EFFECT OF CONCENTRATED LOADS ON SHALLOW BURIED POLYVINYL CHLORIDE AND POLYETHYLENE TUBING

by

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ABSTRACT

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Submitted to the Department of Civil Engineering on June 27, 1975 in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

This thesis presents a study of the behavior of PVC and corrugated PE plastic pipe under simulated wheel loads. Data is presented on pipe deflections, soil-pipe interface pressures, and soil strains for pipe buried with 6, 12 and 18 inches of soil cover and surface loaded with 10,000 pounds on a 10 inch diameter plate. In 5 of the tests the PVC pipe was instrumented with strain gages on the inside and outside of the pipe at the crown, invert and springlines. The test results are compared with an elastic theory for deeply buried pipe to observe if the elastic predictions can be used as a design basis for the shallow burial-concentrated load problem.

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CHAPTER 1

INTRODUCTION

Over the past twenty years plastic pipe has seen increasing usage in a wide variety of applications. Among these applications is drainage of roadways, both in the form of storm sewers, and underdrains. Plastic pipes are light, flexible, and, since they are manufactured by extrusion processes, can be produced in any reasonable length desired. These factors help in reducing construction costs by reducing the manpower and equipment needed to place individual lengths, and by reducing the number of field joints. Plastic piping systems can also be desirable due to their resistance to the corrosive effects of many fluids, which would shorten the life of other pipe materials.

Structurally, buried pipes must be designed for two types of load after installation, earth loads, which are simply the weight of soil bearing on the pipe; and live loads, which are primarily wheel loads imposed from the surface. Soil pressures due to wheel loads are generally insignificant for smaller pipes buried over ten feet, but they increase very rapidly with decreasing burial depth until they are predominant at depths less than about four feet. During road construction, particularly highways, the weight of construction equipment rolling over pipe installations before backfilling is completed can create a more severe loading condition than the pipe will ever be subjected to again. To reduce these loads, ASTM specification D2321, for the installation of thermoplastic sewer pipes does not permit heavy equipment to travel over buried pipes until 2.5 feet of cover is placed and compacted over the crown of the pipe. If such provisions were removed or lessened the cost of road construction might be reduced by allowing contractors more freedom; however, culvert technology has not as yet provided a method to design for wheel loads and shallow burial. Until such a method is developed, these restrictions must be used to prevent damage to drainage systems.

The purpose of this thesis is to study the structural performance of buried plastic sewer and drain pipe under simulated wheel loads, and to establish the following:

1. An indication of minimum cover required to protect plastic pipe installations from construction equipment loads. This depth is governed by the magnitude of the load, the rate at which the load attenuates with increasing depth and the physical properties of the pipe and surrounding soil.

2. A rational design approach to treat the concentrated load problem.

Due to the multitude of types and shapes of plastic pipe available, it is necessary to restrict this thesis to two particular piping systems, polyvinyl chloride (PVC) and corrugated polyethylene (PE). Both of these systems are now being considered by many transportation departments for use in drainage of highways.

CHAPTER 2

THEORETICAL ANALYSIS

2.1 Material Properties

Structural analysis of plastic pipe must of necessity begin with a brief look at plastics and the processes that make them into pipes. Plastics are organic compounds whose properties are dependent upon molecular structure. PVC and PE are both thermoplastics, that is their properties vary with temperature. Figure 2.1 shows this variation for polyethylene. The effect on PVC is similar but less pronounced. Thermoplastics are manufactured into pipe form by 1) heating a resin, 2) forcing it through an extruder, 3) performing any molding operations necessary (corrugations, belled ends) and 4) cooling the finished product. The properties of the finished product are affected by such factors as the type and formulation of resin, the manufacturing process, and the processing temperature. The structural properties of plastics vary also with time, however, since this thesis is studying short term loads, only short term properties will be discussed.

Corrugated PE underdrain pipe is specified in ASTM standard F405-74 which in turn calls for material conformance with ASTM D1248. Some of the physical properties of PE from D1248 are shown in Table 2.1.

PVC sewer pipe is governed by several specifications (D2729, D3033, D3034). D2729 is for a thinwalled pipe, while D3033 and D3034 specify pipe with thicker walls and are almost identical. All three specifications refer to the material standard D1784 for the plastic specification. Some of the physical properties for PVC from D1784 are shown in Table 2.1.

TABLE 2.1 PHYSICAL PROPERTIES OF PE AND PVC

Property	PE (D1248)	PVC (D1784)
Tensile Strength (psi)	1800 - 3200	6000 - 7000
Elongation, min. (%)	100 - 500	40 - 80*
Brittleness Temp., max. (°C)	-75 to -60	-----
Impact Strength, min. (Izod) ft-lb/in. of notch	-----	0.65 - 1.5
Modulus of Elasticity tension (psi)	60,000-180,000*	360,000-500,000
Diameter (in.)	3 - 8	4 - 15

*Taken from Ref. 1, not specified by ASTM

Since both of these material specifications were primarily intended for pressure pipes, which are normally stressed mostly in tension, no compressive or flexural strengths are specified. PVC plastics typically have compressive strengths of about 9500 psi and flexural strengths of about 12,000 psi. PE plastics have a compressive strength of about 3100 psi and a flexural strength of about 5000 psi.

2.2 Buried Pipe Design

Historically the design of buried tubes (i.e., pipes, culverts, etc.) has been divided into two fields, one for "rigid" conduits such as concrete pipe and the other for "flexible" conduits such as corrugated metal culverts. Rigid conduits are designed to resist earth and live loads primarily by internal forces. This means that the main form of resistance is through bending moments, resulting in rather stiff sections and therefore the classification "rigid." Flexible conduits however are designed to deflect laterally under load in order to utilize the passive resistance of the surrounding soil. This type of design resists loads more by membrane action than by bending and results in much thinner sections. The major difficulty with these traditional culvert design practices is that they were developed empirically for specific types of pipe and do not provide an adequate design method for all buried pipes. That is, there is an intermediate range of stiffnesses for which neither design method is adequate, as is discussed below.

2.2.1 Soil-Structure Interaction

The traditional design methods mentioned classify installations as flexible or rigid solely from the bending stiffness of the pipe wall, called the "ring stiffness":

$$S = \frac{EI}{D^3}$$

where S = Ring stiffness (psi)

E = Modulus of elasticity of pipe material (psi)

I = Moment of inertia of pipe wall per unit length (in.^{4}/in.) with respect to ring bending

D = Mean diameter of pipe (in.)

In recent years, however, a better understanding of the soil structure interaction around pipes has brought into use a new parameter, the flexibility coefficient:

$$F = \frac{M_s}{S}$$

where F = Flexibility coefficient (dimensionless)

M_s = One-dimensional compression modulus of soil (psi),

(See below for discussion of this parameter.)

The flexibility coefficient is a more suitable parameter for the description of buried pipe installations because it shows that performance is governed by the relative stiffness of soil and pipe, rather than the absolute value of either one. That is, a given pipe installation could be considered either flexible or rigid depending upon the soil in which it is buried.

It has also been shown by Lew (3) and Selig (4), that the performance of a buried pipe can be influenced by the ring compressibility of the pipe. This can be the case in metal culverts where slip can occur in bolted joints, and in plastic pipes, which typically have low elastic moduli. The ring compressibility of a pipe is described by the compressibility coefficient:

$$C = \frac{M_s D}{EA}$$

where C = Compressibility coefficient (dimensionless)

A = Cross-sectional area of pipe wall per unit length (in.^{2}/in.)

Allgood (5) suggests that a system for which F is greater than 1000 be classified as flexible (in the traditional sense) and any system for which F is less than or equal to 10 be classified as rigid. The area in between is the aforementioned "intermediate" range. Considering that a typical value for the ring stiffness of PVC or PE pipe is about 1.0 psi, then by Allgood's classification any pipe buried in a soil with M_s less than 1000 psi would be in the intermediate range. This indicates that for plastic pipe it is desirable to find a new design method which can treat the entire range of pipe stiffness.

Both of the above coefficients use the one-dimensional modulus (M_s) as the parameter to describe soil stiffness. This replaces the soil parameter traditionally used in flexible conduit design, the modulus of soil reaction (E'). The modulus of soil reaction is used with the Iowa formula for determining flexible conduit deflections, but is highly empirical. Krizek (6) presented a number of attempts at correlating E' with other soil properties but these have never proved entirely satisfactory. The one-dimensional modulus however can be determined from a simple laboratory test. The procedure calls for determination of a stress-strain curve for the test soil in its natural state. The one-dimensional modulus is then selected as the slope of the secant from zero to the point on the curve which represents the calculated load. This procedure is shown in Figure 2.2. It is important to note that this procedure involves the assumption of linear stress-strain characteristics in the soil. This is adequate in general, however some investigators (6, 7) have introduced methods of treating the true non-linear behavior of soil.

Methods that are presently available for the analysis and design of buried pipe that can treat circumferential stiffness, as well as the intermediate ring stiffness range, fall into two general categories, closed form elasticity solutions, and numerical methods in the form of finite element analyses. Photoelastic and holographic interferometric (8) methods have also been used to analyze pipe problems, but these are not intended as design methods.

2.2.2 Finite Elements

Perhaps the most comprehensive method available for the analysis of buried pipes is the computerized finite element approach. Finite elements in their most complete form can be used to analyze almost any situation, such as concentrated loads, poor bedding, non-uniform soil or non-linear soil behavior. Some solutions have even been made to treat longitudinal effects along the length of the pipe. Katona et al. (7) are presently developing a three-level computer analysis/design

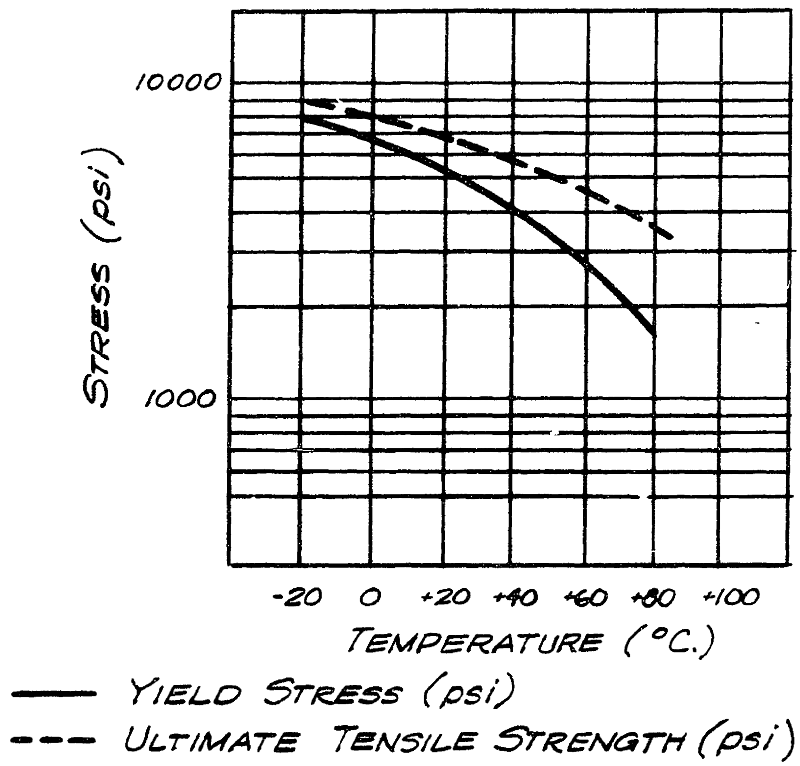


FIGURE 2.1 STRENGTH VARIATION WITH TEMPERATURE IN PE (FROM REF. 2)

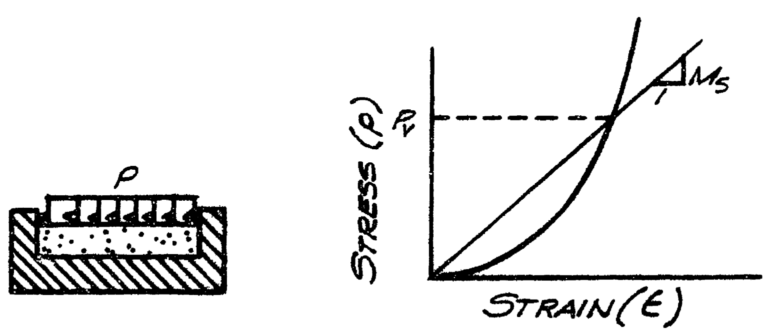


FIGURE 2.2 DETERMINATION OF ONE-DIMENSIONAL MODULUS

program for culverts called CANDE (Culvert Analysis and Design). Two of these levels use finite elements, one having a standard, internally defined mesh representing the system while the other allows the mesh to be completely defined by the user. The third level of this program is an elasticity solution by Burns and Richard which will be discussed in Section 2.2.3.

Although finite element analyses could provide such a thorough solution to the shallow burial-concentrated load problem, there are two major factors which make it less desirable as a design method:

1. Computerized solutions can frequently be time-consuming and expensive. In the case of small pipe, which is frequently governed by other criteria (e.g., handling), it is only rarely that one can justify this expense.

2. Any analysis is only as accurate as its input. The variability of soil compaction and bedding effects are extremely difficult to predict, and are highly variable along the length of a pipe. The result of this variability is that an engineer using the finite element method must either check numerous possible conditions, or else accept a sophisticated analysis using mostly estimated properties and conditions, and leaving as much doubt as to the accuracy of the solution as an approximate analysis.

In view of the above facts, finite element methods will not be considered further as a potential design method for shallow pipes of the type being considered here.

2.2.3 Elasticity Solutions

Most theoretical analyses of buried cylinders consider the soil as a linearly elastic, isotropic homogeneous medium with either a cylindrical inclusion (Burns and Richard (9), Höeg (10), Dar and Bates (11), or an equivalent ring of soil representing the cylinder (Richards and Agrawal (12)). The latter solution models the pipe wall as a ring of soil with a thickness calculated to give the model the same ring stiffness as the pipe. The problem with this solution is that the ring compressibility and ring flexibility cannot be modelled simultaneously. This method will not be discussed further.

The remaining analyses are all similar and only the Burns and Richard solution (which is presented briefly in Appendix A) will be referred to hereafter, as it is the most commonly quoted. This solution was intended for deeply buried cylinders but has been shown both theoretically (9) and in tests (10) to be accurate for depths of burial as small as one pipe diameter (for uniform loads), making it applicable in that respect to the shallow burial problem to be studied here. Drawbacks associated with the Burns and Richard analysis are:

1. This solution was developed for a uniform surface loading and needs to be modified to treat concentrated loads applied at the surface.

2. This solution considers the cylinder to be perfectly bedded in the soil and also considers the soil to be constant in its properties. Very few pipes, particularly small ones such as underdrains, are bedded perfectly into the supporting soil, and uniform soil conditions are rarely achieved in the field.

Despite its limitations, the Burns and Richard solution is a potential design method for the shallow burial-concentrated load problem because:

1. It treats all stiffness ranges of pipe.

2. It addresses the soil-structure interaction of buried pipe.

3. It is desirable to use the same design method for all phases of pipe design and the Burns and Richard solution is accepted by some as an adequate method for designing deeply buried pipes.

2.3 Previous Studies

Analyses of the shallow burial-concentrated load problem have been conducted by Anand (13) and Richards and Agrawal (12) using the finite element method. The Anand study modeled a concrete pipe with one pipe diameter of soil cover over the crown. The loading was a strip load, varying in width from 1 to 3 pipe diameters and located directly over the crown. The Richards and Agrawal analysis used a line load over the crown and varied the depth of cover from 1.2 to 2.0 times the pipe radius and the ratio of pipe material modulus to soil modulus from 1 to 600. Both studies modeled the soil as a linearly elastic material.

Figure 2.3, reproduced from the Anand paper, shows the variation in radial pressures on the pipe as the load width increases and also the Burns and Richard solution for a uniform load over the entire surface. For the narrowest loading condition, Anand found that there is a pressure concentration at the crown. As the load width increases, this concentration decreases, and the pressure distribution about the pipe becomes more uniform. The Burns and Richard solution predicts a nearly uniform pressure about the entire pipe.

Figure 2.4, taken from the Richards and Agrawal study shows the effect on the radial pressure distribution of varying the depth of cover over the pipe. There is a very high positive pressure concentration at the crown and a negative pressure for α between 30 and 90 degrees. The concentration is more pronounced than that in the Anand study because there is less cover over the pipe and the load is concentrated.

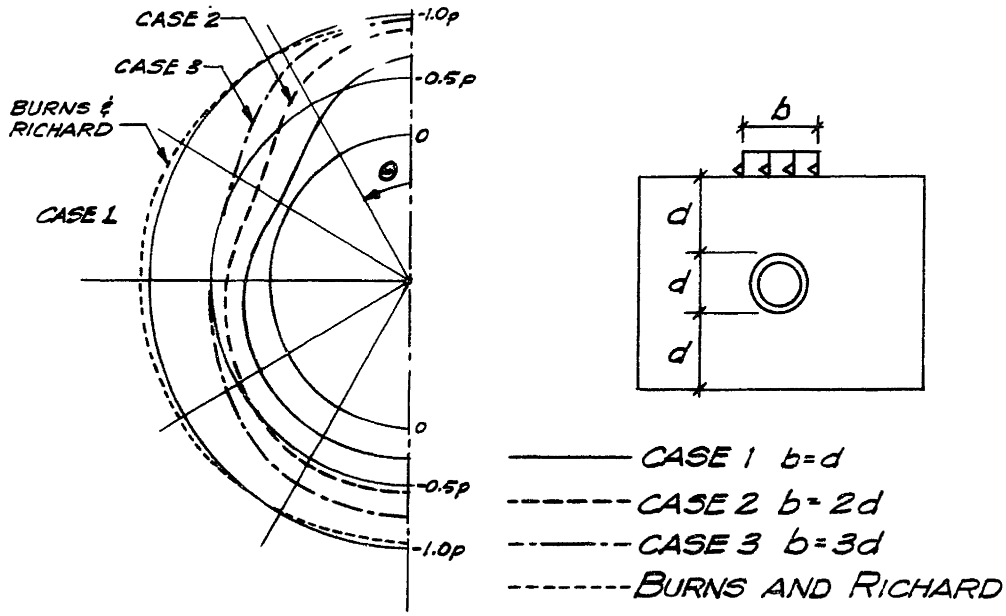


FIG. 2.3 RADIAL PRESSURES ON PIPE FOR VARIOUS LOAD WIDTHS ANAND (13)

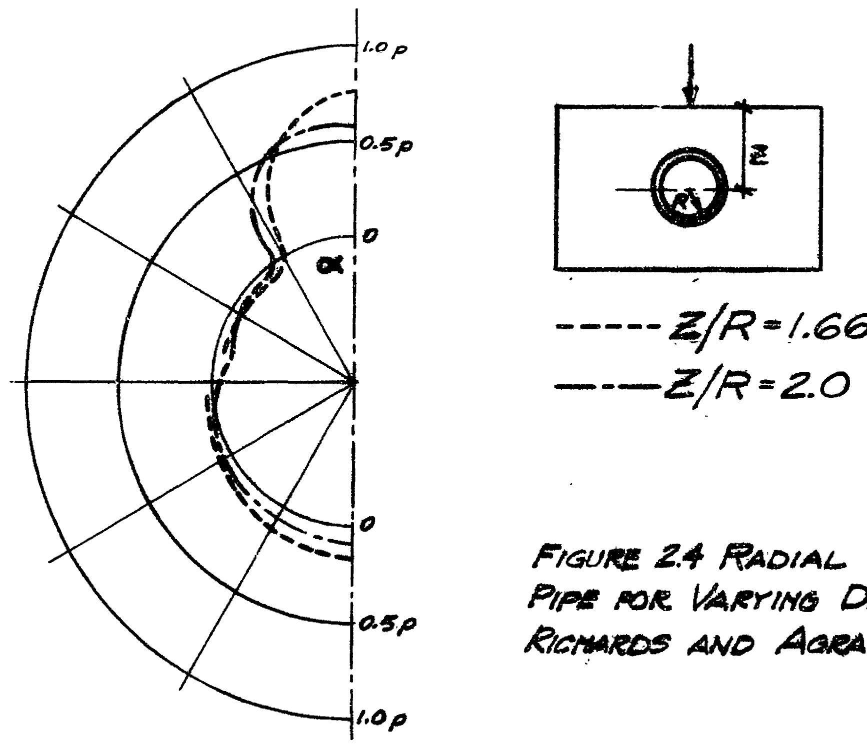


FIGURE 2.4 RADIAL PRESSURES ON PIPE FOR VARYING DEPTH OF COVER RICHARDS AND AGRAWAL (12)

The bending moments in the pipe of the Anand study, shown in Figure 2.5, are greatest at the crown, which is consistent with the pressure distribution. As the load width increases from 1 to 2 pipe diameters (i.e., twice the load) the moments also increase significantly, due to the additional surface load. However, as the load width increases from 2 to 3 pipe diameters, very little change in moments occurs, and, as the load width increases further, to the Burns and Richard solution, the moments actually reduce to less than those for Case 1. This behavior can be explained by considering the Poisson effect of a stress on an elastic material. That is, a normal stress on an element creates a lateral strain, as shown in Figure 2.6. In Anand's Case 1, the applied load is carried primarily by the pipe, such that the vertical soil stresses, adjacent to the pipe are low, and no lateral strains are created to resist pipe deformations. The situation is similar in Case 2 where the pipe carries more of the load and develops larger bending moments. The effect of increasing the load width further, however, produces a lesser increase in the load on the pipe, but does increase the vertical stresses in the soil. This increase in confining pressure produces a Poisson effect in the soil, which means greater lateral support for the pipe, and subsequently lower bending moments. This effect of increasing the load width, is even more pronounced in true soil, which has non-linear stress-strain characteristics. Soil at low confining pressures (e.g., a narrow load) has a low modulus and subsequently provides little lateral support for the walls of an embedded pipe.

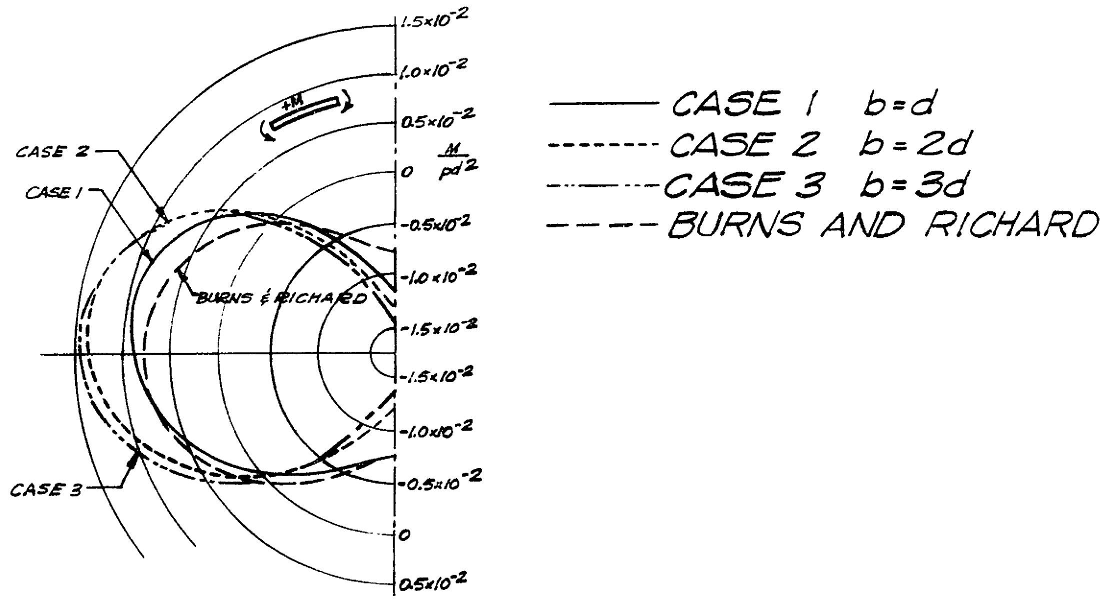


FIG. 2.5 HOOP BENDING MOMENT IN PIPE FOR VARIOUS LOAD WIDTHS ANAND (13)

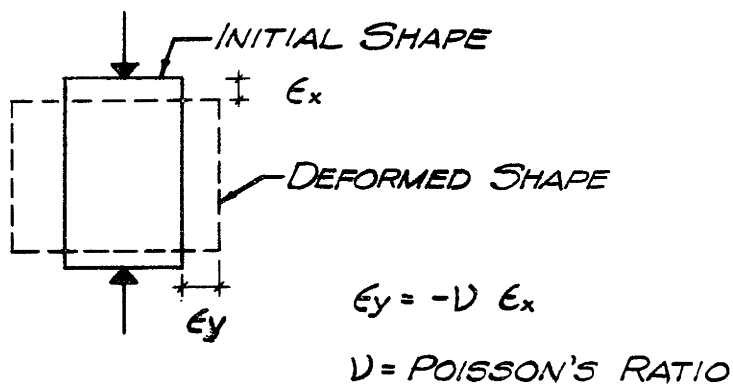


FIGURE 2.6 POISSON EFFECT IN AN ELASTIC MATERIAL

The above analysis suggests that a design method for the shallow burial-concentrated load problem should address two points:

1. Soil Modulus: As discussed in Section 2.2.1, deep culvert design typically assumes linear soil behavior and uses the secant modulus associated with the vertical load on the pipe to define soil stiffness. If the load is a concentrated wheel load, however, the soil behaves as if only the earth load were confining it. This means that the soil modulus selected should either ignore the effect of the live load, or consider only part of it, depending on the degree to which the load has attenuated.

2. Load Attenuation: Load attenuation with increasing depth has a significant effect not only on the load on the pipe, but also on the manner in which the soil reacts to pipe deformations. That is, as the load spreads laterally through the soil, it increases the confinement of the soil and hence the soil stiffness which aids the pipe in resisting load.

CHAPTER 3

TESTING PROGRAM

A small scale test program was developed to observe directly the behavior of plastic pipe under concentrated loads, and to determine the accuracy of elastic theory as discussed in Chapter 2. Besides the two types of plastic being considered two other variables were introduced:

1. Depth of burial: Tests were performed with 6, 12 and 18 inches of soil over the crown of the pipe. This produced a wide range of pipe behavior.

2. Soil Density: For each depth of cover, tests were performed with two soil densities, hereafter referred to as "loose" and "dense" respectively. By varying the soil density 8 to 10 pcf a wide range of soil stiffness was achieved. (As will be discussed later the "loose" soil condition is a good field condition while the "dense" soil condition is probably better than could be achieved in the field for the soil used in this study.)

3.1 Apparatus

The tests were performed in the 30-inch deep 35-inch diameter cylindrical steel tank shown in Figure 3.1. A frame was constructed over the tank to support the mechanical screw jack used to apply the concentrated load. Load was transferred

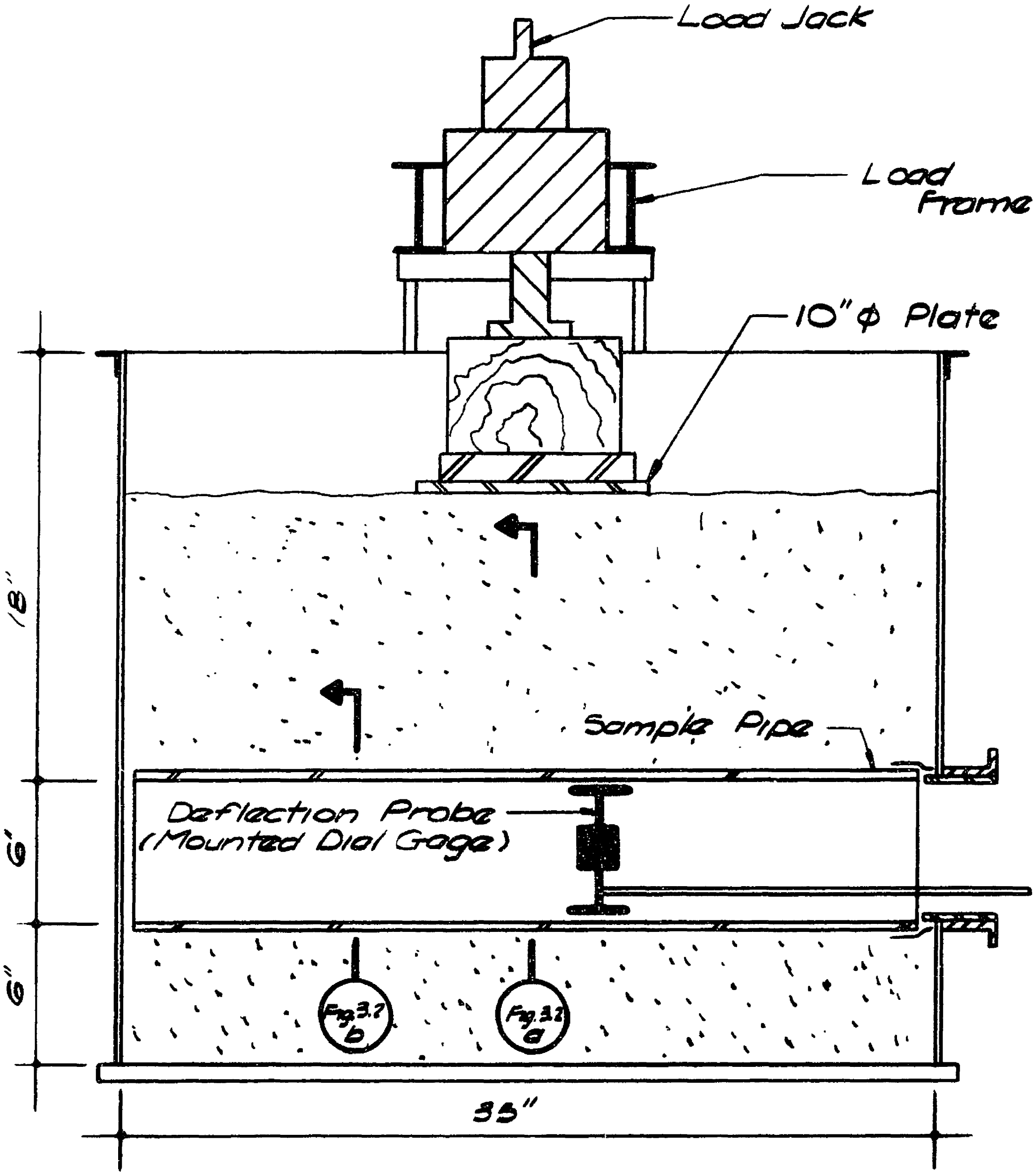


Figure 3.1 Cross Section Through Test Tank And Load Apparatus

from the jack to the soil through a series of shims with a 10-inch diameter circular plate in direct contact with the soil.

The test soil used was concrete sand from a local sand and gravel supplier. This choice was based on two factors:

1. It is specified as embedment material in several states, especially for underdrains.

2. More uniform materials, such as Ottowa sand, which are frequently used in such studies do not have the bearing capacity to support the intended load, as shown in exploratory tests.

Tests were performed on the sand to determine the maximum standard density, grain size distribution and stress strain curves f .he determination of the one-dimensional modulus (M_s). This data is presented in Appendix B. In place, sand densities were measured by the rubber balloon method, ASTM D2167.

Vertical and horizontal pipe deflections were measured with dial gages which were mounted in sleds such that they could be moved along the pipe length through an access hole in the side of the tank. Deflections were taken at the pipe centerline and at 6 and 12 inch offsets (See Figure 3.1).

Interface pressures were measured with $1\frac{1}{2}$ inch diameter stress gages installed in the soil $\frac{1}{2}$ to 1 inch away from the soil-pipe interface and located as shown in Figure 3.2. These gages utilize a pressure sensitive diaphragm mounted with strain gages and read on a standard strain indicator.

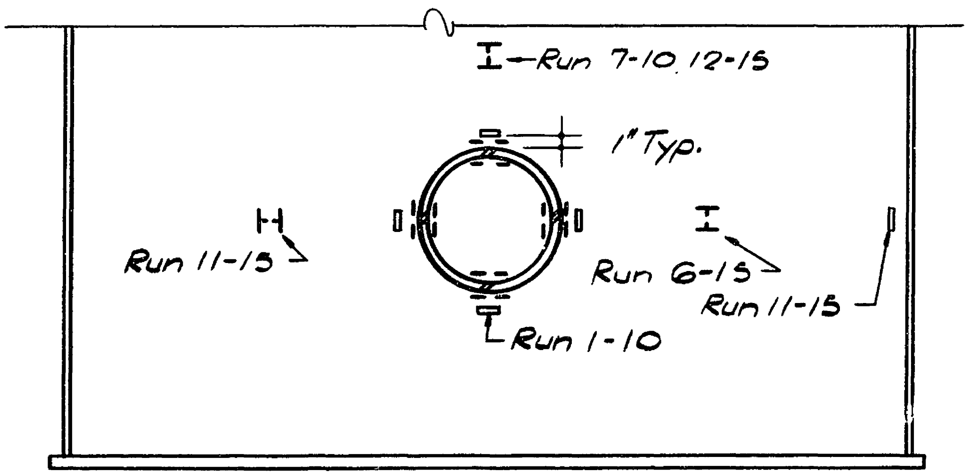
Soil strains were measured around the soil-pipe interface at the locations shown in Figure 3.2. The gages used were of the coil-inductance type, 1 inch in diameter. In this system, one coil is excited with a known voltage, which induces a voltage in a second coil proportional to the distance between the two.

For the tests on PVC with 12 and 18 inches of cover, strain gages were attached to the interior and exterior of the pipe at the crown invert and both springlines, as shown in Figure 3.2. These provided measurements on axial and bending strains in the pipe.

3.2 Samples

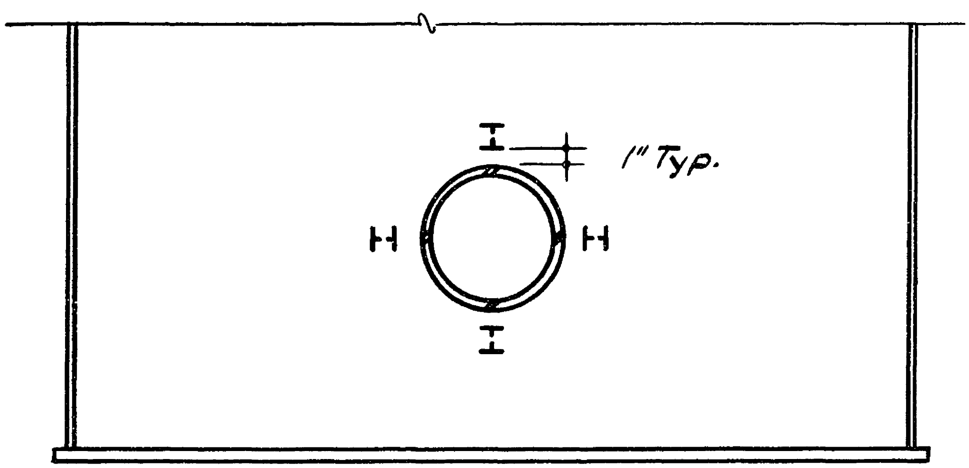
Sample size was limited to 6 inches in diameter. Using this size pipe assured that there would always be over two pipe diameters of soil on either side of the pipe.

The PVC sample pipes used were manufactured by the Johns-Manville Company in accordance with ASTM Specification D3034, SDR-35. The PE sample pipes were manufactured by Advanced Drainage Systems and conformed structurally to



a. Pipe Centerline

Soil Strain Gages
Stress Gages
-- Material Strain Gages – Runs 7-11 Only
Note: Gages used for all runs except where noted.



b. 6" Offset From Pipe Centerline

Figure 3.2 Instrumentation Locations Around Test Pipe

ASTM F405. All sample dimensions were measured to check conformance to the applicable requirements. Twelve inch segments were removed from each sample for parallel plate tests, to check conformance with specified strength requirements. After the load tests were conducted, 12 inches were removed from the central portion of the tested samples and parallel plate tested to determine if any strength loss took place. Results of these measurements and tests are compared with ASTM requirements in Table 3.1.

3.3 Test Procedure

Different sand densities were achieved by varying the lift thickness and the number of tampings. The end of a 4 x 6 timber was used as to compact the soil. The "loose" condition was installed in six inch lifts and tamped twice, producing dry densities of about 102 pcf (90% AASHTO T-99). This density would be considered a good field installation for this material. The sand for the "dense" condition for this test program was installed in 2 inch lifts and tamped 3 times, producing dry densities of about 110 pcf (100% AASHTO T-99). This density is greater than could normally be achieved in the field for this material but is useful in examining the benefit of a very stiff soil.

Tests with no pipe embedded in the sand were run for each density condition to compare load attenuation with elastic theories.

TABLE 3.1 PHYSICAL PROPERTIES OF SAMPLES USED IN TEST PROGRAM

Sample	A-1	J-2	J-3
Pipe Material	PE	PVC	PVC
Applicable ASTM Specification	F405	D3034	D3034
Diameter (in.)			
ASTM	6.0 $\pm$3% (inner)	6.275 $\pm$ .0110 (outer)	6.275 $\pm$ .011 (outer)
Min. measured	5.98	6.270	6.248
Max. measured	6.01	6.286	6.302
Wall Thickness (in.)			
ASTM (min.)	NR	.180	.180
Min. measured	-	.188	.186
Pipe Stiffness (psi)			
ASTM (5% Deflection)	30 (min)	46 (min)	46 (min)
Before Test	40	68	69
After Test	45	74	NT
ASTM (10% Deflection)	25 (min)	NR	NR
Before Test	33	-	-
After Test	35	-	-
Pipe Flattening			
ASTM	NR	No Visible Defect @ 60% Deflection	
Before Test	-	None	None
After Test	-	None	NT

NR = No Requirement

NT = Not Tested

At the start of the tests with pipe, 6 inches of bedding sand was placed in the test tank and compacted to the dense condition. This sand was left in place throughout the test program so that all pipes would have the same support. The bedding sand was grooved and the sample pipes installed, such that the sand supported the lower 90° arc of the pipe. After the pipe was installed the remaining instrumentation and sand were placed as described.

To determine the effect of departing from the 90° bedding condition, one test was performed with "flat" bedding. This test was made with 12 inches of cover in loose sand.

In the loose sand tests, the load was applied in 1000 pound increments, and measurements were taken after each increment was applied. The load was increased until bearing failure occurred in the sand. When bearing failure did occur, the load was removed, and the system was allowed to rebound for one-half hour, after which a final reading of all instruments was made. In the dense sand tests, the load was applied in 2000 pound increments, until either bearing failure, or the maximum intended load of 10,000 pounds was reached. At this point the load was removed, and the system was allowed to rebound for one half hour, after which the full load was reapplied in a single increment. Measurements were taken, the load was removed and the system was again allowed to rebound for one-half hour, after which a final reading of all instrumentation was made. In all but two instances all the sand (except that for bedding) was removed after each test. Since the pipe deflections measured during the tests with 18 inches of dense sand were slight ($<0.5\%$), the top 6 inches of sand was removed and the tests with 12 inches were performed.

3.4 Test Results

Table 3.2 presents a summary of the key parameters of all tests, including sand densities, moisture contents and the maximum load. Bearing failure occurred at a load of about 5000 pounds in the loose tests and at 8000 pounds in one dense test (Run 6). Bearing failure did not occur in any other tests with dense sand.

3.4.1 Loads

Load attenuation with depth, as determined from the load tests on sand without an embedded pipe are presented in Figure 3.3. The loose and dense conditions produced the same soil stresses until the loose sand began bearing failure at a load of about 3000 pounds. The load at which this occurred varied from test to test, depending upon the actual sand density. The sharp increase in the stresses at the point of failure is primarily due to the large displacements in the bearing plate necessary to increase the load, that is, the distance between the stress gage and load plate is decreasing considerably with each load increment.

TABLE 3.2 SUMMARY OF TEST CONDITIONS

Test #	Plastic	Sample #	Depth	Dry Density	% Moisture	Max. Load
1	PVC	J-3	6"	L-101 pcf	4.5	4,000 lb
2	None	None	-	L-101	3.2	4,700
3	PVC	J-3	6"	D-111	5.0	10,000
4	None	None	-	D-110	4.3	10,000
5	PE	A-1	6"	L-100	4.3	5,000
6	PE	A-1	6"	D-111	4.2	8,000
7	PVC	J-2	18"	D-108	4.2	10,000
8	PVC	J-2	12"	D-108	4.2	10,000
9	PVC	J-2	18"	L-102	4.6	5,000
10	PVC	J-2	12"	L-105	4.1	5,000
11*	PVC	J-2	12"	L-102	5.1	4,000
12	PE	A-1	18"	D-108	4.3	10,000
13	PE	A-1	12"	D-108	4.3	10,000
14	PE	A-1	18"	L-100	4.7	3,000
15	PE	A-1	12"	L-98	4.2	5,000

* Flat Bedding L = Loose Sand D = Dense Sand

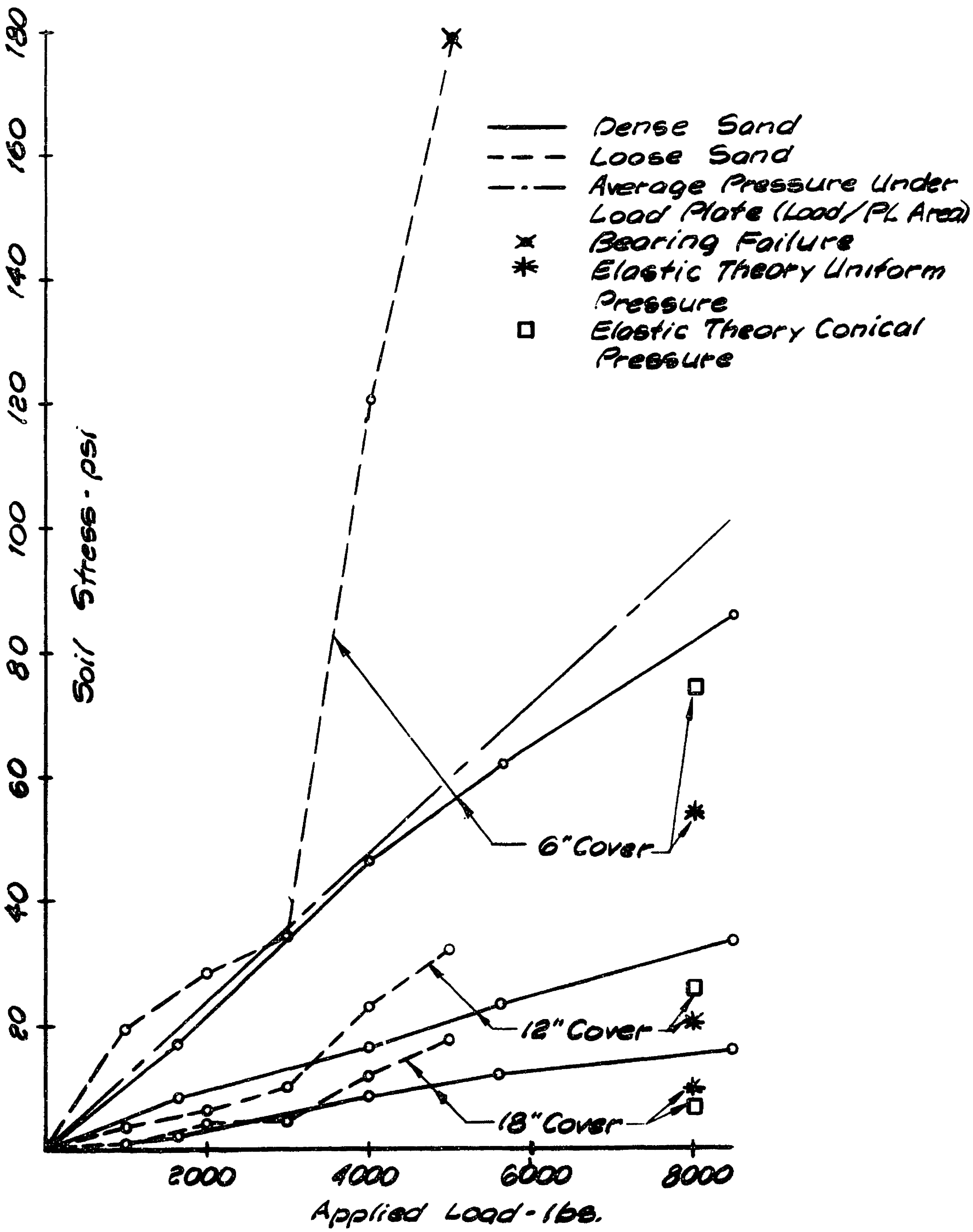


Fig. 3.3 Stresses in Sand Without Pipe

The predicted stresses from elastic theory (Reference 14) for a uniform pressure over a circular area, and for a conical pressure distribution over a circular area are also presented in Figure 3.3. The elastic predictions underestimate the measured stresses in all cases, but the error decreases with increasing depth of cover. The predictions of the conical distribution are more accurate for 6 and 12 inches of cover while at 18 inches of cover the uniform distribution prediction is in closer agreement with the measured stresses.

The results from the stress gages, when used to measure interface radial pressures, were frequently inconsistent with other data taken, such that the results are subject to question. The inconsistency of this data could be attributed to placement techniques, the fact that the gage is a rigid inclusion in the sand and may be effecting the stress pattern locally or, in the case of the PE pipe, the effect of the corrugated surface. Despite the inconsistencies in this data, two observations can be made:

1. As shown in Figure 3.4 the pipes embedded in loose sand carried a significantly larger load than those in dense sand. This would be expected since the flexibility coefficient for the dense sand systems was much higher than that for the loose sand systems.

2. The radial pressures were always highest at the crown, lesser at the springlines and least at the invert. Figure 3.5 shows the general pressure distribution around the sample pipe for the PVC-6

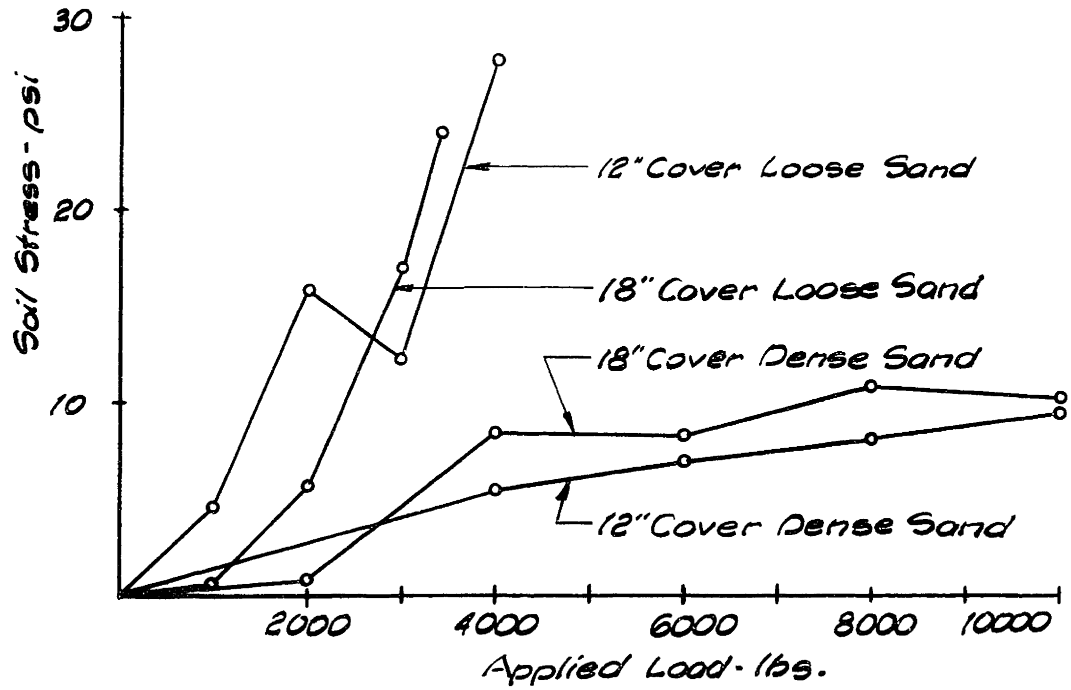


Fig. 3.4 Crown Soil Stresses For PE Pipe

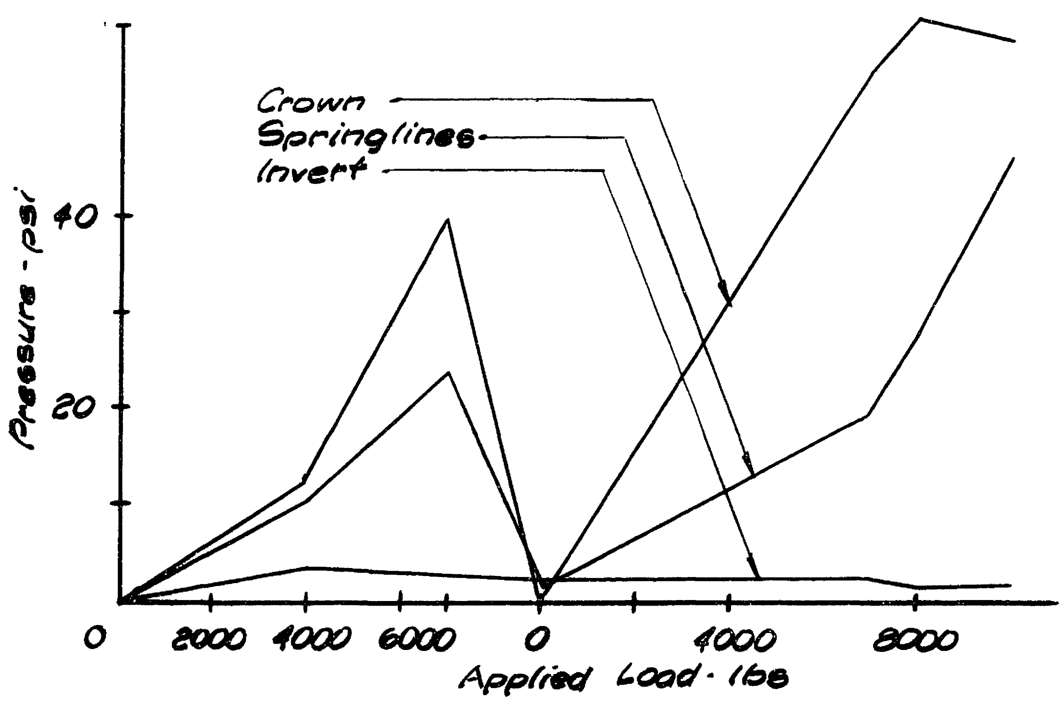


Fig. 3.5 Typical Pressure Distribution Around Pipe

inch cover-dense sand test.

3.4.2 Deflections

Vertical deflections were almost always greater than horizontal for both PE and PVC pipes, however the difference was much more pronounced for the PE pipes where the vertical deflections were approximately twice the horizontal. Both vertical and horizontal deflections displayed the same trends and therefore, for simplicity, only data on vertical deflections will be presented.

The effect of bearing failure in the sand on pipe deflection is shown in Figure 3.6 for the PE - 6 inch cover-dense sand test. After failure, load is transferred onto the pipe resulting in excessive deflections. Since pipe behavior after sand failure is outside the scope of this study further references to deflections will neglect such effects where possible.

Figures 3.7 and 3.8 show the deflection profile along the length of the PVC and PE pipes at maximum load. In all tests, deflections were small at the cross-section 12 inches from the pipe centerline. For PE pipe in loose sand the deflections appear to concentrate more at the pipe centerline and die out quickly, producing a "kink" in the deflection profile, while in all other instances the deflections die out at a more uniform rate. In both six inch cover tests for PE and the dense sand-six inch cover test for PVC, the soil strain gages located over the crown at

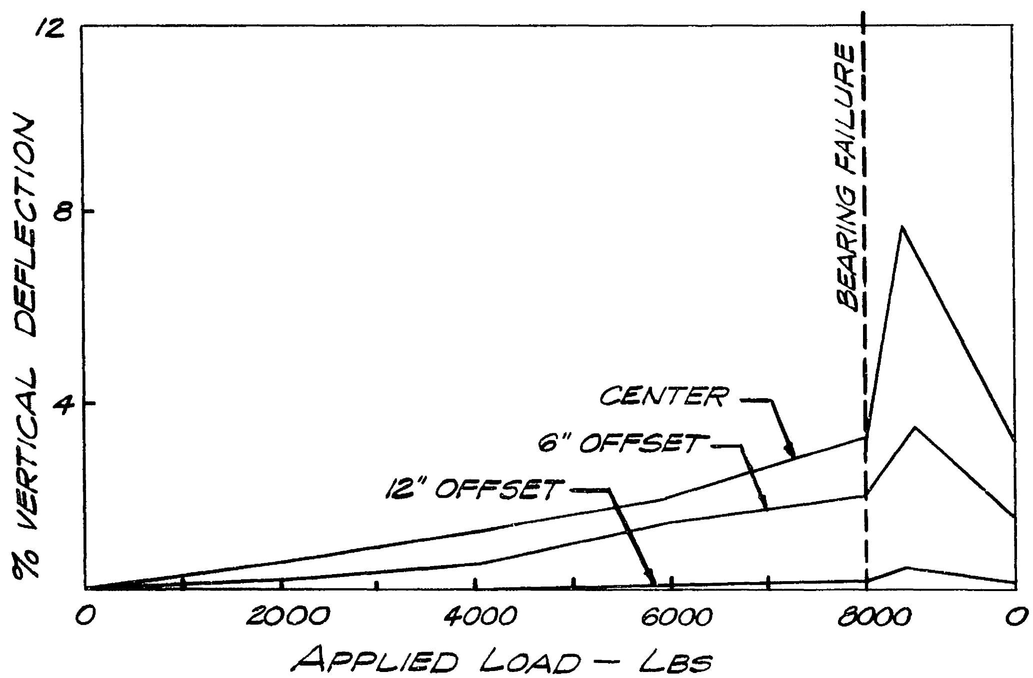


FIGURE 3.6 EFFECT OF BEARING FAILURE ON PIPE DEFLECTIONS

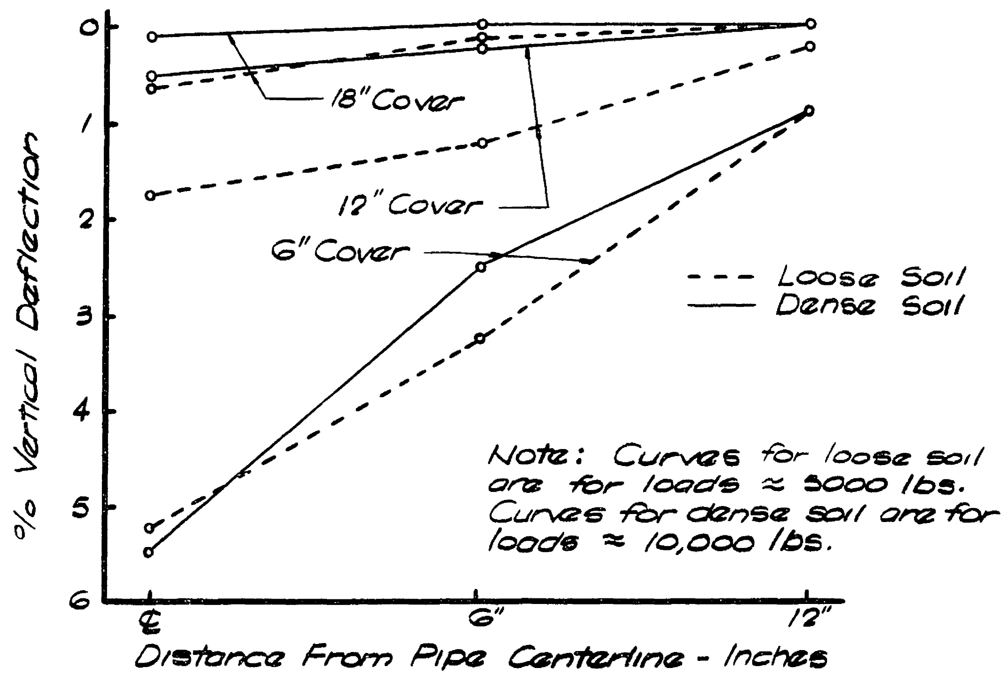


Figure 3.7 Deflection Profile Along PVC Pipe

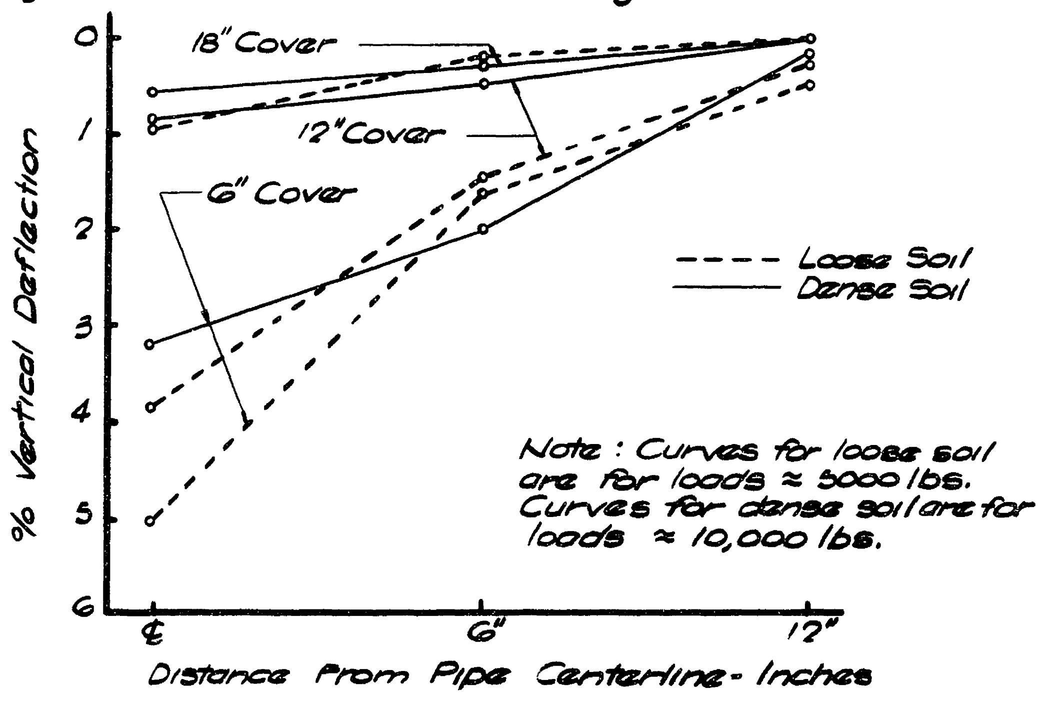


Figure 3.8 Deflection Profile Along PE Pipe

a distance 6 inches from the centerline of the pipe showed positive strains (i.e. tension). This indicates that the pipe at this location has no externally applied load, but is deflecting due to the load at the pipe centerline. This effect was not seen in pipes with greater cover, indicating a more distributed load pattern.

Pipe deflection versus applied load are shown for all cases in Figures 3.9 and 3.10. The PVC and PE pipe deflections are similar for tests with 6 inches of cover but for the tests with 12 and 18 inches of cover the PE pipes deflected more than the PVC. Higer deflections would be expected in the PE pipe since its ring stiffness is about 60% that of the PVC (See Table 3.1). The deflections with 6 inches of cover were similar, probably due to the closeness of the pipe to the load plate, such that local effects controlled the pipe behavior.

In the tests in dense sand, after the maximum load was reached it was removed and then applied again in a single increment. In all except one case the deflections increased further upon second application of the load. The increases ranged from 4 to 25 percent of the deflection due to the first cycle. In the PVC-6 inch cover-dense sand test the deflection decreased 2 percent on the second application.

3.4.3 PVC Strains

Figure 3.11 shows the crown strains due to ring deformations as recorded for test numbers 7 and 8. Axial and bending strains, also shown, were calculated

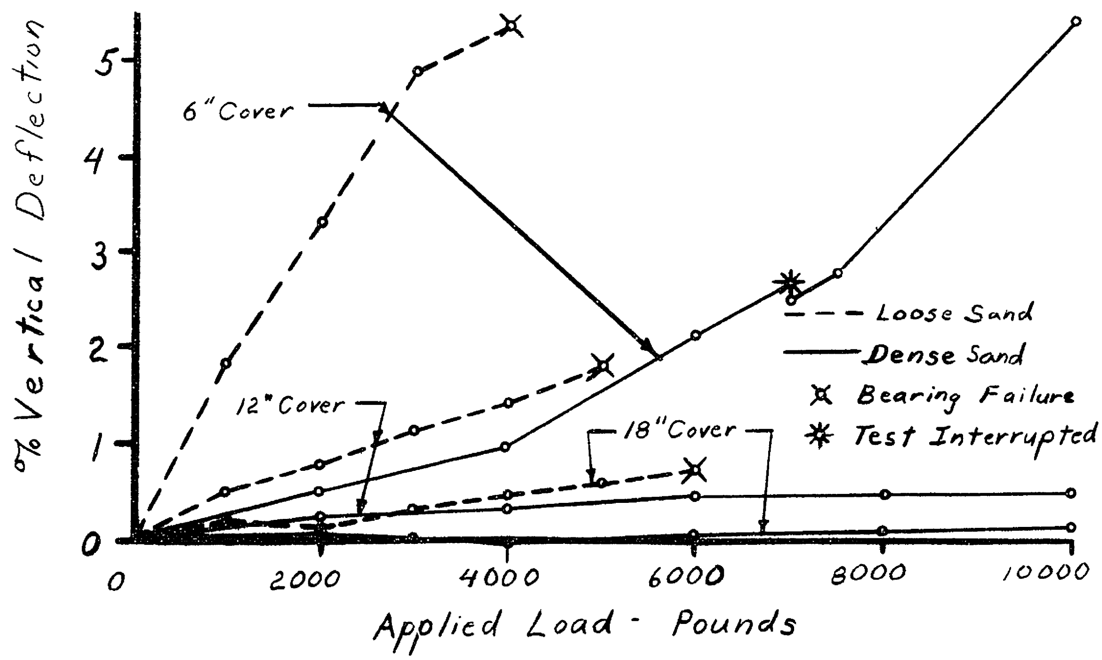


Figure 3.9 PVC - Vertical Deflection Vs. Load

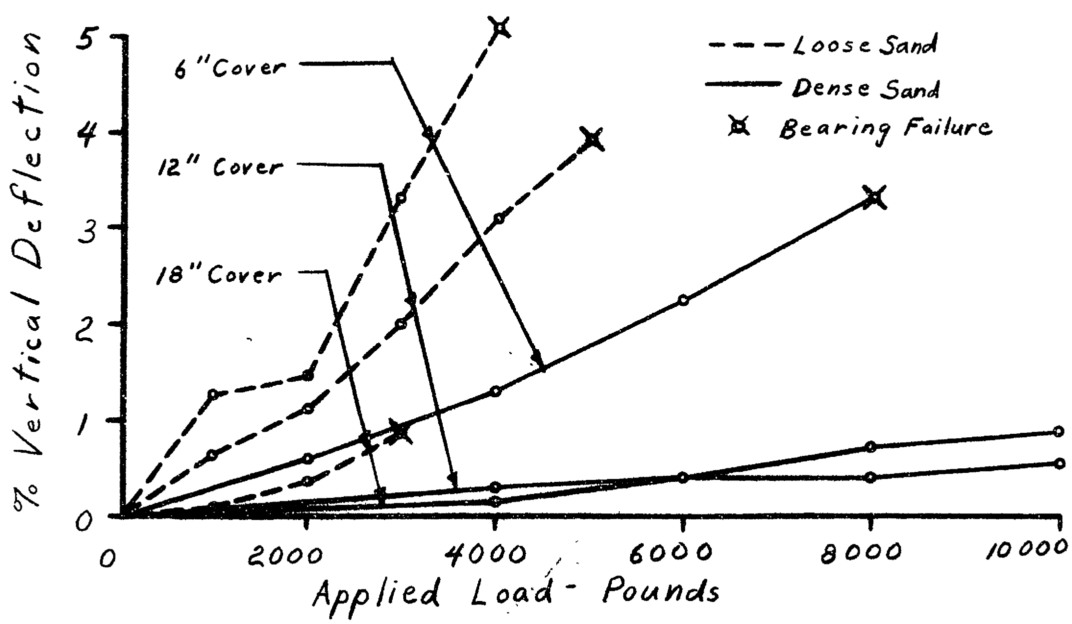


Figure 3.10 PE - Vertical Deflection Vs. Load

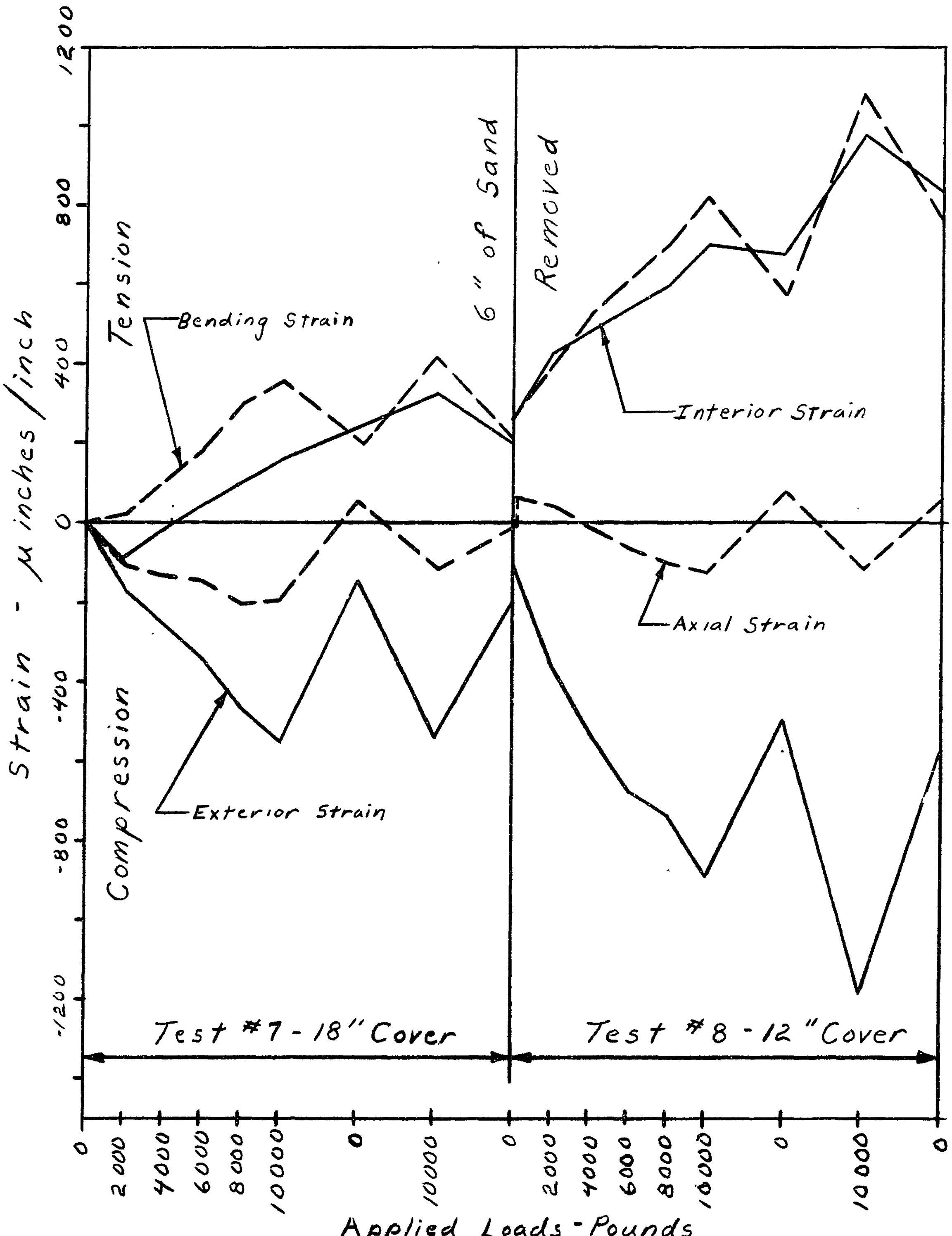


Figure 3.11 Crown Strains - PVC - Dense Sand

assuming a linear strain variation across the cross-section. When the load was removed, the axial strains were released, yet most of the bending strains remained. This suggests that bending stresses may accumulate if the pipe is subjected to repetitive loads. This will be discussed further in Chapter 4.

Figure 3.12 compares bending and axial strains at the crown, invert and springline for the 12 inch cover-dense sand case. For conversion to stresses the ring formula:

$$F/\Delta_V = \frac{EI}{0.149R^3}$$

is used in conjunction with the parallel plate test results (Table 3.1) to calculate E. For these PVC samples E $\approx$ 510,000 psi. Bending stresses are greatest at the crown whereas axial thrusts are largest at the springline. The invert is in almost pure compression. These trends were observed in all other cases except the 18 inch cover-dense sand test, where the springline bending stress was greater than that at the crown. This is due to the increased depth of cover and more uniform loading on the pipe.

Figure 3.13 shows the change in crown bending stress for each load condition. The influence of sand stiffness is shown very clearly here. The 18 inch cover-loose sand pipe was subjected to about half the load as that of the 12 inch cover-dense sand case; however, the former still developed considerably larger strains.

Figure 3.14 shows the effect of load condition on the springline thrusts. This

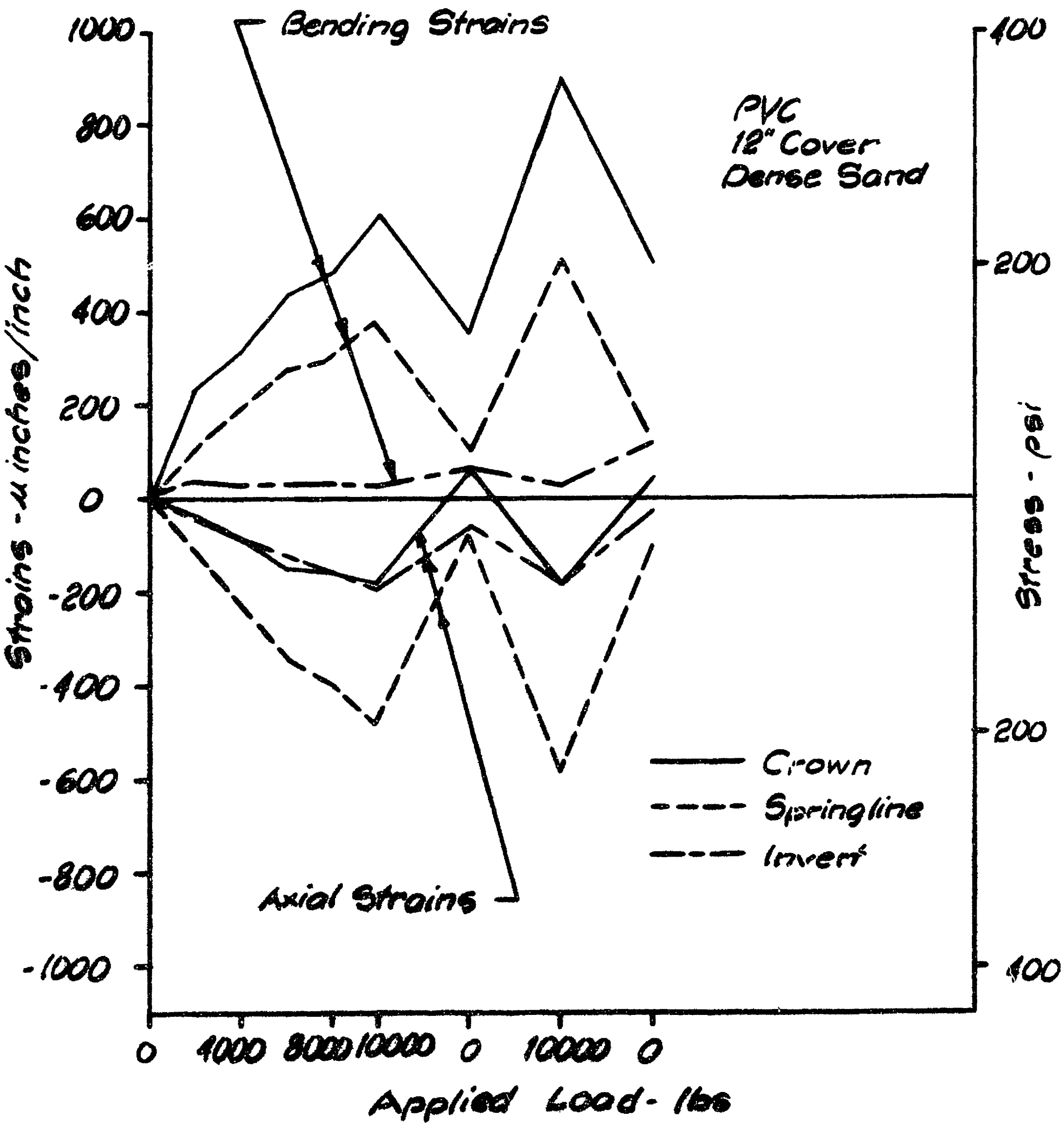


Figure 3.12 Crown Invert and Springline Strains

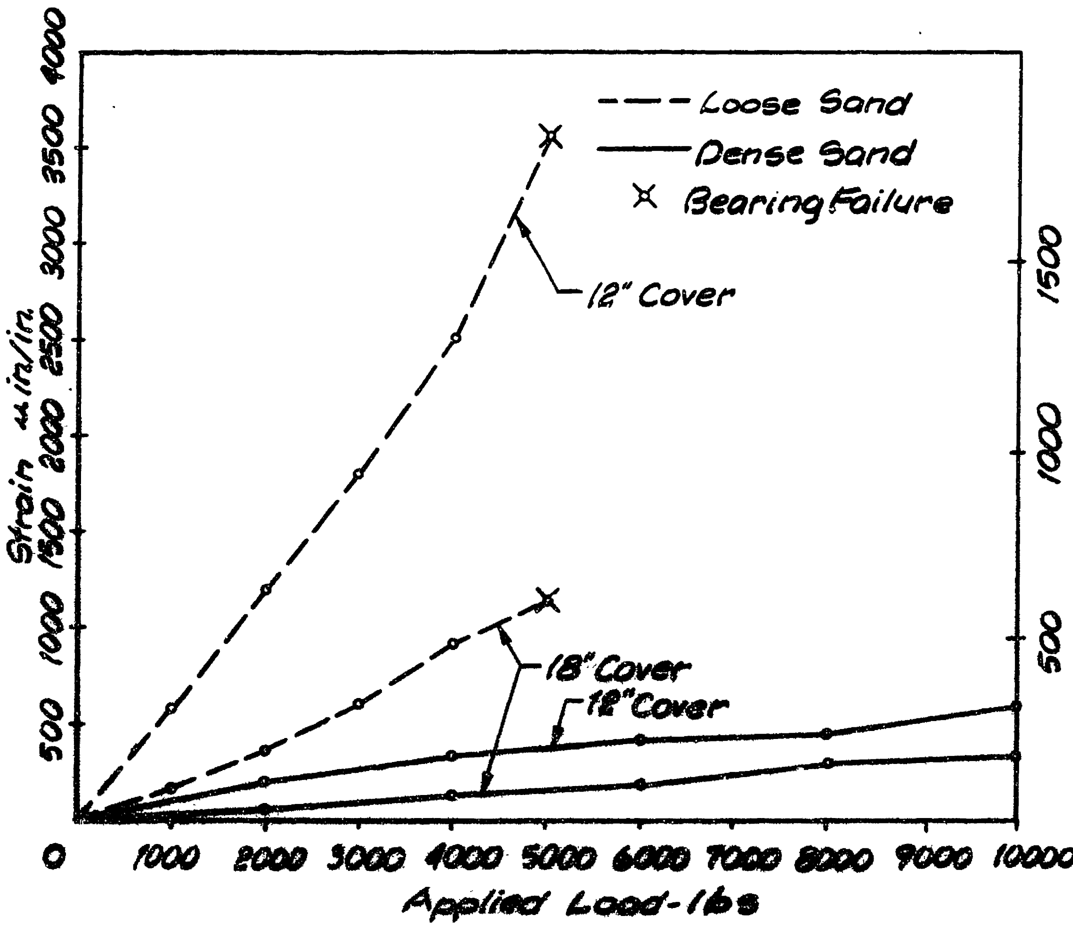


Fig. 3.13 Variation in Crown Bending Strains with Load Condition

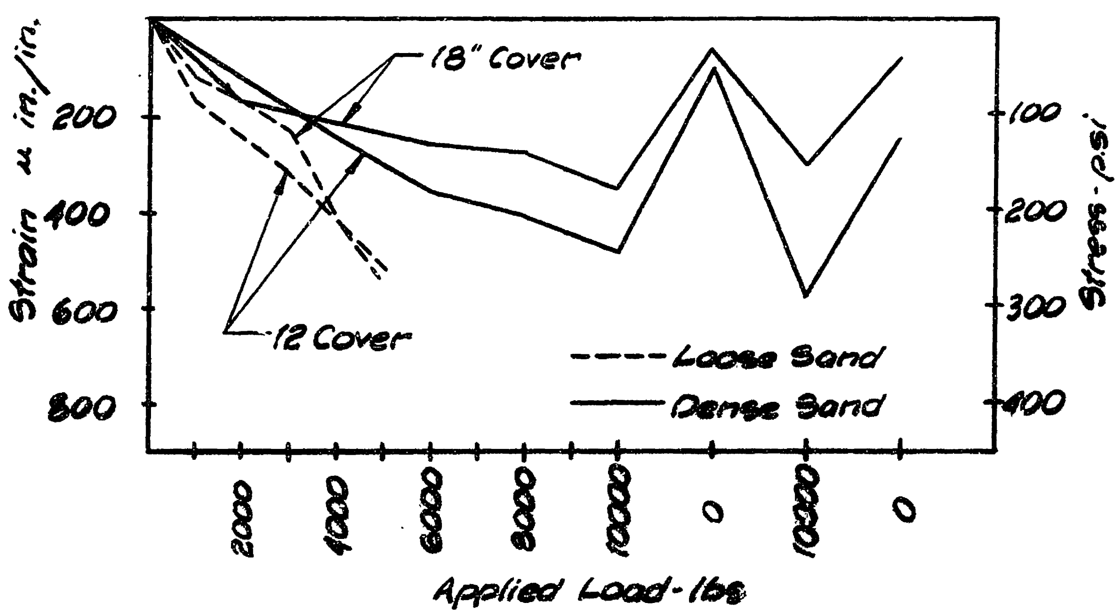


Fig 3.14 Variation in Springline Thrust Versus Load Condition

again shows the change in sand stiffness to be far more significant than the change in load.

3.4.4 Effect of Flat Bedding

Figures 3.15 and 3.16 show the effect of flat bedding on deflections and strain levels. Pipe deflection increased slightly over that for 90° bedding, however the major change was at the invert where the bending strains increased about 400%. Crown strains did not increase significantly and remain the largest strains.

3.4.5 Load Plate Settlements

In the dense sand tests, the load plate settlements required to develop the 10,000 pound load averaged 0.25 inches. In the loose sand tests average plate settlements of 0.75 inches were necessary to develop a 5000 pound load. These settlements probably had an effect on the pipe performance, as will be discussed in Chapter 4.

3.4.6 Effect of Tank Size on Pipe Performance

During test numbers 11 through 15 a stress gage was placed at the level of the springline, near the edge of the tank. A pair of soil strain coils were also placed at the springline level, halfway between the pipe and tank walls. The purpose of these gages was to observe any lateral effects produced by the tank

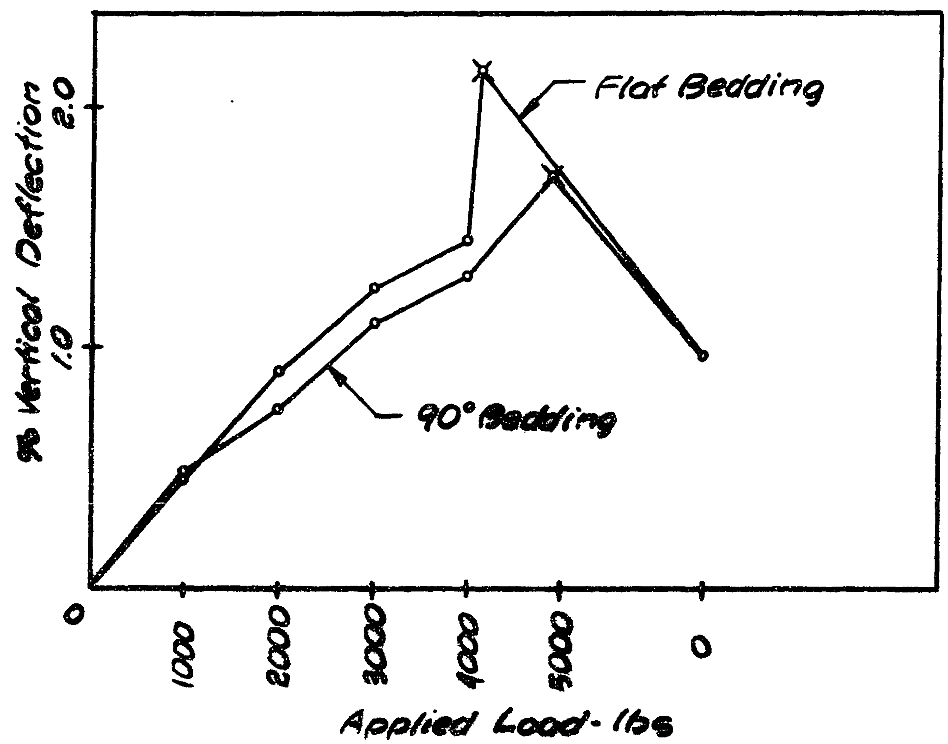


Fig. 3.15 Comparison of Flat and 90° Bedding Influence on Vertical Deflections

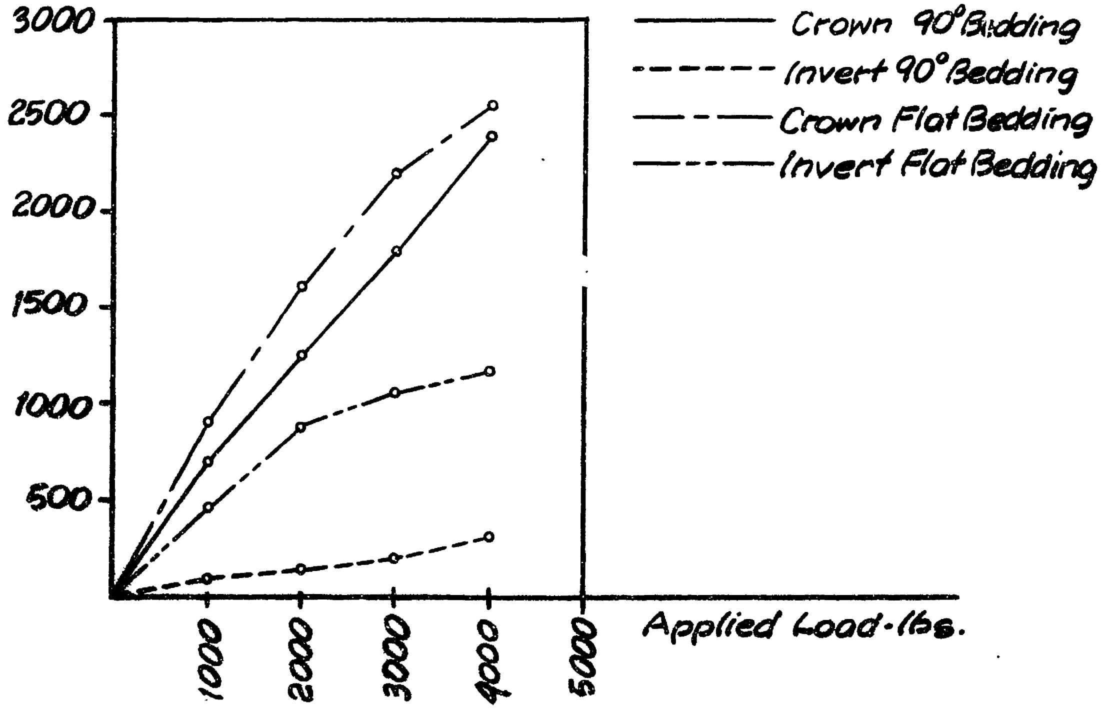


Fig. 3.16 Comparison of Flat and 90° Bedding Influence on Bending Strains

which would influence the pipe performance. In the dense sand tests the stress gage showed pressures of about 10 to 12 psi, while the strain gages showed neglible strains. This seems to indicate that lateral pressures are transmitted to some extent to the sidewall of the tank. Some of these pressures would be present in an unconfined test and some were due to the constraint of the tank but the two cannot be separated. In the loose sand tests the stress gage indicated pressures about 6 psi and the strain gage indicated higher strains. This indicates that in the loose sand less of the pipe effects are transmitted to the tank. Sidewall friction was not considered to be a factor in these tests due to the nature of the load applied.

CHAPTER 4

DISCUSSION

4.1 Criteria for Establishing a Minimum Depth of Cover

Criteria for acceptable pipe performance under the load and burial conditions considered here fall into 2 basic catagories:

1. Deflections: In flexible pipe design, deflections of up to 5% are typically considered acceptable. This rule was developed from experience with metal culverts, which normally fail at deflections of about 20%, on the theory that a limit of 5% provides a safety factor against failure of about 4. This 5% limit is also normally used as a limit on plastic pipe deflections. This limit is applicable to deeply buried pipes, which are subjected to a single static loading cycle, through the addition of backfill. When designing for wheel loads however the designer must consider the effects of multiple loadings, with the pipe deflecting and rebounding during each cycle. Loudon (15) conducted a study in which a 47,000 pound gravel truck (33,000 pounds on the tandem rear axles), was repeatedly driven over 4 inch diameter plastic tubing with 12 inches of cover. The tubing was

embedded in gravel up to 2 inches over the crown, over which 10 inches of a clay loam soil was added. He reported deflections less then 0.5% on the first pass, but that deflections increased through each of 14 load cycles to almost 5%. The depth which the gravel truck sank into the soil was not reported, but may have influenced the results. The test results reported in Chapter 3 also indicate that deflections increase upon repetitive loading. These cumulative effects cannot be reproduced in an elastic theory, such as Burns and Richard, because they are due to the inelastic behavior of soil. If a design method is to be based on elastic theory therefore, it can only analyze a single load cycle, and should use a limiting deflection of less than 5%. Also to be considered when designing for wheel loads during construction is the addition of more backfill after the wheel loading, which will cause additional deformations. In view of these facts, the limiting deflection used in further discussion will be 2.5%. This limit allows for the occurrence of several load cycles, and additional long term deflections. (This is not intended to represent an allowable deflection in installations subject to a large number of load cycles.)

2. Stresses: Since most research on plastic pipe has focused on

pressure pipe, which is subjected primarily to tensile forces, very little is known about the long term behavior of non-pressure pipe, which resists loads primarily in flexure and compression. This lack of information makes it necessary to be conservative in choosing an allowable stress. The considerations just discussed with respect to deflection limitations also apply here and provide further cause to be conservative in selecting a design stress. Further discussion will neglect the fact that the flexural strengths of PVC and PE are known to be greater than the tensile strengths, as stated in Chapter 2. An allowable stress due to a single cycle of wheel loading will be considered to be 1/3 of the tensile yield strength, or 2200 psi for PVC and 1000 psi for PE. This allows a margin of 3 for uncertainty of the behavior of plastics under flexural and compressive stresses, the effects of several load cycles, and long term effects after the installation is complete. This is a larger margin than used for deflections due to the greater uncertainty.

A third factor which influences the concentrated load-shallow buried problem, but for which no criteria will be established, since it relates to sand behavior, is bearing capacity. Sand density was varied in this study primarily to examine the effects of soil stiffness, as it provides lateral support to an embedded pipe.

Density also has a significant effect on the bearing capacity of sand, and in this respect also has a tremendous influence on the shallow burial problem. In all of the loose sand tests and even in one dense sand test, bearing failure occurred before reaching the intended load of 10,000 pounds. When this occurs, the load plate undergoes large deflections into the sand with additional load being transferred onto the pipe. Figure 3.6 shows the effect of bearing failure on pipe performance. Large plate deflections can also occur before the soil actually fails. As was noted in Section 3.4.5, to develop a 5,000 pound load in loose sand required 3 times the plate movement that was necessary to develop a 10,000 pound load in dense sand. Considering the attenuation of a concentrated load with increasing depth, these large deflections undoubtedly caused an increased load on the pipe. Discussion of recommendations for minimum soil cover, will assume the following:

1. The soil has the bearing capacity to support the design load.

2. The stated depths of cover will refer to the distance from the crown of the pipe to the bottom of the impression left by the tire.

The load condition which will be discussed is the same as that used in the test program, i.e. a 10,000 pound load on a single tire.

4.2 Allowable Minimum Cover

In order to discuss the subject of minimum cover in as general a sense as possible, and since sand behavior is primarily controlled by stiffness, the two density conditions will now be referred to by their one-dimensional modulus, rather than as loose and dense. In this way the test results can be applied to any soils (under the conditions discussed in Section 4.1) with similar stress-strain characteristics. As discussed in Chapter 2, the soil modulus which best describes sand behavior under concentrated loads is at or near the minimum possible value. Computing the minimum modulus from the curves for 110 pcf and 102 pcf sand in Figure B2 yields:

$$M_{s\text{-Loose}} = 2000 \text{ psi}$$

$$M_{s\text{-Dense}} = 9000 \text{ psi}$$

These values will be used to describe the soil stiffness in the following discussion.

Table 4.1 presents the maximum deflections, axial stresses and bending stresses recorded for all of the 9000 psi sand tests and the extrapolated deflections (Figure 4.1) and stresses for all of the 2000 psi sand tests, assuming that no bearing failure had occurred prior to reaching the 10,000 pound load. From these results the following observations can be made:

1. 6 Inch Cover: None of the pipes tested with 6 inches of cover performed within the limits set forth in Section 4.1.

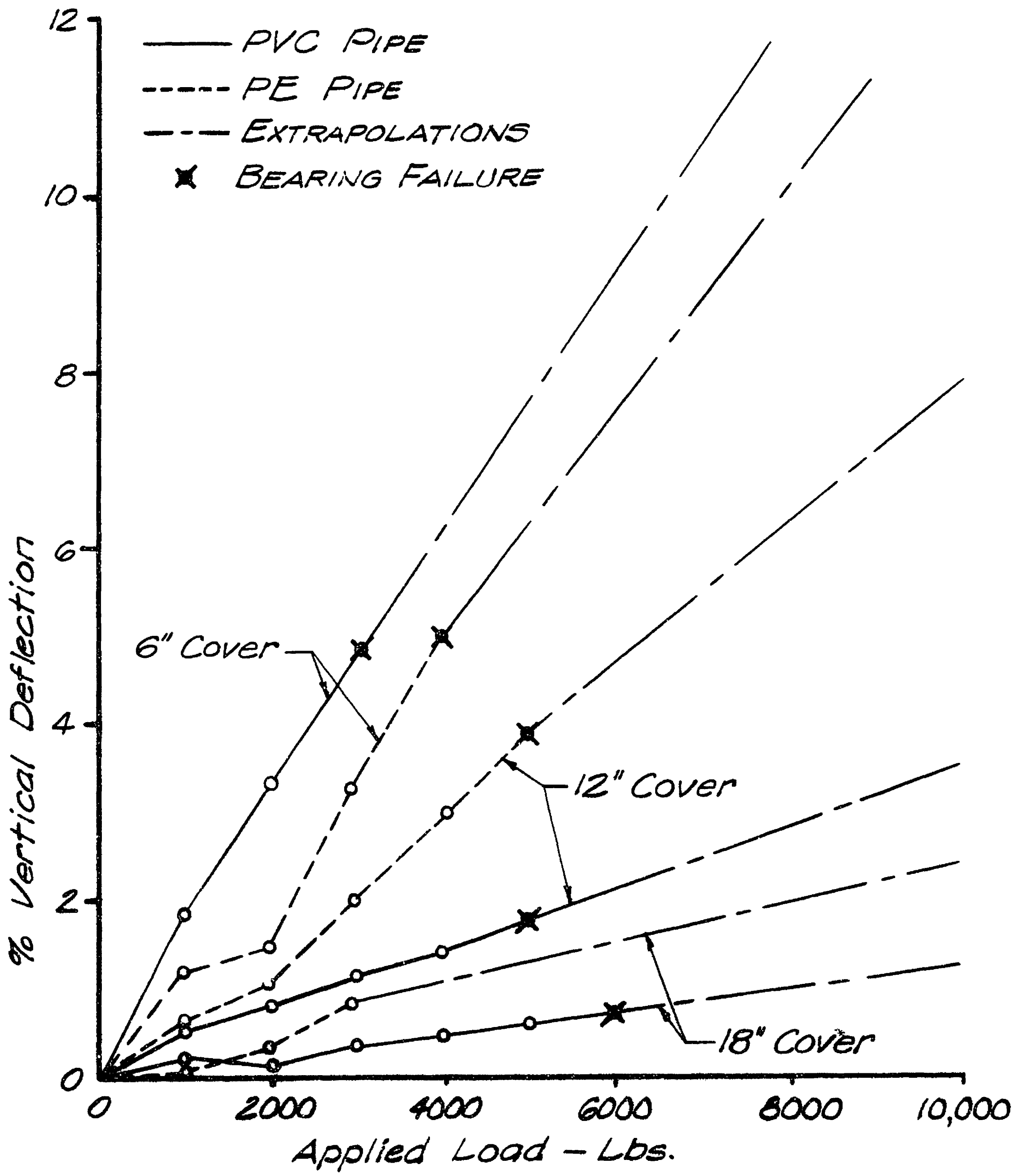


FIGURE 4.1 EXTRAPOLATION OF DEFLECTIONS IN LOOSE SOIL TO 10000 POUND LOADS

TABLE 4.1 DEFLECTIONS AND STRESSES AT 10,000 POUND LOAD

Depth of Cover	Measurement	PVC		PE	
		M_s = 2000 psi	M_s = 9000 psi	M_s = 2000 psi	M_s = 9000 psi
6 inches	Deflection %	12	5.25	12.5	4.75
	Axial Stress psi	-	-	-	-
	Bending Stress psi	-	-	-	-
12 inches	Deflection %	3.5	0.5	8.0	0.75
	Axial Stress psi	490	235	-	-
	Bending Stress psi	3480	280	-	-
18 inches	Deflection %	1.2	0.25	2.5	0.55
	Axial Stress psi	280	175	-	-
	Bending Stress psi	1060	200	-	-

2. 12 Inch Cover: In 9000 psi sand both the PE and PVC pipes performed satisfactorily. In 2000 psi sand, the PE pipe deflected well over the specified limit while the PVC pipe deflected only slightly too much but was 80% overstressed.

3. 18 Inch Cover: With 18 inches of cover over the crown all pipes performed satisfactorily. Deflection in the PE pipe in 2000 psi sand was marginal but did meet the limits.

To make recommendations concerning allowable minimum cover from these results requires consideration of several factors which were not studied here, such as the potential variations in construction practices, embedment material quality, and the dynamic effects of a tire actually rolling over a pipe installation. One must also consider what happens to the pipe after a wheel loading during construction. For example, the test with a flat bedding condition showed only slight increases in deflection and peak stresses over the comparable test with 90° bedding, however, a 400% increase in bending stresses was observed at the invert. If more backfill were added, the increased stresses at the invert could become a governing factor in the design.

Different pipe diameters also have an effect on the response. While the controlling factors in pipe response are load attenuation, and soil stiffness, larger pipes will be subjected to the concentrated effects of a wheel load at greater depths than will smaller pipes, and an allowance should be made for this. Consideration of all these points leads to the following recommendations:

1. 9000 psi sand: If a sand stiffness of 9000 psi can be achieved then 12 to 18 inches (depending on field control) of cover will provide adequate protection for pipes having ring stiffnesses in the range of the PE and PVC tested here.

2. 2000 psi sand: In 2000 psi sand, the variation in ring stiffness between PE and PVC has a greater effect. In the case of PVC, 18 inches of cover is adequate if good control is exercised over the installation. If the quality control is questionable, then this cover should be increased, and although no tests were performed with over 18 inches of cover, 24 inches should be adequate. For PE pipe, since the 18 inch test was marginal, 24 to 30 inches of cover should probably be required, although no tests were conducted at these depths of cover to confirm this.

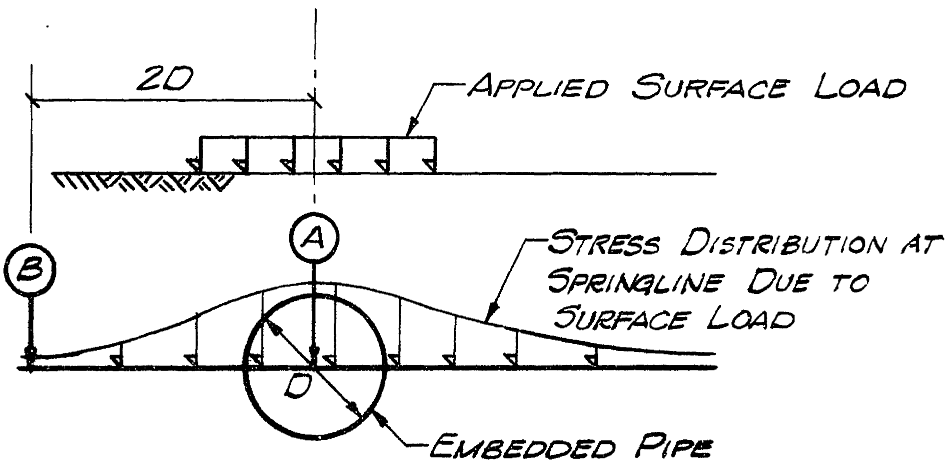
These recommendations should be increased slightly for larger diameter pipes, but since only one diameter was considered here, specific recommendations cannot be made.

4.3 Design for the Concentrated Load-Shallow Burial Situation

An elastic analysis of the test pipes was made to see if the Burns and Richard theory could be used to predict pipe behavior sufficiently accurate for design purposes. Utilizing the ideas put forth in Chapter 2, the load used in the analysis was the vertical soil stress at the springline (Figure 4.2) as measured in the no-pipe 9000 psi sand tests. The soil modulus was calculated from Figure B2 using the vertical soil stress at the depth of the springline, but 2 pipe diameters laterally away from the load axis as shown in Figure 4.2. This stress more accurately reflects the state of the soil which resists pipe movement, than do the soil stresses near the pipe. Table 4.2 compares the Burns and Richard predictions of deflections, axial stress and bending stress with those measured in the burial tests. The following observations can be made:

1. Deflections: The deflection predictions underestimate those measured in all of the 6 inch cover tests, but improve with depth. For the 12 inch cover-9000 psi sand case and all the 18 inch cover cases the predictions are sufficiently close for design purposes.

2. Thrusts: The Burns and Richard thrust predictions are always within 30% of those measured in the burial tests. The predictions are always high in the 9000 psi sand and always low in the 2000 psi sand. This indicates that the 9000 psi sand, being



(A) STRESS FOR USE IN BURNS AND RICHARD ANALYSIS

(B) STRESS FOR USE IN DETERMING ONE-DIMENSIONAL MODULUS

FIGURE 4.2 STRESSES FOR USE IN BURNS AND RICHARD ANALYSIS

TABLE 4.2 COMPARISON OF TEST RESULTS WITH BURNS AND RICHARD THEORY

Soil	Load Condition	Maximum Load Pounds	Method	PVC Deflection %	PVC Axial Stress psi	PVC Bending Stress psi	PE Deflection %	PE Axial Stress psi	PE Bending Stress psi
2000 psi sand	6" Cover	5000	Burial Test	5.5*	-	-	5.1	-	-
			Burns & Richard	1.8	555	820	2.6	880	490
			Test/Theory	3.1	-	-	2.0	-	-
	12" Cover	5000	Burial Test	1.75	245	1740	4.0	-	-
			Burns & Richard	0.7	190	320	1.0	310	190
			Test/Theory	2.5	1.3	5.4	4.0	-	-
	18" Cover	5000	Burial Test	0.6	140	530	1.0*	-	-
			Burns & Richard	0.5	110	220	0.7	190	130
			Test/Theory	1.2	1.3	2.4	1.4	-	-
9000 psi sand	6" Cover	10000	Burial Test	5.25	-	-	4.25*	-	-
			Burns & Richard	1.0	970	390	1.6	850	250
			Test/Theory	5.3	-	-	2.7	-	-
	12" Cover	10000	Burial Test	0.5	235	280	0.75	-	-
			Burns & Richard	0.4	340	150	0.6	320	100
			Test/Theory	1.25	0.7	1.9	1.2	-	-
	18" Cover	10000	Burial Test	0.25	175	200	0.55	-	-
			Burns & Richard	0.24	200	100	0.4	200	65
			Test/Theory	1.0	0.9	2.0	1.1	-	-

*Soil failed before reaching indicated load. Values have been extrapolated to account for this.

Note: Variations in stress and deflections are approximately linear. To compare 2000 psi sand and 10000 psi sand results at same load multiply dense soil results by $\frac{1}{2}$.

much stiffer, is able to arch more load around the pipe than is the 2000 psi sand.

3. Bending Stresses: All of the Burns and Richard predictions of bending stresses underestimate those measured considerably. This is because the peak bending occurs as a local effect at the crown, rather than as a general ring bending, as predicted by Burns and Richard. In the 2000 psi sand the ratio of measured bending stresses to those predicted by Burns and Richard, decreases from 5.4 to 2.4 as the depth of cover increases from 12 to 18 inches. In the 9000 psi sand the ratio is approximately 2 in both cases. If a larger data set were available, these ratios could be established for varying depths of cover, pipe diameter, and soil conditions providing a modification factor to be applied to the Burns and Richard solution.

In general it can be seen that the Burns and Richard analysis gives better predictions for the tests in 9000 psi sand than it does for those in 2000 psi sand. This is because the 9000 psi sand, being more highly compacted and therefore stiffer, is subject to fewer non-linearities and local effects. The 2000 psi sand undergoes a great deal of compaction during testing, and it is sometimes difficult to distinguish exactly where bearing failure begins to influence the tests.

CHAPTER 5

CONCLUSIONS

5.1 Minimum Cover

Burial tests have been conducted to observe the structural behavior of PVC sewer pipe and corrugated PE underdrain pipe under concentrated loads. Based on the results of these tests, recommendations have been made for allowable minimum cover:

1. PVC Sewer Pipe: PVC pipe performs satisfactorily according to the criteria set forth in Section 4.1 under a 10,000 pound single wheel loading, when covered with 12 to 18 inches of sand with a modulus of 9000 psi or with 18 inches of sand with a modulus of 2000 psi and good field control. If the field control is limited than this limit for 2000 psi sand should be increased. Although no tests were run with more than 18 inches of cover it is expected that 24 inches should be adequate.

2. Corrugated PE Underdrain Pipe: Corrugated PE pipe deflects within acceptable limits when covered with 12 to 18 inches of 9000 psi sand or although tests were not performed at these

depths, would be expected to perform acceptably if covered with 24 to 30 inches of 2000 psi sand.

Application of these results should consider possible variations in construction and loading, as well as the influence of soil conditions. Design for such situations should allow for both the soil stiffness and the expected depth of the impression left in the soil by the wheel of a vehicle. The cumulative effect of multiple loadings should also be considered. The suggested depths should be increased for larger diameter pipe, but since only one diameter pipe was used, no recommendation can be made.

The recommendations can be extended to soils other than sand if the stress-strain characteristics are similar, such as a dry or overconsolidated clay. The consolidation effects in a soft clay would have a significant effect in the test results, but a soft clay is a poor backfill material and should not be used under the loads discussed here.

5.2 Design Method

The Burns and Richard elasticity theory has been evaluated for its ability to predict pipe performance under concentrated wheel loads. For the data gathered in this test program, the theory always gave predictions for axial thrust within

30% of those measured, if load attenuation is accounted for when applying load to the Burns and Richard Model. The deflection predictions were quite close if the soil cover was over 12 inches deep in soil with a 9000 psi modulus or over 18 inches deep in soil with a 2000 psi modulus. The Burns and Richard theory did not give good predictions for the bending stresses under any of the conditions considered, but did improve in accuracy with increasing depth of cover.

5.3 Recommendations For Future Research

This study has shown that elastic theory can be used to describe thrusts and in some instances as noted above, deflections in a plastic pipe under wheel loads, but is inadequate to predict peak bending stresses. Further research should be undertaken to determine empirical modification factors for the Burns and Richard Theory to enable it to predict the complete pipe behavior. This research should take two directions:

1. This test program considered only a limited number of variables. Further testing should be done to examine the effects of other variables, such as soil type and pipe diameter. This could be accomplished by either sand box tests, as used in this study, or with finite element studies. Finite element studies could be difficult in the case of the shallow burial-concentrated load problem because of the difficulty in modeling soil behavior properly.

2. Any laboratory or computer studies should be verified with full scale field tests.

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APPENDIX A

BURNS AND RICHARD ELASTICITY SOLUTION

The Burns and Richard equations for buried cylinders are derived using a plane strain formulation from elasticity theory. The system is idealized as an infinite, weightless, homogeneous, isotropic, linearly elastic soil with a circular elastic inclusion. Load is applied as a surface overpressure and lateral loading is proportional to the coefficient of earth pressure at rest for a given soil.

Parameters necessary to define the soil are:

M_s = one-dimensional compression modulus

ν = Poisson's ratio for soil

Poisson's ratio can be easily related to the coefficient of earth pressure at rest by:

$$K = \frac{\nu}{1-\nu}$$

The cylinder is characterized by:

$u = bC$ = extensional flexibility ratio

$v = \frac{cF}{24}$ = bending flexibility ratio where $C = \dfrac{M_s D}{EA}$ = compressibility coefficient

$$F = \frac{M_s D^3}{EI} \quad = \text{flexibility coefficient}$$

$$b = \frac{1}{2} \left(\frac{1}{1-\nu}\right)$$

$$c = \frac{1}{2} \left(\frac{1-2\nu}{1-\nu}\right)$$

Solutions were developed for the limiting cases of full-slip and no-slip at the soil-conduit interface.

Full Slip:

$$w = \frac{pr}{2M_s} \{u(1-a_o) - \frac{2v}{3} (1+3a_2 - 4b_2) \cos 2\psi\}$$

$$T = pr\{b(1-a_o) + \frac{c}{3} (1 + 3a_2 - 4b_2) \cos 2\psi\}$$

$$M = pr^2 \{\frac{cu}{6v} (1-a_o) + \frac{c}{3} (1 + 3a_2 - 4b_2) \cos 2\psi\}$$

$$p_r = p\{b(1-a_o) - c(1+3a_2 - 4b_2) \cos 2\psi\}$$

No Slip:

$$w = \frac{pr}{2M_s} \{u(1-a_o) - v(1-a_1 - 2b_1) \cos 2\psi\}$$

$$T = pr\{b(1-a_o) + c(1+a_1) \cos 2\psi\}$$

$$M = pr^2 \left\{ \frac{cu}{6v} (1-a_o) + \frac{c}{2} (1-a_1-2b_1) \cos 2\psi \right\}$$

$$p_r = p\{b(1-a_o) - c(1-3a_1-4b_1) \cos 2\psi\}$$

where:

w = radial displacement of conduit wall

T = ring compression load per unit length

M = bending moment in conduit wall per unit length

p_r = radial pressure at soil conduit interface

p = vertical soil stress if pipe were not installed

r = mean radius of conduit

ψ = angle relative to horizontal

$$a_o = \frac{u-1}{u+b/c}$$

$$a_1 = \frac{c(1-u)v + 2b - (c/2)(c/b)u}{(1+b+cu)v + 2(1+c) + (1+c/2)(c/b)u}$$

$$b_1 = \frac{(b+cu)v - 2b - (c/2)u}{(1+b+cu)v + 2(1+c) + (1+c/2)(c/b)u}$$

$$a_2 = \frac{(2v-1+1/b)}{(2v-1+3/b)}$$

$$b_2 = \frac{(2v-1)}{(2v-1+3/b)}$$

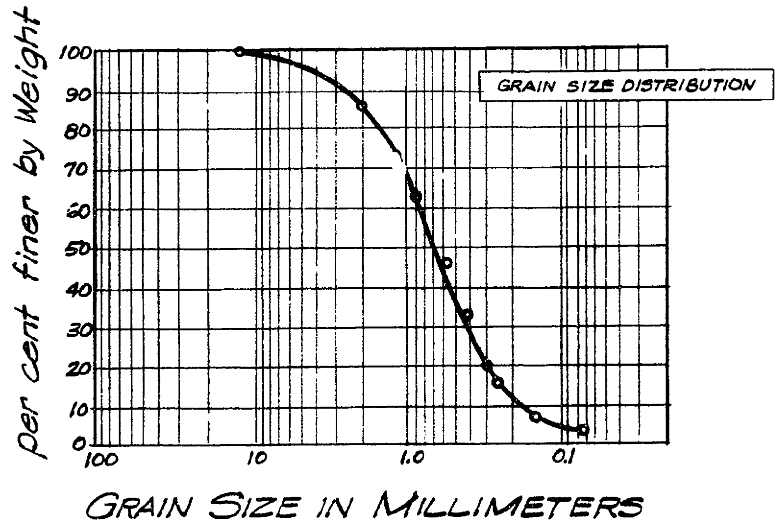
Design charts based upon this formulation have been presented by Krizek (6) and Lew (3). Katona (7) has incorporated this solution into his culvert design program (CANDE) and has modified it to allow for the formation of plastic hinges, which is useful in analyzing metal culverts.

APPENDIX B

SOIL PROPERTIES

Figure B1 shows the maximum sand density determined in accordance with AASHTO Specification T-99. In uniform sands, such as this, it is frequently difficult to determine accurate points on the curve at more than the optimum moisture content, due to moisture seepage through the sand. This difficulty was encountered here.

Figure B2 presents stress-strain curves for the test sand at 3 densities. These curves are used in determining the one-dimensional modulus.



MAX. DRY UNIT Weight 109.1 pcf
OPTIMUM MOISTURE 11.9 %
TEST AASHTO T-99

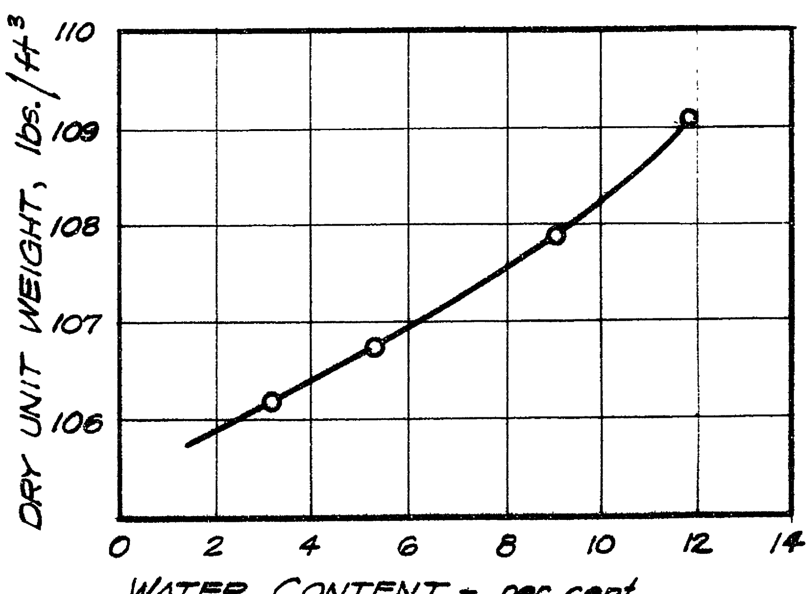


FIGURE B1 STANDARD COMPACTION TEST RESULTS

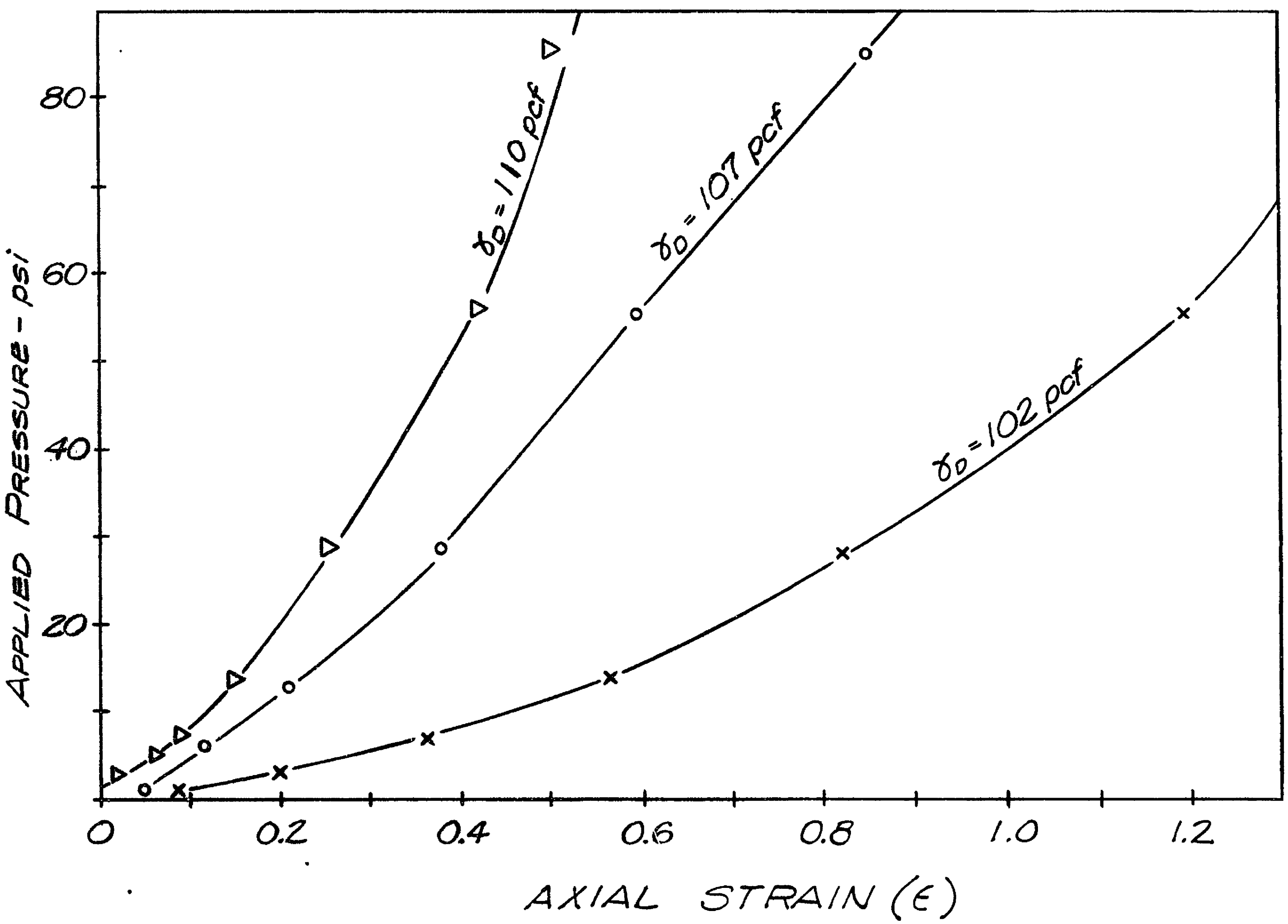


FIGURE B2 ONE DIMENSIONAL MODULUS FOR CONCRETE SAND