Laboratory Investigation on the Static Response of Repaired Sewers

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Abstract

This paper examines and characterizes the structural response of intimately fitted repaired sewers due to disturbance in the vicinity (e.g. vehicle loads). This soil-host pipe-liner interaction is investigated by full scale laboratory testing. The test geometry and measurement scheme are described. Mechanisms affecting local bending at host pipe fractures are identified. A comparison is made between response for the HDPE liner within a fractured non-reinforced concrete pipe, and that same HDPE liner buried alone in soil. Under earth disturbance, the fractures in the host pipe expanded at the inner surface at the crown and invert, and at the outer surface at the springlines. This expansion of fractures caused contacts between the inner edges of the host pipe and the liner at the crown and invert. The liner response was dominated by the local bending that resulted.

Introduction

The trenchless rehabilitation of damaged rigid sewers has long been a competitive alternative to conventional pipeline replacement methods. When a polymer pipe liner is inserted within a damaged sewer, both the hydraulic and structural integrities can be restored. This construction technique minimizes the excavation needed; hence, disturbance to the traffic can be reduced significantly. Preliminary testing in the laboratory has shown that any initial lack of fit (between liner and host pipe) dictates the local strain pattern of the liner (D'Andrea, 1999, Law & Moore, 2000).

This paper reports on two recent tests that have been performed to investigate the static behavior of intimately fitted repaired sewers.

Problem Definition

Stress experienced by buried pipes can be separated into two components: compressive hoop thrust and bending. Both components lead to pipe deflections, while flexible structures can buckle under excessive thrust. Current pipe liner design standards (e.g. ASTM F1606-95) identify buckling due to groundwater or earth pressures as the important performance limits. This is applicable only to cases where deep burial or zero disturbance to the repaired system, such as vehicle loads or excavation in the vicinity, is guaranteed. Since these circumstances may not be realistic, bending may develop and could well be a significant limit state.

There are three components in the pipe liner interaction problem: the soil, the host pipe, and the liner. Soil, of course, serves the dual purposes of applying load to the system while providing support against deformation. The host pipe being repaired is generally rigid and is typically composed of vitrified clay or reinforced concrete. Bending stiffness of clay pipes is immediately compromised after longitudinal cracking. Concrete pipes, on the other hand, may retain their bending resistance shortly after damage occurs due to the presence of reinforcement. Exposed reinforcement will eventually corrode and the behavior of these two types of host pipe will then converge. The liner installed is usually a flexible pipe such as high density polyethylene (HDPE), polyvinylchloride (PVC), or glass reinforced polymer structure.

Slip-lining, cured-in-place, and deform-reform systems are just a few of the many construction techniques that can be used to undertake sewer repairs. A flexible pipe is simply pulled and placed within a host pipe for the case of slip-lining (e.g. ASTM F585 – 94). A certain amount of clearance between the liner and the host pipe is necessary to minimize interference and friction during insertion. This spacing can be eliminated by grouting. When a sewer is repaired using the cured-in-place (e.g. ASTM F1216-93) or deform-reform (e.g. ASTM F1606-95) techniques, the mechanism of response will be similar, because they both feature a liner fitted snugly inside the host pipe. A grouted slip-lined system could also in this manner, once the ground disturbance fractures the grout at locations where it crosses the host pipe damage.

Design procedures provided in ASTM F1216-93 and ASTM F1606-95 are identical. Considerations are given to the conditions of the host pipe. If the host pipe is characterized as "partially deteriorated", it is assumed to be hydraulically compromised but still capable of supporting soil and surcharge loads throughout the design life of the rehabilitated pipe. On the other hand, if the original pipe is denoted "fully deteriorated", it is not assumed to be structurally sound and will not be considered as supporting any of the soil loading. The designation and design of the “fully deteriorated” condition is currently the subject of considerable debate in the industry. The investigation reported here is part of a study to clarify performance limits of structurally damaged pipes.

The design standards mentioned above treat host pipes suffering from classic overloading fractures (longitudinal cracks at the springlines, crown, and invert) as being in the “partially deteriorated” condition. Since buckling under groundwater or earth pressure is viewed as the governing performance limit, this classification is reasonable since host pipes with this fracturing are still capable of carrying thrust. In terms of bending, however, this type of failure deserves further attention, and might need to be considered as “fully deteriorated” because host pipes with overloading fractures have no bending stiffness. This study examines the implications of these fractures. Pipe testing was conducted for liner placed within an artificially damaged host pipe.

The tests are designed to investigate the conceptual deformation of the host pipe-liner system under earth disturbance; the idealization is shown in Figure 1. Strain distributions in the liner are expected to be similar to the ideal hour-glass pattern that occurs around the circumference of buried pipes. The fractures of the host pipe will expand (at the inner surface of the host pipe at the crown and invert, and the outer surface at the springlines), yet maintain contact at one point on the host pipe wall, forming a hinge mechanism. Despite deformations of the damaged host pipe, it is still capable of carrying thrust as long as the segments remain in contact. The "separating" motion at the fractures of the host pipe puts the inner edges of the host pipe and the liner in direct contact at both crown and invert, causing stress concentrations in the liner. This "angular expansion" of the fractures can be expressed by an angle, θ. The flexural response of the host pipe-liner system is expected to be governed by plane strain conditions due to its geometry.

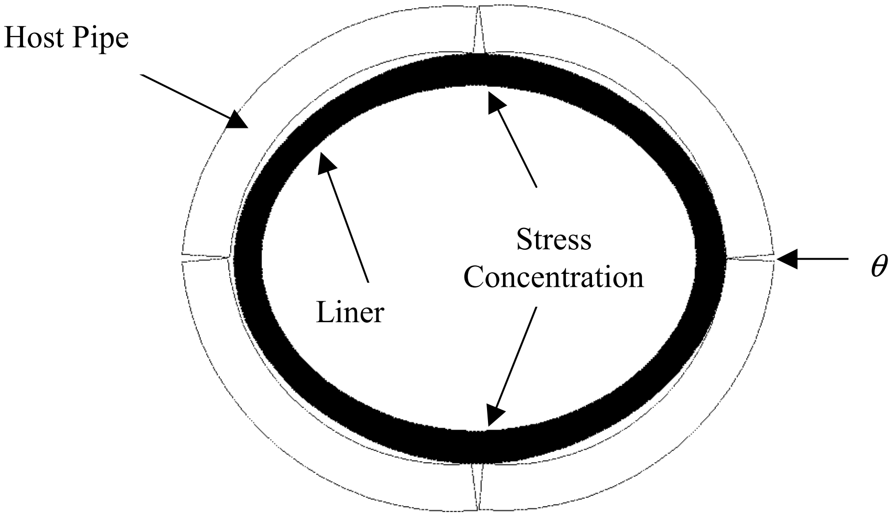


Figure 1 – Idealized Deformation of Repaired Sewers

Preliminary Tests Examining Liner-Host Pipe-Soil Interaction

Objectives

A biaxial compression test cell was used to investigate the mechanisms of intimately fitted soil-host pipe-liner interaction. Two tests have been performed in this study. The first test featured a plain 320mm HDPE SDR 26 liner placed inside a non-reinforced concrete host pipe with artificially induced overloading fractures. The same liner was tested alone in the soil for the second test, to investigate the impact of the presence of the host pipe on the liner response.

Specimen preparation and instrumentation

In order to simulate sewers repaired by techniques such as case-in-place or deform-reform systems, the tests were designed to obtain intimate contact between the liner and the host pipe (no initial gap). Unreinforced concrete pipe segments were cast directly onto an HDPE pipe, 1960mm in length, using the following procedures. The locations of crown, invert, and springlines were marked on the pipe liner, four steel angles (25.4mm by 9.5mm with 2.5mm thickness) were then screwed onto the liner at those locations (Figure 2). The liner was then place inside a steel tube mould (370mm in diameter) and concrete was fed from the top with the sample standing vertically. The product, Quickcrete, was used and high slump was maintained in the concrete mix to ensure its ability to flow down the 25.4mm space between the liner and the steel tube. Before the steel tube mould was removed on the fourteenth day, each concrete segment and its corresponding position on the liner were recorded. The steel angles were then removed from the liner and glued back to the host pipe segments to reduce any gap between fractures and between the liner and the host pipe.

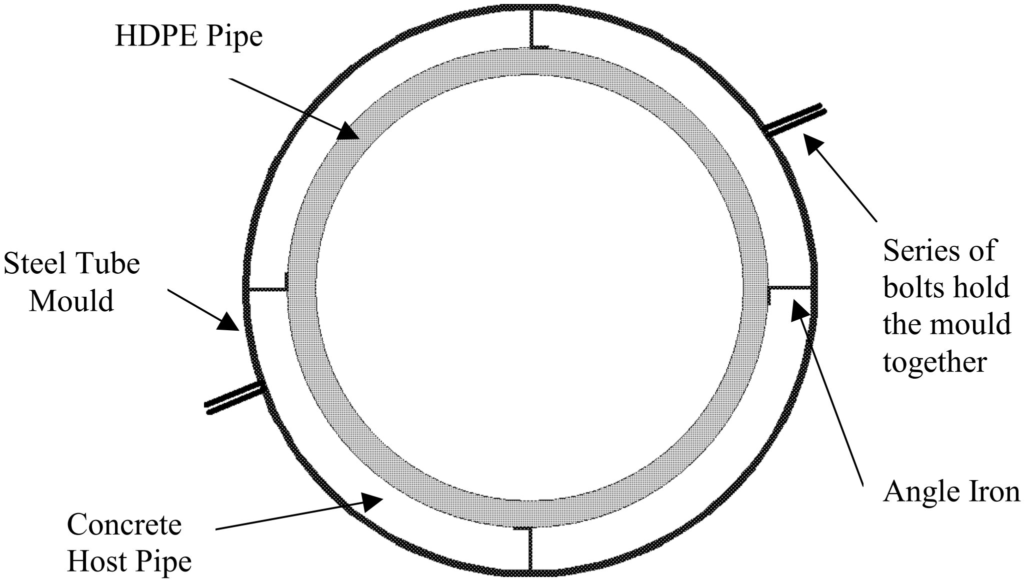
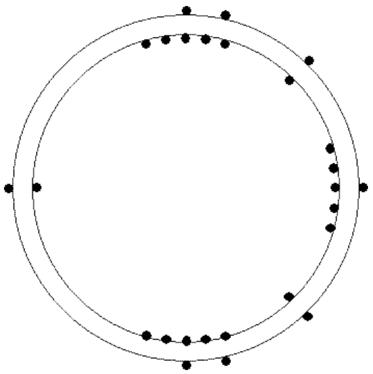
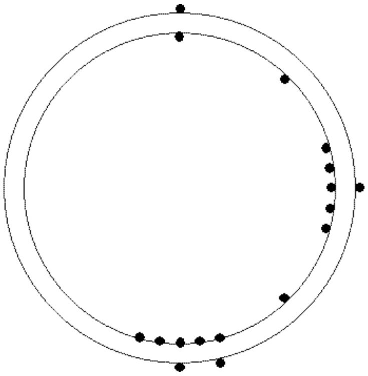


Figure 2 – Configuration for Concrete Host Pipe Casting
(horizontal section through vertically oriented pipe)

The same HDPE pipe liner used as the internal boundary during casting was instrumented with resistance strain gauges and displacement transducers on two test sections, A and B, 330mm from each end (Figure 3). Clusters of five gauges were used to monitor local bending where the liner spanned between the four fractures in the host pipe. Pairs of single gauges were placed on both the interior and exterior surfaces at a number of other locations to evaluate bending and thrust. A total of forty-two biaxial strain gauges were used. Two displacement transducers were installed at both test sections to measure vertical and horizontal diameter changes. Concrete pipe segments and the liner were then assembled and the whole specimen was strapped with steel bands.





Test Section A Test Section B

Figure 3 – Strain Gauge Locations

It is important to investigate and quantify the fracture angular expansions of the host pipe under ground disturbance. Mechanical shutter release cables (1m in length) were modified and mounted onto the host pipe at springline locations to measure the relative movement of the host pipe segments during testing. These cables, though designed for camera use, act like a displacement transducer, but without the casing and electronic components (Figure 4). The stroke of each cable is 14mm. Calibration showed that measurement errors in displacement evaluated by these cable range from 1% to 4%. However, about 30% of their sensitivity may be lost if the cables are bent sharply. The cables were mounted (in a compressed state) on a small piece of fiberglass block and an angle iron, and another piece of angle iron was used as the reaction (Figure 5). Since the host pipe deforms as a rigid body longitudinally, movement of fractures should be consistent in all locations. The cables were glued onto the host pipe near the ends using epoxy adhesive. Dimensions of each cable location were measured in detail to translate displacement readings to the angular expansion of the fractures in degrees.

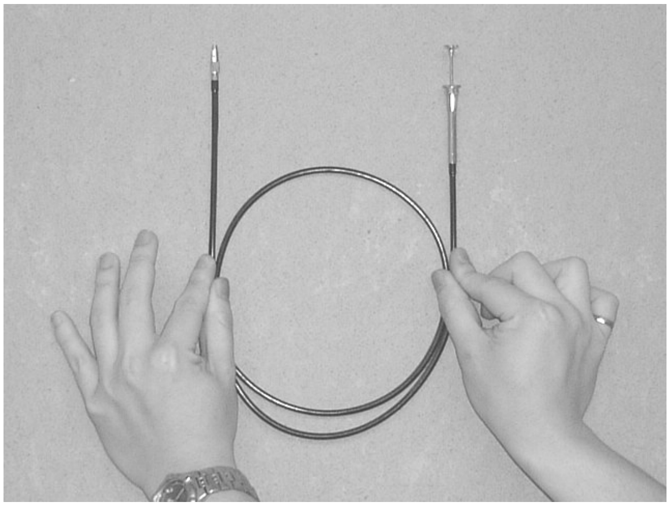


Figure 4a – Mechanical Shutter Release Cable

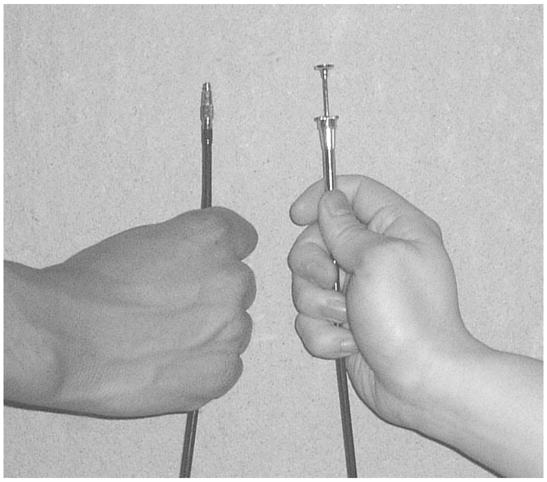


Figure 4b – Cable in Released State

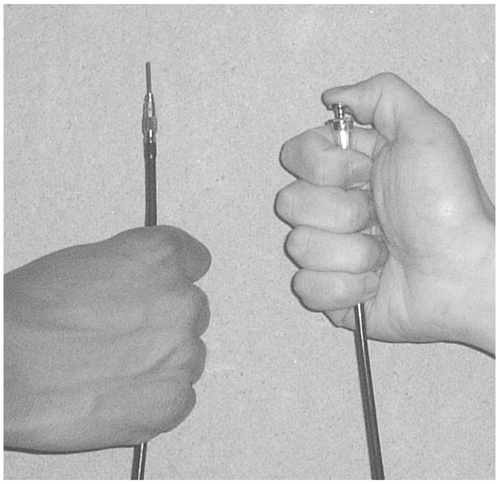


Figure 4c – Cable in Compressed State

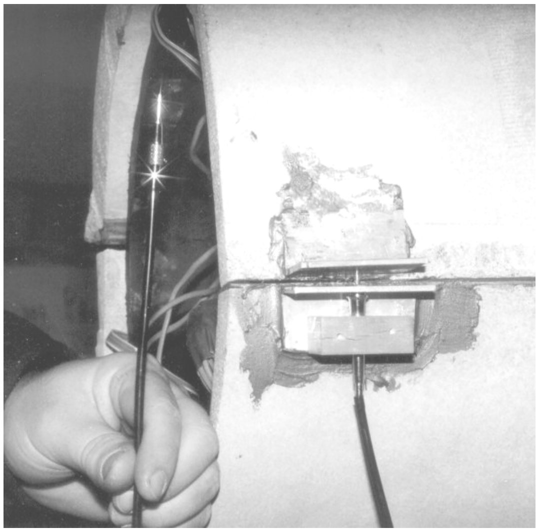


Figure 5 –Mechanical Shutter Release Cable at the Springline of the Host Pipe

Pipe burial

The biaxial compression test cell is designed to simulate deep embankment loadings on a buried pipe. By restraining backfill soil laterally, close to a geostatic stress in the “at rest” condition (K_o) can be achieved (Brachman, 1999). The cell dimensions of 2.0m by 2.0m by 1.6m are capable of providing a reasonable approximation to the geostatic condition for a test pipe up to 600mm in external diameter.

Poorly graded sand (Olimag 30-60, C_u = 1.46, C_c = 0.94, Lapos, 2002) was used as the backfill soil (Figure 6). It was placed in 100 mm lifts at about 3% moisture content, resulting in an average density of 1335 kg/m^3. Soil was not compacted, to simulate poor backfill material in the field. The specimen was placed horizontally once the soil reached a depth of 400mm. The temporary metal straps were removed from the host pipe-liner specimen once backfill was placed just above the springline level. The mechanical shutter release cables were covered to prevent intrusion of backfill soil. Wires for strain gauges and displacement transducers were routed from the sample through an access hole to a data acquisition system. The free ends from the mechanical shutter release cables were also fed through the access hole and were mounted to displacement transducers. Two settlement plates and eight earth pressure cells were also installed in the backfill.

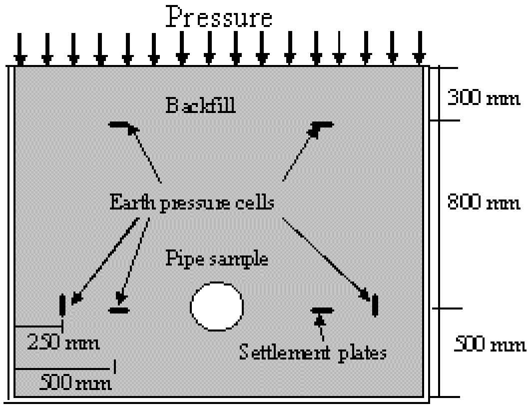


Figure 6 – Test Configuration

Loading

The loading schemes for the two tests were identical. Loading was applied in approximately 55 kPa increments at the surface of the backfill through a flexible bladder system. Each pressure increment was sustained for 900 seconds prior to application of the next load step. Three such increments were applied, resulting in a final surface pressure of about 170 kPa. Strain gauge and displacement transducer readings were recorded every 10 seconds using the data acquisition system. Earth pressure cell readings were also taken towards the end of each load step. This loading corresponds to a situation where additional embankment material or surface live load is placed on the ground surface across the top of the damaged sewer pipe after repair.

Test Results

For the test on the host pipe-liner system, problems with instrumentation meant that deformation measurements were restricted to the mechanical shutter release cable measurements at difference overburden pressures shown in Figure 7. At the highest pressure level of 170 kPa, angular expansion of the fractures was about 3.6°.

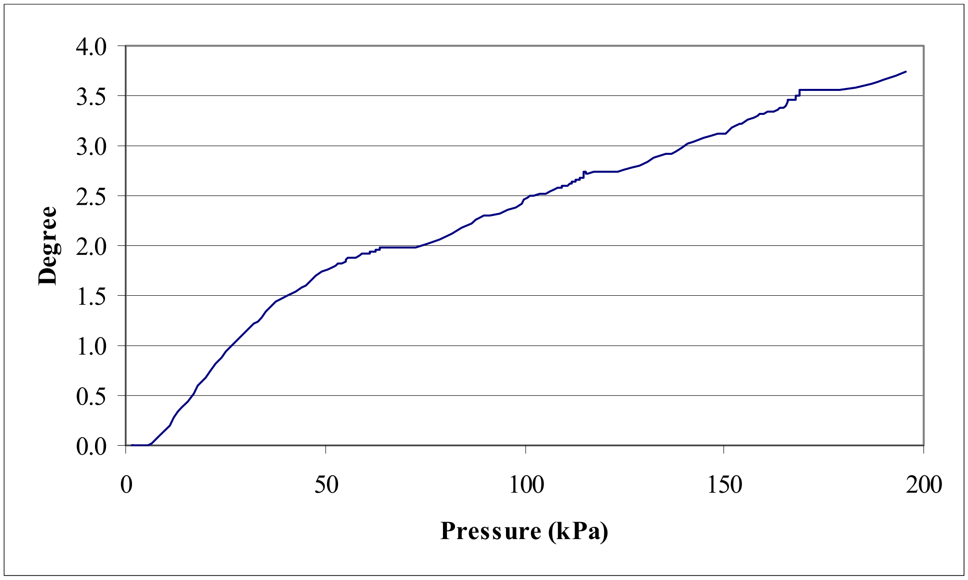


Figure 7 – Angular Expansion of the Fracture (θ) vs. Pressure – Springline Reading

For a rigid host pipe, its decrease in vertical diameter and increase in horizontal diameter would be identical due to the kinematics of the four quadrants, so the fracture angular expansion measurements can be used to approximate diameter changes based on the trigonometric relationships shown in Figure 8. Deformation values for both tests are shown in Figure 9. Values shown for the test without the host pipe are direct measurements obtained using displacement transducers. In that particular test, deformations of the liner at 170 kPa were 5.3% vertically and 4.1% horizontally. In contrast, the presence of the host pipe seems to provide additional resistance against flexural deformations, and therefore, alters the changes in diameter.

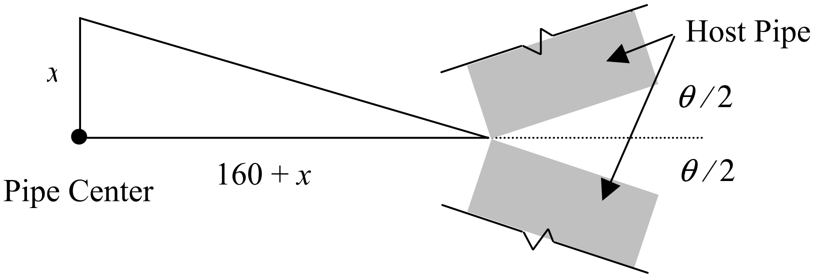


Figure 8 – Relation between Fracture Expansion and Diameter Change
(original liner radius = 160 mm, diameter change = $2x$ mm)

Interior and exterior hoop strains for the two tests are show in Figure 10. Measurements from sections at 170 kPa overburden pressure are shown (tension is positive). Readings from both sections are largely similar, except interior strains at section A for the test where the pipe liner was buried alone (shown by open circles). It seems that there was a zone of low stiffness soil under the haunch on one side of the liner, and so, this set of data is not used for further comparisons.

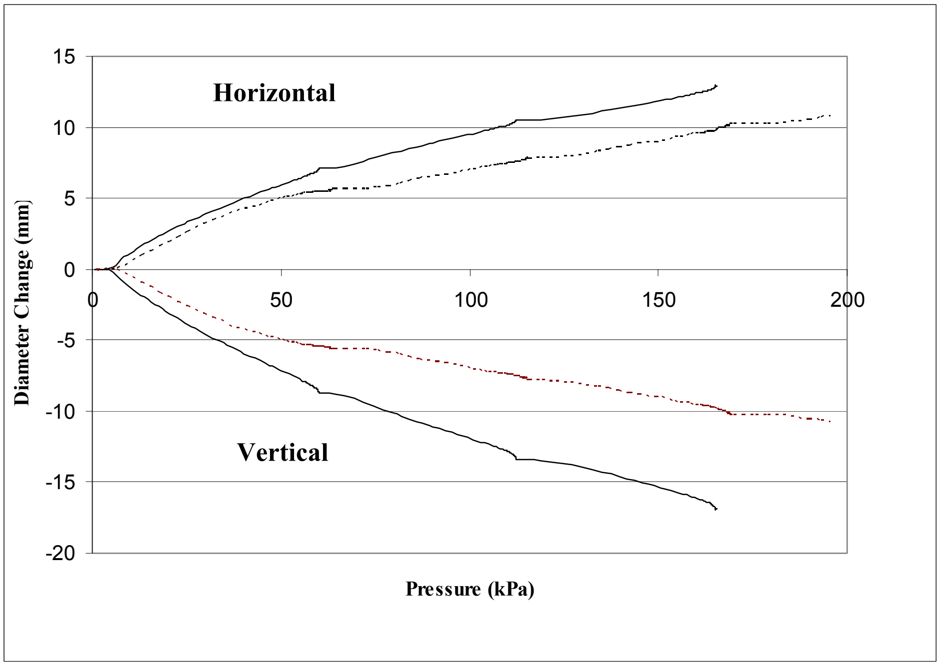


Figure 9 – Average Diameter Change vs. Pressure
(——— Liner alone ·········· Liner with Host Pipe)

Strain readings across the fractures in the host pipe indicate that the liner was almost responding in pure bending at those locations when a host pipe was present (strains at the inner and outer surfaces are almost equal and opposite). This is reasonable, since thrust was carried in the host pipe segments and was transferred between them where they contact one other (at springlines, crown, and invert). At crown and invert, the strain pattern was dominated by local bending caused by the "separating" motion of the host pipe segments (Figure 1).

In contrast, at the springlines, the compressive strain was lower when the liner was placed within the host pipe, again, due to the effect of reduced thrusts.

The presence of the host pipe reduced the thrust on the liner, but the effect of thrust was less dominant than bending. From the strain measurements taken at haunch and shoulder, thrust increased the compressive strain on the liner when it was buried alone, but thrust was small and the compressive strains remained low (about -1500 microstrain, Figure 10).

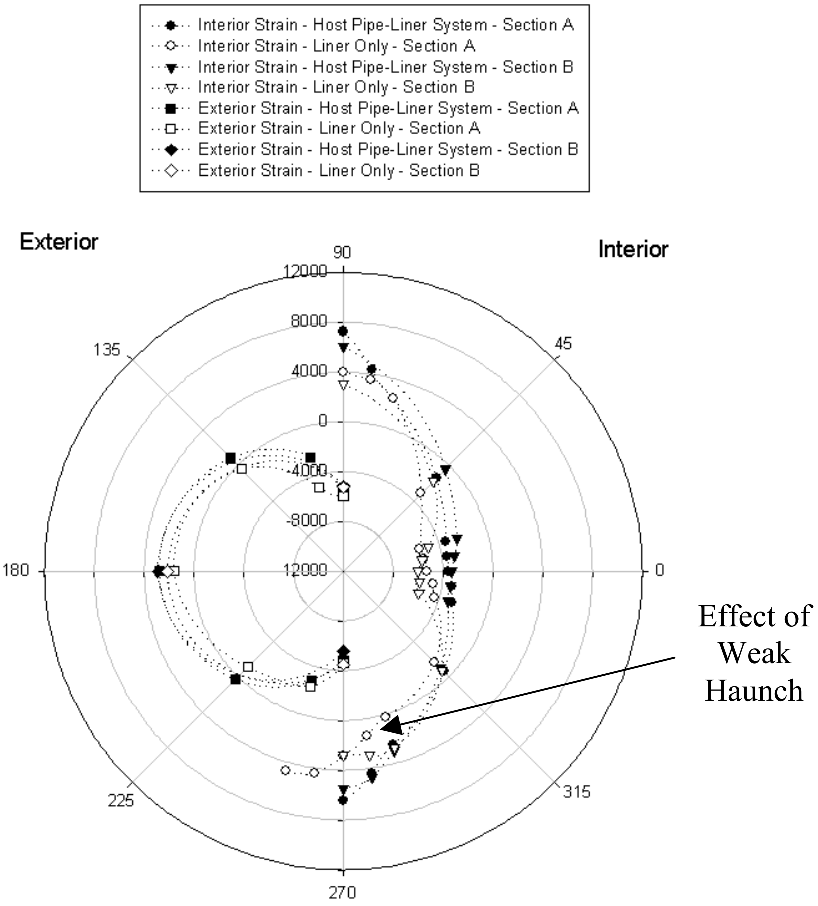


Figure 10 – Hoop Strain Measurements at 170 kPa

Axial strains at overburden pressure of 170 kPa are shown in Figure 11. Only section A is shown since measurements on both sections are similar. The ends of the liner were not in direct contact with the sidewalls of the test cell, so these low axial strain values indicate that the presence of the host pipe may be providing substantial axial restraint to the liner, reducing deformations in the longitudinal direction.

Discussion

The results given here are based on a specific set of samples, test procedures, configurations, geometrical and material conditions. Issues that require particular attention are the load path imposed (the timing of liner insertion) and the condition of the fractures in the host pipe. In the tests performed, the sewer was “repaired” prior to the application of the earth pressures. This load path is adopted for practical considerations in the laboratory, but the load path in the field is likely to be different.

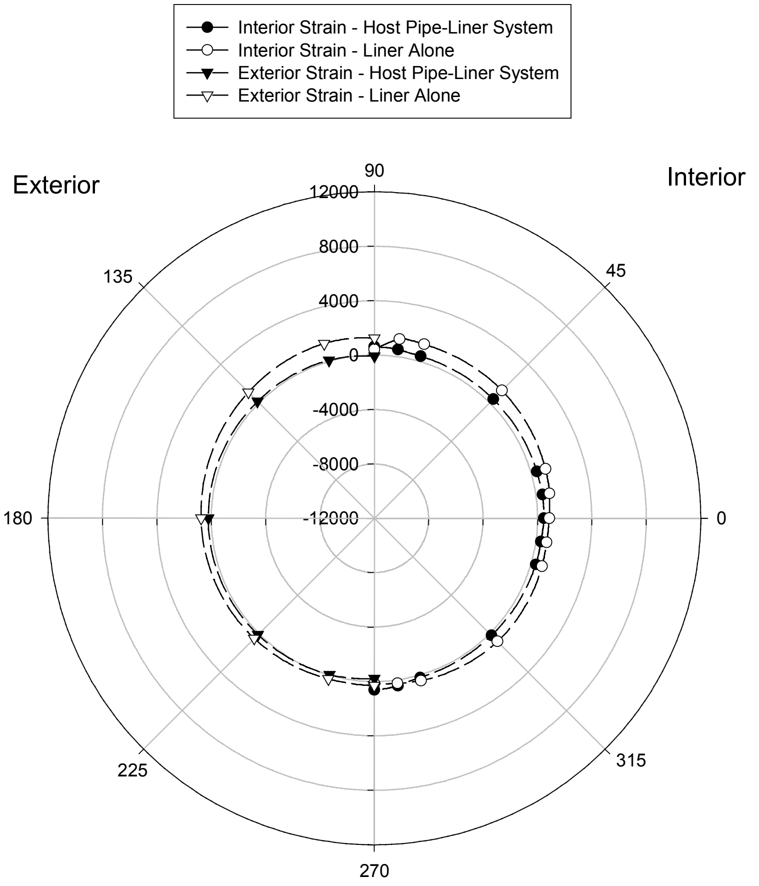


Figure 11 – Axial Strain Measurements at 170 kPa

The 25.4mm thick rigid host pipe used in the test was specially fabricated for use in this investigation. Its material and geometry, together with the artificially induced fractures are reasonable idealizations of both clay and reinforced concrete pipe. Nonetheless, these properties are approximate, and an idealized representation of field characteristics. Future testing using actual exhumed host pipe with realistic damage would definitely be valuable.

Deformation

The presence of the host pipe has changed the deformation of the liner. This implies that the host pipe-liner system has a different stiffness than the liner alone. This difference is limited to the hoop direction only, since the cracked host pipe only has bending stiffness within, not between, the four quadrants.

Effects of Thrust

Theoretically, a pipe with uniform ground support is under pure thrust at the shoulder and haunch (e.g. Burns and Richard, 1964). Since the hoop stiffness of the damaged host pipe is still sustained as long as there are contacts between segments, less thrust is transferred to the liner. Average compressive strain across the liner at the shoulder were reduced from about 1500 microstrain to close to zero (Figure 10), indicating that thrust was almost fully carried by the host pipe.

Effects of Bending

The decrease in hoop compressions and the local bending at the fractures, meant that the interior and exterior strain distribution patterns altered significantly as a result of interaction of the liner with the host pipe.

Local bending in the liner across fractures in the host pipe was distinctly different at the crown, invert and springline locations. At the crown and invert, the fracture of the host pipe expanded at the inner surface, resulting in a direct contact with the liner, so the liner spans between these two separate host pipe segments. Substantial bending results, with peak bending strain of more than +/- 6000 microstrain (Figure 10). At the springlines, the host pipe segments in contact with the liner remained together, and while there was still local bending that developed, the curvature changes and bending strains were distinctly lower than those measured at crown and invert (strain values less than +/- 4000 microstrain, Figure 10)

Response in Axial Direction

The host pipe provided restraint to the liner in the longitudinal direction, further supporting the argument that this soil-host pipe-liner interaction problem responds under a plain strain condition. It is significant as Young's modulus (E) of the liner would be increased by the ratio $1/(1-v^2)$, where v is the Poisson's ratio of the liner. Assuming v of at least 0.4 in HDPE, this increase in liner stiffness is no less than 20%.

Conclusions

The current liner design approach and some of its limitations have been reviewed. Recent work to investigate the static behavior of repaired sewers has been presented. Results from two full scale tests indicate that the host pipe provides most of the hoop stiffness to the repaired sewer. Significant local bending was observed in the liner at crown and invert, where the inner edges of the cracks apply local loads that induce large changes in pipe curvature and local bending stresses. Further studies are planned to examine the behavior of the damaged sewer alone, and the impact of the stress path on the response of the HDPE liner.

Acknowledgements

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