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Finite element analysis of long-term performance of buried high density polyethylene pipes

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Finite Element Analysis of Long-Term Performance of Buried High Density Polyethylene Pipes

By
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Thesis submitted to the
College of Engineering and Mineral Resources
at West Virginia University
in partial fulfillment of the requirements
for the degree of

Master of Science
in
Civil Engineering

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Morgantown, West Virginia
2006

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ABSTRACT

Finite Element Analysis of Long-Term Performance of Buried High Density Polyethylene Pipes

Raj Kumar Gondle

Buried pipes are utilized for various applications such as sewers, water conveyance, and highway drainage systems. Advantages such as lightweight, ease of installation, low cost, higher impact strength and durability have resulted in the extensive use of High Density Polyethylene (HDPE) pipes during the past few years. Time dependent properties such as creep associated with HDPE material may reduce the expected design life of high density polyethylene pipes under field conditions. The objective of this research work was to investigate the time dependent nature of buried high density polyethylene pipes using the finite element analysis. The influence of trench width on the performance of 24 inch (600 mm) and 48 inch (1200 mm) single-wall and double-wall corrugated pipes under different loading conditions were also investigated. Pipes buried up to a depth of 60 feet were considered. Results show approximately 60 to 80% of the creep deformation within one year of the pipe installation. Pipe deformations observed over 50 years were not more than 30% of the initial pipe deformations. Finite element results also show that 24 inch (600 mm) and 48 inch (1200 mm) HDPE pipes can be buried successfully at depths up to 20 feet by using a trench width ratio as low as 1.5.

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CHAPTER 1

INTRODUCTION

1.1 Background

Buried pipes are utilized for various applications such as sewers, water conveyance, highway drainage systems, and landfill drainage systems. With the increasing use of buried pipes to improve living standards, design issues become vital to help enhance the performance of these pipes. Significant amount of resources is spent every year by the departments of highways and federal government for maintenance, rehabilitation of current pipes, development of the new infrastructure, and in order to improve the overall performance of these pipes (Sargand et al., 1993). “Overall performance of buried pipes not only includes the structural performance of pipe but also includes the service life of buried pipes” (Moser, 1990). Performance of buried pipes can only be evaluated based on experience and thorough understanding of pipe-soil interaction. New pipe and backfill materials, improved pipe profiles, and new design procedures are emerging to enhance the durability and service life of these pipes.

Both, the pipe and the soil mass play important roles in the enhancement of overall performance of buried pipes. Structural performance of buried pipe depends on the interaction between the backfill material, the surrounding soil and the pipe. This interaction is normally referred to as soil-pipe interaction (Goddard, 2003). Service life of buried pipe depends on the choice of pipe material, choice of backfill material, pipe profile, environmental conditions, and installation practices. American Association of State Highway and Transportation Officials (AASHTO) recognizes buried pipe as a composite structure made of plastic ring and soil envelope. A part of the load induced on the pipe may be transferred to surrounding soils over time. Therefore, AASHTO recommends consideration of type and behavior of material beneath the structure, over the structure and adjacent to the structure in the design of buried pipes. Schematic of pipe cross-section and trench geometry are shown in Figure 1.1 and Figure 1.2, respectively. Also, a three-dimensional view of a buried pipe is illustrated in Figure 1.3.

Note: ID = Inside Diameter
OD = Outside Diameter

Figure 1.1: Typical cross-section of pipe geometry
(Zoladz, 1995; Watkins, 1999; Mada, 2005).

Figure 1.2: Typical cross-section of trench geometry (Zoladz, 1995).

Figure 1.3: Typical three-dimensional view of a buried pipe.

1.2 Factors influencing pipe performance

Pipe geometry, pipe strength, pipe flexibility, and backfill properties become important parameters in the structural design of buried pipes (Moser, 1990). Condition of soil-pipe interface, soil-backfill interface, and the nature and magnitudes of loads acting on the pipe also become contributing factors to the performance of these pipes. Time dependent properties (like creep) of pipe become important in evaluation of long term performance of flexible pipes (Janson, 1985, Chua, 1986). For many applications, pipes of various sizes are buried at great depths (such as gravity lines) to let smooth delivery of drainage/flow through the pipes. Large pipe deflections and high soil pressures are very likely to occur over time for such added depths (Moser, 1990). Therefore, for a desired function of buried pipe, selection of appropriate pipe material, pipe size and backfill material becomes a difficult task to any contractor or engineer with all these considerations. Advantages such as greater flexibility, higher impact strength and durability have resulted in the extensive use of high density polyethylene pipes (HDPE) during the past few years (ADS, 2006). When designed and applied suitably, the high density polyethylene (HDPE) pipes are expected to have a longer service life often exceeding 50 years (ADS, 2006).

1.3 Problem statement

Time dependent properties like creep and stress relaxation are normally associated with high density polyethylene pipes and other plastic pipes. High density polyethylene (HDPE) pipes have an excellent ability to function under high stresses (Moser, 1990; Goddard and Gabriel, 1998; Mada, 2005). Creep can be defined as continuous deformation in the pipe material when subjected to a constant mechanical load, resulting in the failure of pipe material over time. Stress relaxation can be defined as relaxation of load with time in a material when held in constant deformation. The rate of this deformation or failure in a pipe can be a function of exposed time, applied pressure, and/or the temperature. Depending on the applied pressure and its duration, the

deformation in the pipe becomes so large that the pipe can no longer perform its intended function.

The flexible nature of HDPE pipe under overlying load or surface load allows pipe to deflect and induce passive pressures against the surrounding soils as the sides of the pipe try to move outward with increase in the horizontal deformation (Moser, 1990). Vertical pipe deformation relieves the load acting on the pipe and redistributes around the pipe. A phenomenon called 'soil arching' reduces the load acting on the pipe as the load is redistributed to the surrounding soil (Moser, 1990). This behavior of flexible pipe coupled with time dependent nature of pipe and soil has complicated the process of evaluation of long-term performance of polyethylene pipes. Despite this, there has been continuous increase in the use of high density polyethylene pipes as buried pipes (Chua and Lytton, 1987).

Various analytical, experimental and numerical procedures have been developed over the years to understand soil-pipe behavior, improve the pipe design practice, and enhance the overall performance of buried pipes. Considerable amount of work has been conducted by many researchers (Janson, 1985; Chua, 1986; Hashash, 1991; Moore, 1995) in order to study the time dependent behavior of high density polyethylene pipes. Many theoretical creep models (Janson, 1985; Chua, 1986; Hashash, 1991; Moore, 1995) for high density polyethylene pipes have been reported. Not many field and numerical studies have been performed to evaluate the long-term performance of high density polyethylene pipes under field conditions. Few research studies performed during the past years elucidate that time dependent properties such as creep associated with HDPE material may reduce the expected design life of high density polyethylene pipes under field conditions. Therefore, there is a need to carefully examine and study the time dependent nature of high density polyethylene pipes under field conditions.

With the improvements in the computer technology and the most recent developments in the finite element codes, a finite element approach has been adopted as the method of choice in order to get better an understanding of field performance of high

density polyethylene (HDPE) pipes. The present study investigates the creep behavior of corrugated HDPE pipes buried at both shallow and deep depths with different soil properties for 50 years. Results may help to define safe limits for the current design practice used in the buried pipe industry.

1.4 Recent studies

Earlier research work at West Virginia University (Simmons, 2002; Mada, 2005) included experimental and numerical investigation on the performance of corrugated HDPE pipes using flowable fill as backfill material. Experimental study (Simmons, 2002) was limited to small diameter high density polyethylene (HDPE) pipes [6 inch (15 cm) and 8 inch (20 cm)] under laboratory conditions. Numerical analysis were performed (Mada, 2005) in order to investigate the structural performance of both small and large size diameter pipes under laboratory and field conditions using flowable fill. The scope of this research was limited to single-wall corrugated HDPE pipes. Two-dimensional plane strain analysis and three-dimensional finite element analysis were performed to study the influence of pipe parameters like pipe size and trench width on the structural performance of corrugated HDPE pipe with assumed material and geometric stiffness. Results discussed show that trench width ratios can be reduced to as low as 1.5 for certain depths. Use of controlled low strength material (CLSM) has shown improvement in the performance of the buried pipes. Larger deflections with increasing burial depths have been noticed. Results also show significant increase in the pipe deflection with the use of interface elements between the native and backfill soil. Performance of 18 inch (450 mm) and 24 inch (600 mm) corrugated pipes under HS-20 loading was evaluated by conducting static analysis. Lower values of material properties were assumed in the static analysis to simulate degradation of material properties with time.

1.5 Scope of work

Extensive literature has been reviewed to examine past and present trends to evaluate the short-term and long-term deflections of pipe. Numerical issues related to the

use of beam elements and plane strain elements to formulate pipe model for unconfined and confined conditions are discussed. Pipe deflections computed on the basis of numerical procedures are compared with the deflections calculated using analytical methods to define limits of validity of the ring deflection theory for thin walled flexible pipes.

Larger pipe deflections and high soil pressures are probable for pipes buried quite deep to allow smooth flow of water/drainage (Moser, 1990). Not many attempts have been made in the past to study the long-term behavior of HDPE pipes under these added depths. The finite element program, ABAQUS was used to study the time-dependent behavior of these HDPE pipes due to self-weight of soil and under HS-20 truck loading. A schematic diagram of a pipe under the influence of self-weight is shown in Figure 1.4. A critical review was conducted on the creep behavior in buried high density polyethylene pipes. Different creep models (Janson, 1985; Chua, 1986; Hashash, 1991; Moore, 1995) applicable for polyethylene pipes were investigated and an appropriate model was selected for the present research study.

Figure 1.4: Buried HDPE pipe with self-weight of soil.

Time-dependent analysis for 50 years was performed to study the nature of creep in high density polyethylene pipes. Double-wall and single-wall corrugated pipes have been used for the purpose. This provides valuable information in choosing dual-wall or single-wall HDPE pipe for a particular application considering cost-cutting benefits and service life of the pipe system. Safe trench widths have been investigated under various loading conditions and at different burial depths. Long-term performance of larger pipe diameters at varying burial depths was also investigated.

1.6 Research objectives

- Review the existing literature to understand the complex behavior of pipe-soil composite structure.
- Select an appropriate type of element to formulate the pipe geometry.
- Investigate a suitable creep model for high density polyethylene pipes.
- Perform transient analysis for 24 inch (600 mm) dual-wall and single-wall high density polyethylene (HDPE) pipes under different loading conditions using flowable fill as backfill material. Compare the response of dual-wall and single-wall HDPE pipes under various loading conditions.
- Investigate the influence of trench widths on the performance of 24 inch (600 mm) and 48 inch (1200 mm) dual-wall and single-wall corrugated pipes under various loading conditions and at different burial depths.
- Evaluate the creep response of 24 inch (600 mm) and 48 inch (1200 mm) high density polyethylene pipes over time.
- Compare the short-term and long-term results for the evaluation of the creep response.

CHAPTER 2

DESIGN AND ANALYSIS OF FLEXIBLE PIPES

2.1 Introduction

In the early years of 19th century, pipes were made rigid and thick assuming rigid pipes (like concrete pipes) would provide greater strength and service. With the increasing applications of buried pipes and better understanding of soil-pipe system, thin walled flexible pipes have become superior because of their exceptional advantages such as lightweight, easy installation, low cost and the flexible nature. Thin steel, plastic and thermoplastic pipes are examples of such flexible pipes. It was observed that flexible pipes perform extremely well when buried in soil although they provide little stiffness compared to rigid pipes (Spangler, 1941). With the improved capability for analysis, better installation practices and newer pipe materials developed over the years, there is a need to carefully study and bring together the important aspects of soil-pipe interaction system. Various analytical, experimental and numerical procedures have been developed over the years to improve the design of buried flexible pipes. Therefore, in order to widen the current understanding of soil-pipe interaction system, existing literature was reviewed and is reported in this chapter. This chapter includes a discussion on few fundamental concepts used by designers and engineers in buried pipe industry. The main aspects that were reviewed in this chapter are:

- Design characteristics of flexible pipes
- Overview of soil properties and mechanics
- Soil-pipe interaction
- Soil arching phenomenon

2.2 Design characteristics of flexible pipes

The flexible nature of pipe under the weight of overlying strata or surface loading allows pipe to deflect and induce passive pressures as the sides of the pipe try to move outward against the surrounding soils. Vertical deflection of pipe relieves the load acting on the pipe and redistributes the load to the surrounding soil. This is known as soil arching (Moser, 1990). In this process, a part of the load acting on the flexible pipe is picked up by surrounding soil, therefore, reducing the load acting on the pipe over time (Moser, 1990). This behavior of flexible pipe coupled with time dependent nature of pipe has complicated the understanding of soil-pipe system. Therefore, there is a need to develop rational means of understanding the complex behavior in order to improve pipe performance. Conventional ways of determining the pipe deflections and the other design parameters influencing the pipe response are described briefly in the following sections:

2.3 Pipe Deflections

Determining the pipe deflection after a period of time is an important task in the successful design of soil-pipe system. Engineers have used various methods to determine the deflection of the pipe. These methods are reviewed briefly in the following section.

2.3.1 Deflection of thin flexible pipes using elastic theory (Timoshenko, 1936)

Elastic theory of flexure was applied to thin rings under simple loading conditions (Timoshenko, 1936). If 'w' was the deflection of the curve, following differential equation can be obtained by using simple elastic theory [detailed procedure is described else where (Timoshenko, 1936)]:

$$\frac{w}{r_0^2} + \frac{d^2 w}{ds^2} = \frac{-M}{EI} \dots\dots\dots (2.1)$$

Figure 2.1 shows the schematic of the ring deformation. From the principle of least work, bending moment at any point, A can be expressed as (Timoshenko, 1936):

$$M = \frac{Pr_0}{2} \left(\frac{2}{\pi} - \cos \theta \right) \quad \dots\dots\dots (2.2)$$

Figure 2.1: Ring compression under two identical loads (Timoshenko, 1936).

Solving the above differential equation, the expression for vertical and horizontal ring deflections can be obtained (Timoshenko, 1936):

$$\delta_v = \frac{2Pr_0^3}{EI} \left(\frac{\pi}{8} - \frac{1}{\pi} \right) = \frac{0.148Pr_0^3}{EI} \quad \dots\dots\dots (2.3)$$

$$\text{and } \delta_H = -\frac{2Pr_0^3}{EI} \left(\frac{1}{\pi} - \frac{1}{4} \right) = -\frac{0.136Pr_0^3}{EI} \quad \dots\dots\dots (2.4)$$

where ‘ δ_v ’ is the vertical change in the pipe diameter and ‘ δ_H ’ is the horizontal change in the pipe diameter.

It was reported (Bulson, 1985) that Equation 2.3 and Equation 2.4 use small deflection theory assumptions about the radius of curvature and are strictly valid as long as the deflections are less than the 3% of the ring diameter. For larger deflections the relationship between the load, P and deflection, w becomes non-linear and this has been reported elsewhere (Spangler, 1938) in the experimental work on thin walled pipes.

2.3.2 Pipe Deflections measured in laboratory

Pipe deflections can be calculated from above equations or can be measured in the laboratory by conducting simple pipe bending test (or parallel plate loading test). Parallel plate testing (ASTM D 2412, 2000) is a standardized test procedure used in the laboratory to measure pipe stiffness by the application of two opposing identical loads to the pipe specimen from two rigid plates at a controlled rate as shown in Figure 2.2. The ratio of applied load per unit length of pipe to the change in inside diameter is known as pipe stiffness. In practice, bending stiffness is often called “pipe stiffness”. Pipe stiffness (or ring bending stiffness or flexural stiffness) can be expressed as (Moser, 1990):

$$PS_f = \frac{F}{\delta_v} = \frac{6.7E_f I}{r^3} \quad \dots\dots\dots (2.5)$$

where PS_f = pipe stiffness (psi),
 E_f = flexural modulus of pipe material (psi),
 I = moment of inertia of the pipe wall (in⁴/in),

r = mean radius (in),

F = force applied over unit length (lbs), and

δ_v = vertical change in the pipe diameter (in).

Figure 2.2: Parallel plate testing (ASTM D 2412 Test) for flexible pipe.

Pipe stiffness values are calculated at 5% vertical deflection as per ASTM D 2412 specification. An allowance of 5% change in vertical deflection has been a traditional safe deflection limit for flexible pipe under surface and soil loads (Chambers et al., 1980; Katona, 1993). However, few others (Solenio, 2005) have recommended 7.5% change in vertical deflection as allowable deflection. Pipe stiffness is expressed with different terminology in buried pipe industry as follows (Moser, 1990):

- 1) Stiffness factor = EI
- 2) Ring stiffness = EI/r^3
- 3) Pipe stiffness = $P/\delta_v = 6.7EI/r^3$

where E = modulus of elasticity of pipe material (lb/in^2),

I = pipe wall moment of inertia per unit length (in^4/in),

r = mean radius of pipe (in),

P = applied load (lb/in), and

δ_v = change in inside diameter measured in the direction of applied load (in).

Pipes buried in the soil exhibit not only bending strains but also exhibit circumferential strains. These strains are very small compared to bending strains. Circumferential stiffness (or ring compression stiffness) can be expressed as (Moser, 1990; Zoladz, 1995):

$$PS_H = \frac{P}{\frac{\Delta D}{D}} = \frac{P}{\frac{pr}{E_{rc}A}} = \frac{E_{rc}A}{r} \quad \dots\dots\dots (2.6)$$

where E_{rc} = compression modulus of pipe material (psi),

A = unit area of the pipe wall (in²/in),

r = mean radius (in),

p = radial pressure on pipe (psi),

D = Pipe diameter (in), and

ΔD = Change in inside diameter (in).

2.4 Parameters influencing the pipe response

It has been reported that for design considerations, pipe stiffness (or ring bending stiffness) forms the controlling parameter rather than the strength of pipe (Moser, 1990). Soil stiffness and overlying loads on the pipe also play an important role in the successful installation of soil-pipe system (Gabriel, 1990). For an unconfined/unsupported pipe, deformation in the pipe can be directly linked to the load induced on the pipe and structural rigidity of the pipe using the following relation derived from Equation 2.5 (Solenio, 2005):

$$Deformation(\Delta) = \frac{Loading\ on\ the\ pipe(P)}{Material\ Stiffness(E) * Geometric\ Stiffness(I / r^3)} \quad \dots\dots\dots (2.7)$$

2.4.1 Load induced on the pipe

The load induced on the pipe depends on the flexibility of the pipe and the surrounding soils. It is important to note the fact that it is not only the magnitude of load that determines the stresses and deformations, but also the character, or distribution, of the load acting on pipe that decides structural response of the pipe (Moser, 1990; Gabriel, 1990). The forces acting on the soil-pipe interface can be resolved into normal and tangential force components (Gabriel, 1990). No bending in the pipe wall can be observed if the surface tractions that exist at the pipe-soil interface are radial and are of the same magnitude favoring ring compression (Gabriel, 1990). Two equal and opposite concentrated loads on opposite ends of a diameter of pipe when viewed in cross-section is considered as the worst possible loading of stable equilibrium. The greatest measure of bending stress (parallel plate loading) is the very worst possible loading scenario (Gabriel, 1990).

2.4.2 Structural rigidity of the pipe

Structural rigidity of the pipe (pipe stiffness) refers to the material's ability to resist the deformation in the pipe and is directly related to the material stiffness (E) of the pipe and the geometric stiffness (I/r^3) of the pipe as shown in Equation 2.7 (Gabriel, 1998). These stiffness values can be evaluated based on the response of the unsupported or unconfined pipe. Properties of backfill material and native soil also play an important role in the evaluation of these properties. Material stiffness includes the density (ρ) and the modulus (E) of the pipe material, that depend on the nature of the material used to manufacture the pipe. For an unsupported flexible pipe, Equation 2.7 shows that pipe stiffness increases with the increase in the wall moment of inertia, I or to the decrease of radius, r of pipe. This shows that not only the size of pipe but also pipe profile serves to maximize pipe resistance and pipe stiffness by increasing or decreasing the wall moment of inertia.

2.5 Overview of soil properties and mechanics

Soil stiffness is one of the most important design parameters in the successful installation of buried pipes. “Stability in a soil-structure interaction system requires not only adequate design of the structural barrel; it also presumes a well engineered backfill” (American Iron and Steel Institute, 1971). In the successful design of buried pipes, it is necessary to select a proper backfill material. The selected backfill must be strong enough to hold pipe deformations. Table 2.1 presents the classification of backfill materials. Also, it is important to ensure that in-situ soil is strong enough to hold the pressure of the backfill material (Solenio, 2005).

2.5.1 Soil stiffness

Soil stiffness is characterized by the modulus of soil reaction, E' and is an important parameter in Iowa formula (Hartley and Duncan, 1987) for determining the pipe deflections. Spangler (1941) proposed the Iowa formula after a series of experiments to determine the horizontal deflection of the pipe culverts. This formula has been used to back calculate the modulus of soil reaction, E' . Modulus of soil reaction, E' , is considered theoretically similar to an effective soil modulus (such as Young’s modulus, E or constrained modulus, M_s) and is assumed to behave in a similar fashion (Hartley and Duncan, 1987). Figure 2.3 shows the assumptions used by Spangler (1941) in developing the Iowa formula. Spangler (1941) assumed that Marston’s load applied was uniformly distributed at the top of the pipe as shown in the Figure 2.3. He also assumed a uniform pressure over a part of the bottom of the pipe. The horizontal pressure, p on sides was assumed to be proportional to the deflection of the pipe.

From his original experiments conducted in 1941, Spangler’s Iowa formula has been typically shown as below (Moser, 1990):

$$\Delta X = \frac{D_L K W_c}{\frac{EI}{r^3} + 0.061er} \dots\dots\dots (2.8)$$

where

ΔX = change in the horizontal diameter (in).

D_L = deflection lag factor to account for time dependent deflections.

K = bedding constant.

W_c = Marston's load per unit length of the pipe (lb/in).

E = modulus of elasticity of the pipe material (psi).

R = mean radius of the pipe (in).

I = moment of inertia of the pipe wall per unit length of the pipe (in⁴/in).

e = modulus of passive resistance of the backfill (psi).

**Figure 2.3: Spangler's assumptions to develop the Iowa formula
(Moser, 1990; Bulson, 1990).**

Table 2.1: Class and type of soil used for backfill material⁺

Class	Type	Backfill description
IA [*]	---	Granular material: angular crushed stone or rock, crushed gravel, crushed slag, large voids, 6 to 40mm (1/4 to 1 1/2 inches) with little or no fines.
IB [*]	---	Angular crushed stone or other class 1A material with or without other regional components such as coral, ash, crushed stone, shell and stone/sand mixture, with little or no fines.
II [*]	GW	Well-graded gravel of 40mm (1 1/2 inches) maximum and mixture of sand and gravel, little or no fines. 50 % or more of gross retained by #4 sifters. More than 95 % retained by #200 sifters. Clean.
	GP	Poorly graded gravel, gravel/sand mixtures, little or no fines. 50 % or more of gross retained by #4 sifters. More than 95 % retained by #200 sifters. Clean.
	SW	Well-graded sands, gravel sand; little or no fines. 50 % or more of gross retained by #4 sifters. More than 95 % retained by #200 sifters. Clean.
	SP	Poorly graded gravel, gravelly sands, little or no fines. 50 % or more of gross retained by #4 sifters. More than 95 % retained by #200 sifters. Clean.
III [*]	GM	Silty gravels, gravel/sand/slit mixtures More than 50 % passes through #4 sifters. More than 50 % retained by #200 sifters.
	GC	Clayish sands, gravel/sand/clay mixtures More than 50 % passes through #4 sifters. More than 50 % retained by #200 sifters.
	SM	Silty sands, sand/slit mixtures. More than 50 % passes through #4 sifters. More than 50 % retained by #200 sifters.
	SC	Clayish sands, sand/clay mixtures More than 50 % passes through #4 sifters. More than 50 % retained by #200 sifters.
IVA ^{***}	ML	Inorganic silts and very fine sands, rock flour silty or clayish fine sands, silts with slight plasticity, sand with average to high flow and liquidity limit. Limit of 50 % or less liquid. More than 50 % passes through #200 sifters.
	CL	Inorganic clays of low to moderate plasticity, gravelly, sandy or silty clays, lean clays, sand with moderate to high flow and liquidity limit. Limit of 50 % or less liquid. More than 50 % passes through #200 sifters.
IVB ^{***}	MH	Inorganic silts, macaceous or diamaceous fine sandy or silty soils, elastic soils with moderate to high flow and liquidity limit. Limit of 50 % or less liquid. More than 50 % passes through #200 sifters.
	CH	Inorganic clays of high plasticity with moderate to high flow and liquidity limit. Limit of 50 % or less liquid. More than 50 % passes through #200 sifters.
V ^{****}	OL	Organic silts and organic silty clays with low plasticity. Limit of 50 % or less liquid. More than 50 % passes through #200 sifters.
	OH	Organic clays of moderate to high plasticity, organic silts. Limit of 50 % or less liquid. More than 50 % passes through #200 sifters.
	PT	Peat, manure and other highly organic soils.

* As described in standard ASTM D 2487, with the exception of Class I materials which are described in the ASTM D 2321 standard.

** According to standard ASTM D 2487, less than 5 % of soils pass through #200 sifter.

*** According to standard ASTM D 2487, 5 to 12 % of soils that pass through #200 sifter fall within the limits of the classification which is more characteristic of class II than class III. Soils of types MH, ML, CH and CL are not recommended for the bedding, the haunch or the initial backfill.

**** This class includes frozen soil, debris and rocks bigger than 40mm (1 1/2 inches) in size. All materials OL, OH and PT are not recommended for the bedding, the haunch or the initial backfill.

+Reference: (ASTM D 2321, 2000; Soleno, 2005, Mada, 2005).

Spangler (1941) defined ‘modulus of passive resistance’ as the ratio of the horizontal pressure acting on the culvert to horizontal deflection of the culvert. This parameter has been observed as a constant (Spangler, 1941). It was later observed that ratio, ‘e’ was not constant for a given soil, but that the quantity “e” multiplied by the mean radius of the pipe was approximately constant (Watkins and Spangler, 1958). As a result of this study, a new soil parameter was defined in the buried pipe terminology. The quantity has been denoted as E' and has been termed as ‘modulus of soil reaction’, $E' = er$. This resulted in the development of modified Iowa formula which can be typically given as (Howard, 1977, Moser, 1990, Brewer, 1990, Bulson, 1990):

$$\Delta X = \frac{D_L K W_c}{\frac{EI}{r^3} + 0.061 E'} \dots\dots\dots (2.9)$$

where ΔX = change in the horizontal diameter (in),

D_L = deflection lag factor to account for time dependent deflections,

K = bedding constant,

W_c = Marston’s load per unit length of the pipe (lb/in),

E = modulus of elasticity of the pipe material (psi),

R = mean radius of the pipe (in),

I = moment of inertia of the pipe wall per unit length of the pipe (in⁴/in), and

E' = modulus of soil reaction (psi).

In order to investigate the influence of modulus of soil reaction, many analytical studies have been performed. One such study is using the elastic solution proposed by Burns and Richards (1964). The modulus of soil reaction, E' may be obtained from the Burns and Richards (1964) solution for pressures and deflection by modification of Spangler (1941) formula (Hartley and Duncan, 1987):

$$E' = \frac{P_h r}{w_h} \dots\dots\dots (2.10)$$

where P_h = horizontal pressure at springline (psi),

r = mean pipe radius (in),

w_h = radial horizontal displacement (in).

In the analysis presented by Burns and Richards (1964) analysis, the soil was assumed to be homogeneous, elastic and isotropic. It was also assumed that the soil was subjected to uniform overlying pressure of infinite horizontal extent. Many researchers (Chamber et al., 1980; Krizek et al., 1971) have related modulus of soil reaction, E' to the constrained soil modulus, M_s by using the relationship as given below:

$$E' = kM_s \dots\dots\dots (2.11)$$

where 'k' is a constant between 0.7 and 1.5 determined by elastic analysis. Constrained soil modulus, M_s which forms the more fundamental character has been closely approximated to modulus of soil reaction by other researchers (Chambers et al., 1980; Krizek et al., 1971) based on the practical tests. Assuming that $E' \approx M_s$, the elastic modulus (Young's modulus of the soil) can be calculated using the following relationship (Hartley and Duncan, 1987):

$$E' \approx M_s = \frac{E_s(1 - \nu_s)}{(1 + \nu_s)(1 - 2\nu_s)} \dots\dots\dots (2.12)$$

where E_s and ν_s are elastic modulus (Young's modulus) and Poisson's ratio of the soil, respectively.

2.5.2 Parameters influencing soil stiffness

It has been clearly established experimentally and in practice (Moser, 1995; Howard, 1977) that modulus of soil reaction, E' depends to a great extent on soil properties such as soil density, soil type and moisture content. The degree of compaction of the backfill, the trench geometry and composition of native soil also influence the modulus of soil reaction, E' (Hartley and Duncan, 1987; Soleno, 2005). Table 2.2 shows the values of modulus of soil reaction, E' for different soil types and different compaction efforts (Soleno, 2005). It has been observed that E' is significantly influenced by different compaction techniques used at the site (Howard, 1977; Hartley and Duncan, 1987; Moser, 1995; Faragher et al., 1998). Table 2.3 illustrates the degree of compaction attained by using different compaction procedures (Soleno, 2005). In a study conducted by Hartley and Duncan (1987), it has been observed that the modulus of soil reaction, E' also depends on the backfill depth. Table 2.4 provides the recommended values of soil modulus by various researchers. Based on the information given in the Table 2.4, a soil modulus of 2,000 psi was assumed for the backfill material to investigate the performance of the pipe in this study.

The modulus of soil reaction, E' , is not only considered as a property of the soil but it is also treated as a property of the soil-pipe system. Inspection of Iowa formula (Spangler, 1941) shows that deflection of the pipe is dependent on the overlying loads, and, modulus of the pipe and the surrounding backfill. Therefore, modified Iowa formula has been expressed conceptually as follows (Watkins and Spangler, 1958; Gabriel, 1990):

$$\text{Pipe deflection} = \frac{\text{load on the pipe}}{\text{Pipe Stiffness} + \text{Soil Stiffness}} \dots\dots\dots (2.13)$$

It has been observed that soil stiffness is more dominant than the structural stiffness of the pipe on the overall performance of the buried pipe (Howard, 1977).

Table 2.2: Modulus of the soil reaction (E')*.

Modulus of soil reaction – E'								
Pipe Backfill Material				E' kPa (psi) According to the degree of compaction				
ASTM D2321 Class Description		ASTM D2487 Symbol Description		Minimum Recommended Proctor	None (dumped)	Light < 85 % relative density < 40 %	Moderate 85 to 95 % Relative density 40 to 70 %	High > 95 % Relative density > 70 %
IA IB	Crushed gravel, manufactured	S/O	Crushed gravel, angular and large	Diverse	6895 (1000)	20685 (3000)	20685 (3000)	20685 (3000)
II	Granular soils, clean	GW, GP, SW, SP	Gravel or sand with little or no fine particles	85 %	1379 (200)	6895 (1000)	13790 (2000)	20685 (3000)
III	Granular soils with fines	GM, GP, SW, SP	Mixture of gravel or sand with other components <10% fines	90 %	690 (100)	2758 (400)	6895 (1000)	13790 (2000)
IVA	Granular, fine inorganic soils	ML CL	Cohesive soil with little to moderate plasticity	<p style="text-align: center;">Not Recommended</p> <p style="text-align: center;">Data not available For all usage, request APPROVAL of a soil expert</p>				
IVB	Granular, fine inorganic soils	MH CH	Cohesive soil with high plasticity					
V	Organic or highly organic soils	OL, OH, PT	-----					

*Reference: Soleno, 2005.

Table 2.3: Degree of compaction of backfill materials.*

Compaction of Backfill Materials				
Class of material	I	II	III	IV
Description of material	Angular manufacturing stones	Gravel and sand without fines, clean	Soil mixtures (sand and gravel with small gravel), little fines	Mixture of fine soils (silt and clay), with fines.
Upper limit of water content, % of dry weight.	----	9 to 12	9 to 18	6 to 30
Compaction technique	% proctor density (% relative density)			
Mechanical compactor (roller, rammer, etc)	95 to 100 (75 to 100)	95 to 100 (80 to 100)	95 to 100	90 to 100
Density increased by portable vibrators	80 to 95 (60 to 75)	80 to 95 (60 to 80)	80 to 95	75 to 90
Saturation compaction	80 to 95 (60 to 75)	80 to 95 (60 to 80)	----	----
Placed manually	60 to 80 (40 to 60)	----	----	----
Compressed manually	----	60 to 80 (40 to 60)	60 to 80	60 to 75
Dumped	60 to 80 (40 to 60)	60 to 80 (50 to 60)	60 to 80	60 to 75

*Reference: Soleno (2005).

Table 2.4: Recommended values of different soil modulus used by various researchers.

Author	Type of soil	Soil Modulus, MPa (psi)	Poisson's ratio	Density, kN/m³ (pcf)
Brachman et al. (2000) Moore and Brachman (1994)	Granular Backfill (90% compacted)	30 (4350)	0.3	18 (114)
	Granular Backfill (98 % compacted)	80 (11600)	0.3	18 (114)
Suleiman et al. (2004)	Silty Sand (SM)	6.89 (1000)	0.35	18.8 (120)
Brachman et al. (1996)	Stone	50 (7250)	0.25	--
	Cover soil	20 (2900)	0.20	--
	Clay	10 (1450)	0.35	--
Faragher et al. (1998)	Lightly compacted soil	18 - 94 (2600 – 13600)	--	17.5 (111)
	Gravel surround	29 – 148 (4200 – 21465)	--	14.5 (92.0)
Howard (1977)	Low plasticity clay (CL)	2.8 (400)	--	--
	Low plasticity silt (ML)	7.0 (1000)	--	--
	Well graded sand (SW)	14.0 (2000)	--	--
Selig (1988); Hashash and Selig (1990)	Low plasticity clay (CL)	3.4 – 15.2 (500 – 2200)	--	8.7 – 18.6 (56 – 119)
	Low plasticity silt (ML)	9.6 – 40.0 (1400 – 5800)	--	10 – 20 (66 – 127)
	Well graded sand (SW)	28 – 82 (4060 – 11900)	--	14.2 – 22 (91 – 141)
Sargand and Masada (2000)	Coarse granular soil (85 % compacted)	3.6 – 5.6 (500 – 800)	0.3	20.4 (130)
Hartley and Duncan (1987); Goddard et al., (2003)	Fine grained soils (CL, ML)	3.5 – 18.0 (500 – 2600)	0.35 – 0.40	15 – 24 (100 – 150)
	Coarse grained soils (SP, SW, GP, GW)	4.0 – 26.0 (600 – 3800)	0.3 - 0.35	

2.5.3 Controlled Low Strength Material (CLSM)

Structural performance of any buried pipe may depend on the type, and density of the surrounding backfill around the pipe (Howard, 1977; Hartley and Duncan, 1987). For the satisfactory performance of a buried pipe, backfill placed around the pipe should maintain the specified compacted density and should provide adequate structural and drainage characteristics. Also, backfill should be placed uniformly to eliminate stress concentration in the soil zone around the pipe. Other structural considerations of backfill placed in the soil zone around the pipe include the soil type, compaction effort, and the trench geometry (Hartley and Duncan, 1987, Howard, 1972). Therefore, in order to mitigate structural deficiencies caused by inadequate backfill, controlled low strength material (CLSM) can be used. Moreover, CLSM has excellent characteristics to flow uniformly around the pipe.

CLSM is a mixture of cementitious material, soil, water and sometimes fly ash and admixtures (Hitch, 1998). During the past few years, CLSM has been used instead of compacted backfill because of its exceptional advantages such as self-leveling competence and ability to flow into hard-to-reach areas with minimal effort. Flowable CLSM is the best and economical alternative in conventional earth fills where controlled compaction is mandatory (Howard and Hitch, 1998). In a research study (Webb et al., 1998) as a part of installation procedures for buried pipe; a corrugated HDPE pipe was installed using CLSM as a backfill. Prior to field test, laboratory evaluation of mix designs were undertaken to determine the use of these materials. Excellent flowability of the CLSM has been observed, filling all the voids below the pipe and those (voids) formed by the pipe corrugations. The measured interface pressure has indicated that CLSM provides very uniform and excellent support, especially in the hard-to-reach areas where it is not possible to compact the soil. Not only the flowability and self-leveling capability but also other advantages of CLSM observed are (Howard and Hitch, 1998):

- Ease of mixing
- No special equipment is required to deliver the CLSM

- Fast backfilling operations
- Can be placed under adverse weather conditions

Controlled low strength material is also known with different terminology such as flowable fill and controlled density fill (Howard and Hitch, 1998). It is possible to excavate CLSM easily because of its low strength. Although CLSM provides numerous advantages compared to conventional earth backfilling, there are also few limitations on the use of these materials. Listed are few limitations (Howard and Hitch, 1998):

- Need to anchor lighter-weight pipes
- Confinement needed before setting
- Higher-strength mixtures may not allow excavation

The use of CLSM as a backfill material in the soil zone around the pipe was investigated (Brewer, 1990). To demonstrate the design advantages of CLSM, values for the modulus of soil reaction E' were presented and their effect on horizontal deflections has been compared to conventional backfill using the Iowa formula. Results compared illustrated that for the same value of modulus of soil reaction E' , CLSM backfill could result in a 50% reduction in horizontal pipe deflections. Values for the modulus of soil reaction (E') used for installation of flexible pipe ranged from 1,000psi to 3,000psi for ages from 16 hours to 28 days, respectively (Brewer, 1990).

2.6 Soil-pipe interaction

Soil-pipe interaction is a function of properties of pipe and soil backfill. These properties control the overall performance of a buried pipe, and therefore impacts total system cost (AASHTO, 2002). The design of soil-pipe system is treated as a statically indeterminate problem (Moser, 1990). This signifies that the interface pressure between the soil and pipe cannot be calculated by considering static equilibrium alone.

2.7 Soil arching phenomenon

For flexible pipes buried at great depths, soil arching phenomenon adds an advantage by reducing the load directed to the pipe. In this phenomenon, load attracted to the pipe is directed towards the stiffer soil where it will look for additional support. For the rigid pipes buried in the soil, the load acting on the top of the pipe increases larger than the prism load caused by the soil column resting exactly above the pipe as shown in Figure 2.4(a). The soil prism resting on the top of the pipe settles more compared to the soil columns standing on the sides resulting in a differential settlement as shown in the Figure 2.4(a). This phenomenon is known as negative soil arching (Moser, 1990). On the other hand, the load acting on the flexible pipe is less than the prism load (Figure 2.4). With the ability of the flexible pipes to elongate on the sides, additional passive pressures are developed. These passive pressures developed reduce load attracting towards the pipe by distributing the part of loads acting on the pipe to surrounding soil medium. This phenomenon is known as positive soil arching (Moser, 1990).

Figure 2.4: Vertical soil Arching (Moser, 1990).

The soil arching phenomenon associated with buried thermoplastic pipes has been reported (Sargand and Masada, 2003) in detail to characterize the degree of soil arching above the buried pipe. Elastic solutions obtained by Burns and Richard (1964) were revisited (Sargand and Masada, 2003) to derive expressions for vertical arching factor. The results showed that the elastic solutions predicted vertical soil pressure at the crown well by incorporating both the hoop stiffness and the bending stiffness. Also, it was observed that AASHTO method was conservative and it over-predicts the load on buried thermoplastic pipe by 30% (Sargand and Masada, 2003). In another study, effects of soil arching on the behavior of buried flexible pipe with varying trench widths were investigated (Wijeyesekera and Warnalulasuriya, 2000). It has been observed (Wijeyesekera and Warnalulasuriya, 2000) that the rate of increase of pipe deformation with increasing fill height decreased with increasing trench width.

In another study, bending and circumferential strains in buried thermoplastic pipes were calculated using simplified design equations (Dhar et al, 2002). These simplified design equations have been evaluated by comparing experimental and finite element results. Bending strains and circumferential strains calculated from simplified design equations were observed to be more reasonable. The equation was similar to Iowa equation (Spangler, 1941) except that the hoop and bending components were incorporated. The proposed equation to calculate pipe deflection is given as below (Dhar et al., 2002):

$$\frac{\delta_v}{D} = \left(\frac{VAF \cdot q_v}{\frac{EA}{R} + 0.57M_s} \right) + \left(\frac{D_t K_b q_v}{\frac{EI}{R^3} + 0.061M_s} \right) \dots\dots\dots (2.14)$$

where,

δ_v = Change in vertical pipe diameter (in),

D = Pipe diameter (in),

q_v = Overburden pressure at the springline (psi),

E = Modulus of pipe material (psi),

R = Radius of pipe (in),

M_s = Soil modulus (psi),
 K_b = Bedding coefficient,
 D_1 = Deflection lag factor, and
VAF = Vertical arching factor.

The first term in the above equation was observed to be important for compressible pipes such as HDPE pipe as it provides circumferential shortening. The second term in the design equation quantifies the bending deformations. It was also concluded that the equation for vertical arching factor provided a reasonable estimate of the effect of pipe-soil interaction on the thrust that develops at the pipe spring line.

2.8 Summary

From the literature on soil-pipe interaction system, it is clear that the soil-pipe structure acts as a composite structure. The soil stiffness and pipe stiffness play an important role in the satisfactory performance of the pipe. Also, the magnitude and the method of load application influence the performance of the pipe. Soil arching adds an advantage to flexible pipes by reducing the load acting on the pipe and improving the performance of flexible pipes.

CHAPTER 3

TIME DEPENDENT RESPONSE OF POLYETHYLENE PIPES

3.1 Introduction

Time dependent properties like creep and stress relaxation are normally associated with high density polyethylene (HDPE) pipes and other plastic pipes. Creep in pipe can be defined as continuous deformation in the pipe material when subjected to a constant mechanical load. Creep may result in failure of pipe material over time. Stress relaxation can be defined as relaxation of load with time in a pipe material when held in constant deformation (Arvidsson and Gronvall, 2004). The rate of this deformation in a pipe is a function of exposed time, applied load, and the temperature. Depending on the magnitude of the applied load and its duration, the deformation in the pipe may become so large that pipe can no longer perform its intended function. There has been a continuous increase in the use of high density polyethylene pipes (Chua and Lytton, 1987) despite their time-dependent behavior which may negatively influence long term performance. Therefore, there is a need to develop rational means to understand the time dependent nature of buried high density polyethylene pipes.

3.2 Creep

Creep can be defined as continuous deformation in pipe material when subjected to a constant mechanical load. Creep is not observed with instantaneous sudden loading; it happens over time. As the load is applied, pipe deforms instantaneously up to a certain magnitude after which the pipe material permanently strains over a period of time before it fails. A 5% or 7.5% vertical change of diameter has been assumed as the failure criterion for these pipes (Goddard, 2003; ADS, 2006; Plastic Pipe Institute, 2005; Reddy and Ataoglu, 2002).

In order to investigate the creep behavior of plastic pipes, a pipe specimen is subjected to instantaneous load and is allowed to creep for a period of time. The vertical change in the diameter of the pipe is measured for a period of exposed time. Using a relationship between measured pipe deflection and applied load, the variation in the modulus of material is calculated with time. This modulus is known as creep modulus or time dependent modulus. Creep modulus provides time dependent relationship between stress and strain in the material. Creep modulus is an important material property for determining the short term and long term pipe modulus of the buried pipe (Janson, 1985). Figure 3.1 shows the stages of creep deformation in a material. Time-dependent modulus (or creep modulus) for a linear viscoelastic material like HDPE can be expressed as (Arvidsson and Gronvall, 2004):

$$E(t) = \frac{\sigma}{\varepsilon(t)} \dots\dots\dots (3.1)$$

Significant amount of laboratory tests have been performed on various plastic pipes (Janson, 1985; Chua, 1986; Petroff, 1993; Hashash, 1991; Moore, 1995). Response of such pipes has been divided into three regions as shown in the Figure 3.1 (Hashash, 1991). An initial deflection is observed with instantaneous application of load and as the pipe is allowed to creep over a period of time, the creep rate decreases with continuous increase in strain. This is known as primary creep in the material (Arvidsson and Gronvall, 2004). Beyond the primary creep, the strain in the material continues to increase with the constant application of load as shown in Figure 3.1. This is known as secondary creep. It can be illustrated from Figure 3.1 that creep deformation is rapid (primary creep) during its initial stages and then slows down (secondary creep) as the creep properties of the material tends to dampen the response. As the load is still maintained, the material permanently strains reaching a failure point in tertiary region. Considerable amount of laboratory work has been conducted by many researchers in order to study the creep behavior of high density polyethylene pipes (Janson, 1985; Chua, 1986; Hashash, 1991; DiFrancesco, 1993; Moore and Hu, 1995).

Figure 3.1: Creep deformation in a material (Arvidsson and Gronvall, 2004).

3.3 Stress relaxation

Stress relaxation can be defined as relaxation of load with time in a material when held in constant deformation. Stress relaxation tests feature load measurements over time for a pipe deflected under constant deformation. Various laboratory tests (Janson, 1985; Chua, 1986; Petroff, 1993; Hashash, 1991; Moore, 1995) have shown stress decrease with time under constant strain. Using a relationship between pipe deflection and measured load, the variation in the modulus of material can be calculated with time. If a test specimen is subjected to a constant strain, ε_0 , the stress decrease $\sigma(t)$ over time can be plotted as shown in the Figure 3.2. Stress relaxation modulus for a linear viscoelastic material can be expressed as (Arvidsson and Gronvall, 2004):

$$E(t) = \frac{\sigma(t)}{\varepsilon} \dots\dots\dots (3.2)$$

Figure 3.2: Stress relaxation in a material.

The relaxation modulus $E(t)$ of a linear viscoelastic material like HDPE can be expressed in as (Arvidsson and Gronvall, 2004):

$$E(t) = E_{\infty} + \int_0^{\infty} E(\tau) e^{-\frac{t}{\tau}} d\tau \quad \dots\dots\dots (3.3)$$

where ‘ τ ’ is the relaxation time, and $E(\tau)$ is the relaxation function.

Stress and strain in the material are related using a secant modulus which depends on time (Chua, 1986, Moore and Hu, 1995, Moore and Zhang, 1998). For a viscoelastic material, relationship between stress and strain with time cannot be described reasonably using elastic constants. Hence, in order to describe the time dependent response of the viscoelastic material, time dependent material functions are used such as secant modulus, $E(t)$. Secant modulus, $E(t)$ at any time t can be defined as the total stress, $\sigma(t)$, divided by the total strain, $\varepsilon(t)$ as shown below (Moore and Hu, 1995):

$$E(t) = \frac{\sigma(t)}{\varepsilon(t)} \quad \dots\dots\dots (3.4)$$

For a certain duration after the initial strain, the strain in viscoelastic material increases linearly with increase in stress. When the strains exceed proportional limit, the behavior is assumed to become non-linear (Moore and Zhang, 1998).

Considerable amount of experimental and numerical studies have been conducted over the years in order to study creep behavior of high density polyethylene pipes. Various analytical models have been proposed based on the time dependent response of high density polyethylene pipes (Janson, 1985; Chua, 1986; Petroff, 1993; Hashash, 1991; Moore, 1995). These models are discussed briefly in the following sections:

3.4 Power law model

In the analysis of buried pipes, the pipe material can be expressed by using elastic modulus, $E(t)$ as a function of time. For a viscoelastic material like HDPE, the power law formulation has been used to model elastic modulus as a function of time (Chua, 1986; Petroff, 1993). Power law model are simple and quite appealing. These power law models are formulated based on stress relaxation tests featuring load measurements over a period of time for a pipe deflected under constantly held strain. Based on geometry and stiffness of pipe, modulus of the pipe at that particular period of time is computed. Using a relationship between pipe deflection and measured load, the variation in the modulus of material with time is plotted to logarithmic scale. Equation 3.5 shows a general power law relation obtained by fitting the curve for most pipe materials. The equation is thereafter used to extrapolate the data to 50 or 100 years to predict the long term creep behavior of high density polyethylene pipes (Chua, 1986):

$$E(t) = E_1 t^{-m} \dots\dots\dots (3.5)$$

where E_1 and m are constants that depend on the material type. E_1 is the intercept and m is the creep rate as shown in the Figure 3.3. Creep rate is the rate of degradation of the pipe material (Chua, 1986). Various power law models have been reported in the literature based on stress relaxation tests to describe the time dependent behavior

associated with high density polyethylene pipes (Janson, 1985; Chua, 1986; Petroff, 1993; Hashash, 1991; Moore, 1995).

Figure 3.3: Power law form of creep behavior.

3.4.1 Janson's (1985) power law model:

Stress relaxation tests were performed on three different material grades of polyethylene pipes manufactured by Unifos Kemi AB, Sweden (Janson, 1985). Pipe specimens were held at constant deflection throughout the testing period by the use of two linear loads acting in opposite directions along the centerline. The load that was needed to maintain constant pipe deflection was measured over time. The relaxation modulus based on pipe stiffness calculations were then plotted to logarithmic scale as a function of loading time up to 10,000 hours. By curve fitting, a relation between modulus of the pipe material and time was obtained and then extrapolated to 50 years. A check was made after 20,000 hours of loading time to show that extrapolation is conservative. Recovery of the pipe deflection was also studied as a function of time after the load release. Recovery tests reported that HDPE pipe material behaves like a new material under short term loading even after 10,000 hours of constant deflection. Power law

relations for $E(t)$ at 4.6% and 13.7% diametrical deflections were reported using the following equations:

At 4.6% vertical deflection:

$$E(t) = \frac{\sigma}{\varepsilon(t)} = 520 t^{-0.0795} \dots\dots\dots (3.6)$$

where relaxation modulus, $E(t)$ was expressed in MPa and time, t was measured in hours.

At 13.7% vertical deflection:

$$E(t) = \frac{\sigma}{\varepsilon(t)} = 350 t^{-0.0672} \dots\dots\dots (3.7)$$

where relaxation modulus, $E(t)$ was expressed in MPa and time, t was measured in hours.

3.4.2 Chua's (1986) power law model:

Chua (1986) performed few compression tests on pipes manufactured by Spirolite, United States. Chua (1986) proposed a simple power law model to overcome the complexities of Maxwell and Kelvin model. The proposed power law is given below:

$$E(t) = E_{\infty} + E_1 t^{-m} \dots\dots\dots (3.8)$$

where E_{∞} , E_1 , and m depend on material type and are independent of time. Significance of these constants has been described (Chua, 1986) using a 'master curve' constructed on a relaxation modulus versus time curve plotted on a logarithmic scale as shown in the Figure 3.4. E_1 is the intercept and 'm' is creep rate as shown in Figure 3.4. The value of creep rate, m , is chosen to be about 0.098 for HDPE material.

Figure 3.4: Master curve (Chua, 1986).

For a high density polyethylene material, Chua (1986) proposed a relaxation modulus as shown below:

$$E(t) = \frac{\sigma}{\varepsilon(t)} = 7630 + 99507 t^{-0.097786} \quad \dots\dots\dots (3.9)$$

where modulus, $E(t)$ was calculated in psi and time, t was measured in minutes. This relationship has also been expressed as (Moore and Hu, 1996):

$$E(t) = \frac{\sigma}{\varepsilon(t)} = 52.6 + 460 t^{-0.097786} \quad \dots\dots\dots (3.10)$$

where modulus, $E(t)$ was calculated in MPa and time, t was measured in hours.

3.4.3 Petroff's (1993) power law model

In another study, Petroff (1993) observed that HDPE pipe regains part of its original strength and behaves like a new material after the load release. The recovery of pipe material was constrained to pipes loaded beyond the yield strain. Instantaneous

deformation with rapid creep deformation was observed during the early weeks of installation. When held to constant position, creep rate got dampened with time after the primary creep. Table 3.1 explains the percentage of creep deformation over time. Petroff (1993) concluded that any pipe would not fail by creep within 50 or 100 years, if the pipe passes inspection during the first month. Below is the power law relation obtained after a series of tests (Petroff, 1993):

$$E(t) = \frac{\sigma}{\varepsilon(t)} = 131000 t^{-0.083} \dots\dots\dots (3.11)$$

where modulus, E(t) was calculated in psi and time t was measured in minutes.

Table 3.1: Percentage creep deformation for HDPE pipe as a function of time (Petroff, 1993).

Loading time	Creep deformation
1 day	68%
1 week	76%
1 month	81%
10 years	95%
100 years	100%

3.4.4 Hashash's (1991) power law model:

Few laboratory tests have been performed on corrugated HDPE pipes manufactured by Advanced Drainage System (Hashash, 1991) in order to evaluate short term as well as long term field performance of HDPE pipes. In this study, two different types of laboratory tests - creep and stress relaxation tests were conducted on corrugated HDPE pipes of various sizes. Creep tests were performed at various deformation rates. Tests were conducted according to ASTM D 2412 and, the applied load, vertical and

horizontal deflections were recorded for each test. The deformation rates varied from 0.0004%/min. to 3%/min. Stress relaxation tests were performed at various constant deformation levels. For each stress relaxation test, the load needed to hold the pipe at constant deformation was recorded over a period of seven months. The stress relaxation tests were performed at 5%, 10% and 15% vertical deflections.

Results of creep tests have indicated an increase in pipe stiffness and load per unit deformation as the deformation rate was increased. It has been reported that a decrease of stiffness factor (EI) with time for all deformation rates was noticed (Hashash, 1991). The variation of stiffness factor (EI) with time was plotted and the amount of decrease in stiffness factor was observed to be a function of pipe category tested. Effect of pipe size on material modulus was also studied. An increase in the material modulus was observed with increase in the pipe size. The creep rate was observed to be roughly same for all different pipe sizes.

Results from stress relaxation tests have shown roughly the same rate of stress decay with time for all three deflection levels - 5%, 10% and 15%. The rate of decrease of the stiffness factor was highest for the lowest deflection level (5%). The stiffness factor observed was roughly about the same for higher deflection levels (10% and 15%). The variation between the pipe modulus with time at 5% diametrical deflection was plotted to logarithmic scale as shown in the Figure 3.5. These results follow the relationship given below:

$$E(t) = \frac{\sigma}{\varepsilon(t)} = 96300 \ t^{-0.0859} \quad \dots\dots\dots (3.12)$$

where modulus, E(t) was expressed in psi and time, t was measured in minutes. In metric units:

$$E(t) = \frac{\sigma}{\varepsilon(t)} = 467 \ t^{-0.0859} \quad \dots\dots\dots (3.13)$$

where modulus, $E(t)$ was expressed in MPa and time, t was measured in hours.

The above equation was then used to extrapolate the data to 50 years to evaluate the long term field performance of HDPE pipes. The short term modulus computed and the long term modulus obtained from extrapolation was in good agreement with AASHTO recommended values. AASHTO recommended modulus values of 758 MPa (110,000 psi) and 152 Mpa (22,000 psi) for short-term and long-term conditions.

Figure 3.5: Variation of relaxation modulus with time (Hashash, 1991).

3.4.5 Comparison of power law models

Results obtained from power law models reported above were plotted on log scale as a function of time. In another study, Moore and Hu (1996) also compared different power law models to develop multi-Kelvin model for high density polyethylene pipes. The comparison shows that at 5% deflection, the Janson (1985) and Chua (1986) models

are roughly the same. Hashash's (1991) power law model at the same diametrical followed similar trend with time, but with lower values as shown in Figure 3.6. Hashash's (1990) power law equation (Equation 3.12) shows that the relaxation modulus diminishes with time. Chua's (1986) power law equation shows that pipe reaches a minimum limit of 52.6 MPa (7630 psi).

Figure 3.6: Comparison of various power law models (Moore and Hu, 1996).

3.5 Linear viscoelastic model

In order to conveniently illustrate the viscoelastic behavior of high density polyethylene pipes, a multi-Kelvin linear viscoelastic model was developed by Moore and Hu (1996). This model consists of nine Kelvin elements with an independent spring having different moduli. The constitutive equation governing this multi-Kelvin model (as shown in Figure 3.7) can be written as (Moore and Hu, 1996):

$$\sigma(t) = E_0 \varepsilon_0 = E_i \varepsilon_i + \eta_i \dot{\varepsilon}_i \quad \dots\dots\dots (3.14)$$

$$\varepsilon(t) = \varepsilon_0(t) + \sum_{i=1}^9 \varepsilon_i(t) \quad \dots\dots\dots (3.15)$$

where ‘i’ denotes the Kelvin element number.

The elasticity and damping constants of the Kelvin elements other than first were chosen to be multiples of the elasticity and damping constants of the first Kelvin element, respectively. Instantaneous response of the pipe was predicted by using a independent spring. Each Kelvin element denoted the behavior over time.

Figure 3.7: Multi-Kelvin linear viscoelastic model (Moore and Hu, 1996).

In order to evaluate the time dependent response of HDPE pipes using sets of springs and dashpots, two different rheological models (small deflection rheology and large deflection rheology) were developed using finite element approach (Moore and Hu, 1996). Pipe was subjected to a constant strain rate to develop these models. Small deflection (S) rheology was developed by comparing results of Chua’s (1986) power law model and examining the relaxation data for 5% deflection. Results from Chua’s (1986) power law model were in good agreement with viscoelastic predictions obtained using the small deflection rheology. The study uncovered the fact that S rheology performs well only up to 3% vertical deflection of the pipe. Non-linearity was observed after

continuous increase of vertical deflection of pipe. This non-linear response may limit the use of small deflection (S) rheology beyond 3% of vertical deflection. For this reason, a large deflection (L) rheology was developed to characterize the response of pipe at larger deflection levels (Moore and Hu, 1996). This rheology was developed based on the DiFrancesco's (1993) relaxation tests providing reasonable predictions of response at larger vertical deflections (between 3% and 10%).

3.6 Nonlinear viscoelastic models

The nonlinear time dependant behavior of the HDPE pipe material has been investigated experimentally under a wide range of loading histories in compression (Moore and Zhang, 1998). Extensive experimental studies were conducted and various constitutive models were developed in order to characterize the nonlinear time dependent response of HDPE pipes under different conditions of creep and relaxation (Zhang and Moore, 1997). The stress-strain behavior was found to be highly nonlinear and rate dependent. Also, it was reported that nonlinear constitutive modeling is needed to predict the pipe response under parallel plate loading at large deflections where large strains develop at crown, invert, and spring lines (Moore and Zhang, 1998).

3.7 Summary

Based on the above review, it is evident that a number of power law models can be used to predict the creep behavior of HDPE pipes. As shown in Figure 3.6, a number of these models predict similar behavior. The model developed by Hashash (1991) predicts the lower values of modulus (i.e., higher displacements), and as such the model would provide more conservative results. Moreover, the predictions from their model are close to AASHTO recommended values. Therefore, the model developed by Hashash (1991) was used in the present study. This model can be expressed as:

$$E(t) = 67779t^{-0.0859} \dots\dots\dots (3.16)$$

where E(t) is expressed in psi and t is measured in hours.

CHAPTER 4

DOUBLE-WALL AND SINGLE-WALL CORRUGATED PIPES

4.1 Introduction

According to American Association of State Highway and Transportation Officials (AASHTO), commercially available polyethylene pipes have been mainly classified as follows (AASHTO M294, 2002; Plastic Pipe Institute, 2005):

- Type S – the pipe has a circular cross-section with corrugated outer surface and smoother inner surface as shown in the Figure 4.1a. This pipe is also known as double-wall or dual-wall corrugated HDPE pipe.
- Type C – the pipe has a circular cross-section with inner and outer corrugations as shown in the Figure 4.1b. This pipe is also known as single-wall corrugated HDPE pipe.
- Type D – the pipe has a circular cross-section with smooth inner and outer surface as shown in the Figure 4.1c.

The type of the pipe is selected based on the hydraulic requirements and their applications. With the HDPE pipes being flexible, light weight, durable and economical, they have become popular and are suitable for effective water management. Double-wall corrugated HDPE pipes facilitate maximum hydraulic efficiency because of its smooth interior liner. Single-wall corrugated pipes are suitable for drainage applications as their wall profile facilitates controlled drainage. The published literature on the performance of single and double-wall corrugated HDPE pipes is discussed in the present chapter. Moreover, this chapter describes the modeling techniques of single and double-wall corrugated pipes to evaluate the long term performance using finite element approach.

(a) Section A-A of double-wall corrugated HDPE pipe (Type S)

(b) Section A-A of single-wall corrugated HDPE pipe (Type C)

(c) Section A-A of rectangular plain corrugated HDPE pipe (Type D)

Figure 4.1: Profile geometries of corrugated high density polyethylene pipe.

4.2 Parameters influencing pipe stiffness

Standard tests such as ASTM D 2412 test are used to determine the pipe stiffness and deflection of the unconfined pipe. Pipe stiffness can be defined as the load required in displacing the pipe by one unit. When evaluating the stiffness properties of corrugated HDPE pipes in accordance with AASHTO M-252 and/or AASHTO M-294, a sample of the pipe is loaded at a constant strain rate using two parallel plates as shown in the Figure 4.2. The deflection and the load applied are measured over time. Pipe stiffness is evaluated using the relationship between deflection (δ) measured and the load applied (P) as given below (Moser, 1990):

$$K = \frac{P}{\delta} \quad \dots\dots\dots (4.1)$$

where

K = pipe stiffness (psi),

P = load applied per unit length of pipe (lb),

δ = vertical change in the pipe diameter (in).

Figure 4.2: Parallel plate loading test or ASTM D 2412 test.

By using equation 4.1, the deflection of the pipe has been linked to the applied and structural stiffness of the pipe as shown below (Solenio, 2005):

$$\delta = \frac{Load}{(Material \ Stiffness)(Geometric \ Stiffness)} \dots\dots\dots (4.2)$$

For a buried HDPE pipe, vertical soil arching adds an advantage by reducing the load induced on the pipe. The structural properties also play an important role in reducing the deflections of the pipe. The structural properties like material stiffness (E) and geometric stiffness (I/r^3) of the pipe can be evaluated based on the response of the unsupported pipe. Backfill and the native soil properties also play a vital part in the evaluating the performance of the pipe. Material properties in Equation 4.2 include density (ρ) and modulus (E) of the pipe. Geometric properties include the wall moment of inertia (I), the distance from the inside surface to the neutral axis (c), and the cross-sectional area (A_s) (Solenio, 2005). Usually, these details are provided by the pipe manufacturer. In the present study, the details of pipe geometry for single-wall and dual-wall corrugated HDPE pipes were selected on the basis of available information (ADS, 2006) as shown in Table 4.1 and Table 4.2. These specifications do not include elastic modulus of the pipe material or its effect extended over time.

For an unconfined pipe, stiffness of pipe increases with an increase of the moment of inertia, I or a decrease of the radius, r. The wall profile and the wall thickness influence the stiffness of the pipe by increasing or decreasing the wall moment of inertia. Therefore, the wall profile of the dual-wall corrugated pipes (shown in the Figure 4.1a) is expected to provide higher values of moment of inertia (I) resulting in reduced pipe deflections and an improvement in structural rigidity of the pipe.

Table 4.1: Section properties of single-wall corrugated HDPE pipes (ADS Inc., 2006).

Nominal Diameter	Inside Diameter Average	Outside Diameter, Average	Minimum Pipe Stiffness @ 5% Deflection	Weight kg./6m (lbs./20 ft)	Area of section mm ² /mm	Moment of inertia (I) cm ⁴ /cm	Distance inside diameter to neutral axis (C)
75 mm (3")	79 mm (3.12")	93 mm (3.66")	240 kN/m ² 35 psi	1.97 kg (4.40 lbs)	1.88 (0.074 in ² /in)	0.0066 (0.0004 in ⁴ /in)	2.97 (0.117 in)
100 mm (4")	102 mm (4.03")	120 mm (4.71")	240 kN/m ² 35 psi	2.81 kg (6.30 lbs)	2.06 (0.081 in ² /in)	0.0169 (0.001 in ⁴ /in)	4.29 (0.169 in)
125 mm (5")	127 mm (4.99")	148 mm (5.81")	240 kN/m ² 35 psi	4.43 kg (9.90 lbs)	2.34 (0.092 in ² /in)	0.0229 (0.0014 in ⁴ /in)	4.78 (0.188 in)
150 mm (6")	151 mm (5.95")	176 mm (6.92")	240 kN/m ² 35 psi	6.39 kg (14.30 lbs)	3.15 (0.124 in ² /in)	0.0459 (0.0028 in ⁴ /in)	6.22 (0.245 in)
200 mm (8")	207 mm (8.14")	240 mm (9.45")	240 kN/m ² 35 psi	11.02 kg (24.60 lbs)	3.25 (0.128 in ² /in)	0.1082 (0.0066 in ⁴ /in)	8.08 (0.318 in)
250 mm (10")	255 mm (10.05")	300 mm (11.83")	240 kN/m ² 35 psi	16.23 kg (36.30 lbs)	3.48 (0.137 in ² /in)	0.1966 (0.0120 in ⁴ /in)	11.20 (0.441 in)
300 mm (12")	306 mm (12.04")	366 mm (14.41")	345 kN/m ² 50 psi	26.56 kg (59.40 lbs)	5.23 (0.206 in ² /in)	0.5211 (0.0318 in ⁴ /in)	15.88 (0.625 in)
375 mm (15")	378 mm (14.87")	444 mm (17.49")	290 kN/m ² 42 psi	36.39 kg (81.40 lbs)	4.67 (0.184 in ² /in)	0.5440 (0.0332 in ⁴ /in)	14.86 (0.585 in)
450 mm (18")	454 mm (17.86")	534 mm (21.04")	275 kN/m ² 40 psi	53.51 kg (119.70 lbs)	6.22 (0.245 in ² /in)	1.062 (0.0648 in ⁴ /in)	20.02 (0.788 in)
600 mm (24")	600 mm (23.61")	699 mm (27.50")	235 kN/m ² 34 psi	100.33 kg (224.40 lbs)	8.99 (0.354 in ² /in)	2.161 (0.1318 in ⁴ /in)	25.38 (0.999 in)

*1 psi = 6.89 kN/m²; 1pcf = 16 Kg/m³; 1 inch = 2.54 cm

Table 4.2: Section properties of dual-wall corrugated HDPE pipes (ADS Inc., 2006).

Nominal Diameter	Inside diameter	Outside diameter Average	Inner Liner Thickness, Minimum	Minimum Pipe stiffness at 5% deflection	Weight kg./6m (lbs/20ft)	Area mm²/mm	" I " cm⁴/cm	" C " mm
100 mm (4")	104 mm (4.10")	120 mm (4.78")	0.5 mm (0.020")	340 kN/m ² 50 psi	4.08 kg (9.00 lbs)	1.59 (0.063 in ² /in)	0.01 (0.0006 in ⁴ /in)	3.06 (0.12 in)
150 mm (6")	152 mm (6.00")	176 mm (6.92")	0.5 mm (0.020")	340 kN/m ² 50 psi	7.71 kg (17.00 lbs)	2,15 (0.085 in ² /in)	0.035 (0.0021 in ⁴ /in)	4.94 (0.19 in)
200 mm (8")	200 mm (7.90")	233 mm (9.11")	0.6 mm (0.024")	340 kN/m ² 50 psi	13.97 kg (30.80 lbs)	2.75 (0.108 in ² /in)	0.078 (0.005 in ⁴ /in)	6.36 (0.25 in)
250 mm (10 ")	251 mm (9.90")	287 mm (11.36")	0.6 mm (0.024")	340 kN/m ² 50 psi	20.96 kg (46.20 lbs)	3.48 (0.137 in ² /in)	0.134 (0.008 in ⁴ /in)	7.58 (0.30 in)
300 mm (12")	(308 mm) (12.15")	367 mm (14.45")	0.9 mm (0.035")	345 kN/m ² 50 psi	29.60 kg (65.20 lbs)	5.5 (0.217 in ² /in)	0.574 (0.035 in ⁴ /in)	10.92 (0.43 in)
375 mm (15")	380 mm (14.98")	448 mm (17.57")	1.0 mm (0.039")	290 kN/m ² 42 psi	42.00 kg (92.50 lbs)	6.91 (0.272 in ² /in)	0.901 (0.055 in ⁴ /in)	13.21 (0.52 in)
450 mm (18")	459 mm (18.07")	536 mm (21.20")	1.3 mm (0.051")	275 kN/m ² 40 psi	58.38 kg (128.60 lbs)	6.93 (0.273 in ² /in)	1.327 (0.081 in ⁴ /in)	14.48 (0.057 in)
600 mm (24")	612 mm (24.08")	719 mm (27.80")	1.5 mm (0.059")	235 kN/m ² 34 psi	99.93 kg (220.30 lbs)	8.23 (0.324 in ² /in)	2.245 (0.137 in ⁴ /in)	18.8 (0.74 in)
750 mm (30")	762 mm (30.00")	892 mm (35.10")	1.5 mm (0.059")	195 kN/m ² 28 psi	140.00 kg (308.6 lbs)	9.6 (0.378 in ² /in)	4.539 (0.277 in ⁴ /in)	21.84 (0.86 in)
900 mm (36")	914 mm (36.00")	1059 mm (41.70")	1.7 mm (0.067")	150 kN/m ² 22 psi	180.00 kg (396.8 lbs)	10.19 (0.401 in ² /in)	6.555 (0.400 in ⁴ /in)	25.4 (1.00 in)
1050 mm (42")	1054 mm (41.40")	1212 mm (47.70")	1.8 mm (0.070")	140 kN/m ² 20 psi	230.00 kg (570.10 lbs)	11.64 (0.458 in ² /in)	9.373 (0.572 in ⁴ /in)	30.73 (1.21 in)
1200 mm (48")	1209 mm (47.60")	1361 mm (53.60")	1.8 mm (0.070")	125 kN/m ² 18 psi	283.50 kg (625.00 lbs)	12.58 (0.495 in ² /in)	9.341 (0.570 in ⁴ /in)	29.72 (1.17 in)
1500 mm (60")	1512 mm (59.5")	1684 mm (66.3")	1.8 mm (0.070")	95 kN/m ² 14 psi	410.00 kg (903.90 lbs)	14.68 (0.578 in ² /in)	14.09 (0.860 in ⁴ /in)	33.66 (1.32 in)

*1 psi = 6.89 KN/m²; 1pcf = 16 Kg/m³; 1 inch = 2.54 cm

4.3 Double-wall corrugated HDPE pipes

Double-wall corrugated pipes have a circular cross-section with corrugated outer surface and smoother inner surface as shown in the Figure 4.3. The outer corrugations of the double-wall corrugated pipes play an important role in structural integrity while the inner wall facilitates improving hydraulic efficiency. The nominal diameter of dual-wall pipe ranges from 4 inch (100 mm) to 60 inch (1500 mm) [ADS, 2006]. The increasing use of double-wall corrugated HDPE pipes because of its exceptional advantages has compelled the industry to gain in-depth knowledge. Given below are few advantages gained with the use of dual-walled corrugated HDPE pipes (ADS, 2006):

- Improve the service life of the pipe.
- Facilitate the long term hydraulic efficiency because of its smoother inner surface.

(a) Double-wall corrugated HDPE pipe

(b) Geometry of the dual-wall corrugated HDPE pipe

Figure 4.3: Profile view of double-wall corrugated HDPE pipe.

4.3.1 Performance of double-wall corrugated HDPE pipes

A study has been performed to investigate the performance of dual-wall HDPE pipes (Fleckenstein and Allen, 1990). Cuts or tears were noticed inside the pipe. The reason of cuts or tears was assumed to be improper backfilling and unequal load distribution on the pipe wall. Therefore, Fleckenstein (1993) suggested few guidelines for successful installation of HDPE pipes:

- Pipes installation has to be done according to ASTM 2321 standards
- Use of granular backfill for at least one foot cover above the pipe.
- Continuous long term inspection is suggested for all pipe installations.

In another study (Hashash and Selig, 1990) a continuous supervision (for up to 2 years) was made to examine the structural performance of 24 inch (600 mm) dual-wall corrugated HDPE pipes. Pipe was buried below the center of 100 ft high (30.5 m) embankment. A 4.3% decrease in the vertical diameter and 0.6% increase in the horizontal diameter have been observed. Also, a circumferential shortening of 1.6% was observed. Moreover, a compressive strain of 1.4% and the bending strain of 0.3% were observed in the pipe wall. A finite element approach was later followed to predict the long term performance of the pipe. A time dependent model was recommended to correctly predict the long term response of the soil-pipe system (Hashash and Selig, 1990). This study helped in estimating hoop thrust and predicting stresses in the pipe wall.

Moore (1995) investigated the response of corrugation and internal lining of profiled high density polyethylene pipes in hoop compression. In his study, a corrugated section with a smooth internal liner (Type S) was tested. It has been observed that axial stresses do develop at certain locations in the pipe profile. The junction between the liner and corrugation was observed to have largest axial tensions. The local bending observed in corrugated lined pipe was confined to liner. Corrugated unlined pipe did not lead to the same phenomenon. Shear failure of the soil placed in the corrugation valley was expected

to increase the local bending stress, whereas the stiffening of soil material with increasing applied stress was expected to reduce the local stresses in the pipe (Moore, 1995).

In a similar study, field performance of large diameter high density polyethylene pipes under deep soil fill was investigated (Sargand et al, 2000). The study proved that response of HDPE pipes under constant soil fill loading is more controlled by stress relaxation than by the creep.

4.3.2 Modeling of the double-wall corrugated HDPE pipes

This section provides a method to model double-wall corrugated HDPE pipes in finite element analysis. The geometric details are obtained from Table 4.2. The procedure followed to model double-wall pipes in the present study is described below using a 24 in (600 mm) pipe diameter:

For a 24 inch nominal pipe diameter:

Min. Pipe stiffness @ 5% deflection (K)	= 34 psi (235 kN/m ²)
Inside diameter (ID)	= 24.08 in. (612 mm)
Outside diameter (OD)	= 27.80 in. (719 mm)
Moment of inertia (I _d)	= 0.137 in ⁴ /in. (2.245 cm ⁴ /cm)
Distance from inner wall to neutral axis (C)	= 0.74 in. (18.8 mm)
Flexural modulus of the pipe (E _d)	= 110,000 psi

$$\begin{aligned} \text{Mean diameter (D}_{\text{mean}}) &= \text{Inside diameter (D}_{\text{int}}) + 2C && \dots\dots\dots (4.3) \\ &= 24.08 + 2(0.74) = 25.56 \text{ in.} \end{aligned}$$

Therefore, mean radius (r_{mean}) = 12.78 in.

As wall corrugations are difficult to model in buried pipes, the geometry of the pipe is idealized to be a rectangular plain section without changing the structural stiffness

as shown in the Figure 4.4. The thickness selected is the difference between the inside and outside radius.

$$t = \frac{OD - ID}{2} \dots\dots\dots (4.4)$$

where

t = thickness of pipe (in)

OD = Outside diameter of pipe (in)

ID = Inside diameter of pipe (in)

Therefore, the thickness for a 24 inch pipe diameter can be expressed as:

$$t = \frac{27.80 - 24.08}{2} = 1.86 \text{ in.}$$

As the cross-section of the pipe is idealized as a rectangular section with above thickness, the moment of inertia can be calculated as:

$$I = \frac{t^3}{12} = \frac{1.86^3}{12} = 0.5362 \text{ in}^4 / \text{in.}$$

(a) Double-wall corrugated HDPE pipe

(b) Section A-A of double-wall pipe

(c) Idealized section of double-wall pipe

where

ID = Inside Diameter

OD = Outside Diameter

C = Distance from inside diameter to neutral axis

$t = \text{thickness of the idealized rectangular section} = t = \frac{OD - ID}{2}$

Figure 4.4: Typical cross section of double-wall corrugated HDPE pipe.

Since the geometric stiffness of pipe was changed in order to simplify the modeling process, material stiffness has to be changed to maintain the pipe stiffness. The modification is shown below:

$$E_d I_d = E_p I_p \quad \dots\dots\dots (4.5)$$

where

E_d = Elastic modulus of corrugated section for dual-wall pipe.

E_p = Elastic modulus of idealized rectangular section.

I_d = Moment of Inertia of corrugated section for dual-wall pipe.

I_p = Moment of Inertial of idealized rectangular section.

Therefore,
$$E_p = \frac{E_d I_d}{I_p} = \frac{110000 \times 0.137}{0.5362} = 28105 \text{ psi}$$

Similar procedure was employed to determine the appropriate thickness and material stiffness for various sizes of pipes used in this study. Change in the material stiffness over time can be modeled using the time dependent relationship obtained from Hashash's creep model (Hashash, 1991). Table 4.3 shows the material properties used for double-wall corrugated HDPE pipes in the present study.

4.4 Single-wall corrugated HDPE pipes

For a single-wall corrugated HDPE pipe, the pipe has a circular cross-section with inner and outer corrugations as shown in the Figure 4.5. Generally, pipe sizes ranging from 3 inch (76 mm) to 24 inch (600 mm) are commercially available for the use. These pipes are effective for controlled drainage and other hydraulic applications with Manning's coefficient of 0.015 to 0.022 (ADS, 2006).

(a) Single-wall corrugated HDPE pipe

(b) Geometry of single-wall corrugated HDPE pipe

Figure 4.5: Profile view of single-wall corrugated HDPE pipe.

4.4.1 Performance of single-wall corrugated HDPE pipes

An experimental study was performed to evaluate the long term response of single-wall corrugated HDPE pipes subjected to external loading (Reddy and Ataoglu, 2002). Assuming a 7.5% vertical change in the diameter as the failure criterion, deflections were recorded over time. The results were extrapolated to estimate the service life of single-wall corrugated HDPE pipes

In another study, a two-dimensional plain strain finite element analysis was used (Mada, 2005) to ascertain the 5-year and 50-year performance of large diameter (18 inch and 24 inch) single-wall corrugated HDPE pipes using CLSM (Controlled Low Strength

Material) backfills. A static analysis was performed at 5-year and 50-year time periods with assumed lower pipe modulus to account for creep behavior. Pipe was modeled using plain strain elements. This study has shown that pipes did not meet failure criterion within the 50-year period when buried depth was less than 50 feet. The assumed pipe properties appear to be larger than the values reported in the literature (ADS, 2006). Since the previous study was based on static analysis with assumed properties, a new creep analysis (transient analysis) with actual pipe properties was performed in the present study.

4.4.2 Modeling of the single-wall corrugated HDPE pipes

This section provides a method to model single wall corrugated HDPE pipes in finite element analysis. The geometric details are obtained from Table 4.1. The procedure followed to model single-wall pipes in the present study is described below using a 24 inch (600 mm) pipe diameter:

For a 24 in nominal pipe diameter:

Minimum pipe stiffness at 5% deflection (K)	= 34 psi (235 KN/m ²)
Inside diameter (ID)	= 23.61 in. (600 mm)
Outside diameter (OD)	= 27.50 in. (699 mm)
Moment of inertia (I _s)	= 0.1318 in ⁴ /in. (2.161 cm ⁴ /cm)
Distance from inner wall to neutral axis (C)	= 0.999 in. (33.66 mm)
Flexural modulus of the pipe (E _s)	= 110,000 psi

$$\begin{aligned} \text{Mean diameter (D}_{\text{mean}}) &= \text{Inside diameter (D}_{\text{int}}) + 2C && \text{..... (4.6)} \\ &= 23.61 + 2(0.999) = 25.608 \text{ in.} \end{aligned}$$

Therefore, mean radius (r_{mean}) = 12.804 in.

As wall corrugations are difficult to model in buried pipes, the geometry of the pipe is idealized to be a rectangular plain section without changing the structural stiffness as shown in Figure 4.6. The thickness selected is the difference between the inside and outside radius.

$$t = \frac{OD - ID}{2} \dots\dots\dots (4.7)$$

where

t = thickness of pipe (in)

OD = Outside diameter of pipe (in)

ID = Inside diameter of pipe (in)

Therefore, the thickness for a 24 inch pipe diameter can be expressed as:

$$t = \frac{27.50 - 23.61}{2} = 1.945 \text{ in.}$$

As the cross-section of the pipe is idealized as a rectangular section with above thickness, the new moment of inertia is:

$$I = \frac{t^3}{12} = \frac{1.945^3}{12} = 0.6131 \text{ in}^4 / \text{in.}$$

(b) Single-wall corrugated HDPE pipe

(b) Section A-A of single-wall pipe

(c) Idealized section of single-wall pipe

where

ID = Inside Diameter

OD = Outside Diameter

C = Distance from inside diameter to neutral axis

$t = \text{thickness of the idealized rectangular section} = t = \frac{OD - ID}{2}$

Figure 4.6: Typical cross section of single-wall corrugated HDPE pipe.

Since the geometric stiffness of the pipe was changed in order to simplify the modeling process, material stiffness has to be changed to maintain the same pipe stiffness. The modification is shown below:

$$E_d I_d = E_p I_p \dots\dots\dots (4.8)$$

where

E_d = Elastic modulus of corrugated section for single-wall pipe.

E_p = Elastic modulus of idealized rectangular section.

I_d = Moment of Inertia of corrugated section for single-wall pipe.

I_p = Moment of Inertia of idealized rectangular section.

Therefore,

$$E_p = \frac{E_d I_d}{I_p} = \frac{110000 \times 0.1318}{0.6131} = 23647 \text{ psi}$$

Table 4.3 shows the properties of the pipes used in the present finite element study.

Table 4.3: Pipe properties used in the present study.

Properties	HDPE Pipe		
	24 inch Single-wall	24 inch Double-wall	48 inch Double-wall
Inside Diameter (ID)*	23.61 in.	24.08 in.	47.60 in.
Outside Diameter (OD)*	27.50 in.	27.80 in.	53.60 in.
Distance from the inner wall to the neutral axis (C)*	0.999 in.	0.74 in.	1.17 in.
Actual moment of Inertia - I_d (for a corrugated section)*	0.1318 in ⁴ /in	0.137 in ⁴ /in	0.570 in ⁴ /in
Actual Elastic Modulus - E_d (for a corrugated section)**	110,000 psi	110,000 psi	110,000 psi
Thickness of Idealized rectangular section (t)	1.945 in.	1.86 in.	3.0 in.
Moment of Inertia - I_p (Idealized rectangular section)	0.6131 in ⁴ /in	0.5362 in ⁴ /in	2.25 in ⁴ /in
Elastic modulus - E_p (Idealized rectangular section	23647 psi	28105 psi	27867 psi

* Values are obtained from Table 4.1 (for single-wall corrugated pipes) or Table 4.2 (for double-wall corrugated pipes).

** Modulus of HDPE pipe material.

CHAPTER 5

NUMERICAL METHODOLOGY

5.1 Introduction

During the past few years, finite element method has become a popular tool for solving a wide range of engineering problems. Applications range from simple deformation and stress analysis to complex non-linear analysis. With the latest developments in computer technology, the finite element techniques have become more cost-effective and user-friendly to solve complex problems. In the present research work, a well-known commercially available finite element package known as ‘ABAQUS’ was chosen to analyze the time-dependent behavior of buried high density polyethylene pipes. The present chapter provides an overview of different constitutive creep models available in ABAQUS and describes the methodology used in developing a time dependent pipe-soil model.

5.2 Finite element method (FEM)

The present section reviews few fundamentals of the Finite Element Method (FEM). In a finite element procedure, the actual geometry is discretized into number of small elements known as finite elements. Each element is connected to other element at their common points known as nodal points. The collection of these nodal points and finite elements is recognized as finite element mesh. In each of these finite elements, a displacement field is approximated by use of simple polynomials. As the actual geometry is divided into number of finite elements, a number of elemental equations are generated. In order to achieve an approximate solution of the behavior of actual geometry, these elemental equations are assembled to obtain global governing equations which can be expressed as (Cook et al., 2003):

$$[K]\{r\}=\{R\} \quad \dots\dots\dots (5.1)$$

where

$[K]$ = global stiffness matrix

$\{r\}$ = global displacement vector

$\{R\}$ = global load vector

Selection of these elements depends on thorough physical understanding of the problem (Cook et al., 2003). High speed computers are needed to perform finite element analysis of larger problems.

5.3 Recent studies

In another study performed (Moore, 1994), local strains were measured at springline of high density polyethylene pipes to test the effectiveness of three dimensional finite element analyses for parallel plate loading test. An efficient finite element procedure was developed (Moore, 1994) to study the elastic and viscoelastic behavior of soil-pipe interaction system. Stress and strain distribution in and around the pipe was computed. The circumferential and axial strain values predicted compared well with the experimental values. Elastic analysis provided reasonable predictions for HDPE pipes to estimate the circumferential and axial strains. Viscoelastic analysis was employed in order to estimate strain values at any location and to evaluate the load deflection response (Moore, 1994). Improvements in the constitutive and finite element models were suggested to produce better viscoelastic predictions and to evaluate the long-term structural performance of HDPE pipes under deep burial conditions (Moore, 1994).

In order to understand the effect of profile geometry on soil-pipe interaction three dimensional finite element analysis has been used (Dhar and Moore, 2001). Different HDPE pipe profiles were considered for the purpose of investigation such as lined corrugated, boxed, and tubular. Local bending was observed along the inner wall. Maximum tension near the liner-corrugation junction was noticed. Also hoop strains and cross-sectional area of the pipes were observed to be uniform and similar. Dhar and Moore (2001) suggest the use of two dimensional plane strain finite element analysis to

predict the overall pipe response. However, localized phenomenon was not completely predicted (Dhar and Moore, 2001).

Two-dimensional plane strain finite element analysis and three-dimensional finite element analysis has been performed (Schafer and McGrath, 2003) to determine the allowable fill heights over a buried thermoplastic pipe. Strain demands were computed from a two-dimensional plane strain model for a pipe buried under increasing fill depths. Ultimate strain capacity was determined from a three-dimensional finite element model of a pipe-soil segment under thrust and bending actions. Thrust strains were dominant as opposed to bending strains in the overall behavior the thermoplastic pipes in the embankment condition (Schafer and McGrath, 2003). Analysis of pipe-soil segment under increasing thrust and bending has been limited to local buckling of the pipe (Schafer and McGrath, 2003). The results compared were found to be within the strain limits of the newly adopted AASHTO design methods (Schafer and McGrath, 2003).

Two and three-dimensional finite element analysis has also been performed in another study (Brachman et al., 1996). Influence of boundary conditions of the tests conducted at Ohio University testing facility for small diameter high density polyethylene pipes has been investigated. Method of load application to the soil-pipe system has been considered as a key boundary condition. Also, the interaction between the pipe and surrounding soil was investigated for smooth and rough interfaces (Brachman et al., 1996). The pipe was surrounded by 25 mm crushed stone and underlain by compacted clay. Previous studies performed (Moore and Hu, 1995) reported that pipe deflections are primarily controlled by modulus of the backfill surrounding the pipe, and pipe modulus has little effect on pipe deflections. However, in the study performed by Brachman et al. (1996), it has been observed that stone backfill has a little effect on the pipe deflections. Also, Brachman et al. (1996) concluded that stress distribution imposed by finite, rigid loading is different from uniform distribution expected. Also, it has been found (Brachman et al., 1996), when backfill surrounding the pipe yields at small loads, the granular material starts behaving like a beam. This may result in larger pipe deflections

as the lateral support is reduced. Failure in the soil was observed in larger load conditions (Brachman et al., 1996).

Several other finite element studies have been reported in the literature (Hashash and Selig, 1990; Moore, 1995; Brachman et al., 2000; Sargand and Masada, 2000; Dhar et al., 2002; Suleiman, 2002).

5.4 Geometry

Finite element analyses of soil-structure interaction problems are different from simple deformation and stress analysis. In a soil-structure interaction problem, different element types are used to model the geometry of soil-structure interaction system. Also, the geometry of structure can be linear or nonlinear. Moreover, material properties of soil and/or structure can be linear or nonlinear.

Table 5.1 shows different element types used in previous studies. In the present study, soil surrounding the pipe was modeled as a homogeneous soil using two-dimensional deformable solid elements. Four-noded bilinear plane strain quadrilateral (CPE4) elements were used to model the soil geometry. In the finite element analysis of soil-pipe interaction system, the pipe can be modeled using beam elements or plane strain elements. Beam elements are capable of accommodating shear, moment and thrust whereas the plane strain elements are capable of handling only deformations.

Table 5.1: Element types used in previous studies to model the soil-pipe geometry.

Author	Pipe geometry	Soil
Hashash and Selig (1990)	Beam-column elements	4-noded quadrilateral elements
Moore (1995)	6-noded triangular elements	6-noded triangular elements
Zhang and Moore (1998)	6-noded triangular elements	6-noded triangular elements
Brachman et al. (2000)	8-noded rectangular elements	6-noded triangular elements
Sargand and Masada (2000)	Beam elements	Quadrilateral elements
Dhar et al. (2002)	2-noded beam-column elements	6-noded continuum elements
Suleiman (2002)	Beam column elements	Quadrilateral elements

Several finite element analyses were performed on unconfined rings to select an appropriate element type to model the pipe geometry. For the purpose of this preliminary study, pipe sizes ranging from 24 inch (600 mm) to 60 inch (1500 mm) were chosen. Pipes were assumed to have a rectangular cross-section and different r/t (radius-to-thickness ratio) values. Numerical results obtained from the use of a beam formulation and plane strain formulations are discussed in this section. Two-point parallel plate loading configuration was considered. Pipe deflections found from each formulation were compared to deflections calculated by using ring deflection theory (Equation 2.5). The comparison helps in selecting appropriate element types to model the geometry of buried pipes. Also, limits for the use of ring deflection theory were determined.

For a 24 inch (600 mm) pipe, the thickness of pipe was varied from 0.4 inch (10 mm) to 6.0 inch (150 mm) with r/t (mean radius to thickness) values ranging from 30 to 2.0. FEMAP (FEMAP, 2002) was used to model the geometry. A load of 20 lbs was applied at the crown of the pipe while constraining the bottom of pipe. A material modulus (E) of 110,000 psi was used.

For a 60 inch (1500 mm) pipe, the thickness of the pipe was varied from 1 inch to 15 inch with r/t (mean radius to thickness) values ranging from 30 to 2.0. A load of 50 lbs was applied at the crown of the pipe with material modulus (E) to be 65,000 psi.

Figure 5.1 shows the comparison of theoretical deflections, $\delta_{\text{Theoretical}}$ (obtained by using Equation 2.5) and deflections obtained by using beam formulation, δ_{Beam} for two different pipe sizes – 24 inch (600 mm) and 60 inch (1500 mm). Results show that δ_{Beam} is within 5% of $\delta_{\text{Theoretical}}$ for the r/t (effective radius/ thickness) values above 6.0 (approximately). Figure 5.2 shows with the percentage error between $\delta_{\text{Theoretical}}$ and δ_{Beam} . In order to check the above results, various pipe sizes with different r/t values were analyzed. Figure 5.3 shows the comparison for different r/t values by using different pipe sizes. The error between $\delta_{\text{Theoretical}}$ and δ_{Beam} was same for pipes with the same r/t value irrespective of size, loading or material stiffness used.

Figure 5.1: Comparison of δ_{Beam} and $\delta_{\text{Theoretical}}$ for 24 in and 60 in pipes.

Figure 5.2: Percentage error between $\delta_{\text{Theoretical}}$ and δ_{Beam} for an unconfined pipe.

Figure 5.3: Comparison of δ_{Beam} and $\delta_{Theoretical}$ for different r/t values.

Figure 5.4 shows a comparison of pipe deflections for a 24 inch (600 mm) pipe obtained using beam formulation and plane strain formulation. Percentage error in pipe deflections obtained using plane strain elements for unconfined pipes was higher for larger r/t values. In this study, beam elements were used in all other analyses presented later in this report.

Effect of confinement around the pipe was studied before the development of the final model for the evaluation of long term performance of buried pipes. Figure 5.5 shows the model used to study the effect of confinement. Figure 5.6 shows the percentage of deformation for a 24 inch (600 mm) pipe for an r/t value of 10. Results from Figure 5.7 show a decrease in the deformation with the increase in the modulus of confinement material.

Figure 5.4: Comparison of $\delta_{\text{Theoretical}}$, δ_{Beam} and $\delta_{\text{Plane strain}}$ for a 24 in unconfined pipe.

Figure 5.5: Numerical model used to study the effect of confinement.

Figure 5.6: Effect of confinement.

Figure 5.7: Percentage error between δ_{Beam} and $\delta_{\text{Plane strain}}$.

In order to evaluate the creep behavior, two dimensional transient analyses were performed on 24 inch (600 mm) and 48 inch (1200 mm) diameter single-wall and double-wall corrugated pipes for a time period of 50 years. Different fill heights ranging from 10 feet to 60 feet were considered with varying trench width ratios varying from 1.5 to 2.5. Trench width ratio is defined as the ratio of trench width to nominal pipe diameter.

5.5 Element type and mesh

ABAQUS consists of several meshing techniques such as structured, swept, and free to efficiently mesh the geometry of the model. In this study, a free meshing technique with quadrilateral element shapes was used to mesh the soil geometry. Free meshing technique was used to allow more flexibility than structured meshing. Four-noded bilinear plane strain quadrilateral (CPE4) elements were used to model the soil geometry and 2-noded linear beam elements (B21) were used to model the geometry of the pipe. Figure 5.8 illustrates the meshed geometry of the soil-pipe system.

Figure 5.8: Finite element mesh for the soil-pipe system.

A brief discussion on the mathematical formulation of these elements is discussed below:

5.5.1 Mathematical formulation of two-noded beam element

A two dimensional beam element has a node at either end. Each node has two degrees of freedom, namely lateral translation and rotation as shown in Figure 5.9 (Cook et al., 2003). In this figure, u_1 and u_2 represent lateral translations and θ_1 and θ_2 represent rotations of the beam element. Figure 5.9 also illustrates the nodal loads and moments associated with nodal points. If ‘E’ was the elastic modulus of the material and ‘I’ was the moment of inertia, the element stiffness matrix for a two dimensional beam element can be expressed as given below (Cook et al., 2003):

$$[k]_{4 \times 4} = \begin{bmatrix} \frac{12EI}{L^3} & \frac{6EI}{L^2} & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ \frac{6EI}{L^2} & \frac{4EI}{L} & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ \frac{6EI}{L^2} & \frac{2EI}{L} & -\frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix}_{4 \times 4} \quad \dots\dots\dots (5.2)$$

$$\{q\} = \begin{Bmatrix} u_1 \\ \theta_1 \\ u_2 \\ \theta_2 \end{Bmatrix}_{4 \times 1} \quad \dots\dots\dots (5.3)$$

Figure 5.9: Two-noded beam element.

More details of the mathematical formulation can be found elsewhere (Cook et al., 2003). The governing element equations can be expressed as:

$$[K] \{q\} = \{Q\} \quad \dots\dots\dots (5.4)$$

where

$[K]$ = element stiffness matrix

$\{q\}$ = displacement vector = $\{u_1 \ \theta_1 \ u_2 \ \theta_2\}^T$

$\{Q\}$ = load vector = $\{P_1 \ M_1 \ P_2 \ M_2\}^T$

5.5.2 Mathematical formulation of bilinear quadrilateral element

In the formulation of bilinear quadrilateral elements, displacements at a given point can be expressed by using interpolations. Figure 5.10 illustrates the global coordinate system and local coordinate system for a four-noded isoparametric element.

The displacements at any point can be expressed as:

$$u = \sum_{i=1}^4 N_i u_i \quad \dots\dots\dots (5.5)$$

$$v = \sum_{i=1}^4 N_i v_i \quad \dots\dots\dots (5.6)$$

where ‘ u_i ’ and ‘ v_i ’ represent the nodal displacements in the x- and y- directions, and
‘ N_i ’ represents the interpolation functions in the local coordinate system
(Figure 5.10).

Equations (5.5) and (5.6) can also be expressed in matrix form as shown below:

$$\{U\} = [N]\{q\} \quad \dots\dots\dots (5.7)$$

or

$$\begin{Bmatrix} u \\ v \end{Bmatrix} = \begin{bmatrix} N_1 & 0 & N_2 & 0 & N_3 & 0 & N_4 & 0 \\ 0 & N_1 & 0 & N_2 & 0 & N_3 & 0 & N_4 \end{bmatrix} \begin{Bmatrix} u_1 \\ v_1 \\ u_2 \\ v_2 \\ u_3 \\ v_3 \\ u_4 \\ v_4 \end{Bmatrix} \quad \dots\dots\dots (5.8)$$

where N_i ($i = 1$ to 4) are interpolation functions which can be written as:

$$N_i = \frac{1}{4}(1 + xx_i)(1 + yy_i) \quad \dots\dots\dots (5.9)$$

where x and y correspond to the local coordinate system.

(a) Global coordinate system

(b) Local coordinate system

Figure 5.10: Four-noded isoparametric element (Desai and Abel, 1972).

The constitutive relationship can be expressed as:

$$\{\sigma\} = [C]\{\varepsilon\} \quad \dots\dots\dots (5.10)$$

where $\{\sigma\}$ represents the stress vector = $\{\sigma_{xx} \ \sigma_{yy} \ \tau_{xy}\}^T$,

$$[C] \text{ is the constitutive matrix.} = \begin{bmatrix} C_{11} & C_{12} & C_{13} \\ C_{21} & C_{22} & C_{23} \\ C_{31} & C_{32} & C_{33} \end{bmatrix},$$

and $\{\varepsilon\}$ represents the strain vector = $\{\varepsilon_{xx} \ \varepsilon_{yy} \ \varepsilon_{xy}\}^T = [B]\{q\}$.

where $[B]$ is the strain-displacement transformation matrix.

The strain vector $\{\varepsilon\}$ can be derived by using the strain-displacement relationships as shown below (Cook et al., 2003):

$$\varepsilon_{xx} = \frac{\partial u}{\partial x} \quad \dots\dots\dots (5.11)$$

$$\varepsilon_{yy} = \frac{\partial v}{\partial y} \quad \dots\dots\dots (5.12)$$

$$\gamma_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \quad \dots\dots\dots (5.13)$$

where ε_{xx} = normal strain along x-axis,

ε_{yy} = normal strain along y-axis, and

γ_{xy} = shear strain.

By using the energy principles, the general expression for potential energy can be expressed as follows (Cook et al, 2003):

$$\pi_p = \iiint_R U(u, v) dv - \iiint_R (\bar{X}u + \bar{Y}v) dv - \iint_{S_1} (\bar{T}_x u + \bar{T}_y v) dS - \sum (P_{ix} u_i + P_{iy} v_i) \quad \dots (5.14)$$

where $U(u, v)$ = strain energy density = $\frac{1}{2} \epsilon \sigma$,

$\bar{X}$ and $\bar{Y}$ are the body forces,

$\bar{T}_x$ and $\bar{T}_y$ are surface tractions or surface loading per unit area,

S represents the portion of the body on which the surface traction is applied,

u, v represent the nodal displacements, and

P_i denotes the load acting at node 'i'.

Equilibrium equations derived by minimizing the potential energy functional, π_p can be expressed as:

$$[K]\{q\} = \{Q\} \quad \dots\dots\dots (5.15)$$

where $[K]$ = Element stiffness matrix = $\iiint_V \{B\}^T [C] \{B\} dv$,

$[C]$ = Constitutive matrix (Desai and Siriwardane, 1984)

$\{Q\}$ = Element load vector

$$= \left\{ \iiint_V [N]^T \{\bar{X}\} dv + \iint_S [N]^T \{\bar{T}\} dS - \iiint_V [B]^T \{\sigma_o\} dv + \{P\} \right\},$$

and $\{q\}$ = displacement vector = $\{u_1 \ v_1 \ u_2 \ v_2 \ u_3 \ v_3 \ u_4 \ v_4\}^T$.

5.6 Loading, boundary conditions and step sequence

In a previous study (Brachman et al., 1996), it has been reported that method of load application to the soil-pipe system forms an important boundary condition in finite element analysis of buried pipes. In the present study, two different loading conditions were used. They were:

1. Self weight of the soil.
2. Self weight of the soil + HS-20 loading at the ground surface.

Body forces were applied throughout the soil medium to account for the self weight of the soil. A pressure of 105 psi was applied on a rectangular strip of 22 inch x 7 inch (56 cm x 18 cm) to account for HS-20 external loading as shown in Figure 5.11 (Watkins, 1999; Mada, 2005).

Adjacent soil elements were tied to nodes of the pipe elements at their common nodal points using tie constraints (ABAQUS, 2006). The nodal points at soil-pipe interface were tied to act as a single composite soil-pipe element. Figure 5.12 shows the boundary conditions applied to the geometry of the model. It may be even necessary in few instances to use interface elements between soil and pipe to obtain a thorough understanding of the soil-pipe system (Brachman et al., 1996). These interface elements have essentially no thickness, but permits slip between the soil and pipe.

Figure 5.11: HS-20 tire pressure (Mada, 2005).

Figure 5.12: Boundary conditions of the soil-pipe system.

To analyze the pipe-soil model for a period of 50 years, several visco-steps were defined. The step sequence provided a convenient way to capture deflections at various time intervals during the course of the analysis. An instantaneous load was applied after which the pipe was allowed to creep for a period of 50 years in subsequent steps at regular time intervals. The 50-year analysis was divided into five steps as given below:

Step 1: Instantaneous application of the load.

Step 2: Pipe was allowed to creep for a period of one hour at regular time intervals of three minutes.

Step 3: Pipe was allowed to creep for a period of 24 hours at regular intervals of 1 hour.

Step 4: Pipe was allowed to creep for a period of one month at regular intervals of one day.

Step 5: Pipe was allowed to creep for a period of 50 years at regular intervals of one month (30 days) .

5.7 Material

Figure 5.13 and Table 5.2 show the material properties used in the present study. These values were chosen from the literature. The model developed by Hashash (1991) was used in the present study. This model can be expressed as:

$$E(t)=67,779t^{-0.0859} \dots\dots\dots (5.16)$$

where modulus, E(t) is in psi and time, t is in hours.

Several creep models are available in ABAQUS based on different theories developed over the years. Power law model is simple to use (Arvidsson and Gronvall, 2004; ABAQUS, 2006). The power law models consist of two versions of hardening: time-hardening and strain-hardening. The time-hardening creep law is the simplest and very convenient to use when the stress in the material remains constant. The strain-hardening creep law is suitable under fluctuating load (Arvidsson and Gronvall, 2004;

ABAQUS, 2006). However, both the versions of power-law are good for low stresses (Arvidsson and Gronvall, 2004). Hyperbolic sine creep law is applicable if the stress in the material exceeds the yield stress (Arvidsson and Gronvall, 2004).

Figure 5.13: Elastic parameters used in the finite element analysis.

Table 5.2: Material properties used in the present study.

Property/ Material		HDPE pipe	Backfill soil	Insitu soil	CLSM
Elastic Modulus, E (psi)		110,000*	2,000	800	1,000
Poisson's ratio, ν		0.46	0.3	0.3	0.3
Mass density, ρ (pcf)		60	125	115	140
Creep parameters for Power-law Model	A	1.267354e-06	-	-	-
	n	1	-	-	-
	m	-0.9141	-	-	-

Note: Additional information on the properties of pipes used in the finite element analysis is shown in Table 4.3.

* Short term modulus of HDPE pipe material

In the time-hardening creep law, the creep strain rate, $\dot{\varepsilon}^{cr}$ depends on deviatoric stress, σ and time, t (Arvidsson and Gronvall, 2004; ABAQUS, 2006). The creep strain rate in the time-hardening form has been described as given below (ABAQUS, 2006):

$$\dot{\varepsilon}^{cr} = A \sigma^n t^m \quad \dots\dots\dots (5.17)$$

where A , n , and m are constants that depend on the temperature. These constants can be determined experimentally. ‘ A ’ determines the level of overall creep deformation in the material. The value of ‘ A ’ may be very small for typical creep strain rates. However, if ‘ A ’ is less than 10^{-27} , numerical difficulties may result (Arvidsson and Gronvall, 2004; ABAQUS, 2006). The parameter ‘ n ’ describes how the creep rate depends on stress level whereas ‘ m ’ changes the curvature of the creep curve. The value of ‘ n ’ is always positive and the value of ‘ m ’ always lies between -1 and 0. Table 5.3 shows the limits of A , n and m to avoid numerical difficulties (Arvidsson and Gronvall, 2004).

Table 5.3: Values of A , n and m (Arvidsson and Gronvall, 2004; ABAQUS, 2006).

Variables	Values
A	$> 10^{-27}$
n	> 0
m	$-1 < m < 0$

In the strain-hardening creep law, the creep strain rate, $\dot{\varepsilon}^{cr}$ depends on the deviatoric stress and strain. The creep strain rate in the strain-hardening form can be described as (ABAQUS, 2006):

$$\dot{\varepsilon}^{cr} = \left(A (\sigma^{cr})^n [(m+1) \varepsilon^{cr}]^m \right)^{\frac{1}{m+1}} \quad \dots\dots\dots (5.18)$$

where A , m and n are constants that depend on temperature.

In the present study, the time-hardening creep law (Equation 5.17) model was used since the external load on the pipes was assumed to be a constant.

5.7.1 Mathematics of creep formulation

In a time-dependent material like high density polyethylene, when the same amount of stress is maintained continuously on the material, the strain vector $d\{\bar{\varepsilon}\}$ can be divided into elastic strain, $d\{\bar{\varepsilon}\}^e$ and creep strain, $d\{\bar{\varepsilon}\}^{cr}$ as written below (Bathe, 2002):

$$d\{\bar{\varepsilon}\} = d\{\bar{\varepsilon}\}^e + d\{\bar{\varepsilon}\}^{cr} \quad \dots\dots\dots (5.19)$$

Therefore, the basic constitutive equation for the stress increment can be expressed by using the following relationship (Bathe, 2002):

$$d\{\bar{\sigma}\} = [C]^e \left[d\{\bar{\varepsilon}\} - d\{\bar{\varepsilon}\}^{cr} \right] \quad \dots\dots\dots (5.20)$$

where $d\{\bar{\sigma}\}$ = incremental stress vector,

$[C]^e$ = elastic constitutive matrix,

$d\{\bar{\varepsilon}\}$ = incremental total strain vector, and

$d\{\bar{\varepsilon}\}^{cr}$ = creep strain vector.

The incremental creep strain, $d\{\bar{\varepsilon}\}^{cr}$ can be calculated as given below (Bathe, 2002):

$$d\{\bar{\varepsilon}\}^{cr} = \Delta t \gamma \bar{\sigma} \quad \dots\dots\dots (5.21)$$

where $\bar{\sigma}$ is the total effective stress $= (1 - \alpha)' \bar{\sigma} + \alpha^{t+\Delta t} \bar{\sigma}$, α being an integration parameter ($0 \leq \alpha \leq 1$). The function γ is given by $\gamma = \frac{3}{2} \frac{\{\dot{\bar{\varepsilon}}\}^{cr}}{\{\bar{\sigma}\}}$ and is based on the creep

law. For an incompressible material, a typical uniaxial creep law can be described as below (Bathe, 2002):

$$\{\varepsilon\}^{cr} = a_0 \{\sigma\}^{a_1} t^{a_2} \{\varepsilon\}^{-\frac{a_3}{T+273.16}} \dots\dots\dots (5.22)$$

where $\{\varepsilon\}^{cr}$ = creep strain,

$\{\sigma\}$ = total stress,

T = temperature in degree Celsius,

a_0 , a_1 , a_2 , and a_3 are constants.

For a multiaxial condition, the creep law can be written as (Bathe, 2002):

$$\{\bar{\varepsilon}\}^{cr} = a_0 \{\bar{\sigma}\}^{a_1} t^{a_2} \{\bar{\varepsilon}\}^{-\frac{a_3}{T+273.16}} \dots\dots\dots (5.23)$$

where $\{\bar{\varepsilon}\}^{cr}$ = effective creep strain,

$\{\bar{\sigma}\}$ = effective stress,

T = temperature in degree Celsius,

a_0 , a_1 , a_2 , and a_3 are constants.

All the other creep laws can be expressed in total creep strain or incremental creep strain using the relationships as given below (Bathe, 2002):

$$\{\bar{\varepsilon}\}^{cr} = f_1 \{\bar{\sigma}\} f_2(t) f_3\{\theta\} \dots\dots\dots (5.24)$$

or

$$d\{\bar{\varepsilon}\}^{cr} = \Delta t f_1 \{\bar{\sigma}\} \dot{f}_2(t) f_3\{\theta\} \dots\dots\dots (5.25)$$

The incremental creep law described in Equation (5.25) is evaluated based on experimental evidence and corresponds to time differentiation of the function, f_2 only. This version of the creep law corresponds to the time-hardening law. In the strain-

hardening version, the creep strain rate can be expressed in terms of effective creep strains by the use of pseudotime instead of time, t (Bathe, 2002). Strain-hardening version shows better results for the variable stress conditions. More information on the creep models shown in Equation 5.24 and Equation 5.25 can be found elsewhere (Bathe, 2002).

5.7.2 Parametric study to determine creep constants

A back calculation procedure was used to determine the constants in the time hardening version by using Hashash's power-law model (Section 3.4.4). Equation (5.16) can also be expressed as given below:

$$\varepsilon(t) = \frac{\sigma}{E(t)} = \frac{\sigma}{67779 t^{-0.0859}} = \left(\frac{1}{67779} \right) \sigma^{(1)} t^{0.0859} \quad \dots\dots\dots (5.26)$$

Therefore, the creep strain rate can be obtained as below:

$$\begin{aligned} \dot{\varepsilon}(t) &= \frac{d\varepsilon(t)}{dt} = \left(\frac{1}{67779} \times 0.0859 \right) \sigma^{(1)} t^{0.0859-1} \\ &= \left(1.267354 \times 10^{-6} \right) \sigma^{(1)} t^{-0.9141} \quad \dots\dots\dots (5.27) \end{aligned}$$

Table 5.4 shows the derived constants by comparing the Equation (5.27) with Equation (5.17).

Table 5.4: Constants used to model time dependent nature of pipe using time hardening creep law.

Variables	Values
A	$1.267354e^{-06}$
n	1
m	-0.9141

5.8 Validation of the time-dependent model

Results from static analysis were in good agreement with the Burns and Richard's (Burns and Richards, 1964) solution reported in the literature. No test protocols were available to compare the time dependent analysis for 50 years. AASHTO recommends a value of 110,000 psi for short-term modulus (at 0.05 years) and 22,000 psi for long-term (at 50 years) of high density polyethylene pipes. Therefore, a two dimensional static analysis with reduced modulus was performed to check the pipe deflections obtained after 50 years. Static results obtained by using reduced moduli were close to that obtained from a creep analysis. However, the static analysis with a reduced elastic modulus over-predicted the deflections.

CHAPTER 6

RESULTS AND DISCUSSIONS

6.1 Introduction

In the present study, two-dimensional finite element analyses were performed to evaluate the long-term performance of single-wall and double-wall corrugated HDPE pipes. Time dependent analysis (transient analysis) was performed on 24 inch (600 mm) and 48 inch (1200 mm) pipes for a period of 50 years. Analysis was performed using two different loading conditions with fills reaching as high as 60 feet. Performance of pipe was investigated due to dead loads (self-weight of the soil) and live loads (HS-20 loading) for a period of 50 years. Trench width ratio was varied from 1.5 to 2.5. Trench width ratio is defined as the ratio of the trench width to nominal pipe diameter.

6.2 Numerical results for a 24 inch single-wall corrugated HDPE pipe

Analysis was performed on a 24 inch (600 mm) single-wall corrugated high density polyethylene pipe due to self-weight of the soil and HS -20 truck loading. Trench width ratio was varied from 1.5 to 2.5 to determine the effect of trench width.

6.2.1 Response due to self-weight of the soil

Figure 6.1 shows the deformation in the soil-pipe system immediately after the installation of the pipe. Figure 6.2 shows the vertical pipe deflection over a period of 50 years for a 24 inch (600 mm) single-wall corrugated pipe buried at a depth of 20 feet in a trench width of 48 inches. Vertical pipe deflection is defined as the vertical change in the pipe diameter. Vertical change in the pipe diameter was computed by taking the difference in the vertical displacements of nodal points at the crown and bottom of the pipe. Figure 6.3 shows the percentage of creep deformation at any time, T. Percentage of creep deformation at any time, T can be expressed as $\left(\frac{U_T - U_i}{U_{50} - U_i} \times 100 \right)$. U_i is the initial

pipe deflection, U_{50} is the total pipe deflection after 50 years and U_T is the vertical pipe deflection at any time, T . Figure 6.4 shows the vertical pipe deflection at different burial depths -10 feet, 15 feet and 20 feet. Figure 6.5 illustrates the effect of trench width for a period of 0 years, 5 years and 50 years.

In this report, the pipe deflection based on a static analysis is defined as U_i (initial vertical pipe deflection) and the time dependent deflection as U_T . Since it is convenient to perform a static analysis, the pipe deflection at time, T is expressed as a percentage of U_i . This percentage would be useful in estimating the increase in the pipe deflection due to creep behavior. The quantity $\frac{U_T - U_i}{U_i}$ was used as a measure of increase in the pipe deflection due to creep. Figure 6.6 shows the effect of trench width on creep deformation after a period of 5 years and 50 years.

Figure 6.1: Deformation in the soil-pipe system due to self-weight of the soil.

Figure 6.2: Vertical pipe deflection for a period of 50 years.

Figure 6.3: Percentage of creep deformation up to a time period of 50 years.

Figure 6.4: Pipe deflections for a 24 inch single-wall corrugated pipe with increasing fill heights.

Figure 6.5: Effect of trench width for a 24 inch single-wall corrugated pipe.

Figure 6.6: Effect of trench width ratio on creep deformation after a period of 5 years and 50 years.

Results obtained from Figure 6.2 show a gradual increase in the vertical pipe deflection over a period of 50 years. Figure 6.2 also shows that most of the pipe deflection can be observed in the early stages after the installation of pipe. Figure 6.3 shows that almost 60% to 80% of the creep deformation takes place within one year of pipe installation. Figure 6.4 shows similar increase in the creep deformation for a period of 50 years for different fill heights. Pipe deformations seem to be increasing with increasing fill heights as expected. Figure 6.5 shows a decrease in pipe deflections with the increase in trench width ratio. Also, Figure 6.5 illustrates that single-wall corrugated HDPE pipes can be buried up to 20 feet with the trench width ratio as small as 1.5. Pipe deflections from Figure 6.5 shows that single-wall corrugated HDPE pipes do not fail due to self-weight when buried up to a depth of 20 feet.

6.2.2 Response due to HS -20 truck loading + Self-weight of the soil

Figure 6.7 shows the deflection in the soil-pipe system due to HS-20 truck loading and self-weight of the soil. Figure 6.8 shows the vertical pipe deflection over a period of 50 years for a 24 inch (600 mm) single-walled corrugated pipe buried at a depth of 20 feet under HS-20 loading in a trench width of 48 inches. Figure 6.9 shows the percentage of creep deflection over a period of time, T. Percentage of creep deflection at any time, T can be expressed as $\left(\frac{U_T - U_i}{U_{50} - U_i} \times 100 \right)$. U_i is the initial pipe deflection, U_{50} is the total pipe deflection after 50 years and U_T is the vertical pipe deflection at any time, T. Figure 6.10 explains the effect of trench width for a period of 0 years, 5 years and 50 years under HS-20 loading.

In this report, the pipe deflection based on a static analysis is defined as U_i (initial vertical pipe deflection) and the time dependent deflection as U_T . Since it is convenient to perform a static analysis, the pipe deflection at time, T is expressed as a percentage of U_i . This percentage would be useful in estimating the increase in the pipe deflection due to creep behavior. The quantity $\frac{U_T - U_i}{U_i}$ was used as a measure of increase in the pipe deflection due to creep. Figure 6.11 shows the effect of trench width on creep deformation after a period of 5 years and 50 years due to HS-20 loading.

Figure 6.7: Deformation in the soil-pipe system under HS-20 truck loading.

Figure 6.8: Vertical pipe deflection for a period of 50 years due to HS-20 loading and self-weight of the soil.

Figure 6.9: Percentage of creep deformation for a period of 50 years.

Figure 6.10: Effect of trench width for 24 inch single-wall corrugated pipe due to HS-20 truck loading and self-weight of the soil.

Figure 6.11: Effect of trench width ratio on creep deformation after a period of 5 years and 50 years due to HS-20 loading and self-weight of the soil.

Results from Figure 6.8 show a gradual increase in vertical pipe deflection over a period of 50 years. Figure 6.8 also shows that most of the pipe deflection takes place in the early stages of pipe installation. Figure 6.9 also explains that almost 60% to 80% of the creep deformation takes place within one year of the pipe installation. Figure 6.10 shows a decrease in pipe deflections with the increase in trench width ratio. At trench width ratio equal to 1.5 and 2.0, the pipe deflections exceed 5% allowable deflection after a period of 50 years. Therefore, for the HDPE pipe to perform successfully under HS-20 loading, a good backfill material may be needed. Figure 6.11 shows less than 30% increase in the pipe deflection due to creep even under HS-20 loading buried at a depth of 20 feet.

6.3 Numerical results for a 24 inch double-wall corrugated HDPE pipe

Analysis was performed on a 24 inch (600 mm) double-wall corrugated high density polyethylene pipes due to self weight of the soil and HS -20 truck loading. Trench width ratio was varied from 1.5 to 2.5 to determine the effect of trench width. Also, fills heights up to 40 feet were considered.

6.3.1 Response due to self-weight of the soil

Vertical deformation in the soil-pipe system is similar to that of single-wall corrugated HDPE pipes. Figure 6.12 shows the vertical pipe deflections over a period of 50 years for a 24 inch (600 mm) double-wall corrugated pipe buried at a depth of 20 feet in a trench of 36 inches. Vertical pipe deflection is defined as the vertical change in the pipe diameter. Figure 6.13 shows the percentage of creep deformation over a period of time, T. Percentage of creep deflection at any time, T can be expressed as

$$\left(\frac{U_T - U_i}{U_{50} - U_i} \times 100 \right). U_i \text{ is the initial pipe deflection, } U_{50} \text{ is the total pipe deflection after 50}$$

years and U_T is the vertical pipe deflection at any time, T. Figure 6.14 shows the vertical deformation in the pipe over a period of 50 years at different burial depths ranging from

10 feet to 40 feet. Figure 6.15 illustrates the effect of trench width for a period of 0 years, 5 years and 50 years.

In this report, the pipe deflection based on a static analysis is defined as U_i (initial vertical pipe deflection) and the time dependent deflection as U_T . Since it is convenient to perform a static analysis, the pipe deflection at time, T is expressed as a percentage of U_i . This percentage would be useful in estimating the increase in the pipe deflection due to creep behavior. The quantity $\frac{U_T - U_i}{U_i}$ was used as a measure of increase in the pipe deflection due to creep. Figure 6.16 shows the effect of trench width on creep deformation after a period of 5 years and 50 years.

Figure 6.12: Vertical pipe deflection for a period of 50 years.

Figure 6.13: Percentage of creep deformation up to a time period of 50 years.

Figure 6.14: Pipe deflections for a 24 inch double-wall corrugated pipe with increasing fill heights.

Figure 6.15: Effect of trench width for a 24 inch double-wall corrugated pipe.

Figure 6.16: Effect of trench width on creep deformation after a period of 5 years and 50 years.

Results from the Figure 6.12 show a gradual increase in the vertical pipe deflection over a period of 50 years. Response of double-wall corrugated pipes was similar to that of single-wall corrugated HDPE pipes. Figure 6.13 shows that almost 60% to 80% of the creep deformation takes place within one year of pipe installation. Figure 6.14 shows a continuous increase in the creep deformation for a period of 50 years for different fill heights. Pipe deformations seem to be increasing with increasing fill heights as expected. The pipe seems to be performing well within 5% of the deflection for fill heights up to 20 feet. A failure (deflection reaching above 5%) is observed for fill heights at 40 feet. Figure 6.15 shows a decrease in pipe deflections with the increase in trench width ratio. Also, Figure 6.15 shows that double-walled corrugated HDPE pipes can be buried up to a depth of 20 feet with the use of trench width ratio as small as 1.5. Response of single-wall and double-wall corrugated pipes seems to be similar except the fact that creep rate in double-walled corrugated pipes is higher. Increase in pipe material may be one of the causes. Also, Figures 6.6 and 6.16 illustrate a small increment in the percentage increase in pipe deflections due to creep with the increase in trench width ratio.

6.3.2 Response due to HS -20 truck loading and self-weight of the soil

Figure 6.17 shows the vertical change in the pipe diameter over a period of 50 years for a 24 inch (600 mm) double-walled corrugated pipe buried at a depth of 20 feet in a trench width of 36 inch due to HS-20 truck loading and self-weight of the soil. Figure 6.18 the percentage of creep deflection over a period of time, T for the pipes buried at a depth of 20 feet and 40 feet. Percentage of creep deflection at any time, T can be expressed as $\left(\frac{U_T - U_i}{U_{50} - U_i} \times 100 \right)$. U_i is the initial pipe deflection, U_{50} is the total pipe deflection after 50 years and U_T is the vertical pipe deflection at any time, T. Figures 6.19 and 6.20 explain the effect of trench width for a period of 0 years, 5 years and 50 years under HS-20 loading at depths of 20 feet and 40 feet respectively.

In this report, the pipe deflection based on a static analysis is defined as U_i (initial vertical pipe deflection) and the time dependent deflection as U_T . Since it is convenient to perform a static analysis, the pipe deflection at time, T is expressed as a percentage of U_i . This percentage would be useful in estimating the increase in the pipe deflection due to creep behavior. The quantity $\frac{U_T - U_i}{U_i}$ was used as a measure of increase in the pipe deflection due to creep. Figure 6.21 shows the effect of trench width on creep deformation after a period of 5 years and 50 years due to HS-20 loading and self-weight of the soil.

Figure 6.17: Vertical change in the pipe diameter for a period of 50 years.

Figure 6.18: Percentage of creep deformation for a period of 50 years.

Figure 6.19: Effect of trench width for a 24 inch double-wall corrugated pipe buried at a depth of 20 feet due to HS-20 loading and self-weight of the soil.

Figure 6.20: Effect of trench width for a 24 inch double-wall corrugated pipe buried at a depth of 40 feet due to HS-20 loading and self-weight of the soil.

Figure 6.21: Effect of trench width on creep deformation after a period of 5 years and 50 years due to HS-20 loading and self-weight of the soil.

Results obtained from the Figure 6.17 demonstrate a gradual increase in the vertical change in the pipe diameter over a period of 50 years due to HS-20 loading. Figure 6.18 confirms that almost 60% to 80% of the creep deformation takes place within one year of the pipe installation for the material properties used in the present study. Figure 6.19 and Figure 6.20 shows a decrease in pipe deflections with the increase in trench width ratio. The pipe deflections after a period of 50 years exceed 5% allowable deflection. Therefore, for the HDPE pipe to perform under HS-20 loading a good backfill material may be needed. Figures 6.6, 6.11, 6.16 and 6.21 show less than 30% increase in the pipe deflection due to creep. Therefore, Figures 6.6, 6.11, 6.16 and 6.21 would be useful in estimating the increase in the pipe deflection due to creep behavior.

6.4 Numerical results for a 48 inch double-wall corrugated HDPE pipe

Analysis was performed on a 48 inch (1200 mm) double-walled corrugated high density polyethylene pipes due to self-weight of the soil and HS-20 truck loading. Overburden was further increased to 60 feet. Different pipe sizes (24 inch and 48 inch) were compared for the pipes buried at 20 feet. Trench width ratio was varied from 1.5 to 2.5 to determine the effect of trench width.

6.4.1 Response due to self weight of the soil

Figure 6.22 compares the pipe deflections of 24 inch (600 mm) and 48 inch (1200 mm) double-wall corrugated pipes buried at depth of 20 feet for a trench width ratio of 1.5. Figure 6.23 shows the percentage of creep deformation for 24 inch (600 mm) and 48 inch (1200 mm) double-wall corrugated HDPE pipes. Figure 6.24 and 6.25 shows the effect of trench for a 48 inch (1200 mm) double-wall corrugated HDPE pipe buried at 20 feet and 40 feet respectively. Figure 6.26 and 6.27 shows effect of trench on creep deformation for a period of 5 years and 50 years when buried at depth of 20 feet and 60 feet respectively.

Figure 6.22: Vertical pipe deflections for a 24 inch and a 48 inch double-wall corrugated pipe buried at 20 feet.

Figure 6.23: Percentage of creep deformation for a 24 inch and a 48 inch double-wall corrugated pipe buried at a depth of 20 feet.

Figure 6.24: Effect of trench width for a 48 in pipe buried at 20 feet.

Figure 6.25: Effect of trench width for a 48 in pipe buried at 40 feet.

Figure 6.26: Effect of trench width ratio on creep deformation after a period of 5 years and 50 years for a 48 inch double-wall pipe buried at 20 feet.

Figure 6.27: Effect of trench width ratio on creep deformation after a period of 5 years and 50 years for a 48 inch double-wall pipe buried at 60 feet.

Results from Figure 6.22 show reduced pipe deflections for larger diameter pipe buried when buried at same depth. Figure 6.23 shows the same amount of creep deformation in 48 inch (1200 mm) pipe as seen in 24 inch (600 mm) pipe after a period of time, T. Figure 6.24 shows that pipes (of 24 inch and 48 inch diameter) can be buried at 20 feet with trench width ratio as low as 1.5. Figure 6.25 shows a failure in the pipe when buried at a depth of 40 feet. Therefore, for pipes buried at depths greater than 20 feet, use of good backfill material may be needed. Also, Figures 6.26 and 6.27 show less than 30% increase in the pipe deflections due to creep.

6.4.2 Response due to HS -20 truck loading and self-weight of the soil

Analysis was performed on a 48 inch (1200 mm) double-wall corrugated HDPE pipes due to HS-20 loading at different burial depths. The pipe deflections obtained were following the same trend as in smaller diameter (24 inch) pipes. The rate of creep deformation was same for 24 inch (600 mm) and 48 inch (1200 mm) pipe diameter. No failure in the pipe was observed even under HS-20 loading for the pipes buried at 20 feet with trench width ratios as low as 1.5. Figure 6.28 shows the results for pipe buried at 20 feet under HS-20 loading. Failure in the pipe was observed for pipes buried at greater depths (at depths of 40 feet and 60 feet). Less than 30% increase in the pipe deflections due to creep was observed for the pipes buried at greater depths.

Figure 6.28: Effect of trench width for a 48 inch double-wall HDPE pipe buried at depth of 20 feet due to HS-20 loading and self-weight of the soil.

CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1 Summary

The primary objective of this research work was to investigate the time dependent behavior of buried single-wall and double-wall corrugated high density polyethylene pipes by using the finite element analysis. Secondary objective was to investigate the influence of trench width for different pipe sizes and at different depths. A critical literature review was conducted to widen the current understanding of the soil-pipe system and time dependent nature of high density polyethylene pipes. Different creep models reported for high density polyethylene pipes were investigated. Hashash's creep model (1991) was selected for the pipe used in this study based on the comparison of different creep models. Controlled low strength material (CLSM) was considered as a backfill around the pipe. Respective properties of backfill materials and native soils were obtained from the literature.

For the purpose of this study, pipes ranging from 24 inch (600 mm) to 48 inch (1200 mm) were selected. Both single-wall and double-wall corrugated pipes were investigated. Trench width ratios ranging from 1.5 to 2.5 were selected. Analysis was conducted on pipes buried at depths as high as 60 feet under two different loading conditions – only the self-weight of the soil and under the HS-20 truck loading. Results obtained are compared using available analytical solutions (Timoshenko, 1936; Burns and Richards, 1964) and AASHTO recommendations (AASHTO, 2002) for the short-term and long-term performance of buried pipes.

7.2 Conclusions

Following conclusions were drawn based on the results presented in this research work:

- Beam elements seem to be more appropriate to model the geometry of the unconfined pipe. Results obtained using beam formulation fall within 5% of analytical solutions for the values of r/t (radius-to-thickness ratio) greater than 6.0.
- As expected, results show a decrease in the deformation with the increase in the modulus of confinement material.
- Results obtained compared well with available analytical solutions (Burns and Richards, 1964).
- Results obtained from transient analysis compared well with the static analysis performed with reduced modulus for 50 years.
- Rate of creep deformation is similar for different pipe sizes. A continuous change in the vertical pipe diameter is noticed for a period of 50 years.
- Pipe deformations seem to be increasing with increasing fill heights as expected.
- 60 – 80% of the creep deformation was observed within one year of the installation. Not much of creep deformation in the material was observed in the later years ($t > 1$ year).
- Pipe deformations decreased with the increasing trench width ratios.

- High density polyethylene pipes (HDPE) pipes can be buried successfully at depths up to 20 feet under its own weight with the use of trench width ratio as low as 1.5.
- Response of single and double-wall corrugated pipes seems to be similar except for the fact that creep rate in double-walled corrugated pipes is higher.
- Less than 30% increase in the pipe deformations was noticed after initial pipe deformations even under the extreme loading conditions for a period of 50 years. However, further investigation is required.

7.3 Recommendations

- Perform numerical studies by using different backfill materials to investigate the performance of pipes buried in different backfill conditions.
- Investigate the soil-pipe interaction by considering the soil-pipe interface properties.
- Investigate pipes buried under the embankments as shown in Figure 7.1. Not all the pipes are installed and buried in trenches. Therefore, it is necessary to investigate pipes that are buried under the embankments as well. Typical examples of pipes not installed in trenches are railway and highway culverts.
- Investigate the potential use of geosynthetic materials in the buried pipe installation. Use of geosynthetic material may help in reducing vertical deflection of the pipe and improving overall structural performance of the pipe.

Figure 7.1: Buried HDPE pipe under embankment.

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