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ASSESSMENT OF DAMAGE TO RIGID SEWER PIPES AND EROSION VOIDS IN THE SOIL, AND IMPLICATIONS FOR DESIGN OF LINERS

Ian D. Moore[1]

[1] GeoEngineering Centre at Queen's-RMC, Queen's University, Kingston, ON

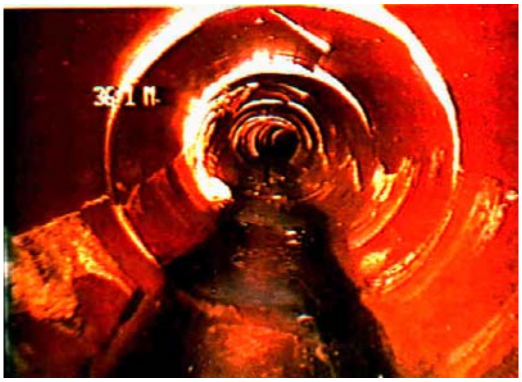
ABSTRACT: CCTV images are taken routinely during inspections of damaged rigid sewers, and assessments are made regarding the need for pipe repair. One key challenge with visual assessment inside the pipe has been estimation of the condition of the surrounding ground, and the possibility of erosion voids in the vicinity of the damaged sewer resulting from water ingress through joints and fractures. Furthermore, the effect on liner design and performance of the visible damage to the pipe structure and the invisible damage to the surrounding ground has not been quantified.

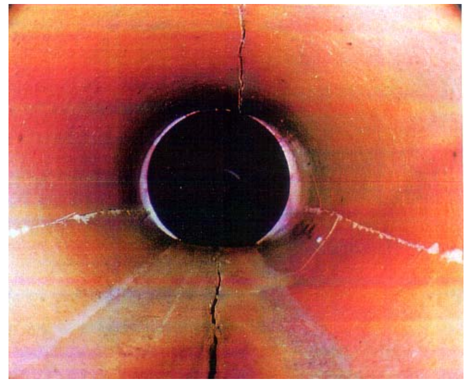
New findings are reported quantifying the effect of erosion voids on pipe fracture and post-fracture deformations. Calculations are made regarding increases in bending moment in rigid pipe as water inflow leads to erosion voids adjacent to the pipe. The increases in horizontal sewer diameter are then used to infer the extent of backfill erosion. Two potential procedures are proposed for quantifying the extent of damage to the backfill, based on the level of deformation observed in the fractured pipe. One procedure provides values of decreases in average modulus for the surrounding soil, as an index of the level of ground deterioration. The other procedure provides estimates of the amount of the outer pipe circumference that has lost ground support through erosion. The effect on liner design of the damage observed in the pipe and inferred in the soil is then discussed, with explicit reference to the ability of the liner to resist both external fluid loads and external earth pressures.

1. INTRODUCTION

Methods of rigid pipe repair have been actively under development and use for decades and much is known regarding the resistance of polymer sewer liners to both external fluid loads (e.g. Glock, 1977; Moore, 1998; El Sawy and Moore, 1997) and external earth loads (e.g. Law and Moore, 2003, 2007). Video (closed circuit television or CCTV) inspection is used routinely to determine the extent of the damage visible within the sewer (Figures 1a and 1b), and many standards exist to guide the design of liners for insertion within the sewer (e.g. ASTM F1216-06). It would perhaps seem that there is little need for further research on these topics.

However, rigid pipe failure over time can be caused by the development of erosion voids in the vicinity of the damaged sewer resulting from water ingress through joints and fractures (e.g. Tan and Moore, 2007), and the response of fractured rigid pipe to earth loads and the role of erosion on the deformations of deteriorated and repaired pipes are only now being studied (e.g. Spasojevic et al., 2006; Tan, 2007; El Taher and Moore, 2008).





a. Sewer in Toronto. b. Sewer in Germany (with permission).

Figure 1. CCTV images of damaged rigid sewers showing longitudinal fractures and changes in shape.

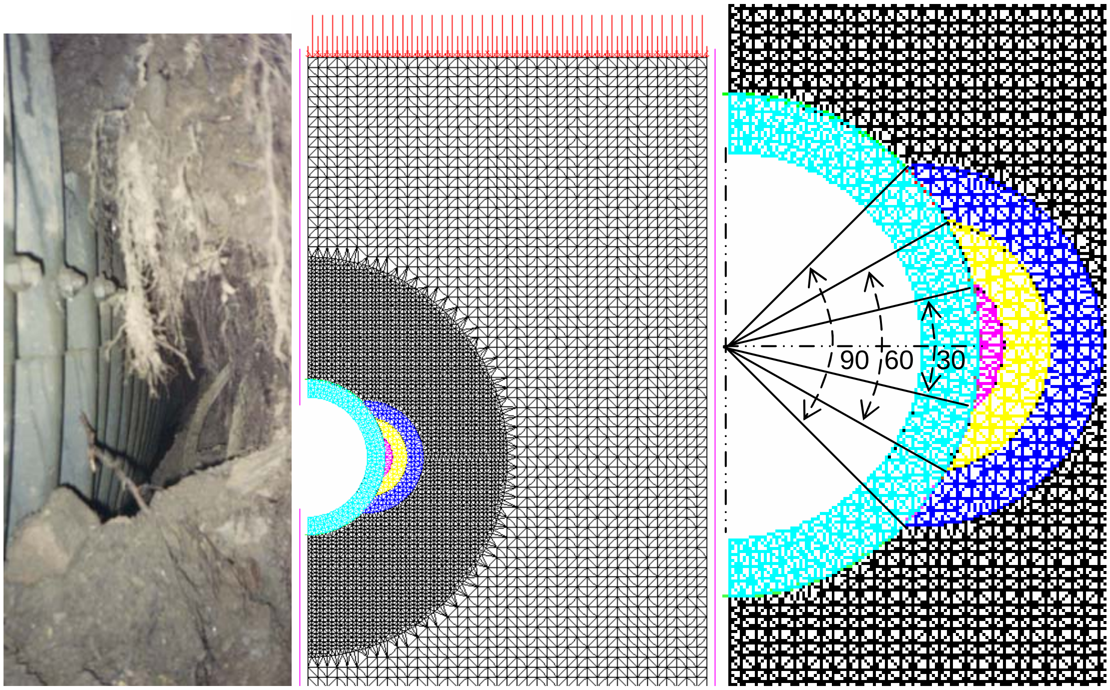
It is clear that the deterioration of soil support around a rigid pipe is the most critical feature leading to the structural damage visible during video inspections, and it is understood that the stability of the deteriorated sewer before and after repair is dependent on the composite action of the soil (intact or deteriorated), the damaged rigid pipe, and the liner within (e.g. Law and Moore, 2007). These recent findings are therefore used here to initiate a re-examination of the results of CCTV inspections, to determine what can be inferred from the extent of damage seen in the video images, and to examine how the likely consequences of both soil and pipe deterioration can be taken into consideration in the design of sewer repairs.

2. FRACTURE AND DEFORMATION OF RIGID PIPE DUE TO SOIL EROSION

While it is difficult to obtain images of erosion voids, it is reasonably certain that critical voids develop at the springline of rigid and flexible culverts, Figure 2a. These develop long before they become sufficiently large to produce sinkholes or other features visible at the ground surface. Furthermore, experiments reported by Spasojevic et al. (2006) indicate that if voids develop at the invert, they are filled by soil moving down from the springlines, so that the final result is loss of ground support at the springlines.

Tan and Moore (2007) used linear elastic and elastic-plastic finite element analyses to examine the stresses within a rigid (typically clay or concrete) sewer pipe as voids grow at the springline. Figure 2b shows a typical mesh used in their study, with three different sized circular zones at the springline used to model progressive growth of the void. In their study, the primary geometrical variable found to control pipe deformation was the angle of contact between the void and the outside surface of the pipe, α. Solutions for contact angles of zero (intact ground), 30^{o}, 60^{o} and 90^{o} voids were reported, Figure 2c.

Figure 3 shows how the maximum tensile and compressive stresses at the crown and invert were found to increase as the void expanded, for elastic-plastic modeling of the granular soil surrounding the pipe. Since most concrete and clay pipes are designed to have a factor of safety of 2 against reaching the ultimate moment capacity, it is expected that stress increases of more than 100% (which double the peak tension) are sufficient to induce longitudinal fractures in the rigid pipe. Solutions for voids of three different widths (A: wide; B: those shown in Figure 2c; C: narrow) produced somewhat different stress changes, and the level of slip or adhesion at the soil-pipe interface also has some effect. However, the dominant factor is the size of the soil void, expressed using the contact angle. Tan and Moore (2007) conclude that a critical long term failure mechanism for concrete and clay pipes is the erosion of surrounding backfill as water leaks through joints between pipe segments. Once longitudinal fractures develop, the processes of erosion and further void growth are expected to accelerate.



a. Springline void. b. Finite element mesh (void set B) c. Void geometry and contact angle (degrees)
Figure 2. Finite element analysis for rigid sewers considering erosion voids (after Tan and Moore, 2007).

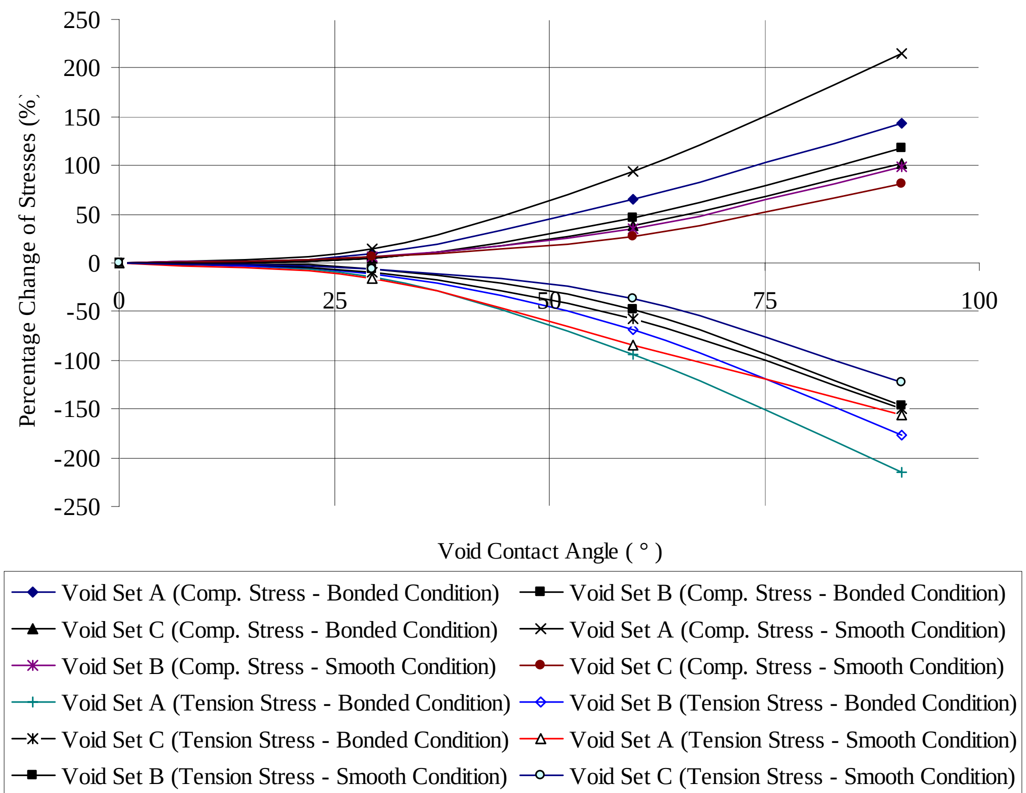
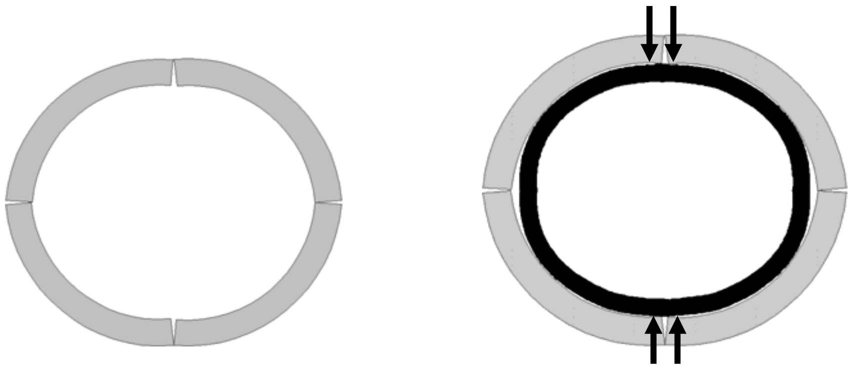


Figure 3. Increase in circumferential stresses at crown due to erosion voids, Tan and Moore (2007).



a. Fracture pattern and kinematics b. Loads on the liner due to pipe deformation

Figure 4. Four segment hinge mechanism expected for rigid pipe after fracturing (Law and Moore, 2003).

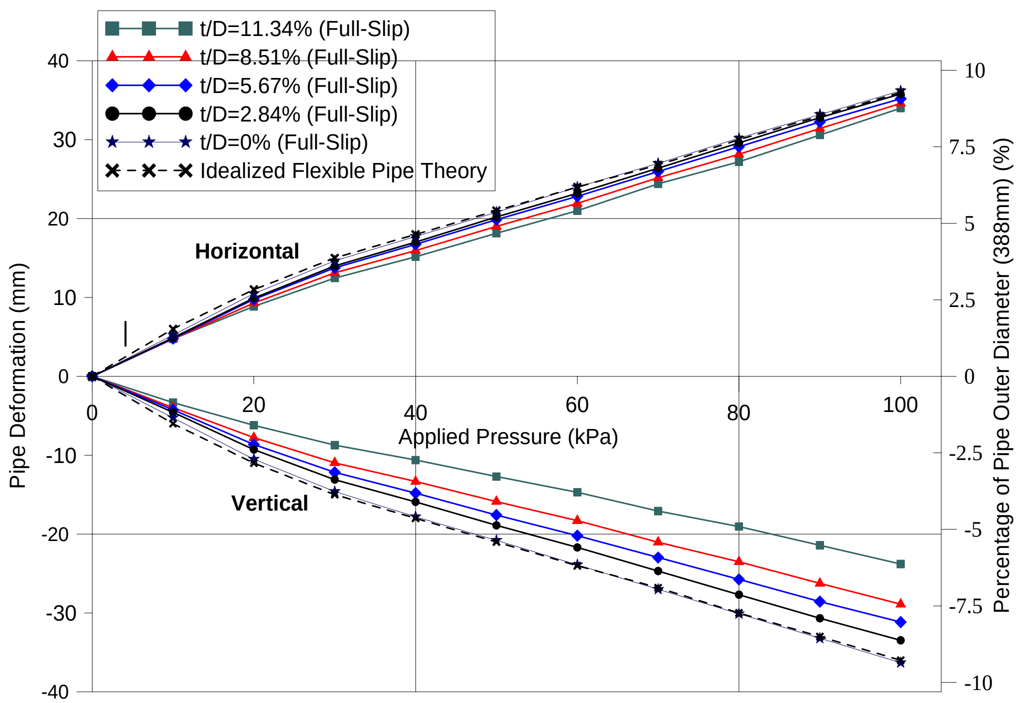


Figure 5. Fractured pipe deformation (finite element analysis and flexible pipe theory), Tan (2007).

Finite element analysis has been used to calculate the deformations (decreases in vertical pipe diameter, and increases in horizontal pipe diameter) that then result after the pipe fractures. Idealizing the pipe as fracturing into four quadrants, Figure 4a, finite element analyses were used by Tan (2007) to calculate pipe response, Figure 5. Following comparison with the kinematics of fracture pipe deformation (Law and Moore, 2003), it was found that the fracture pipe responds much like a flexible pipe; changes in horizontal pipe diameter can therefore be expressed as (adapted from Moore, 2001):

$$\Delta D_H / OD_{pipe} = 4.\sigma_V.(1-K)\frac{(1-\nu_S)}{(3-2\nu_S)(1-2\nu_S)M_S} \approx 1.6\frac{\sigma_V}{M_S}$$

$$\Delta D_H / ID_{pipe} \approx 1.6\frac{\sigma_V}{M_S}(1+\frac{2t_{pipe}}{ID_{pipe}}) \quad [1]$$

where overburden stress σ_V, lateral earth pressure ratio *K*, constrained modulus M_s and Poisson's ratio of the soil ν_s all influence the response, and the expression 1.6 is based on *K* =0.45, and ν_s=0.3. Other combinations of *K* and ν_s result in values ranging from 1.4 to 1.7. The kinematics of pipe deformation can then be used to provide a relationship between changes in vertical and horizontal diameters:

$$\Delta D_V = -\Delta D_H .(1 - \frac{2.t_{pipe}}{OD_{pipe}}) \qquad [2]$$

Further finite element analyses of fractured pipe deformations were conducted, examining the increase in pipe deformations as erosion voids like those seen in Figure 2c develop at the springlines, Tan (2007). Figure 6 presents the resulting calculations based on either elastic soil response, or elastic-plastic soil response (where shear failure in the soil around the void is considered). These calculations were performed for a fine grained backfill, where coefficient of lateral earth pressure is either K=0.5 or K=0.67. Shear strength in the elastic-plastic analysis was based on drained response, and angle of internal friction ϕ=20^o.

Figure 6 also includes two approximate relationships between void size and displacement increase which are introduced in a subsequent section.

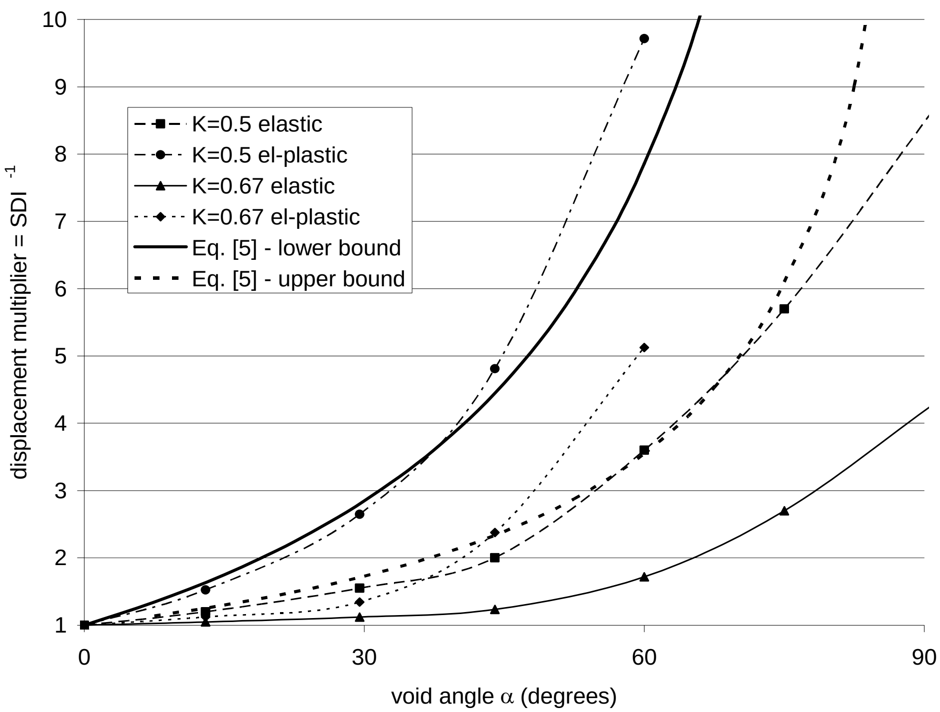


Figure 6. Displacement increase due to development of erosion voids, Moore (2008).

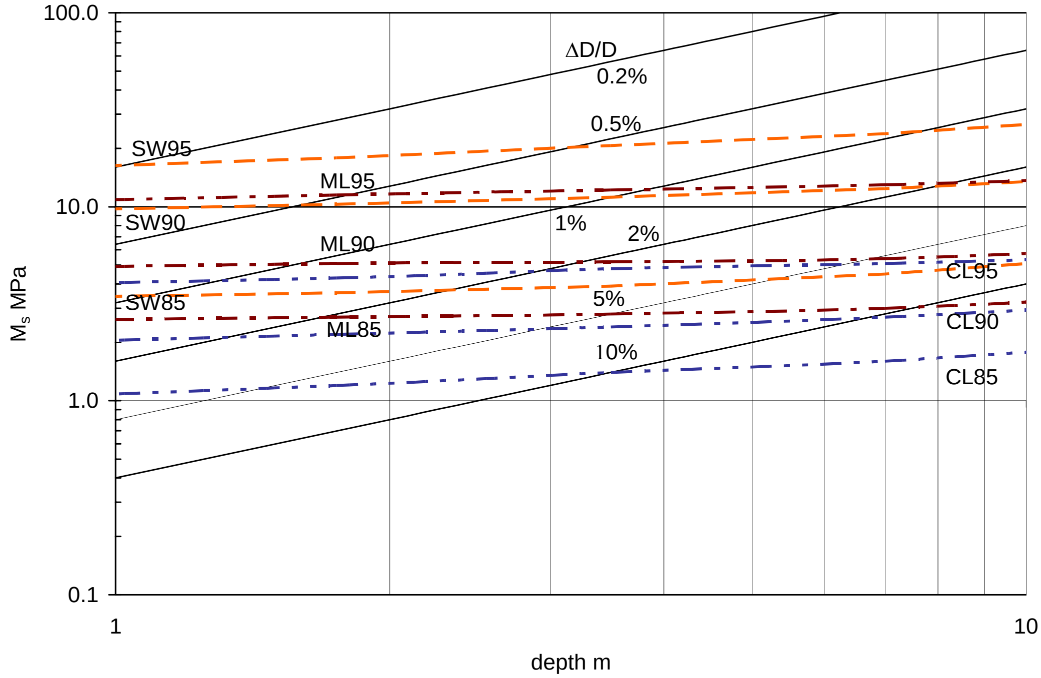


Figure 7. Constrained soil modulus and fractured rigid pipe deformations with depth, Moore (2008).

3. INTERPRETATION OF SOIL CONDITION – DETERIORATED MODULUS

Combining equations 1 and 2, the relationship is obtained between the secant value of constrained soil modulus M_s, (the modulus relating total change in pipe diameter to total value of vertical overburden stress), the change in vertical pipe diameter $\Delta D_V/D$, and the depth of burial of the pipe. Figure 6 illustrates how depth increases correspond to increases in the level of normalized pipe deformation for any given modulus. Figure 6 also includes expressions for typical constrained soil moduli for different pipe backfills, obtained from the work of McGrath et al. (2002).

This figure can be used in a number of ways to estimate the current condition of the soil surrounding the damaged sewer.

i. If the soil material used in original sewer construction is known, then the amount of deformation for a fractured rigid pipe within intact (undeteriorated) ground can be obtained from the figure, $(\Delta D_v/D)_{intact}$.

ii. The actual level of observed deformation $(\Delta D_v\ D)_{CCTV}$ can then be used to define an index quantifying the soil deterioration, denoted here as *SDI* or soil degradation index:

$$SDI \quad = \quad (\Delta D_v/D)_{CCTV} / (\Delta D_v/D)_{intact} \qquad [3]$$

iii. While soil erosion is expected to develop in a nonuniform manner, with greater degradation in some areas and less in others, a single equivalent degraded soil modulus can be found from the figure, based on the depth of burial and the level of observed deformation in the fractured pipe, $(\Delta D_v/D)_{CCTV}$

iv. The ratio of the intact soil modulus and the equivalent degraded modulus can also be used to define or calculate the soil degradation index:

$$SDI \quad = \quad M_{S.equivalent} / M_{S.intact} \qquad [4]$$

Examples of the use of these expressions are provided by Moore (2008).

4. INTERPRETATION OF SOIL CONDITION – VOID SIZE

Using the relationships between void size and pipe deformation discussed earlier in section 2 in relation to Figure 6, the size of the void can be estimated from the damage index (that is, the amount of deflection observed in a CCTV inspection). Curve fitting to the relationships shown on Figure 6, an expression has been developed between deflection increase and the size of the erosion void (defined as the angle of contact between the void and the outside surface of the pipe, α):

$$(SDI)^{-1} = \frac{A + B\alpha}{A - \alpha} \quad \text{or} \quad \alpha = \frac{A(SDI^{-1} - 1)}{B + SDI^{-1}} \qquad [5]$$

Two parameter sets were selected for the results presented in Figure 6:

i. *A* = 95, *B* = 3 which provide 'lower bound' values of void angle for a given level of displacement increase; these correspond to results for elastic-plastic ground and *K* = 0.5; and

ii. *A* = 100, *B* = 0.7 which provide 'upper bound' values of void angle for a given level of displacement increase; these correspond to results for stronger material and/or *K* = 0.67.

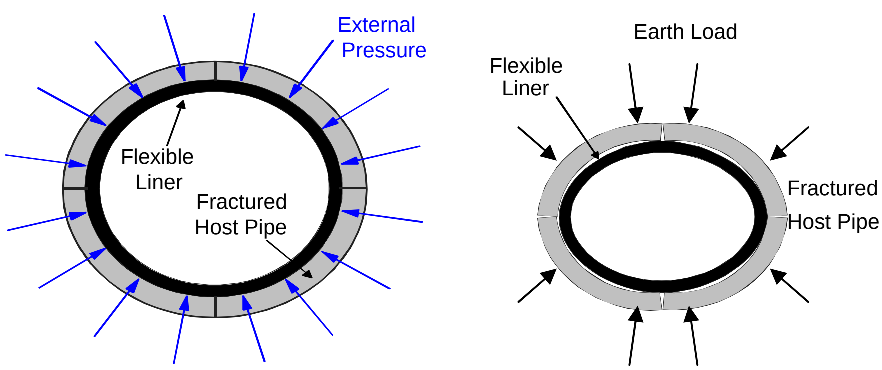
Figure 6 includes these two curve fits from Eq. [5]. Examples of the use of these expressions are provided by Moore (2008).

Now, these parameters are based on just four sets of finite element calculations, and their use provides only a preliminary estimate of void size. Further work is needed to obtain direct experimental results to evaluate the effectiveness of the finite element analysis and these curve fits.

5. INFLUENCE OF PIPE AND SOIL CONDITIONS ON LINER DESIGN

5.1 Performance limits for liner within gravity flow sewer

There are two performance limits that need to be considered when designing the polymer liner. First, the liner needs to be able to support the external fluid pressures in the surrounding ground without buckling, Figure 8a. Second, the liner needs to withstand tensile bending strains at crown and invert as the vertical diameter of the fractured sewer decreases under the influence of the earth loading, Figure 8b.



a. Liner under external groundwater pressure b. Liner bending as vertical diameter decreases

Figure 8. Performance limits for liner within gravity flow sewer, Moore (2005).

5.2 Reductions in buckling strength

For a close-fitting liner, Moore (1998, 2005) indicates that the buckling pressure is given by

$$P_b = 1.0E_{liner}\left(\frac{t_{liner}}{D_{liner}}\right)^{2.2} e^{-q/0.18} e^{-0.56\Delta/t_{liner}} \qquad [6]$$

for dimensionless ovality *q* (0.18 is changed to 18 if the percentage ovality from ASTM F1216 is used). Now, for a fractured circular sewer, dimensionless ovality can be calculated as:

$$q = \frac{D_H - D_V}{D_H + D_V} \approx \frac{\Delta D_H}{ID} \quad [7]$$

if the effect of the thickness of the old sewer is neglected (which will be conservative).

If the liner drapes over the fractured sewer at the invert, then a wavy imperfection results with amplitude

$$\Delta_{cr} \approx 0.5ID(\frac{\Delta D_H}{ID})^2(1+\frac{2.t_{pipe}}{ID})^{-1} \quad [8]$$

where *ID* is the internal diameter of the uncracked sewer pipe. This can be further simplified by ignoring the thickness of the old sewer (providing a conservative approximation).

The reduction in stability as a result of deformations in the fractured sewer are given in Figure 9 for a range of liner dimension ratios and fractured sewer deformations, based on the expression

$$e^{-\Delta D_H/0.18D} e^{-0.28\frac{D}{t_{liner}}(\frac{\Delta D_H}{D})^2} \quad [9]$$

Figure 9 indicates that reductions in stability become significant when fractured pipe deformation is greater or equal to 5%. It is not necessary to distinguish between the dimension ratio *DR=D/t* of the liner calculated using its average diameter and its external diameter, since the correction factor that results is hardly changed. Internal sewer diameter (i.e. external liner diameter) is the easiest to employ. Examples of the use of this approach are provided by Moore (2008).

5.3 Reductions in resistance to bending

Law and Moore (2003, 2007) examined the local bending that results from contacts between pipe and liner at the crown and invert, Figure 4b, and demonstrated that bending strains in the liner can be effectively calculated using the solution for a ring under parallel plate loading:

$$\varepsilon_{cr} = \varepsilon_{in} = \pm \frac{\Delta D_v c}{2\pi \bar{R}_{liner}^2 \left(\frac{\pi}{8} - \frac{1}{\pi}\right)} = \pm \frac{2.139 \Delta D_v c}{\bar{R}_{liner}^2} \quad [10]$$

where the liner has mid-surface radius of R_{liner}, and *c* is the distance from the mid-surface to the extreme fiber responding in tension. Here, the change in vertical pipe diameter ΔD_v is assumed to take place in some sections of the sewer after the liner is inserted. Use of the value of ΔD_v determined from a CCTV inspection of the deformed pipe prior to lining, Moore (2005), is recommended if, at the time of liner construction, there is a possibility of similar amounts of soil deterioration behind segments of undamaged pipe in the vicinity of the damaged pipe segments. An implicit assumption here is that the presence of the liner has a negligible effect on the amount of vertical diameter decrease, a reasonable assumption since the flexural stiffness of the liner is negligible compared to that of the soil surrounding the fractured sewer. Restoration of the hydraulic integrity of the sewer should arrest further backfill erosion (assuming the erosion results from inflows of groundwater), and it is therefore reasonable to assume that magnitudes of vertical diameter decrease that are greater than those already observed for the sewer are unlikely.

Figure 8 illustrates how the percent change in vertical diameter being considered and the allowable strain for the polymer used in the liner control the maximum thickness *t* for a plain liner (where *t=2c*). This bending limit state sets an upper bound to liner thickness to prevent excessive wall strain, since greater liner thickness increase the distance to the extreme fiber and thus the maximum bending strains. Moore (2008) illustrates the use of this approach with calculations for two example problems.

6. CONCLUSIONS

The typical form of damage in deteriorated rigid sewers is discussed, including the impact of soil erosion associated with ingress of water to the leaking sewer. This erosion decreases lateral ground support to the sewer, and bending moment increases eventually fracture the sewer. Once fractures develop, erosion likely accelerates. Finite element calculations explain the rigid pipe fracture process, and provide preliminary guidance on bending moment increases for erosion voids of specific size and shape.

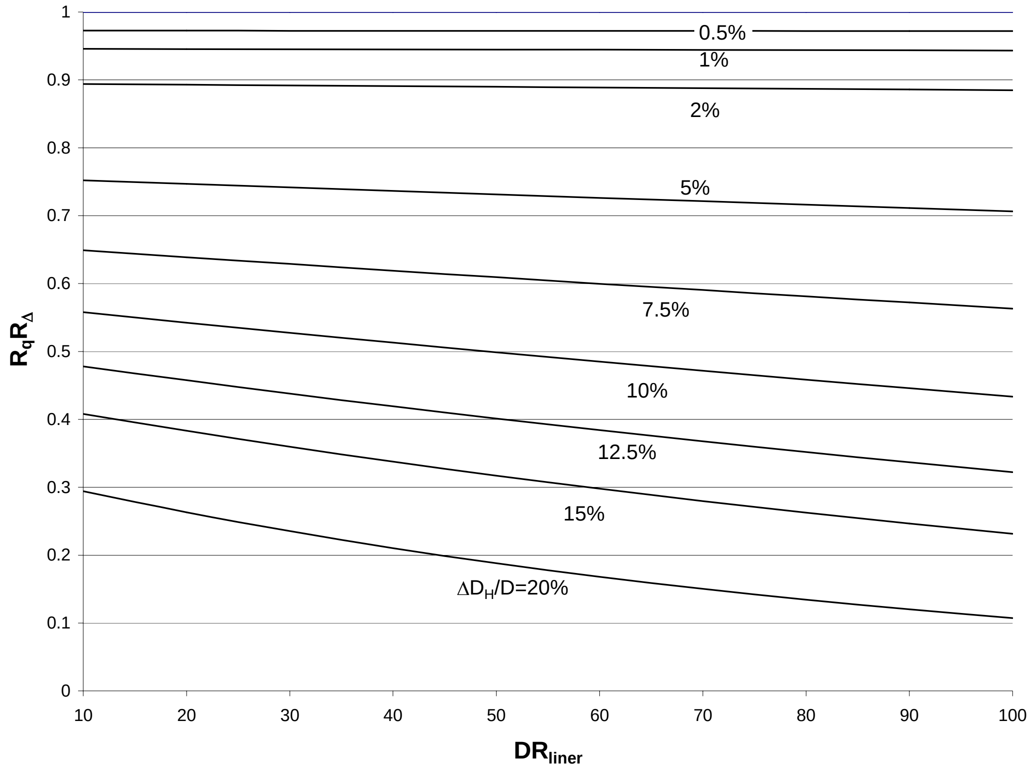


Figure 9. Buckling strength reduction for fractured sewer with close fitting liner, Moore (2008).

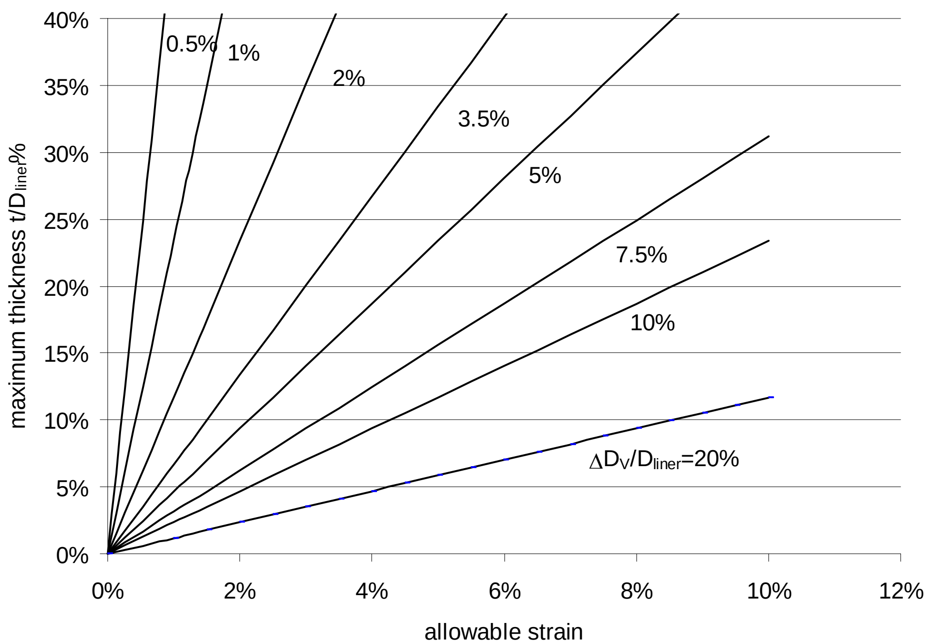


Figure 10. Relationship of pipe deformation and allowable strain to maximum thickness, Moore (2008).

Longitudinal fractures at the pipe crown, invert and springlines act as hinges, and the damaged sewer then deforms in a manner similar to a flexible pipe. Finite element analysis has demonstrated that increases in horizontal diameter can be estimated using flexible pipe theory, and vertical diameter decrease from the increase in horizontal diameter. Pipe deformations accelerate as erosion voids grow, and an approximate relationship is reported between change in vertical diameter and void size.

Design issues for a tight fitting polymer liner installed within the damaged, deformed sewer include reduced buckling resistance to external fluid pressure as the sewer deformations increase. A design equation and chart have been provided to quantify those decreases. There is increased potential for bending in the liner at crown and invert as the pipe deforms, and the reductions in maximum liner thickness that result have also been quantified.

7. ACKNOWLEDGEMENTS

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