Structural Performance of Buried Polyvinyl Chloride Pipes Under Large Distributed Load

SHAD M. SARGAND, GLENN A. HAZEN, XUEGANG LIU, TERUHISA MASADA, AND JOHN O. HURD

Two polyvinyl chloride (PVC) pipes of nominal diameters 457 and 914 mm (18 and 36 in.) were fully instrumented and loaded to failure at the Ohio University load frame facility site as part of a major field testing program of plastic pipes. Before load testing, each pipe was backfilled with crushed limestone material meeting the Ohio Department of Transportation no. 310 subbase specifications. Height of backfill soil over crown was 610 mm (24 in.) for the 457-mm (18-in.) diameter pipe and 305 mm (12 in.) for the 914-mm (36-in.) diameter pipe. A specially designed rotational linear variable differential transformer (LVDT) deflection measurement system was inserted into the pipe to monitor the cross-sectional deformation. During the incremental stages of backfilling and load testing, readings from the strain gages and the LVDT system were recorded by computerized data acquisition units. Test data were analyzed to establish load-deflection curves, bending moment and thrust responses, and failure modes of the buried PVC pipes. In addition, analytical results from modified Iowa formula and elastic solutions of J. Q. Burns and R. M. Richard were compared with the experimental results to evaluate general applicability of these analytical methods. The findings of this study indicated that the elastic solutions are valuable for analyzing flexible pipes buried in a homogeneous soil with loaded boundary more than one pipe diameter away.

Increasing use of profile-wall, plastic pipes in the transportation facilities has led to numerous studies on performance of these plastic pipe products. Adams et al. (1) studied a 610-mm (24-in.) plastic pipe under high fill conditions and noted that the change in vertical diameter was in the 3 to 5 percent range, whereas variation in the horizontal diameter was only 0.5 percent. The vertical deflection was determined to result from compression instead of bending. Minimal fill conditions were examined by Katona (2). He suggests that a conservative allowable deflection criteria of 7.5 percent can be used to determine minimum fill height. Because all pipes depend on the mechanical properties of the soil, a minimum overburden of 305 mm (12 in.) is recommended. Culley (3) suggests that pipe stiffness be incorporated in design procedures for diameters over 610 mm (24 in.) and viscoelastic behavior be considered for depths of over 15.24 m (50 ft).

This study presents both experimental and analytical results for two profile-wall polyvinyl chloride (PVC) pipes. The reason that this report is focused on the PVC pipes is that it appears in recent years an increasing number of publications has been made by others on high density polyethylene (HDPE) pipes than on PVC pipes. The research program, initiated by Ohio University, was founded on multiple objectives. The first objective was to examine failure

S. M. Sargand, G. A. Hazen, X. Liu, and T. Masada, Civil Engineering Department, 145 Stocker Center, Ohio University, Athens, Ohio 45701-2979. J. O. Hurd, Ohio Department of Transportation, 25 South Front Street, Columbus, Ohio 43215.

modes of profile-wall design plastic pipes. The second aim was to establish design parameters based on true mechanical properties of backfill material and flexibility of pipe. The third objective was to develop design criteria and analytical procedures to determine soil and pipe performance. And, if necessary, recommendations may be made as to the material and design specifications for the various profile-wall plastic pipe types. The last objective was to develop laboratory test procedures to simulate actual field behavior for quality assurance testing of profile-wall plastic pipes.

TEST PIPES AND INSTRUMENTATION METHOD

The first pipe, PVC-1, was a profile-wall PVC pipe with a nominal diameter of 457 mm (18 in.). Structural design of this pipe combines a seamless uniform cross-section wall with perpendicular radial ribs. The second pipe, PVC-2, had a nominal diameter of 914 mm (36 in.). The wall profile was smooth (rib-free), consisting of hollow core I-beam design. Table 1 summarizes some of the basic properties of the pipes, and typical wall profile is graphically shown in Figure 1. Both of these pipes are being used as gravity sewer pipes in the United States.

Figure 1 illustrates a typical strain gage instrumentation plan applied to the pipes. Each pipe was instrumented with strain gages at two circumferential cross-sections along 4.268 m (14 ft) length. A total of 16 biaxial strain gages were placed at the primary section, located at the mid-length at eight equally spaced points (crown, shoulders, springlines, haunches, and invert) on both interior and exterior pipe wall surfaces. The secondary section, located 152 mm (6 in.) away from the primary section, received six biaxial strain gages. The purpose of instrumenting this additional section was to provide a backup strain gage data for the crown and springline regions. The strain gages installed on the exterior surface of PVC-1 and all surfaces of PVC-2 were type EA-50-125TQ-350, manufactured by Micro-Measurement. The resistance and gage factor of this gage were 350 ohms and 2.18, respectively. A different type of strain gages (KFG-5-120-D16-11L3M3S by Kyowa Electronic Instruments) was applied to the interior surface of PVC-1 because the space was limited and this gage came with presoldered 3-m long lead wires. The resistance and gage factor of this gage were 120.4 ohms and 2.13, respectively.

PIPE BACKFILLING PROCEDURE

In this study, the PVC test pipes were installed in the field according to the current Ohio Department of Transportation (DOT) stan-

TABLE 1 Basic Properties of Test Pipes

|	PVC-1	PVC-2
Average Inside Diameter	448 mm (17.656 inch)	896 mm (35.275 inch)
Average Outside Diameter	502 mm (19.766 inch)	952 mm (37.475 inch)
Wall Area	7.8 mm²/mm (0.3066 in.²/in.)	14.4 mm²/mm (0.5676 in.²/in.)
Moment of Inertia	464 mm⁴/mm (0.0283 in.⁴/in.)	17190 mm⁴/mm (0.1049 in.⁴/in.)
Minimum Pipe Stiffness	413.7 KN/m/m (60 lbf/in/in)	317.17 KN/m/m (46 lbf/in/in)
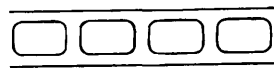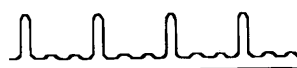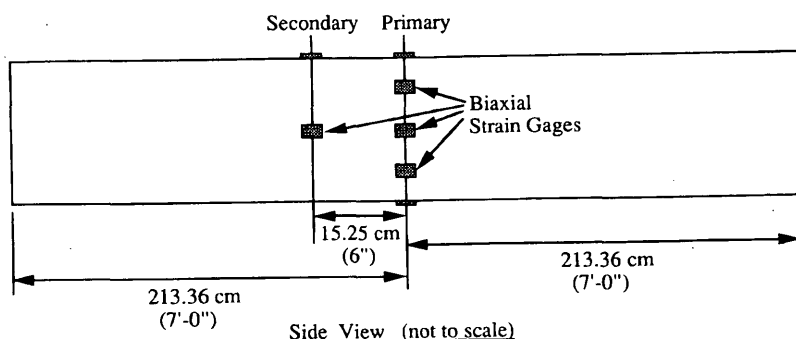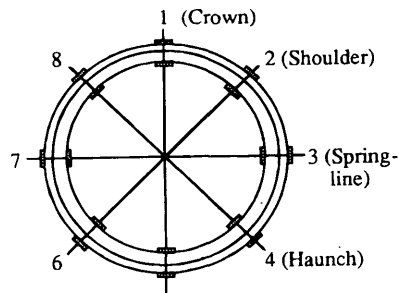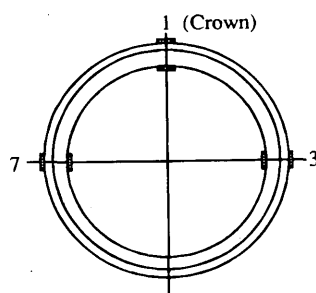
FIGURE 1   Typical wall profile and strain gage instrumentation plan for PVC-1 and PVC-2.

dard practices and subjected to loading incrementally by utilizing an unique load frame facility. This load frame facility, shown in Figure 2, was designed by the Center for Geotechnical and Environmental Research (CGER), Ohio University. It consists of two major components—two high capacity hydraulic cylinders and a structural frame anchored deeply into bedrock. The facility is some-what similar to the loading system often used when load tests are performed on pile foundations. Instead of selecting a dead weight, a total of eight tension rock anchors were installed to supply large uplifting capacity. Four concrete columns, spaced 7.622 m (25 ft) center-to-center, rest directly on top of a shaley bedrock. They function as a support for the structural I-beam frame. The two hydraulic
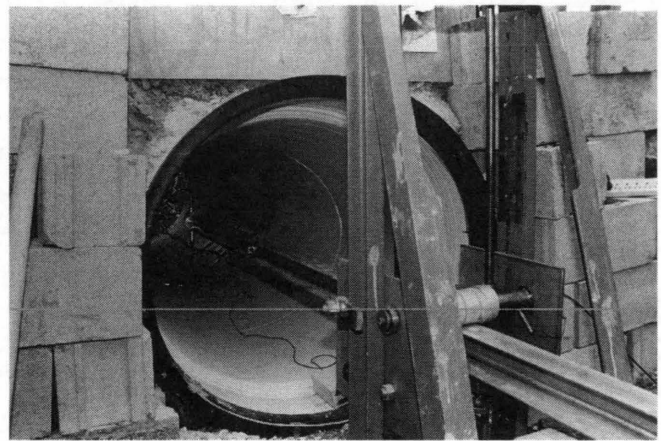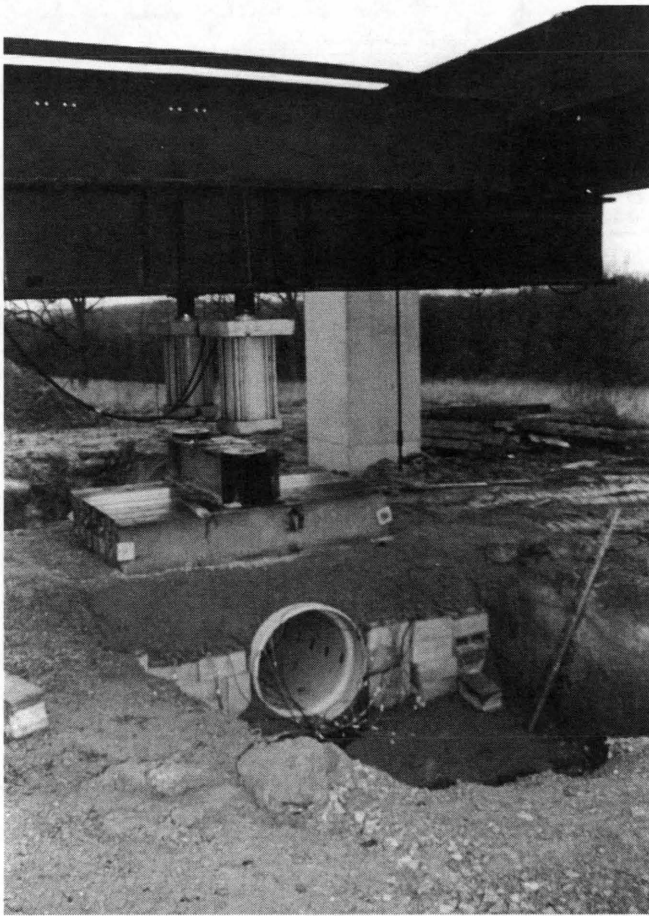
FIGURE 2   Load frame facility and LVDT deflection measurement system.

cylinders were connected to a hydraulic power supply/control unit for synchronized operation.

Installation procedures for the two pipes were basically the same. First, a trench of approximately 7.622 m (25 ft) length and 3.658 m (12 ft) width was excavated by backhoe under the load frame facility. The depth of the trench was extended down to the top of bedrock so that no soft soil layer might support the test pipe. A bedding layer of suitable thickness was formed by spreading and compacting in lifts crushed limestone material meeting the Ohio DOT no. 310 subbase material specifications. Actual bedding layer thickness was 508 mm (20 in.) for PVC-1 and 914.4 mm (36 in.) for PVC-2. Initial laboratory gradation test results (in terms of percent finer in mass) on the backfill soil, Table 2, were 100 percent on a 2.5-in. sieve, 100 percent on a 1.0-in. sieve, 25.5 percent on a no. 4 sieve, 5.5 percent on a no. 40 sieve, and 2.7 percent on a no. 200 sieve. The test pipe was carefully set over the bedding under the hydraulic cylinders. Then, the pipe backfilling work commenced by placing the Ohio DOT no. 310 material with a vibrating plate compactor. Lift thickness was usually 254 mm (10 in.) or less when compacted. It was controlled so that the top of lifts would coincide with the springline and crown elevations. The backfill height was extended 610 mm (24 in.) and 305 mm (12 in.) beyond the pipe crown for PVC-1 and PVC-2, respectively. This difference in the installation conditions was introduced to examine general influence of burial depth on the pipe performance and reliability of some of the

existing analytical methods. During backfilling, moisture content and dry density of each lift were recorded. The average moisture content and compaction rate were 5.1 percent and 90.8 percent for PVC-1 and 5.3 percent and 91.9 percent for PVC-2, respectively. Efforts were made manually to place the backfill soil densely in the haunch areas.

Once the backfilling was completed, a welded I-beam loading platform was positioned at the top of the pipe/soil system. This platform covered a contact surface area of 1.83 m (6 ft) width by 2.744 m (9 ft) length. The platform placement applied a surface pressure of about 6.89 kPa (1 lb/in.^{2}).

LOAD TEST PROCEDURES

Typical test set-up included a specially designed LVDT/stepper motor system and the biaxial strain gages all connected to computerized data acquisition units and soil pressure cells and a settlement profile-meter read by a readout box. Each pressure cell was embedded within a thick lens of fine sand to prevent bridging effect problem. Cross-sectional deformation of the pipe at the instrumented section was monitored with a specially designed LVDT driven by a stepper motor. This LVDT/stepper unit was secured on an aluminum I-beam section and suspended in air into the center of the pipe through a holding stand (see Figure 2). The advantage of this LVDT unit was that it provided deformation of the pipe at 72 or

TABLE 2   Gradation Data on Backfill Soil

U.S. Sieve No.	Sieve Size (mm)	Percent Finer in Mass (%)	
		Ohio D.O.T. Requirement	Typical Lab Data
2.5 inch	63	100	100
1.0 inch	25	70-100	100
4	4.75	25-100	25.5
40	0.425	5-50	5.5
200	0.075	0-10	2.7

more points along the pipe circumference within 30 sec. A settlement profile-meter (GEOKON Model 4651) probe was attached onto the loading platform to monitor settlement of the platform during the pipe loading test.

After backfilling and placement of the loading platform, a load test was initiated by pressurizing the two hydraulic cylinders. The hydraulic pressure was increased typically in 1379 kPa (200 lb/in.^{2}) increments. This translates to an increase of 54.47 kPa (7.9 lb/in.^{2}) per increment in the intensity of the surface pressure applied, based on the number of hydraulic cylinders (= 2) used, the bore size inside each cylinder (= 355.6 mm = 14 in. in diameter), and the surface area covered by the loading platform (= 1.83 m by 2.74 m = 6 ft by 9 ft). Duration of each load increment was 15 min, during which three sets of readings were taken simultaneously from all the sensors every 5 min.

TEST DATA ANALYSIS

Output voltage readings from the strain gages were transformed to corresponding level of strains. Then, the bending moment (M) and thrust (P) across the pipe wall were computed through the strain readings obtained from two biaxial gages located distances C_1 (positive) and C_2 (negative) away from the neutral axis of the pipe wall:

$$M = \frac{E}{1-\mu^2}\left(\frac{I}{C_1+C_2}\right)\left[(\varepsilon_{C_2}-\varepsilon_{C_1})+\mu(\varepsilon_{L_2}-\varepsilon_{L_1})\right] \qquad (1)$$

$$P = \frac{E}{1-\mu^2}\left(\frac{A}{C_1+C_2}\right)\left[C_2(\varepsilon_{C_1}+\mu\varepsilon_{L_1})+C_1(\varepsilon_{C_2}-\mu\varepsilon_{L_2})\right] \qquad (2)$$

where

E = Young's modulus of pipe material;
μ = Poisson's ratio of pipe material;
I = moment of inertia of pipe wall per unit length;
ε_C = circumferential strain in pipe wall;
ε_L = longitudinal strain in pipe wall;
A = cross-sectional area of pipe wall per unit length.

Here, sign conventions of Timoshenko are used consistently. In other words, bending moment is positive if its action tends to deflect the beam concave upward. Thrust is positive if it is in tension.

ANALYTICAL METHODS APPLIED

Analytical tools applied to analyze the two PVC pipes were modified Iowa formula and elastic solutions of Burns and Richard (4). Iowa formula has been used widely by engineers and researchers. The elastic solutions mentioned above have received little attention since publication in 1964. These solutions are available as Level 1 Solution in CANDE (Culvert ANalysis and DEsign)-89 finite element computer code.

The Iowa formula was originally established by Spangler (5) on the basis of elastic ring theory and fill-load hypothesis he reached after testing numerous flexible steel culverts. The formula was later modified by Watkins and Spangler (6) through a similitude study to the current form of:

$$\frac{\Delta \chi}{d}(\%) = \frac{100 \cdot D_L \cdot K \cdot P}{0.149(PS_o)+0.061(E')} \qquad (3)$$

TABLE 3   Summary of Backfill Data

	PVC-1			PVC-2		
	Thickness (mm)	Moisture Content (%)	Percent Compaction (%)	Thickness (mm)	Moisture Content (%)	Percent Compaction (%)
Bedding	457	5.3	75.0	229	6.0	86.4
Layer 1	229	4.5	100.0	254	5.0	96.1
Layer 2	229	5.0	93.8	254	4.5	94.5
Layer 3	203	5.3	92.2	254	4.2	87.0
Layer 4	203	5.3	92.2	254	4.6	89.0
Layer 5	203	4.9	91.4	305	7.2	98.4

where

Δ/d = horizontal deflection (percent);
D_L = deflection lag factor;
K = bedding constant;
P = pressure on pipe;
PS_o = pipe stiffness;
E' = modulus of soil reaction.

The above equation may be also applied to evaluate the vertical deflection on the assumption (Uni-Bell PVC Pipe Association) (7) that the pipe deforms equally in both horizontal and vertical directions.

More rigorous theoretical solutions were formulated by Burns and Richard (4) for an elastic circular ring deeply buried in an infinite elastic soil material. They applied extensional shell theories to the pipe and Mitchell's stress function to the elastic soil medium. First, they assumed that radial displacements are continuous at the pipe/soil interface. Under the full-slip condition, zero shear stress exists at the pipe/soil interface, whereas equality exists at the interface between tangential displacement of the pipe and tangential displacement of the soil medium under the no-slip conditions. Solutions for the two conditions are summarized as follows:

Full-Slip Condition

$$W = \frac{P_v R}{2M_S}\left[UF(1-a_0) - \frac{2}{3}VF(1+3a_2-4b_2)\cos 2\theta\right]$$

$$N = P_v R\left[B(1-a_0) + \frac{C}{3}(1+3a_2-4b_2)\cos 2\theta\right] \quad (4)$$

$$M = P_v R^2\left[\frac{CUF}{6VF}(1-a_0) + \frac{C}{3}(1+3a_2-4b_2)\cos 2\theta\right]$$

No-Slip Condition

$$W = \frac{P_v R}{2M_S}\left[UF(1-a_0) - VF(1-a_2-2b_2)\cos 2\theta\right]$$

$$N = P_v R\left[B(1-a_0) + C(1+a_2)\cos 2\theta\right] \quad (5)$$

$$M = P_v R^2\left[\frac{CUF}{6VF}(1-a_0) + \frac{C}{2}(1-a_2-2b_2)\cos 2\theta\right]$$

where

P_v = pressure applied at top and bottom boundaries;
R = radius of pipe;
M_s = confined soil modulus; and
θ = counterclockwise angle taken from right springline.

Definitions of the other terms, such as UF and VF, are listed below.

$$B = 0.5(1+K) = \frac{1}{2(1-v)}$$

$$C = 0.5(1-K) = \frac{1-2v}{2(1-v)}$$

$$UF = \frac{2BM_S R}{EA} = (1+K)\frac{M_S R}{EA} \quad (6)$$

$$VF = \frac{2CM_S R^3}{6EI} = (1-K)\frac{M_S R^3}{6EI}$$

$$a_0 = \frac{UF-1}{UF+(B/C)}$$

for both no-slip and full-slip

$$a_2 = \frac{C(1-UF)VF-(C/B)UF+2B}{(1+B)VF+C(VF+1/B)UF+2(1+C)}$$

for no-slip

$$a_2 = \frac{(2VF-1+1/B)}{(2VF-1+3/B)}$$

for full-slip

$$b_2 = \frac{(B+CUF)VF-2B}{(1+B)VF+C(VF+1/B)UF+2(1+C)}$$

for no-slip

$$b_2 = \frac{2VF-1}{2VF-1+3/B}$$

for full-slip

Hoeg (8) also developed his elastic solution for deflection (W) of a pipe placed in an infinite elastic media, along with performing some experiments in a test bin. He used a similar model, but he did not impose any specific ratio for the horizontal vertical loads. His solutions are useful only for uncorrugated pipes, since the moment of inertia is being computed based on the pipe wall thickness. Galili and Shmulevich (9) conducted an extensive experimental study of flexible to semirigid pipes in a laboratory soil-box and compared the results to the elastic solutions of Burns and Richard (4) and of Hoeg. Here, flexible pipes are defined with $S < 0.005$. Pipes with midrange stiffness are characterized with $0.005 < S < 0.5$. S is a pipe/soil stiffness ratio expressed in terms of $EI/(M_S r^3)$ where M_S is a confined one-dimensional modulus and r is the pipe radius. Their outcome indicated that the elastic solutions agree relatively well with the experimental for flexible to midrange stiffness pipes.

DISCUSSION OF RESULTS

The backfilling and load test was conducted on June 5 and 6, 1992, for PVC-1 and on April 29 and 30, 1992, for PVC-2. During backfilling the pipe diameter in the vertical direction increased by 0.2 percent for PVC-1 and by 0.11 percent for PVC-2. This indicates that the backfilling operation induced relatively insignificant structural responses in the pipes.

Upon application of increased loading, the pipes did not respond instantaneously; it generally took several minutes for the effects of the loading to be felt within the pipe/soil system. Once the load influence appeared, the pipe response stabilized quickly. This was due to the use of a stiff, granular backfill material. The test results presented in this section represent for each load increment the final stabilized pipe response. Figure 3, a and b, presents horizontal and vertical deflections measured during the load test for the two test pipes. A vertical deflection of 7.5 percent was reached under surface pressure of 245.46 kPa (35.6 lb/in.²) for PVC-1 and under surface pressure of 382 kPa (55.4 lb/in.²) for PVC-2. These surface pressure intensities correspond to at least 13.1 and 20.4 m (43 and 67 ft) of depth for PVC-1 and PVC-2, respectively, using an average backfill density of 1.92 g/cm³ (120 lb/ft³). These plots also indicate deflections computed from the modified Iowa formula and the elastic solutions of Burns and Richard (4). Parametric values used for the modified Iowa formula were:

PS_o = 413.7 kPa (60 lb/in.^{2}); E' = 20.68 MPa (3,000 lb/in.^{2});
   D_L = 1.0; K = 0.11 for PVC-1
PS_o = 317.17 kPa (46 lb/in.^{2}; E' = 20.68 MPa (3,000 lb/in.^{2});
   D_L = 1.0; K = 0.11 for PVC-2.

The E' value of 20.68 MPa (3,000 lb/in.^{2}) was selected according to the study by Howard (*10*) as a standard value for relatively well compacted crushed rock material. The K value of 0.11 was recommended by Moser (*11*) and Uni-Bell Pipe Association (*7*), which corresponded to a nearly unhaunched condition. For the elastic solutions, the following input values were utilized:

E = 2.758 GPa (400,000 lb/in.^{2}); u = 0.30 for PVC-1 and PVC-2
M_s = 13.79 MPa (2,000 lb/in.^{2}); u_s = 0.25;
K = 0.333 for backfill soil.

Based on the experimental plots of surface pressure vs. horizontal deflection, the initial E' value of the backfill soil may be back calculated to be about 6.205 MPa (900 lb/in.^{2}) for PVC-1 and about 16.62 MPa (2,410 lb/in.^{2}) for PVC-2. This indicates that the backfill soil was somewhat less stiff around PVC-1, contrary to the data

presented in Table 3. The backfill soil, which contained aggregates as large as 25 mm in diameter, could not be compacted effectively in the zone immediately next to PVC-1 due to the presence of narrowly spaced ribs. The field density tests for the backfill were typically performed about 0.6 m (2 ft) away from each test pipe. So, the backfill data presented in Table 3 may not be representing the actual conditions that existed immediately next to PVC-1. Magnitude of the vertical deflection remained at 1.2 to 1.3 times the horizontal deflection for PVC-1, whereas this deflection ratio decreased from 2.5 to 1.1 for PVC-2 as the surface pressure was increased. This contrasts with vertical to horizontal deflection ratio assumed to be 1.0 for the modified Iowa formula. The elastic solutions produced ratios of 1.28 for no-slip condition and 1.23 for full-slip condition between the vertical and horizontal deflections. Progressive stages of actual deformed shape of PVC-1 recorded during the load test is shown in Figure 4. The pipe deformed nearly symmetrically.

Review of bending moment and thrust around the pipe circumference can indicate the overall response of the pipes to the load for both pipes. The moment responses of the pipes were consistent with the basic principles of engineering mechanics. Positive moment was measured at the crown and the invert, whereas the springline regions experienced negative moment. The magnitudes of moment were more pronounced for PVC-2. Thrust remained entirely compressive in the wall of both pipes, except for the haunch regions where increasing surface pressure induced extensional thrust of small magnitudes. Comparisons between experimental results and elastic solutions for bending moment and thrust at crown and springlines are made in Figures 5 through 12. For PVC-1 the bending moments were relatively close, both at the crown and at the springlines. For PVC-2, agreement between the theory (full-slip) and the experimental was good at the crown. But, at the springlines the theoretical moments were much less than the experimental. General agreement for thrust was somewhat less satisfactory between the experimental and the theoretical. Overall, these plots indicate that for pipe deflections and bending moment level of agreement between the experimental values and elastic solutions was better for PVC-1, the pipe under deeper soil cover. And, between the no-slip and full-slip solutions, the latter predicted values closer to the experimental results. Hoeg stated that the boundary conditions assumed in his elastic analysis are not very severe if the loaded boundaries are at last one pipe diameter away from the pipe wall. The ratio of the soil cover over the nominal pipe diameter was 1.333 for PVC-1 and 0.333 for PVC-2. This may explain the less satisfactory agreement observed for PVC-2 between the experimental and theoretical results. Also, the interface friction angle between the smooth PVC pipe surface and the backfill soil should be relatively small.

During the load test PVC-1 continued to deflect without developing wall crushing or buckling. This pipe reversed its curvature slightly in the crown region during the last load increment (surface pressure 546 kPa = 79.2 lb/in.^{2}, equivalent fill height 29 m = 95 ft or more, vertical deflection 17 percent). Upon removal of the pipe after the test, close examination revealed no physical damage to the pipe wall and ribs. On the contrary, PVC-2 experienced structural failure under the surface pressure of 436.45 kPa (63.3 lb/in.^{2}) through seam failure and longitudinal cracking which was concentrated in the springline regions. This surface pressure is equivalent to more than 23.2 m (76 ft) of cover over the pipe. The pipes are shown in Figure 13.
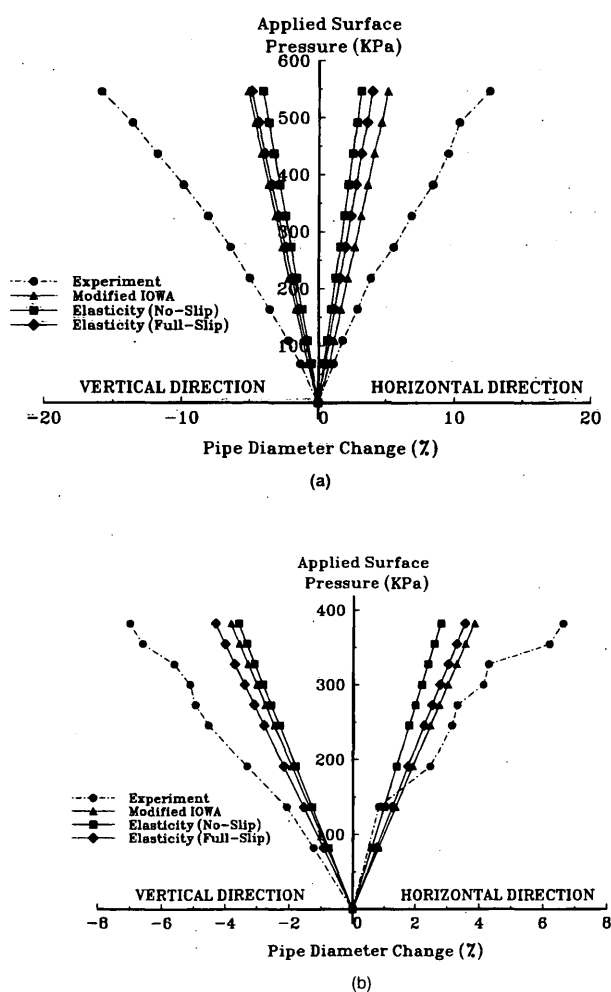

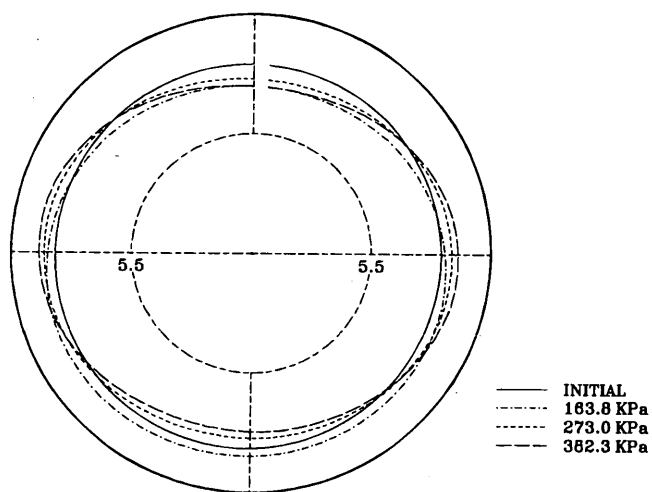**FIGURE 3 Deflection of PVC-1 (*a*) and PVC-2 (*b*) pipe during field load test.**

FIGURE 4   Deflected shape of PVC-1 during field load test.

FIGURE 7   Comparison between experimental and theoretical thrust at crown section for PVC-1 pipe.
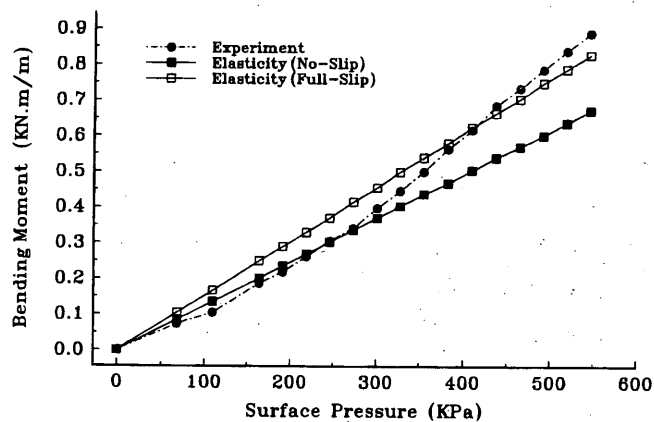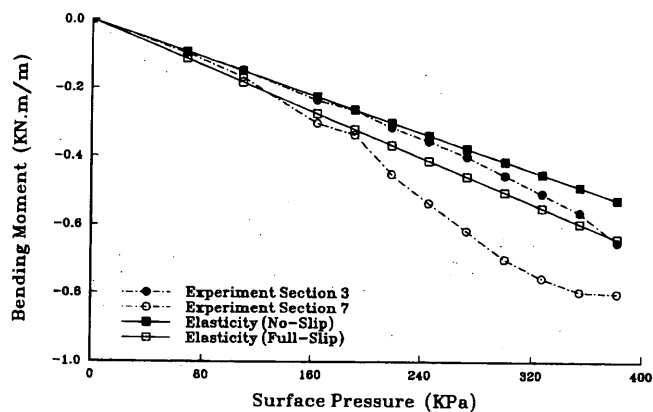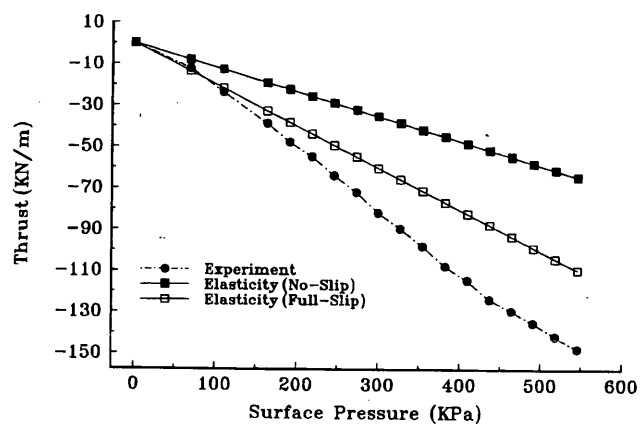

FIGURE 5   Comparison between experimental and theoretical bending moment at crown section for PVC-1 pipe.
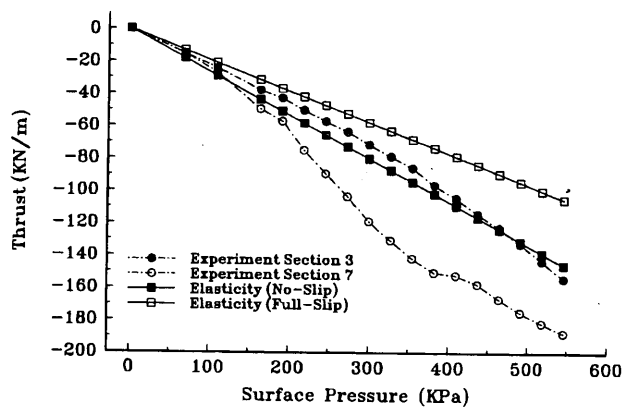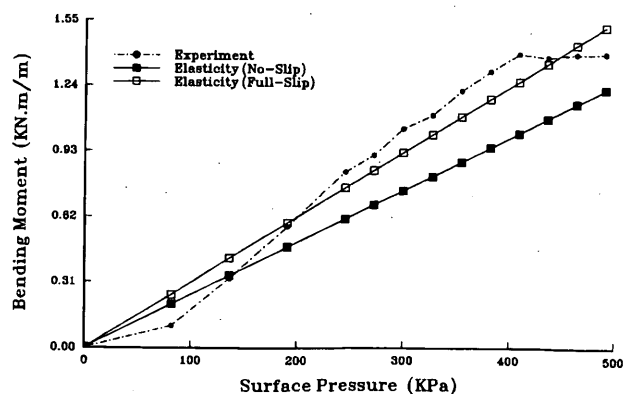

FIGURE 8   Comparison between experimental and theoretical thrust at springline section for PVC-1 pipe.

FIGURE 6   Comparison between experimental and theoretical bending moment at springline section for PVC-1 pipe.

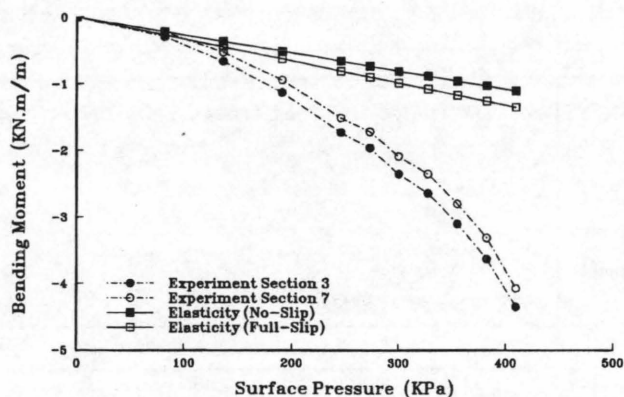FIGURE 9   Comparison between experimental and theoretical bending moment at crown section for PVC-2 pipe.

FIGURE 10 Comparison between experimental and theoretical bending moment at springline section for PVC-2 pipe.
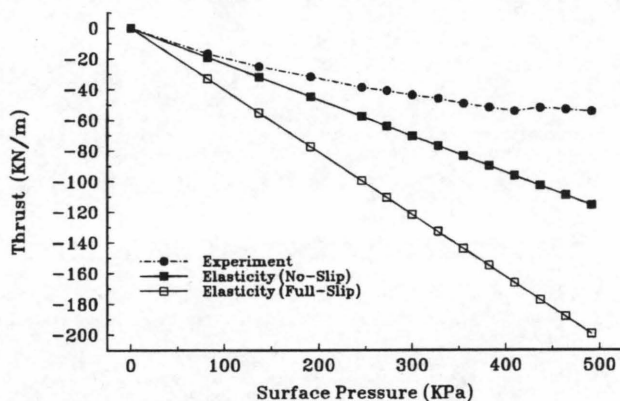

FIGURE 11 Comparison between experimental and theoretical thrust at crown section for PVC-2 pipe.
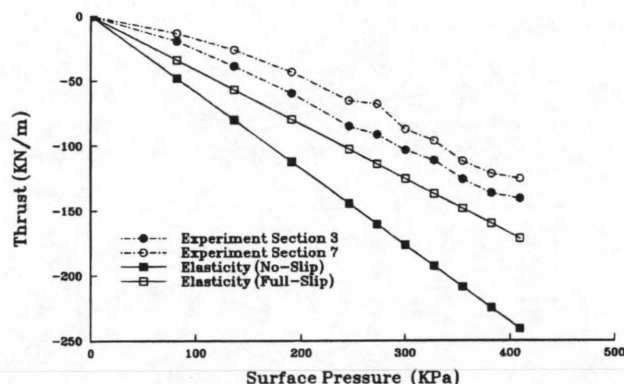

FIGURE 12 Comparison between experimental and theoretical thrust at springline section for PVC-2 pipe.

CONCLUSIONS

Two PVC pipes of diameter 457 mm and 914 mm (18 in. and 36 in.) were instrumented, buried in a stiff backfill soil, and subjected to large loads. An unique, large-scale, load frame facility was utilized to conduct the pipe tests. The loading conditions

simulated in this study reflected possibly the worst loading conditions that these pipes can experience during service. These pipes are rarely installed under shallow cover conditions and are seldom subjected to extremely large surface loading. Extremely large surface loading applied only a few feet above the pipe does not allow any development of significant level of soil arching. The profile design differed significantly between the two pipes. The 457-mm (18-in.) diameter pipe (PVC-1) was seamless, solid-walled, and ribbed outside. The 914-mm (36-in.) diameter pipe (PVC-2) had seams running spiral along the length, with its wall section hollowed in the core. The final soil cover was 610 mm (24 in.) for PVC-1 and 305 mm (12 in.) for PVC-2. Both pipes performed satisfactorily during the backfilling, and the field load tests demonstrated that they could both carry substantial loading. Under applied surface pressure, PVC-1 continued to deform without showing any structural distress. Very slight reversal of curvature was observed in the crown region at 17 percent change in vertical diameter. PVC-2 deformed less under the same load. However, its load carrying capacity was limited by seam strength. Deflections predicted by the modified Iowa formula were slightly larger than the deflections obtained from the full-slip elastic solutions of Burns and Richard (*4*). For both pipes, the experimental deflections were larger than those resulted from both analytical methods. The elastic solutions (full-slip) agreed reasonably well
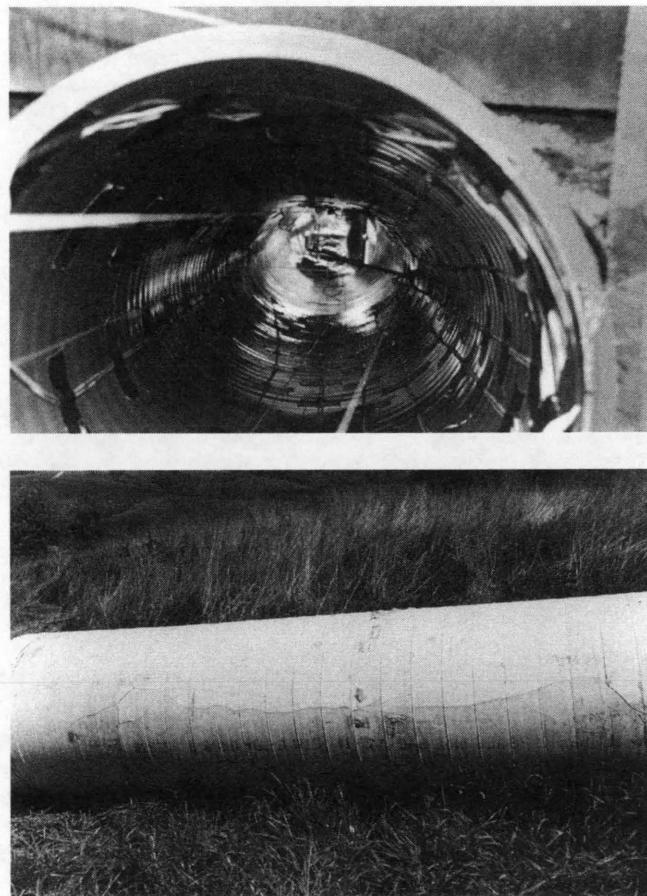

FIGURE 13 Test pipes during and after load test: (*top*) cross-section, PVC-1 (under surface pressure of 546 kPa); (*bottom*) side view, PVC-2 after load test.

with the experimental bending moment results for PVC-1, the pipe under more than one pipe diameter thick soil cover. These findings indicate that the elastic solutions are valuable for analyzing flexible circular pipes buried in a homogeneous soil with loaded boundary more than one diameter away.

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