Structural Performance of Profile-Wall Drainage Pipe—Stiffness Requirements Contrasted with Results of Laboratory and Field Tests

C. D. F. ROGERS, P. R. FLEMING, M. W. J. LOEPPKY, AND E. FARAGHER

This paper describes the development of the current United Kingdom (UK) stiffness requirements for profile-wall flexible pipes and assesses their limitations. Laboratory testing of flexible pipes ranging in diameter from 100 to 375 mm is described. The results indicate that deformations and circumferential strains are small, even under severe loading, and generally fall well within the current limits specified in the appropriate U.K. standards. Creep stiffness specifications and design and installation standards are assessed in light of the collected data, and recommendations for improved criteria are propounded.

United Kingdom requirements for nonpressure drainage pipes cover a range of properties including impact resistance, watertightness, material properties and time-dependent stiffness. This last property is of particular importance to pipe manufacturers in engineering their product and to civil engineers for specification and design purposes.

Profile-wall pipes are a relatively recent innovation and have not yet been considered in British Standards (BS), although they will be included in forthcoming Euro Norms. In the absence of an applicable BS, "fitness for purpose" assessments for construction products are carried out by the British Board of Agrément (BBA), frequently in consultation with the U.K. Department of Transport (DOT). The BBA's mandate also includes conformance testing, using standard, adapted or ad hoc testing methods, to ensure compliance with the established specifications.

DEVELOPMENT OF EXISTING U.K. STANDARDS

Specification criteria for materials used on projects controlled by the DOT are contained in the *Specification for Highway Works* (*1*). This document provides specification requirements for use in public purchasing contracts and is therefore the specification for the majority of applications for the pipes under consideration. A related document, DOT Highway Advice Note HA40/89, *Determination of Pipe and Bedding Combinations for Drainage Works* (*2*) states that the DOT requires profile-wall, non-pressure drainage pipes to meet a minimum 50-year extrapolated stiffness of 1,400 Pa (0.2 lb/in.^{2}) when tested in accordance with Appendix B of BS4962:1989, *Specification for Plastics Pipes and Fittings for use as Subsoil Field Drains* (*3*). Tabulated safe burial depth recommendations in HA40/89 were developed using the Transport Research Laboratory (TRL) method, an analytical approach which treats the pipe-soil structure as the basic structural unit (*4,5*). The calculations are based on conservative assumptions and, predictably, pipes conforming to these requirements have been found to experience long-term diametral strains less than the widely accepted limit of 5 percent. The 5 percent limit is based on a factor of safety of four applied to the historically accepted limit of 20 percent deformation to avoid snap through buckling of large diameter steel corrugated culverts. It should be noted that the term "pipe stiffness" in U.K. standards is analogous to the term "stiffness factor" defined in ASTM D2412-87 (*6*), which refers to a short-term constant rate of deflection test. It is therefore fundamentally different from the long-term constant load creep test specified in the United Kingdom.

The approach of HA40/89 (*2*) is excessively conservative by accepting the traditional 5 percent diametral strain limit in addition to applying a factor of safety of two to the pipe stiffness and assuming worst case installation conditions. This "belt and braces" approach, coupled with the long-term creep test requirements of BS4962 (*3*), has resulted in the substantial overdesign of pipes to meet material and structural criteria. An appraisal of the functional requirements for a pipe in use under load, based on engineering principles, would yield a better engineered, hence more economical design. Flexible pipe design would then follow the route taken by other branches of civil engineering, which have economized on the use of materials by a more sophisticated and rational appraisal of basic structural requirements.

DESCRIPTION OF U.K. TESTS AND REQUIREMENTS FOR FLEXIBLE PIPE

BS4962 (*3*) specifies a parallel-plate loading test, one test of many employed by the BBA in certifying flexible pipe products for use in roads and bridges, in which a constant load is applied to a laterally unrestrained pipe sample for 1000 hr, with deflection readings being taken at set times. A 50-year deformation value is extrapolated from the data, using a computerized nonlinear optimization technique, and the design stiffness calculated.

It is the authors' opinion that the rationale of the test, and of the test method itself, is questionable. The creep performance of the pipe is considered out of context (i.e., with the pipe not buried in a soil surround) and therefore no account is taken of the lateral support provided to the pipe by the sidefill, or of load shedding by the pipe due to arching effects in the soil above the pipe crown as the

Loughborough University of Technology, Loughborough, Leicestershire, England LE11 3TU.

pipe deflects under load. Furthermore, the plate load applied is based not on the pipe diameter, expected static or dynamic loading, or other known parameter, but on the deflection after five minutes, and hence on the short-term stiffness itself. Consequently, two otherwise identical pipes made of different materials may be tested with vastly differing applied loads. Another weakness is the fact that, by using the results of testing over 1000 hr to determine a stiffness fifty years into the future, small experimental inaccuracies will significantly affect the extrapolated result. The test is, however, required for certification and is often justified by its ease of repeatability under controlled laboratory conditions. In addition, there is evidence to indicate that pipes passing the test do perform adequately when installed in accordance with the Specification for Highway Works.

The Specification for Highway Works (*1*) stipulates acceptable products for drainage and ducting applications, in addition to detailing acceptable materials and practices associated with pipe installation. A companion document, *Highway Construction Details* (*7*), provides standard drawings of acceptable trench dimensions and bedding, haunch, and surround criteria. These documents do not make pipe design recommendations, leaving an explanation of flexible pipe design and tabulated safe depth ranges for various installation conditions to HA40/89 (*2*).

The safe depth ranges in HA40/89 (*2*) are calculated using the TRL method (*5*) and assume worst cases of pipe stiffnesses and installation practices, although no precise details of the base data used are given. U.K. pipe manufacturers tend to use the Iowa formula (*8*) which, although regarded as being less theoretically sound, is nevertheless widely accepted as a valid method due to the data accumulated over the years for the modulus of soil reaction (E'), particularly by Howard (*9*), and because it has proved to be a relatively reliable predictor of flexible pipe deflection. The lack of usable back-analyzed soil stiffness data for the more theoretically justifiable TRL method implies uncertainty in the allowable pipe installation conditions tabulated in HA40/89. Indeed, HA40/89 admits explicitly that the charts contained therein are based on conservative design parameters. There is a wide range of factors to consider in predicting the performance of the pipe-soil structure using the TRL method, including installation procedures, site conditions, trench geometry, and withdrawal of trench support. Whichever design method is used, the soil stiffness dominates the design and thus pipe stiffness is not the principal variable. A thorough appraisal of the various design methods is given elsewhere (*10*).

LABORATORY TESTING OF FLEXIBLE PIPE

Testing Equipment

Pipes with an internal diameter of 300 mm or less were tested in a 1.0 × 1.1 × 1.0-m deep box, whereas larger pipes were tested in a 1.5 × 1.8 × 1.5-m deep box. Test boxes should ideally have rigid sides, if zero lateral strain conditions are required. It is appreciated that trench walls in practice will deflect when stressed, and thus a small lateral deflection of the box walls would be acceptable. Measured lateral deflections of the test box walls were very small, typically less than 2 mm under maximum load conditions, and similar to those expected for a natural soil forming the walls of a trench.

The loading arrangement provided an approximately uniform vertical stress, achieved using a natural rubber membrane mounted to the underside of the test box lids. Water was forced between the lid and the rubber membrane until the desired pressure (loading) was achieved. Two magnitudes of loading were applied, 70 and 140 kPa, to simulate burial at two different depths. For cyclic loading, to simulate the passage of a vehicle over the pipe, an automated system applied a pressure varying sinusoidally between 0 and 70 kPa.

Diametral strain measurements were taken using three linear variable differential transformers, mounted on a self-righting sledge. Circumferential strains were measured on the 375-mm pipe using uniaxial, foil-type strain gauges (mounted in epoxy resin) with coefficients of thermal expansion balanced to the pipe material. The gauges were affixed to the internal wall of the pipe at both single- and twin-wall sections to determine any differences in behavior between them. All data were recorded on a dedicated data acquisition system.

Test Procedures

Twin-wall annular corrugated HDPE pipes with inside diameters ranging from 100 to 375 mm were tested. Pipe stiffnesses for 5 percent deflection [as defined by ASTM D2412-87 (*6*)] were 97.8 lb/in.^{2} (674.4 kPa), 66.7 lb/in.^{2} (460.0 kPa), 71.9 lb/in.^{2} (495.5 kPa), 65.4 lb/in.^{2} (450.6 kPa), and 49.1 lb/in.^{2} (338.2 kPa), for 100-, 150-, 225-, 300-, and 375-mm diameters, respectively. The placing and compaction of the surround and backfill to the pipes, carried out in layers in accordance with typical site practice, constitute the installation phase. The bed, surround, and backfill materials used were a well graded river sand ($c_u = 4.37$, $c_c = 0.65$, $D_{10} = 0.19$ mm) and river gravel ($c_u = 1.55$, $c_c = 0.96$, $D_{10} = 5.5$ mm). Bedding layers were 100 mm thick for all tests. The river sand surround and backfill was placed either virtually uncompacted or heavily compacted in layers not exceeding 150 mm in depth. This represents both very poor and very good site practice. The river gravel, being relatively uniform 10 mm sub-rounded (pea) gravel, is essentially self-compacting, and compaction on site would only be justified if required to bed the material into soft trench walls. The river gravel was placed carefully on both sides of the pipe before completion of the backfill in one continuous operation. This represents typical U.K. installation conditions.

There were three loading phases:

1. Application of a static 70-kPa stress, to simulate a stationary heavy vehicle or burial to a depth of approximately 4 m.
2. Application of a cyclic 70-kPa stress, to simulate heavy vehicle loading over a shallow buried pipe. The frequency of the cycle was 0.01 Hz, 1000 cycles being applied.
3. Application of a static 140-kPa stress, to simulate a burial depth of approximately 8 m.

The static stresses were applied for 12 hr and, after unloading, a period of 4 hr was allowed for recovery.

Pipe Deflections

Selected test data are presented in Table 1. The values of vertical and horizontal diametral strain (VDS and HDS) are given after installation (I), just before the load is released at the end of the 70-kPa static load (70 S), 70-kPa cyclic load (70 C), and 140-kPa static load (140 S) sequences, and at the end of the test after final recovery. A set of vertical and horizontal test data is illustrated in Figures

TABLE 1 Experimental Data at Critical Points of Tests

PIPE SIZE	SOIL	SIDEFILL COMPACTION	VDS HDS	I	70S	70C	140S	END
100	RS	Not compacted	VDS HDS	0.07 -0.12	0.8 -0.8	2.7 -2.4	2.8 -2.4	2.6 -2.4
100	RS	Heavily compacted	VDS HDS	-0.17 -0.01	-0.11 -0.03	0.08 -0.19	0.10 -0.16	0.04 -0.17
100	RG	Not compacted	VDS HDS	0.03 -0.03	0.4 -0.3	1.2 -0.9	1.2 -0.9	1.2 -0.9
150	RS	Not compacted	VDS HDS	0.10 -0.10	1.6 -1.0	2.6 -1.9	3.3 -2.1	3.0 -2.1
150	RG	Not compacted	VDS HDS	0.15 -0.05	1.3 -0.7	1.9 -1.1	2.3 -1.3	2.1 -1.3
225	RS	Heavily compacted	VDS HDS	-0.12 0.06	0.1 0.0	0.1 -0.1	0.2 -0.1	0.1 -0.1
225	RS	Not compacted	VDS HDS	0.04 -0.07	1.2 -1.0	2.0 -1.9	2.7 -2.1	2.1 -1.8
225	RG	Not compacted	VDS HDS	0.14 -0.11	1.0 -0.7	1.5 -1.1	1.9 -1.3	1.6 -1.2
300	RS	Heavily compacted	VDS HDS	-0.31 0.40	0.0 0.3	0.1 0.2	0.4 0.1	0.2 0.2
300	RS	Not compacted	VDS HDS	-0.01 -0.05	2.7 -2.6	4.2 -4.2	5.1 -4.7	4.1 -4.2
300	RG	Not compacted	VDS HDS	0.18 -0.08	1.5 -1.2	2.2 -1.9	2.8 -2.5	2.3 -2.4
375	RS	Not compacted	VDS HDS	0.14 -0.03	1.3 -0.6	3.9 -2.6	4.4 -2.7	4.0 -2.6
375	RS	Heavily compacted	VDS HDS	-0.70 0.80	-0.6 0.7	-0.6 0.7	-0.5 0.7	-0.7 0.8
375	RG	Not compacted	VDS HDS	-0.30 0.30	0.0 0.1	0.3 -0.1	0.6 -0.2	0.2 -0.1

Legend and Sign Convention

Soil Types

RS = Well graded river sand
RG = Relatively uniform, sub-rounded 10 mm gravel

Deflection

VDS = Vertical diametral strain (% of mean external diameter)
HDS = Horizontal diametral strain (% of mean external diameter)
(Positive diametral strain values indicate a decrease in pipe diameter.)

1 and 2 for a pipe with an internal diameter of 375 mm installed in uncompacted, 10-mm pea gravel (typical U.K. site practice).

The test results indicate minimal deformations (less than 1 percent) during the installation phase. Negative values of VDS often occurred, particularly in heavily compacted sand installations, indicating an increase in diameter along the vertical axis [see also Rogers (*11,12*)]. The 70-kPa static load, representing a parked vehicle or a relatively deep burial in the United Kingdom, produced very small deformations, with the maximum VDS for this phase being 2.7 percent (300-mm pipe in a poor surround).

It is appreciated that construction traffic loading subsequent to pipe installation can be significant, particularly on road construction sites, and, depending on the cover depth of the pipe, significant deformations are possible. Minimum burial depths for pipes not protected by special measures are specified (*13*) and implied (*2*) to ensure that such loading is not critical, the 70-kPa cyclic load being used here to simulate the maximum loading under minimum burial depths. During the 70-kPa cyclic load phase, the rate of increase of VDS and HDS decays exponentially. Associated field trial data, in which cyclic loading was applied to shallow buried pipes using a heavily laden vehicle, demonstrate similar trends and degrees of deformation, thus providing a high degree of confidence in the laboratory simulations. A progressive reduction in the amplitude of the elastic deformation caused by the cyclic load was also observed as the tests progressed.

Tests using heavily compacted well graded river sand demonstrated remarkably good performance. Deflections during all phases of the test were extremely small, typically only just becoming positive at the end of testing (following negative installation deflections). This is because a pseudo-elastic system is in existence in well compacted material, in which little further fill compaction can take place under a subsequently applied load. Tests in which the river sand was placed without any sidefill compaction applied, simulating very poor (and, according to U.K. specifications, unac-

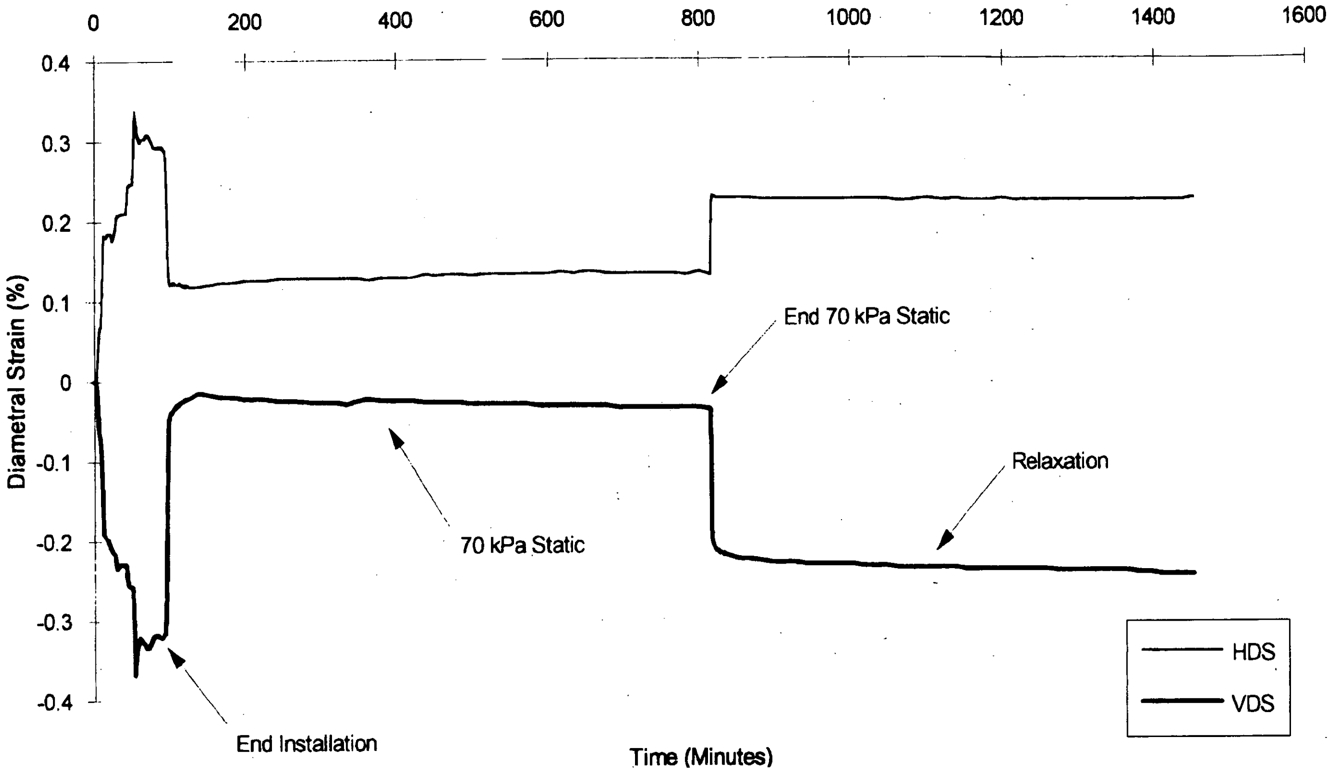

FIGURE 1 Diametral strains due to installation and 70-kPa static stress phases.

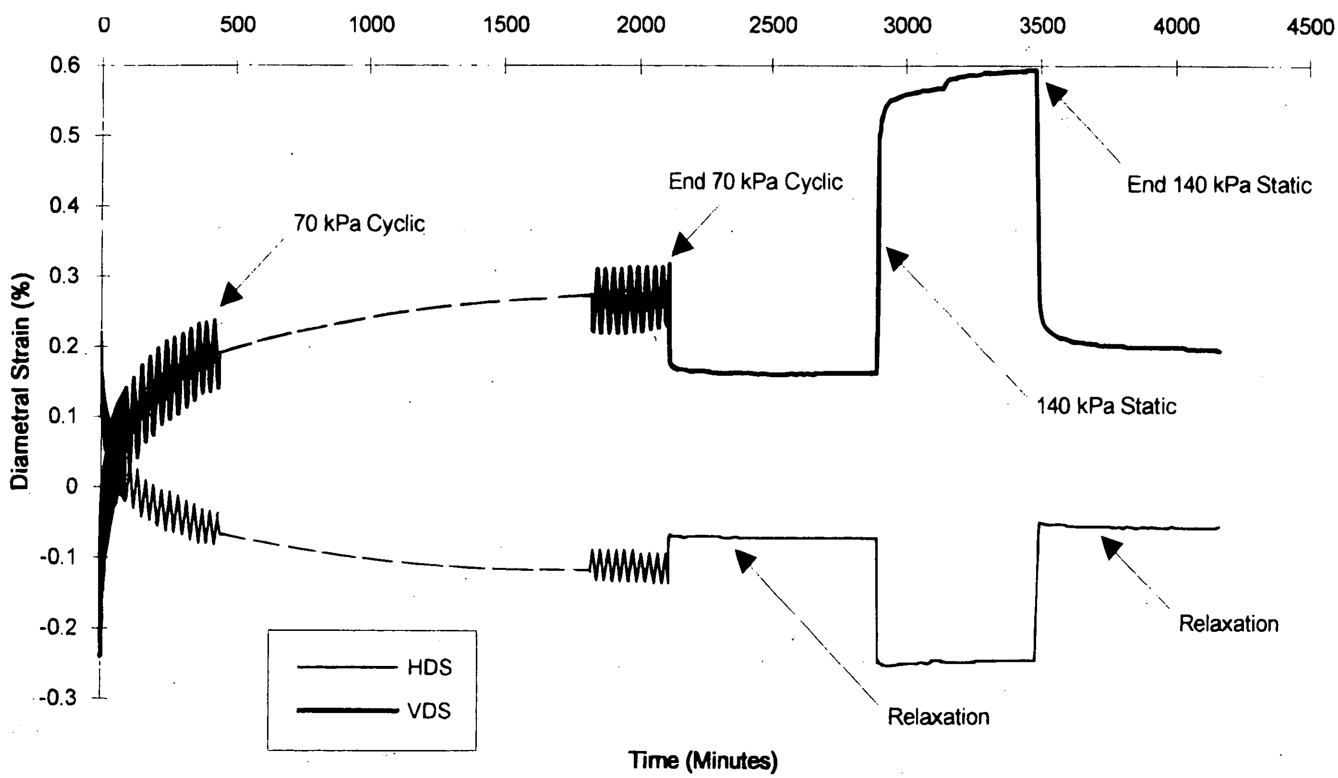

FIGURE 2 Diametral strains due to 70-kPa cyclic and 140-kPa static stress phases.

ceptable) site practice, yielded much greater deformations (because significantly greater pipe deformation is required to generate the supporting equilibrium passive earth pressure). However, in only one case (300-mm pipe in uncompacted sand) does the VDS exceed the benchmark value (5.0 percent), before falling to 4.1 percent at the end of the test. However, this result must be interpreted in the context of the very poor (unacceptable) site practice simulated and the large stresses applied. A similar pattern of results, though with typically far higher VDS for comparable loading, has been indicated by Rogers (*11*) for smooth wall polyvinyl chloride (PVC)-U pipe, which has a standard dimension ratio (SDR, the ratio of external diameter to wall thickness) of 41. Corrugated HDPE pipe, with equivalent SDRs ranging from 12.2 to 20.4 (based on an equivalent single wall pipe with the same moment of inertia as the corrugated profile) is thus structurally superior. In tests simulating typical UK site practice using an uncompacted 10-mm pea gravel surround, the maximum VDS values (2.8 percent under maximum load and 2.3 percent at the end of testing) were recorded for 300-mm pipe.

The best performance was achieved by the 100-mm pipe, indicating that this pipe/soil system is superior because of the existence of a narrower structural span exposed to the applied loading, and a narrower span over which to induce arching in the surrounding soil. The 100-mm pipe is also somewhat stiffer than larger diameter pipes, although still proportionally small compared to the total stiffness of the pipe-soil structure.

Several tests were duplicated under identical conditions and these achieved a very high degree of repeatability, providing confidence in the procedures used. Potential reductions in loads transmitted to the pipes due to frictional effects of the test box walls have also been investigated. The large steel-sided test box was lined with phenolic film-faced plywood, similar to that of the smaller test box, and a polyethylene sheet was placed against the smooth surface to ensure a low friction interface. The results demonstrated that box wall friction effects were insignificant for installations using uncompacted river sand (anticipated to be the critical case), the differences in the comparative tests lying well within normal experimental ranges. It should also be noted that the large test box is four times wider than the largest pipe tested, further minimizing boundary effects.

Pipe Wall Strains

Pipe wall strains were measured beneath the corrugation, or ridge (single wall) and the valley (twin wall) for the tests using the 375-mm pipe only. The data for the two (repeat) tests on the pipe buried in uncompacted river gravel will be presented here to demonstrate the behavior under different applied stress conditions. It should be noted that twin wall strains are plotted to demonstrate behavior since these are in all cases more extreme. Single wall data for full test loading are subsequently presented to illustrate this point. In addition, all strain data are for the pipe with the stress removed and after recovery.

The wall strains caused by installation were expected to be relatively low since no compaction was applied to the gravel. The twin wall strains shown in Figure 3 confirm this. Test 2 exhibits small strains at all points except the invert (180°), where a compression of 1000 με is recorded. In contrast, the pipe used in Test 1, although recording almost identical invert strains, exhibits a compression of 1430 με at 90°. This was caused by the accidental tipping of gravel sidefill at one side of the pipe only when filling the box, subsequent shovelling being required to feed the uncompacted gravel carefully

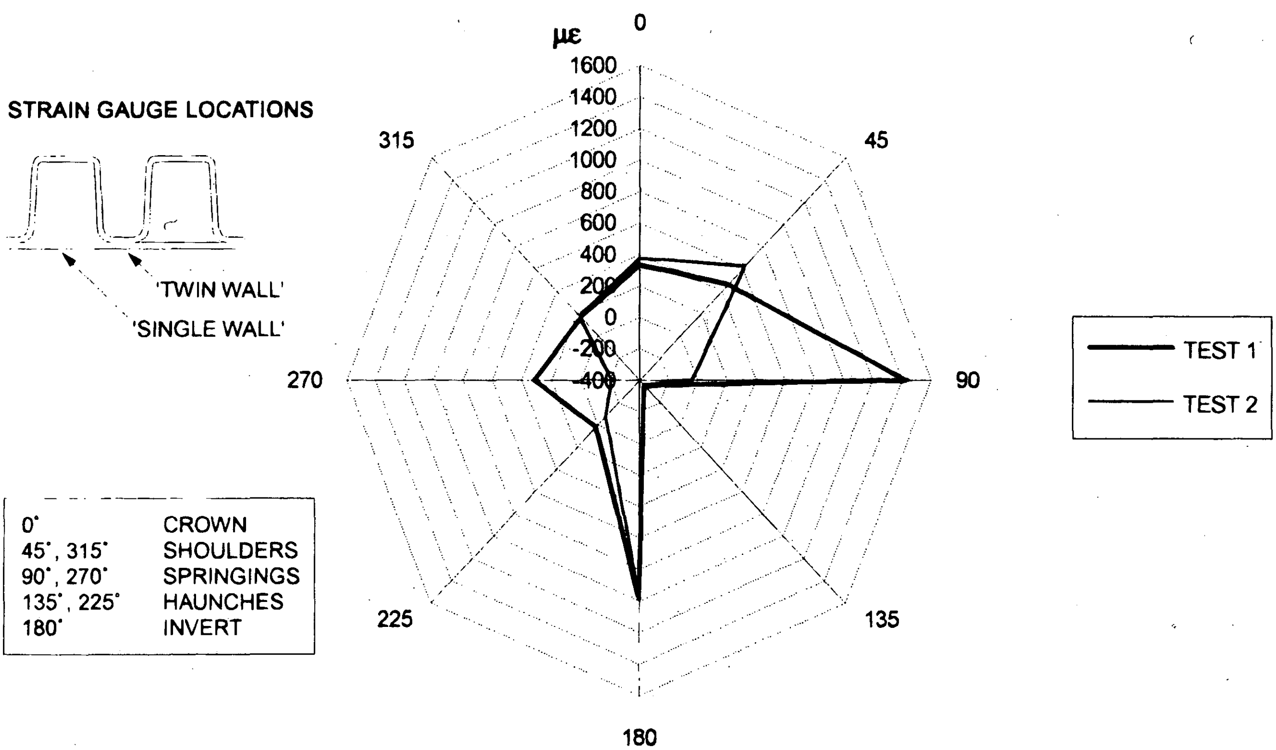

FIGURE 3 Twin wall strains caused by pipe installation.

to the other side. This demonstrates the importance of filling trenches uniformly at both sides of the pipe simultaneously, which practice was used for Test 2. The expected pattern of broadly uniform small compressive strains is otherwise generally demonstrated with the exception that the haunches in both cases exhibit small or markedly tensile strains and the invert relatively large compression. This implies that the careful feeding of gravel beneath the haunches, as specified in good site practice guidance, has caused the haunches to provide the majority of the vertical support from the bedding. Indeed the installation might have caused the pipe invert to have been raised off the underlying bedding layer by a small amount, thus allowing curvature (exhibited by compression on the internal surface) to occur readily at this point.

The wall strain data caused by the loading sequences are presented hereafter as readings averaged about the vertical axis for clarity in description of behavior, although the readings were typically broadly similar about this axis. Readers should note that, for Figures 3 to 8, positive readings indicate compression, and negative readings tension. Strain gauges were placed at the pipe crown (0°), invert (180°), springings (90° and 270°), haunches (135° and 225°), and shoulders (45° and 315°). The dotted lines are lines of equal strain. The overall effect of all three load sequences (i.e., the strain data at the end of the test minus those strains caused by installation) is illustrated in Figure 4, which indicates remarkably similar trends for the two tests. Also shown for comparison are the results of constant rate of deformation parallel-plate tests, in which elliptical deformation was expected. Linearity was demonstrated for all gauges, both during loading and unloading in the plate test, thereby demonstrating that the gauges were working properly. The results indicate tension at the crown and invert (indicating flattening of the pipe wall), compression at the springings (indicating an increase in curvature) and very small compressive strains at the haunches and shoulders. This wall strain distribution indicates elliptical deformation, and thus confirms the link between deformed shape and wall strain (*12*). The expected strain pattern for the entire loading sequence, given in Figure 4, of tensile strains at the crown, high compressive strains at the shoulders and lower compressive strains at the springings (90° and 270°) is a classic demonstration of what was termed "heart-shaped" deformation by Rogers (*12*). This illustrates most action in resisting applied stresses occurring in the upper half of the pipe, with the crown tending to flatten and the shoulders tending to bulge. Although these descriptions sound dramatic, they are not in reality since the deformations are remarkably low (0.5 and 0.65 percent VDS caused by loads) and indeed are indicative of remarkably good pipe performance. The effect of haunch support and invert curvature noticed above is reiterated and causes a pattern of higher strains (although still low in absolute terms) that would otherwise be unexpected if the pipe received uniform support throughout its lower half. Relatively low compressive strains would be expected below the horizontal axis.

The effect of the 70-kPa static load sequence is illustrated in Figure 5. Again the pattern for the two tests is similar, with Test 1 exhibiting a greater degree of hoop compression than Test 2. The heart-shaped deformation pattern modified by haunch support is clearly initiated by this loading. The effect of the cyclic load sequence (Figure 6), in contrast, is much less severe with typically smaller strains being more uniformly distributed. Virtually no change in curvature occurred at the springings (90° and 270°) in Test 1, with flattening at the crown and invert and compression at the shoulders. This pattern lies between that of heart-shaped deformation and elliptical deformation, a tendency to elliptical deformation being expected (*12*). The data for Test 2 are more extreme with a much greater flattening at the crown and much more pronounced heart-shaped deformation. The data for the 140-kPa static load sequence (Figure 7) show relatively small additional strains

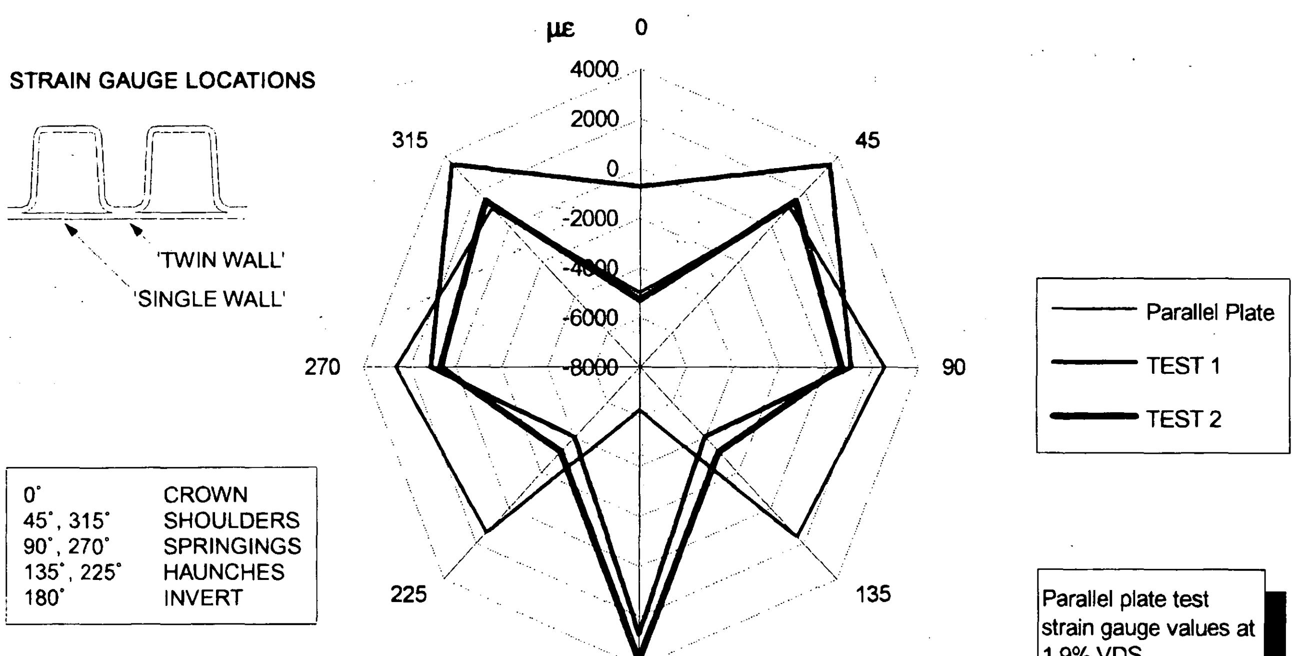

FIGURE 4 Twin wall strains caused by the complete stress sequence (end of test minus installation) and parallel plate testing.

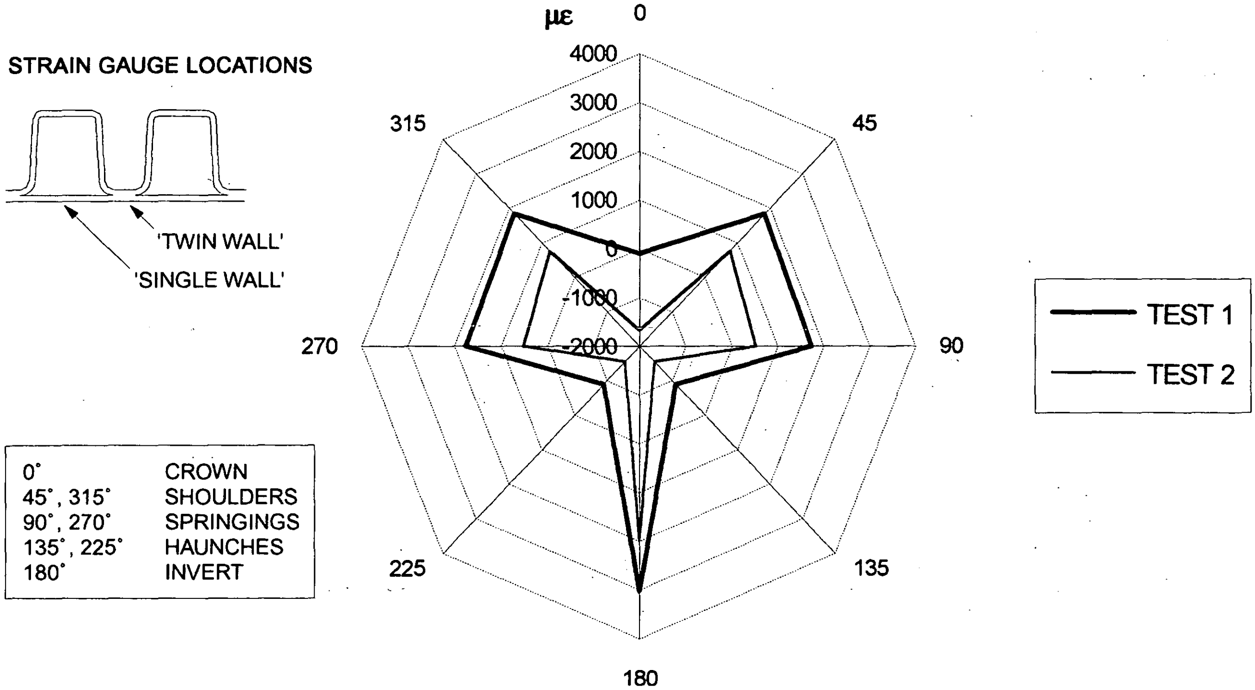

FIGURE 5 Twin wall strains caused by the 70-kPa static stress sequence (end of 70 S minus installation).

that conform broadly to the pattern of the 70-kPa static load sequence, although definitive conclusions are difficult to draw because of their small magnitude in relation to the previous two load sequences.

The wall strain data for the single wall (i.e., beneath the ridge) were in all cases less extreme and exhibited a greater degree of hoop compression than those for the twin wall sections. The effect of the complete load sequence on the single wall is shown in Figure 8, which should be compared with Figure 4. It is apparent that the strain profile here is more uniform for both tests although the fundamental pattern exhibited by the twin wall section is broadly followed. The tendency toward hoop compression is best exhibited at

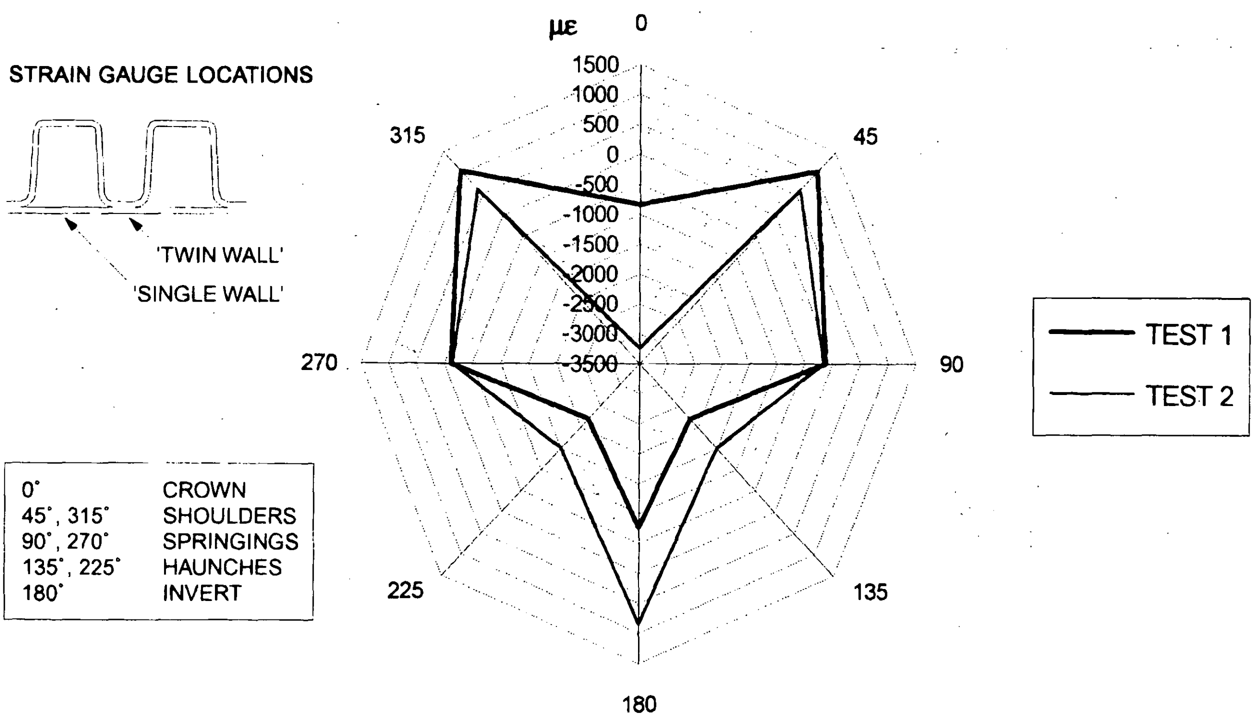

FIGURE 6 Twin wall strains caused by the 70-kPa cyclic stress sequence (end of 70 C minus end of 70 S).

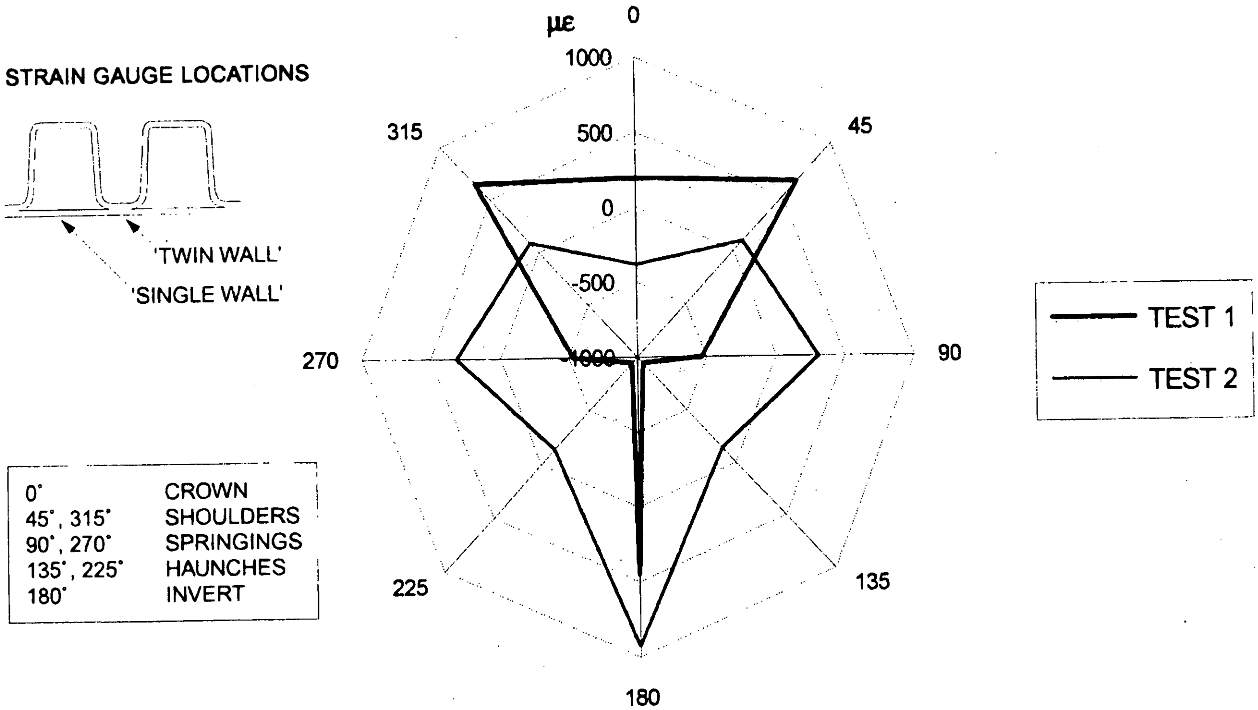

FIGURE 7 Twin wall strains caused by the 140-kPa static stress sequence (end of 140 S minus end of 70 C).

the crown in Test 1. These observations indicate that the ridges and valleys provide a large proportion of the resistance to external loading and that the single wall beneath the ridges is structurally less important.

In well compacted sand, the benefit of good, uniform support has been found to result in a virtually uniform (compressive) strain distribution at all points of the pipe. This implies hydrostatic stress distribution. Even at high loads, the strains were small (typically less than 400 με), indicating the high compressive stiffness of the pipe ring. Circumferential shortening can be approximated from strain gauge data, and for the 375-mm diameter pipe was found not to exceed 0.25 mm under the 140-kPa static stress.

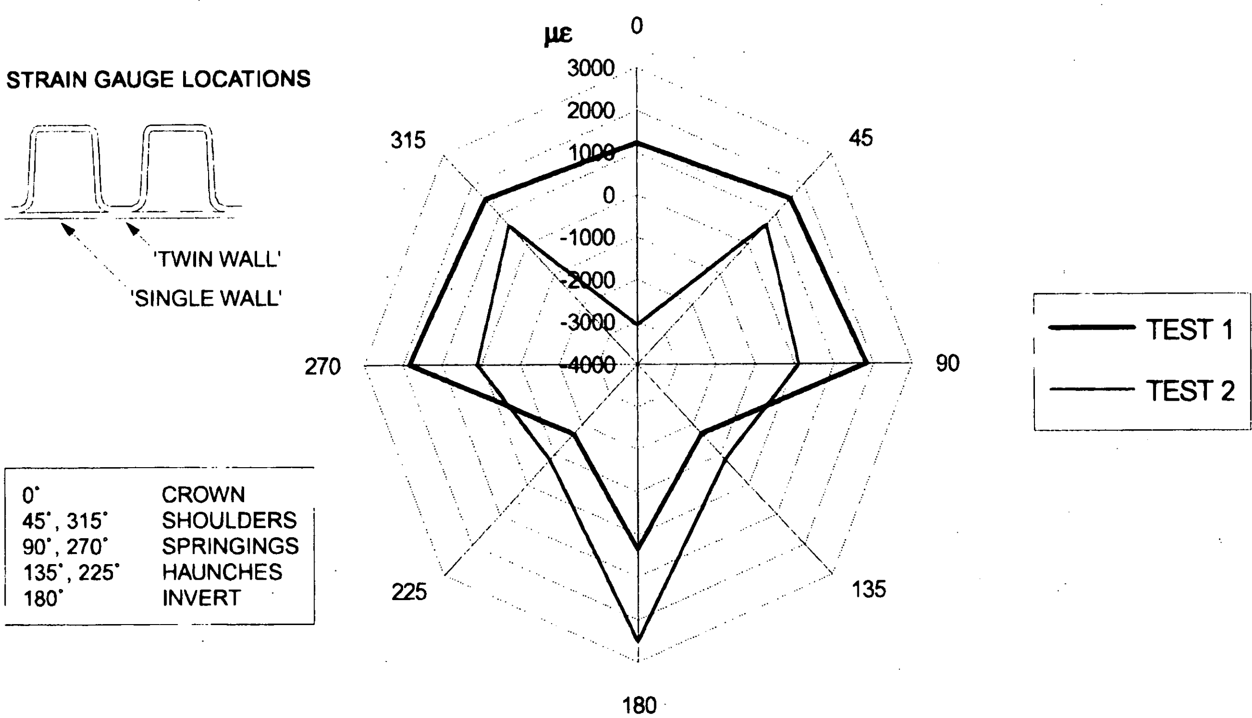

FIGURE 8 Single wall strains caused by the complete stress sequence (end of test minus installation).

Discussion of the Test Results in Relation to Other Work

The results support previous findings in the United Kingdom and elsewhere. Magnitudes of plastic pipe deformation are widely reported in the literature [for example, Rogers (*11*) and Gehrels and Elzink (*14,15*)], and it is now firmly established that remarkably good performance can be achieved with plastic pipes when buried with care, in a wide variety of pipe surround materials. In addition, they confirm the results of the work carried out by Rogers (*10,12*) that the shape of the deformed pipe is a function of the properties of the surround medium. Lightly compacted sand produced the largest deflections, as would be expected due to the inability of arching mechanisms to form in loose material and greater pipe deflection being required to mobilize equilibrium passive earth pressures. Pipes in gravel exhibit far less vertical diametral reduction and deform to a "heart" shape, because of the high degree of lateral support provided to all parts of the pipe circumference by this medium. The analyses of the strain profiles for all of the tests indicate that the greatest tensile strains always occurred at the pipe crown, whereas the distribution of strain around the circumference depended on the type of surround and the type of loading. Good support to the pipe typically resulted in deformation that deviated from an ellipse, most notably under static load. Cyclic loading appears to permit reorientation of the soil particles and cause deformations of a more elliptical nature to be superimposed on the deformed shape. The results of this work thus help in understanding how a flexible pipe resists applied loading in the field.

Implications of the Test Results

Trott and Stevens (*16*) concluded from their series of loading tests that the creep behavior of PVC-U pipes under sustained load is controlled by the properties of the pipe surround instead of those of the pipe. Gehrels and Elzink (*14*) state that the pipe class (i.e., pipe stiffness) is of minor importance in relation to the influence of bedding and backfill on the rate of deformation. These conclusions raise serious questions as to the validity of using 50-year pipe creep stiffness moduli for certification, specification and design purposes, and indicate that a soil creep stiffness is more relevant. Joekes and Elzink (*17*) propose the use of a 2-year stiffness value, based on a single logarithmic model that specifies a minimum value of correlation coefficient. They also reiterate earlier conclusions, stating that pipe stiffness has a minimal contribution to increasing deflection of the pipe after installation caused by settlement of the fill. Trafficking is said to result in the earlier establishment of equilibrium of the pipe-soil structure, adding that the equilibrium condition is reached within 2 years in virtually all cases studied and, under certain conditions, within a month. These observations are further supported by the various pipe design methods, which indicate that the pipe stiffness has only a very small influence on predicted deformation.

The recently published ISO 9967 (*18*) adopts the 2-year stiffness value and other minor improvements, yet, despite acknowledging a virtual cessation of deflection after a short period (and certainly within 2 years), persists in specifying pipe testing in isolation from the soil and applied loads based on the initial stiffness of the pipe. Although the authors acknowledge the need for a repeatable and relatively simple performance test for pipes, the use of a test that does not reflect performance in situ would appear to be poor engineering practice. It is worth noting in this context that pipes certified for use in sewer applications are subjected to a 10,000-hr (nearly 14 month) creep test. In this case simplicity does not equate to facility since the test clearly represents a major constraint on the pipe industry, in terms of development programming and costs, as modifications to the design of the pipe must await the results.

The deformations recorded at the end of all tests were less than 5 percent. Therefore it would appear that the long-term performance of the twin wall corrugated HDPE pipes tested will be excellent under most installation conditions, especially when considered in the context of the large magnitude of the stresses applied. These results are even more impressive when viewed in the context of Gehrels and Elzink's conclusion that diametral deflections of 10 percent to 20 percent have never been shown to cause problems for the proper function of pipelines (*15*). They add that the failure due to deflection is unknown with SDR 41 PVC-U pipes, pipes that are structurally inferior to twin wall HDPE pipes because of their higher SDRs.

CONCLUSIONS

It is concluded that current specification and design criteria used in the United Kingdom are conservative in light of laboratory and field data. The historically accepted limiting deflections of 5 percent of original diameter over the longer-term are still widely held in the United Kingdom, in spite of a large body of evidence indicating that this, too, is excessively conservative. More recent moves to relax this specification to 5 percent of the original diameter at the end of the construction period (*13*) indicate a better appreciation of the structural performance of such pipes. The U.K. Water Research Centre recommends a deformation limit of 6 percent 12 months after construction and accepts that the greatest degree of increase in deformation after installation will occur in the first 2 years (*19*). This demonstrates the incorporation by the water industry of research in their specifications, although drainage pipe specifications have remained unchanged. The test results additionally indicate that a wider range of soil surrounds could be used in practice, which would in turn reduce the costs of pipeline construction.

A major concern with current pipe testing methods has been found to be the fact that the creep stiffness test methods currently available do not address the fundamentals of pipe-soil interaction. Creep stiffness should not be considered in isolation as the installed behavior is more complex, and dependent to a major extent on the properties of the soil surround. Greater consideration must therefore be given to the formulation of representative, and repeatable, tests that take account of the behavior of the pipe-soil structure. Finite element methods would also lend themselves to this problem, and work in this field is being undertaken at many establishments.

It is thus apparent that advances in engineering of pipes and pipeline installation have the potential to reduce the cost of pipeline construction although maintaining the required levels of performance over the full design life of the pipeline. For this to happen, the current standards and specifications must be revised in the light of extensive research data to permit properly engineered solutions based on structural performance.

REFERENCES

1. *Specification for Highway Works*. Department of Transport, HMSO, London, 1991.
2. *Highway Advice Note HA40/89 Determination of Pipe and Bedding Combinations for Drainage Works*. Department of Transport (Highways and Traffic), HMSO, London, UK, 1990.
3. *Specification for Plastics Pipes and Fittings for Use as Subsoil Field Drains*. British Standards Institution, BS4962:1989.
4. Smith, G. N. *Buried Flexible Pipes 2: The Analytical Method Developed by Gumbel for the TRRL*. Contractor Report 229, TRRL, Crowthorne, Berkshire, UK, 1991.
5. Smith, G. N. *Buried Flexible Pipes 3: Applications of a New Design Method*. Contractor Report 230, TRRL, Crowthorne, Berkshire, UK, 1991.
6. *Standard Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading*. ASTM D2412-87, American Society for Testing and Materials, Philadelphia, 1987.
7. *Highway Construction Details*. Department of Transport, HMSO, London, UK, 1991.
8. Spangler, M. G. The Structural Design of Flexible Pipe Culverts. *Bulletin 153*, Engineering Experiment Station, Iowa State College, Iowa, 1941.
9. Howard, A. K. Modulus of Soil Reaction (E′) Values for Buried Flexible Pipes. *Journal of the Geotechnical Division, ASCE*, Vol. 103, 1977.
10. Rogers, C. D. F. *The Response of Buried uPVC Pipes to Surface Loading*. Ph.D. thesis, University of Nottingham, U.K., 1985.
11. Rogers, C. D. F., The Influence of Surrounding Soil on Flexible Pipe Performance. In *Transportation Research Record 1129*, TRB, National Research Council, Washington, D.C., 1987.
12. Rogers, C. D. F. Deformed Shape of Flexible Pipe Related to Surround Soil Stiffness. *Transportation Research Record 1191*, TRB, National Research Council, Washington, D.C., 1988.
13. *Plastics Pipework (Thermoplastics Materials) Part 6. Code of Practice for the Installation of Unplasticized PVC Pipework for Gravity Drains and Sewers*. British Standards Institution, BS5955,1980.
14. Gehrels, J. F., and W. J. Elzink. More Than 15 Years of Distortion Measurement Results with uPVC Pipes. *Proc., Europipe '82 Conference*, Basel, Switzerland, 1982.
15. Gehrels, J. F., and W. J. Elzink. Experience of Long-Term Behaviour of uPVC Gravity Sewers. *Proc., 4th International Plastics Pipes Conference*, University of Sussex, Brighton, U.K., 1979.
16. Trott, J. J., and J. B. Stevens. Loading Tests on uPVC pipes Installed in Cohesive and Non-Cohesive Bedding Materials. *Proc., 4th International Plastics Pipes Conference*, University of Sussex, Brighton, U.K., 1979.
17. Joekes, D., and W. J. Elzink. Deflection of PVC Sewer Pipes and a New Method for Measuring and Specifying Stiffness of Plastics Pipes. *Proc., 6th International Plastics Pipes Conference*, University of York, U.K., 1985.
18. *Thermoplastics Pipes—Determination of Creep Ratio*. ISO 9967, International Standards Organisation, 1994.
19. *ER201E Guide to the Water Industry for the Structural Design of Underground Non-Pressure UPVC Pipelines*. Water Research Centre Engineering, Swindon, Wilts, U.K., 1986.

Publication of this paper sponsored by Committee on Culverts and Hydraulic Structures.