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KINEMATIC RESPONSE OF DAMAGED RIGID SEWER UNDER EARTHLOAD

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ABSTRACT

Preliminary testing in the laboratory has been used to investigate the interaction between the soil and a damaged rigid pipe under earth disturbance. The test geometry and measurement scheme to evaluate movement at fractures are described. Conclusions are drawn regarding the impact of the four 90° segments of damaged pipe on the deformation of the system when disturbed. The flexural stiffness of a rigid pipe with overloading fractures is similar to an idealized flexible buried pipe and simplified design calculations can be made using flexible pipe theory.

RÉSUMÉ

Des essais préliminaires en laboratoire sont utilisés pour étudier l'interaction entre le sol et un tuyau rigide endommagé sous des charges de sol. On présente la géométrie de l'essai et la grille de mesure pour évaluer le mouvement aux fissures. Des conclusions sont tirées concernant l'impact des quatre segments de 90º du tuyau endommagé sur la déformation du système quand il est chargé. Le module de résistance à la flexion du tuyau rigide avec les fissures est similaire à un tuyau flexible enterré et qu'une méthode simplifiée de calcul de conception basée sur la théorie des tuyaux flexible peut être utilisée.

1. INTRODUCTION

As part of a study on the behavior of pipe repair using polymer liners, the flexural response of damaged rigid sewers has been investigated. The design of pipe liners is currently performed neglecting the contribution of the damaged host pipe in the assessment of resistance to earth loads (Moore, 1999). However, this damaged rigid pipe may behave as a buried flexible pipe and provide some structural support. This paper reports on one recent test performed to investigate the kinematic response of a rigid sewer with artificially induced longitudinal fractures, disturbed by changes in earth loads (these may result during installation of adjacent utilities or by application of vehicle loads).

2. PROBLEM DEFINITION

Rigid pipes are generally composed of vitrified clay or reinforced concrete. A pipe is considered "rigid" when its bending and hoop stiffnesses are very large relative to the surrounding soil. Pipe damage can result in a loss of both hydraulic and structural capacity. One type of common damage is overloading fractures, where cracks are formed at the springlines, crown, and invert (Figure 1). When a clay pipe suffers from this kind of fractures, its bending stiffness is compromised immediately. Conversely, bending resistance in a concrete pipe may be retained shortly after damage occurs, due to the presence of reinforcement. When the exposed reinforcement corrodes, the behaviour of these two types of rigid pipe will then converge. Horizontal fracturing should have no effect on the hoop stiffness as long as contacts are maintained between the pipe segments.

Pipe rehabilitation design procedures depend on the condition of the damaged pipe (e.g. ASTM F1216-93). The designation of the deterioration condition is currently the subject of considerable debate in the industry.

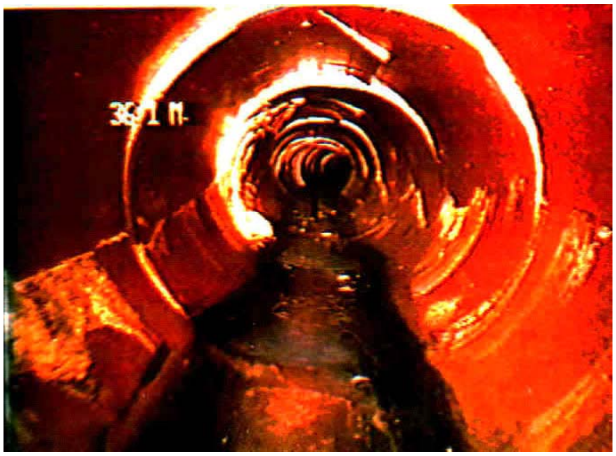

Figure 1. Rigid Pipe with Overloading Fractures

The test reported here was designed to investigate the conceptual deformation of damaged sewers under earth disturbance. The idealization is show in Figure 2. The fractures in the rigid pipe will expand at the inner surface of the pipe at the crown and invert, and the outer surface at the springlines. However, one contact point is maintained at each crack during deformation, forming a hinge mechanism. An angle, θ, can be used to express the magnitude of this "angular expansion". The flexural response of damaged sewers is expected to be governed by plane strain conditions due to its geometry.

Interpretation of the test result may be extended to the tunnelling application. Even though the stress path experienced by a pipe and a tunnel is different, their deformation kinematics are identical.

It can be important to clarify the specific pipe dimensions that are being evaluated during design calculations of pipe deflections. Often, the focus of attention is the inner pipe wall, since this controls the hydraulic performance of the structure, e.g. the internal pipe measurements reported earlier by the authors for an HDPE liner (Law and Moore, 2000). For a fractured pipe, it is straight forward to define horizontal pipe diameter to the inner surfaces of the structure. However, the vertical pipe diameter is best defined to the outer pipe surface, since that is the location at crown and invert where the pipe segments remain in contact. This designation can be crucial when the pipe is thick and θ becomes large.

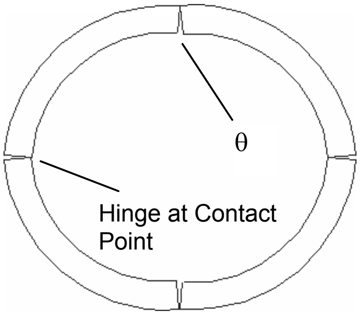

Figure 2. Idealized Deformation of Damaged Sewer

3. PRELIMINARY TEST EXAMINING DAMAGED PIPE-SOIL INTERACTION

3.1 Objectives

A biaxial compression test cell was used to investigate the mechanisms of damaged pipe-liner interaction. One test has been performed in this study. The test featured a concrete pipe with artificially induced overloading fractures.

3.2 Specimen Preparation and Instrumentation

Four concrete pipe segments, 25mm nominally thick and 1960mm in length, were cast using the following procedures. The locations of crown, invert, and springlines were marked on a piece of HDPE pipe, which was used as the internal boundary during casting. Steel angles (25.4mm by 9.5mm with 2.5mm thickness) were screwed onto the HDPE pipe at those four locations. The sample was then placed 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 25mm space between the HDPE pipe and the steel tube. Steel wire mesh was used as reinforcement within the concrete pipe segments, but no reinforcement was placed over the span of any fractures. The steel tube mould and the HDPE pipe were removed after fourteen days. Concrete pipe segments were assembled and the whole specimen was temporarily strapped with steel bands.

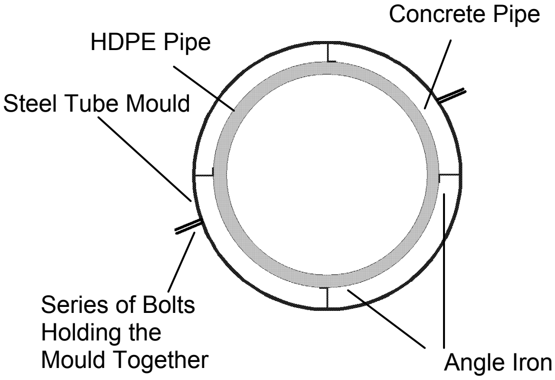

Figure 3. Configuration for Concrete Pipe Casting (Horizontal Section through Vertically Oriented Pipe)

It is important to investigate and quantify the angular expansion of the fractures in the damaged pipe under ground disturbance. To measure the relative movement of the pipe segments during testing, eight mechanical shutter release cables (1m in length) were modified and were mounted onto the host pipe (internal surfaces at the crown and invert, and external surfaces at the springline). 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 cables range from 1% to 4%. However, when bent sharply, the cables are 30% less sensitive, and this can affect their performance. 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). Four cables were glued onto the sample at each end using epoxy adhesive. Dimensions of each cable location were measured in detail to convert displacement readings to the angular expansion of the fractures in degrees.

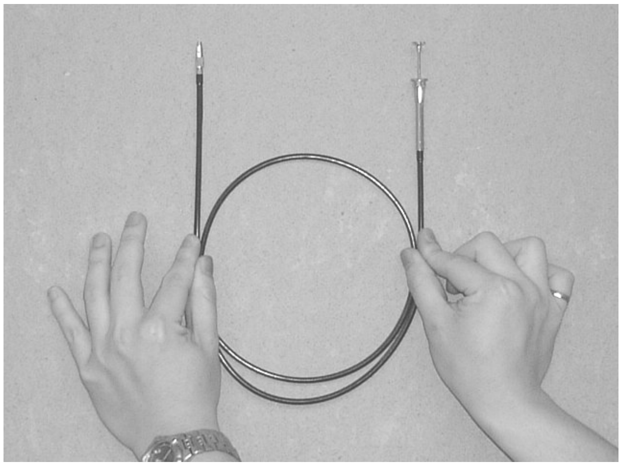

Figure 4a. Mechanical Shutter Release Cable

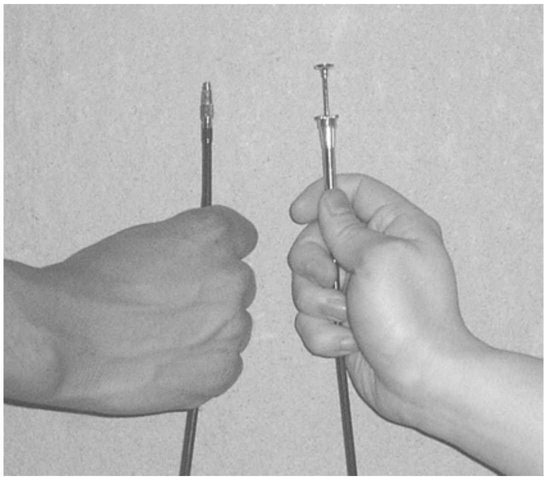

Figure 4b. Cable in Released State

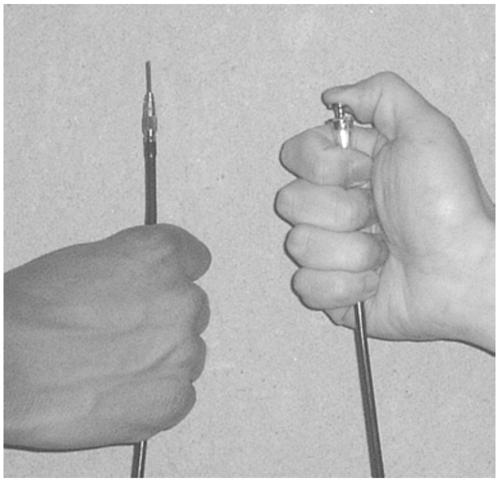

Figure 4c. Cable in Compressed State

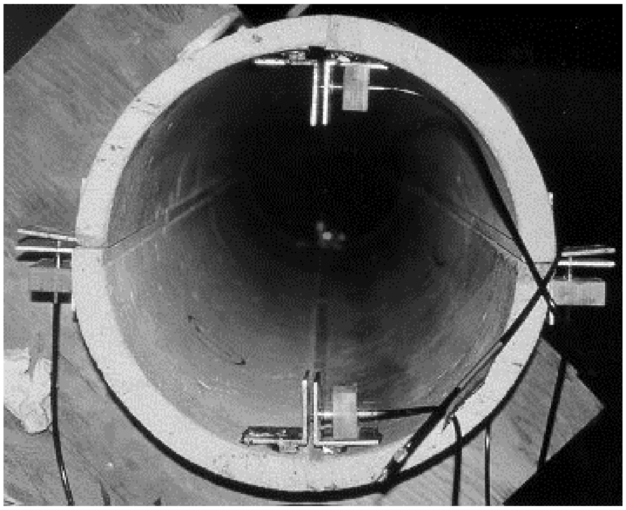

Figure 5. Section 'A' of Specimen, with Instrumentation

3.3 Pipe Burial

The biaxial compression test cell is designed to simulate deep embankment loadings on a buried pipe. It features special side wall treatment to reduce its friction to approximately 5^{o} (Tognon et al., 1999). By restraining backfill soil laterally, close to a geostatic stress in the "at rest" condition (k_o) can be achieved (Brachman, 1999).

Olimag 30-60, poorly graded sand with C_u of 1.46 and C_c of 0.94 (Lapos and Moore, 2002), was used as the backfill soil. It was placed in 100mm lifts at about 3% moisture content, resulting in an average density of 1353 kg/m^3. To simulate poor backfill material in the field, soil was not compacted. The specimen was placed horizontally when the soil reached a depth of 400mm. Once backfill was placed just about the springline level, the temporary metal straps were removed from the specimen. The external mechanical shutter release cables were covered to prevent intrusion of backfill soil. The free ends from the cables were routed from the sample through an access hole and were mounted to displacement transducers. A data acquisition system was used to record all test results. Two settlement plates and eight earth pressure cells (measuring both vertical and horizontal pressure) were also installed in the backfill (Figure 6)

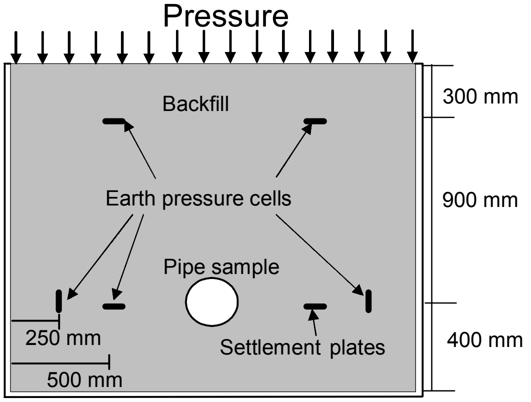

Figure 6 – Test Configuration

3.4 Loading

Loading was applied in 25 kPa increments at the surface of the backfill through a flexible bladder system. It 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. Each pressure increment was sustained for 900 seconds prior to application of the next load step. Six increments were applied, resulting in a final surface pressure of 150 kPa. Displacement transducer readings were recorded every 10 seconds using the data acquisition system. Earth pressure cell readings were also taken prior to the end of each load step.

3.5 Test Results

Based on generalized Hooke's Law, soil modulus (E) and Poisson's ratio ($\nu = k_o / (1+k_o)$) were calculated using the average readings from the settlement plates and the stress cells.

Table 1. Soil Response

Applied Pressure (kPa)	k_o	ν	E (kPa)
25	0.48	0.32	2105
50	0.49	0.33	2421
75	0.50	0.33	2861
100	0.50	0.34	3246
125	0.51	0.34	3530
150	0.51	0.34	3730

Angular expansion readings of the fractures are shown in Figure 7. It shows that readings from the crown and invert at both sections are more consistent than those at the springlines. These measurements can be used to approximate diameter changes based on the trigonometric relationships. Readings from the springline can be converted to vertical diameter changes (Figure 8a), while the same thing can be done to relate readings from the crown and invert to horizontal diameter changes (Figure 8b). The results are shown in Figure 9. For a rigid pipe with overloading fractures, its decrease in vertical diameter and increase in horizontal diameter would be identical due to the kinematics of the four quadrants. Since measurements at the springlines require placement of the mechanical shutter release cables on the outer surface of the pipe, where soil intrusion and cable distortion may interfere with the measurements, the following discussion focuses on readings at the crown and invert.

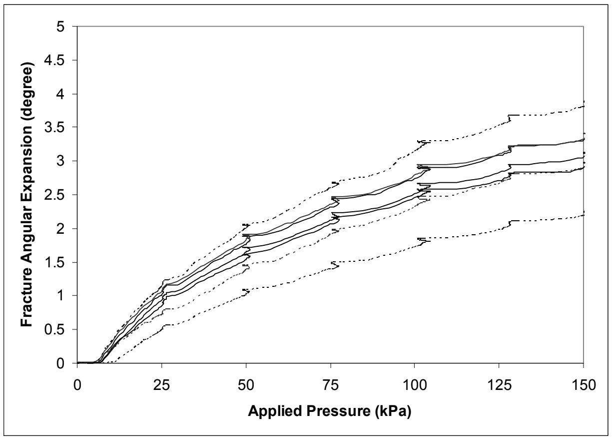

Figure 7. Angular Expansion Readings vs. Applied Pressure
(Crown and Invert ——, Springlines ——)

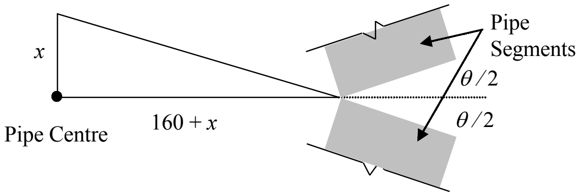

Figure 8a. Springlines, Relationship between Fracture Angular Expansion and Vertical Diameter Decrease
(Original Internal Radius = 160mm,
Vertical Diameter Change = 2x)

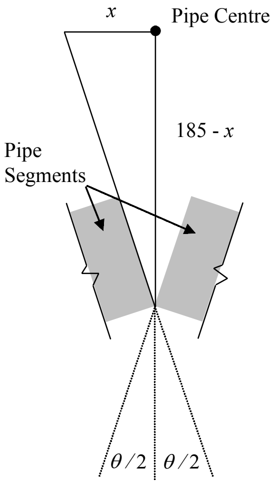

Figure 8b. Invert (same with Crown), Relationship between Fracture Angular Expansion and Horizontal Diameter Increase
(Original External Radius = 185mm,
Horizontal diameter Change = 2x)

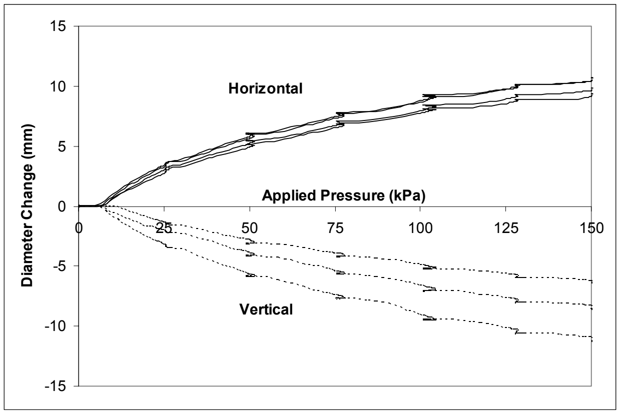

Figure 9. Diameter Changes from Cable Readings vs. Applied Pressure
(Crown and Invert ——, Springlines ——)

At the highest overburden pressure of 150 kPa, average angular expansion of the fractures at the crown and invert was about 3.2^{o} and the average increase in horizontal pipe diameter was 2.7%.

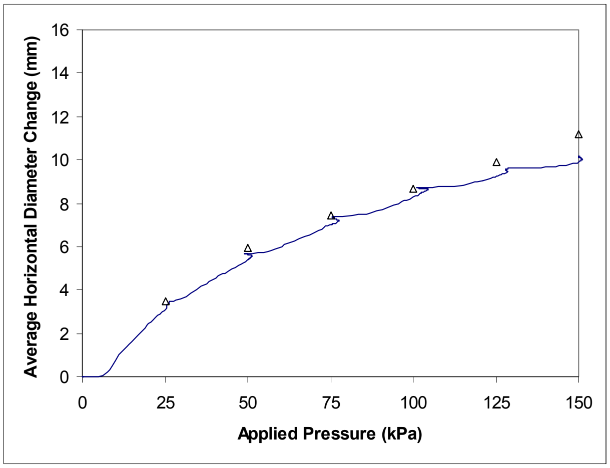

Figure 10. Horizontal Diameter Change vs. Applied Pressure
(Theoretical Response Shown in Open Triangles)

Average horizontal diameter changes and the theoretical response of an idealized flexible pipe (high hoop stiffness, negligible bending stiffness, Burns and Richard, 1964) is shown on Figure 10. At 150 kPa, the deformation of the idealized flexible pipe is about 3.0%, 10% higher than the damaged host pipe, indicating that the damaged host pipe is slightly stiffer.

3.6 Discussion

The results given in this paper are based on a specific set of sample, testing procedures, configurations, geometrical, and material conditions. Issues that require particular attention are the load path imposed and the condition of the damage sewer. In the test performed, the sewer was damaged before application of the earth pressures. This load path is adopted for practical considerations, but the load path in the field is likely to be different.

The 25mm thick rigid pipe used in the test was specially fabricated for this investigation. Its material and geometry, together with the artificially induced fractures are reasonable approximations of both clay and reinforced concrete pipe. Nonetheless, these properties are idealized representation of field characteristics.

It appears that a rigid pipe with overloading fractures behaves like a buried flexible pipe in that deformations are controlled by the surrounding soil. There is additional bending stiffness within the pipe segments, but the changes in vertical and horizontal pipe diameter are very similar to those for flexible pipe.

4. CONCLUSIONS

Some limitations of the current pipe design procedures have been reviewed. Recent work to investigate the flexural behavior of damaged sewers has been presented. Results from a full scale test indicate that the changes in vertical and horizontal pipe diameter of a rigid pipe with overloading fractures are similar to a flexible pipe. Further studies are planned to examine the behavior of damaged sewers under a “tunneling” load path, one that involves application of earth pressures prior to pipe fracture.

5. ACKNOWLEDGEMENTS

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6. REFERENCES

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, USA.

Brachman, R.W.I., 1999, *Mechanical Performance of Landfill Leachate Collection Pipes*, PhD Thesis, The University of Western Ontario, London, Canada.

Burns, J.Q., Richard, R.M., 1964, Attenuation of Stresses for Buried Cylinders, *Proceedings of the Symposium on Soil-Structure Interaction*, University of Arizona, 379-392.

Lapos, B., Moore, I.D., 2002, Classification of Strength and Deformation Parameters of Olimag Synthetic Olivine, *Proceedings for the 55th Canadian Geotechnical Society annual Conference*, Niagara Falls, Ontario, Canada.

Law, T.C.M., Moore, I.D., 2000, Static Response of Slip-Lined Sewers, *Proceedings for the North American NO-DIG 2000 Conference*, Anaheim, California, 51-56

Moore, I.D., 1999, Performance Limits for Polymer Sewer Liners, *Proceedings for the 52th Canadian Geotechnical Society Annual Conference*, Regina, Saskatchewan, 357-362.

Tognon, A.R.M., Rowe, R.K., Brachman, R.W.I., 1999, Evaluation of Side Wall Friction for a Buried Pipe Testing Facility, *Geotextiles and Geomembranes*, 17, 193-212.