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16. Abstract

This research developed new specifications for the use of large diameter (36 in to 48 in) HDPE pipe in TxDOT construction projects. One of the primary tasks in the project was to identify backfill materials that would provide both reliability and economy at the same time. The research plan for accomplishing this task included: a) survey of other State DOT practices, b) monitoring of several HDPE pipe installation projects, c) a constructability review, d) an economic analysis, and e) full-scale field load testing. Three types of backfill were selected based on the findings from this study: a) granular backfill, b) cement stabilized backfill, and c) flowable fill. Among these, granular backfill provides the best economy. The specified gradation band for granular backfill was selected such that good reliability can be achieved in pipe installations without the need for elaborate quality control measures. Cement stabilized backfill and flowable backfill are much less economical but may be used to meet special construction needs. The proposed specifications also developed maximum fill height and minimum cover criteria.

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Evaluation of Backfill Materials and Installation Methods for High Density Polyethlene Pipe

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Report Number: 1809-3

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IMPLEMENATATION STATEMENT

This research developed specifications for the use of large diameter high density polyethylene, or HDPE pipe in TxDOT construction projects. The primary considerations in the development of the above specifications were first, the reliability and secondly, economy. The proposed specifications are based on: (a) other state DOT practices, (b) data collected from actual installations and (c) field load tests on pipe. Specifications allow the use of three types of backfill materials, granular backfill, cement stabilized backfill and flowable fill. Among the 3 backfill material options granular backfill will provide the best economy. The gradation specifications for granular backfill has been selected so that good pipe performance will be achieved with minimum quality control during installation. Specifications all provide maximum fill heights and minimum cover to protect the pipe from vehicular loading. Proposed specifications will be ready for implementation once it is approved by the TxDOT Specifications Committee.

Prepared in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

AUTHOR DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view of policies of the Texas Department of Transportation. This report does not constitute a standard, specification, or regulation.

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ENGINEERING DISCLAIMER

Not intended for construction, bidding or permit process.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate

(Revised September 1993)

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER I
INTRODUCTION

1.1 Use of HDPE Pipe in Highway Drainage Applications

Traditionally, the Texas Department of Transportation (TxDOT) has used concrete or corrugated metal pipes in highway drainage applications. In the recent times however, large-diameter High Density Polyethylene (HDPE) thermoplastic pipes have emerged in the marketplace as a viable alternative. However, these pipes require more care and control during installation. Because of this reason, and due to the lack of statewide experience with this type of pipe, TxDOT has taken a conservative approach and specified flowable backfill as the only acceptable type of backfill material for thermoplastic pipe. Flowable backfill, which is a mixture of sand, cement, fly ash and water, is considerably more expensive than conventional pipe backfill materials such as sand and gravel. As a result when flowable backfill is used, HDPE pipe is no longer an economically attractive option. Therefore, this research project was initiated with the primary objective of investigating the viability of using alternative backfill materials such as granular materials in HDPE pipe installations. This report documents the work accomplished in and the findings from the above research project.

Thermoplastic pipe offers many advantages over metal and concrete pipes. HDPE pipe is 40% less heavy than steel pipe and 90% less heavy than concrete pipe and therefore can be transported and handled at lower costs (*1*). Therefore, price of HDPE pipe, delivered at the job site, is significantly lower when compared with that of reinforced concrete and corrugated metal pipes. Also, HDPE is manufactured in longer length and therefore, fewer joints are required in a given pipe installation. This and the ease of handling of HDPE pipe allow faster installation of the pipe. Moreover, according to HDPE pipe manufacturers' literature, HDPE pipe is chemically inert and therefore, can be used in both acidic and alkali soils.

On the other hand, there are some drawbacks in the use of HDPE pipe as well. HDPE pipe falls into the category of flexible pipe and, therefore can undergo relatively large deflections under load. As the pipe deforms, it mobilizes more support from the surrounding backfill. Unlike rigid pipes, that carry nearly all of the load by itself, the flexible pipes rely on the support from the surrounding backfill to withstand the load. Therefore, the quality of the pipe embedment materials plays a key role in the performance of HDPE pipe and good quality control measures are needed during installation. In fact, construction considerations may have more influence on deflection performance than the embedment type, in situ soil stiffness or pipe stiffness (*2*). If excessive deflection does occur as a result of inadequate pipe backfill support, that may in turn lead to loss of joint integrity and constriction of flow. The dependence of HDPE pipe on the installation quality is a drawback because good quality control cannot always be ensured during routine installations.

Another concern with the use of HDPE pipe is its tendency for flotation during installation when flowable fill is used. For this reason, some state highway agencies do not allow the use of flowable fill with HDPE pipe. Also, care must be taken to avoid the installation of this type of pipe when water is present in the pipe trench. The combustibility of HDPE pipe has also been a concern although there have not been many reports of pipe combustion in actual field installations.

Finally, it must be noted that the technology and material used in the manufacture of HDPE pipe has seen significant change over the time period during this type of pipe has been in use. Therefore, our knowledge of the long-term performance of HDPE pipe is limited compared to that of reinforced concrete and corrugated metal pipes. While this does not necessarily reflect negatively on HDPE pipe, it necessitates a more conservative approach to be taken during the design and installation of this type of pipe.

1.2 Research Objectives

The primary objective of this research was to develop specifications to allow the use of large diameter HDPE pipe in TxDOT construction projects. Such specifications and construction guidelines have also been developed by other agencies such as AASHTO and ASTM. However, these procedures require the design of each pipe system on a project by project basis by considering project specific design parameters such as pipe stiffness, moment of inertia, soil and loading conditions. It is not common practice in Departments of Transportation (DOTs) to perform such customized designs. Therefore, to get the optimum benefit from the use of HDPE pipe, more generalized design procedures that can be applied over broad range of soil and loading conditions must be developed. Therefore, the primary objective of this research was to come up such specifications and construction guidelines.

The requirements for the new standard specifications are as follows: (a) it should ensure integrity of the pipe system during both their installation and long-term service, (b) the specifications should only require backfill material that is readily available statewide, (c) it must ensure cost effectiveness of the pipe system, and (d) it must be applicable to the broad range of climatic and native soil conditions that is found in Texas. One of the specific objectives of the research was to develop specifications for backfill type and compaction in such a manner that density checks, backfill compaction control, or pipe deflection measurements to verify proper installation would not be needed.

1.3 Research Methodology

The general research approach used was as follows. At the outset, a comprehensive survey of the pertinent background information on the use of thermoplastic pipe in gravity flow drainage applications was conducted. This effort included two separate tasks. First, a survey was carried out among all State DOTs. Secondly a comprehensive survey of published literature was carried out. Based on the above, the current state of practice with regard to the use of thermoplastic pipe in transportation applications was established. First, specifications used currently by state DOTs and other highway agencies for the installation of buried thermoplastic pipe were reviewed. Secondly long term and short term experience with HDPE pipe was evaluated. Based on the above information, draft specifications were developed. Subsequently, several construction projects were carried out throughout the state of Texas. The projects were monitored to establish the effectiveness and any shortcomings of the draft specifications. However, as all types of backfill and compaction could not be duplicated in the field test projects, full scale load testing of buried HDPE pipe was carried out in a test facility that was specially constructed for that purpose. Thereafter, the data from the full scale tests were combined with finite element analyses to evaluate the strength of different types of backfill under different levels of compaction. This was accomplished by back calculating the strength of the backfill from the loads-deflection data collected during each full scale load test. Knowing the mechanical properties of the backfill, guidelines were developed for maximum fill heights and minimum cover required using a probabilistic approach.

1.4 Organization of the Report

Chapter II of this report contains the findings from the literature review that was carried out at the beginning of the research project. It contains information on the following:

(a) Mechanical and chemical properties of HDPE
(b) Materials currently used to backfill HDPE pipe
(c) ASTM and AASHTO recommendations for the installation of HDPE pipe
(d) Criteria for the design of HDPE pipe such as allowable stress and allowable strain
(e) Current methods of design of flexible pipe installations

Chapter III contains the findings from the State DOT survey. Based on the findings from the above tasks, a draft specification for the installation of large diameter HDPE pipe was developed. This draft specification can be found in Appendix A of this report. It was developed based on guidance available through specifications developed by other agencies such as AASHTO, ASTM and other state DOTs as well as data collected from experimental work conducted in this research. Chapter IV contains a description of the full-scale field load testing program that was carried out by the researchers. Under the testing program two different series of tests were carried out. The first series of tests was carried out to evaluate the effectiveness of different compaction equipment and compaction effort when compacting granular materials. The second series of tests was carried out to determine the performance of buried HDPE pipe backfilled with several different types of backfill and compacted using alternative equipment and effort.

Chapter V documents the findings from pilot constructions projects. In this task, eight TxDOT pipe installation projects were selected for the installation of large diameter (36-48 inch diameter) HDPE pipe. These eight projects were monitored to determine pipe performance both during and after installation. Chapter VI describes the application of constructibility review concepts to evaluate the practicality of the draft specification that was developed previously in this research project. The primary objective of constructibility review was to examine the draft specification from a constructibility viewpoint and hence identify any elements in the specifications that may create difficulties during its field implementation.

In Chapter VII, HDPE pipe is compared with reinforced concrete pipe and corrugated metal pipe in terms of material and installation costs. This chapter documents data collected on prices of different pipe products and backfill materials in various parts of Texas. It also presents findings from a detailed economic analysis that was conducted to compare as-installed costs of HDPE pipe versus RCP when pipe installation is performed according to TxDOT specifications. It examines the influence of pipe material (HDPE vs.

RCP), pipe diameter and backfill material price on overall project cost by varying each parameter within the complete range of values found within Texas. Several useful conclusions were drawn based on the findings from the above parametric study.

Chapter VIII contains a description of the finite element analysis that was carried out to determine the properties of the backfill types tested in the full-scale field tests. Results from the finite element analysis can then be used to determine the performance of buried HDPE pipe under several installation and loading conditions. The reliability of an installation can be evaluated using a statistical and probabilistic approach.

Chapter IX contains conclusions and recommendations stemming from the research described herein.

The proposed final specification is found in Appendix C.

CHAPTER II
LITERATURE REVIEW

This chapter presents the findings from review of relevant background literature. It begins with a brief overview of flexile pipe behavior under loads. This overview is followed by a discussion on mechanical properties of high density polyethylene as well as AASHTO and ASTM specifications for this type of pipe. Subsequent sections of this chapter deal with backfill material requirements, modes of pipe failure, design criteria and existing design procedures for thermoplastic pipe.

2.1 Use of Flexible Pipe for Gravity Flow Drainage

Flexible pipe, by definition, is pipe that can undergo deflections of at least two percent of pipe diameter before any structural distress such as rupture or cracking of the pipe occurs (*3*). Rigid pipes, made of cast iron, asbestos cement, concrete, and clay, on the other hand, are more brittle and hence do not show similar large deformation before failure occurs by cracking and rupture (*4*). Figure 2.1 shows a buried pipe and the terminology used to describe different zones of backfill (*5*). If the pipe zone and the haunch area backfill has not been well compacted in a rigid pipe installation, the load above the pipe crown will generate excessive stresses on the pipe, thus leading to failure of the pipe by rupture. Under similar circumstances, flexible pipe made of Polypropylene, HDPE, and Poly vinyl chloride will undergo much larger deformations. Larger deformations allow mobilization of support from surrounding soil backfill. The support generated from the soil as the pipe deforms attenuates any concentrated loading, and minimizes the strain on the pipe (*6*). This phenomenon is illustrated in Figure 2.2. Therefore, the failure in flexible pipe is more likely to be controlled by excessive deflection and serviceability problems resulting from that (joint failure, constriction of flow etc.) rather than material rupture.

2.2 Material Properties of HDPE

Corrugated plastic tubing was developed in Europe and introduced to the United States in 1967 (*7*). They helped in overcoming a number of problems associated with the use of conventional materials, such as corrosion, exfiltration and infiltration (*4*). However, HDPE pipe's pimary advantages are its light weight and good mechanical performance (*4*). Pipe with profiled walls offers greater ring stiffness and buckling strength than pipe with solid walls. Most large diameter HDPE pipe is manufactured with profiled walls, thus generating savings in material and transportation costs.

High Density Polyethylene is a type of plastic, and as such, is a polymer made up of long chain molecules that contain a large amount of carbon atoms. HDPE, a thermoplastic, softens when heated, and is formed into pipe by extruding. HDPE is resistant to corrosion, biodegradation and chemical attack (*8*).

2.2.1 Cell Classification

AASHTO M 294M-98 (*9*), which covers AASHTO requirements and test methods for polyethylene pipe, specifies the cell class requirements for HDPE. The material that the pipe is manufactured with, must meet the criteria set forth in ASTM D-3350 "Standard Specification for Polyethylene Plastics Pipe and Fittings Materials" (*10*). The cell class classification is done by ascertaining the following properties of the material with which the pipe is manufactured meets specified criteria.

(a) Density (ASTM D 1505)
(b) Melt Index (ASTM D 1238)
(c) Flexural Modulus (ASTM D 790)
(d) Tensile Strength at Yield (ASTM D 638)
(e) Environmental Stress Crack Resistance (ASTM D 1693)
(f) Hydrostatic Design Basis (ASTM D 2837)

The cell classification specified according to AASHTO 294 M – 98 is 335420C with the exceptions that the carbon black content shall not exceed 5 percent and the density shall not be less than 0.945 g/cm^3 (0.546 oz/in.^{3}) nor greater than 0.955 g/cm^3 (0.552 oz/in.^{3}). This is a higher cell classification than the previous classification of 324420C specified in AASHTO M 294-94.

2.2.2 Stress Strain Behavior

HDPE has stress relaxation properties under constant stress (*7*). When deformation of the pipe increases, more and more load is transferred to the sidefill resulting in relaxation of stress and strain in the pipe (*8*). The backfill acts as an arch, similar to a masonry arch. HDPE has stress/strain relationships that are nonlinear and time dependent. The stress that can be applied continuously for fifty years without rupture is called the fifty-year tensile strength. This is calculated from hydrostatic design basis. The modulus of the material is also specified as initial and fifty year. The lower fifty-year modulus does not indicate a softening of the pipe but the time dependency of the stress strain relationship. Short-term deflections will be governed by the initial modulus regardless of the age of the installation. AASTHO Section 18 in *Standard Specifications for Highway Bridges* (*11*) states that long term modulus should be used for the analysis of buckling. The choice of an appropriate modulus for other types of analysis, however, is left to the discretion of the design engineer. The minimum values of short-term (initial) and long-term (fifty year) moduli, as specified in AASHTO Section 18, are given in Table 2.1. Because of stress relaxation and load distribution effects, the stress levels in underground flexible pipe is low, and the linear part of the stress strain curves can be used (*4*). For HDPE, the creep modulus for fifty years is approximately 29,000 to 43,000 psi and the short term values are 116,000 to 145,000 psi (*4*).

In HDPE, which is considered a viscoelastic material, the modulus appears to diminish with constant stress or constant strain. However, this loss of modulus is due to using a linear elastic model to fit a viscoelastic material. There has been some discussion amongst engineers as to what modulus should be used when there is a combination of a short-term and a long-term load. This type of loading occurs when a pipe is buried and then is subjected to vehicular loading acting at the surface level. The principle of superposition is applicable to thermoplastics in the linear viscoelastic range. Linear viscoelastic materials exhibits the same response to an increment of stress or strain regardless of the previous history of stress or strain (*12*). Some characteristics of linear viscoelastic materials are shown in Figure 2.3 (adapted from *12*).

2.2.3 Ultraviolet Degradation

HDPE undergoes degradation when exposed to ultraviolet radiation. Exposure to ultraviolet radiation initially increases the tensile strength and decreases the ductility, and eventually causes a decrease in the tensile strength. To reduce the loss in the pipe's strength resulting from exposure to ultraviolet rays, a small quantity of carbon black is added during the manufacture of HDPE pipe. According to HDPE pipe manufacturers, present products treated with carbon black can withstand several decades of exposure to sunlight without significant distress.

2.3 Backfill Materials

The deflection of a flexible pipe is highly dependent on the stiffness of the soil embedment. The pipe deflection is controlled by the stiffness of the material up to 2.5 pipe diameters from the pipe side wall. Therefore, both the stiffness of the backfill inside the pipe trench and the stiffness of the in-situ soil material immediately outside the trench are important. If the in-situ soil is stiffer than the backfill material, it is desirable to keep the trench as narrow as possible. This situation occurs if the in-situ soil consists of rock such as claystone, mudstone, or siltstone, and soils with densities that are greater than that of the backfill (*13*). For installations where the in situ soil is soft, the stiffness of the backfill and the amount (or thickness) of backfill between the pipe and the trench wall must be sufficiently large to provide necessary support.

2.3.1 AASHTO Recommendations for Backfill Materials

AASHTO Section 18, which contain specifications for the installation of thermoplastic pipe (*11*) specifies that backfill used shall be granular material that is free of organics, frozen lumps and stones larger than 1.5in. in greatest dimension. It further specifies that the backfill moisture content shall be in the range of optimum (typically -3% to +2%) and that backfill shall be compacted to a minimum of 90% standard density as established by AASHTO T 99. The compaction requirements are such that the soil envelope will have a soil modulus (E') in excess of 1700 psi. According to AASHTO Section 30 (*14*), bedding should have a maximum particle size of 1.25 in. and meet the requirements of AASHTO M 145, A-1 or A-3. Backfill for thermoplastic pipe should be granular material that are free of organic material, stones larger than 1.5 in. in greatest dimension, or frozen lumps, and the moisture content should be in the range of optimum (typically –3% to +2%) permitting thorough compaction.

AASHTO specifications for the installation of corrugated metal pipe (*15*) require granular material compacted to a minimum of 90% standard density as established by AASHTO T 99. AASHTO specifications for the installation of concrete pipe (*16*) require granular material or fine material, depending upon the type of the installation. Compaction is controlled by specifying the density to be achieved as a percentage of standard density, varying according to the type of installation.

2.3.2 ASTM Specifications for Backfill Materials.

Specifications for the underground installation of thermoplastic pipe are also found in ASTM D 2321-89 (*17*). ASTM D 2321 classifies all potential bedding and backfill materials into four classes (I, II, III and IV). Among these, backfill material classes I and II consist of non-plastic, granular soils whereas classes III and IV consist of fine-grained soils with plasticity. Since the primary focus in this paper is on granular backfill that is generally accepted for use as backfill for large-diameter thermoplastic pipe subjected to vehicular loading, at this point, attention is devoted on ASTM backfill material classes I and II only. Table 2.2 summarizes the specifications for these materials. Table 2.3 presents the ASTM D 2321 recommendations for their use as embedment and backfill materials in pipe installation.

Based on the information given in this table it is evident that, once again, the basis for compaction control used in ASTM D 2321 is the moisture-density relationship of the soil. ASTM D 2321 requires 85% maximum standard Proctor density when granular materials belonging to Classes I (I-A and I-B) or II are used as embedment materials. The compaction requirement has been formulated to provide a soil envelope with a minimum soil modulus (E') of 1000 psi.

2.3.3 Limiting Migration of Fines

Hydraulic flow may cause fines to migrate from a fine grained material into a coarse grained material placed adjacent to it. Significant loss of pipe support has been attributed to migration in several cases. Migration is of importance where significant groundwater flow is expected. Methods can be employed to impede migration such as the use of an appropriate stone filter or filter fabric in the interface between the fine and coarse material. ASTM D 2321 suggests the following filter gradation criteria to preclude migration.

1. $D_{15}/d_{85} < 5$ where D_{15} is the sieve opening size passing 50% by weight of the coarser material and d_{85} is the sieve opening size passing 85% by weight of the finer material.
2. $D_{50}/d_{50} < 25$ where D_{50} is the opening sieve size passing 50% by weight of the coarser material and d_{50} is the sieve opening size passing 50% by weight of the finer material. This criterion need not apply if the coarser material is well graded.
3. If the finer material is a medium to high plasticity clay without sand or silt partings (CL or CH), then the following criterion may be used in lieu of (1). $D_{15} < 0.02$ inches where D_{15} is the sieve opening size passing 15% by weight of the coarser material.

2.3.4 Maximum Particle Size

Maximum particle size should be limited because oversize particles can abrade the pipe during the placement of the backfill or cause point loads on the pipe. According to ASTM D2321, the maximum particle size for embedment should be limited to material passing a 1.5 in. sieve. Smaller pipe may require smaller maximum particle sizes. AASHTO Section 30 recommends that bedding should be 1.25 in. in maximum particle size. Pipe with corrugated exteriors should be backfilled with material that allows the filling of the corrugation valley.

2.3.5 Alternative Backfill Materials

Controlled low strength mortar (CLSM), also referred to as controlled density fill (CDF), may be used for backfill and bedding provided adequate flotation resistance can be achieved by restraints, weighting or placement techniques. With CLSM backfill, trench width can be reduced because this type of backfill does not require compaction. Flowable backfill, a mixture of sand, cement, fly ash, and water is occasionally used in installations. Upon hardening, flowable fill provides excellent support for the pipe. Cement stabilized materials are used in situations where extra support of the pipe is needed, e.g. Pipe installation with shallow cover.

All of the above alternative materials are significantly more expensive than granular materials, and therefore, the use of granular materials is the preferred whenever possible.

2.4 Design Criteria

Flexible conduits are less likely to fail by rupture, cracking or crushing (*4*). Plastic materials, especially HDPE, endure deformation to the point of total collapse without cracking or rupture. Deflection of a pipe is defined as the reduction in diameter from the nominal due to construction and dead loads, divided by the nominal diameter, expressed as a percentage. Buckling failure and excessive bending strains do not occur until the deflections in the pipe are about thirty percent (*4*). Therefore, the limiting deflection used in the design of thermoplastic pipe installations is not controlled by material or construction failure, but rather by factors such as geometry, flow characteristics, and joint integrity. According to European and American field experience, deflections of 5 to 7.5 percent can be tolerated without detrimental effects to the functioning and joints of the pipe (*4*). In an analysis of the failure of many pipe installations, Bjöklund and Janson found that most failures in HDPE pipe occur at the joints (*18*).

Smooth wall plastic pipe were used before large diameter corrugated wall plastic pipe superceded them (*6*). Smooth walls were suitable for pipe of smaller diameter. However, corrugated walls were more suitable for pipe of larger diameters because of the greater buckling stiffness and ring bending stiffness that they provide. Buckling stiffness and ring bending stiffness is critical for pipe of larger diameter, and the cost of building smooth wall pipe of sufficient thickness to satisfy these structural requirements would be prohibitive. By using corrugated pipe walls instead of smooth walls, the stiffness of the pipe can be increased threefold (*4*). Furthermore, handling, transportation and installation costs can be reduced by manufacturing lighter pipe sections. Installation of light pipe sections can be accomplished easily, with smaller construction equipment. Corrugating pipe walls gives great longitudinal flexibility to pipe, thus assuring that no high stresses are developed due to longitudinal bending. Since corrugated plastic pipe adjusts to the bedding, beam design for installations is not considered necessary (*7*).

2.4.1 Modes of Distress in HDPE Pipe

The modes of distress that must be considered in the design of thermoplastic pipe include: (a) Excessive deflection, (b) Wall buckling, (c) Wall crushing and (d) Excessive wall strain.

Excessive deflection may disturb the integrity of the pipe joints and cause leaks. Large deflections may also cause loss of pavement support or lead to the restriction in the use of standard size pipe cleaning equipment.

Wall buckling indicates that the pipe stiffness is not adequate. It may govern the design when the pipe is subjected to internal vacuum, external hydrostatic pressure, or high soil pressures in compacted soil.

Wall crushing occurs when the in-wall ring compression stress reaches the yield stress of the pipe material. Wall crushing is likely to be limiting only for stiffer plastic pipes installed in highly compacted backfill and subject to deep cover.

Excessive wall strain could be due to a bending strain, ring compression strain, or hoop strain; in gravity flow pipe, bending is the largest component. Figure 2.4 shows the these four major forms of distress.

Formation of breaks or networks of fine breaks in the liner that is visible to the unaided eye is called "liner cracking or crazing." A "rupture" is a break extending through or partially through the wall. "Wall cracking" is the formation of a break in the wall visible to the unaided eye. "Wall delamination" is the separation of sections of the pipe wall that is visible to the unaided eye. The above definitions are as given in ASTM D 2412 (*19*).

<u>2.4.2 Control of Deflection</u>

Excessive deflection may affect the joint integrity of pipes with gaskets at their joints. Large deflections also sometimes cause settlement of the overlying pavement. Moreover, deflections can be used as a method for determining the stress strain conditions within the pipe wall; large deflections are indicative of large stresses and strains within the pipe which may indicate that collapse is imminent. Therefore, it is desirable to limit and monitor pipe deflections within an installation.

It is extremely important that construction deflections are taken into account, as they are highly variable and difficult to predict (*3*). In the authors' experience, HDPE pipe decrease in the horizontal diameter and increase in the vertical diameter during compaction of the backfill. This deformation aids significantly in the reduction of deflection during service life, where typically the vertical diameter reduces and the horizontal diameter increases. The vertical diameter of a flexible pipe can increase by as much as five percent during the placing and compaction of the backfill. It has commonly been proposed that the deflection limit for flexible pipe buried in earth should be between 5% and 10%. This limit is intended to provide a factor of safety against failure by collapse at a deflection of about twenty percent (*3*). However, since soil exhibits nonlinear stress strain characteristics, the factor of safety is actually much higher (*20*).

Flexible pipe may deflect in one of two possible modes; either elliptically or rectangularly (*21, 22*). Elliptical deformation occurs when parallel plate or three edge testing is carried out, and theoretically, the horizontal deflection is 91.3% of the vertical deflection. Elliptical deformation occurs if the soil stiffness is low. If the soil stiffness is high, rectangular deformation occurs, and the horizontal deflection can be as low as twenty percent of the vertical deflection (Figure 2.5).

Long-term deflections in an HDPE pipe installation with properly compacted embedment do not exceed the initial deflection by more than fifty percent (*4*). Therefore, it is possible to design and test for initial deflections only, with the assurance that long term deflections will be within the required tolerance (*4*).

2.4.3 Pipe Stiffness

Pipe stiffness is usually considered to be the total vertical load applied to a pipe segment divided by its length and then by the vertical deformation of the pipe under the load.

$$\text{Pipe Stiffness} = \frac{\text{Load}}{(\text{Length of Pipe Segment}) \times \text{Vertical Deflection}} \tag{2.1}$$

The "stiffness" of HDPE pipe is measured by using the parallel plate test (See Figure 2.6). Pipe stiffness gives an indication of the pipe's resistance to bending deformation and also acts as a quality control measure in the manufacturing process (*22*). A minimum pipe stiffness is required for the integrity of an installation as well as the ease of construction of a pipeline (*19*).

Generally thin circular ring or shell analysis is used to interpret the results of the parallel plate tests but HDPE pipe is not strictly "thin" because its profile may be a significant fraction of the diameter. If plane stress conditions are assumed, the relationship between the pipe modulus E, the pipe radius r, the second moment of inertia I, and the pipe stiffness PS, can be derived from the thin elastic ring representation. The corresponding pipe stiffness, PS is given by the following formula (*22*).

$$PS = (EI/r^3)\,(\pi/4 - 2/\pi)^{-1} \tag{2.2}$$

However, results from tests indicate that the above equation is over simplified. The reasons for the discrepancy are the viscoelastic nature of HDPE pipe, the non-linearity of the deformations and the significant depth of the HDPE profile (*23*). Because of the viscoelastic nature of HDPE, the deformation rate of the parallel plate test has to be kept at a standardized value. ASTM D 2412, which gives a standard test method for the parallel plate load test, specifies that the upper plate move down at a constant rate of 0.50 ± 0.02 inches per minute.

Related to pipe stiffness is the "stiffness factor, SF," which is the product of E and I. The stiffness factor had widespread usage in the early stages of development of the theory of flexible pipe (*19*).

Pipe stiffness does not significantly affect pipe deflection in most cases as the contribution from the soil stiffness is much higher (*21*).

2.4.4 Tensile Strength of HDPE Pipe

According to AASHTO Section 18, the allowable stress for wall thrust should be minimum tensile strength, divided by a safety factor. AASHTO Section 18 suggests a factor of two for the service load design method. Table 2.1 gives the AASHTO recommendations for the minimum tensile strength of HDPE pipe material.

However, researchers observed that the stresses in HDPE diminish at a faster rate at constant strain than the rate which the strength diminishes (*24*). This phenomenon is significant in designing installations for long term loads.

2.4.5 Allowable Strain in HDPE Pipe

The extreme fiber tensile strength of the pipe wall may determine the allowable deflection in an HDPE pipe. According to AASHTO Section 18, the allowable long-term strain of 5% for polyethylene should be checked against the tension strain in a pipe that has deflected significantly. Compression thrust should be deducted from deflection bending stress to obtain net tension action. AASHTO Section 18 states that the allowable long-term strain given above should not be reached in pipe designed and constructed in accordance with AASHTO Section 18.

2.4.6 Handling and Installation Rigidity

Pipe may suffer damage during handling as well as during installation. AASHTO Section 18 specifies that handling and installation rigidity be measured by a flexibility factor, FF, determined by the formula given below. The flexibility factor, according to the specifications, must be 9.5×10^{-2}.

$$FF = (D_e)^2/EI \tag{2.3}$$

where

- FF = Flexibility factor, in./lb,
- D_e = Effective diameter, in.,
- E = Modulus of elasticity of the pipe material, lb/in.^{2},
- I = Average moment of inertia per unit length of cross section of the pipe wall in.^{4}/ in.

2.4.7 Jointing Systems

Several jointing systems are available, such as thermal welding, elastomeric gaskets, and chemical adhesives (*4*). Stiffness and strength of joints in plastic pipe vary significantly from the strength and stiffness of its barrel (*5*). The joints of HDPE pipe is typically specified as water-tight or soil-tight. Water-tightness is a more stringent requirement than soil-tightness in joints. HDPE pipe with water-tight joints are now being manufactured by major plastic pipe manufacturing companies. Soil-tight, silt-tight, leak-resistant and water-tight jointing systems are available in the marketplace. ASTM standards D3212 (*25*) and F 477 (*26*) are used for the specifications for water-tight joints and elastomeric seals respectively. AASHTO MP6-95 (*27*) and AASHTO M294 (*9*) do not require that the pipe be water-tight. Instead they state that the type of joint must be selected to ensure that soil infiltration into the pipe does not occur. These specifications refer to AASHTO *Standard Specifications for Highway Bridges*, Division II, Section 26 "Metal Culverts" (*16*) for criteria on soil tightness. The new AASHTO Section 30 "Thermoplastic Pipe: Construction/Installation also gives criteria on soil-tightness and water-tightness.

2.5 Methods of Design of Flexible Pipe Installations

2.5.1 Iowa Formula

The strain that a structural element undergoes is related to the stress it is under and the modulus of elasticity of the material it is made of. The modulus of elasticity of most materials is available, and can be found by laboratory experimentation. In buried flexible pipe installations, the pipe response is determined by the stiffness of the pipe-soil system.

$$\text{Pipe deflection} = \frac{\text{Load}}{\text{Pipe Stiffness} + \text{Soil Stiffness}} \tag{2.4}$$

The Iowa Formula, which is used to predict deflections in flexible pipe installations, has the above general form. The Iowa Formula was developed by Professor M.G. Spangler of Iowa State University who published a landmark paper describing a design procedure for the underground installation of flexible pipe in 1941(*28*). Spangler and Watkins later modified the formula to include a more realistic value for the soil parameter. The modified Iowa formula (*21*) is given as:

$$X = \frac{D_l KW}{EI/R^3 + 0.061E'} \tag{2.5}$$

where

- X = horizontal deflection of the pipe, in.,
- D_l = deflection lag factor to compensate for the volume change of soil with time, dimensionless,
- K = bedding constant, which varies with the angle of bedding, dimensionless,
- W = load on the pipe per unit length, lb/in.,
- EI = pipe wall stiffness per unit length, lb in.,
- E' = modulus of soil reaction, lb/in.^{2},

or,

$$X = \frac{\text{Load Factor}}{\text{Ring Stiffness Factor} + \text{Soil Stiffness Factor}} \tag{2.6}$$

The parameters used in the Iowa Formula are explained below.

(a) *Load Factor $D_l KW$* – The load factor incorporates the parameters that have to do with the magnitude and distribution of the soil pressures on a buried pipe. Changes in construction procedures or bedding materials along a pipeline could significantly vary the load factor.

(b) *Deflection lag factor D_l* – Loaded soil continues to reduce in volume with time. The conversion of immediate deflection to long term deflection is accomplished by the use of the deflection lag factor. Spangler recommends a value of 1.5 for D_l. However, the value varies according to the time the initial deflection was measured, the volume change rate of the soil, and the load the soil is under. It has been found that the deflection lag factor can range from 1.0 to 6.

(c) *Bedding constant, K*– The bedding constant can vary from 0.083 to 0.110 depending on the shaping of the bedding for placement of the pipe. It has been observed that as the angle of bedding increases, the pipe deflects less due to the increase in the loaded area. Most investigators use 0.1 as a typical value.

(d) *Load on the pipe, W* – The Marston theory is commonly used for calculating the load on the pipe and is recommended by Spangler for the Iowa Formula. According to the Marston theory, the load used in design depends on whether the pipe is a trench or embankment installation and the backfill soil, amongst other factors. In recent years the load on the pipe has been considered to be the weight of the column of the earth directly above the pipe.

(e) *The ring stiffness* EI/R^3 – This is considered to have very limited effect on the pipe deflection because the soil stiffness factor is relatively much larger. Therefore the use of the nominal values for E, I and R is generally considered sufficient for use with the Iowa Formula.

(f) *Soil Stiffness Factor* $0.061E'$ – E′ was considered to be a pipe-soil interaction modulus by Spangler and Watkins. A constant E′ = 700 psi was considered representative for soils over 90% of their maximum laboratory dry density. However, Spangler later recommended that experience and judgment be used in selecting values of E′. Chambers and McGrath recommend that E′ be taken as 3000 psi for Class 1 Material (according to ASTM D2321) compacted to a density greater than 85% standard Proctor density and Class 2 materials compacted to greater than 95% standard Proctor density. For Class 2 material compacted to between 85% and 95% standard Proctor density, they recommend that a value of 2000 psi be assigned to E′ (*5*).

According to ASTM D 2412, the deflection given by the Iowa formula is both the vertical and horizontal deflections. However, data from field tests and installations carried out under roads in this research indicated that the vertical deflections are consistently significantly greater than the horizontal deflections.

2.5.2 USBR Equation

The USBR (United States Bureau of Reclamation) equation can also be used to predict deflections of buried flexible pipe (*3*). This equation was developed purely on an empirical basis by considering many field installations. However, the equation can be used only to predict the deflection due to the dead load on the pipe. The equation is:

$$\Delta Y(\%) = T_f\left(\frac{0.07\,\gamma h}{EI/R^3 + S_f D_f} + C_f\right) + I_f \qquad (2.7)$$

where,

$\Delta Y(\%)$ = percent vertical deflection,

T_f = time-lag factor, dimensionless,

0.07 = combination of conversion factors and bedding constant, ft^2/in^2,

γ = backfill density, lb/ft^3,
h = depth of cover, ft,
EI/R^3 = pipe stiffness factor, lb/in^2,
S_f = soil stiffness factor, lb/in^2,
D_f = design factor, dimensionless,
C_f = construction factor, percent vertical deflection,
I_f = inspection factor, percent vertical deflection.

The prism load should be used with the equation. The equation can be used only when the sidewall support is as stiff or stiffer than the bedding material. McGrath found that the soil prism load gives conservative results. He argued that the assumption of pure creep and constant load is conservative and actual behavior is a mixture of creep and relaxation (*7*).

Because of the great variability in the loads that act on a pipe and the soil stiffness, the use of nominal values for the values of E, I and R is acceptable in the USBR equation.

Values of the Soil Stiffness Factor, S_f, are suggested by Howard (*3*) according to the type of material and level of compaction. The values were back-calculated using data from over 120 installations, and then finding the cases which best represent the type of soil and compaction effort. The Design Factor D_f, also varies according to the level of compaction and type of soil.

The Time-lag Factor, T_f, is used to take into account the increase of deflection in a buried pipe with time. The increase is caused by two factors; one is the increase in the soil load on the pipe, and the other is the consolidation of the soil next to the pipe (*3*). Spangler showed that the load on a pipe does not peak until months or even years after the installation has been completed. Deflections continue even after the maximum deflection has been reached, just as in the settlement of footings (*3*). Spangler found a maximum deflection lag of 1.5. When the same data was evaluated to find time-lag factors, T_f ranged from 1.5 to 2.0.

The Construction Factor, C_f, is incorporated to take into account the inherent variability in the deflections that occur in a pipeline. There are indications that a variation of two percent from the average can be expected, but deflections of three percent from the average have occurred (*3*). There are no instances where there is more than $\pm$ 1 percent where the backfill material has been highly compacted coarse grained soils with little or no fines. C_f, decreases with higher compaction and with higher quality backfill.

The Inspection Factor, I_f, is placed in the equation to take into account the quality of inspection at the time of installation. A low value of I_f, is used if qualified personnel are continuously available to assert that the contractor is compacting the bedding according to specification, density checks carried out, and deflection measurements taken. On the other hand, if control over the practices of the contractor is not maintained, and density and deflection measurements are not taken, a higher value of I_f, is recommended.

A comparison was carried out to find the accuracy of the USBR equation in predicting deflections by considering data from actual field constructions (*3*). The average long-term deflections calculated using the USBR equation were higher than the actual deflections in ninety percent of the cases studied. Ninety-five percent of the predicted initial maximum deflections were higher than the actual maximum deflections reported from the field. All the values that were calculated for the long-term maximum deflections using the USBR equation were higher than the values reported from the field.

2.5.3 The Watkins' and Gaube's Method

In this method, the ratio R_s of the soil modulus to the pipe stiffness factor is calculated first.

$$R_s = \frac{E_{soil}}{E_{pipe}\dfrac{I}{D^3}} \tag{2.8}$$

Then the ratio between pipe deflection, δ_v, and soil deformation, ε_b, is found from Figure 2.7 which is a result of theoretical and experimental work carried out by Watkins and continued by Gaube (*29, 30*). Subsequently ε_B is found from

$$\varepsilon_B = \frac{P_v}{E_{soil}} \tag{2.9}$$

where,

P_v = load acting on the pipe, $\gamma \,.\, H + \psi \,.\, p_l$ (psi)
γ = unit weight of soil, (psi),
H = height of soil cover
ψ = impact factor of traffic load (see Reference *30*)
p_l = traffic load effective on pipe's top level (see Reference *31*)

Since the ratio between δ_v and ε_B is known (the ratio was derived from Figure 2.7), δ_v can be calculated. The soil moduli for various soil materials and levels of compaction as derived from reference (*30*) is presented in Table 2.4.

2.5.4 Design for Hydrostatic Pressure

HDPE pipe becomes unstable at about 10% deflection under external hydrostatic loading, as does all flexible pipe. The buckling or collapse occurs after the application of a critical hydrostatic pressure (dependent on pipe dimensions and properties) over a period of time (*32*). The hydrostatic pressure under which a flexible pipe collapses is given by the equation

$$P = \frac{(1 - \mu^2)\, R^3}{3\, EIC} \tag{2.10}$$

where

P = critical buckling pressure
E = modulus of elasticity of the pipe material,
μ = Poisson's ratio of the pipe material,
I = moment of inertia of the cross section,
R = mean radius of the pipe,
C = reduction factor for out of roundness,

as given by to Timoshenko and others (*34*). Allman (*34*) gives an equation for C which he derived based on experiments on polyethylene pipe obtained from a single manufacturer.

$$C = \left(\frac{D_{\min}}{D_{\max}} \right)^{4.62} \tag{2.11}$$

The above equations can be used only when the pipe is unconstrained, i.e., surrounded by liquid only. In most underground installations, the pipe has benefit from the surrounding soil envelope. Therefore, the hydrostatic pressure which causes buckling of buried pipe is higher than given by the above two equations. Allman (35) suggests the following equation for the analysis of a soil surrounded pipe under hydrostatic pressure.

$$P_k = 0.67\,(E'P)^{0.5} \tag{2.12}$$

where

P_k = Buckling pressure in constrained soil,

P = Unconstrained buckling pressure from the previous equations,

E′ = Tangent modulus of the soil medium.

Table 2.1. Tensile Strength and Modulus Values Specified by AASHTO Section 18.

Initial		50 Year	
Minimum Tensile Strength (psi)	Minimum Modulus of Elasticity	Minimum Tensile Strength (psi)	Minimum Modulus of Elasticity (psi)
3000	110,000	900	22,000

Table 2.2. ASTM Specifications for Backfill Material Classes I and II.

Soil Class	Type	Soil Group Symbol D2487	Description	Percent Passing Sieve Sizes			Atterberg Limits		Coefficients	
				1 1/2 in. (40mm)	No. 4 (4.75mm)	No. 200 (0.075 mm)	LL	PI	Uni-formity Cu	Curva-ture Cc
IA	Manufactured Aggregates: Open-graded, clean.	None	Angular, crushed stone or rock, crushed gravel, broken coral, crushed slag, cinders or shells; large void content, contain little or no fines	100 %	≤10 %	<5 %	Non Plastic			
IB	Manufactured, Processed Aggregates; dense-graded, clean.	None	Angular, crushed stone (or other Class IA materials) and stone/sand mixtures with gradations selected to minimize migration of adjacent soils; contain little or no fines	100 %	≤50 %	<5 %	Non Plastic			
Class II	Coarse-Grained Soils, clean	GW	Well-graded gravels and gravel-sand mixtures; little or no fines	100 %	<50 % of "Coarse Fraction"	<5 %	Non Plastic		>4	1 to 3
		GP	Poorly-graded gravels and gravel-sand mixtures; little or no fines						<4	<1or>3
		SW	Well-graded sands and gravelly sands; little or no fines		>50 % of "Coarse Fraction"				>6	1 to 3
		SP	Poorly-graded sands and Gravelly sands; little or no fines						<6	<1or>3
	Coarse-Grained Soils, Borderline clean to w/fines	e.g. GW-GC, SP-SM	Sands and gravels which are Borderline between clean and with fines	100 %	Varies	5 % to 12 %	Non Plastic		Same as for GW,GP ,SW and SP	

Table 2.3. ASTM D 2321 Recommendations for Embedment and Backfill Materials of Classes I and II.

Soil Class (see Table 1)	Class IA	Class IB	Class II
General recommendations and restrictions	Do not use where conditions may cause migration of fines from adjacent soil and loss of pipe support. Suitable for use in a drainage blanket and underdrain in rock cuts where adjacent material is suitably graded.	Process materials as required to obtain gradation which will minimize migration of adjacent materials. Suitable for use as drainage blanket and underdrain.	Where hydraulic gradient exists check gradation to minimize migration. "Clean" groups suitable for use as drain-age blanket and underdrain
Foundation	Suitable as foundation and for replacing over excavated and unstable trench bottom as restricted above. Install and compact in 6-inch maximum layers.	Suitable as foundation and for replacing over excavated and unstable trench bottom . Install and compact in 6-inch maximum layers.	Suitable as foundation and for replacing over excavated and unstable trench bottom as restricted above. Install and compact in 6-inch maximum layers.
Bedding	Suitable as restricted above. Install in 6-in maximum layers. Level final grade by hand. Minimum depth 4 in (6 in. in rock cuts)	Install and compact in 6-in maximum layers. Level final grade by hand. Minimum depth 4 in (6 in. in rock cuts)	Suitable as restricted above. Install in 6-in maximum layers. Level final grade by hand. Minimum depth 4 in (6 in. in rock cuts)
Haunching	Suitable as restricted above. Install in 6-in. maximum layers. Work in around pipe by hand to provide uniform support	Install and compact in 6-in. maximum layers. Work in around pipe by hand to provide uniform support	Suitable as restricted above. Install and compact in 6-in. maximum layers. Work in around pipe by hand to provide uniform support
Initial Backfill	Suitable as restricted above. Install to a minimum of 6in. above pipe crown	Install and compact to a minimum of 6 in above pipe crown.	Suitable as restricted above. Install to a minimum of 6in. above pipe crown
Embedment Compaction [A]	Place and work by hand to ensure all excavated voids and haunch areas are filled. For high densities use vibratory compactors	Minimum density 85% std. Proctor.[B] Use hand tampers or vibratory compactors	Minimum density 85% std. Proctor.[B] Use hand tampers or vibratory compactors
Final Backfill	Compact as required by the engineer	Compact as required by the engineer	Compact as required by the engineer

[A] When using mechanical compactors avoid contact with the pipe. When compacting over pipe crown maintain a minimum of 6 in. cover when using small mechanical compactors. When using larger compactors maintain minimum clearance as required by the engineer.

[B] The minimum densities given in the table are intended as the compaction requirements for obtaining satisfactory embedment stiffness in most installation conditions

Table 2.4. Soil Moduli for Various Soil Types and Compaction Efforts *(30)*.

Soil Class	Specific Weight (lbs / ft^3)	Soil Modulus, E_{soil}, (psi) for various proctor density values, D_P,					
	D_P	85%	90%	92%	95%	97%	100%
1	127	360	870	1300	2300	3300	5800
2	127	175	435	580	1150	1600	2900
3	127	115	290	435	725	1150	2025
4	127	85	220	290	580	875	1450

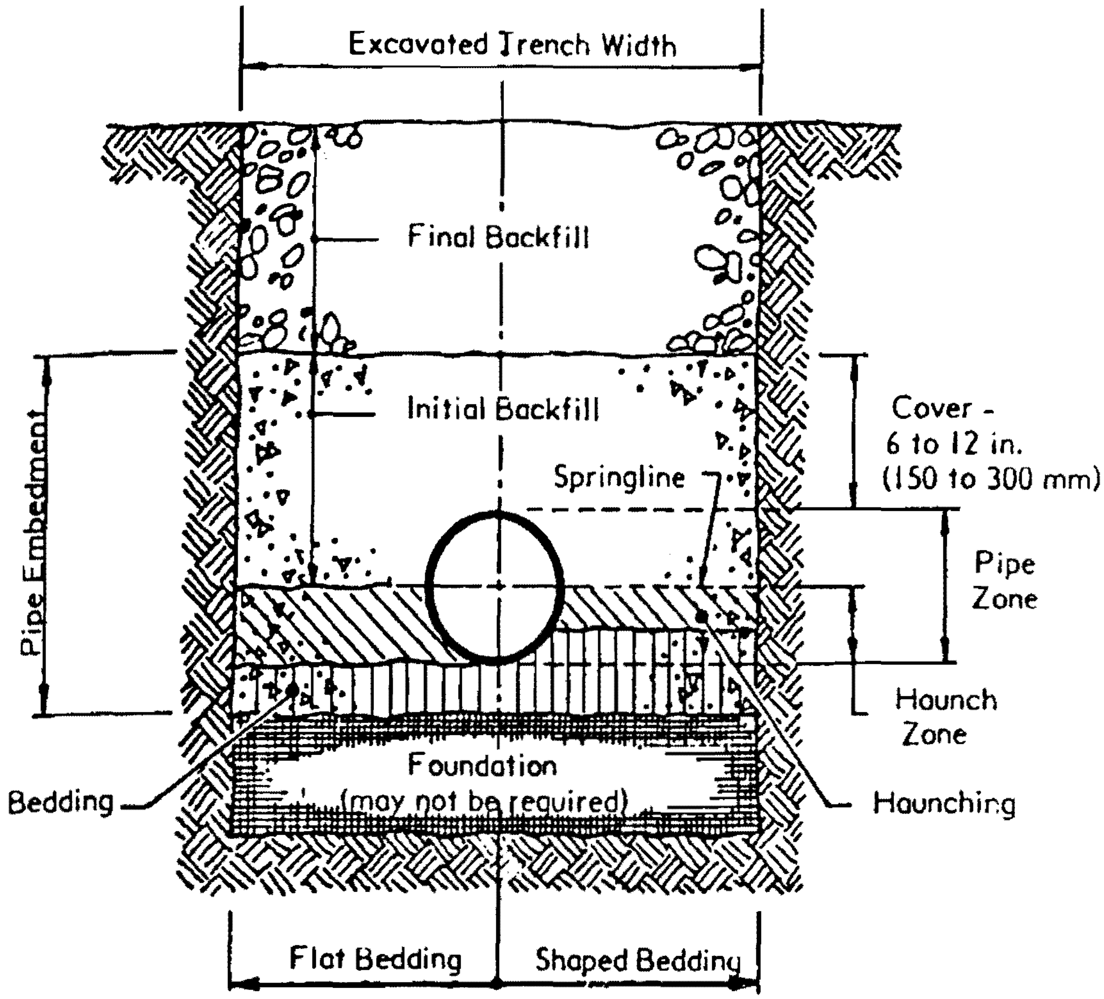


Figure 2.1. Typical Cross Section of Trench Installation (5).

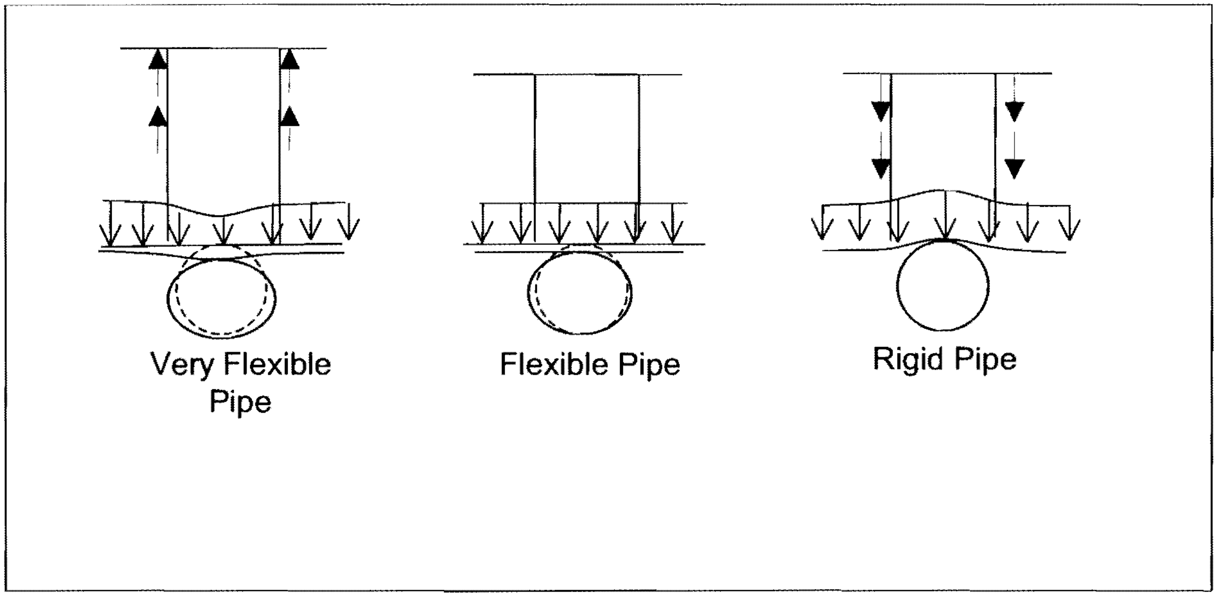


Figure 2.2. Influence of the Ratio Pipe Stiffness/Soil Stiffness on the Vertical Soil Pressure (adapted from *6*).

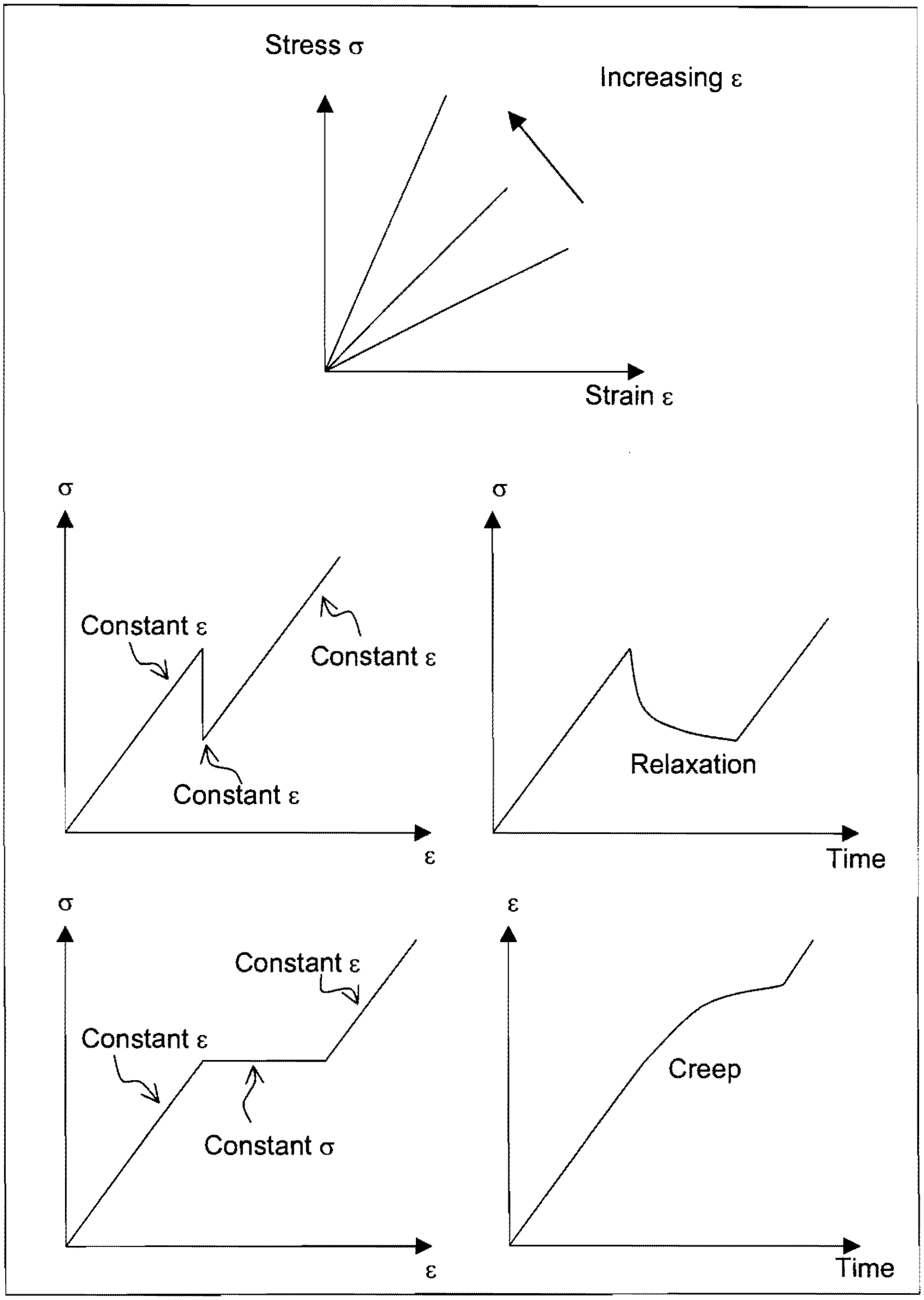


Figure 2.3. Characteristics of Linear Viscoelastic Materials (*12*)

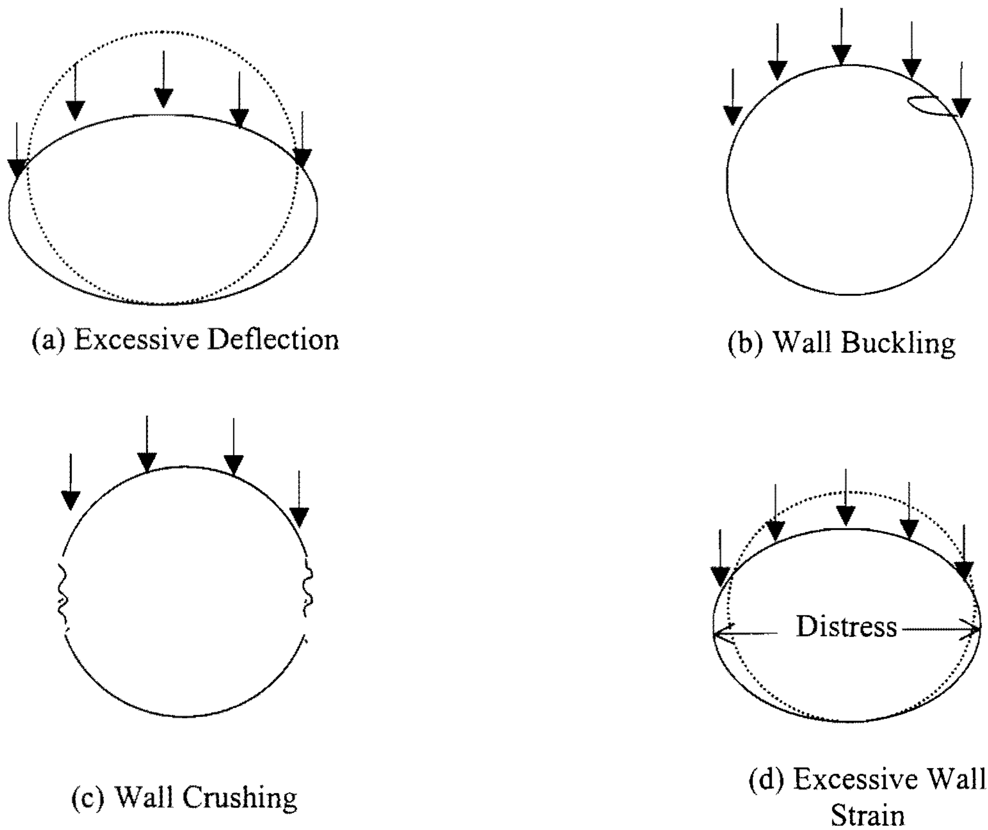
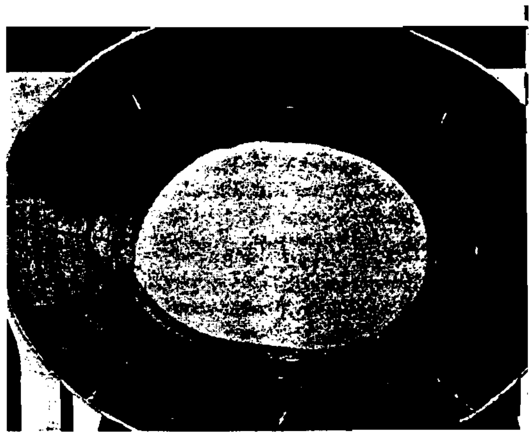
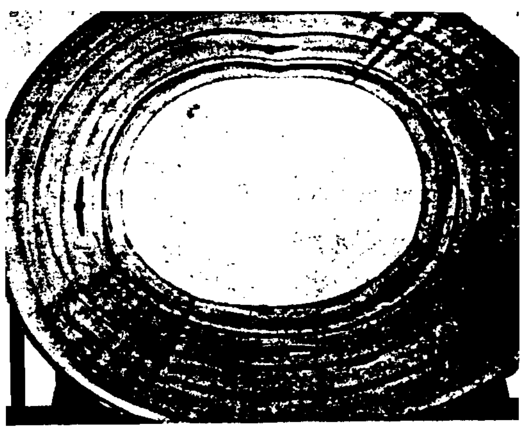


Figure 2.4. Modes of Distress of HDPE Pipe.



(a) Elliptical Deformation



(b) Rectangular Deformation

Figure 2.5. Elliptical and Rectangular Deformation of Flexible Pipe (*21*).

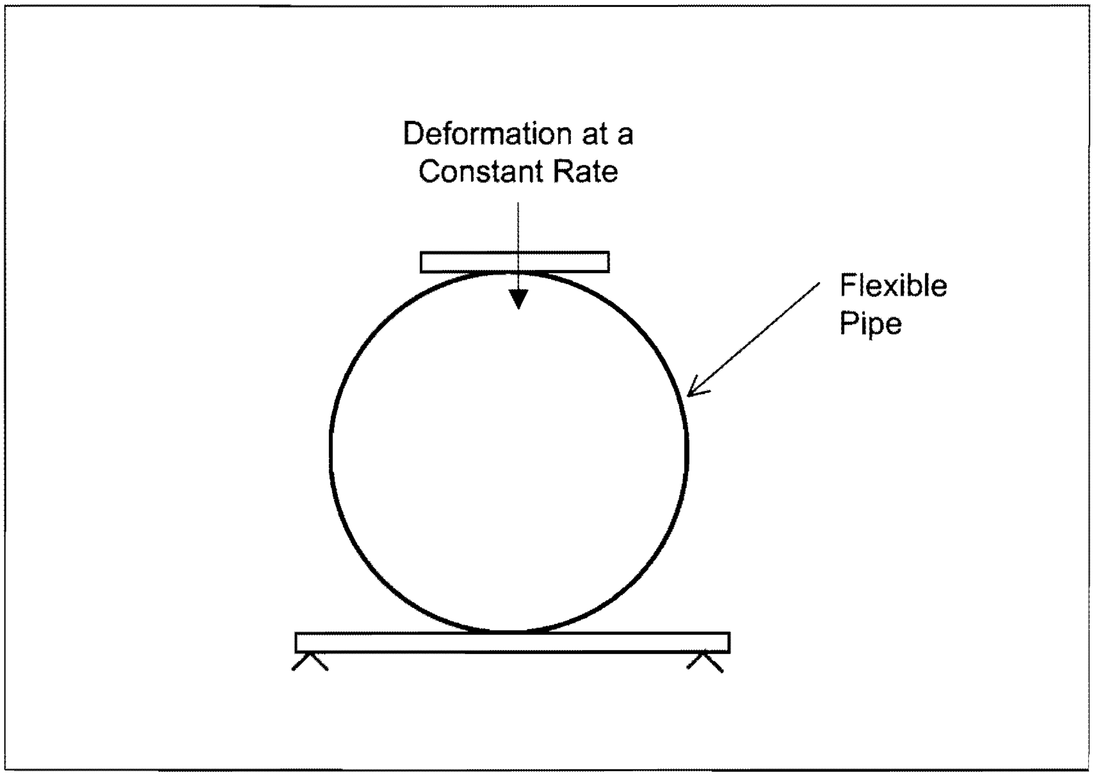
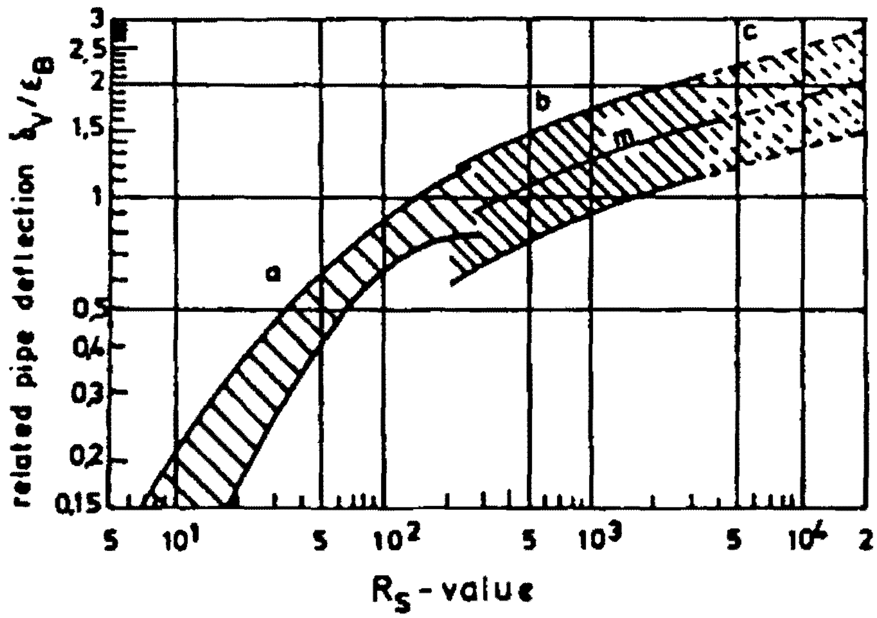


Figure 2.6. Parallel Plate Test Methodology.



Note: a- range of flexible steel pipes, b-range of PVC and PE pipes, c-extrapolated range, m- mean curve

Figure 2.7. Watkins' Diagram Extended to the Plastic Pipe Range *(29, 30)*.

CHAPTER III
USE OF LARGE DIAMETER HDPE PIPES IN HIGHWAY CONSTRUCTION: CURRENT STATE OF PRACTICE

The researchers performed a nationwide survey to document the current state of practice in the use of thermoplastic pipe. This survey was conducted during the time period between September 1997 and March 1998. Transportation officials in all 50 state DOTs with expertise in design and installation of pipe systems were contacted. A preliminary telephone survey was carried out to develop an understanding of the critical issues involved. Based on the findings from the telephone survey, a questionnaire was developed and sent to all fifty states. Figure 3.1 shows the above questionnaire. After the responses to the questionnaires were received and analyzed, further information on several points of interest was collected through a second round of telephone calls.

3.1 General Overview of Survey Findings

3.1.1 Experience with HDPE Pipe

Out of the 50 state DOTs contacted 32 responded to the questionnaire. Of the state DOTs that responded, 22 included sections from their specifications that were relevant to the installation of HDPE pipe. Eighteen out of 32 states have used large diameter HDPE pipe for subsurface gravity flow drainage for between five and ten years. Six states have used HDPE pipe for less than five years and eight states for more than ten years. The number of years that each state DOT has used HDPE pipe is given in Table 3.1. The same information is presented in Figure 3.2 in summary form.

3.1.2 Specifications Used in HDPE Pipe Design and Installation

AASHTO and ASTM have developed specifications for the design of pipe installations and for pipe materials. Most states depend on AASHTO *Standard Specifications for Highway Bridges* Section 18: Soil Thermoplastic Pipe Interaction Systems for the structural design of pipe installations (*11*). This specification relates the loading on the pipe and the backfill characteristics to pipe design. Some states have amended the AASHTO specification to suit their specific requirements and the most common amendment involves the minimum cover for the pipe. The minimum cell classification of the pipe material according to Section 18 is now 335420C as defined in ASTM D 3350 (*10*). This is a higher cell classification than the previous one. In their own specifications some state DOTs have specified cell classifications that are higher than the new 335420C while other states have specified cell classifications that are lower.

Most state DOTs use AASHTO M294: Corrugated Polyethylene Pipe 12 in. to 36 in. diameter for pipe material and properties (*9*). AASHTO MP 6-95: Corrugated Polyethylene pipe, 42 in. and 48 in. was in use for HDPE pipe of diameter 42 in. and 48 in. (*26*). In the latest AASHTO specification, pipes with diameters 42 in. and 48 in. have been included in AASHTO M294. AASHTO MP7-97 was introduced for pipe diameters 54 in.

and 60 in. Even though some states specify water-tight joints, AASHTO M294 and AASHTO MP 6-95 require only soil-tight joints.

ASTM D2321, "Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity Flow Applications" (*17*) is used as a guideline for the installation procedures by some states. There were favorable comments from state DOTs regarding this specification. AASHTO and ASTM specifications are evolving rapidly as technological advances are made.

A summary of specifications used by each state DOT is found in Table 2.1.

3.1.3 General Assessment of HDPE Pipe

Most state DOTs that have used large-diameter HDPE pipe in their subsurface drainage applications indicated that HDPE pipe systems have provided good performance. A few pointed out problems associated with finding a qualified contractor and with line and grade maintenance. There appears to be a general consensus that HDPE will perform well if appropriate precautions are taken during installation to prevent the pipe being disturbed by the construction traffic.

3.2 Significant Issues Identified

3.2.1 Applications

Twenty-five state DOTs indicated that they use HDPE pipe for culverts as well as side drainage while four other state DOTs use HDPE for side drainage only. The term, "culverts," in the present context refers to pipes that provide cross drainage underneath the pavement. They run in perpendicular to the direction of travel. The term, "side drains," on the other hand, refers to pipes that are installed outside the pavement and run parallel to the direction of travel. Most of the states that use HDPE culverts use them on both primary highways and side roads. Only three states specify that HDPE pipe used for culverts should only be used under minor cross roads and driveways.

Although, as indicated above, many state DOTs allow HDPE pipe for culverts some have imposed other restrictions that may limit the use of HDPE pipe in the above application. One state DOT specifies that HDPE culverts should not be used on interstate highways. Seven other state DOTs specify maximum ADT on the roadway in which this type of pipe may be used for culverts. The maximum allowable ADT limits used by different state DOTs vary from 250 to 1700. One state specifies the usage of HDPE according to the Design Hourly Volume (DHV) of the road (DHV smaller than or equal to 200). Table 3.2 presents HDPE pipe applications and corresponding ADT restrictions for all of those states that responded to the questionnaire.

3.2.2 Pipe Diameter

Table 3.3. shows the number of state DOTs that allow HDPE pipe of a given maximum diameter for each of the three different applications: side drainage, culverts under entrance roads and culverts under primary highway. It can be seen that, at the present time, most states use only up to 36 in. diameter HDPE pipe.

3.2.3 Backfill Material

As mentioned previously proper backfill support is crucial to the successful performance of the pipe. If native soil is used it is important that the backfill material is properly compacted to the desired density around the pipe. However, proper compaction of the backfill in the haunch area is difficult to achieve. Therefore, some state DOTs have relied on flowable backfill to ensure adequate pipe support. Flowable backfill is a mixture of sand, cement, fly ash and water. It does not require compaction but gains strength upon hardening. Survey responses revealed that backfill material specifications vary significantly from one state to another.

Seventeen states allow native soil as backfill for HDPE pipe. Eighteen states allow select backfill that usually consisted of granular material. Some of the commonly used select backfill materials are as follows.

a. Sand or well graded granular material,

b. A-1, A-2, A-3 according to AASHTO classification,

c. Granular material with 100% passing 1.5 in. sieve, <5% passing No200 sieve, PI=0,

d. Processed aggregate,

e. Stone screenings,

f. Granular backfill passing a 1 in. sieve,

g. Crushed stone.

Fifteen states allow flowable backfill. Most states allow more than one type of backfill material. The type of backfill material that may be used in a given installation is selected based on additional criteria. For example, some states use flowable backfill when the cover over the pipe is limited. Similarly, stabilized backfill is used when it is required to get the road open to traffic as soon as possible. Table 3.4 provides details with regard to the backfill material types used by each state DOT. The number of states allowing each type of backfill is summarized in Figure 3.3.

3.2.4 Joints

The joints of HDPE pipe is typically specified as water-tight or soil-tight. Water-tightness is a more stringent requirement than soil-tightness in joints. HDPE pipe with water-tight joints are now being manufactured by major plastic pipe manufacturing companies. Soil-tight, silt-tight, leak-resistant and water-tight jointing systems are available in the marketplace. ASTM standards D3212 (*25*) and F 477 (*26*) are used for the specifications for water-tight joints and elastomeric seals respectively. AASHTO MP6-95 and AASHTO M294 do not require that the pipe be water-tight. Instead they state that the type of joint must be selected to ensure that soil infiltration into the pipe does not occur. These specifications refer to AASHTO *Standard Specifications for Highway Bridges*, Division II, Section 26 "Metal Culverts" (*15*) for criteria on soil tightness. The new AASHTO Section 30 "Thermoplastic Pipe: Construction/Installation also gives criteria on soil-tightness and water-tightness. The information on joint criteria used by state DOTs was not requested in the original questionnaire. However, once this was identified as one of the important issues to be addressed, attempt was made to collect information on joint criteria through a second round of telephone calls. A summary of information collected is presented in Table 3.5. Based on the information provided in this table it is apparent that about half of the state DOTs require water-tightness while the others require only soil-tightness. Some state DOTs, such as California and Colorado select the type of joint depending on backfill soil conditions, hydrostatic potential within the pipe and whether there is a special concern over infiltration/exfiltration or not. According to information provided by HDPE pipe manufacturers, water-tight joints are significantly more expensive to construct than soil-tight joints. Therefore, states have to weigh all options carefully before specifying the joint type.

3.2.5 Minimum Cover and Maximum Fill Height

AASHTO Section 18 specifies a minimum cover of the larger of 1 ft or the internal pipe diameter divided by eight (*11*). Of the states that responded, fourteen have specified minimum soil cover values between 0.75 ft and 1 ft. Fifteen states have specified minimum cover values between 1 ft and 2 ft. Two states have specified minimum soil cover of more than 2 ft.

Some states change their minimum cover requirements according to the type of pavement (rigid or flexible). Also, some other states specify the same minimum cover, but change the location to which the cover is measured (to the top of the pavement for rigid pavements and to the top of the subgrade for flexible pavements). Minimum cover requirements used by each state DOT is listed in Table 3.4.

There is a wide range of maximum fill heights specified for HDPE pipes in different states. Once again this information was not requested in the original questionnaire but collected by telephone calls at a later stage. The maximum fill height specifications used by each state DOT are shown Table 3.6.

3.2.6 Performance

Most state DOTs reported they had positive experience with HDPE pipe. Some problems with the use of HDPE pipe were reported, but many of these were not limited to HDPE pipe. Among the problems that affected HDPE pipe, maintenance of the line and grade of the pipe during installation appear to be the most common. This issue comes up when laying pipe in the presence of water. Of the 32 states that responded, two states identified this as a frequent problem while seven others reported it to be an occasional problem. The next most problematic aspect of using HDPE pipe is finding a qualified contractor with one state DOT finding it to be a frequent problem and three others labeling it as an occasional problem. Joint leakage has been detected in some installations. This was frequently attributed to improper installation and backfilling practices. There was one instance of pipe wall cracking that occurred as a result of improper handling of the pipe, i.e., pipe being dropped off the delivery truck.

One state DOT indicated that they have had very bad experience with the use of HDPE pipe. Excessive deflection was their greatest concern. One specific problem area identified by another state DOT involved the flared end sections of HDPE pipe. They reported that these flared ends sometimes bend and obstruct pipe openings. Although fire hazard was cited as a concern by some state DOTs, there have been very few cases reported fires in field installations. Some state officials claim that enough air is not available within the pipes for them to burn. This, however, may not apply to all types of pipe installations because experimental fires set up by some state agencies resulted in the culvert pipes burning right through (*1*). It has been found that HDPE pipe with concrete headwalls burn less readily (*1*). An important consideration is that bitumen coated metal pipe has been used for some time with very few reported incidents of fire. Bitumen burns at a lower temperature than HDPE.

Information on problems reported by each state DOT is provided in Table 3.7. Their frequency of occurrence is summarized in Table 3.8 and Figure 3.4.

3.2.7 Inspection

Some states use mandrel testing and video inspection to check whether the installation is satisfactory. Georgia, Indiana, Louisiana, North Carolina, Wyoming and Wisconsin are some of the states that use mandrel testing. It is not possible to mandrel test the larger pipe diameters as the mandrel may damage these pipes. Georgia, Indiana, Michigan, North Carolina, New Jersey and South Carolina use video cameras to inspect installations.

3.3 Summary

Conventionally, corrugated metal and concrete pipe has been used for subsurface gravity flow drainage. Thermoplastic pipe is relatively new in the market. Thermoplastic pipe offers a number of advantages, most importantly its light weight, which reduces transportation and handling costs. On the other hand, there are disadvantages as well. The pipe performance is highly dependent on installation quality. Among the drawbacks identified difficulty in maintaining line and grade and finding qualified contractors were the most common. Many of the other problems such as joint leakage, excessive deflection, and pipe wall cracking appear to be linked to inadequate care during pipe handling and installation.

A survey was carried out to document the current state of practice in the use of thermoplastic pipe. A questionnaire was sent out to all 50 states and 32 states responded. According to the responses, most states have used HDPE for between five to ten years and many states have had positive experiences in the use of thermoplastic pipe.

Most of the states that responded use HDPE culverts across roads. A vast majority of the states use HDPE pipe for side drainage. Most states use HDPE pipe of up to 36 in. diameter for drainage and few states allow larger diameters. Native backfill is used in fifteen of the states. Others use either stabilized materials or imported backfill such as stone screenings. Both soil-tight and water-tight joints are specified. Some major HDPE pipe manufacturing companies are now producing pipe with water-tight joints. Minimum cover specified is mostly between 12 in. and 24 in. This is comparable to the minimum cover of 12 in. specified in AASHTO Section 18. The maximum fill heights specified vary from 10 ft to 61 ft. Maintaining line and grade, especially when water is present is a more frequent problem with using HDPE pipe. Though HDPE pipe is combustible, there have been few cases reported of actual fires.

Table 3.1. Information on the Largest Pipe Diameter Used, Number of Years of Use, and Specifications for HDPE Pipe.

DOT	Largest Diameter (inches)	Number of Years HDPE Has Been Used	Developed Own Specifications	Specifications Used
Ohio	60	13	Yes	M294
California	48	12	Yes	18
Connecticut	48	10	No	M252,M294
Florida	48	7	Yes	18,M294
New York	48	9	Yes	M294,18, MP 6-95
South Dakota	48	5	No	ALL
Washington	48	13	Yes	M294 Type S, 18
Kentucky	36[a]	14	Yes	18, M294
Arkansas	36	10	No	M294 Type S
Colorado	36	10	No	M294
Georgia	36	3	Yes	M252, M294,Type S
Hawaii	36	2	No	M294 or M252
Indiana	36	<10	Yes	M294, F894, F714
Louisiana	36	8	Yes	M294
Maryland	36	18	No	M294-Type S
Mississippi	36	10	Yes	NG
Missouri	36	5	Yes	M294
Montana	36	2	No	M294
New Hampshire	36	5	Yes	M294 Type S
North Carolina	36	5	Yes	M294
Rhode Island	36	0	No	M 294, 18
South Carolina	36	10	Yes	M294
Tennessee	36	7-10	Yes	18
Texas	36	8	Yes	M294, 18
Utah	36	10+	Yes	M-294
Wisconsin	36	8	Yes	M294
Wyoming	36	1	Yes	M294
Idaho	24	5	Yes	M294, F894
Kansas	24	2	Yes	18, M294
Minnesota	24	10	Yes	18
North Dakota	24	10+	Yes	M252
Oregon	24	2	No	M294 Type S, M252

[a]48 in. being field tested, F714=ASTM F714 Cell Class 335420c, F894=ASTM F894
18= Section 18 of AASHTO Specifications, Highway Bridges (*1*), M294, MP 6-95, and M252 refer to sections in AASHTO Specifications for Transportation Materials and Methods for Sampling and Testing.

Table 3.2 HDPE Pipe Applications: Maximum Pipe Diameters, Pipe Profiles and ADT Restrictions.

DOT	Entrance Road Culvert (inches)	Cross Road Culvert (inches)	Side Drain (inches)	Pipe Profile Corrugated	Pipe Profile Smooth	ADT Limit
Ohio	>48	N	>48	N	Y	No Limit
California	48	48	48	Y	Y	No Limit
Connecticut	NG	≤36	>48	NG	Y	NA
Florida	NG	48	48	Y	NG	None
New York	48	48	48	Y	Y	NA
South Dakota	48	48	NG	Y	NG	800
Washington	NG	36	36	N	Y	No Restriction
Kentucky	36	36	36	NG	Y	NG
Arkansas	NG	NG	36	Y	Y	NA
Colorado	NG	NG	NG	Y	Y	None
Georgia	36	36	36	N	Y	<250 for Cross Drain
Hawaii	36	36	36	Y	Y	NG
Indiana	36	36	36	N	Y	No Restriction
Louisiana	NG	NG	36	N	Y	NG
Maryland	N	N	36	Y	Y	Unknown
Mississippi	36	36	36	NG	Y	DHV<200 for Culvert
Missouri	36	36	36	N	Y	<1700
Montana	NG	NG	24	Y	Y	Local Access
New Hampshire	NG	36	36	Y	NG	All
North Carolina	36	24	36	Y	N	NA
Rhode Island	NG	NG	Testing	Y	NG	NG
South Carolina	NG	36	36	N	Y	1000
Tennessee	NG	NG	36	NG	Y	NG
Texas	36	36	36	Y	N	2000/lane
Utah	36	36	36	Y	Y	NG
Wisconsin	NG	36	36	NG	NG	1500 for Culverts
Wyoming	36	36	NG	NG	Y	NA
Idaho	NI/24	NI/24	24	Y	Y	NA
Kansas	24	N	NG	NG	NG	NG
Minnesota	24	NG	NG	NG	NG	Unlimited
North Dakota	24	24	24	NG	NG	NG
Oregon	24	24	24	NG	Y	NA

Y = Allowed, N = Not Allowed, NI = Not Interstate, NG = Information Not Given, NA= Not Applicable

Table 3.3. Maximum Pipe Diameter for Each Pipe Application and Number of State DOTs.

Largest Diameter Used (inches)	Number of States		
	Side Drainage	Minor Cross Roads/ Driveways	Cross Road Culverts
>48	2	1	0
48	3	3	4
36	17	10	14
24	4	5	4
Not Used	0	1	3
Testing	1	0	0
Information Not Given	5	12	7

Table 3.4. Types of Backfill Used, Field Compaction Densities and Minimum Cover.

DOT	Native	Flowable	Backfill Minimum (inches)	Compaction Density	Description of Backfill Material
Ohio	N	Y	18	96%max in Field	Sand or Well Graded Granular
California	NG	Y	24	95%	Specification Defined Structure Backfill
Connecticut	NG	NG	24	NG	
Florida	NG	NG	9	95%	A-1,A-2,A-3
New York	UI	UI	12	95%	Select Granular
South Dakota	NG	NG	17	95%	Sand
Washington	Y	Y	24	90%	
Kentucky	Y	NG	12	NG	
Arkansas	Y	NG	12	95%	
Colorado	NG	Y	12	95%	Granular Backfill in Non Curb Areas
Georgia	N	N	18	95%	
Hawaii	Y	Y	24	95%	
Indiana	F	M	24	95%	Borrow for Backfilling of Driveways
Louisiana	NG	Y	12	95%	Granular
Maryland	N	Y	24	95% T-180	Cradle w/ AASHTO #57
Mississippi	Y	NG	12	Not Specified	Granular, 100% Passing 1.5 in. Sieve
Missouri	NG	NG	12	90%	100% Passing 1.5 in. Sieve,<5% Passing No 200 sieve, PI=0
Montana	Y	NG	18	95%	

NG= No Information Given, Y=Allowed, N= Not Allowed, UI= Used Infrequently, F=Field Entrance Only, M=Mainline

Table 3.4. Continued.

DOT	Native	Flowable	Backfill Minimum (inches)	Compaction Density	Description of Backfill Material
New Hampshire	Y	Y	24	95%AASHTO -T99C	Processed Aggregate
North Carolina	Y	NG	18	NG	Stone Screenings
Rhode Island	Y	NG	NG	90-95%	
South Carolina	Y	N	48	95%	
Tennesee	NG	Y	24	95%	
Texas	NG	Y	12	Not Specified	Crushed Stone or Pea Gravel <3/8 inch
Utah	Y	Y	12	96% T-99	State Specific Grading
Wisconsin	NG	NG	12 When Paved	90%	Granular Backfill Passing a 1 inch Sieve
Wyoming	Y	Y	25	95%	
Idaho	Y	NG	24	100%,95%[a]	
Kansas	Y	NG	36	90%	Crushed Stone to 6in. to 12 in. Above Top of Pipe
Minnesota	F	NG	PP	95%	For Storm Sewers Granular Soil Required
North Dakota	Y	Y	12	NG	
Oregon	NG	Y	12 to subgrade	95%	3/4"-0, 1"-0

NG= No Information Given, Y=Allowed, N= Not Allowed, UI= Used Infrequently, PP=1ft to 20ft m Private Entrance, 2ft to 20ft Public Entrance, F=Field Entrance Only, M=Mainline [a]100% for dry density>110 pcf, 95% otherwise.

Table 3.5. Types of Joints Specified by Different State DOTs.

State	Joint Type Specified
California	Water-Tight or Soil-Tight Depending on Soil Conditions, Hydrostatic Potential etc.
South Carolina	Water-Tight
Colorado	Water-Tight for Storm Drains, Soil-Tight for Culverts
Utah	HDPE Prohibited Where Watertight Required
Connecticut	Soil-Tight
New Hampshire	Soil-Tight
North Carolina	Soil-Tight
Texas	Soil-Tight
Wyoming	Soil-Tight

Table 3.6. Maximum Fill Heights Specified by Different State DOTs.

State	Maximum Fill Height (ft)
Michigan	10
Missouri	13
New York	15
Washington	15
Idaho	15
New Hampshire	15
Georgia	20
California	30 ft. for Up to 36 in. Diameter and 18 ft. for 42 in. and 48 in. Diameter.
Nebraska	40
North Dakota	61

Table 3.7. Problems with the Use of HDPE Pipe.

DOT	Excessive Deflection	Joint Leakage	Wall Cracking	Fire Hazard	Chemical Attack	Line and Grade	Qualified Contractor
Ohio	OP	NP	NP	NP	NP	OP	OP
California	NP	NP	OP	OP	NP	NI	NP
Connecticut	NI	NI	NI	NP	NI	OP	OP
Florida	NP	OP	OP	NP	NP	FP	FP
New York	NP	OP	OP	OP	NP	OP	NP
South Dakota	NP	NP	NP	NP	NP	NP	NP
Washington	NP	NP	NP	NI	NP	NP	NP
Kentucky	OP	NP	OP	NP	NP	NP	NP
Arkansas	NI	NI	NI	NI	NI	NI	NI
Colorado	NI	NI	OP	OP	NI	FP	NP
Georgia	NP	NP	NP	NP	NP	NP	NP
Hawaii	NI	NI	NI	NI	NI	NI	NI
Indiana	NI	NI	NI	NI	NI	NI	NI
Louisiana	NP	NP	NP	NP	NP	OP	NP
Maryland	NP	NP	NP	NI	NI	NI	NI
Mississippi	NP	NP	NI	NP	NP	NP	NP
Missouri	NI	NI	NI	OP	OP	NI	NI
Montana	NI	NI	NI	NI	NI	NI	NI
New Hampshire	NP	NP	NP	NP	NP	NP	NP
North Carolina	OP	NI	NI	NP	NP	OP	OP
Rhode Island	NI	NI	NI	NI	NI	NI	NP
South Carolina	NP	OP	OP	NP	NP	NP	NP
Tennesee	NI	NI	NI	NI	NI	NI	NP
Texas	NI	OP	NI	NP	NI	NI	NI
Utah	NI	NI	NI	NI	NI	OP	NP
Wisconsin	NP	NP	NP	NP	NP	NP	NP
Wyoming	NI	NI	NI	NI	NP	NI	NI
Idaho	NI	NI	NI	NI	NI	NI	NI
Kansas	OP	NP	NP	NP	NP	OP	NP
Minnesota	NP	OP	NP	NP	NP	NP	NP
North Dakota	NI	NI	NI	NI	NI	NP	NI
Oregon	NP	NP	NP	NP	NP	NP	NP

FP=Frequent Problem, OP= Occasional Problem, NP= Not a Problem, NI=No Information

Table 3.8. Frequency of the Occurrence of Specific Problems.

Frequency	Excessive Deflection	Joint Leakage	Wall Cracking	Fire Hazard	Chemical Attack	Line and Grade	Qualified Contractor
Frequent problem	0	0	0	0	0	2	1
Occasional problem	4	5	6	4	1	7	3
Not a problem	14	13	11	16	18	11	18
No information	14	14	15	12	13	12	10

Survey on the Use of Large Diameter (36" and up) High Density Polyethylene (HDPE) Pipes for Gravity Flow Drainage Applications

Conducted by

Department of Civil Engineering, Texas Tech University

Box 41023, Lubbock, TX 79409-1023

Page 1 of 3

1. Number of years your Agency has used HDPE pipes for gravity flow drainage: ________
2. The maximum diameter of HDPE pipe allowed in your Agency: ______
3. Please complete the HDPE pipe application matrix below :

Pipe Diameter (in.)	Pipe Profile		HDPE Pipe Applications in Your Agency			
			Culvert		Side Drainage	Others (Pl. specify)
	Corrugated	Smooth	Entrance Road Only	Cross Road		
24						
36						
48						
>48						

4. Please complete the HDPE pipe backfill material matrix below :

Pipe Diameter (in.)	Backfill Material			Minimum Soil Cover	Compaction Density	Maximum Allowable Traffic (e.g., ADT)
	Native Soil	Flowable Backfill	Others (Pl. specify)			
24						
36						
48						
>48						

Figure 3.1. Questionnaire Used in Survey of State DOTs.

Page 2 of 3 (page break)

5. Specific problems encountered during long-term service of the HDPE pipe:

HDPE PIPE PERFORMANCE	Don't know	Not a problem	Occasional problem	Frequent problem
Excessive deflection				
Joint leakage due to excessive pipe deformation				
Wall cracking on pipe				
Fire hazard (Combustibility)				
Degradation due to acidity				
Others (Pl. specify) a. b. c. d.				

6. Specific difficulties encountered by your Agency during HDPE pipe installation:

INSTALLATION DIFFICULTIES	Don't know	Not a problem	Occasional problem	Frequent problem
Difficulty to maintain proper line and grade				
Availability of qualified contractor for HDPE pipe installation				
Others (Pl. specify) a. b. c. d.				

Figure 3.1. Continued.

7. Please write down HDPE pipe specifications (e.g., AASHTO 18, etc.) used in your Agency:

   a. Material:

   b. Structural design:

   c. Installation:

8. Does your Agency have its OWN SPECIFICATION or SUPPLEMENTARY PROVISION to existing AASHTO or ASTM specifications for HDPE pipe installation/design?

   Yes: ________ No: ________

   If "Yes," could you please send a copy to:

Dr. P.W. Jayawickrama
Texas Tech University
Department of Civil Engineering
Box 41023
Lubbock, TX 79409-1023

9. To ensure proper installation of the HDPE pipe, the approach(s) used in your Agency (Pl. check all that applies):

   a. Video tape the installation
   b. Developed HDPE installation specification for the contractors to follow
   c. Others (Pl. specify):

10. Please provide the following information:

    Your job title:
    Division:
    e-mail address:

11. Please FAX the completed survey to:

Dr. P.W. Jayawickrama
Texas Tech University
Department of Civil Engineering
FAX No: (806) 742-3488

Figure 3.1. Continued.

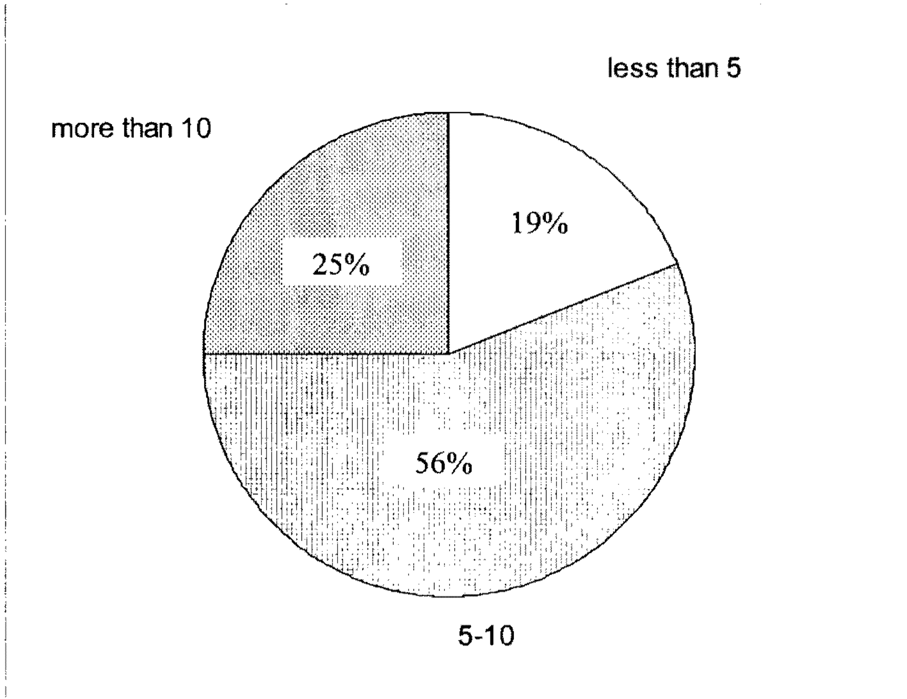


Figure 3.2. Number of Years of Experience with the Use of HDPE Pipe.

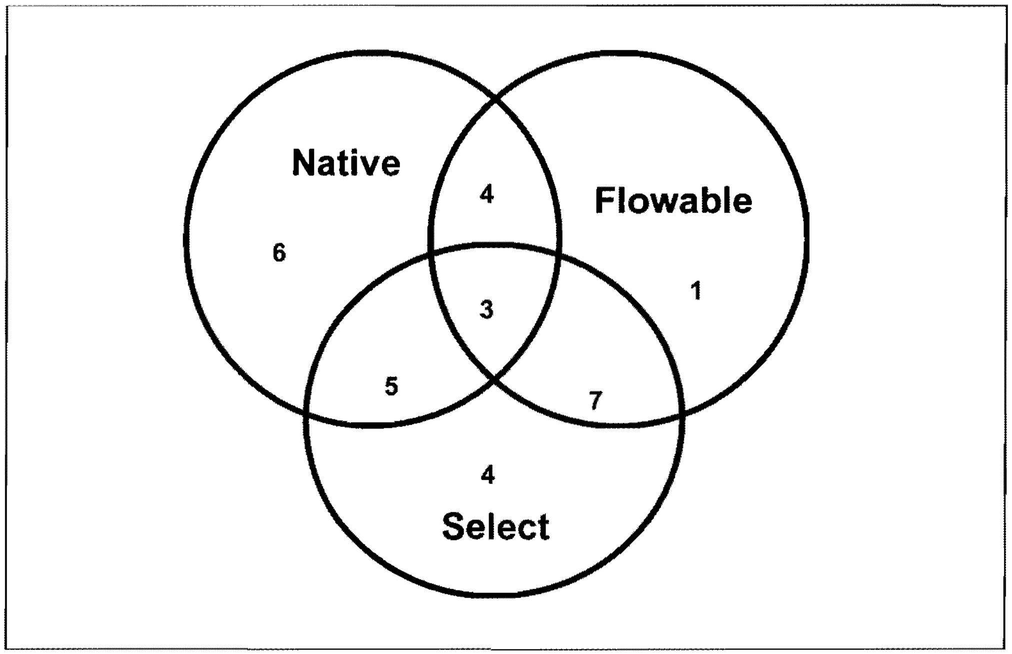


Figure 3.3. Types of Backfill Materials Allowed by Various State DOTs.

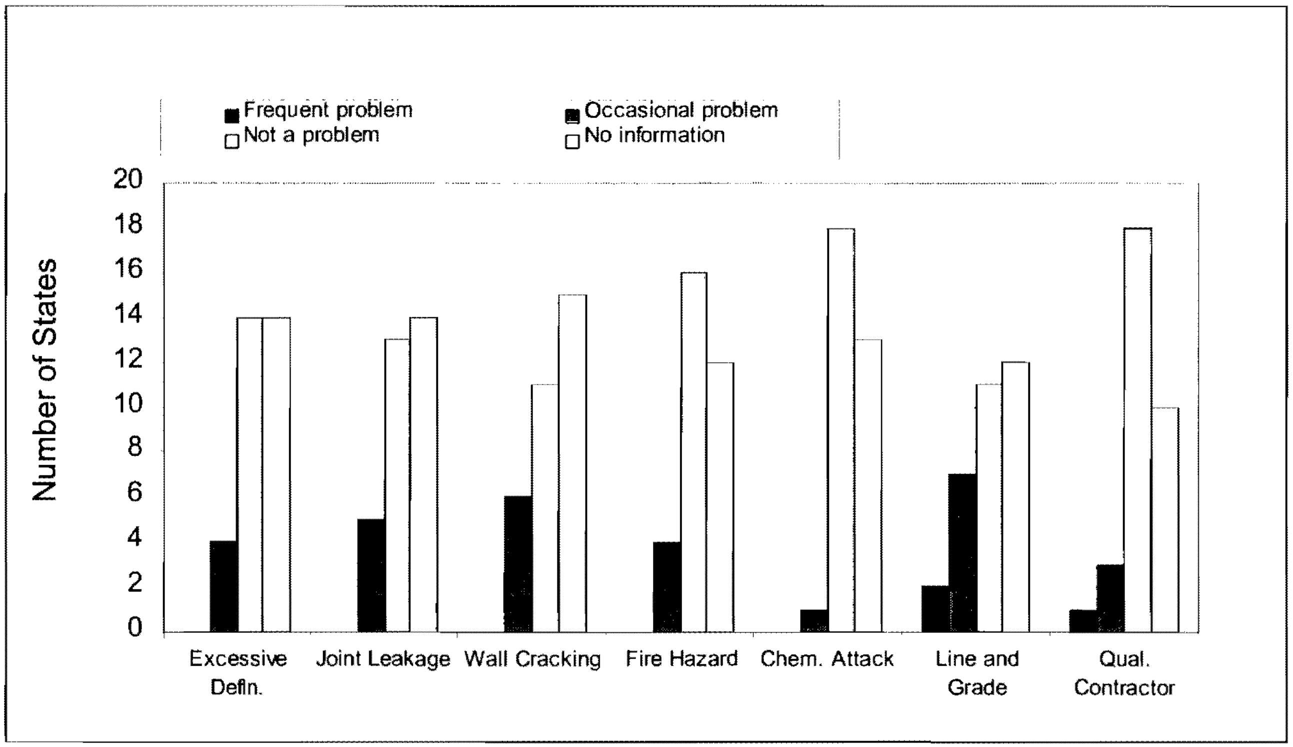


Figure 3.4. Problems Experienced During the Use of HDPE Pipe and Their Frequency of Occurrence.

CHAPTER IV
FIELD TESTING PROGRAM

4.1 Objectives

The primary objective of this research was to develop specifications for the installation of large diameter (up to 48in. nominal diameter) HDPE pipe for gravity flow applications. Among the important issues that must be addressed in these specifications are the selection of suitable backfill materials and proper methods for their placement and compaction. Furthermore, specifications should also address the maximum fill height and minimum cover requirements corresponding to specified backfill material and placement conditions. The data needed for the development of such specifications were obtained from a series full-scale field load tests. This chapter describes the above field load test program in detail.

4.2 Compaction Control of Granular Fill

In geotechnical engineering practice, it is customary to use the dry density of the compacted fill to control the field compaction operation. Accordingly, a standard Proctor density test, AASHTO T-99 (*36*) or ASTM D698 (*37*) is performed on the soil and the maximum dry density of the soil determined. The target dry density to be achieved in the field is then expressed as a percentage of the above maximum dry density. This is also the approach recommended by the AASHTO and ASTM specifications that specifically deal with the selection and field compaction of backfill materials for thermoplastic pipe. Chapter II contained a detailed discussion regarding the AASHTO and ASTM specifications for backfill materials.

4.3 Limitations in the Use of Density Control Approach

Minimum dry density approach recommended by AASHTO and ASTM has several limitations as far as its application in routine thermoplastic pipe installation projects are concerned. First of all, this approach requires density measurements on each lift of the compacted fill throughout the entire length of the pipe. Such a requirement will place additional demand on manpower and will slow down the pipe installation process considerably. On the other hand, good control on the backfill compaction is necessary in thermoplastic pipe installation in order to ensure satisfactory pipe performance. Thus, there is a need for alternative criteria for the selection of suitable granular backfill materials and methods to ensure their proper compaction. Such alternative criteria should eliminate the need for intensive testing at the jobsite.

A second limitation in the application of density control approach for coarse granular materials is associated with the difficulty in establishing a well defined moisture-density relationship for such materials. To demonstrate this, data is presented from tests conducted on two well-graded sand gravel mixtures that are typical of backfill material used in pipe installations. Both materials classify as well-graded gravel (GW) in the USCS classification and therefore, are identified as GW1 and GW2. The results obtained for the two gravels based on moisture-density tests conducted according to ASTM D 698 are shown in Figure 4.1.

Review of data presented in Figure 4.1 reveals that the water contents used in these tests are lower than those typically used in moisture-density tests on finer grained soils. In each case, the range of water contents used in testing varied between approximately 2.0% and 8.0%. The upper limit (i.e., water content = 8.0%) represents the maximum water content that each soil was able to retain. Although more water was added to the sample during testing in an effort to raise the water content even further, this did not change the final outcome because the additional water readily drained away during compaction. Review of the density-water content data presented in Figure 4.1 shows that there is a general trend of increasing dry density with increasing water content. Unlike in fine-grained soils, the data do not show an optimum water content at which the dry density reaches a maximum nor a decrease in dry density beyond the optimum water content. Therefore, based on the data presented above, it is clear that for free draining material such as those discussed above, an alternative method for compaction control must be found. ASTM offers two alternative test methods that may be used for such coarse-grained materials. They are: ASTM D 4253: Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table (*38*) and ASTM D 4254: Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density (*39*). However, AASHTO and ASTM guidelines on the underground installation of plastic pipe do not give any indication as to how the relative density of a compacted material can be used as a measure of adequate compaction. Furthermore, published data indicate that these test methods have a high degree of variability making them less attractive options for use in routine applications (*39*).

4.4 Testing Methodology

4.4.1 Overview

The following is an overview of the field test program. As a first step, several candidate materials that represent the complete range of backfill types belonging to ASTM Classes I and II were selected.

Secondly, necessary field testing was carried out on each of these candidate materials to establish Dynamic Cone Penetrometer (DCP) blow count profiles (i.e. blow count versus depth plots) for selected compaction conditions.

Accordingly, the same backfill material was compacted using different compaction equipment and under different amounts of compaction energy (such as 2-passes, 4-passes of compaction equipment, etc.) and a separate DCP profile developed for each of these combinations. These tests and the results obtained are described in a subsequent section under the heading Test Series A. In the next step, selected combinations of backfill material-compaction condition were used in full-scale load tests on HDPE pipes. This series of load tests is described later in this report as Test Series B. Testing was conducted under two types of loading conditions; (a) uniform loading to represent situations where the pipe is subjected to overburden pressures from earthfill and (b) concentrated loading to represent situations where pipe is subjected to construction wheel loads under minimum cover. This test series provided load-deflection curves for pipes installed under different backfill conditions. These data were used to develop guidelines for the selection of backfill materials, compaction equipment and number of passes with each type of compaction equipment as well as to develop maximum fill height charts. The data from Test Series with concentrated loading were used to develop guidelines for minimum cover required over HDPE pipe installations.

4.4.2 Backfill Materials

Four different types of granular backfill materials were included in this research. The particle size distribution curves for all these are shown in Figure 4.2. The coarsest of these four is labeled as "Coarse Gravel" in the figure. Coarse Gravel consisted of angular particles of crushed rock with sizes ranging from 0.25 in. to 1.0 in. The second backfill material used in testing was a river gravel that consisted of sub-rounded particles with sizes ranging from 0.01in. to 0.5in. In Figure 4.2, this material is labeled as "Medium Gravel." The third material that was included in the testing program was an aggregate blend consisting of 50% Coarse Gravel and 50% sand. This material is referred to as the "Gravelly Sand" throughout this report. The particle size distributions corresponding to both the sand and the Gravelly Sand are shown in Figure 4.2. The Gravelly Sand had a broad range of particle sizes with particle sizes ranging from 0.01in. to 1.0in. The fourth material tested, which was the finest, consisted of 17 % Medium Gravel, 66% Sand, and 17% of a sandy clay and met the gradation requirements for Flexible Base, Grade 5 (TxDOT Standard Specifications, 1993, Item 247). This material is referred to as the " Clayey Sand" throughout the rest of the report. The Gravelly Sand and the Clayey Sand had the broadest range of particle sizes.

4.4.3 Field Compaction Equipment

The field compaction devices that are most commonly used in the compaction of pipe embedment materials are: (a) impact rammers and (b) vibratory plates. These equipment can be operated within the narrow, confined space between the pipe and the trench wall to achieve good compaction. A third type of compactor that can be used in very tight spaces, such as the pipe haunch area, is the compressed air tamper. Testing performed in this research used all three types of compaction devices mentioned above. However, DCP data obtained during preliminary testing revealed that the levels of compaction provided by the compressed air tampers were not adequate for the specific application concerned. Therefore, no further testing was performed using this equipment. Figure 4.3(a) shows a photograph of a model of the impact rammer that was used in this study. Impact rammers (also called Wacker Packers or Jumping Jacks) can provide effective compaction for a broad range of soils. They have a reciprocating shoe that comes off the ground approximately 2 to 3 inches and then slaps down on the soil that is being compacted. Typically, these machines deliver 3000-4000 lbs of impact force per blow and operate at the rate of about 600-700 blows/min.

Figure 4.3 (b) shows a vibratory plate compactor that was used in this research. Vibratory plate compactors are well suited for compaction of granular material but not for cohesive soils. They have a rotating offset weight that creates vibrations in the plate. The plate rests on the soil being compacted and the vibrations of the plate reduces the friction between sand and gravel particles thus allowing it to compact under its own weight and the weight of the machine. The vibratory plate compactors typically weigh 135-700 lbs. The plate dimensions typically vary from about 15in. x 20in. to about 24in. x 34in.

Figure 4.3(c) shows the air tamper that was used in the research. While this type of compactor is very useful for compacting backfill in very narrow spaces and around other jobsite obstructions, they do not deliver the same level of compaction energy as the other two types of compactors discussed previously.

The manufacturers' specifications for all three compactors are shown in Table 4.1.

4.5 Test Series A: DCP Blow Count Profiles

4.5.1 The Dynamic Cone Penetrometer

In the research work described in this report, a Dynamic Cone Penetrometer (DCP) was used as a means of comparing the levels of compaction achieved in different granular backfill materials when compacted with different types of compaction equipment and different number of passes. The DCP was invented by A.J. Scala of Australia during the 1950's (*40*). Subsequently the DCP was used in South Africa and in the United States. Useful correlations of DCP blow counts with CBR and SPT values are available (*41*). The cone penetrometer used in this study is a WILDCAT Dynamic Cone Penetrometer with a 35lb safety drop hammer and 15 in. of free fall. The cone has a 90° apex angle and a projected area of 1.6 in.^{2}. It is mounted on a 1.1 in. O.D. sounding rod that has groove marks at 3.9in. (10 cm) increments. To drive the cone, the hammer is raised manually by two handles until it just encounters the end of its maximum possible stroke and then released. This raising and dropping operation is repeated as the cone is driven into the soil. The number of hammer blows per 10cm of drive is recorded as the DCP blow count at that specific depth. The above dynamic cone penetrometer provided a simple, efficient, and inexpensive means of evaluating and comparing the levels of compaction. A photograph of the DCP being operated is shown in Figure 4.4.

4.5.2 Test Procedure

The variation of the DCP blow count with depth was obtained for the four types of materials given above, using different compaction equipment and numbers of passes. The trench that was used for these tests was 4 ft. deep, 3.3 ft. wide and 9 ft. long. The width of the trench was selected to approximate the average distance between a plastic pipe and a trench wall in a typical installation. Material was poured into the trench in 8 in. lifts and compacted and subsequently, DCP readings were obtained for the entire depth of backfill placed up to that point. The DCP readings taken at each level was then combined to develop the blow count profile for that particular combination of backfill, compaction equipment, and number of compaction passes. DCP profiles for four types of compaction for each material were obtained. They are: (1) loose (no-compaction), (2) 5 passes of vibratory plate, (3) 2 passes of impact rammer, and (4) 4 passes of impact rammer. The DCP profile for the Clayey Sand with no compaction was not obtained as this combination of backfill and compaction could not be successfully used in any installation.

4.5.3 Results and Review of Findings

The DCP blow count profiles for Coarse Gravel, Medium Gravel, the Gravelly Sand, and the Clayey Sand are shown in Figure 4.5, Figure 4.6, Figure 4.7, and Figure 4.8, respectively. A preliminary review of the DCP blow counts obtained for all four backfill materials revealed that they are quite sensitive to the depth of measurement. In other words, it was observed that, in a given test, the DCP blow counts obtained at larger depths were significantly higher than those obtained at shallower depths. Since the material, the method of compaction and compaction energy were identical at all depths, it is reasonable to conclude that the increase in the blow count at larger depths was due to the effects of the confining pressure and further densification from the compaction of subsequent lifts. Because of the depth sensitivity of the DCP blow counts, plots of DCP count versus depth were prepared before any further review of data. These DCP blow count-depth profiles, shown in Figures 4.5, 4.6, 4.7, and 4.8, lead to a number of useful conclusions.

First of all, DCP blow counts from a given test, when plotted against depth, fit within a fairly narrow band. Although some scatter within these bands exists, differences between different compaction equipment and compaction energy levels can be clearly discerned. One interesting observation that can be made is that 4 passes of impact rammer has consistently provided best compaction for all four materials. Comparison of the DCP profiles obtained for 2 passes versus 4 passes of the impact rammer suggest that in the case of the two uniformly graded materials (Coarse Gravel and Medium Gravel) 2 passes provided approximately 60-70 percent of the compaction that was achieved with 4 passes. In contrast, similar data for the Gravelly Sand shows that the compaction achieved with 2 passes is nearly the same as that with 4 passes. However, the data for the Clayey Sand shows that the compaction achieved with 4 passes of impact rammer was appreciably greater than obtained with 2 passes of impact rammer.

Another interesting observation involves a comparison between the DCP profiles obtained for impact rammer and vibratory plate compactor. Although the specific models of these two compaction equipment used here were very comparable in weight (155lbs for impact rammer versus 165lbs for vibratory plate), the levels of compaction achieved with the vibratory plate were consistently lower than those with the impact rammer. This difference was most pronounced in the cases of the more well-graded Gravelly Sand and the Clayey Sand. Data obtained for these material show that the compaction level achieved with 5 passes of the vibratory plate was much lower than that achieved with 2 passes of the impact rammer. For the more uniformly graded materials, however, the DCP blow counts obtained from 5 passes of vibratory plate and 2 passes of impact rammer were comparable.

From the results of Test Series A, several important conclusions can be made. Firstly, differences between the efficiency of different compaction equipment and compaction effort are apparent. It is clear that the vibratory plate compactor is not as effective as the impact rammer when utilized on materials such as the ones tested in the above test program. An examination of the productivity (3650 sq.ft per hour for the impact rammer compared to 636 sq.ft per hour for the vibratory plate compactor) of the two types of equipment also establishes the superiority of the impact rammer. Though larger models of vibratory plate compactors may compact backfill more efficiently, they cannot be operated in the narrow spaces in a pipe installation. After obtaining the results from Test Series A, it was decided to perform full scale load tests of HDPE pipe with backfill at three levels compaction. High compaction, medium compaction, and low compaction was adjudged to be 4 passes of impact rammer, 2 passes of impact rammer and no compaction, respectively.

4.6 Test Series B: Full-Scale Load Testing

4.6.1 Summary of Test Procedure

After Test Series A was completed, full-scale load testing of buried HDPE pipe was carried out to examine the behavior of the pipe under different backfill conditions when subjected to high surcharge load. The tests were conducted in a field test facility that was specially developed for this purpose. The performance of all four materials tested under Test Series A was evaluated. As mentioned previously, three levels of compaction, high, medium, and low were considered to be represented by 4 passes of impact rammer, 2 passes of impact rammer, and no compaction, respectively. Two sizes of pipe were tested, that is, pipe of nominal diameter 36 in. and 48 in. The properties of the pipes tested are shown in Table 4.2.

4.6.2 Description of Test Facility

Figure 4.9 and Figure 4.10 show a schematic cross section and plan view of the above test facility. The loading facility consisted of a 6.5ft deep, 65ft long test trench, a movable reaction frame, and 4 pairs of 12ft deep reinforced concrete belled piers for anchoring the reaction frame. The reaction frame was constructed with steel I sections; the beam had a web of 13 in. made of ½ in. thick steel, the flanges of 14.75 in. made of $^{5}/_{8}$ in. thick steel while the column had webs 9 in. wide made of 3/8 in. thick steel and flanges 8 in. wide made with $^{3}/_{8}$ in. thick steel. A photograph of the reaction frame is shown in Figure 4.11. Each column of the reaction frame was held down by 6 "J" bolts $^{5}/_{8}$ in. in diameter embedded in the concrete pier. A neoprene pad $^{3}/_{8}$ in. in thickness was placed between the top of the concrete pier and the steel column of the reaction to preclude any crushing of the concrete and to distribute the load evenly over the concrete pier – steel column interface. The steel beam was connected to each column by six $^{5}/_{8}$ in. bolts. Loading was accomplished by using a hydraulic cylinder with a piston diameter of 10 in. The top of the hydraulic cylinder was bolted to a steel plate which in turn was bolted to the bottom flange of the steel beam. The steel plate had elongated slots through which bolts were placed so that the hydraulic cylinder could be moved a limited distance along the beam to align the load directly over the pipe. The load was controlled by increasing the hydraulic pressure which could be monitored using a pressure gauge connected to the supply line from the pump. The pump had a Fenner 2HP motor, and Lubriguard 3000 AW 32 hydraulic fluid was used.

As mentioned previously, pipes were subjected to two types of loading conditions; (a) uniform loading and (b) concentrated loading. These represent two critical loading conditions. The uniform loads represented overburden pressures from earth fill over the pipe while the concentrated loads simulated wheel loading under minimum cover conditions.

In tests where uniform loads were used, the pipes were subjected to loads up to 120 kips (generated by 1500 psi of hydraulic pressure). This corresponds to an average surcharge pressure on the backfill of 5300 psf, which was approximately equal to the overburden pressure generated by 40 ft of fill. The load was transferred from the bottom plate of the hydraulic ram to the top of the backfill over the pipe by a welded steel structure. This structure can be seen in Figure 4.11. The bottom of the structure comprised a ¾ in. thick steel base plate 4.5 ft square. At the top of the structure there was a 11 in. square steel top plate, also ¾ in. thick. Four 4 in. x 4 in. angle struts (0.4 in. thick steel) and six channel sections (5 in. web and 2 in. flanges, ¼ in. thick steel) connecting the base plate and the top plate. To test pipes under concentrated loading, a smaller steel frame structure with a 2.0ft × 2.0ft base plate was used. These dimensions approximate the tire footprint of an Elevator Scraper Model CAT 615. Hydraulic pressures up to 600psi were used to simulate axle loads up to 100kips.

Each pipe section tested was 5 ft in length. The pipe sections were contained within two steel plate sections, made from ¼ in. steel, that allowed backfill to be placed and compacted against it. The location of the steel plate sections are shown on Figure 4.10. The steel plates were held at a 5ft distance apart by steel angles, and were supported from the outside by timber beams of 4 in. x 4 in. cross section. These details are also shown in Figure 4.10. A hole 2ft in diameter was cut in each plate to allow deflection measurements to be taken. A photograph of a pipe section in place between two plates is shown in Figure 4.13.

The vertical and horizontal diameters were measured with a "deflectometer." This instrument consisted of a dial gauge mounted on a metal rod. The distance between the tip of the fully extended traveling head to the bottom of the rod was measured. As the pipe deflected, the head was pushed in, and the variations in the diameters could be monitored by recording the dial gauge reading. The dial gauges allowed the measurements to be made to an accuracy of 0.001 in. The deflectometer is shown in Figure 4.14.

4.6.3 Test Procedure

Uniform loadings conditions were used in thirteen tests. The backfill material types, compaction equipment and compaction levels were varied from one test to another. The procedure used in the installation of pipe in the test trench was as follows. First, the in-situ soil was leveled out prior to the placement of the bedding. The height of bedding materials was such that there would be 6 inches of material between the in-situ soil and the bottom of the pipe. However, the bedding was extended to a level higher than the bottom of the pipe, so that it would cover 10% of the height of the pipe (see Figure 4.9). The bedding was subsequently grooved with a wooden template to fit the bottom of the pipe. Then the bedding was compacted in the same manner as the rest of the backfill. The pipe sections were lowered into the trench once the bedding had been placed, compacted and shaped.

All materials were placed in 8 in. (loose measurement) lifts and then compacted. The backfill material was built up simultaneously on either side of the pipe. Initially problems were experienced with rising of the pipe as the backfill was compacted between the pipe and the trench wall. To counteract this tendency, two angles of 3 in. length each were attached to the steel end sections to hold the top of the pipe down during compaction.

The material was extended to 1 ft above the crown of the pipe for all tests. Subsequently the locations inside the pipe where diametrical measurements were to be taken were marked. The vertical and horizontal diameters were located with the aid of a bubble level, and then the locations were marked with a permanent marker. Deflection measurements were taken at three sections for each test: at the Northern and Southern ends and in the center of the pipe. The measurements were taken before any load was placed, and then at each 24 kip increment in load (corresponding to a 300 psi increase in hydraulic pressure). At each load increment, the load was kept constant for a duration of 10 minutes.

Measurements were taken at the beginning and end of each load increment. Loading was carried out to the full 120 kips except for the two occasions where the pipes showed distress before that. The two occasions were when Medium Gravel and Clayey Sand were used as backfill material without any compaction. In these tests, rippling of the inner liner of the pipe took place.

Data collected from these tests were then presented in the form of Load-Deflection curves. Figure 4.14 shows the load-deflection curves (vertical and horizontal) obtained from one of the full-scale load tests. The deflection measurements obtained for all thirteen tests are shown in Table 4.3. The data is presented in the order that the tests were carried out.

There were eight load tests where concentrated loads using the smaller 2ft × 2ft base plate were applied. In these tests, a maximum load of 100 kips was used. This load was applied in five or six increments. At each increment, vertical and horizontal pipe deflection were measured. Once the maximum load was reached, the final deflection measurements were taken and then the load was applied cyclically to simulate repeated passes of a heavy construction equipment wheel over the pipe. Data from full-scale load tests with concentrated loading are presented in Table 4.4.

All data analysis procedures and results obtained are described in Chapter V.

Table 4.1. Manufacturers Specifications for Three Compactors.

Impact Rammer	
Weight (lbs)	155
Dimensions (LxWxH) (in.)	27.5 x 14 x 37
Shoe Size (WxL) (in.)	13 x 13.5
Impact Blow (lbs/blow)	3525
Frequency (VPM)	660-680
Stroke (in.)	3.15
Working Speed (ft./min)	27-50
Compaction Depth (in.)	22
Productivity (ft.^{2}/hr)	3650
Vibratory Plate Compactor	
Weight (lbs)	165
Plate Size (WxL) (in.)	17 x 22.4
Centrifugal Force (lbs)	3350
Max. Speed (ft./min)	75
Productivity (ft.^{2}/hr)	636
Air Tamper	
Weight with Butt (lbs)	35
Length with Butt (in.)	53
Piston Stroke (in.)	4
Blows per Minute	1550

Table 4.2. Properties of Pipe Tested.

Inside Diameter (in.)	Outside Diameter (in.)	Moment of Inertia (in.^{4}/in.)	Area (in.^{2}/in.)	Wall Thickness –minimum (in.)
36	42.46	0.55	0.361	0.05
48	55.0	0.543	0.440	Unavailable

Table 4.3. Deflection Data from Full Scale Load Testing.

Test No	Backfill	Compaction	Pipe, Trench	Load Kips	Vertical Deflection (in.) North	Vertical Deflection (in.) Center	Vertical Deflection (in.) South	Horizontal Deflection (in.) North	Horizontal Deflection (in.) Center	Horizontal Deflection (in.) South
1	Coarse Gravel	None	48 in., Narrow	24	0.646	0.697	0.621	0.452	0.419	0.418
				48	0.815	0.900	0.779	0.588	0.491	0.512
				72	0.986	1.086	0.936	0.729	0.586	0.586
				96	1.165	1.280	1.101	0.836	0.632	0.677
				120	1.397	1.547	1.317	1.033	0.719	0.799
2	Medium Gravel	None	48 in., Narrow	24	0.174	0.388	0.386	0.068	0.130	0.274
				48	0.397	0.743	0.750	0.187	0.327	0.570
				72	0.731	1.201	1.223	0.367	0.603	0.959
				96	1.132	1.779	1.844	0.566	0.897	0.475
3	Coarse Gravel	4 IR[A]	36 in. Wide	24	0.127	0.047	0.184	-0.061	0.013	0.099
				48	0.240	0.122	0.316	0.186	0.046	0.193
				72	0.382	0.259	0.456	0.255	0.105	0.312
				96	0.488	0.411	0.606	0.310	0.182	0.430
				120	0.629	0.585	0.728	0.377	0.271	0.552
4	Gravelly Sand	4 IR[A]	36 in. Wide	24	0.016	0.132	0.126	0.009	0.023	0.487
				48	0.114	0.251	0.26	0.072	0.06	0.552
				72	0.201	0.331	0.34	0.113	0.053	0.611
				96	0.295	0.429	0.43	0.166	0.045	0.624
				120	0.359	0.523	0.476	0.178	0.038	0.652
5	Coarse Gravel	2 IR[B]	36 in. Wide	24	0.078	0.118	0.088	0.038	0.031	0.152
				48	0.191	0.214	0.182	0.129	0.068	0.193
				72	0.347	0.375	0.308	0.201	0.132	0.286
				96	0.515	0.552	0.468	0.314	0.22	0.351
				120	0.675	0.717	0.593	0.385	0.275	0.42
6	Medium Gravel	2 IR	36 in. Wide	24	0.13	0.104	0.218	0.042	0.05	0.07
				48	0.25	0.176	0.333	0.103	0.096	0.132
				72	0.355	0.3	0.449	0.143	0.147	0.196
				96	0.436	0.389	0.546	0.207	0.21	0.262
				120	0.557	0.525	0.682	0.259	0.267	0.318
7	Gravelly Sand	2 IR	36 in. Wide	24	0.11	0.056	0.109	-0.055	0.016	0.038
				48	0.238	0.158	0.233	0.101	0.059	0.079
				72	0.465	0.349	0.422	0.225	0.17	0.181
				96	0.609	0.495	0.581	0.322	0.24	0.267
				120	0.827	0.744	0.807	0.47	0.356	0.278

[A]4IR= 4 passes of impact rammer

[B]2IR= 2 passes of impact Rammer

Table 4.3. Continued.

Test No	Backfill	Compaction	Pipe, Trench	Load	Vertical Deflection (in.)			Horizontal Deflection (in.)		
				Kips	North	Center	South	North	Center	South
8	Coarse Gravel	2 IR[A]	48 in. Wide	24	0.282	0.222	0.232	0.112	0.1	0.079
				48	0.372	0.297	0.302	0.162	0.129	0.115
				72	0.607	0.502	0.491	0.287	0.229	0.197
				96	0.8	0.704	0.675	0.407	0.322	0.274
				120	1.078	0.971	0.879	0.587	0.448	0.417
9	Medium Gravel	2 IR	48 in. Wide	24	0.091	0.124	0.173	0.055	0.075	0.062
				48	0.163	0.255	0.302	0.125	0.145	0.143
				72	0.259	0.405	0.468	0.19	0.257	0.252
				96	0.375	0.59	0.655	0.274	0.374	0.359
				120	0.49	0.78	0.875	0.387	0.532	0.518
10	Gravelly Sand	2 IR	48 in. Wide	24	0.142	0.298	0.058	0.061	0.048	0.025
				48	0.357	0.605	0.273	0.208	0.166	0.151
				72	0.467	0.756	0.394	0.292	0.223	0.224
				96	0.628	0.974	0.593	0.432	0.338	0.364
				120	0.899	1.345	0.922	0.683	0.555	0.609
11	Clayey Sand	4 IR[B]	36 in. Wide	24	0.251	0.265	-0.068	0.208	0.089	0.065
				48	0.344	0.339	0.082	0.25	0.118	0.087
				72	0.453	0.452	0.125	0.302	0.151	0.101
				96	0.806	0.77	0.345	0.537	0.318	0.238
				120	0.891	0.837	0.386	0.577	0.244	0.251
12	Clayey Sand	2 IR	36 in. Wide	24	0.064	0.214	0.115	0.013	0.043	0.053
				48	0.218	0.567	0.387	0.129	0.205	0.24
				72	0.406	0.981	0.714	0.284	0.399	0.471
				96	0.618	1.416	1.072	0.453	0.613	0.739
				120	0.812	1.761	1.406	0.616	0.812	0.980
13	Clayey Sand	None	36 in. Wide	24	0.505	0.482	0.303	0.479	0.448	0.256
				48	1.824	2.009	1.348	N.A.	N.A	1.106
				72	3.839	4.436	2.950	2.250	2.767	2.474

[A]2IR= 2 passes of impact rammer

[B]4IR= 4 passes of impact Rammer

N.A. = Not Available

Table 4.4. Deflection Data from Full Scale Load Testing for Minimum Cover.

Test	Backfill	Compaction	Load Kips	Cycle No	Vertical Deflection			Horizontal Deflection		
					North	Center	South	North	Center	South
1	Coarse Gravel	2 IR[A], 48 in	8		0.073	0.289	0.032	-0.033	-0.040	-0.005
			16		0.082	0.352	0.043	-0.042	-0.043	-0.021
			24		0.131	0.548	0.083	-0.054	-0.049	-0.022
			31		0.219	0.805	0.133	-0.081	-0.064	-0.033
			39		0.303	1.112	0.201	-0.104	-0.091	-0.072
				3	0.357	1.266	0.263	-0.151	-0.178	-0.100
				6	0.400	1.417	0.319	-0.172	-0.209	-0.113
				9	0.433	1.408	0.337	-0.190	-0.242	-0.135
				12	0.445	1.582	0.361	-0.200	-0.254	-0.142
				15	0.461	1.642	0.385	-0.208	-0.274	-0.150
			47		0.542	1.879	0.455	-0.236	-0.298	-0.171
			55		0.682	2.350	0.565	-0.276	-0.328	-0.195
2	Medium Gravel	2 IR, 48 in	12		0.054	0.322	0.112	-0.025	-0.048	-0.046
			16		0.061	0.392	0.139	-0.024	-0.058	-0.055
			24		0.095	0.507	0.187	-0.039	-0.073	-0.069
			31		0.139	0.726	0.255	-0.054	-0.095	-0.090
				3	0.184	1.016	0.352	-0.070	-0.235	-0.112
				6	0.262	1.311	0.462	-0.116	-0.238	-0.193
				9	0.288	1.428	0.510	-0.142	-0.293	-0.227
				12	0.311	1.558	0.554	-0.157	-0.334	-0.257
				15	0.338	1.633	0.594	-0.173	-0.363	-0.282
			47		0.354	1.731	0.606	-0.187	-0.385	-0.306
			55		0.448	2.349	0.795	-0.210	-0.442	-0.343
3	Gravelly Sand	2 IR, 48 in	8		0.057	0.177	0.025	-0.047	-0.015	-0.005
			16		0.090	0.298	0.032	-0.032	-0.020	-0.010
			24		0.135	0.487	0.067	-0.051	-0.042	-0.036
			31		0.183	0.820	0.111	-0.064	-0.061	-0.058
			39		0.255	1.267	0.184	-0.093	-0.082	-0.086
				3	0.310	0.999	0.230	-0.124	-0.154	0.150
				6	0.319	2.261	0.232	0.033	-0.215	-0.259
				9	0.343	2.604	0.258	0.020	-0.235	-0.292
				12	0.373	3.026	0.267	-0.176	-0.240	-0.156
				15	0.370	3.330	0.280	-0.175	-0.251	-0.156
4	Coarse Gravel	2 IR, 36 in	10		0.028	0.158	0.030	-0.020	-0.015	-0.019
			16		0.056	0.255	0.052	-0.033	-0.030	-0.018
			24		0.106	0.448	-0.212	-0.062	-0.066	-0.045
			31		0.201	0.648	0.220	-0.014	-0.123	-0.080
			39		0.321	0.964	0.297	-0.202	-0.193	-0.121
			47		0.493	1.378	0.393	-0.294	-0.293	-0.152
			55		0.765	2.261	0.566	0.526	-0.452	0.814

[A]2IR= 2 passes of impact rammer

Table 4.4. Continued.

Test	Backfill	Compaction	Load Kips	Cycle No	Vertical Deflection			Horizontal Deflection		
					North	Center	South	North	Center	South
5	Medium Gravel	2 IR[A], 36 in	9		0.016	0.046	-0.040	-0.012	-0.004	-0.012
			16		0.023	0.134	-0.010	-0.008	-0.022	-0.016
			24		0.082	0.331	0.059	-0.045	-0.059	-0.050
			31		0.186	0.606	0.232	-0.061	-0.119	-0.107
			39		0.226	0.923	0.327	-0.124	-0.179	-0.133
				2	0.282	1.074	0.329	-0.139	-0.247	-0.215
				4	0.291	1.079	0.320	-0.184	-0.280	-0.226
				6	0.313	1.116	0.435	-0.204	-0.322	-0.972
				8	0.330	1.127	0.415	-0.204	-0.329	-0.275
				10	0.334	1.116	0.462	-0.216	-0.342	-0.274
			47		0.250	1.318	0.544	-0.246	-0.380	-0.278
			55		0.457	1.647	0.596	-0.373	-0.427	-0.325
6	Gravelly Sand	2 IR, 36 in	12		0.107	0.246	0.032	-0.033	-0.054	-0.024
			16		0.124	0.284	0.038	-0.034	-0.056	-0.047
			24		0.149	0.418	0.066	-0.074	-0.080	-0.052
			31		0.212	0.602	0.111	-0.082	-0.119	-0.076
			39		0.319	0.877	0.152	-0.134	-0.179	-0.106
				2	0.381	1.049	0.194	-0.184	-0.229	-0.145
				5	0.517	1.174	0.228	-0.230	-0.274	4.838
				8	0.483	1.220	0.278	-0.242	-0.297	-0.171
				11	0.483	1.259	0.225	-0.254	-0.304	-0.187
				15	0.495	1.304	0.347	-0.268	-0.319	-0.183
			47		0.583	1.479	0.299	-0.279	-0.353	-0.219
			55		0.680	1.890	0.333	-0.330	-0.303	-0.239
7	Clayey Sand	2 IR, 36 in	8		0.028	0.221	0.074	-0.009	-0.051	-0.022
			16		0.095	0.466	0.153	-0.030	-0.083	-0.093
			24		0.093	0.530	0.168	-0.034	-0.107	-0.078
			31		0.144	0.981	0.264	-0.081	-0.201	-0.157
			39		0.169	1.578	0.329	-0.079	-0.315	-0.200

[A]2IR= 2 passes of impact rammer

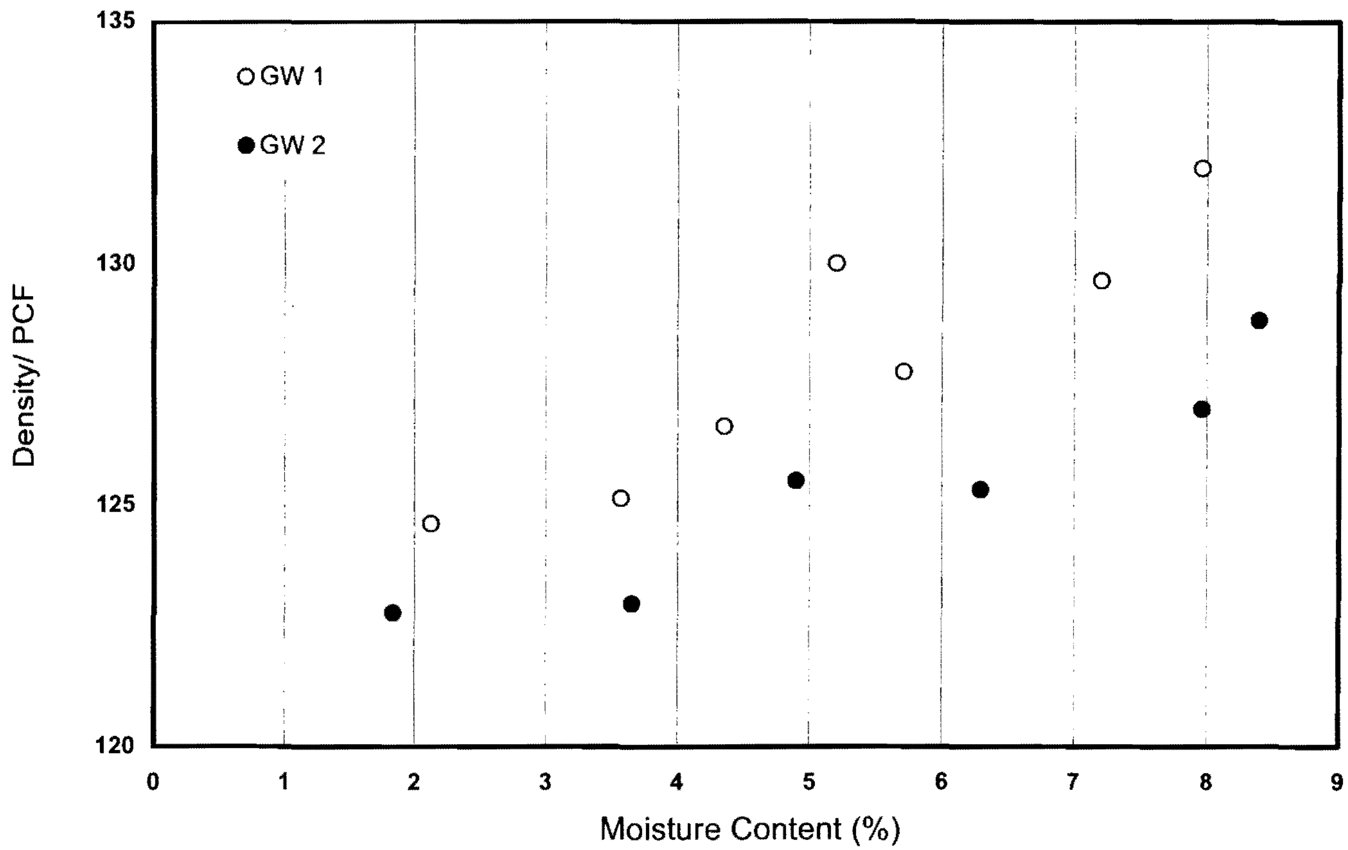


Figure 4.1. Moisture Density Relationship for Well Graded Gravel Mixtures.

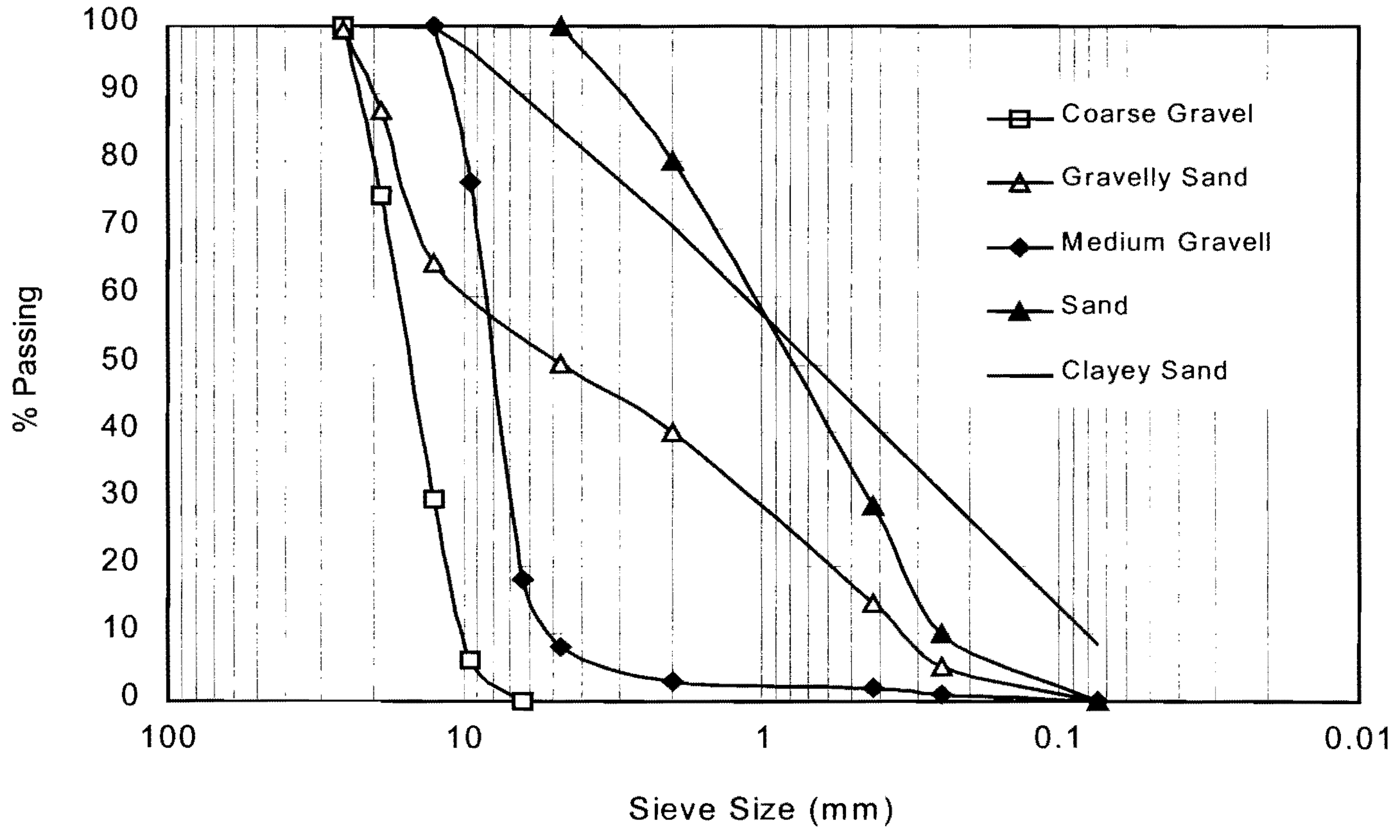
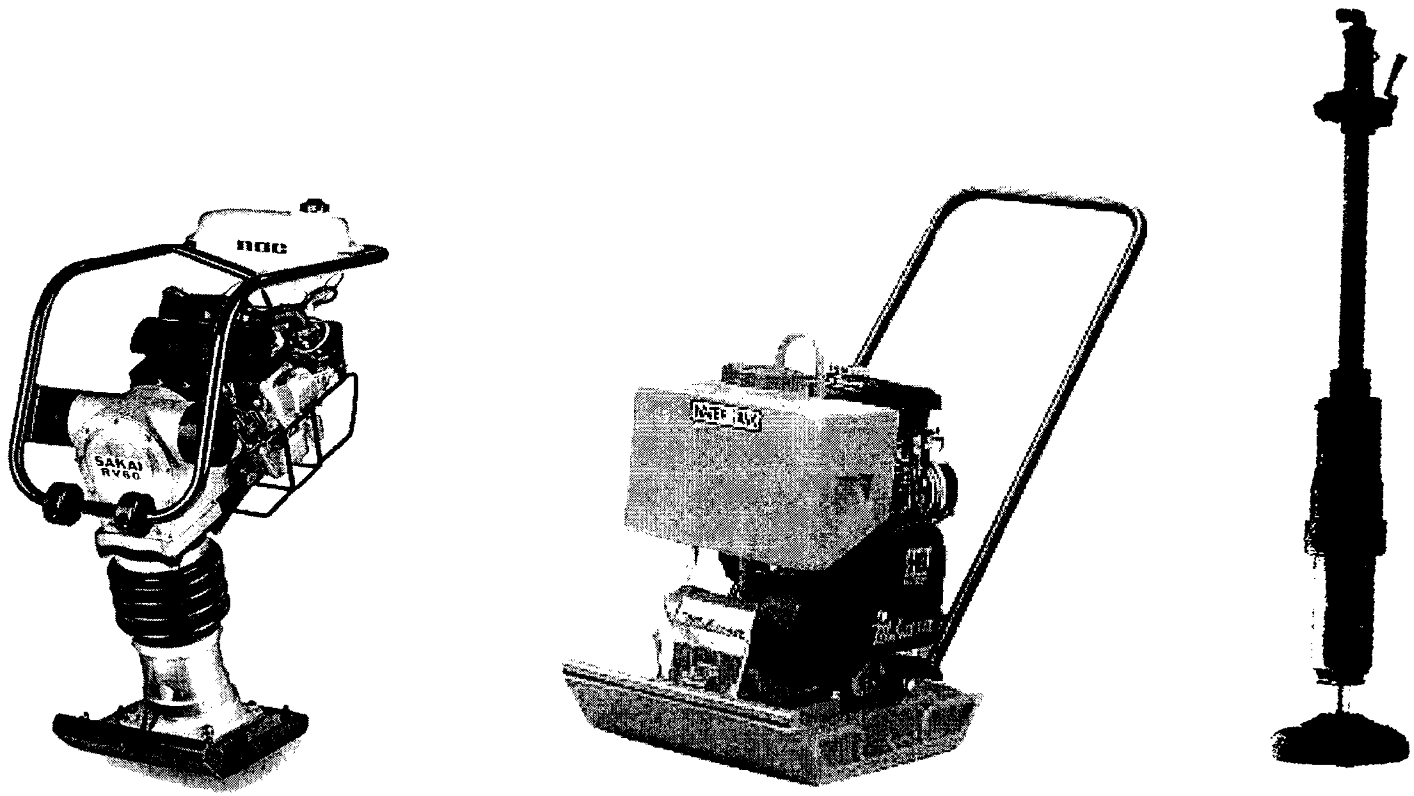


Figure 4.2. Particle Size Distribution Curves of Materials Tested.



(a) (b) (c)

Figure 4.3. Compaction Equipment; (a) Impact Rammer, (b) Vibratory Plate Compactor, (c) Air Tamper.

Figure 4.4. The Dynamic Cone Penetrometer Being Operated.

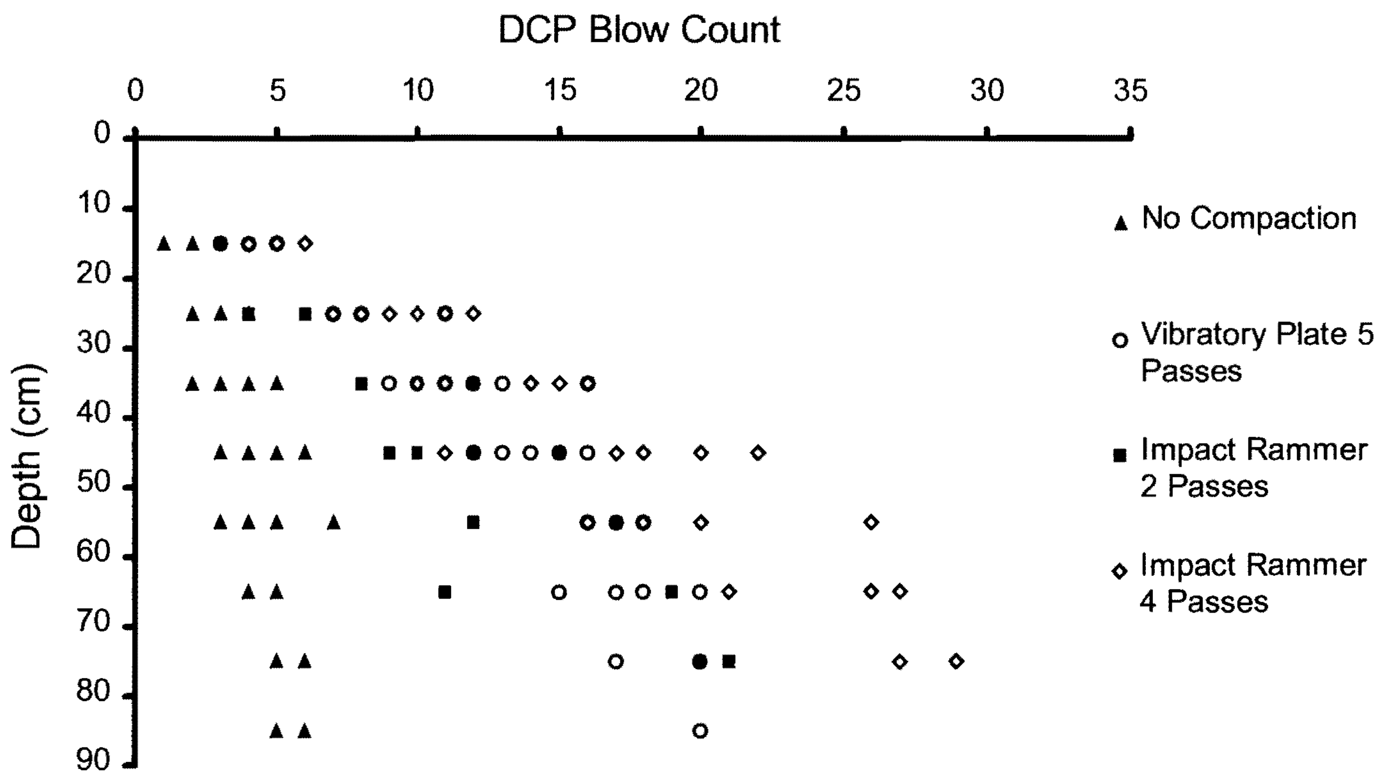


Figure 4.5. DCP Blow Count Profiles for Coarse Gravel.

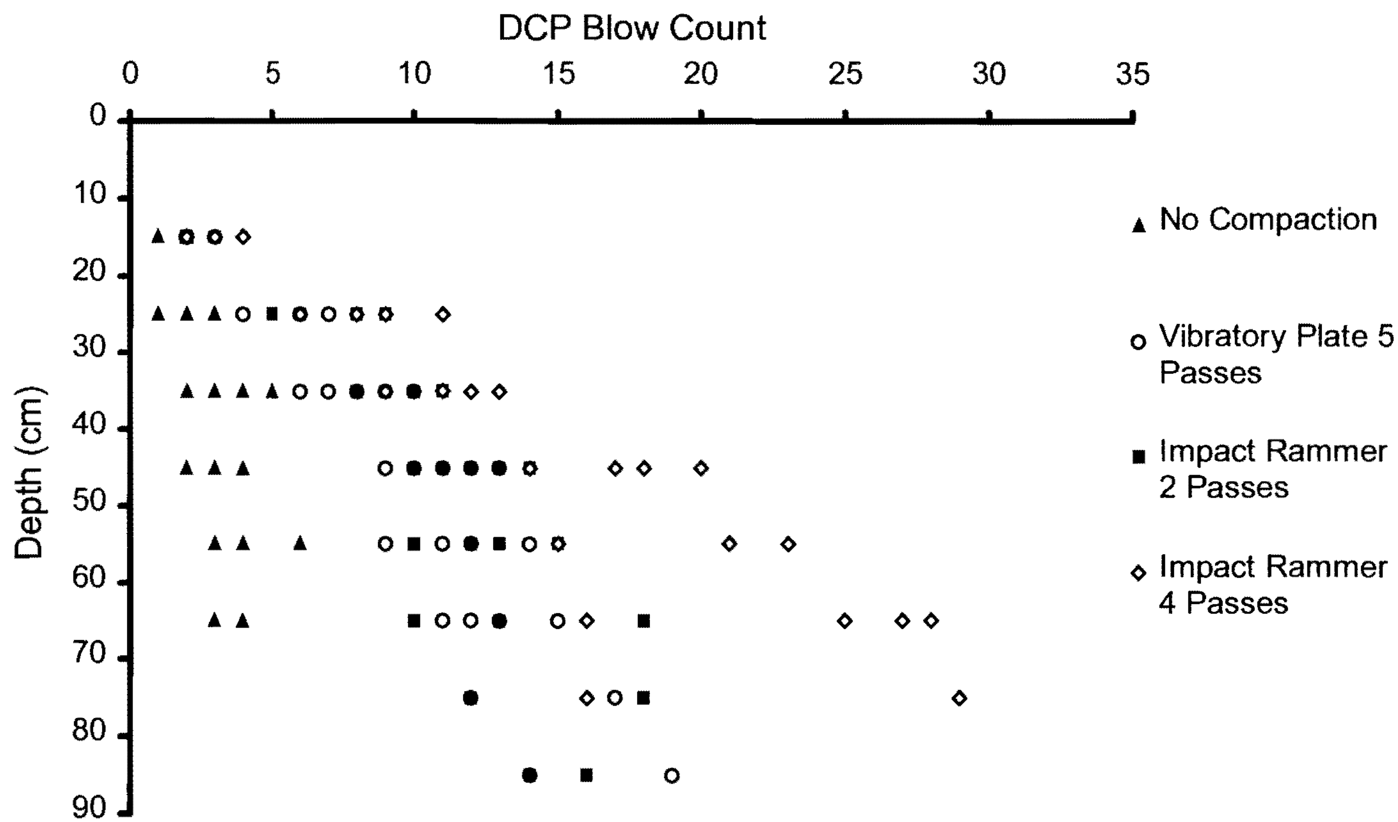


Figure 4.6. DCP Blow Count Profiles for Medium Gravel.

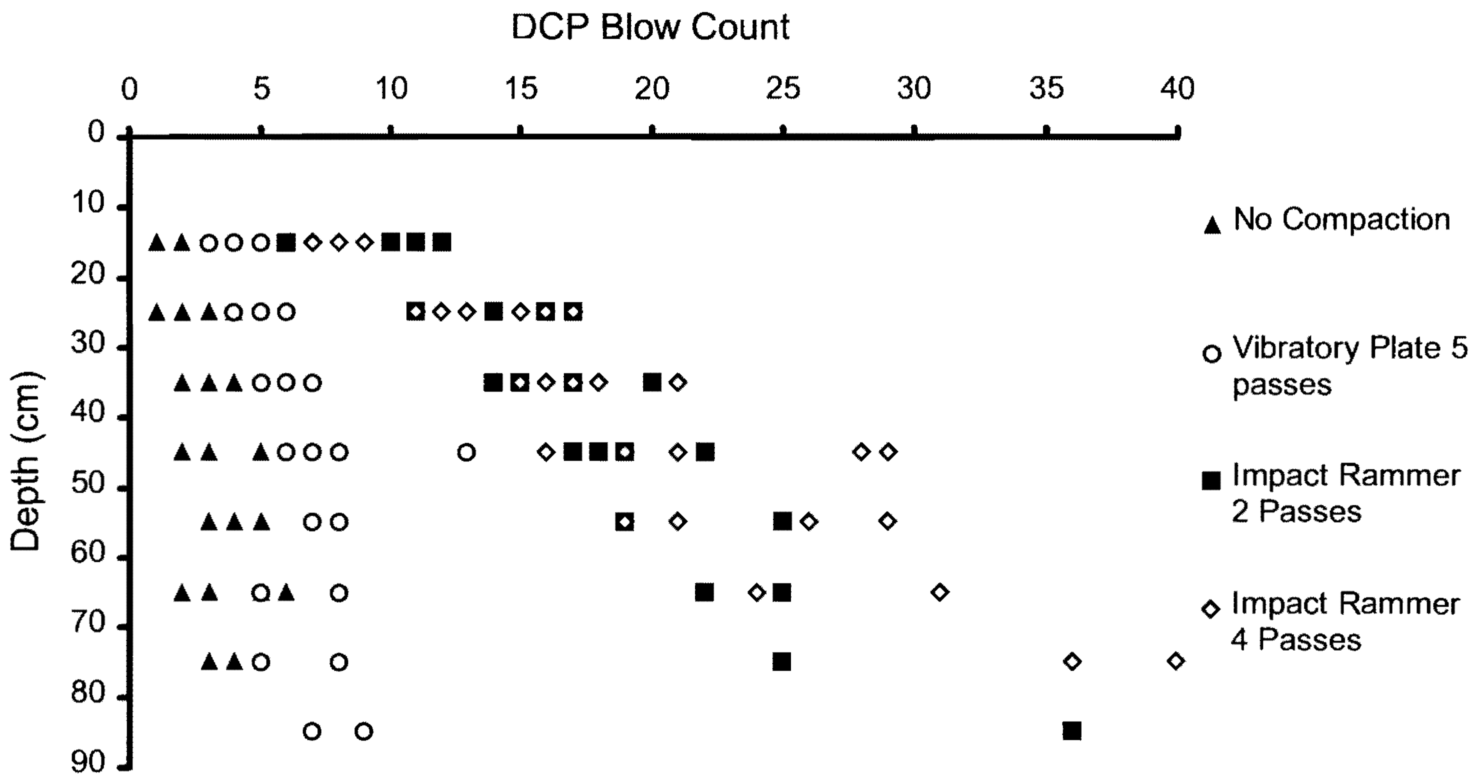


Figure 4.7. DCP Blow Count Profiles for Gravelly Sand.

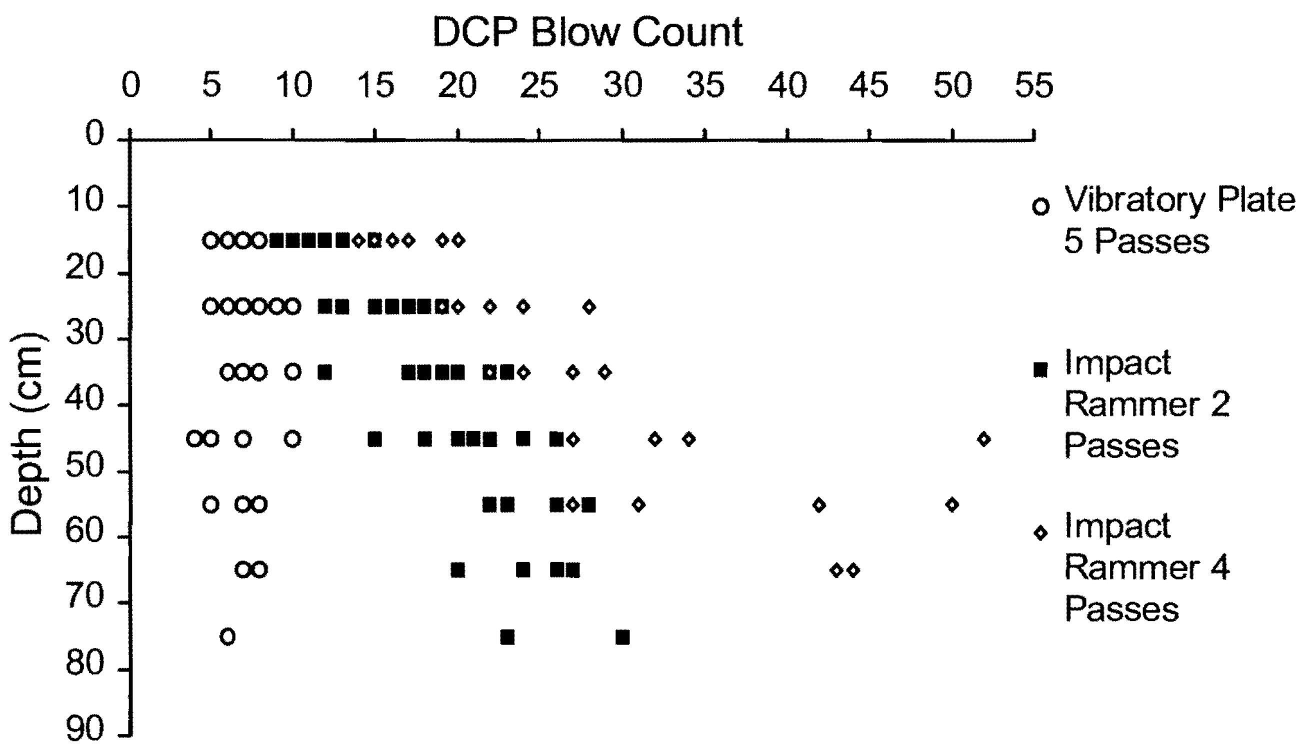


Figure 4.8. DCP Blow Count Profiles for Clayey Sand.

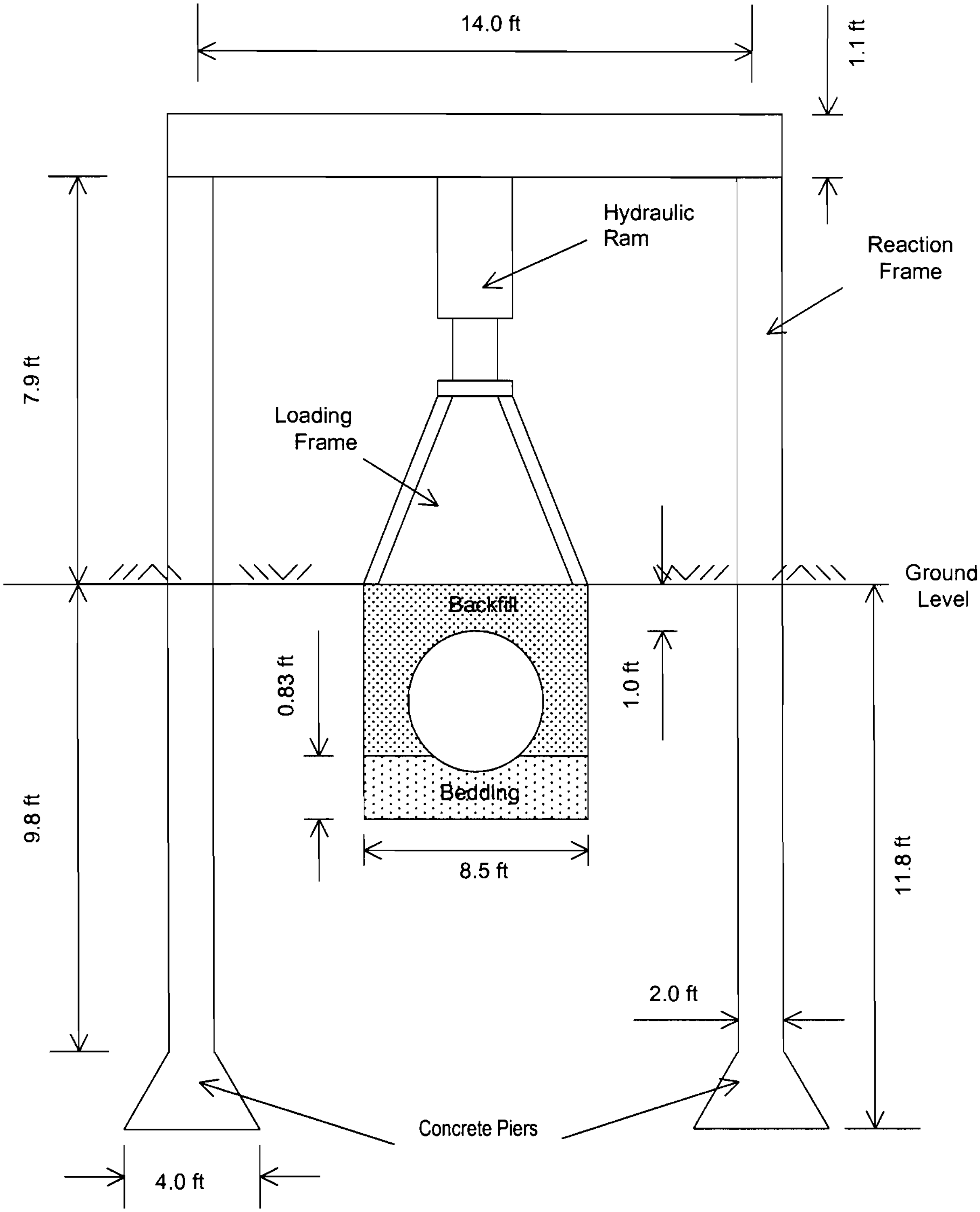


Figure 4.9. Schematic Cross Section of Test Facility, Testing a 36 in. Pipe.

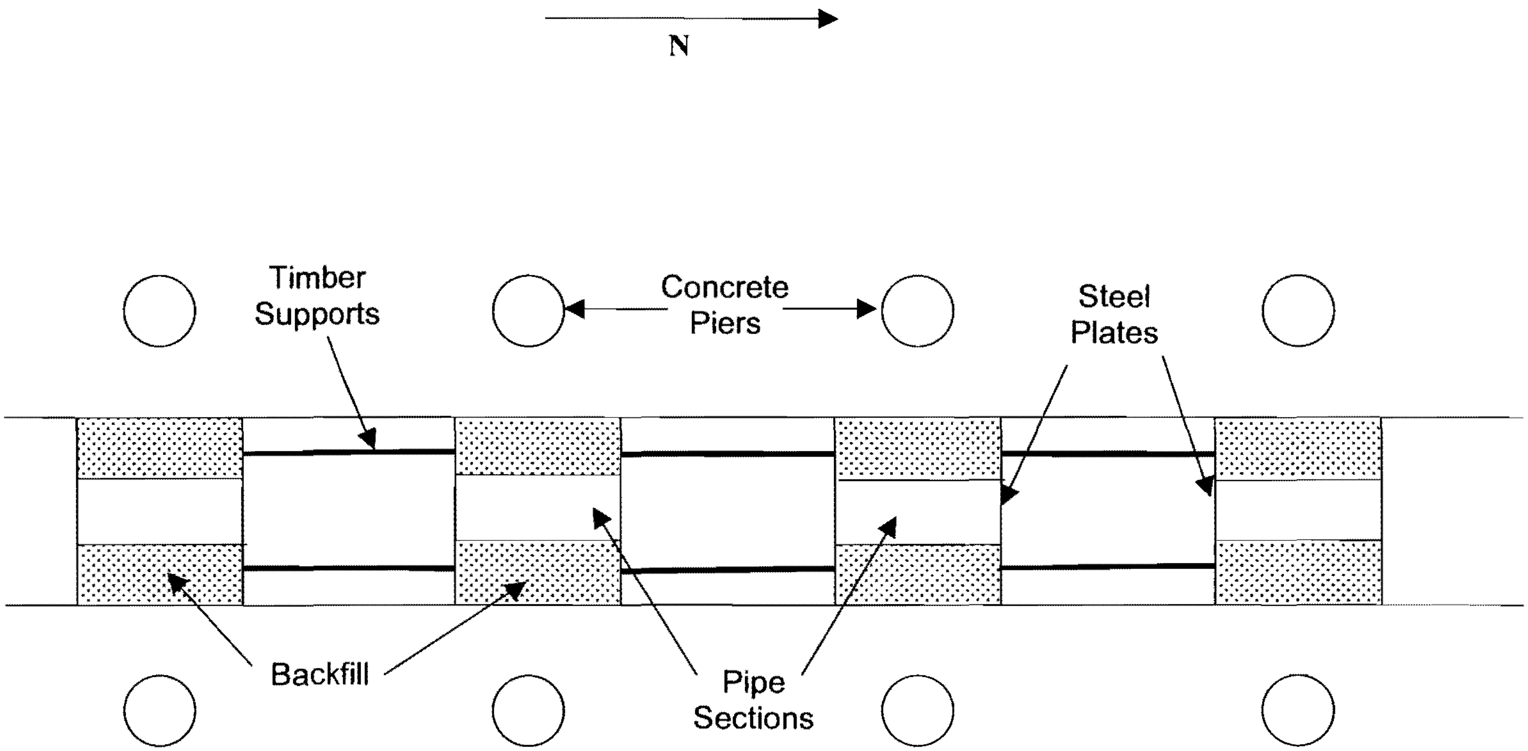


Figure 4.10. Schematic Plan View of Test Facility.

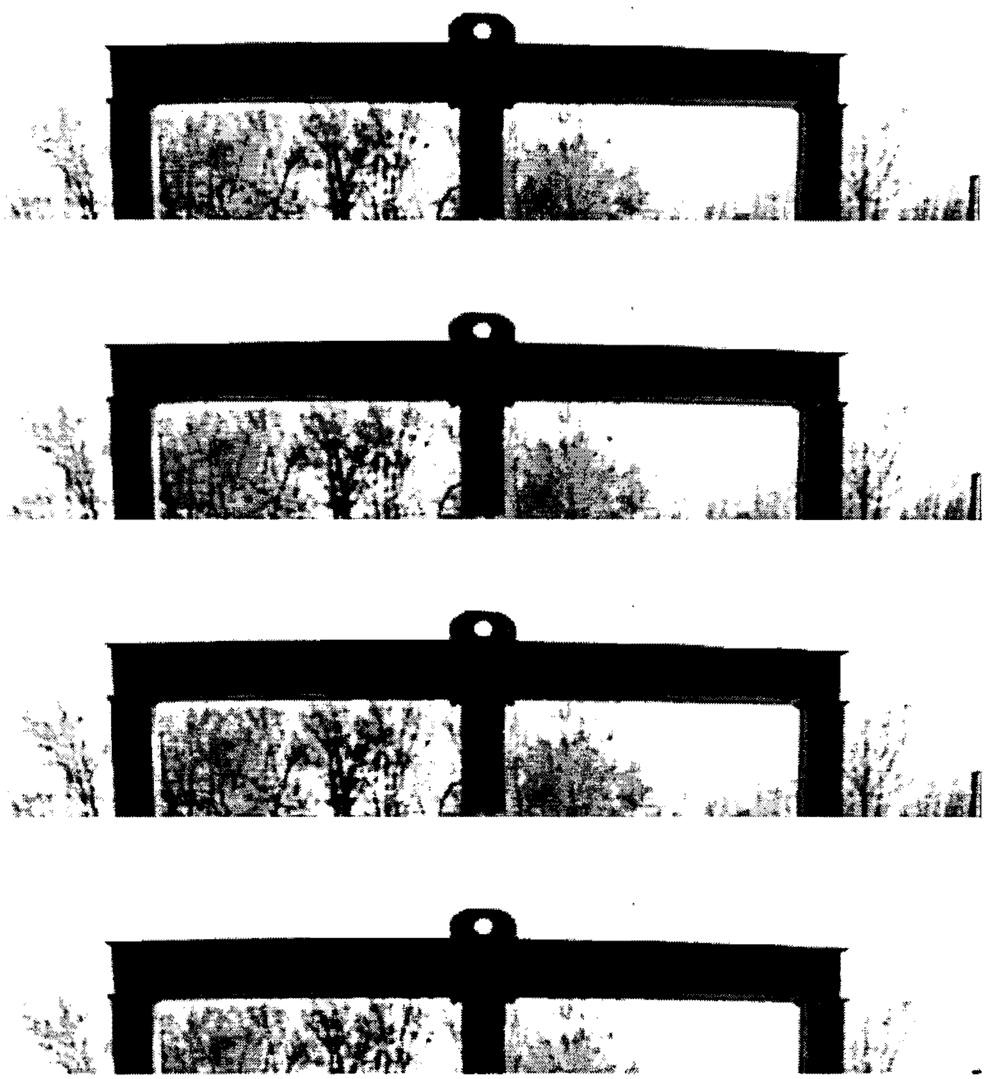


Figure 4.11. The Reaction Frame and Loading Apparatus.

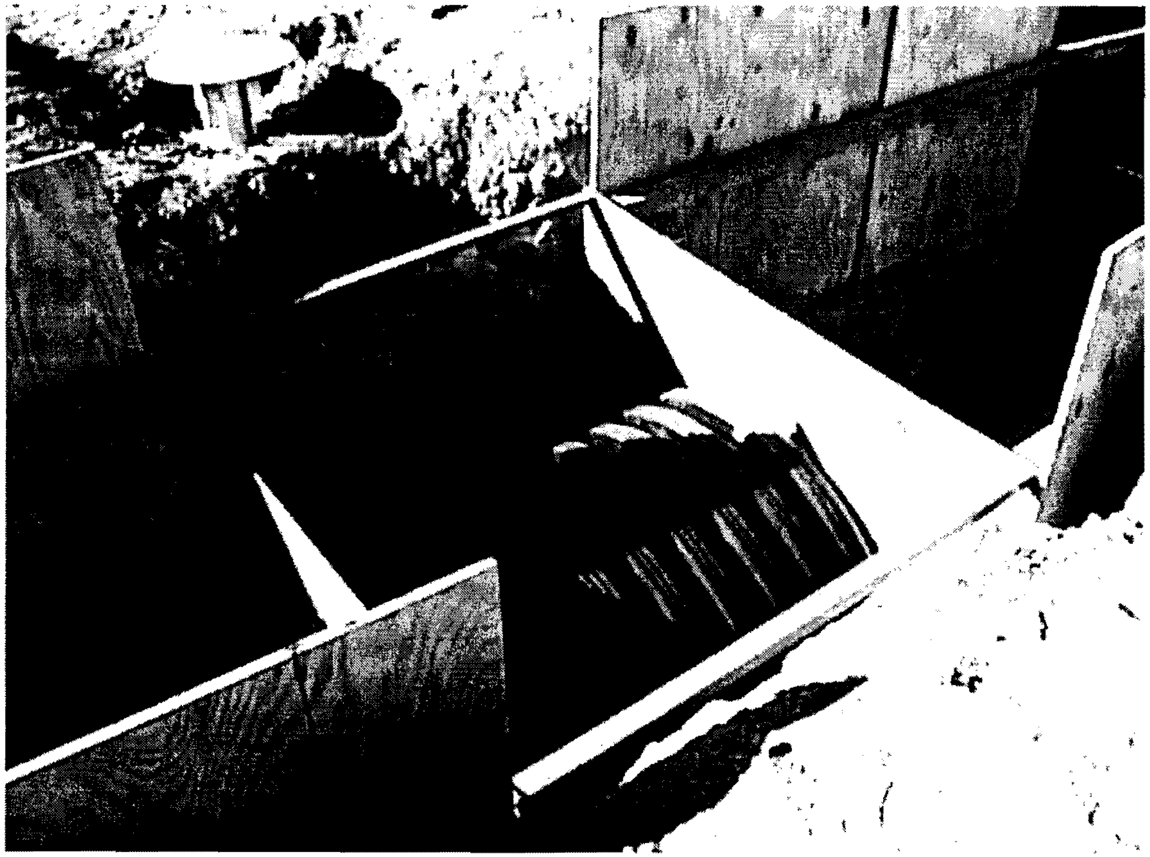


Figure 4.12. A Pipe Section Ready for the Placement of Backfill.

Figure 4.13. A Pipe Section Backfilled and Ready for Loading.

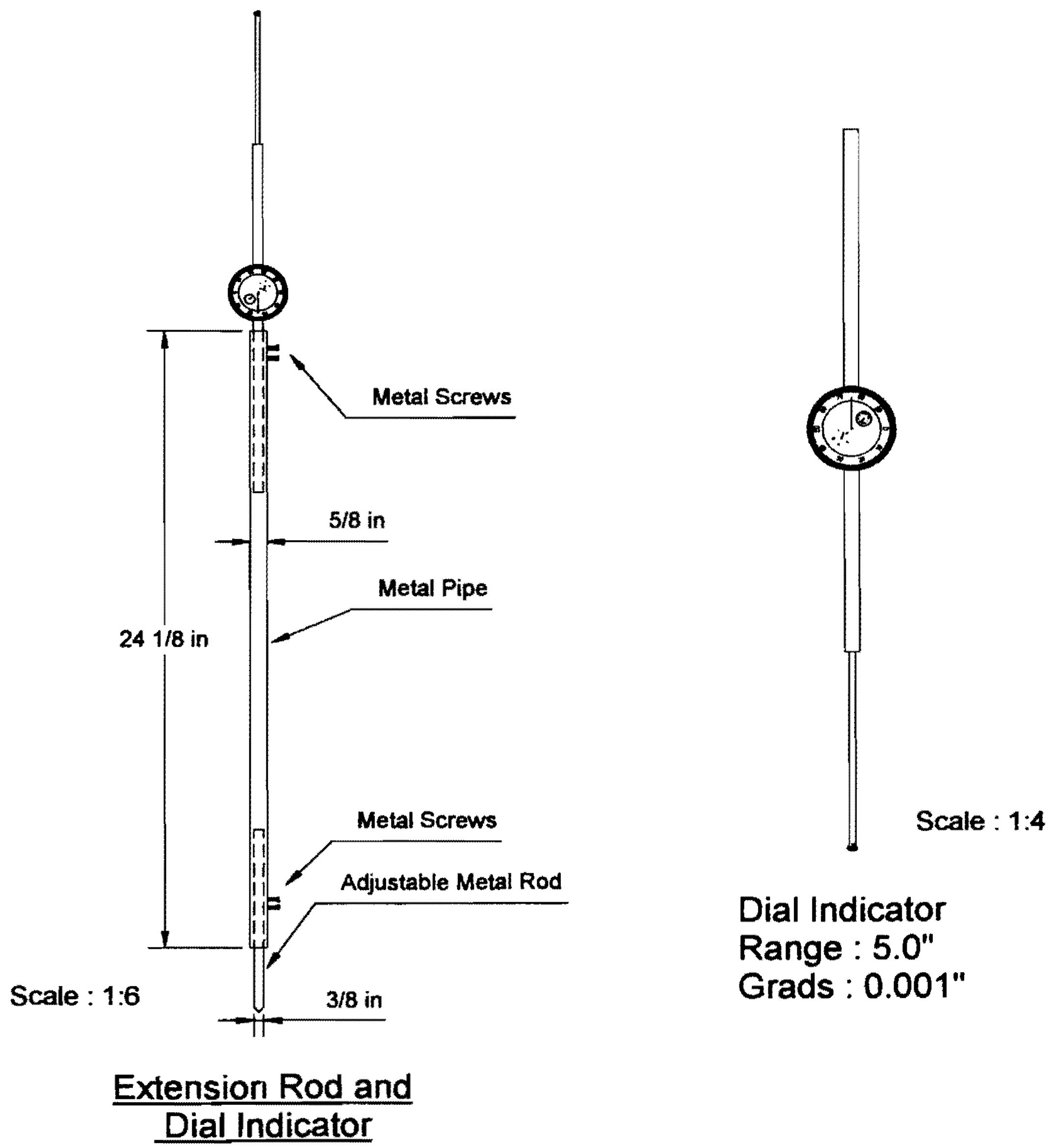


Figure 4.14. The Deflectometer.

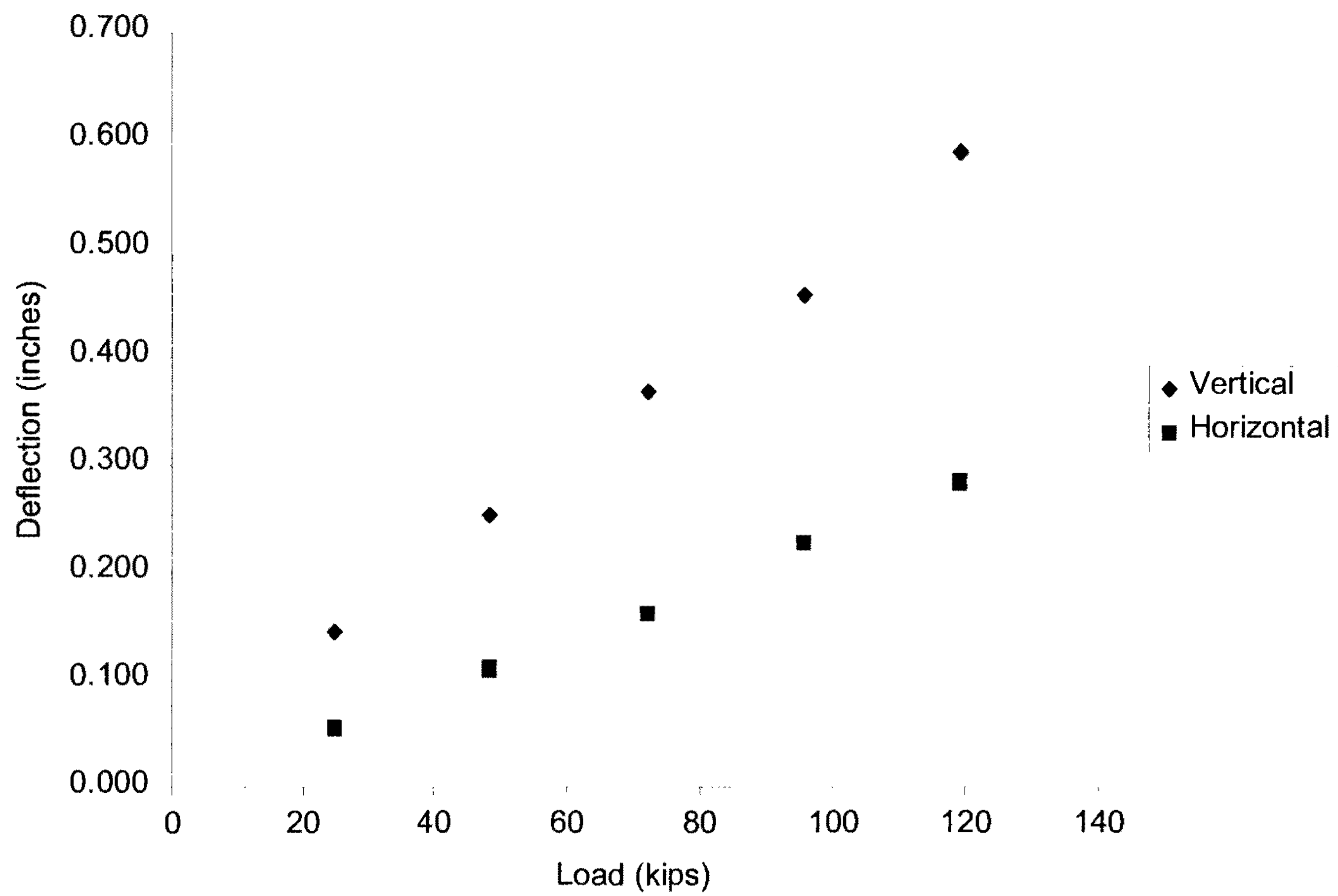


Figure 4.15. The Variation of the Average Deflection with Load for a 36 in. Pipe Backfilled with Gravelly Sand with Medium Compaction

CHAPTER V
PILOT CONSTRUCTION PROJECTS

5.1 Introduction

Eight culvert installation projects were selected from five different TxDOT districts for use as pilot construction projects. The purpose of the pilot construction projects was to install HDPE pipe according to the proposed specifications, observe the installation process and monitor the performance of the pipe. The data collected included information on site conditions, construction procedures, backfill materials and any specific problems encountered during construction. As a part of this monitoring effort, vertical and horizontal deflections were measured inside the pipe at several cross-sections shortly after installation and after the installations have been in use for several months. These data were subsequently used in a constructibility review of the proposed the specifications.

There are many variables that influence the successful installation and performance of the pipe. Therefore, it was important that the pilot construction projects would represent the broad range of these variables. Accordingly, the following factors were considered during the evaluation of the candidate construction projects:

a. Nominal pipe diameters, 36 in., 42 in. and 48 in.,
b. Single or multiple barrel installations,
c. Pipe manufacturers,
d. Installations with minimum cover and maximum fill height conditions,
e. Climatic conditions,
f. Native soil conditions
g. Types of backfill.

Based on this evaluation, eight installations located in five TxDOT districts were selected. These districts included: San Angelo, Laredo, Atlanta, Wichita Falls and Yoakum. The locations of the pilot construction projects are shown in Figure 5.1.

5.2 Monitoring Program

The monitoring program undertaken in this study consisted of a minimum of two visits to each construction site; the first visit at the time of pipe installation, and a second visit after the pipe had been in-service for several months. In some projects where the installations were completed during the first year of the research study, there was sufficient time to make two visits to monitor post-construction performance. With that exception, the approach used in all pipe installation projects were identical. The essential steps involved in the monitoring program are briefly explained in the following paragraphs.

Once the project had been scheduled for construction, TxDOT personnel contacted the researchers to inform them about the date of construction and exact location of the installation. The research team then traveled to the construction site so that they may collect necessary information during pipe installation. This information included:

a. General information (such as location of the project, highway number, directions to the job site, type of installation, number of installations, diameter of the pipe, pipe manufacturer, and length and/or number of installed pipes),
a. Construction procedures and equipment,
b. Trench dimensions and excavation process,
c. Placement and compaction of bedding,
d. Handling and installation of pipe,
e. Pipe jointing,
f. Placement and compaction of backfill. (Special attention was given to the haunching zone),
g. Vertical and horizontal pipe deflections at marked points,
h. Dynamic cone penetrometer readings,
i. Collection of backfill samples, and
j. General information of final installation.

The documentation of the construction process was generally in the form of video recordings and photographs whereas other details were recorded in field books. Samples of backfill were collected at the site, brought to Texas Tech University and tested to determine gradation.

The next step in the monitoring program was the post-construction inspection. Accordingly, the research team visited each construction site a few months after construction to evaluate the general performance of the pipe. The inspection checklist that was used during post-construction evaluation included the following items:

(a) Excessive pipe deflection,
(b) Joint separation,
(c) Wall buckling,
(d) Cracking of the pipe,
(e) Backfill washing into the pipe through the pipe joints,
(f) Erosion of Backfill,
(g) Pavement depression due to backfill settlement, and
(h) Pavement cracks related to the pipe installation.

As a part of pipe performance monitoring, deflection measurements were made prior to the installation of the pipe, after the placement of the backfill, and during post-construction visits. In some cases, the deflection readings prior to the backfill placement could not obtained because of the delay that it would cause in the contractor's construction schedule. The deflection measurements were made using the "deflectometer" that was shown in Figure 4.11. During these deflection measurements, as a first step, two diametrically opposite sides of the inside of the pipe were sprayed with white paint. This was done in both vertical and horizontal directions and at several cross-sections of the pipe along its length. Then, using a black permanent marker the exact points used in measurements were marked with a cross. Subsequently, the deflectometer was adjusted to the appropriate length and the initial distance from the tip of the traveling head to the pointed bottom of the steel rod was measured. Subsequently, readings were taken at the points marked inside the pipe. To obtain the inside pipe diameter, the reading at each point was subtracted from the initial length of the deflectometer.

5.3 Field Installations

As mentioned earlier, the eight pilot construction projects were located in five TxDOT districts. Of the five districts, San Angelo, Laredo and Wichita Falls had one installation each, while Atlanta and Yoakum had two and three installations respectively. All pipe installation projects were to be used for cross drainage purposes. The following sections describe each pilot construction project in detail.

5.3.1 San Angelo District Installation

5.3.1.1 General Information

This project was located on US 83 about 40 miles south from Junction, Texas. The culvert is 1.3 miles south from the intersection of US Highway 83 and Highway 39. The culvert was installed on a roadway widening project. Two parallel HDPE pipelines were installed in the same trench in a northwest-southeast direction. Each pipeline consisted of four 20 ft. sections of pipe. Figure 5.2 shows a plan view of the pipe installation. The HDPE pipe used in this installation was manufactured by Advanced Drainage Systems (ADS). The inside diameter of the pipes was 36 in. while the outside diameter was 41 in. The clear distance between the two parallel lines of pipe was 24 in. The width of the pipe trench varied between 13 and 14 ft. Accordingly, the clear distance between the outside of the pipe and the trench wall varied between 25 and 30 in. Figure 5.3 shows a cross sectional view. Table 5.1 presents the general information pertaining to the installation.

5.3.1.2 Equipment

This installation was completed by Jascon Contractors. The trench was excavated with a "CAT 320L" excavator equipped with a 30 in. wide bucket. A "P&H" 28-ton crane was used to place backfill material and pipe sections in the trench. Backfill was hauled to the trench with a one cubic yard capacity bucket. The loaded bucket was lifted by the crane, and moved to the location where the material would be dumped. A "CAT 110" front-end loader was used to move backfill material from the stockpile to the crane's range. Two types of compactors were used. One was a Mikasa tamping rammer operating at 600 vpm (vibrations per minute) and equipped with a four horsepower (HP) engine. The foot of the compactor was 12 in. wide and 14 in. long. The other compactor was a vibratory plate that had a five HP motor, and a plate 18 in. wide and 15 in. long. A "GX 120" water pump was used to pump water from a water truck into the backfill material that had already been placed in the pipe trench.

5.3.1.3 Construction Procedure

This pipeline was laid in a rock cut. The contractor started the day's work by placing a layer of crushed limestone varying between 1 inch and 8 in. in thickness to fill parts of the trench that had been over excavated. This layer was compacted before the bedding was placed. Sieved river gravel was used as backfill material and bedding for the installation. The bedding material was brought to the trench from a nearby stockpile using a front-end loader. The material was poured into the bucket with one cubic yard capacity; then the bucket was lifted by the crane and the material dumped into the trench. The layer of bedding was not compacted, instead placed loosely over the crushed limestone layer.

After the bedding was placed, the contractor shaped the bedding by using a template as per the specifications as shown in Figure 5.4.

A front-end loader was used to transport the pipe from its on-site storage location to within the range of the crane. The pipe was tied to the front-end loader using the following procedure. First, two chains were laid on the ground in a straight line and the pipe was rolled on top of the chains; the two ends of chain were then brought together over the top of the pipe and attached to the front-end loader. Once the pipe section was brought to the crane, the pipe was lifted by the crane and placed in the trench. Chains were hitched to the one third and two third points of the pipe section, as recommended by the manufacturer. Once the first pipe section was placed on the bedding, the second pipe section was transported to the trench to be connected to the first. To minimize potential damage to the pipes, HDPE pipe manufacturers recommend the use of nylon slings, rather than chains during their handling. However, such nylon slings were not available with the contractor at this project site.

It is customary to place plastic pipe with the spigot end facing downstream to minimize joint leakage. The laying of the pipe started from the downstream end following the usual practice. The spigot end (with the gasket) and the bell were lubricated using lubricant supplied by the pipe manufacturer. The spigot end of the second pipe section was then pushed into the pipe section already lying on the ground. This was accomplished while the second pipe was still a couple of in. above the bedding as it was partially carried by the crane. Once the joint was completed, the second pipe section was allowed to rest on the bedding. Subsequently, the chains that had been used to carry the second pipe length were pulled from beneath the pipe.

The contractor experienced some difficulty in assembling pipe joints as the spigot end refused to go all the way into the bell. When the pipe section with the spigot end was pushed into the bell, both sections of pipe moved in the direction of the applied force. A representative of the pipe manufacturer who was present at the jobsite determined that the problem due to incompatibility between the design of the bell and the gasket at the spigot end. Pipes with gaskets of newer design were supplied by the manufacturer allowing the joints to be assembled in a satisfactory and expeditious manner.

A second minor problem arose at this site because of failure to allow for the overlap length at pipe joints. The length of the each pipe section is generally specified as 20 feet. However, when you allow for the overlap length of 4 in, each pipe section is only 19ft. and 8 in. long. As a result, the total length of four pipe sections that had been ordered for this installation was less than 80 ft. length specified in the contract. This may become a common mistake among contractors who are not familiar with this particular pipe product.

The contractor compacted the backfill in layers of four to six in. (loose measurement) though the draft specification from TxDOT allowed layers of up to 8 in. thick (loose measurement). The number of passes on each lift of backfill material varied between two to four. Figure 5.5 shows the backfill being compacted with an impact rammer. In this installation, there was ample room between the outside of the pipe and the trench wall to allow compaction equipment. The available clearance (approximately 25 to 30 inch) was much larger than that provided by the minimum trench width specifications. The contractor made a further mistake while calculating the horizontal clearance between the two pipe barrels. The center to center distance was calculated using the inner pipe walls, while TxDOT specifications called for two feet clear distance between the outer walls. This may also become a frequent mistake as pipes are generally specified and referred to by their inner diameters. Figure 5.6 shows both pipes with the backfill compacted up to the crown of the pipe.

Deflection measurements were taken of the southern pipeline using the deflectometer. Pipe diameter was measured before any backfill was placed and after the backfill had been placed and compacted up to the crown of the pipe. Figure 5.7 shows the percent deflections at the marked points relative to the initial pipe diameter. The sign convention in reporting pipe deflection is as follows. The positive values in the "deflection" axis, represent a diametrical reduction in the pipe and the negative numbers represent an increase in the pipe diameter. This diameter change in both directions will result in alteration of the shape of the pipe from circular to elliptical.

After the backfill was completed, dynamic cone penetrometer (DCP) readings were taken to measure the level of compaction of the backfill. Three different locations were chosen to perform the test; one location was in the space between pipes, and the other two on the outer side of each pipe. The readings were taken with the backfill up to the crown of the pipe. On the northern side of the installation, the tip of the DCP refused to penetrate further than 30 cm. Refusal was judged to be when the tip did not advance any further into the soil despite repeated blows. Table 5.2 shows the results of the DCP readings at the three different locations.

As mentioned before, a sample of the backfill was taken to the soils laboratory at Texas Tech to determine the particle size distribution. The results were compared with the draft specifications to verify if the backfill gradation met specification requirements. The particle size distribution of the backfill is shown in Figure 5.8. The backfill material meets the draft specifications except for the particles larger than 3 mm in size. Based on the ASTM D-2487 designation (the "Unified Soil Classification System"), the backfill can be classified as "SW", (Well-graded sands with little or no fines).

5.3.1.4 Post Construction Monitoring

This pipe installation was inspected twice after the construction had been completed. The first inspection was performed on this installation three months after the date of construction. The embankment was built up to the level required by the projected specifications. The pavement had not been fully constructed, and no safety end treatments were in place at that time. The height of the fill at the highest point was approximately 12 ft. The pipelines had been constructed only halfway across the road; the downstream end of the pipeline was buried, and blocked off with a wooden board. The contractor, after the initial construction, had placed a board on the downstream end of the pipelines since the remainder of the installation was going to be built at a later stage. As a result, some water and silt had accumulated inside the pipe, making it difficult to make the pipe deflection measurements. Pipe joints were examined and no evidence of distress was found.

Figure 5.9 shows the deflections, in percent of the initial pipe diameter, with the overfill in place. It is apparent that the vertical deflections had changed significantly from negative values (increase in diameter) immediately after backfill had been placed to the top of the pipe, to a positive deflection (reduction in diameter) at the time of the first inspection. This increase of the vertical deflection was due to the placement of the 12 feet of fill. On the average, the deflections changed from -0.5 % to 1.7 % in the vertical direction. On the horizontal direction, less change occurred; the initial deflection of 0.8 % increased to 1.2 %. However, all of these deflections were well below the generally accepted limit of 5%.

The second post-construction visit was done 16 months after construction. At the time of this visit, both pipelines were completely installed according to the project design. Safety end treatments were constructed at both ends of the pipelines, as can be seen in Figure 5.11.

The pipelines that were monitored in this project were on the northbound lanes. The northbound lane was completely paved and open to traffic although the contractor was still working on the southbound lane pavement. The pavement over the pipe installation was inspected and no distress was observed. Figure 5.11 shows the condition of the pavement during the second post-construction monitoring. The pipe was inspected and no distresses of any type were found on the pipe or the joints. However, significant accumulation of silt up to 1/3 of the height of the pipe was found on the downstream end as shown in Figure 5.12.

The pipe deflections were obtained again to gauge the performance of the pipe. The measured deflection are shown in Figure 5.13. It can be noted that the vertical and horizontal deflections did not change significantly between the two post-construction visits. In the first inspection, the deflections in the horizontal and vertical directions were 1.7 % and 1.2 % respectively. In the second inspection the deflections in the horizontal and vertical directions were of 1.6 % and 1.1 % respectively. Evidently, the pipe buried under 12 feet of fill, maintained the same shape and developed no distresses 13 months after the first inspection.

5.3.2 Laredo District Installation

5.3.2.1 General Information

This project was located in Laredo in the southwest corner of the intersection of US Highway 83 and Sierra Vista Boulevard. The culvert was installed on the new southbound lanes of a highway-widening project. One HDPE pipeline for cross drainage was installed and connected to an existing concrete catch basin. Cherokee Bridge and Road, Inc. was the contractor for this pipe installation. Advanced Drainage Systems (ADS) pipe with inside diameter of 36 in. was used in this project. The pipeline started at a concrete catch basin at the median of the proposed road and ended on the west side of the road. A plan view of the installation is shown in Figure 5.14. The locations where deflection measurements were taken are also marked on this figure. The bedding comprised of compacted granular material, identified as Type D aggregate. The bedding layer varied from 6 to 7 in. in depth. The backfill was placed up to 1 ft. above the top of crown of the pipe, followed by three feet of native soil. Six inches of lime stabilized base was placed then above the native soil, followed by 2 in. of asphalt concrete. A cross sectional view of the installation is shown in Figure 5.15. Table 5.3 summarizes the general information pertaining to this the installation.

5.3.2.2 Equipment

The equipment used for construction is as follows. A "Molt CAT 416 Series II" front-end loader with a backhoe excavator was used to excavate the trench and to place the backfill inside the trench. A "P&H" 32-ton crane was used to place the pipe sections in the trench, while a "Ali Chambers" forklift was used to move the pipe sections from the stockpile to within the crane's range. A "Mikasa MT 8.5 H" impact tamper was used to compact the backfill. Water, dispensed from a truck, was used to increase the moisture content of the backfill.

5.3.2.3 Construction Procedure

The trench, which was 11 feet in width, was excavated in a sandy soil with fines. The trench had vertical sides for a height of 5 feet, then a flat portion of 2 feet, then another section of 2 feet height at a slope of one vertical to one horizontal.
After the excavation was completed, bedding material was placed and compacted. Although it is customary to place the pipe starting from the downstream end, in this case, the contractor started installation from the upstream end at the catch basin to avoid problems later on in jointing the pipe with the basin. Figure 5.16 shows the HDPE pipe after it had been connected to the existing concrete catch basin. The pipe sections were brought from the stockpile to within the crane's range with the forklift, and then fastened with a chain from the middle point of the pipe section. Once the pipe had been secured, the pipe was lifted by the crane and placed in the trench.

After the pipe section had been placed and aligned correctly, some backfill material was poured over the pipe with the front-end loader and then dispersed to hold the pipe in place. In addition, some 2 in × 4 in timbers were placed near the catch basin to hold the pipe in place. After the backfill was placed in this pipe section, it was wetted to increase the moisture content, thus making the compaction process more efficient. The spigot of the section installed was lubricated and the next pipe section was placed into the trench. The bell of the following section was lubricated and aligned with the pipe already in place. To joint both pipes properly, the second pipe section was pushed against the first pipe section with the front-end loader, and to avoid damaging the pipe, 2 in × 4 in timbers were placed between the pipe and the loader. This procedure greatly facilitated the pipe jointing. The remaining sections were installed in the same manner. Backfill was placed in 8 to 10 inch lifts (loose measurement) and compacted with 2 to 3 passes with the impact tamper. Backfill placement and compaction process is shown in Figure 5.17.

Deflection measurements were taken at the marked points prior to placement of the backfill. The deflections were measured at two points after the placement of the backfill up to the crown. Figure 5.18 shows the two points measured prior to and after the placement of the backfill. Once again, it is apparent that the vertical diameter had increased and the horizontal diameter decreased due to the placement and compaction of the backfill is apparent; the pipe had an elliptical shape after construction. It can be seen that the deflections in both directions changed by approximately 1 % of the average, due to the placement of the backfill. The rest of the marked points were measured in the post-construction visits only.

A sample of the Type D backfill was taken to the laboratory to determine its particle size distribution and the results are shown in Figurte 5.19. The backfill material, which can be classified as "Well-graded gravel with little or no fines" (GW) according to the ASTM D-2487 designation, met the gradation requirements of the draft specifications. Dynamic cone penetrometer (DCP) readings were not taken in this project.

5.3.2.4 Post-Construction Inspection

This installation was inspected two times after its initial construction. The first of these post-construction inspection took place three months after the initial installation of the pipe. At the time of this inspection, the pipe had close to 5 feet of cover and the pavement had already been completed. However, the road was being utilized by construction traffic only. The safety end treatment was still not in place. The pavement and the pipe were inspected and no signs of distress were found. Figure 5.20, shows the installation at the time of the first post-construction inspection.

Pipe deflection measurements were taken at all marked points, and the deflections expressed as a percentage of the initial diameter prior to placement of the backfill were calculated. Figure 5.21 shows the pipe deflections at marked points after three months of the installation. Comparing points 3 and 3C, a small reduction in the vertical diameter can be noticed when compared with the measurements made soon after installation. However, the elongation of the vertical diameter due to compaction had not fully been negated by the placement of fill and the passage of construction vehicles. In the horizontal direction, there was no significant change. On the average, deflections in the vertical and horizontal directions were in the order of -0.4 % and +0.6 % respectively.

The second post-construction inspection was carried out 16 months after the initial construction date. At this time, the road was open to regular traffic and the safety end treatment had been constructed (see Figure 5.22). The pavement was found to be in good condition with no noticeable distresses. Figure 5.23 shows the condition of the pavement at the time of the second post construction monitoring. There were no distress on the pipe or pipe joints.

Pipe deflections were taken and no significant change was noticed. Figure 5.24 shows the pipe deflections sixteen months after construction. The deflections in the most critical region, just below the traffic (points 3, and 3C), remained almost the same as it was in the previous visit. On the average, the deflections in the vertical and horizontal direction were 0.2 % and 0.5 %, respectively which were well within the acceptable limit of 5%.

<u>5.3.3 Atlanta District Installation</u>

5.3.3.1 General Information

Two multiple barrel installations were made in this district. One was a triple barrel installation and the other a double barrel installation. The sites were located on FM 997 approximately 3.5 miles south of Daingerfield. The first installation, referred to as Installation A1, is 2.4 miles south of the intersection of FM 997 and FM 144 and the second one, referred to as Installation A2, is 600 feet south of the same intersection. In both installations existing corrugated metal pipe culverts were replaced by HDPE pipe. Installation A1 consisted of three HDPE corrugated pipelines with a diameter of 42 in. and installation A2 consisted of two HDPE corrugated pipelines with a diameter of 36 in. Table 5.4 shows the general information corresponding to both installations. Both culverts were

installed by the TxDOT Atlanta District Maintenance crew. HDPE Pipes were manufactured by Quail Piping Products, Inc.
Figure 5.25 shows the plan view of installation A1. Also shown in this figure are the locations of the deflection measurement points.

5.3.3.2 Equipment

The construction equipment used on both installations were the same. The trenches were excavated using two "Badger 460" hydroscopic excavators working in tandem from either side of the culvert. The material excavated from the trench was removed from the job site by using hauling trucks. A front-end loader with a backhoe "New Holland 675 E" was used to bring the pipe sections from the stockpile to the trenches.). The compaction of the backfill between the pipe sections was accomplished with a vibratory plate, "Wacker 4 HP" weighing 184 pounds, while an air tamper was used to compact the haunching zone on the first lifts of backfill. The same pneumatic wheeled New Holland 675 E front-end loader that was used to move pipe sections was used to compact the road base above the pipe. The base material was leveled with a "Galion 8'0" grader before the road was opened to traffic.

5.3.3.3 Construction Procedure

The trench was excavated on a clayey-sandy soil, and had a width varying between 19 and 20 feet and a depth of 8 feet. The backfill material was brought from Granite Mountain, Arkansas. TxDOT maintenance crew indicated that a 6 inch layer of this material was going to be used in the bedding, and in the region 1 to 1.5 feet above the crown of the pipe. A depth of between 1.5 ft and 2.5 ft of salvage material from the rehabilitation of Highway 11 was used as a base material. The clear distance between pipe barrels was set at 2 feet. There was 2 feet of clearance between the pipe and the trench on the West side of the installation. However, on the East side, there was just 1 foot of clearance, making the compaction in that area somewhat difficult. Figure 5.26 shows the cross section of the trench of the installation A1.

After the trench was excavated, 6 in. of bedding was placed without compaction at the bottom of the trench. The bedding was not shaped to fit the bottom of the pipe. The front-end loader transported the pipe sections from the stockpile to the trench. The pipes were attached to the front bucket with chains. The pipe sections were subsequently detached from the front-end loader, and were attached to the hydroscopic excavator. A chain was wrapped around the center point of each pipe section, which was then lowered into the trench. As mentioned previously, this was a three barrel installation, and the first center pipeline was placed first. After the alignment of the first section, some backfill material was poured on the sides of the pipe to hold it in place. The crew then proceeded to install the west and the east pipe sections in that order. The distance between the pipes and the trench walls was checked, and the crew found that more material from the east wall of the trench would have to be removed in order to clear the path for the east pipeline; this widening was carried out before any further backfilling work was continued. Backfill material was brought from the stockpiles by the hauling trucks and was then placed inside the trench using the buckets of the excavators. After all the pipe sections had been placed, a lift of 8 to 10 in. in thickness (loose measurement) was placed at the bottom of the trench and compacted with the vibratory plate compactor. The loose material left adjacent to the pipe in the haunching zone was compacted with the air tamper. The lifts that were placed up to the spring line were approximately between 10 and 12 in. in thickness. The lifts were not of uniform thickness at the sides of the installation between the pipes and the trench above the spring line. In these areas, the lift thicknesses varied from 12 to18 in. On the east side of the installation, where the spacing between the pipe and the trench wall was 1 foot, compaction was carried out with the air tamper. After the backfill material had been placed almost up to the crown of the pipe, the hauling trucks were used to discharge the material from the sides of the trench directly over the pipe. The backfill was then spread uniformly with shovels and the excavator buckets. The material from an elevation of 1.5 to 2 feet above the crown of the pipe was compacted with the pneumatic tires of the front-end loader. After this material had been placed and compacted, base material was placed in lifts of 1.5 to 2 feet and compacted with the front-end loader. At least ten passes from the front-end loader was used to compact the cover and the base material. On the following days, the maintenance crew placed between1 to 2 in. of cold mix asphalt concrete on top.

DCP readings were taken in the northern side of the installation between the pipes. Table 5.5 shows these test results. In addition, deflection readings were taken at the points marked within the pipe. No readings were taken just after the pipes had been laid on the bedding and prior to the placement of the backfill. As shown in Figure 5.27, the vertical diameter has decreased and the horizontal diameter has increased. It should be noted that this was different from the trend seen in San Angelo and Laredo installations. In these previous installations, the vertical diameter increased as a result of backfill compaction on either side. The deflection pattern seen in the Atlanta District installation may be an indication that there was less compaction at these installations compared to the previous projects. On average, deflections in both directions were in the range of 0.4 %.

Generally, the same construction procedure used in installation A1 was used in installation A2. As mentioned before, this installation replaced a double barrel culvert of corrugated metal pipe. The plan view of installation A2 in Atlanta District can be seen in Figure 5.28.

The pipe trench was approximately 15.6 feet wide and 6 feet deep and was excavated in a clayey-sandy soil. The bedding was compacted with 1 to 2 passes of the vibratory plate. After the pipe sections had been placed, the backfill was placed in lifts varying from 12 to 18 in. in thickness and compacted with the vibratory plate. The air tamper was used in the haunching zone. After the backfill material had been placed up to the spring line, the two western sections of the installation were backfilled. The maintenance crew then realized that more backfill material would be needed; the amount of backfill available at that time was insufficient to fill the eastern sections beyond the spring line. One foot of cover was placed above the crown of the pipe and compacted with the pneumatic tires of the front-end loader; at least 10 passes were given to the cover. Later, base material was placed in a lift varying from 1.0 to 1.5 feet in thickness and compacted with the pneumatic tires of a fully loaded hauling truck; again, at least 10 passes were given. By the time this operation was complete, a truck containing more backfill material had arrived at the site. The truck discharged all the material on the northern side of the installation between the northern pipe and the trench wall. Later, the material was dispersed between the pipes and the southern side of the installation. The backfill material on that section was neither placed in lifts nor compacted. One to two passes of vibratory plate were given at the top surface of the backfill material. Later, the base material was placed and compacted as the previous section. The grader leveled the base material to match the existing pavement, and the road was opened to traffic. The cold mix asphalt pavement was to be placed in the following days. The cross section of the trench of Installation A2 in Atlanta District can be seen in Figure 5.29.

Deflection readings were taken in this installation as well. They are shown in Figure 5.30. Once again, the deflection readings indicated that there was reduction in the vertical diameter. In some locations, the deflections were as high as 1.5%.

The particle size distribution of the backfill was determined and is shown in Figure 5.31. Although, there were some particles larger than 1 inch (25mm), the gradation fitted quite well within the specifications band specified in the draft specifications. According to the ASTM D-2487 designation, the material could be classified as Well-graded gravel with little or no fines (GW).

DCP readings were taken in both ends of the installation between the pipes, and the results are shown in Table 5.6.

5.3.3.4 Post-Construction Inspection

The research team visited both installations in Atlanta District for post-construction inspection 5 months after the date of construction. However, during this visit, no deflection measurements could be made in installation A1 (42" triple barrel), because the pipelines were full of water. No headwalls had been constructed at that time. It was noticed that some erosion of the backfill material and base material had occurred from the sides of the installation. The eroded material had deposited on the pipe openings on the downstream side. Figure 5.32 shows backfill erosion at downstream end of installation, A1. It was apparent that the granular backfill material used in the installations was susceptible to erosion, especially in locations where there is potetial for flooding. The accumulation of material at the downstream end of the pipe may then obstruct the free flow of water and may cause future problems. This problem can arise in any culvert installation regardless of the type of pipe used in it. To avoid this problem, it is recommended that backfill be confined through appropriate measures such as rip rap and headwalls to protect the material from erosion. At the upstream end of the installation, the entrance to the pipelines was partially blocked by dead branches from a tree.

A new pavement layer had been placed on the road section where the trench was excavated. The cold mix pavement had been expanded to cover up to 65 feet on each side of the installation. No signs of distress were found on the new pavement section. The new layer of pavement can be seen in Figure 5.34. Installation A2 (36" double barrel) was examined as well. The installation presented some erosion on the west end of the pipelines. No headwalls had been constructed on this culvert either. Figure 5.35 shows the downstream end of the installation A2 at the time of the inspection. The bituminous pavement on this installation had also been replaced.

The newly constructed pavement layer extended approximately 60 feet on each side of the installation. A maximum settlement of about 1 inch on the pavement section was observed over the trench. A narrow crack of 2 feet in length was found in the northbound lane on the edge of the depressed area. The width of the crack was approximately 0.04 in. (1 mm). It was clear that some settlement had occurred in this area. Figure 5.36 shows a plan view of the area where settlement and the crack was observed.

Deflections were measured in the installation; it is apparent that the pipe experienced more deformation in both directions (See Figure 5.37). High deflections in the order of 3 % can be seen in the vertical and horizontal directions. On the average, deflections in both directions were 2.1 %, and stayed within the specified limits of 5.0 %.

5.3.4 Yoakum District Installations

5.3.4.1 General Information

The selected pilot projects included three single barrel installations in Yoakum District. All of these projects were culvert replacement projects where existing metal pipe were replaced with HDPE pipe. They were all located on FM 530 with approximately 0.5miles from one pipe installation to the next. These projects were located approximately 8.4 miles south from the intersection of FM 530 and road 90A in Halletsville. Thirty-six inch, (900mm), 42" (1050mm) and 48" (1200mm) corrugated HDPE pipes were used for installations Y1, Y2 and Y3, respectively. The manufacturer of the HDPE pipe was Hancor. Table 5.7 provides the general information pertaining to the Yoakum District installations. By the time the researchers arrived at the job site, two out of the three pipelines had been installed already (installations Y2 and Y3), and the third one (installation Y1) was installed half way across the road. Therefore, the documentation of the installation was based on the remaining half of the pipe installation Y1. According to the information provided by the TxDOT inspector, same procedures were used in all three installations.

5.3.4.2 Construction Procedure

At each pipeline installation, the contractor worked on one lane of the road at a time. The other lane was left open for traffic. The construction procedure used on the east half of installation Y1 is described below. A plan view of this installation can be seen in Figure 5.38.

The existing metal pipe was uncovered by excavating with a "Komatsu PC 300LC" excavator. The trench was excavated in a clayey-sandy soil. Based n the information collected from TxDOT inspector, the trench widths of the completed installations are as shown in Table 5.8. These trench widths meet the requirements specified in the draft specifications. After excavation, the subgrade was compacted with 2 to 3 passes of an impact tamper (Ground Pounder Model R450/1). Next, 6 in. of bedding was placed and compacted. The backfill used in these installations consisted of Flexible Base Type D Grade 1 as per item 247 of TxDOT standard specifications. This material was used as backfill up to the crown of the pipe. Backfill was placed in lifts of 10 to 12 in with the exception of one lift that was 18 in. thick. The compaction of the layers consisted of typically 2 to 3 passes of impact tamper. Once the backfill had been compacted up to the crown of the pipe, 1 foot of cement stabilized limestone base with 7% Portland cement was placed and compacted. . Soon afterwards, a 2-inch premix layer of asphalt concrete was placed above the base layer. Figure 5.39 shows the cross section of installation Y1 with the 36 in. pipe.

Deflection readings were taken in all three installations. Figure 5.40, Figure 5.41 and Figure 5.42 show the deflections for the installations Y1, Y2 and Y3, respectively. It can be seen that installations Y1 (36" pipe) and Y2 (42" pipe) generally have negative vertical deflections (an increment in the vertical diameter) and positive horizontal deflections (a reduction in the horizontal diameter). On the average, deflection values were approximately 1.2 % and 1.0 % for Y1 and Y2 installations, respectively. In installation Y3 (48" pipe), deflections were not very uniform along the pipeline; however, the deflections are quite small (not exceeding 0.35 %).

A sample of the Type III backfill material was obtained and tested to determine its particle size distribution. The results are shown in Figure 5.43. The backfill material met the requirements of the draft specifications except for particles larger than 1 inch (25mm). According to the ASTM D-2487 designation, the material could be classified as "Well-graded gravel with little or no fines" (GW).

DCP readings were taken in two out of the three locations. One test was performed at installation Y2 at the southwest corner, where the cement-stabilized material was placed (point A). The readings at that point were extremely high and the test was abandoned. Table 5.9 shows the blow counts recorded in this incomplete test. Two other tests were conducted on installation Y1. One test was on the southwest corner of the installation (point B), where the material was not cement-stabilized, and the other one in the middle of the northbound lane (point C), before the foot of cover was placed. Table 5.10 provides these results.

5.3.4.3 Post-Construction Inspection

A site visit was made six months after the initial construction for post-construction inspection. At the time of this visit, the pipes were partly filled with water. Also, there was significant silt accumulation in the pipe (See Figure 5.44). Concrete headwalls had been constructed around the sections of safety end treatment of all installations, and metal pipes had been added to help prevent obstruction at the pipe entrances.

It was found that in installation Y3 (48" pipe), one of the joint gaskets had come loose on the west side of the installation (See Figure 5.45). The problem occurred at the joint between the eastern inner pipe and the safety end treatment section (see Figure 5.46).

The replaced pavement sections appeared to be in good conditions and did not show any type of distress related to the pipe installations. It was not possible to take deflection readings in the installations due to the water and silt accumulation inside the pipes.

5.3.5 Wichita Falls District Installation

5.3.5.1 General Information

Wichita fall District pilot project was located on FM 1197 approximately 3.7 miles north from the intersection of US 82 and FM 1997 in Henrietta. The project was a multiple barrel installation consisting of four pipelines of 48" (1200 mm) HDPE corrugated pipe. These new pipelines replaced existing concrete pipelines. A summary of the information pertaining to this project is given in Table 5.11.

The culvert was located in a flood zone, and there was some concern regarding the backfill material proposed for this installation. There was some risk that the backfill materials could be easily washed out by the water from the stream. Therefore, the district planned to construct headwalls at the ends of the culvert. A Plan View of the installation can be seen in Figure 5.47.

5.3.5.2 Construction Procedure

This installation was completed by TxDOT maintenance crew. The pipe was manufactured and delivered to the TxDOT yard by Quail Piping Products, Inc. The trench was excavated with two hydroscopic excavators and the material was removed with hauling trucks. The existing concrete pipes were removed with a "Case 621" front-end loader. One impact tamper "Wacker Packer BS 700" was used to compact the backfill material. Once the trench was excavated, approximately 6 in. of bedding material were placed. The bedding was not compacted or shaped. The northern pipeline was installed and aligned, and the rest of the pipelines were placed 2 feet apart (measured from the outer diameters) from each other. The distance between the outer pipelines and the trench walls was approximately 42 in. A cross section of the trench can be seen in Figure 5.48.

It is important to note that the pipe sections were transported from the TxDOT yard to the job site at the time of the construction. At this construction site, chains or slings were not used to unload the pipe from the delivery truck. Instead, they were pushed from the side of the delivery truck and allowed to fall on the ground. This procedure is not recommended by pipe manufacturers. Nevertheless, there was no visible damage to the pipe as a result of improper handling.

Once all four pipes had been installed and aligned, the backfill material was discharged directly into the trench from the end dump trucks. The backfill material used at this site was coarse granular material that was quite uniform in gradation. Some attempt was made to compact the material but compaction did not prove to be very effective because of the uniformity of backfill material. Consequently, most of the backfilling took place with minimal compaction. Figure 5.49 shows the backfill placement in the pipe trench. Once the backfill material was brought up to the existing pavement grade, a layer of asphalt pavement was placed.

Deflection readings were taken only at the middle pipe sections of each pipeline (directly under the roadway) and are shown in Figure 5.50. It can be seen that in locations 1, 3 and 4 the vertical pipe diameters tended to decrease. Only location 2 had an increase in the vertical diameter (or a negative deflection). On the average, pipe deflections were in the range of 0.38 % in both directions.

DCP readings were taken in two locations between the southern pipelines, and it should be noted that the values obtained are very low in comparison with the values obtained in other districts. Table 5.12 shows the results of the DCP tests.

A sample of the backfill material was taken and its particle size distribution was determined (see Figure 5.51). The material used as backfill met the requirements of the draft specifications. According to ASTM D-2487, the material can be classified as "Poorly-graded gravel with little or no fines" (GP).

5.3.5.3 Post-Construction Inspection

This site was visited three months after the date of construction. Concrete headwalls had been constructed at the ends of the culvert (see Figure 5.52). The pavement above the installation was in good condition and did not reveal any type of distress related to the culvert installation. Figure 5.53 shows the condition of the pavement above the pipe installation 3 months after construction. No distresses were noticed in the pipes or their joints.

Pipe deflections were measured and no significant change was observed (See Figure 5.54). Locations 1 and 3 had reduction in the vertical diameter, while location 1 experienced an increment in the horizontal diameter. The average deflections were in the range of 0.6 % and 0.4 % in the vertical and horizontal direction, respectively.

Table 5.1. General Information on the Pilot Project in San Angelo District

District	San Angelo
Location	US 83
No. of Installations	1
Installation Type	Multiple (2 barrel)
Diameter of the Pipe (in)	36
Length of the Pipe (ft)	120
Safety End Treatment	4-SET (900 mm)(1:3)
Height of fill (ft)	12
ADT	600
Date of Installation	25-Jan-99
Date of 1st Inspection	24-Apr-99
Date of 2nd Inspection	24-May-00

Table 5.2 DCP Readings from San Angelo District.

Depth (cm)	South Side (blows / 10 cm)	Center (blows / 10 cm)	North Side (blows / 10 cm)
10	4	5	3
20	6	11	8
30	10	18	24
40	12	52	Refusal
50	15	38	
60	21	17	
70	20	14	
80	14	11	
90	13	15	
100	21	10	
110	Refusal	62	

Table 5.3 General Information on the Pilot Project in Laredo District

District	Laredo
Location	US 83
No. of Installations	1
Installation Type	Single
Diameter of the Pipe (in)	36
Length of the Pipe (ft)	94
Safety End Treatment	CH-FW-O(M) HDWLS
Height of fill (ft)	5
ADT	11,600 (1997 count)
Date of Installation	28-Jan-99
Date of 1st Inspection	25-Apr-99
Date of 2nd Inspection	24-May-00

Table 5.4. General Information on the Pilot Projects in Atlanta District.

District	Atlanta	
Location	FM 997	
Installation No.	A1	A2
Installation Type	Multiple (3 barrel)	Multiple (2barrel)
Diameter of the Pipe (in)	42	36
Length of the Pipe (ft)	50	50
Safety End Treatment	NA	NA
Height of fill (ft)	3	3
ADT	940	940
Date of Installation	1-Nov-99	2-Nov-99
Date of 1st Inspection	22-Apr-00	22-Apr-00

Table 5.5 DCP Readings of Installation A1 in Atlanta District

Depth (cm)	West Side (blows / 10 cm)	East Side (blows / 10 cm)
10	11	12
20	14	15
30	18	20
40	19	19
50	16	20
60	15	15
70	14	12
80	10	7
90	8	7
100	10	5
110	5	6
120	7	9
130	12	10
140	10	12
150	7	8
160	5	8
170	11	14
180	12	11
190	9	6

Table 5.6 DCP Readings of Installation A2 in Atlanta District.

Depth (cm)	East Side (blows/10 cm)	West Side (blows /10 cm)
10	20	20
20	14	20
30	12	23
40	14	23
50	10	29
60	6	22
70	6	20
80	6	17
90	8	13
100	6	8
110	5	9
120	7	12
130	6	7
140	5	8
150	5	14
160	7	13
170	8	11
180	11	10
190	9	4

Table 5.7 General Information on the Installations in Yoakum District.

District		Yoakum	
Location		FM 530	
Installation No.	Y1	Y2	Y3
Installation Type	Single	Single	Single
Diameter of the Pipe (in)	36	42	48
Length of the Pipe (ft)	60	67	65
Safety End Treatment	CH	CH	CH
Height of fill (ft)	1	1	1
ADT	840	840	840
Date of Installation	11-Nov-99	10-Nov-99	10-Nov-99
Date of 1st Inspection	25-May-00	25-May-00	25-May-00

Table 5.8 Trench Widths Used in Yoakum District Installations

Installation	Trench Width (in)
Y1	78
Y2	84
Y3	90

Table 5.9 DCP Readings at Installation Y2
At Yoakum District.

Depth (cm)	Point A (blows / 10 cm)
10	42
20	30
30	60
40	50

Table 5.10 DCP Readings at Installation Y1 in Yoakum District.

Depth (cm)	Point A (blows / 10 cm)	Point B (blows / 10 cm)
10	7	35
20	8	22
30	11	22
40	12	26
50	12	28
60	30	20
70	32	10
80	32	17
90	10	-
100	27	-
110	8	-
120	10	-
130	12	-
140	12	-
150	7	-
160	7	-
170	8	-
180	8	-
190	7	-

Table 5.11 An Overview of the Installation in Wichita Falls District.

District	Wichita Falls
Location	FM 1197
No. of Installations	1
Installation Type	Multiple
Diameter of the Pipe (in)	48
Length of the Pipe (ft)	52
Safety End Treatment	CH
Height of fill (ft)	2
ADT	-
Date of Installation	10-Jan-00
Date of 1st Inspection	23-Apr-00

Table 5.12 DCP Readings in Wichita Falls District.

Depth (cm)	Point A (blows / 10 cm)	Point B (blows / 10 cm)
10	2	5
20	5	5
30	7	4
40	5	4
50	5	2
60	4	1
70	4	4
80	4	3
90	3	4
100	2	-
110	2	-
120	3	-
130	3	-
140	6	-
150	5	-

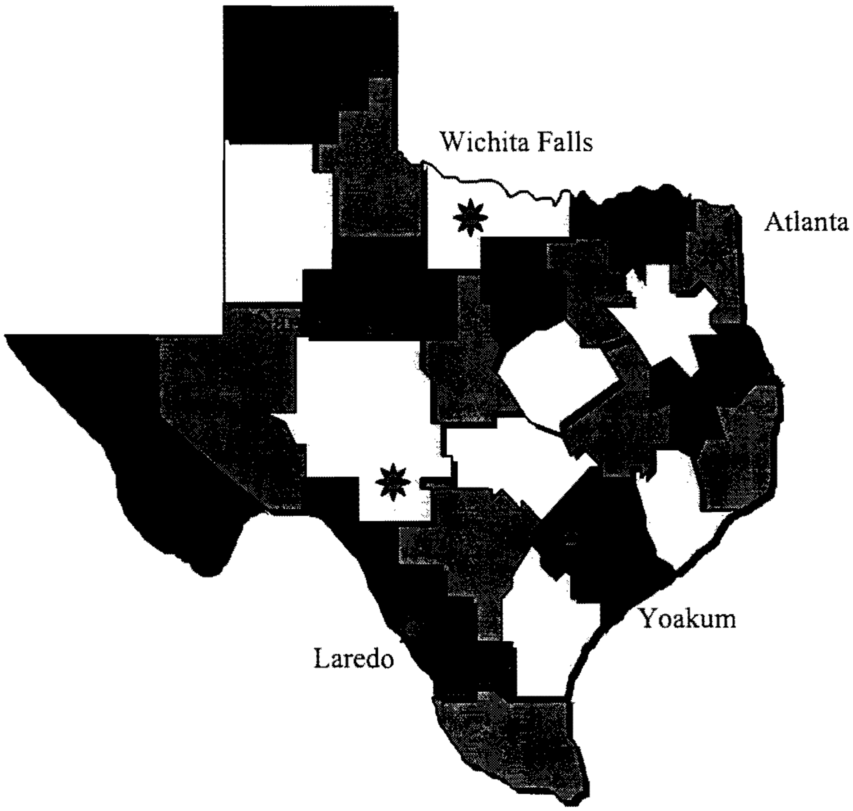


Figure 5.1 Locations of Pilot Construction Projects

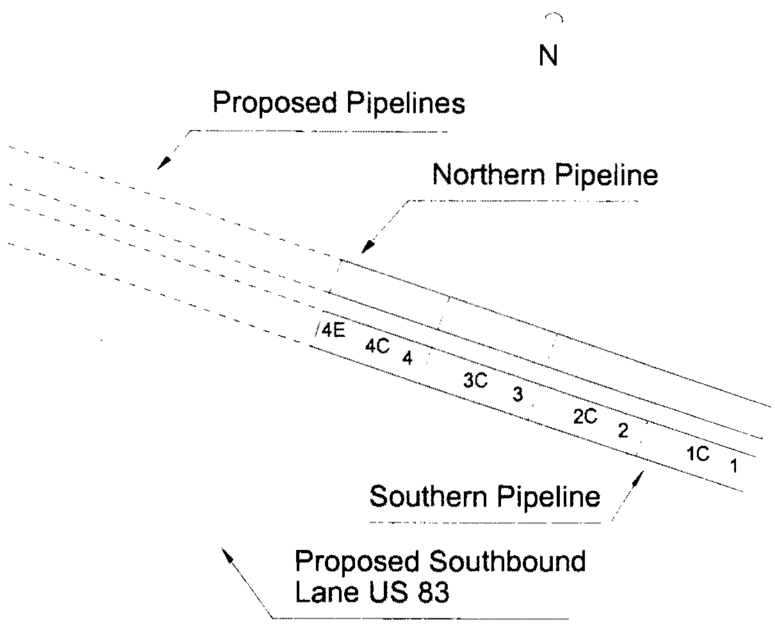


Figure 5.2. Plan View of San Angelo District Installation.

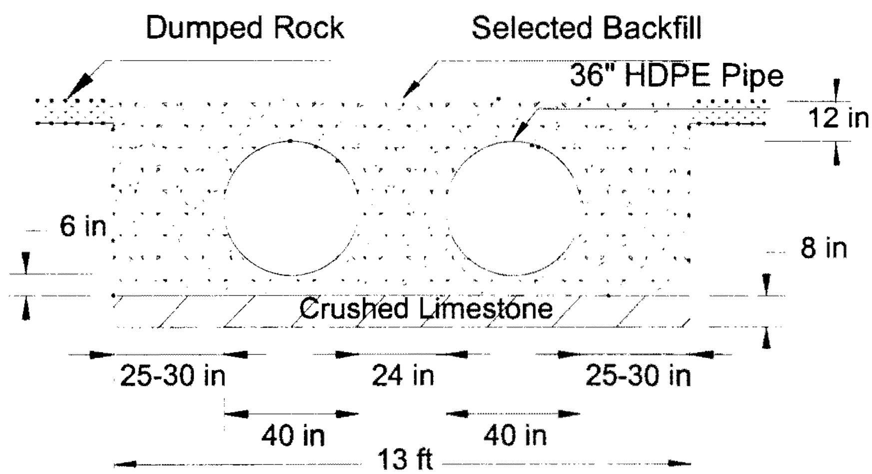


Figure 5.3. Cross-Section of Trench in San Angelo District.

Figure 5.4. Shaping of the Bedding.

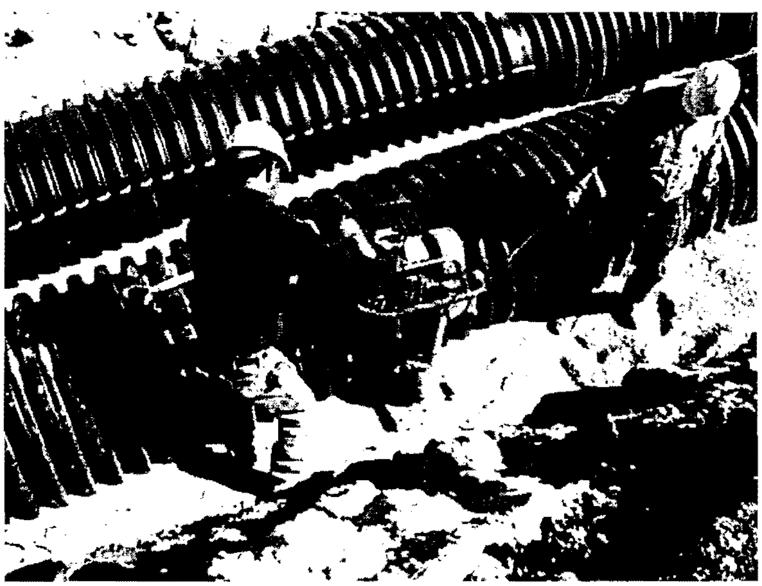


Figure 5.5 Compaction of the Bedding.

Figure 5.6 Pipe with Backfill up to the Crown.

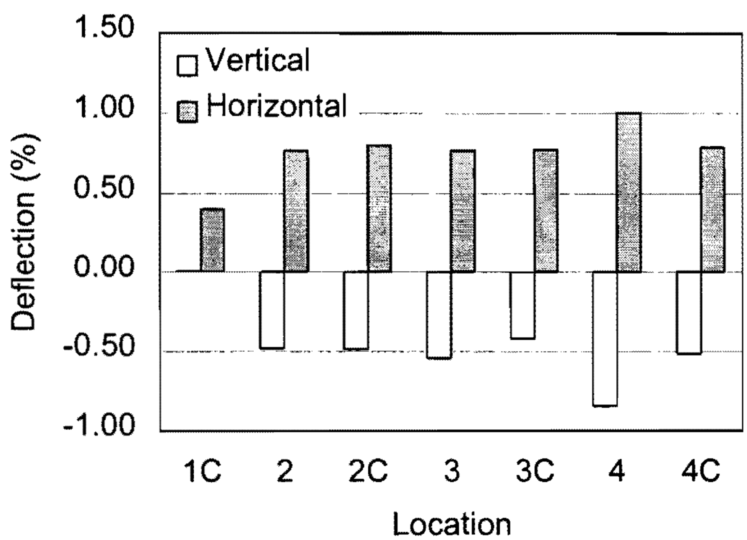


Figure 5.7 Pipe Deflections Immediately after Installation in San Angelo.

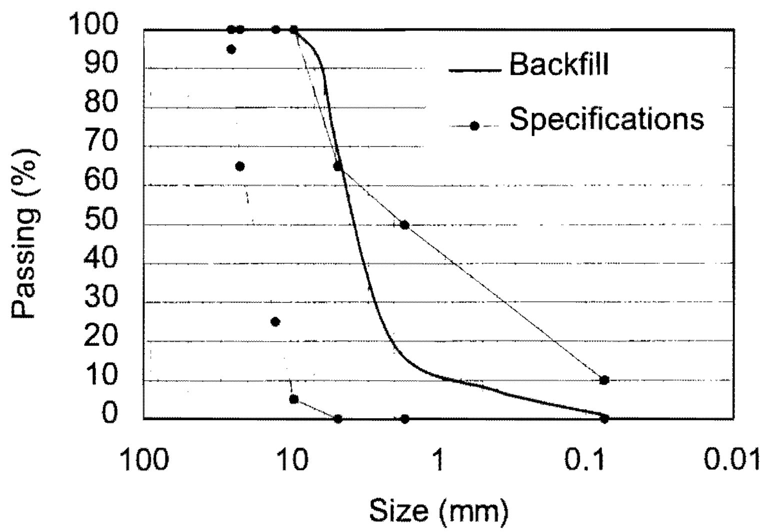


Figure 5.8 Particle Size Distribution of Backfill in San Angelo.

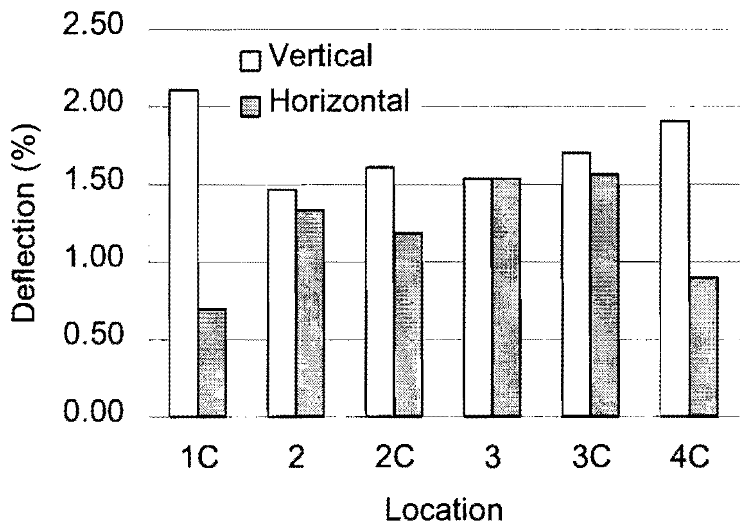


Figure 5.9 Pipe Deflections after Three Months in San Angelo.

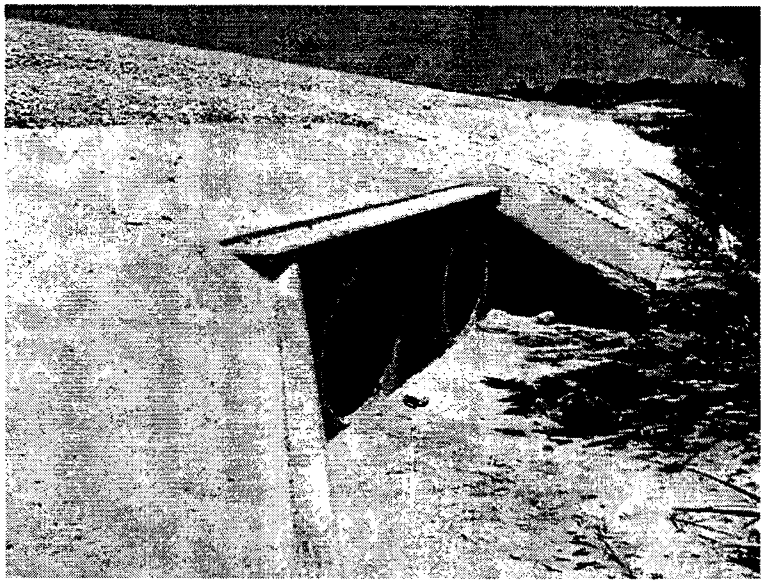


Figure 5.10 Safety End Treatment in San Angelo District.

Figure 5.11 Pavement Section in San Angelo District.

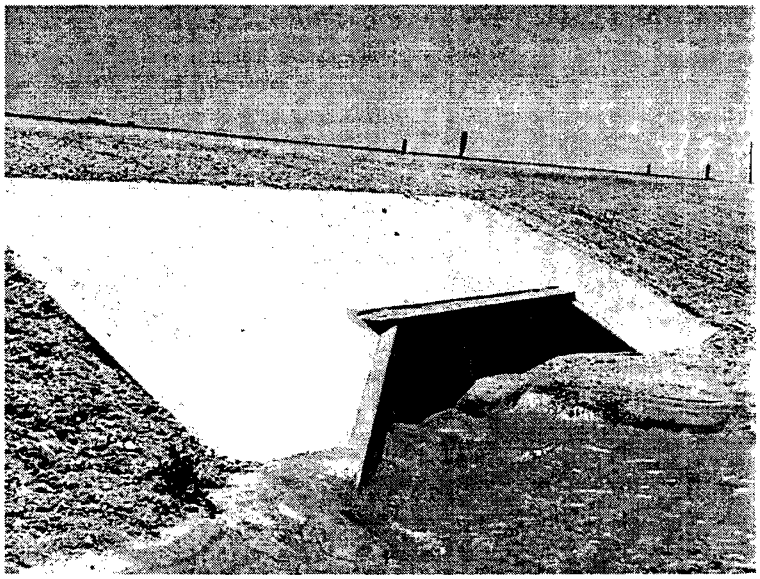


Figure 5.12 Silt Accumulation in Downstream End.

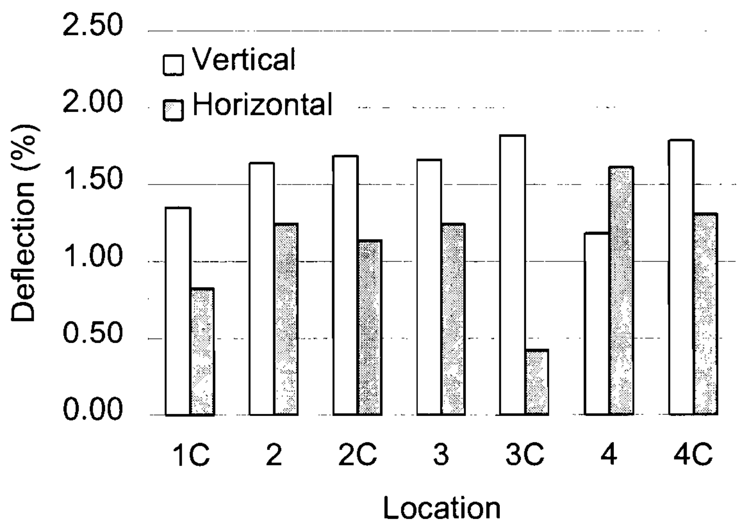


Figure 5.13 Pipe Deflections after Sixteen Months in San Angelo.

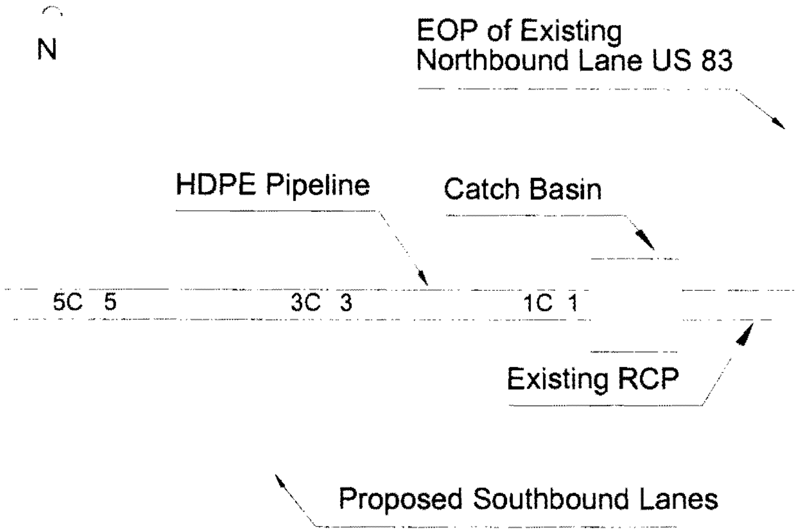


Figure 5.14 Plan View of the installation in Laredo District.

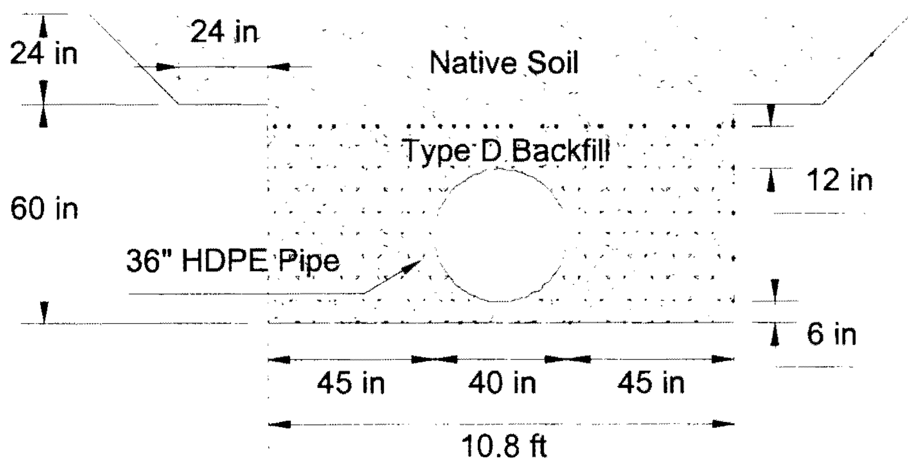


Figure 5.15 Cross-Section of the Trench in Laredo District.

Figure 5.16 Existing Concrete Catch Basin and HDPE Pipe.

Figure 5.17 Backfill Placement and Compaction.

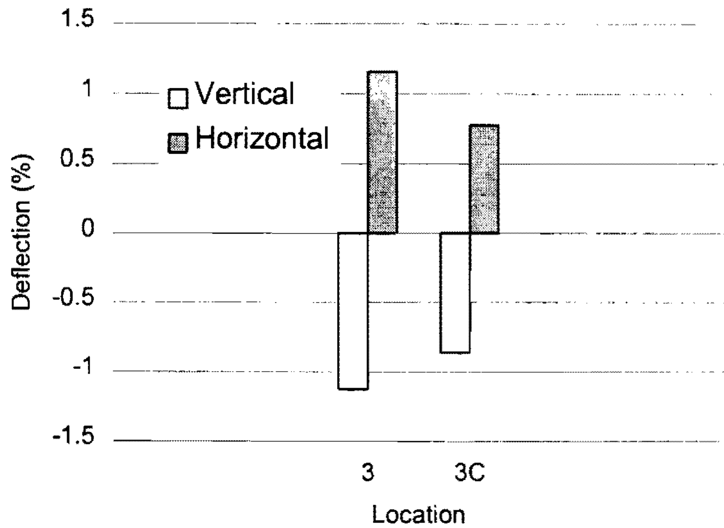


Figure 5.18 Pipe Deflections after Installation in Laredo.

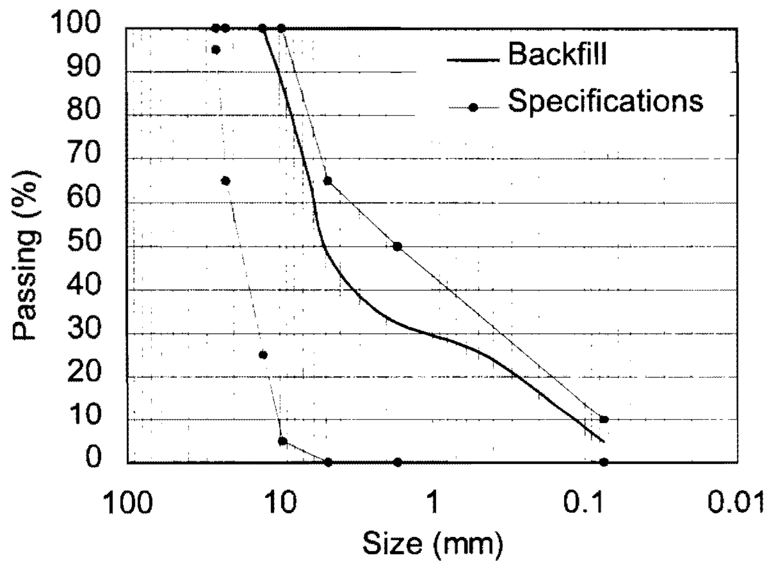


Figure 5.19 Particle Size Distribution of Backfill in Laredo.

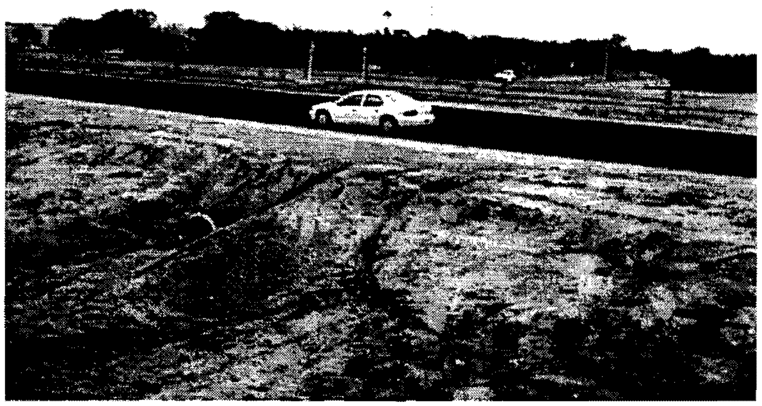


Figure 5.21 Installation in Laredo District Three Months after Construction.

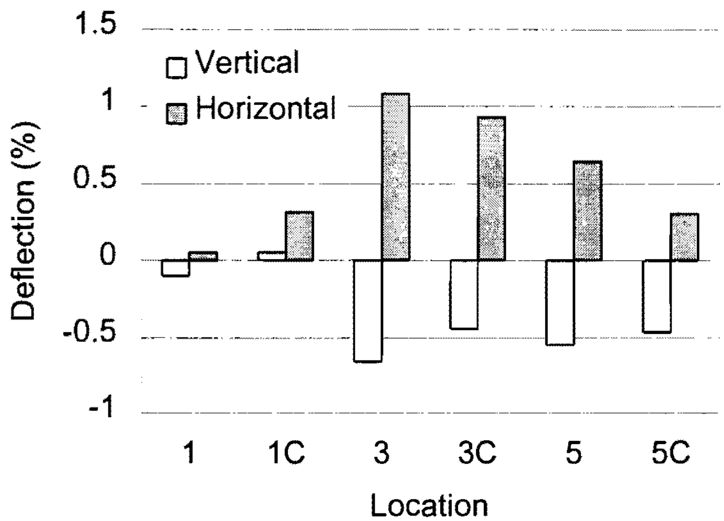


Figure 5.21 Pipe Deflections in Laredo Three Months after Construction.

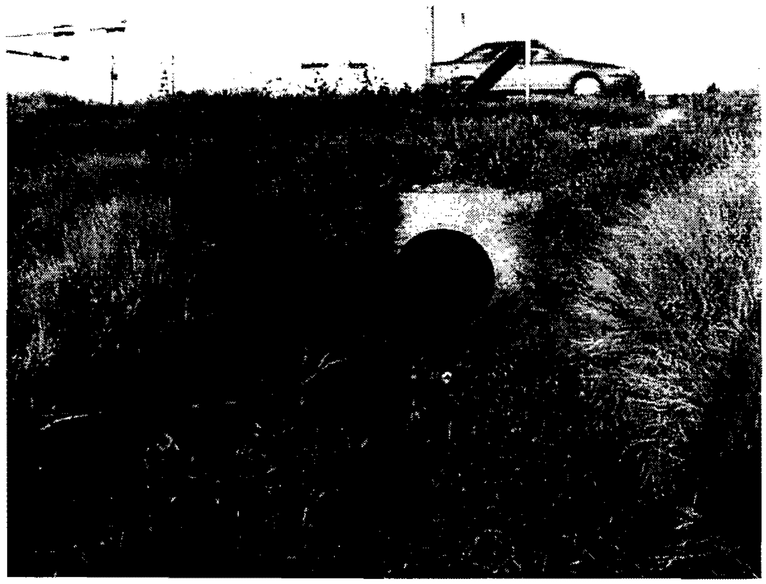


Figure 5.22 Safety End Treatment in Laredo District.

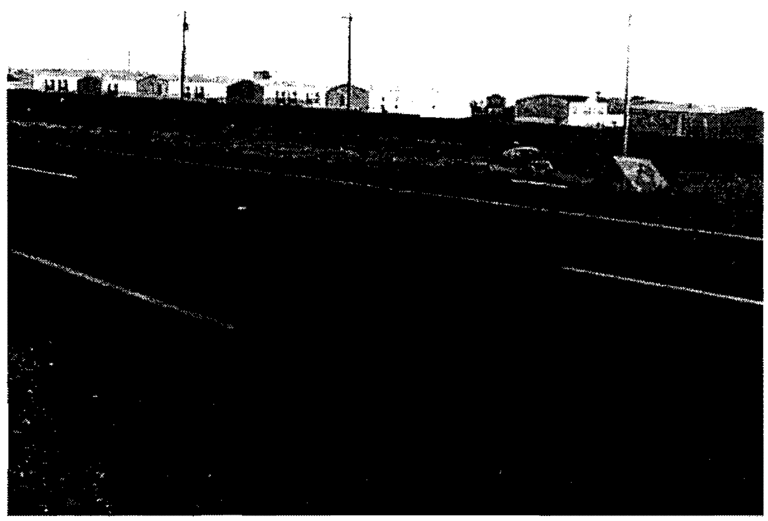


Figure 5.23 Pavement Section In Laredo District.

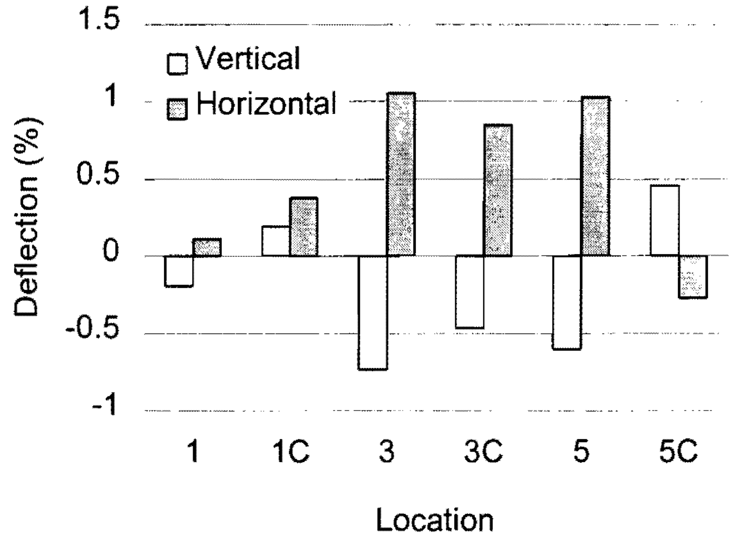


Figure 5.24 Pipe Deflections in Laredo Sixteen Months after Construction.

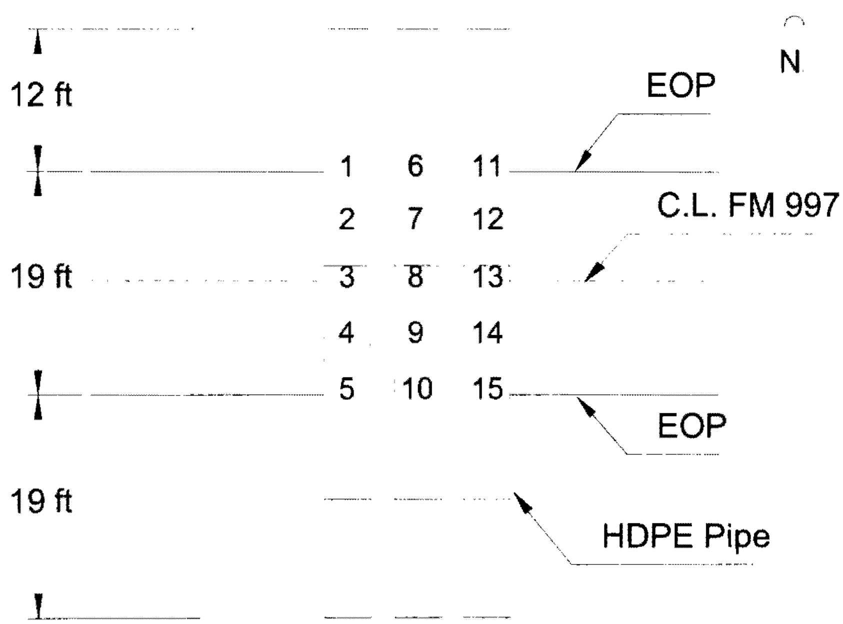


Figure 5.25 Plan View of Installation A1 in Atlanta District.

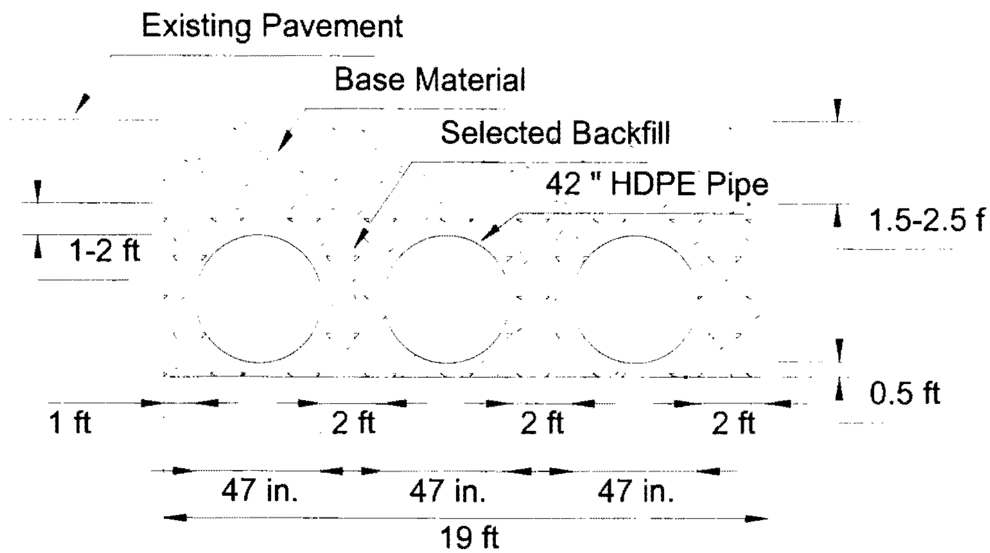


Figure 5.26 Cross-Section of Installation A1 in Atlanta District.

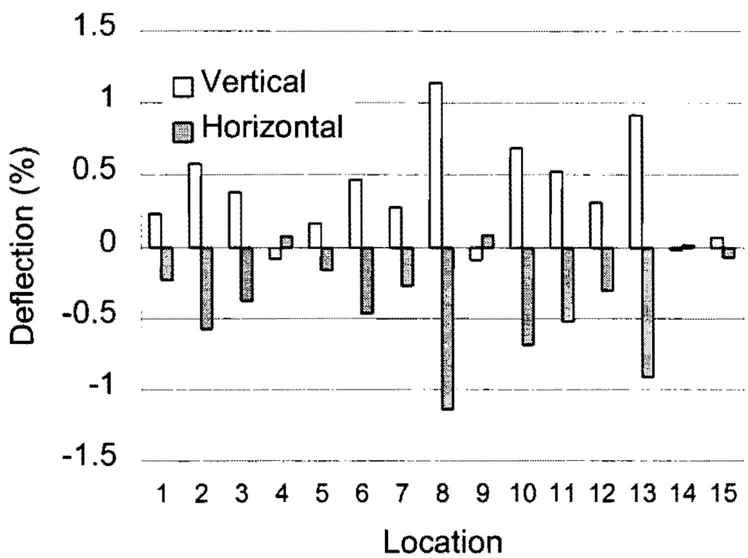


Figure 5.27 Deflections of Installation A1 in Atlanta District.

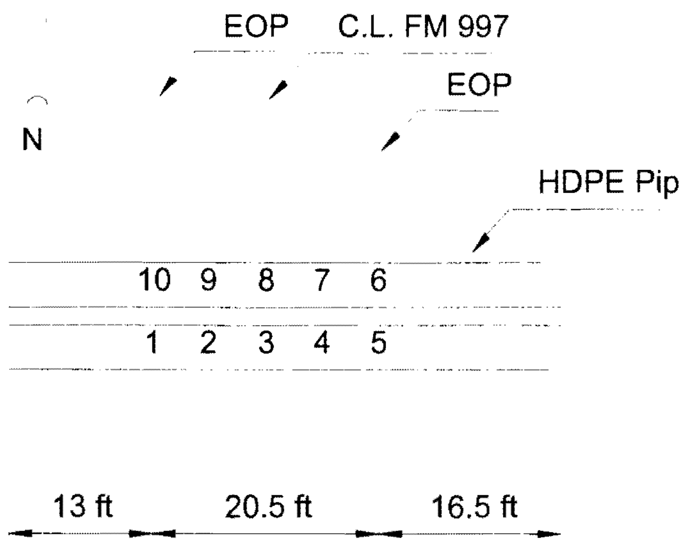


Figure 5.28 Plan View of Installation A2 in Atlanta District.

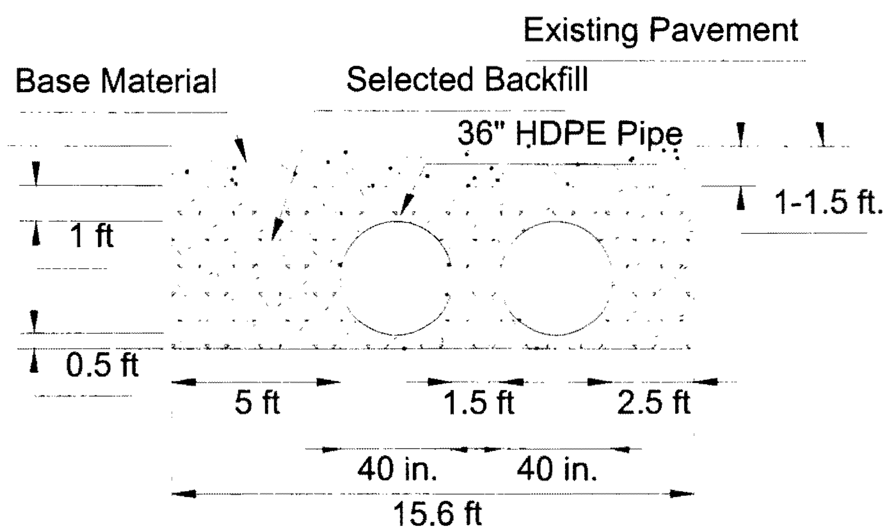


Figure 5.29 Cross-Section of Installation A2 in Atlanta District.

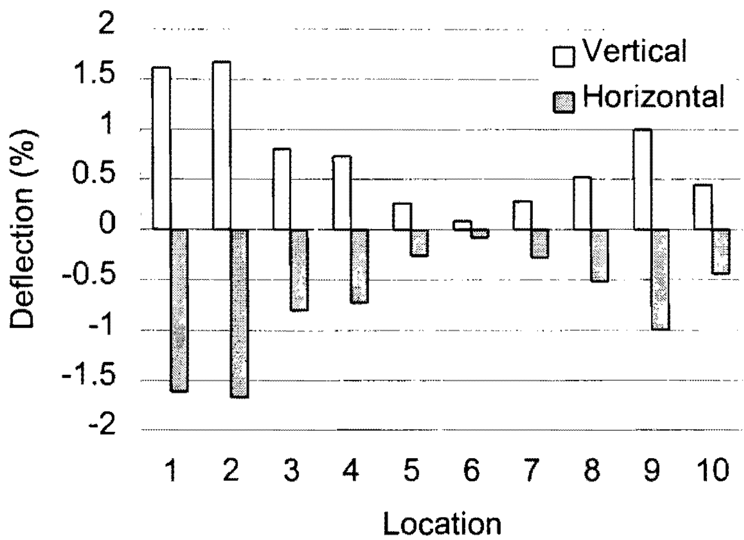


Figure 5.30 Deflections of Installation A2 in Atlanta District.

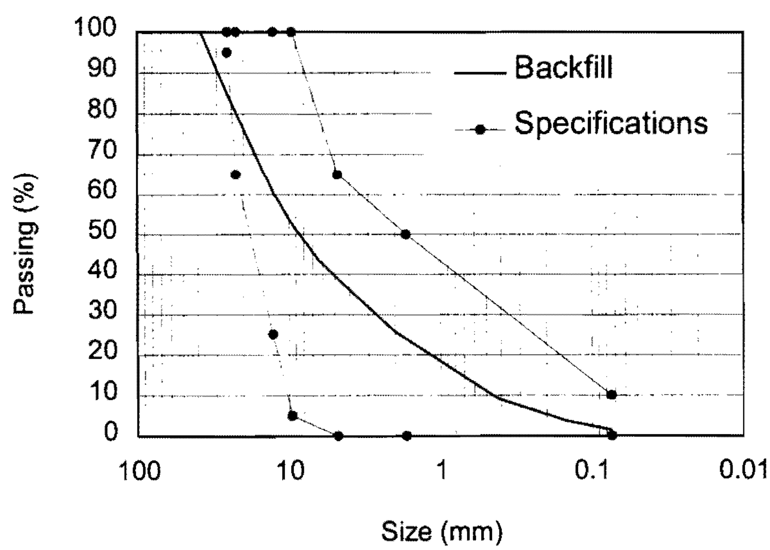


Figure 5.31 Particle Size Distribution of Atlanta Backfill.

Figure 5.32 Erosion in Installation A1 in Atlanta District.

Figure 5.33 Upstream Side of Installation A1 in Atlanta District.

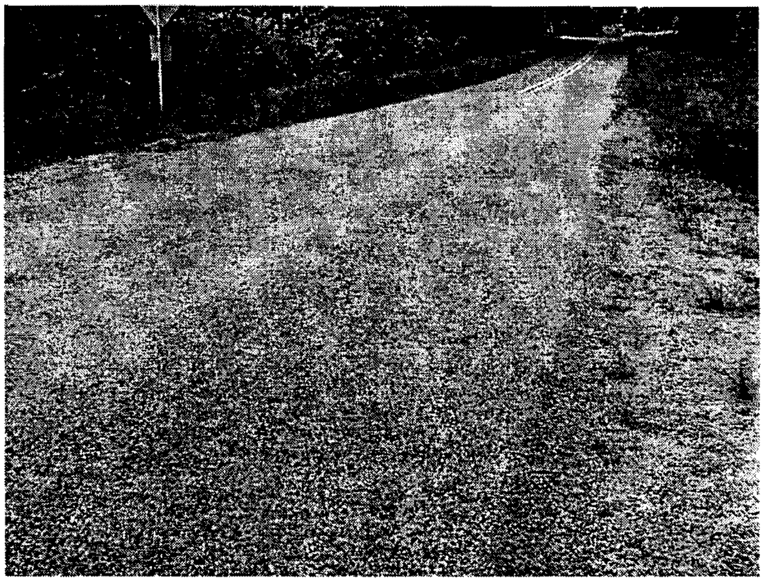


Figure 5.34 Pavement Section over Installation A1 in Atlanta District.

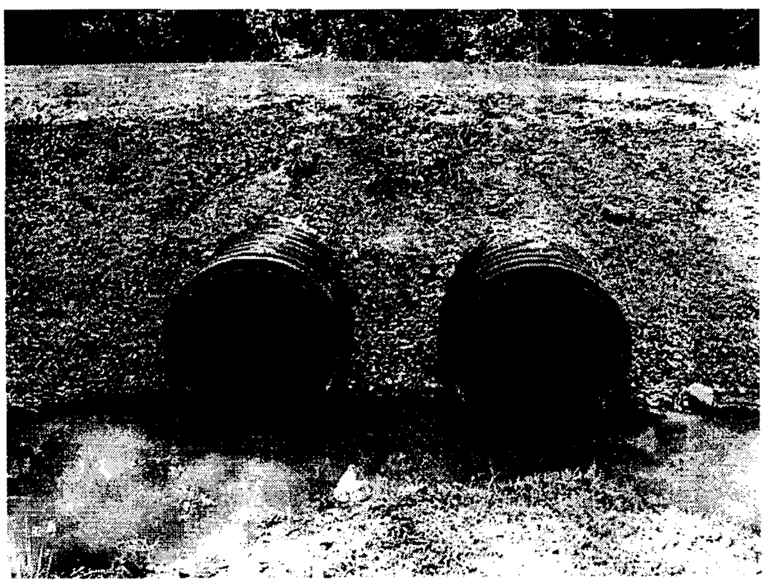


Figure 5.35 Installation A2 in Atlanta District.

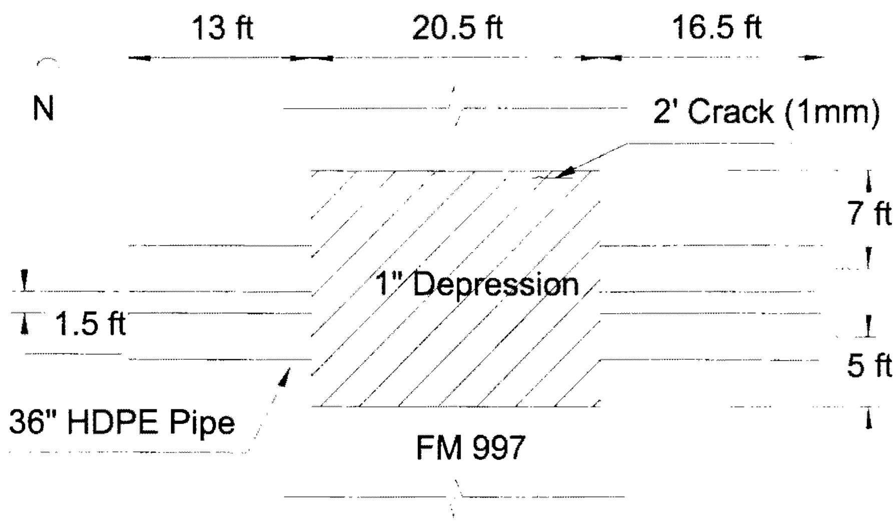


Figure 5.36 Pavement Settlement at Installation B in Atlanta District.

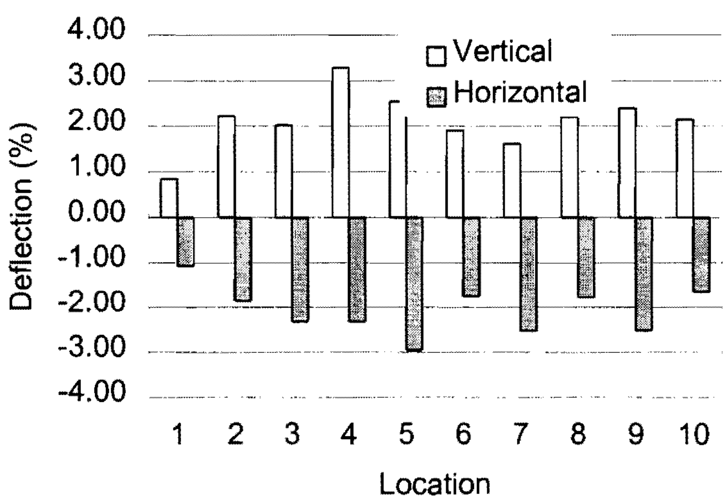


Figure 5.37 Deflections in Installation A2 in Atlanta District.
Five Months after Construction.

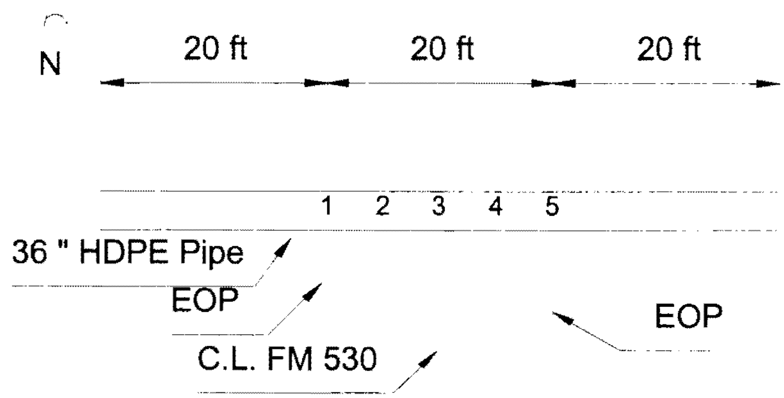


Figure 5.38 Plan View of Installation Y1 in Yoakum District

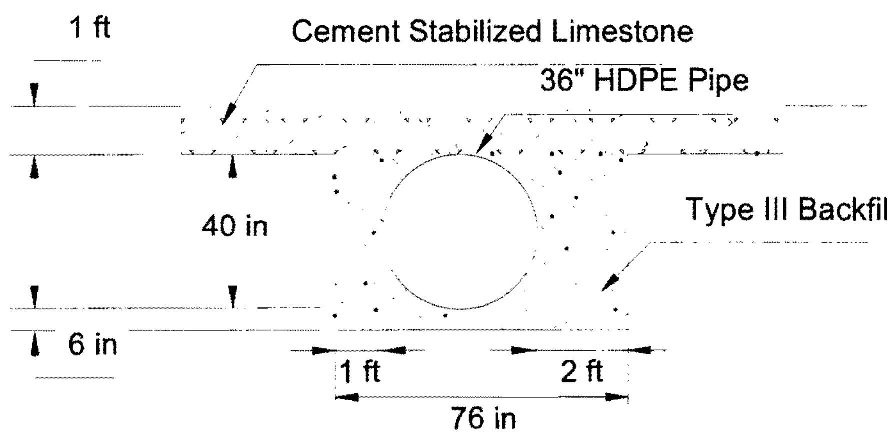


Figure 5.39 Cross-Section of Installation Y1 in Yoakum District.

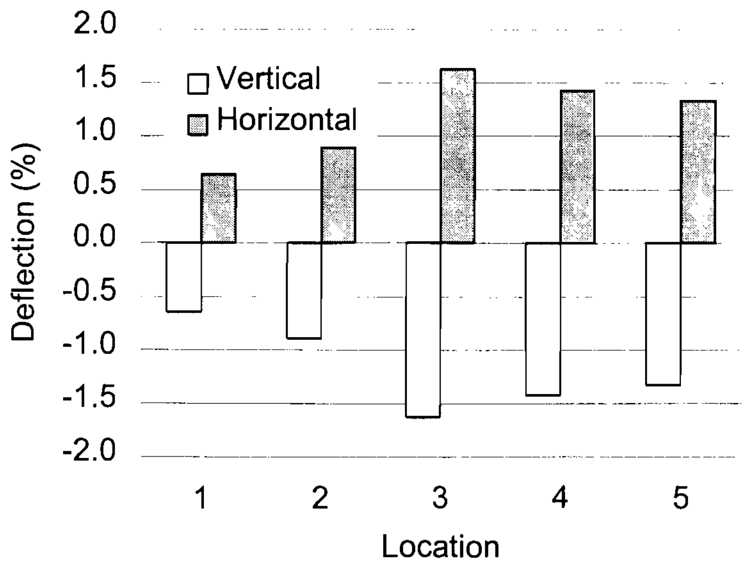


Figure 5.40 Deflections in Installation Y1 in Yoakum District.

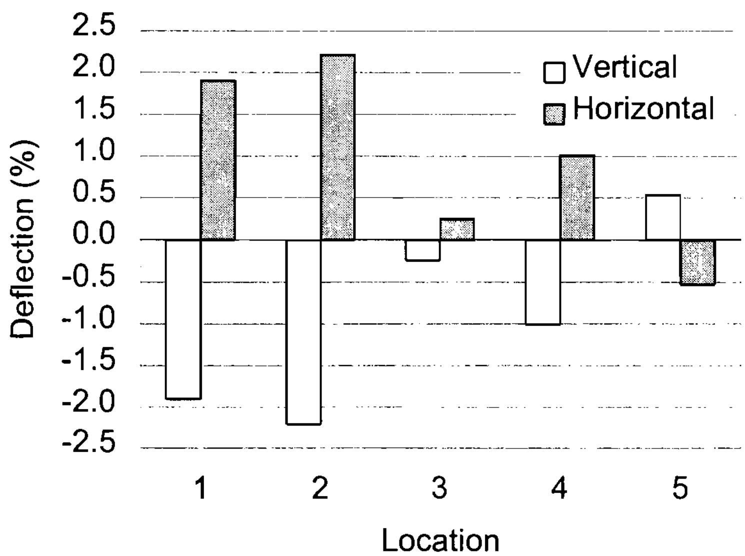


Figure 5.41. Deflections in Installation Y2 in Yoakum District.

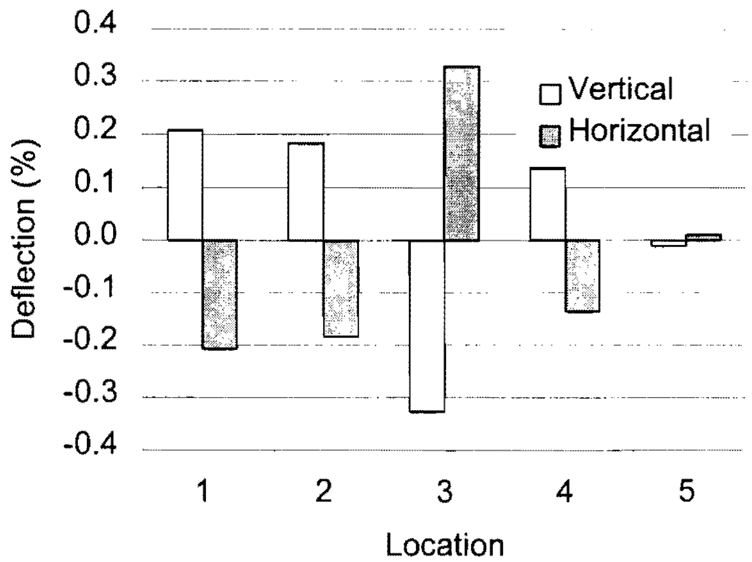


Figure 5.42 Deflections in Installation Y3 in Yoakum District.

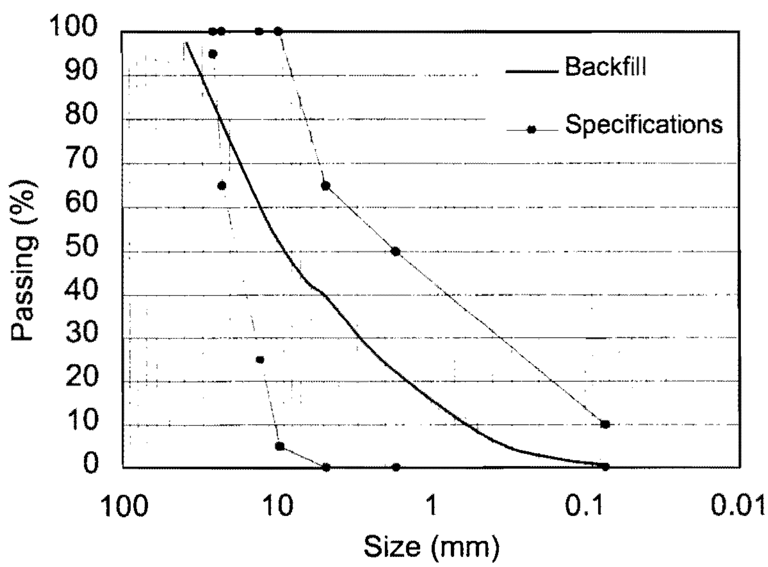


Figure 5.43 Particle Size Distribution of Yoakum Backfill.

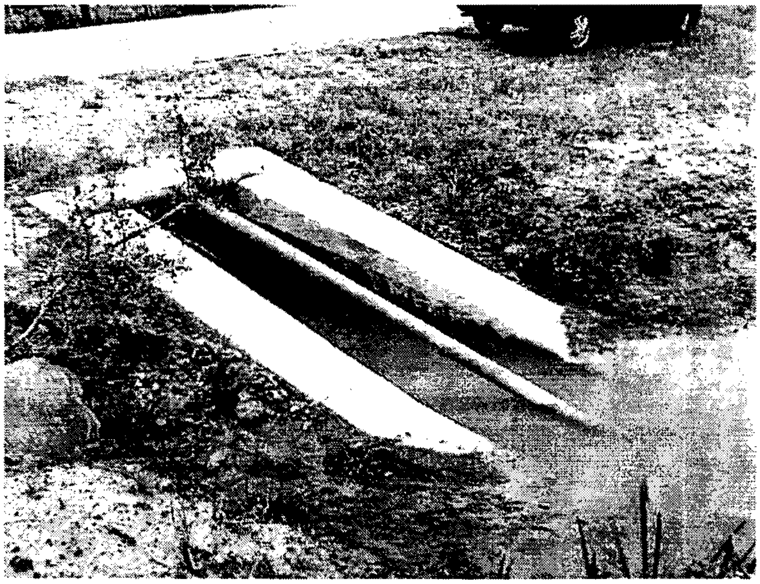


Figure 5.44 Water and Silt Accumulation in Yoakum District Installation

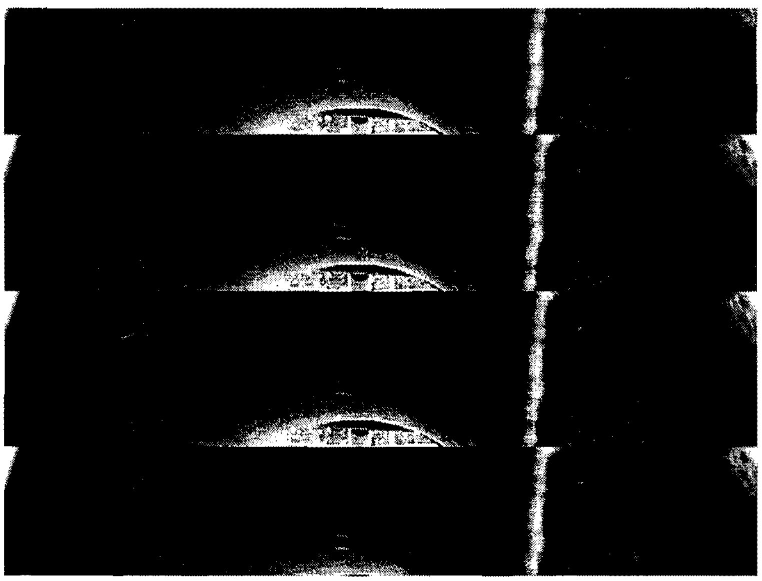


Figure 5.45 Loose Gasket in Yoakum District Installation.

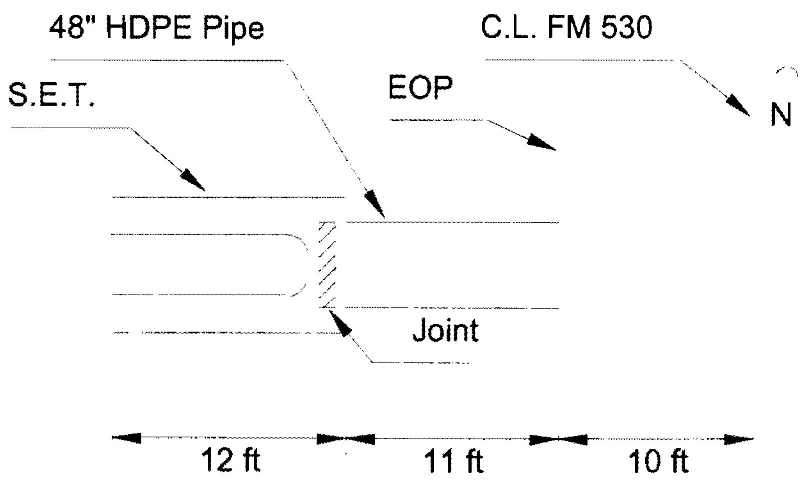


Figure 5.46 Plan View of the Section with a Dislodged Gasket in Installation Y3.

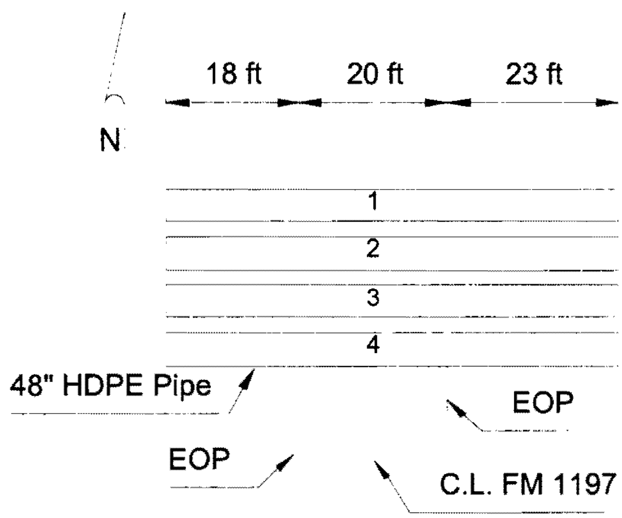


Figure 5.47 Plan View of Installation in Wichita Falls District.

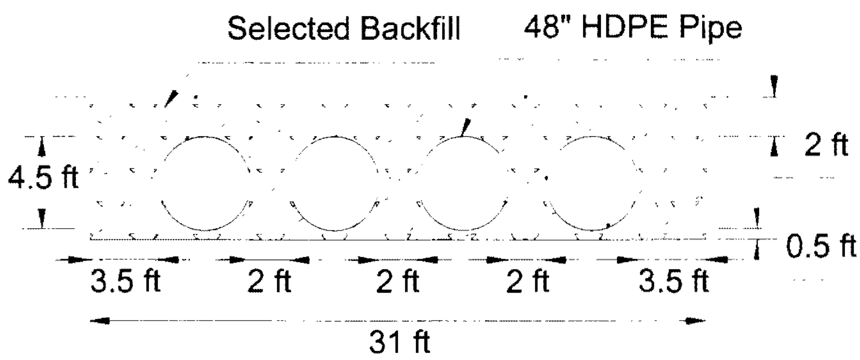


Figure 5.48 Cross Section of the Trench in Wichita Falls.

Figure 5.49 Wichita Falls Installation.

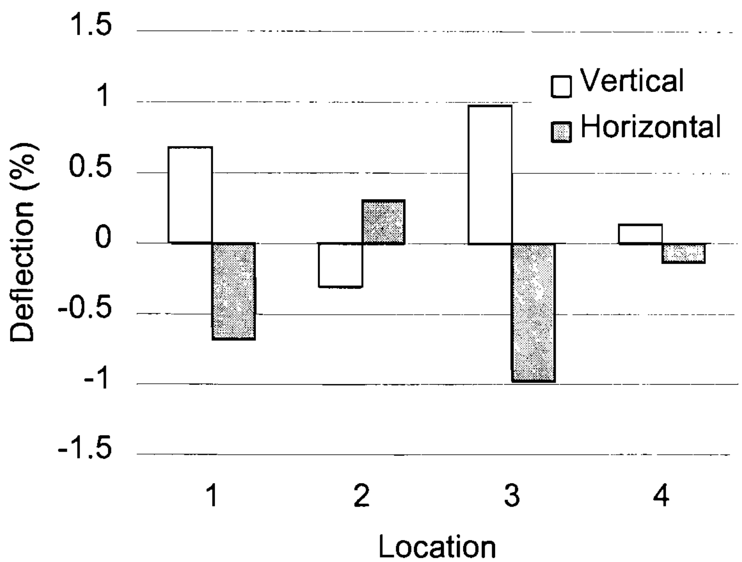


Figure 5.50 Pipe Deflections in Wichita Falls District.

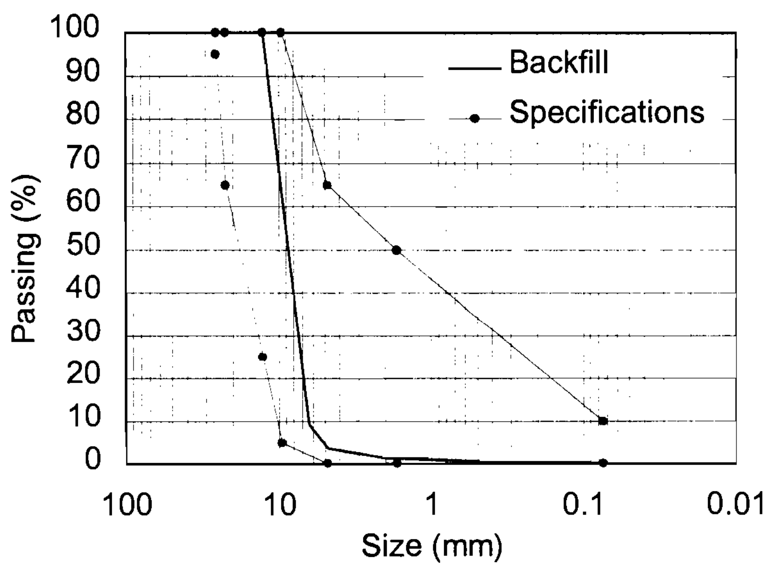


Figure 5.51 Particle Size Distribution of Backfill in Wichita Falls.

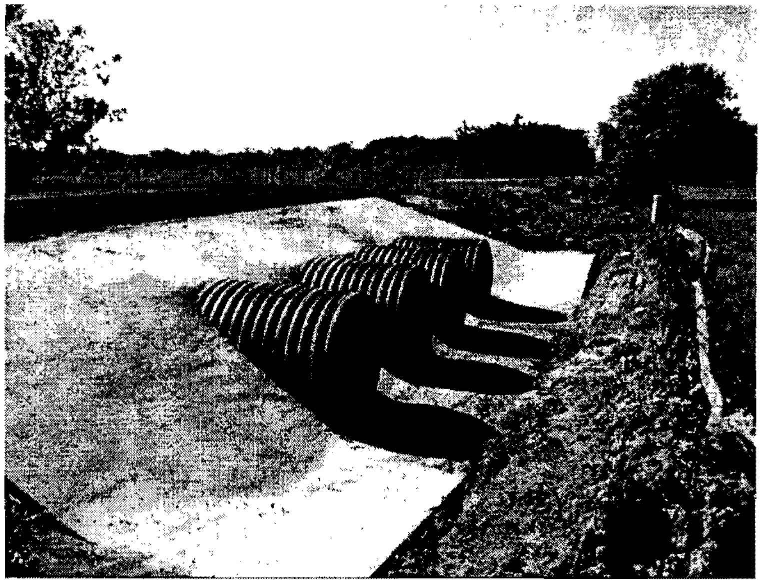


Figure 5.52 Concrete Headwall in Wichita Falls.

Figure 5.53 Pavement Section in Wichita Falls.

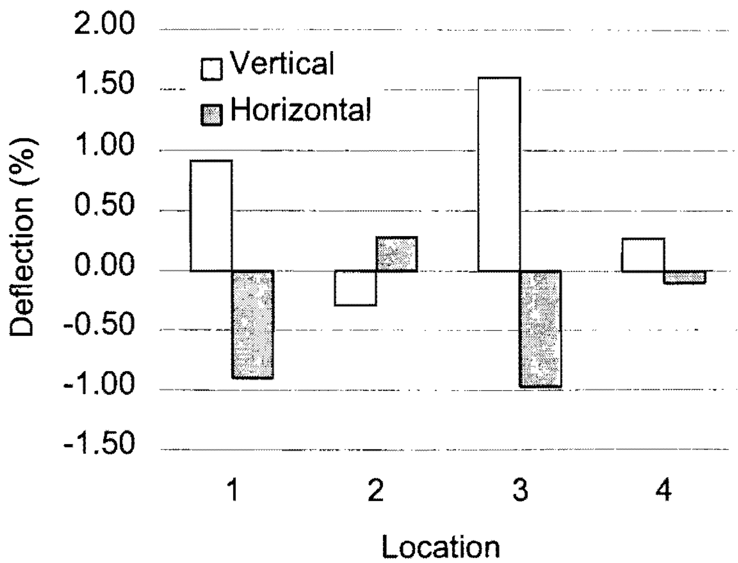


Figure 5.54 Deflections in Wichita Falls Three Months after Construction.

CHAPTER VI
CONSTRUCTIBILITY REVIEW

This chapter describes the application of constructibility review concepts to evaluate the practicality of the draft specification that was developed previously in this research project. This draft specification can be found in Appendix A of this report. It was developed based on guidance available through specifications developed by other agencies such as AASHTO, ASTM and other state DOTs as well as data collected from experimental work conducted in this research. The primary objective of constructibility review was to examine the draft specification from a constructibility viewpoint and hence identify any elements in the specifications that may create difficulties during its field implementation. The various steps involved in the above constructibility review and the recommendations are presented in the following sections.

6.1 Constructibility Review Team

As a first step in the constructibility review process, it was necessary to identify a number of qualified individuals to serve on a constructibility review team. In a typical field construction project, the constructibility review will be performed by individuals who have significant field experience in the specific construction processes involved. However, the constructibility review described here was different in a number of ways. First, it did not involve a specific pipe installation project. Instead, it involved a new specification that has been developed for such pipe installations. Secondly, the review was performed as a part of a research project. Therefore, this constructibility review team consisted of two groups of individuals: (1) members of the research team, (2) individuals with experience in the field installation of HDPE pipe. The researchers included: (a) research assistant charged with the primary responsibility of conducting the constructibility review, (b) the research study supervisor of the study and (c) other key investigators. The field construction personnel included (a) members of the TxDOT project monitoring committee, (b) contractor representatives from TxDOT pilot construction projects in San Angelo and Laredo districts and (c) representatives from HDPE pipe manufacturing companies. The primary role of the researchers was to collect necessary information from the field construction personnel, published literature, phone survey, and latest estimating catalogs and then perform the review based on this data. Table 6.1 below identifies the members of the constructibility review team.

6.2 Development of the Work Breakdown Structure (WBS)

Constructibility review begins with the development of a Work Breakdown Structure (WBS) for the particular construction project. During this task, the pipe installation process is analyzed in detail and each individual construction activity is listed in a sequential manner. As a first step, the pipe installation process is divided into five major tasks. They are:

(a) Trench excavation
(b) Installation of trench support system
(c) Preparation of the trench bottom

(d) Laying and joining pipe
(e) Placement and compaction of backfill

The next step involves the development of the detailed work breakdown structure. In the detailed work breakdown structure, each task is divided into several sub-tasks. Table 6.2 presents the above detailed work breakdown structure. It identifies all the steps that the site engineer, contractor, and the construction crew must go through from the time they receive the plans to the time of project completion. Table 6.2 also provides information on the resources needed for the completion of each sub-task. These resources include: equipment, construction crew and material. Subsequently, in Chapter 7 Economic Analysis, this work breakdown structure is further expanded to include the costs associated with each construction activity.

6.3 Equipment

As shown in Table 6.2, one of the important resources needed for successful pipe installation includes construction equipment. The construction equipment required for each itemized work process, as listed in that table, include the following:

(a) Trench excavators
(b) Trench support system: trench boxes, drag boxes, slide rails, trench sheeting.
(c) Pipe-layers, cranes
(d) Backfilling equipment: loaders, backhoes, backhoe loaders
(e) Compaction equipment for initial backfill: vibratory plate compactors, impact rammers
(f) Earth moving vehicles: elevating scrapers, belly-dump trucks

The following sections provide a preview of the equipment listed above. The discussion includes the selection of right type of equipment for each work process and their effects on the installation process as a part of constructibility review

6.3.1 Trench Excavators

The backhoe is the most commonly used equipment for the excavation of pipe trenches. It is used for excavating below the track elevation. It is also used as a small crane for handling pipe and trench support units. When excavating larger and deeper trenches, an operating plan should be developed in advance for the removal of the spoil and the construction of any ramps needed for the construction vehicles. Selecting an excavator of right capacity is important to achieve maximum efficiency in the excavation job. Table 6.3 lists typical maximum digging depths for hydraulic backhoes of varying capacity. Accordingly, the maximum digging depth for a backhoe with ¾ CY capacity is 18 ft (6m). In other words, greatest construction efficiency may not be achieved if the above equipment is used to excavate a trench with depth of say 25 ft (approximately 8m).

Another important factor to be taken into consideration in the selection of the appropriate excavation equipment is their operating weight. This is because any equipment that traverses the already installed pipe has the potential to disturb the structural integrity of buried HDPE pipe. This issue will be reviewed in Section 6.8 under the heading "minimum cover." Table 6.4 below lists typical operating weights for backhoes of various capacities.

6.3.2 Trench Support Systems

Pipe installation projects may involve excavation to large depths. In such projects, safety of the construction crew working inside the trench require special attention. Safety must be ensured through the selection of a trench support system that is appropriate for the specific depth of excavation.

OSHA regulations, 29 CFR Part 1926 Subpart P, Standard no. 1926.652 (*42*), provide guidelines that must be followed in this regard. These are presented below in the form of a graphical flow chart in Figure 6.1. According the OSHA flow chart presented above, some kind of trench protection system is required whenever the trench depth exceeds 5 ft. Consequently, many large diameter HDPE pipe installations will require such trench support. The following section presents some of the most commonly used trench support systems. These include: trench boxes, drag boxes, or wood plank and struts.

6.3.2.1 Drag Boxes

Figure 6.2 shows how a drag box is utilized in construction. This method of installation requires the trench to be cut slightly wider than the box and the drag box is lowered into position. Subsequently, backfilling inside the drag box and excavation of the next segment of the trench proceeds simultaneously. After the backfilling has been completed, the drag box is pulled forward into position by a large excavator into the new excavation.

6.3.2.2 Trench Boxes

Figure 6.3 is a sketch of a trench box. It is a modular system composed of two support walls separated by props. Figure 6.4 demonstrates the procedure used in the installation of trench boxes. Contractors have generally found that three boxes are sufficient to operate in an efficient work cycle – one box going down as trench is excavated, a second box already in place to provide protection for pipe installation and the third box coming up as the backfilling proceeds.

Figure 6.5 shows the use of slide rails in the installation of trench boxes. The trench box system has been developed a stage further to incorporate slide posts driven ahead of the slide plates to act as guides. This method overcomes difficulties in trench box withdrawal in granular soil.

6.3.3 Earth Moving Equipment: Bulldozers, Loaders, Scrapers and Graders

The construction sites are often uneven and require leveling. When construction takes place in such rough terrain, excavated material is removed, transported and deposited in a cycle. A fairly broad range of earth moving equipment is available and the most suitable equipment must be selected depending on specific site conditions. The bulldozer is very versatile machine and is used frequently for stripping top soil, clearing vegetation, pushing scrapers, spreading and grading. The bulldozer can be used effectively for moving earth over short distances up to 300ft (100m). However, many projects necessitate a combined load, haul and discharge system of at least up to a distance of 2 miles (3 km).

This situation calls for a robust excavator, capable of traveling over rough terrain to eliminate the use of trucks and wagons on public roads. The scraper has been developed specially to cater to this medium distance haul. Essentially the earth is cut and loaded directly into the scraper box (or bowl), transported to the discharge area and finally spread in layers. The whole process takes place in a continuous cycle. The type of machine to be adopted depends upon traveling distance. The loader is a machine which serves the purpose of both the fixed-position excavator and transporter over shorter distances of perhaps 30-60ft (10-20m).

6.3.4 Compaction Equipment

Compaction of the backfill within the trench requires special equipment because of the limited space available between the pipe and the trench wall. Walk-behind Vibratory plate compactor and impact rammers are the most common and convenient types of compactors contractors use for compaction of backfill inside a trench for pipeline installation.

Rammers are the best type of compactor for clay and cohesive soils, where we need to squeeze out air and excess water. The shoe or foot of rammer will come off the ground, approximately 2 or 3 inches and then slap down about 600 to 700 times a minute providing very effective soil compaction. The vibratory compactors are well suited for granular material. However, they do not work well in clay soil because their compacting action tends to pump water to the surface and create mud. The machine bogs down because it doesn't have enough amplitude to separate itself from the clay.

The backfill materials recommended in the specification other than the flowable backfill are coarse granular materials and hence, there should be no problem with using vibratory plate compactors for compacting backfill of HDPE pipe installation. Vibratory compactors and, vibratory plates in particular, rest directly on the ground. The compactor with smallest plates dimension measure 12 inches wide by 25 inches long. A rotating offset weight in the plate creates vibrations. These vibrations reduce the friction between the soil or gravel particles in backfill, then allow gravity and the weight of the machine to compact that material.

There are two types of vibratory plate compactors: forward-plate compactors and reversible-plate compactors. Forward-plate machines use one counterbalanced weight to produce compaction energy and it is designed to go in only one direction only. A reversing plate machine uses two. Changing the pitch on one of those weights allows the machine to go from forward to reverse simply by pulling a lever. This eliminates the need to turn the machine around at the end of the run. This is a significant advantage when working in a confined place such as in a pipe trench. Therefore, reversing-plate machines are more productive than forward-plate.

6.3.5 Weight of Equipment

Once the pipe soil envelope is prepared, any of the equipment discussed above may traverse buried pipe zone and hence affect the structural integrity of the installed pipe. Therefore, proper cover must be provided to avoid potential damage to the pipe. The selection of the minimum cover required, as explained in Section 6.8 is done based on the operating weight of the equipment. Table 6.5 that provides approximate weight of most of the construction equipment that might traverse that pipe-soil zone. Data in this table has been obtained from Caterpillar Performance Handbook, Edition 26 (*43*).

6.4 Deficiencies in the Draft Specification

The next step in the constructibility review involved careful examination of each sub task along with the relevant portions of the draft specification to identify potential problems in implementation. During this process, the researchers relied heavily upon the input they collected from the field construction personnel. This review identified a number of deficiencies or problem areas that deserve special attention. These are as follows:

(a) Minimum trench width requirements
(b) Types of backfill material allowed
(c) Granular backfill gradation specifications
(d) Minimum cover specifications

Each of these topics is discussed in detail in the following sections.

6.5 Minimum Trench Width Requirements

Section 6.1 of the draft specification deals with trench excavation. It specifies that "*the trench width shall be sufficient, but not greater than necessary, to allow working room to properly and safely compact haunching and other embedment materials.*" Since, in most cases, the native material does not meet the specifications for pipe backfill suitable material has to be obtained and then transported to the jobsite at a cost. Therefore, from a project economics standpoint, it is important to keep the trench width to a minimum. At the same time, however, the trench should be wide enough to allow placement and proper compaction of the backfill. The minimum trench width requirements found in the draft specification depend on the type of backfill used. They are as follows:

Type I Backfill	Outside pipe diameter + 12 inches
Type II Backfill	Outside pipe diameter × 1.25 + 12inches

One of the issues that was addressed during this constructibility review involved the minimum trench width specifications for installations where Type II backfill (i.e. granular backfill) was used. The current specification was based on the guidelines found in ASTM D 2321: *Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications*. However, during the field tests conducted in this research, it was observed that the above minimum trench width specifications did not provide adequate room for the operation of backfill compaction equipment. Therefore, this problem was investigated during the constructibility review.

As a first step, the minimum trench width guidelines developed and used by various agencies were compiled. These data are shown in Table 6.6. The last column in this table represents the trench widths calculated by the draft specification. Comparison of these numbers reveals that there is significant variation in the minimum trench width recommendations developed by different agencies. For large diameter pipe (i.e. 36in and above), the AISI Handbook, NCSPA installation brochure and UniBell Handbook provide the smallest minimum trench widths. Minimum trench widths specified in AASHTO

Bridge Design Manual Sections 12 and 26 are largest. Figure 6.7 shows a comparison of some of the more commonly used minimum trench width guidelines.

Secondly, the overall dimensions of the more commonly used backfill compactors were reviewed to determine, the minimum space that would be required to operate these equipment. Based on the information reviewed it was determined that a minimum of 18 inches will be needed to operate most vibratory plate compactors or impact rammers without disturbing the pipe. Consequently, the minimum trench widths were calculated allowing 18in space between the pipe and the trench wall. These calculations are summarized in Table 6.7. Based on this review, it is recommended that the minimum trench width in the specification be modified according to Table 6.8. Comparison of the minimum trench widths recommended in Table 6.8 and those shown in Table 6.6 reveal that the new guidelines closely match with AASHTO Section12 and Section 26 guidelines.

6.6 Types of Backfill Material

Section 6.8 of the draft specifications deals with backfill materials. It allows the following two types of backfill materials.

Type I Backfill - Flowable Backfill in accordance with Special Specification 4005
Type II Backfill - Granular Material that meets the gradation requirements of Type B, C, D or F aggregate mixtures in Item 334 or Item 340

One of the key issues addressed during this constructibility review involved the availability of the specified backfill material at economical prices in various parts of Texas. To examine this issue, a survey was conducted among the materials or laboratory engineers in all TxDOT districts. As a first step, a copy of the draft specification was sent to each district lab engineer. Then they were asked to provide information on the availability and the cost of each specified backfill material in their district. They were also asked to identify alternative materials that are economically available within the region that may be used as HDPE pipe backfill. The information collected from this survey is summarized in Appendix B. Review of the information received during this survey lead to the following important findings.

a) Cement stabilized sand is a common backfill material that is widely used by many TxDOT districts. In some districts, such as Houston and Beaumont this material is found to be more economical than conventional granular backfill. This was confirmed by data collected during the economic analysis phase of this research. Therefore, cement stabilized sand should be included in the specifications as an acceptable backfill material.
b) The mix design of the Special Specification 4005 flowable backfill is designed to provide a much higher strength than is necessary for pipe backfill purposes. This flowable backfill is considerably more expensive than other backfill materials. A TxDOT special task force has examined a number of different flowable backfill specifications and identified Special Specification 4438: flowable backfill as a more suitable backfill material for this application. Item 4438 corresponds to a lower strength (28-day compressive strength of 80-150 psi) and therefore a more economical flowable fill. Therefore, Item 4438 should be used instead of Item 4005 in the specifications for Type I backfill materials. However, there is some

concern with respect to lack of control on the short-term strength of this particular flowable fill. Item 4438 specifies the 28-day strength to be a minimum of 80 psi but does not specify a minimum short-term, say 24 hr, strength. Short-term strength of the flowable fill is of importance because, in many pipe installation projects, the trench must be covered up and the highway opened to traffic as soon as possible. Therefore special attention should be paid to the short-term strength of the flowable fill when this type of backfill is used in pipe installation projects.

6.7 Granular Backfill Gradation

Gradation of the Type II granular backfill is specified in Section 6.8 of the draft specification. In this section it is stated that "Type II backfill consists of granular material that meets the gradation requirements of Type B, C, D, or F aggregate mixtures in Item 334: Hot Mix-Cold Laid Asphalt Concrete Pavements and Item 340: Hot Mix Asphalt Concrete Pavements." Although the specified gradation bands match the HDPE pipe backfill requirements very well, there is a major difficulty associated with the use of this specification. In the preparation of an asphalt concrete mix of a specified type, aggregate of different size fractions are fed into the plant in the correct proportions and then blended inside the hot mix plant. Accordingly, aggregate blending to achieve specific gradation requirements of Type B, C, D and F aggregate mixtures is accomplished within the hot mix plant. Thus, achieving the same Type B, C, D and F gradations for another application is difficult.

Item 334 and Item 340 were used in the backfill material gradation specification with the expectation that they will make the task of finding the appropriate material easier for the contractor. However, experience from the pilot construction projects proved that it created more ambiguity and confusion for the contractor than it helped him identify suitable granular backfill. Therefore, the following alternative approach is recommended for use in the granular backfill specification. Table 6.9 below specifies the gradation band for granular backfill. In addition, items from the current TxDOT Standard Specification that may meet the specified gradation requirements are identified in a footnote to the table.

6.8 Minimum Cover

One of the important aspects that must be addressed in the specification for installation of the pipe involves the minimum cover requirements to protect the pipe from vehicular loading. In arriving at a suitable thickness for the minimum cover, one must consider two types of loadings; (a) loadings from off-road vehicles, such as construction vehicles that may traverse the pipe during construction, (b) repeated loading from vehicles that travel on the highway once the pipe installation is complete.

In the original version of the specification that was developed by the research team and presented to the TxDOT project monitoring committee on April 30, 1998, minimum cover issues were addressed in Sections 3.1 and 4.5.4. Figure 6.8 on the following page presents the relevant sections from the above specification. During the constructibility review, the project monitoring committee (PMC) was asked to review the specifications and provide their comments. Based on their review, the PMC raised a question with respect to lack of a clear definition of "heavy construction vehicles." This was an important issue because the specification requires the construction of a special ramp to provide a minimum 4 ft (1.2 m) cover before heavy construction vehicles could traverse the pipe. However, building of such a ramp will require the use of construction vehicles. This raises a question as to which construction vehicles are not considered "heavy" and therefore may be used in the construction of the ramp.

In response to the above comment, the researchers conducted a thorough review of the existing information on effects of heavy vehicle loading on the thermoplastic pipe with minimum cover. The findings indicated that, in general, 1 ft (300mm) minimum cover has generally been found to be adequate to protect the pipe from loading due to many commonly used construction equipment such as excavators, rollers, front-end loaders, backhoes etc. However, larger cover is needed to protect the pipe from construction equipment such as earth movers, elevating scrapers and cranes. However, since each type of construction vehicle comes in a broad range of models, it is not possible to categorize construction vehicles into two classes as "light, for which 1 ft (300mm) is adequate" and "heavy, for which minimum cover larger than 1ft (300mm) is required." Therefore, it is recommended that the specifications be revised in the following manner.

"The backfill material shall be placed evenly and simultaneously on both sides of the pipe to not less than 1ft (300mm) above the top of the pipe. No heavy construction equipment with axle loads equal to or larger than 40 kips shall be permitted to traverse the pipe trench. If the passage of such heavy construction equipment over an installed pipeline is necessary during construction, compacted fill in the form of a ramp shall be constructed to depth of one pipe diameter above the crown of the pipe."

It must be noted that the recommendations given above are based on information available in published technical literature. In latter parts of this research study, appropriate field testing was conducted to check the validity of the above specification for the specific types of backfill material used in TxDOT pipe installation projects. Based on the findings from these field tests, minimum cover requirements recommended here were further revised. These revisions are described in Chapter VIII .

Table 6.1. Constructability Review Team.

Researcher/s	Field Personnel
Research Assistant Mohd. D. Alam	TxDOT Project Director Victor Pinon, P.E.
Research Supervisor P.W. Jayawickrama, Ph.D.	District Construction Engineers
Other Key investigators D.G. Gransberg, Ph.D., P.E.	Representatives of the Contractors Involved in TxDOT Pilot Construction Projects
	Representatives from the HDPE Pipe Manufacturers

Table 6.2. Itemizing Major Activities.

Item Number	Item	Equipment / Heavy construction vehicle	Crew	Material
1	Surveying	Leveling survey equipment	Surveyor.	
2	Site cleaning	Site cleaning equipment	Laborer	
3	Trench Excavation, Spoil is piled adjacent to the trench, part of which will be used for backfilling above the pipe zone. (Trench depth, width, side slopes, and longitudinal grading are determined in planning and design phase following the draft specification):	Crawler mounted or wheel mounted backhoe with bucket size ranging from ½ CY to 2-1/2 CY	Tractor operator and laborer	
4	Dewatering (if ground water flows into the trench during excavation)	Dewatering equipment	Laborer	
5	Hauling away the unused Part of excavated material	rock	Truck driver, laborer	
6	Trench Box Installation (At least three trench boxes):			
6.1	Install trench box 1	Comparatively high capacity backhoe with bucket size ranging from 1-1/2CY to 3CY	Tractor operator and laborer	

Table 6.2. (Continued)

6.2	Install trench box 2 next to trench box 1	Backhoe with bucket size ranging from 1-1/2CY to 3CY	Tractor operator and laborer	
6.3	Install trench box 3 next to trench box 2	Backhoe with bucket size ranging from 1-1/2CY to 3CY	Tractor operator and laborer	
6.4	Remove trench box 1 and install it next to trench box 3. Repeat 6.2, 6.3 and continue.	Backhoe with bucket size ranging from 1-1/2CY to 3CY	Tractor operator and laborer	
7	Trench Bottom Preparation			Backfill material
7.1	Level the trench bottom maintaining the longitudinal grade that will ensure the self cleansing velocity of the storm water through the HDPE pipe.	Instrument to check longitudinal slope	Site engineer, Supervisor, Laborer	
7.2	Procure bedding material that meets the gradation requirement as recommended the draft specification same way as it will be obtained for backfilling later on.	Truck	Truck driver, laborer	
7.3	Place bedding material on the trench bottom, spread it all over until a uniform minimum thickness of 6 in. is obtained as recommended in the draft specification	Backhoe, Loader, Crane.	Tractor operator and laborer	

Table 6.2. (Continued)

7.4	Compact the bedding with vibratory plate compactor or tamping rammer to ensure an stable foundation of granular material for HDPE pipe to be installed as recommended in the draft specification.	Vibratory plate compactor, tamping rammer	Laborer	
7.5	To reduce the possibility of Lateral movement of light weight HDPE pipe, trim the trench bottom in a concave shape to a height of 1/10 of pipe diameter as recommended in the draft specification.	A curved template might be used to obtain a concave shaped trench bottom	Laborer	
8	Laying and joining of pipe:			HDPE pipe, Joints
8.1	Procure HDPE pipe and store it in a shaded area to prevent ultra violet degradation as close to the trench as possible.	Truck	Truck driver, laborer	
8.2	Tie the pipe with two loops at each ¼th position from pipe ends and then join this loops with hook of backhoe or crane; start laying of pipe at the outlet that will proceed to the inlet end (up stream)	Backhoe, Crane	Tractor operator and laborer	
8.3	Joining: use Integral Bell-N- Spigot joint or Exterior Bell-N-Spigot joint as recommended in the draft specification. At each joint position	Manually	Laborer	

Table 6.2. (Continued)

		some bedding material can be removed so that any offset from joint can rest on lowered foundation without disturbing the pipe alignment. This will also eliminate the possibility of intrusion of any soil from the bedding into the pipe during joining.			
9		Placement and compaction of backfill material:			Backfill material
	9.1	Procure suitable backfill material that meets the strength or gradation requirements as specified in the draft specification.	Truck	Laborer	
	9.2	Provide a mixing plant to obtain the recommended gradation of the granular backfill and bedding material if it is not vailable in a 'ready made' condition but the ingredients are available.	Mixing plant	Laborer, site engineer	
	9.3	Place the backfill material and spread it uniformly so that 8 inch thickness is obtained every time it is placed. Continue backfilling until the backfill height reaches 1 ft above the pipe	Backhoe, Loader or Crane.	Tractor operator, laborer	

Table 6.2. (Continued)

	crown. Above that height use excavated native soil		
9.4	Compact the backfill material placed in the trench in every 8 inch lift as recommended in the draft specification. The zone under the laid pipe is difficult to compact using a compactor, compact manually by hand in that critical zone.	Vibratory plate compactor or tamping rammer.	Laborer.

Table 6.3. Maximum Digging Depth.

Bucket capacity (not heaped)	¼ CY-5/8 CY	¾ CY	1 CY	½ CY	2 CY	2 ½ CY	3 CY
Backhoe's Maximum digging depth	5 m	6 m	7 m	8 m	8 m	8 ½ m	9 m

Table 6.4. Backhoe's Operating Weight.

Maximum bucket capacity	Operating weight
0.82 CY	27,910 lbs
0.97 CY	35,100 lbs
1.83 CY	50,000 lbs
2.12 CY	59,560 lbs
2.75 CY	73,880 lbs
3.40 CY	110,420 lbs

Source: Caterpillar Performance Handbook, Edition 26 (*43*).

Table 6.5. Equipment Weight.

Equipment	Capacity/Diemenstion Range	Corresponding Operating Weight
Hydraulic backhoe	0.82 CY-3.40 CY bucket capacity	27,910 lb. -110,420 lb.
Backhoe loader	14'6''-21'5'' digging depth	13,700 lb. - 19,603 lb.
Pipelayers	40,000 lb.- 230,000 lb. Lifting capacity	34,600 lb. –1 49,600 lb.
Wheel tractor-scrapers	20 CY - 44 CY heaped capacity ..	70,700 lb. – 146,770 lb.
Construction and mining trucks	35 – 240 US ton	68,750 lb. – 323,709 lb.[1]
Wheel loaders	1.6 CY – 40 CY	15,836 lb. – 395,900 lb.
Duel drum vibratory asphalt compactor	39.4'' – 78'' drum width	5115 lb. – 26,920 lb.
Padded drum vibratory asphalt compactor	48'' – 88'' drum width	9300 lb. – 25,700 lb.
Pneumatic tire asphalt compactors	6612 – 11020 lb. wheel load	46,285 lb. – 77,140 lb.[2]
Single drum vibratory compactor	67'' drum width	29,106 lb.
Smooth drum vibratory soil compactors	48'' – 84'' drum width	9200 lb. – 33,590 lb.
Cold planers(Paving)	79'' – 150'' cutting width	58,000 lb. – 103,130 lb.
Reclaimer mixtures/stabilizer mixtures	96'' cutting width	39,800 lb.
Asphalt pavers	3' – 32' paving width	9000 lb. – 40,000 lb.
Road widener	10'-14' laydown width	29,500 lb. – 37,500 lb.

Note:
[1]Maximum corresponding gross weight ranges from 149,000 lb. – 830,000 lb.
[2]Operating weight when full
Source: Caterpillar Performance Handbook, Edition 26 (*43*).

Table 6.6. Minimum Trench Width(inch) Recommendation in Various Specifications and Suggested by Various Pipe Manufacturers.

	ADS[1]	CPPA[2]	Sec 30[3]	Hancor[4]	AISI[5]	NCSPA[6]	Unibell[7]	Section 26&12[8]	ACPA[9]	Sec 27 & 17[10]	ASTM D2321[11]	A.H.[12]	Draft Specs[13].
Dia (in)													
12	31	24	30	24	24	24	30	60	21	21	27	32	-
15	34	30	35	30	27	27	30	63	25	25	31	35	-
18	39	36	39	36	30	30	32	66	30	30	35	38	38.50
24	48	48	48	48	36	36	36	72	41	41	42	60	46.75
30	66	54	57	54	42	42	42	78	51	51	50	66	57.08
36	78	60	66	60	48	48	48	84	61	61	57	72	65.00
42	83	66	75	66	54	54	54	90	65	65	65	78	70.43
48	89	78	84	72	60	60	60	96	75	75	72	84	77.87
54		83	93	78	66	66		102	84	84	80	90	-
60		84	102	84	72	72		108	93	93	87	96	-

Notes:

[1]ADS values from the current installation instructions
[2]CPPA Structural Integrity of Non-Pressure Corrugated Polyethylene Pipe.
[3]Sec. 30 is the ballot copy of the new Section 30 for the AASHTO Bridge Committee.
[4]Hancor is from their literature (*44*).
[5]AISI is from the AISI Handbook of Steel Drainage & Highway Construction.
[6]NCSPA is from their installation brochure.
[7]Unibell Handbook of PVC Pipe, Chapter X.
[8]Section 26 and Section 12 take the trench widths from the AASHTO Bridge Design Manual.
[9]The ACPA trench widths are from the SIDD program.
[10]Section 27 is from the AASHTO Bridge Design Manual.
[11]ASTM D2321 is the thermoplastic pipe installation practice from ASTM
[12]A.H. is from Amster Howard's book
[13]Draft Specification, dated May 1998.

Table 6.7. Minimum Trench Width Required to Use Compactors.

Nominal diameter	Outer diameter	Minimum trench width calculated according to the draft specification	Minimum space between trench wall and pipe	Width of a typical reversible vibratory plate compactor	Minimum trench width required to operate a compactor
18 in (450mm)	21.20 in (536 mm)	38.5 in	8.65 in	12 in	21.2+2*18= 57 in
24 in (600mm)	27.80 in (719 mm)	46.75 in	9.475 in	12 in	27.8+2*18= 64 in
30 in (750mm)	36.07 in (917mm)	57.08 in	10.5 in	12 in	36.07+2*18= 72 in
36 in (900mm)	42.46 in (1073mm)	65 in	11.3 in	12 in	42.46+2*18= 79 in
42 in (1050mm)	46.75 in (1187mm)	70.43 in	11.84 in	12 in	46.75+2*18= 83 in
48 in (1200mm)	52.7 in (1339mm)	77.875 in	12.6 in	12 in	52.7+2*18= 89 in

Table 6.8. Minimum Trench Width.

Nominal Pipe Diameter		Minimum Trench Width	
inches	mm	inches	mm
18	450	44	1100
24	600	54	1350
30	750	66	1650
36	900	78	1950
42	1050	84	2100
48	1200	90	2250

Table 6.9. Gradation Requirements for Type III Backfill.

Sieve No.	Percent Retained (Cumulative)
1 inch	0-5
$^{7}/_{8}$ inch	0-35
½ inch	0-75
$^{3}/_{8}$ inch	0-95
No. 4	35-100
No.10	50-100
No.200	90-100

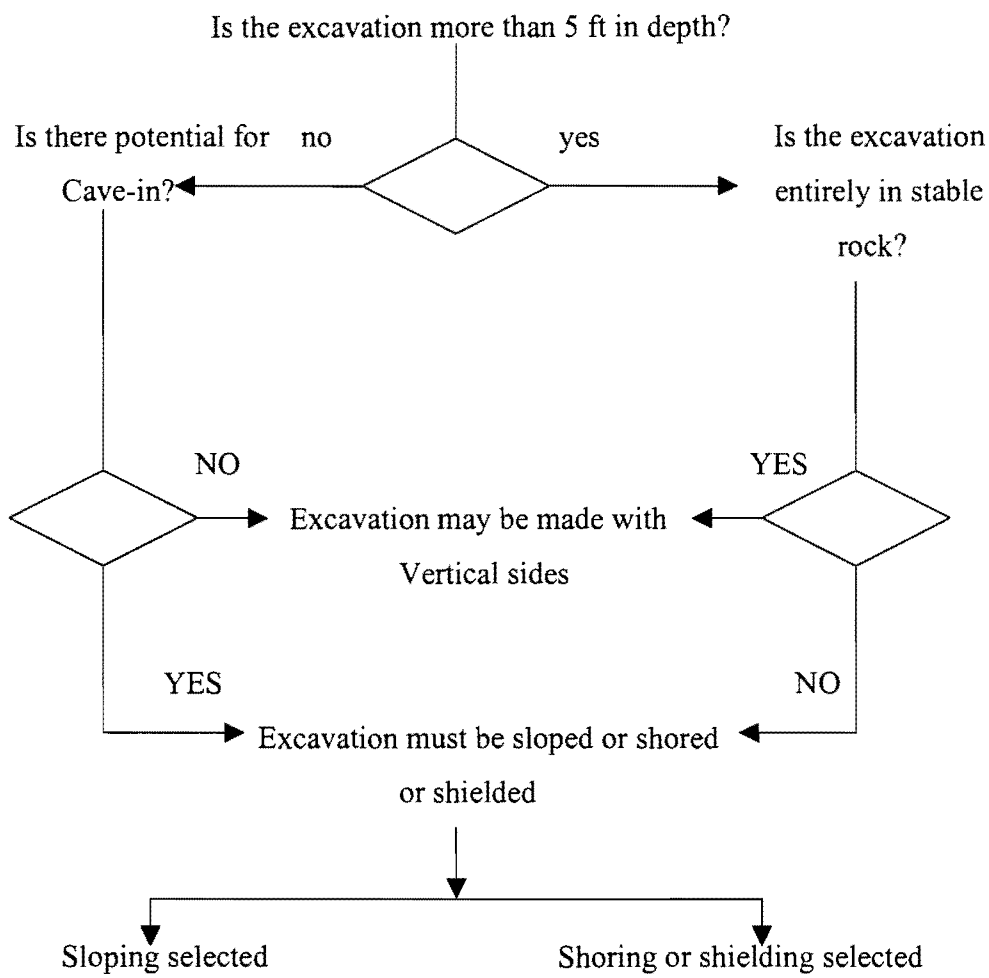


Figure 6.1. Flow Chart for Selection of a Trench Protection System.

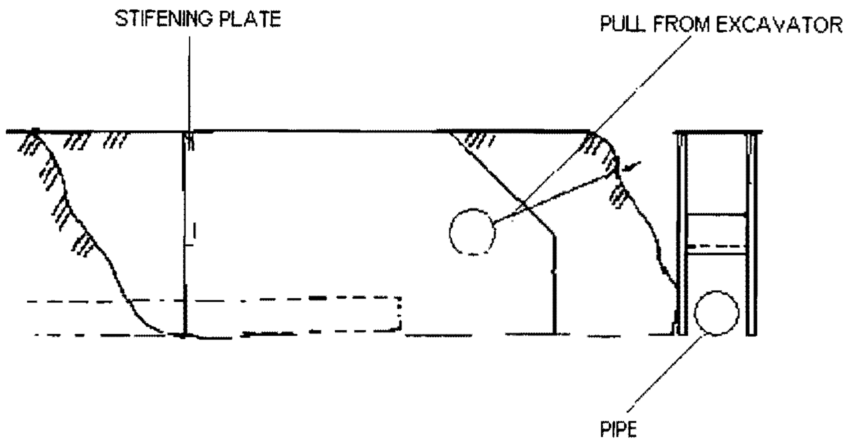


Figure 6.2. Drag Box Installation.

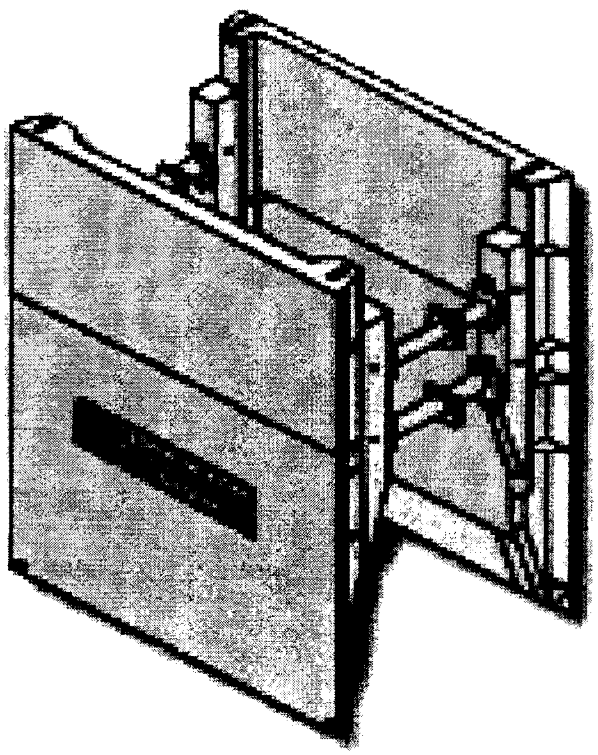


Figure 6.3. Trench Box Module.

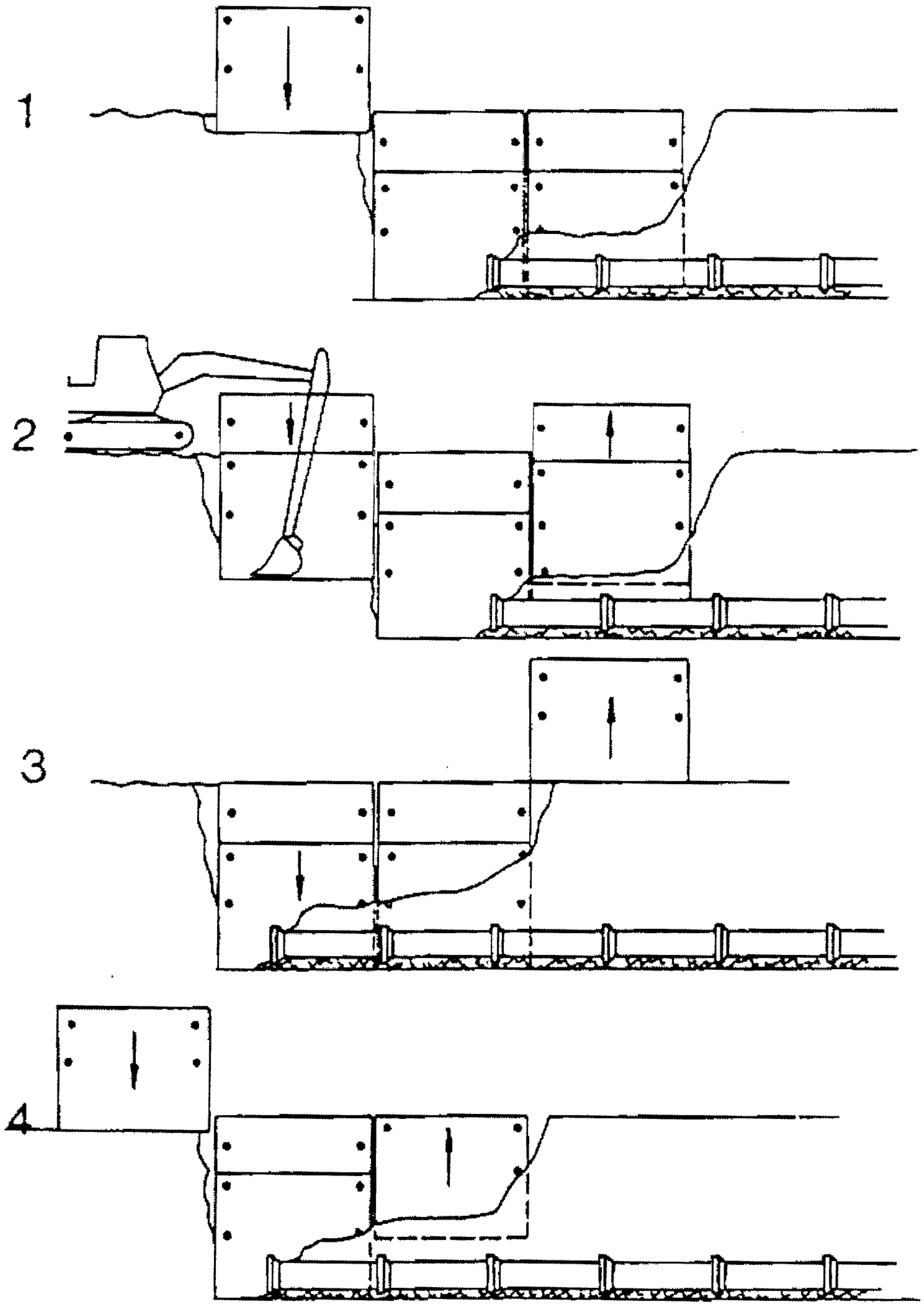


Figure 6.4. Pipe Installation with Trench Boxes.

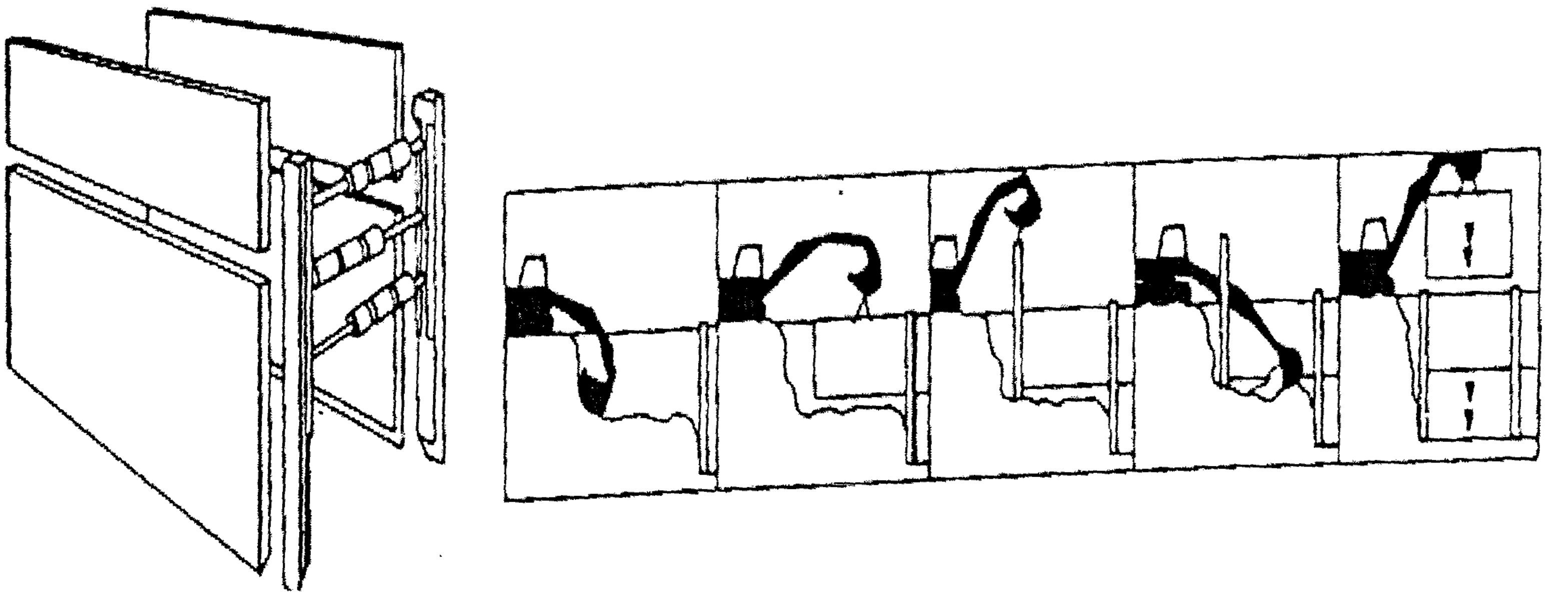


Figure 6.5. Pipe Installation with Slide Rails.

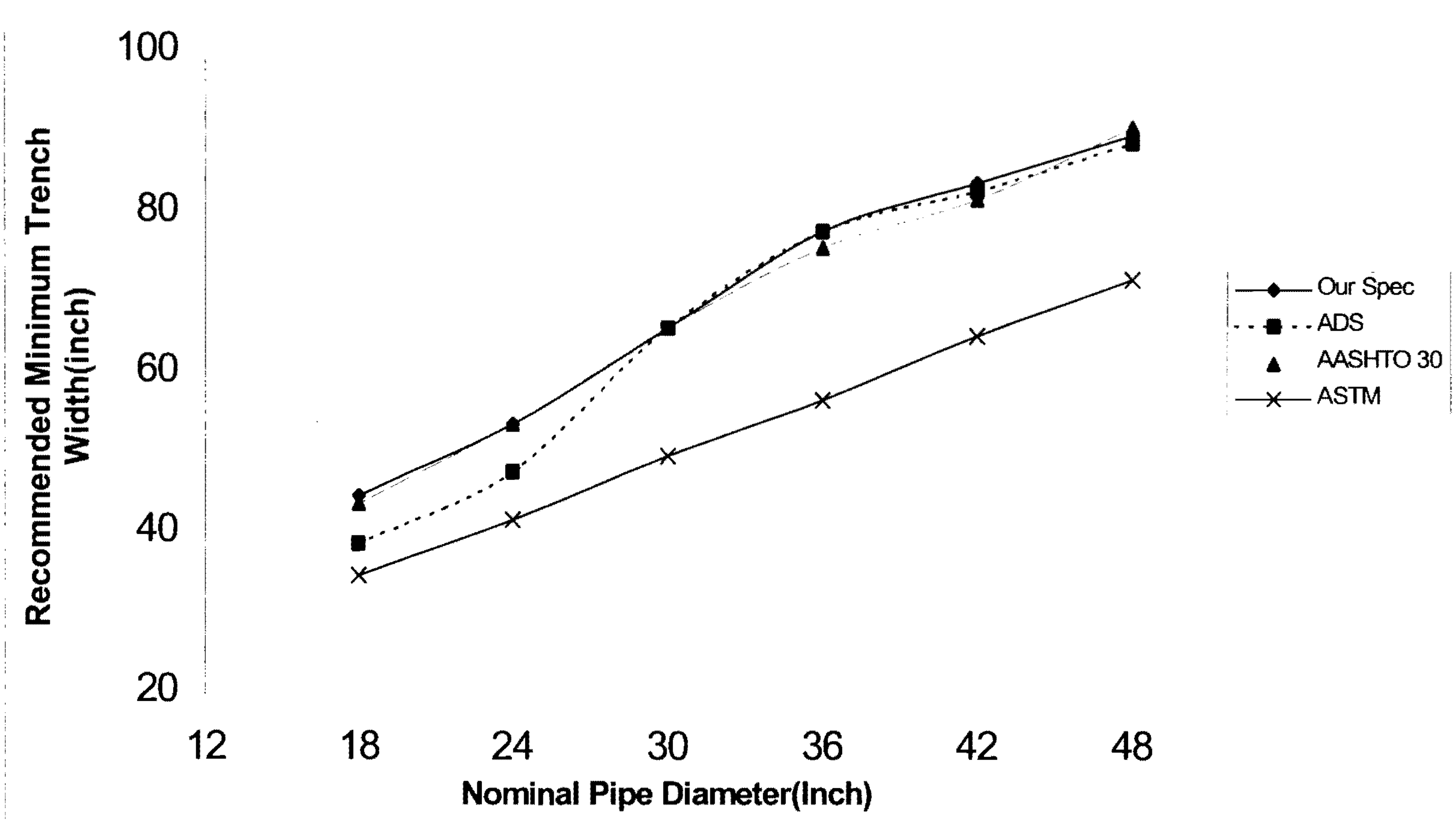


Figure 6.6. Comparison of Some of the More Commonly Used Minimum Trench Width Guidelines.

3.1 Minimum Soil Cover

The minimum cover from the pipe crown to the top of the road subgrade or ground surface should be as specified Table 2 below.

Table 2. Minimum Soil Cover

Type of Pavement	Minimum Cover	
	inches	mm
Rigid Pavement	12	300
Flexible Pavement	18	450
Unpaved Roadway	27	675
No Vehicular Loading	21	525

4.5.4 Final Backfill - Final backfill consists of the zone that extends from 1 ft (300 mm) above the crown of the pipe to the base or final grade. Placement and compaction of the final backfill should be performed according to specifications provided in the plans. Heavy construction vehicles should not be allowed to cross over the pipe until the compaction has been completed to the finished earthwork grade or to an elevation of at least 4 ft above the crown the pipe. If the passage of construction equipment over an installed pipeline is necessary during construction, compacted fill in the form of a ramp shall be constructed to a minimum elevation of 4 ft (1.2 m) above the crown of the pipe. Any damaged pipe shall be replaced at the contractor's expense.

Figure 6.7. Excerpts from Draft Specifications that Address Minimum Cover Requirements.

CHAPTER VII
ECONOMIC ANALYSIS

7.1 Introduction

The new specifications that were being developed in this research were expected to meet two primary requirements. First, it must ensure reliability of the HDPE pipe installation. Secondly, construction according to the specifications should be economical. Therefore, economics related to HDPE pipe installation according to the new specifications was one of the key issues to be addressed in this research. This chapter describes a detailed analysis that was conducted to determine the costs associated with HDPE pipe installation. This analysis includes a comparative analysis of as-installed costs of large diameter HDPE pipe and RCP pipe.

It must be pointed out that, ideally, a comparison between two pipe products should not only consider the cost of installation but also include the costs associated with their maintenance and operation. However, such a comparison is not feasible at this time because long-term data on pipe performance and maintenance is not available for HDPE pipe. Although HDPE pipe has been in use for some time, the use of HDPE pipe in the large diameter category is quite recent. Moreover, the technology and the material that is currently being used in HDPE pipe production and the designs of pipe joints are very recent. As a result, any data available on performance of exiting pipe systems is not likely to be very representative of the performance of HDPE pipes in the future. Therefore, the analysis conducted in this research was limited to a comparison between as-installed costs for the two pipe products.

Section 7.2 below includes a general review of factors that influence the pipe installation costs. The two main material resources required in large diameter pipe installation are pipe and trench backfill material. The unit prices for both of these, especially backfill material, depend largely on the project location. Therefore, the economic analysis conducted in this study also involved a study of how these parameters vary within the state. Accordingly, data on backfill material prices were collected from different TxDOT districts and reviewed. Section 7.3 of this chapter presents the results from the above data review and analysis.

Section 7.4 provides a preliminary comparison performed using a software named PipePac 2000. The last section of this chapter, Section 7.5, presents the findings from a parametric study. This parametric study is based on a detailed analysis that was conducted to obtain estimates of as-installed costs for a hypothetical pipe installation project when pipe installation is performed according to TxDOT specifications. It examines the influence of pipe material (HDPE vs. RCP), pipe diameter and backfill material price on overall project cost by varying each parameter within the complete range of values found within Texas. Several useful conclusions were drawn based on the findings from the above parametric study.

7.2 Factors Influencing Cost of Pipe Installation

Concrete pipe has thicker pipe wall and is heavier than a HDPE pipe of equivalent diameter. The heavier weight of concrete pipe results in higher transportation cost per mile. In other words, a truck with given weight carrying capacity can transport a much larger quantity (i.e. total pipe length) of HDPE pipe than concrete pipe. Very often, HDPE pipe are nested and stacked up higher on the truck allowing a larger quantity of HDPE pipe to be transported in a single trip. However, it is important to note that this advantage is somewhat offset by the longer distances that HDPE pipe must be transported in comparison to conventional RCP pipe. There are fewer number of HDPE pipe manufacturing plants when compared with RCP pipe manufacturing plants. For example, there are only two HDPE pipe manufacturing plants within the entire state of Texas: ADS pipe manufacturing plant in Enis (near Dallas) and Hancor pipe manufacturing plant in Yoakum. In comparison, there are more than seven RCP manufacturers within Texas as listed in Highway Dope Book and& Directory published by Whitley & Siddons (Dec. 1998). Some of those RCP manufacturers have more than one production plants located within Texas. Therefore, the average distance from the point of production to the project site is lower for RCP hat it is for HDPE pipe. A more detailed discussion of pipe prices can be found later in a subsequent section of this chapter.

In addition to possible savings in transportation costs, the lower weight of the HDPE pipe also results in savings in two other ways. First, HDPE pipe requires less labor in handling and placement. Secondly, the lighter weight of HDPE pipe allows the pipe to be manufactured in longer pipe lengths. The typical length of HDPE pipe (also called "stick length") is 20ft whereas the typical length of RCP is in the range of 8-10ft. Accordingly, HDPE pipe installation involves fewer joints. This results in savings in overall project time and therefore, project cost.

HDPE pipe, being a more flexible product than reinforced concrete pipe, depends more on the surrounding backfill material for structural support. Therefore, quality control during the placement and compaction of backfill is a very important aspect of HDPE pipe installation. For this reason, specifications for installation tend to be stricter for HDPE pipe than those for rigid pipe systems such as RCP. These specifications typically include requirements for special backfill materials, special precautions during handling and placement of the pipe, special precautions to avoid potential problems due to pipe floatation, requirements on larger minimum covers to prevent pipe damage, requirements to measure pipe deflection to ensure satisfactory installation etc. Such requirements tend to drive the cost of HDPE pipe installation higher. Another drawback that is often cited by highway agencies with regard to the use of HDPE pipe on a routine basis is that their inability to provide the necessary close supervision. As a result, their HDPE pipe installation specifications tend to be conservative. Such conservative practices increase the cost of HDPE pipe installation even further.

7.3 Pipe and Backfill Material Prices in Texas

Data available from other state DOTs suggest that there may be significant savings from the use of HDPE pipe in highway drainage applications (*45, 46*). At the same time, however, it is important to point out that the pipe installation costs vary with the installation specifications used by each agency. The pipe installation costs also vary from one location to another depending on local conditions such as availability of specific types of backfill materials required by the specifications.

Therefore, before any conclusions could be reached concerning the potential savings to be gained by TxDOT from the use of HDPE pipe, it is necessary to perform a closer review of the specific conditions that exist within Texas. Consequently, a survey was conducted to determine the pipe prices and backfill material prices within various parts of Texas. Among all the parameters that influence the overall pipe installation cost, these two, *viz.* pipe price and the backfill material price are most liable to vary from one region to another. The findings are summarized in the following two sections 7.3.1 Pipe Price and 7.3.2 Backfill Material Price. Subsequently, in Section 7.5 this information is used in a detailed economic analysis to determine cost of pipe installation according to draft specifications developed in this research.

7.3.1 Pipe Price

In order to have the most recent pipe price information in different parts of Texas, the two largest HDPE pipe manufacturers in Texas and two concrete pipe manufacturers were contacted. Each of these companies was asked to provide price quotations for their products. Table 7.1 lists price of smooth interior corrugated HDPE pipe for pipe diameters ranging from 18in to 48in. The listed prices include the price of pipe, joints and freight cost. Manufacturer 1 recommend that if delivery is requested in less than 5 days after the order, extra freight cost will be added. For overnight delivery extra \$200+\$1.67 per mile is required. The prices listed for manufacturer 2 could be considered delivered prices anywhere within Texas, provided the order was in full truck load quantities. If the order was less than truck load amounts, an extra freight charge could be applied. This extra charge would be no more than \$150.

Similarly, prices per ft of ASTM C-76 Class III (Tongue and Groove Joint) reinforced concrete pipe from the two RCP companies are shown in Tables 7.2, 7.3 and 7.4. Table 7.2 shows the price quotations provided by the first RCP manufacturer in the vicinity of San Antonio while Table 7.3 shows the RCP pipe prices for Dallas Fort Worth and vicinity. Table 7.4 are price quotations obtained by a second RCP manufacturer, also for Dallas Fort Worth metropolitan area and its vicinity. These figures include the price of the joints.

One significant difference that can be observed between the price tabulations for HDPE pipe and RCP is that RCP prices vary with the distance between the manufacturing plant and the project location whereas HDPE prices do not. This difference is due to the significant costs associated with the transportation of RCP. This is an interesting observation because, in comparison to RCP, the HDPE pipe manufacturing plants are few and far between. For example, each of the two concrete pipe suppliers mentioned above, has six RCP manufacturing plants in different parts of Texas. The HDPE pipe manufacturer, on the other hand, has only one manufacturing plant in Texas that serves the whole state and some of the neighboring states as well. Nationwide, they have 33 storage locations and 4000 independent distributors. Nevertheless, the average distance from the point of production of HDPE pipe to point of pipe installation is much larger than for RCP pipe. The information presented in Tables 7.2, 7.3 and 7.4 shows that despite longer transportation distance, HDPE pipe price is lower than concrete pipe.

It was also reported that price of RCP varies with the quantity ordered. In other words, contractors receive discounts when they order large quantities. Based on the information received from RCP manufacturers, these discounts may be in the order of 10-20%. According to information presented in Table 7.2, the minimum listed price for RCP corresponds to RCP pipe produced at San Antonio plant and delivered within Zone 1. Accordingly, the least possible price of RCP could be calculated by applying percent discount to that minimum RCP price. Figure 7.1 presents a comparison of HDPE pipe price against RCP pipe prices for pipe diameters ranging from 18in to 48in. In this comparison the HDPE prices from both the manufacturers were used. Because RCP prices varied from one delivery zone to another, the maximum price, the minimum price and minimum price with 15% discount are plotted. It shows that HDPE pipe price listed by each of the two manufacturers is lower than RCP pipe price even when a high discount rate of 15% is applied.

7.3.2 Backfill Material Price

Backfill material is one of the most important among all the factors that control the as-installed cost of the HDPE pipe installation. The new backfill material specifications have been developed so that satisfactory pipe performance can be ensured without the need for elaborate quality control measures during backfill placement and compaction. Most native soils will not meet these specifications and therefore, suitable backfill materials must be obtained from outside sources and transported to the site. Thus, the specified backfill material comes at a cost and it is an important element that must be considered in the estimation of the as-installed cost of the pipe. The availability of a given type of backfill material and its price vary significantly from one location to another. Therefore, as part of this economic analysis, information on availability and price of each type of specified backfill material types within different regions of the state was collected and reviewed. The findings from the above review is presented in this section of the report.

The following section povides a detailed description of the data collection procedure. It also presents all the data collected in tabular form. Subsequently, it presents the findings from the data review and analysis. The data review was performed with the following specific objectives in mind. First of all, the prices of different types of backfill material allowed in the specifications are reviewed to establish the general price range for each. Secondly, the price quotations obtained from different regions are examined to determine whether they show any trends or patterns with respect to geographical location. Finally, the findings from this review are used to determine the overall as-installed cost of the pipe with each type backfill material allowed by the current specifications. These as-installed costs are also compared with as-installed costs of RCP pipes of the same diameter.

The first step in the review process involved the collection of data on current backfill material prices in various parts of Texas. This was accomplished in two different ways:

(a) Contacting laboratory engineers in TxDOT districts and collecting their input on prices of suitable types of backfill materials that are economically available in each district
(b) Extracting data from TxDOT's website that publishes 12-month average unit bid prices based on construction projects that have been completed in each district.

Backfill price data obtained from district lab engineers in all 25 TxDOT districts are listed in Appendix B. Table 7.6 is an excerpt from Appendix B that lists some of the more economical materials available in various parts of Texas. Similarly, data collected from average bid price tabulations published in TxDOT's website are summarized in Tables 7.7, 7.8, 7.9 and 7.10. They represent the average unit bid prices compiled from numerous projects during the year, 1999. Table 7.9 shows unit bid prices for cement stabilized backfill (also referred as Type II backfill in the specification for thermoplastic pipe) whereas Table 7.10 summarizes the bid prices for flex base materials.

However, not all the data that were collected in this manner could be directly used in further analysis. First of all, it could be easily seen that unit bid prices for backfill material were quite sensitive to the quantity of material supplied. In general, the unit price decreases as the quantity supplied increases. As a result, unit price applicable to one project may not be directly compared with unit price for another unless the quantities of backfill material supplied in the two cases are similar. Another factor that makes direct comparison of backfill material difficult involves the form in which backfill prices are reported. For example, in Table 7.10, unit bid prices of flexible base material are tabulated in two forms: 'roadway delivery' price and 'complete in place' price. Roadway delivery price refers to the price of flexible base material delivered at the job site and thus, it includes the cost of transportation of the material. On the other hand, price of flex base in 'complete in place condition' includes cost of material, cost of transportation and cost of placing of placing the material and compacting it to specified density. Additionally, because of the volume reduction associated with compaction of the material, it will be incorrect to compare the 'roadway delivery" price with 'complete in place' when these prices are reported in $/CY. An even greater difficulty arises with unit prices of coarse aggregate material used in bituminous mixes. The prices quoted by most laboratory engineers represented the price of the bituminous mix that included both aggregate and asphalt binder. Many times these prices represented the price of the bituminous mix in 'complete in place' condition. Because of these ambiguities, the bituminous coarse aggregate prices were not included in any further review or analysis.

Among all the backfill materials prices, flowable fill (i.e. Backfill Material Type I in the specifications) were the highest. Table 7.7 shows flowable backfill prices for several districts of Texas. The unit price of flowable fill varied in the range from $53/CY to $130/CY. However, it must be noted that although the unit price of flowable fill is higher than that for any other backfill material type, it does not necessarily mean that flowable backfill will result in higher as-installed cost. First, the specifications allow the use of a smaller trench width when flowable fill is used. Therefore, the volume of flowable fill required in a given installation is less than the volume of any other backfill material type. Secondly, this type of backfill does not require any compaction thus allows faster installation. These factors may partly offset the higher cost of backfill material.

Review of unit price tabulations found on TxDOT website reveals that cement stabilized material, i.e. Type II Backfill in the specifications, is more widely used in TxDOT construction projects than flowable backfill. Table 7.8 shows the cement stabilized backfill prices for 24 TxDOT districts. The last column in the table shows the weighted average price calculated for each district. Comparison of these unit prices show that the price of cement stabilized backfill vary significantly. Most of the data lie in the range between \$26/CY and \$87/CY although there are a few data points can be found outside this range. Secondly, it can be observed that the prices of cement stabilized backfill show a clear price trend according to different geographical location. Table 7.9 categorizes the cement stabilized backfill price into three different ranges; low (less than \$40/CY), medium (between \$40/CY and \$70/CY) and high (greater than \$70/CY). Subsequently, based on the weighted average price calculated for each TxDOT district, they are divided into different price zones. These different price zones are depicted in Figure 7.2. It shows that the price of cement stabilized is lower near the gulf coast. The prices show a general increasing trend as you move away from the gulf coast with some of the West Texas and northern districts showing the highest prices.

The third type of backfill allowed in the specifications is granular backfill. In most regions, granular backfill is found to be the most economical option among the three types of backfill materials specified. Once again, review of granular backfill material prices was performed based on input received from district laboratory engineers as well as unit bid price tabulation available on TxDOT website. Table 7.6 provides a summary of information collected from district lab engineers. Review of these data shows that the unit price of granular backfill range from about \$9.50/CY to about \$25.00/CY. The next step involved review of information available on TxDOT unit bid price tabulations. Although, granular material with gradations similar to those specified is used in many applications including Hot Mix Asphalt Concrete, Surface Treatment, Portland Cement Concrete, the unit prices for these items cannot be used in this review because they do not represent cost of aggregate material alone. The only exception was Item 247: Flexible Base. Therefore, overall average bid prices for flex base in 'roadway delivery condition' were obtained for 15 TxDOT districts. They are listed in Table 7.10. The typical price range appears to be between \$7.00/CY and \$25.00/CY. It is worthwhile noting that unit prices reported for 11 out of 15 districts are less than \$15/CY. At the same time, however, it must be pointed out that not all flexible base materials may qualify for use as backfill under the proposed specifications because of higher fines (minus 200) content. Table 7.10 also lists overall bid prices of flex base as 'complete in place' condition for all 25 district which varies in the range of \$14.51-\$35.51. Data presented in Table 7.10 indicate that flex base is an economically available backfill material at an average price in almost every district of Texas. Based on the above review, \$10.00/CY was selected as a typical price for granular backfill at the low end while \$15.00/CY was selected as a representative figure for medium price granular backfill.

Based on the above review, the unit prices listed in Table 7.11 were selected as representative figures for each backfill material category. These unit prices are subsequently used in the estimation of as-installed costs of HDPE pipe. This analysis is presented in Section 7.5 below.

7.4 'PipePac 2000' Comparison of HDPE and RCP As-installed Cost

As a preliminary step, a comparison of HDPE and RCP as-installed costs was performed using Cost Analysis of Pipe Envelope (referred to as CAPE) of the software named PipePac 2000. The copyright for this software is held by American Concrete Pipe Association. 'PipePac 2000' allows calculation of as-installed costs for both RCP and HDPE pipe.

This section summarizes the results from the analysis performed for several example-projects of HDPE and RC pipe-installation in order to get a prelimnary comparison. The main parameters required as input for the analysis are pipe price, pipe diameter, hauling and tipping rate and granular backfill price. Trench dimensions such as trench width, bedding depth, excavation height receive default values once a diameter is selected for an analysis. As-installed cost of HDPE and RC pipe estimated using this software is listed in Table 7.12. RCP Installation type Class C is listed which assumes the use of native soil for backfilling. RCP price considered in this analysis is the minimum listed pipe price of Hanson Concrete Products (*48*). Whereas HDPE pipe price is typical price for Texas region listed by Advanced Drainage System, Inc. Savings from using HDPE pipe were calculated accordingly. Table 7.12 shows that approximately 7% to 9% savings on using HDPE pipe is possible over RCP.

This analysis was considered preliminary because of several significant limitations. The software estimates the total pipe installation cost as the sum of cost of the pipe, cost of backfill material and cost of removal and disposal of excavated native soil. It does not consider cost of labor and equipment for the installation. Therefore, it does not appropriately consider reduction in cost due to less labor and faster installation of HDPE pipe resulting from the lighter weight and longer joint spacing for this type of pipe. A more comprehensive analysis that includes all of the factors mentioned above is described in the next section.

7.5 As-Installed Costs for HDPE and Concrete Pipe

This section describes an economic analysis that was conducted in this study to estimate the as-installed costs for both HDPE pipe and RCP according to TxDOT specifications. This analysis was performed for a hypothetical pipe installation project where the pipe diameter, pipe price and backfill material price were varied within the typical ranges as established in Section 7.3. The findings from this analysis is presented graphically so that comparisons can be made between installation costs for HDPE and RCP for a variety of pipe diameter, pipe price, backfill price combinations. Section 7.5.1 presents details with regard to sources of data used and assumptions made during analysis.

7.5.1 Sources of Data and Assumptions

A detailed work breakdown structure for the entire pipe installation process was developed in Chapter VI as the part of the constructibility review. This work breakdown structure is presented in Table 6.2. Table 6.2 lists sub-tasks associated with different stages of construction as well as resources (i.e. manpower, equipment and materials) needed for the completion of each sub-task. In this cost analysis the above work breakdown structure was further expanded to include the costs associated with each construction activity. This enhanced work break down structure for installation of HDPE pipe is given in report as Appendix D. This new itemization forms the primary basis for the economic analysis presented here. The analysis in this section calculates approximate as-installed cost of HDPE pipe and concrete pipe installation of 18 in., 24 in., 30 in., 36 in., 42 in. and 48 in diameter for identical utility. The whole analysis is done in one MS Excel Spread Sheet which has been attached as Appendix E. The sources of relevant resource price information for this analysis are as follows.

(a) Information on equipment rental price, equipment capacity, equipment size, workers wage and installation cost per unit for each sub-task is available in Table D.1 of Appendix D. Appendix D was compiled following National Construction Estimator edited by Killey and Allyn (1997) (*49*) and Means Heavy Construction Cost Data (1998) (*50*).

(b) Backfill material price of six categories has been used from Table 7.11 that was selected after a thorough review on all available backfill price data presented in section 7.3.2.

(c) From Table 7.1, HDPE pipe price of manufacturer 1(Advanced Drainage Systems, Inc.) (*47*) was used for estimating as-installed cost. Among the prices of RCP listed in Table 7.2, 7.3 and 7.4, RCP unit prices of the following categories have been selected to be used in this estimating:

   i. Minimum RCP price, Zone 1 of CSR Hydro-conduit, with 15% discount
   ii. Minimum RCP price, Zone 1 of CSR Hydro-conduit without discount.
   iii. Minimum RCP price (FOB), priced by Hanson Concrete Products.
   iv. Minimum RCP price, Hanson Concrete Products with 15% discount.
   v. Maximum RCP price, priced by Hanson Concrete Product.
   vi. Discounted RCP price of Zone2, CSR Hydro Conduit.
   vii. Discounted RCP price of Col. 3, Table 7.4, Hanson Concrete Products.
   viii. Discounted RCP price of Col. 4, Table 7.4, Hanson Concrete Products.
   ix. Discounted RCP price of Col. 6, Table 7.4, Hanson Concrete Products.

The model construction project used for this analysis assumes a flat terrain and the trench depth required to be the minimum. For concrete pipe, it was assumed that the excavated native soil could be used as backfill. For HDPE pipe backfill material of six selected backfill types with prices as shown in Table 7.11 were considered. Nine different concrete pipe prices, mentioned previously, were used. For 18 in., 24 in. and 30 in diameter pipe two temping rammers with 10 in. plate width have been used. And for 36 in., 42 in., and 48 in. diameter pipe two vibratory plate compactors with 12 in. plate width have been used. It was further assumed that two large capacity backhoes would be rented for the job. Backhoe1 would be doing the whole excavation. Backhoe2, the larger of the two backhoes, would perform three tasks: backfilling, installing trench boxes and laying pipe in the trench. The project duration will be determined by the total busy hours of Backhoe2. Backhoe1 would do some other subtasks like putting the excavated material in the hauling truck during its idle time.

MS Excel Spreadsheet was chosen as a convenient environment to perform this analysis. The following section provides a review of the findings from the as-installed cost comparison.

7.5.2 Review on Findings from As-installed Cost Comparison

Table 7.13 presents the as-installed costs in $/linear ft for both HDPE and RCP while Table 7.14 documents the percent savings from each comparison. Analysis was conducted for pipe diameters ranging from 18 in. to 48 in. for both types of pipe. In the case of HDPE pipe, which requires special backfill, the as-installed costs varied with backfill price used in analysis. Six different backfill prices that were previously identified in Table 7.11 as representative were used. In the case of RCP, the backfill was assumed to be native soil and therefore backfill price was did not enter into analysis. Instead, RCP as-installed cost varied with pipe price. Pipe price was varied depending on the delivery zone as well as on whether a discount was applied or not.

The results are also presented in the form of graphs for easy review. Figures 7.4, 7.5 and 7.6 summarize this information. For HDPE pipe as-installed cost curves corresponding to following backfill materials are shown in these figures; (a) granular fill (at $10/CY and $15/CY), (b) flex base (at $18/CY), (c) cement stabilized backfill (at $32/CY and $60/CY) and (e) flowable fill (at $85/CY). The above HDPE as-installed cost data is compared with RCP as-installed cost estimates corresponding to various RCP price conditions. This comparison is presented in three separate figures to avoid over crowding of information.

The analysis shows that the more cost effective pipe product for a given project and the margin of savings offered by one type of pipe over the other will depend on conditions specific to that project. Nevertheless, some general conclusions can be draw from these findings. In many situations, HDPE pipe constructed with granular backfill may provide lower as-installed costs compared with RCP pipe installed with native backfill. This would be particularly true if granular backfill can be found at $15/CY or less. HDPE pipe as-installation cost are much higher when cement stabilized or flowable backfill are specified. It should also be noted that the lower as-installed cost for HDPE pipe with flowable fill at $85/CY compared to HDPE pipe with cement stabilized backfill at $60/CY is not an anomaly but a result of the narrower trench width and the faster rate of installation that is possible when flowable fill is used.

Table 7.1. Typical Smooth Interior Wall Corrugated HDPE Pipe Pricing (May 12, 1999).

Diameter	Length	Price/ft ($) Manufacturer 1 (*47*)	Price/ft ($) Manufacturer 2 (*44*)
18''	20'	6.30	5.95
24''	20'	9.71	9.44
30''	20'	15.97	16.65
36''	20'	19.43	20.00
42''	19.5'	30.45	27.00
48''	19.5'	36.71	34.00

Table 7.2. 1998 Price of ASTM C-76 Class III RC Pipe (Supplied from San Antonio Plant, CSR Hydro Conduit).

Size	Joint	Length	Weight (lbs /ft)	Listed price ($/LF) For Supply Zone 1	Listed price ($/LF) For Supply Zone 2
18''	T & G	8'	168	10.30	10.90
24''	T & G	8'	265	15.50	16.50
30''	T & G	8'	384	22.10	23.80
36''	T & G	8'	524	31.90	34.50
42''	T & G	8'	685	44.80	47.10
48''	T & G	8'	867	55.30	58.40

Table 7.3. Price of ASTM C-76 Class III RC Pipe (Supplied from Dallas/Ft. Worth Metro Area Plant, CSR Hydro Conduit).

Size	Joint	Length	Weight (lbs /ft)	Listed price ($/LF) FOB Plant	Listed price ($/LF) Col 1	Listed price ($/LF) Col 2	Listed price ($/LF) Col 3	Listed price ($/LF) Col 4
18''	T & G	8'	168	8.77	9.02	9.19	9.36	9.53
24''	T & G	8'	265	13.02	13.51	13.81	14.10	14.40
30''	T & G	8'	384	18.52	19.30	19.79	20.27	20.76
36''	T & G	8'	524	26.58	27.83	28.58	29.34	30.10
42''	T & G	8'	685	38.15	39.57	40.23	40.89	41.56
48''	T & G	8'	867	46.97	48.82	49.74	50.66	51.59

Table 7.4. Price of ASTM C-76 Class III RC Pipe (Supplied from Dallas/Ft. Worth Metro Plants, Hanson Concrete Products, *48*).

Size (inch)	Joint	Approx. Weight (lbs /ft)	Listed price ($/LF) FOB	Listed price ($/LF) Col. 1	Listed price ($/LF) Col. 2	Listed price ($/LF) Col. 3	Listed price ($/LF) Col. 4	Listed price ($/LF) Col. 5	Listed price ($/LF) Col. 6	Listed price ($/LF) Col. 7	Listed price ($/LF) Col. 8	Listed price ($/LF) Col. 9	Listed price ($/LF) Col. 10	Listed price ($/LF) Col. 11
18	T & G	170	11.75	12.25	12.50	12.75	13.00	13.25	13.75	14.00	14.25	14.50	14.75	15.25
24	T & G	268	17.5	18.25	18.50	19.00	19.25	19.75	20.00	20.25	20.50	20.75	21.25	21.50
30	T & G	389	24.75	25.75	26.50	27.25	27.75	28.50	29.00	29.50	30.25	30.75	31.50	32.00
36	T & G	533	35.25	37.00	38.00	39.25	40.00	41.00	42.00	42.75	43.75	44.50	45.50	46.50
42	T & G	697	50.50	52.75	53.25	54.25	55.00	56.00	57.00	57.75	58.75	59.75	60.50	61.50
48	T & G	883	62.25	64.75	66.00	67.25	68.25	69.75	71.25	72.25	74.25	75.75	77.25	78.75

Table 7.5. Delivery Zones by County (Deliveries from Dallas/Fort Worth Plants of RCP pipe).

Column Number	Counties	Distance from the factory
Col. 1	Dallas, Tarrant	20 - 30 miles
Col. 2	Collin, Denton, Ellis, Hood, Johnson, Kaufman, Parker, Rockwell, Wise	40 - 60 miles
Col. 3	Hill, Navarro, Somervell	70 - 85 miles
Col. 4	Anderson, Bosque, Cooke, Erath, Fannin, Grayson, Henderson, Hunt, Jack, Montague, Palo Pinto, Van Zandt,	90 – 110 miles
Col. 5	Clay, Freestone, Limestone	110 – 120 miles
Col. 6	Archer, Comanche, Eastland, Stephens, Young	120 – 135 miles
Col. 7	Baylor, Callahan, Shackelford, Throckmorton, Wichita,	135 - 150 miles
Col. 8	-	
Col. 9	Haskell, Jones, Knox, Taylor, Wilbarger	180 – 225 miles
Col. 10	Fisher, Ford, Stonewall,	230 – 270 miles
Col. 11	Kent, King, Nolan	Above 270 miles

Table 7.6. Economically Available Granular Materials in Texas.

Backfill Material	Unit Price ($/ton)	Unit Price ($/CY)	District
Flowable Fill	40.00	64.80	Amarillo
River Gravel (round)	6.00	9.72	Atlanta
River Gravel (crushed)	8.00	12.96	Atlanta
Coarse Component of Item 340 Type B, C, D	8 - $12	12.96-19.44	Atlanta, Waco
Concrete Fine Aggregate	6.00	9.72	Austin
1 inch Pea Gravel	7.50	12.15	Austin
Item 340 without Asphalt	14.00	22.68	Austin
Cement Stabilized Backfill	15		San Angelo
Sand	9.25	14.98	Corpus Christie
Siliceous River Gravel	10.00	16.20	Pharr
Select Fill	3.12	$5.00	Tyler
Crushed Limestone (not readily available)	12.00	19.44	Waco
Pea Gravel (readily available)	7.00	11.38	Waco
Pea Gravel Used for Pipe bedding	15.00	24.30	El Paso
Grade 4 Aggregate	13.00	21.06	Wichita Falls
Flex Base	9.89	16.02	Wichita Falls
Flex Base	12.00	19.44	Yoakum

Table 7.7. Overall Average Bid Price of Flowable Fill in Texas Districts in 1999.

District	Item Number	Quantity	Overall Unit Bid Price of Flowable Backfill ($/CY)
Austin	4156	3475	53.47
San Antonio	4156	840	55.85
Beaumont	4158	381	68.85
Wichita Falls	4438	10	75.00
Corpus Christie	4438	5	100.00
Childress	4438	157	120.00
Bryan	4438	8	125.00
Pharr	4438	237	127.00
Fort Worth	4438	47	130.00

Table 7.8. Overall Average Bid Price of Cement Stabilized Backfill in Texas Districts in 1999.

District	*Quantity (CY)*	*Average Bid Price ($/CY)*	*Weighted Average Bid Price ($/CY)*
Abilene (ABL)	45.26	87.00	87.00
Atlanta (ATL)	215.70	95.27	66.70
	1,538.00	62.74	
Austin (AUS)	248.00	36.23	36.23
Beaumont (BMT)	752.56	40.60	25.92
	30213.00	25,56	
Brownwood (BWD)	78.20	28.00	28.00
Bryan (BRY)	3,018.00	66.20	66.71
	581.00	69.45	
Childress (CHS)	583.12	81.88	81.88
Corpus Christie (CRP)	47.12	85.00	32.00
	1293.5	30.10	
Dallas (DAL)	42.00	60.00	67.80
	11.40	60.00	
	861.00	68.28	
El Paso (ELP)	256.00	75.30	84.23
	5,724.00	84.63	
Fort Worth (FTW)	16.10	128.00	128.00
Houston (HOU)	31,052.73	27.94	26.12
	51,292.34	27.65	
	5,600.66	10.75	
	77,770.00	35.21	
Laredo (LRD)	1,086.84	58.10	56.85
	537.78	54.34	
Lubbock (LBB)	745.90	78.47	67.57
	1786.96	63.00	
Lufkin (LFK)	722.50	68.56	54.65
	892.6	43.40	
Odessa (ODA)	842.65	67.35	75.73
	1056.40	82.43	

Table 7.8. (Continued)

Paris (PAR)	298.20 718.50	132.6 87.00	109.7
Pharr (PHR)	1,935.00 3,767.40	62.54 37.67	46.00
San Angelo (SJT)	936.26 278.76	36.74 83.33	47.43
San Antonio (SAT)	1,272.53 234.90 512.00	71.53 67.20 68.00	70.13
Tyler (TYL)	48.40 100.00	120.00 141.00	134.00
Waco (WAC)	301.00 610.30	61.34 45.18	50.51
Wichita Falls (WFS)	51.00	79.50	79.50
Yoakum (YKM)	3,412.30 2886.00	39.73 33.87	36.65

Table 7.9. Different Price Category of Cement Stabilized Backfill in Different Parts of Texas.

Districts	Overall Unit Bid Price of Cement Stabilized Backfill ($/CY)	Price Level
AUS, BMT, BWD, CRP, HOU, YKM	Below $40 per CY	Low
PHR, SJT, WAC, LFK, LRD, ATL, BRY, DAL, LBB, SAT	Between $40 and $70 per CY	Medium
ABL, AML, CHS, ELP, FTW, ODA, PAR, TYL, WFS	Above $70 per CY	High

Table 7.10. Overall Average Bid Price of Flex Base.

Districts	Overall Average Bid Price ($/CY), Roadway Delivery	Overall Average Bid Price ($/CY), Complete in Place
Abilene (ABL)	-	20.53
Amarillo (AMA)	11.74	29.20
Atlanta (ATL)	-	16.67
Austin (AUS)	-	20.64
Beaumont (BMT)	-	35.51
Brownwood (BWD)	15.57[1]	16.59
Bryan (BRY)	-	27.88
Childress (CHS)	19.31	22.68
Corpus Christie (CRP)	12.41	31.96
Dallas (DAL)	-	33.30
El Paso (ELP)	-	20.51
Fort Worth (FTW)	24.60	24.87
Houston (HOU)	-	31.04
Laredo (LRD)	13.00[1]	14.51
Lubbock (LBB)	-	23.81
Lufkin (LFK)	7.00	32.24
Odessa (ODA)	6.27[1]	21.00
Paris (PAR)	-	26.60
Pharr (PHR)	12.51	29.96
San Angelo (SJT)	7.14	19.18
San Antonio (SAT)	43.47[1]	17.15
Tyler (TYL)	7.10[1]	31.23
Waco (WAC)	2.51	16.94
Wichita Falls (WFS)	28.351	25.54
Yoakum (YKM)	12.82	24.7

[1] The numbers reported represent only a single construction project

Table 7.11. Suitable Backfill Materials Selected for HDPE Pipe As-installed Cost Estimation.

Suitable Backfill Material	Price ($/CY)
Flowable Backfill (Type I Backfill as specified in revised specification), typical price in Texas region, source: Appendix B	70.00
Cement Stabilized Backfill (Type II Backfill as specified in revised specification), low price zone, source: Table 6.12	32.00
Cement Stabilized Backfill (Type II Backfill as specified in revised specification), medium price zone, source: Table 6.12	60.00
Flex Base, (typical price), source: Table 6.14	18.00
Granular Backfill Conforming Type III Backfill as specified in revised specification, source: Table 6.10	10.00
Granular Backfill Conforming Type III Backfill as specified in revised specification, source: Table 6.10	15.00

Table 7.12. As-installed Cost of HDPE and RC Pipe Estimated by Using 'PipePac' 2000 and Savings from HDPE.

HDPE					RCP				Savings from Using HDPE
Dia. (inch)	[1]Pipe Price($/ft)	Backfill Price($/CY)	Haulage+ Tipping($/CY)	As-installed Cost($/CY)	Installation Type	[2]Pipe Price($/ft)	Haulage+ Tipping($/CY)	As-installed Cost	
18''	6.30	10.00	4.50	11.54	Class C	11.75	4.50	12.66	8.8%
24''	9.71	12.00	4.50	17.20	Class C	17.50	4.50	17.18	8.8%
30''	15.97	12.00	4.50	25.13	Class C	24.75	4.50	25.13	5.6%
36''	19.43	18.00	5.50	34.80	Class C	35.25	5.50	34.80	9.0%
42''	30.45	15.00	5.50	50.60	Class C	50.50	5.50	50.60	7.4%
48''	36.70	15.00	5.50	61.50	Class C	62.25	5.50	61.50	8.8%

[1]Listed HDPE Pipe Price of Advanced Drainage Systems, Inc for Texas Area.
[2]Listed RCP Price(Freight On Board) of Hanson Concrete Products, Dallas, Texas.

Table 7.13. Estimated As-installed Cost of HDPE and RCP.

Pipe Product	Resource Price as Parameter	As-installed Cost ($/LF)					
		18in	24in	30in	36in	42in	48in
HDPE	When Flowable Fill @ $85/CY is Used	32.68	43.4	58.8	70.5	88.2	103.6
	When Cement Stabilized Backfill @$32/CY is Used	23.9	33.2	46.7	58.6	75.5	88.2
	When Cement Stabilized Backfill @$60/CY is Used	33.7	46.6	65.0	82.4	102.3	118.3
	When Flex Base @$18/CY is Used	18.9	26.5	37.5	46.8	62.0	73.1
	When Type III Granular Backfill @$10/CY is Used	16.1	22.6	32.2	40.0	54.4	64.5
	When Type III Granular Backfill @$15/CY is Used	17.9	25.0	35.5	44.2	59.2	69.9
RCP	Minimum RCP Price, Zone 1, CSR Hydro Conduit, 15% Discount, is Used	17.9	22.7	32.6	42.4	55.3	66.3
	Listed Minimum RCP Price, Zone 1 of CSR Hydro-conduit without Discount, is Used	19.5	25.0	35.9	47.2	62.0	74.6
	Minimum RCP Price(FOB), Hanson Concrete Products with 15% Discount, is Used	19.2	24.5	34.8	45.3	60.1	72.2
	Minimum RCP Price, Priced by Hanson Concrete Product(FOB), is Used.	20.9	27.0	38.5	50.6	67.7	81.5
	Listed Maximum RCP Price, Supply to Remote City, Priced by Hanson Concrete Product, is Used	24.5	31.0	45.8	61.8	78.7	98.0
	Discounted RCP Price, Col. 3 (Project-site within 75 miles) Hanson Concrete, is used	20.0	25.7	37.0	48.7	63.3	76.4
	Discounted RCP Price, Col. 4 (Project-site within 110 miles), Hanson Concrete, is used	20.26	25.9	37.4	49.3	63.9	77.26
	Discounted RCP Price, Col 6 (Project-site within 130 miles), Hanson Concrete, is used	20.9	26.6	38.5	51.0	65.6	79.8

Table 7.14. Percent Estimated Savings from Using HDPE.

Type of Backfill Used and Its Unit Price ($/CY)	% Estimated Savings for Using HDPE instead of Using RCP[1]					
	18''	24''	30''	36''	42''	48''
When Type III Granular Backfill @$10/CY is Used for HDPE Project						
Savings Relative to RCP As-installed Cost with Discounted (15%) Minimum RCP Price, CSR Hydro Conduit was Used	10%	-	-	5.7%	-	2.7%
Savings Relative to RCP As-installed Cost with Discounted (15%) Minimum RCP Price(Freight On Board), Hanson Concrete was Used	16%	7.7%	7.5%	11.7%	9.3%	10.7%
Savings Relative to RCP As-installed Cost with Discounted (15%) RCP Price, to Supply 70 Miles away from Plant , Hanson Concrete, was Used	19.5%	12%	13%	17.8%	14%	15.6%
Savings Relative to RCP As-installed Cost with Discounted (15%) RCP Price, to Supply 100 Miles away from Plant, Hanson Concrete, was Used	20.5%	12.7%	13.9%	18.9%	14.7%	16.5%
Savings Relative to RCP As-installed Cost with Discounted (15%) RCP Price, to Supply 130 Miles away from Plant, Hanson Concrete, was Used	23%	15%	16.4%	21.6%	17%	19.2%
Savings Relative to RCP As-installed Cost with Minimum RCP Price, (without discount) CSR Hydro Conduit was Used	17.4%	9.6%	10.3%	15.3%	12.2%	13.5%
When Type III Granular Backfill @$15/CY is Used for HDPE Project						
Savings Relative to RCP As-installed Cost with Discounted (15%) RCP Price, to Supply 70 Miles away from Plant, Hanson Concrete, was Used	10.5%	2.7%	4.0%	9.2%	6.5%	8.5%
Savings Relative to RCP As-installed Cost with Discounted (15%) RCP Price, to Supply 100 Miles away from Plant, Hanson Concrete, was Used	11.6%	3.5%	5.1%	10.3%	7.3%	9.5%

Table 7.14 (Continued)

Savings Relative to RCP As-installed Cost with Discounted (15%) RCP Price, to Supply 130 Miles away from Plant, Hanson Concrete, was Used	14.4%	6%	7.8%	13.3%	9.7%	12.4%
Savings Relative to RCP As-installed Cost with Minimum RCP Price, (without discount) CSR Hydro Conduit was Used	8.2%	-	-	6.3%	4.5%	6.3%
When Flex Base @$18/CY is Used for HDPE Project						
Savings Relative to RCP As-installed Cost with Discounted (15%) RCP Price, to Supply 100 Miles away from Plant, Hanson Concrete, was Used	6.7%	-	-	5%	3%	5.3%
Savings Relative to RCP As-installed Cost with Discounted (15%) RCP Price, to Supply 130 Miles away from Plant, Hanson Concrete, was Used	10%	-	2.6%	8.2%	5.5%	8.4%

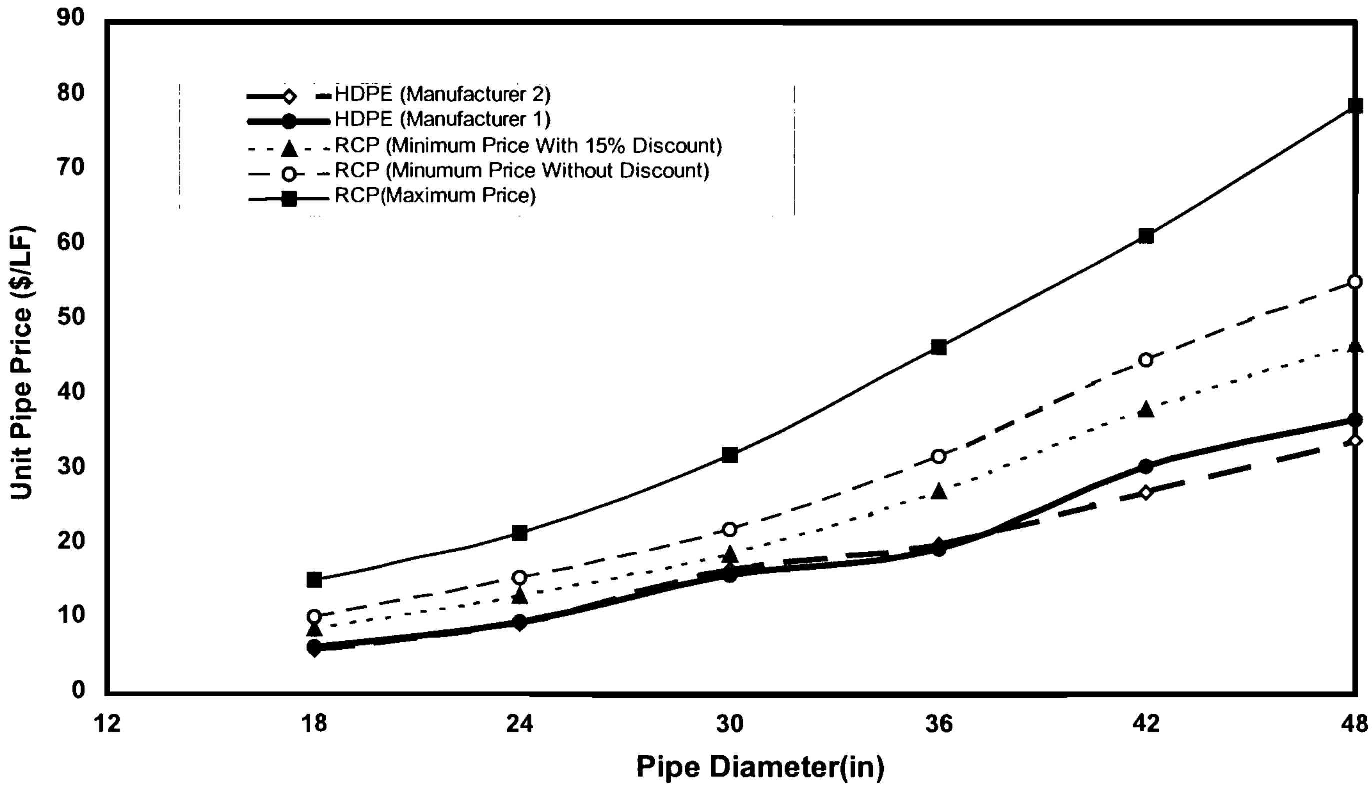


Figure 7.1. Unit Pipe Price Comparison in Texas: HDPE versus RC Pipe.

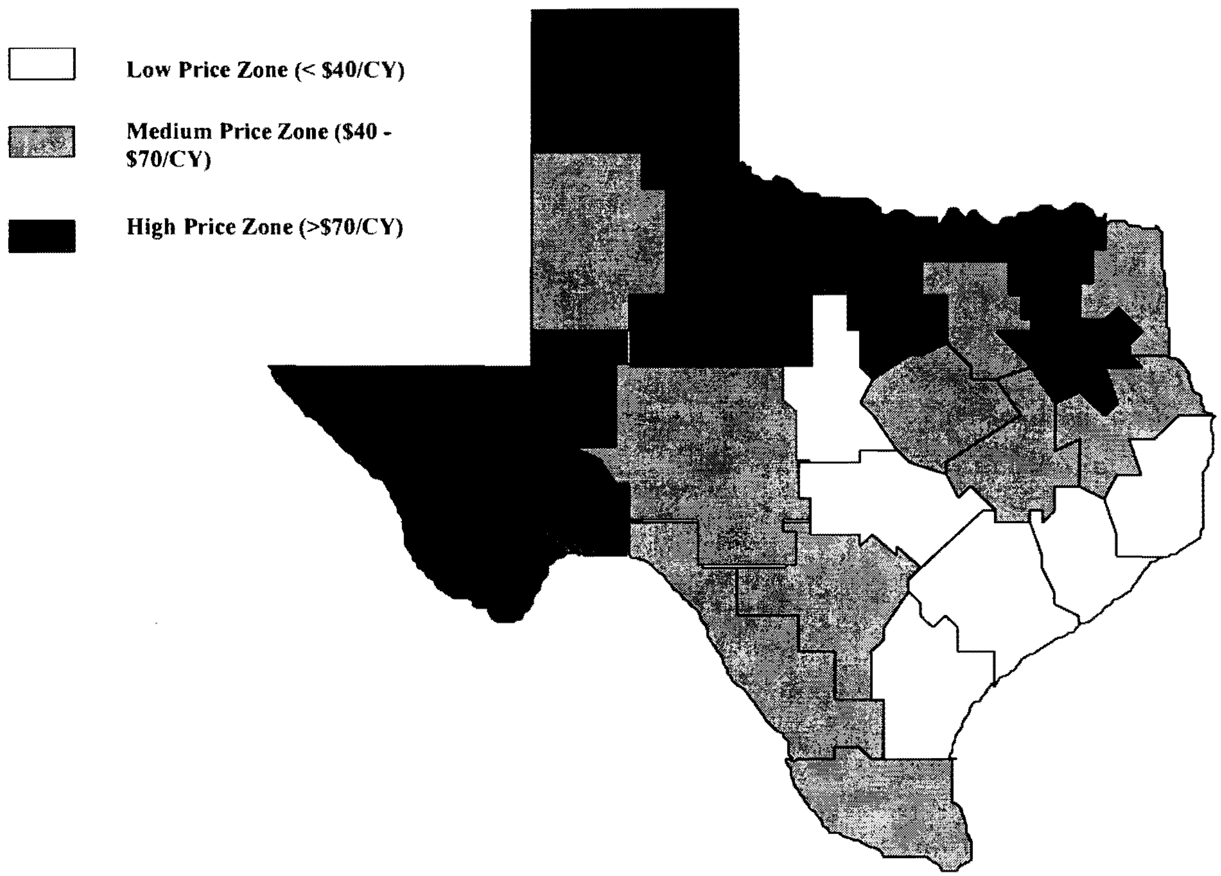


Figure 7.2. Price Zones of Cement Stabilized Backfill in Texas.

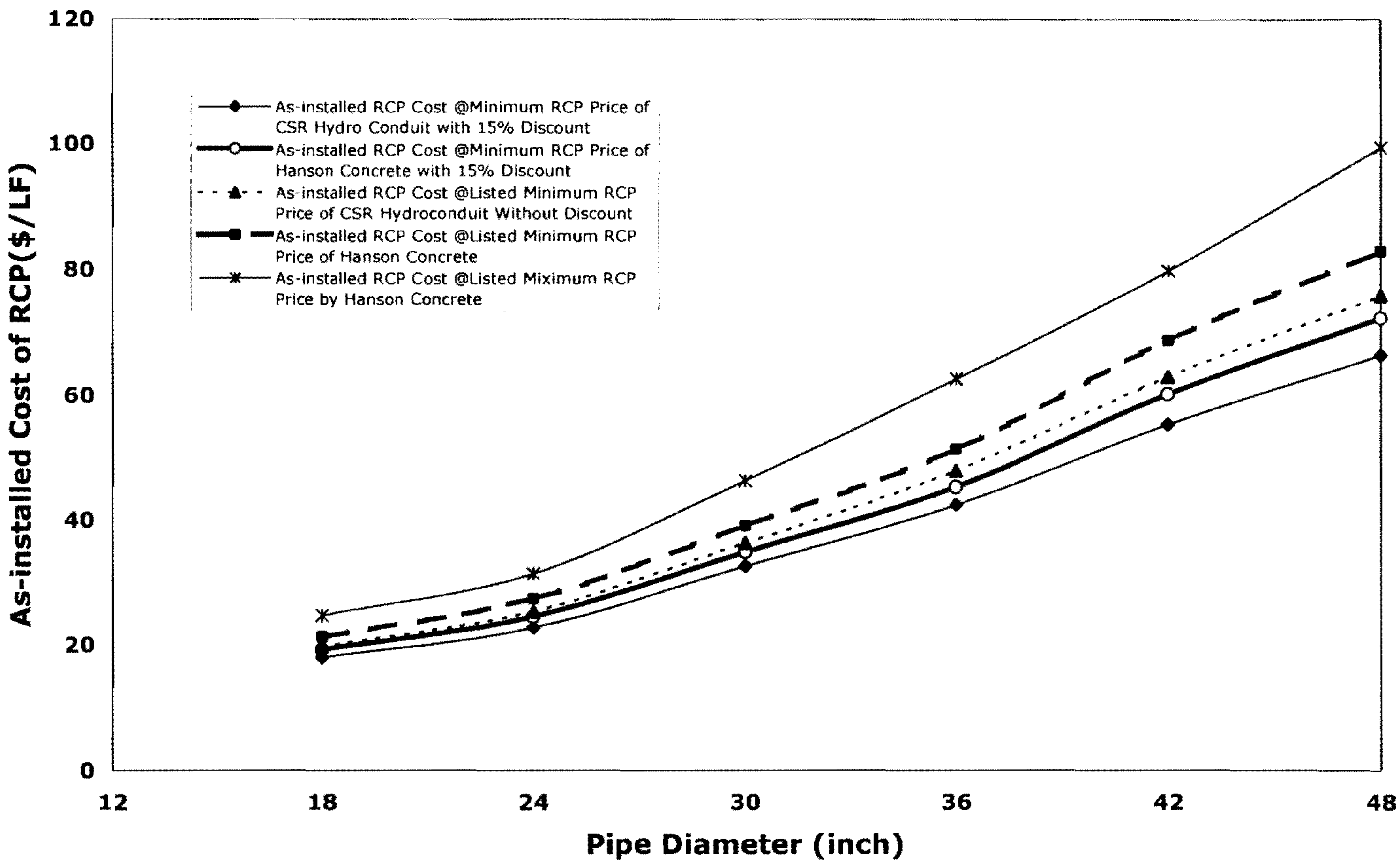


Figure 7.3. As-installed Cost of RCP at Varying Pipe Price Conditions.

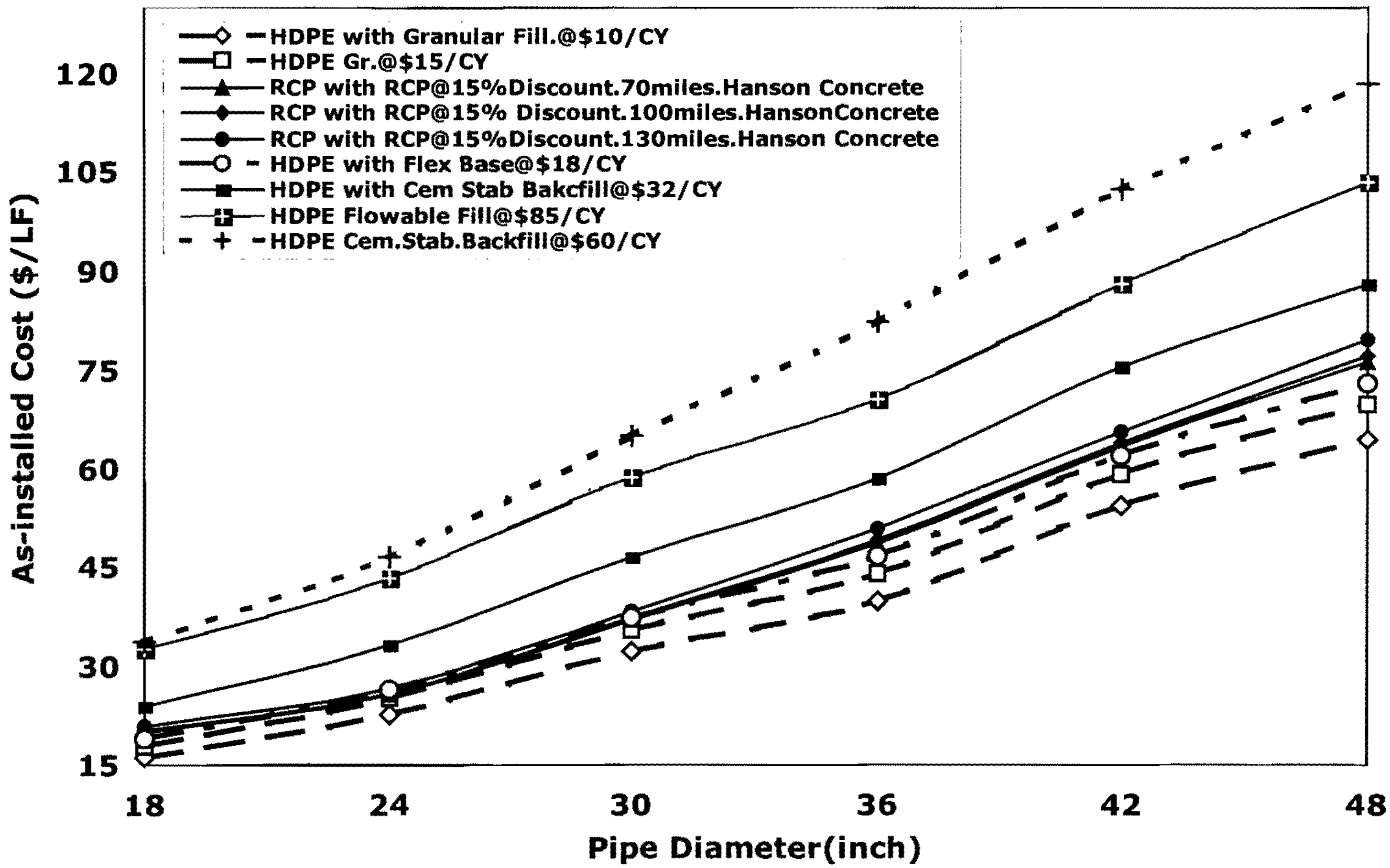


Figure 7.4. Estimated As-installed Cost of HDPE versus As-installed Cost of RCP

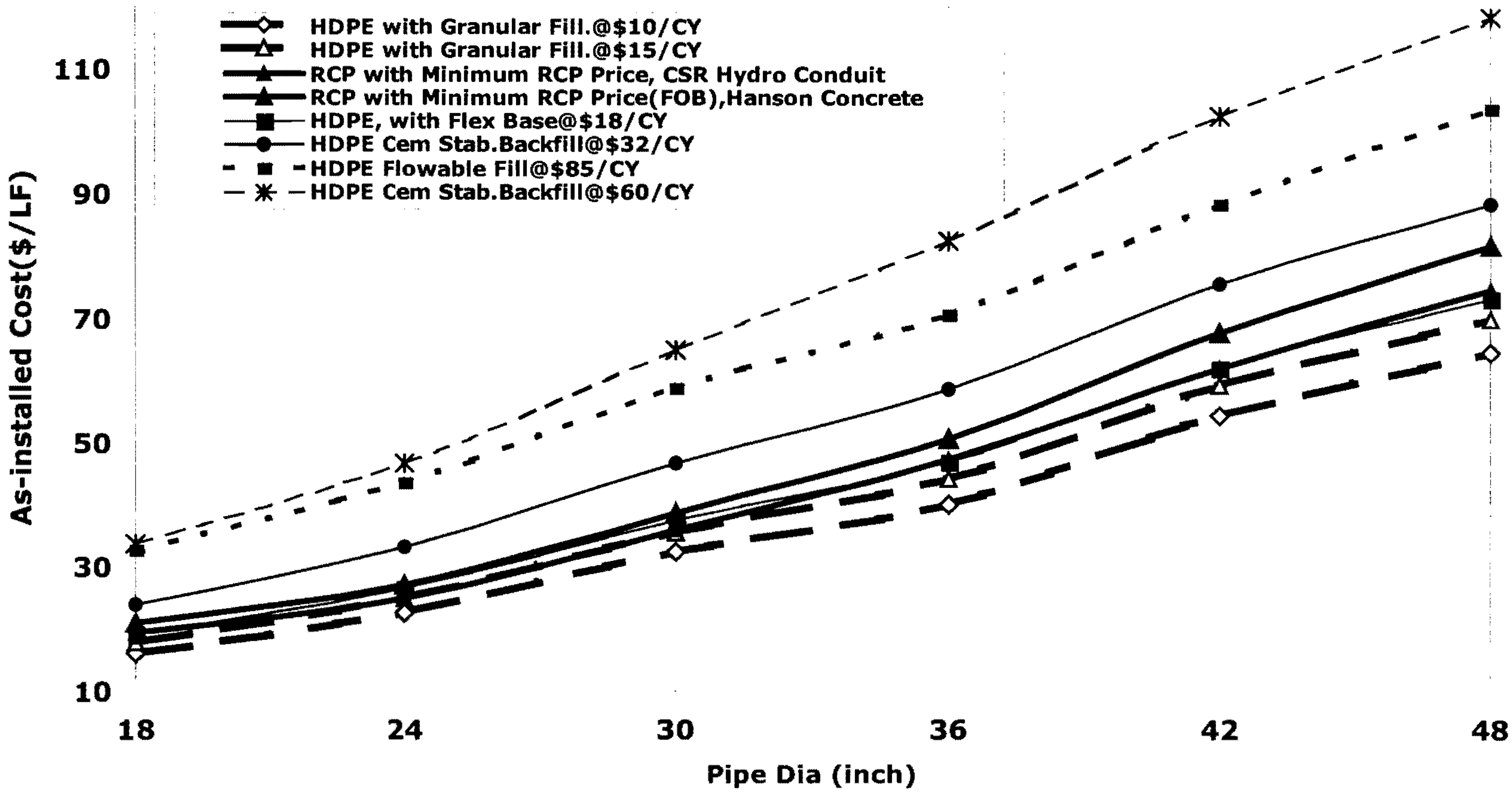


Figure 7.5. Estimated As-installed Cost of HDPE versus As-installed Cost of RCP

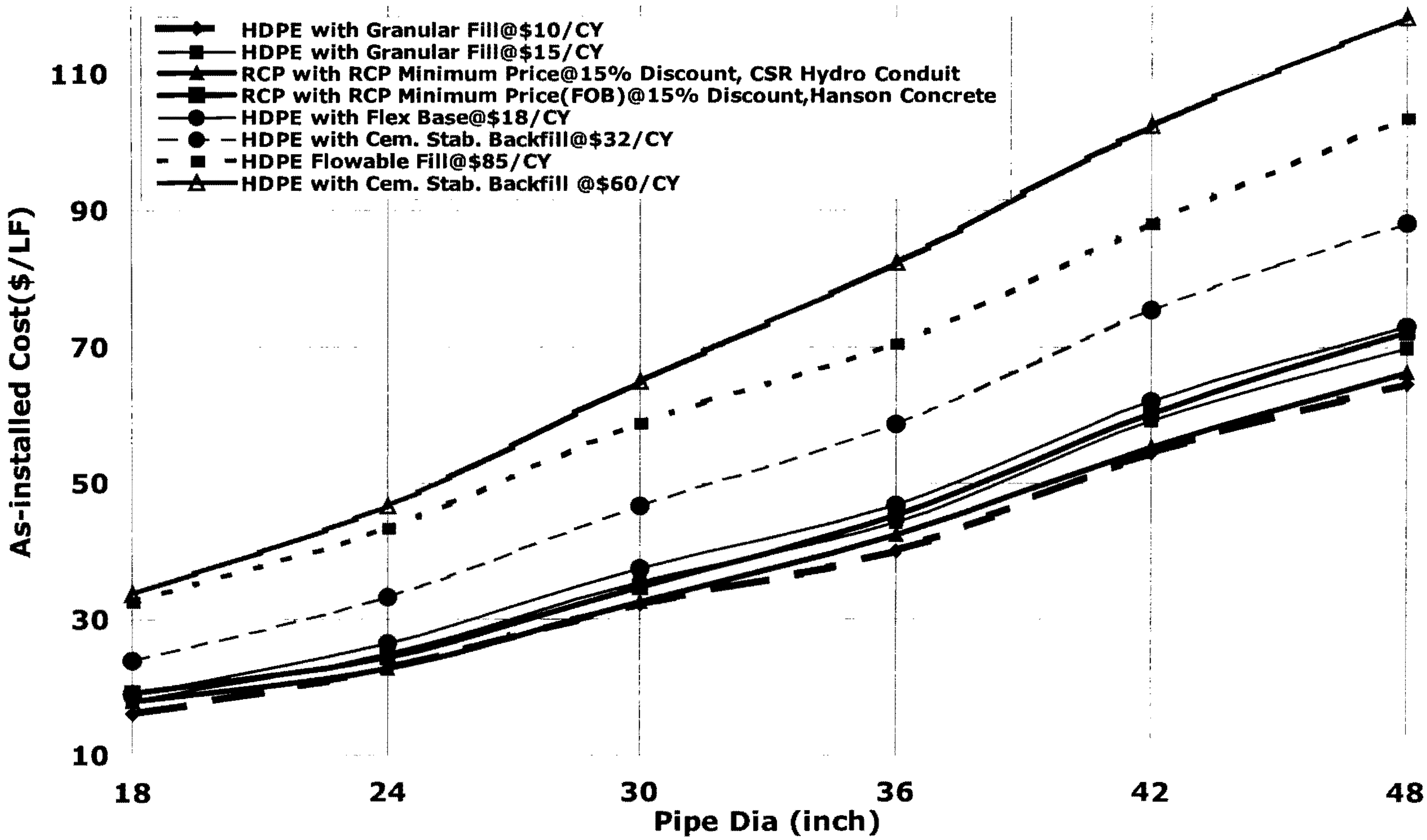


Figure 7.6. Estimated As-installed Cost of HDPE versus As-installed Cost of RCP

CHAPTER VIII
ANALYSIS OF TEST DATA

8.1. Introduction

The results from the field analysis were used to derive the physical properties of the backfill using "CANDE," a computer software that can be used to design and analyze culverts and other soil structure interaction systems. CANDE has been written in ANSI Fortran, and operates in a batch mode environment and can be run on mainframes, minicomputers, and microcomputers (*51*). Its name is derived from the acronym "Culvert Analysis and Design." CANDE was introduced in 1976, and has been widely distributed by the Federal Highway Administration (FHWA). It is used by state highway departments, domestic and military Federal agencies, research laboratories, consulting firms, industry, and universities in the United States, Canada and Europe. Users have favorably assessed CANDE and the FHWA has funded additional developments and enhancements to it since its inception. Buried structures made of reinforced concrete, corrugated metal and structural plastic can be designed and analyzed to withstand soil loading, temporary construction loads, and loads induced by traffic. CANDE uses boundary values to generate solutions based on the assumption of plane strain conditions.

CANDE has three solution levels that allow the user to choose the rigorousness of the desired solution. Level 1, which is the simplest to use, utilizes a closed form solution of an analysis of a cylindrical conduit buried in an elastic half space, while Levels 2 and 3 solutions are based on a two-dimensional finite element mesh. Level 2 has a completely automated mesh generation routine that is adequate for most of the common culvert configurations. Generation of circular, elliptical, rectangular, and arch geometries are possible, and the user need not have any specialized knowledge of the Finite Element Method to analyze/ design installations with these geometries. A limitation of Level 2 is the assumption that the culvert is symmetrical about the centerline; however this assumption would be valid for most culvert installations. For the analysis of asymmetrical and arbitrary shapes, a Level 3 solution is required, where the user creates the finite element mesh manually. However, the Level 3 option is more time consuming and requires that the user have knowledge of the Finite Element Method.

In CANDE, the culvert wall is modeled by using beam-column elements with nonlinear material properties that allow for elasto-plastic yielding of corrugated metal and cracking and crushing of concrete. Several different models are available to model the soil, including the newest hyperbolic models. Interface elements are also available to allow for the separation, bonding, and friction of the pipe wall and the backfill, and the backfill and the trench wall.

Traditional methods of designing culverts depend heavily on empirical methods. The disadvantage of these methods is that the designer is constrained to a limited database. CANDE does not suffer from such limitations, and almost all configurations can be assessed. In a study conducted by Federal and State agencies, CANDE was found to be the most versatile, and rigorously correct program available for buried culverts (*52*).

The input to CANDE consists of following parameters:

1. The culvert size and shape (circular, elliptical, arch, etc.),
2. The culvert wall section properties per unit length (area and moment of inertia),

3. Culvert wall material properties,
4. The soil geometry, and material properties for various zones (e.g., in situ, backfill, bedding, etc.),
5. Loading, including the gravity weight of the soil, and live load pressures due to construction equipment and vehicular traffic.

Standard sizes and shapes can be used in CANDE analysis and design or, if needed, the size and shape can be customized. The culvert wall section properties can be specified, and can be varied around the circumference if required. A library of culvert wall material properties is available for corrugated aluminum, corrugated steel, reinforced concrete, and structural plastic. In the analysis carried out for this research, the material properties were given by the user, as the use of the material properties given in the library for "plastic" material were limited to smooth, non corrugated pipe walls only.

Many models are available to represent the physical behavior of soil, and these models include the following:

1. Isotropic linear elastic,
2. Orthotropic linear elastic,
3. Overburden dependent,
4. Hardin (linear elastic parameters, but varied according to the mean stress and strain),
5. Duncan (tangent Young's modulus formulation),
6. Selig (hyperbolic bulk modulus formulation).

The last three models listed above are nonlinear, and the solution is generated in an iterative manner. As recommended in the user manual for CANDE, the in situ soil was assumed to behave in an isotropic linear elastic manner. This choice could be made because of the lack of significant overburden pressure on the in situ soil. However, for the backfill material, a model where (a) the elastic modulus of the material increases with increasing confining stress and decreases with increasing shear stress, and (b) the bulk modulus of the material increases with confining stress was chosen, as the loading on the top of the backfill produces high vertical stresses. This model, the Duncan formulation, is particularly suited for the analysis of flexible pipe, as large horizontal deflections of the pipe generates significant lateral pressures in the backfill.

The Duncan model employs the following equation to calculate the Young's Modulus of the soil.

$$E_t = \left(1 - \frac{R_f(1-\sin\phi)(\sigma_1 - \sigma_3)}{2C\cos\phi + 2\sigma_3\sin\phi}\right)^2 KP_a(\sigma_3/P_a)^n \tag{8.1}$$

where

E_t = Tangential Young's Modulus,

R_f = Ratio between the deviator stress at failure in a triaxial test and the deviator stress at failure as predicted by the hyperbolic model (see Figure 8.1),

$\phi = \phi_o - \Delta\phi \log_{10}(\sigma_3/P_a)$ = Angle of internal friction (see Figure 8.2),

C = Soil Cohesion,

K, n = Non dimensional parameters, found by fitting triaxial or full scale test data,

P_a = Atmospheric pressure.

In the Duncan Model the Bulk Modulus is calculated for a given stress state in the following manner.

$$B = K_b\, P_a\, (\sigma_3 / P_a)^m \qquad (8.2)$$

where

B = Bulk Modulus,

K_b, m = Non-dimensional parameters found by fitting triaxial or full scale test data.

8.2 The Back Calculation Procedure

A typical finite element mesh used in the CANDE analysis is shown in Figure 8.3 (*51*). As shown in the figure, there are four distinct material zones. The zones are: (a) the in-situ soil, (b) the bedding, (c) the backfill, and (d) the pipe material.

The material and section properties of the pipe were obtained from the pipe manufacturers. Hence, the physical properties of the backfill materials and in-situ soil had to be back calculated. As a starting point, the properties suggested in the CANDE instruction manual were used. The manual includes recommended values for the parameters used in the Duncan Formulation (K, n, m, K_b, etc.) in CANDE for granular material (GW, GP, SW, and SP according to the Unified Classification) compacted to different levels as given in Table 8.1. These parameters have been derived by incorporating data from both field tests and triaxial tests. For coarse aggregates, three levels of compaction are considered, corresponding to compaction to 105%, 95%, and 90% standard Proctor. The three materials are named CA105, CA95, and CA90 respectively in the CANDE users' guide.

From the diametrical measurements taken while loading the buried pipe sections, the deflection at the Northern, Central, and Southern cross sections were determined for a load level of 120 kips. Both the vertical and horizontal deflections were computed and used in the analysis. The imposed load was converted into force per unit length of pipe by dividing the 120 kips by the length that the load acts upon, that is, the length of the pipe section tested (5 ft). The calculated force per unit length was then distributed over the uppermost nodes of the mesh, as shown in Figure 8.3, in such a manner that the vertical deflection at each node was equal. This procedure was adopted because the loading plate was sufficiently rigid to ensure that the loads would not induce the bottom of the plate to develop curvature.

First, a preliminary analysis was carried out primarily to determine the Young's Modulus of the in situ soil. As mentioned previously, the in situ material was presumed to have a constant Young's Modulus. The average (over the North, Center, and South cross sections) deflection for each pipe section tested was utilized in this preliminary analysis. This procedure had to be carried out in an iterative manner. First, a trial value for the Young's Modulus of the in situ soil had to be selected. Then trial values for the strength parameters for the Duncan Model needed to be selected. The bedding and the backfill was assigned the same material properties as the same compaction effort and compaction equipment were used in the compaction process. Then these trial values were varied until the best match between the horizontal and vertical deflection could be found. Trial values were selected by interpolating the parameters from between "CA105" and "CA95". For instance, let us assume that the deflections calculated with the properties of "CA105" assigned to the backfill were lower than the measured values. Also, let us assume that the deflections calculated with the properties of "CA95" assigned to the backfill were higher than the measured. Then, the parameters midway between "CA105" and "CA95" would be used in the next analysis. The balancing between "CA105" and "CA95" was accomplished by the using an interpolating ratio. The manner in which the interpolating ratio was utilized can be defined in the following manner. Suppose that some parameter has been assigned values of X_1 and X_2 in Table 8.1 for backfill types "CA105" and "CA 95" respectively. Then, the value of the parameter used in the analysis, X_{new}, can be given in terms of the interpolating ratio as follows:

$$X_{new} = X_1 (r) + X_2 (1-r) \tag{8.3}$$

An example of the parameters used where the interpolating ratio was set to 0.5 (midway between "CA105" and "CA95") is given in Table 8.2. Subsequently, the stiffness of the in situ material and of the backfill were varied until the horizontal deflection and the vertical deflection both matched the values that were obtained during field testing. For instance, if the vertical deflection predicted by CANDE was higher than the actual deflection measured in the field, and the horizontal deflection was lower than measured in the field, the next simulation would be carried out with a lower stiffness for the in-situ soil and a higher stiffness for the backfill. When the stiffness of the backfill and the in-situ soil is changed, the vertical deflection at the nodes that the load was applied to change, and thus, the load distribution must be adjusted. This iterative procedure was used until a good match was obtained between the horizontal and vertical deflections calculated by CANDE with the field deflections.

As an example, the output from the CANDE analysis of one test is presented in Table 8.3. This analysis was carried out on a 48 in. diameter pipe, backfilled with loose Coarse Gravel, and installed in a trench 72 in. in width. The vertical and horizontal deflection calculated by the computer program for the strength of backfill and in-situ soil conditions assigned were 1.391 and 0.843, respectively. The average measured deflections from full scale testing were 1.420 and 0.850 for vertical and horizontal deflections, respectively. The modulus of the in-situ soil which yielded the above match between the horizontal and vertical deflections was 2650 psi, while the interpolating ratio between "CA105" and "CA95" was 0.15. As an example as to how the interpolating ratio was utilized, consider the manner in which the "k" parameter was calculated for this analysis. The values suggested in CANDE for k was 600 and 300 for "CA105" and "CA95", respectively (k is dimensionless). Since the interpolating ratio was 0.15, the modified k value was calculated by

$$k = (0.15 \times 600) + (0.85 \times 300) \tag{8.4}$$

$$k = 345.$$

The modulus of the in-situ material was estimated by considering the results from 5 full-scale tests. The results from this analysis is presented in Table 8.4. The average Young's modulus for the in-situ material obtained from these five tests was 2367 psi. Therefore the Young's modulus of the in-situ material was assigned a value of 2400 psi in all subsequent analyses.

Analysis of data from all 13 full-scale tests followed the same procedure described above. Deflection measurements at all North, Center, and South sections were analyzed separately. Since the in-situ soil's Young's modulus was now fixed at 2400 psi, and only the interpolating ratio between "CA105" and "CA95" was changed, an exact matching of both horizontal and vertical deflections could not be achieved always. In the cases where a good match could not be found, the error was distributed between the horizontal and vertical directions. In every case the deflections in the horizontal directions were always less than in the vertical deflections, the error was distributed to the ratio of the measured horizontal and vertical directions. For example, when matching the deflections at the Northern end of the first test, (Coarse Gravel with no compaction, 48 in. diameter pipe, 72 in. wide trench) it was found that the best match for the measured deflections of 1.397 in. and 1.033 in. was 1.576 in. and 0.903 in. The discrepancy between the vertical deflection measured in the field and predicted by CANDE, and the discrepancy between the horizontal deflection measured in the field and predicted by CANDE have a ratio close to the ratio of the vertical and horizontal deflection measurements taken in the field.

$$\frac{(1.576 - 1.397)/(1.033\text{-}0.903)}{(1.397/1.033)} = 1.01 \qquad (8.5)$$

All the deflections measured in the 13 full-scale tests were matched in the above manner. The following levels of accuracy were sought when the analysis was carried out. The deflection at each node where the loads were applied was matched to within 6% of the largest deflection. When matching the horizontal and vertical deflections however, it was noted in some cases that small alterations in the interpolating ratio between "CA105" and "CA95" caused large changes in the error ratio as calculated in Eq.(8.5) above. In these cases, the iterative procedure was stopped when it was found that, in two consecutive runs with the interpolating ratio within 0.02 of each other, the error ratio was higher than 1.0 in one case and lower than 1.0 in the other. In cases where the error ratio was not substantially affected by changes in the interpolating ratio between "CA105" and "CA95", the error ratio was kept within 1.0 ± 0.08.

The matches that were found for the 39 sections of the 13 tests are presented in Tables 8.5, 8.6, and 8.7. Table 8.6 contains data from tests where backfill was compacted with 4 passes of impact rammer. Table 8.7 has data from tests where backfill was compacted 2 passes of impact rammer, and Table 8.8 has data from tests where the backfill was not compacted.

In the next step of the research, an analysis was carried out to determine whether the Duncan parameters for the backfill vary according to a normal distribution. The mean and standard deviation of the back calculated "k" parameter was used in this analysis. A comparison between the spread of the back calculated k values and the spread expected if the parameters were to vary according to the normal distribution are shown in Figures 8.4, 8.5, and 8.6 for high, medium, and low compaction, respectively. The best fit for the variation of "k" is given by normal distributions with (a) for high compaction a mean of 790 and a standard deviation of 280, (b) for medium compaction a mean of 615 and a variance of 170, (c) for no compaction a mean of 365 and a standard deviation of 70. The three probability density curves are shown in Figure 8.7.

Thus, the mean and the standard deviation of the strength parameters are known. How this information can be used in a statistical method to design HDPE installations is discussed in the next section.

8.3 Analysis of Installation Configurations

8.3.1 Overview

The results obtained from the analyses presented in the previous sections of the report can be used to design buried HDPE pipe installations. As the strength of the backfill is not constant, and varies in a stochastic manner, a statistical analysis is required to determine the reliability of a given installation. In this section the statistical basis for the design of HDPE pipe installations will be discussed, followed by the analysis of selected installations to demonstrate how the suggested approach can be used.

8.3.2. Statistical Basis for Analysis

During field installation of pipes, there can be significant variability in the amount of compactive effort used by different contractors and in different pipe installations. Even within a given pipe installation some sections of backfill may receive higher compactive effort than others. Observations during pilot installation showed that some sections may even receive no compaction at all. Instead of focusing attention on minimizing variability in compactive efforts, this research investigated the amount of variability and incorporating such variability into the design method. In other words, variability in backfill conditions is built into the charts that were developed for prediction of pipe performance.

As a first step, the data from all the tests, i.e. tests with high, medium, or low compaction were combined. The pooled data was then used to examine the spread of the back calculated parameters. This procedure was decided upon as the compaction effort delivered to the backfill in a pipeline cannot be established in advance, and varies according to the conscientiousness of the contractors and the level of inspection carried out. However, it was assumed that backfill with gradation similar to Clayey Sand will receive a minimum level of compaction equivalent to 2 passes of an impact rammer. Therefore, data from the full scale load test where Clayey Sand was used as backfill without any compaction was not included. Similarly, data collected from tests with 4 passes of impact rammer was excluded because it was determined that that level of compaction is rarely used in actual field installation. For the pooled field data, the variance of "k" in the Duncan model was found to have an approximate normal distribution, with a mean of 590 and a standard deviation of 252. The probability density distribution of the parameter K in this analysis is shown in Figure 8.8. The cumulative probability distribution of the strength of the backfill is also of importance. Knowledge of the 5^{th} percentile of the strength of the backfill would facilitate establishing what the 95^{th} percentile of the deflections would be, in a manner which will be explained later in this chapter. The cumulative distribution of the parameter k for all the tests is shown in Figure 8.9, while the value of k and other parameters corresponding to a number of selected percentiles are given in Table 8.8.

The factors that influence the performance of a buried HDPE pipe installation are the physical properties of the pipe (such as wall cross sectional area, and moment of inertia), strength of the native soil, strength of the backfill, and expected loading due to soil fill and vehicular load. The physical properties of HDPE pipe can be obtained from pipe manufacturers, and can be predicted accurately, as pipe is manufactured in plants under good quality control. The strength of the native soil can be determined from a knowledge of the geology of the region; any variance of the strength used in the analysis will not affect the results to a great extent as the backfill that is placed and compacted around the pipe has a much greater effect on pipe deflections. Table 8.9 contains recommended values for the Young's Modulus of a variety of soils (*53*). The load on the pipe can be determined from the height of fill expected and/or the weight of heavy construction equipment traversing the trench during construction. Thus the strength of the compacted backfill has a great influence on the performance of the pipe, and is the only unknown factor. The analyses carried out in the preceding sections has determined the expected strength of backfill and the variance of the strength of the backfill for a selected range of backfill gradations. For any installation, the deflections in the pipeline depend on the strength of the backfill. The deflections in the vertical direction, ΔY, will vary as a function of the strength of the backfill.

$$\Delta Y = F \text{ (Backfill Strength)} \tag{8.6}$$

If, in the design of an installation, the strength of the backfill is taken to be the only variable, the 5^{th} percentile of the strength of the backfill would correspond to the 95^{th} percentile of the predicted deflection, the 1^{st} percentile of the strength of the backfill would correspond to the 99^{th} percentile in the predicted deflection and so on. The above analysis can be used to (a) given the installation configuration, determine the probability that a certain magnitude of deflection be exceeded (b) the magnitude of the dead and live loads that can be applied in an installation, to limit the largest deflection within the pipeline.

8.4. Pipe Performance under Maximum Fill Height Conditions

For pipes with significant overfill, the main source of loading comes from the weight of the soil above the pipe. Stresses produced by vehicular traffic loads are dissipated rapidly with depth and do not need to be taken into account. However, HDPE pipe can experience very high deflections due to the soil weight and soil settlement. Therefore, in the study of the effect of the fill height only the soil weight will be considered as the load acting over the pipe.

Table 8.12 and Table 8.13 present the model input parameters corresponding to two backfill material specifications. The first of these is based on a gradation band that would include some flexible base materials similar to the Clayey Sand used in load testing. The second excludes such Flex Base materials. These parameters were developed by pooling the corresponding field test data. The physical properties of the backfill materials were assigned to correspond to the 5^{th}, 10^{th}, 25^{th} and 50^{th} percentile of the strength of the backfill in order to predict the 95^{th}, 90^{th}, 75^{th} and 50^{th} percentile deflection, respectively.

In the preliminary analysis, a pipe with a diameter of 48 inches was used. The width of the trench was 90 inches. The Young's Modulus of the in-situ material was assumed to be 500 psi. The use of this value that corresponds to a medium stiff clay adds some conservatism to the analysis. Four levels of fill height were considered: 10, 20, 30 and 40 feet.

Figure 8.10 presents the predicted deflections by CANDE at different fill heights for the coarse granular material with medium or low compaction, or Flex Base material with medium compaction.

In Figure 8.11, where only the coarse granular material with medium or low compaction was used, the predicted deflections present a small decrease. This was expected since the incorporation of Flex Base type materials to the coarse granular material caused a small decrease in the strength parameters to be used in the analysis.

In the same manner, different curves can be generated to determine the maximum fill allowed for a specific design. For instance, if Figure 8.10 is used to determine the maximum fill height admissible when the deflection is limited to 5% with a 95% reliability (e.g., just 5% of the backfill strength is assumed to be used) the allowable maximum fill height is approximately 24 feet.

Actual deflections in the San Angelo District installation, where the fill height is 12 feet, agree with the deflections predicted by CANDE. The maximum deflection measured in San Angelo after 16 months of service was 1.8%. The deflection predicted by CANDE for a fill height of 12 feet and considering the mean value of the backfill strength corresponds to 1.7%. The use of the 50th percentile of the backfill strength can be considered appropriate since the compaction control observed in the installation was considered fair.

In the previous paragraphs, deflections were predicted for different fill heights using the deflection criteria as the performance limit. However, the criterion governing the maximum fills allowed for corrugated HDPE pipe is the thrust stress, and not the deflection, as demonstrated by Michael G. Katona (*53*). In this study, charts for maximum fill height were developed based on structural considerations such as pipe size, corrugation geometry, backfill soil quality and design life using the finite element program CANDE.

The strength of a corrugated pipe can be assessed by the hoop stiffness, which is controlled by the corrugated section area and by the flexural stiffness, which is proportional to the corrugation moment of inertia. However, in the analysis carried out by Katona, the hoop stress was almost always the controlling design criteria. Thrust stress is treated as the major design variable in conformance with AASHTO design criteria and specifications (*11*).

Four design criteria were considered for the study carried out by Katona: (1) thrust stress, (2) flexural strain, (3) relative deflection, and (4) buckling pressure. These criteria were applied to long and short-term design life. Nine pipe diameters ranging from 4 to 30 in. were used with three different corrugation areas for each pipe size. Moreover, two soil models were considered for the analysis, good and fair soil.

Combinations of the variables mentioned above were analyzed until each of the design criteria was exceeded. For the thrust stress criteria the allowable limit considered was half of the yield stress recommended by the AASHTO M294 specification (*9*). The results obtained by Katona showed consistently that the thrust stress governed the maximum fill height over the rest of the criteria.

As Katona did not consider large diameter pipes in his analysis, the researchers carried out an analysis where the thrust stress was considered a performance limit to determine the maximum fill height for corrugated HDPE pipe.

The analysis carried out considered coarse granular material with medium and low compaction including flex base with medium compaction. The backfill strength parameters

from Table 8.12 were analyzed at different fill heights and the thrust forces were obtained for each combination of fill height and backfill strength. Three levels of in-situ material stiffness were considered, that is, 500 psi, 1250 psi, and 2500 psi corresponding to weak, medium stiff, and stiff soil. The hoop stress was obtained by dividing the thrust force by the corresponding corrugated area and was then compared with the allowable tensile stress. The analysis was facilitated by computing a stress ratio, defined as the ratio between the computed thrust stress, and the allowable stress. AASHTO Section 18 specifies that the allowable stress in HDPE be half the tensile strength (900 psi for long term analysis). The variation of the stress ratio with fill height for the three in-situ soil strengths considered is shown in Figures 8.12, 8.13, and 8.14.
It can be observed that with 50 % reliability, the allowable fill heights are 14 ft, 20 ft, and 24 ft respectively.

8.5 Pipe Performance under Minimum Cover Conditions

Vehicular loads provide the most critical loading condition for any buried pipe with minimum soil cover. The most critical loading is likely to come from heavy construction equipment that may travel over the pipe that has just been installed but has been provided with only the minimum soil cover. Once the pavement structure is in place, it will distribute the loading from vehicles and therefore reduce potential for damage significantly. Therefore, loading from construction equipment that are applied before the construction of the pavement structure was considered to be the most critical.

The first step in the development of minimum cover specifications involved a comparison between the results from the full-scale testing and the wheel footprint structure against the predicted deflections obtained with CANDE. As mentioned before, CANDE is a computer program that assumes plane strain analysis and there is no consideration for the effect of discontinuous load areas such as the footprint of a tire. CANDE assumes an infinite load along the pipe. The load must be assigned as a point load and will act as a linear load (knife-edge load) above the cover of the pipe. Since the soil stress produced by a loaded area diminishes more rapidly than the one produced by a linear load, a reduction factor needs to be computed to incorporate the full-scale testing loads into the CANDE program. This reduction factor is given by the following equation.

$$2P/b_{CANDE} = r\,P/BL_{Full\text{-}scale} \tag{8.7}$$

where

P/b_{CANDE} = Load per unit length required for CANDE,
$P/BL_{Full\text{-}scale}$ = Load per contact area used in the full-scale testing,
r = reduction factor.

The reduction factor was determined by using Boussinesq's method *(53)*. This method can calculate stresses induced by external loads in an infinite elastic half-space. The solution to obtain the stress induced by a linear load is given by:

$$\sigma_z = \frac{2z_f^{\,3} P/b}{\pi(x_f^{\,2} + z_f^{\,2})^2} \tag{8.8}$$

where

σ_z = Vertical stress produced by the linear load,

P/b $= 2P/b_{CANDE}$ = Vertical load per unit length along the line,
z_f = Depth from bottom of loaded line to the point,
x_f = Horizontal distance from the load to the point.

In the same manner, the vertical stress induced by an area load can be obtained by the following equation.

$$\sigma_z = q\left[1-\left(\frac{1}{1+\left(\frac{B}{2z_f}\right)^{1.38+0.62B/L}}\right)^{2.60-0.84B/L}\right] \tag{8.9}$$

where

σ_z = Vertical stress beneath the center of a loaded area,
$q = P/BL_{Full\text{-}scale}$ = Bearing pressure,
B = Width or diameter of loaded area,
L = Length of loaded area,
z_f = Depth from bottom of loaded area to point.

By substituting the corresponding values in Equations 8.8 and 8.9 for an area of 2 feet by 2 feet (tire footprint loading dimensions) and a depth of 1 foot, the linear load required for CANDE can be obtained by

$$P/b_{CANDE} = 0.5535\, P_{Full\text{-}scale} \tag{8.10}$$

Once the reduction factor was determined, the actual loads used in the full-scale testing with the wheel footprint structure were converted to the corresponding linear load and the analysis with CANDE was carried out for 7 full-scale tests separately. For each single analysis, the physical properties of the corresponding backfill material for the specific test were used. The results are presented from Figure 8.15 to Figure 8.21.

As can be seen in the figures, the predicted deflections by CANDE are very close to the actual average deflections. Subsequently, the same approach was used to predict deflections for different axle loads at different levels of reliability.

The contact area and load applied to the roadway must be known to determine the minimum soil cover. Due to the wide range of construction equipment and truck configurations that exist for load and contact area provided by the tires, it was assumed that the square contact area was proportional to the load transmitted by the truck. A tire pressure of 100 psi was assumed for all trucks based on a manufacturer's catalog (*43*). Since the tire air pressure is approximately equal to contact area pressure between the road and the tire, the contact area can be computed by dividing the tire load by the tire air pressure. Also, the static load provided by the tire was multiplied by a dynamic factor, as proposed by AASHTO *(55)*, depending on the depth of the cover to take into account the movement of the vehicle over the pipe.

Subsequently, different axle loads were chosen to represent different truck configurations. The surface contact area and dynamic load were computed for each load as mentioned in the previous paragraph. The corresponding vertical stress due to the dynamic load for a specific cover was calculated based on Equation 8.9. Then, by using Equation 8.8, the linear load corresponding to the stress obtained by Equation 8.9 was back calculated. Finally, to take into account the use of half mesh in CANDE the linear load was divided by two. Table 8.14 shows the linear load required by CANDE for different axle loads and soil covers assuming a tire pressure of 100 psi.

With the linear loads for CANDE corresponding to the different axle loads listed in Table 8.14, an analysis was carried out for three different minimum covers: 1, 2 and 3 feet. A 48-inch pipe diameter was considered. The trench width selected was 90 inches. The Young's modulus of the in-situ soil was 500 psi. Curves for different levels of reliability for each cover were obtained. For this analysis, the strength parameters for the coarse granular soil with medium and low compaction including Flex Base with medium compaction were considered as the backfill material.

The results for 1, 2 and 3 feet cover are shown in Figure 8.22, Figure 8.23 and Figure 8.24, respectively.

The results obtained in the graphs can be used in the following ways:

1. The deflection produced by a known axle load with a specific cover can be predicted for different backfill strengths, and
2. The maximum axle load allowed over a pipe can be established for a specific deflection limit and cover.

For instance, if the deflection limit for a specific installation is 5% for a 48-inch pipe with a minimum cover of 1 foot, the maximum axle load allowed when the 5th percentile (95% reliability) of the backfill strength is used is approximately 45 kips (from Figure 8.19).

8.6. Pipe Performance under Repeated Wheel Loading Conditions

In the analysis described on the previous section, the deflections predicted by CANDE correspond to a single load application. In actual field installations the pipe may experience the traffic of heavy construction equipment several times before the pavement structure is placed. As part of the full-scale testing with the wheel footprint structure, 6 tests were subjected to repeated loading. The procedure is described in the following paragraphs.

The repeated loading phase was carried out after the single load application of 500 psi as described in the full-scale testing section. The wheel footprint structure was raised above the ground level and then lowered again to apply a load of 500 psi (approximately 40 kips) up to 15 times. Diameter readings were taken at every 3rd, 6th, 9th, 12th and 15th load cycle. The deflections obtained for the different tests are presented in Figure 8.25.

The initial deflection reading is the one obtained at the first 500-psi reading before the repeated loading and the final deflection is the one obtained at the last load cycle. Some observations can be made from the results: (1) the pipes with larger diameter experienced a higher percent in average of change in the deflection, (2) the gravelly sand experienced the highest deflection of all, and (3) the lowest change in deflection was obtain by the medium gravel when the 36-inch pipe was used.

However, comparing the change in deflection between consecutive readings when the repeated load is applied it seems to decrease as the number of repetitions increase as shown in Figure 8.25.

Table 8.1. Parameters for the Duncan Model Recommended in CANDE for Granular Materials with Varying Degrees of Compaction.

Backfill Type	γ_m	C	ϕ_o	$\Delta\phi$	K	n	R_f	K_b	m
	kip/ft^3	kip/ft^2	deg	deg					
CA105	0.15	0	42	9	600	0.4	0.7	175	0.2
CA95	0.14	0	36	5	300	0.4	0.7	75	0.2
CA90	0.135	0	33	3	200	0.4	0.7	50	0.2

Table 8.2. Parameters when the Strength of the Backfill is Halfway Between "CA105" and "CA95."

Backfill Type	γ_m	C	ϕ_o	$\Delta\phi$	K	n	R_f	K_b	m
	kip/ft^3	kip/ft^2	deg	deg					
Midway of CA105 and CA95	0.145	0	39	7	450	0.4	0.7	125	0.2

Table 8.3 Example Output from an Analysis Carried out Using CANDE.

Measurement Type	Deflection at Nodes Where Load is Applied (in.)			Deflection of Culvert (in.)		
	(1)	(2)	(3)	Top (Δy)	Spring Line (Δx)	Bottom (Δy)
Before Loading	0.085	0.079	0.058	0.253	0.071	0.071
After Loading	1.851	1.858	1.744	1.990	0.493	0.275
Deflection Due to Loading	-1.766	-1.779	-1.685	1.737	0.422	0.204

Table 8.4. Summary of Analysis Carried out to Determine the Modulus of the Insitu Material.

Test No.	1	5	8	10	12
Backfill Type	Coarse Gravel	Coarse Gravel	Coarse Gravel	Gravelly Sand	Clayey Sand
Trench Width	72	102	102	102	102
Pipe Diameter	48	36	48	48	36
Compaction	None	Medium	Medium	Medium	Medium
Measured Vertical Deflection	1.391	0.681	0.964	1.033	1.272
Calculated Vertical Deflection	1.420	0.662	0.976	1.055	1.326
Measured Horizontal Deflection	0.843	0.349	0.500	0.617	0.797
Calculated Horizontal Deflection	0.850	0.360	0.484	0.616	0.803
Insitu Modulus	2650	3000	2500	1200	2485
Interpolating Ratio	0.15	1.05	1.00	1.18	0.10
K	345	615	600	654	330

Table 8.5 Results from CANDE Analysis of Field Tests with Backfill Subject to High Compaction

Test No	Backfill Material	Pipe Diameter (in.)	Trench Width (in.)	Section	Interpolating Ratio	K
3	Coarse Gravel	36	102	North	1.20	660
3	Coarse Gravel	36	102	Center	1.63	789
3	Coarse Gravel	36	102	South	0.73	519
4	Gravelly Sand	36	102	North	3.00	1200
4	Gravelly Sand	36	102	Center	3.00	1200
4	Gravelly Sand	36	102	South	1.00	600
11	Clayey Sand	36	102	North	0.51	453
11	Clayey Sand	36	102	Center	1.42	726
11	Clayey Sand	36	102	South	2.30	990

Table 8.6. Results from CANDE Analysis of Field Tests with Backfill Subject to Medium Compaction.

Test No	Backfill Material	Pipe Diameter (in.)	Trench Width (in.)	Section	Interpolating Ratio	K
5	Coarse Gravel	36	102	North	1.10	630
5	Coarse Gravel	36	102	Center	1.42	726
5	Coarse Gravel	36	102	South	1.15	645
6	Medium Gravel	36	102	North	1.83	849
6	Medium Gravel	36	102	Center	1.83	849
6	Medium Gravel	36	102	South	1.28	684
7	Gravelly Sand	36	102	North	0.73	519
7	Gravelly Sand	36	102	Center	1.08	624
7	Gravelly Sand	36	102	South	1.28	684
8	Coarse Gravel	48	102	North	0.77	531
8	Coarse Gravel	48	102	Center	1.08	624
8	Coarse Gravel	48	102	South	1.23	669
9	Medium Gravel	48	102	North	2.37	1011
9	Medium Gravel	48	102	Center	1.15	645
9	Medium Gravel	48	102	South	1.05	615
10	Gravelly Sand	48	102	North	0.8	540
10	Gravelly Sand	48	102	Center	0.58	474
10	Gravelly Sand	48	102	South	0.86	558
11	Clayey Sand	36	102	North	0.55	465
11	Clayey Sand	36	102	Center	-0.05	285
11	Clayey Sand	36	102	South	-0.05	285

Table 8.7. Results from CANDE Analysis of Field Tests with Low Compaction.

Test No.	Backfill Material	Pipe Diameter	Trench Width	Section	Interpolating Ratio	K
1	Coarse Gravel	48	72	North	0.1	330
1	Coarse Gravel	48	72	Center	0.33	399
1	Coarse Gravel	48	72	South	0.38	414
2	Medium Gravel	48	72	North	0.45	435
2	Medium Gravel	48	72	Center	-0.19	243
2	Medium Gravel	48	72	South	0.27	381
13	Clayey Sand	36	72	North	-1.64	36
13	Clayey Sand	36	72	Center	-1.75	25
13	Clayey Sand	36	72	South	-1.58	42

Table 8.8. The Backfill Strength Corresponding to Selected Percentiles.

Percentile %	Interp. Ratio	γ_m kip/ft^3	C kip/ft^2	ϕ_o deg	$\Delta\phi$ deg	K	n	R_f	K_b	m
2.5	-0.67	0.133	0.0	32.0	2.3	100	0.4	0.7	8	0.2
5.0	-0.40	0.136	0.0	33.6	3.4	179	0.4	0.7	35	0.2
10	-0.10	0.139	0.0	35.4	4.6	270	0.4	0.7	65	0.2
50	0.98	0.150	0.0	41.9	8.9	593	0.4	0.7	173	0.2

Table 8.9. Recommended Values for Young's Modulus of Insitu Soil (*51*).

In-situ Material	Young's Modulus (psi)
Soft Clay	35 – 210
Medium Clay	70 – 485
Stiff Clay	175 – 2800
Loose Sand	1390 – 3475
Medium Dense Sand	2800 – 8300
Dense Sand	6900 – 14000
Sandstone	$9.7\times10^{5} - 2.8\times10^{6}$
Granite	$3.5\times10^{6} - 6.9\times10^{6}$

Table 8.10. Maximum Deflections Predicted by CANDE for Two HDPE Pipe Installation Configurations with Shallow Cover.

Load (kips)	Width of Wheel (ft)	Cover (ft)	Deflection (%)
18	2.0	1.0	2.22
18	2.0	1.5	2.10
18	2.0	2.0	1.96
32	1.5	1.0	6.13
32	1.5	1.5	5.65
32	1.5	2.0	4.89

Table 8.11. Maximum Deflections Predicted by CANDE for a 48 in. Diameter Pipe Buried in Deep Fill.

Fill Height (ft)	Deflection (%)
10	1.71
20	3.19
30	4.21
35	4.63
40	5.12

Table 8.12 Backfill Strength for Coarse Granular Soils Including Flex Base at Different Percentiles.

Percentile (%)	γm kip/ft^3	C kip/ft^3	ϕo deg	$\Delta\phi$ deg	K	n	R_f	Kb	m
5	0.137	0	34.0	3.6	198	0.4	0.7	41	0.2
10	0.139	0	35.3	4.5	263	0.4	0.7	63	0.2
25	0.142	0	37.4	5.9	371	0.4	0.7	99	0.2
50	0.146	0	39.8	7.5	491	0.4	0.7	139	0.2

Table 8.13 Backfill Strength for Coarse Granular Soils without Flex Base at Different Percentiles.

Percentile (%)	γm kip/ft^3	C kip/ft^3	ϕo deg	$\Delta\phi$ deg	K	n	R_f	Kb	m
5	0.137	0	34.3	3.9	215	0.4	0.7	47	0.2
10	0.139	0	35.6	4.7	281	0.4	0.7	69	0.2
25	0.143	0	37.8	6.2	391	0.4	0.7	105	0.2
50	0.147	0	40.3	7.8	513	0.4	0.7	146	0.2

Table 8.14. Linear Load for CANDE Corresponding to Different Axle Loads and Cover.

Axle Load (kips)	Load per Wheel (kips)	Contact Area (in^2)	B = L (ft)	Cover (ft)	Dynamic Load (kips)	Stress (kips/ft^2)	P/b $_{CANDE}$ (lb/in)
16	8	80	0.74	1.0	10.4	3.83	250.7
32	16	160	1.05	1.0	20.8	6.56	429.3
64	32	320	1.49	1.0	41.6	10.11	662.2
128	64	640	2.10	1.0	83.2	13.69	896.3
16	8	80	0.74	2.0	9.6	1.00	131.9
32	16	160	1.05	2.0	19.2	1.92	252.1
64	32	320	1.49	2.0	38.4	3.53	462.8
128	64	640	2.10	2.0	76.8	6.05	792.6
16	8	80	0.74	3.0	8.8	0.42	82.7
32	16	160	1.05	3.0	17.6	0.82	162.0
64	32	320	1.49	3.0	35.2	1.58	311.2
128	64	640	2.10	3.0	70.4	2.93	576.2
250	125	1250	2.94	3.0	137.5	5.00	983.7

Table 8.15. Results from Repeated Loading Phase.

Test	Material	Compaction Level	Pipe Diameter (in)	Initial Deflection (%)	Final Deflection (%)	Change (%)
8	Coarse Gravel	Medium	48	2.317	3.421	47.6
9	Medium Gravel	Medium	48	2.117	3.606	70.3
10	Gravelly Sand	Medium	48	2.640	6.938	162.8
11	Clayey Sand	High	36	2.490	4.920	97.6
15	Medium Gravel	Medium	36	2.564	3.131	22.1
16	Gravelly Sand	Medium	36	2.436	3.622	48.7

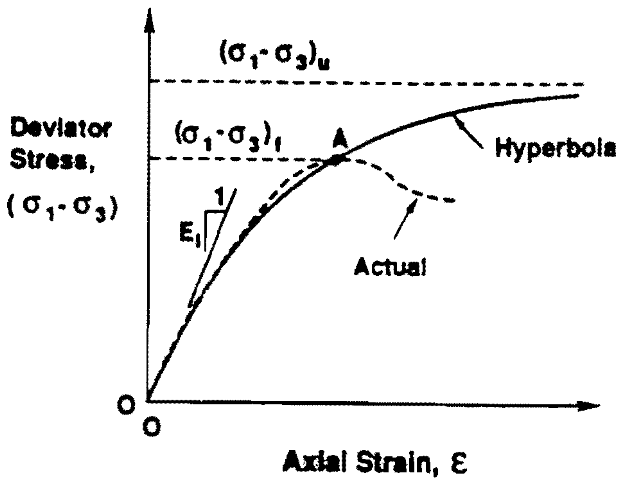


Figure 8.1. The Calculation of R_f (*1*).

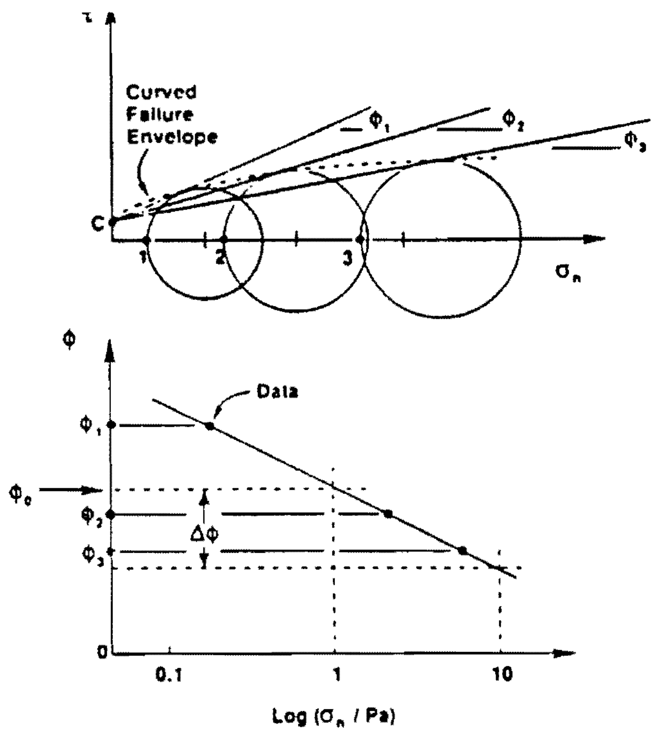


Figure 8.2. The Calculation of the Modified (*1*).

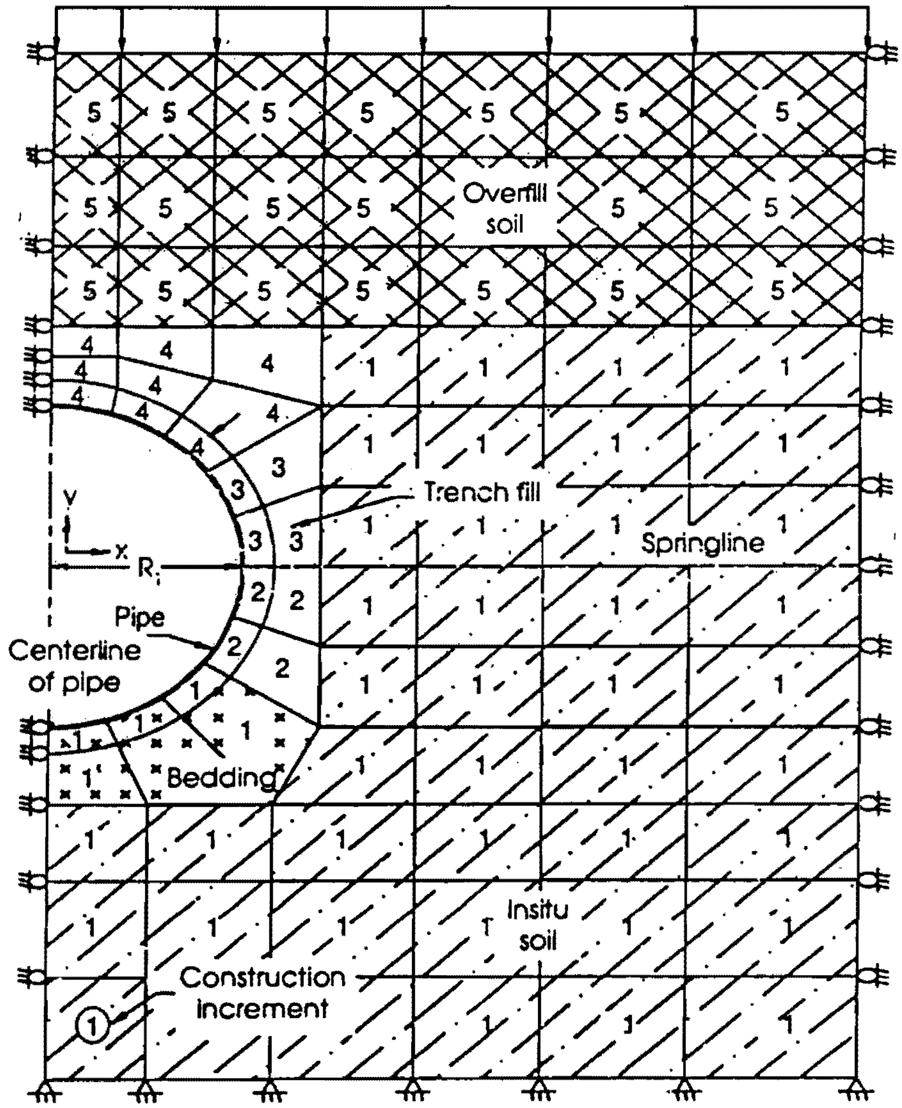


Figure 8.3. A Finite Element Mesh Used in the Analysis (*49*).

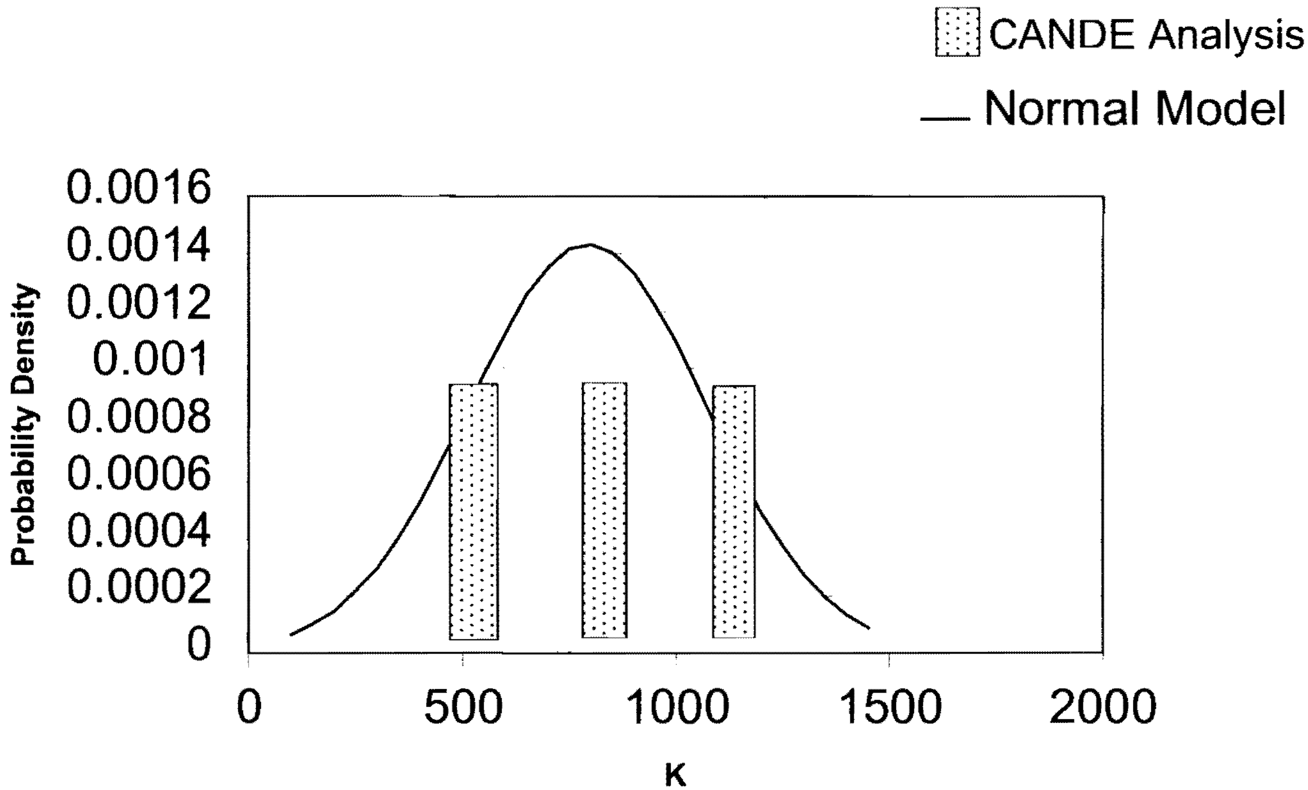


Figure 8.4. Variation of K Back Calculated from CANDE for Backfill with High Compaction in Comparison with a Normal Model.

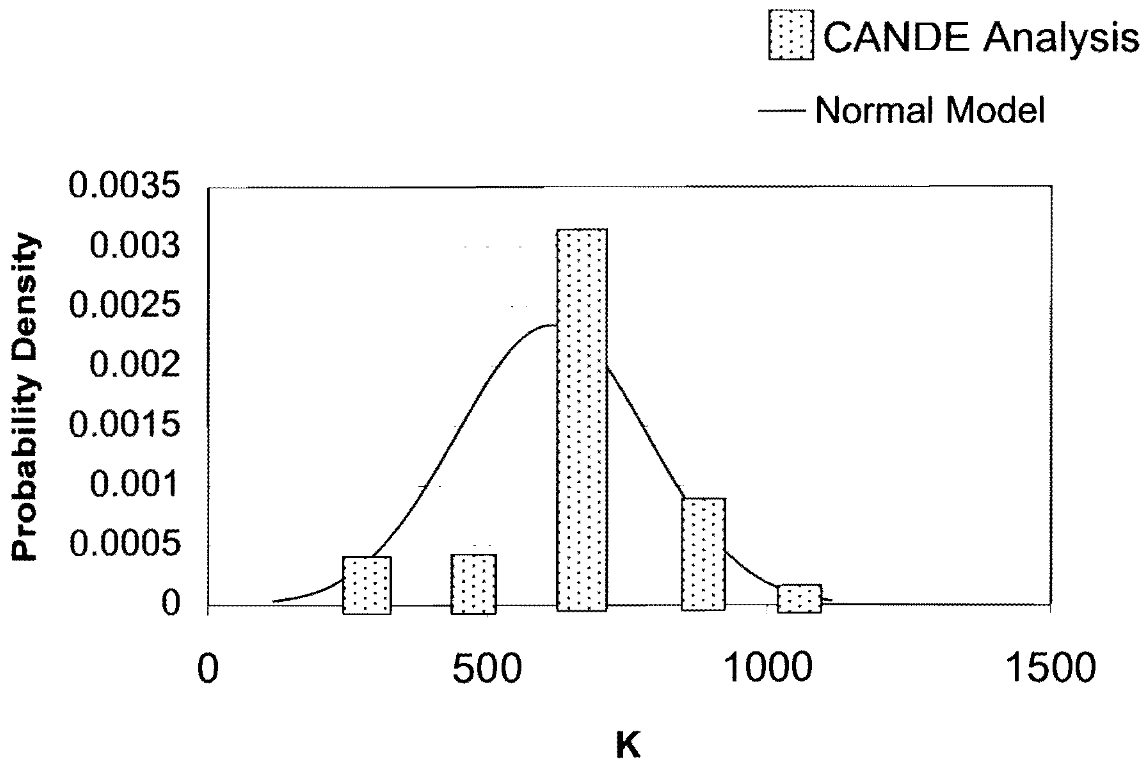


Figure 8.5. Variation of K Back Calculated from CANDE for Backfill with Medium Compaction in Comparison with a Normal Model.

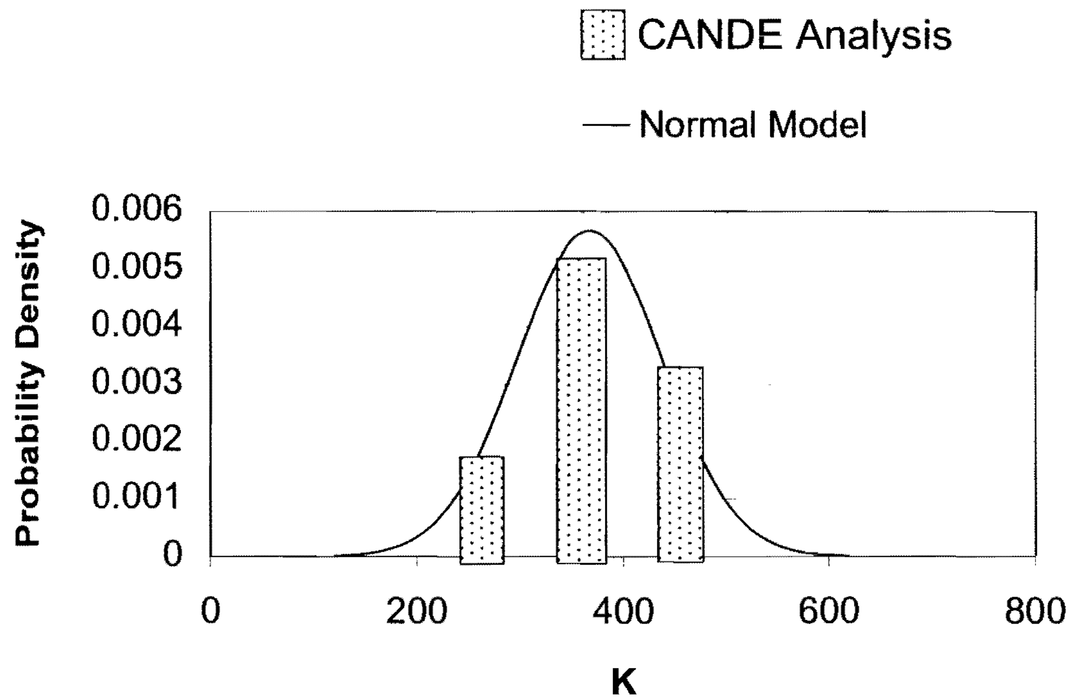


Figure 8.6. Variation of K Back Calculated from CANDE for Backfill with Low Compaction in Comparison with a Normal Model.

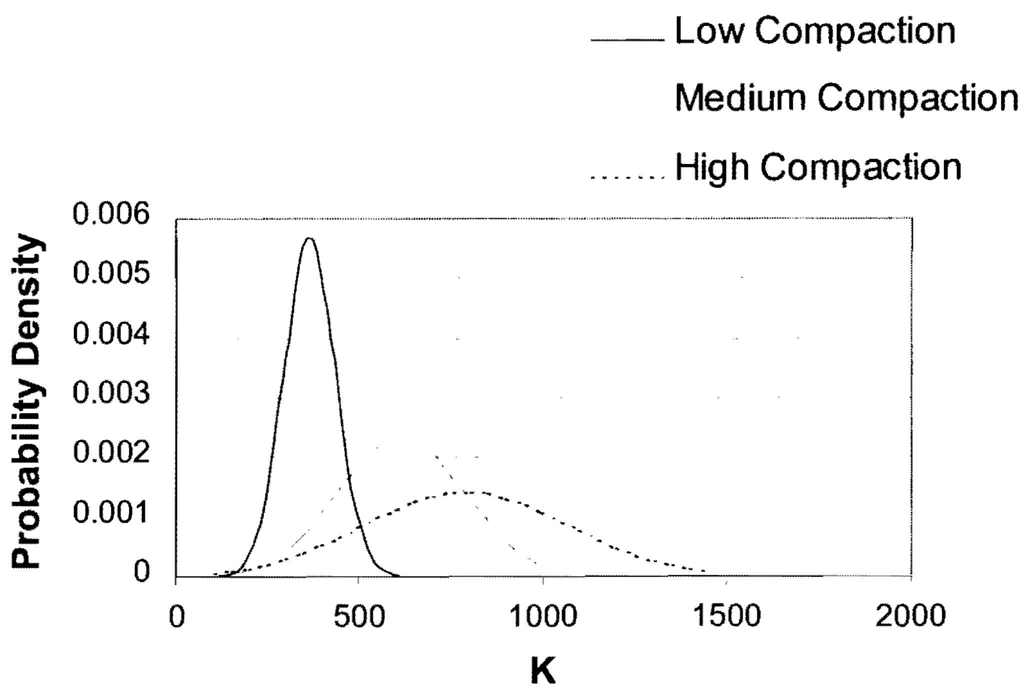


Figure 8.7. The variation of K with Compaction Effort.

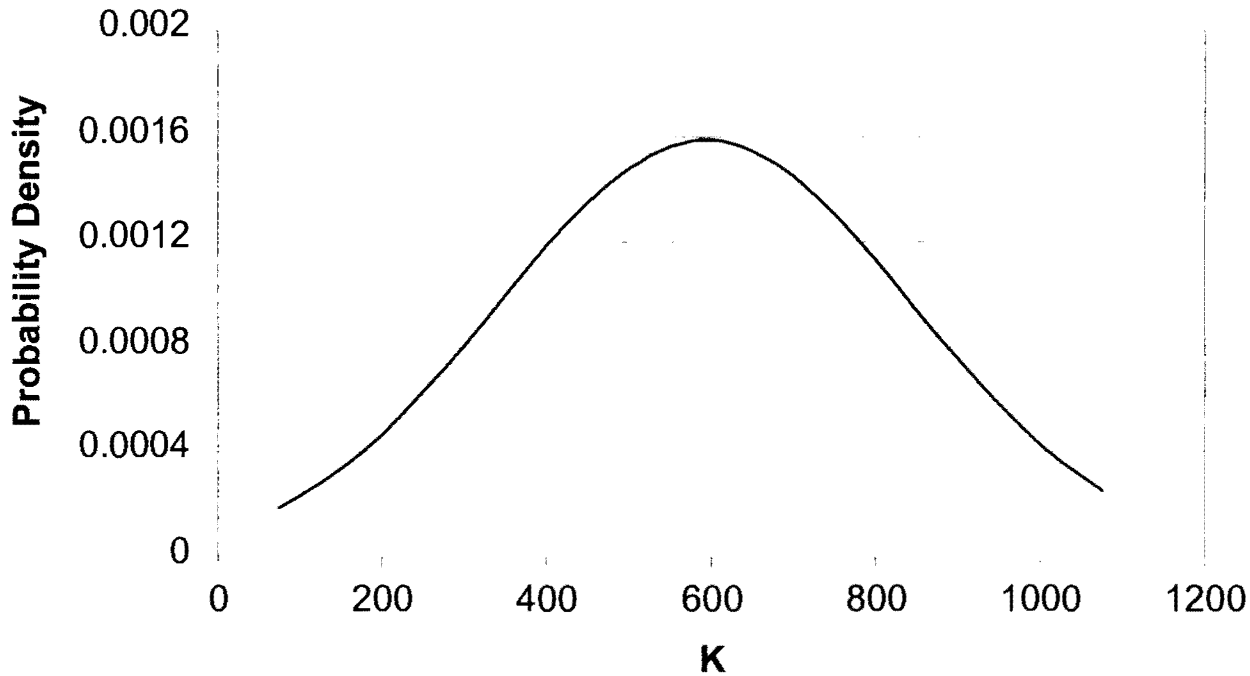


Figure 8.8. Probability Distribution of K Including All Compaction Levels.

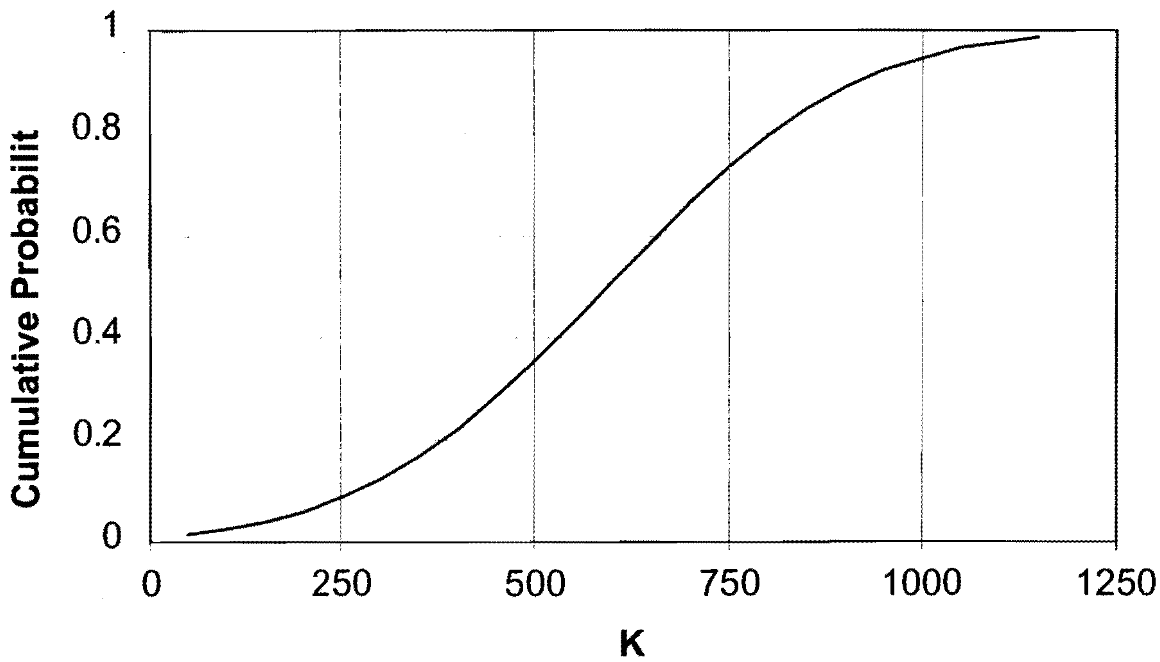


Figure 8.9. Cumulative Probability Function of K Including All Compaction Levels.

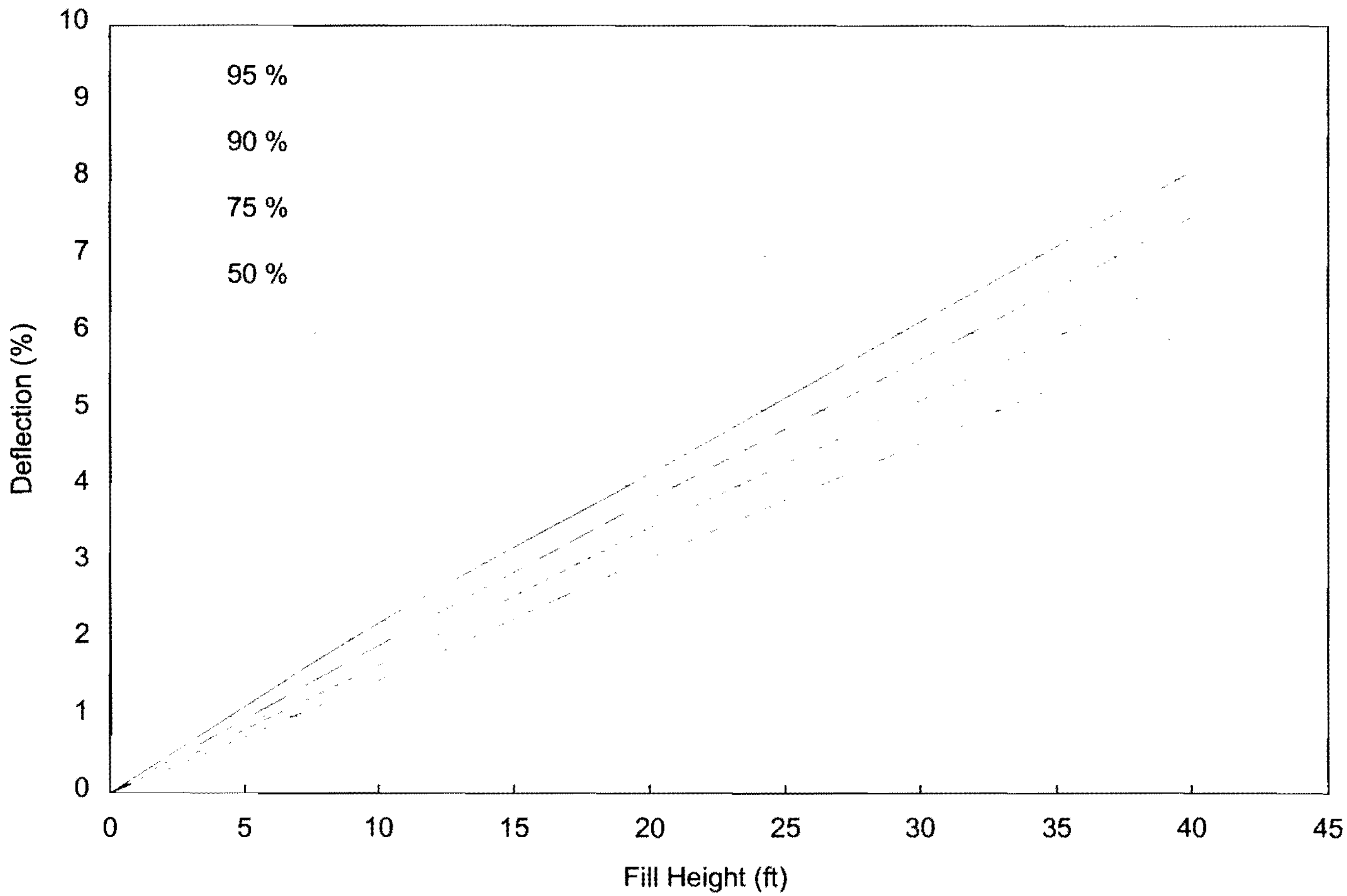


Figure 8.10 Pipe Deflections for Coarse Granular Material Including Flex Base.

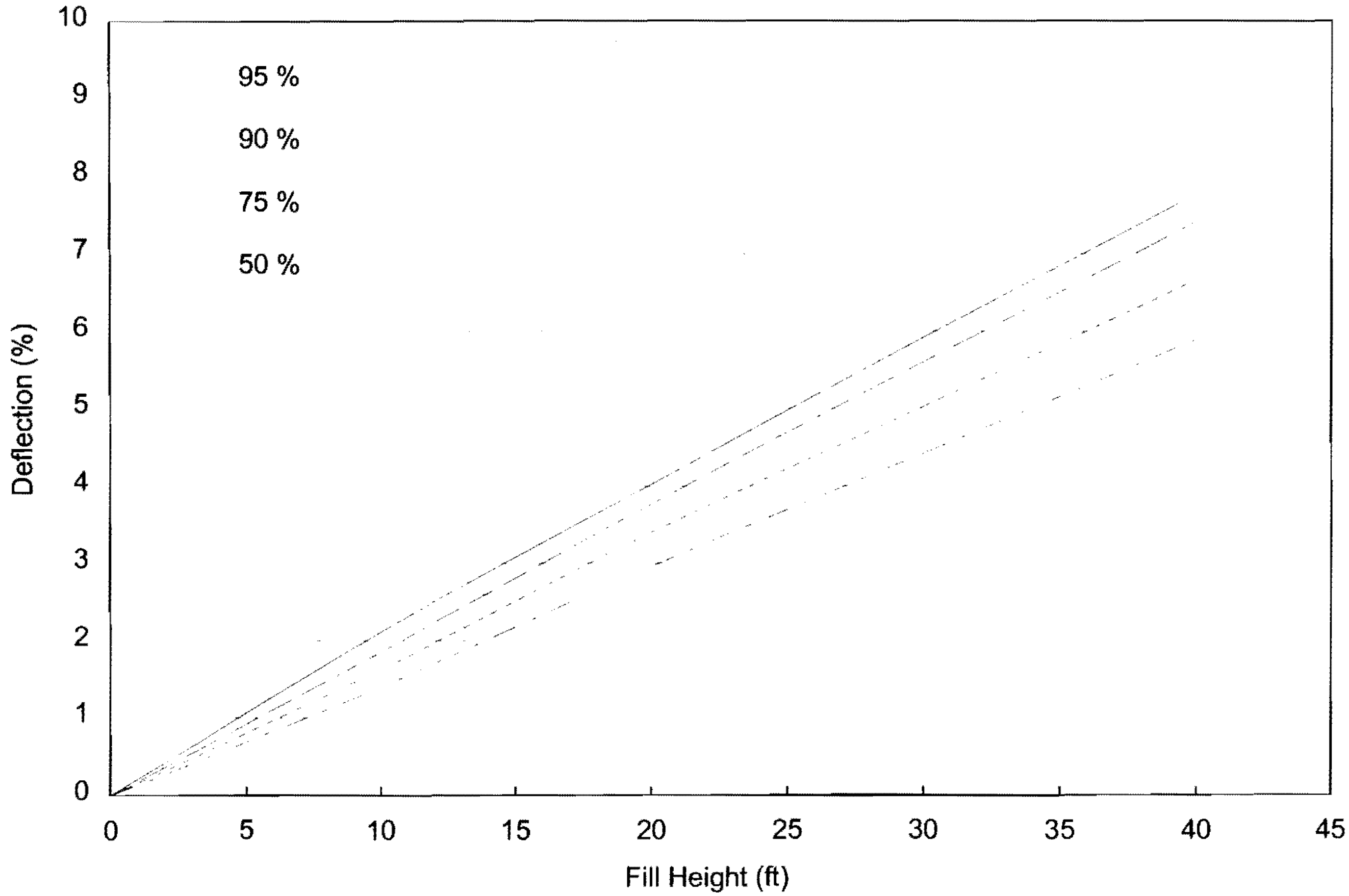


Figure 8.11 Pipe Deflections for Coarse Granular Material Without Flex Base.

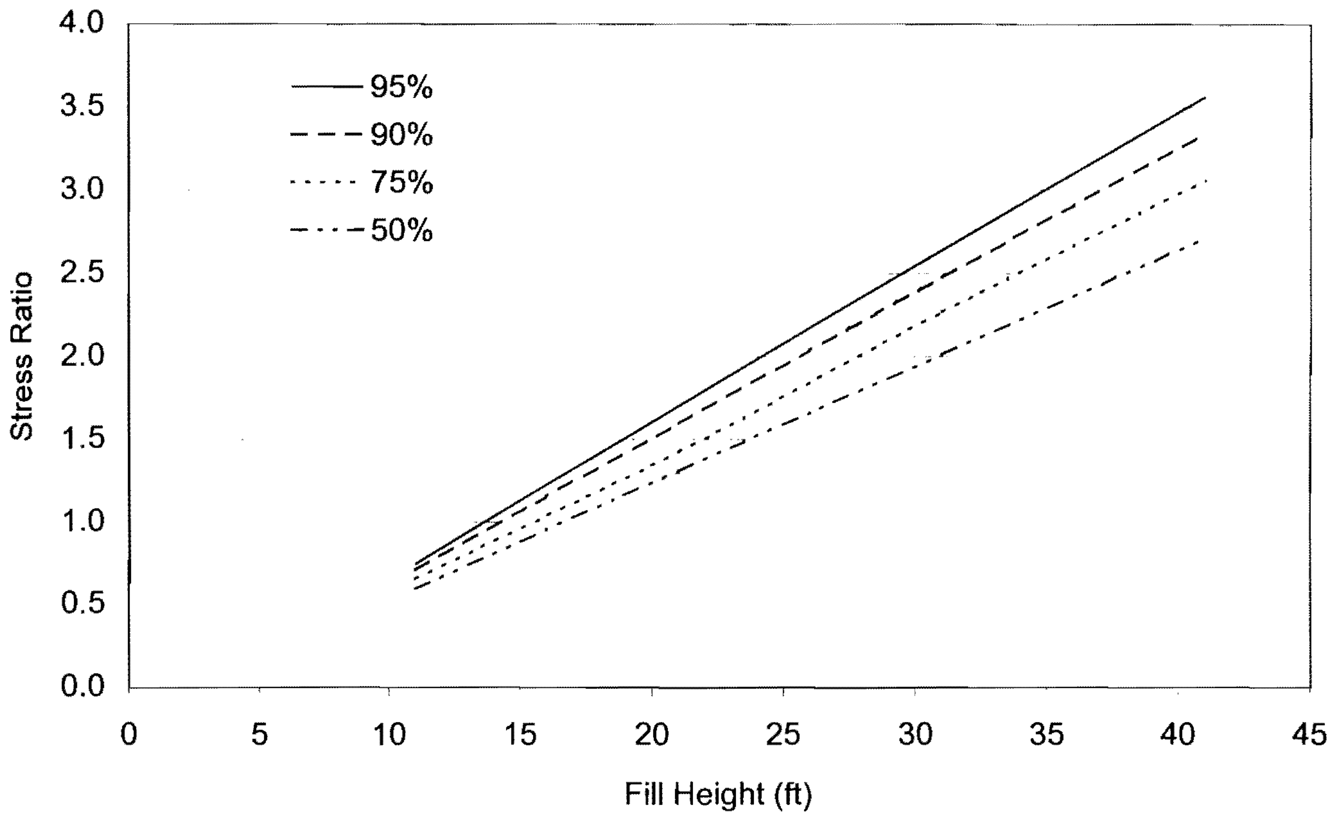


Figure 8.12 Variation of Stress Ratio with Fill Height and Reliability - Weak In Situ Soil.

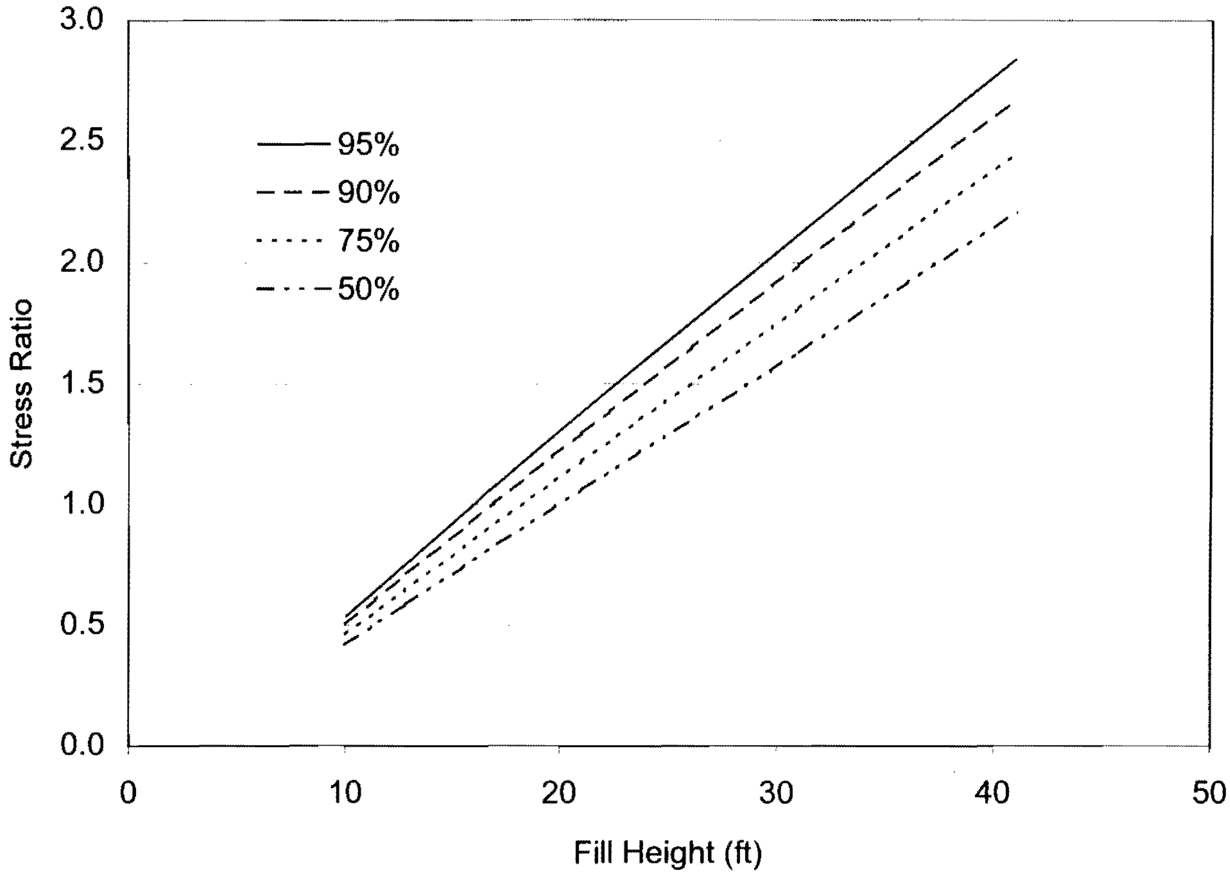


Figure 8.13 Variation of Stress Ratio with Fill Height and Reliability- Medium Strength In situ Soil

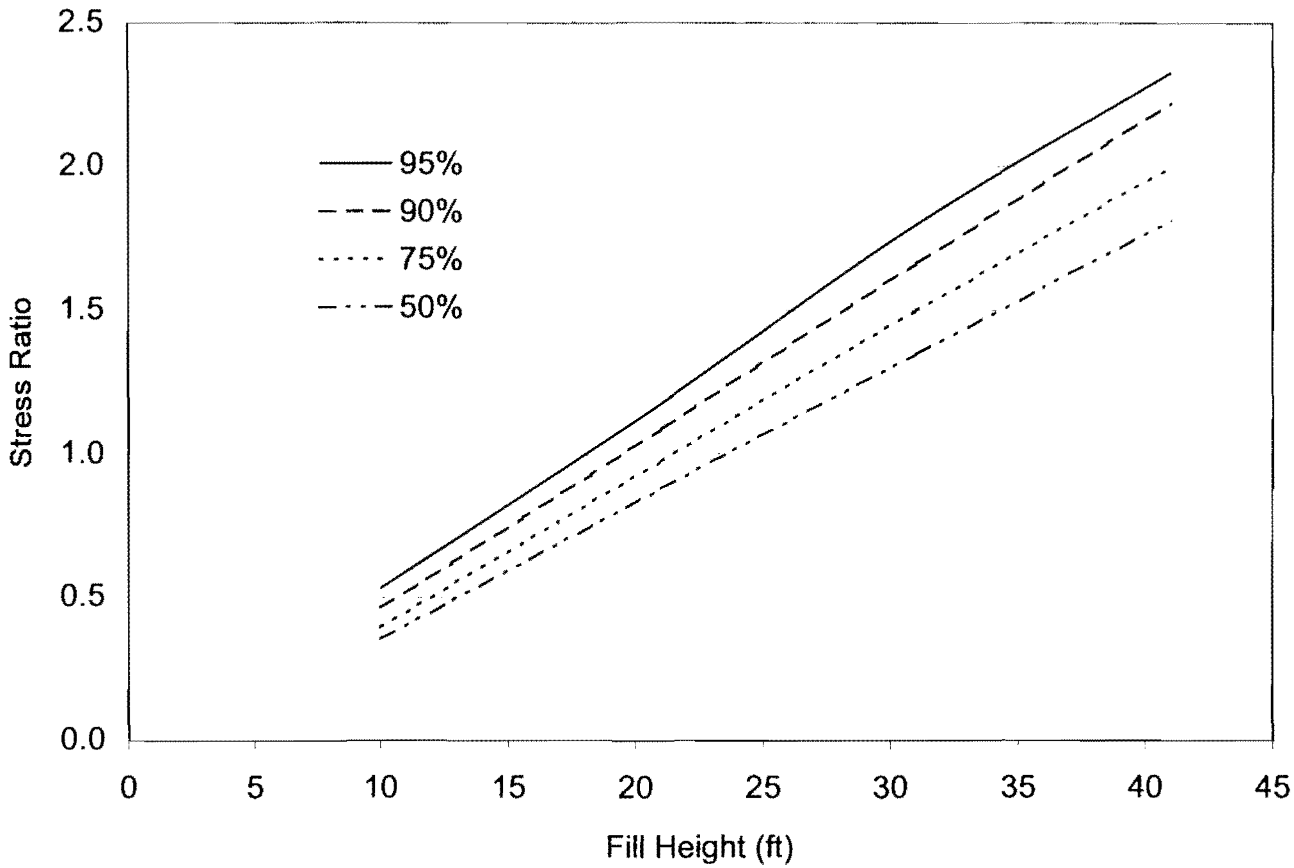


Figure 8.13 Variation of Stress Ratio with Fill Height and Reliability-Stiff In situ Soil.

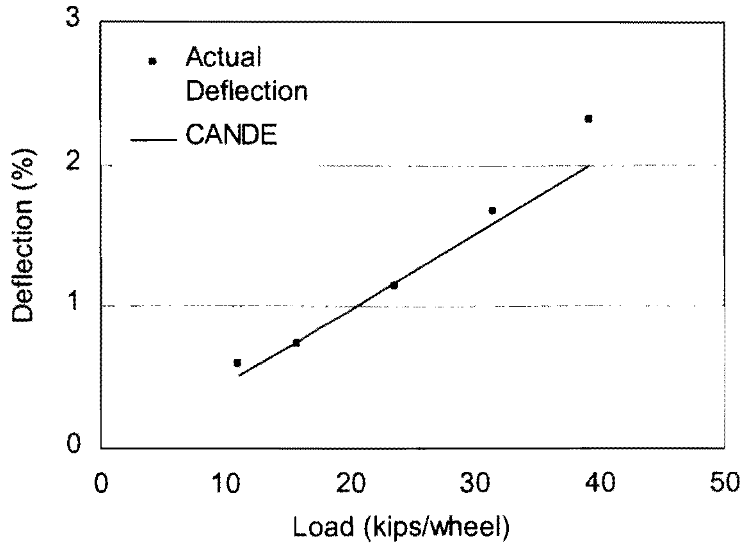


Figure 8.15. Deflection by CANDE versus Actual Deflection for Test 8.

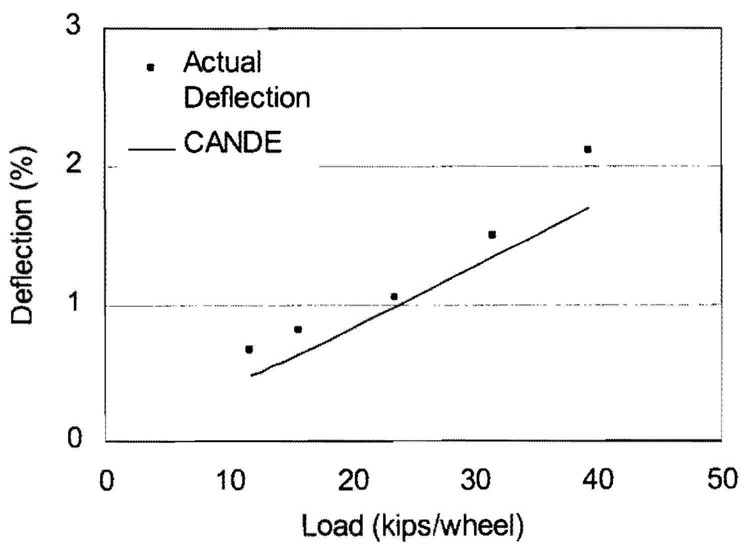


Figure 8.16. Deflection by CANDE versus Actual Deflection for Test 9.

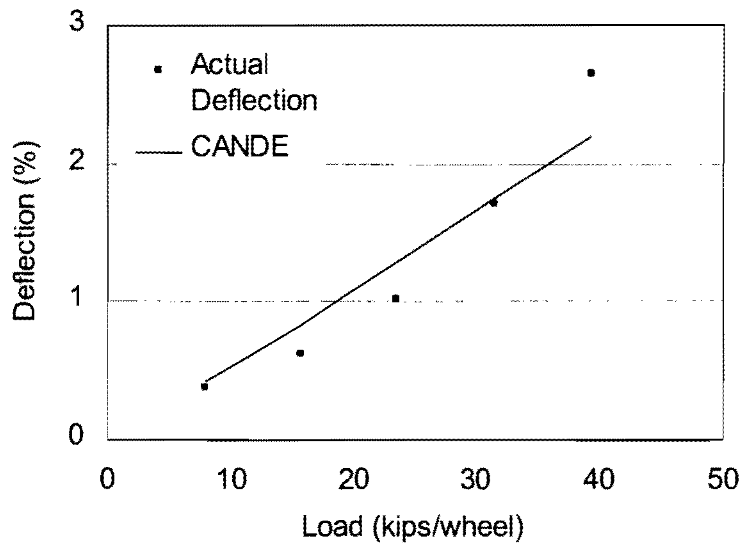


Figure 8.17. Deflection by CANDE versus Actual Deflection for Test 10.

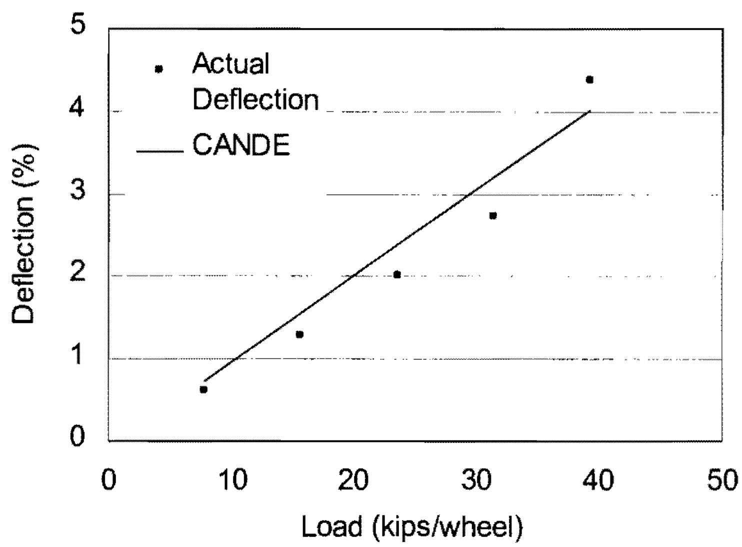


Figure 8.18 Deflection by CANDE versus Actual Deflection for Test 12.

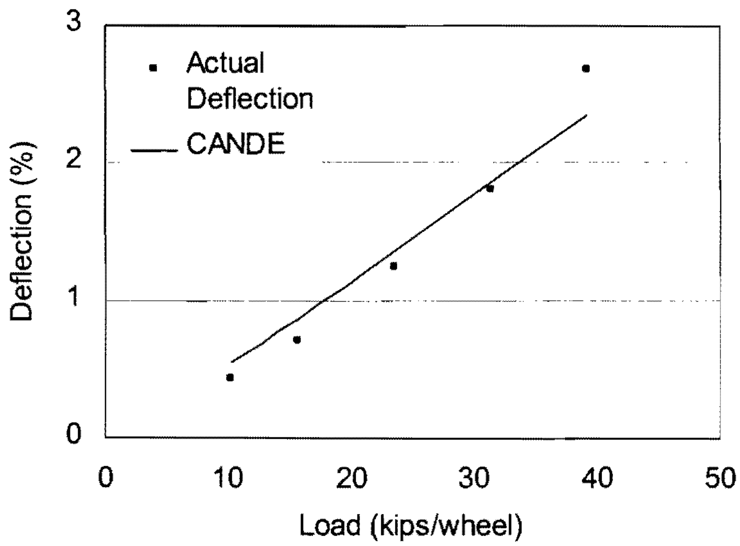


Figure.8.19. Deflection by CANDE versus Actual Deflection for Test 13.

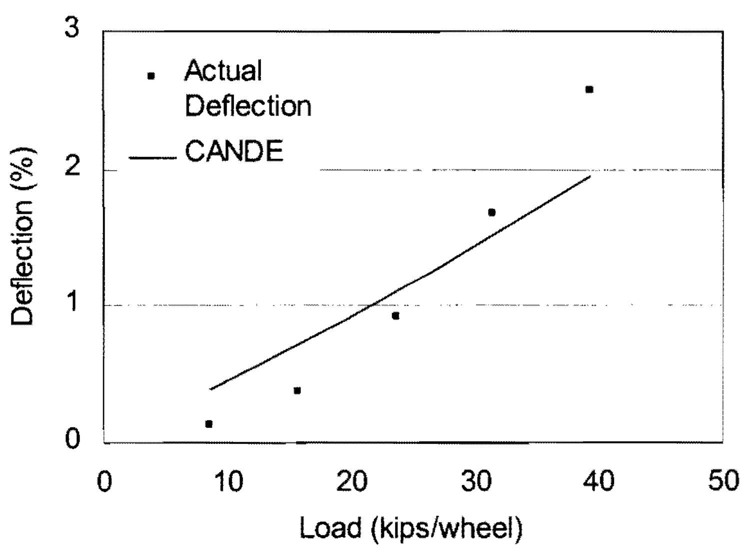


Figure 8.20 Deflection by CANDE versus Actual Deflection for Test 15.

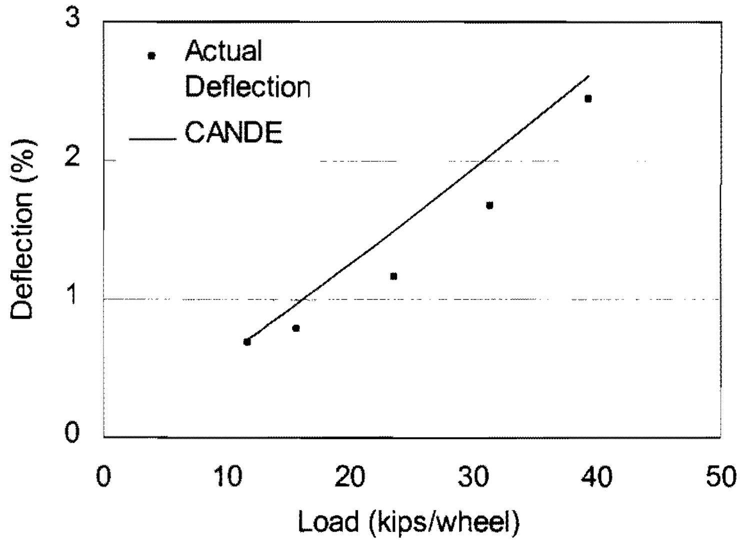


Figure 8.21 Deflection by CANDE versus Actual Deflection for Test 16

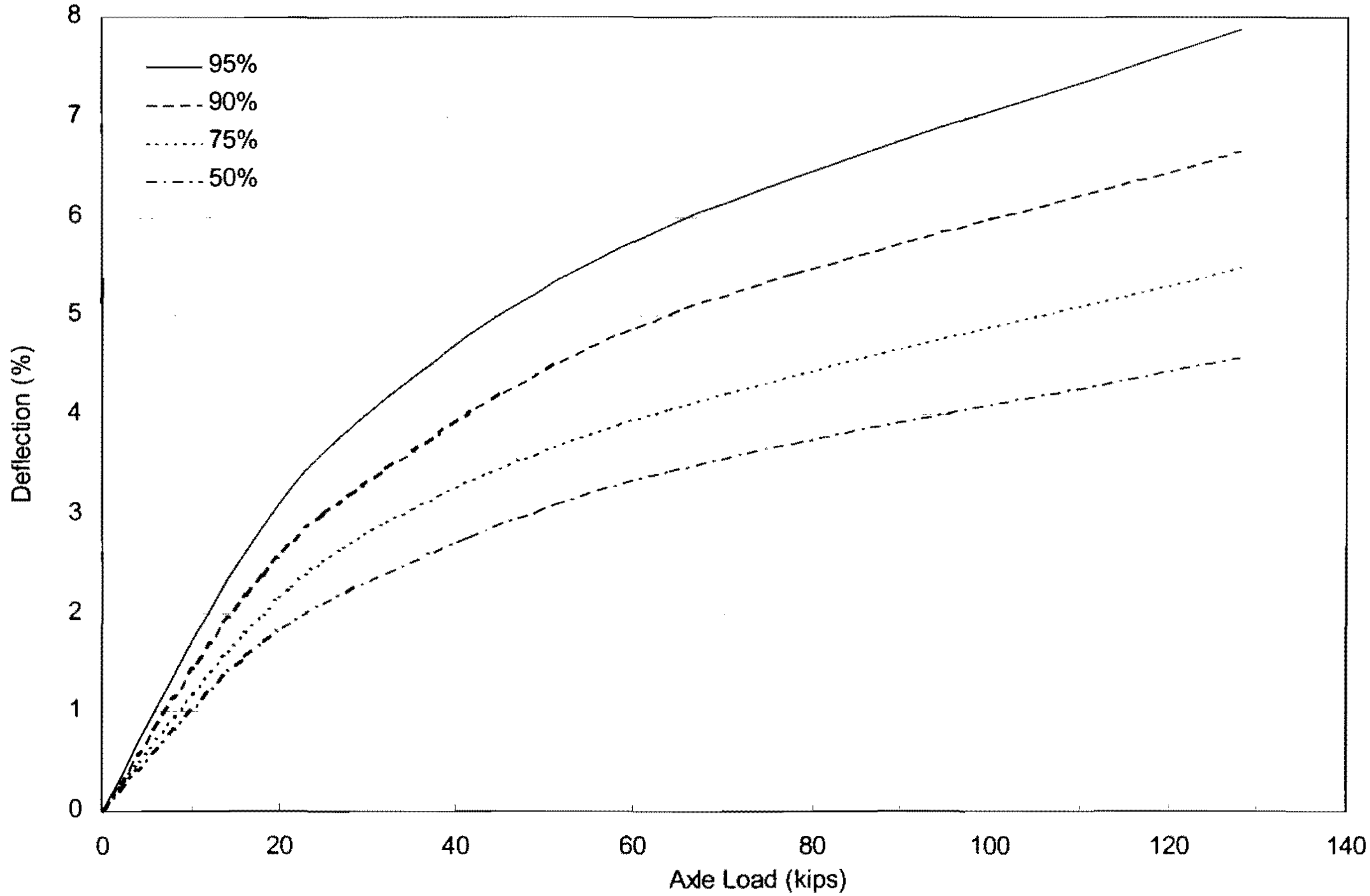


Figure 8.22 Deflections Predicted by CANDE for 1 Foot Cover.

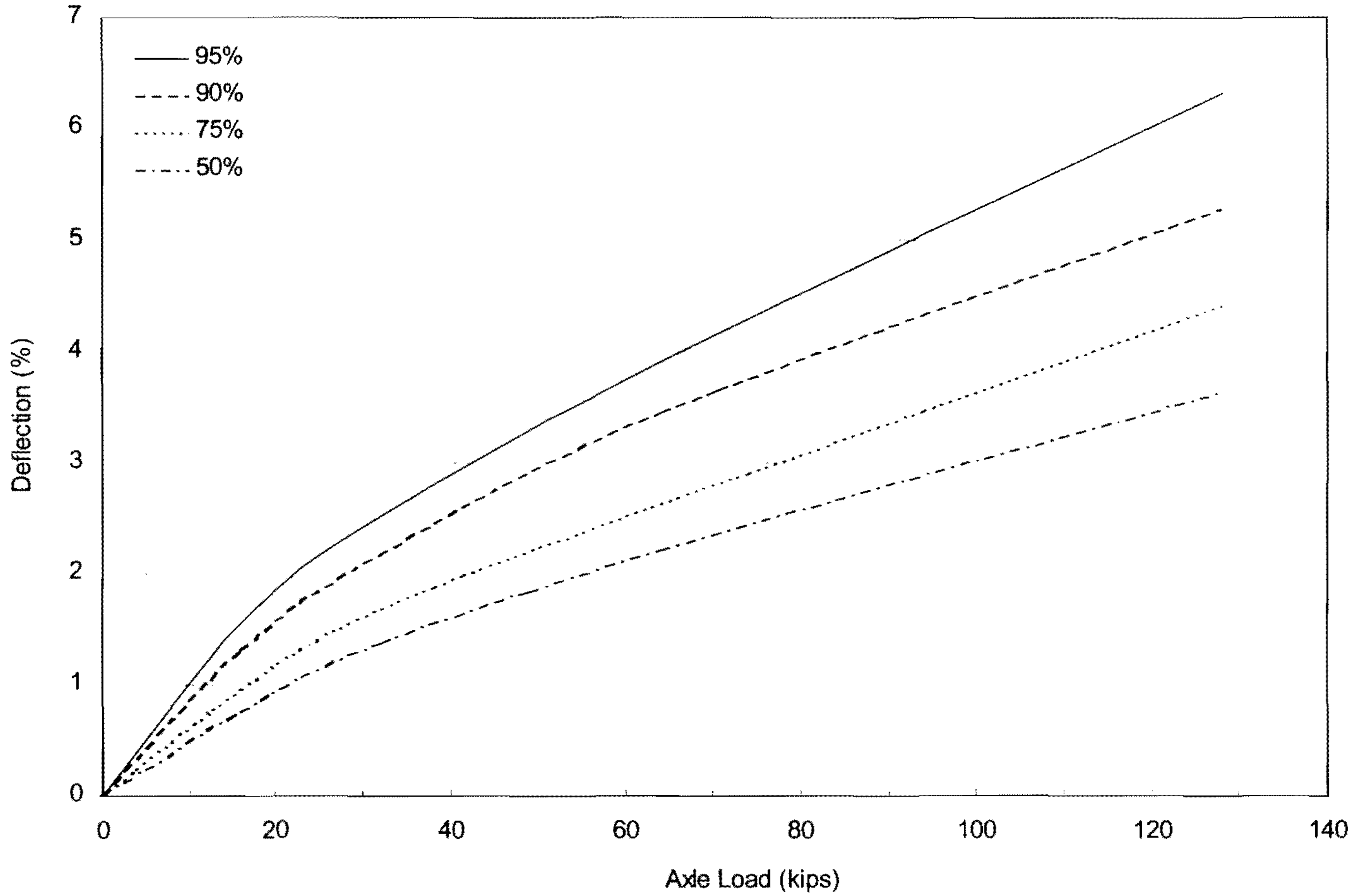


Figure 8.23 Deflections Predicted by CANDE for 2 Feet Cover.

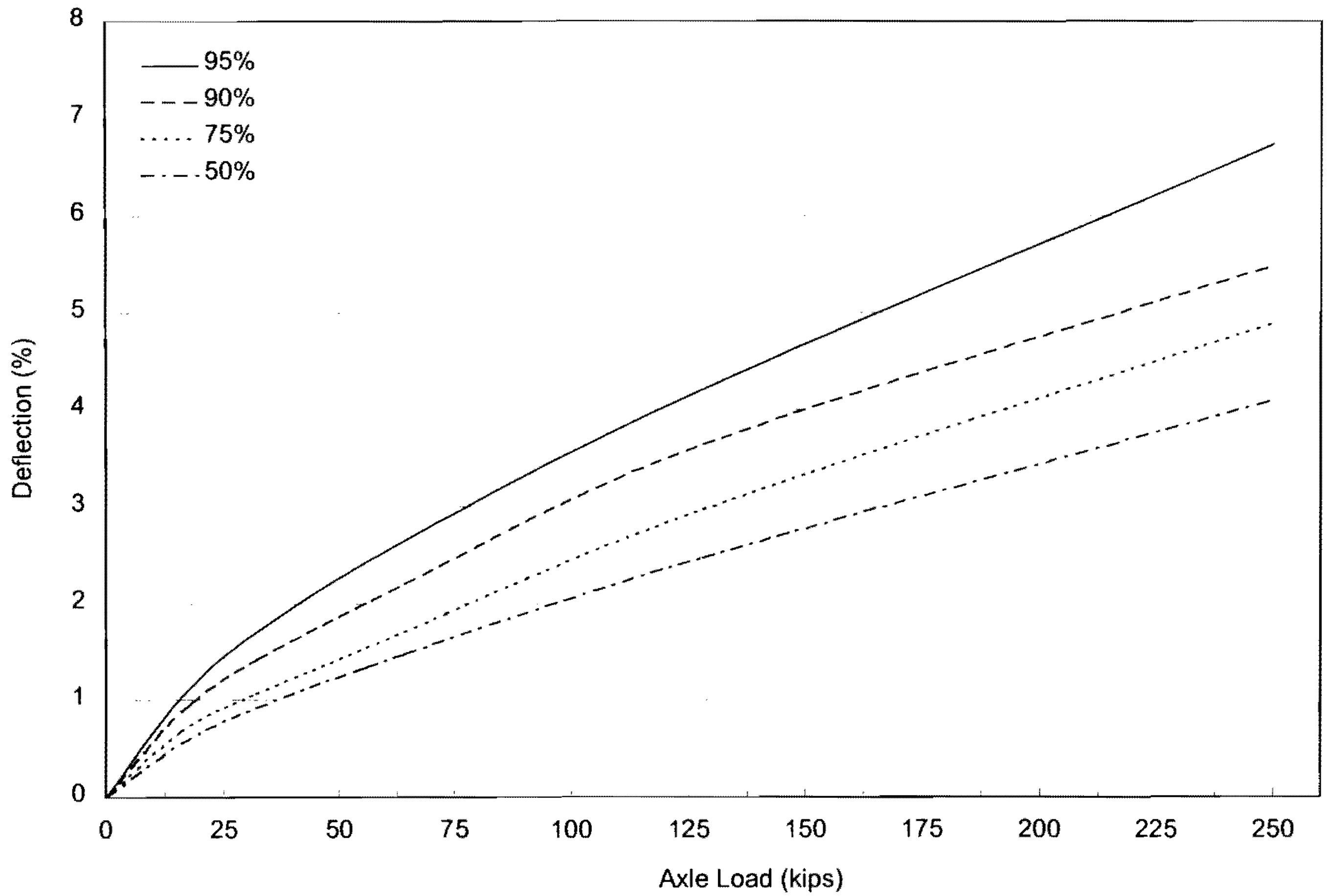


Figure 8.24 Deflections Predicted by CANDE for 3 Feet Cover.

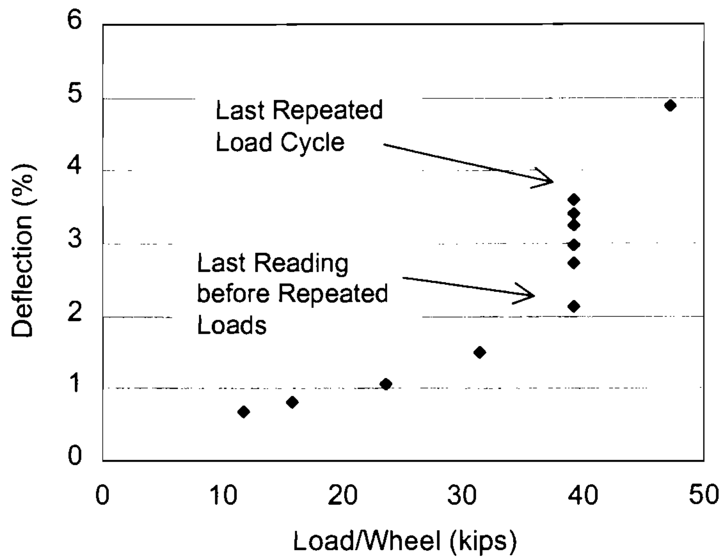


Figure 8.25. Results from Repeated Load Simulation.

CHAPTER IX
CONCLUSIONS AND RECOMMENDATIONS

The primary goal of this research study was to develop new specifications for the use of large diameter (36 to 48 inch diameter) High Density Polyethylene (HDPE) pipe in highway drainage applications. The research plan to accomplish this goal included the following tasks: (a) a national survey to document other state DOT practices with regard to use of large diameter HDPE pipe, (b) development of a draft specification based on existing AASHTO, ASTM and other specifications, (c) identify 8 TxDOT pipe installation projects, install large diameter HDPE pipe, monitor construction process as well as pipe performance, (d) 21 full scale field load tests to collect data necessary to develop maximum fill height and minimum cover requirements, (e) perform a constructibility review and hence make necessary changes in the draft specifications, (f) perform an economic analysis to collect data on prices of various types of backfill, RCP and HDPE pipe and hence compare as-installed costs for HDPE pipe installations against those for RCP installations.

The proposed specification for the installation of large diameter HDPE pipe is found in Appendix C of this report. The conclusions and recommendations from this study are summarized below.

9.1 Survey of Other DOT Practice

Out of 50 state DOTs contacted 32 responded to the survey. 18 out of the 32 state DOTs that responded indicated that they have specifications in place to allow the use of large diameter HDPE pipe. 14 out of those 18 states allowed use of HDPE pipe only up to 36in diameter. California, New York, Florida (max. 48in) and Ohio (max. 60in) were exceptions to this rule. Most states reported positive experience with the use of this type of pipe. They stated that the majority of the problems that they have experienced in their HDPE installations are typical of those experienced with other types of pipe and are not unique to HDPE pipe. Among the problems unique to HDPE pipe, difficulty in maintaining line and grade during installation and finding qualified contractors were cited most often.

9.2 TxDOT Pilot Construction Projects

Eight TxDOT pilot construction projects were selected to serve as pilot construction projects for this research study. There were located in 5 separate TxDOT districts (i.e. San Angelo, Laredo, Atlanta, Yoakum and Wichita Falls) so that the broad range of climatic and soil conditions that are found within the state is represented. The pipe installations included: single barrel and multiple barrel installations, 36 in, 42 in, and 48 in diameter pipes and pipe products from 3 different manufacturers. Some of these pipe installations were carried out by contractors while others were carried out by TxDOT maintenance crew. The reaction of the TxDOT engineers and the construction personnel who participated in pilot construction projects was generally favorable toward the new product. In their view, the speed of installation of HDPE pipe was the most significant advantage of HDPE pipe. When HDPE pipe is used, a culvert installation project across a two-lane highway can be completed in one day. A second advantage identified by construction personnel was ease of handling of the HDPE pipe. At some project sites, there was concern over the use of granular backfill that may erode during flooding. In fact, some erosion of the backfill was observed during the post-construction monitoring of one of the pilot construction projects in Atlanta District. Therefore, the researchers recommend that backfill material be properly confined by using rip rap and other suitable end treatment where flooding is anticipated.

All 8 pilot installations were completed successfully. However, the contactor's familiarity with the specifications and care taken during pipe installation varied significantly from one installation to another. For example, the level of compactive effort used on pipe embedment material varied from three or four passes of impact rammer to no compaction at all. This confirmed the concern expressed by many TxDOT engineers regarding the difficulty in enforcing strict quality control measures during field installation of pipe.

In spite of limited construction control observed at some of these sites, all 8 pipe installations met the desired performance limits. The primary performance limit used in monitoring was the maximum pipe deflection. The vertical deflection measurements were made at several cross-section of the pipe shortly after installation and after the pipe had been in-service for several months. All of the measured deflections were found to be within the acceptable limit of 5% of nominal pipe diameter. This observation supports the view that, when granular material with specified gradation is used, satisfactory pipe performance can be achieved even with minimal control during installation. The pipe deflection measurements in the eight pilot installations varied from a minimum of – 0.75% to a maximum of 3%. The – 0.75% deflection represents an increase in vertical diameter due to lateral compression. In addition to deflection measurements, pipes were also examined for cracking and other distresses, evidence of joint failure, backfill erosion, and backfill settlement. Inspections revealed no significant problems in these areas.

9.3 Economic Analysis

The economic analysis performed in this study examined the economic competitiveness of HDPE pipe versus the most widely used pipe product, RCP. This analysis was based on the costs associated with the pipe installation. A life cycle cost analysis that considers not only the initial installation cost but also cost of maintenance and operation of the pipe system was not considered feasible because long-term data on pipe performance and maintenance is not available for HDPE pipe. Although HDPE pipe has been in use for some time, the use of HDPE pipe in the large diameter category is quite recent. Moreover, the technology and the material that is currently being used in HDPE pipe production and the designs of pipe joints are very recent. As a result, any data available on performance of exiting pipe systems is not likely to be very representative of the performance of HDPE pipes in the future. Therefore, the analysis conducted in this research was limited to a comparison between as-installed costs for the two pipe products. Data collected on pipe prices reveal that HDPE pipe is significantly cheaper than RCP. However, HDPE pipe installation according to proposed specifications requires special kind of backfill material that must be obtained and transported to the project site at a cost. Among the backfill material types allowed by the specifications, the granular fill is the most economical. Data was collected on other two types of backfill allowed in the proposed specifications; cement stabilized backfill and flowable fill. The data revealed that the availability and price of cement stabilized backfill follows a geographic pattern. It is a readily available at economical prices in the TxDOT gulf coast districts such as Houston and Beaumont. The price of cement stabilized backfill is much higher in TxDOT districts away from the Gulf coast. Flowable fill is the most expensive among all three backfill materials. Although there are many factors that control the cost of a pipe installation, the analysis showed that, in general, HDPE pipe is the more cost effective option when granular backfill is available at $20/CY or less. When cement stabilized backfill or flowable fill is used, HDPE installations are more expensive than RCP installations that use native backfill.

9.4 Backfill Materials

The proposed specification allows the use of three types of backfill; (a) Type I Flowable backfill that meets TxDOT Special Specification Item 4438, (b) Type II Cement stabilized backfill that meets TxDOT Specification Item 400.6, (c) Type III Granular backfill that meets the gradation requirements specified in Table 2 of the proposed specifications. As mentioned earlier, for greatest economy Type III granular backfill must be used whenever possible.

Type III backfill specified in the proposed specification consists of coarse granular materials that include no more than 10% of minus No.200 materials. Materials that meet this specific gradation are less sensitive to placement and compaction conditions and, as a result, can provide good pipe support even when there is minimal control during backfill placement. In the selection of a backfill material, preference should be given to more well-graded materials. Compaction is less effective on uniformly graded materials and therefore when such material is used, construction personnel tend to avoid the compaction operation altogether. Data collected from pilot construction project show that, uniformly graded materials that have received minimal compaction can still provide adequate pipe support once it is properly confined. However, the pipe deflections measured in these cases were higher. This is because good compaction of pipe backfill in the haunch area and on the sides causes the pipe to increase its vertical diameter (i.e. negative deflection) initially. Such negative deflection helps in reducing subsequent positive deflection (reduction in vertical diameter) due to loads.

9.5 Maximum Fill Height and Minimum Cover Requirements

Chapter 8 of this report presents design charts that may be used to determine the maximum fill heights to be allowed and minimum cover required in HDPE pipe installations. One of the unique features in these design charts is that they incorporate construction variability. To use the charts, the design engineer must first select a desired level of reliability. For example, if 95% reliability is selected, then there is only a 5% probability that the desired pipe performance limit may be exceeded in an actual installation. The maximum fill heights were found to be controlled by thrust stress in the pipe wall whereas minimum cover was controlled by pipe deflections.

9.6 Recommendations for Implementation

The findings from this research indicate it will be beneficial for the Department to allow large diameter (up to 48in diameter) HDPE as a biddable alternative in TxDOT construction projects. Appendix C of this report presents the specifications that can be used in implementation.

This specification is largely based on the data collected from full-scale load tests and experience gained from 8 pilot construction projects. Because of limited experience, the researchers have taken a somewhat conservative approach in specifying backfill material gradation, maximum fill heights and minimum cover. During implementation, it will be useful to know whether any of these specifications will prove to be a significant constraint that will prevent the Department from getting the maximum benefit from the use of this product. It will also be useful to know whether there are other special situations that were not encountered during the pilot construction projects but should be addressed in the specifications.

Therefore, the researchers recommend that the initial implementation be conducted under the careful control of a monitoring program. This can be conducted as a part of a research implementation program where the construction and performance of a larger number of new HDPE pipe will be monitored. The Department may consider using appropriate restrictions on the ADT or class of highway during the initial round of pipe installation and then gradually relax these requirements as more pipe monitoring data become available. It will also be beneficial to include a training program in the implementation program to familiarize TxDOT engineers and construction personnel on various aspects of HDPE pipe installation.

APPENDIX A

DRAFT SPECIFICATION FOR INSTALLATION
OF HDPE PIPE, DATED MAY 15, 1998

DRAFT SPECIFICATIONS

HIGH DENSITY POLYETHYLENE (HDPE) PIPE FOR GRAVITY FLOW DRAINAGE APPLICATIONS

1. Description. This specification shall govern for the furnishing and installing of all 18 in (450 mm) to 48 in (1200mm)[1] high density polyethylene (HDPE) pipe used in the construction of thermoplastic pipe culverts, sewer mains, laterals, stubs and inlet leads. The pipes shall be of the sizes, types, design and dimensions shown on the plans and shall include all connections and joints to new or existing pipes, sewer, manholes, inlets, headwalls and other appurtenances as may be required to complete the work.

2. Materials. Unless otherwise specified on the plans or herein, the HDPE pipes and fittings used for gravity flow drainage applications shall conform to the following specifications.

2.1 High density polyethylene pipes and fittings shall meet the requirements as in AASHTO M 294M-96 (for pipes up to 36 inches/900mm in diameter) and AASHTO MP6-95 (for pipes of 42 inches/1050mm and 48 inches/1200 mm in diameter).

2.2 *Raw Materials* - The pipes and the fittings shall be manufactured from virgin PE compounds which conform to the requirements of cell class 335420C[2] as defined and described in ASTM D 3350, except that carbon black content shall not exceed 5 percent.

2.3 *Designation of Type* – The HDPE pipes used for gravity flow drainage applications shall be of Type S (outer corrugated wall with smooth inner liner) or Type D (inner and outer smooth walls braced circumferencially or spirally with projections or ribs).

2.4 *Section Properties* - Minimum wall thickness of the inner walls of Type S pipe and inner and outer walls of Type D pipe shall be as specified in Section 7.2.2 of the AASHTO M 294M-96 and MP6-95 respectively. The pipe stiffness at 5% deflection, when determined in accordance with ASTM designation D 2412, shall be as specified in Section 7.4 of AASHTO M 294M-96 and AASHTO MP6-95.

The manufacturer shall perform appropriate test procedures on representative samples of each type of pipe furnished, and hence verify that the pipe complies with the specifications. A certificate of compliance shall be prepared and submitted to the Department for review and approval. It shall include the following information: manufacturing plant, date of manufacture, pipe unit mass, material distribution, pipe dimensions, water inlet area, pipe stiffness, pipe flattening, brittleness, environmental stress crack resistance, and workmanship.

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[1] Nominal pipe size is the nominal inside diameter of the pipe

[2] This new cell classification (i.e. 335420C) which is required in AASHTO Section 18 is a higher classification than that found in AASHTO M 294M-96 (i.e. 324420C).

3. Inspection. The quality of the materials, the process of manufacture, and the finished pipe shall be subject to inspection and approval by the Engineer at the manufacturing plant. In addition, the finished pipe shall be subject to further inspection by the Engineer at the project site prior to and during installation.

4. Marking. All pipe shall be clearly marked at intervals of not more than 12 ft (3.5 m), and fittings and couplings shall be clearly marked as follows:

4.1 Manufacturer's name or trade mark
4.2 Nominal size
4.3 Specification designation (e.g. M 294M-96)
4.4 Plant designation code
4.5 Date of manufacture

5. Joints. Joints shall be installed such that the connection of pipe sections will form a continuous line free from irregularities in the flow line. Joints shall meet the soiltightness definition in accordance with AASHTO Section 26.4.2.4. Suitable joints are the following:

5.1 *Integral Bell-N-Spigot* – The bell shall overlap a minimum of two corrugations of the spigot end when fully engaged. The spigot end shall have an O-Ring gasket that meets ASTM F 477: Specifications for Elastomeric Seals (Gaskets) for Joining Plastic Pipe.

5.2 *Exterior Bell-N-Spigot_* - The bell shall be fully welded to the exterior of the pipe and overlap the spigot end so that flow lines and ends match when fully engaged. The spigot end shall have an O-Ring gasket that meets ASTM F 477: Specifications for Elastomeric Seals (Gaskets) for Joining Plastic Pipe.

6. Construction Methods. The location of private driveway and side road pipe shall be constructed at locations shown on the plans or as directed by the Engineer.

6.1 *Excavation* – All excavation shall be in accordance with the requirements of Item 400, "Excavation and Backfill for Structures."

The width of the trench for pipe installation shall be sufficient, but no greater than necessary, to ensure working room to properly and safely place and compact haunching and other embedment materials. The space between the pipe and trench wall must be wider than the compaction equipment used in the pipe zone.

When Type I backfill (*See section 6.8 below*) is used, the minimum trench width is the pipe outside diameter plus 12 inches (300 mm).

When Type II backfill (*See section 6.8 below*) is used, the minimum trench width is the pipe outside diameter times 1.25 plus 12 inches (300 mm). The contractor can use any trench width above the pipe zone.

6.2 *Installation in Embankment* - If any portion of the pipe projects above the existing ground level, an embankment shall be constructed as shown in the plans or as directed by the Engineer for a distance outside each side of the pipe location of not less than five times the diameter and to a minimum elevation of 2 ft (0.6 m) above the top of the pipe. The trench shall then be excavated to a width as specified in section 6.1 above.

6.3 *Shaping and Bedding* – The pipe shall be bedded in a foundation of compacted granular material that meets the gradation requirements of Type B, C, D or F aggregate mixtures in Item 334, "Hot Mix-Cold Laid Asphalt Concrete Pavements" and Item 340, "Hot Mix Asphalt Concrete Pavements." This material shall extend a minimum of 6 inches (150 mm) below the outermost corrugations or ribs and shall be carefully and accurately shaped to fit the lowest part of the pipe exterior for at least ten percent (10%) of the overall height. When requested by the Engineer, the Contractor shall furnish a template for each size and shape of the pipe to be placed for use in checking the shaping and bedding. The template shall consist of a thin plate or board cut to match the lower half of the cross section of the pipe.

6.4 *Handling and Storage* – Handling and Storage of HDPE pipe shall be in accordance with the pipe manufacturer's instructions. Proper facilities shall be provided for hoisting and lowering the pipe into the trench without damaging the pipe or disturbing the bedding or the walls of the trench.

6.5 *Laying Pipe* – Unless otherwise authorized by the Engineer, the laying of pipe on the bedding shall be started at the outlet (or downstream) end and shall proceed toward the inlet (or upstream) end with separate sections firmly joined together. The pipe should be laid in conformity with the established line and grade and shall have a full, firm and even bearing at each joint and along the entire length of the pipe. The pipe should not rest on the bells at the end and therefore it may become necessary to excavate for the pipe bells. Any pipe which is not in alignment or which shows any undue settlement after laying shall be removed and relaid at the Contractor's expense.

Multiple installation of HDPE pipe shall be laid with the centerlines of individual barrels parallel. Unless otherwise indicated on the plans, the minimum clear distance between the outer surfaces of adjacent pipes shall be equal to 24 inches (600 mm).

6.6 *Reuse of Existing Appurtenances* – When exiting appurtenances are specified on the plans for reuse, the portion to be reused shall be severed from the existing culvert and moved to new position previously prepared, by approved methods.

Connections shall conform to the requirements for joining sections of pipes as indicated herein or as shown on the plans. Any headwalls and any aprons or pipe attached to the headwall that are damaged during moving operations shall be restored to their original condition at the Contractor's expense. The Contractor, if he so desires, may remove and dispose of the existing headwalls and aprons and construct new headwalls at his own expense, in accordance with the pertinent specifications and design indicated on the plans or as furnished by the Engineer.

6.7 *Sewer Connections and Stub Ends* – Connections of pipe sewer to existing sewers or sewer appurtenance shall be as shown on the plans or as directed by the Engineer. The bottom of the existing structure shall be mortared or concreted if necessary, to eliminate any drainage pockets created by the new connection.

Where the sewer is connected into existing structures which are to remain in service, any damage to the existing structure resulting from making the connection shall be restored by the Contractor to the satisfaction of the Engineer. Stub ends, for connections to future work not shown on the plans, shall be sealed by installing watertight plugs into the free end of the pipe.

6.8 *Backfilling* - Backfill from the pipe bedding up to 1 ft (300 mm) above the top of the pipe is critical for the successful performance of the pipe. It provides necessary structural support to the pipe and controls pipe deflection. Therefore, special care should be taken in the placement and compaction of the backfill material. Special emphasis should be placed upon the need for obtaining uniform backfill material and uniform compacted density throughout the length of the pipe so that unequal pressure will be avoided. Extreme care should be taken to insure proper backfill under the pipe in the haunch zone.

Backfill material shall meet the following specifications.

Type I - Backfill consists of Special Specification Item 4005, "Flowable Backfill." The flowable backfill shall be placed across the entire width of the trench and shall maintain a minimum depth of 1ft (300 mm) above the pipe. A minimum of 24 hours shall elapse prior to backfilling the remaining portion of the trench with other backfill material in accordance with Item 400, "Excavation and Backfill for Structures."

Type II – Backfill consists of granular material that meets the gradation requirements of Type B, C, D or F aggregate mixtures in Item 334, "Hot Mix-Cold Laid Asphalt Concrete Pavements" and Item 340, "Hot Mix Asphalt Concrete Pavements." The backfill material shall be placed evenly and simultaneously on both sides of the pipe to not less than 1 ft (300 mm) above the top of the pipe. The backfill shall be placed in uniform layers not exceeding 8 inches (200mm) of thickness (loose measurement), wetted if required, and thoroughly compacted between the pipe and the side of the trench. Until a minimum cover of 1 ft (300 mm) is obtained, only hand operated tamping equipment will be allowed within vertical planes 2 ft (600 mm) beyond the horizontal projection of the outside surfaces of the pipe.

In the selection of appropriate backfill material, consideration should also be given to possible migration of fines from adjacent native soil materials into the backfill. Where potential for such migration exists, separation geotextiles that meet the requirements of ASSHTO M 288 Section 7 shall be installed between the native soil and the backfill.

6.9 *Protection of Pipe* – No heavy construction equipment, such as earth hauling equipment shall be permitted to traverse the pipe trench until a minimum depth of cover above the pipe has been established. Unless otherwise specified on the plans, the minimum depth of cover shall consist of fill compacted to a depth of at least one pipe diameter above the pipe.

Prior to adding each new layer of loose backfill material, until a minimum of 1 ft (300 mm) of cover is obtained, an inspection will be made of the inside periphery of the structure for local or unequal deformation caused by improper construction methods. Evidence of such will be reason for such corrective measures as directed by the Engineer.

Pipe damaged by the Contractor shall be removed and replaced at no additional cost to the State.

7. Measurement. This item will be measured by the linear foot (meter). Such measurements will be made between the ends of the barrel along its flow line, exclusive of safety end treatments. Safety end treatments shall be measured in accordance with item 467, "Safety End Treatment". Where spurs, branches or connections to existing pipe lines are involved, measurement of the spur or new connecting pipe will be made from the intersection of its flow line with the outside surface of the pipe into which it connects. Where inlets, headwalls, catch basins, manholes, junction chambers, or other structures are included in lines of pipe, that length of pipe tying into the structure wall will be included for measurement but no other portion of the structure length or width will be so included.

For multiple pipes, the measured length will be the sum of the lengths of the barrels, measured as prescribed above.

This is a plans quantity measurement Item and the quantity to be paid for will be that quantity shown in the proposal and on the "Estimate and Quantity" sheet of the contract plans, except as modified by Article 9.8. If no adjustment of quantities is required additional measurements or calculations will not be required.

Flowable backfill will not be measured, but considered subsidiary to this item.

9. Payment. The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit price bid for "HDPE Pipe (Type I backfill)" of the type (if required) and size specified or "HDPE Pipe (Type I or II backfill)" of the type (if required) and size specified. This price shall be the full compensation for furnishing, hauling, placing and joining of pipes; for all connections to new or existing structures; for moving and reusing headwalls where required, for removing and disposing of portions of existing structures as required; for the bedding and Type I or II backfill material as required, for cutting of pipe ends on skew; and for all labor, tools, equipment and incidentals required to complete the work.

Excavation and backfill above the Type I or II backfill will be paid for in accordance with Item 400, " Excavation and Backfill for Structures".

Safety end treatment will be paid for in accordance with Item 467, "Safety End Treatment".

APPENDIX B

TYPES OF BACKFILL MATERIAL ECONOMICALLY AVAILABLE IN THE DISTRICTS OF TEXAS

APPENDIX B: Types of Backfill Material Economically Available in the Districts of Texas

	Backfill	Unit Price
Abilene	[1]Item 340 Type C (with asphalt)	$71.50 per Mg
	[1]Item 340 Type D (with asphalt)	$44.00 per Mg
Amarillo	Flowable backfill	$40 per ton
	Granular backfill	$20 per ton
	Item 340 Type B,C,D,F(with asphalt)	$38 per ton
[2]Atlanta	River gravel (round)	$6 per ton
	River gravel (crushed)	$8 per toin
	Coarse component of Item 340 Type B, C, D	$8 per ton
	Fine component of Item 340 Type B,C,D	$6 per ton
	Item 330, Type D	$33.30 per ton
Austin	Concrete fine aggregate	$6 per ton
	1 inch Pea gravel	$7.50 per ton
	Item 340 Type B, C, D (with asphalt)	$35 per ton
	Item 340 Type B, C, D (without asphalt)	$14 per ton
Beaumont	No information	
Brownwood	Item 340 Type B or Item 3116	$36.00 per ton, Aggregate only is $20
	Item 340 Type D or Item 3116	$46.00 per ton, Aggregate only is $20
	Item 340 F, Aggregate only	$20.00 per ton
Childress	No information available	
Corpus Christie	Sand	$12.5 per CY, $9.25 per ton
	Cement stabilized sand	$35 per CY, $25.9 per ton
	Item 340 Type B (with asphalt)	$37 per ton
	Item 340 Type C, D (with asphalt)	$35 per ton
	Item 330 Type AA	$35 per ton
	Item 330 Type C	$33 per ton
El Paso	2 sack cement	$40 per CY
	Pea gravel (used for pipe bedding)	$15 per CY
	Shredded tire 3 inch and 1 inch	------------

Fort Worth	M.S.E Retaining wall backfill specs 423.2+5% cement	
Houston	Crushed concrete	
	Recycled iron-core	
	Recycled asphalt pavement (R.A. V)	
	Recycled existing base from project(similar to Item 330 Type AA)	
Laredo	Item 247 flexible base	$20 per ton
	2 sack cement	
Lubbock	Special spec 3022 QCQA HMA Type B	$35 per ton
	Special spec 3022 QCQA HMA Type C	$40 per ton
Lufkin	Cement stabilized material	$99.67 per cubic meter
Odessa	Cement stabilized material (2 sack)	$100 per cubic meter
	CMHB 3022 (with asphalt)	$38 per cubic meter
Paris	No information	
Pharr	Silecious River Gravel	$6 per ton ($10 per ton with hauling)
	Item 340 B, C, D, F (with asphalt)	$25-$30 per ton
San Angelo	Cement stabilized embankment	$15 per ton
Tyler	Flexible base	$20 per ton
	Select fill	$5 per CY
	Cement stabilized backfill	$55 per CY
	Item 340 B, C, D, F (aggregate only)	$35 per ton
Waco	Pea gravel (readily available)	$7 per ton
	Crushed limestone (not readily available)	$12 per ton
	Item 340 Type B, C, D, F(aggregate only)	$10-$12 per ton
	Item 330	$40 per ton
Wichita Falls	Item 340(with asphalt) without delivery	$24 per ton
	Item 340 (aggregate only, not blended) without delivery	$12.5 per ton
Yoakum	Flexible base	$12 per ton
	Cement stabilized sand	$25 per CY

Item 340 BCDF (without aggregate)	$15 per ton
Item 330	$33 per ton

[1]Prices cannot be estimated accurately as backfill is subsidiary to the price of the pipe.
[2]No known aggregate source in district

APPENDIX C

PROPOSED FINAL SPECIFICATION FOR THE INSTALLATION OF HDPE PIPE
(November 7, 2000)

PROPOSED FINAL SPECIFICATIONS

HIGH DENSITY POLYETHYLENE (HDPE) PIPE FOR GRAVITY FLOW DRAINAGE APPLICATIONS

1. Description. This specification shall govern for the furnishing and installing of all 18 in (450 mm) to 48 in (1200mm)[1] high density polyethylene (HDPE) pipe used in the construction of thermoplastic pipe culverts, sewer mains, laterals, stubs and inlet leads. The pipes shall be of the sizes, types, design and dimensions shown on the plans and shall include all connections and joints to new or existing pipes, sewer, manholes, inlets, headwalls and other appurtenances as may be required to complete the work.

2. Materials. Unless otherwise specified on the plans or herein, the HDPE pipes and fittings used for gravity flow drainage applications shall conform to the following specifications.

2.1 High density polyethylene pipes and fittings shall meet the requirements as in AASHTO M 294-98 (for pipes up to 48 inches/1200mm in diameter).

2.2 *Raw Materials* - The pipes and the fittings shall be manufactured from virgin PE compounds which conform to the requirements of cell class 335420C as defined and described in ASTM D 3350, except that carbon black content shall not exceed 5 percent.

2.3 *Designation of Type* – The HDPE pipes used for gravity flow drainage applications shall be of Type S (outer corrugated wall with smooth inner liner) or Type D (inner and outer smooth walls braced circumferencially or spirally with projections or ribs).

2.4 *Section Properties* - Minimum wall thickness of the inner walls of Type S pipe and inner and outer walls of Type D pipe shall be as specified in Section 7.2.2 of the AASHTO M 294-98. The pipe stiffness at 5% deflection, when determined in accordance with ASTM designation D 2412, shall be as specified in Section 7.4 of AASHTO M 294-98.

The manufacturer shall perform appropriate test procedures on representative samples of each type of pipe furnished, and hence verify that the pipe complies with the specifications. A certificate of compliance shall be prepared and submitted to the Department for review and approval. It shall include the following information: manufacturing plant, date of manufacture, pipe unit mass, material distribution, pipe dimensions, water inlet area, pipe stiffness, pipe flattening, brittleness, environmental stress crack resistance, and workmanship.

3. Inspection. The quality of the materials, the process of manufacture, and the finished pipe shall be subject to inspection and approval by the Engineer at the manufacturing plant. In addition, the finished pipe shall be subject to further inspection by the Engineer at the project site prior to and during installation.

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[1] Nominal pipe size is the nominal inside diameter of the pipe

4. Marking. All pipe shall be clearly marked at intervals of not more than 12 ft (3.5 m), and fittings and couplings shall be clearly marked as follows:

4.1 Manufacturer's name or trade mark
4.2 Nominal size
4.3 Specification designation (e.g. M 294-98)
4.4 Plant designation code
4.5 Date of manufacture

5. Joints. Joints shall be installed such that the connection of pipe sections will form a continuous line free from irregularities in the flow line. Joints shall meet the soiltightness definition in accordance with AASHTO Section 26.4.2.4. Suitable joints are the following:

5.1 *Integral Bell-N-Spigot* – The bell shall overlap a minimum of two corrugations of the spigot end when fully engaged. The spigot end shall have an O-Ring gasket that meets ASTM F 477: Specifications for Elastomeric Seals (Gaskets) for Joining Plastic Pipe.

5.2 *Exterior Bell-N-Spigot* – The bell shall be fully welded to the exterior of the pipe and overlap the spigot end so that flow lines and ends match when fully engaged. The spigot end shall have an O-Ring gasket that meets ASTM F 477: Specifications for Elastomeric Seals (Gaskets) for Joining Plastic Pipe.

6. Construction Methods. The location of private driveway and side road pipe shall be constructed at locations shown on the plans or as directed by the Engineer.

6.1 *Excavation* – All excavation shall be in accordance with the requirements of Item 400, "Excavation and Backfill for Structures."

The width of the trench for pipe installation shall be sufficient, but no greater than necessary, to ensure working room to properly and safely place and compact haunching and other embedment materials. The space between the pipe and trench wall must be wider than the compaction equipment used in the pipe zone.

When Type I backfill (*See section 6.8 below*) is used, the minimum trench width is the pipe outside diameter plus 12 inches (300 mm).

When Type II or Type III backfill (*See section 6.8 below*) is used, the minimum trench width shall be as specified in Table 1. The contractor can use any trench width above the pipe zone.

6.2 *Installation in Embankment* - If any portion of the pipe projects above the existing ground level, an embankment shall be constructed as shown in the plans or as directed by the Engineer for a distance outside each side of the pipe location of not less than five times the diameter and to a minimum elevation of 2 ft (0.6 m) above the top of the pipe. The trench shall then be excavated to a width as specified in section 6.1 above.

6.3 *Shaping and Bedding* – The pipe shall be bedded in a foundation of compacted granular material that is free of organic matter, clay lumps, and other deleterious matter. Such bedding material shall meet the gradation requirements shown in Table 2. This material shall extend a minimum of 6 inches (150 mm) below the outermost corrugations or ribs and shall be carefully and accurately shaped to fit the lowest part of the pipe exterior for at least ten percent (10%) of the overall height. When requested by the Engineer, the Contractor shall furnish a template for each size and shape of the pipe to be placed for use in checking the shaping and bedding. The template shall consist of a thin plate or board cut to match the lower half of the cross section of the pipe.

6.4 *Handling and Storage* – Handling and Storage of HDPE pipe shall be in accordance with the pipe manufacturer's instructions. Proper facilities shall be provided for hoisting and lowering the pipe into the trench without damaging the pipe or disturbing the bedding or the walls of the trench.

6.5 *Laying Pipe* – Unless otherwise authorized by the Engineer, the laying of pipe on the bedding shall be started at the outlet (or downstream) end and shall proceed toward the inlet (or upstream) end with separate sections firmly joined together. The pipe should be laid in conformity with the established line and grade and shall have a full, firm and even bearing at each joint and along the entire length of the pipe. The pipe should not rest on the bells at the end and therefore it may become necessary to excavate for the pipe bells. Any pipe which is not in alignment or which shows any undue settlement after laying shall be removed and relaid at the Contractor's expense.

Multiple installation of HDPE pipe shall be laid with the centerlines of individual barrels parallel. Unless otherwise indicated on the plans, the minimum clear distance between the outer surfaces of adjacent pipes shall be equal to 24 inches (600 mm).

6.6 *Reuse of Existing Appurtenances* – When exiting appurtenances are specified on the plans for reuse, the portion to be reused shall be severed from the existing culvert and moved to new position previously prepared, by approved methods.

Connections shall conform to the requirements for joining sections of pipes as indicated herein or as shown on the plans. Any headwalls and any aprons or pipe attached to the headwall that are damaged during moving operations shall be restored to their original condition at the Contractor's expense. The Contractor, if he so desires, may remove and dispose of the existing headwalls and aprons and construct new headwalls at his own expense, in accordance with the pertinent specifications and design indicated on the plans or as furnished by the Engineer.

6.7 *Sewer Connections and Stub Ends* – Connections of pipe sewer to existing sewers or sewer appurtenance shall be as shown on the plans or as directed by the Engineer. The bottom of the existing structure shall be mortared or concreted if necessary, to eliminate any drainage pockets created by the new connection. Where the sewer is connected into existing structures which are to remain in service, any damage to the existing structure resulting from making the connection shall be restored by the Contractor to the satisfaction of the Engineer. Stub ends, for connections to future work not shown on the plans, shall be sealed by installing watertight plugs into the free end of the pipe.

6.8 *Backfilling* - Backfill from the pipe bedding up to 1 ft (300 mm) above the top of the pipe is critical for the successful performance of the pipe. It provides necessary structural support to the pipe and controls pipe deflection. Therefore, special care should be taken in the placement and compaction of the backfill material. Special emphasis should be placed upon the need for obtaining uniform backfill material and uniform compacted density throughout the length of the pipe so that unequal pressure will be avoided. Extreme care should be taken to insure proper backfill under the pipe in the haunch zone.

Backfill material shall meet the following specifications.

Type I - Backfill consists of Special Specification Item 4438, "Flowable Backfill." The flowable backfill shall be placed across the entire width of the trench and shall maintain a minimum depth of 1ft (300 mm) above the pipe. A minimum of 24 hours shall elapse prior to backfilling the remaining portion of the trench with other backfill material in accordance with Item 400, "Excavation and Backfill for Structures."

Type II - Backfill consists of Specification Item 400.6, "Cement Stabilized Backfill." Cement Stabilized Backfill shall be placed and compacted to ensure that all voids are filled completely.

Type III – Backfill consists of hard, durable, clean granular material that is free of organic matter, clay lumps, and other deleterious matter. Such backfill shall meet the gradation requirements shown in Table 2. The backfill material shall be placed evenly and simultaneously on both sides of the pipe to not less than 1 ft (300 mm) above the top of the pipe. The backfill shall be placed in uniform layers not exceeding 8 inches (200mm) of thickness (loose measurement), wetted if required, and thoroughly compacted between the pipe and the side of the trench. Until a minimum cover of 1 ft (300 mm) is obtained, only hand operated tamping equipment will be allowed within vertical planes 2 ft (600 mm) beyond the horizontal projection of the outside surfaces of the pipe.

In the selection of appropriate backfill material, consideration should also be given to possible migration of fines from adjacent native soil materials into the backfill. Where potential for such migration exists, separation geotextiles that meet the requirements of TxDOT Material Specification D9-6200, Type I shall be installed between the native soil and the backfill. Consideration shall also be given to potential for flooding and erosion of backfill. Granular backfill must be properly confined by using rip rap and other suitable end treatment where flooding is anticipated.

6.9 *Protection of Pipe* – No heavy construction equipment with axle loads equal to or larger than 30-kips shall be permitted to traverse the pipe trench. If the passage of such heavy construction equipment over an installed pipeline is necessary during construction, compacted fill in the form of a ramp shall be constructed to as per minimum cover requirements specified in construction plans.

Prior to adding each new layer of loose backfill material, until a minimum of 1 ft (300 mm) of cover is obtained, an inspection will be made of the inside periphery of the structure for local or unequal deformation caused by improper construction methods. Evidence of such will be reason for such corrective measures as directed by the Engineer.

Pipe damaged by the Contractor shall be removed and replaced at no additional cost to the State.

7. Measurement. This item will be measured by the linear foot (meter). Such measurements will be made between the ends of the barrel along its flow line, exclusive of safety end treatments. Safety end treatments shall be measured in accordance with item 467, "Safety End Treatment". Where spurs, branches or connections to existing pipe lines are involved, measurement of the spur or new connecting pipe will be made from the intersection of its flow line with the outside surface of the pipe into which it connects. Where inlets, headwalls, catch basins, manholes, junction chambers, or other structures are included in lines of pipe, that length of pipe tying into the structure wall will be included for measurement but no other portion of the structure length or width will be so included.

For multiple pipes, the measured length will be the sum of the lengths of the barrels, measured as prescribed above.

This is a plan quantity measurement Item and the quantity to be paid for will be that quantity shown in the proposal and on the "Estimate and Quantity" sheet of the contract plans, except as modified by Article 9.8. If no adjustment of quantities is required additional measurements or calculations will not be required.

Flowable backfill will not be measured, but considered subsidiary to this item.

9. Payment. The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit price bid for "HDPE Pipe (Type I backfill)" of the type (if required) and size specified or "HDPE Pipe (Type I, II or III backfill)" of the type (if required) and size specified. This price shall be the full compensation for furnishing, hauling, placing and joining of pipes; for all connections to new or existing structures; for moving and reusing headwalls where required, for removing and disposing of portions of existing structures as required; for the bedding and Type I, II or III backfill material as required, for cutting of pipe ends on skew; and for all labor, tools, equipment and incidentals required to complete the work.

Excavation and backfill above the Type I, II or III backfill will be paid for in accordance with Item 400, " Excavation and Backfill for Structures".

Safety end treatment will be paid for in accordance with Item 467, "Safety End Treatment".

TABLE 1. Minimum Trench Width

Nominal Pipe Diameter		Minimum Trench Width	
inches	mm	inches	mm
18	450	44	1100
24	600	54	1350
30	750	66	1650
36	900	78	1950
42	1050	84	2100
48	1200	90	2250

TABLE 2. Gradation Requirements for Type III Backfill Material

Sieve No.	Percent Retained (Cumulative)
1 inch	0-5
$^{7}/_{8}$ inch	0-35
½ inch	0-75
$^{3}/_{8}$ inch	0-95
No. 4	35-100
No.10	50-100
No.200	90-100

Note: Material that qualify under the following TxDOT specifications may meet the gradation requirements specified in this table.

1. ITEM 247: FLEXIBLE BASE, Grades 1, 4 and 5.
2. ITEM 421: PORTLAND CEMENT CONCRETE, Coarse Aggregate Grades 4, 5, 6, 7 and 8.
3. ITEM 556: PIPE UNDERDRAINS, Filter Material Type B

APPENDIX D

WORK BREAK DOWN STRUCTURE

Table D.1. Itemizing major activities

Item number/ alternatives	Item	Equipment / Heavy construction vehicle	Unit	Per hour output	Crew	Man hour per unit	Installation cost =workers wage per unit + equipment cost per unit (in $)	Material Price	Total cost per unit overhead not included ($/unit)
1	Trench Excavation								
1.1	Trench Excavation with Backhoe of ¾ CY bucket Capacity. Spoil is piles adjacent to the pile.	Crawler mounted hydraulic backhoe with ¾ CY bucket at $41 per hour			S1 crew: 1 laborer and one tractor operator at $35.41 per man-hour				
1.1.1	Light soil		CY	33CY		S1@.061			
1.1.2	Medium soil		CY	27CY		S1@.074	2.16+1.25	-	$3.41
1.1.3	Heavy or wet soil		CY	22CY		S1@.091	2.62+1.52	-	$4.14
1.1.4	Loose rock		CY	18CY		S1@.112	3.22+1.87	-	$5.09
							3.97+2.30	-	$6.27
1.2	Trench Excavation with Backhoe of 1 CY bucket Capacity. Spoil is piles adjacent to the pile.	Crawler mounted hydraulic backhoe with 1 CY bucket at $46 per hour			S1 crew: 1 laborer and one tractor operator at $35.41 per man-hour				
1.2.1	Light soil		CY	65CY		S1@.031	1.1+.71	-	1.81
1.2.2	Medium soil		CY	53CY		S1@.038	1.35+ .87	-	2.22
1.2.3	Heavy or wet soil		CY	43CY		S1@.047	1.66+1.08	-	1.76
1.2.4	Loose rock		CY	37CY		S1@.055	1.95+1.27	-	3.22
1.3	Trench Excavation with Backhoe of 1.5 CY bucket Capacity. Spoil is piles adjacent to the pile.	Crawler mounted hydraulic backhoe with 1.5 CY bucket at $56 per hour			S1 crew: 1 laborer and one tractor operator at $35.41 per man-hour			-	
1.3.1	Light soil		CY	83 CY		S1@.024	.85+.67	-	1.52
1.3.2	Medium soil		CY	70 CY		S1@.029	1.03+.81	-	1.84
1.3.3	Heavy or wet soil		CY	57 CY		S1@.035	1.24+.98	-	2.22
1.3.4	Loose rock		CY	48 CY		S1@.042	1.49+1.18		2.67

Table D.1 (Continued)

1.4	Trench Excavation with Backhoe of 2 CY bucket Capacity. Spoil is piles adjacent to the pile.	Crawler mounted hydraulic backhoe with 2 CY bucket at $46 per hour			S1 crew: 1 laborer and one tractor operator at $35.41 per man-hour				
1.4.1	Light soil		CY	97CY		S1@.021	.74+.91	-	1.65
1.4.2	Medium soil		CY	80CY		S1@.025	.89+1.08	-	1.97
1.4.3	Heavy or wet soil		CY	65CY		S1@.031	1.10+1.33	-	2.43
1.4.4	Loose rock		CY	55CY		S1@.036	1.27+1.55	-	2.82
1.5	Trench Excavation with Backhoe of 2.5 CY bucket Capacity. Spoil is piles adjacent to the pile.	Crawler mounted hydraulic backhoe with 2.5 CY bucket at $46 per hour			S1 crew: 1 laborer and one tractor operator at $35.41 per man-hour				
1.5.1	Light soil		CY	122 CY		S1@.016	.57+.84	-	1.41
1.5.2	Medium soil		CY	100 CY		S1@.020	.71+1.05	-	1.76
1.5.3	Heavy or wet soil		CY	82 CY		S1@.024	.85+1.26	-	2.11
1.5.4	Loose rock		CY	68 CY		S1@.029	1.03+1.52	-	2.55
2	Trench Box Renting (labor cost and other equipment cost not included)	Comparatively high capacity Backhoe with bucket size ranging from 1-1/2CY to 3CY							
2.1	Each 16 ft long 6 ft deep trench box		Day		Tractor operator and laborer				100
2.2	Each 20 ft long 10 ft deep trench box		Day		Tractor operator and laborer				150

Table D.1 (Continued)

2.3	Each 12 ft long 8 ft deep trench box		Day		Tractor operator and laborer				129
2.4	Each 12 ft long 8 ft deep trench box		Week		Tractor operator and laborer				494
2.5	Each 16 ft long 8 ft deep trench box		Day		Tractor operator and laborer				153
2.6	Each 16 ft long 8 ft deep trench box		Week		Tractor operator and laborer				460
3	Trench Bottom Preparation							Backfill material.	
3.1	Prepare 6" sand and gravel bedding (material that meets the gradation requirement for final backfilling as recommended in the revised specification)	Wheel loader 55hp at $22 per hour	CY	80CY	S1 crew: 1 laborer and 1 tractor operator at $35.41 per man-hour	S1@.025	025*35.41+22/80=$1.17	16.5	17.67
3.2	Prepare 6" sand and gravel bedding (material that meets the gradation requirement for final backfilling as recommended in the revised specification)	3/4CY Wheel loader at $16 per hour.	CY	80CY	B8 crew: 1 laborer, 1 operating engineer at $24.67 per hr	B8@.025	24.67*.025+16/80= .81	16.5	17.32
3.3	Compact the bedding with vibratory plate compactor or tamping rammer to ensure an stable foundation	Vibratory plate compactor, tamping rammer							

Table D.1 (Continued)

3.4	Trim the trench bottom in a concave shape to a height of 1/10 of pipe diameter as recommended in the draft specification.	A curved template might be used to obtain a concave shaped trench bottom	SF	100 SF	Crew type:CL:1 laborer at $30.6 per man-hour	CL@.01	.01*30.6+ 0/200=$.31	0	$0.31
4	Lifting, Laying and joining of 24 in HDPE pipe	Wheel mounted ½ CY backhoe at $21.00 per hr for lifting and placing of the pipe	LF	20 ft	Crew type: U1: 1 plumber 2 laborers I tractor Operator At $35.96 Per man-hour	U1@.212	.212*35.96+21 /20=7.62+1.1= $8.72	37.7	46.43
5	Placement and compaction of backfill material:								
5.1	Backfill with well-graded granular backfill material in conformance with the specification	Wheel loader 55 hp at $22 per hour	CY	80 CY	S1 crew : 1 laborer and one tractor operator at $35.41 per man hour	S1@.025	.025*35.41+22 /80=.89+ .28= $1.17	16.5	17.67
5.2	Backfill trenches from loose material piles adjacent to trench (soil previously excavated)	Wheel loader 55 hp at $15 per hour	CY	50 CY	B8 crew : 1 laborer and one operating engineer at $24.67 per man hour	B8@0.04	24.67*.04+ 15/50=1.28	0	1.28
5.3	Backfill trenches from loose material piles adjacent to trench (soil previously excavated)	Wheel loader 55 hp at $22 per hour	CY	50 CY	S1 crew : 1 laborer and one tractor operator at $35.41 per man hour	S1@0.04	35.41*.04+ 22/50=1.84	0	1.84

Table D.1 (Continued)

5.4	Backfill trenches from loose material piles adjacent to trench (soil	¾ CY crawler loader at $23 per hr	CY	33 CY	S1 crew : 1 laborer and one tractor operator at	S1@.061	35.41*.061+ 23/33=2.86	0	2.86

	previously exacavted) ¾ CYcrawler loader				$35.41 per man hour				
5.5	Compaction of soil in trenches in 8 in layer Pneumatic tempers	Pneumatic tampers at $5.7 per hr	CY	40 CY	Crew type: BL: I laborer at $21.08 per hour	BL@.05	1.05+.19	0	1.24
5.6	Compaction of soil in trenches in 8 in layer Vibrating rammers	Vibrating rammers at $5 per hr	CY	20 CY	Crew type: BL: I laborer at $30.60 per hour	BL@.10	2.11+.61	0	2.72

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