COMPARISON BETWEEN FIELD AND ANALYTICAL RESULTS ON THE STRUCTURAL PERFORMANCE OF DEEPLY BURIED 42&60-INCH DIAMETER HIGH DENSITY POLYETHYLENE PIPES

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of the Requirement for the Degree
Master of Science

by
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CHAPTER 1
INTRODUCTION

1.1 INTRODUCTION

Large diameter thermoplastic pipes have been utilized increasingly to convey drainage under roadways in place of conventional masonry short-span bridges and conventional pipe materials such as reinforced concrete and corrugated metal. For example according to Goddard [1], the New York State Department of Transportation reported in 1996 that pipe usage was about 6% for aluminum, 20% for steel, 35% for concrete, and 39% for thermoplastic. This trend is due to constant improvements being made in manufacturing technologies and advantages of plastic pipes in terms of lightweight, cost efficiency, and long-term chemical stability. Although there have been numerous papers and reports published on the plastic pipes, a lack of comprehensive field performance data still exists. It is widely known that the plastic pipes are flexible structures and derive their strength and stability through interaction with surrounding soil media. Most of theories applicable to the buried plastic pipes were established before 1970s, based on performance of flexible metal pipes under relatively high fills in the field or in laboratory test conditions.

The structural performance of the thermoplastic pipes has been investigated by the Ohio Research Institute for Transportation and Environment (ORITE). In 1999 (ORITE) launched the thermoplastic pipe deep burial project at a site next to the Ohio
University airport in Albany, Ohio. A total of eighteen thermoplastic (12 HDPE, 6 PVC) pipes, ranging in nominal diameter from 30 to 60 inches, were instrumented with various sensors and buried under either a 20-ft. or 40-ft. embankment fill. Installation conditions for the test pipes were purposefully varied to examine the effects of fill height, bedding thickness, backfill material type, and relative compaction of the backfill on their structural performance. Typical instrumentation applied to these test pipes included two types (electric, fiber-optic) of strain gages for pipe wall strain measurements, linear potentiometers for pipe deflections and circumferential shortening, and pressure cells for measuring soil pressure acting against pipe. Initial backfilling of the test pipes started during the summer of 1999. The soil fill placement over the test pipes began in early December 1999 and saw its rapid completion before the end of January 2000.

1.2 LITERATURE REVIEW

This section presents a review of past studies on the structural performance of HDPE pipes under deep burial. This review includes early theories and methods and how each of them evolved into a popular analytical technique for predicting structural performance of buried thermoplastic pipes. It also mentions some research studies that have been conducted to examine the validity of these methods. More details on the analysis by the elastic solutions and finite element will be discussed in a later chapter.
1.2.1 Conduit Load Theory and Iowa Formula

In 1913, Marston and Anderson [2] published their work on a method to estimate the vertical soil load acting on a buried rigid conduit. They considered the pipe as a “ditch” conduit, because the pipe was often buried in a narrow trench made into the native soil. The basis of their conduit-load theory was that the vertical load acting on the conduit was created by the settlement of the pipe and the soil prism above the pipe relative to the adjacent soils.

In 1941, Spangler [3] published his work on the structural behavior of flexible culverts, such as corrugated metal. It had been known for many years that flexible pipe can carry large loads when installed in relatively stiff soils, Prior to his study, there had been no analysis made to explain this phenomenon. He incorporated the elastic ring theory developed by Filkins and Fort [4] to determine pipe deflections. Spangler also considered the effects of the surrounding soil in his analysis and determined that the stiffness of soil is different from that of the pipe. The vertical load on the pipe was determined by the Marston’s conduit-load theory and assumed to be uniformly distributed across the top of the pipe. The reaction at the bottom of the pipe was equated to the vertical load and assumed to be distributed over the width of the bedding. Spangler assumed a passive horizontal soil pressure distributed parabolically over the middle 100 deg. arc section of the pipe on each side. The magnitude of this pressure was considered to be a function of soil type and pipe deflection. The outcome of the Spangler’s analysis was a horizontal deflection formula commonly known as the “Iowa formula.”
In 1958, Watkins and Spangler [5] reexamined the Iowa formula through a dimensional analysis. They discovered that the modulus of passive resistance, $e$, did not have the correct dimensions to be a modulus. With this finding, they revised the Iowa formula by replacing the modulus of passive resistance $e$ with the modulus of soil reaction $E'$.

1.2.2 Modulus of Soil Reaction $E'$

Howard [6] investigated the load-deflection relationship of buried flexible pipe, using data from available laboratory tests and special field installations. His study specifically focused on the modulus of soil reaction, which is part of Iowa formula.

Two approaches were used in the Howard’s work: (1) varied pipe modulus with a constant soil modulus; and (2) constant pipe modulus with soil having different soil modulus values. Data from one hundred and thirteen field installation tests were collected to back calculate the $E'$ values. These tests determined that varying the pipe moduli had negligible effect on the pipe deflection, whereas, increasing the density of the surrounding soil greatly reduced the pipe deflection. The results were tabulated for use in the Iowa Formula. These values of $E'$ were used to predict flexible pipe deflection for fills up to 50 ft. with no time dependency. These values can be found in Table 1.1.
Table 1.1 Average Values of Modulus of Soil Reaction $E'$ [6]

<table>
<thead>
<tr>
<th>Soil Type-Pipe Bedding Material (Unified Classification System)</th>
<th>$E'$ for Degree of Compaction of Bedding, lb/in$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dumped</td>
</tr>
<tr>
<td></td>
<td>Slight, &lt; 85% Proctor, &lt; 40% Relative Density</td>
</tr>
<tr>
<td></td>
<td>Moderate, 85%-95% Proctor, 40%-70% Relative Density</td>
</tr>
<tr>
<td></td>
<td>High, &gt;95% Proctor, &gt;70% Relative Density</td>
</tr>
<tr>
<td>Fine-grained soils (LL&gt;50) Soils with medium to high plasticity (CH, MH, CH-MH)</td>
<td>No data available; consult a competent soils engineer; Otherwise use $E'$ = 0</td>
</tr>
<tr>
<td>Fine-grained soils (LL&lt;50) Soils with no to medium plasticity, with less than 25% coarse-grained particles (CL, ML, ML-CL)</td>
<td>50 200 400 1,000</td>
</tr>
<tr>
<td>Fine-grained soils (LL&lt;50) Soils with no to medium plasticity, with more than 25% coarse-grained particles (CL, ML, ML-CL)</td>
<td>100 400 1,000 2,000</td>
</tr>
<tr>
<td>Coarse-grained soils with fines, with more than 12% fines (GM, GC, SM, SC)</td>
<td></td>
</tr>
<tr>
<td>Coarse-grained soils with little or no fines, with less than 12% fines (GW, GP, SW, SP)</td>
<td>200 1,000 2,000 3,000</td>
</tr>
<tr>
<td>Crushed rock</td>
<td>1,000 3,000 3,000 3,000</td>
</tr>
<tr>
<td>Accuracy in terms of percentage deflection</td>
<td>+/-2 +/-2 +/-1 +/-0.5</td>
</tr>
</tbody>
</table>

1.2.3 $E'$ and its Variation with Depth

In 1988, Hartley and Duncan [7] presented a paper on the study that they conducting on the modulus of soil reaction, $E'$. As with the Howard's study, they
believed that $E'$ not only depended on the soil type and density, but also on the depth of backfill as well.

Hartley and Duncan’s investigation contained four parts: (1) an in-depth review of literature on the dependence of $E'$ on backfill depth; (2) an examination of data for individual pipes buried under a range of backfill depths; (3) a study of the theoretical relationship of $E'$ to fundamental soil moduli using elastic analysis; and (4) a back-calculation of $E'$ using a finite element model of simulated buried pipelines.

As stated earlier, it had already been established in published literature that $E'$ varied with both soil type and density, and yet there was still no agreement on whether $E'$ increased with fill height. $E'$ was studied empirically from the deflection data of three different pipeline installations that had a range of backfill heights with the same burial conditions. From these examinations, it was apparent that $E'$ increased with depth. The modulus of soil reaction was then examined using the Burns and Richard elastic solutions [8], which determined that since all soil moduli were dependent on confining pressure, $E'$ varied with depth. $E'$ was also analyzed using the finite element Computer software, SSTIPN. $E'$ was calculated using three different methods and showed a good agreement. Again, this analysis showed that $E'$ varied with depth.

Hartley and Duncan also determined that the modulus of soil reaction, $E'$, was nearly equal to the constrained modulus, $M_s$, for the material next to the pipe. Therefore, a simple laboratory test could be conducted to determine $E'$ when desired. Results of their study on $E'$ are summarized in Table 1.2.
Table 1.2  Design Values of $E'$ (psi) [7]

<table>
<thead>
<tr>
<th>Type of Soil</th>
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<th>Standard Compaction</th>
<th>AASHTO</th>
<th>Relative</th>
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<td>Fine-grained soils with less than 25% sand content (CL,ML,CL-ML)</td>
<td>0-5</td>
<td>500</td>
<td>700</td>
<td>1,000</td>
</tr>
<tr>
<td></td>
<td>5-10</td>
<td>600</td>
<td>1,000</td>
<td>1,400</td>
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<tr>
<td></td>
<td>10-15</td>
<td>700</td>
<td>1,200</td>
<td>1,600</td>
</tr>
<tr>
<td></td>
<td>15-20</td>
<td>800</td>
<td>1,300</td>
<td>1,800</td>
</tr>
<tr>
<td>Coarse-grained soils with fines (SM, SC)</td>
<td>0-5</td>
<td>600</td>
<td>1,000</td>
<td>1,200</td>
</tr>
<tr>
<td></td>
<td>5-10</td>
<td>900</td>
<td>1,400</td>
<td>1,800</td>
</tr>
<tr>
<td></td>
<td>10-15</td>
<td>1,000</td>
<td>1,500</td>
<td>2,100</td>
</tr>
<tr>
<td></td>
<td>15-20</td>
<td>1,100</td>
<td>1,600</td>
<td>2,400</td>
</tr>
<tr>
<td>Coarse-grained soils with little or no fines (SP,SW,GP,GW)</td>
<td>0-5</td>
<td>700</td>
<td>1,000</td>
<td>1,500</td>
</tr>
<tr>
<td></td>
<td>5-10</td>
<td>1,000</td>
<td>1,500</td>
<td>2,200</td>
</tr>
<tr>
<td></td>
<td>10-15</td>
<td>1,050</td>
<td>1,600</td>
<td>2,400</td>
</tr>
<tr>
<td></td>
<td>15-20</td>
<td>1,100</td>
<td>1,700</td>
<td>2,500</td>
</tr>
</tbody>
</table>

1.2.4 Elastic Solutions

In 1964, Burns and Richard [8] presented a paper on the elastic solutions for buried pipe in homogenous soil. Their analysis was a plane-strain solution using an extension of the shell theory for the cylindrical shell and Mitchell’s [9] formulation of Airy’s stress function for the elastic soil medium. The loading was applied along the boundaries that were located at an infinite distance away from the pipe. This allowed for the modeling of deeply buried pipe. The solutions were applicable to both “rigid” and “flexible” pipes. The results of the analysis yield the following interactions between the shell and medium; the thrusts, moments, and displacements in the shell; and the stresses and displacements throughout the soil medium. The solutions were established under the “free-slip” and “full-bond” interface conditions between the shell and the soil medium.
1.2.5 Finite Element Computer Code CANDE

Katona and his associates [10] developed a computer program called CANDE (Culvert ANalysis and DEsign) in 1976. CANDE was developed for the structural analysis, design, and evaluation of buried soil-structure systems. It has three options for solution levels: Level 1 consists of the elastic solutions of Burns and Richard [8] for buried circular pipes. Level 2 is a plain-strain finite element analysis using a mesh that is automatically generated by the program. The analysis is available to pipes that are circular, elliptical, rectangular, or arch-shaped. Level 3 is also a plain-strain finite element analysis, except it allows the user to generate the mesh that may be non-symmetric. Due to its non-automatic nature of defining the problem domain, Level 3 allows flexibility in specifying the pipe geometry, material properties, and loading conditions.

1.2.6 Performance of Large Diameter HDPE Pipes

Watkins [11] presented a paper on the structural performance of a 36-inch diameter corrugated HDPE pipe. At the time this study was released, only pipes of diameter 24 inches and less had proven performance under high fill. Watkins' test was performed in the Utah State University soil cell. It used fine sand containing about 20% silt for a backfill; this percentage is on the lower limit allowed by the California Department of Transportation. The results were consistent with what they observed previously for small diameter HDPE pipes. Dimpling occurred at the springline at vertical deflection of approximately 5%.
1.2.7 HDPE Pipe Under High Fill

Adams, Muindi, and Selig [12] presented a paper in 1989 on the performance of a 24-inch diameter corrugated HDPE pipe under a 100-ft. high soil fill. The investigation took place at an embankment construction site on Interstate 279, north of Pittsburgh, Pennsylvania. The pipe's interior was mostly corrugated, with a smooth hydraulic liner added in some sections. The pipe was backfilled with a compacted crushed limestone. Data collection included pipe wall strain, pipe diameter changes, pipe circumferential shortening, pipe wall temperature, earth pressure acting on the pipe, and vertical soil strain adjacent to the pipe. It took a total of 722 days to complete the construction of the embankment. No signs of structural distress were found inside the pipe, and the pipe shape remained relatively round. At the end of embankment construction, the vertical deflection was about - 4 %, the horizontal deflection was 0.4 %, and the circumferential shortening was 1.4%. The geostatic pressure appeared to be approximately five times greater than the vertical soil pressure at the pipe crown, and about twice the horizontal pressure acting at the springline of the pipe.

1.2.8 Analysis of Buried HDPE Pipe

Hashash and Selig [13] presented a study on the long-term performance of the HDPE pipe, up to two years following pipe installation, and the interpretations of the results using finite element analysis. The observed long-term vertical and horizontal deflections were - 4.3 % and 0.6 %, respectively. The circumferential shortening was 1.6 %. There were no visual signs of structural distress. They used two finite element codes,
SOILCON and CANDE to simulate the field installation conditions, and they found that the short-term analysis showed reasonable agreement with the experimental values during construction. Both SOILCON and CANDE gave similar results, except CANDE over predicted soil pressure.

1.2.9 Performance of HC-HDPE Pipe

Sargand and Masada [14] presented a paper on the performance of a 42-inch diameter (Honeycomb) HC-HDPE pipe that was buried under a 52-ft. of fill at a highway construction site in southeast Ohio. The data collected in the study included the strains in the pipe wall during the backfilling process, earth pressure readings surrounding the pipe, and the vertical and horizontal deflections of the pipe. The investigation took place in 1995, and the pipe performance was monitored for 386 days. The horizontal deflection stabilized within 40 days under the maximum soil fill. However, a longer time was needed for the vertical deflection to converge. The final pipe deflections were - 10% vertical and 3% horizontal. Visual inspection revealed that the pipe suffered from localized buckling in the springline area. The researchers felt that this was due to a combination of bending and ring compression actions. Two different granular materials were used in backfilling the pipe; the change from one material to the other was made in the vicinity of the springline, which may have caused a loss in lateral support. A comparison was then made between the field results and the analytical results from the elastic solutions by Burns and Richard and the finite element computer software.
elastic solutions by Burns and Richard and the finite element computer software CANDE-89. Both methods showed partial success in predicting the field pipe performance.

1.3 OBJECTIVES

The objectives of this study are summarized below:

*Record the pressure distribution around the perimeter of the pipes subjected to deep burial.

*Record the deflection profile and circumferential shortening of the pipes subjected to deep cover.

*Record the pipe response as a function of time.

*Evaluate reliability of empirical and numerical methods developed for the analysis of buried flexible pipe in light of the latest field test data
CHAPTER 2
PROJECT OVERVIEW

2.1 INTRODUCTION

A total of 18 thermoplastic pipes were instrumented and monitored under deep cover at the ORITE deep burial site in Albany, Ohio. Four thermoplastic pipe manufacturers each supplied a 300 linear ft. of 30 in. diameter thermoplastic pipe. Advanced Drainage Systems (ADS), Inc. and Lane Enterprises (Lane), Inc. supplied 30 in. diameter High-Density Polyethylene (HDPE) pipes, while Contech and Lamson & Sessions Vylon supplied 30 in. diameter Poly Vinyl Chloride (PVC) pipes. ADS, Inc. also supplied 42 in. and 60 in. diameter HDPE pipes. Table 2.1 provides a list of test pipes involved in the study along with their installation plans, and Figure 2.1 shows an overall embankment construction plan.

2.2 SITE DESCRIPTION

The ORITE deep burial site is approximately 13 miles southwest of the Ohio University Campus in Athens, Ohio, on Section 13, Town 10, Range 15, Lee Township, Albany, Ohio. Athens County is part of the unglaciated Allegheny Plateau Region. The Albany area is found to have gently sloping, high, level terraces. According to Soil Survey of Athens County Ohio [15], the soil is a preglacial till of Omulga silt loam (OtB). Shale bedrock is found at an estimated depth of 20 ft.
Table 2.1  List of Test Pipes [16]

<table>
<thead>
<tr>
<th>Pipe #</th>
<th>Pipe Type**</th>
<th>Degree of Compaction</th>
<th>Backfill Type</th>
<th>Bedding Thick.</th>
<th>Fill Height</th>
<th>Pressure Cell Config. *</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>96%</td>
<td>Sand</td>
<td>6”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>A</td>
<td>96%</td>
<td>Gravel</td>
<td>6”</td>
<td>40’</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>A</td>
<td>86%</td>
<td>Gravel</td>
<td>6”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>B</td>
<td>86%</td>
<td>Sand</td>
<td>6”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>B</td>
<td>96%</td>
<td>Gravel</td>
<td>6”</td>
<td>40’</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>B</td>
<td>96%</td>
<td>Gravel</td>
<td>6”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>7</td>
<td>C</td>
<td>96%</td>
<td>Sand</td>
<td>6”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>8</td>
<td>C</td>
<td>96%</td>
<td>Sand</td>
<td>6”</td>
<td>40’</td>
<td>3</td>
</tr>
<tr>
<td>9</td>
<td>C</td>
<td>86%</td>
<td>Gravel</td>
<td>6”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>D</td>
<td>86%</td>
<td>Sand</td>
<td>6”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>11</td>
<td>D</td>
<td>96%</td>
<td>Gravel</td>
<td>6”</td>
<td>40’</td>
<td>2</td>
</tr>
<tr>
<td>12</td>
<td>D</td>
<td>96%</td>
<td>Gravel</td>
<td>6”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>13</td>
<td>E</td>
<td>90%</td>
<td>Sand</td>
<td>0-12”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>14</td>
<td>E</td>
<td>96%</td>
<td>Sand</td>
<td>3-15”</td>
<td>40’</td>
<td>1</td>
</tr>
<tr>
<td>15</td>
<td>E</td>
<td>90%</td>
<td>Gravel</td>
<td>0-12”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>16</td>
<td>F</td>
<td>90%</td>
<td>Gravel</td>
<td>3-9”</td>
<td>20’</td>
<td>1</td>
</tr>
<tr>
<td>17</td>
<td>F</td>
<td>96%</td>
<td>Gravel</td>
<td>6-12”</td>
<td>40’</td>
<td>1</td>
</tr>
<tr>
<td>18</td>
<td>F</td>
<td>96%</td>
<td>Sand</td>
<td>3-9”</td>
<td>20’</td>
<td>1</td>
</tr>
</tbody>
</table>

* See Figure 2.8 for Pressure Cells Configuration  Invert pressure cell used for Pipe 11 was located at Pipe 8

** Pipe A: Lamson & Sessions Vylon – 30” Dia.
** Pipe B: Contech A2000 – 30” Dia.
** Pipe C: Lane HDPE – 30” Dia.
** Pipe D: ADS N12 – 30” Dia.
** Pipe E: ADS N12 – 42” Dia.
** Pipe F: ADS – 60” Dia.

= Additional pressure cells were placed at various fills heights above the pipe.
Figure 2.1 Overall Embankment Construction Plan [16]
2.3 DATA RECORDED IN THE FIELD

During initial backfill and subsequent embankment construction, the following data were recorded in the field:

* The changes in horizontal and vertical diameters at the mid-length section of the test pipes. Also, a circumferential shortening was recorded at the mid-length section inside selected test pipes.

* Soil pressures acting against the pipe were measured at the crown and springline of all test pipes. Soil pressure acting at the invert was measured for several pipes. Two test pipes had additional pressure cells placed at different fill heights above the crown in the overburden.

* Pipe's cross-sectional shapes were recorded with a laser profilemeter while the embankment was under construction. The profilemeter was designed and built by the ORITE.

* Pipe wall strains were recorded at the inside and outside surfaces of the thermoplastic pipe near the mid-length section, using electric resistance strain gages. The outside strain readings were taken from strain gages placed at the crest of exterior corrugation. The inside strain readings were taken from strain gages placed on the interior lining, directly oppose to the outside strain gage location. Figure 2.2 illustrates a typical strain gage installation plan. For smooth-walled pipes, a location near the mid-length of the pipe was instrumented on the inside and outside faces.
*Additional circumferential strain readings were taken with the use of fiber optic strain gages placed also near the mid-length section. The fiber optic strain gages were placed at this section added as backup sensors, because of their long-term stability.

*It took about one and half months to complete the initial backfilling of all test pipes. The construction history data are summarized in Tables 2.2 and 2.3.

**Table 2.2 Summary of Construction Data for Initial Pipe Backfilling**

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Dates of Backfilling</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>16,17,18</td>
<td>10/06/99 – 10/18/99</td>
<td>Removed and then replaced Lifts 1 on Pipe 17 on 10/08/99. A major tension crack was seen running through the trench wall next to Test Pipe 17.</td>
</tr>
<tr>
<td>13,14,15</td>
<td>10/20/99 – 10/27/99</td>
<td></td>
</tr>
</tbody>
</table>
Table 2.3  Rate of Embankment Construction Data for Test Pipes

<table>
<thead>
<tr>
<th>Height of Fill (ft.)</th>
<th>Completion Date</th>
<th>Time Completed</th>
<th>Date of Data Collection</th>
<th>Time of Data Collection</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>11/30/99</td>
<td>2:00 P.M.</td>
<td>12/2/99</td>
<td>10:30 A.M.</td>
</tr>
<tr>
<td>4.5</td>
<td>12/8/99</td>
<td>2:30 A.M.</td>
<td>12/8/99</td>
<td>7:45 A.M.</td>
</tr>
<tr>
<td>6</td>
<td>12/8/99</td>
<td>3:30 P.M.</td>
<td>12/8/99</td>
<td>3:50 P.M.</td>
</tr>
<tr>
<td>7.5</td>
<td>12/9/99</td>
<td>2:30 A.M.</td>
<td>12/9/99</td>
<td>8:00 A.M.</td>
</tr>
<tr>
<td>8</td>
<td>12/9/99</td>
<td>4:30 P.M.</td>
<td>12/9/99</td>
<td>6:00 P.M.</td>
</tr>
<tr>
<td>9.5</td>
<td>12/10/99</td>
<td>2:30 A.M.</td>
<td>12/15/99</td>
<td>8:00 A.M.</td>
</tr>
<tr>
<td>10</td>
<td>12/15/99</td>
<td>4:30 P.M.</td>
<td>12/15/99</td>
<td>5:00 P.M.</td>
</tr>
<tr>
<td>11</td>
<td>12/16/99</td>
<td>2:30 A.M.</td>
<td>12/16/99</td>
<td>7:30 A.M.</td>
</tr>
<tr>
<td>12</td>
<td>12/16/99</td>
<td>4:30 P.M.</td>
<td>12/16/99</td>
<td>4:45 P.M.</td>
</tr>
<tr>
<td>13</td>
<td>12/17/99</td>
<td>2:30 A.M.</td>
<td>12/17/99</td>
<td>8:00 A.M.</td>
</tr>
<tr>
<td>14</td>
<td>12/17/99</td>
<td>2:30 P.M.</td>
<td>12/17/99</td>
<td>3:15 P.M.</td>
</tr>
<tr>
<td>14.5</td>
<td>12/18/99</td>
<td>2:30 A.M.</td>
<td>12/20/99</td>
<td>9:40 A.M.</td>
</tr>
<tr>
<td>17</td>
<td>12/21/99</td>
<td>2:30 A.M.</td>
<td>12/21/99</td>
<td>9:40 A.M.</td>
</tr>
<tr>
<td>18</td>
<td>12/21/99</td>
<td>3:30 P.M.</td>
<td>12/21/99</td>
<td>3:45 P.M.</td>
</tr>
<tr>
<td>20</td>
<td>12/22/99</td>
<td>10:00 A.M.</td>
<td>12/22/99</td>
<td>11:00 A.M.</td>
</tr>
<tr>
<td>22</td>
<td>12/22/99</td>
<td>4:00 P.M.</td>
<td>12/22/99</td>
<td>4:30 P.M.</td>
</tr>
<tr>
<td>23</td>
<td>12/23/99</td>
<td>2:30 A.M.</td>
<td>12/23/99</td>
<td>7:30 P.M.</td>
</tr>
<tr>
<td>28</td>
<td>12/24/99</td>
<td>2:30 A.M.</td>
<td>12/24/99</td>
<td>7:30 A.M.</td>
</tr>
<tr>
<td>29</td>
<td>12/24/99</td>
<td>12:00 P.M.</td>
<td>12/27/99</td>
<td>7:30 A.M.</td>
</tr>
<tr>
<td>32</td>
<td>12/27/99</td>
<td>3:30 P.M.</td>
<td>12/27/99</td>
<td>3:30 P.M.</td>
</tr>
<tr>
<td>38</td>
<td>12/28/99</td>
<td>2:30 A.M.</td>
<td>12/27/99</td>
<td>7:30 P.M.</td>
</tr>
<tr>
<td>40</td>
<td>12/28/99</td>
<td>3:00 A.M.</td>
<td>12/29/99</td>
<td>11:00 A.M.</td>
</tr>
</tbody>
</table>

2.4  PIPE PROPERTIES

Six thermoplastic pipe products, provided by the four manufacturers were either manufactured of HDPE or PVC. Four of the pipe products were corrugated on the outside and smooth-lined on the inside. Remaining two pipe products had smooth
surfaces on both sides of the pipe wall. Figure 2.3 illustrates the profile-wall design of each test pipe. All the pipes in this study have different designs and meet the design requirements. A section was cut from each pipe to determine the shape, wall thickness, moment of inertia, and wall area. Table 2.4 lists the basic properties of the pipes.

Pipe stiffness (PS) estimated by the following formula:

\[ PS = \frac{6.71EI}{R^3} \]  

(2.1)

Where \( I \) = moment of inertia of the pipe wall per unit length, \( E \) = modulus of elasticity of the pipe material, and \( R \) = mean radius of pipe.
Table 2.4  Basic Properties of Test Pipes[16]

<table>
<thead>
<tr>
<th>Make *</th>
<th>Nominal Diameter</th>
<th>Pipe Material</th>
<th>Wall Area (in²/in)</th>
<th>Wall Moment of Inertia (in⁴/in)</th>
<th>Short-Term Modulus (ksi)</th>
<th>Pipe Stiffness (lb/in/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>30 in.</td>
<td>U-PVC</td>
<td>0.434</td>
<td>0.0514</td>
<td>400</td>
<td>40.9</td>
</tr>
<tr>
<td>B</td>
<td>30 in.</td>
<td>U-PVC</td>
<td>0.475</td>
<td>0.111</td>
<td>400</td>
<td>88.3</td>
</tr>
<tr>
<td>C</td>
<td>30 in.</td>
<td>HDPE</td>
<td>0.377</td>
<td>0.285</td>
<td>110</td>
<td>62.3</td>
</tr>
<tr>
<td>D</td>
<td>30 in.</td>
<td>HDPE</td>
<td>0.392</td>
<td>0.287</td>
<td>110</td>
<td>62.8</td>
</tr>
<tr>
<td>E</td>
<td>42 in.</td>
<td>HDPE</td>
<td>0.430</td>
<td>0.521</td>
<td>110</td>
<td>41.5</td>
</tr>
<tr>
<td>F</td>
<td>60 in.</td>
<td>HDPE</td>
<td>0.657</td>
<td>0.850</td>
<td>110</td>
<td>23.2</td>
</tr>
</tbody>
</table>

*Refer to Table 2.1 for the make of each test pipe.

2.5  SOIL TYPE AND PROPERTIES

Two different backfill materials were utilized in installing the pipes. One of the materials used as bedding and backfill was a crushed limestone. The crushed limestone met the 1997 ODOT Item 603.02 specifications for pipe bedding and backfill. Under this specification, the material was identified as Granular Material, Type 1. The second backfill material used as bedding and backfill was sand. The sand also met ODOT 1997 Item 603.02 specifications for pipe bedding and backfill. Under this specification, the material was identified as Granular Material, Type 2. Tables 2.5. (A) and (B) summarize the sieve analysis results on the crushed limestone and sand. The maximum dry density unit weight ($\gamma_{d,max}$) of the crushed limestone and the sand was 140.6 lb/ft³ and 120.5 lb/ft³, respectively. Figures 2.4 and 2.5 present the actual compaction curves for these materials. Engineering properties of each backfill material will be presented in more details in Chapter 3.
For each test pipe, the backfill material was first placed as bedding material and compacted only on the outer 1/3 section of the trench width (leaving the middle 1/3 section loose). The pipe sections were placed on top of the loose bedding material. The backfill material was placed in lifts, leveled and compacted with a vibratory plate compactor.

To ensure that the compaction of the backfill material was achieved consistently as planned along each pipeline, stiffness gage readings were taken on the backfill lifts placed for the test pipes, where the degrees of compaction had been known from the sand cone test results. This established a direct correlation between the stiffness gage reading and the actual degree of compaction. Then, the correlation was applied to monitor quickly the quality of backfill compaction for the pipes located between the test pipes.

### Table 2.5.A  Sieve Analysis of ODOT Granular Material, Type 1 (Crushed Limestone)

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Actual Mass Retained (g)</th>
<th>Actual % Passing</th>
<th>Required %Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>2&quot;</td>
<td>0</td>
<td>100</td>
<td>100%</td>
</tr>
<tr>
<td>1&quot;</td>
<td>19.5</td>
<td>97.96</td>
<td>70-100%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>70.9</td>
<td>90.54</td>
<td>50-90%</td>
</tr>
<tr>
<td>No. 4</td>
<td>418.2</td>
<td>46.76</td>
<td>30-60%</td>
</tr>
<tr>
<td>No. 30</td>
<td>308.9</td>
<td>14.42</td>
<td>9-33%</td>
</tr>
<tr>
<td>No. 200</td>
<td>89.8</td>
<td>5.02</td>
<td>0-13%</td>
</tr>
<tr>
<td>Pan</td>
<td>48</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

[Note] This material was classified to be A-1-a by the AASHTO method
Table 2.5.B  Sieve Analysis of ODOT Granular Material, Type 2  
(Sand)

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Actual Mass Retained (g)</th>
<th>Actual % Passing</th>
<th>Required % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5&quot;</td>
<td>0</td>
<td>100</td>
<td>100%</td>
</tr>
<tr>
<td>1&quot;</td>
<td>0</td>
<td>100</td>
<td>70-100%</td>
</tr>
<tr>
<td>No. 4</td>
<td>0</td>
<td>100</td>
<td>25-100%</td>
</tr>
<tr>
<td>No. 40</td>
<td>259</td>
<td>43.62</td>
<td>5-50%</td>
</tr>
<tr>
<td>No. 200</td>
<td>191.4</td>
<td>1.96</td>
<td>0-10%</td>
</tr>
<tr>
<td>Pan</td>
<td>9</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

[Note] This material was classified to be A-1-b by the AASHTO method.

Figure 2.4  Compaction Curve for ODOT Granular Material, Type 1  
(Crushed Limestone)

OMC = 7.63%  
$\gamma_{\text{dmax}} = 140.57 \text{ lb/ft}^3$
Figure 2.5  Compaction Curve for ODOT Granular Material, Type 2(Sand)
CHAPTER 3
LABORATORY TESTING OF EMBANKMENT FILL AND BACKFILL SOILS

3.1 INTRODUCTION

Laboratory testing was conducted in the ORITE Laboratory. The main purpose of the laboratory testing was to characterize the engineering properties of the backfilling materials placed around the buried pipes as well as those of the embankment fill material. This allowed for a more accurate analysis and modeling of the structural performance of the thermoplastic pipes.

The laboratory tests performed on the embankment soil sample consisted of:

- Atterberg limits
- Soil classification
- Triaxial compression test

The laboratory tests performed on each granular backfill material consisted of:

- Mechanical sieve analysis
- Standard Proctor compaction test
- One-dimensional compression test
- Triaxial compression test

The one-dimensional compression test was performed to obtain the one-dimensional constrained modulus values, which will be useful in the analysis of the buried flexible pipe by the modified Iowa formula and the elastic solutions. A series of triaxial...
compression test was performed to determine the values of parameters that will be used in the numerical simulation of the pipe-soil interaction. The mechanical sieve analysis was made to check if the material met the applicable ODOT gradation specification.

3.2 SOIL CLASSIFICATION

The determination of the soil properties surrounding the flexible pipe is essential in the analysis of its structural performance. The sieve analysis was to ensure the quality of the material that is being used for backfill.

3.2.1 Atterberg Limits

ASTM-D 4318 procedures were used to determine the Atterberg limits. Casagrande developed the equipment and procedures for determining the liquid limit of soils in 1932, known as the Casagrande cup method. It has been shown that the physical and mechanical behavior of fine-grained soils has four distinct states: solid, semisolid, plastic, and liquid. This change of state occurs with an increase in moisture content. In 1911, a Swedish scientist named Atterberg developed the method to describe this behavior. As a soil initially in a liquid state is allowed to dry, its moisture content and volume will decrease. When the soil reaches a point that it becomes too stiff and can no longer flow, it has reached a limit. This moisture content limit is referred to as the liquid limit. As the soil continues to dry, there is a range of moisture contents at which the soil can be remolded without suffering from any cracks or ruptures. This plastic behavior continues until the moisture content becomes low enough that cracks appear. This
moisture content is referred to as the plastic limit, and the range in between is known as the plastic index.

3.2.1.1 Classification of Embankment Fill Soil

A representative sample of the soil fill was collected from the site during the construction of the embankment. The Atterberg limit tests yielded the following results: liquid limit (LL) of 27.2%, plastic limit (PL) of 16.5%, and plasticity index (PI) of 10.7%. The results from the Atterberg limit tests agree with the range specified by the local soil survey reports. Figure 3.1 illustrates the results from the liquid limit test. The results were used to classify the soil using the Unified Classification System (ASTM D-2487). The soil was classified as CL (lean clay). From in-situ tests using the Shelby tube, the average compacted unit weight of the soil fill was determined to be 130.0 pcf.

3.2.2 Sieve Analysis of Backfill Soils

The particle size analysis was performed on each backfill soil in accordance with ASTM D 421 and ASTM D 422. This test was performed to ensure the integrity of the material received and used as a backfilling around the test pipes. The results from the sieve analysis have been presented in Chapter 2 --- see Tables 2.5.A and 2.5.B. Figures 3.2 and 3.3 are graphical plots of the data summarized in these tables. The results showed that both materials met the required ODOT gradation specifications.
Figure 3.1  Liquid Limit Test Results for Embankment Fill Soil [16]

Figure 3.2  Sieve Analysis Results for ODOT Granular Material-Type 2 (Sand) [16]
3.2.3 Standard Proctor Compaction Tests

In 1933, R. R. Proctor presented the procedure for the Standard Proctor Test described in ASTM D 698-78. This method performed to determine the maximum dry unit weight for the backfill material in this study:  ODOT Granular Material-Type 1 (Crushed Limestone) and ODOT Granular Material-Type 2 (Sand).

Compaction of loose materials is an important process because it increases the soil’s shear strength and unit weight. It lowers the compressibility and reduces the permeability of the soil by rearranging its structure into a denser configuration. It also improves the soil’s behavior and is usually expressed in terms of a percentage of the
maximum dry unit weight. Water is added to the soil to achieve a greater compaction because it allows particles to slip over each other and densification occurs more easily. However, if the moisture content becomes greater than the optimum moisture at which maximum dry density occurs, the dry unit weight be reduced.

Results from the standard Proctor test on the backfill material identified as the ODOT Granular Material-Type 2 (sand) were as follows: the optimum moisture content (OMC) of 11.5 %, and the maximum dry unit weight ($\gamma_{d \text{ max}}$) of 120.5 pcf. Figure 2.5 presented the compaction curve for this soil.

Results from the standard Proctor test on the backfill material identified as the ODOT Granular Material-Type 1 (Crushed Limestone) were as follows: the optimum moisture content (OMC) of 7.6%, and the maximum dry unit weight ($\gamma_{d \text{ max}}$) of 140.6 pcf. Figure 2.4 presented the compaction curve for this soil.

3.3 ONE DIMENSIONAL COMPRESSION TEST

The one-dimensional compression test was performed using a MTS load frame system. Either the ASTM or AASHTO did not issue any standardized test procedures for this test. This test was performed on both types of the backfill material at 86 %, 90%, and 96 % relative compactions.

The samples were prepared in a 4-inch diameter Proctor mold for the sand and in a 6-inch diameter CBR mold for the crushed limestone. All aggregates in the crushed limestone larger than $\frac{1}{2}$-inch were removed through a sieve before compaction. From the results shown in Figures 3.4 and 3.5 and the known volume of the mold, necessary
amounts of water and dry soil solid were determined. The test specimen was then prepared in the same procedure as the standard Proctor test sample. A steel circular plate with a diameter approximately 1/4-inch smaller than the inside diameter of the mold was used to load the specimen in each type of molds. The loading rate was set at -0.5 % strain/minute. Load readings were taken every 15 seconds during the first 2 minutes of loading and then every 30 seconds afterward.

Results from the one-dimensional compression tests were plotted in terms of stress vs. strain for each compaction and material. The tangent slope or constrained modulus (M*) was determined from the stress-strain plots.

Table 3.1 Values of $E'$ (psi) Determined from One-Dimensional Compression Tests

<table>
<thead>
<tr>
<th>Type of Material</th>
<th>Pressure (psi)</th>
<th>E' @ Relative Compaction of:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>90 %</td>
</tr>
<tr>
<td>ODOT Granular</td>
<td>0-5</td>
<td>1,520</td>
</tr>
<tr>
<td>Material-Type 2</td>
<td>5-10</td>
<td>1,650</td>
</tr>
<tr>
<td>(Sand)</td>
<td>10-15</td>
<td>1,725</td>
</tr>
<tr>
<td></td>
<td>15-20</td>
<td>1,865</td>
</tr>
<tr>
<td>ODOT Granular</td>
<td>0-5</td>
<td>1,896</td>
</tr>
<tr>
<td>Material-Type 1</td>
<td>5-10</td>
<td>2,300</td>
</tr>
<tr>
<td>(Crushed Limestone)</td>
<td>10-15</td>
<td>2,805</td>
</tr>
<tr>
<td></td>
<td>15-20</td>
<td>3,238</td>
</tr>
<tr>
<td></td>
<td>20-25</td>
<td>3,888</td>
</tr>
<tr>
<td></td>
<td>25-30</td>
<td>4,105</td>
</tr>
<tr>
<td></td>
<td>30-35</td>
<td>4,538</td>
</tr>
</tbody>
</table>

From the previous work summarized by ORITE [17], it was concluded that the constrained modulus (M*) was approximately equal to the modulus of soil reaction ($E'$). The plots from the tests are all presented in Appendix B. The results show an increase in
as the stress increases, which agree with the view expressed by Hartley and Duncan [7]. The test results are summarized in Table 3.1.

3.4 TRIAXIAL COMPRESSION TESTS

Flexible pipe derives its strength from the surrounding soil material. The strength of the soil material is measured by its shear strength or ability to resist deformation. The triaxial test offers an excellent shear strength measurement of soils. An advantage of using the triaxial compression test is that the results can be further analyzed to derive hyperbolic model parameters for finite element analysis. The conventional triaxial tests were performed using two separate systems. Testing on the embankment fill soil and the ODOT Granular Material-Type 2(sand) were performed using the SBEL HX-100 Triaxial Cell with the GeTS testing software. This system was also used to test the ODOT Granular Material-Type 1 (Crushed Limestone) at 86% relative compaction. Due to limited load cell capacity (1500 lb.) in this system, the crushed limestone specimens compacted at 90% and 96% relative compactions were tested with the MTS- Model 655.05A-01 triaxial cell with the Test Star II software.

3.4.1 Triaxial Compression Test Results on Embankment Fill Soil

ASTM D 4767 – 88 testing procedures for consolidated – undrained (CU) triaxial compression test on cohesive soils was used for testing the embankment fill soil. Consolidation of each test specimen was performed before applying the deviator stress and loading to failure. Applying an equal pressure to all faces of the sample carried out
an isotropic consolidation. Due to the low permeability of the clay, this consolidation was performed over a 24-hour period. During this time period, the drainage valve was left open. The drainage valve was then closed before loading.

A 2.8-inch diameter by approximately 6-inch height specimen was prepared for the testing. The specimen had average moisture content of 15.9% and unit weight of 130.0 pcf to comply with the findings in the field. Testing was performed at a rate of 2% strain/minute. Three different confining pressures of 5, 10, and 15 psi were used during the test program. To obtain additional test data, unconfined compression tests were also performed on the embankment fill. Efforts were made to repeat each test to check the validity of the results Figures 3.4 and 3.5 present the typical test results graphically.

![Figure 3.4](image)

**Figure 3.4** Deviator Stress vs. Axial Strain Plots for Embankment Fill Soil [16]
3.4.2 Triaxial Compression Test Results on ODOT Granular Material-Type 2 (Sand)

The consolidated-drained (CD) triaxial compression test was performed on the ODOT Granular Material-Type 2 (sand). The test procedures were very similar to the CU test, except that the drainage valve was open during consolidation, and during the deviatoric loading. This allows for both air and water to drain out of the test specimen during the test. The testing was performed at a rate of 2.0% strain/minute.

A 2.8-inch diameter mold was utilized in the preparation of each test specimen. The specimen had a height of approximately 6 inches. Test specimens were prepared at
relative compactions of 90% and 96%. Again, the tests were performed at 5, 10, and 15psi confining pressure. The specimens compacted at relative compactions of 90%, and 96% had dry unit weight of 108.5 and 122 pcf, respectively. The standard proctor compaction method was applied to prepare each test specimen. In some cases, the rubber mallet was used to tap the sides of the mold to increase the compaction. The specimen was prepared inside the rubber membrane, and a small vacuum was used to keep the test specimen intact during the removal of the mold and placement of the specimen into the triaxial chamber. Typical Triaxial test results are shown in Figures 3.8 through 3.11. As expected, the angle of internal friction ($\phi$) increased for the specimens prepared at higher relative compactions.

![Figure 3.6 Deviator Stress Vs. Axial Strain Plots for ODOT Granular Material-Type 2 (Sand) at 90% Relative Compaction](image)
Figure 3.7  Shear Failure Envelope for ODOT Granular Material-Type 2 (Sand) at 90% Relative Compaction

Figure 3.8  Deviator Stress Vs. Axial Strain Plots for ODOT Granular Material-Type 2 (Sand) at 96% Relative Compaction
3.4.3 Triaxial Compression Test Results on ODOT Granular Material-Type 1 (Crushed Limestone)

The consolidated-drained (CD) triaxial compression test was performed on the ODOT Granular material-Type1 (crushed limestone) sample. The test procedures applied to test the sand were followed for the crushed stone material. Each specimen was loaded at a rate of 2.0 % strain/minute. The test specimens were prepared at relative compactions of 90% and 96 %. Confining pressures of 5, 10, and 15psi were introduced in the chamber during the test program.

The crushed limestone specimens prepared at higher (90% and 96%) relative compaction were tested inside the MTS Model 655.05 A-01 triaxial champer.
The specimen size of 6-in. diameter by 12-in height was chosen for the tests. The specimens had moist unit weight of 126.5 pcf (moisture content 2.5 %) and 135 pcf (moisture content 5.3%) at relative compactions of 90% and 96%, respectively. All aggregates larger than 1-inch were removed through sieving. The compaction of the specimen was achieved by the use of an air hammer attached to a 3.5-inch diameter plate. Typical test results are shown in Figures 3.10 through 3.13. Comparing all the results, it can be stated that higher degree of compaction influenced the angle of internal friction ($\phi$) more than the cohesion.

![Figure 3.10 Deviator Stress Vs. Axial Strain Plots for ODOT Granular material-Type 1 (Crushed Limestone) at 90% Relative Compaction](image-url)
Figure 3.11 Shear Failure Envelope for ODOT Granular Material-Type 1 (Crushed Limestone) at 90% Relative Compaction

Figure 3.12 Deviator Stress Vs. Axial Strain Plots for ODOT Granular Material-Type1(Crushed Limestone) at 96% Relative Compaction.
3.5 DETERMINATION OF HYPERBOLIC MODEL PARAMETERS

In 1980, J. M. Duncan et al. [18] published a report on the hyperbolic stress-strain parameters for nonlinear finite element analysis in soil masses. In 1980, CANDE (Culvert ANalysis and DEsign) implemented Duncan’s formulation of a tangent Young’s modulus and bulk modulus into its analysis. CANDE is a computer program for the structural analysis, design and evaluation of buried culverts and other soil-structures. In 1988, Selig [19] further extended and modified Duncan’s formulations. Selig’s alternative method agreed with Duncan’s technique of determining the parameters from

**Figure 3.13** Shear Failure Envelope for ODOT Granular Material-Type 1 (Crushed Limestone) at 96% Relative Compaction
the same tangent modulus from the triaxial compression, but gave a different method for determining the bulk modulus. Selig obtained the bulk modulus from a hydrostatic compression test, whereas Duncan proposed using the volume change and deviator stress from the triaxial test results. More recent studies have shown that the hydrostatic compression test gives better results for representing soil behavior than the power formulation developed by Duncan. CANDE offers both the Selig and Duncan formulation in its subroutine. Further details on the procedures for deriving the hyperbolic stress-strain parameters are given in the next section.

The hyperbolic stress-strain relationships are developed to strengthen the finite element analysis. The Generalized Hooke’s Law of elastic deformation governs the relationship between the stress and strain [18].

Using the deviator stress vs. axial strain data obtained from a triaxial test, the Duncan – Selig hyperbolic parameters were obtained. These parameters include the modulus number (K), the modulus exponent (n), the failure ratio (R_f), cohesion (c), the initial friction angle (φ_0), and the reduction in friction angle (Δφ). The method for determining these parameters will be explained in this section.

Figure 3.14 illustrates a hyperbolic stress-strain curve for a tri-axial compression test. Mathematically, this curve can be represented by Eq. (3.1) shown below.

\[
\sigma_1 - \sigma_3 = \frac{\varepsilon}{1 + \left(\frac{\varepsilon}{E_i}\right)} \left(\frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_u}\right)
\]  

(3.1)
where \( E_i \) = initial tangent modulus and \( (\sigma_1 - \sigma_3)_u \) = ultimate deviator stress.

To determine the values of \( E_i \) and \( (\sigma_1 - \sigma_3)_u \), the non-linear data shown above was transformed into a linear relationship. Eq. (3.1) was transformed to Eq. (3.2) below and Figure 3.15 illustrates the linear transformation.

\[
\frac{\varepsilon}{(\sigma_1 - \sigma_3)} = \frac{1}{E_i} + \frac{\varepsilon}{(\sigma_1 - \sigma_3)_u} 
\]  

(3.2)
The actual point of soil failure is always smaller than \((\sigma_1 - \sigma_3)_u\). This point is represented by \((\sigma_1 - \sigma_3)_f\). The relationship between these points is expressed by a failure ratio \((R_f)\) shown below in Eq. (3.3). The value of \(R_f\) is typically between 0.5 and 0.9 for most soils.

\[
R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_u}
\]  

(3.3)
For all soils except those fully saturated and tested under unconsolidated-undrained (UU) conditions, the strength will become greater with an increase in confining stress ($\sigma_3$). Therefore, the values of $E_i$ and $(\sigma_1-\sigma_3)_u$ will increase. This variation of $E_i$ and $\sigma_3$ is represented by Eq. (3.4) below. The relationship can also be seen in Figure 3.16.

\[ E_i = KP_a \left( \frac{\sigma_3}{P_a} \right)^n \]  

(3.4)

where $K =$ modulus number, $n =$ modulus exponent, and $P_a =$ atmospheric pressure.

Figure 3.16  Variation of Initial Tangent Modulus with Confining Pressure
The variation of \((\sigma_1 - \sigma_3)_f\) and \(\sigma_3\) is represented by the Mohr-Coulomb shear failure envelope. This relationship can be expressed by Eq. (3.5) and Figure 3.17.

\[
(\sigma_1 - \sigma_3)_f = \frac{2c \cos \phi + 2\sigma_3 \sin \phi}{1 - \sin \phi}
\]  

(3.5)

where \(c\) = cohesion and \(\phi\) = friction angle.

Figure 3.17 Variation of Strength with Confining Pressure
The failure envelope shown above is actually curved in a concaved down formation. This indicates that the value of \( \phi \) may vary with the confining stress. Therefore, either a best-fit straight line or Eq. (3.6) can be used to determine \( \phi \) and \( \Delta \phi \) (reduction in \( \phi \) for a ten-fold increase in \( \sigma_3 \)). The method used to determine these parameters is illustrated in Figure 3.18.

\[
\phi = \phi_o - \Delta \phi \log_{10} \left( \frac{\sigma_3}{P_o} \right)
\]  

(3.6)

where, \( \phi_o = \) value of \( \phi \) for \( \sigma_3 = P_o \).

Figure 3.18 Variation of \( \phi \) with Confining Pressure
The tangent Young's modulus ($E_t$) may be determined for any stress state by differentiating Eq. (3.1), which yields Eq. (3.7):

$$E_t = \left(1 - \frac{R_f (1 - \sin \phi)(\sigma_1 - \sigma_3)}{2c \cos \phi + 2\sigma_3 \sin \phi}\right)^2 \cdot KP_a \left(\frac{\sigma_3}{P_a}\right)^m$$  \hspace{1cm} (3.7)

The volumetric parameters such as $K_b$ and $m$ for Duncan's model can also be determined from the tri-axial results. This is achieved by using the values of volume change and deviator stress at which either the point of 70% of the maximum strength is reached on the deviator stress vs. strain plot, or when the plot of volumetric strain vs. axial strain has a slope of zero. These values can then incorporated into Eq. 3.8 and Eq. 3.9.

$$B = \frac{\sigma_1 - \sigma_2}{3\varepsilon_{vol}}$$  \hspace{1cm} (3.8)

$$B = K_b P_a \left(\frac{\sigma_3}{P_a}\right)^m$$  \hspace{1cm} (3.9)

Selig's volumetric parameters of $B/P_a$ and $\varepsilon_u$ were determined by a hydrostatic compression test. This test was performed using a conventional triaxial cell without applying a load to the specimen. The confining pressure was increased and the cell fluid
volume change was monitored to obtain these parameters. It was found that the slope of the hydrostatic stress-strain curve gave the tangent bulk modulus \(B\). Selig developed Eq. (3.10) to represent the relationship of \(\sigma_m\) and \(\varepsilon_{vol}\).

\[
\sigma_m = \frac{B_i \varepsilon_{vol}}{1 - \left(\frac{\varepsilon_{vol}}{\varepsilon_u}\right)}
\]

(3.10)

where \(B_i = \) initial tangent bulk modulus, \(\varepsilon_u = \) ultimate volumetric strain at large stress.

Taking the derivative of Eq. (3.10) and substituting for \(\varepsilon_{vol}\) from the same equation yields the following. Eq. (3.11), which is Selig’s expression for bulk modulus.

\[
B = B_i \left[1 + \frac{\sigma_m}{(B_i \varepsilon_u)}\right]^2
\]

(3.11)

Since the hydrostatic parameters were estimated from the CANDE - 89 Manual [20], no further explanation of how to determine the bulk modulus will be provided. Further explanation for the determination of the bulk modulus expressions can be found in the CANDE - 89 Manual.

The results shown in Table 3.2 are based on the procedures described previously to determine the Duncan – Selig Hyperbolic stress – strain parameters. Appendix E shows the plots used to determine the Duncan-Selig Hyperbolic Model Parameters for each material. Again, the hydrostatic test parameters were estimated using the default values from the CANDE – 89 Manual. Table 3.3 shows additional soil type parameters
from the default values in the CANDE – 89 Manual that are incorporated into the finite element analysis for a more exact model of each test pipe.
Table 3.2  Duncan – Selig Hyperbolic Stress – Strain Parameter values Used in CANDE – 89 Analysis (Determined From Laboratory Testing).

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>K</th>
<th>n</th>
<th>R_f</th>
<th>C (lb/in²)</th>
<th>( \phi_0 ) (deg)</th>
<th>( \Delta\phi ) (deg)</th>
<th>B_p/P_a</th>
<th>( \varepsilon_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td>CL</td>
<td>95</td>
<td>0.78</td>
<td>0.78</td>
<td>5</td>
<td>17.2</td>
<td>4</td>
<td>25.5</td>
<td>0.121</td>
</tr>
<tr>
<td>Granular Material-Type 2(sand) (90 %)</td>
<td>695</td>
<td>0.43</td>
<td>0.795</td>
<td>0</td>
<td>41.2</td>
<td>4.2</td>
<td>46</td>
<td>0.071</td>
</tr>
<tr>
<td>Granular Material-Type 2(sand) (96 %)</td>
<td>775</td>
<td>0.588</td>
<td>0.632</td>
<td>0</td>
<td>44.8</td>
<td>5.1</td>
<td>187.0</td>
<td>0.014</td>
</tr>
<tr>
<td>Granular Material-Type 1(sand (90 %)</td>
<td>1685</td>
<td>0.145</td>
<td>0.86</td>
<td>8</td>
<td>42.25</td>
<td>7.5</td>
<td>102.0</td>
<td>0.036</td>
</tr>
<tr>
<td>Granular Material-Type 1(sand (96 %)</td>
<td>1713</td>
<td>0.14</td>
<td>0.641</td>
<td>10</td>
<td>46</td>
<td>5.6</td>
<td>187.0</td>
<td>0.014</td>
</tr>
</tbody>
</table>

Table 3.3  Additional Duncan – Selig Hyperbolic Stress – Strain Parameter values Used in CANDE – 89 Analysis (CANDE – 89 Manual Default Values).

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>K</th>
<th>n</th>
<th>R_f</th>
<th>C (lb/in²)</th>
<th>( \phi_0 ) (deg)</th>
<th>( \Delta\phi ) (deg)</th>
<th>B_p/P_a</th>
<th>( \varepsilon_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td>CL 95</td>
<td>120</td>
<td>0.45</td>
<td>1.00</td>
<td>9</td>
<td>15.0</td>
<td>4</td>
<td>53.0</td>
<td>0.092</td>
</tr>
<tr>
<td>SW 61</td>
<td>54</td>
<td>0.85</td>
<td>0.90</td>
<td>0</td>
<td>29.0</td>
<td>0</td>
<td>4.3</td>
<td>0.163</td>
</tr>
<tr>
<td>SW 80</td>
<td>320</td>
<td>0.35</td>
<td>0.83</td>
<td>0</td>
<td>36.0</td>
<td>1</td>
<td>15.3</td>
<td>0.078</td>
</tr>
</tbody>
</table>
CHAPTER 4
EVALUATION OF CURRENT ANALYTICAL METHODS

4.1 INTRODUCTION

In this study, three different analytical methods were applied to predict the structural responses of the thermoplastic pipes placed under deep burial conditions at the ORITE project site. These methods included the finite element computer code CANDE, elastic solutions by Burns and Richard, and the modified Iowa formula by Spangler and Watkins. These methods were chosen because of their continued popularity among the practicing engineers and academicians. Since thermoplastic is made of a viscoelastic material, the time and temperature effects must be considered in the analysis. To determine the effect of the viscoelastic property of the pipe material on the analysis, both the elastic solutions and the modified Iowa formula were applied using the short-term pipe modulus and a long-term modulus that is adjusted to the time and temperature effects. The CANDE analysis was conducted using only the short-term pipe modulus. Section 4.2 discusses the considerations of the time and temperature effects on the pipe modulus, and the following sections describe how each analytical method was applied in the current study.
4.2 TEMPERATURE AND TIME ADJUSTMENTS FOR PIPE MODULUS

Previous research project conducted by the ORITE investigated the variations of the modulus for thermoplastic pipe material with respect to the loading duration and temperature [17]. These results were utilized in the analysis here.

An equation was developed for HDPE pipe material. This was done by combining the temperature vs. instantaneous Young's modulus relationship for HDPE material published by the Plastic Pipe Institute [21] and stress relaxation test results established by the ORITE. The combination yielded the equation below:

$$E(T,t) = 0.85 \left[257,000 - 2,150.5 \times T(\circ F) + 4.800 \times T(\circ F)^2 \right] \times t(min.)^{-8.257E-2} \quad (4.1)$$

Equation (4.1) was also used to determine the short-term pipe modulus that has been corrected for the temperature effect. Table 4.1 lists the computed instantaneous (short-term) Young's modulus and the Young's modulus at the completion of the backfilling process used in the analysis.

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Young's Modulus of Pipe Material (psi):</th>
<th>Average Temperature (\circ F)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Instantaneous*</td>
<td>At Completion of Backfilling**</td>
</tr>
<tr>
<td>13</td>
<td>158,902</td>
<td>62,526</td>
</tr>
<tr>
<td>14</td>
<td>158,994</td>
<td>62,486</td>
</tr>
<tr>
<td>15</td>
<td>158,488</td>
<td>62,111</td>
</tr>
<tr>
<td>16</td>
<td>158,289</td>
<td>61,282</td>
</tr>
<tr>
<td>17</td>
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<td>61,639</td>
</tr>
<tr>
<td>18</td>
<td>157,982</td>
<td>61,657</td>
</tr>
</tbody>
</table>

* Adjusted for temperature only, ** Adjusted for both time and temperature.
4.3  **FINITE ELEMENT ANALYSIS BY CANDE-89**

The CANDE (Culvert ANalysis and DEsign) [20] is a computer program developed by Katona et al. [10] for numerical analysis of buried culvert problems. Level 3 Solution was selected for the numerical analysis of each test pipe, due to the greatest flexibility allowed at this level. In the half-mesh, the pipe was represented by a series of ten 2-node beam elements. The soils were represented by 4-node quadrilateral elements having 131 or 163 nodal points, depending on whether the pipe was buried under the 20-ft. or 40-ft. of cover. The boundary condition along the bottom of the mesh was specified through a series of pins. The boundary condition along each vertical side of the mesh was specified using rollers. The width of the mesh was at least three times the pipe diameter to minimize the effect of the boundary conditions on the pipe responses. No rotation was permitted for the rollers inline with the vertical centerline of the pipe. Without this restriction, plastic hinges would form instantly at the crown and invert of the pipe. No interface elements were inserted into the mesh in the analysis for rapid convergence.

The construction sequence was simulated in eight increments for the 20-ft. cover and twelve increments for the 40-ft. cover. The first four construction increments were needed for the steps taken in the field to achieve the initial backfilling of the pipe. 5-ft. lifts were used to simulate the embankment construction sequence above the pipe. Figure 4.1 shows a typical half-mesh used in the CANDE analysis. The CANDE accepts one value as the pipe modulus at the beginning of the analysis. The instantaneous (constant) pipe moduli shown in Table 4.1 were used in the CANDE finite element analysis. In the
CANDE analysis, the wall moment of inertia is computed from the wall thickness, assuming a smooth wall type. Therefore, the wall thickness was adjusted to correspond to the actual moment of inertia determined from geometrical analysis of the pipe wall section.

Figure 4.1 Typical Half-Mesh Used in CANDE-89 Analysis
The half mesh contained three different soils – bedding/backfill soil, in-situ soil, and embankment fill soil. Behaviors of each soil type were modeled through the Duncan-Selig hyperbolic approach. The hyperbolic model parameter values listed in Tables 3.2 were used for the bedding/backfill soil and embankment fill soil. The in-situ soil was considered to be similar to CL-95 contained in the CANDE default material property library. In the field, it was not possible to compact the backfill soil very well in the haunch area. Depending on the actual field conditions, different material specifications were applied to the backfill material in the haunch zone. Table 4.2 below summarizes the soil types specified as the backfill and in the haunch area in the CANDE simulation of each test pipe.

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Backfill Material</th>
<th>Material Specified in Haunch Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>Granular Material Type 2 (Sand) @ 90% Compaction</td>
<td>Granular Material Type 2 (Sand) @ 86% Compaction</td>
</tr>
<tr>
<td>14</td>
<td>Granular Material Type 2 (Sand) @ 96% Compaction</td>
<td>Granular Material Type 2 (Sand) @ 86% Compaction</td>
</tr>
<tr>
<td>15</td>
<td>Granular Material Type 1 (Crushed Limestone) @ 90% Compaction</td>
<td>Granular Material Type 1 (Crushed Lime-stone) @ 86% Compaction</td>
</tr>
<tr>
<td>16</td>
<td>Granular Material Type 1 (Crushed Limestone) @ 90% Compaction</td>
<td>Granular Material Type 1 (Crushed Lime-stone) @ 86% Compaction</td>
</tr>
<tr>
<td>17</td>
<td>Granular Material Type 1 (Crushed Limestone) @ 96% Compaction</td>
<td>Granular Material Type 1 (Crushed Lime-stone) @ 86% Compaction</td>
</tr>
<tr>
<td>18</td>
<td>Granular Material Type 2 (Sand) @ 96% Compaction</td>
<td>Granular Material Type 2 (Sand) @ 86% Compaction</td>
</tr>
</tbody>
</table>

[Note] “CL” = Crushed Limestone.
4.4 ANALYSIS BY ELASTIC SOLUTIONS OF BURNS AND RICHARD

The one-dimensional compression test results presented in Chapter 3 showed that the modulus of the backfill material was not constant and increased with the magnitude of the overburden pressure (see Table 3.1). To account for this nonlinear stress-strain behavior of the backfill material, each analysis with the Burns and Richard’s elastic solutions [8] was performed in a piecewise linear fashion. The important input parameters for both the soil medium and pipe material were the modulus of elasticity (E) and Poisson’s ratio (ν). Additional input parameters for the pipe included the wall thickness, wall area, and average pipe radius. In the elastic analysis, the wall moment of inertia (I) is computed automatically from the wall thickness (t). Therefore, the wall thickness was adjusted in a manner it was previously done for the CANDE analysis.

To perform realistic analysis, the modulus of elasticity (E) of the pipe material was estimated using Eq. (4.1) for HDPE pipes. To view the effect of the pipe material modulus, the instantaneous pipe modulus was used in the first series of the elastic analysis. And, the pipe modulus at the end of embankment construction was inputted in the second series of the analysis. These pipe modulus values were previously listed in Table 4.1. The surface loading was modeled from the prism-load theory.

The elastic solutions of Burns and Richard for the pipe deflection, circumferential shortening, and the pressure in the soil medium against the pipe at the crown and springline locations are summarized below. The solutions are given under two extreme pipe/soil interface conditions of “free-slip” and “full-bonding.”
In the elastic solutions, general input parameters for the soil medium and the pipe include: Poisson’s ratio and the constrained modulus for the soil ($\nu_s, M_s$), Poisson’s ratio and modulus of the pipe material ($\nu, E$), wall moment of inertia ($I_p$), wall area ($A$), and the average pipe radius ($R$). New shell-medium parameters ($E_p$ and $K$) and other parameters ($B, C, UF$, and $VF$) are then introduced in Eqs. (4.2) through (4.7):

\[ K = \frac{\nu_s}{1 - \nu_s} \quad (4.2) \]
\[ E_p = \frac{E}{1 - \nu^2} \quad (4.3) \]
\[ B = \frac{1 + K}{2} \quad (4.4) \]
\[ C = \frac{1 - K}{2} \quad (4.5) \]
\[ UF = 2B \frac{M_s R}{E_p A} \quad (4.6) \]
\[ VF = 2C \frac{M_s R^3}{6E_p I} \quad (4.7) \]

Where $UF =$ extensional flexibility ratio, and $VF =$ bending flexibility ratio.

For the “full-bonding” condition, there is a continuity of the radial and tangential displacements between the pipe and the soil at the interface. Constants are introduced in Eqs. (4.8) through (4.10):

\[ A_0 = \frac{UF - 1}{UF + \left(\frac{B}{C}\right)} \quad (4.8) \]
\[ A_2 = \frac{C(1 - UF)VF - \left(\frac{C}{B}\right)UF + 2B}{(1 + B)VF + C\left\{VF + \left(\frac{1}{B}\right)\right\}UF + 2 + 2C} \quad (4.9) \]
\[ B_2 = \frac{\{B + C(UF)\}VF - 2B}{(1 + B)VF + C\left\{VF + \left(\frac{1}{B}\right)\right\}UF + 2 + 2C} \quad (4.10) \]
The "full-bonding" interface solutions for the pipe deflection ($w$), circumferential shortening ($cs$), and radial pressure ($\sigma_r$) are provided by Eqs. (4.11) through (4.13):

$$w = \frac{PR}{2Ms} \left[ UF (1 - A0) - VF \left\{ 1 - A2 - 2(B2) \right\} \cos 2\theta \right]$$  \hspace{1cm} (4.11)

$$cs = \frac{2PR}{Ms} \left\{ 1 - A2 + \left( \frac{2C}{B} \right) (B2) \right\}$$  \hspace{1cm} (4.12)

$$\sigma_r = P \left[ B(1 - A0) - C \left\{ 1 - 3(A2) - 4(B2) \right\} \cos 2\theta \right]$$  \hspace{1cm} (4.13)

where $\theta = \text{an angle taken counterclockwise from the right springline} = 0$ for the springline and $\pi/2$ radians for the crown.

The "free-slip" interface solutions are obtained with zero shear stress at the interface instead of compatibility condition of zero relative tangential displacement. Therefore, new non-dimensional constants ($A0^*$, $A2^*$, and $B2^*$) are determined. They are expressed in Eqs. (4.14) through (4.16):

$$A0^* = \frac{UF - 1}{UF + \left( \frac{B}{C} \right)}$$  \hspace{1cm} (4.14)

$$A2^* = \frac{2VF - 1 + \left( \frac{1}{B} \right)}{2VF - 1 + \left( \frac{3}{B} \right)}$$  \hspace{1cm} (4.15)

$$B2^* = \frac{2VF - 1}{2VF - 1 + \left( \frac{3}{B} \right)}$$  \hspace{1cm} (4.16)
The “free-slip” solutions for the pipe deflection (w), circumferential shortening (cs), and radial pressure (σr) are listed below:

\[
\begin{align*}
  w &= \frac{PR}{M_1} \left[ UF \left( 1 - A0^* \right) - \left( \frac{2VF}{3} \right) \left\{ 1 + 3 \left( A2^* \right) - 4 \left( B2^* \right) \right\} \cos 2\theta \right] \\
  cs &= \frac{3PR}{2M_i} \left\{ VF + \left( \frac{C}{2B} \right) UF \right\} \left\{ 1 + 3 \left( A2^* \right) - 4 \left( B2^* \right) \right\} \\
  \sigma_r &= P \left[ B \left( 1 - A0 \right) - C \left\{ 1 + 3 \left( A2 \right) - 4 \left( B2 \right) \right\} \cos 2\theta \right]
\end{align*}
\] (4.17) (4.18) (4.19)

4.5 ANALYSIS BY MODIFIED IOWA FORMULA

In Spangler’s original study from 1941[3], it was stated that as the overburden was placed on the buried pipe, the ratio of the horizontal pressure to the horizontal movement was constant for a given soil. This constant was referred to as passive soil resistance (e). In 1958, Watkins and Spangler began to study the original formula (4.20) dimensionally. Their findings showed that the ratio e, modulus of passive resistance, did not have the correct dimensions to be a modulus nor is it constant for a given soil. They concluded that when e was multiplied by the radius of the pipe (r), the modulus of passive soil resistance became dimensionally correct and appeared approximately constant. This value was referred to as the modulus of soil reaction, \( E' = er \). Hence, the original Iowa formula became the modified Iowa formula given by Eq. (4.21).
\[
\Delta x = \frac{D_LKW_cr^3}{EI + 0.061Er^4}
\] (4.20)

\[
\Delta x = \frac{D_LKW_c r^3}{EI + 0.061E'r^4}
\] (4.21)

Where \(\Delta x\) = change in horizontal diameter, \(W_c\) = Marston’s load per unit length of pipe,

\(D_L\) = deflection lag factor, \(K\) = bedding constant (0.096 for 90 degree bedding angle), \(r\) = mean radius of pipe, \(E\) = modulus of elasticity of the pipe material,

\(I\) = moment of inertia of the pipe wall per unit length, and \(E'\) = modulus of soil reaction.

\(E'\) values estimated from one-dimensional compression test, Table 3.1. \(E\) values estimated from Eq. 4.1, Table 4.1. \(I\) values are listed in Table 2.4. The deflection lag factor takes into account the consolidation of the soil located on the sides of the pipe. This consolidation causes the pipe structure to continue to deflect over time. It was recommended by Spangler to use a deflection lag factor of 1.5 for design when using Marston’s load, which may under estimates the actual load on the pipe. The calculations performed in this study incorporated the prism-load theory for simplicity. Research at Utah State University [11] has indicated that the load on a flexible pipe may approach the prism load over time. Therefore, when using the prism load, the deflection lag factor can be set equal to 1.0.

The modified Iowa formula can easily be adjusted to accept the prism-load theory. Substituting Eqs. (4.22) – (4.24) below into Eq. (4.21) results in an alternative
expression of the modified Iowa formula. Eq. (4.23) comes from the elastic theory of a loaded ring.

\[ W_e = 2rP \quad (4.22) \]

\[ EI = 0.149(PS)r^3 \quad (4.23) \]

\[ \frac{\Delta x}{d} (\%) = \frac{100D_iKP}{0.149(PS) + 0.061E'} \quad (4.24) \]

Where, \( d \) = pipe diameter (in.) and \( \frac{\Delta x}{d} (\%) \) = percent horizontal deflection.

\[ PS = \frac{6.71EI}{r^3} \quad (4.25) \]

Where, \( P \) = pressure (psi), \( r \) = pipe radius (in.), and \( PS \) = pipe stiffness (lb/in/in) = \( P/\Delta x \).

The pipe stiffness can either be measured from the parallel plate test (ASTM D-2412) or estimated from Eq. (4.25). The alternative form of the modified Iowa formula is shown in Eq. (4.24).
CHAPTER 5

RESULTS OF FIELD TESTING AND ANALYTICAL METHODS

5.1 INTRODUCTION

The field structural performances of six HDPE pipes (Test Pipes 13-18) buried under different conditions were compared to the predictions from three analytical methods: the finite element computer code (CANDE-89), the elastic solutions of Burns and Richard, and the modified Iowa formula. A description of these analytical methods has been given in Chapter 4. Each pipe was monitored for its deflection profile, circumferential shortening, and pressure distribution around the pipe.

5.2 FIELD PERFORMANCE OF HDPE PIPES

The following sections discuss the results of the structural performance of each pipe.

5.2.1 Test Pipe 13

Test Pipe 13 was a 42-in. nominal diameter, corrugated HDPE pipe, whose profile wall design (see Table 2.1). Its initial horizontal and vertical inside diameters were 41.28 in. and 40.92 in., respectively. It was backfilled with the ODOT Granular Material - Type 2 (sand) having 92.7% average relative compaction ($\gamma_{dry} = 111.7 \text{pcf}$). The pipe was subjected to 20.5 ft. of soil fill placed over its 12-in. thick backfill soil cover. Figures in Appendix C present soil pressure and deflection data recorded for test Pipe 13 with respect to the elapsed time.
The pipe exhibited a minor peaking behavior during the initial backfilling. This is evidenced by the fact that the vertical diameter increased by about 0.33 in. (0.8%), and the horizontal diameter decreased by 0.21 in. (- 0.5%). At the end of construction, the vertical and horizontal deflection values were – 0.63 in. and 0.1 in. (or – 1.54% and 0.24%), respectively. And, during the subsequent monitoring period, the pipe deflections increased slightly to – 0.88 in. (- 2.14%) vertical and 0.25 in. (0.61%) horizontal. No circumferential shortening data were available for this pipe.

When the final fill height was reached, the measured pressures were about 8.5 psi both at the crown and 11.3 psi at the springline. This crown pressure value represented 46.25% of the estimated geostatic pressure (18.4 psi). Both the crown and springline pressures decreased continuously during the course of the monitoring period. At the end of the monitoring period, the crown and springline pressures were about 7.8 psi (42.44% of the geostatic pressure) and 8.7 psi, respectively.

5.2.2 Test Pipe 14

Test Pipe 14 was also the 42-in. nominal diameter, corrugated HDPE pipe (see Table 2.1). Its initial horizontal and vertical inside diameters were 40.94 in. and 41.56 in., respectively. It was backfilled with the ODOT Granular Material - Type 2 (sand) having 94.9% average relative compaction ($\gamma_{\text{dry}} = 114.4$ pcf). The pipe was subjected to 40.0 ft. of soil fill placed over its 12-in. thick backfill soil cover. Figures in Appendix C present soil pressure and deflection data recorded for test Pipe 14 with respect to the elapsed time.
The pipe exhibited a minor peaking behavior during the initial backfilling. This is evidenced by the fact that the vertical diameter increased by about 0.36 in. (-0.9%), and the horizontal diameter decreased by 0.20 in. (0.5%). At the end of construction, the vertical and horizontal deflection values were -0.98 in. and 0.28 in. (or -2.36% and 0.68%), respectively. And, during the monitoring period, these values fluctuated somewhat but remained about the same. The pipe experienced 0.72 in. of shortening by the end of the construction, which corresponded to about 0.56% reduction in its circumferential length. During the subsequent monitoring period, the circumferential shortening increased to the steady value of about -0.9 in. (-0.7%). Overall, test Pipe 14 deformed very little under the 40-ft. soil cover.

Under the final fill height, the measured pressures were about 14.5 psi at the crown and 12 psi at the springline. This crown pressure value represented 40.3% of the estimated geostatic pressure (36 psi). Both the crown and springline pressure measurements showed a tendency to decline over time beyond the end of construction. Their values stabilized finally after about 140 days under the embankment loading. At the end of the monitoring period, the crown and springline pressures were 13.4 psi (37.22% of the geostatic pressure) and 9.5 psi, respectively.

5.2.3 Test Pipe 15

Test Pipe 15 was also the 42-in. nominal diameter, corrugated HDPE pipe (see Table 2.1). Its initial horizontal and vertical inside diameters were 41.36 in. and 41.01 in., respectively. It was backfilled with the ODOT Granular Material - Type 1 (crushed
limestone) having 89.7% average relative compaction ($\gamma_{\text{dry}} = 126.1$ pcf). The pipe was subjected to 20.0 ft. of soil fill placed over its 12-in. thick backfill soil cover. Figures in Appendix C present soil pressure and deflection data recorded for test Pipe 15 with respect to the elapsed time.

The pipe exhibited a minor peaking behavior during the initial backfilling. This is evidenced by the fact that the vertical diameter increased by about 0.17 in. (-0.4%), and the horizontal diameter decreased by 0.26 in. (-0.6%). At the end of construction, the vertical and horizontal deflection values were -0.54 in. and -0.13 in. (or -1.32% and -0.31%), respectively. And, during the monitoring period, these values increased slightly to -0.68 in. (-1.65%) vertical and 0.18 in. (0.44%) horizontal. No circumferential or strain gage data were available for Test Pipe.

Under the final fill height, the measured pressures were about 8.0 psi at the crown and 9.2 psi at the springline. This crown pressure value represented 44.4% of the estimated geostatic pressure (18 psi). Both the crown and springline pressure measurements showed a tendency to decline slightly over time before increasing at the end of the monitoring period. This increase in their values near the end of the study was a result of a few-week long rainy weather that the site area experienced. At the end of the field monitoring, the crown and springline pressures were both about 9.6 psi (53.3% of the geostatic pressure).

5.2.4 Test Pipe 16

Test Pipe 16 was a 60-in. diameter, honey-comb design HDPE pipe (see Table
2.1). It was backfilled with the ODOT Granular Material - Type 1 (crushed limestone) having 90.1% average relative compaction ($\gamma_{dry} = 126.6$ pcf). The pipe was subjected to 20.1 ft. of soil fill placed over its 12-in. thick backfill soil cover. Appendix C present soil pressure and deflection data recorded for test Pipe 16 with respect to the elapsed time.

The pipe exhibited a moderate peaking behavior during the initial backfilling. This is evidenced by the fact that the vertical diameter increased by about 1.2 in. (2.0%), and the horizontal diameter decreased by 1.0 in. (-1.7%). At the end of construction, the vertical and horizontal deflection values were approximately -0.63 in. and 0.14 in. (or -1.05% and 0.23%), respectively. During the monitoring period, the vertical deflection increased slightly to -1.25 in. (-2.09%). The horizontal diameter changed little beyond the end of construction.

Under the final fill height, the measured pressures were about 8.6 psi at the crown and 14.6 psi at the springline. This crown pressure value represented 47.5% of the estimated geostatic pressure (18.2 psi). Both the crown and springline pressure measurements first decreased and then increased back again before the end of the monitoring period. The increase in the measured soil pressure values near the end of the study was a result of a few-week long rainy weather that the site area experienced. At the end of the field monitoring, the crown and springline pressures were about 10 psi (57.1% of the geostatic pressure) and 15.6 psi, respectively.

5.2.5 Pipe 17

Test Pipe 17 was also the 60-in. diameter, honey-comb design HDPE pipe (see
Table 2.1). It was backfilled with the ODOT Granular Material - Type 1 (crushed limestone) having 95.6% average relative compaction ($\gamma_{dry} = 134.4$ pcf). During the backfilling process, a large tension crack was seen running through the trench wall. A storm event was believed to have destabilized the wall. The pipe was subjected to 40.0 ft. of soil fill placed over its 12-in. thick backfill soil cover. Figures in Appendix C present soil pressure and deflection data recorded for test Pipe 17 with respect to the elapsed time.

The pipe exhibited a minor peaking behavior during the initial backfilling. This is evidenced by the fact that the vertical diameter increased by about 1.2 in. (2 %), and the horizontal diameter decreased by 1.0 in. (- 1.7 %). At the end of construction, the vertical and horizontal deflection values were approximately – 3.05 in. and 0.76 in. (or – 5.08% and 1.27%), respectively. And, during the monitoring period, the vertical deflection increased slightly to – 3.58 in. (- 5.96%). The horizontal diameter changed little beyond the end of construction.

The pipe elongated circumferentially by about 0.8 in. due to the initial backfilling. While the embankment was being built, the circumferential length shortened by 1.9 in. compared to the original value, which corresponded to about 1% reduction in its circumferential length. During the subsequent monitoring period, the circumferential shortening kept increasing at a slow rate. The final reading at the end of the study was close to – 3.0 in. (- 1.6%).

Under the final fill height, the measured pressures were about 13.5 psi at the crown and 20.0 psi at the springline. This crown pressure value represented 37.4% of the
estimated geostatic pressure (36.1 psi). Both the crown and springline pressure measurements declined quickly following the end of construction and then remained relatively unchanged until the end of the monitoring period. At the end of the field monitoring, the crown and springline pressures were about 12.4 psi (34.3% of the geostatic pressure) and 17.0 psi, respectively.

5.2.6 Pipe 18

Test Pipe 18 was also the 60-in. diameter, honey-comb design HDPE pipe (see Table 2.1). It was backfilled with the ODOT Granular Material - Type 2 (sand) having 94.3% average relative compaction (γ_dry = 113.6 pcf). The pipe was subjected to 19.7 ft. of soil fill placed over its 12-in. thick backfill soil cover. Figures in Appendix C present soil pressure and deflection data recorded for test Pipe 18 with respect to the elapsed time.

The pipe exhibited a moderate peaking behavior during the initial backfilling. This is evidenced by the fact that the vertical diameter increased by about 0.6 in. (1.0%), and the horizontal diameter decreased by 0.57 in. (-1%). At the end of construction, the vertical and horizontal deflection values were approximately 0.51 in. and 0.058 in. (or – 0.85% and 0.1%), respectively. And, during the monitoring period, the vertical deflection was about –0.55 in. (0.92%). The horizontal diameter changed little beyond the end of construction. The loading from the 20-ft. high soil fill was just sufficient to reduce the vertical diameter to the original value but not sufficient to increase the horizontal diameter back to at least the initial value. No circumferential shortening and strain gage data were available from Test Pipe 18.
Under the final fill height, the measured pressures were about 6 psi at the crown and 5.5 psi at the springline. This crown pressure value represented 34.3\% of the estimated geostatic pressure (17.7 psi). Both the crown and springline pressure measurements increased before the end of construction. This increase in their values near the end of the study was a direct result of a few-week long rainy weather that the site area experienced. At the end of the field monitoring, the crown and springline pressures were about 8.1 psi (45.8\% of the geostatic pressure) and 6.8 psi, respectively.

5.3 FURTHER DISCUSSIONS ON FIELD PERFORMANCE DATA

Backfill placement of HDPE pipes ranged from 56 to 76 days depending on the construction sequence. The performance of HDPE pipes is time dependant therefore; their performance is continuously monitored during and after the construction. Tables 5.1 through 5.3 summarize the pipe deflections and circumferential shortening data for all the test pipes at the end of embankment construction, 3 months beyond the end of construction, and at the end of monitoring period. Also, Tables 5.4 through 5.5 summarize soil pressure data for all test pipes at the end of embankment construction, 3 months beyond, and at the end of monitoring period. The deflection and pressure time history plots for each pipe are provided in Appendix C.
### Table 5.1  Time-Dependent Vertical Deflection Results

<table>
<thead>
<tr>
<th>Test Pipe No.</th>
<th></th>
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<th>Ratio</th>
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<tr>
<td></td>
<td>Vertical (%)</td>
<td></td>
<td></td>
<td></td>
<td>B/A</td>
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<td>End of Construction (A)</td>
<td>Three Months Later (B)</td>
<td>End of Monitor Period (C)</td>
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<tr>
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<td>Average</td>
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<td>1.32</td>
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</table>

### Table 5.2  Time-Dependent Horizontal Deflection Results

<table>
<thead>
<tr>
<th>Test Pipe No.</th>
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<th>Ratio</th>
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<td>Horizontal (%)</td>
<td></td>
<td></td>
<td></td>
<td>B/A</td>
</tr>
<tr>
<td></td>
<td>End of Construction (A)</td>
<td>Three Months Later (B)</td>
<td>End of Monitor Period (C)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
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<td>0.61</td>
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<td>0.10</td>
<td>0.03</td>
<td>-0.03</td>
<td>0.3</td>
<td>-1.0</td>
</tr>
<tr>
<td>Average</td>
<td>1.23</td>
<td>0.97</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 5.3  Time-Dependent Circumferential Deflection Results

<table>
<thead>
<tr>
<th>Test Pipe No.</th>
<th>Circumferential (%)</th>
<th>Ratio</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>End of Construction (A)</td>
<td>Three Months Later (B)</td>
<td>End of Monitor Period (C)</td>
</tr>
<tr>
<td>13</td>
<td>..................</td>
<td>................</td>
<td>................</td>
</tr>
<tr>
<td>14</td>
<td>-0.72</td>
<td>-0.72</td>
<td>-0.70</td>
</tr>
<tr>
<td>15</td>
<td>..................</td>
<td>................</td>
<td>................</td>
</tr>
<tr>
<td>16</td>
<td>..................</td>
<td>................</td>
<td>................</td>
</tr>
<tr>
<td>17</td>
<td>-1.97</td>
<td>-1.43</td>
<td>-1.60</td>
</tr>
<tr>
<td>18</td>
<td>..................</td>
<td>................</td>
<td>................</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.4  Time-Dependent Crown Pressure Results

<table>
<thead>
<tr>
<th>Test Pipe No.</th>
<th>Crown Pressure (psi)</th>
<th>Ratio</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>End of Construction (A)</td>
<td>Three Months Later (B)</td>
<td>End of Monitor Period (C)</td>
</tr>
<tr>
<td>13</td>
<td>8.5</td>
<td>8.39</td>
<td>7.8</td>
</tr>
<tr>
<td>14</td>
<td>14.5</td>
<td>13.46</td>
<td>13.4</td>
</tr>
<tr>
<td>15</td>
<td>8.0</td>
<td>8.85</td>
<td>9.6</td>
</tr>
<tr>
<td>16</td>
<td>8.6</td>
<td>9.9</td>
<td>10</td>
</tr>
<tr>
<td>17</td>
<td>13.5</td>
<td>12.74</td>
<td>12.4</td>
</tr>
<tr>
<td>18</td>
<td>6</td>
<td>6.14</td>
<td>8.1</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 5.5  Time-Dependent Springline Pressure Results

<table>
<thead>
<tr>
<th>Test Pipe No.</th>
<th>Springline pressure (psi)</th>
<th>Ratio</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>End of Construction (A)</td>
<td>Three Months Later (B)</td>
<td>End of Monitor Period (C)</td>
</tr>
<tr>
<td>13</td>
<td>11.3</td>
<td>9.7</td>
<td>8.7</td>
</tr>
<tr>
<td>14</td>
<td>12</td>
<td>9.1</td>
<td>9.5</td>
</tr>
<tr>
<td>15</td>
<td>9.2</td>
<td>8.7</td>
<td>9.6</td>
</tr>
<tr>
<td>16</td>
<td>14.6</td>
<td>15.45</td>
<td>16</td>
</tr>
<tr>
<td>17</td>
<td>20</td>
<td>18.4</td>
<td>17</td>
</tr>
<tr>
<td>18</td>
<td>5.5</td>
<td>5.5</td>
<td>6.8</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Based on these tables it can be stated that:

* Relative compaction initially achieved on the backfill soil had a significant effect on both the short-term and long-term and long-term deflections of the pipes. The higher the initial relative compaction was, the smaller the pipe deflections remained. Also, the pipe experienced less circumferential shortening when surrounded by a denser backfill material.

* The value of deflection lag factor may be estimated by dividing the vertical deflection at the end of the monitoring period by the vertical deflection at the end of construction. Deflection lag factor varied between 1.0 and 1.3. No clear trend existed between the value of deflection lag factor and the relative compaction initially achieved on the backfill soil. Deflection lag factor was larger in the crushed limestone backfill soil than in the sandy.
* Deflection ratio $|\Delta V/\Delta H|$ appears to be slightly larger for pipes installed in denser backfill.

* The pipe experienced a larger circumferential shortening in the sandy backfill soil than in crushed limestone backfill.

* Relative compaction initially achieved on the backfill soil had measurable effects on the soil pressure distribution around the buried pipes. The higher the initial relative compaction was, the smaller the vertical pressure at the crown and lateral pressures at the springline were.

* The above point didn’t hold true when the HDPE were installed in the crushed limestone. This may be due to the fact that the compaction of the coarse backfill material left a relatively loose zone against the corrugated surface of the HDPE pipes regardless of the intended relative compaction.

* No clear correlation existed between the relative compaction achieved on the backfill soil and the value of soil pressure ratio (Crown/Springline).

* Time dependant change of the vertical pressure measured at the crown offered no clear trend with the relative compaction initially achieved on the backfill soil.

* The soil pressure ratio (Crown/Spring) had a tendency to be larger in the sandy backfill soil than in the crushed limestone material.

* The vertical soil pressure measured at the crown of the HDPE pipes had a tendency to decrease slightly overtime.
5.4 COMPARISONS BETWEEN FIELD AND ANALYTICAL RESULTS

In the following sections, the structural performance of the HDPE pipes predicted by the three analytical methods (finite element computer code, CANDE-89; elastic solutions of Burns and Richard; and the modified Iowa formula) are evaluated in light of the field performance data obtained in the study. Additional results from the analyses carried out by the three methods can be found in the appendices.

5.4.1 Evaluation of Pipe Performance Predicted by CANDE-89

Section 4.3 described some details on how the field installation conditions were numerically simulated for each test pipe, using the CANDE-89 program. Short-term modulus corrected for the temperature effect was used for the pipe material. Figure 5.1 presents a deflection comparison between the field data and the CANDE results for Test Pipe 14. The CANDE deflections were generally less than the actual deflections in the horizontal and vertical direction. Similar plots for the other test pipes are contained in the appendices.

Next, Figures 5.2 and 5.3 compare graphically the soil pressures between the field data and the CANDE analysis results for Test Pipe 14. Figure 5.2 shows that the CANDE simulation predicted the vertical soil pressure at the crown closely over a wide range of fill height. However, the lateral soil pressure at the springline was increasingly underestimated by the CANDE simulation (see Figure 5.3).
Figure 5.1  Deflection Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 14

Figure 5.2  Crown Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 14
Tables 5.6 and 5.7 summarize the comparisons between the field performance data and the CANDE results for the test pipes. According to Table 5.6, the vertical deflection was consistently underestimated by the analysis. The overall average value of the vertical deflection ratio of |Field/Predicted| ratio is about 2.065. The CANDE underestimated the horizontal deflections even more than the vertical deflections. The average value of the |Field/Predicted| ratio for the horizontal deflection was 7.057. As far as the soil pressure is concerned, both the vertical pressure at the crown and the lateral pressure at the springline were slightly overpredicted in the CANDE simulations (see Table 5.7). The average value of (Field/CANDE) ratio is 0.63 for the crown pressure and 0.58 for the springline pressure.

![Figure 5.3](image_url)

**Figure 5.3** Springline Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 14
Table 5.6  Comparison of Pipe Deflections Between Field Data and CANDE Simulation Results at the End of Embankment Construction

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Pipe Deflection (%)*</th>
<th>Field Vertical</th>
<th>Field Horizontal</th>
<th>CANDE Vertical</th>
<th>CANDE Horizontal</th>
<th></th>
<th></th>
<th>Field/CANDE Ratio Vertical</th>
<th>Field/CANDE Ratio Horizontal</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>-1.21</td>
<td>0.43</td>
<td>-1.49</td>
<td>0.37</td>
<td>0.81</td>
<td>1.16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>-1.94</td>
<td>0.69</td>
<td>-1.61</td>
<td>0.15</td>
<td>1.20</td>
<td>4.60</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>-1.24</td>
<td>0.36</td>
<td>-0.46</td>
<td>0.016</td>
<td>2.70</td>
<td>22.50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>-1.00</td>
<td>0.25</td>
<td>-0.49</td>
<td>0.035</td>
<td>2.04</td>
<td>7.14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>-5.1</td>
<td>1.2</td>
<td>-1.07</td>
<td>0.18</td>
<td>4.77</td>
<td>6.67</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>-0.85</td>
<td>0.097</td>
<td>-0.98</td>
<td>0.36</td>
<td>0.87</td>
<td>0.27</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Average Ratio = 2.065 7.057

[Notes]  * Peaking Deflection included

All factor of safety values for elastic buckling were over 10 in the CANDE analyses.

Table 5.7  Comparison of Soil Pressure Acting Against Pipe Between Field Data and CANDE Simulation Results at the End of Embankment Construction

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Soil Pressure (psi)</th>
<th>Field Crown</th>
<th>Field Springline</th>
<th>CANDE Crown</th>
<th>CANDE Springline</th>
<th></th>
<th></th>
<th>Field/CANDE Ratio Crown</th>
<th>Field/CANDE Ratio Springline</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>8.45</td>
<td>11.03</td>
<td>17.1</td>
<td>13.4</td>
<td>0.49</td>
<td>0.82</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>14.45</td>
<td>8.53</td>
<td>29.2</td>
<td>30.1</td>
<td>0.49</td>
<td>0.28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>8.55</td>
<td>7.29</td>
<td>12.2</td>
<td>18.1</td>
<td>0.70</td>
<td>0.40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>8.91</td>
<td>14.5</td>
<td>9.63</td>
<td>15.8</td>
<td>0.93</td>
<td>0.92</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>13.68</td>
<td>20.3</td>
<td>18.5</td>
<td>27.4</td>
<td>0.74</td>
<td>0.74</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>5.75</td>
<td>5.0</td>
<td>12.9</td>
<td>15.9</td>
<td>0.45</td>
<td>0.31</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Average Ratio = 0.63 0.58

[Notes]  * Peaking Deflection included

All factor of safety values for elastic buckling were over 10 in the CANDE analyses.
5.4.2 Evaluation of Pipe Performance Predicted by Elastic Solutions

Section 4.4 summarized the elastic solutions of Burns and Richard and how the values of the key input parameters were determined. Figure 5.4 presents a deflection comparison between the field data and the Elastic Solutions results for Test Pipe 16.

![Figure 5.4](image_url)  
**Figure 5.4** Deflection Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 16

Figure 5.5 compares the circumferential shortening experienced by Test Pipe 14 between the field data and the elastic solutions. The actual circumferential shortening was less than the theoretical values. The full-bond interface solution overpredicted the actual pipe shortening behavior less than the free-slip interface solution.
Next, Figures 5.6 and 5.7 compare the soil pressure measured in the field and predicted by the elastic solutions for Test Pipes 13 and 14. Figure 5.6 suggests that the vertical soil pressure acting at the crown of Test Pipe 13 was initially estimated more closely by the free-slip solution but became similar to the full-bond interface value as the fill height increased (and as more time elapsed). Figure 5.7 shows a similar comparison made for Test Pipe 14 in terms of the lateral pressure at the springline. The elastic solutions overestimated the soil pressure. Similar plots comparing the field pipe behaviors and the predictions made by the elastic solutions for the other test pipes can be found in the appendices.
Figure 5.6  Crown Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 13

Figure 5.7  Springline Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 14
Tables 5.8 through 5.11 summarize the comparisons between the field performance data and the elastic solutions for all the test pipes. According to Tables 5.8 and 5.9, the vertical deflection at the end of the embankment construction was estimated well by the elastic method. The vertical deflection ratio of \(|\text{Field/Predicted}|\) was on the average 0.76 with the full-bond interface solution and 0.93 with the free-slip interface solution. The ratio of \(|\text{Field/Predicted}|\) for the horizontal deflection was on the average 1.2 with the full-bond interface solution and 1.9 with the free-slip interface solution. The circumferential shortening behavior was estimated reasonably well by the full-bond interface solution. As far as the soil pressure is concerned (refer to Tables 5.7 and 5.8), the full-bond interface solution predicted the vertical soil pressure at the crown more accurately than the free-slip interface solution. On the contrary, the lateral soil pressure measured at the springline was more closely estimated by the free-slip solution.

5.4.3 Evaluation of Horizontal Pipe Deflections Predicted by Modified Iowa Formula

Section 4.5 summarized the development of the modified Iowa formula and described how the values for some of its key input parameters were determined. Figure 5.8 presents a horizontal deflection comparison between the field data and the modified Iowa formula for Test Pipe 16. Similar plots for the other test pipes are attached in the appendices. Table 5.12 summarizes the comparison between the field horizontal deflection data and the modified Iowa formula results for all the test pipes. It can be seen easily that the modified Iowa formula consistently overestimated the actual horizontal deflections of the field pipes. The average value of (Field/Predicted) is 0.33.
Figure 5.8  Horizontal Deflection Comparisons Between Experimental Data and Modified Iowa Formula for Test Pipe 16
Table 5.8  Comparison of Pipe Deflections and Circumferential Shortening Between the Field Data and Full-Bond Interface Elastic Solution (Using Adjusted Pipe Modulus) at the End of Embankment Construction

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Field Data (%)*</th>
<th>Full-Bond (%)</th>
<th>[Field/Elastic] Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical</td>
<td>Horizontal</td>
<td>Circum.</td>
</tr>
<tr>
<td>13</td>
<td>-1.21</td>
<td>0.43</td>
<td>.....</td>
</tr>
<tr>
<td>14</td>
<td>-1.94</td>
<td>0.69</td>
<td>-0.55</td>
</tr>
<tr>
<td>15</td>
<td>-1.24</td>
<td>0.36</td>
<td>.....</td>
</tr>
<tr>
<td>16</td>
<td>-1.00</td>
<td>0.25</td>
<td>.....</td>
</tr>
<tr>
<td>17</td>
<td>-5.1</td>
<td>1.2</td>
<td>-1.045</td>
</tr>
<tr>
<td>18</td>
<td>-0.85</td>
<td>0.097</td>
<td>.....</td>
</tr>
</tbody>
</table>

[Note]  * Peaking Deflection included

Average Ratio = 0.92 1.37 0.58
## Table 5.9 Comparison of Pipe Deflections and Circumferential Shortening Between the Field Data and Free-Slip Interface Elastic Solution (Using Adjusted Pipe Modulus) at the End of Embankment Construction

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Field Data (%)*</th>
<th>Free-Slip (%)</th>
<th>[Field/Elastic] Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical</td>
<td>Horizontal</td>
<td>Circum.</td>
</tr>
<tr>
<td>13</td>
<td>-1.21</td>
<td>0.43</td>
<td>....</td>
</tr>
<tr>
<td>14</td>
<td>-1.94</td>
<td>0.69</td>
<td>-0.55</td>
</tr>
<tr>
<td>15</td>
<td>-1.24</td>
<td>0.36</td>
<td>....</td>
</tr>
<tr>
<td>16</td>
<td>-1.00</td>
<td>0.25</td>
<td>....</td>
</tr>
<tr>
<td>17</td>
<td>-5.1</td>
<td>1.2</td>
<td>-1.045</td>
</tr>
<tr>
<td>18</td>
<td>-0.85</td>
<td>0.097</td>
<td>....</td>
</tr>
</tbody>
</table>

Average Ratio = 0.90 1.36 0.35  

[Note] * Peaking Deflection included
Table 5.10  Comparison of Soil Pressure Between Field Data and Full-Bond Interface Elastic Solutions (Using Adjusted Pipe Modulus) at the End of Construction

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Field Data (psi)</th>
<th>Full-Bond (psi)</th>
<th>[Field/Elastic] Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Crown</td>
<td>Springline</td>
<td>Crown</td>
</tr>
<tr>
<td>13</td>
<td>8.45</td>
<td>11.03</td>
<td>7.93</td>
</tr>
<tr>
<td>14</td>
<td>14.45</td>
<td>8.53</td>
<td>14.91</td>
</tr>
<tr>
<td>15</td>
<td>8.55</td>
<td>7.29</td>
<td>7.74</td>
</tr>
<tr>
<td>16</td>
<td>8.91</td>
<td>14.5</td>
<td>7.23</td>
</tr>
<tr>
<td>17</td>
<td>13.68</td>
<td>20.3</td>
<td>10.07</td>
</tr>
<tr>
<td>18</td>
<td>5.75</td>
<td>5.0</td>
<td>7.67</td>
</tr>
</tbody>
</table>

Average Ratio = 1.06  0.57
Table 5.11  Comparison of Soil Pressure Between Field Data and Free-Slip Interface Elastic Solutions (Using Adjusted Pipe Modulus) at the End of Construction

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Field Data (psi)</th>
<th>Free-Slip (psi)</th>
<th>[Field/Elastic] Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Crown</td>
<td>Springline</td>
<td>Crown</td>
</tr>
<tr>
<td>13</td>
<td>8.45</td>
<td>11.03</td>
<td>11.67</td>
</tr>
<tr>
<td></td>
<td>14.45</td>
<td>8.53</td>
<td>22.1</td>
</tr>
<tr>
<td>14</td>
<td>8.55</td>
<td>7.29</td>
<td>11.4</td>
</tr>
<tr>
<td>15</td>
<td>8.91</td>
<td>14.5</td>
<td>10.74</td>
</tr>
<tr>
<td>16</td>
<td>13.68</td>
<td>20.3</td>
<td>15.82</td>
</tr>
<tr>
<td>17</td>
<td>5.75</td>
<td>5.0</td>
<td>11.29</td>
</tr>
<tr>
<td>18</td>
<td>0.72</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.65</td>
<td>0.42</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>0.69</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.83</td>
<td>1.40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.86</td>
<td>1.32</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.51</td>
<td>0.46</td>
<td></td>
</tr>
</tbody>
</table>

Average Ratio == 0.67  0.80

Table 5.12  Comparison of Horizontal Deflection Between Field Data and Modified Iowa Formula (Using Adjusted Pipe Modulus) at the End of Construction

<table>
<thead>
<tr>
<th>Test Pipe</th>
<th>Field (%)</th>
<th>Modified Iowa (%)</th>
<th>Deflection Ratio of (Field/Predicted)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>0.43</td>
<td>1.3</td>
<td>0.33</td>
</tr>
<tr>
<td>14</td>
<td>0.69</td>
<td>2.98</td>
<td>0.23</td>
</tr>
<tr>
<td>15</td>
<td>0.36</td>
<td>1.3</td>
<td>0.28</td>
</tr>
<tr>
<td>16</td>
<td>0.25</td>
<td>1.22</td>
<td>0.21</td>
</tr>
<tr>
<td>17</td>
<td>1.2</td>
<td>1.42</td>
<td>0.85</td>
</tr>
<tr>
<td>18</td>
<td>0.097</td>
<td>1.41</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Average Ratio == 0.49

84
CHAPTER 6
SUMMARY AND CONCLUSIONS

6.1 SUMMARY

This study evaluated the structural performance of the thermoplastic pipe under actual field burial conditions during and beyond the construction process and over the long-term. The objectives of the current study were:

- Measure the deflection profile and circumferential shortening for the pipes subjected to deep burial,
- Measure the pressure distribution around the pipes subjected to deep burial,
- Measure the pipe response as a function of time
- Compare field performance data with the analytical results from the finite element computer code CANDE-89, the elastic solutions of Burns and Richard, and the modified Iowa formula.

A total of eighteen (6 PVC and 12 HDPE) pipes, ranging in nominal diameter from 30 to 60 inches (762 to 1,524 mm), were instrumented with sensors, backfilled with granular backfill soils, and then subjected to overburden pressure from a either 20 or 40 ft. high soil fill. The sensors were applied to measure horizontal and vertical pipe deflections, circumferential shortening, profile changes, strains in pipe wall, soil pressure acting against pipe, and vertical pressure at various fill heights above selected pipes. Effects of some variables on the field pipe performance were considered in the project, which included the pipe material type, pipe diameter, type and relative compaction of the
backfill soil, height of soil fill, duration of time, and bedding layer thickness. Field performances of the pipes were monitored for 540 days, between the late January 2000 and June 2001.

Detailed review of the field pipe performance data yielded numerous conclusions concerning the structural performance of these thermoplastic pipes under relatively deep soil fill. In addition, reliability of three analytical methods (CANDE finite element analysis, elastic solutions, and modified Iowa formula) was evaluated in the light of the latest field performance data. A comprehensive soil testing was conducted previously in the laboratory prior to the analyses to obtain properties of the backfill and embankment fill soils that were needed by the analytical methods.

6.2 CONCLUSIONS

1. In most cases, the field behavior of the thermoplastic pipes stabilized within 3 months beyond the end of construction.

2. Horizontal deflections experienced by the test pipes were mostly less than 1.5%.

3. Circumferential shortening experienced by the test pipes was small, ranging from about 0.1% to 1%.

4. Minor peaking behavior, represented typically by 0.3% to 0.8% vertical diameter increase and 0.2% to 0.6% horizontal diameter decrease, was observed during the initial backfilling of the pipes in the field.

5. Vertical soil pressure measured at the pipe crown ranged from 6 to 14 psi under the 20-ft. soil fill. The crown pressure ranged from 11 to 25 psi under the 40-ft.
soil fill.

6. Lateral soil pressure measured at the pipe springline ranged from 6 to 17 psi under the 20-ft. soil fill. The springline pressure ranged from 9 to 25 psi under the 40-ft. soil fill.

7. Relative compaction initially achieved on the backfill soil had a significant effect on both the short-term and long-term deflections of the thermoplastic pipes. The higher the initial relative compaction was, the smaller the pipe deflections remained. Also, the pipe experienced less circumferential shortening when surrounded by a denser backfill material.

8. Deflection ratio $|\Delta V/\Delta H|$ appears to be slightly larger for thermoplastic pipes installed in a denser backfill material.

9. Deflection lag factor was larger in the crushed limestone backfill soil than in the sandy backfill soil. This is believed to be due to more fines contained in the crushed limestone soil.

10. The pipe experienced a larger circumferential shortening in the sandy backfill soil than in the crushed limestone backfill.

11. The CANDE finite element computer analysis (with the instantaneous pipe material modulus and a combination of hyperbolic soil parameters determined in the laboratory and from the literature) was not successful in simulating accurately the field performance of the thermoplastic pipes. It had a tendency to underpredict the pipe deflections and overpredict the soil pressure acting against the pipe. This may be partially due to the use of the short-term pipe material
modulus throughout the CANDE analysis. It did have some degree of success in the field performance of the PVC pipes relatively more closely than the HDPE pipes.

12. The modified Iowa formula (with the pipe material modulus adjusted to time and temperature effects and the soil modulus measured in the laboratory) overpredicted the horizontal deflections of the field test pipes by a factor of 2. The use of the prism load theory and a crude approach in addressing the soil-pipe interaction phenomena might be the reason for this outcome.

13. The elastic solutions of Burns and Richard, (with the prism load theory, the pipe material modulus adjusted to time and temperature effects, and the laboratory-measured soil modulus entered in an incremental fashion) were by far the most promising analytical tool among the three methods for predicting the field performance of the thermoplastic pipes.

14. The elastic solutions had a tendency to predict the vertical deflection reasonably well and underpredict the horizontal deflection. The vertical soil pressure at the crown was predicted more closely by the full-bond interface solution, while the lateral pressure at the springline was predicted more accurately by the free-slip solution. The circumferential shortening experienced by the larger diameter HDPE pipes was estimated reasonably accurately by the full-bond interface solution of this method.
BIBLIOGRAPHY


21. Plastic Pipe Institute, Catalog No. D-0345, Richardson, Texas, 19
Appendix A

Experimental and Analytical Pipe Deflections
Figure A.1.1 Deflection Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 13

Figure A.1.2 Deflection Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 14
Figure A.1.3 Deflection Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 15

Figure A.1.4 Deflection Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 16
Figure A.1.5 Deflection Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 17

Figure A.1.6 Deflection Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 18
Pipe Modulus Adjusted to Time and Temperature
(Pipe E 90% Compaction on ODOT 310 Sand)

Figure A.2.1 Deflection Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 13

Pipe Modulus Adjusted to Time and Temperature
(Pipe E 96% Compaction on ODOT 310 Sand)

Figure A.2.2 Deflection Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 14
Figure A.2.3  Circumferential Shortening Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 14

Figure A.2.4  Deflection Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 15
Figure A.2.5  Deflection Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 16

Figure A.2.6  Deflection Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 17
Pipe Modulus Adjusted to Time and Temperature  
(Pipe F, 96% Compaction on ODOT 304 Crushed Limestone)

Figure A.2.7  Circumferential Shortening Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 17

Pipe Modulus Adjusted to Time and Temperature  
(Pipe F, 96% Compaction on ODOT 310 Sand)

Figure A.2.8  Deflection Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 18
Figure A.3.1  Horizontal Deflection Comparisons Between Experimental Data and Modified Iowa formula for Test Pipe 13

Figure A.3.2  Horizontal Deflection Comparisons Between Experimental Data and Modified Iowa Formula for Test Pipe 14
Figure A.3.3 Horizontal Deflection Comparisons Between Experimental Data and Modified Iowa Formula for Test Pipe 15

Figure A.3.4 Horizontal Deflection Comparisons Between Experimental Data and Modified Iowa Formula for Test Pipe 16
Figure A.3.5  Horizontal Deflection Comparisons Between Experimental Data and Modified Iowa Formula for Test Pipe 17

Figure A.3.6  Horizontal Deflection Comparisons Between Experimental Data and Modified Iowa Formula for Test Pipe 18
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Experimental and Analytical Soil Pressure
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Figure B.1.2 Springline Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 13
Figure B.1.3  Crown Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 14

Figure B.1.5  Springline Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 14
Figure B.1.3 Crown Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 15

Figure B.1.6 Springline Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 15
Figure B.1.7 Crown Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 16

Figure B.1.8 Springline Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 16
Figure B.1.9  Crown Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 17

Figure B.1.10  Springline Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 17
Figure B.1.11  Crown Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 18

Figure B.1.12  Springline Pressure Comparisons Between Experimental Data and CANDE Solutions for Test Pipe 18
Pipe Modulus Adjusted to Time and Temperature
(Pipe E, 90% Compaction on ODOT 310 Sand)

Figure B.2.1 Crown Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 13

Pipe Modulus Adjusted to Time and Temperature

Figure B.2.2 Springline Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 13
Figure B.2.3  Crown Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 14

Figure B.2.4  Springline Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 14
Figure B.2.5  Crown Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 15

Figure B.2.6  Springline Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 15
Figure B.2.7 Crown Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 16

Figure B.2.8 Springline Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 16
**Figure B.2.9** Crown Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 17

**Figure B.2.10** Springline Pressure Comparisons Between Experimental Data and Solutions for Test Pipe 17
Pipe Module Adjusted to Time and Temperature
(Pipe F 96% Compaction on ODOT 304 Crushed Limestone)

Figure B.2.11 Crown Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 18

Pipe Module Adjusted to Time and Temperature
(Pipe F 96% Compaction on ODOT 304 Crushed Limestone)

Figure B.2.12 Springline Pressure Comparisons Between Experimental Data and Elastic Solutions for Test Pipe 18
Appendix C

Time-Dependant Field Performance
Figure C.1.1 Deflection vs. Time for Test Pipe 13

Figure C.1.2 Deflection vs. Time for Test Pipe 14
20 ft. cover completed in 62 days (Pipe E, 90% Compaction on ODOT 304 Crushed LimeStone)

Figure C.1.3 Deflection vs. Time for Test Pipe 15

20 ft. cover completed in 77 days (Pipe F, 90% Compaction on ODOT 304 Crushed LimeStone)

Figure C.1.4 Deflection vs. Time for Test Pipe 16
20 ft cover completed in 67 days (Pipe F.96% Compaction on ODOT 310Sand)

Figure C.1.5 Deflection vs. Time for Test Pipe 17

40 ft. cover completed in 84 days (Pipe F.96% Compaction on ODOT 304 Crushed Limestone)

Figure C.1.6 Deflection vs. Time for Test Pipe 18
Figure C.2.1 Soil Pressure vs. Time for Test Pipe 13

(Pipe E, 95% Compaction on ODOT 310 Sand)

Figure C.2.2 Soil Pressure vs. Time for Test Pipe 14

(Pipe E, 90% Compaction on ODOT 310 Sand)
Figure C.2.3 Soil Pressure vs. Time for Test Pipe 15

Figure C.2.4 Soil Pressure vs. Time for Test Pipe 16
Figure C.2.5 Soil Pressure vs. Time for Test Pipe 17

Figure C.2.6 Soil Pressure vs. Time for Test Pipe 18
Appendix D

Additional Laboratory Test Results
Figure D.1.1 Results of One Dimensional Compression Tests on 90% Compacted ODOT 310 Sand

Figure D.1.2 Analysis of One-Dimensional Compression Tests Data on 90% Compacted ODOT 310 Sand
Figure D.1.1  Results of One Dimensional Compression Tests on 96% Compacted ODOT 310 Sand

Figure D.1.2  Analysis of One-Dimensional Compression Tests Data on 96% Compacted ODOT 310 Sand
Figure D.1.1 Results of One Dimensional Compression Tests on 90% Compacted ODOT 304 Crushed Limestone

Figure D.1.2 Analysis of One-Dimensional Compression Tests Data on 90% Compacted ODOT 304 Crushed Limestone
Figure D.1.1 Results of One Dimensional Compression Tests on 96% Compacted ODOT 304 Crushed Limestone

Figure D.1.2 Analysis of One-Dimensional Compression Tests Data on 96% Compacted ODOT 304 Crushed Limestone
Appendix E

Determination of
Duncan – Selig Hyperbolic Model Parameters
Figure E.1.1  Stress-Strain Curves for Clayey Overburden Soil

Figure E.1.2  Linear Transformation of Stress-Strain Curves for Clayey Overburden Soil
Figure E.1.3 Determination of $K$ and $n$ for Clayey Overburden Soil

$K = 95$

$n = \log(570) - \log(95) = 0.78$

Figure E.1.4 Determination of $\phi$ and $\Delta \phi$ for Clayey Overburden Soil

$\phi_e = 19.3^\circ$

$\Delta \phi = 8^\circ$
Figure E.2.1 Stress-Strain Curves for ODOT 310 Sand with 90% Compaction

Figure E.2.2 Linear Transformation of Stress-Strain Curves for ODOT 310 Sand with 90% Compaction
Figure E.2.3 Determination of K and n for ODOT 310 Sand with 90% Compaction

Figure E.2.4 Determination of $\phi$ and $\Delta\phi$ for ODOT 310 Sand with 90% Compaction
Figure E.3.1 Stress-Strain Curves for ODOT 310 Sand with 96% Compaction

Figure E.3.2 Linear Transformation of Stress-Strain Curves for ODOT 310 Sand with 96% Compaction
\[ K = \text{Log}(3000) - \text{Log}(775) = 0.588 \]

\[ n = \text{Log}(3000) - \text{Log}(775) = 0.588 \]

\[ K = 775 \]

**Figure E.3.3** Determination of K and n for ODOT 310 Sand with 96% Compaction

\[ \phi = 44.8^\circ \]

\[ \Delta\phi = 5.1^\circ \]

**Figure E.3.4** Determination of \( \phi \) and \( \Delta\phi \) for ODOT 310 Sand with 96% Compaction
Figure E.4.1  Stress-Strain Curves for ODOT 304 Crushed Limestone with 90% Compaction

Figure E.4.2  Linear Transformation of Stress-Strain Curves for ODOT 304 Crushed Limestone with 90% Compaction
Figure E.4.3 Determination of K and n for ODOT 304 Crushed Limestone with 90% Compaction

Figure E.4.4 Determination of $\phi$ and $\Delta\phi$ for ODOT 304 Crushed Limestone with 90% Compaction
Figure E.5.1 Stress-Strain Curves for ODOT 304 Crushed Limestone with 96% Compaction

y = 8.715E-03x + 4.706E-05
y = 8.299E-03x + 4.431E-05
y = 5.726E-03x + 3.855E-05

Figure E.5.2 Linear Transformation of Stress-Strain Curves for ODOT 304 Crushed Limestone with 96% Compaction
Figure E.5.3 Determination of K and n for ODOT 304 Crushed Limestone with 96% Compaction

\[ n = \log(2342) - \log(1713) = 0.14 \]
\[ K = 1713 \]

Figure E.5.4 Determination of \( \phi \) and \( \Delta \phi \) for ODOT 304 Crushed Limestone with 96% Compaction
Appendix F

Input Parameters for Analysis by Elastic Solutions Method
Table F.1 Input Parameters for Test Pipe 13

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Table F.2  Input Parameters for Pipe 14

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## Table F.6 Input Parameters for Pipe 18

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