Evaluation of HDPE and PVC Pipes Used for Cross-drains in Highway Construction

by

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Abstract

The Federal Highway Administration (FHWA) passed regulations mandating that equal consideration be given to the specification of plastic pipe materials if they were judged to be equally acceptable on the basis of engineering and economic analysis. This thesis reports the results of a research project that investigated the use of thermoplastic pipes, namely High-Density Polyethylene (HDPE) and Poly-vinyl Chloride (PVC), as cross-drains under highways. The project consisted of completing three major phases that included a comprehensive literature review, an analytical study into the allowable fill heights for thermoplastic pipes, and finally a field study to observe the installation and performance of the pipe in service conditions. The thesis concludes with recommendations for how and when thermoplastic pipe should be installed in order for it to perform as intended.

Acknowledgments

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This thesis is part of an ongoing research project supported in part by the Alabama Department of Transportation (ALDOT) and the Auburn University Highway Research Center (AU-HRC). As such, the contents and conclusions are considered preliminary at the time of finalizing the thesis. The contents do not necessarily reflect views, opinions, conclusions, or policies of the Alabama Department of Transportation (ALDOT), the State of Alabama, the Federal Highway Administration (FHWA), or the Auburn University Highway Research Center (AU-HRC).

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List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ADT	average daily traffic
DOT	Department of Transportation
^{o}C	degree Celsius
cfs	cubic feet per second
^{o}F	degree Fahrenheit
FE	finite element
FEA	finite element analysis
FHWA	Federal Highway Administration
fps	feet per second
ft	feet
HDPE	High-Density Polyethylene
in	inch
ksi	kips per square inch
lbs	pounds
mm	millimeter
PCC	Portland cement concrete
pcf	pounds per cubic foot

pii	pounds per inch per inch of pipe
PPI	Plastics Pipe Institute
psi	pounds per square inch
PVC	polyvinyl chloride
SF	safety factor

CHAPTER 1

INTRODUCTION

1.1 Background

A culvert can be described as a hydraulically short conduit which conveys water through a roadway embankment or past some other flow obstruction (Normann et al. 2001). Culvert materials are selected depending on the required structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance needed for a particular application (Normann et al. 2001). The three most common culvert materials currently being used are concrete, corrugated aluminum, and corrugated steel (Normann et al. 2001). In 2006, the Federal Highway Administration (FHWA) published a new regulation in the Federal Register that broadened the available types of culvert materials that could be specified and used for drainage applications on federal-aided highway projects (FHWA 2006). The new regulation required that "equal consideration" be given when specifying alternate pipe materials—including plastic and corrugated aluminum—that are "judged to be of satisfactory quality and equally acceptable on the basis of engineering and economic analysis" (FHWA 2006).

In recent years, the use of thermoplastic profile wall pipes for culverts and other highway applications has begun to increase (Gassman et al. 2005). The rise in the use of thermoplastic pipes does not mean that the average civil engineer places a high level of confidence in these pipes (Sargand et al. 2002). Thermoplastic pipes are still relatively new products for civil engineers, who are much more familiar with steel and concrete (Sargand et al. 2002). As with any new material that has not been evaluated over its design life cycle, questions exist about the structural as well as the long-term performance of thermoplastic pipes (Sargand et al. 2002; Gassman et al. 2005). In order to quantify the long-term performance of the soil-pipe system, the performance of both the pipe itself as well as the interaction between the pipe and the soil envelope must be investigated (Gassman et al. 2005).

The reason thermoplastic pipes have been gaining popularity over the years can be explained by the fact that they are generally lighter, more cost-effective, and more resistant to chemical attacks than most of the conventional pipe types (Sargand et al. 2002). According to Sargand et al. (2002) the common questions or concerns of thermoplastic pipe are related to the allowable maximum fill height, the length of time required for stabilization of the pipe responses, and the recommended method for the analysis and design of buried thermoplastic pipe.

1.2 Research Objective

The primary objectives of this research project were to assess the use of high-density polyethylene (HDPE) and polyvinyl chloride (PVC) for use as cross drains under highways and to assist the Alabama Department of Transportation (ALDOT) in developing a methodology for using plastic pipe.

1.3 Scope and Methodology

The objective of the research project was achieved by completing a series of tasks that would investigate the differences between thermoplastic pipes and pipes currently used (ie. concrete and steel) in construction. The evaluation process was accomplished by executing the following tasks:

Task 1: Collect and synthesize previous research as well as recommendations from HDPE and PVC pipe manufacturers.

Task 2: Analytically evaluate maximum fill height and minimum cover recommendations.

Task 3: Select and help coordinate field trial installations and perform construction monitoring. The installations were observed and consisted of mandrel testing or direct inspections by the research team no sooner than 30 days after backfilling is complete.

Task 4: Develop a long-term condition monitoring program for the field trial installations that will begin to accumulate long-term performance history.

Task 5: Prepare a report with recommendations for revisions to ALDOT procedures for selection, design, and construction of plastic pipe installations.

1.4 Organization of Thesis

Chapter 2 consists of a comprehensive literature review. This review will introduce HDPE and PVC, give a brief history of each, give performance limits and design procedures, provide a comprehensive review of the current practices for their installation and use, review past research done in the field and laboratory, and examine current testing and quality control and quality assurance practices.

Chapter 3 describes the analytical study to determine maximum fill height and minimum cover requirements. The minimum cover section consists of requirements for highway loads when there is no pavement, there is a flexible or rigid pavement, and also when construction loads are considered.

Chapter 4 describes the trial field installations. The chapter includes the coordination of activities, information on the actual installation, and a performance review when inspections were conducted.

Chapter 5 consists of conclusions that were formed based on the work completed in this thesis. The chapter concludes with the project teams recommendations.

CHAPTER 2

LITERATURE REVIEW

This chapter provides a comprehensive review of relevant material relating to both high-density polyethylene (HDPE) and polyvinyl chloride (PVC) pipe for use as a drainage material under roadways. This review will give a brief history of each type of pipe, introduce the properties and characteristics of each, give performance limits and design procedures, provide a comprehensive review of the current practices for their installation and use, review past research done in the field and laboratory, and examine current testing and quality control and quality assurance practices.

2.1 History of Thermoplastic Pipe

2.1.1 HDPE

Polyethylene was first synthesized by Hans von Pechman, a German chemist, in 1898 by accident (Goddard 2009). Polyethylene's first commercial application came during World War II, when the British used it to insulate radar cables (Gabriel 2008). It was not until 1951 that high-density polyethylene (HDPE) was first produced by Robert Banks and J. Paul Hogan. The first corrugated plastic pipes commercially produced for drainage were produced in the late 1960s and were used for agricultural drainage. HDPE was first used as highway underdrains in the early 1970s by the Iowa Department of Transportation (DOT) on Interstate 80 and by the Georgia DOT on Interstate 20. The first state DOT to include corrugated polyethylene pipe in their standard specifications was Georgia. The first known corrugated polyethylene cross-drain culvert that was installed under a state highway occurred in 1981 by the Ohio DOT. Today, all of the State DOTs currently specify and use corrugated polyethylene pipe for some application. Most State DOTs allow its use for culvert and storm sewer applications (Goddard 2009).

2.1.2 PVC

PVC was first discovered at the end of the nineteenth century. The PVC pipe industry began during World War II, when German scientists and engineers used PVC pipe to quickly restore essential water and waste water pipelines. The use and availability of PVC pipe has steadily grown since the 1950s. Recently with the ability to produce larger diameter pipe, PVC pipe has expanded into gravity storm sewers and highway drainage (Uni-Bell 2005).

2.2 Introduction to Thermoplastic Pipe

Plastics are made up of two basic groups: thermosetting and thermoplastic materials (Hu 1994). Thermosetting materials form permanent shapes when cured by heat or a curing chemical (Hu 1994). Thermoplastic materials soften when heated and reharden upon cooling and can be formed and reformed repeatedly (Hu 1994). High-density polyethylene and polyvinyl chloride materials belong to the thermoplastic group. Thermoplastic pipes are just one type of flexible pipe.

2.2.1 Viscoelastic Properties

Thermoplastic pipes exhibit viscoelastic properties. An important issue to understand when dealing with thermoplastic pipes is how the material responds to internal and external forces. One of the most common misconceptions surrounding plastics is that they lose strength with time. This idea is construed by applying elastic behavior criteria to a viscoelastic material (Gabriel 2008). Stress is an internal response of a deformable body subjected to external forces and strain is the response to those associated deformations. The relationship between stress and strain is different for all materials. Thermoplastic materials have an inelastic and nonlinear response with a time dependency measured in seconds. Concrete has the same inelastic and nonlinear response but its time dependency is measured in years. Steel on the other hand can essentially be regarded as independent of time as long as the load is small enough to maintain the integrity of its linear stress-strain response. Thermoplastic materials respond to loads much differently than do linear, elastic, time-independent materials. Plastics creep with sustained load and do not fully recover during the relaxation phase when the load is removed as can be seen in Figure 2-1 (Gabriel and Goddard 1999).

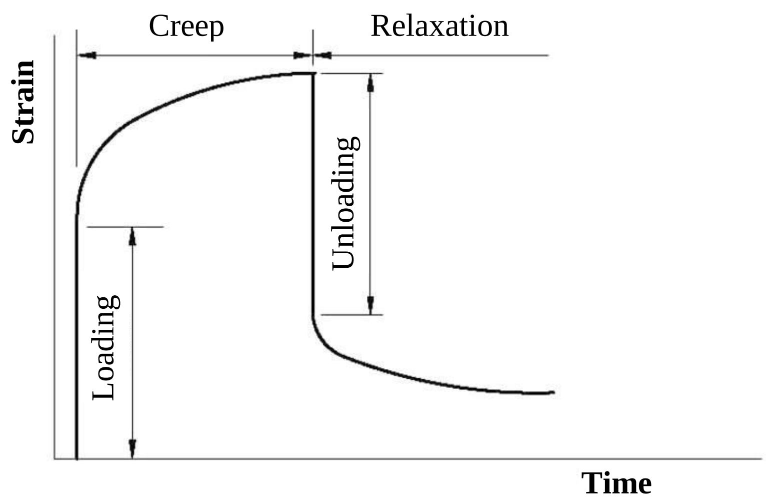


Figure 2-1: Thermoplastic Material, Strain vs. Time (Gabriel and Goddard 1999)

Thermoplastics can therefore be considered a nonlinear viscoelastic material with two moduli: a creep modulus when the load is maintained and a relaxation modulus when the deformation is maintained (Gabriel 2008). According to Gabriel and Goddard (1999), "These modular values decrease with time because of increasing strain due to creep or decreasing stress due to relaxation."

Goddard (1994) summarized a simplified way to describe the differences between rigid, elastic, and viscoelastic materials in the following excerpt:

> To put the differences between rigid, elastic, and viscoelastic materials in the simplest possible terms consider the following; the hard candy stick, licorice, and a hershey bar. The hard candy (the rigid structure) shatters if you attempt to bend it, regardless of loading rate. The hershey bar (the elastic structure) flexes under load but returns to shape unless that load exceeds the yield point. Beyond the yield point, the material takes a permanent set or deformation. At some amount of strain, the elastic material fails. The licorice (the viscoelastic material) responds differently depending on the rate at which the load is applied. If the load is applied very rapidly, the strength of the material is quite high. If a much lower load is placed on the licorice, it will slowly elongate. If the elongation is fixed at some constant strain, the licorice will relieve itself of stress.

While this example is helpful, it is not quite accurate because the pipe walls in non-pressure pipes are normally in compression, not tension. Because it is in compression, the pipe wall has a tendency to compress and thicken under load rather than stretching and necking down. This causes the cross-sectional area to increase while stress relaxation is taking place (Goddard 1994).

According to Rahman (2004), the most common way to evaluate the strength of a material is by defining the direct relationship between stress and strain when a load is applied to a material. Figure 2-2 shows the stress-strain relationship of both elastic and viscoelastic materials.

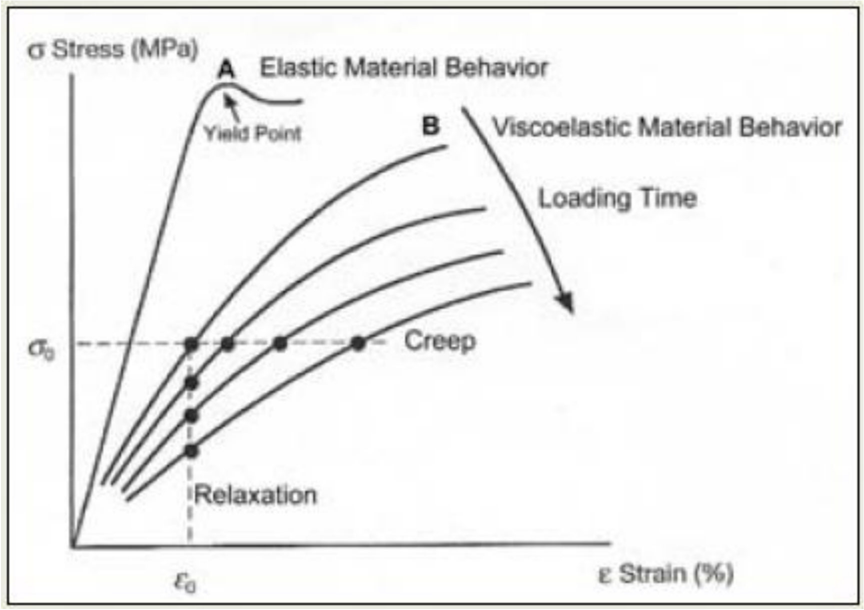


Figure 2-2: Stress-Strain Relationship in Elastic and Viscoelastic Materials (Rahman 2004)

Graph A in Figure 2-2 shows the linear relationship between stress-strain in elastic materials. In an ideal elastic material, the strain returns to zero when the material is unloaded, and the linear relationship is not time-dependent (Rahman 2004). This relationship is only valid up until the yield point. Graph B shows the stress-strain relationship for viscoelastic materials and it can be seen that there is no longer a linear relationship and that the gradients of the curves depend on the loading time. For example, it is seen that for a given stress level, the longer the loading time, the larger the strain reached. Creep can be defined as the continuing deformation (increasing strain) with time when the material is subjected to a constant stress. The relaxation property of thermoplastics can be seen in the figure where the initial stress decreases with time (Rahman 2004).

According to Gabriel (2008), the University of Massachusetts conducted research to address the effect time has on the modulus of polyethylene. In the study, a corrugated polyethylene pipe was placed in a frame that allowed measurement of both stress and strain under repeated load intervals, and for a long period of time (Gabriel 2008). The results of the study are shown in Figure 2-3.

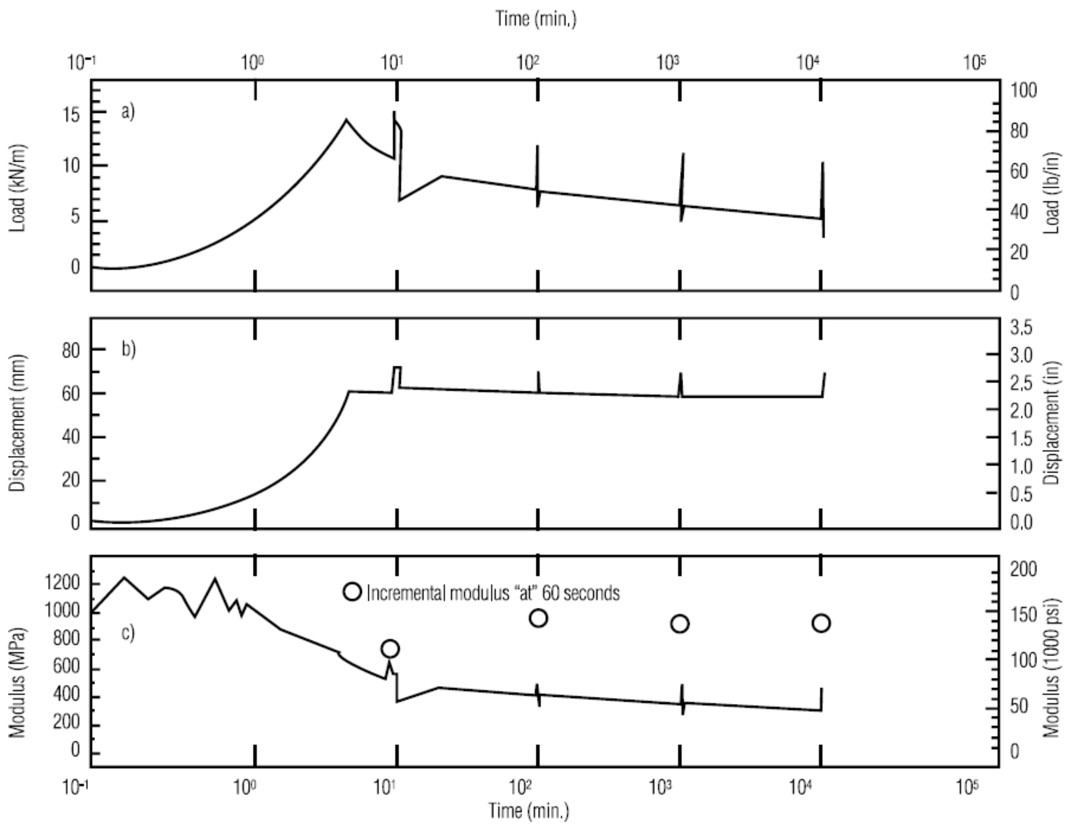


Figure 2-3: Effects of Repeated Loads on Corrugated Polyethylene Pipe (Gabriel 2008)

As can be seen from Figure 2-3, a load was applied to the pipe to create an initial level of deflection. The third graph in the figure shows that the pipe responded with an initial high modulus but it then began to decrease. While the pipe was still deflected, the stress level was increased. The pipe responded with its initial modulus again before immediately decreasing. Several more loads were applied to the pipe, and the pipe responded the same each time (Gabriel 2008). According to Gabriel (2008), this behavior is not indicative of a material that is losing strength.

Polyethylene exhibits significant creep behavior under constant loading. Because of this fact, the effective long-term modulus is significantly lower than the short-term modulus (Katona 1987). The long-term modulus of elasticity of PVC is approximately 30% of the initial, while the long-term modulus of elasticity of HDPE is approximately 16% of the initial (Zhao et al. 1998). The long-term modulus of elasticity values should be used in calculating long-term pipe deflections and critical buckling pressures. The initial and long-term material properties of HDPE and PVC can be seen in Table 2-1.

Table 2-1: Mechanical Properties of Thermoplastic Pipes (AASHTO LRFD 2009)

Type of Pipe	Minimum Cell Class	Allowable Long-Term Strain %	Initial		50-Year	
			F_s min (ksi)	E min (ksi)	F_s min (ksi)	E min (ksi)
Solid Wall PE Pipe--ASTM F 714	ASTM D 3350, 335434C	5.0	3.0	110.0	1.44	22.0
Corrugated PE Pipe--AASHTO M 294	ASTM D 3350, 435400C	5.0	3.0	110.0	0.90	22.0
Profile PE Pipe--ASTM F 894	ASTM D 3350, 334433C	5.0	3.0	80.0	1.12	20.0
	ASTM D 3350, 335434C	5.0	3.0	110.0	1.44	22.0
Solid Wall PVC Pipe--AASHTO M 278, ASTM F 679	ASTM D 1784, 12454C	5.0	7.0	400.0	3.70	140.0
	ASTM D 1784, 12364C	3.5	6.0	440.0	2.60	158.4
Profile PVC Pipe--AASHTO M 304	ASTM D 1784, 12454C	5.0	7.0	400.0	3.70	140.0
	ASTM D 1784, 12364C	3.5	6.0	440.0	2.60	158.4

Note: F_s – tensile strength
E – flexural modulus

As can be seen in Table 2-1, AASHTO Specifications currently provide a modulus of elasticity for "initial" and for 50-year design periods. The values called "initial" values are derived from the material qualification tests and represent very short loading periods. The values termed 50-year values were provided by industry (McGrath et al. 2009). According to PPI (2003), the 50-year values for tensile strength and flexural modulus were determined based on resins used in the 1980s with some additional safety factors, and they are conservative for resins currently required and used by the industry. AASHTO LRFD Bridge Design Specifications (2010) state that the 50-year design tensile strength requirements were derived from hydrostatic design models and indicate a minimum 50-year life expectancy under continuous application of that stress. AASHTO LRFD (2010) goes on to state that the "initial" and "long-term" relate to conditions of loading, and not to the age of installation. The response of pipes to live loads will reflect the initial modulus of the material, regardless of how long the pipe has been installed (AASHTO LRFD 2010). The choice of a "50-year" value was arbitrary and a carry-over from the gas pressure pipe industry (PPI 2003). The current AASHTO specifications are targeting a seventy-five year design life for bridges, so research has been undertaken for HDPE and PVC in order to develop 75 and even 100-year moduli of elasticity (McGrath et al. 2009).

Sharff and DelloRusso (1994) presented data in a report on PVC pipes that were held under constant deflection in ring-bending for over two years and computed coefficients to extrapolate the modulus to long time periods using the viscoelastic model of Horsely (McGrath et al. 2009). According to McGrath et al. (2009), the data show that the 75- and 100-year moduli of elasticity are approximately 98.5% and 97.5% of the 50-year modulus, respectively.

Based on the information included in the above research of the long-term modulus of elasticity, McGrath et al. (2009) proposed the following long-term modulus for 75- and 100-year design periods shown in Table 2-2.

Table 2-2: Long-Term Design Values for Modulus of Elasticity, ksi (McGrath et al. 2009)

Material	Current 50-yr Modulus in AASHTO	Proposed Long-Term Modulus Values	
		75-yr	100-yr
Profile PE Pipe (ASTM D3350, 34433C)	20.0	19	18
Other PE materials, including corrugated	22.0	21	20
PVC 12454C	140.0	137	136
PVC 12364C	158.4	156	154

According to Moser (1994), many erroneously believe that the Young's modulus, for plastics such as PVC and HDPE, decrease with time, but that is not the case. Moser (1994) states that his point can be proven by taking a sample of a pipe that has been under load for a long period of time and running a test for modulus on it. A test was run to find the modulus on a PVC pipe that had been in service for 15 years and it was determined that the modulus was the same as when the pipe was newly manufactured (Moser 1994). Moser (1994) also contends that the creep modulus is an invented term that has almost no application in design.

Sharff and DelloRusso (1994) contend that the short-term pipe stiffness is useful in characterizing the deformation response of buried pipe subjected to loads where the short-term response is of prime interest. For example, this is the case when pipes are subjected to quasi-instantaneous loadings such as traffic live loads or dead weight surcharges after the original soil/pipe installation has been stabilized. Under these loadings, the deformation behavior of the pipe can be reasonably determined using initial stiffness properties as determined by the parallel plate method (Sharff and DelloRusso 1994).

2.3 Plastic Pipe Characteristics

Plastic pipes have many different uses in the world today. The focus of this review will be on their use for gravity flow applications. AASHTO M 294 is a standard specification for corrugated polyethylene pipe that deals with the material requirements and the quality of workmanship. This specification breaks the pipe into classifications and the main ones are described below (AASHTO M 294):

- Type C – Full circular cross section, with a corrugated surface both inside and outside.
- Type S – Full circular cross section, with an outer corrugated pipe wall and a smooth inner liner.
- Type D – Smooth inner liner connected with projections or ribs to a smooth outer wall.

PVC pipes can also be broken down into similar classifications. PVC profile pipes are manufactured with open profiles (ASTM F 794 and F 949), using either ribs or dual wall corrugations, and closed profiles (ASTM F 1803), using a pattern enclosed between smooth internal and external walls. The majority of the pipe being manufactured for gravity flow drainage under roadways consists of Type S pipe or pipes with a smooth inner liner.

2.3.1 Basic Properties

2.3.1.1 HDPE

In order to determine the performance of a pipe in service (stress, strain, deformation responses, and stability), the basic properties along with the profile geometry must be known. According to AASHTO M 294 *Standard Specification for Corrugated*

Polyethylene Pipe 300- to 1500-mm Diameter (2008), the resin used to produce the pipe should conform to the requirements of cell class 435400C as defined and described in ASTM D 3350. ASTM D 3350 *Standard Specification for Polyethylene Plastics Pipe and Fittings Materials* (2006) classifies polyethylene in accordance with density, melt index, flexural modulus, tensile strength at yield, slow crack growth resistance, and hydrostatic strength classification. Table 2-3 lists the properties, the ASTM test method for calculating the property, and the requirements of those classifications. As stated before, AASHTO M 294 requires that the polyethylene conform to the requirements of cell class 435400C. Figure 2-4 gives an example of how to read the cell classification for Class PE233424B. It can be easily seen that the properties are in order and the requirements can easily be picked off Table 2-3 for the required cell classification.

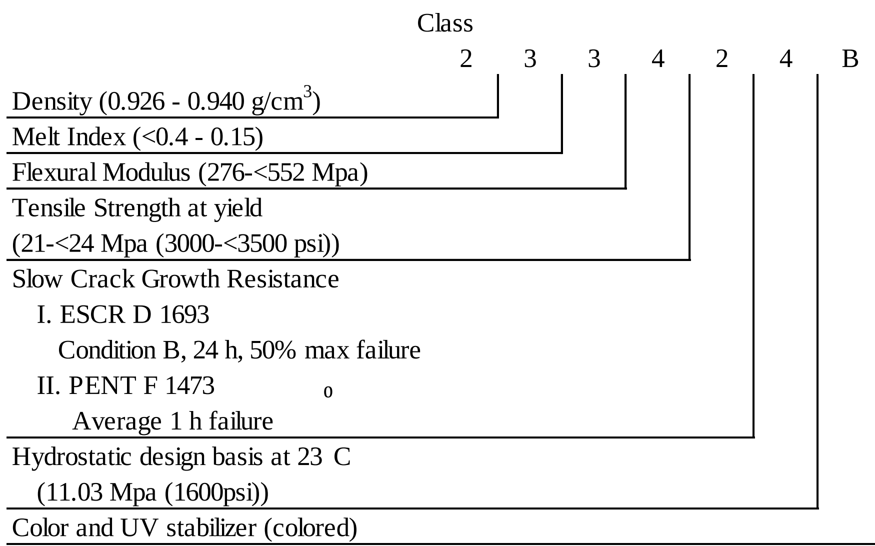


Figure 2-4: Cell Classification Key (ASTM D 3350)

Table 2-3: Primary Properties—Cell Classification Limits (ASTM D 3350)

Property	Test Method	0	1	2	3	4	5	6	7	8
1. Density, g/cm^3	D 1505	Unspecified	0.925 or lower	>0.925-0.940	>0.940-0.947	>0.947-0.955	>0.955	. . .	Specify Value	
2. Melt Index	D 1238	Unspecified	>1.0	1.0 to 0.4	<0.4 to 0.15	<0.15	A		Specify Value	
3. Flexural modulus, Mpa [psi]	D 790	Unspecified	<138 [<20 000]	138-<276 [20 000 to <40 000]	276-<552 [40 000 to 80 000]	552-<758 [80 000 to 110 000]	758-<1103 [110 000 to <160 000]	>1103 [>160 000]	Specify Value	
4. Tensile strength at yield, Mpa [psi]	D 638	Unspecified	<15 [<2200]	15-<18 [2200-<2600]	18-<21 [2600-<3000]	21-<24 [3000-<3500]	24-<28 [3500-<4000]	>28 [>4000]	Specify Value	
5. Slow Crack Growth Resistance										
I. ESCR	D 1693	Unspecified								
a. Test condition (100% lgepal.)			A	B	C	C	. . .	. . .	. . .	Specify Value
b. Test duration, h			48	24	192	600				
c. Failure, max, %		Unspecified	50	50	20	20				
II. PENT (hours)	F 1473									
Molded plaque, 80°C, 2.4 Mpa		Unspecified	. . .	. . .	. . .	10	30	100	500	Specify Value
Notch depth, F 1473, Table 1		Unspecified								
6. Hydrostatic Strength Classification										
I. Hydrostatic design basis, Mpa [psi], (23°C)	D 2837	NPR	5.52 [800]	6.89 [1000]	8.62 [1250]	11.03 [1600]	. . .	. . .		
II. Minimum required strength, Mpa [psi], (20°C)	ISO 12162	. . .	. . .	. . .	. . .	. . .	8 [1160]	10 [1450]		

2.3.1.2 PVC

PVC pipes similarly to HDPE have certain requirements for the primary properties of the material. AASHTO M 304 *Poly(Vinyl Chloride) (PVC) Profile Wall Drain Pipe and Fittings Based on Controlled Inside Diameter* (2007) requires that the pipes be made of PVC plastic that has a minimum cell classification of 12454C or 12364C as defined in ASTM D 1784. The difference between these two classifications of PVC pipe is that one has a lower modulus and higher strength, while the other has a higher modulus and a lower strength (McGrath et al. 2009). ASTM D 1784 *Standard Specification for Rigid Poly(Vinyl Chloride) (PVC) Compounds and Chlorinated Poly(Vinyl Chloride) (CPVC) Compounds* gives requirements for impact resistance, tensile strength, modulus of elasticity, and deflection temperature under load. The following properties and their corresponding cell limits can be seen in Table 2-4. Similarly to HDPE, PVC's classes are designated by the cell number for each property in the order that they are listed in Table 2-4. Figure 2-5 shows an example of how to read the above table using the Class 12454.

Table 2-4: Class Requirements for PVC Compounds (ASTM D 1784)

Desig-nation Order No.	Property and Unit	Cell Limits											
		0	1	2	3	4	5	6	7	8	9	10	11
1	Base resin	unspecified	poly(vinyl chloride) homo-polymer	chlorinated poly (vinl chloride)	vinl co-polymer								
2	Impact resistance (izod), min:												
	J/m of notch under notch	unspecified	<34.7	34.7	80.1	266.9	533.8	800.7					
	ftlb/in. of notch under notch		<0.65	0.65	1.5	5.0	10.0	15.0					
3	Tensile strength min:												
	Mpa	unspecified	<34.5	34.5	41.4	48.3	55.2						
	psi		<5 000	5 000	6 000	7 000	8 000						
4	Modulus of elasticity in tension, min:												
	Mpa	unspecified	<1930	1930	2206	2482	2758	3034					
	psi		<280 000	280 000	320 000	360 000	400 000	440 000					
5	Deflection temperature under load, min, 1.82 Mpa [264 psi];												
	°C	unspecified	<55	55	60	70	80	90	100	110	120	130	140
	°F		<131	131	140	158	176	194	212	230	251	266	284
	Flammability	*A*	*A*	*A*	*A*	*A*	*A*	*A*	*A*	*A*	*A*	*A*	*A*

A - All compounds covered by this specification, when tested in accordance with Test Method D 635, shall yield the following results: average extent of burning of <25 min, average time of burning of <10 s.

Class 1 2 4 5 4

Identification:

Poly(vinyl chloride) homopolymer — 1

Property and Minimum Value

Izod

(34.7 J/m (0.65 ftlbf/in.)) under notch — 2

Tensile strength

(48.3 Mpa (7000psi)) — 4

Modulus of elasticity in tension

(2758 Mpa (400 000psi)) — 5

Deflection temperature under load

(70°C (158°)) — 4

Figure 2-5: Cell Classification Key (ASTM D 1784)

2.4 Rigid Versus Flexible Pipe

A flexible pipe may be defined as a conduit that can deflect at least 2 percent without any sign of structural distress, such as rupture or cracking (AWWA 2002). Flexible pipes derive their soil-load carrying capacity from their flexibility (AWWA 2002). The difference between how flexible pipes respond to loads compared to rigid pipes is shown in Figure 2-6.

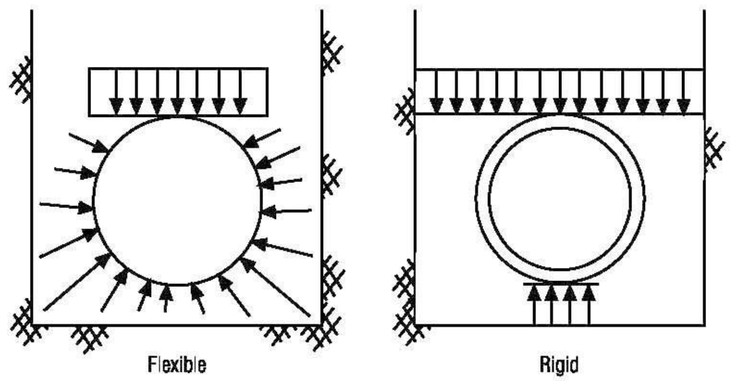


Figure 2-6: Pipe Response to Loading (Gabriel 2008)

When flexible pipes deflect against the backfill the load is transferred to and carried by the surrounding backfill (Gabriel 2008). Rigid pipes transfer loads through the pipe wall into the bedding (Gabriel 2008). The rigid pipe must therefore support the given earth load by the inherent strength of the pipe (Jeyapalan and Boldon 1986). Figure 2-7 shows how the load is transferred from both flexible and rigid pipes to the supporting soil.

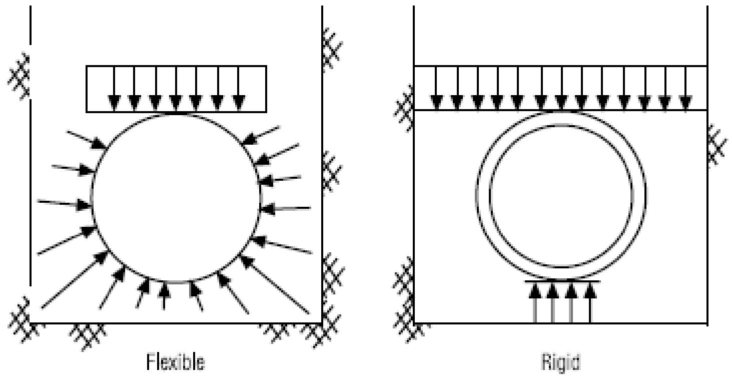


Figure 2-7: Pipe-Backfill Interaction (Gabriel 2008)

Figure 2-8 shows the mechanism of soil arching for both rigid pipes and flexible pipes.

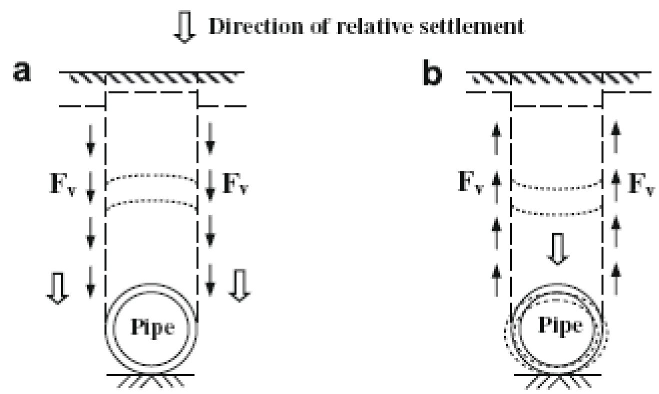


Figure 2-8: Mechanism of Soil Arching within Soil-pipe System (Kang et al. 2009)

The pipe on the left is a rigid pipe where the relative downward deflection of the adjacent backfill soil prism is greater than that of the central soil prism, thereby inducing a negative arching action (Kang et al. 2009). The pipe on the right in the figure is a flexible pipe where the vertical deflection of the central soil prism is greater than that of the adjacent backfill which induces a positive arching action and allows for some of the vertical load to be carried by the surrounding soil (Kang et al. 2009).

2.5 Performance Limits

The performance limits and distress modes are different for different types of pipe materials (Zhao et al. 1998). Table 2-5 was compiled by Zhao et al. (1998) listing the performance criteria and limits for the differing types of pipes currently being used for culvert applications.

Table 2-5: Performance Criteria and Limits of Various Pipes (Zhao et al. 1998)

Type of Pipe	Performance Criteria and Limits
Reinforced-concrete pipe	collapse development of leaky cracks wall crushing joint separation
Corrugated steel pipe	collapse perforation buckling 5% deflection exceeded joint separation
PVC	collapse inverse curvature buckling 7.5% deflection exceeded wall crushing joint separation
HDPE	collapse inverse curvature buckling 5% deflection exceeded wall crushing joint separation

Performance limits of plastic pipe are an important topic to understand before considering how to design the pipe. Some key performance limits for flexible pipe are deflection, wall buckling, stress, and strain (Goddard 1994). The following section will expand on these performance limits to give a comprehensive review.

2.5.1 Deflection

Deflection in plastic pipe soil systems is desirable, but must be maintained within acceptable limits (Uni-Bell 1997). The amount of deflection that will occur in any buried flexible pipe depends on three factors: pipe stiffness, soil stiffness, and the amount of load on the pipe, including both earth load and live load (Uni-Bell 1997). The deflection of the crown of flexible pipes becomes inverted and unable to resist additional load at a deflection of approximately 20% (Moser 2001). The most common types of deflection in flexible pipes are ring deflection and reversal of curvature due to over deflection as can be seen in Figures 2-9 and 2-10 (Goddard 1994).

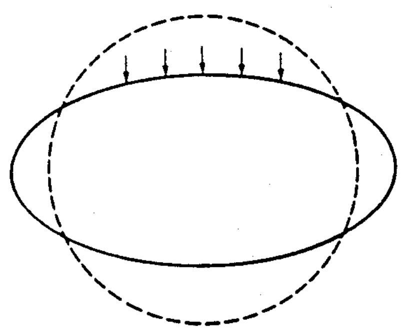


Figure 2-9: Ring Deflection in a Flexible Pipe (Goddard 1994)

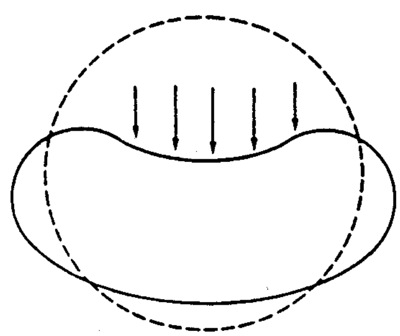


Figure 2-10: Reversal of Curvature Due to Over-deflection (Goddard 1994)

Deflection limits need to be established not only to prevent distress to the pipe but also to limit surface settlement above the pipe (Zhao et al. 1998). Chambers, McGrath, and Heger (1980) suggest selecting deflection limits for plastic pipe for the following reasons:

- To prevent the loss of seals at joints and junctions with ancillary structures.
- To minimize the loss of support to the pavement when a pipe is installed with a shallow cover under a roadway.
- For cleaning, in the case that cleaning equipment requires a minimum diameter.
- Index of stress and strain in a pipe wall. Because deflection of a pipe correlates to the strain in the wall, a deflection limit for the pipe indirectly limits the strain as well.

Jeyalapan and Boldon (1986) recommend an initial deflection limit of 5% of the pipes inside diameter for the following reasons:

- If the pipe is poorly restrained laterally, for example having poor compaction or a weak embedment material was used, failure may occur due to excessive deformation that can cause the pipe to invert at the crown.
- Because of creep characteristics, flexible plastic pipes will continue to deflect over time, thus limiting the initial deflection to 5% will help prevent excessive deformation over the design life of the pipe.
- It will maintain a substantial factor of safety against structural collapse.
- Excessive deflection could cause infiltration or exfiltration to occur as joints become unsealed.

From the above argument for limiting deflection, it can be seen that deflection limits should be set for a variety of reasons to improve the performance. Deflection of flexible pipe is primarily controlled by the quality of installation and the backfill and insitu soil properties (Goddard 1994).

2.5.2 Wall Buckling

Wall buckling can be caused by insufficient bending stiffness of the pipe or insufficient stiffness of the soil envelope surrounding the pipe. It is important that wall buckling be considered because it represents pipe cave in (Reddy 2002). The design of large diameter flexible pipe can be controlled by wall buckling, particularly when the pipe is subjected to high soil pressures in low stiffness soils (Goddard 1994). The effect of wall buckling on a flexible pipe is illustrated in Figure 2-11.

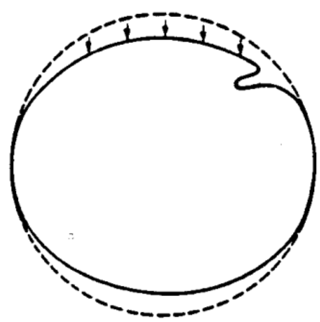


Figure 2-11: Localized Wall Buckling (Goddard 1994)

The more flexible a pipe, the less buckling resistance the pipe displays (Suleiman 2002). The buckling of flexible pipe depends not only on the pipe's material properties but also on the pipe's geometrical properties and the surrounding stiffness of the soil (Suleiman 2002).

2.5.3 Wall Crushing

Wall crushing occurs when wall stresses in compression become excessive. If the ring compressive stress becomes greater than the compressive strength of the wall of the pipe, wall crushing can occur (Goddard 1994). Wall crushing's primary contributor is the ring compression as shown in the equation below (Moser 2001):

$$\text{Ring Compression} = \frac{P_v D}{2A} \qquad \text{Equation 2-1}$$

Where: P_v = vertical soil pressure (psi),

D = diameter (in.), and

A = cross-sectional area per unit length (in^2/in).

Wall crushing can also be influenced by the bending stress whose equation is shown below (Moser 2001):

$$\text{Bending stress} = \frac{Mt}{2I} \qquad \text{Equation 2-2}$$

Where: M = bending moment per unit length (lb-in),

t = wall thickness (in.), and

I = moment of inertia of wall cross section per unit length (in^4/in).

Goddard (1994) states that the viscoelastic properties of thermoplastic make wall crushing very unlikely and field and lab tests completed tend to confirm that view. Figure 2-12 illustrates wall crushing at the 3 and 9 o'clock positions on a flexible pipe.

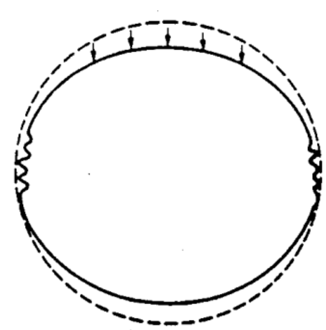


Figure 2-12: Wall Crushing (Goddard 1994)

2.6 Hydraulics

Hydraulics plays a key role in the final design of culverts. When trying to determine the size of pipe required there are several factors that must be analyzed including flow conditions and types of flow control (Normann et al. 2001). The flow of a culvert is controlled by the following geometric variables: cross-sectional size and shape, slope, length, roughness, and entrance and exit hydraulic properties (Bennett 2008). The type of flow control case that gives the least performance is usually designed for in order to provide the needed performance for all conditions (Normann et al. 2001). There are several design methods that are available to determine the design flow and drainage structure size (Bennett 2008). While the hydraulic design of culverts can be quite cumbersome, hydraulic flow research and analysis have established that flow conditions and hydraulic slope in gravity sewer pipe can be designed conservatively using Manning's equation (Uni-Bell 2005). The Manning's equation shown below is based on the conditions of steady flow and open channel flow in determining the discharge of the pipe (Uni-Bell 2005).

$$v = \frac{1.486}{n} r^{2/3} s^{1/2} \qquad \text{Equation 2-3}$$

Where:

v = velocity of flow (ft/s),

n = roughness coefficient,

r = hydraulic radius of the wetted cross section of the pipe (calculated by dividing the cross sectional area of the flow by the wetted perimeter of the pipe in contact with the flow) (ft),

s = hydraulic slope of culvert (ft/ft),

$$= \frac{H_1 - H_2}{L}$$

L = length of pipe section (ft),

H_1 = up-stream pipe elevation (ft), and

H_2 = down-stream pipe elevation (ft).

The “n” value in the above equation is the roughness coefficient (Manning’s) factor. Roughness coefficient values are determined by research and analysis, and they represent the interior surface characteristics of the pipe that account for the frictional losses in the pipe. The greater the frictional loss of a material, the higher the “n” value, and the lower the flow velocity. Table 2-6 shows typical values of Manning’s “n” coefficients for the most common pipe types.

Table 2-6: Typical Value of Manning’s “n” Coefficients (Adapted from Bennett 2008)

Corrugated metal pipe	
Subdrain	0.012-0.014
Riveted CSP	0.024-0.027
Helical CSP	0.011-0.027
Concrete pipe	
Culvert, straight and free debris	0.010-0.013
Culvert with bends, connections and some debris	0.011-0.015
Finished	0.011-0.015
Sewer with manholes, inlet, etc., straight	0.013-0.017
Unfinished, steel form	0.012-0.014
Unfinished, smooth wood form	0.012-0.016
Unfinished, rough wood form	0.015-0.020
Polyvinyl Chloride pipe	0.010-0.015
Polyethlene pipe	
Corrugated	0.021-0.030
Corrugated, smooth interior	0.010-0.015
Smooth wall	0.010-0.015

Studies that have been conducted in laboratories as well as in the field have shown the values of "n" for PVC to range from 0.007 to 0.011. These low values for PVC are attributed to the following (Uni-Bell 2005):

- the non-pourus, smooth surface of the PVC pipe,
- the low profile gap at the joints,
- the longer laying lengths available in PVC pipe, resulting in fewer joints, and
- the chemical and abrasion resistance of PVC.

The Uni-Bell PVC Pipe Association (2005) recommends that the Manning's "n" factor to be used for PVC should be 0.009 for the hydraulic design. Tests that were conducted by the research laboratory at Utah State University show minimum Manning's "n" values of less than 0.010 for corrugated HDPE pipe with a smooth interior liner (ADS 2009). To account for actual field conditions and to incorporate a safety factor, ADS recommends using a Manning's "n" factor of 0.012 for corrugated HDPE pipes with a smooth interior liner (ADS 2009).

In 2002, the Utah Water Research Laboratory at Utah State University conducted research to determine the hydraulic roughness characteristics of the Contech A-2000 PVC sewer pipe. The tests were conducted on a 24 inch diameter pipe that was approximately 200 feet long. The test velocities used in the testing ranged from 1.06 to 22.39 feet per second (fps). The Manning's "n" values calculated ranged from 0.0104 at the lowest velocity down to 0.0082 at the higher velocity. The average Manning's "n" value was 0.0087 (Tullis 2002).

In 2008, Colorado State University conducted hydraulic tests on HDPE pipe to determine the Manning's roughness coefficient for Advanced Drainage Systems pipe.

The study consisted of 120 feet of pipe with eighteen tests completed with varying discharges and bed slopes. The data collected was then used with a standard step fore-water hydraulic model to determine the Manning's roughness coefficient for each test. The Manning's roughness coefficient for discharges greater than 18 cubic feet per second (cfs) was on average 0.0106. At discharges of approximately 2.0 cfs, the average roughness coefficient was 0.0091 (Cox et al. 2008).

Tullis et al. (2005) studied the effects of the change in hydraulic roughness due to wall rippling/corrugation growth. The objective of the study was to quantify the increase in hydraulic roughness due to circumferential pipe wall strain that is typically encountered in field applications. In order to achieve field like conditions in the laboratory, the pipes were wrapped with metal bands that provided better control of the uniformity of the circumferential strain along the length of the test section. The conditions in the lab were also checked with actual field installations to make sure the data collected would be representative of real conditions. The test resulted in Manning's "n" values increasing up to 25% in extreme cases (Tulis et al. 2005). Tulis et al. (2005) goes on to say however that a design Manning's "n" value equal to 0.0012 appears to provide a reasonable factor of safety, relative to variations in hydraulic roughness caused by circumferential strain, for the majority of the data set.

2.7 Plastic Pipe Design

There are two general categories of pipe design which include design of rigid pipe and design of flexible pipe. Rigid pipes are designed to be stiffer than the surrounding soil and resist the applied loads. Flexible pipes are designed to rely on the capacity of the surrounding soil to carry a major portion of the applied load. In all engineering design, performance limits of the material under consideration must be known. To ensure a proper pipe design, the designer must also have an understanding of (Zhao et al. 1998):

- the various pipe materials and products,
- the proper installation procedures,
- the influence of trench width, and
- the quality and compaction of the bedding and backfill materials.

The concept of culvert design and installation requires extensive engineering knowledge in the following fields: hydraulics, soil mechanics, material science, construction methods among others (Malmurugan 1999). The following tasks must be completed for a typical culvert design and installation (Malmurgan 1999):

- Surveying and Planning: Determining the location of the culvert, optimum alignment, depth of burial, etc.
- Hydraulics: Determining the requirements for culvert inside diameter and pipe roughness based on flow considerations.
- Design using classical methods: Determining the required pipe wall sizing to support all loads based on soil-structure interaction.
- Durability: In some cases, it is necessary to provide protective measures for corrosion and abrasion.
- Field Construction: Proper construction procedures should be followed in order to meet the design specifications.

With the above mentioned knowledge, a flexible pipe is designed to prevent the following faults and defects (Zhao et al. 1998):

- Material degradation or environmental stress cracking,

- Seam separation due to excessive ring compression,
- Wall crushing due to excessive external pressure,
- Buckling due to excessive external pressure and/or internal vacuum,
- Excessive deflections leading to leaking joints, and
- Excessive flexural and compressive or tensile strains, leading to yield.

As mentioned previously, flexible pipe responds to external loads by deflection, and by doing so transmits the load on the pipe to the soil at the sides of the pipe.

There are currently several design procedures that are being employed throughout the industry. While all of the procedures are based on the same basic principles, they vary slightly depending on how factors of safety are applied. AASHTO currently has a LRFD design procedure for thermoplastic pipes. This method will not be discussed here, but minimum and maximum fill heights calculated by this method will be shown in subsequent sections. The design methodology shown here will consist of a more general method that has been published by the Plastics Pipe Institute (PPI).

2.7.1 Design Criteria

The design criteria of plastic pipe include pipe section and material properties, installation conditions, and external loads. The following sections will describe these criteria and how they are incorporated into the design procedure. The focus of this section will be on polyethylene pipe, but the same procedure would be used for PVC pipe.

2.7.1.1 Section and Material Properties

Section properties perform an important role in the design of structures, and plastic pipes are no different. The geometry of the pipe wall directly affects the performance of the pipe in the pipe soil-structure system. The pipe properties that are important for design are the following (Gabriel 2008):

- I moment of inertia of the wall profile,
- c distance from inside diameter to the neutral axis, and
- A_s cross-sectional area.

Table 2-7 gives representative values of a range of commercially made pipe meeting the requirements of AASHTO M252, M294, or MP7.

Table 2-7: Representative Section Properties for Polyethylene Pipe (Gabriel 2008)

Inside Diameter, *ID*		Typical Outside Diameter, *OD*		Minimum Pipe Stiffness at 5% Deflection, *PS*		Section Area, A_s		Distance from Inside Diameter to Neutral Axis, *c*		Moment of Inertia, *I*	
in	mm	in	mm	pii	N/m/mm	in^2/in	mm^2/mm	in	mm	in^4/in	mm^4/mm
4	100	4.7	119	35	241	0.0448	1.138	0.139	3.531	0.0	11.5
6	150	7	178	35	241	0.0568	1.443	0.192	4.876	0.0	54.1
8	200	9.9	251	35	241	0.0837	2.126	0.297	7.535	0.0	142.6
10	250	12	305	35	241	0.1044	2.652	0.393	9.97	0.0	303.2
12	300	14.7	373	50	345	0.125	3.175	0.35	8.89	0.0	393.3
15	375	17.7	457	42	290	0.159	4.043	0.45	11.43	0.1	368.5
18	450	21.5	546	40	275	0.195	4.953	0.5	12.70	0.1	1016.0
24	600	28.7	729	34	235	0.262	6.646	0.65	16.51	0.1	1900.9
30	750	36.4	925	28	195	0.327	8.297	0.75	19.05	0.2	2671.1
36	900	42.5	1080	22	150	0.375	9.525	0.9	22.86	0.22	3637.9
42	1050	48	1219	20	140	0.391	9.927	1.11	28.19	0.52	8898.2
48	1200	55	1397	18	125	0.429	10.901	1.15	29.21	0.52	8898.2
54	1350	61	1549	16	110	0.473	12.014	1.25	31.75	0.82	13552.1
60	1500	67.3	1709	14	97	0.538	13.665	1.37	34.798	1.0	16518.2

In addition to section properties, Table 2-7 also includes information on the pipe stiffness (*PS*). Pipe stiffness, rather than crush strength, is usually the controlling pipe material property in flexible pipes (Moser 2001). Pipe stiffness can be defined by the following equation (Moser 2001):

$$\text{Pipe stiffness} = \frac{F}{\Delta y} \qquad \text{Equation 2-4}$$

Where: F = force (lb/in), and
Δy = vertical deflection (in).

The pipe stiffness is determined in the laboratory by using the parallel-plate loading test (ASTM D 2412). The 5% limit is arbitrary and is a quality check which should not be confused with a performance limit (Gabriel 2008).

In addition to section properties, there are also certain soil material properties that affect performance and also must be accounted for in design. One important soil property is the shape factor (D_f), which is a function of pipe stiffness, type of backfill material, and the compaction level (Gabriel 2008). The shape factor was first used in the design of fiberglass pipes (AASHTO LRFD 2010). Its use demonstrates that bending strains are highest in low stiffness pipes backfilled with soils that require substantial compactive effort (silts and clays), and lowest in high stiffness pipes backfilled with soils that require little compactive effort (sands and gravels) (AASHTO LRFD 2010). Table 2-8 shows shape factors for a variety of installation conditions.

Table 2-8: Shape Factors (D_f) (Gabriel 2008)

	Gravel GW, GP, GW-GC, GW-GM, GP-GC and GP-GM		**Sand SW, SP, SM, SC, GM, GC or Mixtures**	
Pipe Stiffness, PS pii (kpa)	Dumped to Slight (<85% SPD)	Moderate to High (≥85% SPD)	Dumped to Slight (<85% SPD)	Moderate to High (≥85% SPD)
14 (97)	4.9	6.2	5.4	7.2
16 (110)	4.7	5.8	5.2	6.8
17 (117)	4.6	5.7	5.1	6.7
20 (138)	4.4	5.4	4.9	6.4
22 (152)	4.3	5.3	4.8	6.3
28 (193)	4.1	4.9	4.4	5.9
30 (210)	4	4.8	4.3	5.8
34 (234)	3.9	4.6	4.1	5.6
35 (241)	3.8	4.6	4.1	5.6
38 (262)	3.8	4.5	4	5.4
40 (276)	3.7	4.4	3.9	5.4
42 (290)	3.7	4.4	3.9	5.3
46 (320)	3.7	4.4	3.9	5.2
50 (345)	3.6	4.2	3.8	5.1

Note: SPD – standard proctor density

In addition to the shape factor, another soil property that must be known is the secant constrained soil modulus (*E'*) (Gabriel 2008). Table 2-9 lists the modulus for different classifications of soil based on various degrees of embedment compaction.

Table 2-9: Soil Modulus (Gabriel 2008)

Pipe Embedment Material						E', psi for Degree of Embedment Compaction				
ASTM D 2321		ASTM D 2487		AASHTO M 43	Min. Std. Proctor	Lift Placement	Dumped	Slightly <85%	Moderate 85% - 95%	High >95%
Class	Description	Notation	Description	Notation	Density (%)	Depth				
IA	Open-graded, clean manu-factured aggregates	N/A	Angular crushed stone or rock crushed gravel, crushed slag; large voilds with little or no fines	5 56	Dumped	18"	1000	3000	3000	3000
IB	Dense-graded, clean manu-factured processed aggregates	N/A	Angular crushed stone orr other Class IA material and stone/sand mixtures; little or no fines							
II	Clean, coarse-grained soils	GW	Well-graded gravel gravel/sand mixtures; little or no lines	57 6 67	85%	12"	N/R	1000	2000	3000
		GP	Poorly graded gravel, gravel/sand mixtures; little or no fines							
		SW	Well-graded sands, gravelly sands; little or no fines							
		SP	Poorly graded sands, gravelly sands; little or no fines							
III	Coarse-grained soils with fines	GM	Silty gravels, gravel/sand/silt mixtures	Gravel and sand with <10% fines	90%	9"	N/R	N/R	1000	2000
		GC	Clayey gravels, gravel/sand/clay mixtures							
		SM	Silty sands, sand/ silt mixtures							
		SC	Clayey sands, sand/clay mixtures							
IVA**	Inorganic fine-grained soils	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, silts with slight plasticity				N/R	N/R	N/R	1000
		CL	Inorganic clays of low to medium plasticity; gravelly, sandy or silty clays; lean clays							

As can be seen from Table 2-9, the secant constrained soil modulus increases both with quality of backfill and degree of compaction.

2.7.1.2 Live Loads

Loads can be considered dead loads or live loads. Live loads that should be considered in plastic pipe design consist mainly of highway loads from trucks. The typical vehicular load consists of the AASHTO H-25 configuration which can be seen in Figure 2-13.

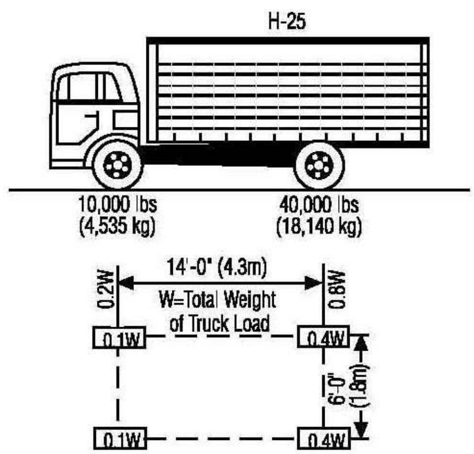


Figure 2-13: AASHTO H-25 Highway Load (Gabriel 2008)

At shallow depths of cover, an impact factor is often used to account for any additional forces caused by the rolling motion of the vehicles (Gabriel 2008). Table 2-10 shows impact factors versus height of cover for highway, railway, runway, and taxiway loads, although the emphasis of this report will be on highways.

Table 2-10: Impact Factor Versus Height of Cover (Moser 2001)

	Installation surface condition			
Height of cover, ft	Highways	Railways	Runways	Taxiways, aprons, hardstands, run-up pads
0 to 1	1.50	1.75	1.00	1.5
1 to 2	1.35	*	1.00	*
2 to 3	1.15	*	1.00	*
Over 3	1.00	*	1.00	*

As you can see from Table 2-10, the impact factor is no longer needed for depths of cover that exceed 3 feet. Table 2-11 shows how the AASHTO H-25 loads are transferred to the pipe based on the height of cover.

Table 2-11: Live Load Data for AASHTO H-25 (Adapted from Gabriel 2008)

Cover, ft	AASHTO H-25 or HS-25		Cover, ft	AASHTO H-25 or HS-25	
	Live Load Transered to Pipe, P_L, psi	Live Load Distribution Width, L_W in		Live Load Transered to Pipe, P_L, psi	Live Load Distribution Width, L_W in
1	15.63	31	14	negligible	N/A
2	6.95	52	16	negligible	N/A
3	5.21	73	18	negligible	N/A
4	3.48	94	20	negligible	N/A
5	2.18	115	22	negligible	N/A
6	1.74	136	24	negligible	N/A
7	1.53	157	26	negligible	N/A
8	0.86	178	28	negligible	N/A
10	negligible	N/A	30	negligible	N/A
12	negligible	N/A	35	negligible	N/A

Notes:
1.) Includes impact where required
2.) N/A indicates that the information is not applicable

In addition to live loads, construction loads also need to be factored in when dealing with live loads. If heavy equipment is used the minimum cover may need to be increased to account for the higher loads (Gabriel 2008).

2.7.1.3 Dead Loads

The dead load that needs to be considered in the design of plastic pipes consists mainly of the soil load over the pipe. There are two different ways to calculate the soil loads, the soil column load (W_C) and the soil arch load (W_A). These two calculations, as well as when to use each, will be discussed in this section.

The soil column load (W_C) is used to determine the deflection and can be defined as the weight of the soil above the outside diameter of the pipe from the pipe's crown to the surface. The equation used to calculate the soil column load is shown below (Gabriel 2008):

$$W_C = \frac{H\gamma_S OD}{144} \quad \text{Equation 2-5}$$

Where:
W_C = soil column load (lb/linear inch of pipe),
H = burial depth to top of pipe (ft),
γ_S = soil density (pcf), and
OD = outside diameter of pipe (in).

The soil arch load (W_A) is a more accurate assessment of the actual load the pipe is experiencing and must be used when determining the wall thrust. The soil arch load is calculated by multiplying a vertical arching factor (VAF) by a geostatic load thereby reducing the earth load to account for the support provided by the adjacent soil columns. The geostatic load is defined as the weight of the soil directly above the outside diameter of the pipe plus a small triangular load extending beyond the outside diameter. The following is the equation for the geostatic load (Gabriel 2008):

$$P_{sp} = \frac{(\gamma_S)\left(H + 0.11\frac{OD}{12}\right)}{144} \quad \text{Equation 2-6}$$

Where:
P_{sp} = geostatic load (psi),
H = burial depth to top of pipe (ft),
γ_S = soil density (pcf), and
OD = outside diameter of pipe (in).

After the geostatic load has been calculated, the vertical arching factor must be determined. The vertical arching factor is based on studies where pipes with a high hoop stiffness ratio (ratio of soil stiffness to pipe hoop stiffness) carry substantially less load than the weight of the prism of soil directly above the pipe (AASHTO LRFD 2010). The following is the equation for the vertical arching factor (Gabriel 2008):

$$VAF = 0.76 - 0.71\left(\frac{S_h - 1.17}{S_h + 2.92}\right) \qquad \text{Equation 2-7}$$

Where:

	VAF	=	vertical arching factor (dimensionless),
	S_h	=	hoop stiffness factor:
		=	$\dfrac{\phi_s M_S R}{EA_s}$
	ϕ_s	=	capacity modification factor for soil (0.9),
	M_S	=	secant constrained soil modulus (psi),
	R	=	effective radius of pipe (in),
		=	$\dfrac{ID}{(2+c)}$
	ID	=	inside diameter of pipe (in),
	c	=	distance from inside diameter to neutral axis (in),
	E	=	modulus of elasticity of polyethylene,
		=	110,000 psi for short-term conditions,
		=	22,000 psi for long-term conditions, and
	A_s	=	section area (in^2/in).

Once the geostatic load and the vertical arching factor have been calculated, the soil arch load can be determined by the following equation (Gabriel 2008):

$$W_A = P_{sp} VAF \qquad \text{Equation 2-8}$$

Where: W_A = soil arch load (psi),

P_{sp} = geostatic load (psi), and

VAF = vertical arching factor (dimensionless).

2.7.2 Design Procedure

Once loads have been calculated, the design procedure can proceed. For a successful design, the pipe must be checked for wall thrust, deflection, buckling, bending stress, and bending strain. This section briefly reviews the process by giving the applicable equations to be checked for each limit state.

2.7.2.1 Wall Thrust

The wall thrust in a pipe can be defined as the stress in the pipe corresponding to the total load (dead and live) on the pipe (Gabriel 2008). The pipe must be able to handle the critical wall thrust in order for it to remain structurally stable. There are two possible cases that must be checked for wall thrust depending on the depth of installation. If the installation only involves dead load, the case when the pipe is generally deeper than 8 ft, then the wall thrust can be checked using the long-term material properties. If the installation is shallow (less than 8 ft) where dead and live loads are present, then two wall thrust analyses must be checked. One analysis would include both the dead and live loads utilizing the short-term material properties, while the other analysis would include only the dead loads and use the long-term material properties. The case that produces the highest thrust would govern. Below is the equation for the critical wall thrust followed by the equation used to calculate the wall thrust. The calculated wall thrust must be less than the critical wall thrust in order for the pipe to remain structurally stable (Gabriel 2008).

Critical Wall Thrust: $$T_{cr} = F_y A_s \phi_p \qquad \text{Equation 2-9}$$

Where: T_{cr} = critical wall thrust (lb/linear inch of pipe),

F_y = minimum tensile strength of polyethylene (psi),

= 3000 psi for short-term conditions,

= 900 psi for long-term conditions,

A_s = section area (in^2/inch of pipe), and

ϕ_p = capacity modification factor for pipe (1.0).

Calculated Wall Thrust: $$T = 1.3(1.5W_A + 1.67P_L C_L + P_W)\left(\frac{OD}{2}\right) \qquad \text{Equation 2-10}$$

Where: T = calculated wall thrust (lb/inch),

W_A = soil arch load (psi)

P_L = See Table 2-11,

C_L = live load distribution coefficient,

= the lesser of (L_W / OD) or 1.0,

L_W = live load distribution width at the crown (in),

OD = outside diameter of pipe (in), and

P_W = hydrostatic pressure at springline of pipe (psi).

2.7.2.2 Deflection

The next limit state that must be checked is the deflection. Deflection in general is controlled through proper construction practices in the field, and the contractor bears most of the responsibility. Even though the burden falls on the contractor, it is the designer’s responsibility to check the feasibility of the specification written (AASHTO LRFD 2010). The vertical deflection of the pipe is usually the most important in design, and it is often limited to between 5 to 7.5% maximum deflection of the base inside diameter. In order to compute the deflection, the pipe stiffness, dead and live loads, and the backfill conditions are needed. Note that the soil prism load is used instead of the reduced load used to compute thrust. The following is the modified Iowa equation that predicts the deflection based on the conditions given (Gabriel 2008):

Deflection: $$\Delta y = \frac{K(D_L + W_C + W_L)}{0.149PS + 0.061E'}$$ Equation 2-11

Where: Δy = deflection (in),

K = bedding constant, dimensionless (often assumed to be 0.1),

D_L = deflection lag factor, dimensionless; 1.0 when the soil column load is used,

W_C = soil column load on pipe (lb/linear inch of pipe),

W_L = live load (lb/linear inch of pipe),

= $OD * P_L$

OD = outside diameter of pipe (in),

PS = pipe stiffness (pii), and

E' = modulus of soil reaction (psi).

2.7.2.3 Buckling

Buckling is the next limit state to be checked in the design process. The critical constraints on buckling deal with the burial conditions and the pipe stiffness. Following are equations for the critical buckling pressure and the actual buckling pressure. The actual buckling pressure must be calculated to be less than the critical buckling pressure in order to have a safe design (Gabriel 2008).

Critical Buckling Pressure: $$P_{CR} = \frac{0.772}{SF}\left[\frac{E^{'}PS}{1-v^{2}}\right]^{1/2}$$ Equation 2-12

Where: P_{CR} = critical buckling pressure (psi),

$E^{'}$ = modulus of soil reaction (psi),

PS = pipe stiffness (pii)

v = poisson ratio, dimensionless; 0.4 for polyethylene, and

SF = safety factor (2.0).

Actual Buckling Pressure: $$P_{V} = \frac{R_{W}H\gamma_{s}}{144} + \frac{\gamma_{w}H_{w}}{144} + \frac{W_{L}}{OD}$$ Equation 2-13

Where: P_{V} = actual buckling pressure (psi),

R_{W} = water buoyancy factor (dimensionless),

= $1-0.33\left(\frac{H_{W}}{H}\right)$

H = burial depth to top of pipe (ft),

γ_{s} = soil density (pcf),

γ_{W} = unit weight of water (62.4 pcf),

H_{W} = height of groundwater above top of pipe (ft),

W_{L} = live load (lb/linear inch of pipe), and

= $OD * P_{L}$

OD = outside diameter of pipe (in).

2.7.2.4 Bending

One last thing the pipe must be checked for is bending. For this check the bending stress and the bending strain must be checked to ensure that they fall within the material's capability. The bending stress calculated in the pipe must not exceed the long-term tensile strength of polyethylene, 900 psi, and the bending strain must not exceed 5%. The equations for calculating both bending stress and bending strain are shown below (Gabriel 2008).

Bending Stress: $$\sigma_b = \frac{(2)(D_f)(E)(\Delta y)(y_O)(SF)}{D_M{}^2}$$ Equation 2-14

Where:

σ_b	=	bending stress (psi),
D_f	=	shape factor (dimensionless),
E	=	long-term modulus of elasticity of HDPE (22,000 psi),
Δy	=	deflection (in),
y_o	=	distance from centroid of pipe wall to the furthest surface of the pipe (in),
	=	the greater of $\frac{OD - D_M}{2}$ or $\frac{D_M - ID}{2}$,
OD	=	outside diameter of pipe (in),
ID	=	inside diameter of pipe (in),
SF	=	safety factor (1.5),
D_M	=	mean pipe diameter (in), and
	=	$ID + 2c$
c	=	distance from inside diameter to neutral axis (in).

Bending Strain: $$\varepsilon_B = \frac{(2)(D_f)(\Delta y)(y_O)(SF)}{D_M{}^2}$$ Equation 2-15

Where:

ε_B	=	bending strain (in/in),
D_f	=	shape factor (dimensionless),
Δy	=	deflection (in),

y_O = distance from centroid of pipe wall to the furthest surface of the pipe (in),

= the greater of $\frac{OD - D_M}{2}$ or $\frac{D_M - ID}{2}$

OD = outside diameter of pipe (in),

ID = inside diameter of pipe (in),

SF = safety factor (1.5),

D_M = mean pipe diameter (in), and

= $ID + 2c$

c = distance from inside diameter to neutral axis (in).

2.8 Current Practice for Installations

The current state of practice for the installation of plastic pipe will be reviewed throughout the next section in order to discover common installation practices as well as the most appropriate installation practices. The two specifications that are most widely used are AASHTO LRFD Bridge Construction Specification (Section 30) and ASTM D 2321--*Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications*. ASTM D 2321 provides recommendations for the installation of thermoplastic pipe. While the standard does not address pipe performance criteria such as minimum pipe stiffness, maximum service deflection, or long term strength, it does give detailed information about the type of soils and installation procedure to be used. Both of the AASHTO and ASTM specifications are reviewed below along with other specifications from plastic pipe organizations and state departments of transportation. The review has been broken into sections similar to those outlined in ASTM D 2321 and the AASHTO LRFD Bridge Construction Specifications.

2.8.1 Terminology

Terminology for plastic pipe installations varies slightly depending upon the source of the specification. This section will attempt to display all necessary terms dealing with the installation of plastic pipe in order to allow complete understanding. Figure 2-14 is taken from ASTM D 2321 (2000) and displays the most common terms dealing with plastic pipe trench installations.

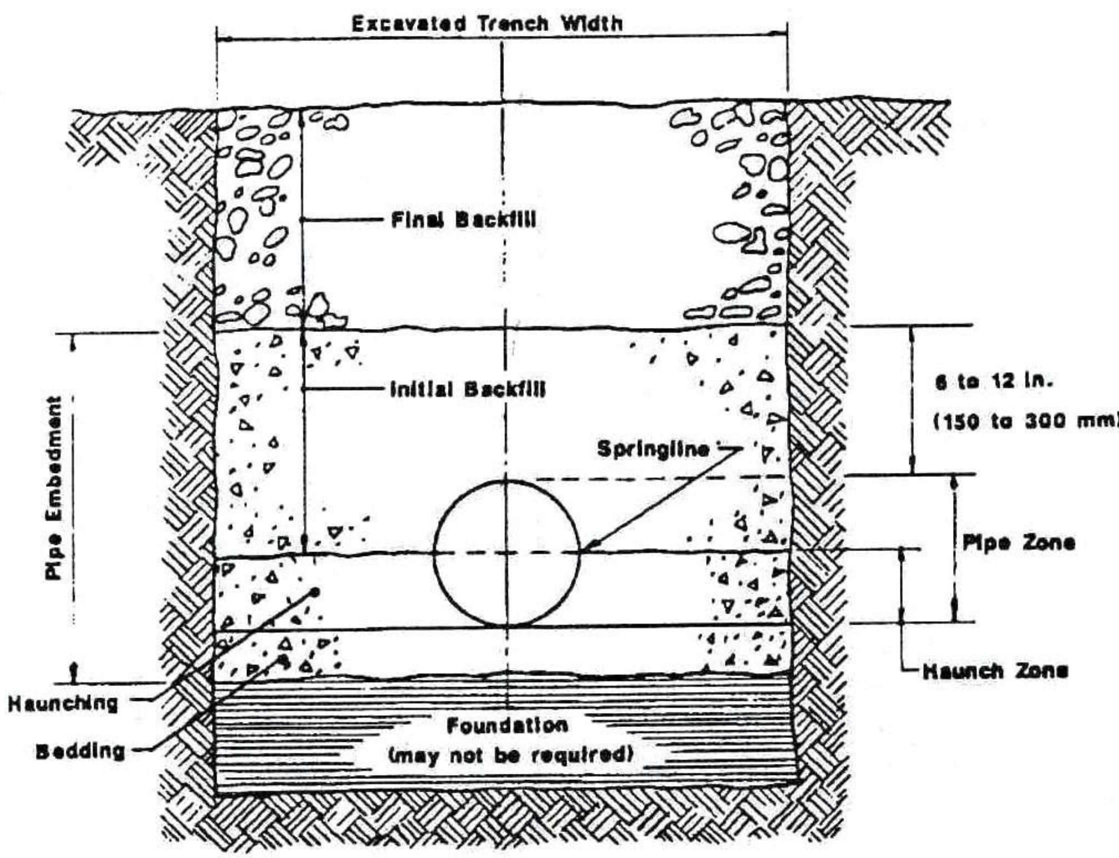


Figure 2-14: Trench Cross Section Showing Terminology (ASTM D 2321)

The following is a list of terms and definitions of common trench terminology (PPI 2010):

- Foundation – A foundation, which may not be required, is used when the native trench bottom does not provide a firm working platform for placement of the pipe bedding material.
- Bedding – The bedding levels the trench bottom to ensure the pipe receives uniform support along its length. The bedding also brings the trench bottom to the required grade.
- Springline – mid height of the pipe.
- Haunching – The haunch zone is part of the initial backfill and refers to the backfill located below the springline of the pipe, which is important in distributing the superimposed loads.
- Initial Backfill – This is a critical zone of embedment soil that reaches from the foundation to a minimum of 12 inches above the pipe. Within the initial backfill zone there are bedding, haunching, sidefill, and topfill zones.
- Final Backfill – The final backfill has less influence on the pipe than the initial backfill, but it should also be a stiff backfill to allow for arching and thus a load reduction.

Two other terms that are not displayed in Figure 2-14 are sidefill and topfill. Sidefill can be defined as the backfill located on the sides of the pipe between the springline and the crown (top) of the pipe. Topfill can be defined as the backfill located between the crown of the pipe and the top of the initial backfill (AASHTO LRFD 2010).

2.8.2 Trench Widths

Trench widths are important in construction because adequate room should be available on either side of the pipe to allow for compaction of the backfill material in the haunching zone. The AASHTO LRFD Bridge Construction Specification recommends that the trench be wide enough to operate the compaction equipment safely on both sides of the pipe, but not less than 1.5 times the outside pipe diameter plus 12 inches. ASTM D 2321 also gives procedures for trench excavation. According to ASTM D 2321, trenches should be excavated so that they remain stable and are of adequate size. The minimum trench width allowed is not less than the greater of the pipe outside diameter plus 16 inches or the pipe outside diameter times 1.25, plus 12 inches. ASTM D 2321 also notes that special care must be taken to control water and that water should be prevented from entering the trench during installation.

Trench widths also become important when multiple buried pipes reside in a common trench. The Plastics Pipe Institute (PPI) recommends using a minimum separation of 12 inches for pipes 24 inches or less, and a separation of the diameter of the pipe divided by 2 for pipes larger than 24 inches in diameter (Gabriel 2008). Gabriel (2008) states that these dimensions may need to be increased depending on the type of backfill, type of compaction techniques, and method of joining.

2.8.3 Bedding and Backfill Materials

ASTM and AASHTO classify soils using different systems. Soils can be classified by either the Unified Soil Classification System (ASTM D 2487) or the AASHTO Soil Classification System (AASHTO M 145). The Unified Soil Classification System is based on laboratory tests to determine the particle-size characteristics, the liquid limit, and the plasticity index. These qualities are then used to describe a soil to aid in the evaluation of its significant properties for engineering use (ASTM D 2487).

ASTM D 2321 divides the soils into different "Classes" (Gassman 2006). Table 2-12 shows the equivalent ASTM and AASHTO Soil Classifications.

Table 2-12: Equivalent ASTM and AASHTO Soil Classifications (Gassman 2006)

Basic Soil Type	ASTM D 2487	AASHTO M 145	ASTM D 2321
Sn (Gravelly sand)	SW, SP, GW, GP Sands and gravels with 12% or less fines	A-1, A-3	Class IB Manufactured processed aggregates, dense graded, clean Class II. Coarse-grained soils clean
Si (Sandy silt)	GM, SM, ML Also GC and SC with less than 20% passing a No. 200 sieve	A-2-4, A-2-3, A-4	Class III. Coarse-grained soils with fines Class IVA: Fine-grained soils with no to low plasticity
Cl (Silty clay)	CL, MH, GC, SC Also GC and SC with more than 20% passing a No. 200 Sieve	A-2-6, A-2-7, A-5, A-6	Class IVA: Fine-grained soils with low to meduim plasticity

Requirements for bedding are important because the pipe along with the surrounding soil make up a soil-structure system. AASHTO LRFD Bridge Construction Specification Section 30 lists several requirements for bedding and backfill materials. First, Section 30.3.2.1 states that the bedding shall be a granular material with a maximum particle size of one inch that is free of organic material, rock fragments larger than 1.5 inches, and frozen lumps. Section 30.3.2.1 also requires the backfill to meet the minimum requirements of AASHTO M145 for A-1, A-2-4, A-2-5, or A-3 soils. In addition to these minimum requirements, it also requires that a maximum of 50% of the particles pass the No. 100 sieve, and a maximum of 20% pass the No. 200 sieve. This last requirement is designed to eliminate soils with large amounts of fine sands and silts. Note that these additional requirements eliminate some A-1b, A-2-4, A-2-5, and A-3 soils for use as backfill materials (AASHTO LRFD 2010).

ASTM D 2321 groups soils into five classes. Class I (angular, crushed stone or rock containing little or no fines), Class II (coarse-grained soils, clean), and Class III (coarse-grained soils with fines) soils are deemed suitable for installation, while Class IV (fine-grained soils) and Class V (organic soils) soils are not recommended. The maximum particle size recommended for embedment is limited to materials passing a 1½ inch sieve. Table 2-13 taken from the ADS, Inc. Drainage Handbook shows the ASTM D 2321 and ASTM D 2487 classes of embedment and backfill materials.

Table 2-13: Classes of Embedment and Backfill Materials (ADS 2009)

ASTM D2321[1] Class	ASTM D2321[1] Description	Notation	ASTM D2487 Description	AASHTO M43 Notation	Min. Compaction Required (Std Proctor Densit %)[3]	ASTM D2321 [1] Percentage Passing Sieve Sizes 1 ½ in. (40mm)	No. 4 (4.75mm)	No. 200 (0.075mm)	Atterberg Limits LL	PI	Coefficients Uniformity Cu	Curvature Cc
IA[4]	Open-graded, clean manufactured aggregates	N/A	Angular crushed stone or rock, crushed gravel, crushed slag; large voids with little or no fines	5 56	Dumped to Slight	100%	≤10%	<5%	Non Plastic		N/A	
IB	Dense-graded clean manufactured, processed aggregates	N/A	Angular crushed stone or other Class IA material and stone/sand mixtures with gradations selected to minimize migration of adjacent soils; little or no fines			100%	≤50%	<5%	Non Plastic			
II	Clean, coarse-grained soils	GW	Well-graded gravel, gravel-sand mixtures; little or no fines	57 6 67	Moderate (85%	100%	<50% of "Coarse Fraction"	<5%	Non Plastic		>4	1 to 3
		GP	Poorly-graded gravels, gravel-sand mixtures; little or no fines								<4	<1 or >3
		SW	Well-graded sands, gravelly sands; little or no fines				>50% of "Coarse Fraction"				>6	1 to 3
		SP	Poorly-graded sands, gravelly sands; little or no fines								<6	<1 or >3
	Coarse-Graine Soils, borderlin clean to w/fines	GW-GC, SP SM	Sands and gravels which are borderline between clean and with fines	N/A		100%	Varies	5% to 12%	Non Plastic		Same as for GW, GP, SV and SP	
III	Coarse-graine soils with fines	GM	Silty gravels, gravel-sand-si mixtures	Gravel & sand with <10% fine	Moderate to High (90%)	100%	<50% of "Coarse Fraction"	12% to 50%	N/A	<4 or <"A" Line	N/A	
		GC	Clayey gravels, gravel-sanc clay mixtures							<7 & >"A" Line		
		SM	Silty sands, sand-silt mixtures				>50% of "Coarse Fraction"			>4 or <"A" Line		
		SC	Clayey sands, sand-clay mixtures							>7 & >"A" Line		
IVA[2]	Inorganic fine-grained soils	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, silts with slight plasticity	N/A	N/R	100%	100%	>50%	<50	<4 or <"A" Line	N/A	
		CL	Inorganic clays of low to medium plasticity; gravelly, sandy, or silty clays; lean clays	N/A						>7 & >"A" Line		
IVB	Inorganic fine-grained soils	MH	Inorganic silts, micaceous c diatomaceous fine sandy o silty soils, elastic silts	N/A	N/R	100%	100%	>50%	>50	<"A" Line	N/A	
		CH	Inorganic clays of high plasticity, fat clays	N/A						>"A" Line		
V	Organic soils o Highly organic soils	OL	Organic silts and organic silty clays of low plasticity	N/A	N/R	100%	100%	>50%	<50	<4 or <"A" Line	N/A	
		OH	Organic clays of medium t high plasticity, organic silts	N/A					>50	<"A" Line		
		PT	Peat and other high organi soils	N/A								

A failure to use proper embedment materials can lead to maintenance and replacement costs in the future (Jeyapalan and Boldon 1986).

2.8.4 Bedding

Bedding is the layer of backfill provided under the pipe. AASHTO Section 30.5.6 suggests compacting the bedding layer to a minimum density equal to 90% of the maximum dry density according to AASHTO T99, except the portion of the bedding layer under the center third of the pipe diameter which should remain uncompacted. AASHTO recommends that the loose bedding layer be a maximum thickness of 6 inches (AASHTO LRFD 2010). ASTM D 2321 recommends a minimum of 4 inches of bedding that provides a firm, stable, and uniform bedding layer for pipe and any protruding features of its joints.

2.8.5 Structural Backfill

Once the bedding has been prepared and the pipe placed, the structural backfill that comprises the soil-structure must be carefully compacted. AASHTO Section 30 breaks down the three stages of backfilling into haunch, sidefill, and topfill. When backfilling these areas it is important for contractors to establish procedures that will achieve the specified degree of compaction without damaging or excessively distorting the pipe. AASHTO recommends limiting the change in vertical diameter of the pipe to a 3% increase when placing and compacting backfill to the top of the pipe. Another reason this requirement is recommended is because the wall stresses in the pipe due to deformation during compaction can be more severe than the wall stresses caused by the ovaling deformation due to the earth load over the pipe (AASHTO LRFD 2010).

2.8.6 Haunch Zone

Backfilling in the haunch zone is important because the haunch backfill supports the pipe below the springline. AASHTO states that the haunch backfill must be placed in 6 inch loose layers and compacted to greater than 85% maximum dry density according to AASHTO T99. It is also recommended that the haunch fill be placed simultaneously on both sides to avoid rolling the culvert. It is recommended that special attention should also be given to make sure the compaction forces do not lift the pipe off grade and that the bottom of the pipe is not damaged. Compaction methods should be chosen carefully because the amount of effort required to achieve a particular compaction is highly dependent on the type of backfill material. AASHTO Section 30 recommends accomplishing the compaction by placing part of the first layer of backfill, working it into the haunches and then placing the remainder the backfill to complete the lift (AASHTO LRFD 2010). ASTM D 2321 also requires 6 inch layers but adds that the layers should be worked in around the pipe by hand to provide uniform support. ASTM D 2321 also states that a lack of adequate compaction of the material in the haunch zone can result in excessive deflections because it is this material that supports the vertical loads applied to the pipe. When backfilling material around the pipe, measures should be taken to not disturb or damage the pipe. The haunching material should be carefully worked in and tamped from the bedding up to the underside of the pipe before placing and compacting the backfill in the embedment zone (ASTM D 2321).

2.8.7 Sidefill

AASHTO Section 30 has several requirements in regards to placing structural backfill in the sidefill zone. First, the backfill must be placed in horizontal, uniform layers not exceeding 6 inch loose thickness. The layers must then be compacted to a maximum density not less than 90% for A-1 and A-3 soils and 95% for A-2 soils. It is also recommended that the sidefill backfill be brought up evenly on both sides with a maximum difference in the sidefill elevations not exceeding the smaller of one-quarter of the diameter of the pipe or 24 inches. Another recommendation is to prevent excessive force on the pipe from equipment used to compact the sidefill (AASHTO LRFD 2010). ASTM D 2321 requires that all materials used in embedment compaction be compacted to a minimum of 85% Standard Proctor by using hand tampers or vibratory compactors. Special care should be taken to ensure that compaction equipment never comes into contact with the pipe to prevent damaging the pipe (ASTM 2321).

2.8.8 Topfill

Topfill is the backfill material found above the sidefill and begins just above the crown of the pipe. AASHTO requires a 6 inch layer of topfill above the top of the pipe. ASTM D 2321 also requires that its "initial backfill" be installed to a minimum of 6 inches above the crown of the pie. Topfill is not considered a minimum cover and additional backfill material should be placed over the pipe to prevent damage of the pipe from construction loads.

2.8.9 Pipe Flotation

While the lightweight of plastic pipe makes it desirable for its ease of handling and installation, this same benefit causes it to be prone to flotation (ADS 2010). While all pipe types can be subjected to flotation, it is much more critical for plastic pipe. The pipe tends to rise or heave when the upward force becomes greater than the downward force of the weight of the pipe along with the load it carries. If a pipe will be placed where flotation is a concern, it is critical to use proper installation techniques and/or anchor the pipe. Pipe flotation can usually be addressed with adequate cover, but in situations where adequate cover cannot be achieved, there are other methods for restraining the pipe. Some possible methods include using a geotextile wrap, using a concrete collar, or by using screw anchors (ADS 2010).

2.9 Minimum Cover

Minimum cover is a minimum depth of backfill that must be used in order to protect the pipe. The thickness of soil above the pipe is the primary factor that controls the intensity of the load that gets imposed on the pipe (Zhao et al. 1998). Currently there are minimum cover requirements used to protect the pipe from construction loads and there are minimum cover requirements that are required for in-service use. Minimum cover specified for each of these reasons varies widely throughout the industry as well as with many state departments of transportation (DOTs).

2.9.1 Research on Minimum Cover

In a 1990 Transportation Research Record journal, Katona describes his research into minimum cover heights for corrugated plastic pipes under vehicle loadings. At that time the tentative guideline for minimum cover of plastic pipe, as suggested by the AASHTO Flexible Culvert Committee, was taken directly from the metal culvert industry, the American Iron and Steel Institute (AISI) (Katona 1990). The AISI specification required a minimum of 12 inches of soil cover for all pipes up to 96 inches. The 12 inches did not include any pavement thickness and that depth was based on long-term observations by the corrugated steel pipe industry of structural performance under live loads. The fact that corrugated plastic pipes are significantly more flexible than corrugated steel pipes led Katona (1990) to perform research using a computer program named Culvert Analysis and Design (CANDE) to determine minimum cover requirements for plastic pipe. Katona (1990) computed minimum fill heights for pipes ranging from12 to 36 inches using both a “fair” and “good” quality soil. Those two cases were represented by the Duncan soil model for silty-clayey sand at 85 percent compaction and silty-clayey sand at 100 percent compaction respectively. For the “good” soil conditions, Katona (1990) found that a minimum cover of 12 inches would be adequate for all pipes from 12 to 36 inches in diameter. For the “fair” soil conditions, Katona (1990) found that the minimum cover could range from 12 to 21 inches depending on the diameter of the pipe. From these results, it is can be seen that the amount of compaction plays an important role in the performance of plastic pipes.

In a 1997 report to the Iowa Department of Transportation, the computer software CANDE was again used to determine minimum fill heights for HDPE (Lohnes et al. 1997). The variables investigated included four HDPE pipes and six backfill conditions. The report states that the deflections are reduced dramatically when the soil cover is increased from 12 to 24 inches. It goes on the state that both the 36 and 48 inch diameter HDPE pipes with a soil cover of at least 24 inches under any of the six backfill conditions investigated meet the 5% deflection criterion. Lohnes et al. (1997) state that it is generally accepted that corrugated polyethylene pipes perform well under live loads with shallow cover, provided that the backfill is well compacted. Even though industry standards require that backfill be carefully compacted, concern exists that poor inspection and/or faulty construction may result in soils that provide inadequate passive restraint at the springlines of the pipes, thereby leading to failure (Lohnes et al. 1997).

In 2002, McGrath et al. published a report on the performance of large diameter pipe under highway vehicle loading. The study consisted of field tests that were conducted at the MnRoad Research Facility, which consists of a two-lane test road traversed by test vehicles. The test consisted of 10 runs of pipe. There were 8 runs of thermoplastic pipe and one each of reinforced concrete and corrugated steel pipe. The pipes were buried with 1 foot and 2 feet of cover, which was measured from the top of the pipe to the top of the pavement. The pipes were backfilled with A-1 and A-2 materials according to AASHTO M 145 and compacted to levels of 85% to 90% compaction based on soil density tests. After construction, there was immediate pavement settlement which was assumed to have resulted from the low levels of compaction used for the backfill and not as a result of pipe deflection. Once installed, the pipes were subjected to loading by the test vehicles which had axle loads that ranged between 18,000 and 24,000 lbs. The deflections measured in the pipes were generally small and averaged 0.31 inches for the vertical deflection and 0.08 inches for the horizontal deflection. The corrugated steel pipe showed less deflection than the polyethylene pipe, but it was not significant since all deflections were less than1%. While the report showed that initial live load response was small and that the pipes were not buried at a limiting depth, the report cautions that continued observation should be undertaken before any conclusions are drawn (McGrath et al. 2002).

In 2006, Arockiasamy et al. published a report describing the procedure and results of a full-scale field test on HDPE, PVC, and metal large diameter pipes subjected to highway loadings. During the study, 36 pipes were buried and tested under the following three burial depths: 0.5D, 1D, and 2D (where D is the diameter of the pipe). The diameters of the pipes used were 36 and 48 inches. The pipes were backfilled with a poorly graded sand with silt while the backfill compaction was tightly controlled to ensure that a minimum dry density of 95% of the Standard Proctor was achieved. The vertical pipe deflections during installation were only taken for the HDPE and metal pipes. The maximum recorded deflections when the backfill was filled up to the pipe crown level were approximately 0.2, 1.1, and 0.5% for the steel, aluminum, and HDPE

48 inch diameter pipes, respectively. Arockiasamy et al. (2006) stated that AASHTO's 5% deflection design limit is found to be adequate for flexible pipes during the installation under shallow burial applications. Once the live loads were applied, the deflections were checked again. The maximum vertical pipe deflection due to the effect of the live load was found to be 0.6% in the 36 inch diameter HDPE pipe with a 0.5D burial depth. Because deflections increase with repeated live loads, Arockiasamy et al. suggest limiting the vertical deflection to 2% for HDPE pipes during the construction phase for roadway and highway applications. In addition to deflection testing, the study also made visual observations of the pipe joints as well as other pipe distresses. After the application of the live load, the pipes were inspected and there were no visible pipe joint openings, and the pipes did not exhibit any visible localized bulging, wall buckling, wall crushing, cracking, or tearing (Arockiasamy et al. 2006). Arockiasamy et al. (2006) suggest that further studies be conducted to assess the effects of repeated loading effects.

2.9.2 Current Standards

Minimum cover for normal use varies somewhat between specifying agencies and state DOTs. As shown in Table 2-14, there is considerable variability among the minimum cover requirements specified by the 38 state DOTs that responded to the 2006 survey administered by the Arizona Department of Transportation (ADOT 2006).

Table 2-14: Minimum Fill Heights for Corrugated HDPE Pipe (Ardani et al. 2006)

State	Minimum fill height, ft	State	Minimum fill height, ft
1. Alaska	2	21. Montana	2
2. California	2	22. Nebraska	1
3. Colorado	2	23. New Mexico	1
4. Connecticut	1	24. New York	1
5. Delaware	2	25. North Carolina	1.5
6. Florida	0.75-1.75	26. Ohio	2
7. Georgia	2	27. Oklahoma	15 to 50 in. to D = 60 in.
8. Hawaii	2-4 to D = 60 in.	28. Oregon	1
9. Idaho	2	29. Rhode Island	3
10. Illinois	1	30. South Carolina	2 to D = 60 in.
11. Indiana	2	31. Tennessee	1
12. Iowa	1	32. Texas	2
13. Kentucky	2	33. Utah	2
14. Louisiana	1	34. Vermont	3
15. Maine	2	35. Virginia	2
16. Massachusetts	4	36. Washington	2
17. Michigan	3	37. West Virginia	2
18. Minnesota	3	38. Wisconsin	1
19. Mississippi	1	39. Hancor	2 to D = 60 in.
20. Missouri	1 – 2 to D = 60 in.	40. PPI	1.5 to D = 60 in.

Note: D = pipe diameter

Table 2-14 shows that minimum heights of fill are generally in the range of 1 to 3 feet, with 2 feet being the most prevalent. Larger diameter sizes typically have higher minimum cover requirements. How states specify minimum cover varies widely across the country. Some states give a single minimum cover value, while other states give minimum cover requirements based on whether the road is unpaved or paved with asphalt or concrete.

AASHTO recommends providing a cover of at least 2 feet before allowing vehicles or construction equipment to cross the trench. The Advanced Drainage Systems (ADS)/Hancor, Inc. Drainage Handbook sets the minimum height of cover to 1 foot for pipes 4 to 48 inches in diameter and to 2 feet for pipes 54 and 60 inches in diameter (Ardani et al. 2006). The Plastics Pipe Institute (Gabriel 2008) sets the minimum heights of fill to 1 foot for pipes that are 4 to 48 inches in diameter and to 1.5 feet for pipes with 54 and 60 inch diameters.

2.9.3 Minimum Cover for Construction Loads

Construction equipment used during the placement of the pipe and for compaction of the soil can cause greater loads on the pipe than the vehicular loads for which the pipe was designed (Katona 1990). Therefore, the pipe sometimes must be protected from those excessive construction loads. Figure 2-15 shows an example of when construction cover might be needed even above final finished grade to allow for heavy construction equipment to pass.

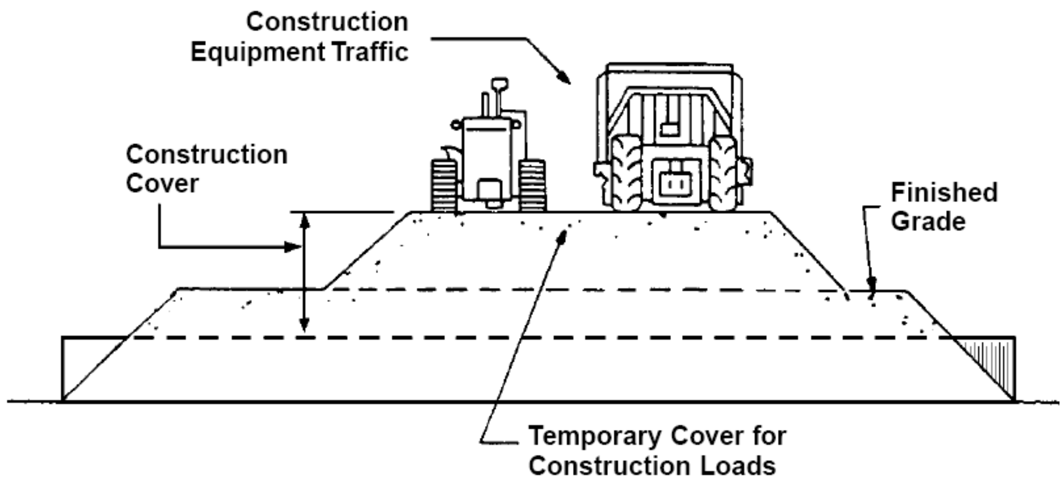


Figure 2-15: Temporary Cover for Construction Loads (UDOT 2008)

ASTM D 2321 specifies a minimum cover of at least 24 inches or one pipe diameter for Class I soils and a cover of at least 36 inches or one pipe diameter for Class II and III soils before allowing vehicles or construction traffic over the trench. ASTM D 2321 also requires at least a 48 inch cover before a hydrohammer can be used for compaction. AASHTO Section 30 currently recommends using Table 2-15 as a guide for construction cover.

Table 2-15: Minimum Cover for Construction Loads (AASHTO LRFD 2010)

	Minimum Cover, in., for Indicated Axle Loads, kips			
Nominal Pipe Diameter, ft	18.0-50.0 kips	50.0-75.0 kips	75.0-110.0 kips	110.0-150.0 kips
2.0-3.0 ft	24.0 in	30.0 in	36.0 in	36.0 in
3.5-4.0 ft	36.0 in	36.0 in	42.0 in	48.0 in
4.5-5.0 ft	36.0 in	36.0 in	42.0 in	48.0 in

Minimum cover is defined by AASHTO as being measured from the top of the pipe to the top of the maintained construction roadway surface (AASHTO LRFD 2010). Most state DOTs specify a cover that is 1 to 2 feet higher than normal for construction loads in order to account for heavy construction equipment. Those values usually range from approximately 3 to 4 feet (Ardani et al. 2006). Advanced Drainage Systems (ADS) also gives recommendations for minimum cover requirements for construction equipment as can be seen in Table 2-16.

Table 2-16: Minimum Cover for Heavy Construction Equipment (ADS 2010)

	SURFACE LIVE LOADING CONDITION	
PIPE DIAM.	H-25	HEAVY CONSTRUCTION (75T AXLE LOAD)
12" - 48"	12"	48"
54" - 60"	24"	60"

* VEHICLES IN EXCESS OF 75T MAY REQUIRE ADDITIONAL COVER

2.10 Maximum Cover

Maximum cover can be defined as the maximum depth of soil that can safely be placed above the crown of pipe. Figure 2-16 shows how the maximum height of cover is determined for a pipe once installed.

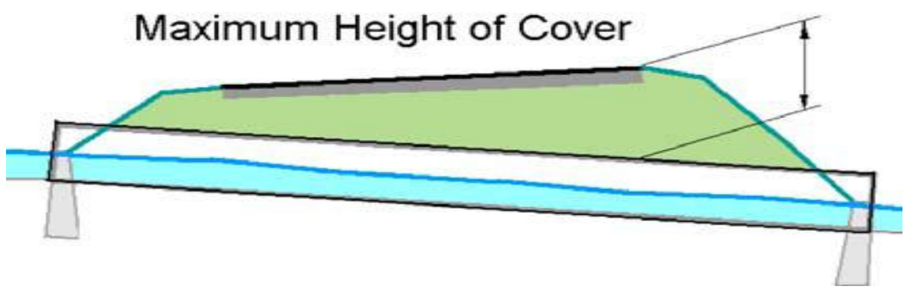


Figure 2-16: Maximum Height of Cover (UDOT 2008)

Currently, there are many different opinions on what the maximum height of cover should be. There has been research done on the subject, but many state DOTs and specifying agencies still have widely varying requirements.

2.10.1 Research on Maximum Cover

In a 1987 Transportation Research Record journal, Katona describes his research in which he developed fill height tables for the maximum allowable burial depth of polyethylene pipe for diameters up to 30 inches. The tables were developed using the computer program CANDE. Katona (1987) computed maximum fill heights for pipes ranging from 4 to 30 inches using both a “fair” and “good” quality soil as well as using both short and long-term properties for the pipes. Those two cases were represented by the Duncan soil model for silty-clayey sand at 85 percent compaction and silty-clayey sand at 100 percent compaction respectively. The long-term properties for the HDPE controlled for all pipe sizes. For the “good” quality soils, maximum fill heights ranged from 22 to 28 feet. For the “fair” quality soils, maximum fill heights ranged from approximately 10 to 13 feet (Katona 1987). While only completing the models for two types of soil, Katona (1987) created graphs and equations to interpolate between “fair” and “good” quality soils.

2.10.2 Current Standards

Maximum cover requirements for plastic pipe installations vary widely between specifying agencies, pipe manufacturers, and state DOTs. The Plastics Pipe Institute and the Drainage Handbook by Hancor, Inc. have constructed Table 2-17 and Table 2-18 respectively listing maximum fill heights. These tables are shown below and are divided up by pipe diameter, type of backfill, and degree of compaction.

Table 2-17: Maximum Fill Height Recommended by PPI, ft (Ardani et al. 2006)

Diameter (inches)	Class I Uncompacted	Class I Compacted	Class II 85%	Class II 90%	Class II 95%	Class II 100%	Class III 85%	Class III 90%	Class III 95%
4	17	59	17	24	37	59	15	18	24
6	16	57	16	24	36	57	15	17	24
8	14	51	14	21	32	51	13	15	22
10	13	50	13	20	31	50	12	14	21
12	13	49	13	20	31	49	12	14	21
15	13	49	13	20	31	49	12	14	21
18	13	49	13	20	31	49	12	14	21
24	13	51	13	21	32	51	12	14	21
30	13	51	13	21	32	51	12	14	21
36	13	50	13	20	31	50	12	14	21
42	11	47	11	19	29	447	10	13	19
48	11	46	11	18	29	46	10	12	19
54	11	44	11	18	28	44	10	12	18
60	11	45	11	18	28	45	10	12	18

Notes:

1. Class I: Manufactured aggregate, open graded, clean, non-plastic
2. Class II: Coarse-grained soils, clean, non-plastic
3. Class III: Coarse-grained soils with fines, very low plasticity

Table 2-18: Maximum Fill Height Recommended by Hancor, Inc., ft (Ardani et al. 2006)

Diameter (inches)	Class I Compacted	Class I Uncompacted	Class II 95%	Class II 90%	Class II 85%	Class III 95%	Class III 90%	Class III 85%
4	55	17	36	25	17	25	18	16
6	54	16	35	24	16	24	17	15
8	53	16	34	23	16	24	17	15
10	54	16	35	23	16	24	17	15
12	56	18	37	25	18	26	19	17
15	55	17	36	24	17	25	18	16
18	54	17	35	24	17	24	18	16
24	53	15	34	23	15	23	16	14
30	50	14	32	21	14	22	15	13
36	48	13	31	20	13	21	14	12
42	46	12	29	19	12	20	13	11
48	47	12	30	19	12	20	13	11
54	43	11	28	18	11	19	12	10
60	44	11	28	18	11	19	12	10

Notes:

1. Calculations assume no hydrostatic pressure and a density of 120 pounds per cubic foot (pcf) for overburden material

Uni-Bell has created a similar table for PVC pipes that can be seen as Table 2-19. This table breaks up the maximum heights of cover based on the embedment soil class as well as the degree of compaction of the soil.

Table 2-19: Maximum Fill Height Recommended by Uni-Bell (Uni-Bell 2005)

PIPE ZONE CONDITIONS		HEIGHT OF COVER (ft)
EMBEDMENT CLASS	% OF PROCTOR DENSITY RANGE	
I	95-100	50
II	90-100	50
	85	40
	80	24
III	90-100	50
	85	36.0
	80	14
IV	85-100	32
	80	12
V	SOIL CLASS NOT RECOMMENDED	

Maximum fill heights are somewhat varied from DOT to DOT, ranging from a few feet to over 50 feet of cover. As shown in Table 2-20, there is considerable variability among the maximum cover limits specified by the 38 state DOTs who responded to a survey given by the Arizona Department of Transportation (Ardani et al. 2006).

Table 2-20: Maximum Fill Heights for Corrugated HDPE Pipe (Ardani et al. 2006)

State	Maximum, ft	State	Maximum, ft
1. Alaska	30 ft for D = 12-36 in. 20 ft for D = 40-48 in.	21. Montana	10 ft to D = 18 in.
2. California	30 ft for D = 12-36 in. 20 ft for D = 48-60 in.	22. Nebraska	40 ft to D = 36 in.
3. Colorado	30 ft to D = 48 in.	23. New Mexico	10 ft to D = 60 in.
4. Connecticut	8 ft to D = 48 in.	24. New York	15 ft to D = 48 in.
5. Delaware	Not Specified	25. North Carolina	Not Specified
6. Florida	17 ft to D = 48 in.	26. Ohio	20 ft to D = 60 in.
7. Georgia	20 ft to D = 36 in.	27. Oklahoma	10 ft to D = 60 in.
8. Hawaii	22–17 ft to D = 60 in.	28. Oregon	15 to D = 60 in.
9. Idaho	15 ft to D = 48 in.	29. Rhode Island	Not Specified
10. Illinois	15 ft to D = 36 in.	30. South Carolina	18 ft to D = 60 in.
11. Indiana	11 ft to D = 36 in.	31. Tennessee	18 ft to D = 48 in.
12. Iowa	15 ft to D = 48 in.	32. Texas	12 ft to D = 48 in.
13. Kentucky	30 ft for D = 12-36 in. 10 ft for D = 42-48 in.	33. Utah	17-11 ft to D=60 in.
14. Louisiana	5 ft to D = 48 in.	34. Vermont	Not Specified
15. Maine	Not Specified	35. Virginia	21-17 ft to D=48 in.
16. Massachusetts	Not Specified	36. Washington	15 ft to D = 60 in.
17. Michigan	16 ft for D = 12-24 in. 10 ft for D = 25-36 in.	37. West Virginia	Not Specified
18. Minnesota	20 ft to D = 36 in.	38. Wisconsin	15 ft to D = 36 in.
19. Mississippi	50 ft to D = 36 in.	39. ADS	44 ft to D = 60 in.
20. Missouri	38 ft for D = 12-36 in. 8 ft for D = 42-60 in.	40. PPI	45 ft to D = 60 in.

Note: D = pipe diameter

It is important to note that the fill heights specified by the state DOTs are generally more conservative (less cover height allowed) than those recommended by either Advanced Drainage System (ADS) (ADS 2009) or the Plastics Pipe Institute (PPI)

(Gabriel 2008). The most widely used fill heights specified by state DOTs were 10 feet and 20 feet. In general, the larger diameter pipes have lower maximum fill heights. The maximum depth of cover is highly influenced by the types of backfill materials used and the degree in which they are compacted (Ardani et al. 2006). Figure 2-17 shows the dispersion of fill heights reported by the State DOTs surveyed by the Arizona Department of Transportation in 2006.

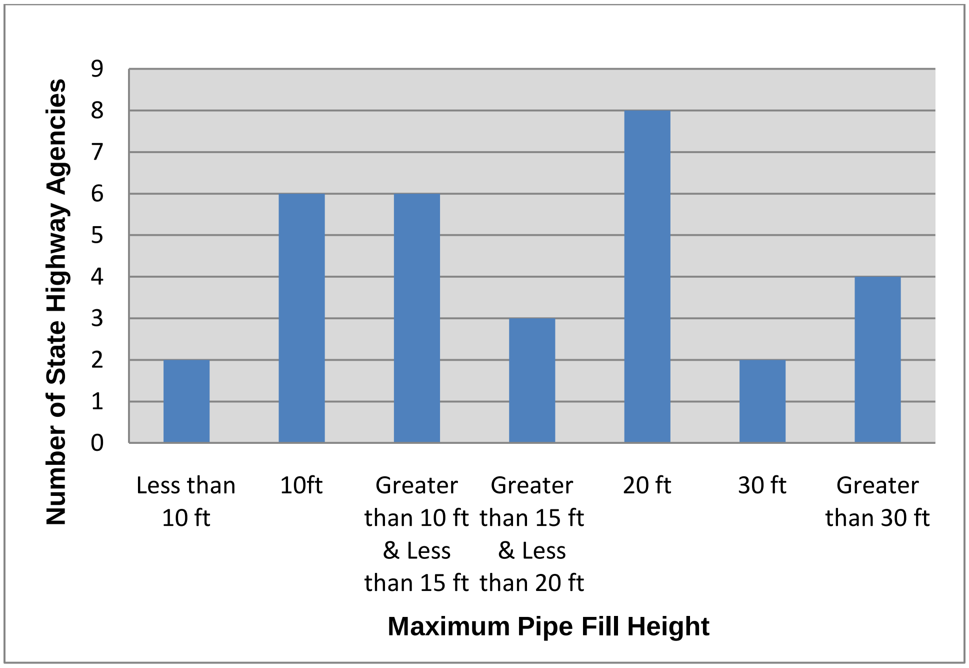


Figure 2-17: Number of States Per Maximum Fill Height (Ardani et al. 2006)

How states choose maximum fill heights varies based on several factors. Some states choose maximums for all pipe sizes, while other states choose maximum cover based on the size of the pipe. The Hawaii DOT takes that a step further and has created a table similar to the tables of Hancor and PPI in which the maximum fill heights are based not only on pipe size but also the type of backfill material and the degree of compaction (Ardani et al. 2006).

2.11 Current Inspection Requirements

Due to the care that must be taken when installing plastic pipe to create the soil-structure system, inspection must be performed during and after the installation in order to ensure proper performance. A summary of current AASHTO recommendations as well as what current state DOTs are requiring will be laid out to help show the current state of practice for these inspections. The current states of practice shown will display the many different strategies being employed and how they compare to one another in terms of time, effort, cost, as well as the amount and quality of information obtained.

2.11.1 AASHTO Specifications

The AASHTO LRFD Bridge Construction Specifications break down their inspection requirements into three sections: visual inspection, installation deflection, and compaction control. The next section will try to summarize those inspection topics to give a comprehensive view of AASHTO's recommendations.

2.11.1.1 Visual Inspection

AASHTO Section 30 recommends visual inspections both during and after installation. During the installation, bedding and backfill materials as well as their placement and compaction should be checked in order to ensure that the specifications are being met. Inspections during the installation are mainly to prevent improper practice and poor workmanship (AASHTO LRFD 2010).

It is required that final internal inspections must be conducted no sooner than 30 days after the completion of the installation and final fill. These internal inspections main purpose is to evaluate issues that may affect the long-term performance of the pipe. If pavement is going to be placed before 30 days, an extra inspection should be done at that time to ensure good construction practices were applied (AASHTO LRFD 2010).

2.11.1.2 Installation Deflection

The deflection must be checked a minimum of 30 days after the installation. AASHTO Section 30 recommends that a minimum of 10 percent of the pipe runs and length should be tested. If a mandrel is used to test deflection, it must be a nine arm (or greater) mandrel that must be pulled by hand through the entire length of the installed pipe. Direct measurements are allowed to be made on pipes larger than 24 inches. These measurements are required to be taken every 10 feet for the length of the pipe. For locations where the pipe deflection exceeds 7.5 percent of the inside diameter, replacement of the pipe is required. If the deflection is measured to be over 5 percent, then the contractor is allowed to evaluate the pipe with a Professional Engineer and submit a report to the Engineer for review and approval based on degree of deflection, structural integrity, environmental conditions, and the desired service life of the pipe (AASHTO LRFD 2010).

2.11.1.3 Compaction Control

Because the compaction of the soil is directly tied to the performance of the pipe system, it is imperative that compaction levels be checked to ensure specifications are met. AASHTO Section 30 requires that compaction in the field be based on compacted density and moisture content obtained from methods, such as the cone replacement (AASHTO T191) or the nuclear gage (AASHTO T238 and T239). For each new soil encountered, it is recommended to perform a density test on a sample to determine a value of maximum dry density and optimum moisture content. AASHTO Section 30 states that the best approach to compaction control is to conduct frequent tests and checks in the early stages to determine the critical parameters that will achieve the desired compaction such as type of compactor, number of passes, and moisture content. Using this process will cut down on the required number of tests needed to be run later as long as initial parameters are still being met (AASHTO LRFD 2010).

2.11.2 ASTM D 2321 Specifications

ASTM D 2321 does not go into as much depth as AASHTO Section 30 but it does give some detail about field monitoring and deflection testing. ASTM D 2321 requires field monitoring to ensure the pipe installation is in compliance with contract documents. Important items to monitor include trench depth, grade, water conditions, foundation, embedment and backfill materials, joints, density of materials in place, and safety. ASTM D 2321 along with AASHTO Section 30 requires waiting a minimum of 30 days after the final installation of the pipe before testing for deflections; however, ASTM D 2321 does not give specific maximum service level deflection requirements. Deflection measurements are allowed to be made directly with extension rulers or tape measures if lines are safe to permit entry, or they can be done with electronic deflectometers, calibrated television or video cameras, or a mandrel (ASTM D 2321).

2.11.3 State Departments of Transportation

This section will review several state specifications in regards to their final inspection requirements. The states were chosen to show the wide variability of thoroughness and complexity of final inspection procedures currently being used by state DOTs.

2.11.3.1 Florida DOT

The Florida Department of Transportation has one of most extensive final inspection procedures of the states reviewed. After the completion of the pipe installation for pipes 48 inches or less in diameter, the Engineer must be provided a video DVD and report using low barrel distortion video equipment with laser profile technology, non-contact video micrometer, and software that provides the following (FDOT 2010):

1. Actual recorded length and width measurements of all cracks within the pipe.
2. Actual recorded separation measurement of all pipe joints.
3. Pipe ovality report.
4. Deflection measurements and graphical diameter analysis report in terms of x and y axis.
5. Flat analysis report.
6. Representative diameter of pipe.
7. Pipe deformation measurements, leaks, debris, or other damage or defects.
8. Deviation in pipe line and grade, joint gaps, and joint misalignment.

When the video or laser profiling shows deflections that appear in excess of that allowed in the Specifications, the Engineer is allowed to require further testing of the pipe. Further testing could require using a mandrel (minimum 9 legs) that would be pulled through the pipe by hand. Pipe failing to meet FDOT's 5% deflection requirement are required to be removed, replaced, and retested at no cost to the Department (FDOT 2010).

2.11.3.2 Georgia DOT

The Georgia Department of Transportation's Standard Specifications have a much less extensive final inspection policy. The specification requires that a nine-point mandrel be used to test a minimum of 25% of the installed length of pipe. The mandrel must be 95% of the base inside diameter and a proving ring to verify the size should be provided to the Engineer. If mandrel testing reveals problems, the Engineer can require that 100% of the pipe installed be checked for deformation. Pipes with a deflection in excess of 5% are required to be removed and replaced at no cost to the department (GDOT 2010).

2.11.3.3 Missouri DOT

The Missouri Department of Transportation's Standard Specifications like the Georgia DOT's Specification requires that the inside diameter not be reduced by more than 5% and that it must be checked by a mandrel. In addition to this requirement, the MoDOT gives the following list of items that constitutes an improper installation:

a. If any horizontal or vertical alignment is in excess of 15 percent from plan alignment, will restrict flow, or will cause excessive ponding within the pipe.
b. Any section of pipe with deflections greater than 5 percent, based upon the units of measurement used in fabricating the pipe.
c. If settlement is greater than one inch at 5 percent or more joints.
d. The pipe shows evidence of being crushed or buckled at any location.
e. The pipe shows evidence of joint separation.

Any section of pipe found to be improperly installed, shall be repaired or replaced to the satisfaction of the engineer at the contractor's expense. The repaired or replaced pipe would then have to be re-inspected by the engineer (MoDOT 2004).

2.12 Pipe Joints

All pipe types are manufactured in limited lengths and are joined together to create a continuous pipeline. Plastic pipes because of their lightweight are able to be produced in longer lengths than pipes made from concrete, which allows for less joints. Joints in a pipe system must provide the following (Zhao et al. 1998):

1. Resistance to infiltration of ground water or soil,
2. Resistance to exfiltration,
3. Flexibility to accommodate lateral deflection or longitudinal movement without creating leakage problems,
4. Resistance to shear stresses between adjacent pipes,
5. Hydraulic continuity, and
6. Ease of installation.

Joints must be designed to have adequate shear strength, flexural strength, tensile strength, joint overlap, and soil or water tightness (Zhao et al. 1998).

The Oregon Department of Transportation sent a survey to other state departments of transportation inquiring about their requirements for watertight joints (Hunt 2000). There were 15 states that responded to the survey. There were 3 states (California, Louisiana, and South Carolina) that required watertight joints for all installations of culverts. One state sometimes requires watertight joints, while the remaining 11 states do not have a watertightness requirement (Hunt 2000). In another survey conducted by the South Carolina Department of Transportation (2007), it was found that Florida also requires watertight joints for all applications. Of the other states responding to the survey, there was a mixture of specifications with the most prevalent one being that soil-tight joints were specified for most installations (SCDOT 2007).

2.13 Pipe End Treatments

End treatments for plastic pipes are an important detail that must receive adequate consideration. By intuition it would seem that the ends of plastic pipe would need more protection from scour, mowers, and fire than concrete or metal pipe. End treatments for all types of culverts deteriorate more rapidly than interior culvert sections (McGrath and Beaver 2004). The deterioration can be attributed to weathering and abuse from vehicles such as mowing equipment. In a survey of highway culverts for the Utah Department of Transportation, it was found that plastic end treatments had an overall poor performance and typically experienced deformation due to lateral soil load (McGrath and Beaver 2004). The report went on to state that it was not a reflection of the structural performance of plastic pipe because they have been used with success when used in conjunction with metal and concrete end treatments (McGrath and Beaver 2004). The Florida DOT has the following requirement in their specifications that tends to agree: "For side drain and cross drain applications, mitered end sections as indicated in the Roadway and Traffic Design, Indexes 272 and 273 requires fabrication from another approved culvert material" (FDOT 2010).

2.14 Durability Characteristics

Durability can be defined broadly as the ability of a pipe to perform its function and withstand degradation (Wyant 2002). Gabriel (2009) defines durability as the property to resist erosion, material degradation, and subsequent loss of function due to environmental or other service conditions (Gabriel 2008). From a material standpoint, a pipe can be degraded in the form of cracking, tearing, spalling, abrading, or corroding (Wyant 2002). Some common durability concerns for plastic culvert pipe include corrosion, chemical resistance, abrasion, fire resistance, ultraviolet radiation, etc. The following sections will address each of these concerns by giving background information as well as how HDPE and PVC pipes perform in reference to the specific issues.

2.14.1 Corrosion

Gabriel (2008) states that chemical corrosion of buried pipelines and culverts may occur when in the presence of soils or waters containing acids, alkalis, dissolved salts, or organic induction wastes. These contaminants are carried by surface water, ground water, sanitary effluent, acid rain, marine environments, and mine drainage. HDPE pipes, unlike corrugated steel pipes, are not conductors and are not susceptible to galvanic corrosion associated with electrochemical attack. HDPE pipes are also not degraded by pH extremes, aggressive salts, or chemically induced corrosion (Gabriel 2008). According to Gabriel (2008), “HDPE pipes are effective for drainage of hostile effluents, such as acid rain, acidic mine wastes, aggressive landfill leachates, and effluents with high concentrations of road salts, fuels, and motor oils.”

PVC like HDPE is also a non-conductor so there are no galvanic or electrochemical effects in PVC pipe (Uni-Bell 2005). PVC also suffers no damage from attack of normal or corrosive soils and is not affected by sulfuric acid in the concentrations found in sewer systems (Uni-Bell 2005).

According to Gabriel and Moran (1998), state DOTs that have reported using plastic as an alternative material for drainage pipes have noted that these pipes are highly resistant to the various corrosive agents, sulfates, chlorides, and other aggressive salts found in soil and highway drainage effluents. The Federal Lands Highway Division of FHWA policy states that plastic alternatives may be specified without regard to resistivity and pH of the site with regard to corrosion (Gabriel and Moran 1998).

2.14.2 Chemical Resistance

A pipe system can be subject to a number of aggressive chemical exposures during its life-span (Uni-Bell 2005). Chemical reactions can be very complex and can be affected by many factors. Some of the factors that can affect the chemical resistance are (Uni-Bell 2005):

1. Temperature
2. Chemical (or mixture of chemicals) present
3. Concentration of chemicals
4. Duration of exposure
5. Frequency of exposure
6. PVC compound (or elastomeric compound) present
7. Geometry of piping system

PVC pipe exhibits resistance to a wide range of chemical reagents in temperatures up to 140^{o}F (Uni-Bell 2005). Through experience it has been seen that PVC pipes are resistant to chemicals that are generally found in water and sewer systems (Uni-Bell 2005).

A test study was performed by Sharff and DelloRusso (1994) investigating the effects of acid environments and constant deflection on PVC sewer pipes. The tests were performed by exposing pipe specimens to a sulphuric acid solution while fixed at 5%

deflection. The two-year study indicated that there was a minimal effect on the short-term stiffness of the pipes (Sharff and DelloRusso 1994).

2.14.3 Abrasion

According to Gabriel (2008), "Chemicals and abrasion are the most common durability concerns for drainage pipes, especially when the effluent flows at high velocities." Abrasion resistance can be defined as the ability of a material to withstand mechanical erosion, a process that tends to progressively remove material from its surface (Zhao et al. 1998). This mechanical erosion is caused by abrasives, such as stones or debris, wearing away at the surface of the pipe while passing through (Gabriel 2008). The extent that the pipe is eroded is dependent on the type of abrasive, the frequency that the material is in the pipe, the velocity of the flow, and the type of the pipe material (Gabriel 2008). Plastic pipe is highly resistant to abrasion (Zhao et al. 1998). Table 2-21 shows the ranking of the four most common types of pipe materials in terms of abrasion resistance based on wear characteristics of these pipe materials reported from laboratory tests (Zhao et al. 1998).

Table 2-21: Abrasion Resistance of Various Pipe Materials (Zhao et al. 1998)

Pipe Material	Abrasion Resistance Ranking
HDPE	4 (best)
PVC	3
Corrugated Steel	2
Concrete	1

Both PVC and HDPE rank above corrugated steel and concrete. HDPE is ranked as the most abrasion resistant with PVC following as the second most resistant to abrasion. According to Uni-Bell (2005), PVC pipes exhibit outstanding resistance to wear and abrasion and have proven to be more durable than metal, concrete, and clay pipe for the transport of abrasive slurries.

2.14.4 Fire Resistance

While the risk of fire in sewer pipe systems is limited, there is a potential for fire to occur in or around culverts (Hancor 2009). The resistance to fire for culvert pipes is an important issue especially for exposed ends. Both HDPE and PVC pipes will burn when there is adequate air flow such as in culverts (Zhao et al. 1998). Pipes can be protected from fire with the use of inflammable end treatments such as the use of Rip-rap, gravel, or concrete headwalls around exposed ends (Hancor 2009). The National Fire Protection Association gives polyethylene a rating of 1 (slow burning) on a scale from 0 to 4, where higher rating indicates more vulnerability (Gabriel 2008).

Zhao et al. (1998) believe that physical resistance to fire plays an important role in the performance and durability of sewer and culvert pipes. Table 2-22 shows the physical resistance of various pipe types to abrasion and fire.

Table 2-22: Physical Resistance of Various Pipe Types (Zhao et al. 1998)

Type of Resistance	Pipe			
	Concrete	Corrugated Steel	HDPE	PVC
Abrasion resistance	low	low	high, 2 and 3 times more resistant than PVC and steel pipe, respectively	high
Fire resistance	high	Most coatings used for corrosion protection are flammable	flammable	flammable with lower flammability rating than HDPE
Freeze-thaw resistance	(Note)			

Note: It is not certain whether concrete culvert pipe is subjected to freeze-thaw damage. Testing is required to clarify this.

According to Uni-Bell (2005), PVC pipes are difficult to ignite and will not continue to burn in the absence of an external ignition source. The temperature for spontaneous ignition of PVC is 850°F, which is much higher than most construction materials. PVC pipe is also referred to as a self-extinguishing material because the products of combustion combine with any available oxygen, thus starving the flame (Uni-Bell 2005).

HDPE deforms at temperatures above 120°C and begins to melt completely at 135°C (Zhao et al. 1998). The ignition temperature of HDPE is 660°F, which is lower than for PVC (Philbin and Vickery 1993). According to Zhao et al. (1998), a flammability test carried out by the North Carolina Department of Transportation in which one end of a corrugated HDPE culvert pipe was exposed to fire caused the pipe to be engulfed in flames within one minute. The pipe was then observed to fuel the fire and burn continuously throughout its entire length (Zhao et al. 1998).

In 1994, the Florida Department of Transportation (FDOT) conducted a study to determine the actual fire risk in typical HDPE pipe installations due to recent concerns expressed relative to the flammability of HDPE pipes (FDOT 1994). The study included field burn tests as well as standard laboratory burn tests on polyethylene coupons. The field tests also included burn tests on mitered end sections with concrete aprons. The results of the study indicated that HDPE pipe installed to present standards is not at significant risk of fire when exposed to fire such as that which would be encountered in a roadside grass fire (FDOT 1994). The report did however say that mitered polyethylene end sections are "… subject to fire damage and destruction when exposed to expected grass fire intensities" (FDOT 1994). The report recommends that the polyethylene pipe terminate in a concrete headwall, drainage structure, or non-plastic mitered end concrete apron (FDOT 1994).

In a South Dakota DOT report, Rumpca (1998) states that in November of 1991 there was a routine fuel reduction burn that destroyed 60 linear feet of 30 inch diameter polyethylene pipe culvert located in the Badlands National Park in South Dakota. The author states that even while highly publicized it was an isolated incident and to his knowledge there have been no other fires that have caused damage to HDPE pipe in South Dakota (Rumpca 1998).

In 1993, Philbin and Vickery wrote a report on the fire performance of HDPE pipe. The report cited past fires dealing with plastics and then explained a full-scale field test of a polyethylene pipe (Philbin and Vickery 1993). The field test consisted of a 30-inch diameter by 20-foot long pipe section installed as a drain culvert pipe with 24 inches of cover. The source of ignition was an ordinary, wood-stick-type kitchen match that was struck and held against the edge of the pipe. The pipe ignited within seconds and the flame began to extend upward. The pipe continued to burn until the pipe was destroyed (Philbin and Vickery 1993). Philbin and Vickery (1993) have recognized a fire hazard with HDPE and do not recommend that it be used in drain and sewer systems, because of the difficulty of fire control, confinement, and extinguishment.

In 1998, Gabriel and Moran conducted a survey of all 50 states with regards to durability issues of which 49 responded. The report stated that Colorado has experienced two cases of damage to HDPE pipes resulting from weed fires (Gabriel and Moran 1998). California reported that an uncontrolled Malibu fire destroyed unprotected HDPE (Gabriel and Moran 1998). The Florida DOT concluded that when exposed to grass fires, HDPE is not at significant risk. The Ohio DOT has used polyethylene pipe as cross drains with exposed ends under roadways since 1982 and has had no recorded incidents of fire. The state of Washington has had no record of fire related failures and believes that the risks associated with the flammability issue are essentially unjustified. New York reported that HDPE and PVC present no significant risk of damage by fire (Gabriel and Moran 1998). The report by Gabriel and Moran (1998) goes on to state that some states require noncombustible exposed ends for plastic pipe.

2.14.5 Other Durability Concerns

2.14.5.1 UV Radiation

Ultraviolet (UV) radiation can cause unprotected plastic materials to degrade over time (Hancor 2009). According to Walker (1981), UV degradation is nature's way of breaking down and reclaiming materials of organic composition. Plastic pipes that are exposed to ultraviolet (UV) radiation for extended periods of time, for example at the ends of culverts, can incur surface damage (Zhao et al. 1998). This degradation can alter the plastic's physical and mechanical properties (Gabriel 2008). These alterations can include color change, a slight increase in tensile strength and elastic modulus, and a decrease in impact strength (Zhao et al. 1998). To help prevent this problem, plastic pipes are created with UV stabilizers to inhibit the physical and chemical processes of the UV degradation. The most common UV stabilizer used in polyethylene pipe is carbon black, which is the most effective at stopping the UV-induced reactions (Gabriel 2008).

Polyethylene is required to have a minimum content of 2% carbon black as required by ASTM D3350 for weather resistant grades. Having the UV stabilizers allows for the pipe to only let the sun's radiation penetrate a thin layer into the pipe wall over the service life of the pipe. UV is only an issue while the pipe is exposed to sunlight. It becomes a non-issue following installation (Hancor 2009).

Walker (1981) reported a 2-year study into the effects of UV aging on mechanical properties of PVC pipe. It was found that after two years of exposure under some of the worst aging conditions in North America that the modulus of tensile elasticity and tensile strength of PVC pipe was unchanged (Walker 1981). This is evidence that PVC pipe's ability to resist external soil loads and traffic loads has not been adversely altered by two years of direct sunlight exposure. The impact strength of the pipes was however found to have decreased by 20.3% over the two years. Walker (1981) contends that even the lowest impact strengths reported during this evaluation should not concern PVC pipe consumers or impair PVC pipe's performance. The report concluded that the desirable mechanical properties of PVC pipe, formulated for buried use, were not adversely affected to a significant extent by two full years of outdoor weathering and direct exposure to sunlight (Walker 1981).

2.14.5.2 Animal Attack

One rare but possible durability issue with plastic pipe involves animal attack. Polyethylene does not attract or act as a nutrient for animals (Hancor 2009). Although the occurrence is rare, rodents can gnaw through plastic pipes if it acts as a barrier to food or water. There are no known microbes that attack polyethylene pipes (Hancor 2009).

2.14.5.3 Biological Attack

Biological attack is defined as the degradation caused by the action of living micro- or macro-organisms (Uni-Bell 2005). Organic materials such as fungi and bacteria would be classified as micro-organisms. Some macro-organisms that could affect pipe underground could include tree roots, insects, and rodents. Many studies have been done on the subject through the years and it has been found that PVC pipe does not deteriorate or break down under biological attack because PVC does not serve as a nutrient for organisms (Uni-Bell 2005). To further that point, investigations have failed to document a single case in which buried PVC pipe products have suffered degradation or deterioration due to biological attack (Uni-Bell 2005). As with PVC pipe, microbial attack is not an issue with polyethylene (Zhao et al. 1998).

2.14.5.4 Temperature Effects

All pipe materials expand and contract with changes in temperature. The amount of expansion and contraction that takes place depends on the material's coefficient of thermal expansion (AWWA 2002). Table 2-23 shows the coefficient of thermal expansion and the amount of expansion that would be seen in 100 ft of pipe with a 10 degree temperature change for several piping materials.

Table 2-23: Coefficients of Thermal Expansion (AWWA 2002)

Piping Material	Coefficient in./in./°F	Expansion in./100 ft/10°F
PVC	3.0×10^{-5}	0.36
HDPE	1.2×10^{-4}	1.44
ABS	5.5×10^{-5}	0.66
Asbestos cement	4.5×10^{-6}	0.05
Aluminum	1.3×10^{-5}	0.16
Cast iron	5.8×10^{-6}	0.07
Ductile iron	5.8×10^{-6}	0.07
Steel	6.5×10^{-6}	0.08
Clay	3.4×10^{-6}	0.04
Concrete	5.5×10^{-6}	0.07
Copper	9.8×10^{-6}	0.12

As can be seen from the Table 2-23, PVC and HDPE have relatively high coefficients of thermal expansion compared to the other traditional pipe materials of metal and concrete. This means that they are more adept to be affected by temperature change.

2.15 Pipe Testing Methods

There are several test methods that apply to PVC and HDPE pipe to ensure quality control of the pipe's material as well as the pipe's performance. The following section will go through some of the test methods and how they apply to the pipe's performance.

2.15.1 Parallel Plate Test

The parallel plate test is a common test used in the quality control process to determine pipe stiffness, stiffness factor, and the load at specific deflections. The parallel plate test is clearly defined in ASTM D 2412 – *Standard Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading*. The test consists of loading a short length of pipe between two rigid parallel flat plates at a controlled rate of compression. The standard test specimen is the smaller of three times the nominal pipe diameter or 12 inches. Both ends of the specimen should also be cut square and shall be free of jagged edges before loading (ASTM D 2412). The pipe stiffness, PS, for any given deflection is calculated as follows:

$$PS = \frac{F}{\Delta y} \qquad \text{(lbf/in./in.)} \qquad \text{Equation 2-16}$$

While McGrath et al. (2009) contends "pipe stiffness is a useful measure for QC/QA during manufacturing," Gabriel (2008) states that "the test is not representative of a typical installation and is not accurate for predicting field performance."

2.15.2 Curved Beam Test

The Curved Beam Test is a test proposed by Gabriel and Goddard (1999) as an alternative to the ASTM D 2412 Parallel Plate Test. The test is used to evaluate time-independent initial stiffness, at the time of load application, for HDPE and PVC drainage pipes using curved-beam sections. The reason for creating this test was to determine more realistic estimates of stiffness for buried thermoplastic pipes by creating a test procedure that is less expensive and that creates a closer approximation to actual field conditions (Gabriel and Goddard 1999). For this test the pipe specimen is cut into 90° arcs and loaded as shown in Figure 2-18.

Figure 2-18: End-loaded Curved Beam (Gabriel 2008)

The specimen can be loaded using the same laboratory equipment that is currently used for the Parallel Plate Test with a few inexpensive modifications. The curved beam test is considered to more closely approximate field conditions of buried pipe because for the same load, the magnitude of wall bending moment at spring line is less in the curved beam than in the full ring. This means that the response of the curved beam is made up of a greater proportion wall compression and a lesser proportion of wall bending moment than the full ring, which is a better approximation for pipes buried in the field (Gabriel and Goddard 1999).

2.15.3 Uniaxial Tension Test

The standard test method for tensile properties of plastics is outlined in ASTM D 638 *Standard Test Method for Tensile Properties of Plastics*. The test method uses standard "dumbbell-shaped" test specimens that are tested under defined conditions of pretreatment, temperature, humidity, and testing machine speed. The primary use for this test method is to produce tensile property data that can be used for the control and specification of plastic materials. The data can also be useful for research and development. The test is run by placing a "dumbbell-shaped" specimen in the grips of the testing machine and then applying tension. The load and elongation are recorded in order to determine the tensile properties (ASTM D 638).

2.15.4 Uniaxial Compression Test

The uniaxial compression test is accomplished by following the guidelines of ASTM D 695 *Standard Test Method for Compressive Properties of Rigid Plastics*. This test determines the compressive properties of the thermoplastic which include modulus of elasticity, yield stress, deformation beyond yield point, and the compressive strength if the material fractures. The standard test specimen is in the form of a right cylinder or prism whose length is twice its width or diameter. The preferred specimen sizes are 12.7 mm by 12.7 mm by 25.4 mm for the prism and 12.7 mm in diameter by 25.4 mm for the cylinder. To complete the test, the specimen is placed between two parallel compression plates with the long axis of the specimen held perpendicular to the plates (ASTM D 695). According to McGrath et al. (2009), "the uniaxial tension and compression tests are useful for qualitative characterization and for research and development to model the mechanical behavior of thermoplastic material."

2.15.5 Stub Compression Test

The "stub compression" test typically referred to a test that assessed the local buckling capacity of cold-formed steel members. The test consists of a short segment of the member that is tested in pure compression, and the peak load is used to determine the local buckling capacity. In 2000, McGrath and Sagan modified the test for use in assessing local buckling of profile wall thermoplastic pipe. The shape they decided to use consists of a pipe section consisting of three corrugations. The test setup features the member fixed at one end and pinned at the other between two steel plates. This test setup allows for a direct way to evaluate the local buckling capacity of thermoplastic pipe under large thrust and small bending demands. The effect that soil would have on this is conservatively ignored (McGrath et al. 2009).

2.15.6 Melt Index

The melt index of thermoplastics is determined through ASTM D 1238 *Standard Test Method for Melt Flow Rates of Thermoplastics by Extrusion Plastometer*. The melt index value provides information regarding the melt flow behavior of the polymer, which is important to the extrusion process used in the manufacturing of pipes. The melt index value is empirically related to the molecular weight of the resin. For similar polymers with a similar molecular weight distribution, a high melt index value indicates a low molecular weight and a low melt index value indicates a high molecular weight (Hsuan and McGrath 2009). This test method is useful as a quality control test on thermoplastics (ASTM D 1238).

2.15.7 Density

The density of polyethylene can be determined by measuring the proportion of crystals within its mass (Gabriel 2008). The density of thermoplastics can be determined by following the specifications found in ASTM D 1505 *Standard Test Method for Density of Plastics by the Density-Gradient Technique*. The density of a solid is an easily measured property that can often be useful as a means of following physical changes of a sample that can be an indication of uniformity among samples, and a means of identification (ASTM D 1505). The test method used in ASTM D 1505 is based on observing the level which a test specimen sinks in a liquid column exhibiting a density gradient, in comparison with a standard of known density. In general, the density of a polymer correlates directly to the percentage of crystallinity, for instance, a high-density would reflect a high percentage of crystallinity (Hsuan and McGrath 2009).

2.15.8 Flexural Modulus

The flexural modulus is a material's stiffness that is predictive of a structural element's resistance to bending under the application of loads (Gabriel 2008). The flexural modulus can be determined from the test method outlined in ASTM D 790 *Standard Test Methods for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials*. The test method consists of a bar of rectangular cross section that rests on two supports and is loaded by means of a loading nose midway between the supports. The specimen is then deflected until rupture occurs in the outer surface of the test specimen or until a maximum strain of 5% is reached. The flexural properties determined by this test method are especially useful for quality control and specification purposes (ASTM D 790).

2.15.9 Environmental Stress Crack Resistance

The environmental stress crack resistance is determined using the procedure outlined in ASTM D 1693 *Standard Test Method for Environmental Stress Cracking of Ethylene Plastics*. Environmental stress cracks can be caused from plastics being in environments where they are present with soaps, wetting agents, oils, or detergents that may cause mechanical failure by cracking. This test method is used for routine inspection purposes by subjecting several specimens to test conditions for a specified amount of time while noting the number that fail. Environmental stress cracking is a property that is highly dependent upon the nature and level of the stresses applied. For this test procedure, high local multiaxial stresses are developed by the introduction of a controlled notch on one surface of the specimen. Environmental stress cracking has been found to occur most often under these conditions (ASTM D 1693). AASHTO M 294 Specifications allow a maximum of 50% failures of the specimens after a 24 hour testing period (Hsuan and McGrath 2009). Husan and McGrath (2009) stated in a report that there are three disadvantages associated with ESCR which are listed as follows:

- The failure time of an individual test specimen cannot be recorded.
- There is a large standard deviation value.
- The actual stress condition varies throughout the test and is not known, because of stress relaxation in the material.

2.15.10 Notched Constant Ligament Stress (NCLS)

The disadvantages of the Environmental Stress Crack Resistance (ESCR) test can be overcome by the new Notched Constant Ligament Stress test (Gabriel 2008). The procedure for this test method can be found in ASTM F 2136 *Standard Test Method for Notched, Constant Ligament-Stress (NCLS) Test to Determine Slow-Crack-Growth*

Resistance of HDPE Resins or HDPE Corrugated Pipe. The procedure begins by molding HDPE resin into a plaque. Dumbbell samples are then machined from the plaque and notched in the midsection. The samples are then placed in a bath at an elevated temperature that contains wetting agents. The samples are then subjected to a constant ligament stress until a brittle failure occurs from the slow crack growth (ASTM F 2136).

2.16 State DOT Surveys

Through the years there have been several surveys conducted by various agencies in order to obtain current information about the use of plastic pipes. This section will briefly summarize some relevant surveys that have been completed recently.

2.16.1 Texas Tech University DOT Survey 1998

In 1998, researchers from Texas Tech University created a survey that was sent to DOTs nationwide in order to document the current state of practice for the use of HDPE pipe. Thirty-two of the 50 states surveyed responded to the questionnaire. The experience level of each of the state DOTs with large diameter HDPE for subsurface drainage varies tremendously. Six states had used HDPE for less than five years; eighteen states had used HDPE from between five and ten years; and eight states had used HDPE for more than ten years. Of the states surveyed, the general consensus was that HDPE pipe provided good performance as long as precautions were taken during the installation to prevent the pipe from being disturbed by construction traffic (Jayawichrama et al. 2001).

The survey identified several issues when dealing with HDPE. The first issue dealt with when and where DOTs allowed HDPE to be installed. Of the states responding, 25 of the state DOTs allowed the use of HDPE for use as cross-drains under roadways. Several of those states had other restrictions that limited the use of HDPE as a cross-drain. Those restrictions varied but included not allowing their use under interstates and/or setting maximum allowable ADT limits on its use. Maximum allowable ADT limit restrictions were given by 7 states and ranged from 250 to 1700 (Jayawichrama et al. 2001).

Another issue identified by the report was with the type of backfill materials used because of the critical role it performs in the successful installation of the pipeline. The survey found that the backfill requirements of each state vary significantly. Of the states responding to the survey, seventeen allowed the use of native soil as a backfill material. Eighteen states required select backfill material such as the following:

- Sand or well graded granular material
- A-1, A-2, A-3 according to AASHTO classification,
- Granular material with 100% passing 1.5 inch sieve, <5% passing No. 200 sieve,
- Processed aggregate,
- Stone screenings,
- Granular backfill passing a 1 inch sieve,
- Crushed stone.

In addition to the above, fifteen states allowed the use of flowable backfill (Jayawichrama et al. 2001).

Minimum and maximum cover requirements were also found to vary from DOT to DOT. In regards to minimum cover, fourteen states specified a minimum cover between 0.75 ft and 1 ft. Fifteen states specified a minimum cover that ranged between 1 ft and 2 ft. Two of the states required a minimum cover that was greater than 2 ft. The maximum cover requirements of the states who responded ranged from 10 ft to 61 ft. The majority of the state DOT's maximum cover requirements ranged from between 10 ft and 20 ft (Jayawichrama et al. 2001).

Another issue covered by the survey dealt with the performance of HDPE pipe installations. The report stated that most of the state DOTs had a positive experience with HDPE pipe. Of the problems with HDPE pipe, maintenance of the line and grade of the pipe during the installation seemed to be the most common. The problem stemmed from trying to lay the pipe in the presence of water. Of the states responding to the survey, two stated it was a frequent problem while seven stated that it was an occasional problem. One state DOT indicated that they had a very bad experience with HDPE pipe. Their greatest concern was the excessive deflection (Jayawichrama et al. 2001).

2.16.2 Alabama DOT Survey 2003

In 2003 the Alabama Department of Transportation compiled a survey that inquired about other state DOTs usage of HDPE pipe. Of the states surveyed, 30 responded with feedback. The survey consisted of three questions asking when and what size HDPE pipe was allowed, if their experience using HDPE pipe particularly as cross drains had been favorable, and if they had any problems/failures with HDPE pipe, particularly diameters larger than 36 inches. Of the 30 states responding, most had some experience with plastic pipe but only approximately half of the states allowed the use of HDPE as cross-drains under roadways. Most states had favorable experiences with HDPE pipe and there were only a few problems/failures reported. Of the failures reported, the cause was often deemed to be caused by improper installation techniques (ALDOT 2003).

2.16.3 South Carolina DOT Survey 2007

In 2007, the South Carolina Department of Transportation compiled responses to a survey of other state DOTs that requested information pertaining to inspection procedures with regards to HDPE pipe. The questions posed asked what the agencies currently specify for deflection testing and what they specify to determine proper line and grade. There were 17 states that responded to the survey. The answers varied on how deflections were tested but consisted of the following: no method is specified--only end result, mandrel testing, video surveillance, laser deflectometer, or a combination of the above. Most of the states that responded on how proper line and grade was determined stated that traditional or laser survey equipment was generally used. The only other question posed in the survey dealt with pipe joints and has already been addressed in the pipe joint section (SCDOT 2007).

2.16.4 Ohio DOT Survey 2007

In 2007, the Ohio Department of Transportation conducted a survey of other state DOTs in regards to their use of HDPE pipe as cross-drains. Of the states surveyed, 21 states responded to a questionnaire that asked what was the maximum diameter of HDPE pipe allowed and if HDPE was approved for use as cross-drains under roadways. There were 7 states that approved diameters up to 60 inches, 6 states that approved diameters up to 48 inches, 7 states that approved diameters up to 36 inch, and 1 other state that only allowed up to 24 inch diameter pipe. There were 14 states that allowed HDPE for cross-drains (some had various restrictions), 6 states that did not allow HDPE for cross-drains, and one state that did not say what their limits of use were (Welker 2007). This survey showed how state DOTs have varying opinions on when HDPE should be used.

2.17 Field Studies

2.17.1 ORITE Field Study

The Ohio Research Institute for Transportation and the Environment (ORITE) at Ohio University conducted research on the time-dependent deflection of thermoplastic pipes under deep burial (40 feet). The field study was performed on both HDPE and PVC pipes that were buried with ODOT 304 crushed limestone or ODOT 310 river sand materials as backfill and had backfill compaction that would normally occur in the field. The objective of the study was to determine the deflections of the pipe in the vertical and horizontal directions as well as determining the circumferential shortening at the time of installation. The horizontal deflection recorded for the pipes ranged from 0.7 to 1.3 percent. The circumferential shortening was measured as being 0.1 percent for PVC pipes and 1.5 percent of HDPE pipes. Circumferential shortening appeared to not correspond with the type of backfill material used. The vertical deflection of the pipes was measured to be 1.5 percent for the PVC pipes and 3 percent for the HDPE pipes. The study also determined by examining deflection-versus-time graphs for HDPE that a portion of the vertical deflection was mainly due to the circumferential shortening of the pipe. For PVC pipes it was found that a portion of the vertical diameter change corresponded to the change in horizontal diameter. The vertical and horizontal deflections and the circumferential shortening were found to have stabilized within 50 days from completion of the construction (Sargand et al. 2001).

2.17.2 Washington State DOT PVC Installation

In 1992, the Washington State Department of Transportation installed PVC pipe in order to evaluate its performance in field trial installation (Miner 2006). For the trial installation, A-2000 PVC pipe was used in lieu of concrete pipe for a project located in Benton County, Washington. Approximately 5,000 linear feet of pipe in sizes of 12, 15, and 18 inches in diameter were used. It was determined that by using PVC pipe instead of concrete pipe that approximately $20,000 was saved on this project. Miner (2006) reported that the installation procedures for the PVC pipe differed from other pipe because less manpower and no equipment was needed to lift and place the pipe. The pipes were backfilled with the sandy native soil to a 95% maximum density. The project engineer on the project reported that the A-2000 could withstand rougher handling because it was "more resistant to breaking, cracking, chipping, or denting then a concrete or metal pipe" (Miner 2006). It was also noted that because the pipe came in 20 foot sections that it was easier to maintain an accurate grade and it provided for better alignment at the joints since there was less tipping or misalignment. After installation, the pipes were inspected by video. While some difficulty was encountered with getting the cameral through the pipes, it was reported that the pipes were in good shape (Miner 2006).

2.17.3 Utah State Investigation

In 1994, Moser performed an investigation of the performance limits of 48 inch diameter N-12 HC polyethylene pipes subjected to external soil pressures. The pipes were tested in Utah State University's large soil cell where the vertical soil load was applied by using hydraulic cylinders (Moser 1994). The observed parameters were ring deflection, visual evidence of distress, and the structural performance limits. The variables tested were soil type, soil density, and the vertical soil load simulating the height of cover. The soil type used was silty sand which is designated as a Class III soil by ASTM D 2321. This soil type was chosen to represent a worst case test by using a lower quality soil. The compaction used for the test consisted of a relatively poor installation (75 percent Standard Proctor), a good installation (85 percent Standard Proctor), and an excellent installation (96.5 percent Standard Proctor). Hinge lines (creases) formed in the pipes at 34, 60, and 180 feet of cover for Proctor densities of 75 percent, 85 percent, and 96.5 percent, respectively. The deflections at which these performance limits occurred were 13 percent, 12 percent, and 5.5 percent for Proctor densities of 75 percent, 85 percent, and 96.5 percent, respectively. It can easily be seen that if deflection is controlled to 5 percent than this performance limit will not occur (Moser 1994). Moser (1994) notes that the high loads can be applied to the pipes without distressing the pipe ring. It is obvious that pipes deflect more in loose soil than in dense soil because loose soil compresses more. It is concluded that pipes should be backfilled with granular soils and should be carefully compacted if the pipe is buried under high soil cover, or under heavy surface loads. Granular pipe zone backfill materials at moderate to high densities assure that the pipes will perform well even at high earth covers (Moser 1994).

2.17.4 NCHRP Report 429

In 1999, Hsuan and McGrath wrote a report through the National Cooperative Highway Research Program entitled HDPE Pipe: Recommended Material Specifications and Design Requirements. Included in this report was a field investigation of 29 HDPE pipe installations. The pipes investigated in the field ranged in size from 12 to 48 inches in diameter and had been installed from between 1 and 16 years. The fill heights of the pipes ranged from 0.3 meters to 30 meters. The maximum horizontal and vertical deflections as a percentage of diameter were found to range from 0.4 to 13.9% and 2.1 to 25% respectively. Circumferential cracks were found to be the most dominant type of cracking and were attributed to the presence of longitudinal tensile stresses. The researchers in this project were able to make some direct correlations between installation conditions and observed behavior. One of correlations was that deflection results from the lack of control of construction procedures and the use of poor backfill materials. Another correlation was that the erosion at the outlet ends of some of the culverts due to unprotected headwalls often resulted in significant loss of material and longitudinal bending and cracking of the corrugated HDPE pipe (Hsuan and McGrath 1999).

2.17.5 Field Performance in South Carolina

In a study conducted by Gassman et al. (2005), the effects of installation procedures on the performance of existing HDPE pipe was investigated. There were 45 HDPE pipes that were inspected in South Carolina, and they were selected based on geographical location, pipe diameter, use, and age. The internal and external conditions of each pipe were evaluated with respect to AASHTO and ASTM specifications. The pipes were inspected with a video camera that revealed circumferential cracks in 18% of the pipes, localized bulges in 20% of the pipes, and tears and/or punctures in 7% of the pipes. Deflections were found to be greater than 5% in 20% of the pipes. Most of the damage that was observed could be considered to be minor because the pipelines were still relatively round and were still performing near the original installation purpose. The causes of the distresses seen were deemed to be caused by a combination of installation problems including poor preparation of bedding, inappropriate backfill materials, and inadequate soil cover. The study showed that the quality of backfill material plays a critical role in the performance of the pipe. For instance, pipes backfilled with Class IV soils exhibited more excessive deflections and circumferential cracks than those backfilled with Class II or III soils. Forty-four percent of the inspected pipes had less than the specified amount of minimum cover. The punctures and localized bulges that were seen indicated the presence of rocks in the backfill soil or careless use of mechanical soil compactors. The main conclusion of the report is that it is highly important to have an appropriate installation procedure in order for HDPE culvert pipes to perform as expected (Gassman et al. 2005).

2.17.6 Condition Assessment of HDPE in 6 States

In 2002, a condition investigation of HDPE pipes in 6 states was conducted by Nelson and Krauss for the American Concrete Pipe Association. The report included the assessment of 39 HDPE pipe installations ranging from 28 to 60 inches in diameter. The installations ranged in age from 3 to 11 years, with most being from 3 to 6 years old. The amount of cover over the pipes ranged from 1.5 to 20 feet. The inspection protocol for each installation consisted of four tasks: diameter measurements, still and video photography, alignment measurements, and distress documentation. Seventy-four percent of the pipes had joint separations greater than 1 inch, 69% had deflections greater than 5%, 62% had buckling, cracking, or bulging, and 41% had noticeable misalignment. The determination of the causes of the deflections and distress was not investigated (Nelson and Krauss 2002).

2.17.7 Performance Evaluation of Ohio and Kentucky DOT Projects

In 2005, field evaluations were conducted by the KY Transportation Center and Pipe Drainage Consultants on Ohio and Kentucky DOT construction projects. The project was partially sponsored by the American Concrete Pipe Association. The evaluations were conducted in order to evaluate the long-term performance of previous HDPE pipe installations. The pipes were inspected using sophisticated equipment that consisted of remote video rovers and pipeline profiling laser ring technology. Seven pipe installations from 3 to 15 years old were inspected in Kentucky in 2005. The following are some excerpts from the Kentucky evaluations (PDC 2005):

"The average maximum recorded corrugation was 0.5 inches."

"Radial cracking is documented in approximately 20% of the pipe sections."

"Sagging and ponding are observed in 26% of the pipe sections."

"The majority of the pipes investigated on this project would not pass a 5% deflection test and most of the sites have pipe sections that would not pass a 10% deflection test."

Thirteen pipe installations from 6 to 13 years old were inspected in Ohio in 2005. The following are excerpts from the Ohio evaluations (PDC 2005):

"The maximum recorded corrugation depth is 0.56 inches, with typical averages of approximately 0.39 inches."

"Cross drains inspected for this project show that cracking has increased by four to seven times since 2001."

"The majority of the pipes investigated on this project would not pass a 5% deflection test and most of the sites have pipe sections that would not pass a 10% deflection test."

The construction procedures, backfill materials, and the compaction methods used on these pipe installations were unknown. The recommendations made were generally focused at construction techniques and quality of backfill. It was also recommended that a quality control/quality assurance inspection program be established and that HDPE pipe installations be further monitored (PDC 2005).

2.17.8 Performance of HDPE Under High Fill

Hashash and Selig (1990) studied the performance of high-density polyethylene pipe under high fill in Pittsburg, Pennsylvania. The study consisted of 24 inch diameter pipes buried under 100 feet of cover with inspections of the pipe occurring 722 days after the installation. By comparing analytical stresses to actual measured stresses it was concluded that there was a local arching effect of the structural backfill directly around the pipe. This arching occurs because of the differences in stiffness properties of the structure and soil around it. The arching factor determined in this case was 0.77, which means that the pipe was only carrying approximately 23% of the prism load (Hashash and Selig 1990).

2.17.9 Pennsylvania Deep Burial Project

In 2007, an Ohio University research team visited the same 24 inch diameter HDPE pipe located under a 100 foot high embankment in Pittsburgh, Pennsylvania. The pipe had been installed in 1987 as part of a research study. There had been several inspections prior to this 20 year inspection and all of them were reported to have been good. Corrugation growth or wrinkling of the liner in the Type S pipe was reported but not quantified or considered problematic. Because the pipeline had been inspected on numerous occasions it provided an interesting look at the development of pipe deflection with time. Figure 2-19 shows the pipe's deflection as a function of time.

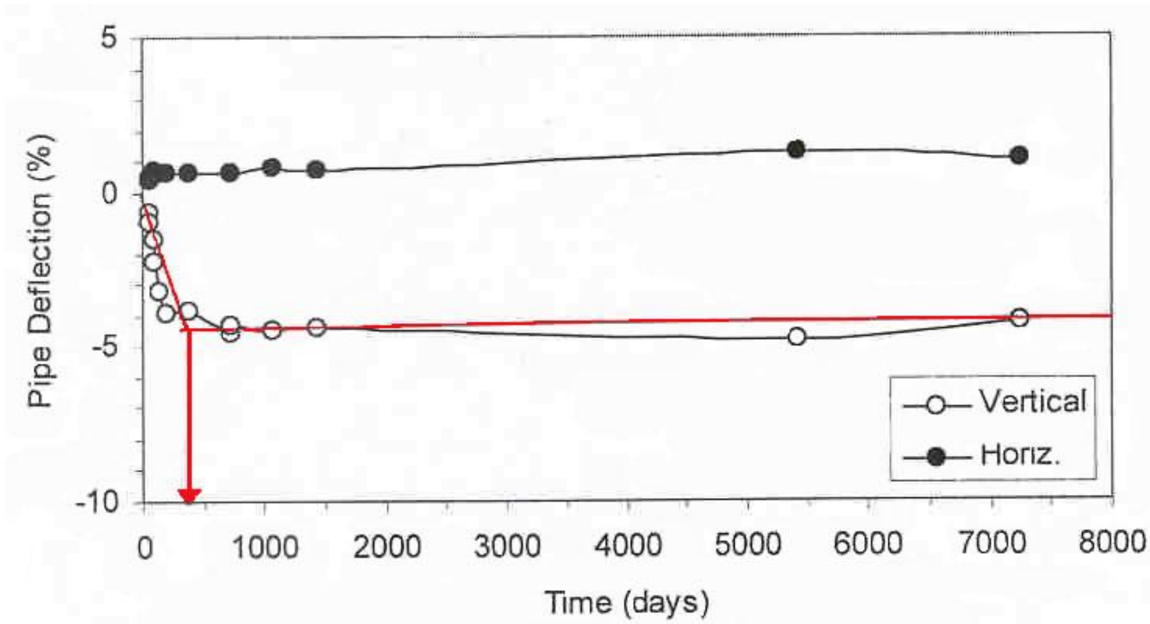


Figure 2-19: Pipe Deflection vs. Time (Sargand et al. 2007)

As can be seen in Figure 2-19, the vertical pipe deflection has been stable at about 4.5% for some years. Also of note is that most of the deflection occurred within the first year. In addition to the field inspection, the research team also conducted basic material tests in the laboratory and found that no noticeable changes took place in the basic engineering properties of the HDPE pipe material over the 20-year period (Sargand et al. 2007).

2.17.10 Performance of HDPE Pipelines in Texas

In 2007, a study was completed by Abolimaali and Motahari investigating the structural performance of 22 HDPE pipelines throughout Texas. The pipes inspected ranged from 36 to 60 inches in diameter, and the pipes had been in service for 6 or 7 years. Twelve of the pipelines were installed during a study by Texas Tech University for the Texas DOT. These installations would have been considered to be “controlled/research conditions.” The remaining ten installations could be considered “normal installations.” The performance of these two types of installations varied greatly which reinforces the observation that the performance of plastic pipe is highly dependent on the quality of the installation (Abolimaali and Motahari 2007). Table 2-24 shows some of the observed distresses found during the inspections, and it is broken down into the types of installation.

Table 2-24: Summary of Observed Pipe Distress (Adapted from Abolimaali and Motahari 2007)

Distress Type	Installation Conditions	
	Controlled	Normal
Cracking	0 of 12	7 of 10
Inverse Curvature	0 of 12	6 of 10
Major Joint Disp.	1 of 12	5 of 10
Buckling	0 of 12	4 of 10
Corrugation Growth	12 of 12	10 of 10
Deflection > 5%	0 of 8*	6 of 10

It is easily seen that the controlled installations performed exponentially better than the pipes installed under normal conditions. One of the conclusions made by the report was that the “structural health and integrity of the installed HDPE pipelines tested are generally below structurally acceptable levels of serviceability” (Abolimaali and Motahari 2007). In addition, while none of the pipelines was reported to be nonfunctional, there were several photographs that indicated that some of pipelines could soon be nonfunctional.

2.18 QC/QA Procedures

While installation procedures are probably the most important attribute of pipe performance, the pipe must also conform to the material specifications set forth by AASHTO M 294, AASHTO M 304, as well as other governing specifications. To ensure that the pipes being manufactured conform to the specifications, adequate quality control and quality assurance programs need to be developed and enforced. According to Gabriel (2008), the first essential and necessary condition for compliance with the specified requirements of the finished product is the control of the quality of the raw material being used. Gabriel (2008) contends that the second essential and necessary condition for compliance is the control of the quality of the pipe manufacturing process. According to McGrath (2008), a proper QC/QA plan should provide assurance to the end user that the pipe product purchased meets the product standard. There are many quality control and quality assurance plans that have been developed, and the review of them below will explain their use and necessity.

2.18.1 PPI's QC/QA Program

The Corrugated Polyethylene Pipe Quality Control/Quality Assurance Program developed by the Plastics Pipe Institute (PPI) gives the pipe producers the responsibility of controlling the quality of their product and the ability to use their quality control information to receive certifications from specifying agencies. The producers perform their own quality control sampling, testing, and record keeping. The specifying agencies then perform quality assurance by sampling and testing to confirm the performance of the producer's quality plan (Gabriel 2008).

The PPI's QC/QA program has three basic requirements. First, the producer should have a specific quality control plan that is site specific and describes in detail the methods the producer plans to use to insure the products meet the specifications. Next, the program requires that all tests be conducted in approved laboratories that have been qualified to perform the required tests. Last, the program requires that each plant should have a quality control technician that tests all samples and has the overall responsibility for implementing the Quality Control Program (Gabriel 2008).

The final component of PPI's QC/QA program consists of the specifying agency reviewing the compliance of the producers with the program. The specifying agency inspects all laboratories and sampling areas, reviews the qualifications of technicians involved with sampling and testing, evaluates raw materials and product quality, verifies facility compliance, and conducts scheduled and random inspections. In regards to the actual product, the specifying agent has the power to evaluate the material before production, after production, and while the product is still in distribution yards. If the specifying agency finds any test failures, they will immediately notify the producer. The producer will then investigate and follow the appropriate steps to rectify the situation (Gabriel 2008).

2.18.2 NTPEP Program

The National Transportation Product Evaluation Program (NTPEP) is a part of AASHTO that was established in 1994 as a Technical Services Program. The program was designed to evaluate materials, products, and devices of common interest for use in highway and bridge construction. One of the primary goals of this program is to provide cost-effective evaluations for state DOTs by eliminating duplication of testing and auditing by the states and the duplication of effort by manufacturers to have their products evaluated. A technical committee for HDPE has created a project work plan and provides oversight and guidance throughout the evaluation process (AASHTO 2010). The current HDPE thermoplastic pipe technical committee consists of 10 different state DOT employees and one member from industry. The project work plan for thermoplastic pipe is a program that establishes a list of manufacturing plants and pipe products that conform to the specifications set forth by AASHTO and ASTM. The program is voluntary and manufacturers are required to pay to have their products tested and their manufacturing process certified. The program consists of an audit of manufacturing plant's quality management system, initial and annual NTPEP audits, and a NTPEP website that lists information about the pipe products by manufacturers that have been found to conform to the requirements of the relevant material specifications. After initial compliance has been verified, the program requires annual NTPEP auditing and testing to ensure that the manufacturer's plants remain compliant. Annual plant audits are thorough and require documentation review, production line inspection, sampling and testing, yard inspection, quality control testing evaluation, a visit from a NTPEP Audit Team, as well as random surveillance visits and testing (AASHTO 2009).

The NTPEP program requires extensive testing by the manufacturer on their product in order to ensure the specifications are satisfied. The required inspections, tests, and measurements along with their required frequencies are shown in Table 2-25 (AASHTO 2009).

Table 2-25: Required Inspections, Tests, and Measurements for NTPEP program (AASHTO 2009)

Inspection	Frequency
• Unit Weight	• continuous, recorded once per shift
• Marking (per AASHTO M252 and M294)	• one per shift
Measurements and Tests	**Frequency**
• Unit Weight	• two per work shift
• Wall Thickness	• two per work shift
• Carbon Black Content (AASHTO M252 or M294 and ASTM D3350)	• one per day
• Inside Diameter	• one per shift
• Pipe Length	• one per shift
• Perforation Locations and Dimensions (Type "CP" and "SP")	• one per shift
• Water Inlet Area (Type "CP" and "SP")	• one per shift
• Pipe Stiffness	• two per week
• Pipe Flattening	• two per week
• Elongation (M252 Only)	• one per year
• Low Temperature Flexibility (M252 Only)	• one per year
• Brittleness	• two per week
• Joint Integrity	• integral bell/spigot, quarterly welded bell/spigot one per week
• Environmental Stress Cracking	• one per year

In addition to the tests made on the pipe, the polyethylene resin must be tested for Density (ASTM D1505 or ASTM D792), Melt Index (ASTM D1238), and Notched Constant Ligament-Stress (ASTM F 2136 & AASHTO M 294) (AASHTO 2009). According to McGrath, “The program has been very successful and provides an independent check on manufacturing quality” (McGrath et al. 2009).

2.18.3 Uni-Bell PVC QC/QA Plan

Uni-Bell PVC Pipe Association is a non-profit technical, educational, and research-oriented organization that was created in 1971 to properly service the future design and technical information needs associated with such a large scale and growing industry (Uni-Bell 2005). Uni-Bell provides a detailed QC/QA plan similar to PPI's plan already described. One of the major differences between the plans is that Uni-Bell's specifications add some QC/QA issues that are necessary for PVC pipe that are not addressed by PPI (McGrath et al. 2009). According to McGrath et al. (2009), these tests include:

- Joint-integrity testing (per ASTM D3212),
- Impact resistance (per ASTM D2444 – similar to "brittleness" ASTM D2444 for HDPE),
- Air test (no air leak at 3.5 psig),
- Gasket material QC/QA procedures,
- Extrusion quality (per ASTM F1057), and
- Specific guidance on testing of helically wound pipe.

The other tests shown in the specification are similar to those outlined in PPI's program. One main difference to note is that even if the test procedures are the same for HDPE and PVC, the requirements may be very different for each material (McGrath et al. 2009).

2.19 Executive Summary

This literature review provides a comprehensive review of relevant materials relating to both high-density polyethylene (HDPE) and polyvinyl chloride (PVC) for use as a drainage material under highways. While it is impossible to give a complete summary, several key topics that have been included in this literature review will briefly be restated.

One important issue dealing with HDPE and PVC pipes is that they are viscoelastic materials. Viscoelastic materials respond much differently to loads than linear, elastic materials (Gabriel and Goddard 1999). According to Moser (1994), many erroneously believe that the young's modulus, for plastics like PVC and HDPE, decrease with time, but he contends that is not the case. Moser (1994) states that his point can be proven by taking a sample of a pipe that has been under load for a long period of time and running a test for modulus on it. A test was run to find the modulus on a PVC pipe that had been in service for 15 years, and it was determined that the modulus was the same as when the pipe was newly manufactured (Moser 1994). Moser (1994) also contends that the creep modulus is an invented term that has almost no application in design. Sharff and DelloRusso (1994) contend that the short-term pipe stiffness is useful in characterizing the deformation response of buried pipe subjected to loads where the short-term response is of prime interest. For example, this is the case when pipes are subjected to quasi-instantaneous loadings like traffic live loads (Sharff and DelloRusso 1994).

Another important topic is how plastic pipes which are flexible differ from rigid pipes ie. concrete pipes. Flexible pipe are defined as pipes that can deflect 2% without any structural distress and are able to carry their soil load through their flexibility (AWWA 2002). Rigid pipes on the other hand must support the earth load by the inherent strength of the pipe (Jeyapalan and Boldon 1986). Because flexible pipes are able to deflect, it induces a positive arching action that allows for some of the vertical load to be carried by the surrounding soil (Kang et al. 2009). Another difference between flexible and rigid pipes is their performance limits. The performance limits for flexible pipe include deflection, wall buckling, wall stress, and wall strain (Goddard 1994).

Hydraulics, while not a structural concern, also plays an important role in the final design of a culvert. One factor that is based on pipe material is the Manning's coefficient. Research has shown that the Manning's coefficient for PVC and HDPE pipe ranges from between 0.010 to 0.015 (Bennett 2008). The Uni-Bell PVC Pipe Association (2005) recommends that the Manning's "n" factor to be used for PVC should be 0.009 for the hydraulic design. Advanced Drainage Systems (ADS) recommends using a Manning's "n" factor of 0.012 for corrugated HDPE pipes with a smooth interior liner (ADS 2009).

The structural design of plastic pipe is another important topic. The concept of culvert design and installation requires extensive engineering knowledge in the following fields: hydraulics, soil mechanics, material science, and construction methods among others (Malmurugan 1999). Once loads have been calculated for a particular pipe installation, the pipe must be checked for wall thrust, deflection, buckling, bending stress, and bending strain.

The actual installation of plastic pipe is probably the single most important aspect when dealing with plastic pipe. The two specifications that are most widely used are AASHTO LRFD Bridge Construction Specification (Section 30) and ASTM D 2321--*Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications*. The most important aspect of the installation is using proper backfill materials and ensuring that they are compacted to the desired specifications.

The minimum and maximum cover requirements for plastic pipes are determined through design in order to ensure that the loads imposed on the pipes are within acceptable levels. Minimum cover requirements deal with live loads caused by moving vehicles and/or construction equipment crossing over the pipe. Most state DOTs specify minimum fill heights in the range of 1 to 3 feet (Ardani et al. 2006). Most state DOTs specify a cover that is 1 to 2 feet higher than normal for construction loads in order to account for heavy construction equipment. Those values usually range from approximately 3 to 4 feet (Ardani et al. 2006). Currently, there are many different opinions on what the maximum height of cover should be. There has been research done on the subject, but many state DOTs and specifying agencies still have different requirements. Maximum fill heights are somewhat varied from DOT to DOT, ranging from a few feet to over 50 feet of cover. The most widely used fill heights specified by state DOTs were 10 feet and 20 feet. In general, the larger diameter pipes have lower maximum fill heights. The maximum depth of cover is highly influenced by the types of backfill materials used and the degree in which they are compacted (Ardani et al. 2006).

Due to the care that must be taken when installing plastic pipe to create the soil-structure system, inspection must be performed during and after installation in order to insure proper performance. The current states of practice show that there are many different strategies being employed, and they vary in terms of time, effort, cost, as well as the amount and quality of information obtained. Because the installation plays such a crucial role in the performance of plastic pipe, it was observed throughout the research performed that careful inspection was important to ensure that the installation specifications were being met.

Durability is always a concern when dealing with construction materials. Some common durability concerns for plastic culvert pipe include corrosion, chemical resistance, abrasion, fire resistance, and ultraviolet radiation. According to Gabriel and Moran (1998), States that have reported using plastic resins as alternative materials for drainage pipes have noted that these pipes are highly resistant to the various corrosive agents, sulfates, chlorides, and other aggressive salts found in soil and highway drainage effluents. Plastic pipes are highly resistant to abrasion (Zhao et al. 1998). While the risk of fire in sewer pipe systems is limited, there is a potential for fire to occur in or around culverts (Hancor 2009). The resistance to fire for culvert pipes is an important issue especially for exposed ends. Both HDPE and PVC pipes will burn when there is adequate air flow such as in culverts (Zhao et al. 1998). Because of the risk associated with this, it is often recommended that plastic pipe be terminated into a concrete headwall, drainage structure, or non-plastic mitered end concrete apron (FDOT 1994). Plastic pipes that are exposed for a long time to ultraviolet (UV) radiation, for example at the ends of culverts, can incur surface damage (Zhao et al. 1998). To help prevent this problem, plastic pipes are created with UV stabilizers to inhibit the physical and chemical processes of the UV degradation (Gabriel 2008). Having the UV stabilizers allows for the pipe to only let the sun's radiation penetrate a thin layer into the pipe wall over the service life of the pipe. UV is only an issue while the pipe is exposed to sunlight. It becomes a non-issue following installation (Hancor 2009).

Through the years, there have been several research projects that have dealt with field installations of plastic pipe. Some have been under research conditions while others have been on previously installed plastic pipe installations. The results varied significantly from project to project. In projects where problems were found with the pipes, the causing factor seemed to always be attributed to a poor installation.

While installation procedures are probably the most important attribute of pipe performance, the pipe must also conform to the material specifications set forth by AASHTO M 294, AASHTO M 304, as well as other governing specifications. To ensure that the pipes being manufactured conform to the specifications, adequate quality control and quality assurance programs need to be developed and enforced. The National Transportation Product Evaluation Program (NTPEP) is a part of AASHTO that was established in 1994 as a Technical Services Program. The program was designed to evaluate materials, products, and devices of common interest for use in highway and bridge construction. One of the primary goals of this program is to provide cost-effective evaluations for state DOTs by eliminating duplication of testing and auditing by the states and the duplication of effort by manufacturers to have their products evaluated. This program is currently only available for HDPE pipe, but hopefully in the future will also evaluate PVC pipe.

With regards to current specifications of state DOTs around the country, a more up-to-date view could be obtained by creating a survey that could be sent to state DOTs for their feedback. While it was decided not to accomplish this survey during this project, a survey of relevant questions that would be valuable has been included in Appendix A. Since many states have been doing research and updating their specifications and standard practices recently, it is suggested that in the future a similar survey be conducted.

CHAPTER 3

FINITE ELEMENT FILL HEIGHT EVALUATION

This chapter provides a comprehensive review of the finite element analyses that were conducted to determine the fill height requirements for both high-density polyethylene (HDPE) and polyvinyl chloride (PVC) pipe when used as a cross-drain under roadways. This chapter will be broken into two parts with the first section providing details on the maximum cover study and the second section detailing the minimum cover study.

3.1 Maximum Cover Study

3.1.1 Introduction

Currently, most state highway agencies allow thermoplastic pipes as side drains, while some allow the use of thermoplastic pipes for under-roadway applications (cross-drains) (Ardani et al. 2006). Because its use has been limited, there have been few rigorous studies on the maximum cover requirements for thermoplastic pipes used in highway construction. The objective of this study was to evaluate the maximum fill heights for plastic pipe used in highway construction. The study was based upon finite element analyses (FEA) that incorporated nonlinear soil models and parameters, the time-dependent material properties of polyvinyl chloride (PVC) and high density polyethylene (HDPE), and the geometric nonlinear behavior of the soil-pipe system. The fill heights suggested as a result of this work may serve as the beginning to a unifying basis for specifying the burial depth limitations for the use of thermoplastic pipes in highway construction.

3.1.2 Soil Arching

Figures 3-1(a) and 3-1(b) illustrate typical soil arching above buried rigid and flexible pipes, respectively.

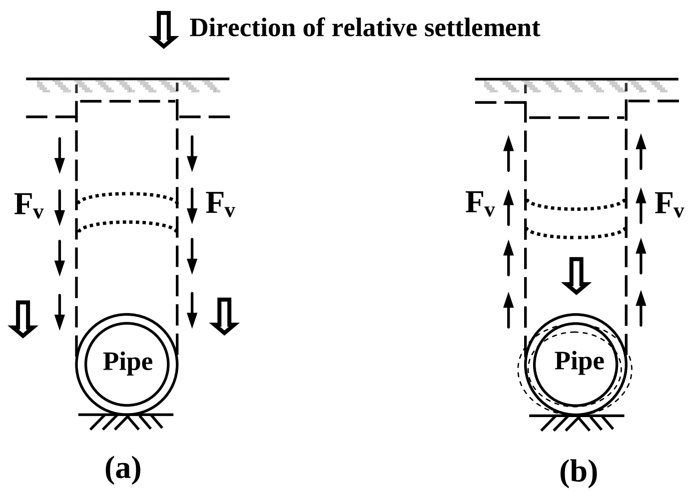


Figure 3-1: Mechanism of Soil Arching within Soil-pipe System (Kang et al. 2009)

Soil arching occurs when the stiffness of the installed structure differs from that of the surrounding soil. If the structure is stiffer than the soil and the relative downward deflection of the adjacent soil prism is thus greater than that of the central soil prism, the soil arches onto the structure as shown in Figure 3-1(a), thereby inducing a negative arching action. If, however, the structure is less stiff than the soil and the vertical deflection of the central soil prism is greater than that of the adjacent soil prism, the soil arches away from the structure as shown in Figure 3-1(b), inducing a positive arching action.

3.1.3 Soil Model and Parameters

The Duncan and Selig soil models are representative of the nonlinear soil behavior in most culvert installations (Mlynarski et al. 2008). They have also been included in CANDE-2007 (Mlynarski et al. 2008), which was developed by research sponsored by the American Association of State Highway and Transportation Officials (AASHTO) in cooperation with the Federal Highway Administration (FHWA). CANDE is a finite element program developed especially for the structural design, analysis, and evaluation of buried structures (Mlynarski et al. 2008). The present study also uses the Duncan and Selig soil models to simulate the nonlinear soil-structure interaction phenomena. The soil stiffness parameters, tangent modulus of elasticity, and bulk modulus are calculated using Equation 3-1 and 3-2, respectively (Duncan et al. 1970, Selig 1988):

$$E_t = \left[1 - \frac{R_f\,(1-\sin\phi)(\sigma_1 - \sigma_3)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]^2 KP_a\left(\frac{\sigma_3}{P_a}\right)^n$$

Equation 3-1

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

Equation 3-2

Where:

	σ_1	=	major principal stress
	σ_3	=	minor principal stress (confining pressure)
	E_t	=	tangent elastic modulus
	ϕ	=	angle of internal friction
	c	=	cohesion
	R_f	=	failure ratio
	K	=	elastic modulus constant

P_a	=	atmospheric pressure
n	=	elastic modulus exponent
B_i	=	initial bulk modulus
σ_m	=	mean stress
ε_u	=	ultimate volumetric strain

Values of the tangent modulus and Poisson's ratio were computed for each layer based on the assumption that vertical and horizontal soil stresses are principal stresses represented by Equations 3-3 and 3-4, respectively (Kim and Yoo 2005).

$$\sigma_1^{(i)} = \gamma_i \ H_i / 2 \ + \sum_{j=i+1}^{n} \gamma_j H_j$$

Equation 3-3

$$\sigma_3^{(i)} = K_0 \sigma_1^{(i)}$$

Equation 3-4

Where:

$\sigma_1^{(i)}$	=	maximum principal stress in the i^{th} layer of soil (numbering commences from the bottom of the backfill)
$\sigma_3^{(i)}$	=	minimum principal stress in the i^{th} layer of soil
H_i	=	depth of the i^{th} soil layer
γ_i	=	density of the i^{th} soil layer
K_0	=	coefficient of lateral earth pressure

These values were back substituted for the principal stresses in Equation 3-1 for the tangent modulus (E_t) and in Equation 3-2 for the bulk modulus (B) for each layer. Soil parameters, such as the internal friction angle and soil cohesion in Equation 3-1 were adopted for a variety of soil types using values provided in the Concrete Pipe Technology Handbook (ACPA 1994) and are shown in Table 3-1.

Table 3-1: Soil Parameters (ACPA 1994)

Soil Type	Std. T99 %	K	n	R_f	B_i/P_a	ε_u	c (psi)	φ (deg)	K_o
SW	95	950	0.60	0.70	74.8	0.02	0	48	1.3
	90	640	0.43	0.75	40.8	0.05	0	42	1.1
	80	320	0.35	0.83	6.1	0.11	0	36	0.8
	60	54	0.85	0.90	1.7	0.23	0	29	0.5
ML	95	440	0.40	0.95	48.3	0.06	4	34	1.2
	90	200	0.26	0.89	18.4	0.10	3.5	32	0.9
	85	110	0.25	0.85	9.5	0.14	3	30	0.8
	80	75	0.25	0.80	5.1	0.19	2.5	28	0.7
	60	16	0.95	0.55	1.3	0.43	0	23	0.5
CL	90	75	0.54	0.94	10.2	0.17	7	17	0.6
	80	35	0.66	0.87	3.5	0.25	5	19	0.4

3.1.4 Numerical Modeling

ABAQUS (2009) was used for the finite element modeling; a schematic of the finite element mesh is shown in Figure 3-2.

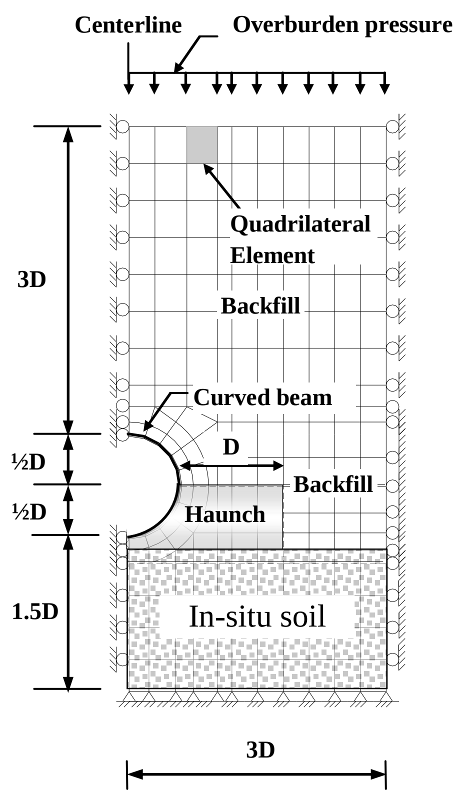


Figure 3-2: Schematic of Finite Element Model for Evaluating Maximum Fill Heights (D = Pipe Diameter)

Since the soil-pipe system is symmetric, only half of the system was modeled. The results from a series of exploratory trial FEA runs showed that it was not necessary for the lateral and top boundaries to extend beyond three times the pipe diameter horizontally from the center of the pipe and three times the pipe diameter vertically above the crown; no difference occurs in the analysis results between a model using additional soil elements and a model with an equivalent overburden pressure applied. Therefore, for deeper fills, an equivalent overburden pressure was used to represent the additional soil weight. The pipe walls were modeled using curved beam elements; plane strain elements were used for the soil. The time-dependent material properties of HDPE and PVC used in the analyses are summarized in Table 3-2.

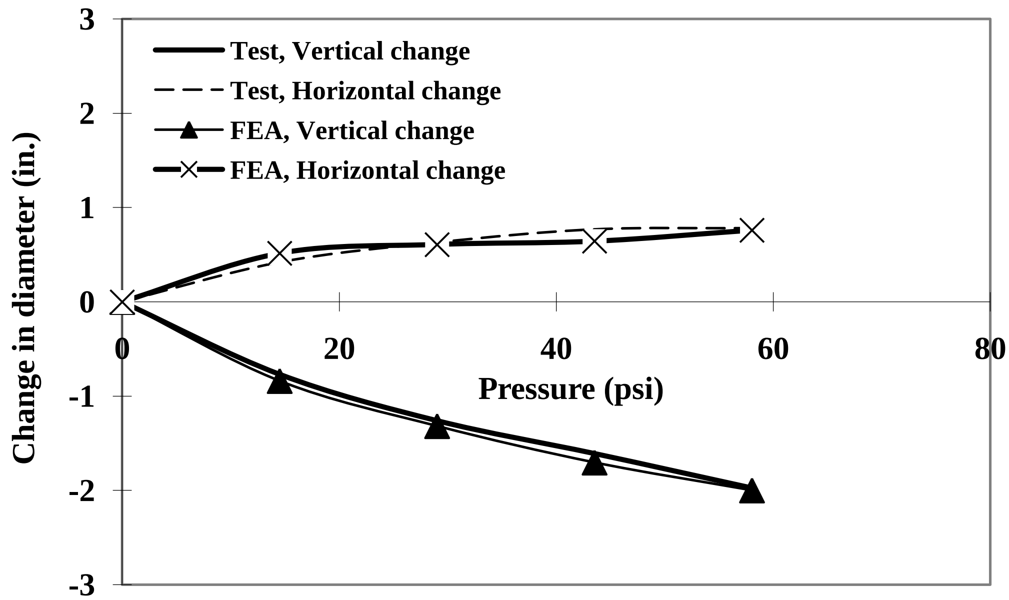
Table 3-2: Time-dependent Material Properties of Corrugated HDPE and PVC Pipes (PPI 2003, PP 2003, AASHTO 2007)

Type of Pipe	Initial			50-Year		
	E_{ini}	ν_{ini}	σ_{yi}	E_{50}	ν_{50}	σ_{y50}
	(psi)		(psi)	(psi)		(psi)
Corrugated HDPE pipe						
AASHTO M 294	110,000	0.35	3,000	22,000	0.45	900
PVC pipe						
AASHTO M 304	400,000	0.30	7,000	140,000	0.30	3,700

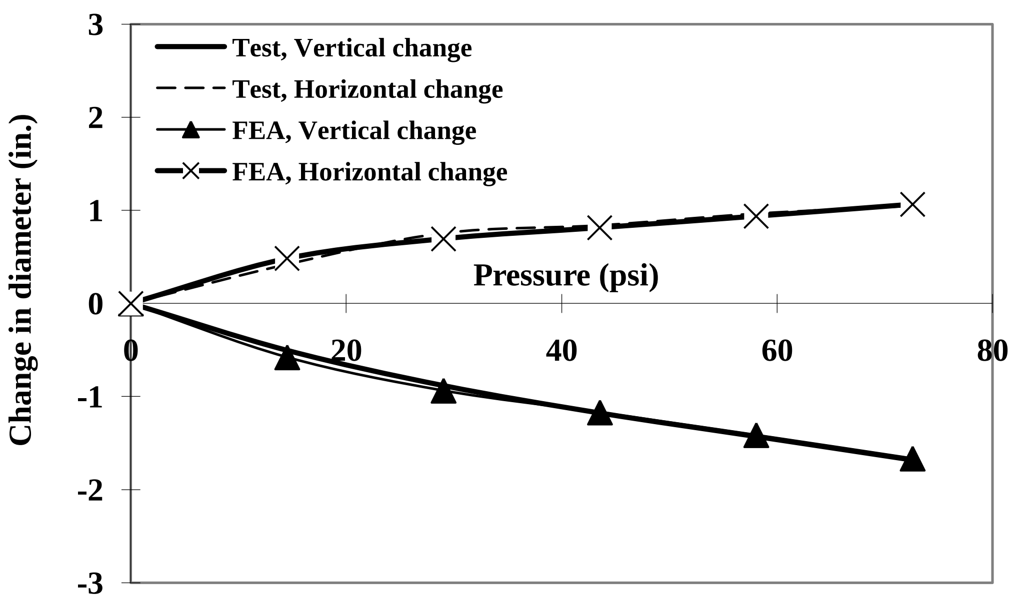
The unit weight (γ) of the pipe materials was taken to be 59.3 pcf. The properties of the soil were described by two stiffness parameters, namely the tangent modulus (E_t) and the bulk modulus (B), which are defined by Equations 3-1 and 3-2, respectively. The unit weight of soil was assumed to be 120 pcf.

3.1.5 Calibration and Validation

Laboratory test data reported in National Cooperative Highway Research Program (NCHRP) Report 631 (McGrath et al. 2009) was employed to calibrate and validate the finite element modeling methodology. The test cell used for these tests was a steel box with dimensions 6 feet by 6 feet in plan and 5.2 feet in height. Pipes of 24 inches in diameter were placed horizontally and centered between the sidewalls. Soil bedding below the pipe was 13-inches thick. The poorly graded sand that was used as the backfill material was simulated with ML85 parameters; ML60 parameters were used in the haunch. The material properties for these backfill materials can be found in Table 3-1. The unit weight of backfill materials was 100 pcf. The modulus of elasticity and Poisson’s ratio for the HDPE were taken as 65 ksi and 0.46, respectively, while the modulus of elasticity and Poisson’s ratio for the PVC were taken as 400 ksi and 0.3, respectively. All model parameters were based on those given in NCHRP Report 631 (McGrath et al. 2009). Figure 3-3 demonstrates that the FEA deflections are in good agreement with the measured deflections in the report for both the HDPE and PVC tests. The final calibrated finite element models were therefore deemed acceptable and used in the subsequent maximum fill height analyses.



(a)



(b)

Figure 3-3: Comparison of Pipe Deflections of NCHRP Test Cell (McGrath et al. 2009) and FEA: (a) HDPE Pipe and (b) PVC Pipe

3.1.6 Analysis Basis

Since deflections and wall stresses are critical performance parameters considered in the design of plastic pipes, these two parameters were evaluated for fill heights varying from approximately 20 feet to 200 feet. The maximum stresses were evaluated against the yield stresses of PVC and HDPE provided in Table 3-2. The maximum wall stresses were calculated by taking the axial force and moment from the finite element results and using the following equation.

$$\sigma = \frac{P}{A} + \frac{Mc}{I} \qquad \text{Equation 3-5}$$

Where:
- σ = maximum wall stress (psi),
- P = axial force (lbs),
- A = cross-sectional area (in.),
- M = moment (lb-in),
- c = distance from inside diameter to neutral axis (in.), and
- I = moment of inertia (in^4/in).

Deflection is quantified in terms of the percentage decrease or increase in pipe diameter (D). In pipe design, the vertical deflection is used as a benchmark and is limited by AASHTO LRFD (2007) to 5%. The response under both the short-term and long-term properties was investigated, and the long-term properties were found to control. The section properties used for the corrugated PVC and HDPE pipes conformed to Section 12 of AASHTO LRFD (2007) and can be seen in Tables 3-3 and 3-4.

Table 3-3: Section Properties for Profile Wall PVC Pipes (AASHTO LRFD 2007)

Nominal Size (in.)	Min. *ID* (in.)	Min. *OD* (in.)	Min. *A* (in.^{2}/ft.)	Min. *c* (in.)	Min. *I* (in.^{4}/in.)	
					Cell Class 12454C	Cell Class 12364C
12	11.7	13.6	1.20	0.15	0.004	0.003
24	23.4	26.0	1.95	0.23	0.016	0.015
36	35.3	39.5	2.60	0.31	0.035	0.031
48	47.3	52.0	3.16	0.37	0.061	0.056
60	59.3	64.0	3.90	0.45	0.080	0.080

Table 3-4: Section Properties for HDPE Corrugated Pipes (AASHTO LRFD 2007 and Gabriel 2008)

Nominal Size (in.)	Min. *ID* (in.)	Min. *OD* (in.)	Min. *A* (in.^{2}/ft.)	Min. *c* (in.)	Min. *I* (in.^{4}/in.)
12	11.8	14.7	1.5	0.35	0.024
24	23.6	28.7	3.1	0.65	0.116
36	35.5	35.5	4.5	0.9	0.222
48	47.5	55.0	5.15	1.15	0.543
60	59.5	65.0	6.46	1.37	1.0

The values found in Tables 3-3 and 3-4 for the 60-inch diameters were obtained by extrapolation for the PVC pipe and from Gabriel (2008) for the HDPE pipes. Section 30.5, "Installation," of AASHTO LRFD specifies the use of 90% as the minimum compaction requirement for HDPE or PVC pipe backfill. The analyses in this study included the results for SW90 (gravelly sand compacted to 90%) in addition to those of SW95 (gravelly sand compacted to 95%). Furthermore, the use of silty sand (ML90 and ML95) was also analyzed in order to evaluate the effects of a lesser quality backfill material.

3.1.7 Safety Factor

Section 3, Table 3.4.1-2 of AASHTO LRFD (2007) specifies a safety factor (*SF*) equal to 2 for wall areas in the service load design of thermoplastic pipes. Therefore, allowable stress, f_a, was calculated as follows:

$$f_a = \frac{f_u}{SF} \qquad \text{Equation 3-6}$$

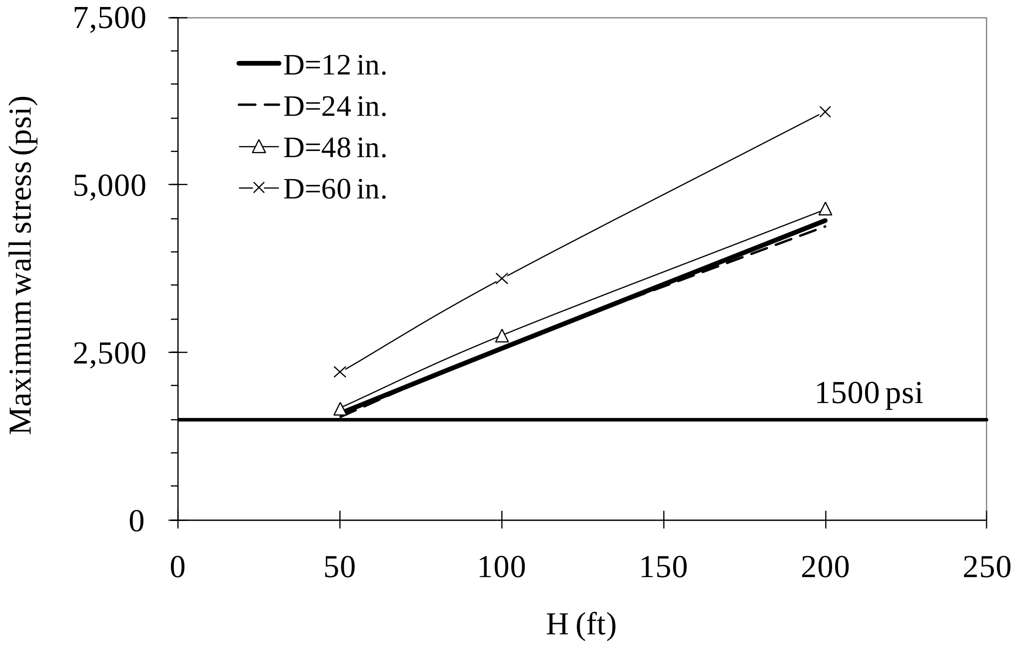
Where: f_a = allowable stress

f_u = specified tensile strength

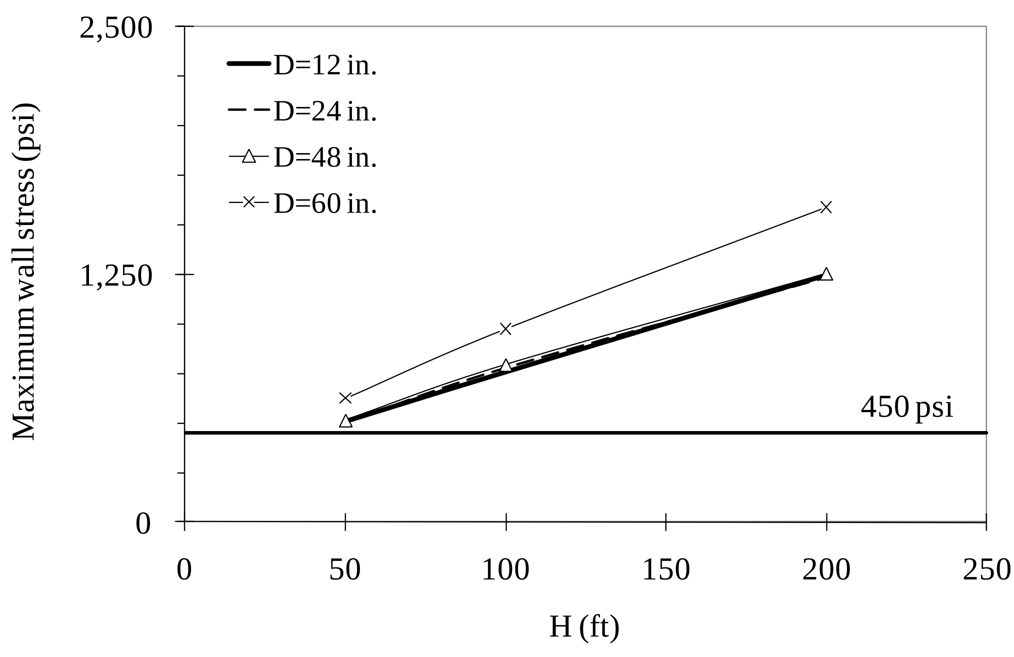
The deflection of the crown of flexible pipes becomes inverted and unable to resist additional load at a deflection of approximately 20% (Moser 2001). Therefore, the AASHTO deflection limit of 5% deflection inherently provides a *SF* of approximately 4.

3.1.8 Discussion of Results

The maximum fill heights that could be constructed without exceeding the deflection and stress limitations described above were evaluated. The maximum stresses were consistently higher in the larger diameter pipes than in the smaller diameter pipes (Figures 3-4 and 3-5). The limits for stress including the SF of 2 have been included in the figures. Let it be noted that these graphs are to show trends. Several hundred finite element runs were made in order to investigate all of the parameters in this study. Plots were not made for each case, and the maximum cover values shown in Table 3-5 were calculated based on the raw finite element data.

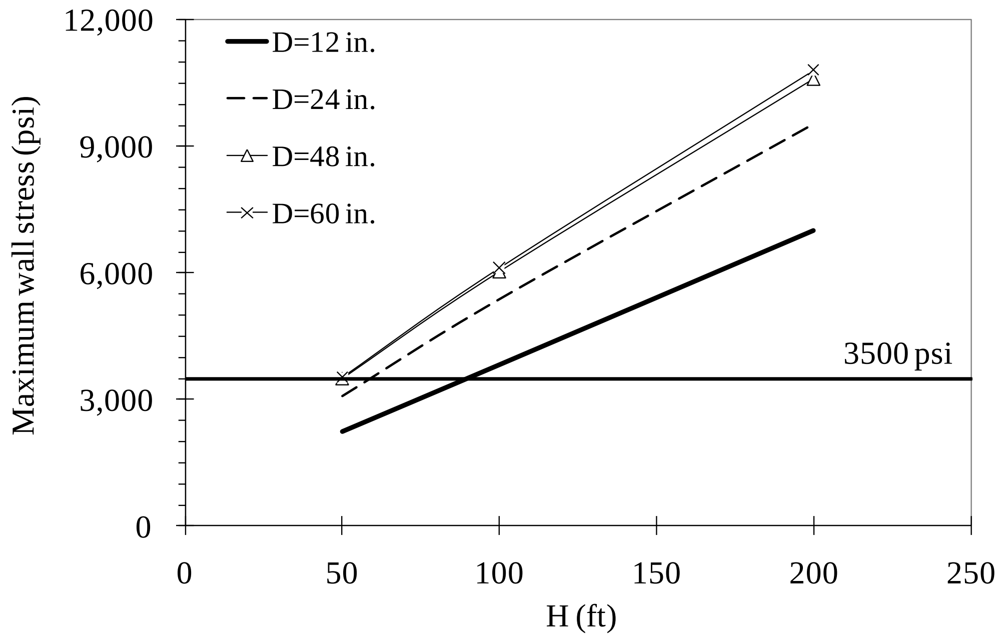


(a)

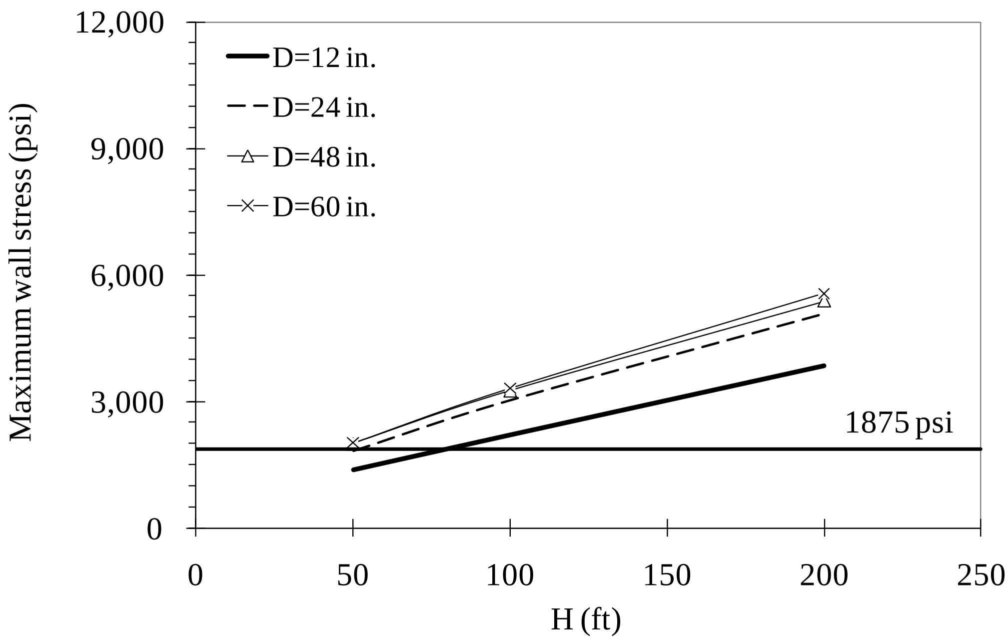


(b)

Figure 3-4: Effects of Pipe Diameters for Maximum Wall Stress of HDPE Pipes: (a) Short-term and (b) Long-term



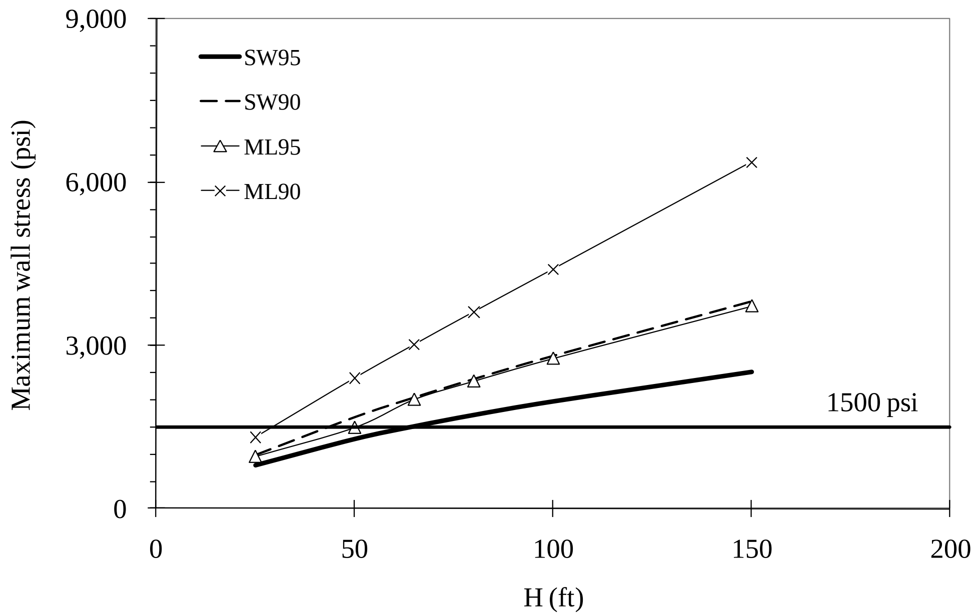
(a)



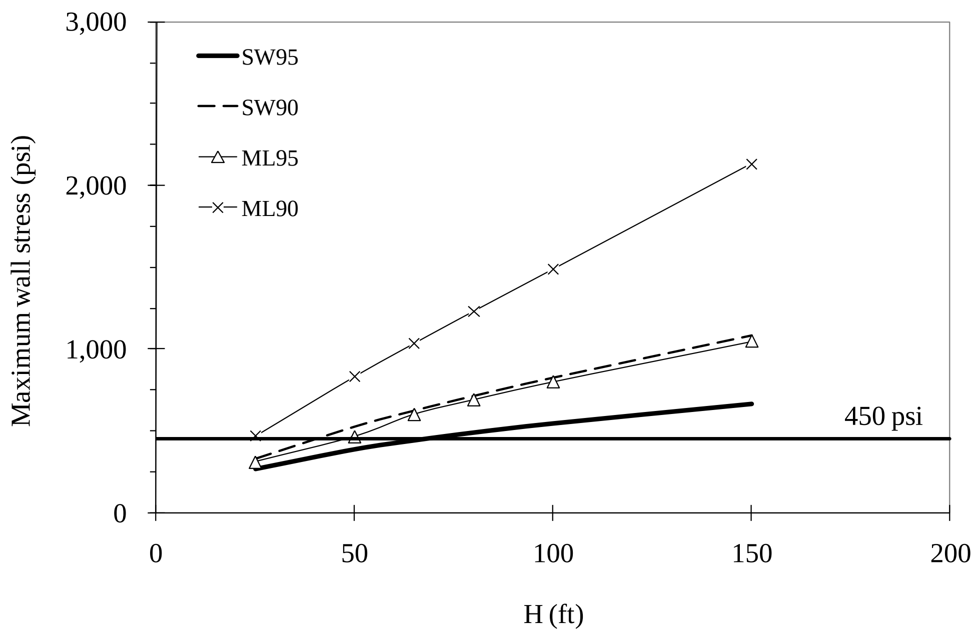
(b)

Figure 3-5: Effects of Pipe Diameters for Maximum Wall Stress of PVC Pipes: (a) Short-term and (b) Long-term

Furthermore, as shown in Figures 3-6 through 3-8, the strength limit using the long-term HDPE and PVC material properties governed the maximum fill heights.

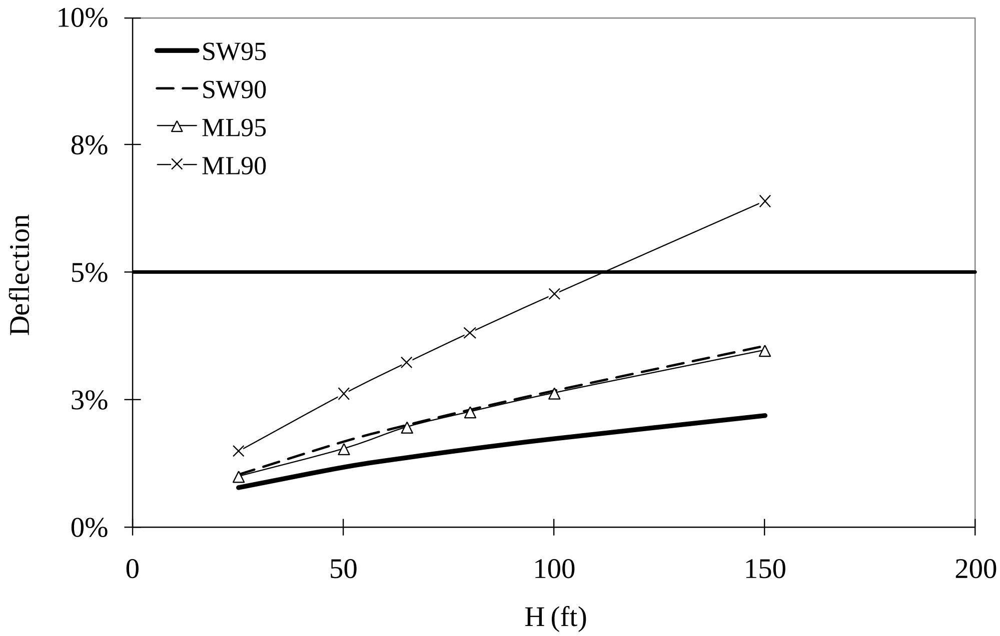


(a)

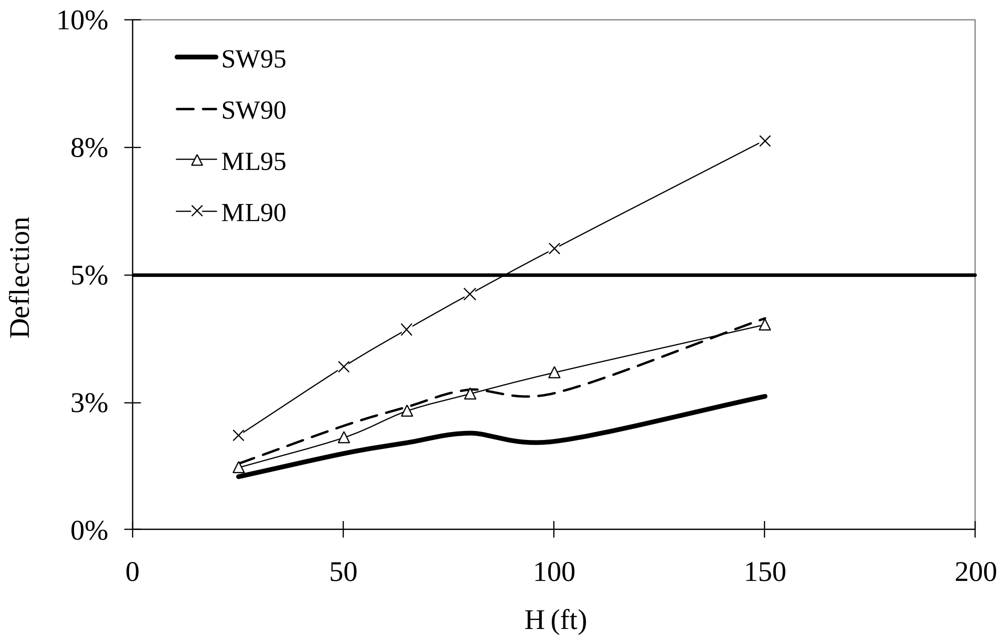


(b)

Figure 3-6: Maximum Wall Stress of Corrugated HDPE Pipes Under Various Fill Heights: (a) Short-term and (b) Long-term

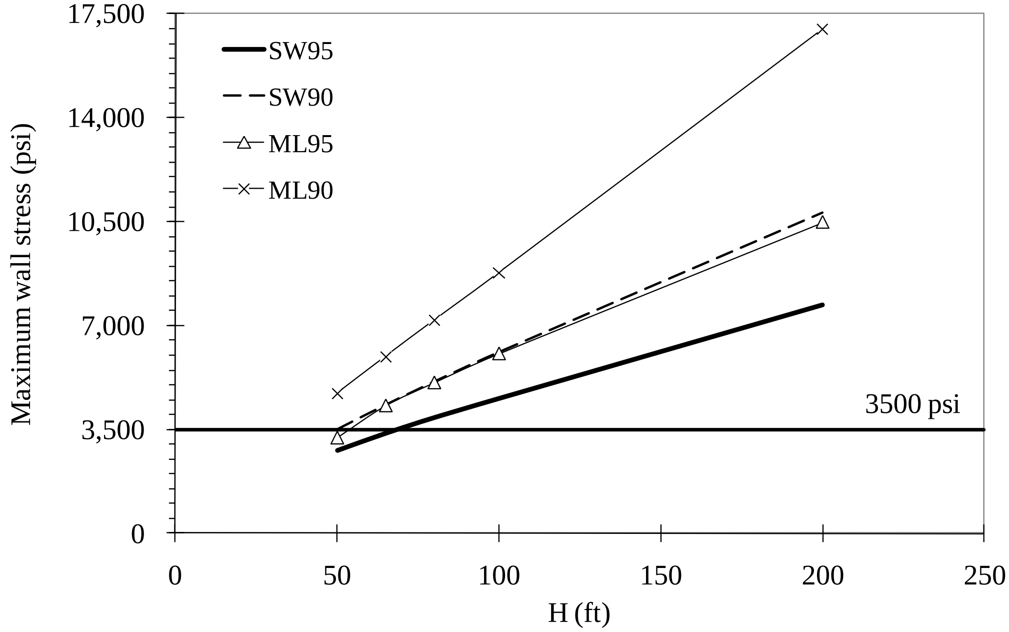


(a)

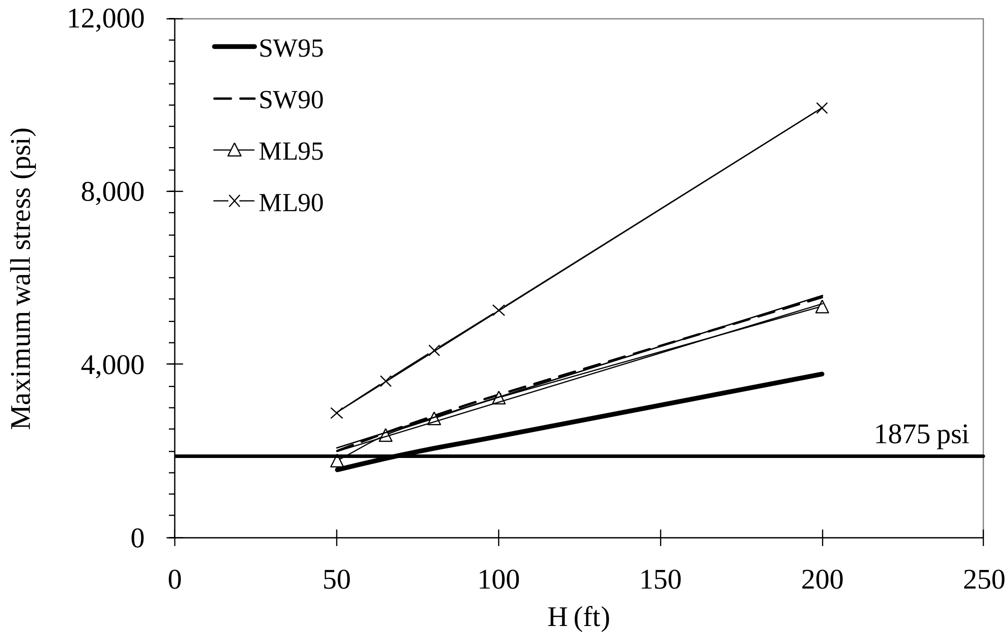


(b)

Figure 3-7: Deflection of Corrugated HDPE Pipes Under Various Fill Heights: (a) Short-term and (b) Long-term (Deflection Limit = 5%)

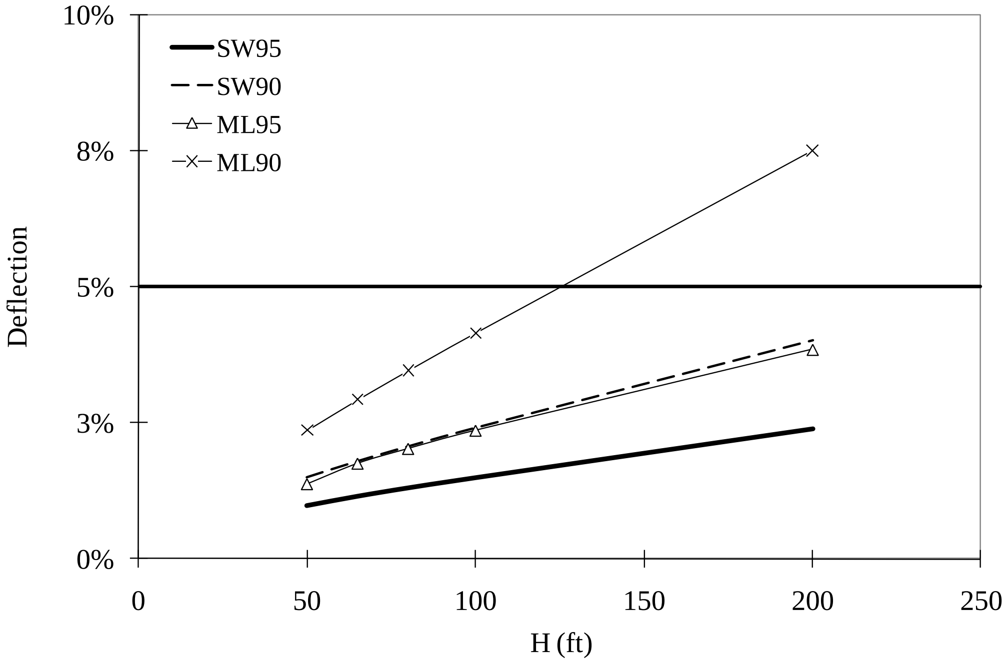


(a)

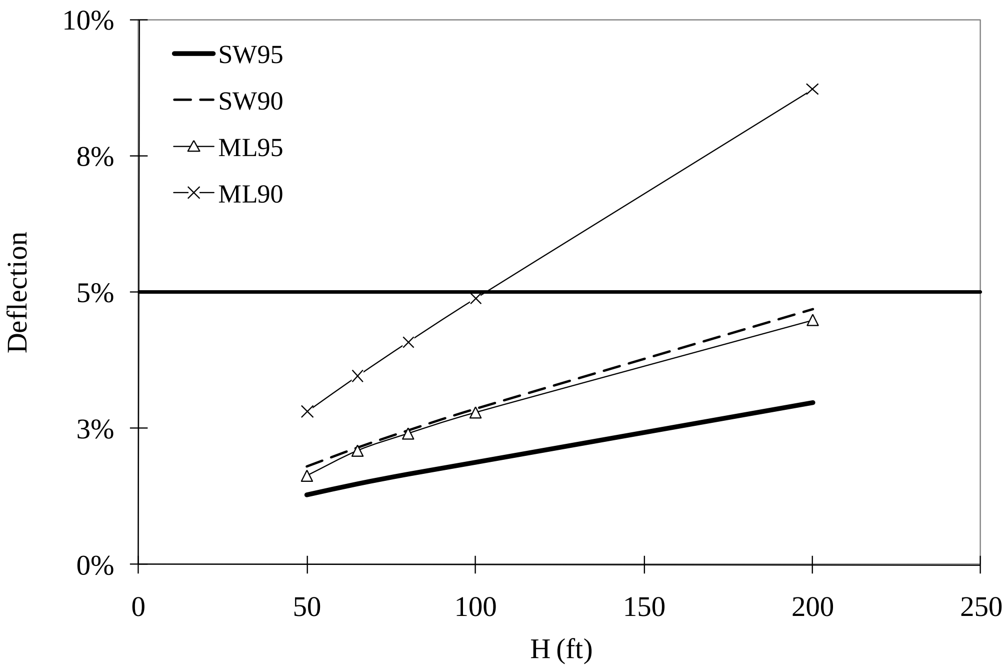


(b)

Figure 3-8: Maximum Wall Stress of Corrugated PVC Pipes Under Various Fill Heights: (a) Short-term and (b) Long-term



(a)



(b)

Figure 3-9: Deflection of Corrugated PVC Pipes Under Various Fill Heights: (a) Short-term and (b) Long-term (Deflection Limit = 5%)

Analyses using the SW95 soil properties resulted in maximum fill heights of up to 60 feet for 60-inch diameter HDPE pipe. This seemingly excessive fill height is primarily due to very high elastic stiffness values that result from the soil models used with 95% compaction. In comparison, the recommended maximum fill height from PPI (Gabriel 2008) for a 60-inch diameter HDPE pipe backfilled with SW95 soil is 28 feet, which is based upon a prism load analysis that results in a vertical arching factor that is double that resulting from the finite element analysis. The axial forces on the springline calculated from the finite element analyses were only 20% of those calculated from equilibrium using the prism load, and 80% of the maximum wall stress is attributed to axial forces.

Maximum fill heights for HDPE and PVC pipes based on the finite element analyses using SW90 and 60 inch diameter pipe were found to be 42 feet and 46 feet, respectively. Maximum fill heights of corrugated HDPE and PVC pipes for ML90 and 60-inch diameter pipe were evaluated to be 20 feet and 26 feet, respectively. These values are slightly higher than the maximum cover limitations currently being used by state DOTs as shown in Table 2-20. The maximum fill height of HDPE pipes specified by PPI is 18 feet for 60-inch diameter pipes under compacted ASTM Class II soil (SW90) and that of PVC pipes was given by Uni-Bell (Uni-Bell 2005) as 50 feet for SW90. A design example using the PPI design procedure is provided as Appendix B in order to help demonstrate the difference in the values determined using the FEA and values using PPI's design procedure. Table 3-5 summarizes the results of the analyses with a delineation made between pipe diameters of 48 inches and less, and those that have a diameter greater than 48 inches.

Table 3-5: Summary of Maximum Fill Heights Based on SF=2

	HDPE		PVC	
Soil Type	*D* = 12 – 48 in.	*D* > 48 in.	*D* = 12 – 48 in.	*D* > 48 in.
SW95	66 ft	60 ft	86 ft	68 ft
SW90	44 ft	42 ft	57 ft	46 ft
ML95	50 ft	48 ft	60 ft	50 ft
ML90	28 ft	20 ft	40 ft	26 ft

3.1.9 Summary

This study evaluated the maximum fill heights for plastic pipes used in highway construction based on 2D finite element analyses that incorporated nonlinear soil models, the time-dependent material properties of thermoplastics, soil-structure interaction, and the inherent geometric nonlinearity of these systems. The most widely used pipe diameters, ranging from 12 inches up to 60 inches, that have been approved by AASHTO and are being considered for cross drain applications by some states, were simulated in the analyses.

This study showed that the strength limit using the long-term pipe material properties governed the determination of maximum fill heights of the thermoplastic pipes. The results indicate that the maximum fill heights for the use of plastic pipes recommended by the Plastics Pipe Institute and the state DOTs that allow plastic pipes for highway drainage applications are generally conservative. Significant disparities between the finite element results and existing fill height recommendations and requirements were attributed to: (1) an overestimated earth load used to develop existing recommendations and standards, (2) an underestimation in current design methodology of the effects of induced positive soil arching due to relative settlement of the plastic pipes, and (3) an underestimation of the supporting strength provided by the sidefill in the procedures used to define existing maximum fill height recommendations and design approach (McGrath et al. 2009).

Maximum fill heights for corrugated HDPE and PVC pipes based on ML90 and a safety factor of 2 were determined to be 20 feet and 26 feet, respectively for pipe diameters greater than 48 inches. The fill heights suggested as a result of these analyses can serve as a useful benchmark for understanding the source of disparities between burial depths currently prescribed for thermoplastic pipes to be used in highway construction. However, it should be noted that, as with all such numerical analyses, many assumptions and limitations are involved, and all conclusions must be verified through full-scaled testing before being implemented in design and construction. Future work may include: (1) using viscoelastic material properties and conducting creep analyses, (2) developing 3-D meshes and models to verify the 2-D modeling approaches used herein, (3) using actual section properties rather than the estimated properties provided by AASHTO, (4) using other advanced, and perhaps more accurate, soil modeling approaches, and (5) conducting additional testing to validate modeling approaches.

3.2 Minimum Cover Study

3.2.1 Introduction

Most state highway agencies have used thermoplastic pipes extensively for side drain applications over the past several decades, and some states have recently begun allowing thermoplastic pipes for under highway cross-drain applications. There has, however, been little rigorous analytical research into the minimum cover required for safe thermoplastic pipe use in highway construction. Thermoplastic pipes have potential drawbacks such as low material strength, buckling and deflection susceptibility, which are balanced with positive characteristics including ease of installation and hauling, safety, and cost.

The objective of this study was to evaluate the minimum fill height requirements for safe use of thermoplastic pipes in highway construction. The following three load scenarios were considered: (1) without the pavement under highway live loads, (2) with flexible and rigid pavements under highway live loads, and (3) with temporary fill under construction equipment loads. The analysis was carried out using the finite element (FE) method based on a 2D plane strain formulation, nonlinear soil model and parameters, time-dependent material properties of thermoplastics, and the geometric nonlinearity of the soil-pipe system. Finite element models were developed and analyzed using the commercial FE program ABAQUS (2009). The comprehensive minimum fill heights resulting from this study will provide a basis for developing guidelines for engineers, designers, and contractors tasked with specifying the burial depths of thermoplastic pipes to be installed for highway cross drain applications.

3.2.2 Finite Element Modeling

Numerical simulations based on the FE method were carried out to evaluate the minimum cover required for safe use of thermoplastic pipes under truck and construction equipment loading. 2D FE analyses were performed based on modeling the soil-pipe interface conditions as fully bonded. Full soil-structure models were needed in order to impose unsymmetrical live loads as illustrated in Figure 3-10.

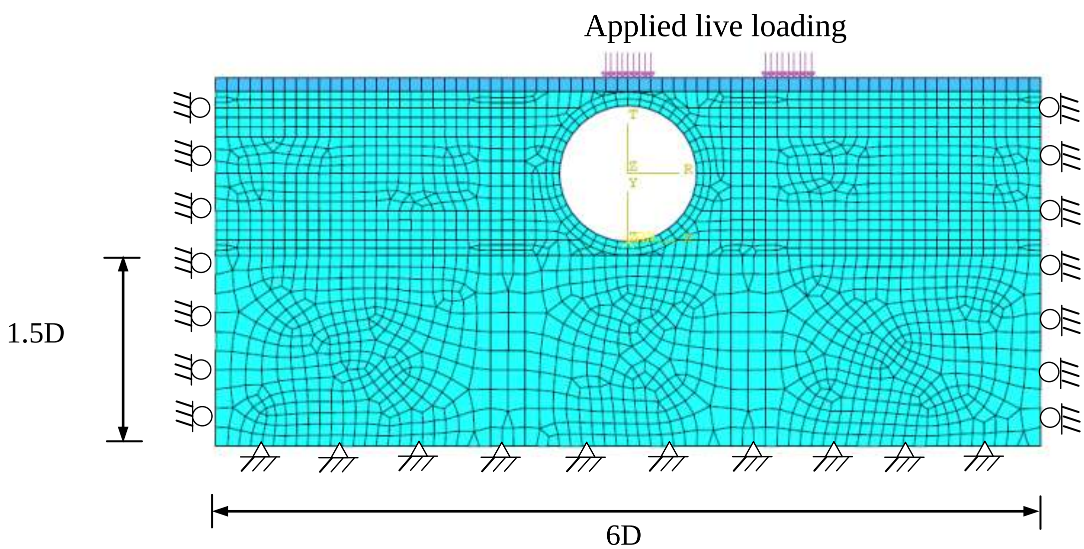


Figure 3-10: Schematic of Finite Element Model for Evaluating Minimum Fill Heights (D = Pipe Diameter)

Two translational degrees of freedom were fixed at the bottom boundary, and only the horizontal translational degrees of freedom were restrained along the two lateral boundaries. The pipe walls were modeled using curved beam elements. The time-dependent material properties of HDPE and PVC used in the analyses are summarized in Table 3-2. The unit weight of HDPE and PVC materials was taken to be 59.3 pcf, but has insignificant impact on the results. The live loads acting on the top of the minimum cover are shown in Figure 3-10, in which the loaded length in the longitudinal direction is assumed to be infinite. A plane strain element was used for the soil. The soil properties were described by two stiffness parameters, namely the tangent modulus and the bulk modulus, which are based on the Duncan and Selig soil models (Duncan and Chang 1970, Selig 1988). The detailed procedure for obtaining the soil parameters was discussed earlier in Section 3.1.3 for the analytical study of maximum cover requirements. The unit weight of soil was assumed to be 120 pcf.

3.2.3 Calibration and Validation

The full-scale field tests carried out by the Florida DOT (FDOT) during December 2001 to May 2002 (Arockiasamy et al. 2004) were employed to calibrate and validate the finite element modeling methodology. The geometry and material properties of a 36-inch diameter HDPE pipe are provided in Table 3-6.

Table 3-6: Properties of HDPE Pipe Used in the Calibration and Validation of the Finite Element Modeling (Arockiasamy et al. 2006)

	HDPE pipe
Nominal pipe diameter (in.)	36
Cross-sectional area (in^2/in)	0.401
Moment of inertia (in^4/in)	0.40
Modulus of elasticity (ksi)	110
Poisson's ratio	0.4

Gravel silty sand with 95% compaction was selected for the bedding and in-situ soil material, which was simulated with in-situ soil parameters (ACPA 1994). Sandy silt was used for both the trench fill and the backfill with the same compaction level, which was simulated with ML95 (silty sand compacted to 95%) parameters. The location of pressure cells used in the tests and finite element modeling are shown in Figure 3-11; all model parameters were based on those given in the FDOT tests (Arockiasamy et al. 2004; Arockiasamy et al. 2006).

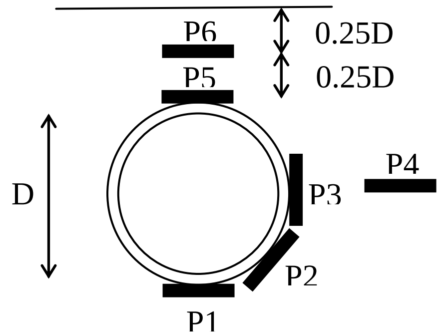


Figure 3-11: Location of Pressure Cells for 36 inch Diameter Pipes (Arockiasamy et al. 2004)

The soil properties were evaluated based on the Duncan and Selig soil models (Duncan and Chang 1970, Selig 1988). Table 3-7 demonstrates that the pressures from the 2D FE analyses of this study and the 3D FE analyses (Arockiasamy et al. 2006) are in the same range as the measured pressures at the pipe crown and springline from the tests.

Table 3-7: Comparisons of Measured Soil Pressures with FE Results for Pipe with 0.5D Burial Depth

Pressure cell position	Field test (Arockiasamy et al. 2006)	FEM (Arockiasamy et al. 2006)	FEM
P1 (psi)	3.05	1.16	2.18
P2 (psi)	5.51	3.77	4.35
P3 (psi)	7.40	7.54	8.99
P4 (psi)	5.66	5.80	6.53
P5 (psi)	18.13	15.08	17.40
P6 (psi)	14.65	73.82	55.84

Note: D = pipe diameter

The vertical and horizontal pipe deflections in this study were measured to be 0.2 inches and 0.11 inches respectively, which compares reasonably with 0.12 inches and 0.06

inches measured from the field tests. The final calibrated finite element models were therefore deemed acceptable and used in the subsequent minimum fill height analyses.

3.2.4 Analysis Method

Since deflections and wall stresses are critical performance parameters considered in the design of plastic pipes, these two parameters were evaluated to determine the minimum safe heights of cover. Deflection is quantified in terms of the ratio of the decrease or increase in diameter to the original pipe diameter. Maximum wall stresses and deflections were evaluated for various AASHTO live loads and construction equipment loads. Yield stresses for PVC and HDPE are given in Table 3-2. In pipe design, the vertical dimension is usually of more concern, and therefore AASHTO LRFD (2007) restricts the allowable total vertical deflection to 5%. AASHTO LRFD (2007) states that the pipe’s response to live loads will reflect the initial modulus, regardless of the age of installation. Gabriel (2008) also showed that the modulus of HDPE under repeated load intervals and for a long period of time remains approximately the same, and that this indicates that the material does not lose strength when not under a constant significant stress condition. However, Gabriel (2008) evaluated the effects of repeated loading for just 7 days, which does not seem to be sufficient time to evaluate the long-term effects. Therefore, the present study analyzed the behavior using both long-term and short-term properties in order to evaluate the potential effects of creep and to provide insight into the influence of initial stiffness loss.

The section properties of corrugated PVC and HDPE pipes used in the FE models are shown in Tables 3-3 and 3-4. The typical section properties of plastic pipes commercially made are generally close to those from AASHTO LRFD (2007) but not exactly the same. Because the values from this study and AASHTO LRFD (2007) are a little less than those from the industry, they provided a conservative design. AASHTO LRFD (2007) specifies 90% as a minimum compaction requirement for HDPE and PVC pipe backfill. The analyses in this study, therefore, included the results for SW90 in addition to those of SW80 for comparative study, which were compared with those of silty sand (ML90 and ML80) and silty clay (CL90 and CL80) in order to evaluate the effects of backfill material properties. The pipe diameters used in the analyses ranged from 12 to 60 inches.

AASHTO LRFD (2007) specifies a safety factor (*SF*) equal to 2 for wall areas in the service load design of thermoplastic pipes. Therefore, allowable stress, f_a, was defined as:

$$f_a = \frac{f_u}{SF} \qquad \text{Equation 3-7}$$

Where: f_a = allowable stress

f_u = specified minimum tensile strength

3.2.5 Minimum Cover Results

3.2.5.1 Minimum Cover without Pavement

The first case investigated using finite element analysis was the situation of a pipe being subjected to traffic loading without a pavement layer. AASHTO H 20 and H 25 live load configurations and surface pressure calculations are shown in Figure 3-12.

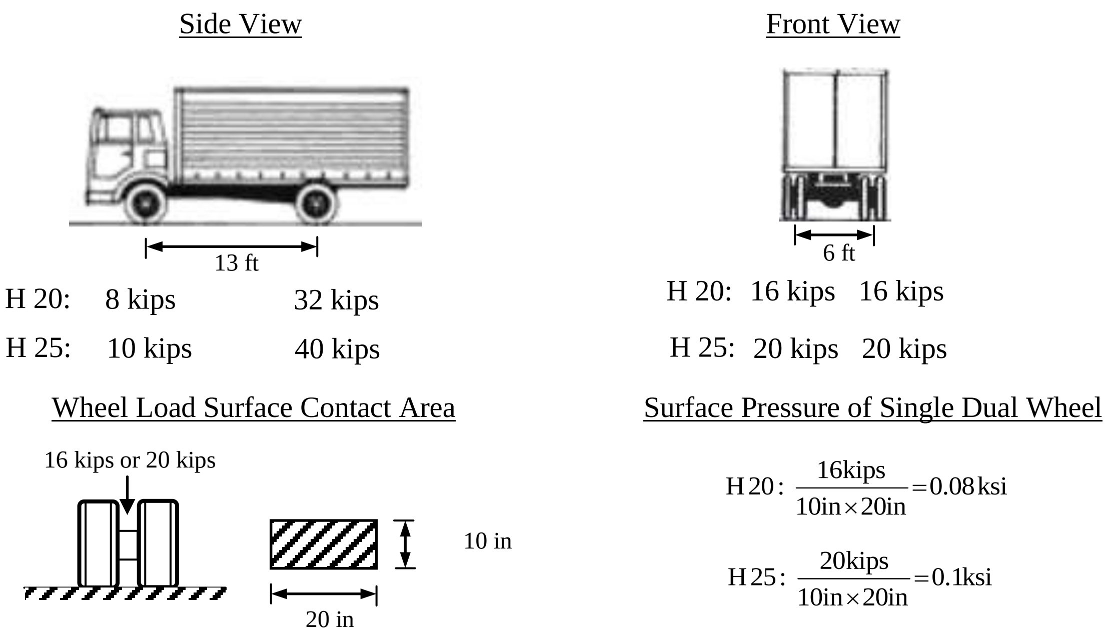


Figure 3-12: AASHTO Live Loads (H20 and H25)

Five loading cases, illustrated in Figure 3-13, were tested in order to identify the critical loading configuration. The H 25 & Alternative load represented in Figure 3-13(e) was found to be the critical loading case and used in the analyses.

Figures 3-14 and 3-15 show the variation of maximum wall stresses and deflection of corrugated HDPE pipes versus the backfill height. These figures demonstrate that the wall stresses, shown in Figure 3-14(b), govern the design for minimum cover. The stress limits including a safety factor of 2 has been included on the figures. Let it be noted that these graphs are to show trends. Several hundred finite element runs were made in order to investigate all of the parameters in this study. Plots were not made for each case, and values shown for minimum covers in Tables 3-8, 3-10, and 3-12 were calculated based on the raw finite element data.

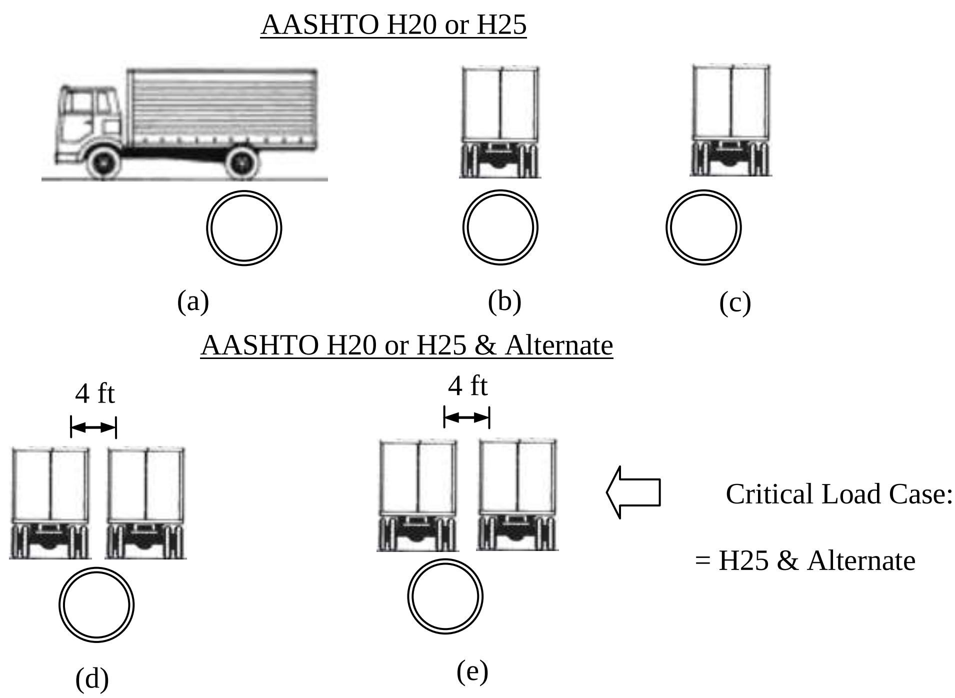


Figure 3-13: Applied Live Load Cases

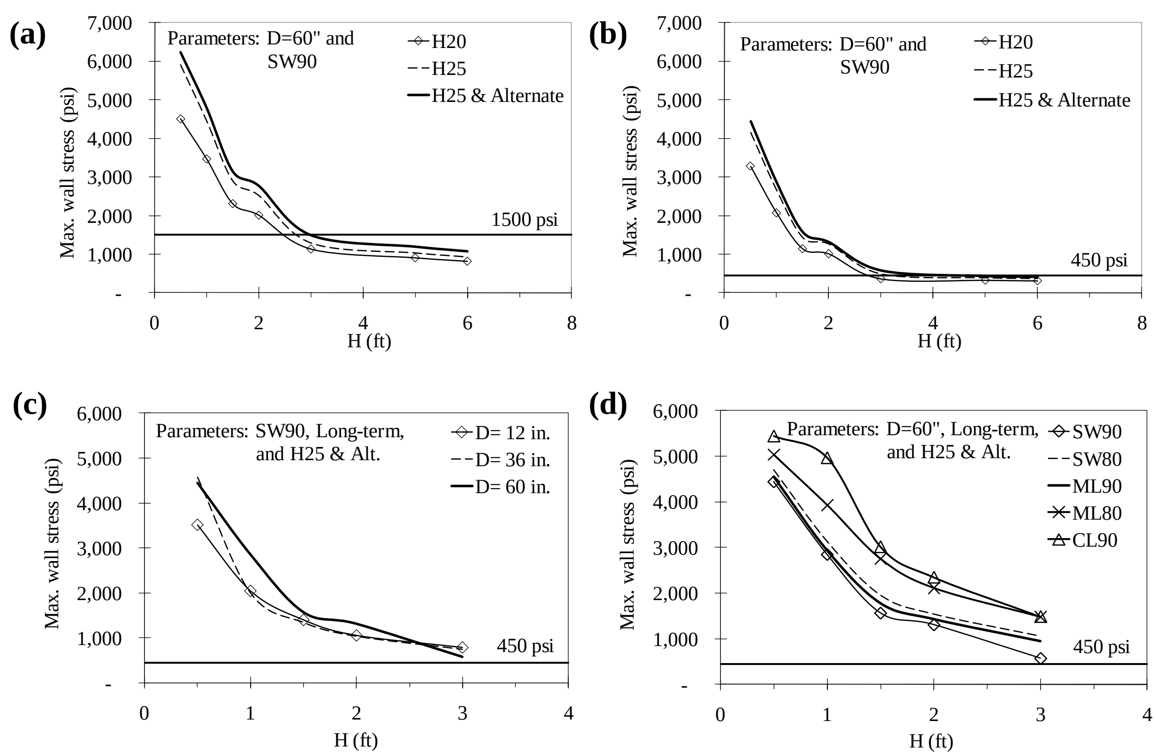


Figure 3-14: Maximum Wall Stress of Corrugated HDPE Pipes Under AASHTO Live Loads: (a) Short-term, Yield Stress = 3,000 psi, (b) Long-term, Yield Stress = 900 psi, (c) Effects of Pipe Diameters, and (d) Effects of Soil Properties

Figure 3-14 shows that the minimum cover under short-term and long-term material properties are 3 feet and 3.3 feet, respectively, for a 60 inch diameter HDPE pipe backfilled with SW90 and loaded with the H 25 & Alternative load. While the long-term properties controlled the design, the results found that minimum cover resulting from the use of the long-term properties was only slightly higher than when using the short-term properties. Therefore, this study agreed with the AASHTO LRFD (2007) and Gabriel (2008) approach reflecting the use of the initial modulus under live loading applications. Figures 3-14 and 3-15 also indicate that the wall stresses reach the yield strength limit before the pipe reaches a 5% deflection.

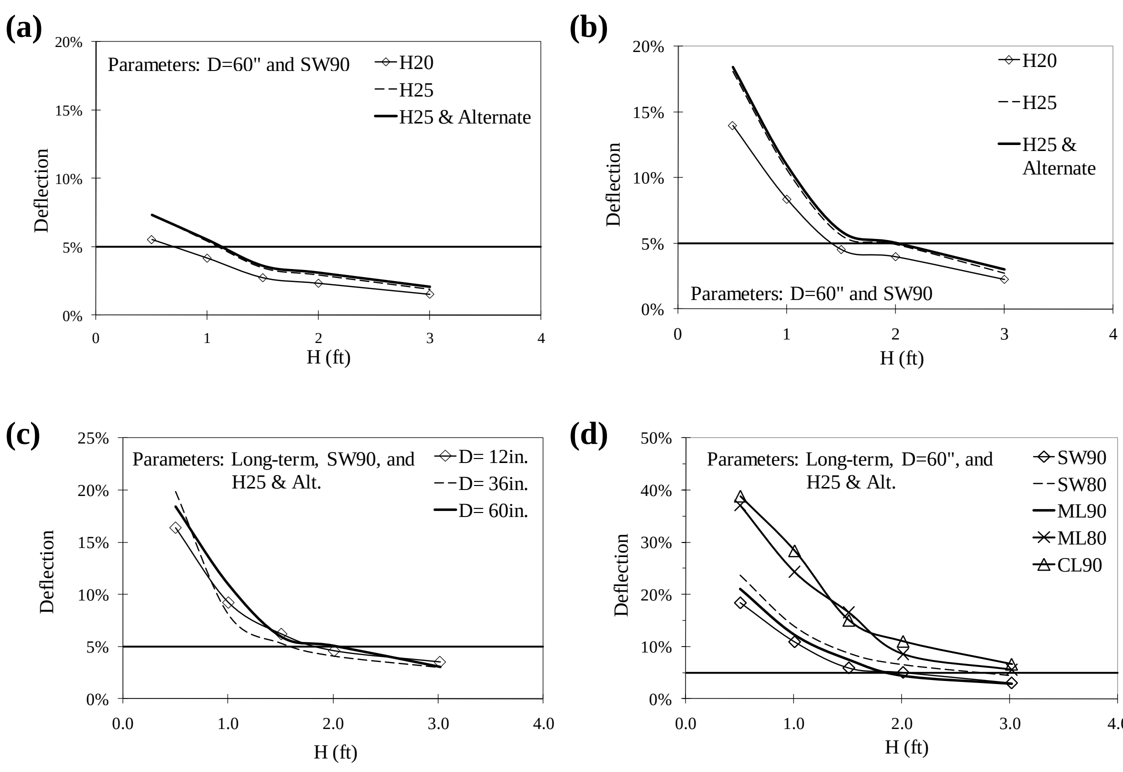


Figure 3-15: Deflection of Corrugated HDPE Pipes under AASHTO Live Loads: (a) Short-term, (b) Long-term, (c) Effects of Pipe Diameters, and (d) Effects of Soil Properties (Deflection Limit = 5%)

Maximum wall stresses and deflections of corrugated PVC pipes showed the same trends as those of corrugated HDPE pipes, and the results can be seen in Figures 3-16 and 3-17.

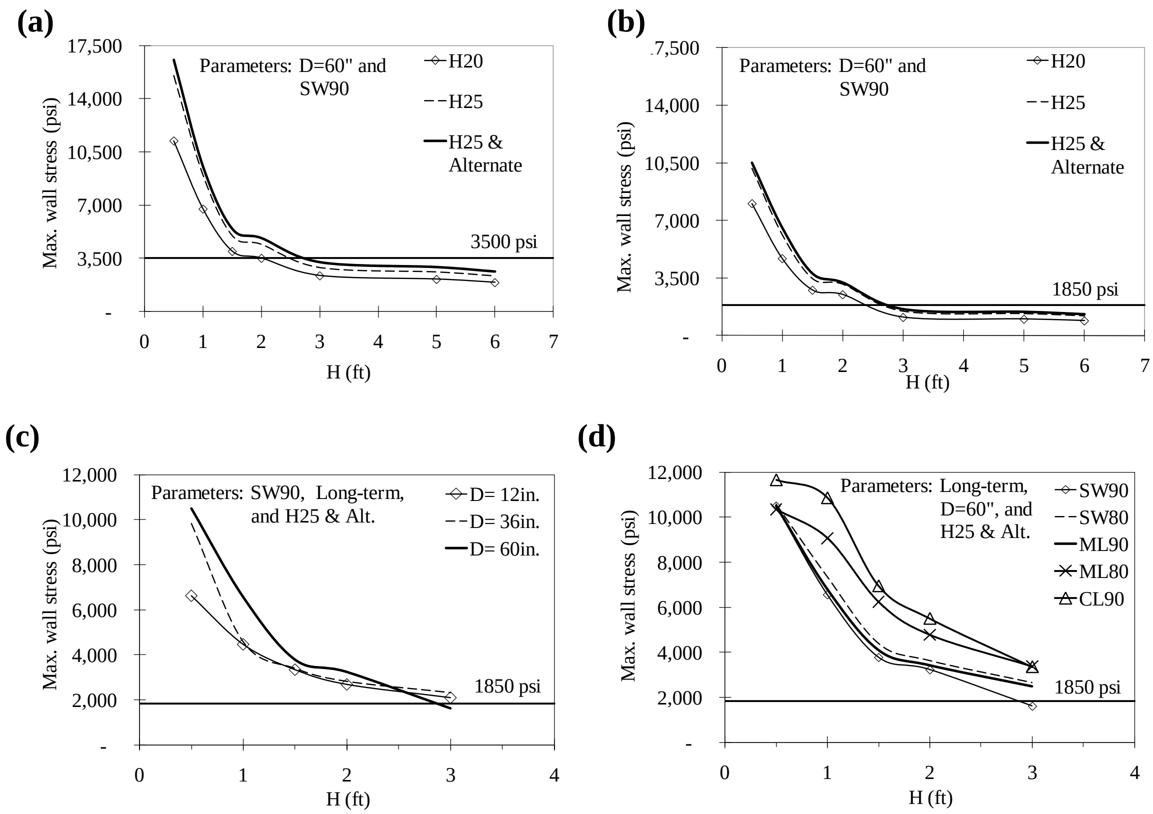


Figure 3-16: Maximum Wall Stress of Corrugated PVC Pipes under AASHTO Live Loads: (a) Short-term, Yield Stress = 7,000 psi, (b) Long-term, Yield Stress = 3,700 psi, (c) Effects of Pipe Diameters, and (d) Effects of Soil Properties

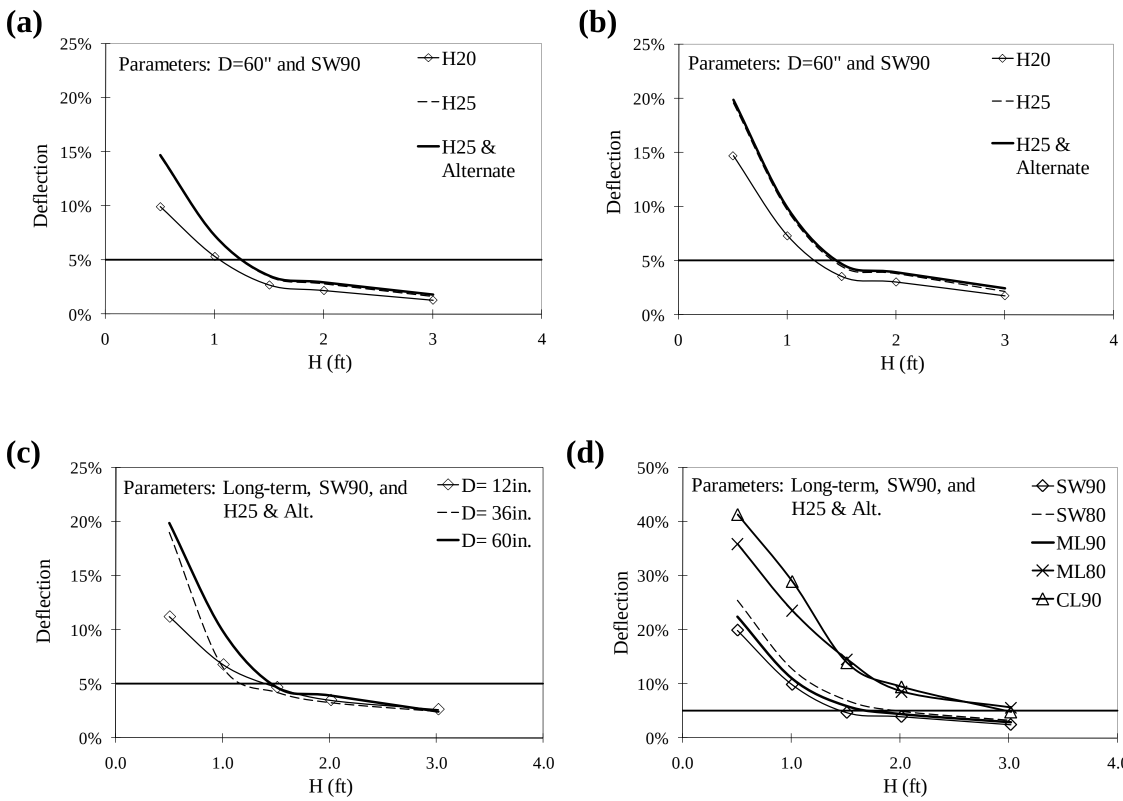


Figure 3-17: Deflection of Corrugated PVC Pipes under AASHTO Live Loads: (a) Short-term, (b) Long-term, (c) Effects of Pipe Diameters, and (d) Effects of Soil Properties (Deflection Limit = 5%)

Minimum cover requirements for the case of a pipe without pavement under the H25 & Alternative live loading are given for various soil properties and pipe diameters in Table 3-8. The minimum fill heights evaluated by this study are slightly higher than those specified by most of the state DOTs for which information is available. This is due to the fact that this study used the AASHTO H 25 & Alternative option as the live load. These results, with the factor of safety, provide a conservative minimum cover requirement for critical highway construction scenarios.

Table 3-8: Minimum Cover without Pavement under Live Loads (SF = 2)

Minimum Cover (ft)												
HDPE	Short-term properties						Long-term properties					
	SW90	SW80	ML90	ML80	CL90	CL80	SW90	SW80	ML90	ML80	CL90	CL80
D= 12 in.	3.3	6.6	4	7.2	5	6.6	4	6.9	5	7.8	5.9	6.9
D= 36 in.	4	6.6	4	7.2	5	6.6	4	6.9	5	7.8	5.9	6.9
D= 60 in.	3	4	4	5	5	6.6	3.3	4.6	4	5	5	6.9
PVC	Short-term properties						Long-term properties					
	SW90	SW80	ML90	ML80	CL90	CL80	SW90	SW80	ML90	ML80	CL90	CL80
D= 12 in.	3.3	4	3	5	4	5	3.3	5	3	5.9	4	5.9
D= 36 in.	3.3	5	3	5.9	3.3	5.9	4	5	3	5.9	4	5.9
D= 60 in.	3	4	4	5	4	4	3	4	4	5	4	5

Units = feet

3.2.5.2 Minimum Cover Including Rigid and Flexible Pavements

Because most pipes buried under a roadway will be covered with some type of pavement, the effect of having a pavement above the pipe was investigated using finite element analyses. The pavement thickness can be included as part of the minimum cover, and in this investigation the word "cover" refers to the summation of the soil layer and the pavement thickness, as illustrated in Figure 3-18.

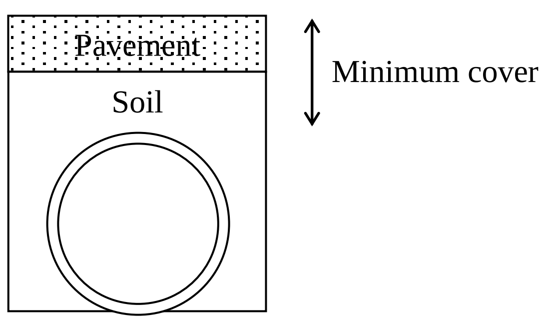


Figure 3-18: Geometry of Minimum Cover Including Rigid or Flexible Pavements

Hard surfaced pavement types can be categorized into flexible and rigid. Flexible pavements are surfaced with bituminous materials such as asphalt concrete (AC). These types of pavements are called flexible because the total pavement structure bends or deflects under traffic loads. Rigid pavements are composed of a portland cement concrete (PCC) surface course. Such pavements are substantially stiffer than flexible pavements due to the high modulus of elasticity of the PCC materials. The assumed material properties of both PCC and asphalt are shown below in Table 3-9.

Table 3-9: Material Properties of Portland Cement Concrete (PCC) and Asphalt

	Modulus of elasticity psi	Density pcf
PCC	3,600,000	145
Asphalt	656,000	145

In this study, PCC and AC properties were used for rigid and flexible pavements, respectively. The effect of pavement thickness was evaluated by using both 6-inch and 12-inch thicknesses. The wall stress distributions when subjected to the H25 & Alternative loading were highly affected by pavement types and pavement thicknesses as shown in Figure 3-19.

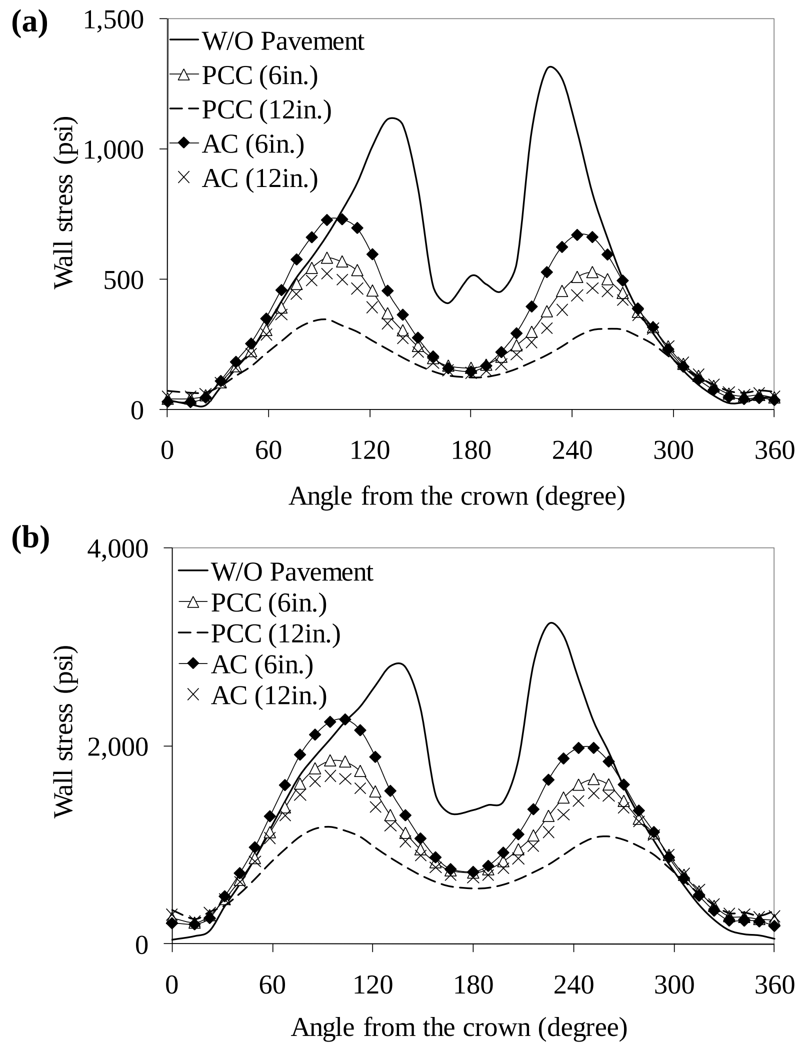


Figure 3-19: Comparison of Wall Stress Distributions of Thermoplastic Pipes Between Without Pavement and With Pavement Conditions: (a) HDPE Pipe and (b) PVC Pipe (Parameters: D = 60 in., SW90, Long-term Properties)

As expected, the addition of pavements reduces the maximum wall stresses, and PCC is more effective than AC. In the case of PCC with a thickness of 6 inches, the maximum stress was reduced by 50% relative to that of no pavement. As the thickness of PCC pavements was doubled, the maximum stress was reduced by an additional 50%. The unsymmetrical wall stress distributions were due to the unsymmetrical loading case shown in Figure 3-13(e). It should be noted that the maximum wall stresses occur below the springline at about 30° from the invert, which emphasizes that the soil in the haunch area from the foundation to the pipe springline provides significant support to the pipe and greatly influences pipe stresses. Table 3-10 shows minimum cover requirements including a safety factor of 2 based on the case for pipes that are under a flexible or rigid pavement subjected to the H25 & Alternative live loading. These requirements were controlled by yield stresses of the HDPE and PVC.

Table 3-10: Minimum Cover Including Pavement under Live Loads (SF=2; Short-term Properties)

Minimum Cover (ft)												
	Thickness of pavement= 6 in.											
	PCC						AC					
HDPE	SW90	SW80	ML90	ML80	CL90	CL80	SW90	SW80	ML90	ML80	CL90	CL80
D= 12 in.	3	4	4	5	5	6.6	4	5	5	6.6	5.2	6.9
D= 36 in.	3	5	4	5	5	6.9	4	5.9	5	6.6	5.9	7.9
D= 60 in.	3	4	4	5	5	6.6	4	5	5	6.6	5.2	6.9
	Thickness of pavement= 12 in.											
	PCC						AC					
HDPE	SW90	SW80	ML90	ML80	CL90	CL80	SW90	SW80	ML90	ML80	CL90	CL80
D= 12 in.	1.6	2.6	2.6	4.6	5	5.9	3	4	4	5	5	6.9
D= 36 in.	1.6	2.6	2	5	5	5.9	3	5	4	5	5	6.9
D= 60 in.	1.6	2.6	2	4.6	5	5.9	3	4	4	5	5	6.9
	Thickness of pavement= 6 in.											
	PCC						AC					
PVC	SW90	SW80	ML90	ML80	CL90	CL80	SW90	SW80	ML90	ML80	CL90	CL80
D= 12 in.	1.6	4.6	4	5	5	6.9	3	5.9	5	5.9	5	7.9
D= 36 in.	1.6	4.6	4	5	5	6.9	3	5.9	5	5.9	5	7.9
D= 60 in.	1.6	4.6	4	5	5	6.9	3	5.9	5	5.9	5	7.9
	Thickness of pavement= 12 in.											
	PCC						AC					
PVC	SW90	SW80	ML90	ML80	CL90	CL80	SW90	SW80	ML90	ML80	CL90	CL80
D= 12 in.	1.6	2	1.6	4	5	5.9	2	4	4	5.9	5	6.9
D= 36 in.	1.6	2	2	4	5	5.9	2	4	4	5.9	5	6.9
D= 60 in.	1.6	2	2	4	5	5.9	2	4	4	5.9	5	6.9

An interesting result was found that the minimum cover requirements under flexible pavements can be a little higher than or close to those without pavements depending on soil properties and the thickness of pavement. As shown in Table 3-8 and Table 3-10, the minimum cover for a HDPE pipe under no pavement and with a 6 inch

AC pavement were 4 feet and 5 feet respectively for a ML90 backfill using short-term properties of the pipe. Minimum cover under the 12 inch AC pavement was 4 feet, which was the same as that under no pavement. It can be explained by the fact that as the thin flexible pavement distributes loads over a smaller area relative to the thick flexible pavement or rigid pavement and acts like a beam surrounding and pushing down the backfill soil; it can induce more concentrated loads on the buried pipe than the case without pavement. Minimum cover requirements under rigid pavements are less than those when no pavement is considered.

3.2.5.3 Minimum Cover Considering Construction Equipment Loads

ASTM D2321 specifies a minimum cover of at least 24 inches or one pipe diameter (whichever is greater) for Class I soils and a cover of at least 36 inches or one pipe diameter for Class II and III soils. AASHTO LRFD (2007) recommends using the minimum cover requirements provided in Table 3-12 as a guide for construction cover. Most state DOTs specify a cover of 1 to 2 feet higher than normal for construction loads in order to account for heavy construction equipment. Those values usually range from 3 to 4 feet (ADOT 2006).

Construction loading cases were investigated in order to find the most critical load scenarios. As illustrated in Figure 3-20, equipment travelling both parallel and perpendicular to the pipe was considered in the FE models.

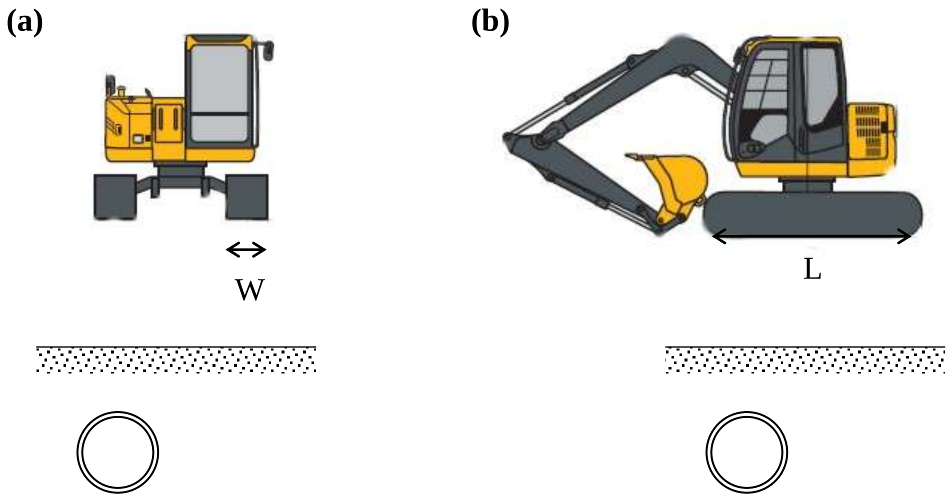


Figure 3-20: Construction Loads Used in FE Models: (a) Equipment Traveling Parallel to Pipe and (b) Equipment Traveling Perpendicular to the Pipe (W=Width of Track, L=Length of Track)

The width and length of track and tire used in the analyses are given for each axle load in Table 3-11.

Table 3-11: Surface Contact Area of Construction Equipment (John Deere 2010)

Axle load kips	Width of track or tire (W) in.	Length of track (L) in.	Length of tire (L) in.
18 – 50	24	100	12
50 – 75	32	160	16
75 – 110	36	180	18
110 – 150	36	230	18

The FE results showed that the critical loading case was when the equipment was travelling perpendicular to the pipe as is shown in Figure 3-20(b), so the minimum cover considering construction equipment loads was evaluated based on this loading.

There are two primary differences between live loading and construction equipment loading application for evaluating the minimum cover. First, construction equipment loads are applied on temporary cover, which may not be well-compacted, while live loads are applied to the well-compacted permanent cover. The appropriate application of soil properties for temporary soil cover is important in order to simulate the behavior of the uncompacted soil. The FE models used in this study were developed using various soil properties and compaction levels, and SW60 was chosen to simulate the temporary cover above the crown. Three different structural backfill cases were investigated for construction loads. Those cases included using SW60 to simulate very little compactive effort, and also using SW90 and ML90 to represent more representative cases of pipe being backfilled to specifications. Second, short-term properties of HDPE and PVC were used in the analyses.

Figure 3-21 shows the maximum wall stresses and deflection versus temporary fill height under various construction equipment loads without structural backfill. Figures 3-21(a) through 3-21(d) illustrate that the minimum cover under construction loads without structural backfill was governed by the deflection limit, and not wall stresses.

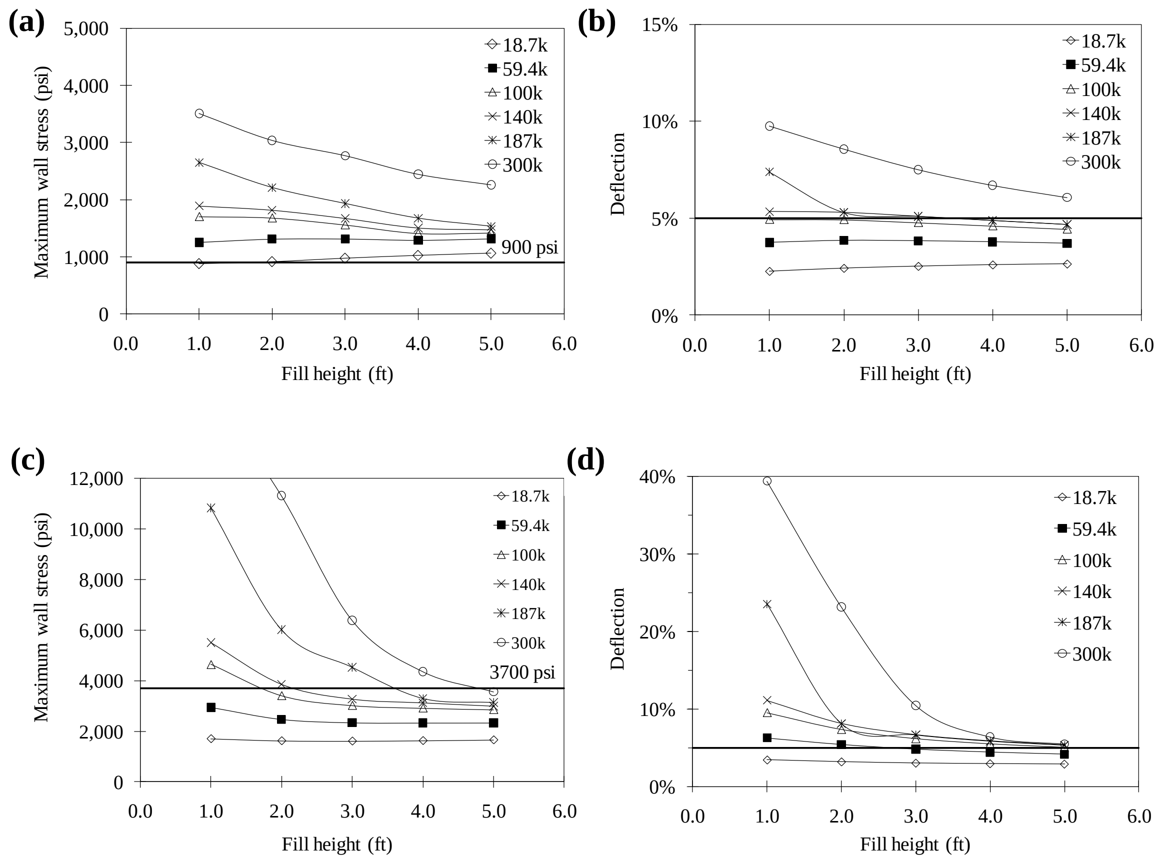


Figure 3-21: Maximum Wall Stresses and Deflection Versus Temporary Fill Height Under Construction Equipment Loads: (a) Stresses on HDPE Pipes, (b) Deflection on HDPE Pipes, (c) Stresses on PVC Pipes, and (d) Deflection on PVC Pipes (Parameter: D = 60 in., SW60, Short-term Properties)

The loose temporary backfill and unsymmetrical heavy construction equipment loads induced severe deformation. Minimum cover under construction loads with SW90 and ML90 structural backfill were governed by the wall stress limit. Therefore, both wall stress and deflection limits should be checked in the design since the minimum cover is highly affected by the quality of backfill material and compaction around the pipe. Table 3-12 provides a summary of minimum cover under construction equipment loads based on the short-term properties of thermoplastics with and without structural backfill, and compares them with the AASHTO recommendations.

Table 3-12: Minimum Cover under Construction Equipment Loads

AL (kips)	18-50			50-75			75-110			110-150		
	AASHTO	FE (HDPE)	FE (PVC)	AASHTO	FE (HDPE)	FE (PVC)	AASHTO	FE (HDPE)	FE (PVC)	AASHTO	FE (HDPE)	FE (PVC)
D (in.)	SW60 used in the temporary fill and around the pipe & track pressure applied											
12	-	1	1	-	1	2	-	2	4	-	5	5
36	2	3	3	2.5	3	4	3	4	4	3	5	5
60	3	3	4	3	3	4	3.5	4	5	4	5	5
D (in.)	SW60 used in the temporary fill and SW90 in the structural backfill & track pressure applied											
12	-	1	1	-	1	2	-	2	3	-	4	4
36	2	2	2	2.5	2	2	3	3	3	3	4	4
60	3	2	2	3	2	3	3.5	3	4	4	4	4
D (in.)	SW60 used in the temporary fill and SW90 in the structural backfill & tire pressure applied											
12	-	7	7	-	7	7	-	7	7	-	7	7
36	2	6	6	2.5	6	6	3	7	7	3	7	7
60	3	5	4	3	5	4	3.5	6	5	4	6	5
D (in.)	SW60 used in the temporary fill and ML90 in the structural backfill & track pressure applied											
12	-	1	1	-	1	2	-	2	3.5	-	4	4.3
36	2	2.5	3	2.5	3	3	3	3.5	4	3	4.3	4.3
60	3	3	3	3	3	3.5	3.5	3.5	4	4	4.3	4.3
D (in.)	SW60 used in the temporary fill and ML90 in the structural backfill & tire pressure applied											
12	-	9	8	-	9	9	-	9	9	-	10	9
36	2	8	7	2.5	8	8	3	9	8	3	9	8
60	3	6	5	3	7	6	3.5	7	6	4	8	6

Unit: feet Note: AL = Axial Load

The FE results (Table 3-12) demonstrated that the minimum fill heights for PVC pipes are slightly higher for some cases than those for HDPE pipes, which initially may seem to be counterintuitive as PVC is stiffer than HDPE. However, it can be explained using Figure 3-22, which shows the contributions of axial force and bending moment for total wall stresses. Wall stresses induced by bending moment are over 50% for both HDPE and PVC pipes, which is a result of severe deformation. Even though the modulus of elasticity of PVC is 4 times higher than that of HDPE, the moment of inertia of the PVC pipe wall is approximately one-eighth to one-tenth the moment of inertia of HDPE for the same pipe diameter (AASHTO LRFD 2007). PVC pipes, therefore, may experience more deflection than HDPE pipes.

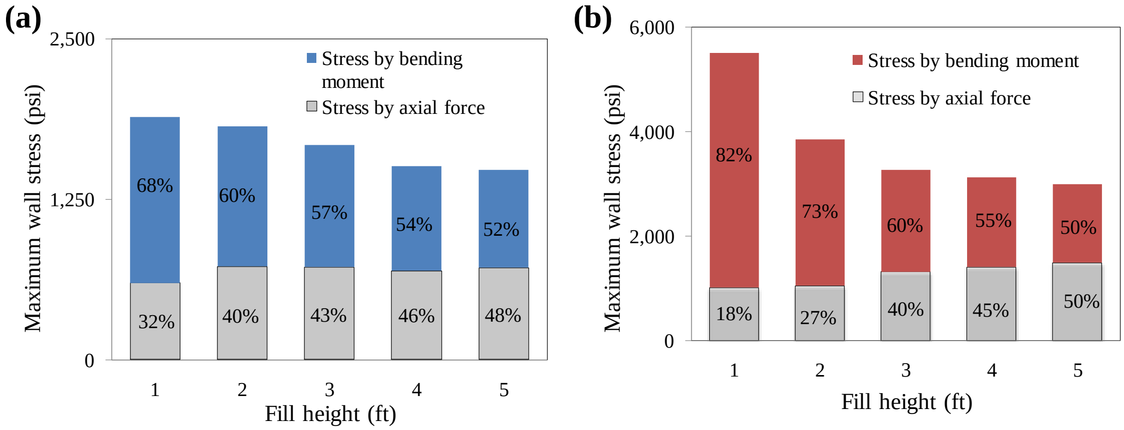


Figure 3-22: Comparison of Wall Stresses Induced by Axial Force and Bending Moment: (a) HDPE Pipes and (b) PVC Pipes (Parameters: D = 60 in.)

3.2.6 Summary and Conclusions

This study evaluated minimum cover requirements based on 2D finite element analyses that incorporated nonlinear soil models and took into account the time-dependent material properties of thermoplastics and the geometric nonlinearity of the soil-structure system. Minimum cover required for thermoplastic pipe used in highway construction was evaluated for the following three cases: 1) without the pavement under highway live loads, 2) with flexible and rigid pavements under highway live loads, and 3) with temporary fill under construction equipment loads. Based on the analytical results presented in this study, the following conclusions can be drawn:

- Minimum cover calculated from this study was slightly higher than those of current standards. This can be attributed to the use of H25 & Alternative live loads and the critical construction equipment loads that were used to ensure a conservative design.
- Minimum cover under AASHTO live loads were found to be slightly higher using the long-term properties than when using the short-term properties. The difference in minimum cover between short-term and long-term properties,

however, was negligibly small. Therefore, this study agreed with the use of the initial modulus under the live loading applications as reflected in AASHTO LRFD (2007) and Gabriel (2008). Field tests may be warranted to support the analytical results of this study.

- Minimum cover under construction equipment loads was governed by either the deflection limit or by the wall stresses using the short-term properties of thermoplastics.
- Maximum wall stresses under AASHTO live loads occur below the springline and approximately 30° from the invert, which emphasizes that the soil in the haunch area from the foundation to the pipe springline provides significant support to the pipe and reduces pipe stresses.

CHAPTER 4

FIELD STUDY OF THERMOPLASTIC PIPE

4.1 Introduction

The objective of the field study was to monitor the construction and installation of thermoplastic pipes and to assess their performance once the installation was complete. The project team worked closely with engineers from the Alabama Department of Transportation (ALDOT) to locate and coordinate a project that would allow the use of thermoplastic pipes to be investigated. Several locations were investigated, and it was decided by ALDOT to conduct the project on Beehive Road in Lee County, Alabama. This project consisted of adding a new exit to Interstate 85 along with new approach roads. The research project at this site consisted of installing five runs of plastic pipe that spanned under the roadway. These pipe installations were not required for drainage and were added into the project for the sole purpose of evaluating their performance in a cross drain application. The construction of the five trial installations was monitored and documented to evaluate the design and construction variables dealing with HDPE and PVC pipe. Some of the variables included pipe diameter, fill height, and bedding and backfill materials. In addition to observing the installation, a monitoring program was created that will consist of at least two site visits and will be described in more depth later.

4.2 Monitoring Program

The field investigations of HDPE and PVC pipe for use as cross drainage under highways consisted of five trial installations to assess the applicability and use of the pipes. The first site visit took place at the time of the pipe installation, and the second visit was approximately 30 days after the completion of the installation and final backfill. In addition, a long-term condition monitoring program for the field trial installations will be designed and implemented to begin accumulating performance history data.

4.2.1 Field Objectives During Pipe Installation

The inspection made during the pipe installation was to monitor the process and document the construction procedure. It was done with the use of a video camera to record the installation while all relevant information needed was documented through notes and digital photography. Emphasis was placed on completing and/or recording the following information:

- Record general background information. Examples include location, type and size of pipe, pipe manufacturer, type of joints, depth of backfill material, size of trench, number of pipes installed, and any other pertinent information,
- Check pipe for roundness and damage before installation,
- Construction procedures and equipment,
- Placement and compaction of foundation and bedding,
- Handling and installation of pipe,
- Results of tests run on samples of structural backfill taken by ALDOT shall be observed to determine if specifications are met,

- Placement and compaction of backfill as well as quality control procedures. Noting such things as thicknesses of lifts and number of passes with compaction equipment. A copy of all in-place density and moisture measurements collected by ALDOT (or representative) was collected, and
- Visual inspection of pipe for any deficiencies due to construction.

4.2.2 Post-Construction Inspection

Once the pipes had been installed, the post-construction inspection was completed approximately 30 days after the installation of the pipe to evaluate the general performance of the pipe. Post-construction inspections consisted of a variety of tasks which are listed below:

- Per specifications, a mandrel will be pulled through the pipe no sooner than 30 days after final backfilling, which will be monitored by the project team.
- Inspection of the interior of the pipe by direct man-entry. Typical items observed would include pipe deflections, evaluation of the shape (deflection, distortion), pipe wall surface conditions (cracking, buckling), and inlet end and outlet conditions, pipe joint conditions (offset, opening).
- Monitoring of the vertical and horizontal deflections of the buried pipe. This was achieved by direct measurement and was done at 10-ft intervals throughout the length of the pipes. In order to provide consistent information for comparison, the same locations will be measured during each subsequent inspection. This was completed by marking the locations of interest with a permanent marker or paint pen.

- Inspection of joint offset and/or openings between joints. Measurements of openings will be recorded. Infiltration and/or exfiltration will be noted when encountered.
- Check for erosion of the backfill around the pipe.
- If pavement has been placed, check and note any issues with pavement directly above the pipe installation.
- Photo-documentation using a digital camera. All photos will be documented, detailing the location of the picture as well as a description.

4.2.3 Long-term Monitoring Program

The long-term monitoring program will consist of periodically continuing to do the post-construction inspections. By continuing these inspections, it will provide long-term performance data that will be used to determine the performance of the thermoplastic pipe over the design life of the system.

4.3 Field Study

The five pipeline trial installations were selected in order to evaluate the most design and construction variables possible. The parameters that were varied include:

- Plastic pipe type (HDPE and PVC),
- Pipe diameter,
- Fill height, and
- Bedding and backfill materials.

The fact that only one suitable location was identified by ALDOT for the field study somewhat limited the possible parameters that could be varied.

The five trial installations consisted of the following variations. There were two runs of 36-inch diameter pipe split into half HDPE and half PVC that were buried under a depth of fill ranging from 25 to 30 feet. One of the lines was backfilled along the entire length using an ASTM Class II soil. The other line consisted of half PVC with an ASTM Class III backfill and half HDPE with an ASTM Class I backfill. There was one additional run under the deep fill that consisted of half 48-inch diameter and half 54-inch diameter HDPE pipe backfilled with an ASTM Class I backfill. In addition to the pipes buried under deep fill, there was one run of 36 inch diameter PVC and one run of 36-inch diameter HDPE placed under a shallow fill ranging from 4.5 to 8 feet. The shallow lines of pipe were backfilled using an ASTM Class III soil. For the pipelines that were split by pipe size or material, the pipes were connected in the middle using a cast-in-place concrete junction box. Each of the pipes was installed with a standard concrete end treatment constructed with a 4:1 slope. The variations of pipe installations are summarized below in Table 4-1.

Table 4-1: Pipe Installation Details

			PVC		HDPE	
STA	**COVER**	**LENGTH**	**DIA**	**Backfill**	**DIA**	**Backfill**
222+00	25-28 ft	254 ft	36"	Class III	36"	Class I
223+00	26-30 ft	258 ft	36"	Class II	36"	Class II
224+00	25-28 ft	260 ft	N/A (entire length HDPE)		48"/54"	Class I
230+00	4.5-8 ft	128 ft	36"	Class III	N/A (entire length PVC)	
231+00	4.5-8 ft	128 ft	N/A (entire length HDPE)		36"	Class III

The backfill materials chosen above were agreed upon by both members of ALDOT and members of the pipe manufacturers who were required to certify that the installations met their respective specifications. The HDPE pipes were manufactured by ADS, Inc., and the PVC pipes were manufactured by Contech. The use of the lowest quality soil allowed by the manufacturers' specifications for each burial depth was chosen to allow the research team to evaluate the pipe's performance under the most lenient requirements.

4.3.1 Location

The trial installations were located in Lee County, Alabama. As can be seen from Figure 4-1, Lee County is located approximately at the mid height of the state adjacent to the Alabama-Georgia state line.

Figure 4-1: Map of Alabama Showing Location of Lee County (digital-topo-maps.com)

The actual project location was on Beehive Road which is just south of Auburn, Alabama. The approximate location of the project site is shown in Figure 4-2.

Figure 4-2: Project Location (Google Maps)

Beehive Road will cross over Interstate 85 approximately one mile southwest of Exit 51. The construction project consists of adding a new exit along Interstate 85, and the pipes are installed under the road approaching the new exit from the south.

4.3.2 Construction Inspections

The construction inspections were carried out by the project team during the months of December 2010 and January 2011. The findings of those inspections are shown below and are organized by the station numbers of the location along the road where the pipes were installed.

4.3.2.1 STA 224+00

Construction for the field study began at Station 224 on December 21, 2010. The first pipe installed for the field study was a line of HDPE that consisted of half a line of

54-inch diameter pipe joined by a concrete junction box to the remaining half line of 48 inch diameter HDPE pipe. The 54-inch diameter pipe had a length of approximately 125 feet, and the 48-inch diameter pipe had a length of approximately 132 feet. These pipes were installed under a deep fill that ranged between 25 and 28 feet. The 54-inch diameter pipe was installed on December 21st, and the 48-inch diameter pipe installation was completed on December 22nd. The weather conditions for December 21st were sunny with the temperature ranging between 50 and 63 degrees Fahrenheit. On December 22nd, the sky was overcast with temperatures ranging from 47 to 70 degrees Fahrenheit. Both pipes were installed in trench conditions. The trench for the 54-inch diameter pipe had to be stepped as shown in Figure 4-3 with the width ranging between 8 and 10 feet.

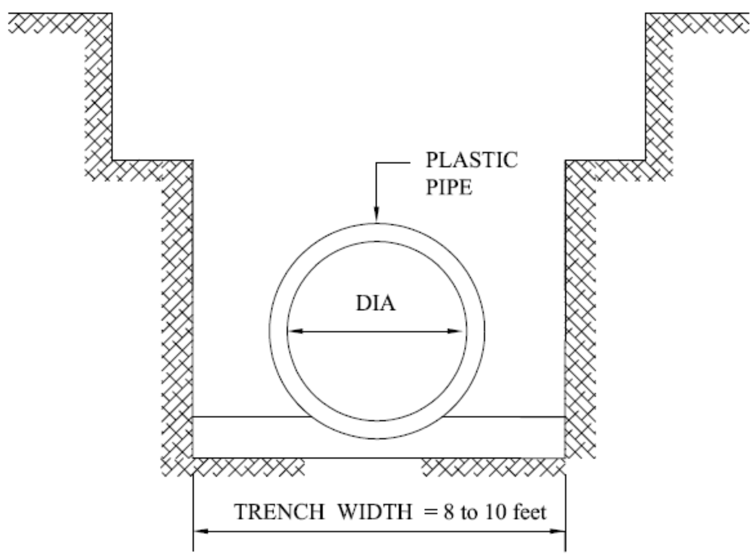


Figure 4-3: Stepped Trench Used for 54-inch Diameter Pipe

The step in the trench was added for safety reasons to prevent from having such a tall, steep trench wall. The trench for the 48-inch diameter pipe also required the step, and the width of the trench ranged between 8 and 9 feet. The bedding and structural backfill used for this line of pipe consisted of an ASTM Class I backfill, which in this case was crushed #57 stone. Figure 4-4 shows the angular nature of the stone used for the backfill.

Figure 4-4: ASTM Class I Backfill (#57 Stone)

The actual construction process began by digging out the trench for the installation starting from the downstream end using the excavator shown in Figure 4-5.

Figure 4-5: Excavator Used During Construction

The trench was only dug for one pipe section at a time, so only about 25 feet of new trench was open at a time. Once the trench was open, the line and grade of the pipeline was set using a Grade Light 2500 which can be seen in Figure 4-6.

Figure 4-6: Grade Light 2500 Utility Alignment Laser

The grade for this pipeline was set to be 1.46%. Once the grade of the trench bottom was set, a 6-inch layer of bedding using #57 stone was placed in the trench bottom. The middle third of the bedding under the pipe was then loosened in order to provide a cradle for the pipe. The pipe was then lowered into place with the bell-end facing upstream. The line and grade of the pipe was then checked to prepare for backfilling. Once in place, the backfill material was dumped over the middle of the pipe so that it spread evenly to both sides. This was done to ensure that the pipe did not roll to one side of the trench which would force the pipe off-line. The backfill was brought up in lifts of approximately 8 to 16 inches and was then compacted using a Wacker WP 1500 plate compactor which is shown in Figure 4-7.

Figure 4-7: Wacker WP 1500 Plate Compactor

After the first lift of backfill was placed on each side of the pipe, special care was taken to ensure that the backfill material was compacted in the haunch region to provide adequate support of the pipe. This was accomplished by using a shovel to hand compact the stone under the bottom side of the pipe. After the first pipe was laid and secured, the next piece of pipe would be trenched and bedded like the one before. The pipes were joined together using a bell and spigot system. Each pipe in the series would have the spigot end lubricated and then pushed in by the excavator that was holding the pipe at third points using nylon slings. An illustration of this technique can be seen in Figure 4-8.

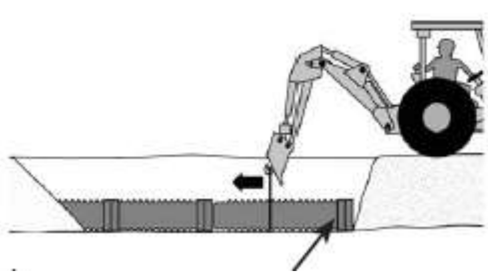


Figure 4-8: Excavator and Sing Method of Joining Pipe (ADS 2010)

The last section of 48-inch diameter HDPE pipe was required to be field cut to the required length. This was observed, and there were no problems with the cutting or the installation of the pipe with a shorter length. Once the pipes creating the pipeline had been jointed together, progressive lifts of backfill were placed and compacted around the pipe until the stone reached approximately one foot above the top of the pipe. Once a foot above the pipe, standard native backfills were used, and it was compacted like a standard embankment. A picture gallery that steps through this process to allow complete understanding can be found in Appendix C.

It is interesting to note that this pipeline cut across one of the main roads that the contractor had running through the project site. Because of this, the 54-inch diameter side of the pipeline needed to be covered to provide a passage way for heavy construction equipment to service other operations of the construction project. Before construction equipment was allowed to cross the trench, a sufficient amount of cover as deemed acceptable by the manufacturer's representative was placed over the pipe. The picture shown below in Figure 4-9 shows a large dump truck crossing above the newly installed 54-inch diameter HDPE pipe. As can be seen from Figure 4-9, the pipe had approximately 2 feet of #57 stone and approximately 4 feet of dirt backfill above the pipe.

Figure 4-9: Dump Truck Crossing Pipeline

4.3.2.2 STA 222+00

The installation of the pipeline at station 222 began on January 14th and was completed on January 15, 2011. This line of pipe consisted of approximately 127 feet of 36 inch diameter HDPE connected by a concrete junction box to 127 feet of 36 inch diameter PVC pipe. The HDPE pipe was backfilled using an ASTM Class I backfill, which in this case was #57 stone. The PVC pipe was backfilled using an ASTM Class III backfill which in this case was a yellow sand with gravel that was a native soil cut out of another portion of the job. The standard proctor density tests run by ALDOT on this backfill material resulted in a maximum dry density of 100.8 pcf and an optimum moisture content of 16.4%. The Class III backfill used is shown in Figure 4-10.

Figure 4-10: Yellow Sand with Gravel – ASTM Class III Backfill

The final fill height for this line of pipe ranged from approximately 25 to 28 feet. The grade of the pipe was 0.46%. The weather for the installation consisted of clear skies with temperatures that ranged from 21 to 47 degrees Fahrenheit. The installation began at the downstream end with the HDPE pipe. The trench for both the HDPE and PVC pipes ranged from approximately 7 to 8 feet in width. Since the HDPE pipe's backfill consisted of #57 stone like the pipe installed at station 224, the installations were conducted in a similar manner. The #57 stone backfill was brought up in approximately 12-inch lifts until one foot above the top of the pipe and compacted as before.

The PVC pipe at this station was installed differently than the previous pipes because it was backfilled with a granular material instead of stone. This portion of the pipeline was bedded using the ASTM Class III backfill which can be seen in Figure 4-11.

Figure 4-11: PVC Pipe Bedded with ASTM Class III Soil

The first pipe section was then held into place to keep the line and grade of the pipe when the next section was installed by dumping a large mound of backfill over the top. This technique is shown in Figure 4-12.

Figure 4-12: PVC Pipe Held in Place with Backfill

Once the first pipe was held in place, the excavation for the next pipe began, and the bedding was leveled. The spigot end of the next pipe was then lubricated and connected to the bell-end of the first pipe using the excavator sling method as before. Once the pipe was laid, backfill was hand compacted into the haunch region. The backfill was then added in approximately 12- to 18-inch lifts until the backfill reached one foot above the top of the pipe. In addition to the plate compactor that was previously used, a jumping jack compactor was also utilized to achieve the required compaction. The jumping jack compactor that was used can be seen in Figure 4-13.

Figure 4-13: Jumping Jack Compactor

A nuclear density gauge was used by a representative of ALDOT to check the compaction of each lift. The compaction achieved for the Class III backfill was 100 percent and 98 percent of the maximum density per AASHTO T-99 for each respective lift. Each of the pipes that were backfilled with a granular backfill followed similar procedures, and a detailed look at that procedure can be found in Appendix D.

4.3.2.3 STA 223+00

The construction of the pipeline at station 223 began on January 15, 2011. The weather consisted of clear skies with temperatures ranging from 38 to 53 degrees Fahrenheit. The line of pipe consisted of approximately 129 feet of 36-inch diameter HDPE pipe connected by a junction box to a line of approximately 129 feet of 36 inch diameter PVC pipe. This line of pipe was under a height of fill that ranged between 25 and 30 feet. The grade of the pipe was 0.95%. This entire line of pipe was backfilled using an ASTM Class II backfill which in this case consisted of a sandy clay mixture as seen in Figure 4-14.

Figure 4-14: ASTM Class II Sandy Clay Backfill

The standard proctor density tests run by ALDOT on this backfill material resulted in a maximum dry density of 113 pcf and an optimum moisture content of 4.4%. The trench width for this line of pipe ranged from 7 to 8 feet. The same Class II material was used as bedding, but it was not compacted. The backfill was placed in approximately 12-inch lifts up until a foot above the top of the pipe and then compacted to meet specifications.

To achieve the desired compaction, two passes were made by the jumping jack compacter along with two passes with the plate compactor. The same compaction techniques were used for the backfill on both the HDPE side of the pipe as well as the PVC side of the pipe. The compaction was found to range between 95 and 100 percent of the maximum density per AASHTO T-99 by the nuclear density gage.

4.3.2.4 STA 230+00

Construction began on the pipeline located at station 230 on January 17, 2010. The line consisted of approximately 128 feet of 36 inch diameter PVC pipe that spanned the entire distance under the roadway. The final fill height ranged between 4.5 to 8 feet along the pipe's length. The grade of this pipe was 0.5%. This pipe was backfilled using an ASTM Class III backfill, but it was not the same Class III backfill used for the PVC pipe at station 222. This Class III backfill was the native material that was used for the embankment construction of the roadway as can be seen in Figure 4-15.

Figure 4-15: ASTM Class III Native Clay Backfill

The standard proctor density tests run by ALDOT on this backfill material resulted in a maximum dry density of 112.8 pcf and an optimum moisture content of 13.4%.

A problem that arose with this backfill material was that there were large clumps of dirt that are not allowed when installing plastic pipe. To remedy the situation, extra care had to be taken when backfilling with this material. The large clumps had to be broken into smaller pieces or picked out of the trench by hand. The weather for this installation consisted of overcast skies with temperatures ranging between 38 and 53 degrees Fahrenheit. This pipe was installed similarly to the other pipes installed with a granular material. The trench width for the installation was approximately 8-feet wide. When placing the first pipe, a mound of #57 stone was placed on top of the pipe to secure it in place and to allow for the connection of the remaining pieces of pipe as shown in Figure 4-16.

Figure 4-16: Stone Used to Hold Pipe in Place

The bedding material used for this pipeline consisted of the same native Class III material and was loosened in the area under the pipe. The backfill was brought up in approximately 12-inch lifts and compacted to meet the manufacturer's specifications. The nuclear density gage testing resulted in readings of 95 and 96 percent maximum density according to AASHTO T-99 on consecutive lifts. An interesting thing to note on this particular installation is that once the backfill had been brought to a height just above the springline of the pipe, the rest of the backfill was pushed over the pipe using a bulldozer as can be seen in Figure 4-17.

Figure 4-17: Bulldozer Backfilling PVC Pipe

Once the fill was over the pipe, the bulldozer was used to compact the fill over the top of the pipe. The next day on January 18, 2011, the pipe received heavy construction traffic with approximately 5 feet of cover. A large dump truck loaded with dirt can be seen below in Figure 4-18 traversing over the top of the PVC pipe.

Figure 4-18: Loaded Dump Truck Crossing PVC Pipe

4.3.2.5 STA 231+00

The installation of the pipeline at station 231 began in the afternoon on January 17th and was completed the morning of January 18, 2011. The line of pipe consisted entirely of 36 inch diameter HDPE backfilled with the same ASTM Class III native soil backfill as the PVC pipe at station 230. Since it was backfilled with the same Class III native soil, it also had the same problem with the large clumps. The problem was solved in the same manner for this pipe as it was for the installation of the pipe at station 230. The weather for this installation consisted of temperatures ranging between 38 and 53 degrees Fahrenheit both days with sprinkling rain on January 17th. The amount of cover for this pipe ranged between 4.5 and 8 feet which is identical to the line of PVC pipe that was installed at station 230. Like the pipe installed at station 230, the first pipe of this line was held into place by placing a mound of #57 stone over the pipe. One difference between this installation and the one completed at station 230 was that this pipe's backfill was brought up and compacted in 12-inch lifts all the way to a foot above the top of the pipe. The compactions measured by the nuclear density gage ranged from 95 to 97 percent of maximum density according to AASTHO T-99 for each successive lift. The backfill for this pipeline was brought up to the halfway point on January 17th and the remainder of the backfill was installed on January 18th. This installation took much longer than the installation at station 230 due to the care and amount of lifts that were carried out to achieve the desired compaction from the base of the pipe until a foot above the pipe. Once the backfill was compacted a foot above the pipe, a bulldozer was used to push the remaining backfill above the pipe and it was compacted like a standard embankment.

4.3.3 Post-Construction Inspections

The post-construction inspections were carried out by the project team during the months of January and February of 2011. The findings of those inspections are shown below and are organized by the station numbers of the location along the road where the pipes were installed.

4.3.3.1 STA 224+00

The post-construction inspection for the pipe installed at station 224 was completed on January 29, 2011. This pipeline consisted of half a line of 54 inch diameter HDPE pipe and half a line of 48-inch diameter HDPE pipe. When inspecting this pipeline, there were no major problems found. Some openings in the joints were found and they ranged from 3/8 of an inch up to 2½ inches. Because of the large bells on the HDPE pipes, these openings ranged within the allowable tolerances. The largest joint opening observed is shown in Figure 4-19.

Figure 4-19: Joint Opening in HDPE Pipe

When marking the locations for deflection recordings, the following procedure was followed. First, a mark was placed at the top of the pipe, and a plumb bob on a string was used to mark the corresponding mark on the bottom of the pipe. The marks that were placed for the horizontal measurements were marked by using a string with a level attached to ensure that the measurements were taken in a straight line. The deflection measurements as a percentage of the nominal diameter for the 54 inch diameter pipe are shown in Figure 4-20.

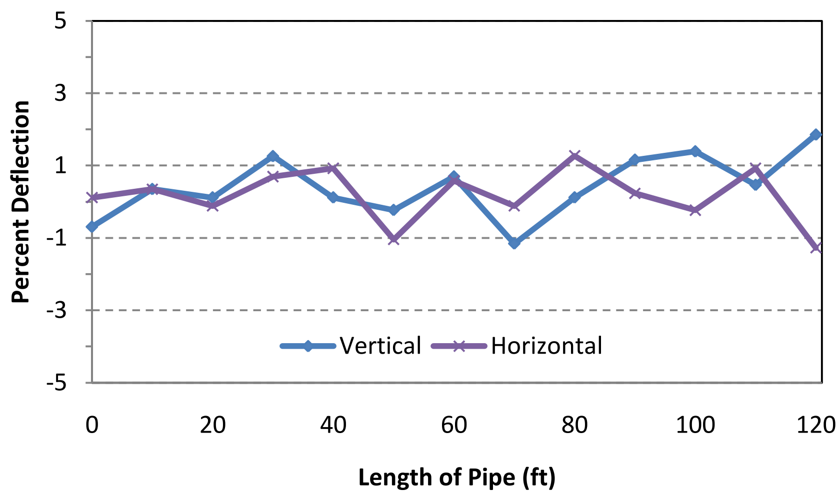


Figure 4-20: Percent Deflection for 54 inch HDPE Pipe at Station 224

As can be seen from Figure 4-20, the percent vertical deflection along the length of the pipe ranged from -1.16% to 1.85%. The percent horizontal deflection along the length of the pipe ranged between -1.27% and 1.27%. Let it be noted that a positive percentage indicates that the pipe was deflected inward, and the dimension measured was smaller than the nominal dimension of the original pipe. The deflections as a percent of the diameter for the 48-inch diameter pipe are shown in Figure 4-21.

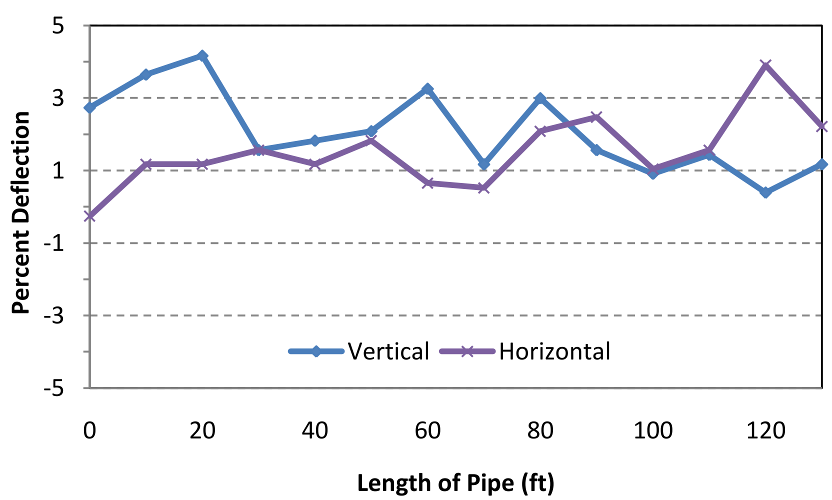


Figure 4-21: Percent Deflection for 48 inch HDPE Pipe at Station 224

As can be seen from Figure 4-21, the percent vertical deflections ranged from 0.39% to 4.17%, while the horizontal deflections ranged from -0.26% to 3.91%. No problems were encountered for this pipeline with infiltration/exfiltration, settlement, or erosion at the time of this inspection.

4.3.3.2 STA 222+00

The post-construction inspection for the pipe installed at station 222 was completed on January 30, 2011. This pipeline consisted of half a line of 36 inch diameter HDPE pipe and half a line of 36 inch diameter PVC pipe. The procedure for marking this pipeline and all subsequent pipelines was carried out in the same fashion as the pipeline at station 224. There were no major problems observed when inspecting this pipeline. The joint openings for the HDPE part of the line ranged from having no gap to a gap of approximately 5/8 of an inch, which is well within tolerance. Joint openings for the PVC pipeline could not be measured because of how the pipes come together, but all of the PVC joints were tight and no issues were found. The deflections as a percent of the nominal diameter for the 36 inch diameter HDPE pipe are shown in Figure 4-22.

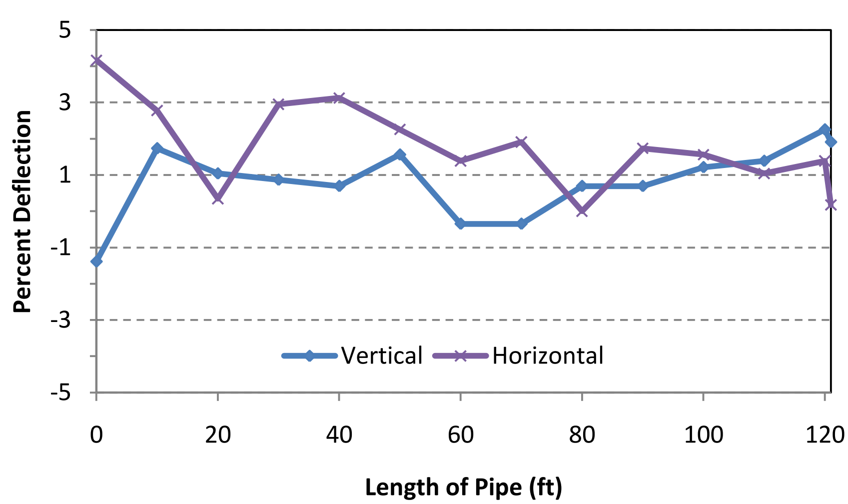


Figure 4-22: Percent Deflection for 36 inch HDPE Pipe at Station 222

As can be seen from Figure 4-22, the percent vertical deflection along the length of the pipe ranged from -1.39% to 2.26%. The percent horizontal deflection along the length of the pipe ranged from 0% up to 4.17%. The deflections as a percentage of the diameter for the 36 inch diameter PVC pipe are shown in Figure 4-23. Let it be noted that the nominal dimension for the HDPE pipes are the stated dimension of the pipe, while the nominal dimension for the PVC pipes is actually a half-inch less than the stated dimension for the 36 inch diameter pipes.

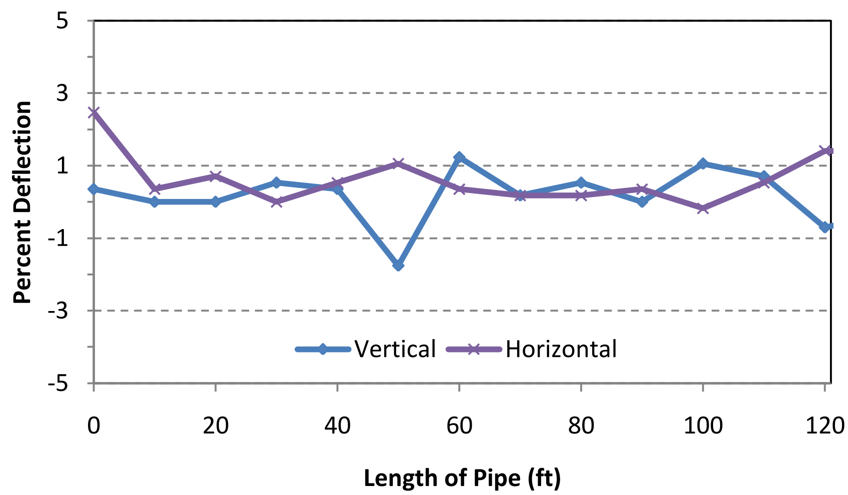


Figure 4-23: Percent Deflection for 36 inch PVC Pipe at Station 222

As can be seen from Figure 4-23, the percent vertical deflection along the length of the pipe ranged from -1.76% to 1.23%. The percent horizontal deflection along the length of the pipe ranged between -0.18% and 2.46%. No problems were encountered for this pipeline with infiltration/exfiltration, settlement, or erosion at the time of this inspection.

4.3.3.3 STA 223+00

The post-construction inspection for the pipe installed at station 223 was completed on January 30, 2011. This pipeline consisted of half a line of 36 inch diameter

HDPE pipe and half a line of 36 inch diameter PVC pipe. There were no major problems observed when inspecting this pipeline. The joint openings for the HDPE part of the line ranged from 1/4 inch up to 5/8 of an inch, which is well within tolerance. The joint openings for the PVC pipeline again could not be measured because of how the pipes come together, but all of the PVC joints were tight and no issues were found. The deflections as a percent of the nominal diameter for the 36 inch diameter HDPE pipe are shown in Figure 4-24.

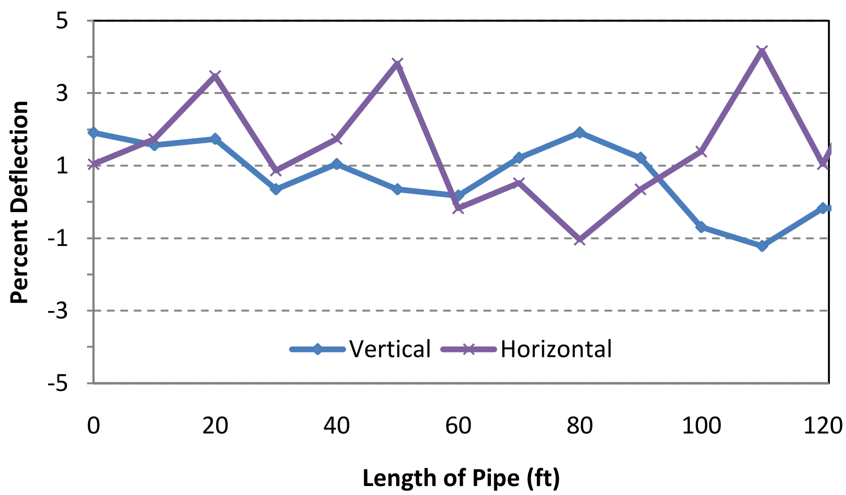


Figure 4-24: Percent Deflection for 36 inch HDPE Pipe at Station 223

As can be seen from Figure 4-24, the percent vertical deflection along the length of the pipe ranged from -1.22% to 1.91%. The percent horizontal deflection along the length of the pipe ranged from -1.04% up to 4.17%. The deflections as a percentage of the nominal diameter for the 36 inch diameter PVC pipe are shown in Figure 4-25.

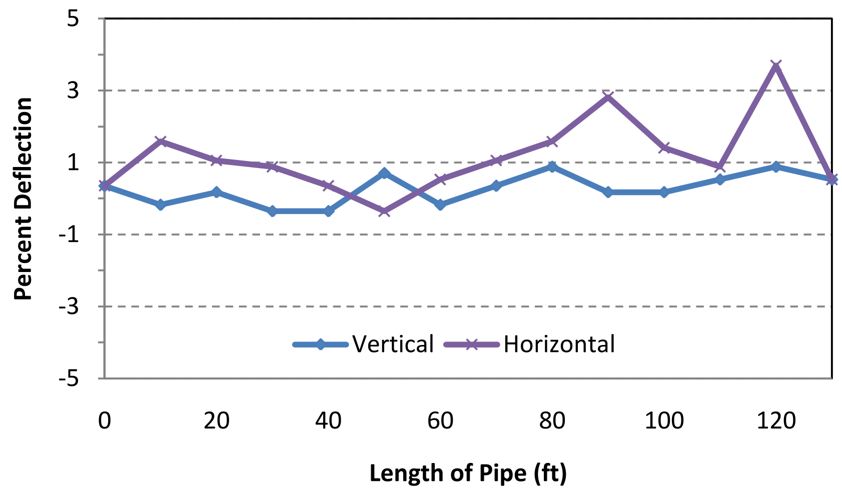


Figure 4-25: Percent Deflection for 36 inch PVC Pipe at Station 223

As can be seen from Figure 4-25, the percent vertical deflection along the length of the pipe ranged from -0.35% to 0.88%. The percent horizontal deflection along the length of the pipe ranged between -0.35% and 3.7%. Again, no problems were encountered for this pipeline with infiltration/exfiltration, settlement, or erosion at the time of this inspection.

4.3.3.4 STA 230+00 / STA 231+00

The post-construction inspection for the pipes installed at station 230 and 231 has not yet been completed. The delay in the inspection for these two pipelines has been caused by several factors including: 1) the pipes cannot be inspected with construction traffic crossing over the pipe, 2) weather conditions have prevented access, and 3) a layer of mud has covered the bottom of the pipelines. Once the pipes have been cleaned out, the post-construction inspection will be conducted on both these pipelines.

4.4 Conclusions and Future Works

Since this is a long-term study, complete conclusions cannot be made at this time. While long-term conclusions cannot be made at this time, the pipes seemed to initially be performing well. At this point in time, all of the pipes inspected had deflections that were less than the specified maximum deflection of 5% of the nominal inside pipe diameter. The pipe deflections will continue to be monitored to see the rate of change in the deflections over the life of the pipe. The experience gained through performing this field study showed that the quality of installation of thermoplastic pipes is much more critical than that of concrete pipes. It should be noted that the pipes installed during this field installation were carefully monitored by representatives of the pipe manufacturers.

Inspection of the pipes installed will continue over the coming months and years to evaluate the long-term performance of the plastic pipes installed. The long-term monitoring plan will be carried out with inspections that are similar to the post-construction inspections that have already been completed. In addition to checking deflections, the pipelines will also continue to be monitored with respect to infiltration/exfiltration, settlement, and erosion. One portion of the post-construction inspections that has not been completed is the mandrel testing. The mandrel testing was written into the project specifications and is to be completed by the contractor under the supervision of the project team, ALDOT representatives, and the plastic pipe manufacturers. In addition to the mandrel testing, the project team has been coordinating with the plastic pipe manufacturers to have the pipelines inspected using laser profiling. The laser profiling would give the deflections at all points along the length of the pipe and also show distortions of the pipe's cross-section along its length. Having the pipes laser profiled will give a clear view of how the pipes have been performing since their installation.

CHAPTER 5

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

5.1 Research Summary

The primary objective of this research project was to assess the use of high-density polyethylene (HDPE) and polyvinyl chloride (PVC) for use as cross-drains under highways and to assist the Alabama Department of Transportation (ALDOT) in developing a methodology for using plastic pipe. This objective was accomplished by completing three major phases that included a comprehensive literature review, an analytical study into the allowable fill heights for thermoplastic pipes, and finally a field study to observe the installation and performance of the pipe under in-service conditions.

The first phase was the literature review which consisted of introducing both HDPE and PVC, giving a brief history of each, giving performance limits and design procedures, providing a comprehensive review of the current practices for their installation and use, reviewing past research done in the field and laboratory, and examining current testing and quality control and quality assurance practices.

The second phase of the research effort focused on analytically developing minimum and maximum cover requirements for both HDPE and PVC pipe using the FE program ABAQUS. The analytical study for the minimum cover requirements investigated three distinct cases: (1) without the pavement under highway live loads, (2) with flexible or rigid pavements under highway live loads, and (3) with temporary fill under construction equipment loads. All of the analyses were carried out using the finite element method based on a 2D plane strain formulation, nonlinear soil model and parameters, time-dependent material properties of thermoplastics, and the geometric nonlinearity of the soil-pipe system.

The final phase of the research study consisted of a field study to monitor the installation and to evaluate the performance of the pipe in an in-service condition. The field study consisted of 5 trial installations that in addition to assessing the applicability and use of the pipes were used to evaluate the design and construction variables that affect thermoplastic pipes. Some of the variables evaluated include pipe diameters, fill heights, bedding and backfill materials, and acceptance testing methods and criteria. The monitoring program consisted of several site visits. The first site visit took place at the time of the pipe installation, and the second visit was taken approximately 30 days after the completion of the installation and final backfill. In addition to those inspections, a long-term condition monitoring program for the field trial installations was designed and will be implemented in order to begin accumulating long-term performance history.

5.2 Conclusions

The results of this study and an explanation on how each of the three phase's findings were used to make the following conclusions will be discussed in this section. The literature review laid the foundation with information about the materials, the design, the construction, the quality control, etc. of both the HDPE and PVC pipes. The literature review showed how the quality of installation determines how well a plastic pipe will perform. The conclusions made consisted of integrating information gained from the literature review, the analytical study, as well as the experience gained from the field study.

One important item to note is that thermoplastic pipes must be installed in a trench. If a pipe is to be installed in a fill, then the fill must be brought up a foot above the elevation of the top of the pipe before the trench is dug. The bedding for the pipe should be placed and compacted with a density of at least 95% of AASHTO T-99 maximum density. The center third of the bedding under the pipe shall then be loosened prior to placing the pipe. The placing and compaction of backfill for plastic pipe is much more critical than when installing concrete pipe, therefore there are special requirements that should be followed in order to ensure that the proper performance of the plastic pipe is achieved.

From the review of available literature, it was apparent that the quality of the installation was crucial for plastic pipe performance. Therefore, quality assurance testing after installation is one of the important elements for the specification of flexible pipes. Many states have adopted the use of a mandrel that has an effective diameter of 95% of the nominal diameter of the pipe being pulled through the entire length of pipeline by hand no sooner than 30 days after the completion of the installation and the completion of the embankment up to grade.

Several preliminary findings from this ongoing research project will now be discussed regarding minimum and maximum covers. Table 5-1 deals with minimum cover requirements for HDPE and PVC pipe. The minimum cover here is defined as the distance from the top of the buried pipe to the top of the finished roadway surface.

Table 5-1: Minimum Cover

HDPE Pipe	PVC Pipe
Minimum Cover, in. (Height of Fill)	Minimum Cover, in. (Height of Fill)
48	36

The preliminary findings suggest that the minimum fill heights for HDPE and PVC pipes should be 48 inches and 36 inches, respectively. These values are slightly higher than the minimum cover specified by the plastic pipe manufacturers, but these values agree with the upper range of values specified by other state DOTs. These values were based on the finite element analyses using conservative restraints. The values from the finite element analyses were based on using the H25 & Alternative loading while it is unknown how the minimum cover requirements were determined from other sources.

One of the most critical loading scenarios a pipe experiences can come during the construction phase. Construction loads are often higher than highway loadings, so it is important that an adequate amount of cover be supplied before allowing heavy construction equipment to cross over a pipeline. The preliminary findings of this research on the minimum cover requirements for heavy construction equipment loads are shown in Table 5-2.

Table 5-2: Minimum Cover for Heavy Construction Equipment Loads

	Minimum Cover, ft (Height of Fill)	
Nominal Pipe Diameter, in.	18 - 75 kips	75-150 kips
12-18	7	7
24-54	6	7

Again, these values are slightly higher than values given by the pipe manufacturers. These values were taken from the results of the finite element analyses using typical sized heavy construction equipment. The cover requirements for construction equipment loads are very important to ensure that the pipes are not damaged before beginning to perform their required function.

The preliminary findings from this research for maximum cover requirements are shown as Table 5-3.

Table 5-3: Maximum Cover

	Maximum Cover, ft (Height of Fill)		
Nominal Pipe Diameter, in.	Class I	Class II	Class III
0-18	43	20	15
24-54	31	14	10

The values in Table 5-3 are in a simplified form chosen conservatively off the pipe manufacturer's maximum fill height table (ADS 2010). Based on the research completed so far, this is considered a reasonable starting point maximum cover requirements. These values allow HDPE and PVC pipes to be installed at depths in the upper range of other state DOT specifications as long as certain levels of compaction and quality backfill materials are used. To achieve the level of performance required, all of the backfill materials shown in the table above must be compacted to at least 95% of AASHTO T-99 maximum density. The values found by completing the finite element analysis were higher than these values and therefore would be less conservative. The pipe manufacturer's maximum fill height table is based on calculations from AASHTO Section 12 assuming a unit weight of soil equal to 120 pcf and installation in accordance with ASTM D 2321. The maximum cover table is not divided between HDPE and PVC because the PVC manufacturer does not give complete information for fill heights broken down based on varying qualities of backfill.

It can be seen from the fill height requirement tables that the maximum pipe diameter shown is 54 inches. The FEA included pipe diameters up to 60 inches but it was chosen by ALDOT not to include pipe diameters above 54 inches because of current policies in place.

Minimum trench widths are important when installing plastic pipe, and based off the research conducted, Table 5-4 has been created.

Table 5-4: Minimum Trench Width

Nominal Pipe Diameter, in.	Minimum Trench Width, in.
12	35
15	40
18	45
24	55
30	65
36	75
42	85
48	95
54	105

The values for minimum trench widths shown in Table 5-4 are based on the most restrictive case given by either AASHTO Section 30 or ASTM D 2321. The clear distances needed when multiple pipes are installed in the same trench is another important installation requirement, and it is shown as Table 5-5. The values shown in Table 5-5 are based off recommendations made in PPI (Gabriel 2008). As with the trench width table (Table 5-4), these values are minimums and enough space should be granted to allow adequate room to safely operate compaction equipment between pipes as well as between the pipes and the trench wall.

Table 5-5: Clear Distances for Multiple Pipe Installations

Nominal Pipe Diameter, in.	Minimum Clear Distances Between Pipes And Between Pipes and Trench Walls, in.
12	12
15	12
18	14
24	18
30	18
36	20
42	22
48	24
54	26

5.3 Recommendations

The research conducted in conjunction with this project has shown that any DOT that specifies HDPE or PVC pipe should be note that a correct and quality installation of these pipes is critical for them to perform to their design capacity. This idea was shown time and again in the field study and performance reviews presented. Because of the critical nature of the installation, it is recommended that the initial implementation of plastic pipe for use in cross-drain applications should be carefully monitored. It should be noted that the trial installations were carefully installed and were installed under the supervision of representatives of the pipe manufacturers. While more long-term data is being collected from the field test, thought should be given to restrict the unlimited use of thermoplastic pipes likes some state DOTs have already incorporated in their specifications as found in a survey conducted by Jayawichrama et al. (2001). Such restrictions could include type of highway or by limiting the allowable Average Daily Traffic (ADT). These restrictions could be relaxed as more performance history has been judged satisfactory. The following are recommendations and suggestions based on the author's research:

- Continued monitoring of the 5 trial installations in order to generate long-term performance data.
- To validate and refine the finite element analyses, future work could include: using viscoelastic material properties and conducting creep analyses; developing 3-D meshes and models to verify the 2-D modeling approaches used; or using actual section properties rather than the estimated properties provided by AASHTO.
- A carefully design and controlled field test could be conducted to verify both the minimum and maximum fill height requirements found by the analytical study. If favorable results are found, then the specifications of both the minimum and maximum fill height requirements could be revisited and adjusted to match the findings of the study.
- With regards to current practices of state DOTs around the country, it is in the project team's opinion that a more up-to-date view could be obtained by creating a survey that could be sent to state DOTs for their feedback. Since many states have been doing research and updating their specifications and standard practices recently, it is suggested that in the future a survey similar to the one outlined in Appendix A should be conducted.

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APPENDIX A: SURVEY OF STATE DOTs

Survey of the Use of High Density Polyethylene (HDPE) and Polyvinyl Chloride (PVC) Pipes for Drainage Applications Under Roadways

Conducted by:
Department of Civil Engineering, Auburn University
238 Harbert Engineering Center
Auburn University, Alabama 36849-5337

STATE: ____

CONTACT PERSON: ____

TELEPHONE: _________________________ EMAIL: ____________________________

NOTE: All of the following questions deal with pipes installed as crossdrains underneath roadways.

1. Does your state currently use the following pipes as crossdrains?
   HDPE__________ PVC__________

2. For each pipe type selected above, what is the maximum diameter pipe allowed in your Agency for cross drains: HDPE___________ PVC_____________

3. What are your current restrictions on the <u>maximum</u> fill height for HDPE and PVC pipes? How were the <u>maximum</u> fill heights established (ex. Research, engineering judgment, etc.)? Please attach any relevant documentation.

4. What are you current restrictions on the <u>minimum</u> fill height for HDPE and PVC pipes? How were the <u>minimum</u> fill heights established (ex. Research, engineering judgment, etc.)? Are there additional requirements for minimum cover dealing with construction loads? Please attach any relevant documentation.

5. Are there any restrictions your Agency has on the use of plastic pipes under roadways (ex. ADT limitations, interstate use, etc.)?

6. Can you please fill out the following table with regards to Plastic Pipe performance observed by your Agency?

PLASTIC PIPE PERFORMANCE	**Frequent Problem**	**Occasional Problem**	**Not a Problem**	**Unsure**
Excessive deflections				
Joint Openings				
Wall cracking				
Fire hazard				
Degradation due to Abrasive Flows				
Others:				

7. Can you please fill out the following table dealing with installation difficulties of plastic pipe encountered by your Agency?

INSTALLATION DIFFICULTIES	**Frequent Problem**	**Occasional Problem**	**Not a Problem**	**Unsure**
Availability of qualified contractors				
Maintained proper line and grade				
Others:				

8. Does your state currently give requirements on the type of joints specified, for example "soil-tight" versus "water-tight" joints? If yes, please explain when each type of joint is required.

9. What does your Agency currently require for final inspections of Plastic Pipe?

   _____ Mandrel Testing
   _____ Video Inspection of pipe
   _____ Laser-profiling
   _____ Other:

   Please Explain:

10. What are your agencies requirements for deflection testing? When are deflections checked? Are there different requirements depending on the size of the pipe (ex. mandrel for small diameter pipe)?

11. Can you please help us obtain a copy (ex. pdf, hardcopy, link etc.) of your current Specifications for Plastic Pipe and any Special provisions that would apply to its use?

APPENDIX B: EXAMPLE CALCULATION OF CURRENT MAXIMUM COVER

REQUIREMENTS USING THE DESIGN PROCEDURE OUTLINED BY PPI

A 60-in corrugated HDPE pipe is proposed as a culvert. The fill height will be 18 ft. Backfill material will be the native soil which, in this situation, is categorized as a Class II (SW) material. Density of this material is 120 pcf. Minimum compaction will be 90% Standard Proctor Density. Calculate the wall stress, deflection, buckling, bending stress and bending strain and determine whether this pipe can endure the fill height given. The soil parameters and properties of HDPE pipe were taken from AASHTO LRFD Section 12.

1. Wall stress by thrust

$$T_{cr} = f_y A_s \phi_p \qquad \text{Equation B-1}$$

Where T_{cr} = critical wall stress; f_y = tensile strength for long-term conditions, 900 psi; A_s = section area, 0.538 in^2/in; and ϕ_p = capacity modification factor for HDPE pipe, 1.

Substituting:

$$T_{cr} = (900)(0.538)(1) = 484.2 \text{ lb/in.}$$

To check whether the calculated wall thrust is in excess of this value, use the following equation.

$$T = 1.3\left[1.5W_A\right]\left(\frac{OD}{2}\right)$$ Equation B-2

$$W_A = P_{sp}VAF = (15.51)(0.47) = 7.29\,\text{psi}$$ Equation B-3

$$VAF = 0.76 - 0.71\left(\frac{S_h - 1.17}{S_h + 2.92}\right) = 0.76 - 0.71\left(\frac{4.05 - 1.17}{4.05 + 2.92}\right) = 0.47$$ Equation B-4

$$S_h = \frac{\phi_s M_s R}{EA} = \frac{(0.9)(1{,}700)(31.37)}{(22{,}000)(0.538)} = 4.05$$ Equation B-5

$$P_{sp} = \gamma_s \frac{\left[H + 0.11(OD/12)\right]}{144} = 120\frac{\left[18 + 0.11(67.3/12)\right]}{144} = 15.51\,\text{psi}$$ Equation B-6

Where T = calculated wall thrust; W_A = soil arch load; P_{sp} = geostatic load; VAF = vertical arching factor; S_h = hoop stiffness factor; ϕ_s = capacity modification factor for soil, 0.9; M_s = secant constrained soil modulus, 1,700 psi; R = effective radius of pipe = ID/2+c; c = distance from inside diameter to neutral axis; E = long term modulus of elasticity of polyethylene, 22,000 psi; γ_s = soil density, 120pcf; H = fill height; OD = outside diameter.

Substituting:

$$T = 1.3\left[1.5W_A\right]\left(\frac{OD}{2}\right) = 1.3\left[1.5(7.29)\right]\left(\frac{67.3}{2}\right) = 478.33\text{ lb/in.} < T_{cr} = 484.2\,\text{lb/in.}$$

Wall stress is within limit. Design OK.

2. Deflection

$$\Delta_y = \frac{K\left(D_L\right)W_c}{0.149\,PS + 0.061E^{'}}$$

Equation B-7

$$W_c = \frac{H\left(\gamma_s\right)OD}{144} = \frac{18\left(120\right)67.3}{144} = 1{,}009.5 \text{ lb/in.}$$

Equation B-8

Where Δ_y = deflection; K = bedding constant, 0.1; D_L = deflection lag factor, 1.0; W_c = soil column load on pipe; PS = pipe stiffness, 14 psi from PPI design manual (*4*); $E^{'}$ = modulus of soil reaction, 1,700 psi.

Substituting:

$$\Delta_y = \frac{K\left(D_L\right)W_c}{0.149\,PS + 0.061E^{'}} = \frac{1.0\left(1\right)1{,}009.5}{0.149\left(14\right) + 0.061\left(1{,}700\right)} = 0.95 \text{in.}$$

$$\therefore \frac{\Delta_{\mathrm{y}}}{D} = \frac{0.95}{60} \times 100 = 1.58\% < 5\%$$

Deflection is well within 5% limit. Design OK.

3. Buckling

$$P_{cr} = \frac{0.772}{SF}\left[\frac{E^{'}PS}{1-\nu^{2}}\right]^{1/2}$$

Equation B-9

Where P_{cr} = critical buckling pressure; ν = poisson ratio; SF = safety factor, 2.0.

Substituting:

$$P_{cr} = \frac{0.772}{2}\left[\frac{(1{,}700)(14)}{1-0.4^{2}}\right]^{1/2} = 64.97\,\text{psi}$$

To check whether the actual buckling pressure is in excess of this value, use the following equation.

$$P_{V} = \frac{H\gamma_{s}}{144}$$

Equation B-10

Where P_{V} = actual buckling pressure.

Substituting:

$$P_{V} = \frac{(18)(120)}{144} = 15\,\text{psi} < P_{cr} = 64.97\,\text{psi}$$

Actual buckling pressure is less than allowable. Design OK.

4. Bending Stress

Bending stress should be less than the long term tensile stress, 900 psi.

$$\sigma_b = \frac{(2)\left(D_f\right)(E)\left(\Delta_y\right)(y_O)(SF)}{D_M^2}$$ Equation B-11

$$y_O = \text{the greater of } \frac{OD - D_M}{2} \text{ or } \frac{D_M - ID}{2}$$ Equation B-12

$$D_M = ID + 2c$$ Equation B-13

Where σ_b = bending stress; D_f = shape factor, 7.7 from PPI; E = modulus of elasticity of polyethylene, 22,000 psi; y_O = distance from centroid of pipe wall to the furthest surface of pipe; SF = safety factor, 1.5; D_M = mean pipe diameter; c = distance from inside diameter to neutral axis, 1.37in.

Substituting:

$$D_M = ID + 2c = 60 + 2\left(1.37\right) = 62.74\,\text{in.}$$

$$y_O = \text{the greater of } \frac{OD - D_M}{2} = \frac{67.3 - 62.74}{2} = 2.28 \text{ in.}$$

$$\text{or } \frac{D_M - ID}{2} = \frac{62.74 - 60}{2} = 1.27 \text{ in.}$$

$$= 2.28 \text{ in.}$$

$$\sigma_b = \frac{(2)(7.2)(22{,}000)(0.95)(2.28)(2)}{(62.74)^2} = 348.65 \text{ psi} < 900 \text{ psi}$$

Actual stress is less than allowable long-term tensile stress. Design OK.

5. Bending Strain

$$\varepsilon_B = \frac{(2)(D_f)(\Delta_y)(y_O)(SF)}{D_M^2} \qquad \text{Equation B-14}$$

Where ε_B = bending strain.

Substituting:

$$\varepsilon_B = \frac{(2)(7.2)(0.95)(2.28)(2)}{(62.74)^2} = 0.016 = 1.6\% < 5\%$$

Actual strain is less than allowable 5%. Design OK.

Conclusion:

This is a suitable application for 60 inch corrugated HDPE pipe. All criteria are within allowable values

APPENDIX C: Typical Installation for Pipe Backfilled with Stone

Figure C-1: Stone Backfill Installation at Station 224+00

Beginning of trench has been dug, and the line and grade instrument is being set up at the end of the pipeline.

Figure C-2: Stone Backfill Installation at Station 224+00

Bedding material is placed and leveled to set the grade of the pipeline.

Figure C-3: Stone Backfill Installation at Station 224+00

The first pipe is lowered into the trench using the excavator.

Figure C-4: Stone Backfill Installation at Station 224+00

Stone backfill being placed over the center of the pipe to prevent the pipe from rolling off line. The stone is also mounded on the first pipe in order to hold it in place while connecting the next pipe.

Figure C-5: Stone Backfill Installation at Station 224+00

The next pipe in the line is being lowered into the trench that has already had bedding placed and leveled.

Figure C-6: Stone Backfill Installation at Station 224+00

The spigot end of the next pipe is being lubricated in order for it to slide into the bell end of the pipe that was previously laid.

Figure C-7: Stone Backfill Installation at Station 224+00

The excavator is being used to slide the pipe into the bell of the previously laid pipe.

Figure C-8: Stone Backfill Installation at Station 224+00

The stone backfill is being carefully shoveled into the haunch region of the pipe and then being compacted.

Figure C-9: Stone Backfill Installation at Station 224+00

The vibratory plate compactor is being used to compact the #57 stone into place.

Figure C-10: Stone Backfill Installation at Station 224+00

The first lift of backfill has been compacted by receiving several passes with the plate compactor.

Figure C-11: Stone Backfill Installation at Station 224+00

The next lift has been placed and is waiting to be compacted.

Figure C-12: Stone Backfill Installation at Station 224+00

The final lift of backfill is being placed over the top of the pipe.

Figure C-13: Stone Backfill Installation at Station 224+00

The final lift of backfill that has been brought to approximately a foot above the top of the pipe is now being compacted with the plate compactor.

Figure C-14: Stone Backfill Installation at Station 224+00

Once the structural backfill has been brought up to a foot above the pipe, the native backfill is placed over the top.

Figure C-15: Stone Backfill Installation at Station 224+00

A sheep foot compactor was then used to compact the layer of soil backfill.

Figure C-16: Stone Backfill Installation at Station 224+00

A bulldozer was then used to push more backfill above the pipe and level with the existing embankment. From this point on, the fill above the pipe will be compacted just as a normal embankment.

APPENDIX D: Typical Installation Using a Granular Material

Figure D-1: Granular Backfill Installation at Station 223+00

The beginning of the trench has been dug, and the line and grade instrument is being set up at the end of the pipeline. Bedding material is being placed and leveled.

Figure D-2: Granular Backfill Installation at Station 223+00

The first pipe is set in place using the excavator, and then the alignment and grade is checked.

Figure D-3: Granular Backfill Installation at Station 223+00

The jumping jack tamp is being used to compact the first lift of backfill adjacent to the pipe.

Figure D-4: Granular Backfill Installation at Station 223+00

The bedding has already been graded, and the next pipe is being lowered in by the excavator.

Figure D-5: Granular Backfill Installation at Station 223+00

The loader is dumping the first lift of backfill over the center of the pipe to prevent the pipe from rolling off its line.

Figure D-6: Granular Backfill Installation at Station 223+00

The trench has been dug for the next piece of pipe, and the bedding material is being leveled to the correct grade.

Figure D-7: Granular Backfill Installation at Station 223+00

The excavator is being used to lower the pipe into the trench where the workers then lubricate the ends of the pipe.

Figure D-8: Granular Backfill Installation at Station 223+00

The excavator is being used to slide the pipe into the bell of the previously laid pipe.

Figure D-9: Granular Backfill Installation at Station 223+00

The first lift of backfill is being compacted with both the jumping jack tamp and the plate compactor.

Figure D-10: Granular Backfill Installation at Station 223+00

A representative of ALDOT is using a nuclear density gage to check the compaction of the backfill.

Figure D-11: Granular Backfill Installation at Station 223+00

The second lift of the backfill has been placed and is now being compacted using the jumping jack and the plate compactor.

Figure D-12: Granular Backfill Installation at Station 223+00

The next lift of soil is being placed by the excavator and is waiting to be compacted.

Figure D-13: Granular Backfill Installation at Station 223+00

This lift of backfill is being compacted using the jumping jack tamp and the plate compactor.

Figure D-14: Granular Backfill Installation at Station 223+00

The final lift of backfill is being placed by the excavator, and it is brought to approximately a foot above the top of the pipe.

Figure D-15: Granular Backfill Installation at Station 223+00

The final lift of structural backfill for this pipeline was compacted using the bucket of the excavator.

Figure D-16: Granular Backfill Installation at Station 223+00

A bulldozer was then used to push more backfill above the pipe and level with the existing embankment. From this point on, the fill above the pipe will be compacted just as a normal embankment.