The performance of a laboratory facility for evaluating the structural response of small-diameter buried pipes

R.W.I. Brachman, I.D. Moore, and R.K. Rowe

Abstract: The performance of a new laboratory facility for testing small-diameter buried pipes (less than 300 mm diameter) subject to the biaxially compressive earth pressures expected to prevail under deep and extensive overburden is examined. The new facility consists of a prism of soil 2.0 m wide × 2.0 m long × 1.6 m high contained within a stiff steel structure. Laboratory tests were performed in the new test facility to examine the appropriateness of the boundary conditions imposed during testing. Overburden pressures are successfully simulated with a pressurized air bladder. Boundary friction was limited to only minimal effects with lubricated polyethylene sheets. The stiffness of the lateral boundary is sufficiently large to induce lateral stresses close to those for zero lateral strain conditions. Overall, the effects on the pipe arising from the idealizations involved in the laboratory model were found to be small. The application of the new test cell is illustrated by using it to assess the response of a small-diameter landfill leachate collection pipe under two different backfill conditions. This comparison showed that the structural response of the pipe is significantly impacted by the coarse gravel backfill used in landfill drainage layers. Maximum pipe deflections and strains were nearly twice as large when tested in the coarse gravel compared with the sand backfill. Much greater variations of deflection and strain were also measured with the coarse gravel when compared with the sand backfill due to local bending effects from the coarse gravel.

Key words: buried pipes, soil–structure interaction, laboratory testing, leachate collection pipes.

Résumé : On examine la performance d'un nouvel équipement de laboratoire pour tester des tuyaux enfouis de petits diamètres (moins de 300 mm de diamètre) soumis aux pressions des terres biaxiales en compression qui peuvent être escomptées sous un dépôt profond et vaste. Le nouvel équipement consiste en un prisme de sol de 2,0 m de largeur × 2,0 m de longueur × 1,6 m de hauteur contenu dans une structure rigide en acier. Des essais de laboratoire ont été réalisés dans ce nouvel équipement d'essai pour voir si les conditions imposées aux frontières durant l'essai étaient appropriées. Des pressions sus-jacentes ont été simulées avec succès au moyen d'une vessie en caoutchouc pressurisée à l'air. Le frottement aux frontières a été limité à des effets minimes au moyen de feuilles de polyéthylène lubrifiées. La rigidité de la frontières latérale est suffisamment importante pour induire des contraintes latérales qui se situent tout près de celles correspondant à des conditions de déformations latérales nulles. On a trouvé que globalement, les effets sur le tuyau résultant des idéalisations impliquées dans le modèle de laboratoire étaient faibles. L'application de cette nouvelle cellule d'essai est illustrée en l'utilisant pour évaluer la réponse d'un tuyau de captage de lixiviant de petit diamètre dans un enfouissement sanitaire sous deux différentes conditions de remblayage. Cette comparaison montre que la réponse structurale du tuyau est influencée de façon significative par le remblayage de gravier grossier dans les couches de drainage de l'enfouissement sanitaire. Les déflexions et déformations maximales étaient presque deux fois plus importantes lorsque l'essai avait lieu dans le gravier grossier par rapport au remblai de sable . Des variations beaucoup plus grandes des déflexions et des déformations ont aussi été mesurées dans le gravier grossier en comparaison du remblai de sable à cause des effets du fléchissement imposé par le gravier grossier.

Mots clés : tuyaux enfouis, interaction sol-structure, essai en laboratoire, tuyaux de captage de lixiviant.

[Traduit par la Rédaction]

Introduction

Large-scale testing of buried pipes is often undertaken to evaluate the soil–structure interaction under simulated field conditions. Various facilities have been used to test buried structures (e.g., Utah State University, University of Massachusetts at Amherst, and Ohio University in the United States; The University of Western Ontario in Canada; and

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LGA Geotechnical Institute in Germany). However, each of the existing facilities has limitations related to the boundary conditions in the facility, and none can closely approximate the expected field conditions with respect to the stress state associated with deep and extensive burial in a zone of soil surrounding a pipe.

The design of a new laboratory facility for testing small-diameter pipes under simulated field conditions was presented by Brachman et al. (2000). This test facility was designed to closely simulate the biaxial earth pressures that act on the vertical and horizontal soil boundaries at some distance from the pipe. This new facility was required to investigate the structural performance of small-diameter pipes (less than 300 mm; e.g., perforated landfill leachate collection pipes) under controlled experimental conditions in the laboratory. Issues such as the selection of dimensions, the simulation of earth pressures, and limiting boundary effects from interface roughness and lateral stiffness were discussed by Brachman et al.

This paper reports on the performance of the new laboratory facility. The results of two tests conducted on a small-diameter leachate collection pipe are examined to assess the suitability of the new laboratory model. The measured response of the soil, pipe, and test facility are considered to evaluate how closely the response of the pipe tested in the new facility compares with that expected to occur under field conditions of deep and extensive overburden pressures, and how the measured response compared with the analysis conducted for the design of the test facility. The effects on the pipe arising from the idealizations involved in the laboratory model are small. Overburden pressures are successfully simulated with a pressurized air bladder. Boundary friction is limited to only minimal effects with lubricated polyethylene sheets. The stiffness of the lateral boundary of the new facility is sufficiently large to induce lateral stresses close to those for zero lateral strain conditions.

The application of the new test cell is illustrated by using it to assess the response of a small-diameter landfill leachate collection pipe for two very different backfill conditions. This comparison showed that the mechanical response of the pipe is significantly impacted by the coarse gravel backfill used in landfill drainage layers. Maximum surface strains and pipe deflections are nearly twice as large when tested in the coarse gravel compared with the sand backfill. Much greater variations of strain and deflection occur with the coarse gravel when compared with the sand backfill due to local bending effects from the coarse gravel.

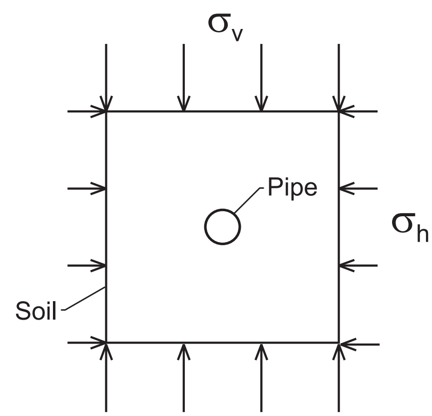
Description of laboratory facility

Laboratory apparatus

Burial of a pipe under deep and extensive ground materials leads to large vertical and horizontal stresses that act at some distance from the pipe. Idealized in Fig. 1 are the vertical stresses that arise from the weight of the overburden material and horizontal stresses that develop in the ground from resistance to lateral movement. The new test facility was designed to closely simulate these biaxial earth pressures.

Figures 2 and 3 show transverse and longitudinal sections of the test cell. The pipe specimen is placed within a prism of soil that is 2.0 m wide × 2.0 m long × 1.6 m high. The pipe may be placed at different elevations in the cell depending on the test requirements. The soil is contained within a stiff steel structure. Lid and base units are connected with twelve 25 mm diameter high-strength tie rods capable of resisting the large vertical forces that develop with the application of large vertical stresses (up to 1000 kPa). The side walls consist of 40 mm thick steel plates stiffened by four support frames that are welded to the side plates and also have welded moment connections in each corner.

Fig. 1. Idealized vertical σ_v and horizontal σ_h earth pressures acting distant from a deeply buried pipe.



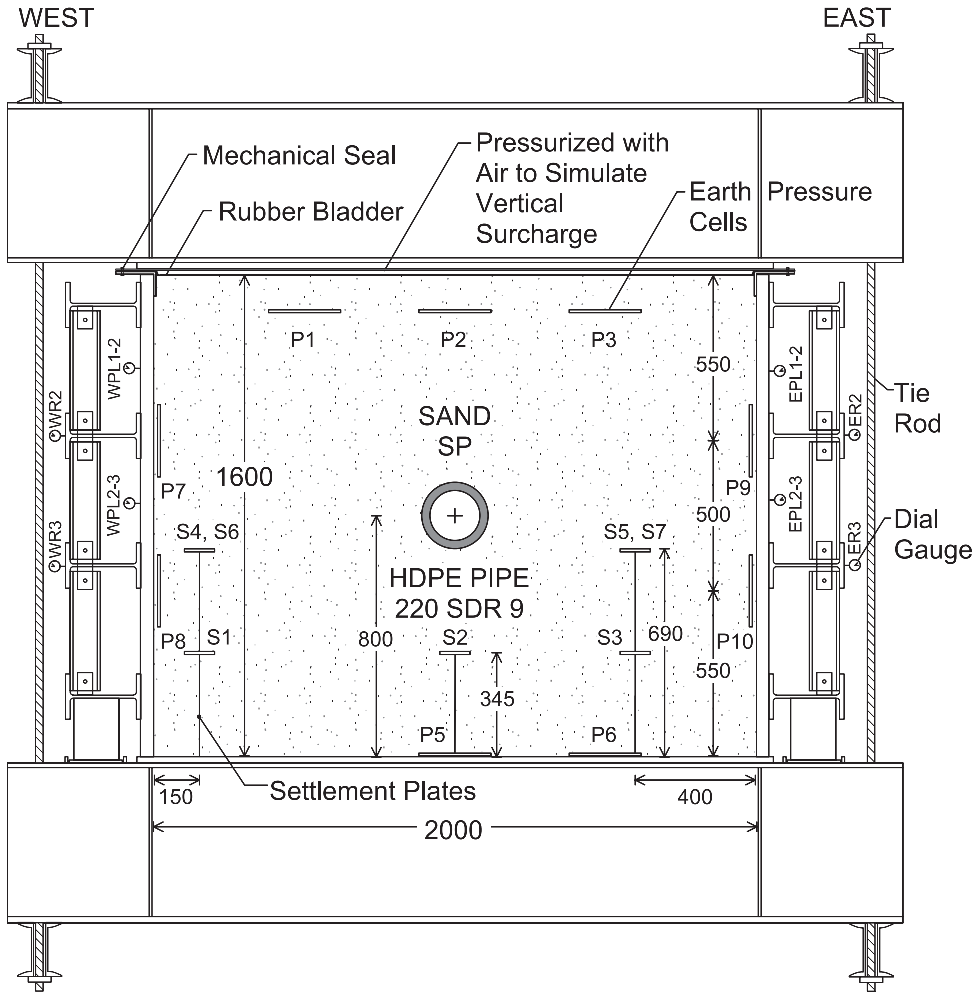
Simulation of vertical and horizontal stresses

Vertical stresses

Vertical loading is applied by a pressurized air bladder that provides a uniform vertical stress across the top soil boundary. The bladder consists of a large rubber sheet sandwiched between a square frame, composed of steel angle sections welded together in the corners and a thin steel plate (Fig. 3). The leg of the angle lies inside the side walls and provides lateral confinement for the bladder as the lid deflects upwards, otherwise the bladder would be unconfined and rupture. The leg of the angle was selected to be larger than the estimated sum of the upward movement of the lid unit (governed by the elongation of the tie rods) and the downward movement of the soil. A mechanical seal was obtained around the perimeter of the bladder by bolting the angle to the steel plate. The relatively thin steel plate is supported by the stiff lid unit during testing. The top plate contains an inlet connection and an outlet connection where the bladder pressure can be monitored.

Several rubber materials were evaluated using a cylindrical pressure vessel (400 mm diameter) with a bolted steel lid. Elongation of the rubber was the main criterion for material selection. Reinforced materials provided excellent strength but are undesirable to use to apply a uniform pressure to the soil, since any tensile force mobilized within the membrane reduces the external pressure applied to the soil. Commercially available lifting bags (normally sealed neoprene bags heavily reinforced with either steel or kevlar) were also evaluated; based on this evaluation, caution is advised on the use of these bags for simulation of vertical

Fig. 2. Transverse section through the pipe test facility designed to closely simulate deep burial conditions. The soil, pipe, and instrumentation used for test T1 are shown. Dimensions in millimetres.



pressures because, as a result of the load transfer to the reinforcement, (*i*) the pressures applied to the soil are unknown, and (*ii*) the distribution of contact stresses may be highly nonuniform.

Natural gum rubber provided the best performance (even for a relatively thin rubber sheet 1.6 mm thick) but was difficult to seam. Seaming is an important consideration, given that the manufactured width of these materials is less than the width of the test cell. Buna-N rubber (3 mm thick) was selected for use as the bladder material because of its good elongation to failure and relative ease of seaming. The sheets of rubber were joined with a 50 mm wide chemically vulcanized lap seam. Rubber cement (Rema Tip-Top SC2000) was used to chemically vulcanize the seam together. Surface preparation (involving scoring and cleaning) and stitching (pressing the seam together to remove air pockets) were critical factors for obtaining a reliable seam.

This diaphragm-type arrangement for the bladder was superior to trials of sealed bags composed of various materials. Larger pressures and more uniform loading conditions were obtained with this arrangement. Proper sealing of bag-type bladders often required many layers of additional reinforcement in the corners, resulting in a stiffer bladder and hence a deviation from uniform pressures applied to the soil.

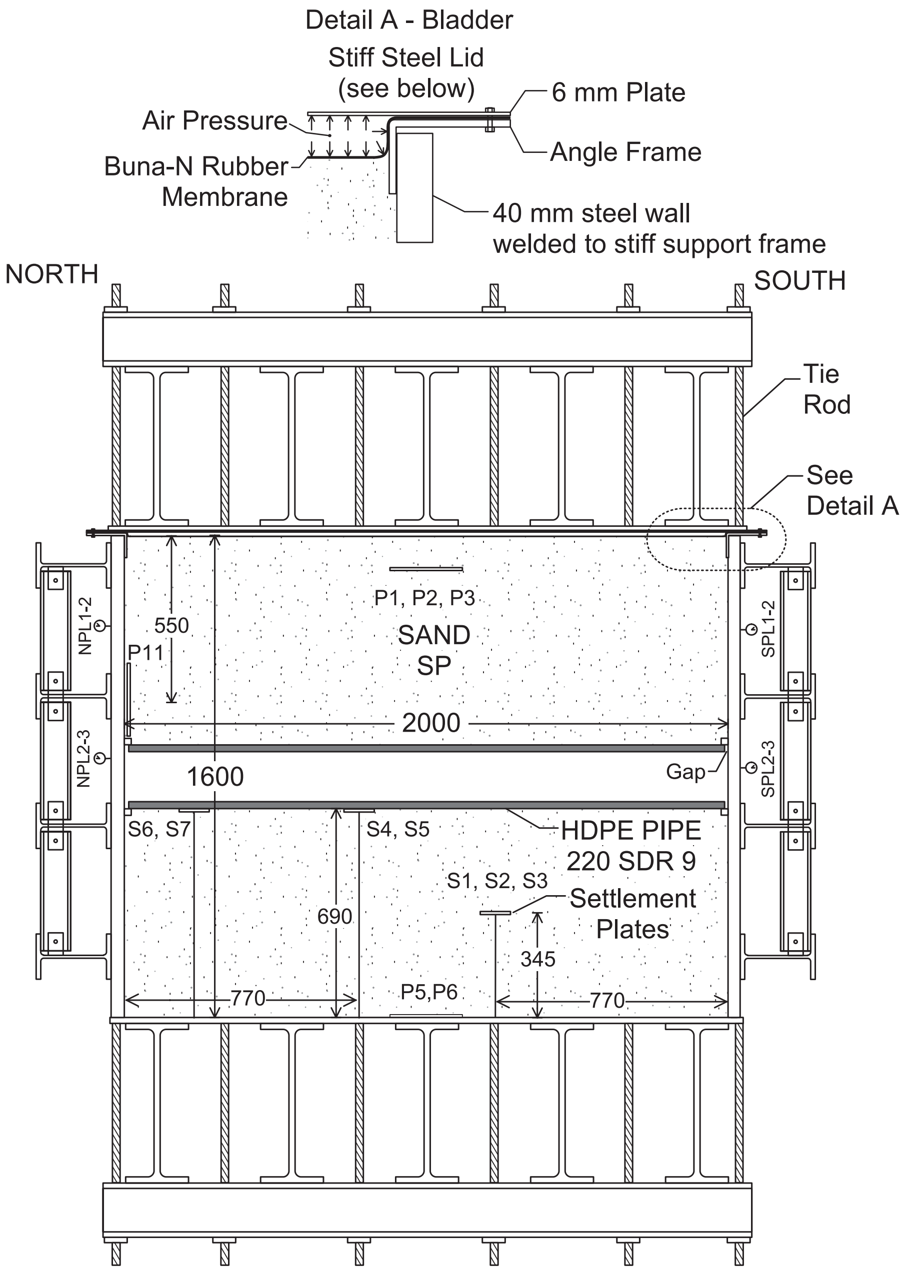
Horizontal stresses

Horizontal stresses develop in the soil as a result of restraint against lateral movements by the side walls. The stiffness of the lateral boundaries was one important consideration during design of the new test facility (Brachman et al. 2000). Simulation of lateral pressures with pressurized bladders (in a manner similar to that for the vertical pressures) was contemplated during the conceptual design of the test cell but was not adopted given the potential for damage of the bladder from the backfill material (especially when the coarse gravel material, shown in Fig. 4, was tested). The method employed in the new laboratory model permits the development of lateral earth pressures appropriate to each material, which is important when different layers of soil materials that may generate different lateral earth pressures are tested (e.g., the simulated landfill conditions shown in Fig. 4).

Limiting boundary friction

The side walls of the test cell required treatment to reduce the boundary effects from friction. Figure 5 shows the angle of friction ϕ_{sw} mobilized for various treated interfaces considered by Tognon et al. (1999). Limited surface treatment involved a thin polyethylene (PE) sheet (0.1 mm thick) and a nonwoven geotextile GT1 (with mass per unit area of

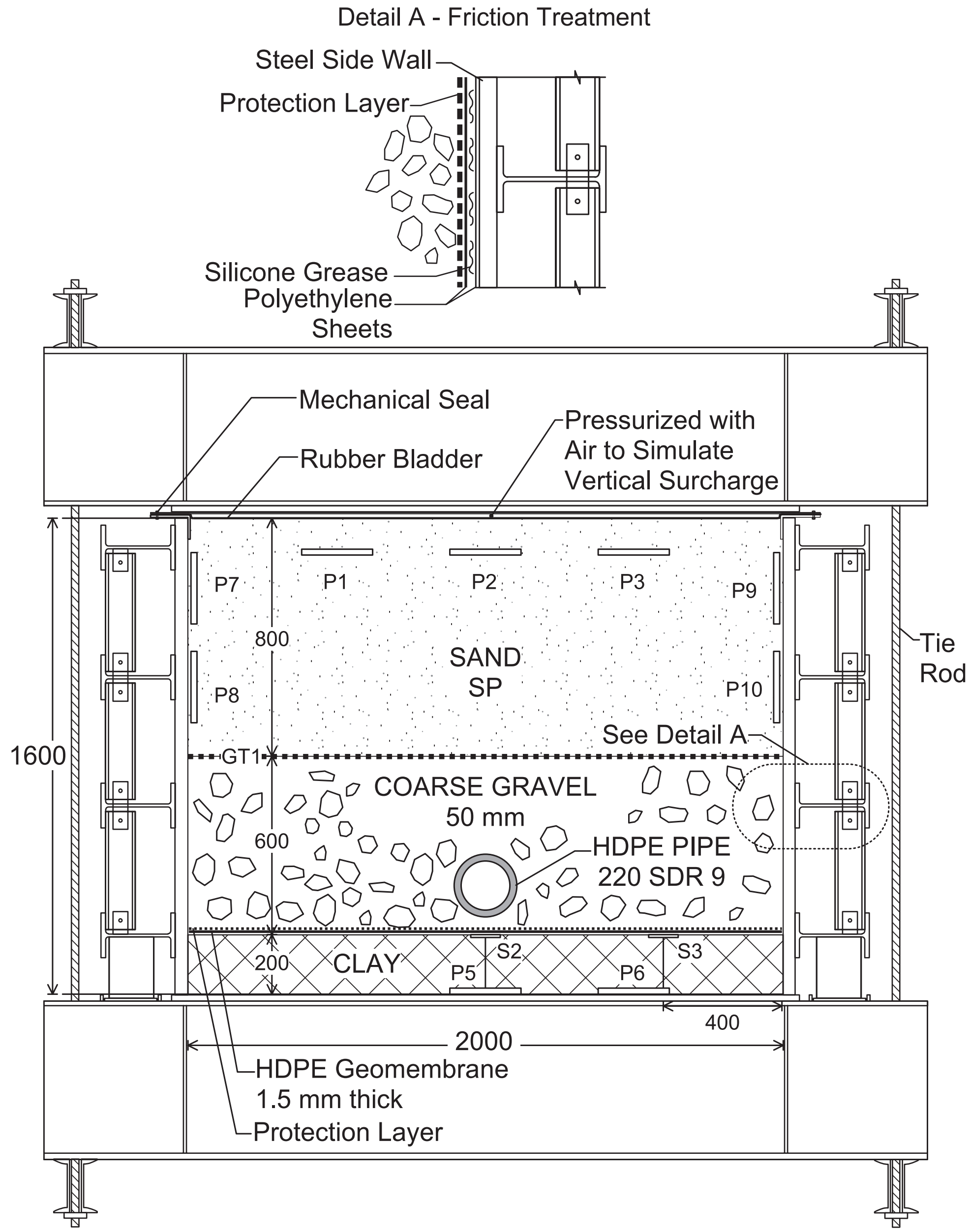
Fig. 3. Longitudinal section through the pipe test facility showing the soil, pipe, and instrumentation used for test T1. Dimensions in millimetres.



435 g/m^2); slip was to occur along the interface between these two materials. When tested in contact with medium sand, the resulting friction angle was 16°. Tests with the coarse gravel material (described later) yielded a higher interface friction angle of 21°. This increase in ϕ_{sw} relative to the sand backfill results from discontinuous interface contact. Damage to the polyethylene layer was noticed because of the coarse gravel particles. This demonstrated a need to protect the friction treatment from impingement by backfill materials.

The adopted interface treatment consisted of two polyethylene sheets (0.1 mm thick) lubricated with silicone grease (Dow Corning 44 high-temperature bearing grease). This arrangement (illustrated in Fig. 4) yielded a friction angle of roughly 5°, as shown in Fig. 5. Adequate protection of the interface treatment from the possible impingement of the backfill material was obtained with a thinner nonwoven geotextile GT2 (mass per unit area of 120 g/m^2) and a 2 mm thick geomembrane GM (Tognon et al. 1999). The potential for a deleterious interaction between the PE sheets and the silicone was investigated by conducting direct shear tests on samples of the PE sheets in contact with the lubricant for longer time frames (up to 30 days). No discernable increase in interface friction was found for the Dow Corning 44 grease (Tognon et al. 1999). It was estimated that this level of friction reduces the vertical stresses reaching the pipe by less than 5% and has only a small effect on the pipe response (Brachman et al. 2000). Since direct measurement of boundary friction would be very difficult (if not impossible) to undertake on the large scale involved in the pipe tests, an

Fig. 4. Transverse section through the biaxial compression testing facility showing the soil materials, pipe, and instrumentation for tests conducted with simulated landfill conditions. Dimensions in millimetres.



assessment of the friction angle obtained during tests in the new facility and the effect on the pipe are achieved through measurements of the pipe and soil response, as well as from visual observations, and are reported later in this paper.

Description of laboratory tests

Soil materials tested

Test T1

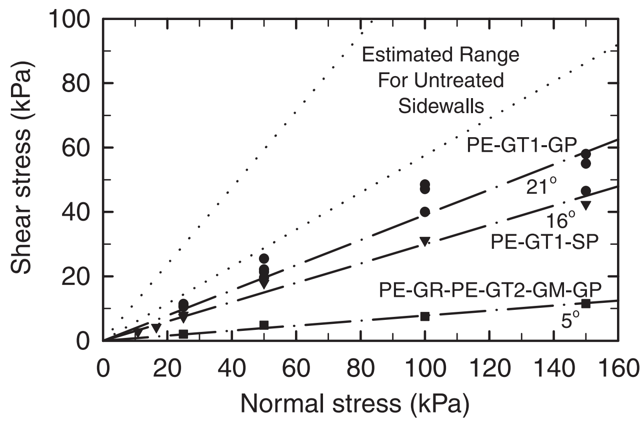
Two different arrangements were tested. The first involved the small-diameter pipe centrally located within the test cell and backfilled with a relatively uniform medium sand (SP). Figures 2 and 3 show the arrangement of materials used for the first test (T1).

Sand (poorly graded medium sand) was selected because it could be consistently placed at a relatively uniform density with moderate compactive effort. The sand was placed in 150 mm lifts (dumped from a constant height) and then compacted by dropping a 9 kg mass a height of 300 mm, with two passes made over each lift. This resulted in a material with relatively uniform density, with an average dry density of 1760 kg/m^3 and water content of 2%. The measurements of density (using nuclear density and sand cone methods) compared well with the density obtained by recording the total mass of sand placed in the cell divided by the volume it occupied.

Test T2

The arrangement of materials for the second test (T2) is shown in Fig. 4. Three different soil materials were used to

Fig. 5. Angle of friction ϕ_{sw} mobilized for various treated and untreated interfaces. Data obtained from direct shear testing reported by Tognon et al. (1999). PE, 0.1 mm thick polyethylene sheet; GT1, nonwoven geotextile (mass per unit area of 435 g/m^2); GT2, nonwoven geotextile (mass per unit area of 120 g/m^2); GR, silicon grease; GM, 2 mm thick geomembrane; GP, gravel; SP, sand.



attempt to simulate the conditions expected in a municipal solid waste landfill. Here, the pipe was placed within a 600 mm deep layer of uniform coarse gravel to simulate the drainage layer. Crushed (40–50 mm) dolomitic limestone, similar to that used at a number of Ontario (Canada) landfills (e.g., Rowe et al. 1993, 2000), was used as the backfill material. This material is a poorly graded coarse gravel (GP) and consists of large angular particles with 70% finer than the 51 mm sieve size and only 8% finer than 38 mm. The use of this coarse gravel is desirable to minimize the biological clogging of leachate collection systems in municipal solid waste landfills (Rowe et al. 1995). The coarse gravel was placed uncompacted (typical for field placement) at an average bulk density of 1520 kg/m^3, which was calculated by recording the total mass of gravel placed in the cell divided by the volume it occupied. The pipe was placed with approximately 100–120 mm of coarse gravel material between the pipe invert and the clay layer.

The drainage layer was underlain by a 200 mm thick clay layer that was included to simulate the effect of a more compressible layer beneath the coarse gravel and the pipe. Finite element analysis of this arrangement was used to select the thickness of the clay layer. It was found that there was little difference in pipe response once the thickness of the clay was such that the pipe was at least one pipe diameter away from the stiff steel base. This silty clay till of low plasticity (liquid limit of 24% and plastic limit of 14%) was placed at a water content near the plastic limit (corresponding to 2–4% wet of optimum, which is standard practice for placing a compacted clay liner; e.g., see Rowe et al. 1995) and an average bulk density of 2100 kg/m^3. This till was from the Halton landfill site and its use as a liner has been described by Rowe et al. (1993, 2000).

The poorly graded medium sand (SP) used in the first test was used to fill the remainder of the test cell. A needle-punched nonwoven geotextile (GT1), with mass per unit area of 435 g/m^2 and equivalent opening size of 75–150 µm, was used to separate the sand and coarse gravel materials.

The arrangement shown in Fig. 4 was also used for four additional tests conducted on two other pipe samples of the same diameter but containing perforations. Results pertaining to the performance of the new facility from tests T3 and T4 are reported in this paper. Details and extensive results from these tests can be found in Brachman (1999).

Details of pipe specimens

The tests were conducted on a thick-wall, small-diameter, high-density-polyethylene (HDPE) pipe. This pipe was made with a polyethylene material with classification PE 345434C in accordance with American Society for Testing and Materials (ASTM) Standard D3350, and Class PE 3408 according to the Plastic Pipe Institute. These pipes are often used as drainage pipes in the leachate collection systems for municipal solid waste landfills, where they would normally be perforated to allow the collection of the leachate. The tests reported in this paper were conducted on pipes without perforations. Tests conducted on perforated leachate collection pipes have been reported by Brachman (1999).

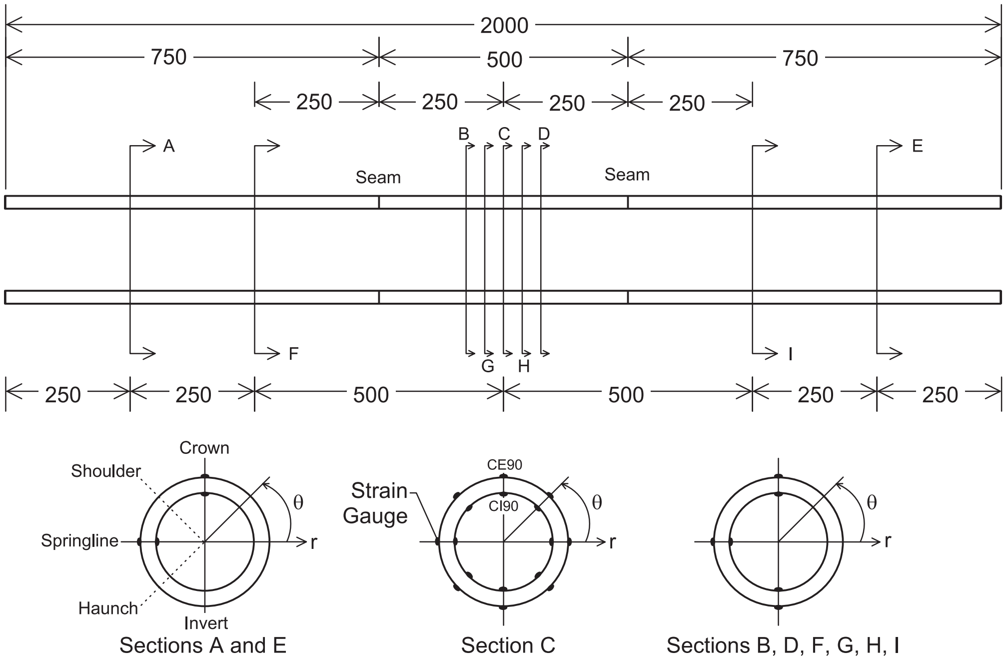
The pipe sample had an average outside diameter of 220 mm and an average wall thickness of 25 mm (nominal pipe size of 220 mm o.d. SDR 9, where SDR is the ratio of outside diameter to the minimum wall thickness). The 2 m long specimen contained two butt-fusion joints, each located 250 mm from the centre of the pipe. This was required to facilitate the application of strain gauges on the interior surface of the pipe. It was also used to test the effect of the seam on the pipe performance. Seaming was conducted in accordance with ASTM Standard D-2657.

Pipe end conditions

Careful consideration was given to the boundary condition at the ends of the pipe specimen. If plane strain conditions prevailed in the axial direction for both the soil and the pipe, axial strains corresponding to the axial elongation of the pipe would be zero. Such conditions are normally assumed to occur under deep burial, when conditions of long and prismatic geometry exist along the pipe axis. However, some situations may arise where the pipe can experience axial elongation, for example at expansion joints or where the pipe enters a manhole. In this case, axial extension of the pipe will lead to tensile axial strains and larger pipe deflections relative to axial plane strain conditions. Tensile axial strains may lead to tensile axial stresses in the pipe depending on the magnitude of the hoop strains that also occur. Since tensile stresses may be more critical for polyethylene pipes (related to long-term potential for stress cracking), consideration of nonzero axial strains is important for assessing the potential performance of the pipe.

Axial strains due to the axial elongation of the pipe can occur when subjected to overburden stresses if the pipe is not fully restrained in the axial direction, or if the soil surrounding the pipe is allowed to deflect in the axial direction relative to the pipe, thereby producing axial strains from the transfer of shear stresses between the soil and the pipe. In such cases these components of axial strain would be tensile. The stiff steel walls of the test cell were used to limit the outward deflection of the soil, and hence the axial strains that are induced in the pipe from the relative movement of the soil and pipe.

Fig. 6. Location of strain gauges on 220 mm o.d. SDR 9 pipe specimen for tests T1 and T2. Pipe deflections recorded at sections B, C, and D. Dimensions in millimetres.



Two alternatives were considered for the restraint imposed at the end of the pipe during testing. The first attempted to provide axial restraint for the pipe. This involved the development of an adjustable end plate, located on either end of the pipe, that could be tightened up against the pipe before testing. The axial strains of the pipe were monitored during this process to impose small compressive axial strains (approximately –0.005%). Although this method did limit the axial elongation of the pipe, very small axial deformations still led to tensile axial strains. For example, 0.1 mm axial elongation of the pipe at each end produces tensile axial strains of 0.01%. Also, despite the fact that measures were taken to limit the friction between the ends of the pipe and the steel plate, some end effects were noted when measurements of pipe deflection were compared at various locations along the pipe (these tests are not reported here).

The second option was to provide no axial restraint for the pipe. This was achieved by providing a gap (approximately 10 mm wide) on both ends of the pipe (see Fig. 3). Measures were taken to ensure that the backfill soil materials were not permitted to fill the space between the ends of the pipe and the lateral boundaries. This arrangement successfully prevented the pipe from jamming up against the side walls as confirmed by observations during testing.

The hoop response of the pipe should not be substantially affected, regardless of the axial conditions imposed for the pipe. This assertion was verified based on numerical modelling of a buried pipe subject to radial pressures with varying deflection at the ends of the pipe specimen. Results from preliminary tests, conducted with the pipe restrained and unrestrained at its ends, also support the fact that the hoop response of the pipe was not sensitive to the end conditions for the pipe. Therefore tests with zero end restraint provided to the pipe are reported here.

Pipe instrumentation

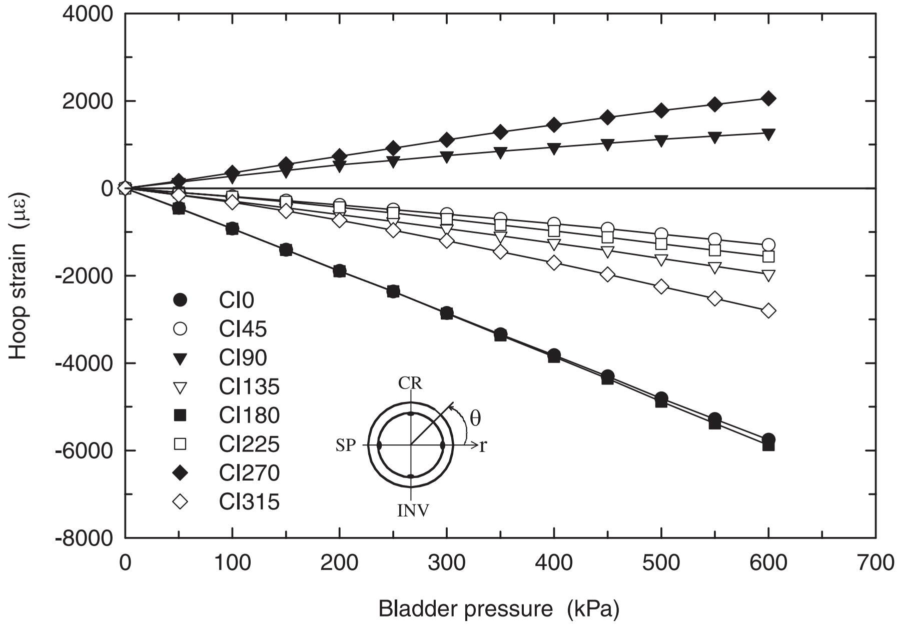
Strains were measured at many locations on the interior and exterior surfaces of the pipe as shown in Fig. 6. Measurements were made using electrical foil strain gauges (2 mm gauge length) in stacked biaxial arrangements. A central section C was instrumented with eight gauges each on the interior and exterior surfaces, located at the springlines (θ = 0 and 180°), shoulders (θ = 45 and 135°), crown (θ = 90°), haunches (θ = 225 and 315°), and invert (θ = 270°). The notation used to identify these measurements, for example, has the strain measured at section C on the interior pipe surface and located at the crown represented by the abbreviation CI90. Sections B and D, each located 75 mm from the centre of the pipe, had strain gauges on the interior and exterior locations of the pipe at the crown, invert, and springline (θ = 180°). The other sections shown in Fig. 6 were selected to measure the variation of strain along the pipe as one means of assessing the influence of the test cell boundaries on the pipe response.

Strains were monitored and recorded using a data acquisition system. The error associated with the strain measurements (gauge sensitivity, lead wire effects, and data recording) was estimated to be ±50 microstrain. Wherever possible, the lead wires from the exterior gauges were located at least one pipe diameter away from the pipe to limit interference with the sand response around the pipe.

Measured pipe strains for test T1

Results of measured surface strains of the pipe are presented to study the structural response of the pipe when tested in the new laboratory facility. Strains are reported in dimensionless units of microstrain ($\mu\varepsilon$), where 1000 $\mu\varepsilon$ equals 0.1% strain. Compressive strains are taken as negative

Fig. 7. Measured hoop strains ε_θ on the interior surface of the pipe at section C during test T1 (compression negative). CR, crown; INV, invert; SP, springline.



values. The reported values of strain versus applied bladder pressure represent the strain averaged over the last 30 s of each 40 min pressure increment. Each pressure step involved the rapid application of an incremental bladder pressure of 50 kPa, which was then held constant for the duration of the pressure step.

Hoop strains at section C

Circumferential (or hoop) strains ε_θ measured on the interior surface of the pipe during test T1 are plotted versus the applied bladder pressure in Fig. 7 for the eight locations around the pipe circumference at section C. Of all the measurements made, the maximum compressive strains occurred at the interior springline and the maximum tensile strains occurred at the interior invert.

Figure 7 shows that compressive hoop strains (negative values) occur at the springlines (θ = 0 and 180°) when the pipe is subject to the biaxial earth pressures. The average hoop strain measured at the springlines is –5800 $\mu\varepsilon$ at an applied vertical surcharge of 600 kPa. The two values recorded at the interior springlines are practically identical (only 2% difference between CI0 and CI180 at 600 kPa), reflecting the close to symmetric response (about the vertical axis of the pipe) expected with uniform vertical and horizontal pressures acting on the soil boundaries and good soil support provided to the pipe by the sand backfill.

Tensile hoop strains occur at the crown and invert (θ = 90 and 270°) on the interior surface of the pipe (Fig. 7). Larger tensile strains occur at the invert (2000 $\mu\varepsilon$) than at the crown (1400 $\mu\varepsilon$) at 600 kPa surcharge, resulting from larger circumferential bending at the invert.

Strains at the quarter points (shoulders θ = 45 and 135° and haunches θ = 225 and 315°) are compressive and smaller in magnitude than those measured at the springline. The magnitudes of these strains are of interest, since it is at these locations that perforations (holes to permit collection of fluid) are typically located. These measurements do not show the symmetry observed at the springlines, as a 40% difference was measured between CI225 and CI315 at 600 kPa. These differences may arise from local variations in density of the sand material, especially at the haunch location where it is difficult to compact the soil.

Variation of strains along pipe

The strain gauges shown in Fig. 6 were located to assess the variation in strains along the pipe. This allows an assessment of the effect of the test cell boundary conditions on the pipe. Figure 8 plots the hoop strains measured at the interior crown location (θ = 90°) at various positions along the pipe. Excellent agreement was found for the three readings near the centre of the pipe (sections B, C, and D), with a mean of 1350 $\mu\varepsilon$ and a coefficient of variation of less than 1% at an applied vertical pressure of 600 kPa.

There is some reduction in hoop strains closer to the ends of the pipe, as the measured values of hoop strain at sections A and E are 80% of the values recorded near the middle of the pipe. Axial strains measured at these locations were tensile and larger towards the ends of the pipe. The increase in axial strains and decrease in hoop strains near the ends of the pipe are a result of the end conditions for the pipe and not the test cell. Hoop stresses (computed from both hoop and axial strains) showed little difference between sections C and E, as the hoop stress at section C is only 3% larger than that at E (using Poisson's ratio of 0.46 for the polyethylene pipe). Axial stresses were almost 15% larger near the end of the pipe than at the centre. These calculations show that the hoop stress at the interior crown is similar along the length of the pipe and largely independent of the axial end conditions of the pipe. No substantial effects from boundary friction or axial end conditions are discernable from these calculated values of stress, especially towards the centre of the cell where the main instrumented sections are located.

Fig. 8. Variation of hoop strain measured at the interior crown location along the pipe during test T1.

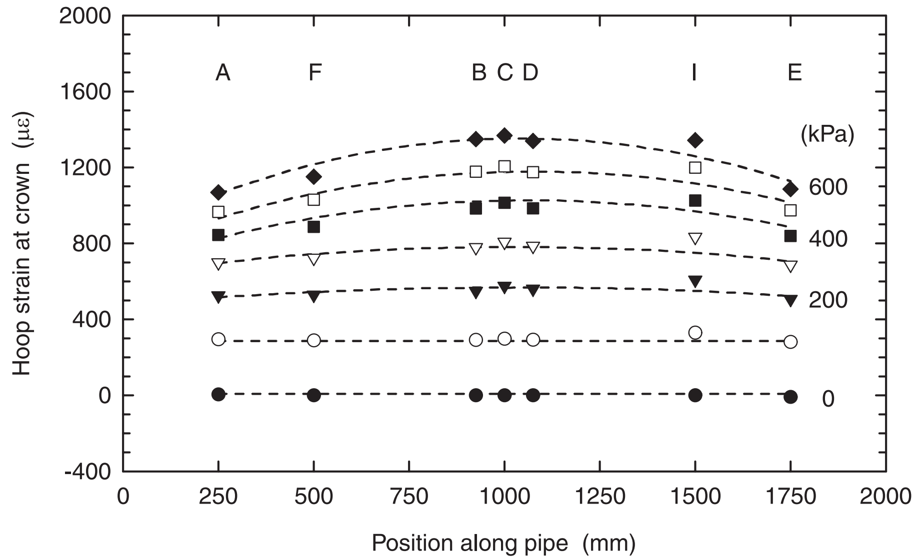
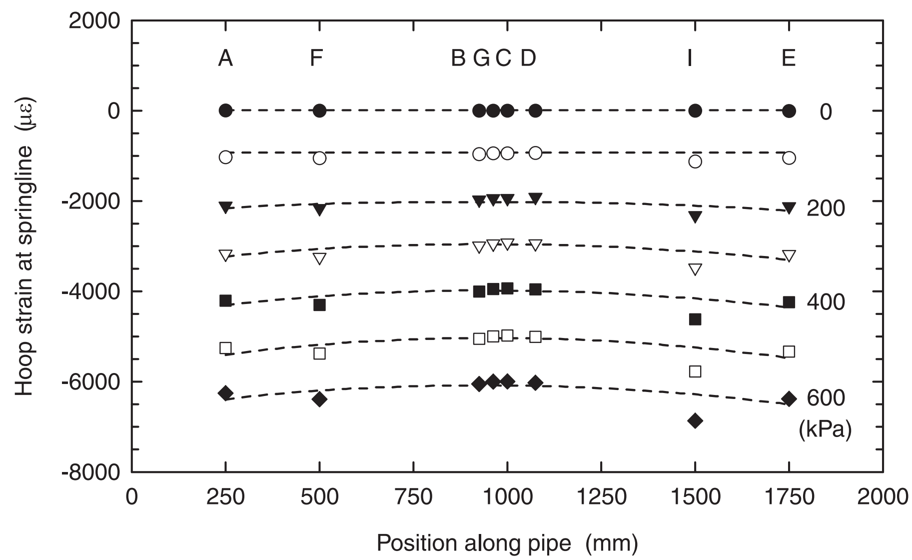


Fig. 9. Variation of hoop strain measured at the interior springline location along the pipe during test T1.



The hoop strains measured at the interior springline (θ = 180°) along the pipe are shown in Fig. 9. Again, excellent agreement between values measured near the centre was found. At a vertical pressure of 600 kPa, a mean of –6000 με and a coefficient of variation of less than 0.5% was calculated from sections B, C, D, and G. The strains measured towards the end of the pipe are slightly larger than those near the centre. However, this difference is not large, as the mean of all the readings is –6200 με with a coefficient of variation of only 5%. Axial strains are tensile and are larger towards the ends of the pipe than at the centre. As found for the interior crown, this results in essentially uniform hoop stresses along the pipe and greater axial stresses near the ends of the pipe.

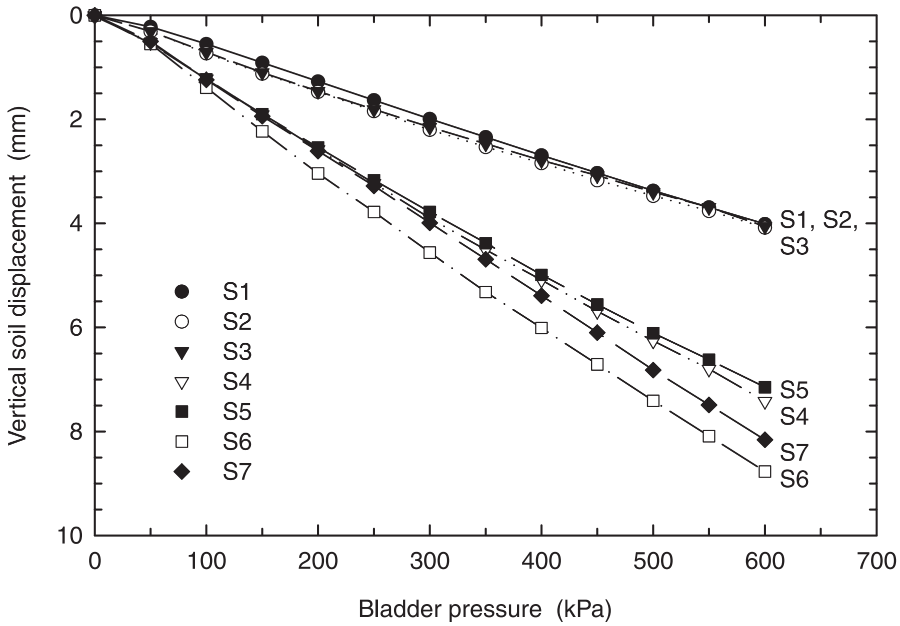
The strain measured at section I is larger than at other locations (Fig. 9). This trend was also observed at the interior crown location (Fig. 8), which suggests that the sand may have been placed at a lower density near section I, resulting in slightly larger strains.

Similar observations of relatively uniform strain along the pipe were made for measurements on the pipe exterior. Overall, the strain response did not vary greatly along the pipe, suggesting only small effects from the boundary conditions imposed in the new laboratory facility. Very good agreement was obtained between results near the centre, indicating that differences observed in subsequent tests with coarse gravel or perforations are due to the coarse gravel or perforations and not the test cell.

Assessment of boundary conditions

Although the free end axial pipe boundary condition did locally influence the strains near the ends of the pipe (but not the hoop stresses), an evaluation of the available data

Fig. 10. Vertical displacement of the soil measured at various locations during test T1.



indicates no significant effect of the test cell boundary conditions on the central portion of the pipe response. Thus it appears that the new facility is performing in accordance with the design expectation. Results from additional instrumentation placed to assess the boundary conditions and visual observations made following the tests are now considered to further demonstrate the effectiveness of the new facility in closely simulating pipe response under biaxial earth pressures.

Soil deformations

Vertical displacements in the sand were measured using settlement plates at seven different locations during test T1 shown in Figs. 2 and 3. Each settlement plate comprised a 100 mm square steel plate (6 mm thick) connected to a 6 mm diameter stainless steel shaft. The shaft extended through the base of the test cell where the vertical movement was measured with a displacement transducer. The shafts were protected from the sand backfill with a stainless steel casing, and grease was used to limit the friction between the shaft and the casing.

The vertical displacements of the sand measured with the settlement plates are plotted in Fig. 10. Measurements at S1, S2, and S3 (all 345 mm from the base) showed very similar results. The average of these measurements was 4.1 mm at an applied vertical pressure of 600 kPa. The coefficient of variation was less than 1% for these readings. No effect from the boundaries is evident from these measurements, even though they correspond to very different locations with respect to the soil boundaries (see Fig. 2).

Settlement plates at S4, S5, S6, and S7 were all located at the same elevation as that of the invert of the pipe. The average of these four readings was 7.9 mm at 600 kPa vertical surcharge. The coefficient of variation of 9% is larger than that calculated from the measurements at the lower elevation. Soil deformations near the north wall (S6 and S7) are larger than those measured at S4 and S5. This difference may be a result of the soil being less dense around these settlement plates, as it may have been more difficult to compact the sand around locations P6 and P7, given the close proximity to the north wall. These results do not show any detrimental effect arising from the lateral boundary. If substantial shear stresses were mobilized along vertical walls, the settlement at S4 would be expected to be less than the value at S5, and the settlement at S6 would be less than that at both S5 and S7; however, this is not the case. The similarity of these values confirms that the relatively low friction angles measured from direct shear tests were mobilized during the tests and that the walls are sufficiently smooth.

Measurements of soil deformation provide a method of inferring the modulus of the sand. Simple estimates of soil modulus between 45 and 50 MPa were made from calculations using isotropic Hooke's Law for the limiting cases of plane strain and plane stress in both orthogonal directions.

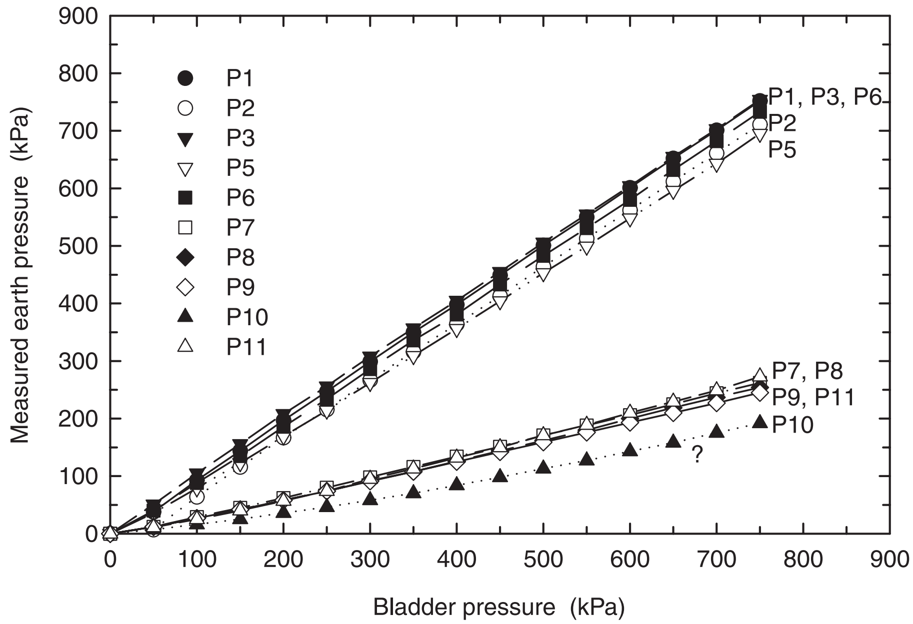
The deformations of the top boundary (i.e., just below the bladder) were observed after each test and showed essentially uniform deflection of the soil surface. Some small local variations were noted, likely arising from small variations in the sand density. Overall, these observations provided visual evidence that the bladder applies a relatively uniform pressure across the top surface, and that the methods of controlling boundary friction appear to be reducing the mobilization of shear stresses to acceptable levels.

Boundary stress measurements

Earth pressure cells with vibrating-wire pressure transducers were used to obtain estimates of the stresses acting on the boundaries of the soil. The location of the earth pressure cells for test T1 are shown in Figs. 2 and 3. Lubricated polyethylene sheets were placed behind the pressures cells that rested against the lateral boundaries to reduce effects from interface friction.

Difficulties with measurements of earth pressures arise because the earth pressure cells (hollow steel discs filled with fluid) typically have a different stiffness to that of the surrounding soil. Additional problems arise with the placement

Fig. 11. Measurements of earth pressures made at various locations around the test cell boundaries during test T4.



of backfill soils around the earth pressure cells at the same density as the soil away from the instrument. Dunnicliff (1988) comments that it is usually impossible to measure the total stresses in the soil with great accuracy.

Typical results are given in Fig. 11 for the backfill conditions used in tests T2 and T4 (those shown in Fig. 4). Variations of earth pressures in the order of 20% were found. Also some nonsensical measurements were obtained. Earth pressure cells were therefore not relied upon to provide precise measurements, but rather were useful in making more qualitative observations regarding the boundary stresses.

The earth pressure cells placed near the top surface of the soil (locations P1, P2, and P3) showed that the vertical stress measured at these locations was essentially the same and equal to the pressure applied by the bladder. These measurements affirm that the simulation of a uniform vertical overburden pressure is successfully attained with the flexible air bladder.

The measurements of lateral earth pressure along the side walls of the facility indicated that the pressures acting on the vertical boundaries parallel to the axis of the pipe were similar and were near to horizontal stresses expected if conditions of zero lateral strain prevailed. A coefficient of lateral earth pressure of 0.3 was found for the sand backfill.

Measurements of vertical stress at the base of the cell were made, with attempts to estimate the proportion of vertical stress reaching the base as another method of assessing the boundary friction acting on the side walls. Vertical stresses measured at P5 and P6 were generally less than those located close to the surface, but were consistent with reductions expected because of boundary friction of 5° (as measured from the direct shear tests on the interface treatment).

Test cell deflections

Outward deflections of the lateral boundaries of the test cell will reduce the lateral stresses that develop in the ground. These deflections were monitored using dial gauges located on all four sides of the test cell during each test to assess the magnitude of these boundary movements. The location of the boundary measurements are shown in Figs. 2 and 3. For example, the boundary deformations measured on the west face of the test cell are denoted as WR2 for the steel ring second from the top and WPL1-2 for the plate at midspan between rings 1 and 2.

The deformation of the steel boundaries results from four components. The first is local plate deformation that occurs between the steel frame supports. Thick (40 mm) steel plates were used to limit this deformation. The second component arises from the bending of the side wall unit (composite plate and frame). The third component occurs because of the axial elongation of the adjacent walls joining it with the opposite side (i.e., elongation of north and south walls from pressures acting on the east and west walls, and vice versa). The fourth component is deflection that results from small distortions of the box-like structure in the horizontal plane.

The deflections measured during test T1 are plotted in Fig. 12. Boundary deflections were measured at a few more locations during test T3, the results of which are shown in Fig. 13. It is apparent that structure does not deflect in a symmetric manner about the centre of the test cell, as opposite walls do not deflect the same amount. Deflections of the west wall are larger compared with those of the east wall, for the boundary parallel with the axis of the pipe; the deflections of the south wall are larger than those of the north wall, for the boundaries perpendicular with the pipe axis. This was a consistent observation for the other tests conducted.

For test T1, the deflection of the east wall measured at the plate in between the second and third rings from the top (EPL2-3) was 0.24 mm, whereas the deflection on the opposite wall at this location was 1.18 mm (WPL2-3), both at an applied bladder pressure of 600 kPa (Figs. 12*a* and 12*b*, respectively). Despite this difference in boundary deflections the response of the pipe was nearly the same at the east and

Fig. 12. Lateral deflections (δ_{sw}) of the exterior walls of the test cell measured during test T1: (*a*) east wall; (*b*) west wall; (*c*) average of east and west walls; (*d*) north wall; (*e*) south wall; (*f*) average of north and south walls.

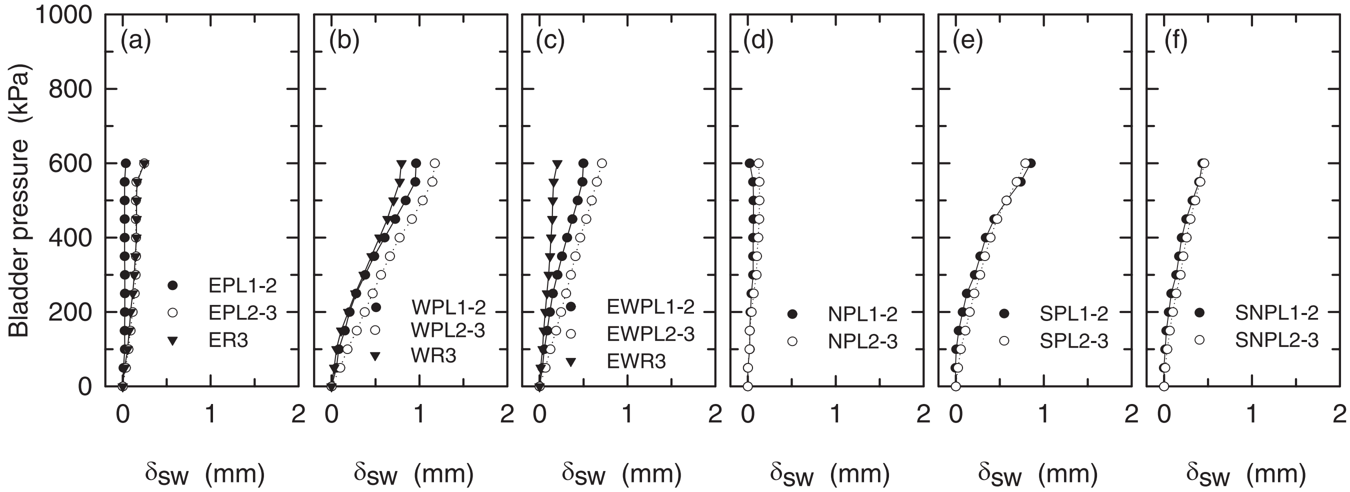
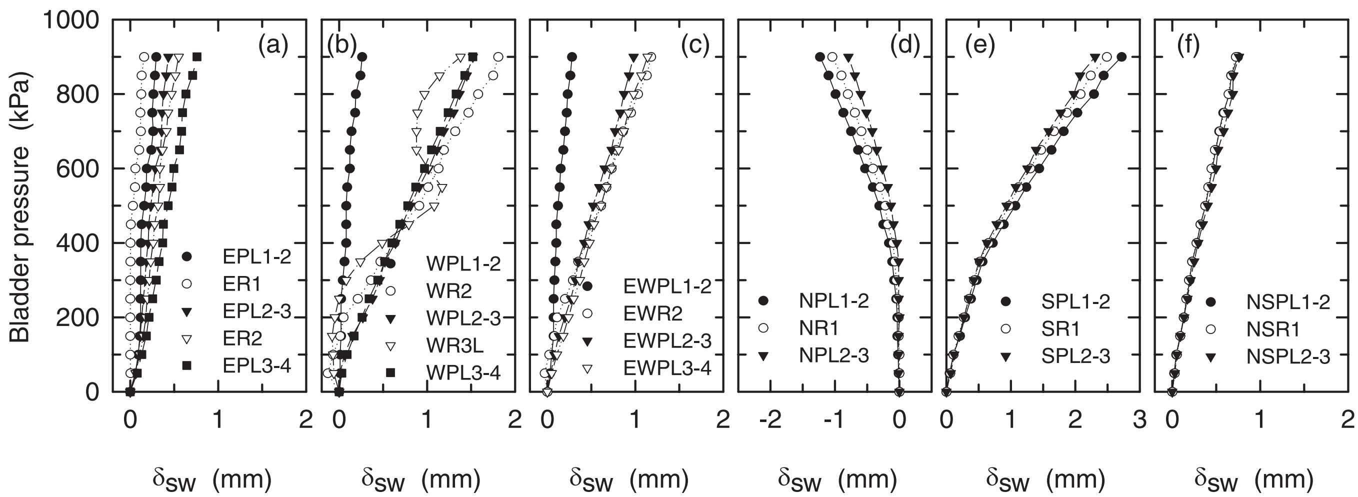


Fig. 13. Lateral deflections of the exterior walls of the test cell measured during test T3: (*a*) east wall; (*b*) west wall; (*c*) average of east and west walls; (*d*) north wall; (*e*) south wall; (*f*) average of north and south walls.



west springlines (refer back to Fig. 7). Measurements of earth pressures at locations P7 and P9 (made near the measurement of test cell deflections) were similar, which implies that the ground pressures are similar along these boundaries. Despite the large difference in test cell deflections, the evidence that the pressures are nearly equal along these walls suggests that the soil is not greatly impacted by the different boundary movements.

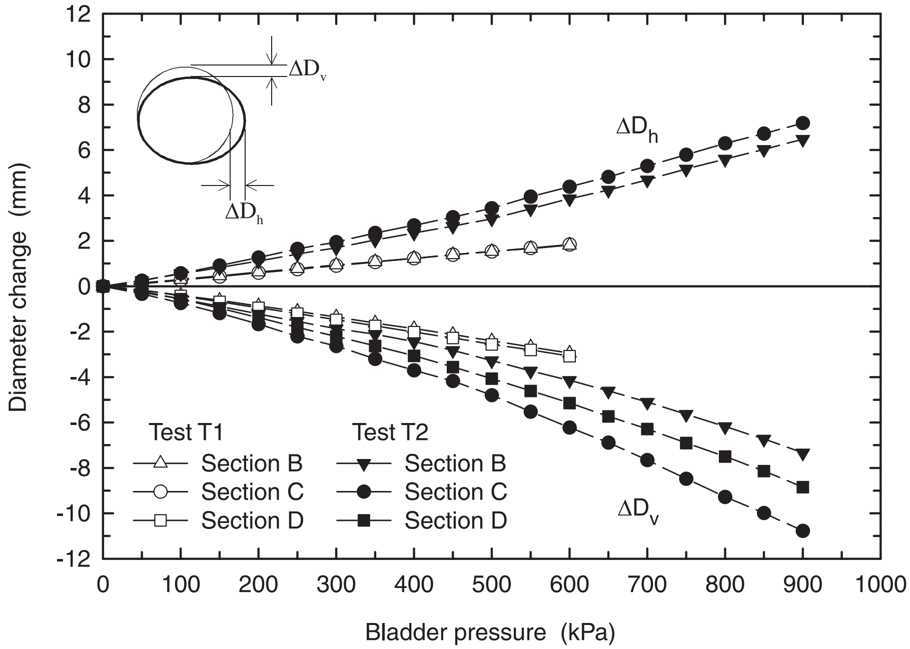
The average of EPL2-3 and WPL2-3 yield a value of 0.7 mm at 600 kPa vertical surcharge (denoted as EWPL2-3 in Fig. 12*c*). This is similar to the average boundary deformation found at this location during test T3 (Fig. 13*c*). At 900 kPa the average outward displacement of the plates is roughly 1 mm.

Deflections measured at the north wall are very small (roughly 0.1 mm at NPL2-3). In fact, during test T3 the north wall actually deflected inward (i.e., towards the centre of the test cell), accompanied by the larger deflections measured at the south face. The average of the north and south boundary deformations is less than 0.8 mm at 900 kPa. Similar results were obtained for the other tests that were conducted.

Measurements of strain in the steel frames (not reported here) indicate relatively uniform response around the perimeter of the test cell. This suggests that the difference in deflections between the east and west walls and the north and south walls arises from a very small distortion of the test cell in a horizontal plane.

The measured deflections were slightly larger than anticipated compared with design calculations, as full composite action between the beams and plates was not likely mobilized (especially for the axial elongation of the wall which comprises the majority of the deflection and may lead to distortion of the structure). It was shown by Brachman et al. (2000) that boundary deformations of 1 mm at 900 kPa do induce stresses that deviate from the boundary stresses as idealized in Fig. 1 but that these differences are small. Reexamining the analysis of Brachman et al., it was estimated that lateral stresses are reduced by roughly 10% as a result of these boundary deformations. Lateral pressures smaller

Fig. 14. Vertical (ΔD_v) and horizontal (ΔD_h) diameter changes of the pipe measured at sections B, C, and D during tests T1 and T2.



than those for zero lateral strain conditions would result in larger pipe deflections. It was estimated that the vertical diameter decrease would be 10% larger in the test cell relative to that expected for perfectly rigid walls. Larger pipe deformations lead to greater bending stresses in the pipe. The pipe response in the new laboratory model is therefore more severe than the behaviour expected under deep and extensive overburden pressures (i.e., the measured pipe deflections and stresses are larger than would be expected under conditions of zero lateral strains).

Comparison between sand and coarse gravel backfill

With the suitability of the new test cell demonstrated, its application is now illustrated by examining the performance of a landfill leachate collection pipe. Comparison of tests T1 and T2 provides a direct measure of the effect of how the conditions in a landfill influence the global pipe response. These tests involved the same pipe but were conducted with two different backfill conditions that provide different loading and support for the pipe. The support provided by the sand backfill will tend to be more uniform, with the small sand particles providing almost continuous loading and support for the pipe. With coarse gravel the loading and support will be discontinuous, arising from fewer contact points that are randomly distributed around the circumference. The presence of a clay layer beneath the gravel may also potentially influence the results in test T2. Observed differences between the results from T1 and T2 can therefore be attributed to the simulated landfill conditions (i.e., gravel backfill and clay layer).

Pipe deflections

Vertical (ΔD_v) and horizontal (ΔD_h) diameter changes of the pipe measured during tests T1 and T2 are plotted in Fig. 14 against the applied bladder pressure. These measurements were made using potentiometers at sections B, C, and D. The tolerance associated with the deflection readings was ±0.01 mm. Some circumferential shortening of the pipe occurred, since the decrease in vertical diameter is larger than the increase in horizontal diameter of the pipe when subject to the biaxial earth pressures.

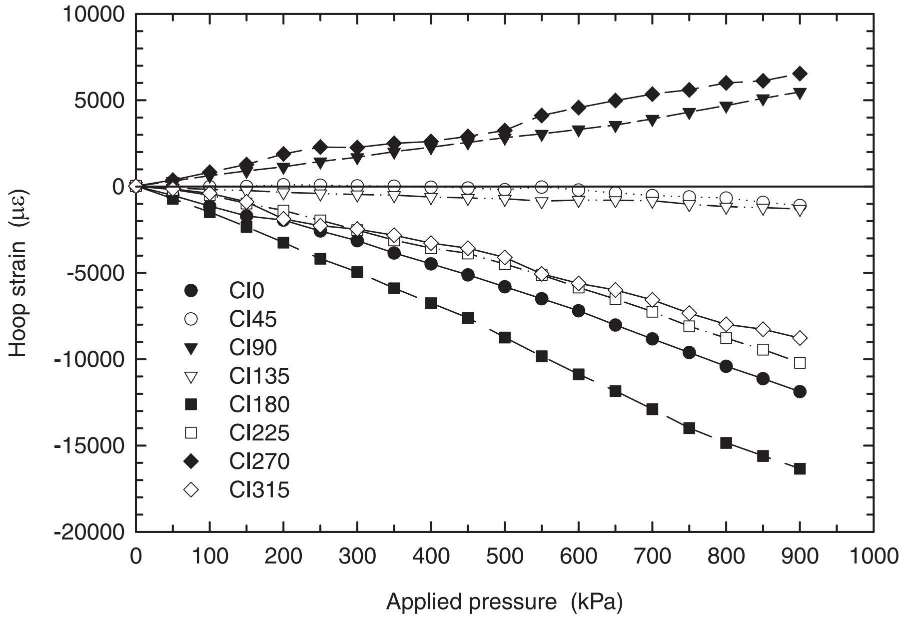
With sand backfill (T1), the measured vertical diameter changes at sections B and D are essentially identical, with a magnitude of –3.0 mm at an applied vertical pressure of 600 kPa. Excellent agreement was also observed for the horizontal diameter change, as increases in diameter of 1.8 mm were recorded at sections B and C (instrumentation problems precluded measurements of ΔD_v and ΔD_h at section C). The observed consistency between pipe deflections is similar to the results observed for the measured strains and is consistent with the premise that, when uniformly compacted, sand essentially provides continuous loading and support to the pipe.

For the simulated landfill conditions of test T2, greater variations and larger magnitudes of pipe deflections were measured relative to those with the sand backfill. Greater variations in measured pipe defections arise predominately from local backfill effects and are discussed further with regard to the pipe strains. Pipe deflections are larger for the simulated landfill conditions, largely because the coarse gravel has a lower lateral earth pressure coefficient K than the sand. The possibility that the soft clay beneath the pipe also produces larger deflections cannot be excluded, however similar results were found with tests conducted with only coarse gravel and no clay (Brachman et al. 1998).

Hoop strains

Hoop strains measured at section C around the interior of the pipe are plotted in Fig. 15. These values do not increase at a steady rate like the results from the test with sand backfill

Fig. 15. Measured hoop strains ε_θ on the interior surface of the pipe at section C during test T2.



(Fig. 7) but vary during the test, with numerous and varied changes. These fluctuations during the test are attributed to the rearrangement of gravel particles within the backfill as evidenced by audible sounds of particles moving during testing and some particle breakage of the coarse gravel found while excavating the materials after the test.

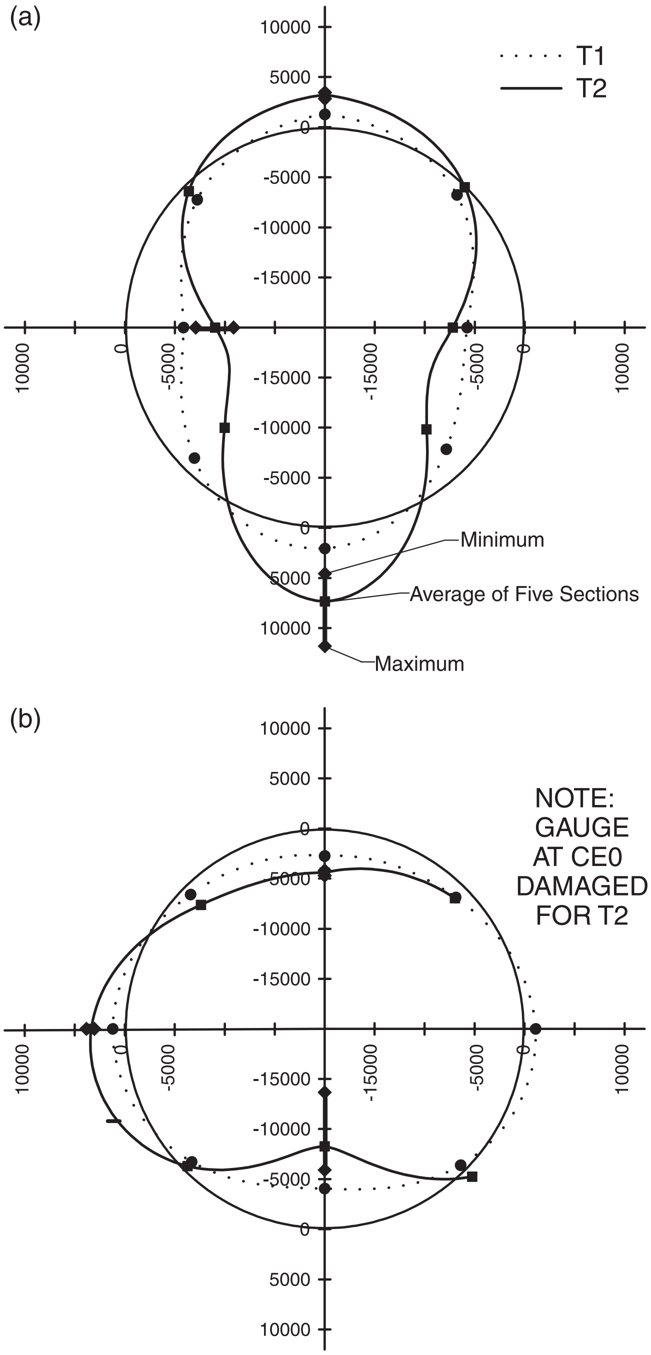
More important, however, are variations in pipe response caused by the coarse gravel. For example, a large difference between the readings at the springlines (CI0 and CI180) was found with the coarse gravel backfill (Fig. 15). This differs greatly from the response observed when tested with sand backfill, where the two measurements at the interior springlines were nearly identical (Fig. 7). The large difference between strains at the springlines for test T2 are caused by local bending effects from the coarse gravel backfill. The spacing between gravel contacts and varying forces from contact to contact induce local bending strains (incremental compressive and tensile strains) that result in either increased or decreased compressive strain at the springline.

Figure 16 plots the hoop strains measured at section C for test T2 on both the pipe interior and the exterior at an applied bladder pressure of 600 kPa. Also shown are the average, maximum, and minimum strains at the five central sections of test T2 (B, G, C, H, and D) for crown, springline (180°), and invert locations to provide a measure of the influence of the coarse gravel on the strains within the pipe. Greater variations occurred at the invert compared with the crown or springline. For example, at the interior invert the hoop strains vary from 4600 to 11 800 με along an axial distance of only 150 mm. Perhaps it is not surprising that the response near the invert is particularly sensitive to the gravel backfill, given that the contacts with the backfill are likely to be more varied at the invert than at other locations around the pipe. Consistent with field installations, the bedding material below the pipe invert was prepared as level as possible with the 50 mm gravel, the pipe was then placed on top, and then the remainder of the gravel was placed. The gravel material placed around the pipe has a greater potential to rearrange itself with respect to the pipe during construction than the material that is already placed at the invert.

A possible approach to account for variations in strain during pipe design may be to multiply the average strain by a strain magnification factor obtained from laboratory tests to provide an estimate of the maximum strain. For example, using the data from test T2, the maximum strain at the interior invert is 1.6 times the average value. However, care is required because the strains measured with the coarse gravel backfill are highly variable. Consequently, four additional tests were conducted on two different pipe samples to better quantify the effects from the coarse gravel backfill, and this work has been reported by Brachman (1999). The observations made from the limited number of measurements from test T2 are consistent with the additional tests that were conducted. It is important to account for increases in strains in the pipe from local bending effects when designing leachate collection pipes.

The hoop strains measured at section C for test T1 are also plotted in Fig. 16. These polar plots illustrate the variation of hoop strain around the pipe. On the interior surface, strains are compressive except for tensile strains at the crown and invert. Strains on the exterior surface are also predominantly compressive, except for tensile strains at the springlines. Hoop strains at the crown, springline, and invert locations of the pipe are all larger when tested with coarse gravel backfill compared to the sand backfill. This is because of the lower lateral confinement provided by the gravel in a state of biaxial compressive pressures, as it has a lower coefficient of lateral earth pressure (K) compared with the sand, as discussed with regards to the pipe defections. The lower K value leads to greater differences between vertical and horizontal stresses in the soil and hence greater circumferential bending in the pipe. This produces larger hoop strains at the locations where bending is greatest (i.e., at crown, springline, and invert locations). The greater strain with the

Fig. 16. Measured hoop strains ε_θ on the (*a*) interior and (*b*) exterior surfaces of the pipe for tests T1 and T2 at an applied vertical surcharge of 600 kPa.



simulated landfill conditions can be accounted for in design by using the appropriate value of *K*.

Figure 16 also shows that hoop strains at the haunches are larger for the gravel backfill relative to the sand backfill. Since it is difficult to place coarse gravel in good contact with the pipe in the haunch region, this lack of backfill support likely produces the greater circumferential bending in the pipe. This type of pipe response is consistent with the "inverted heart shape deformation" arising from poor support at the haunches reported by Rogers (1988). Larger strains resulting from poor backfill support may become significant if perforations are located at the haunches. Additional work is required to resolve this issue.

Summary and conclusions

The performance of a new laboratory facility for testing small-diameter buried pipes (less than 300 mm diameter) subject to the biaxially compressive earth pressures expected to prevail under deep and extensive overburden was examined. The new facility consists of a prism of soil 2.0 m wide × 2.0 m long × 1.6 m high contained within a stiff steel structure. Overburden pressures are simulated with a pressurized air bladder. Lateral earth pressures are developed by limiting the lateral soil strains.

Measurements of pipe and soil response, in addition to visual observations after testing, indicate that the boundary conditions imposed during testing in the new facility closely simulate those expected to prevail under deep and extensive burial. The bladder design adopted consists of a flexible rubber membrane with a mechanical seal around the perimeter and was found to provide a uniformly distributed pressure across the top surface of the soil. Effects of boundary friction were limited to minimal levels by using lubricated polyethylene sheets that had adequate protection from damage caused by the backfill soil. The stiffness of the lateral boundary was sufficiently large to induce lateral stresses close to those for zero lateral strain conditions. Overall, the effects on the pipe arising from the idealizations involved in the laboratory model are small. The deviations from those expected to occur if plane strain conditions were realized in the field involve slightly larger pipe deflections (10% larger for the vertical diameter change) and, consequently, somewhat larger bending stresses in the pipe.

No major effects on the pipe response were discernable from the boundary conditions for the particular conditions tested. Greater effects may occur for significantly larger diameter pipes or much stiffer soil materials. In such cases monitoring of the pipe and structure and careful consideration of the measured results guided by finite element analysis like that conducted by Brachman et al. (2000) are warranted.

The application of the new test cell was illustrated by using it to assess the response of a small-diameter landfill leachate collection pipe under two different backfill conditions. This comparison showed that the performance of the pipe is significantly impacted by the coarse gravel backfill used in landfill drainage layers. Maximum pipe deflections and surface strains were nearly twice as large when tested in the coarse gravel compared with the sand backfill. Much greater variations of deflection and strain were measured with the coarse gravel when compared with the sand backfill due to local bending effects from the coarse gravel and poor backfill support at the haunches. The measured variations of deflection do not, however, preclude the use of coarse 50 mm gravel in direct contact with leachate collection pipes. In fact, the 220 mm o.d., SDR 9 HDPE pipe would be expected to perform well in a medium-size landfill (vertical pressures up to 250 kPa) for the given gravel and perforation size and pattern tested. Further study of these variations and the practical implications for a perforated leachate collection pipe are presented by Brachman (1999).

Acknowledgements

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