GUIDANCE ON DESIGN OF FLEXIBLE LINERS TO REPAIR STRUCTURALLY COMPROMISED GRAVITY FLOW SEWERS

T.C. Michael Law, Graduate Student
Geotechnical Research Centre
Department of Civil and Env. Engineering
The University of Western Ontario
London, Ontario, Canada N6A 5B9
Phone: (613) 533-6000 ext 77142
Fax: (613) 533-2128
Email: tcmlaw@alumni.uwaterloo.ca

and

Ian D. Moore, Professor
Geo Eng Centre at Queen’s - RMC
Department of Civil Engineering
Queen's University
Kingston, Ontario, Canada K7L 3N6
Phone: (613) 533-3160
Fax: (613) 533-2128
Email: moore@civil.queensu.ca

ABSTRACT

Design of polymer and other liners to repair structurally compromised gravity flow sewers and culverts requires consideration of the potential impact of changes in earth loads on the liner-damaged pipe system. Laboratory research studies have identified the need to consider how changes in earth loads induce liner deformations as well as local bending strains at its crown and invert. These issues are discussed, and recommendations made regarding the design approach needed to account for these local bending strains. Reference is made to full scale laboratory tests simulating the behavior of a “tight fitting” damaged pipe-liner system. Discussion is included on the impact of liner thickness, small initial gaps between host pipe and liner, estimation of changes in vertical earth pressure and liner modulus in relation to the recommended design approach.

Keywords: design, pipe liner, local bending, sewer rehabilitation, full scale laboratory testing

INTRODUCTION

The use of trenchless technology to repair damaged rigid sewers is often a competitive alternative to conventional pipeline replacement methods. Trenchless rehabilitation may involve placing a liner within a damaged sewer, or "host" pipe. Both the hydraulic and structural integrities of the sewer can then be restored. This technique minimizes excavations; therefore, disturbance to vehicle traffic and construction costs can be reduced. Currently, buckling caused by groundwater and earth pressures are considered as the controlling limit states (e.g. ASTM F1216-93 [1]). There has been considerable debate regarding the potential of the liner to buckle under earth loads. While there is now general agreement that earth pressures will not induce buckling instability in normal liner installations, the real action of earth pressures on the lined sewer has not been clear, and no design approach has been available to account for earth load effects. This paper summarizes a recent study examining bending in polymer liners due to earth loads and introduces a design approach to quantify these local bending strains to account for this performance limit.

MECHANICS OF SOIL-HOST PIPE-LINER BEHAVIOR

Repaired sewer components

The static behavior of a repaired sewer involves three components: the soil, the host pipe, and the liner. Soil serves dual purposes. It transfers load to the repaired sewer, while providing support against sewer deformations. The host pipe being repaired is generally rigid, and is typically composed of vitrified clay or concrete. Sewer pipes with classic "overloading fractures" are the focus of attention in this study, Figure 1. These longitudinal fractures are located at the crown, invert, and springlines of the host pipe. Bending stiffness in a rigid pipe (resistance against decrease of vertical diameter and increase of horizontal diameter) is clearly compromised after damage of this type occurs. However, as long as the fractured pipe segments are in contact with each other, the host pipe will retain its hoop stiffness, so that it resists circumferential shortening. Further deterioration from processes such as infiltration may result in movement of the host pipe segments and widen the fractures, therefore, eliminating these contacts. Hoop stiffness of the host pipe will then be compromised. The liner installed is usually a flexible pipe such as high density polyethylene (HDPE), polyvinylchloride (PVC), or resin impregnated fabric structure. Both plain and profiled pipes can be used as the liner.

Construction techniques

The host pipe-liner system assumed in this study is a "tight fitting system", i.e. following construction, the liner sits perfectly within the host pipe, which can still support thrust. This configuration can develop when the damaged sewer is repaired by construction techniques such as slip-lining, cured-in-place, and deform-reform. For the case of slip-lining (e.g. ASTM F585 – 94 [2]), the flexible pipe liner is simply pulled in place within the host pipe. A certain amount of clearance between the liner and the host pipe is necessary to minimize interference and friction during insertion. This spacing is often eliminated by grouting. Conversely, both cured-in-place (e.g. ASTM F1216-93 [1]) and deform-reform (e.g. ASTM F1606-95 [3]) techniques result in a liner fitted snugly inside the host pipe, or with a small gap as a result of shrinkage in the cured-in-place system, or only partial contact between a deform-reform liner and an irregularly shaped host pipe. While it is now understood that a small gap between the host pipe and the liner can significantly reduce liner buckling strength under external fluid pressure [4], it will be demonstrated in a subsequent section that it is conservative to neglect these small gaps for the bending limit state being examined here. A grouted slip-lined system will also behave in this manner, once ground disturbance cracks the grout at locations where it crosses the longitudinal host pipe fractures.

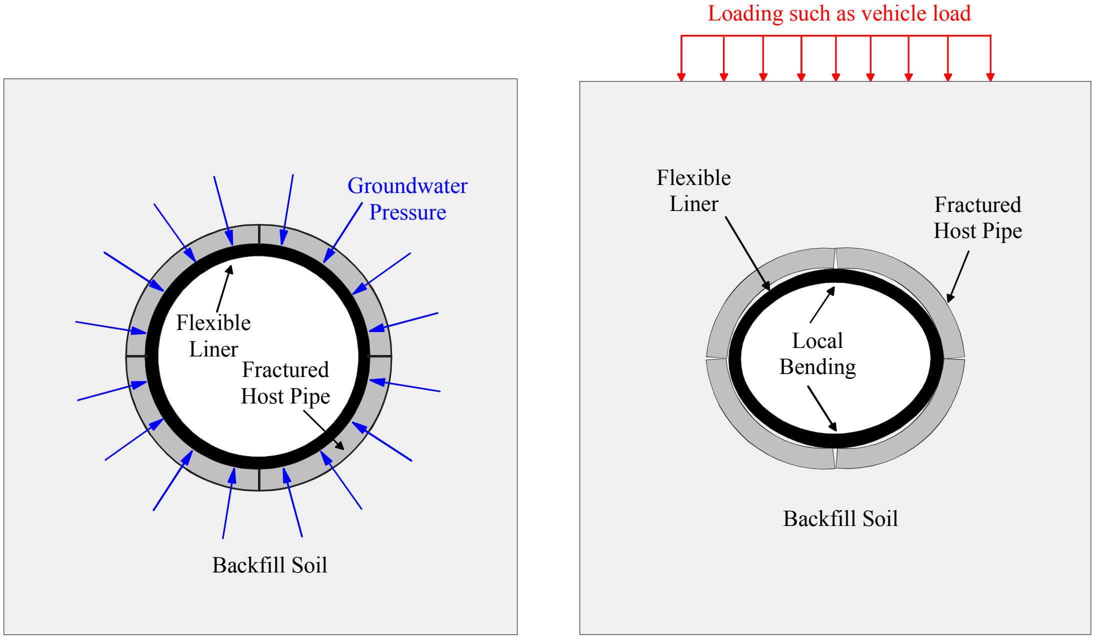
Figure 1 – Rigid Pipe with Overloading Fractures
(with permission, [5])

Loads on repaired sewers

When a liner is inserted into a host pipe which has already deformed under earth pressures, hoop thrust will be generated by the external groundwater pressure, which may cause buckling (Figure 2a). However, the liner will not experience any earth loads unless the host pipe is disturbed.

The host pipe could be disturbed by surface loadings such as additional backfill or vehicle loads. In that case, the repaired system will experience an increase in both vertical and horizontal pressures. Most soils have coefficient of lateral earth pressure less than 1 ($k < 1$), so the increase of vertical pressure on the repaired sewer will be higher than the horizontal pressure increase, causing deformations like those illustrated in Figure 2b. Both hoop thrust and bending moment must be carried by the repaired sewer. For situations where a trench is excavated in the vicinity of the repaired sewer, there will be a decrease in horizontal stress, again resulting in a deformation shape like that shown in Figure 2b.

A recent test at Queen's University shows that a simulated partially damaged sewer deforms under the full overburden pressure when the damage worsens, so that the sewer loses its remaining bending stiffness [6]. Disturbance to the liner resulting from further deterioration of the host pipe can be equated to the effect of applying the full overburden pressure to a lined-fully damaged host pipe.



a – Hoop Thrust Resulting from Groundwater Pressure

b – Thrust and Bending Caused by Surface Loads or other Ground Disturbance

Figure 2 – Loads on the Soil-Host Pipe-Liner System

Deformation mechanism

When a tight fitting host pipe-liner system is subjected to disturbance, the fractures in the host pipe expand at the inner surface of the host pipe at the crown and invert, and the outer surface at the springlines (Figure 3). Yet, contact at one point on the host pipe wall is maintained at each fracture, forming a hinge mechanism. Despite deformations of the host pipe, it is still capable of carrying thrust through those contacts. The “separating” motion at the fractures in the host pipe puts the inner edges of the host pipe and the liner in direct contact at both crown and invert, causing local bending in the liner.

An angle, θ, is used here to quantify this “angular expansion” of the fractures. Due to the geometry of the problem, the flexural response of this host pipe-liner system is expected to be governed by plane strain conditions. The nature of the local bending in the liner has been investigated in a laboratory test program discussed in a subsequent section.

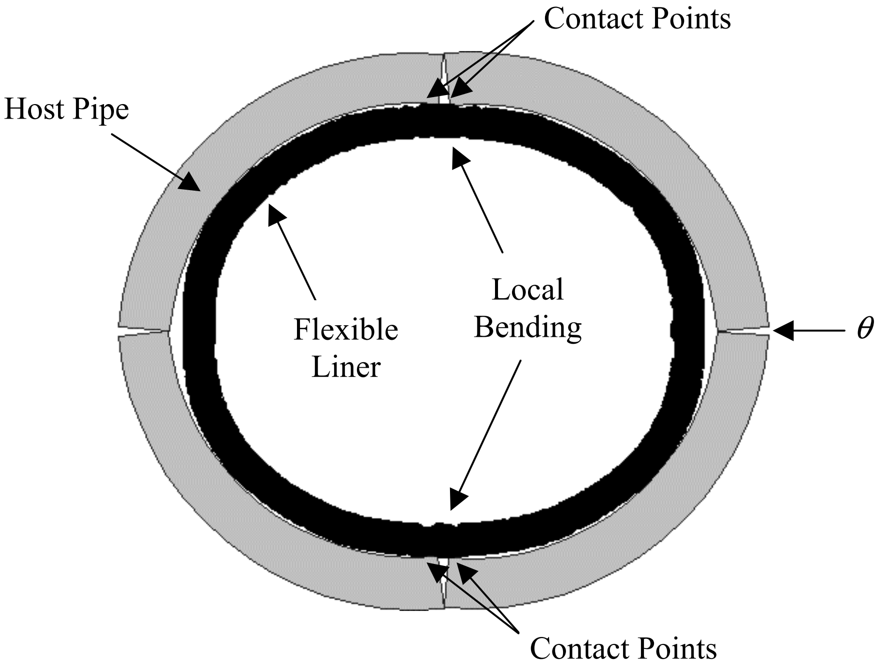


Figure 3 – Deformed Shape for a Tight Fitting Host Pipe-Liner System

Limit states

Current pipe liner design standards (e.g. ASTM F1216-93 and F1606-95) identify buckling due to groundwater and earth pressures as the important performance limits. The choice of which performance limit to consider is dictated by an assessment of this structural condition. 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. In that case, it is only designed to prevent buckling due to groundwater pressures. On the other hand, if the pipe is denoted "fully deteriorated", it is assumed to be structurally compromised. The liner is then also designed to prevent buckling under earth and live loads, using the semi-empirical relationship reported by Glascock and Cagle [7]. The designation of pipes as "fully deteriorated" and the potential for liner buckling under earth pressures is the subject of considerable debate in the industry.

Liner placed within host pipes having the longitudinal fractures at the crown, invert, and springlines seen in Figure 1 will be subjected to local bending if there is disturbance to loads or structural condition, regardless of whether the host pipe is designated as "partially deteriorated" (diameter changes less than 10%) or "fully deteriorated" (diameter changes greater than 10%). This paper describes an investigation of the liner bending mechanism. The relevance of this mechanism to host pipes classified as "partially" or "fully" deteriorated will be discussed in a subsequent section.

MEASUREMENTS OF LOCAL STRAINS AND DEFORMATIONS

Full scale laboratory tests have been performed to investigate the behavior of the tight fitting host pipe and liner system buried in soil [8]. A host pipe (370mm outer diameter and 25mm thick) was cast in concrete and pre-fractured at the crown, invert, and springlines. A plain 320mm SDR 26 HDPE pipe liner was instrumented with resistance strain gauges at two test sections, A and B, to measure curvature change and hoop strain. In addition, displacement transducers were installed in the host pipe to monitor angular expansion. Two tests were performed in a biaxial pipe test cell. With its stiff sidewalls, the biaxial cell can simulate close to field conditions [9]. Test 1 featured the idealized repaired sewer sample buried in the soil. Vertical pressure was applied at the top surface of the test cell using an air bladder system. To study the effect of the host pipe on the flexural behavior of the liner, a comparison test, Test 2, was performed. This test featured the same liner as Test 1, but tested alone in the soil (without the host pipe).

Settlement plates were installed in the backfill to monitor backfill deformation, and earth pressure cells were used to measure increase in vertical and horizontal pressures during the tests. Using these test results, secant modulus of the backfill soil, E_s, can be estimated for these conditions of negligible lateral soil strain [10]. Also, Poisson's ratio, v_s, at each load step was estimated from $v_s = k / (1 + k)$, based on elastic modeling of the soil and k, the lateral earth pressure coefficient. Backfill soil modulus, lateral earth pressure coefficient, and Poisson's ratio for Tests 1 and 2 are summarized in Table 1.

Table 1 – Soil Parameters for Test 1 and Test 2

Applied Pressure (kPa)	Test 1			Test 2		
	Modulus (kPa)	k	Poisson's Ratio	Modulus (kPa)	k	Poisson's Ratio
60	2490	0.40	0.29	2450	0.38	0.27
115	3320	0.42	0.29	3270	0.40	0.28
170	4020	0.43	0.30	3920	0.41	0.29

Angular expansion values in Test 1 were 2.0° at 60 kPa, 2.7° at 115 kPa, and 3.5° at 170 kPa. Figure 4 shows interior and exterior circumferential strains for the two tests. Resistance strain gauge measurements at 170 kPa overburden pressure are shown (tension is positive). Readings from both sections are largely similar. However, interior strains at the invert of section A for Test 2, where the pipe liner was buried alone without the host pipe (shown by open circles), are not symmetric about the vertical diameter. It appears that there was a zone of low stiffness soil under the haunch on one side of the liner.

In Test 1, strain readings of the liner at the inner and the outer surfaces across the fractures in the host pipe are approximately equal and opposite, and these indicate that the liner was almost responding in pure bending at those locations when the host pipe was present. This is reasonable, since thrust was largely carried in the host pipe segments and was transferred between them at the contact points (at crown, invert, and springlines).

At crown and invert, the strain pattern was dominated by local bending caused by the "separating" motion of the host pipe segments and their contact with the liner (Figure 3). In contrast, the compressive strain was lower at the springlines when the liner was placed within the host pipe, again, since most of the thrust is carried by the host pipe.

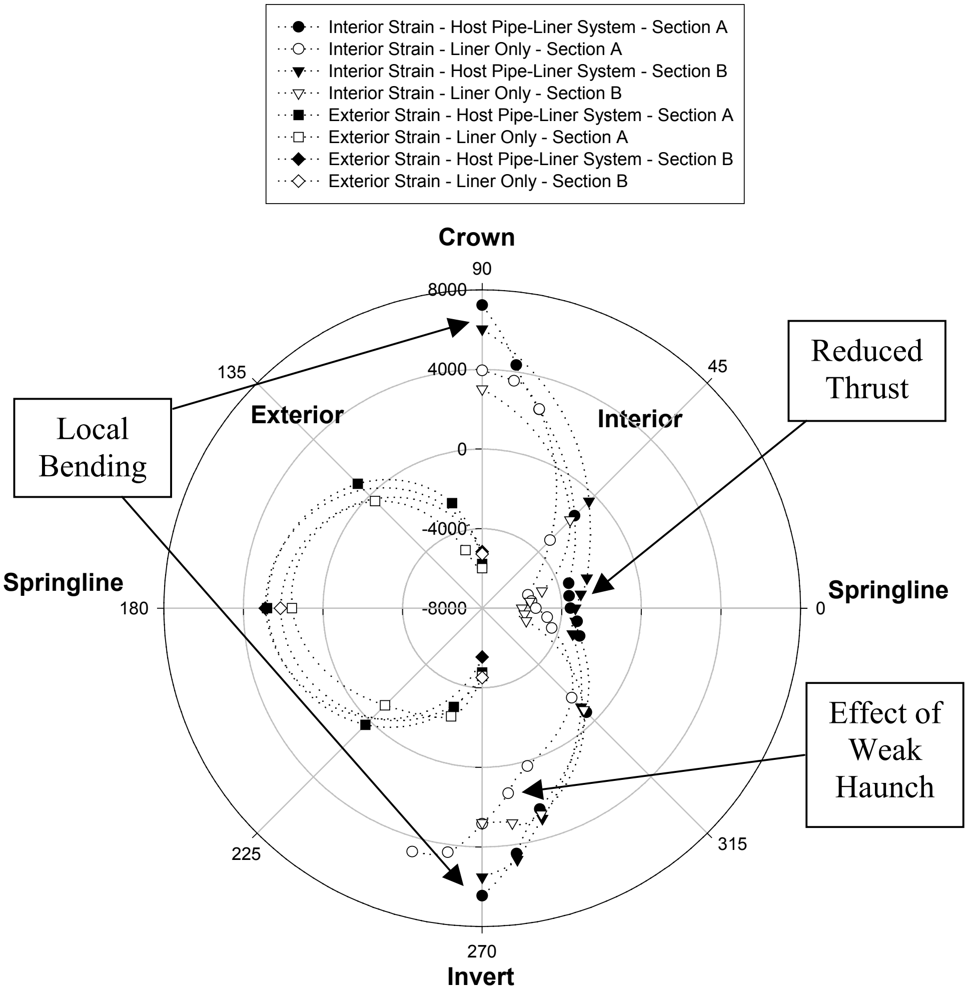
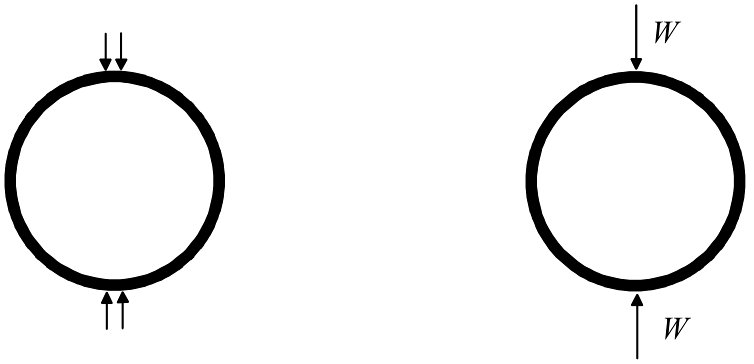


Figure 4 – Circumferential Strain Readings at 170 kPa Overburden Pressure
(Test 1 – Host Pipe-Liner System and Test 2 – Liner Only)

DESIGN APPROACH

In current liner design practice, the performance limits of concern involve buckling due to earth and water pressures. While it is important to ensure stability under external water pressure, buckling as a result of external earth pressures is not possible for a tight fitting structure where the damaged host pipe is still able to support thrust. However, any disturbance to the soil-host pipe-liner system due to changes in earth pressure or further damage to the host pipe after the liner is installed, results in host pipe-liner deformations, Figure 3. These lead to strains in the liner, and Figure 4 reveals that the presence of the host pipe almost doubles the strains that are induced by the response of the soil-host pipe-liner system. Strains are concentrated where the liner spans the fractures in the host pipe, at the crown and invert, and this local bending should be taken into consideration during design to ensure the maximum strains that occur are under the material limits. A new design approach is introduced in this section which quantifies this local bending in the liner.

The static response of a repaired sewer can be separated into two interaction components. The stiffness of both the soil and the repaired sewer controls the sewer deflections, while the local strains that develop in the liner depend on the magnitude of those deflections. Direct contacts between the host pipe segments and the liner at the crown and invert result in two pairs of concentrated loads being imposed on the liner, Figure 5a.



a – Actual

b - Approximation

Figure 5 – Host Pipe-Liner Interaction Idealization

The test discussed earlier demonstrated that the angular expansion of the fractures at these positions, θ, remains small (3.5° at 170 kPa), so that the forces imposed at each edge of the crown and invert fractures remain close together. Calculations of liner deformation and strain can therefore be undertaken treating these force pairs as single forces W at the crown and invert (Figure 5b). Theoretical solutions available for a ring under parallel plate loading, Equations 1 and 2 [11], can be used to determine the decrease in vertical pipe diameter, D_v, and bending moments at crown and invert, M_{cr} and M_{in}.

$$D_v = \frac{2W\overline{R}_{liner}^3}{E_{liner}I_{liner}}\left(\frac{\pi}{8}-\frac{1}{\pi}\right) = 0.1488\frac{W\overline{R}_{liner}^3}{E_{liner}I_{liner}} \quad (1)$$

$$M_{cr} = M_{in} = \frac{W\overline{R}_{liner}}{\pi} \quad (2)$$

where $\overline{R}_{liner}$ is the mean radius of the liner and $E_{liner}I_{liner}$ is its flexural rigidity.

The relationship between bending moment and bending strain (ε) can be expressed as

$$\varepsilon_{cr} = \pm \frac{M_{cr} c}{E_{liner} I_{liner}} \tag{3}$$

where c is the distance from the neutral axis to the extreme fiber. For a plain pipe liner, c is half the liner thickness ($c = t_{liner} / 2$). By rearranging Equations 1, 2, and 3, the magnitude of maximum bending strain in the liner at the crown and invert can be expressed as a function of vertical deflection.

$$\varepsilon_{cr} = \varepsilon_{in} = \pm \frac{D_v c}{2\pi \overline{R}_{liner}^2 \left(\frac{\pi}{8} - \frac{1}{\pi} \right)} = \pm \frac{2.139 D_v c}{\overline{R}_{liner}^2} \tag{4}$$

To determine the deflections of a sewer pipe under earth pressures, the solution of Hoeg [12] can be used. It has been demonstrated elsewhere that the flexural stiffness of the fractured host pipe is similar to an idealized flexible pipe, where hoop stiffness is very high but bending stiffness is negligible [13]. Moore [14] has applied these stiffness limits to Hoeg's solution, finding an expression for the decrease in vertical pipe diameter of a repaired sewer

$$D_v = \frac{8 \Delta \sigma_v R_{host} (1-k)(1-v_s)(1+v_s)}{(3-2v_s) E_s} \tag{5}$$

where $\Delta\sigma_v$ is the change in vertical stress, R_{host} is the external radius of the host pipe, k is the coefficient of lateral earth pressure, v_s and E_s are the Poisson's ratio and Young's modulus of the backfill soil respectively. By combining Equations 4 and 5, bending strains in the liner at the crown and invert can be obtained from the change in vertical stress.

$$\varepsilon_{cr} = \varepsilon_{in} = \pm \frac{4 \Delta \sigma_v R_{host} (1-k)(1-v_s)(1+v_s) c}{\pi \overline{R}_{liner}^2 \left(\frac{\pi}{8} - \frac{1}{\pi} \right) (3-2v_s) E_s}$$

$$= \pm \frac{17.116 \Delta \sigma_v R_{host} (1-k)(1-v_s)(1+v_s) c}{\overline{R}_{liner}^2 (3-2v_s) E_s} \tag{6}$$

Figure 6 shows the predicted bending strain values obtained using Equation 6 (solid lines) and strains measured in the liner at the crown and invert in Test 1, plotted against applied vertical pressure (tension positive). Soil parameters given in Table 1 and the sample geometry mentioned earlier are used for the calculations. Measured strains shown here have been adjusted up by the factor of 1.43 developed by Brachman [9] to account for wall stiffening caused by the adhesive used when gluing strain gauges onto

HDPE. The non-linear increase in strain with overburden pressure is due to strain hardening (modulus increases) of the backfill soil.

Equation 6 captures the trend of the measured data well at overburden pressure less than 150 kPa, except for the interior strain reading for the crown at section A. Discrepancies between the predicted values and strain measurements likely result from the use of the parallel plate approximation for the host pipe-liner interaction, and because contact forces between the host pipe and the liner have been neglected (at the shoulders and haunches for example).

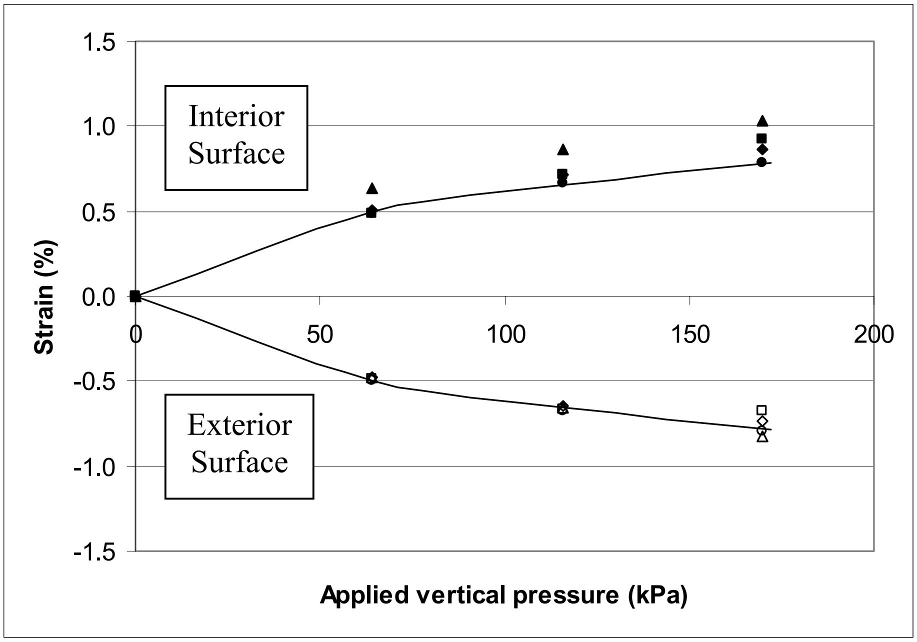


Figure 6 – Predicted and Measured Strains in the Liner at Crown and Invert

DISCUSSIONS

The laboratory test results provided in this paper are based on one specific sample, testing procedure and configuration, and tight fitting sewer repairs can feature many material and geometrical conditions. The equations derived here to capture local bending in the liner are based on ring theory, so are dependent on its assumptions. First, the liner is assumed to be circular. Pipe wall thickness is assumed to be uniform, and thin in relation to the radius. Consideration of thick ring theory [15] indicates that Equation 6 may be unconservative for thick rings. The tests used here to investigate this phenomenon were conducted on a liner with DR = 26. Design using this equation should include additional strain enhancement if $DR \leq 20$.

For a tight fitting system, the fracture type investigated in this study is considered to produce the lowest bending stiffness where four fractures occur in the host pipe. If the host pipe suffers from other four-fracture configurations, its bending stiffness should increase. To achieve the same amount of deflection, higher pressure would be necessary, resulting in higher concentrated loads (W) and higher bending strains in the liner. Equation 4 will then provide unconservative measures of bending strain, for a given level of vertical diameter decrease. However, those same alternative fracture patterns will have increased bending stiffness in the host pipe-liner composite, leading to a decrease in pipe deflection. As a result, the expression relating change in vertical pressure and deflection, Equation 5, is expected to be very conservative for other fracture patterns. The combination of these two expressions to form Equation 6 will lead to a more reasonable result, since these errors associated with stiffer fracture patterns will partly compensate for one another. Equation 6 is expected to be somewhat conservative for alternative fracture patterns, based on the premise that increased host pipe stiffness will shield the liner, though this should perhaps be the subject of further study.

When designing for the liner to sustain bending strains due to earth pressures, a decision is required regarding the incremental overburden pressure $\Delta\sigma_v$ to use in Equation 6. It is obvious that a liner is usually installed to restore the function of an already fractured host pipe, so that no liner strains will result unless there is subsequent disturbance. However, the liner may pass through certain sections of the host pipe where the fractures are only starting to develop, and where pipe deformations have not commenced or have only partly begun. Those sections then control the design, since local bending can develop as the host pipe deteriorates during the design life of the repair, and pipe deformations then develop under the overburden pressure. Safe designs could then be obtained using the full value of overburden pressure. Designers of some structures, such as culverts at shallow cover, might choose to employ incremental earth pressures associated with live loads only. This decision would require the fractured culvert to have fully deformed under the weight of overlying soil, prior to lining.

It is well established that buried flexible pipes utilize both the stiffness and strength of the surrounding ground to resist deformations and distribute the overburden pressures around the flexible structure, e.g. Kay and Abel [16] and Moore [14]. An examination of soil-structure interaction equations like those developed by Hoeg [12] and Spangler [17] demonstrates that the flexural properties of most buried flexible structures have little affect on the resulting changes in pipe diameter. The use of Equation 5 will be conservative for less flexible liners where $E_{liner}I_{liner}/E_sR^3 > 0.01$, and a less conservative calculation can be obtained using the Hoeg or Spangler equations instead of Equation 5. For the SDR26 HDPE pipe liner used in this study, the absence of the structural stiffness term in Equations 5 and 6 increases the calculated deflections and liner strains by less than one percent.

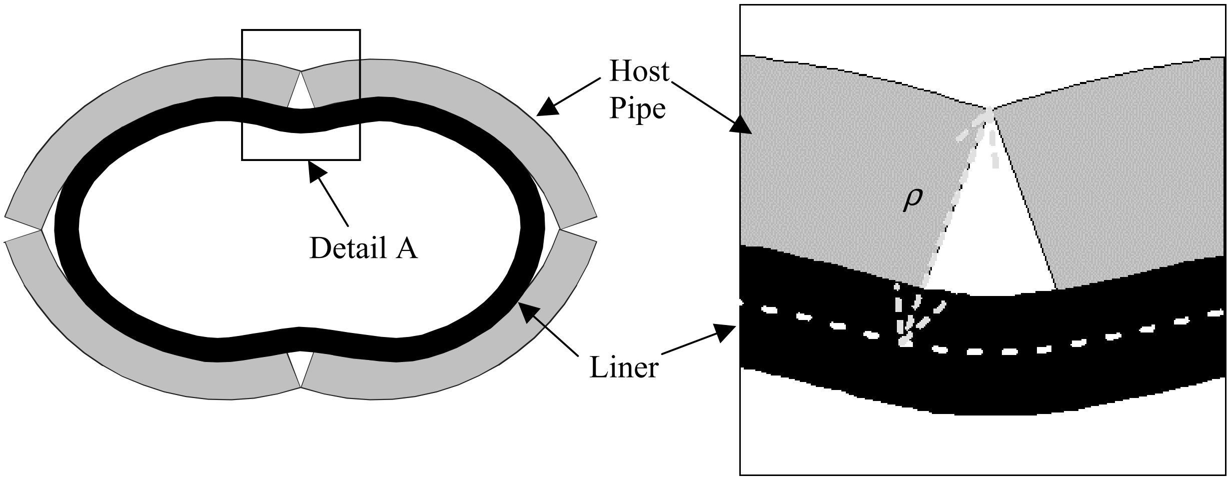
One convenient feature of flexible liner design using Equation 6 is that liner modulus does not influence the liner-host pipe-soil interaction, and there is no need to evaluate the time dependent modulus of the polymer material.

Testing and analysis undertaken in this study is based on the assumptions that the liner is not bonded to the host pipe and that residual stresses in the liner after installation are negligible. The local bending mechanism examined here and quantified in Equation 6 is also based on the assumption that there is no initial gap between liner and host pipe. The only effect of a small initial gap would be to delay the application of vertical forces W at crown and invert as the host pipe deforms under ground disturbance, so Equation 6 would produce slightly conservative strain estimates.

Further examination on the host pipe-liner deformation mechanism reveals that there is a limit to the value for bending strain that can develop at the crown and invert in the liner, Figure 7. This upper bound value is controlled by the curvature geometry,

$$\varepsilon_{\max} = \frac{t_{liner}}{2}\frac{1}{\rho} = \frac{t_{liner}}{2\left(t_{host} + {}^{t_{liner}}\!/_{2}\right)} \tag{7}$$

where ρ is the local radius of curvature, which equals the thickness of the host pipe t_{host} plus half the liner thickness. Equation 7 indicates that the maximum strain that can develop in the liner tested in this study is about +/- 19.5%. Based on the trend observed in Figure 6, strains are not expected to reach this kinematic limit (i.e. it is unlikely that the liner will deform sufficiently to contact the host pipe adjacent to the fracture edges).



a – Deformed Shape b - Detail A

Figure 7 – Host Pipe-Liner System under High Deformations; Peak Local Bending

In all of the preceding discussion, there is no indication of any difference in the mechanism of liner bending for host pipes currently classified as “partially” or “fully” deteriorated, as long as the segments of host pipe are in contact across the fractures. It appears, therefore, that this mechanism is relevant to both levels of pipe distress. This work to develop a rational design limit to account for earth pressure effects in pipe liners provides evidence to support calls to discard the current condition classifications, so that all damaged sewers are evaluated for all potential limit states.

IMPLEMENTATION IN DESIGN

The deflection of the repaired sewer is a function of the backfill soil modulus. However, determining a suitable value may be difficult as many factors, such as material type and construction procedure, can alter the modulus significantly. Designers can make use of some published data for guidance. For instance, McGrath et al [18] proposed the design values of one-dimensional backfill soil modulus (M_s) for three general classes of soil (SW, ML, and CL) at different levels of vertical stress. The relationship between E_s and M_s is expressed as

$$E_s = \frac{M_s(1+v_s)(1-2v_s)}{(1-v_s)} \qquad (8)$$

In terms of M_s, Equation 6 becomes

$$\varepsilon_{cr} = \varepsilon_{in} = \pm\frac{17.116\Delta\sigma_v R_{host}(1-k)(1-v_s)(1+v_s)c}{\overline{R}^2_{liner}(3-2v_s)} \cdot \frac{(1-v_s)}{M_s(1+v_s)(1-2v_s)}$$

$$= \pm\frac{17.116\Delta\sigma_v R_{host}(1-k)(1-v_s)^2 c}{\overline{R}^2_{liner}(3-2v_s)(1-2v_s)M_s} \qquad (9)$$

For typical values of $v_s = 0.3$ and $k = 0.4$,

$$\varepsilon_{cr} = \varepsilon_{in} = \pm\frac{17.116\Delta\sigma_v R_{host}(1-0.4)(1-0.3)^2 c}{\overline{R}^2_{liner}(3-0.6)(1-0.6)M_s}$$

$$= \pm\frac{5.242\Delta\sigma_v R_{host} c}{\overline{R}^2_{liner} M_s} \qquad (10)$$

Equations derived in this study can be used for both plain and profiled liners, by specifying *c*, the distance from the neutral axis to the extreme fiber.

CONCLUSIONS

Recent research work to investigate the static behavior of the tight fitting host pipe-liner system has been presented. Various liner design standards commonly adopted have been reviewed, and a new design approach has been introduced to capture the local bending strains that develop in the liner due to earth loads. Results from two full scale laboratory tests indicate that local bending was observed in the liner at the crown and invert, where the host pipe segments are in direct contact with the liner. The relationship between applied vertical pressure and local bending strains in the liner is established using a parallel plate loading analogy to idealize the host pipe-liner interaction. Further studies are planned to examine the liner design approach on systems where the damage in the host pipe is more severe.

ACKNOWLEDGEMENTS

This work has been supported by the Natural Sciences and Engineering Research Council of Canada through research funds granted to Dr. Ian D. Moore and through a PGS-B scholarship to T.C. Michael Law. Dr. Moore's position at Queen's University is funded by the Canadian Government through the Canada Research Chairs Program.

REFERENCES

1. ASTM (1998), 'Standard practice for rehabilitation of existing pipelines and conduits by the inversion and curing of a resin-impregnated tube', F1216 – 93, ASTM, Philadelphia, PA.

2. ASTM (1998), 'Standard practice for insertion of flexible polyethylene pipe into existing sewer', F585 – 94, ASTM, Philadelphia, PA.

3. ASTM (1998), 'Standard practice for rehabilitation of existing sewers and conduits with deformed polyethylene liner', F1606 – 95, ASTM, Philadelphia, PA.

4. El Sawy, K. and Moore, I.D. (1997), 'Parametric study for buckling of liners: effect of liner geometry and imperfections', Trenchless Pipelines Projects: Practical Applications; ASCE, Boston MA, pp 416 – 423.

5. Digital Sewer Damage Catalogue due to the German Worksheet ATV-M 143-2 (08.01).

6. Lee, J.P.K. (2002) 'Investigation of Soil-Damaged Rigid Pipe Interaction under Tunneling Load Path', Master of Science (Engineering) Thesis, Queen's University, Kingston, Canada.

7. Glascock, B.C. and Cagle, L.L. (1984), 'Recommended design requirements for elastic buckling of buried flexible pipe', Proceedings of the 39th Annual Conference, Reinforced Plastic/Composites Institute, The Society of the Plastic Industry, Inc.

8. Law, T.C.M. and Moore, I.D. (2002) 'Laboratory Investigation on the Static Response of Repaired Sewers', Proceedings of the ASCE Pipelines 2002 Conference, Cleveland, Ohio, USA, 2002.

9. Brachman, R.W.I. (1999), 'Mechanical performance of landfill leachate collection pipes', PhD Thesis, Department of Civil and Environmental Engineering, The University of Western Ontario, London, Canada.

10. Dhar, A.S. (2002) 'Limit States of Profiled Thermoplastic Pipes under Deep Burial', PhD Thesis, Department of Civil and Environmental Engineering, The University of Western Ontario, London, Canada.

11. Young, O.C. and Trott, J.J. (1984) 'Buried Rigid Pipes: Structural Design of Pipelines', Elsevier Applied Science Publishers, 234 pp.

12. Hoeg, K. (1968) 'Stress Against Underground Cylinder', Journal of Soil Mechanics and Foundation Engineering, ASCE, Vol. 94, SM4, pp 833-858.

13. Law, T.C.M. and Moore, I.D (2002) 'Kinematic Response of Damaged Rigid Sewer under Earthload', Proceedings of the 55th Canadian Geotechnical Society Annual Conference, Niagara Falls, Ontario, Canada, Oct 20-23, 5 pp.

14. Moore, I.D. (2000) 'Buried Pipe and Culverts', Chapter 18, Geotechnical and Geoenvironmental Handbook, Edited by R.K. Rowe, Kluwer Publisher, pp 541-568.

15. Moore, I.D. (1985), 'The Stability of Buried Tubes', PhD Thesis, Department of Civil Engineering, The University of Sydney, Australia.

16. Kay, J.N. and Abel, J.F. (1976), 'A Design Approach for Circular Buried Conduits', Transportation Research Record 616, pp 78-80.

17. Spangler, M.J. (1956), 'Stresses in Pressure Pipe-lines and Protective Casting Pipes', Journal of Structural Engineering, ASCE, Reston, VA, 82:pp 1-33.

18. McGrath, T.J., Selig, E.T., Webb, M.C., and Zoladz, G.V. (1999) 'Pipe Interaction with the Backfill Envelop', Publication No. FHWA-RD-98-191, Federal Highway Administration, US Department of Transportation, 269 pp.