3 After Tested to Failure, Note the Crack (Fracture) Line.

Figure C-25. CMP-F#3 at Failure.

Specimen Four (CMP-F#4)

The specimen failed at a peak load of 2,250 lbs. (10.01 kN) at peak stress of 1,204 psi (8,301 kPa). The crack line was located at the center line of the beam. The liner was attached to the concrete part at failure.

Figure C-26. CMP-F#4 at Failure.

Summary of Results

Table C-4 indicates a summary of the results of the flexural strength tests applied on the cement mortar-epoxy composite (CMP) lined specimens. The failure line in samples started at the center of the concrete beam where the concentrated load was applied on top of the specimen and advanced vertically through the bottom of the specimen, where the concrete is lined with cement mortar-epoxy composite material. The average ultimate strength of the three control specimens and five lined specimens were 917 psi (6,322 kPa) and 1,253 psi (8,640 kPa), respectively.

Table C-4. Summary of the Preliminary Flexure Test Results for the CMP Beam Specimens.

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs/sec	kN	lbs
CMP-F#1	W_1=81 W_2=81 W_3=82 W_{ave}=81 H_1=85 H_2=87 H_3=88 H_{ave}=87 L_1=279 L_2=277 L_3=275 L_{ave}=277	W_1=3.2 W_2=3.1 W_3=3.3 W_{ave}=3.2 H_1=3.4 H_2=3.4 H_3=3.5 H_{ave}=3.4 L_1=11.0 L_2=10.9 L_3=10.8 L_{ave}=10.9	3.6	142	08/15/13	ASTM C293	7,118.5	10.54	18	4	9.80	2,205

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs/sec	kN	lbs
CMP-F#2	W_1=82 W_2 =83 W_3=83 W_{ave}=82 H_1=79 H_2=79 H_3=79 H_{ave}=79 L_1=276 L_2=276 L_3=276 L_{ave}=276	W_1=3.2 W_2 =3.3 W_3=3.3 W_{ave}=3.3 H_1=3.1 H_2=3.1 H_3=3.1 H_{ave}=3.1 L_1=10.9 L_2=10.9 L_3=10.9 L_{ave}=10.9	3.3	130	09/16/13	ASTM C293	6,493	10.04	18	4	11.34	2,551
CMP-F#3	W_1=84 W_2 =84 W_3=83 W_{ave}=84 H_1=78 H_2=79 H_3=80 H_{ave}=79 L_1=280 L_2=280 L_3=282 L_{ave}=281	W_1=3.3 W_2 =3.3 W_3=3.3 W_{ave}= 3.3 H_1=3.1 H_2=3.1 H_3=3.2 H_{ave}=3.1 L_1=11.0 L_2=11.0 L_3=11.1 L_{ave}=11.0	3.5	138	09/16/13	ASTM C293	6,620.7	10.26	18	4	10.63	2,390

Beam Sample Number	Sample Dimensions W*H*T		Lining Thickness		Test Date	Test Type	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm²	in.²	N/s	lbs/sec	kN	lbs
CMP-F#4	W_1=79 W_2 =80 W_3=79 W_{ave}=79 H_1=81 H_2=82 H_3=81 H_{ave}=81 L_1=278 L_2=280 L_3=282 L_{ave}=280	W_1=3.1 W_2=3.1 W_3=3.1 W_{ave}=3.1 H_1=3.2 H_2=3.2 H_3=3.2 H_{ave}=3.2 L_1=11.0 L_2=11.0 L_3=11.1 L_{ave}=11.0	3.4	134	09/16/13	ASTM C293	6,415	9.92	18	4	10.01	2,250

EPX1 – Cylinder Specimens

All of the specimens were observed carefully with a magnifier prior to testing. No visible cracks or holes were observed in the concrete or liner on any of the specimens. The liner seemed to be firmly adhered to the surface of the specimen. Minor changes in thickness of the liner were noted. After the appropriate measurements, (Table C-5) the specimen was capped using sulfur capping material three days before the test. The specimen was setup in the 500-kip compression test machine (Figure C-27). Load was applied to the specimen with a constant rate of 100 lbs./sec (445 N/s). Observations and test results for each cylinder specimen are discussed below.

Figure C-27. Lined Cylinder Specimen Mounted on the Admet 500 Kip Compression Test Machine.

Specimen One (EPX1-C#1)

The first specimen failed at a peak load of 85,520 lbs., and peak stress of 6,229 psi. At failure, the specimen fractured into three pieces along the circumference and height. The concrete core failed in a cone shape (Figure C-28).

a **b**

Figure C-28. Specimen EPX1-C#1 at Failure – Top View (a) and Side View (b).

Specimen Two (EPX1-C#2)

The specimen failed at a peak load of 80,730 lbs., and peak stress of 5,967 psi. At failure, the specimen broke into three pieces along the length with a concrete core inside failing in a cone shaped mode (Figure C-29).

a

b

Figure C-29. Specimen EPX1-C#2 at Failure (a) with a Cone-Shaped Fracture Pattern in the Concrete Core (b).

Specimen Three (EPX1-C#3)

The specimen failed at a peak load of 75,800 lbs., and peak stress of 5,457 psi. The specimen failed in a cone shaped pattern in the concrete core (Figure C-30).

Figure C-30. Specimen EPX1-C#3 Failure Pattern.

Specimen Four (EPX1-C#4)

The specimen failed at a peak load of 82,510 lbs., and peak stress of 5,881 psi. Figure C-31a presents the circumferential failure of the specimen. In the concrete core, the specimen failed in a cone shaped pattern (Figure C-31b).

a b

Figure C-31. Specimen EPX1-C#4 at Failure (a) with a Cone-Shaped Fracture Pattern in the Concrete Core (b).

Specimen Five (EPX1-C#5)

The specimen failed at a peak load of 64,940 lbs., and peak stress of 4,632 psi. As was the case for the other EPX1-C specimens, the specimen failed in a cone shaped pattern in the concrete core with the lining fracturing vertically (Figure C-32).

Figure C-322. Specimen EPX1-C#5 Failure Pattern.

Summary of Results

Table C-5 indicates a summary of the results of the compressive strength tests applied on the high-build epoxy (EPX1-C) lined specimens. All of the specimens failed in a similar mode; i.e., the lining fractured vertically and the concrete susbtrate split in a cone-shaped pattern. The average ultimate strenghts for the three control specimens and five lined specimens were 5,084 psi (35,053 kPa) and 5,633 psi (38,838 kPa), respectively.

Table C-5. Summary of the Preliminary Flexure Test Results for the EPX1 Cylinder Specimens.

Cylindrical Sample Number	Sample Dimensions D*L		Lining Thickness		Test Type	Test Date	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	Mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
R1 (EPX1-C#1)	D_1=106 D_2=106 D_{ave}=106 L_1=205 L_2=205 L_{ave}=205	D_1=4.2 D_2=4.2 D_{ave}=4.2 L_1=8.1 L_2=8.1 L_{ave}=8.1	3.1	124	C39	03/20/2013	8,861	13.73	445	100	380.42	85,520
R2 (EPX1-C#2)	D_1=106 D_2=104 D_{ave}=105 L_1=202 L_2=207 L_{ave}=205	D_1=4.2 D_2=4.1 D_{ave}=4.2 L_1=7.9 L_2=8.1 L_{ave}=204.5	3.2	128	C39	03/20/2013	8,730	13.53	445	100	359.10	80,730
R3 (EPX1-C#3)	D_1=107 D_2=107 D_{ave}=107 L_1=201 L_2=202 L_{ave}=202	D_1=4.2 D_2=4.2 D_{ave}=4.2 L_1=7.9 L_2=8.0 L_{ave}=8.0	2.8	111	C39	03/20/2013	8,966	13.89	445	100	337.18	75,800
R4 (EPX1-C#4)	D_1=108 D_2=107 D_{ave}=107 L_1=205 L_2=205 L_{ave}=205	D_1=4.2 D_2=4.2 D_{ave}=4.2 L_1=8.1 L_2=8.1 L_{ave}=8.1	2.97	0.1169	C39	03/20/2013	9,0523	14.03	445	100	367.02	82,510
R5 (EPX1-C#5)	D_1=108 D_2=107 D_{ave}=107 L_1=205 L_2=206 L_{ave}=206	D_1=4.2 D_2=4.2 D_{ave}=4.2 L_1=8.1 L_2=8.2 L_{ave}=8.1	4.035	0.1588	C39	03/20/2013	9,048	14.02	445	100	288.87	64,940

CIP-C – Cylinder Specimens

Before each test, the entire specimen was carefully observed with a magnifier. No visible cracks or holes were observed in the concrete or liner. The liner seemed to be intact and tightly adhered to the surface of the specimens. After the appropriate measurements (Table C-6) the specimens were capped using sulfur capping material three days before the test and then setup in the machine (Figure C-33). The load was applied to each specimen with a constant rate of 100 lbs./sec. Observations and test results for each cylinder specimen are described below.

Figure C-33. Lined Cylinder CIP-C Specimen Mounted on the Admet 500 Kip Compression Test Machine.

Specimen One (CIP#1)

The specimen failed at a peak load of 76,470 lbs. and peak stress of 5,051 psi. The lining has failed in three different places with hairline cracks. The liner was still attached to the substrate upon failure (Figure C-34).

Figure C-34. Specimen CIP-C#1 at Failure, Note the Liner Did Not Separate From the Concrete Substrate.

Specimen Two (CIP-C#2)

The specimen failed at a peak load of 85,940 lbs., and peak stress of 5,193 psi. Failure mode was the same as the first specimen with three hairline cracks (Figure C-35).

Figure C-35. Specimen CIP-C#2 at Failure, Note the Circumferential Cracking.

Specimen 3 (CIP-C#3)

The specimen failed at a peak load of 81,180 lbs., and peak stress of 5,457 psi. The specimen had multiple hairline cracks with minor fracturing at failure. The liner was still attached to the substrate upon failure (Figure C-36).

Figure C-36. Specimen CIP-C#3 at Failure, Note the Hairline Cracking.

Specimen Four (CIP-C#4)

The specimen failed at a peak load of 80,700 lbs., and peak stress of 5,351 psi. Similar to the preceding specimens, CIP-C#4 failed with multiple hairline cracks and minor fracturing (Figure C-37).

Figure C-37. Specimen CIP-C#4 Failure Pattern.

Specimen Five (CIP-C#5)

The fifth specimen failed at substantially higher peak load (111,330 lbs.) and peak stress (6,936 psi). As such, it ruptured vertically with a larger gap in comparison with the fracturing observed on the other specimens. Likewise, the liner was still attached to the substrate upon failure (Figure C-38).

Figure C-38. Specimen CIP-C#5 Failure Pattern, Note the Vertical Splitting.

Summary of Results

Table C-6 indicates a summary of the results of the compressive strength tests applied on the cured-in-place (CIP) lined specimens. The average ultimate strenght for the three control specimens and five lined specimens were 5,084 psi (35,053 kPa) and 5,599 psi (38,604 kPa), respectively. Results obtained from the first four specimens were close to each other; however, the fifth specimen had signicantly, if not substantially, higher ultimate strength than the others, which also resulted in a somewhat different failure pattern than the other four specimens.

Table C-6. Summary of the Preliminary Flexure Test Results for the CIP Cylinder Specimens.

Cylindrical Sample Number	Sample Dimensions D*L		Lining Thickness		Test Type	Test Date	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
CIP#1	D_1=111 D_2=112 D_{ave}=111 L_1=203 L_2=203 L_{ave}=203	D_1=4.4 D_2=4.4 D_{ave}=4.4 L_1=8.0 L_2=8.0 L_{ave}=8.0	6.3	249	C39	04/15/2013	9,768	15.14	445	100	340.16	76,470
CIP#2	D_1=113 D_2=117 D_{ave}=115 L_1=204 L_2=203 L_{ave}=204	D_1=4.5 D_2=4.6 D_{ave}=4.6 L_1=8.1 L_2=8.1 L_{ave}=8.1	6.4	249	C39	04/15/2013	10,677	16.55	445	100	382.28	85,940

Cylindrical Sample Number	Sample Dimensions D*L		Lining Thickness		Test Type	Test Date	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
CIP#3	D_1=111 D_2=110 D_{ave}=111 L_1=207 L_2=208 L_{ave}=208	D_1=4.4 D_2=4.3 D_{ave}=4.4 L_1=8.2 L_2=8.3 L_{ave}=8.3	6.2	242	C39	04/15/2013	9,587	14.86	445	100	361.11	81,180
CIP#4	D_1=112 D_2=111 D_{ave}=111 L_1=203 L_2=206 L_{ave}=205	D_1=4.4 D_2=4.4 D_{ave}=4.4 L_1=8.0 L_2=8.1 L_{ave}=8.1	7.0	269	C39	04/15/2013	9,729	15.08	445	100	358.97	80,700

Cylindrical Sample Number	Sample Dimensions D*L		Lining Thickness		Test Type	Test Date	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
CIP#5	D_1=114 D_2=116 D_{ave}=115 L_1=204 L_2=205 L_{ave}=205	D_1=4.5 D_2=4.6 D_{ave}=4.5 L_1=8.1 L_2=8.1 L_{ave}=8.1	6.6	260	C39	04/15/2013	10,355	16.05	445	100	495.22	111,330

PU – Cylinder Specimens

Before each test, specimens were carefully observed with a magnifier. No visible cracks or holes were observed in the concrete substrate or liner. The liner seemed to be intact and tightly adhered to the surface of the substrate. After the appropriate measurements (Table C-7) each specimen was capped using sulfur capping material three days before the test and then setup in the machine (Figure C-39). The load was applied to the specimen with a constant rate of 100 lbs./sec. Observations and test results for each cylinder specimen are described below.

Figure C-39. Lined Cylinder CIP-C Specimen Mounted on the Admet 500 Kip Compression Test Machine.

Specimen One (PU-C#1)

The specimen failed at a peak load of 115,020 lbs. (511.6 kN) and a peak stress of 7,883psi (54,351.4 kN). The sample failed with a vertical fracture (Figure C-40).

Figure C-40. PU-C#1 Failed with a One-Inch (25 mm) Wide Fracture

Specimen Two (PU-C#2)

The specimen failed at a peak load of 87,380 lbs. (388.7 kN), and peak stress of 6,102 psi (42,072 kPa). The liner had two vertical fractures at failure. Figure C-41a indicates the ¾-inch (19 mm) wide fracture occurred at failure. The liner detached from the concrete substrate at the location of fracture (C-41b).

a b

Figure C-41. A 3/4 –Inch (19 mm)-Wide Vertical Fracture Formed at Failure on PU-C#2 (a). The Liner Detached from the Substrate at this Location (b).

Specimens 3, 4, and 5 (PU-C#3, #4, #5)

PU-C specimens #3, #4, and #5 failed in a similar pattern as in #1 and #2 with vertical fractures (Figure C-42). PU-C#3 failed with two vertical fractures at a peak load of 101,340 lbs. (450.8 kN) and a peak stress of 7,317 psi (50,449 kPa). As was the case for PU-C#2, the liner on PU-C#5 detached from the substrate at failure (Figure C-43).

a b

Figure C-42. Vertical Fracture in PU-C#3 (a) and PU-C#4 (b).

Figure C-43. Liner Detachment from the Concrete Substrate (PU-C#5).

Summary of Results

Table C-7 indicates a summary of the results of the compressive strength tests applied on the polyurea (PU) lined specimens. The average ultimate strength for the three control specimens and five lined specimens were 5,084 psi (35,053 kPa) and 7,282 psi (50,208 kPa), respectively. The PU linings contribution to the ultimate compressive strength of the concrete cylinder substrate was from 42 to 88 percent. The relatively consistent results observed on PU lined specimens could be attributed to the narrow range in the thickness of the lining applied (from 3.6 mm/140 mils to 4.6 mm/179 mils).

Table C-7. Summary of the Preliminary Flexure Test Results for the PU Cylinder Specimens.

Cylindrical Sample Number	Sample Dimensions D*L		Lining Thickness		Test Type	Test Date	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
PU1	D_1=109 D_2=110 D_{ave}=109 L_1=194 L_2=193 L_{ave}=194	D_1=4.3 D_2=4.3 D_{ave}=4.3 L_1=7.8 L_2=7.8 L_{ave}=7.8	4.4	173	C39	05/13/2013	9,393	14.59	445	100	511.63	115,020
PU2	D_1=109 D_2=108 D_{ave}=108 L_1=204 L_2=204 L_{ave}=204	D_1=4.275 D_2=4.254 D_{ave}=4.265 L_1=8 L_2=8 L_{ave}=8	3.6	140	C39	05/13/2013	9,217	14.32	445	100	388.68	87,380

Cylindrical Sample Number	Sample Dimensions D*L		Lining Thickness		Test Type	Test Date	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	mils			mm^2	$in.^2$	N/s	lbs./sec	kN	lbs.
PU3	D_1=108 D_2=108 D_{ave}=107 L_1=205 L_2=206 L_{ave}=206	D_1=4.3 D_2=4.1 D_{ave}=4.2 L_1=8.1 L_2=8.1 L_{ave}=8.1	4.6	179	C39	05/13/2013	8,967	13.85	445	100	450.78	101,340
PU4	D_1=109 D_2=109 D_{ave}=109 L_1=204 L_2=203 L_{ave}=204	D_1=4.3 D_2=4.1 D_{ave}=4.2 L_1=8 L_2=8 L_{ave}=8	4.3	170	C39	05/13/2013	9,335	13.88	445	100	516.74	116,170

Cylindrical Sample Number	Sample Dimensions D*L		Lining Thickness		Test Type	Test Date	Cross Section Area		Loading Rate		Peak Load	
	mm	in.	mm	in.			mm^2	$in.^2$	N/Sec	lbs./Sec	kN	lbs.
PU5	D_1=109 D_2=109 D_{ave}=109 L_1=205 L_2=206 L_{ave}=206	D_1=4.3 D_2=4.3 D_{ave}=4.3 L_1=8.1 L_2=8.1 L_{ave}=8.1	4.4	174	C39	05/13/2013	9,285	14.39	445	100	431.30	96,960

APPENDIX D

MAIN TESTS STANDARD FIELD FORM

Day	Date	Weather	Temp (°F)	Wind	Holiday?	Prepared by:
Tuesday	2/25/2014	Cloudy	50	8 mph	No	Shyam

Personnel Name	Equipment	Own/Cont
Mechanical Jobbers	Generator, Air pump, Van, Lining equipment.	Owned

Description of Work Performed

Description	Quantity
D-Loading with Instrumentation	5 Pipes
Lining of Pipes with Glass Fiber	3 Pipes

Issues/Problems/Concerns

As the weather was cloudy the epoxy took a long time to set before another coat of epoxy was applied over it.

Visits/Meetings

Names	Organization
Jorge Legra	Mechanical Jobbers
Dr. Najafi	UTA (CUIRE)

Description	Results
Lining of pipes with glass fiber in two layers with epoxy to get a thickness of 250 mills.	Successfully Done

Task Performed	Task Performed By
1. Testing on Pipes and Video Recording	Ali and Neehar
2. Lining of Pipes	David and Crew

Description and Result

The peak load was 64,946 lbs. and minimum failure load was 22,050 lbs.

Figure D-1. Application of Epoxy over Glass Fiber.

Figure D-2. Lining Using Glass Fiber.

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Old North State Water Company Inc.
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Ohio
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Columbus, City of
Dayton, City of
Metropolitan Sewer District of Greater Cincinnati
Northeast Ohio Regional Sewer District
Summit County

Oklahoma
Oklahoma City Water & Wastewater Utility Department
Tulsa, City of

Oregon
Bend, City of
Clean Water Services
Gresham, City of
Lake Oswego, City of
Oak Lodge Sanitary District
Portland, City of
Water Environment Services

Pennsylvania
Philadelphia, City of, Water Department
University Area Joint Authority

South Carolina
Beaufort - Jasper Water & Sewer Authority
Charleston Water System
Greenwood Metropolitan District
Mount Pleasant Waterworks
Spartanburg Water
Sullivan's Island, Town of

Tennessee
Cleveland Utilities
Murfreesboro Water & Sewer Department
Nashville Metro Water Services

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Kentucky
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May 2015