The Influence of Surrounding Soil on Flexible Pipe Performance

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The response of 160-mm-diameter, shallowly buried, unplasticized polyvinylchloride (uPVC) pipes to surface loading has been investigated in full-scale tests. Standard installation and loading conditions were used to study the influence of the surrounding soil below, beside, and above the pipe on pipe performance. The type of soil used to surround the pipes was found to have a considerable influence on pipe behavior. Various uncompacted granular materials provided good support, whereas uncompacted silty clay and silty sand did not. Compaction of the surrounding soil had a variable influence wholly dependent on soil type. Silty clay performed well only when thoroughly compacted in thin layers. Light compaction of a broadly graded granular soil improved performance slightly, whereas uniform gravel is generally considered to be unresponsive to compaction. The use of a thin bedding layer was shown to be beneficial in reducing pipe deformation, whereas a thicker bedding layer was shown to be less beneficial. The problems of measuring pertinent soil properties on site for fill selection are discussed and assessment of two empirical methods is made.

The basic structural requirement of a buried pipe is the provision of an opening of dimensions and permanence suited to the function it is to perform. In general, it must maintain its size, shape, line, level, and integrity.

Two fundamental methods of meeting the structural, economic, and other requirements have developed. The first uses rigid pipes on a rigid bed such that the inherent strength of the pipes is sufficient to withstand applied loads by bending action in the pipe walls. Such pipes are subject to brittle fracture if the applied loads are too great or if significant differential settlement occurs.

The second method involves the use of flexible pipes that reach equilibrium in the ground by deforming under load. Such pipes rely on the properties of the surrounding materials to provide support and can fail by buckling, by excessive compressive hoop strain in surrounds providing relatively good support, or by excessive deformation. Failure of the pipe in these cases can be difficult to define if complete collapse does not result.

The division between rigid and flexible pipe behavior has become blurred with the advent of many new forms of pipeline construction. These include flexible pipes with a wide variety of strengths and stiffnesses, and flexible jointing systems for rigid pipes. The distinction between rigid and flexible pipes is not important, however, if the composite pipe-soil system is treated as the structure under consideration rather than as two distinct entities. In this way, loads are resisted by composite action according to the relevant stiffness.

From these ideas of relative flexibility, it can be seen that to design a pipeline an appreciation of both pipe and fill properties and their interaction is necessary. In the case of an inflexible pipe, design is primarily concerned with the structural properties of the pipe ring and the material on which the pipe is laid. Where the pipe has great flexibility, the structural properties of the material both beside and above the pipe also become important, particularly their ability to resist both vertical and horizontal pressures without large deformations. It is at this point that the relative costs of specific structural solutions must be considered. Because there is complex soil-structure interaction, the properties of the soil in this system and the cost of their provision must be carefully considered, particularly for more flexible pipes.

In 1842, John Roe, surveyor of the Holborn and Finsbury districts of London, put his mind to the design of sewer systems at the request of Edwin Chadwick (1800-1890), who was then secretary to the Poor Law Board set up in 1834. Roe's brief was to work on a series of experiments to ascertain for different discharges the most economical sizes of pipes and the best materials for their construction.

Chadwick wrote at that time:

As the old formulae, now in use, are founded on imperfect data and experiments, and not only give results so far above what experience shows to be the fact, but which, even if they were correct, would be of most limited application, they are obviously uncertain guides, and it is better to trust to our observations of what actually takes place. This is in fact experimenting on the largest and safest scale.

In reply to later questioning on the value of these experiments, Chadwick replied that drainage was a matter of gauging and experimenting that if carefully conducted would eventually remove all grounds for differences of opinion. Although the technology has progressed in many ways over the last 140 years, Chadwick's words still have a relevance and no more eloquent apology for the work presented hereafter could be devised.

The research at Nottingham has concerned full-scale tests on 160-mm-diameter, shallow-buried, unplasticized polyvinylchloride (uPVC) pipe under conditions simulative of building drainage. The work was instigated and sponsored by the British Plastics Federation. Such pipes are considered to

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be flexible and in qualitative terms the results can be extrapolated to other forms of flexible pipe construction. The results are presented in general terms to facilitate this extrapolation.

When discussing the results of the work, the terminology shown in Figure 1 is used. The diametral strain is defined as the change in the diametral measurement divided by the original diameter.

RESEARCH PROGRAM

Research Philosophy

The development of the test facilities at Nottingham has been described in detail by Rogers (1) and only scant details will be presented herein. The British Plastics Federation wished to investigate the performance of small-diameter uPVC pipes when buried in various soil surrounds and subjected to the most serious conditions that were likely to occur on a

building drainage site. Based on their experience of such sites, the pipes were buried in a 500-mm-wide trench at a cover depth of 500 mm below the underside of a granular site access road, which was assumed to be approximately 200 mm thick. Surface loads of 5.5 and 7.0 tonnes were applied both statically and cyclically to simulate the loads applied by the rear wheel arrangement of a construction lorry. The lower load was applied statically for 30 min, was removed for 45 min, and was then cycled 150 times at approximately 12 cycles/min, at the end of which pipe deformation was found to have stabilized. The installation was allowed to recover for 2 hr before the process was repeated with the higher load, the final recovery period being at least 18 hr.

The trenches were cut in Keuper Marl, a silty clay having a liquid limit of 32 percent and a plastic limit of 19 percent. The trenches were backfilled using the excavated silty clay placed in one layer and thoroughly compacted at the surface. In accordance with the wishes of the sponsors, no effort was made to vary the type of pipe used in the tests because the properties of uPVC pipe were not considered to vary



FIGURE 1 Terminology and definitions: (a) Pipe terminology, (b) Definition of VDS, and (c) Installation terminology.

Rogers

significantly between manufacturers. The pipes were obtained from a local supplier. The parameters that were of most importance to the investigation were the type and compaction of the sidefill and the depth of the bedding layer. These were varied between tests and are described in greater detail in a later section of this paper.

Equipment and Instrumentation

The first requirement was a facility of sufficient size to permit pipes to be tested free from boundary influences. The pit had a testing area 3 m long, 2.1 m wide, and 1.9 m deep and an inspection chamber at one end to allow access to the pipes during tests (Figure 2). The pit was filled with Keuper Marl at its natural water content of 17.5 percent. Load was applied to the surface of the backfill through a 700-mm-diameter semiflexible platen by a hydraulic ram housed in a loading rig. The platen was designed to represent the stress at the base of a granular subbase layer. The load regime was applied at two points 1 m apart in succession along the surface of each installation in order that duplicate data could be obtained from each installation. Load was measured independently by a load cell.

Further tests were performed in a reinforced box from

which comparative data were obtained under constant boundary influences. The box had internal dimensions 750 mm long, 500 mm wide, and 550 mm deep and pipes were buried with a cover depth of 250 mm (Figure 3). The same load regime was applied through a 480-mm-diameter rigid platen, a dead load being applied over the remaining area to simulate a total depth of cover of 500 mm. The loading was consequently considerably more severe in the box than in the pit.

The deformed shape of the pipe was recorded throughout each test using a ring flash camera developed at the Transport and Road Research Laboratory. The ring flash head was mounted on a boom and inserted into the pipe below the load platen. The head, which appears as a silhouette to provide a datum measurement, produced a thin band of light that was recorded photographically (Figure 4). Diametral change, shape of deformation, and settlement of the pipe invert were measured from the sequence of photographs. Vertical diametral change was also recorded by a linear potentiometer mounted on a self-righting sledge. Internal pipe wall strains were measured in the circumferential direction by eight equally spaced, single, active strain gauges glued directly to the wall of the pipes used in the box. Soil strain was measured at five strategic points in the trench cross section using inductance strain coils, and total pressures were measured at five points around the trench walls. Settlement of the load platen was measured using a linear potentiometer.





FIGURE 3 The test box.



FIGURE 4 A ring flash photograph.

Experimental Program

The pit installations provided the most important research data, the pipe surround configurations for which are summarized in Table 1 and will be described first.

The installation representing current site practice was tested first and was used as a standard against which the

TRANSPORTATION RESEARCH RECORD 1129

performance of other installations was measured. The pipe was laid on a 100-mm-thick bedding of uncompacted 10-mm pea gravel and surrounded by uncompacted pea gravel to the level of the pipe crown. This installation was repeated towards the end of the test program to provide replicate data.

The influence of the bedding layer was investigated by repeating the standard installation configuration with a 50mm-thick and no bedding layer. It was considered that hand trimming of the trench bottom, required where no bedding was laid, was likely to be impractical to specify and that a bedding layer of 50 mm would be necessary for leveling purposes. For this reason, only the more practical installation was repeated. The effect of removing a 50-mm-thick bedding layer from an uncompacted concrete ballast installation was also investigated.

Three levels of compaction were applied during the tests: thorough compaction by two passes of a pneumatic rammer, light compaction by treading, and no compaction. It has been shown by Gaunt et al. (2) in their tests on 300-mm-diameter uPVC pipe that a thoroughly compacted sandy clay sidefill performs comparably with uncompacted pea gravel, whereas an uncompacted sandy clay sidefill leads to large deformations. Performance in a poorer silty clay sidefill subjected to thorough compaction in one layer was thought to be a somewhat intermediate case and was measured in two installations. The effect of light compaction on concrete ballast was investigated.

Perhaps the most important series of tests concerned alternative imported pipe surround materials. The criteria for their choice were that they should be distinctly different in character, they should be widely available in Great Britain, and they should be relatively inexpensive. The materials chosen were concrete ballast (broad grading), washed quarry tailings, building sand (uniform), and reject sand (a silty sand). The grading curves for the coarse fractions of all the materials used in the tests are shown in Figure 5. Each of the materials was used as a 50-mm bedding layer and uncompacted sidefill, and each installation was repeated with the exception of reject sand, which was a marginal material.

The test box was used to gain comparative results from, and hence to indicate likely behavior in the pit of, several of the above installation types. Box installations were generally tested in advance of those in the pit. In addition to these tests, a further avenue of investigation was followed. A series of installations were tested in which good (pea gravel) and poor (silty clay) materials were juxtaposed around the pipe. This was done to isolate the areas in which good support to the pipe was most effective at reducing deformation. The configurations of all box installations are given in Table 2.

Associated soil testing, both in laboratory equipment and in situ, was performed concurrently with the experimental work, with several aims in mind. It was important to gain a good basic understanding of the soil properties in order to plan the test program. Soil properties were also necessary to prime a theoretical model, which was developed using a finite element program based on a linearly elastic soil model. A separate aim of the work was to investigate the means by which the relevant properties of soil that reduce deformation could be measured quickly and simply on site. The current method of soil selection in Great Britain refers only to the

Reference	Bedding Type	Bedding Thickness (mm)	Sidefill Type	Sidefill Compaction	Comments	
1A	Pea gravel	100	Pea gravel	none	standard practice	
2	none	0	Pea gravel	none		
ЗA	none	0	Silty clay	thorough	insitu soil as sidefill	
3B	none	0	Silty clay	thorough	insitu soil as sidefill	
4	none	0	Concrete ballast	light		
5	none	0	Concrete ballast	none	clay refurbishment	
6A	Pea gravel	50	Pea gravel	none	TUTTOWED LESC NO. 5	
68	Pea gravel	50	Pea gravel	none		
7A	Concrete ballast	50	Concrete ballast	none		
7B	Concrete ballast	50	Concrete ballast	none		
88	Building sand	50	Building sand	none		
8B	Building sand	50	Building sand	none		
18	Pea gravel	100	Pea gravel	none	standard practice	
9A	Quarry tailings	50	Quarry tailings	none		
9B	Quarry tailings	50	Quarry tailings	none		
10	Reject sand	50	Reject sand	none	marginal material	

 TABLE 1
 DETAILS OF EXPERIMENTAL INSTALLATIONS IN THE PIT



FIGURE 5 Particle size distribution curves.

Reference	Bedding Type	Bedding Thickness (mm)	Sidefill Type	Sidefill Compaction	Comments
1	Pea gravel	100	Pea gravel	none	standard practice
2	Pea gravel	50	Pea gravel	none	
3	none	0	Pea gravel	none	
4	Pea gravel	50	Pea gravel to springings, silty clay above	none none	support to mid-height of pipe only
5	none	0	Pea gravel to springings, silty clay above	none thorough	split sidefill
6	none	0	Silty clay to springings, pea gravel above	thorough none	split sidefill
7	none	0	Silty clay to crown, pea gravel 50mm above	thorough none	arching layer over 9
8A and 8B	none	0	Silty clay to springings, pea gravel to 50mm above pipe crown	thorough none	arching layer over 6; 8A had low water content, 8B high
9	none	0	Silty clay	thorough	
10	none	0	Silty clay	thorough	compacted in 2 layers
11	none	0	Silty clay	light	
12	Pea gravel	100	Silty clay	thorough	
13	none	0	Concrete ballast	light	
14	none	0	Concrete ballast	none	
15	none	0	Reject sand	light	
16	none	0	Reject sand	none	

TABLE 2 DETAILS OF EXPERIMENTAL INSTALLATIONS IN THE BOX

compaction properties of the material. The measurement of soil stiffness, or compressibility, has presented problems ever since Spangler (3) presented his classic work on deformation prediction and much effort has been expended to this end, notably by Watkins and Spangler (4), Watkins and Nielson (5), Bhandhausavee (6), Jaaskelainen (7), and Howard (8). While no immediate solution to the problem was expected from the work, some thoughts on the subject are presented later in the results section.

EXPERIMENTAL RESULTS

General Observations

Preliminary tests were conducted to provide information on loading rates and duration, recovery periods, and boundary conditions. These tests showed that for installations subjected to large static surface loads for several days there was no apparent increase in the magnitude of pipe deformation under load, but that permanent deformation on removal of the load was significantly increased. It was also discovered that when a long recovery period of 24 hr or more was introduced during the cyclic load sequences, elastic recovery of the pipe was greater and was not wholly removed on the immediate application of the next cyclic load. Thus, accelerated cyclic loading is significantly worse than that applied at the more normal rates experienced on site; extended static loads likewise provide a more severe case.

The internal pipe wall strain measurements were found to

correlate well with the deformed shape of the pipes tested in the box. The shape of deformation varied significantly between tests, most deformation occurring above the springings of pipes buried in competent sidefills, whereas approximately elliptical deformation occurred in sidefills offering poor support.

When the results were assessed in terms of pipe deformation, an arbitrary load regime was necessarily adopted. Any extension of the results must consequently be based on engineering judgment and local experience of site conditions. Values of vertical diametral strain (VDS) quoted herein might differ slightly in certain cases from those published previously by Rogers et al. (9). This variation simply reflects better definition of deformation patterns from more detailed measurements, thereby illustrating inaccurate results.

Pipe deformation data use as the datum either that at the end of installation, representing deformation caused by the applied loads alone, or that when positioning the pipe. The installation process was found to produce somewhat variable pipe deformation, even in pipes with the same soil configuration, whereas deformation caused by the applied loads alone was remarkably consistent between tests.

The Influence of Soil Surround Type

The influence of sidefill type on the VDS of the pipe is demonstrated by the results of installations in which five uncompacted granular materials were used as sidefill and bedding, a summary of which is shown in Figure 6. In this

Rogers

figure, VDS values at various stages of the tests are plotted using the point at which the pipe was positioned (PP) as the datum. Values thereafter are given at the end of installation (EOI), on application of the 55- and 70-kN loads (55 ON and 70 ON), and after they had been applied for 30 min (55/30 and 70/30). Values are also given after recovery at the end of the static load sequences (EO 55, and EO 70,) and the cyclic load sequences (EO 55, and EO 70,). The average curves for uncompacted pea gravel (UCPG) and concrete ballast (UCCB) are of similar type and magnitude throughout, with the cyclic loads having far more effect on the pipe-soil structure than the static loads. The average curve for washed quarry tailings (UCQT) exhibits greater strains at each stage, with a progressively increasing difference between it and the two mentioned, and in particular a greater susceptibility to creep under load. The average curve for the pipes buried in building sand (UCBS) shows higher strains still, with proportionally greater strain under static load, much of which was subsequently recovered. The average curve for uncompacted reject sand (UCRS), which was used in one installation only, exhibits strains of roughly twice those of the pipe in building sand. Installation deformation became progressively greater with poorer material, and it was apparent that creep effects under load were greater in materials having a broader grading or flatter grading curve.

The influence of sidefill type was further investigated in three pit installations in which no bedding layer was used. The pipe buried in uncompacted concrete ballast deformed least, with most deformation occurring under cyclic load. The pipe buried in uncompacted pea gravel was similarly more affected by the cyclic loads, but demonstrated a greater elastic response under static load. The pipe buried in well-compacted silty clay produced a deformation at the end of the test that was only slightly greater than the pea gravel installation, the pipe having a negative deformation at the end of installation caused by compaction of the soil beside the pipe. The permanent VDS caused by the static loads was relatively more significant to the overall value, with the pipe creeping considerably under load.

A further study of the influence of sidefill was conducted in the box in which the performance of three uncompacted granular materials was compared with that of silty clay thoroughly compacted in two layers (WCSC). A summary of the results is presented in Figure 7. In contrast to the pit results, the behavior of the pipes buried in the gravel and ballast was remarkably similar throughout the tests. The reason for this may lie in the inability of the pea gravel to bed in laterally in the box when loaded, whereas in the pit the gravel is found to penetrate the clay trench walls thereby allowing horizontal movement. Concrete ballast, being a broadly graded material, showed no tendency to bed in when used in the pit. Where the clay sidefill was used, a large negative VDS at the end of installation was followed by large positive deformation on application of the 55-kN load, and because of creep only a small part of this was recovered on removal of the load. Thereafter the installation behaved in a similar manner to the others, though with somewhat greater deformations. The curve for UCRS exhibited a large initial response to static load but little creep under load.

Several behavioral trends were apparent from this series of experiments. A pipe buried in an uncompacted sidefill that produced ultimately good performance tended to have a small positive VDS at the end of installation, relatively small deformations caused by static load application and creep effects, and a large elastic recovery on removal of the load. Cyclic load tended to produce greater permanent deformations than the static load. Where the sidefill was compacted, a negative VDS occurred at the end of the installation process. Where a cohesive material was used as sidefill, a large VDS tended to occur on application of the static load and this was increased considerably by creep movements. A relatively small proportion of this deformation recovered on removal



FIGURE 6 Vertical diametral strains (VDS) for five sidefill types in the pit.



FIGURE 7 Influence of sidefill type on pattern of VDS measurements in the box.

of the static load, cyclic load causing a relatively small permanent increase in VDS in comparison with the static load.

The Influence of Sidefill Compaction

Where light compaction was applied to a concrete ballast sidefill (LCCB) beside a pipe with no bedding, the VDS at the end of the test reduced considerably from that where no compaction was applied (Figure 8). The reduction was mainly due to markedly reduced installation deformation and a lesser response to the 55-kN loads. Additional experiments in the box showed that a pipe supported by an uncompacted silty clay sidefill deformed excessively before the full 55-kN load could be applied. Compaction improved the performance of this material to the extent that thorough compaction of a silty clay sidefill in two layers produced a negative deformation at the end of installation of 1.8 percent and a subsequent deformation under surface loads that compared favorably with the better granular materials (Figure 7).

It was clear from these results that compaction of the sidefill, even that effected by systematic treading, was of considerable use in reducing the VDS of the pipe and that different soils respond to compaction by differing amounts. In a clay sidefill, where the voids between lumps had to be removed, thorough compaction in thin layers was necessary before suitable lateral restraint was mobilized. Where concrete ballast, a broadly graded material, was used, light compaction made a significant difference to an otherwise good material, a result confirmed by experience in highway engineering. Pea gravel, however, has proved in practice to be largely unaffected by compaction because a favorable soil structure would be taken up when laid, compaction probably only affecting the bedding in effect at the interface with the surrounding soil.

The Influence of Bedding

The influence of the bedding layer on pipe performance was investigated in three pit installations using uncompacted pea gravel sidefills with 0, 50-, and 100-mm-thick bedding layers (Figure 9). The pipes laid on the 50-mm-thick bedding produced the lowest VDS, with little response to static load. The pipes laid on the thicker bedding produced slightly higher deformations throughout, with a VDS of 2.7 percent at the end of the test compared with 2.3 percent for the 50-mm layer. The pipes on the thicker bedding showed a greater response to static loads, both elastic and permanent. Where no bedding layer was used, a final value of 4.4 percent occurred by progressively greater deformation throughout the test. This ranking of installation types was confirmed in the box (Figure 10), though with curves of similar form throughout and a closer grouping of results showing that the differences were less exaggerated.

A further investigation concerned pipes laid in uncompacted concrete ballast sidefills both with and without a 50-mm-thick bedding. Where no bedding layer was used, installation deformation was 0.6 percent greater, the curves being separated by an offset of roughly this magnitude thereafter (Figure 8). The average VDS values at the end of the tests were 3.0 percent without bedding and 2.4 percent with bedding.

These limited results indicate that where good support is afforded the performance of the pipe with a 50-mm-thick bedding layer is slightly better than that with a 100-mm-thick bedding. Omission of the bedding layer altogether leads to a





FIGURE 8 Influence of bedding and compaction on VDS for concrete ballast sidefill.



FIGURE 9 Influence of bedding on VDS measurements in the pit.

worse case, albeit perhaps only from installation effects. In practice, where a 50-mm-thick bedding layer is necessary to level the trench bottom, its increase to 100 mm on the grounds of structural performance would seem to be inappropriate.

Soil Investigations

The current British Standard (10) installation recommendations include the compaction fraction test, a method of fill selection, which relates the uncompacted and fully compacted heights of soil in a 250-mm-long, 160-mm-diameter tube. The difference between the two heights divided by the original height is known as the compaction fraction of the soil; judgments are made based on this figure. While this test is wholly concerned with the susceptibility of the soil to compaction (which is recommended for sidefill and bedding in the British Standard), it provides a method of ranking the



FIGURE 10 Influence of bedding on VDS measurements in the box.



FIGURE 11 Influence of empirical soil factors on VDS measurements: (a) Compaction fraction, (b) Grading curve factor.

materials and a likely indication of their compressibility. With these considerations in mind, the ability of the compaction fraction to rank sidefill materials in order of performance was investigated. The average results of installations using five uncompacted granular materials as sidefill and 50-mmthick bedding were used for the comparison. The average compaction fractions were plotted against the VDS caused by the application of the 55-kN static load (55 ON) and the VDS at the end of the test (EOT), both related to the strains at the end of installation (EOI). These are presented in Figure 11(a), in which a clear relationship is apparent.

Both the grading and the particle size of the soil surround were found to influence the performance of the pipes. Asecond method of ranking the soils based on the grading curves of the soil (Figure 5) was sought. The product of the difference between the diameters below which 80 and 20 percent (D_{80} and D_{20}) of the particles lie (which reflects the gradient of the curves) and D_{50} (which reflects the absolute

Rogers

size) was used. Plotting this factor on a logarithmic scale against VDS produced smooth curves [Figure 11(b)], as before.

Although no serious conclusions should be drawn from these limited results, the findings perhaps add some ideas to the debate on soil selection.

CONCLUSIONS

The type of soil used to surround the pipe has most influence on its performance. Various uncompacted granular materials were shown to provide good support to the pipe. In order of performance, these included pea gravel, concrete ballast, washed quarry tailings, and building sand. Relatively large deformations were obtained in silty sand and silty clay. The better, granular surrounds were more affected by the cyclic loads than the static loads, whereas the pipes in poorer soils were influenced more by the static load sequences.

The benefit of sidefill compaction was wholly dependent on the soil type. Thorough compaction of a silty clay sidefill in thin (80-mm) layers greatly improved the support afforded by this soil, the performance being comparable to that of the better uncompacted granular soils. Light compaction of a broadly graded soil (concrete ballast) produced a significant improvement in pipe performance, whereas a more uniform soil (pea gravel) has proved to be largely unaffected by compaction.

A bedding layer was found to be beneficial in reducing pipe deformation, although deformation increased as the bedding thickness increased from 50 to 100 mm. A thin bedding layer would appear to be the optimum design solution from these limited data.

Two empirical methods for ranking soils in order of performance, based on compactability and grading, produced encouraging results.

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