A STUDY OF ADHESION
BETWEEN
P.V.C. PIPE AND SAND

by

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B.S., Kansas State University
1972

A MASTER'S THESIS

submitted in partial fulfillment of the
requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1975

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ACKNOWLEDGMENTS

I wish to express my sincere appreciation and gratitude to each of the following individuals for their valuable help and assistance. To Dr. Robert Snell for making it possible for me to enter Graduate School. To Professor Wayne W. Williams for exposing me to the basic ideas and concepts of this research, for providing me guidance and encouragement, and for various forms of assistance. To Russell Gillespie for constructing the testing device. And lastly, to Wayne Duryee for helping me with various phases of the research.

INTRODUCTION

Many modern conveniences which mankind enjoys today are partially dependent on the use of buried conduits. These conduits include sewage lines, water and gas mains, irrigation pipes, and drainage culverts.

In many instances, because of natural and manmade topography, conduits must be buried to great depths and withstand high overburden pressures. Also, they must be utilized in a wide variety of soils with many differing strength characteristics. These factors contribute to the many problems and failures which man experiences when using buried conduits.

Today, through technological advancements in the plastics industry, man is able to use polyvinyl chloride (P.V.C.), along with other plastics, for buried conduits. In order to insure the proper use and design of P.V.C. conduits, knowledge concerning their interaction with the surrounding soil must be acquired. This knowledge will enable man to use the most economical type of P.V.C. conduit for each given situation.

SCOPE

The scope of this research was limited to the study of adhesion between P.V.C. pipe and only one type of soil, a medium graded sand found along rivers and streams in the midwestern United States.

Included in this study was a thorough literature review of past research concerning buried conduits. In addition, a thorough testing program involving the interaction of P.V.C. pipe and sand was carried out. Specifically, tests were conducted on the previously mentioned sand to determine the adhesion between P.V.C. pipe and soil in the longitudinal and radial directions.

PURPOSE

The purpose of this study was to:

1) Determine the adhesion, in terms of cohesion and angle of friction, which occurs at the soil-P.V.C. conduit interface in a longitudinal direction.

2) Determine the adhesion, in terms of cohesion and angle of friction, which occurs at the soil-P.V.C. conduit interface in a radial direction.

3) Relate these values to the normal values of cohesion and internal angle of friction which the soil possesses.

4) Mathematically model the effects of the soil to pipe adhesion on the buried stiffness of the pipe.

5) Relate, in a practical sense, these findings to the existing theories concerning predicted P.V.C. deflection due to overburden loads.

LITERATURE REVIEW

In reviewing the literature related to buried conduits, a brief statement of their early use was necessary. However, the major efforts of this review pointed toward other related areas, commencing with the early research which attempted to establish a rational method for determining and predicting the buried conduit's deflection and suitability for a given situation. Following this initial work, further research up to the present time was noted. For a complete understanding of all the factors and problems involved in designing conduits, familiarity with lateral soil pressure concepts, characteristics of various types of soil, and structural concepts concerning the conduit-soil system must be acquired.

It is pointed out by Spangler (1) and Spangler and Handy (2) that buried conduits were included among the earliest examples of engineering structures built by mankind. During the 14th century B.C., the ancient Greeks utilized open canals for their irrigation systems. Later, in 530 B.C., the Samian aquaduct of Polyerates was constructed. This structure was a tunneled conduit which led from a source of water to feed open pools (3). Other uses of these structures included sewers, culverts, drains, and gas lines.

MARSTON'S LOAD THEORIES

During the early 1900's, the first work on a logical method for designing buried conduits was attempted. This work was conducted by Anson Marston, Dean of Engineering, Iowa State University. It was a result of the necessity to get the state of Iowa "out of the mud."

Marston's load theories are based on the concept of dividing conduits into two general classes: 1) a projecting conduit, both positive and negative, and 2) a ditch conduit. A projecting conduit was defined as one placed under an embankment. If the top of the projecting conduit extended above the natural ground surface, it was defined as being positive while if the top of the conduit was placed in a shallow ditch below the natural ground surface, it was classed as being negative. A ditch conduit is one simply placed in a ditch without an embankment placed over it. Figure 1 illustrates these different classes of conduits.

Conduits are also grouped into two general types: 1) flexible, and 2) rigid. Flexible conduits are usually thin walled structures which are made of steel or different types of plastics. Rigid conduits are very stiff and unyielding, commonly made of concrete, steel, or cast iron. Specific characteristics of each of the two types will be discussed later in this review.

Marston derived his theories of loading on the principle of three separate prisms of soil about the conduit. Two of these prisms were located adjacent to the conduit, one on each side, while the third was located directly over the conduit. The load theory completely ignores all cohesion between the sides of the ditch and the interior prism of soil. This premise is based on two assumptions which are:

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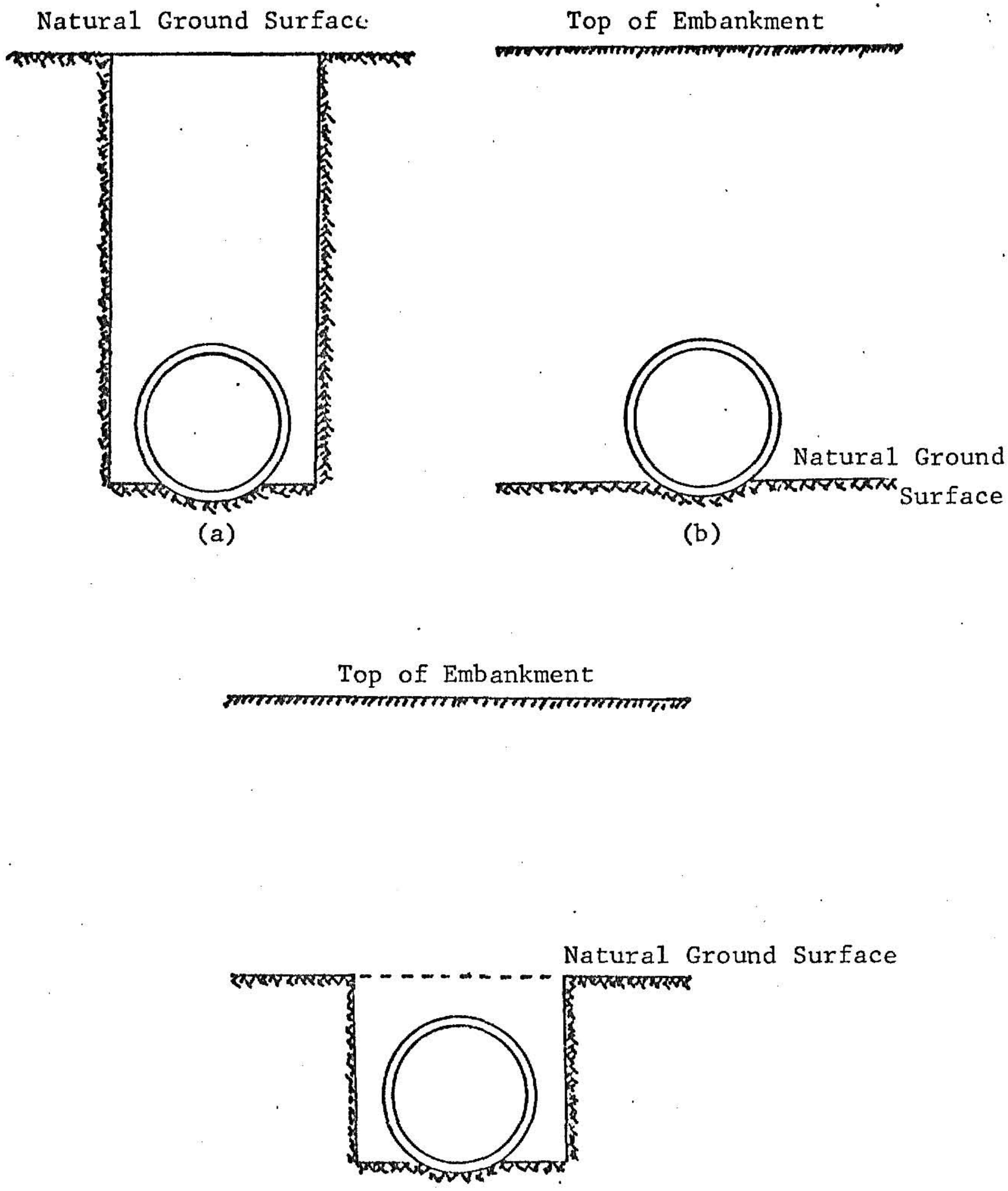


Figure 1

Classes of Conduits: (a) Ditch Conduit, (b) Positive Projecting Conduit, (c) Negative Projecting Conduit. [Taken from Spangler & Handy (2)]

1) that the cohesion takes time to develop, and 2) that by ignoring the possible cohesion, the conduit would be designed for the maximum load (4). This assumption is further clarified by Marston (5) in a later paper when he states that the cohesion is completely destroyed when the ditch is saturated, although in a dry state some cohesion is developed which reduces the load on the conduit.

However, Marston's load theory does take into some account the shearing forces between the sides of the interior prism and the sides of ditch which are developed by the lateral soil pressure multiplied by the tangent of internal friction of the soil. The direction of this force might be downward adding load to the conduit or upward reducing the load on the conduit, depending on the relative movement of the internal prism to the external prisms of soil. Shearing forces will increase the load on the conduit in the case when the external prisms of soil settle more than the internal prism. If the situation is reversed and the internal prism settles more than the two exterior prisms, the shearing forces will be oriented upward, thus reducing the load on the conduit.

The basic equation which Marston developed for the calculation of the maximum load which a rigid conduit will experience in a ditch is:

$$W_c = C_d \gamma B_d^{\,2}$$

W_c = load on conduit in pounds per linear foot

γ = unit weight of soil in pounds per cubic foot

B_d = width of ditch in feet

C_d = load coefficient which is

$$C_d = \frac{1 - e^{-2K_a u'(H/B_d)}}{2K_a u'}$$

e = natural log base

K_a = coefficient of active pressure which is

$$K_a = \mathrm{Tan}^2(45 - 0/2)$$

u' = coefficient of friction between ditch walls and interior prism

H = height of fill to top of conduit

Marston's load coefficient, C_d, takes into account several factors which include: 1) the ratio of the height of the fill to the width of the trench or conduit, 2) the shearing forces which develop between the interior prism of soil and the ditch sides, and 3) the direction and amount of settlement relative to the exterior and interior earth prisms (6).

Graphs have been developed to expedite the determination of the load coefficient, C_d. Such a graph is contained in Figure 2 (2).

When dealing with a flexible ditch conduit, the above equation should be multiplied by the ratio of the width of the conduit to the width of the ditch (B_c/B_d) which results in the equation

$$W_c = C_d \gamma B_d B_c$$

However, when using this equation, it is necessary that the soil immediately adjacent to the conduit be compacted to a condition where it offers the same resistance to settlement as that of the conduit. If a flexible conduit is used in soil

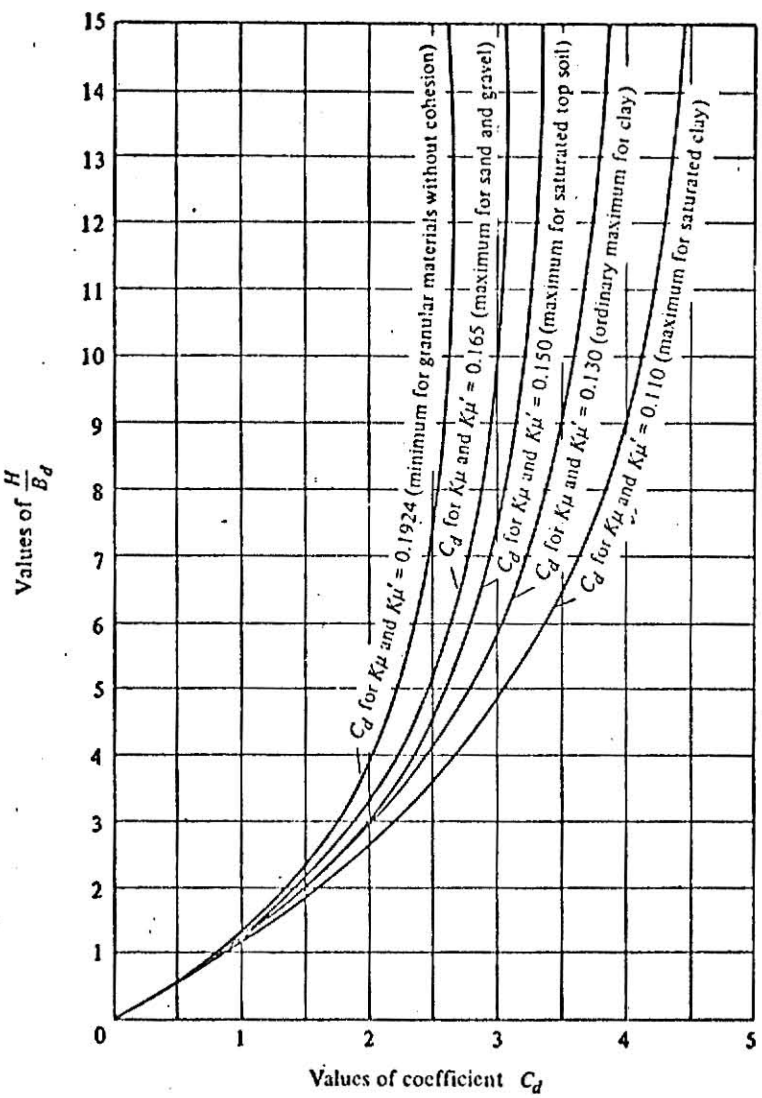


Figure 2

Graph for Coefficient "C_d" for Ditch Conduits [Taken from Spangler & Handy (2)] compacted to a stiffness significantly less than that of the pipe, W_c should be assumed to be equal to 80 percent of the load calculated for a rigid conduit (4).

When calculating loads on conduits, even in sloped-sided ditches, the critical width of the ditch is located just below the top of the conduit, or at the height of the 45 degree point on the pipe's circumference. Thus, this width should be utilized for the value B_d in the Marston load equations (5).

The other classes of conduits, previously mentioned, have similar corresponding load equations and graphs which are very well explained by Spangler and Handy (2) and Spangler (4). This study will mainly concentrate on flexible ditch conduits.

In order to grasp a comprehensive understanding of Marston's load theories, several important concepts must be noted. As may be deduced from the load equation for flexible ditch conduits, the total load on the conduit is dependent on the width of the ditch, the height of the fill, the diameter of the conduit, the coefficient of lateral active pressure, and the shearing forces, but independent of the conduit's shape and composition. It is noted later in a discussion about deflection, that the load is evenly distributed over the full width of the top of the conduit (4). Another important concept is the existence of a time lag between the initial loading or installation of the conduit and the occurrence of its maximum load. These delayed loadings might account for up to 20 to 25 percent of the total load in a granular soil (5). Marston (7) also notes that these maximum loads finally occur due to the ditch being completely flooded and saturating the fill. In some types of soils, this process may not occur until one or two years after the installation of the conduit.

Marston indicates that additional static loads on the surface over conduits should be handled through the use of the Boussinesq stress distribution theory based on the assumption of soil being a semi-infinite elastic solid. This theory and procedure is presented very well by Taylor (8).

In general, the load on the ditch conduit is dependent on overburden and surface loads, geometry of the system, relative stiffness of the structure to the soil, and the soils various characteristics (9). Marston (5) lists these characteristics as moisture content, internal angle of friction, temperature, and cohesion.

SUPPORTIVE STRENGTHS OF CONDUITS

Now, since the Marston load theory for conduits has been briefly discussed, it is appropriate to examine the characteristics and sources of supporting strength which enable conduits to resist such loads. Unlike the instance when the loads on conduits were considered, the supporting strength of conduits is greatly dependent on their shape and composition.

As previously noted, the vertical load on the top of the conduit is evenly distributed across the breadth of the pipe.

However, the vertical reaction through the bottom of the conduit is subject to the bedding characteristics of the ditch. As an example of this phenomenon, the maximum bending moment on a conduit lying flat on top of the surface without an indentation into the surface is $.294rW_c$. For the same conduit with the same identical load, however supported by a 90 degree bedding, the maximum moment of the conduit is $.157rW_c$ or 1.87 times less than the former described situation. In other words, the conduit in the latter situation could support a load almost twice as large as the conduit in the former case (2).

Thus, the supporting strength of ditch conduits is subject to some or all of three primary criteria: 1) the internal structural strength of the conduit, 2) the bedding characteristics of the conduit, 3) the lateral pressures which could develop (2).

Rigid and flexible conduits depend on different sources for development of their respective supportive strengths. The rigid conduit is solely dependent on its inherent strength and active lateral earth pressure, while the flexible conduit is subject only to the passive lateral earth pressure which may develop (4).

The rigid conduit, as already stated, depends mostly on its inherent strength which is largely based on its bedding characteristics. Load factors are assigned to a number of different types of bedding based on the 3-edge bearing test. These load factors vary from 1.5 for an ordinary bedding situation to 2.25 to 3.4 for a concrete-cradle bedding condition. Additional load factors for other specific bedding conditions for ditch and other classed conduits may be found in Spangler and Handy (2). Generally, the average supporting strength of conduits in a ditch is approximately 1.5 times greater than the strength found by the 3-edge bearing test (10). Proper bedding of conduits plays an additional role in cold areas of the country where severe frost penetration problems exist. When the bedding is uneven, the conduits experience points of high pressure concentrations which cause breakage and failures of the conduits (11). Because of its structural characteristics of not bending or deforming, the rigid conduit cannot develop lateral passive earth pressure, but must depend only on lateral active earth pressure (2).

The flexible conduit, usually a thin-walled structure, has very little inherent strength. Instead, it relies on its flexibility which enables it to rapidly deform, taking full advantage of the lateral passive earth pressure which develops along the sides of the conduit as it deforms. These lateral passive forces are parabolically distributed over the sides of the conduit (4). Terzaghi stated that movement is essential for the mobilization of these forces. He noted that for retaining walls, the movement or deflection required to take full advantage of passive forces is 1/1000 of the height of the wall (12). This supportive strength system allows these conduits to be light weight and easy to work with, yet still be able to support heavy loads such as high earth embankments.

In general, when considering the structural supportive strength of flexible conduits, it is imperative that the surrounding soil characteristics adjacent to the conduit be studied to the same extent or more than the characteristics of the conduit itself. Any conclusions which may be drawn from the examination or design of a buried conduit will be guided by several concepts pertaining to the conduit-soil system. They are: 1) vertical loads acting on the top of the conduit are distributed evenly over the full diameter or width of the conduit and may be calculated by using Marston's load theories, 2) the vertical reaction at the bottom of the conduit is equal to the vertical load distributed evenly over the width of the bedding contact of the conduit, and 3) the horizontal pressure is parabolically distributed over the middle 100 degrees of the sides of the conduit, since the top and bottom 40 degrees experience little movement (13). The maximum pressure occurs at the ends of the horizontal diameter where the maximum horizontal deflection occurs. As may be noted from Figure 3, the maximum force is equal to the modulus of passive resistance of the soil multiplied by one half the horizontal deflection. This theory will be thoroughly discussed later in this paper.

TYPES OF FAILURES

When the loading of a conduit is greater than the supportive strength available to the structure, failure will occur. The type of failure is controlled by the type of conduit involved. That is, a rigid conduit will fail by the

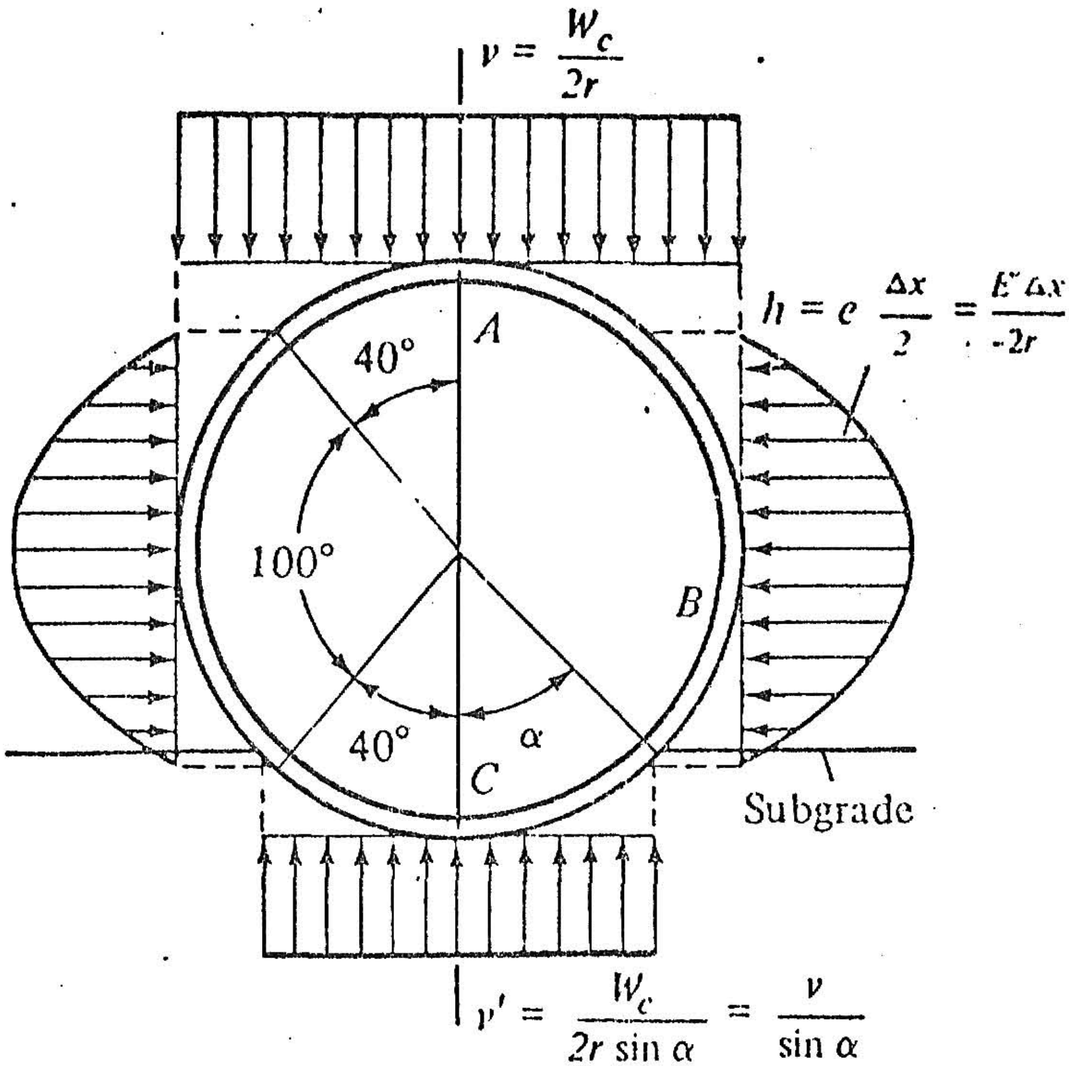


Figure 3

Assumed Pressure Distribution on
a Flexible Conduit

[Taken from Spangler & Handy (2)]

rupturing of its walls and a flexible conduit will fail by excessive deflection (4).

In the case of the rigid conduit, be it either made of steel or concrete, failure is induced by overstressing the walls of the conduit which causes cracking and eventual rupturing of the walls. The overloading or overstressing is usually attributable to pressure concentrations occurring along the top or sides of the conduit. These pressure concentrations occur because of the inability of the rigid conduit to deform which redistributes the loads.

In contrast to rigid conduits, flexible conduits do not fail by rupture because of their ability to accept deformation without structurally damaging themselves. However, there is a limit to the deflection which defines its point of failure on the basis that the conduit is unable to perform up to its designed standards. Flexible conduit failures usually are defined as deflections ranging from 5 to 20 percent (percentage based on the ratio of the vertical deflection to the diameter of the conduit) (14), depending on the actual flexibility of the structure.

When the vertical load is applied to a flexible conduit, the vertical diameter decreases and the horizontal diameter increases, triggering the lateral passive pressure which was described above. After this process has continued, the top of the conduit will be completely flattened. If more load is applied to the top of the conduit, the curvature of the pipe will become reversed or inverted. When this situation occurs, the sides of the conduit will be eventually pulled in toward the center of the conduit, thus negating the passive pressure developed in the adjacent soil. At this point, the only source of supportive strength available to the conduit is its own inherent strength. Obviously, this strength is not sufficient to resist the load and complete collapse will occur. Figure 4 illustrates the various stages of failure which a flexible conduit passes through.

SPANGLER'S IOWA FORMULA

In order to be able to properly design buried conduits, a rational method for the prediction of the conduit's performance under load must be available. To evaluate the rigid conduit's expected performance, basic concrete and steel structural design methods may be employed. But for the flexible conduit, a more complicated situation exists because of the interaction and mutual dependency of the surrounding soil and the conduit. The best procedure for evaluating the suitability of flexible conduits is to predict the deflection which may be expected to occur (2).

Between 1937-1942, M. G. Spangler of Iowa State University in conjunction with the Bureau of Public Roads developed the first and still most viable theory of predicting deflections for buried flexible conduits (15). This theory which is popularly known as the "Iowa Formula" took the form of this equation

$$\Delta X = \frac{D_1 \, K \, W_c \, r^3}{EI + .061 e r^4}$$

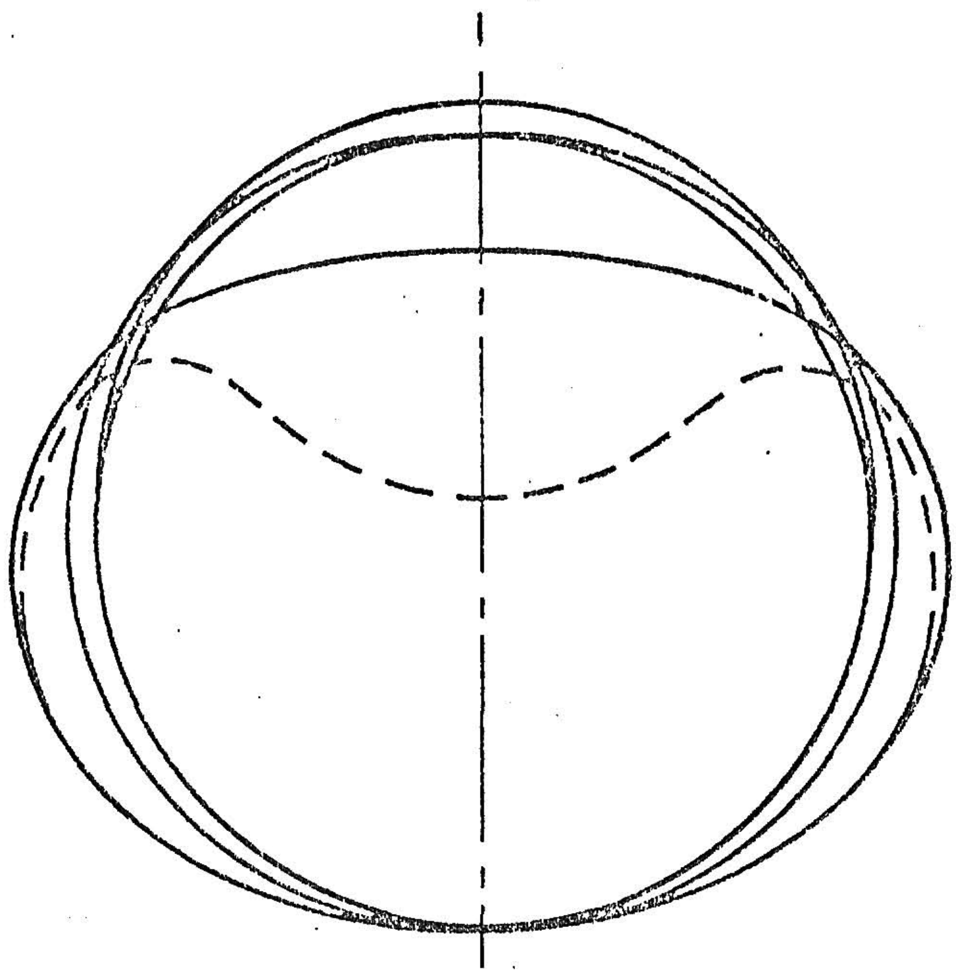


Figure 4

Stages of Deflection of a Flexible Conduit
[Taken from Spangler & Handy (2)]

ΔX = horizontal deflection in inches

D_1 = deflection time lag factor

K = bedding factor

W_c = Vertical load in pounds per inch

r = mean radius of pipe in inches

E = modulus of elasticity of pipe material in pounds per square inch

I = moment of inertia in $(\text{inch})^4$ per inch

e = modulus of passive resistance of soil in pounds per square inch per inch

The deflection time lag factor, "D_1" in the above equation may amount up to 2.0 but usually ranges between 1.25 and 1.50. Spangler (16) notes on the basis of working with flexible steel culverts that the deflection time lag factor, D_L, varies inversely with "e", the modulus of passive resistance. "K" or the bedding factor is subject to the bedding angle shown below (6):

Bedding Angle (degrees)	K_1 Constant
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

"W_c", the vertical load on the conduit is calculated by Marston load theory. The modulus of elasticity and moment of inertia for various types of conduits may be readily secured from tables in pertinent journals and books. This leaves the term "e" or the modulus of passive resistance as the only variable left in the Iowa Formula to be evaluated.

MODULUS OF PASSIVE RESISTANCE

The modulus of passive resistance is the most complicated variable to analyze. It is the measure of the rate of change of lateral pressure with respect to lateral displacement (17). In other words, the modulus describes the constant relationship of the horizontal pressure on the pipe to the horizontal movement of that pipe. It is further described as being similar to Westergaard's modulus of subgrade reaction (18) for concrete pavements and Cumming's modulus of foundation (19) related to his work on the stability of foundation piles. Being a function of the density of the soil, the modulus of passive resistance is developed through consolidation of the sidefill by lateral and vertical pressures. In working on various corregated pipe culverts, Spangler found that hand tamping the soil would double "e" (13). For an extreme example, Spangler (4) notes that the modulus for passive resistance of a heavy clay in its natural state varied from 4 to 8 pounds per square inch. However, when the clay was pneumatically packed, the value of "e" increased to 51 pounds per square inch. Moser, Watkins, and Bishop (20) verified the importance of density to the successful performance of flexible conduits by stating that the density of the soil is the most important soil characteristic. The degree of compaction is as important as compaction itself. Spangler and Donovan (15) found that the modulus of passive resistance in soil compacted at 75 percent to 80 percent Proctor density contributed very little to the strength of flexible pipe culverts. However, when the soil was densified to 90 percent Proctor, the modulus of passive resistance increased tremendously.

In his extensive work on corregated pipe culverts, Spangler (13) noted that measurements at 1 foot increments in the fill indicated that the ratio of the modulus of passive resistance to the horizontal deflection was constant regardless of the height of the fill. He also found that this ratio, $e/\Delta X$, for gravel was 2.5 times greater than the $e/\Delta X$ for loamy soil. Spangler listed several average values of "e" for some typical types of soil. This data included the following:

TYPE OF SOIL	VALUE OF "e" (in pounds per square inch per inch)
Black silty loam (not compact)	14
Well-graded gravel (not compact)	32
Yellow sandy clay loam (not compact)	13
Yellow sandy clay loam (tamped and dry)	27

As may be noted from the above table, the values for "e" varied considerably for different types of soil.

Thus, some of the early conclusions concerning the modulus of passive resistance included: 1) that "e" varies proportionly with the density of the soil, 2) that "e" is independent of height of fill, and 3) that "e" depends on soil properties.

MODULUS OF SOIL REACTION

After much research and work through the years, Watkins and Spangler applied the principle of similitude to the problem of properly evaluating "e". The results of their work indicated that "e" varied inversely with "r" (where "r" equals the radius of the conduit). Also, they found that instead of "e" remaining constant for a given set of conditions, as previously thought, "er" was in reality constant. The product of "er" is called the modulus of soil reaction (E') measured in pounds per square inch. This new modulus was found to obey the conclusions listed above which referred to the modulus of passive resistance (17).

Thus, with this new development, the Iowa Formula was slightly revised to the following form (2):

$$X = \frac{D_1 \; K \; W_c \; r^3}{EI + .061E'r^3}$$

X = horizontal deflection in inches

D_1 = deflection time lag factor

K = bedding factor

W_c = vertical load in pounds per inch

r = mean radius of pipe in inches

E = modulus of elasticity of pipe material in pounds per square inch

I = moment of inertia in $(\text{inch})^4$ per inch

E' = er = modulus of soil reaction in pounds per square inch

e = modulus of passive resistance in pounds per square inch per inch

In order to more easily understand the meaning of this equation, it may be rewritten as follows:

$$\frac{\Delta X}{D} = \frac{D_L\ K\ W/D}{\frac{EI}{r^3} + 0.061E'}$$

or

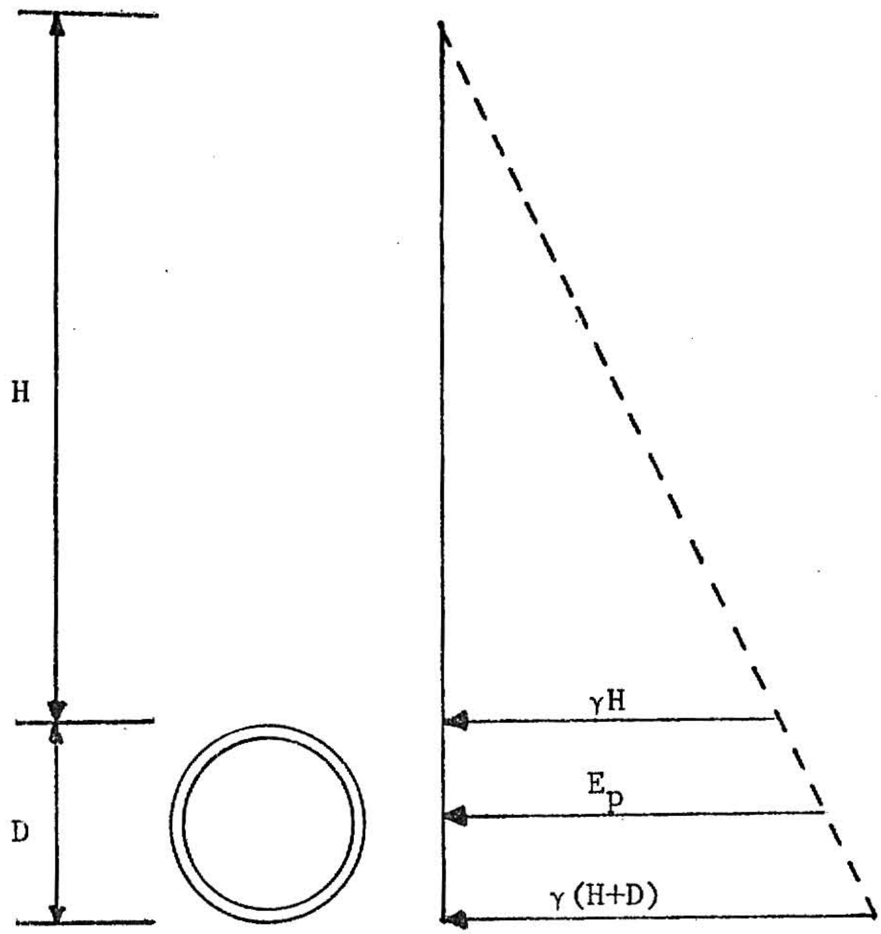
$$\begin{array}{c}\text{Percentage}\\ \text{of}\\ \text{Pipe Deflection}\end{array} = \begin{array}{c}\text{Deflection}\\ \text{Time Lag}\\ \text{Factor}\end{array} \quad \frac{\text{Load or pipe}}{\begin{array}{c}\text{Pipe}\\ \text{modulus}\end{array} + \begin{array}{c}\text{Soil}\\ \text{modulus}\end{array}}$$

As maybe noted, the load on the pipe is determined by the overburden load and the bedding constant. The pipe stiffness or pipe modulus $\left(\frac{EI}{r^3}\right)$ may be determined by the 3-edge test (21).

The modulus of soil reaction has a maximum limiting value defined by Coulomb's and Rankine's classical earth pressure theories (22). Figure 5 illustrates the position and derivation of this maximum limiting force on the conduit (23). However, the amount of movement or deflection required to develop this force is yet unknown. For further explanation of Coulomb's and Rankine's work, Bowles (24), Tschebotarioff (25), or the Navy Design Manual (26) may be consulted.

TESTS TO DETERMINE MODULUS OF SOIL REACTION

Since the prediction of deflection by the use of the Iowa Formula is so dependent on the proper value of the modulus of soil reaction, E', intensive research has been conducted to determine a relationship between E' and typical soil tests. This accomplishment would enable a standard, accurate procedure for the determination of E', thus expediting the calculation of flexible conduit deflection.



E_p = Passive Earth Pressure $\qquad$ H = Height of Fill

γ = Unit Weight of Soil $\qquad$ D = Diameter of Pipe

$K_p = \tan^2(45° + \phi/2)$

$$E_p = \frac{\gamma K_p}{2}(H+D)^2 - \frac{\gamma}{2} K_p (H)^2$$

$$E_p = \frac{\gamma}{2} K_p D(2H+D)$$

Figure 5

Passive Earth Pressure Diagram
[Taken from Williams (23)]

Nielson (27) tried to correlate the triaxial shear test to the determination of E'. He used the theory of elasticity with a fine sand and developed this equation (28)

$$E' = 1.5M^* = \frac{1.5\ E_s\ (1 - u_s)}{(1 + u_s)(1 - 2u_s)}$$

E' = modulus of soil reaction in pounds per square inch

E_s = modulus of elasticity of soil in pounds per square inch

u_s = Poisson's ratio of soil

M* = constrained modulus of elasticity

Two years later, in 1969, Nielson, Bhandhausavee, and Yeb (28) published a paper concerning the relationships of E' to the California Bearing Test (CBR), various soil characteristics, and the Hveem's stabiliometer and the consolidation test. In working with the CBR test, the strength of the subgrade soil was measured by penetrating the soil by forcing a plunger of standard size at a specified rate. Based on the theory of a rigid piston against a plane boundary of semi-infinite elastic solid, the following equation was formulated

$$\Delta = \frac{a\ p\ (1 - u_s^{\,2})}{2E_s}$$

Δ = displacement of piston

a = radius of piston

p = unit load

u_s = Poisson's ratio of the soil

E_s = modulus of Elasticity of soil

However, they found that satisfactory results from this equation were produced only from granular soils. Their work with other soil properties found that E' is a function of dry density, the density of the soil at AASHO T180 compaction, moisture content, plastic index, mean radius of pipe, and the difference between AASHO T180 and T99 soil compaction. Nielson, Bhandhausavee and Yeb also concluded that the density at the soil's optimum moisture content was the most important factor in developing E' although indicating little confidence in their correlation by composing a table of E' and their corresponding d/180 values and stating that an error of 100 percent was possible. After they worked with the Hveem's stabiliometer and the consolidation test, they indicated reasonable results could be expected for the determination of E' for only small deflections of 3 percent or less. It should be noted that the results of these research works should be considered only with a limited amount of confidence, since it isn't valid to assume that any or all types of soils are purely elastic.

CHARACTERISTICS OF POLYVINYL CHLORIDE

In the past, most flexible conduits were manufactured from corregated steel. However, through advancements in the plastics industry, conduits now can be readily made of a variety of materials. Presently conduits are made of three general materials: 1) reinforced thermoset resin (RTR), 2) thermoplastic, and 3) corregated steel. The RTR pipe includes reinforced-plastic-mortar (RPM) pipe and fiberglass reinforced (FRP) pipe. Thermo plastic pipe includes polyethylene (PE) pipe and polyvinylchloride (PVC) pipe (21).

Research has indicated that PVC pipe has very desirable characteristics. These include its ability to retain its flexibility at -40°F and its rigidity at 150°F (29). In addition to these properties, PVC can attain a tensile strength of 3000 psi at 212°F, possess a modulus of elasticity of 400-480 KSI, and be highly resistant to many chemicals (30).

RECENT RESEARCH IN FLEXIBLE CONDUITS

Intensive research on the actual behavior and characteristics of deflected flexible buried conduit started by dealing exclusively with corregated steel culverts. However, with the aforenoted developments in the plastics industry, researchers have begun work with plastic conduits and their relationships to the flexible steel conduit.

White and Layer (31) noted that the compression experienced in a flexible steel culvert is equal to the normal pressure or overburden multiplied by the radius of the structure or pipe. They also seemed to verify the results of previously noted research by Marston, Spangler, and others by reaching three conclusions: 1) the minimal load on the corregated steel culverts will occur when the conduit is placed on a "soft" foundation between two "hard" sides, 2) the maximum load on the corregated steel culvert will occur when the conduit is placed on a "hard" foundation between two "soft", compressible sides, and 3) deflection will be minimized by placing the culvert on a uniform foundation with compacted sides.

Pressure on the top of a corregated culvert was found to equal only 60 percent of the height of the fill multiplied by its density. In order to account for the reduction of load on the culvert, Nielson (32) worked on the theory of soil arching occurring over the culvert. He used the theory of elasticity to determine the location of the maximum shearing stress in the soil arch above the conduit and developed a differential equation to evaluate its value. However, he concluded that more actual field tests were required to properly corroborate his lab findings.

Also, concerning the phenomenon of a reduction in loading by backfill, Tschebotarioff (33) theorizes that after the ratio of the height of the fill to the width of the ditch, H/B_d, reaches 9, no additional pressure will be experienced on the conduit. This is based on the soil to pipe friction angle being equal to the soil's internal angle of friction.

Watkins and Smith (34) conducted research on the behavior of several types of cement-mortar lined steel pipe. They attempted to design these flexible conduits on the basis of buckling strength in their walls. However, this failed because of the ring's deflection, due to the compressibility of the soil.

A table was developed which correlated deflection reduction factors to the ratio of the width of the ditch to the width of the conduit as follows:

B_d/B_c*	Reduction Factor
1.5	.86
2.0	.92
3.0	.98
∞	1.00

*For rigid, vertical trench walls

However, Watkins and Smith surmised that these factors should be ignored and serve only as safety factors.

In summary, Watkins and Smith concluded that: 1) the ring stiffness factor of the pipe becomes more important in loose soil than compact soil, 2) the wall thickness has minimal effect in high density backfills, 3) relative to deflection, ideal densities occur at 90 percent proctor density, and 4) deflections in flexible conduit may be further reduced at 95 percent Proctor density, but the reduction is insignificant.

In further research on corregated culverts, Watkins and Moser (35) stated the most important characteristics of the soil-pipe system includes: 1) soil density, 2) yield point of steel, and 3) ring stiffness. Also, as a result of later work, they concluded that the critical soil density of a backfill surrounding a corregated steel culvert occurred at 75 to 80 percent standard Proctor (36). This value is slightly less dense than that stated by Watkins and Smith as previously noted.

Howard (37), in working on steel pipe, discovered that in a 90 percent Proctor densified backfill, flexible pipe with a ring stiffness factor (RSF) of over 10 psi failed elliptically while pipe of a RSF of 2 psi or less failed rectangularly with its corners at 45°. When deflecting pipe in a 100 percent Proctor density backfill, pipe with a RSF of 9 psi or greater failed elliptically while pipe with RSF of less than 4 psi failed rectangularly. Pipe with ring stiffness factors varying between the two limits failed in shape somewhere between a rectangle and an ellipse. The failure of these steel pipes varied between 16 to 22 percent deflections. Howard noted that in the 100 percent Proctor densified backfill, none of the elliptical shaped steel pipe collapsed, whereas the rectangular failed pipe buckled and collapsed.

An 84-inch wide soil testing bin was used with 18, 24, and 30-inch diameter steel pipe. The ratio of the horizontal to vertical deflection (x/y) varied between 0.6 to 0.9. Howard also found that 1/2 of the soil movement occurred within 6 to 9 inches of the pipe with the amount of movement varying linearly to the wall. After comparing the different curves describing these movements, Howard concluded that the walls of his soil testing bin did interfere with the horizontal deflection of the pipe.

Concerning pressures which developed on the pipe around its circumference, Howard found the stiffer the pipe, the more pressure was exerted on its top. However, this pressure amounted to only 50 to 75 percent of the applied load. The pressure on the bottom of the pipe is largely independent of the other pressures applied to the pipe, but is related only to its bedding condition. Again pointing to the interference of the soil bin walls, soil pressures on the walls were determined to be 50 percent of the surcharge load in a 90 percent Proctor densified backfill and 75 percent of the applied surcharge in a 100 percent Proctor density backfill.

Howard and Selander (21) conducted work for the Bureau of Reclamation on steel, PVC, PE and other various kinds of pipe. They noted that all types of pipe deflected the same in high density backfills. However, when the pipe was deflected in a low density backfill, different types of flexible pipe with similar stiffness values experienced deflections which varied up to 300 percent. This result again verifies the fact that proper backfilling procedures are of the utmost importance.

A one-day testing period is used for all measurements. The pipe modulus of the various types of pipes tested was found by the 3-edge bearing test. All of the sides of the soil tank were coated with petrolatum and covered with a 2 millimeter polyethene film. The pipes were loaded in a large soil tank by a universal testing machine. Densification of the soil was completed after being placed in the soil bin in loose lifts. During the soil compaction phase of the study, stiffeners and braces were placed in and around the sample to prevent deformation and movement. These braces and stiffeners were removed when actual testing commenced.

Howard and Selander discovered that E' for a given densified soil is constant regardless of the diameter of the pipe. They also drew several conclusions concerning the behavior of steel pipe. These results included: 1) that the term EI/r^3 (the pipe modulus) has a decreased effect on flexible pipe deflection when E' (modulus of soil reaction) is increased, 2) that x, the horizontal deflection, is inversely proportional to EI/r^3 in low E' soil, and 3) that steel pipe of equal stiffness have equal deflections independent of diameter or wall thickness. Other notable results of their work were: 1) the discovery that PVC conduit deflected similarly to steel pipe with the same stiffness in a low E' clay, 2) that PVC pipe deformed elliptically to semi-elliptically, and 3) that PVC pipe deflected up to 29 percent without structural damage (38).

It is pointed out by Williams (23) that the research results by Howard and Selander could be slightly questionable because of two significant facts: 1) the adhesion in the pipe and soil system was not considered, 2) the walls of the soil tank were too close to the pipe being tested. The adhesion between the soil tank walls and soil was complicated by the application of the petrolatum and polyethylene film on the tank's interior. In addition, the close proximity of the soil bin walls influenced the lateral stress distribution of the soil.

In contrast to the previously noted results, Moser, Watkins and Bishop (20) concluded that there is an optimum diameter to thickness ratio (D/t) of 37 to 38 for minimal deflection at a constant soil pressure or overburden load. This result is illustrated in a graph plotting D/t versus Y/D (strain or percentage of deflection) which shows that the strain in the pipe walls decreases with an increasing D/t ratio until the curve reaches a ratio of 37. Then the strain increases with increasing D/t ratios. This phenomena is explained by stating that if the ring is too flexible (D/t > 37 - 38), the pipe will correspond its shape with the hard and soft spots in the soil and lose its "ideal elliptical shape." On the other hand, if the pipe is too stiff (D/t < 37), then it will act like a rigid body and tend to concentrate additional pressure on its circumference. In completing additional work for Johns-Manville (39), Bishop and Moser compiled a tabulation of predicted long-term deflection values for various percentages of ASTM D-698-72 densities. These values, again, point out the importance of compacting the soil around the pipe for sand and clay.

CHARACTERISTICS OF IDEAL EMBEDMENTS

Figure 6 illustrates an ideal situation which minimizes deflection in flexible conduits by utilizing to the fullest advantage the principles and conclusions previously discussed in this review. Note, if the conduit was placed in a ditch, the "A" portion would consist of the natural ground.

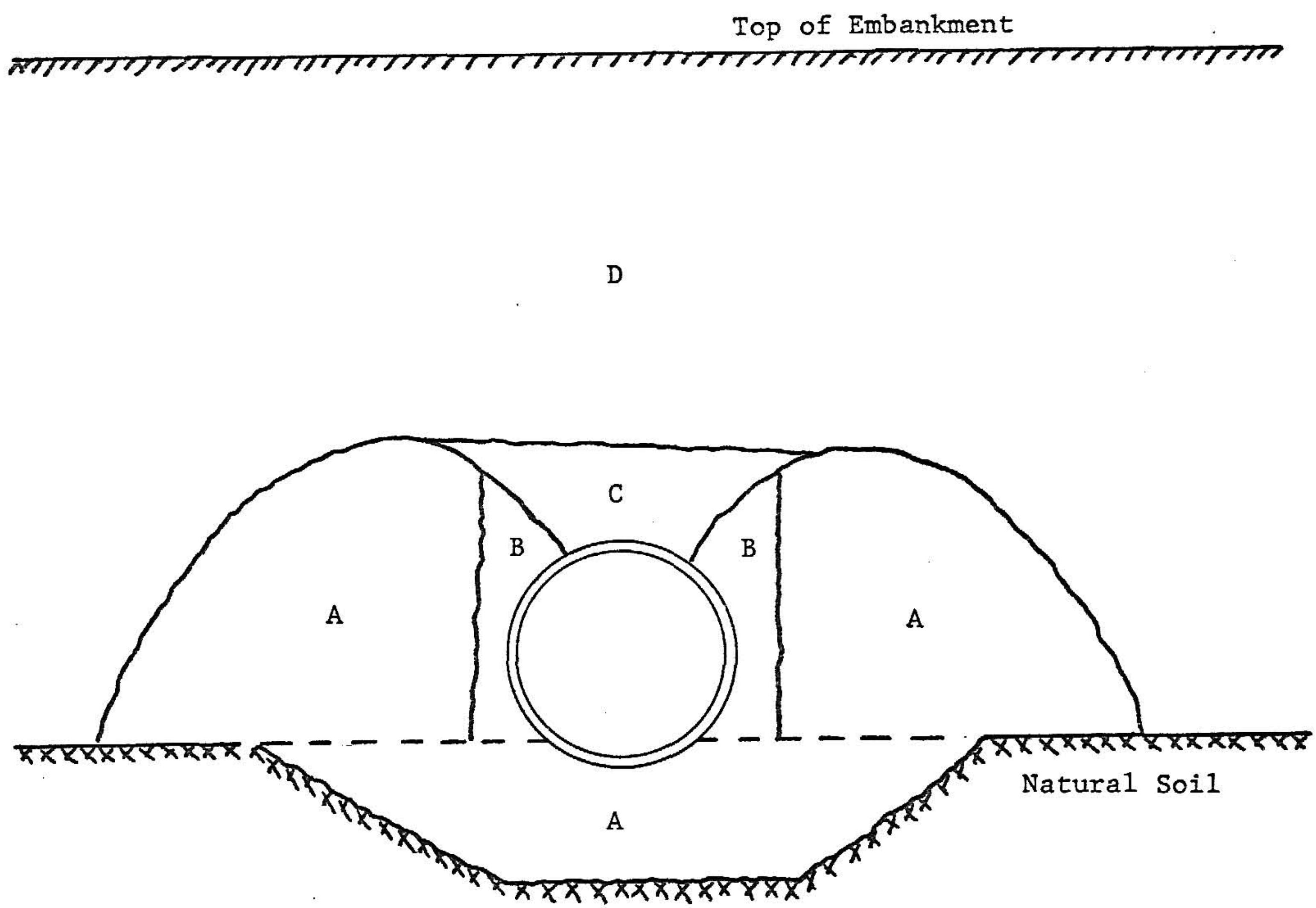


Figure 6

Ideal Conduit Emplacement
[Taken from Tschebotarioff (33)]

Tschebotarioff recommends using granular material for portions "B", "C", and "D", since clay cannot be easily compacted in the narrow confines of a trench (33).

IMPORTANCE OF PIPE TO SOIL ADHESION

The theories concerning loading and deflection of buried conduits have tended to ignore the adhesion between the pipe and the soil. This omission has had little effect on the validity of the theories when applied to concrete, vitrified clay, or even corregated steel because their wall friction angles are of a significant magnitude as may be noted from the following wall friction values set forth by Teng (40).

MATERIAL	ANGLE OF WALL FRICTION
Steel pile coated with tar or bitumen	30°
Concrete or brick walls	20°
Uncoated steel pile	15°

Also in reference to these materials, Terzaghi and Peck (41) stated that the wall friction should be treated as two-thirds the normal internal angle of friction. In addition, Huntington (42) agreed with that assumption by stating that in the absence of any laboratory data, the wall friction angle should be considered as two-thirds ϕ. Thus, the wall friction angle does not vary significantly with the soil's internal angle of friction. However, these values of wall friction do not necessarily apply to PVC pipe.

The following is an explanation illustrating the importance of friction and adhesion between the pipe and soil to the performance of that pipe.

Based on Spangler's theories, the flexible conduit deforms as pictured in Figure 4. This deformation is the result of the pressure distributions illustrated in Figure 3. Spangler (13), in developing this theory, assumed that the flexible circular conduit would deform as a circular ring would deform when subjected to vertical compressive loads at its top and bottom. Thus, the assumed parabolically shaped lateral pressure distribution comes into existence. In addition, he formulated the following equations which evaluate the thrust stresses which occur at any point "D" in the pipe wall due to the applied vertical loads.

$(0° < \theta < \alpha)^*$

$$R_D = W_c\left(\frac{.5 \sin^2 \theta}{\sin \alpha} - B \cos \theta\right)$$

$(\alpha < \theta < 90°)^*$

$$R_D = W_c(.5 \sin \theta + B \cos \theta)$$

$(90° < \theta < 180°)^*$

$$R_D = W_c(.5 \sin^2 \theta + B \cos \theta)$$

*θ commences at point "C" in Figure 3.

The equations for the thrust stresses due to the horizontal loads are as follows:

$(0° < \theta < 40°)$

$$R_D = .511\ hr \cos \theta$$

$(40° < \theta < 140°)$

$$R_D = hr(\cos^2\theta - .568\cos^4\theta)$$

$(140° < \theta < 180°)$

$$R_D = .511\ hr\cos\theta$$

R_D = Thrust stress

W_c = Marston's load on the conduit

α = Embedment angle

r = Radius of the conduit

h = Lateral pressure on the conduit

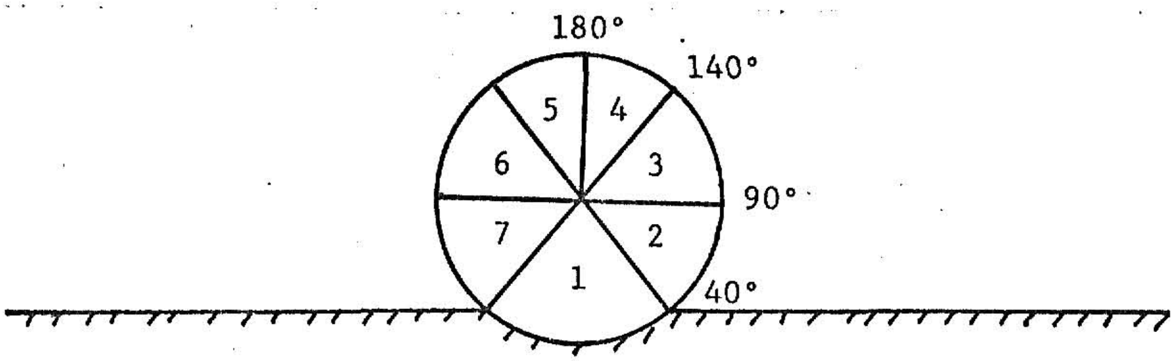
B = Coefficient based on the embedment angle listed in the following table

Embedment Angle (α)	B
0	0.053
15	0.050
30	0.040
45	0.026

The thrust force determined by these equations is the force which acts perpendicular to the surface of the pipe. In other words, R_D is the normal force which acts through the soil particles which surround the pipe. Thus, when the pipe deforms, any slippage between the pipe surface and the adjacent soil particles will be opposed by a frictional shearing force.

$$\text{Friction (F)} = R_D \text{ Tan } \phi_{p.s}$$

This frictional force will extend around the periphery of the pipe with the exception of the embedded section of pipe. There, the soil particles and the pipe surface will not experience significant slippage. Figure 7 is a summary of the results of the integration of the frictional forces around the circumference of a pipe with an embedment angle of 40°. Note that the frictional forces as a result of the horizontal loading in sections 4 and 5 are of a negative value, thus they are equal to zero. This is because a negative frictional force is a tensional force, and soil cannot exert a tensional force.



SECT.	FRICTIONAL FORCES From Horizontal Loading	From Vertical Loading	Total
1	0	0	0
2	.150 hr. tan $\phi_{p.s}$	.441 W_c tan $\phi_{p.s}$	.150 hr. tan $\phi_{p.s}$ + .44 W_c tan $\phi_{p.s}$
3	.148 hr. tan $\phi_{p.s}$	.328 W_c tan $\phi_{p.s}$	.148 hr. tan $\phi_{p.s}$ + .328 W_c tan $\phi_{p.s}$
4	-.643 hr. tan $\phi_{p.s}$ = 0	.040 W_c tan $\phi_{p.s}$	.040 W_c tan $\phi_{p.s}$
5	-.643 hr. tan $\phi_{p.s}$ = 0	.040 W_c tan $\phi_{p.s}$	.040 W_c tan $\phi_{p.s}$
6	.148 hr. tan $\phi_{p.s}$	.328 W_c tan $\phi_{p.s}$	.148 hr. tan $\phi_{p.s}$ + .328 W_c tan $\phi_{p.s}$
7	.150 hr. tan $\phi_{p.s}$	.441 W_c tan $\phi_{p.s}$	.150 hr. tan $\phi_{p.s}$ + .441 W_c tan $\phi_{p.s}$

Figure 7

Frictional Forces Acting on a Buried Conduit

EXPERIMENTAL DESIGN AND PROCEDURE

The sample of sand used in this experiment was obtained from Riley County, near Manhattan, Kansas. It was found in a water-deposited land form.

This experimental research was composed of two phases: a preliminary phase which entailed the determination of the basic physical properties of the soil, and the final phase which primarily involved the evaluation of P.V.C. pipe-to-soil adhesion in terms of cohesion and angle of friction. The basic physical properties of the soil were defined by Specific Gravity, Atterberg Limits, Gradation, Moisture Content, and Internal Angle of Friction. For the final phase, the soil-to-pipe adhesion at various normal pressures was determined. This procedure allowed the evaluation of the cohesion and angle of friction at the pipe-soil interface.

The following is a summary of the procedural specifications followed while conducting the various tests in both phases. Tests which were run during the preliminary phase followed A.S.T.M. criteria. During the final phase, the soil was screened through a No. 4 sieve to remove any large, irregularly-shaped particles which would cut or punch through the diaphragm when air pressure was applied. There was no attempt to compact or densify the soil in the device other than insuring that there were no large pockets or gaps in the soil blanket surrounding the pipe.

Before attempting the final phase of the research, a special device as shown in Figure 8 was built to accomplish

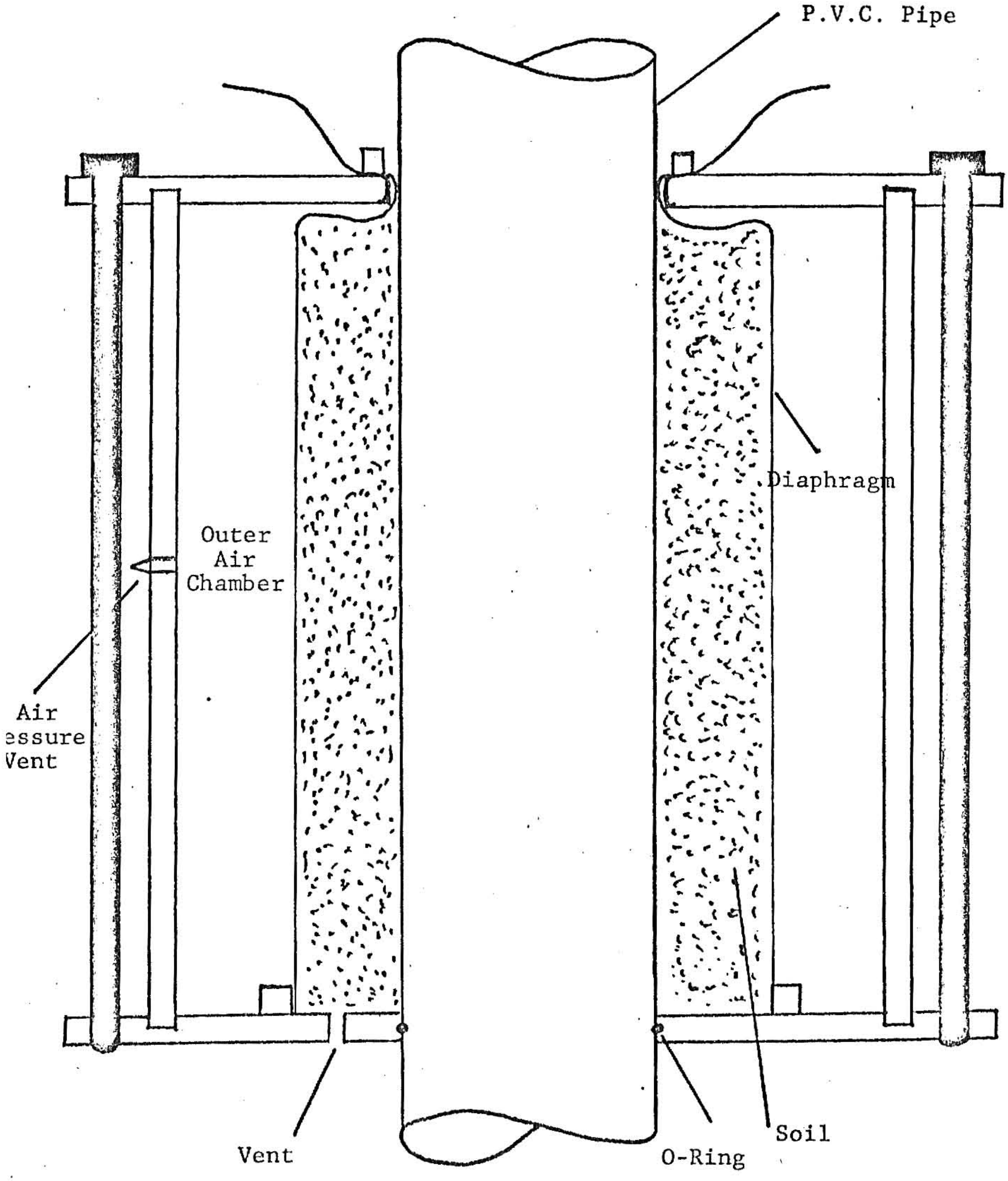


Figure 8

Shear Testing Device

Not to
Scale

the work. It was constructed of 1/4-inch steel plate. Two layers of polyethylene film (.0084 in. thick) were used for the diaphragm. The pressure on the soil was obtained by introducing air pressure produced by two small pumps into the space between the outer chamber wall and the diaphragm. In turn, the diaphragm applied pressure to the soil which was located between the diaphragm and the pipe. The air pressures used in these experiments varied up to 13.5 pounds-per-square-inch.

Loading of the pipe in the longitudinal direction was accomplished by the use of a hydraulic jack with a hand pump. The pressure was measured by a load ring which was placed between the jack and a frame. Inconsistency was encountered in using the hand pump due to the difficulty in applying pressure at a uniform rate. A brief description of the testing procedure for this portion of the research included: 1) introduction of normal pressure, 2) application of shearing force, 3) continuation of shearing force until a displacement of the pipe approximately .1 to .3 of an inch was reached, and 4) the relocation of the pipe to its original position.

The torque was applied to the pipe for the evaluation of the radial shearing stress by the use of a system consisting of a lever and weights. Two holes, 180° apart, were drilled through one end of the P.V.C. pipe. A 1-inch steel pipe was then placed through these two holes providing a lever. Then weights were added to the end of the lever at a measured distance. This was continued until a slippage between the soil and the pipe occurred. As for the testing procedure, it was similar to the testing procedure for the longitudinal shear which was previously described in this section. Although in this case, the pipe was not returned to its original position after each successive pressure application. Difficulty in acquiring accurate results was also encountered in this portion of the study due to the crude fashion in which the torque was applied to the pipe.

RESULTS

DESCRIPTION OF SAND

A brief description of the sand utilized in this research is set forth by the following summary of its physical characteristics:

Moisture Content (w):	0.24%
Specific Gravity (G_s):	2.64
Atterberg Limits:	Non-Plastic
Cohesion (c):	0.0
Internal Angle of Friction (ϕ):	40.5°
Unit Weight:	101.5 Pcf.
Classification:	A.A.S.H.O. - A-3 UNIFIED - SP

TABLE I

MOISTURE CONTENT

Determination No.	1	2	3
Weight of can plus wet sand	447.20g	331.12g	433.42g
Weight of can plus dry sand	446.21g	330.46g	432.60g
Weight of can	52.90g	52.24g	53.89g
Weight of water	.99g	.66g	.82g
Weight of sand	393.31g	278.22g	378.71g
Moisture content (percentage)	.25	.24	.22

Moisture Content = .24%

TABLE II

SPECIFIC GRAVITY

Determination No.	1	2	3
Bottle No.	4	4	4
Weight of Bottle + water + soil, W_1, in grams	767.54	773.44	761.24
Temperature, T, in °C	23.2°	28.0°	23.6°
Weight of Bottle + water, W_2, in grams	674.22	673.64	674.17
Weight of Soil, W_s, in grams	150.00	160.00	140.00
Specific Gravity of water at T, G_T	.9975	.9963	.9974
Specific Gravity of soil, G_s	2.64	2.65	2.64

$$G_s = \frac{G_T W_s}{W_s - W_1 + W_2} \qquad G_s = 2.64$$

TABLE III

SIEVE ANALYSIS

Sieve No.	Sieve Opening (in mm.)	Weight Soil Retained (in grams)	Percent Retained	Cumulative Percent Retained	Percent Finer
1st DETERMINATION					
4	4.76	0.00	0.0	0.0	100.0
1.7 mm.	1.70	28.97	4.1	4.1	95.9
1.4	1.40	16.19	2.3	6.4	93.6
1.18	1.18	25.43	3.6	10.0	90.0
1.0	1.00	37.80	5.3	15.3	84.7
850 um.	0.85	179.23	25.4	40.7	59.3
90	0.09	419.45	59.3	100.0	0.0
63	0.063	1.34	0.0	--	
53	0.053	0.59	0.0	--	
		0.72*	0.0	--	
2nd DETERMINATION					
4	4.76	0.00	0.0	0.0	100.0
10	2.00	19.12	2.7	2.7	97.3
20	0.84	120.99	17.2	19.9	80.1
40	0.42	362.69	51.3	71.2	28.8
60	0.25	146.83	20.8	92.0	8.0
140	0.105	50.30	7.1	99.1	0.9
200	0.074	3.73	0.5	99.6	0.4
		3.08*	0.4	100.0	0.0
3rd DETERMINATION					
4	4.76	0.00	0.0	0.0	100.0
10	2.00	18.85	2.7	2.7	97.3
20	0.84	126.33	18.2	20.9	79.1
40	0.42	391.50	56.4	77.3	22.7
60	0.25	124.72	18.0	95.3	4.7
140	0.105	26.71	3.8	99.1	0.9
200	0.074	3.23	0.5	99.6	0.4
		2.57*	0.4	100.0	0.0

*Passed through last sieve.

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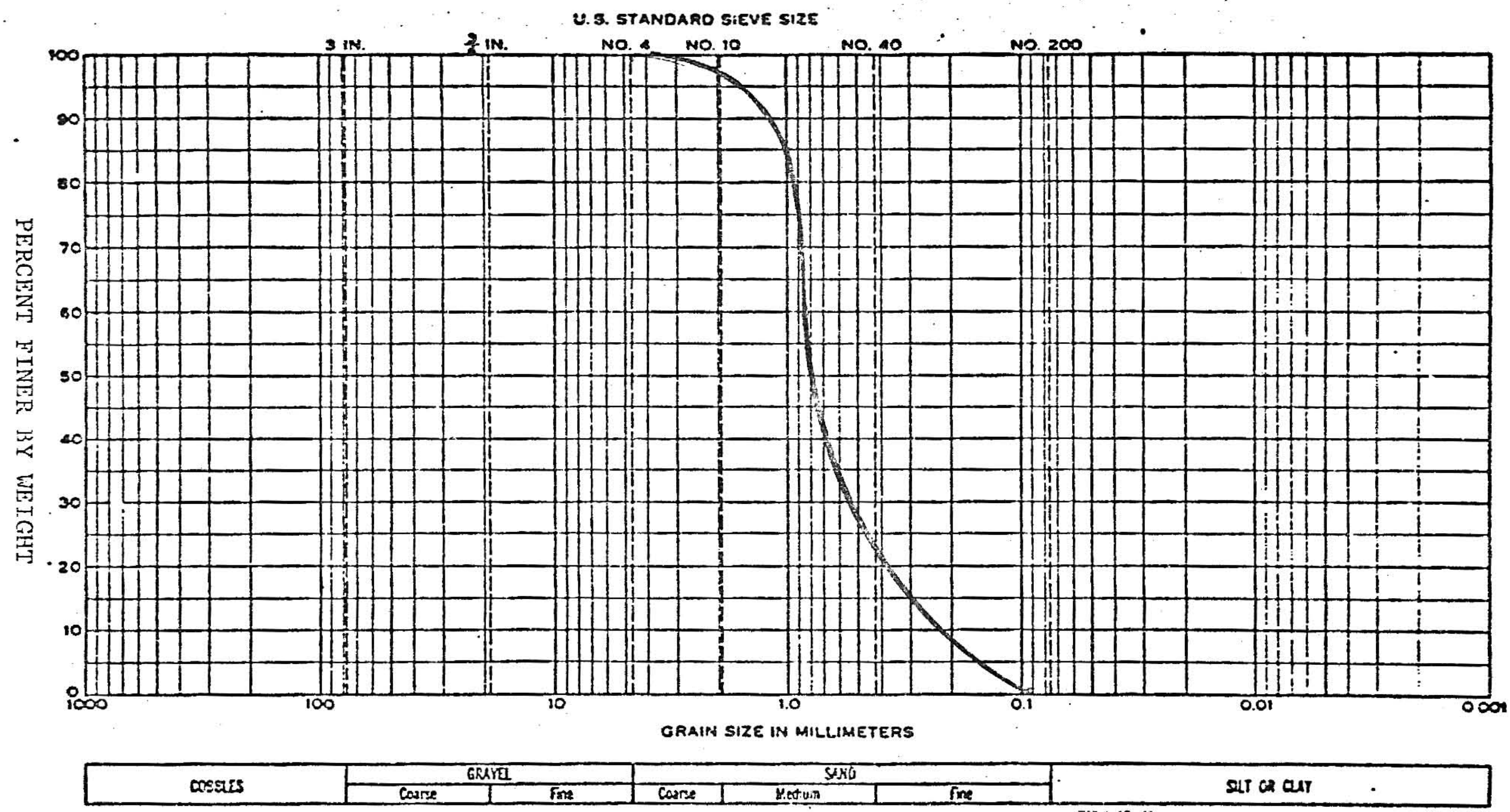


Figure 9

Sieve Analysis
1st Determination

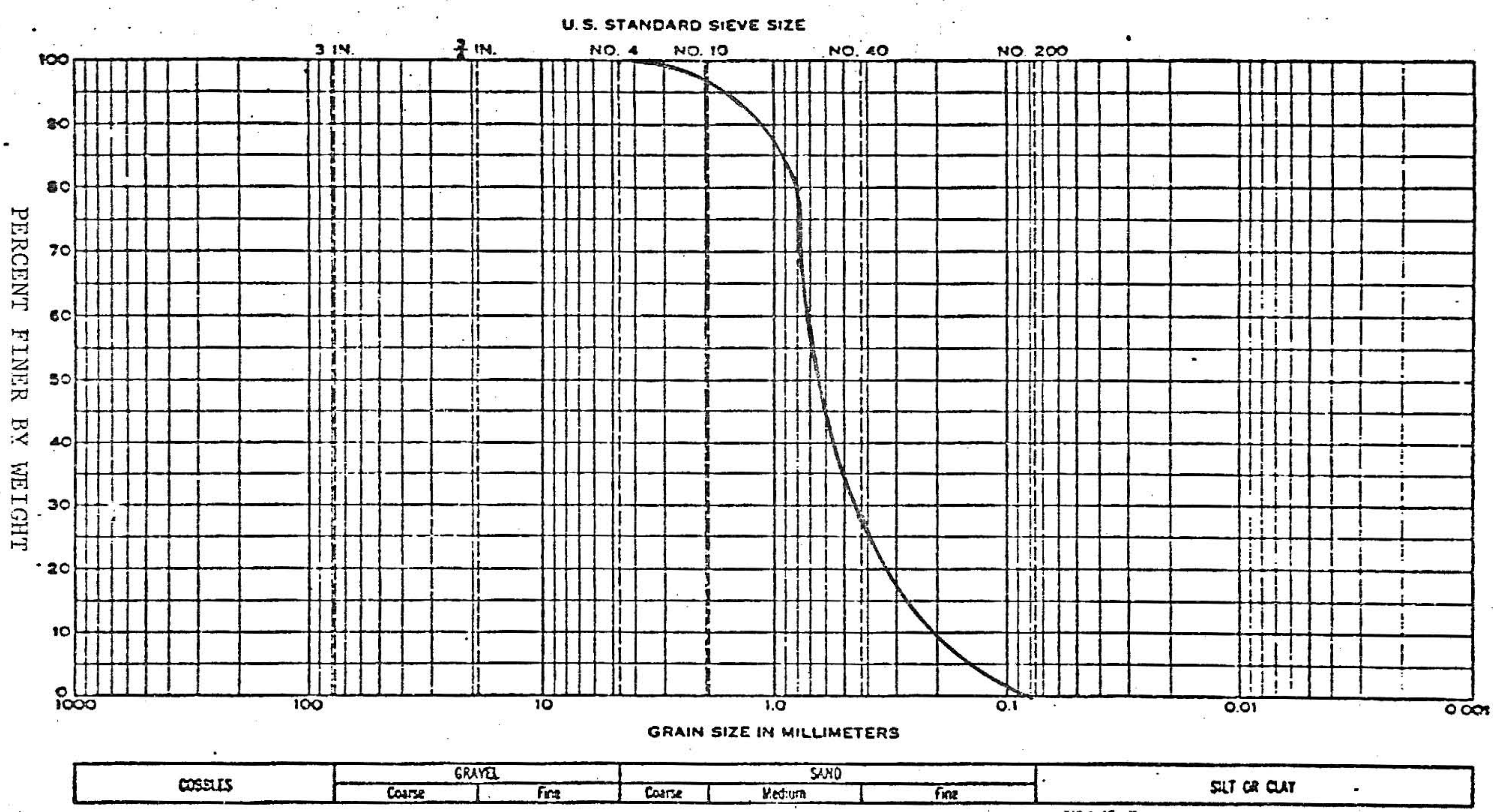


Figure 10

Sieve Analysis
2nd Determination

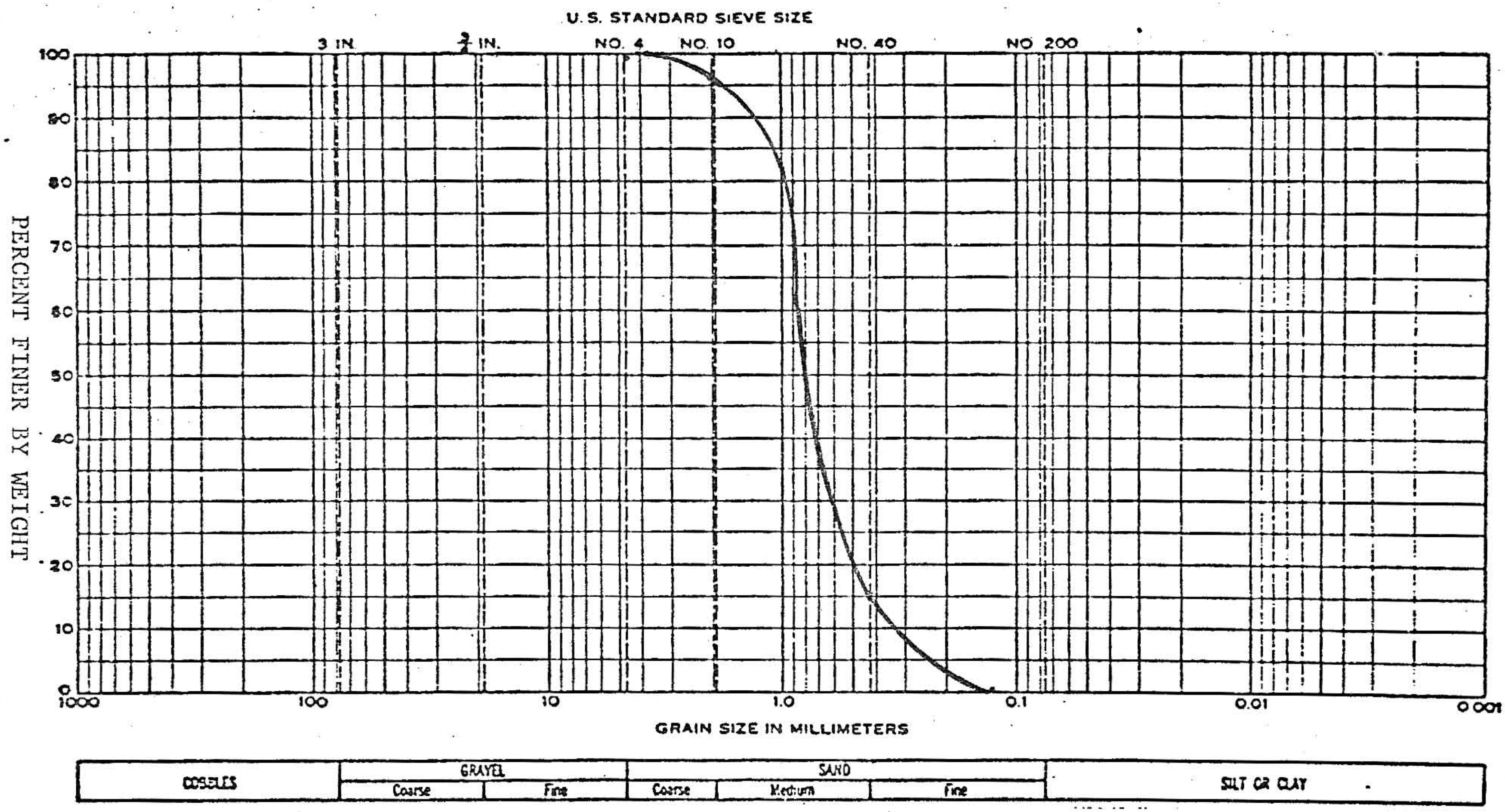


Figure 11

Sieve Analysis

3rd Determination

TABLE IV

DIRECT SHEAR

Shear Specimen Data:

Diameter: 2.5"
Area: 4.91 in^2
Thickness: 1.25"

Volume: 6.14 in^3
Weight of Sample: .414 pound

Vertical Load (p)	Normal Stress σ, (psi)	Load Ring Dial Reading	Horizontal Shear Force (p)	Shear Stress τ, (psi)
1st DETERMINATION				
9.68	1.97	38	12.58	2.56
18.48	3.76	60	19.86	4.04
27.28	5.56	70	23.17	4.72
36.08	7.35	81	26.81	5.46
44.88	9.14	121	40.05	8.16
53.68	10.93	128	42.37	8.63
2nd DETERMINATION				
9.68	1.97	41	13.57	2.76
18.48	3.76	56	18.54	3.78
27.28	5.56	69	22.84	4.65
36.08	7.35	76	25.16	5.12
44.88	9.14	117	38.73	7.89
53.68	10.93	142	47.00	9.57
3rd DETERMINATION				
9.68	1.97	43	14.23	2.90
18.48	3.76	55	18.21	3.71
27.28	5.56	70	23.17	4.72
36.08	7.35	90	29.79	6.07
44.88	9.14	112	37.07	7.55
53.68	10.93	125	41.38	8.43

$\phi_1 = 40.5°$

$\phi_2 = 40.0°$

$\phi_3 = 41.0°$

ϕ average = 40.5°

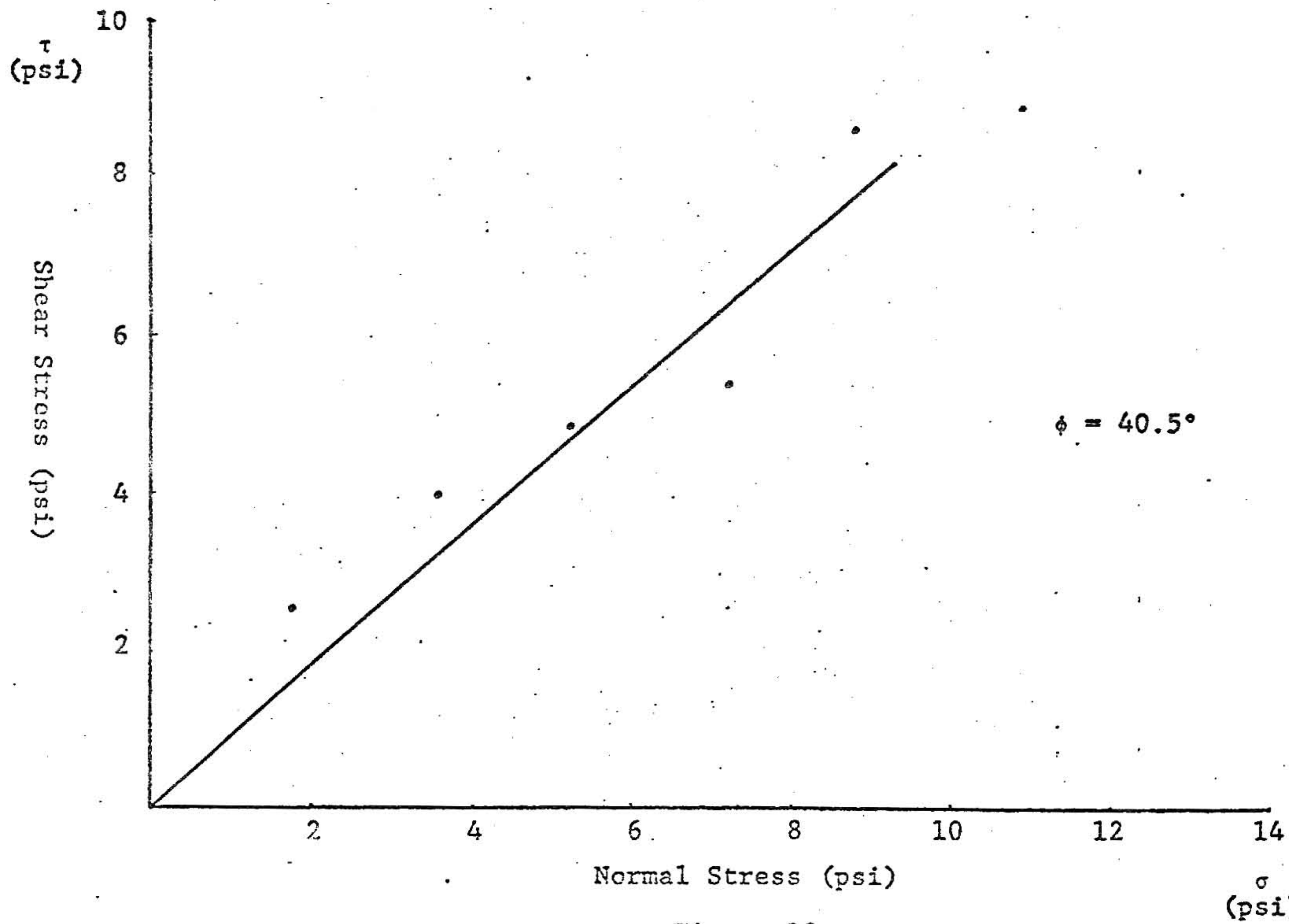


Figure 12

Direct Shear
1st Determination

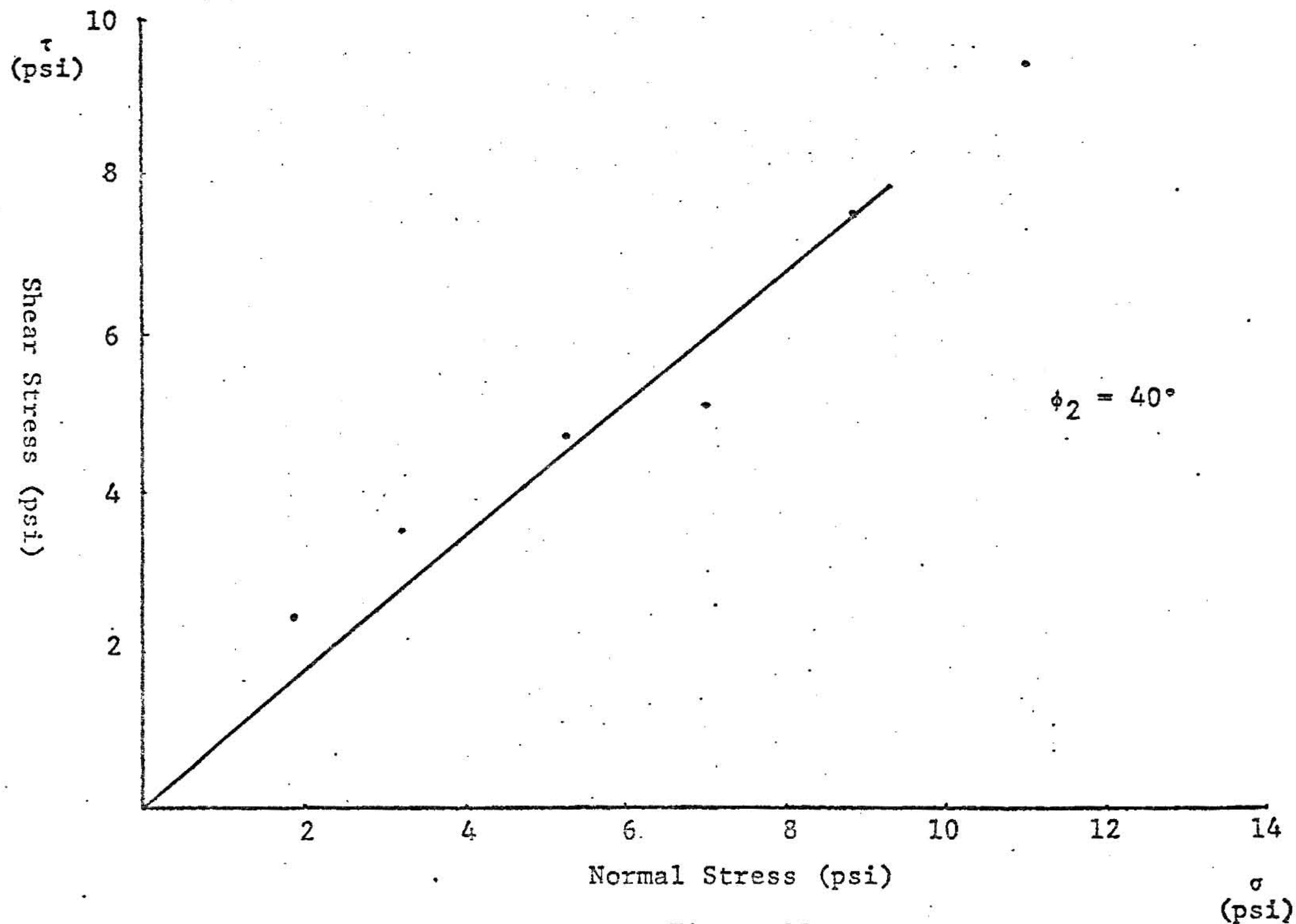


Figure 13

Direct Shear
2nd Determination

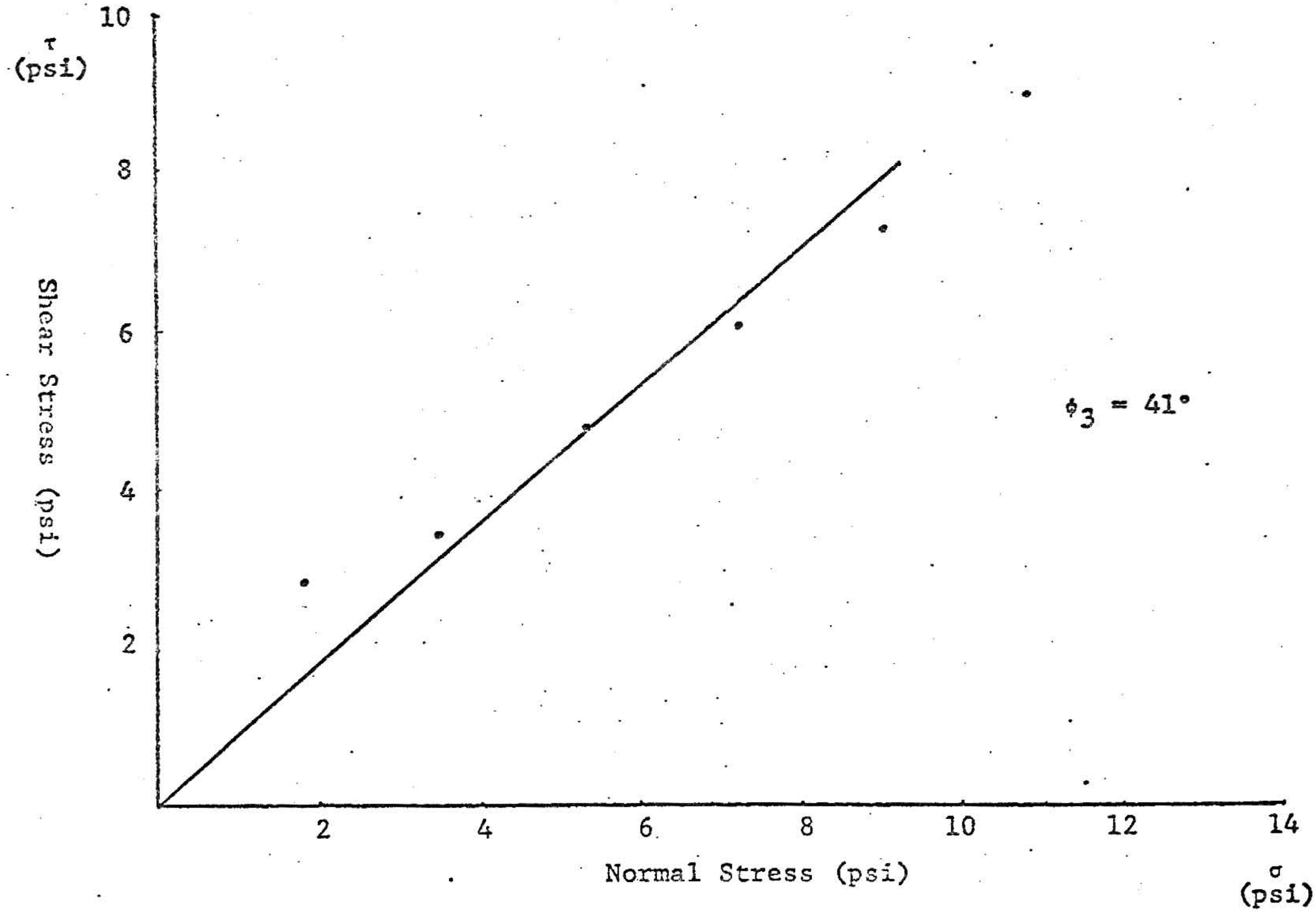


Figure 14

Direct Shear
3rd Determination

TABLE V

LONGITUDINAL SHEAR

Soil Contained in Device: 11.05 pounds

Surface Area of Pipe: 299.61 in^2

Volume of Soil Chamber: .08 ft^3

Normal Stress (psi)	Load Ring Reading	Shear Load (p)	Shearing Stress (psi)
1st DETERMINATION			
0	113	99.44	0.33
5	405	356.40	1.19
7	515	453.20	1.51
10	770	624.80	2.09
2nd DETERMINATION			
0	70	61.60	0.21
5	345	303.60	1.01
7	510	448.80	1.50
10	700	616.00	2.06
3rd DETERMINATION			
0	79	69.52	0.23
5	410	360.80	1.20
7	540	475.20	1.59
10	750	660.00	2.20

$\phi_{p.s_1} = 8.5°$

$\phi_{p.s_2} = 8.9°$ Average longitudinal $\phi_{p.s} = 8.9°$

$\phi_{p.s_3} = 9.4°$

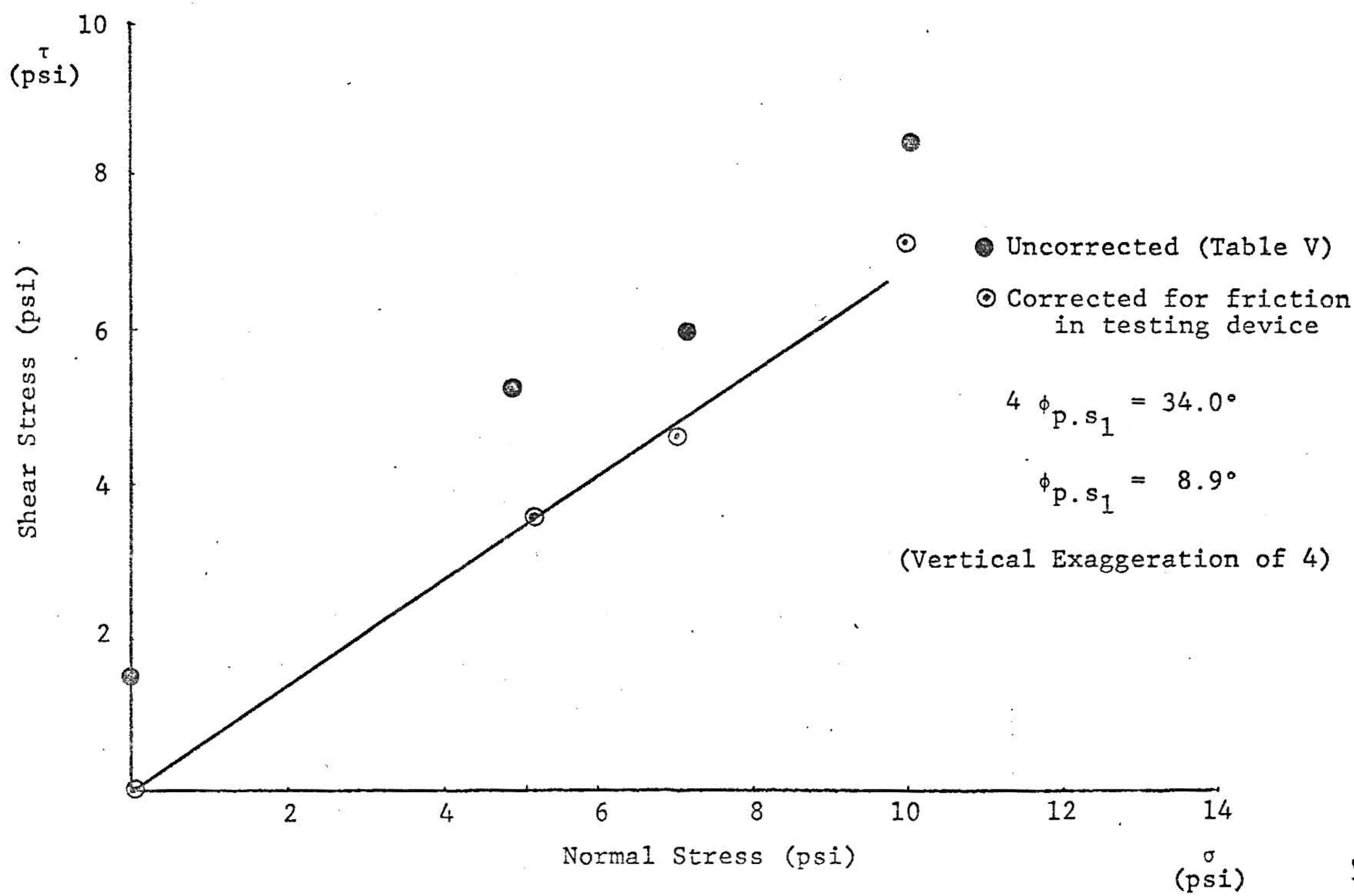


Figure 15

Longitudinal Shear
1st Determination

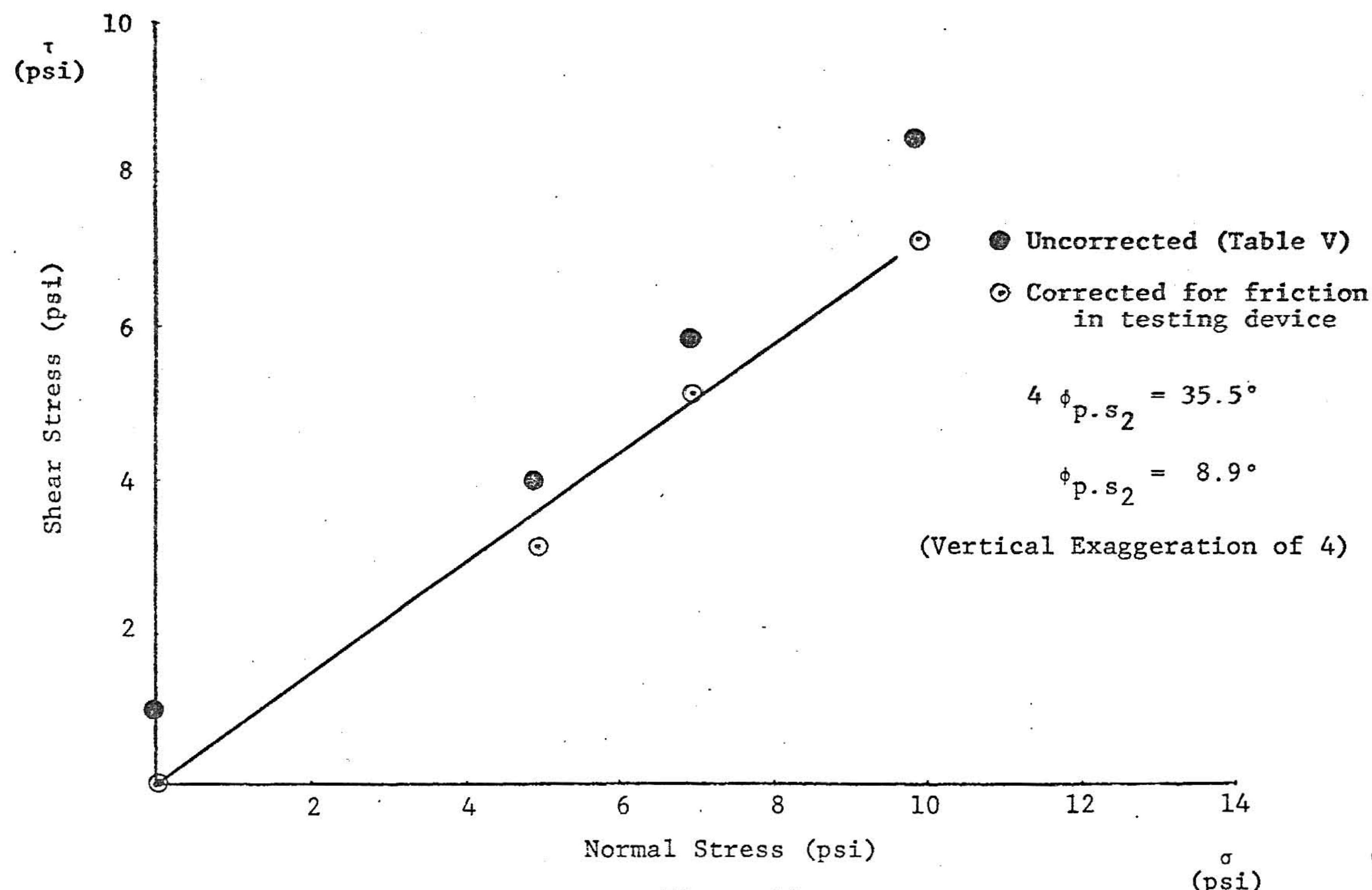


Figure 16

Longitudinal Shear
2nd Determination

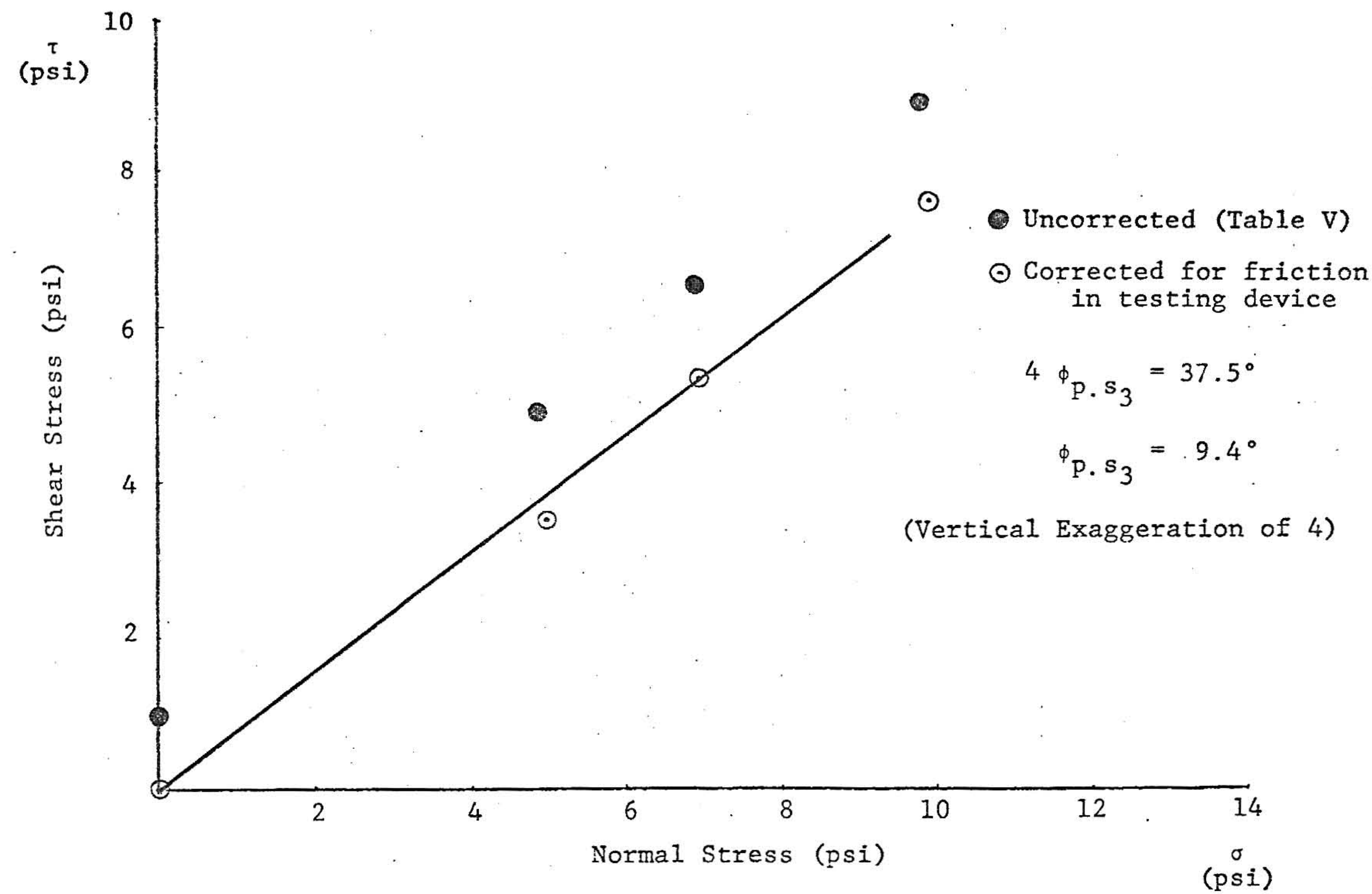


Figure 17

Longitudinal Shear
3rd Determination

TABLE VI

RADIAL SHEAR

Soil Contained in Device: 11.05 pounds

Surface Area of Pipe: 299.61 in^2

Volume of Soil Chamber: .08 ft^3

Normal Stress (psi)	Applied Torque (p·in)	Constant X * $(in)^3$	Shearing Stress (psi)
1st DETERMINATION			
0	370.74	1300.30	0.29
5	735.42	1300.30	0.57
10	1252.10	1300.30	0.96
13.5	1389.25	1300.30	1.07
2nd DETERMINATION			
0	411.30	1300.30	0.32
5	738.82	1300.30	0.57
10	1296.21	1300.30	1.00
13.5	1415.64	1300.30	1.09
3rd DETERMINATION			
0	424.29	1300.30	0.33
5	752.45	1300.30	0.58
10	1255.10	1300.30	0.97
13.5	1348.36	1300.30	1.04

*Constant X = (radius of pipe)(surface area of pipe)

Applied Torque = (radius)(surface area)(shearing stress)

$\phi_{p.s_1} = 3.6°$

$\phi_{p.s_2} = 3.3°$

$\phi_{p.s_3} = 3.2°$

Average radial $\phi_{p.s} = 3.4°$

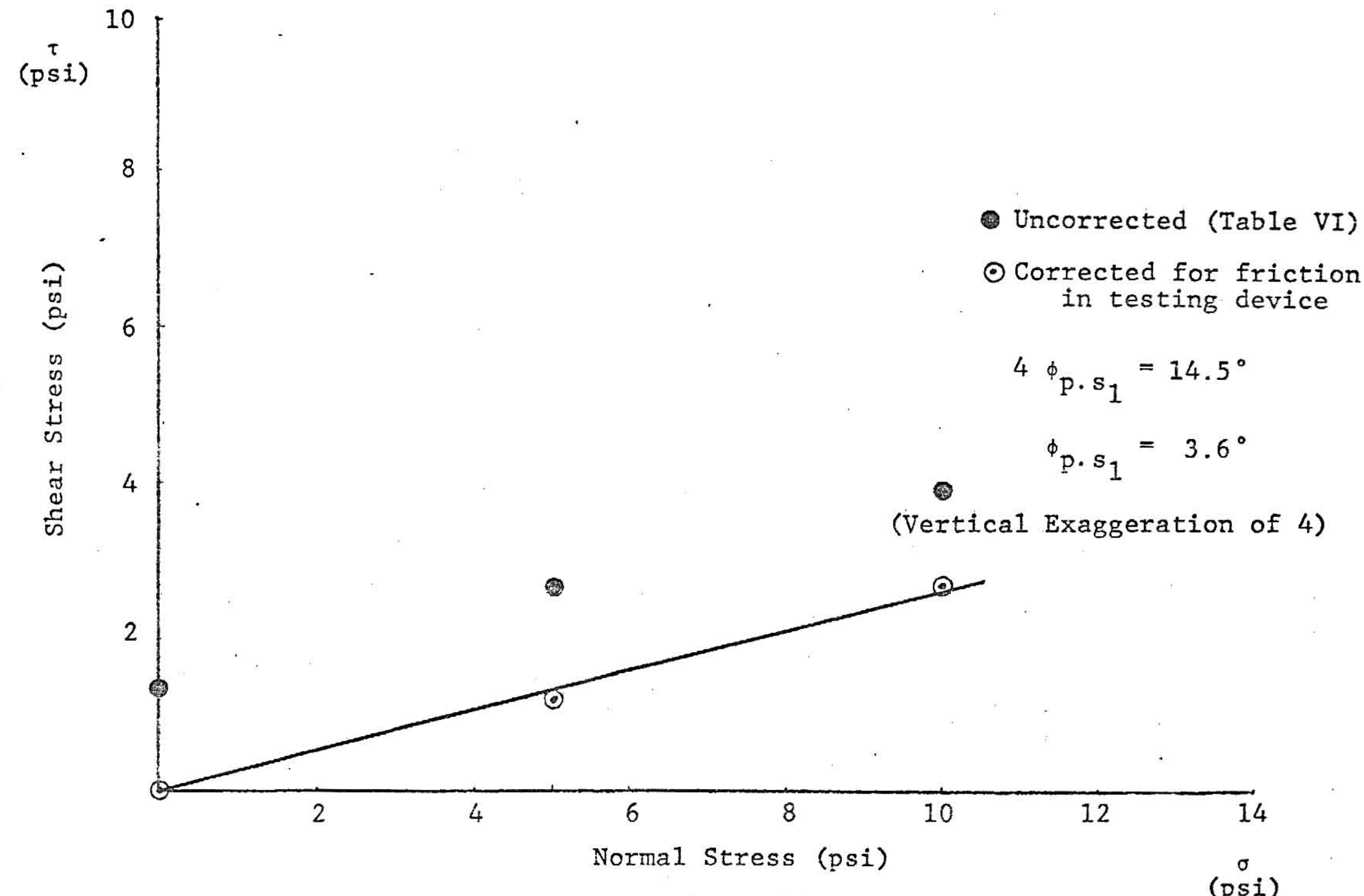


Figure 18

Radial Shear
1st Determination

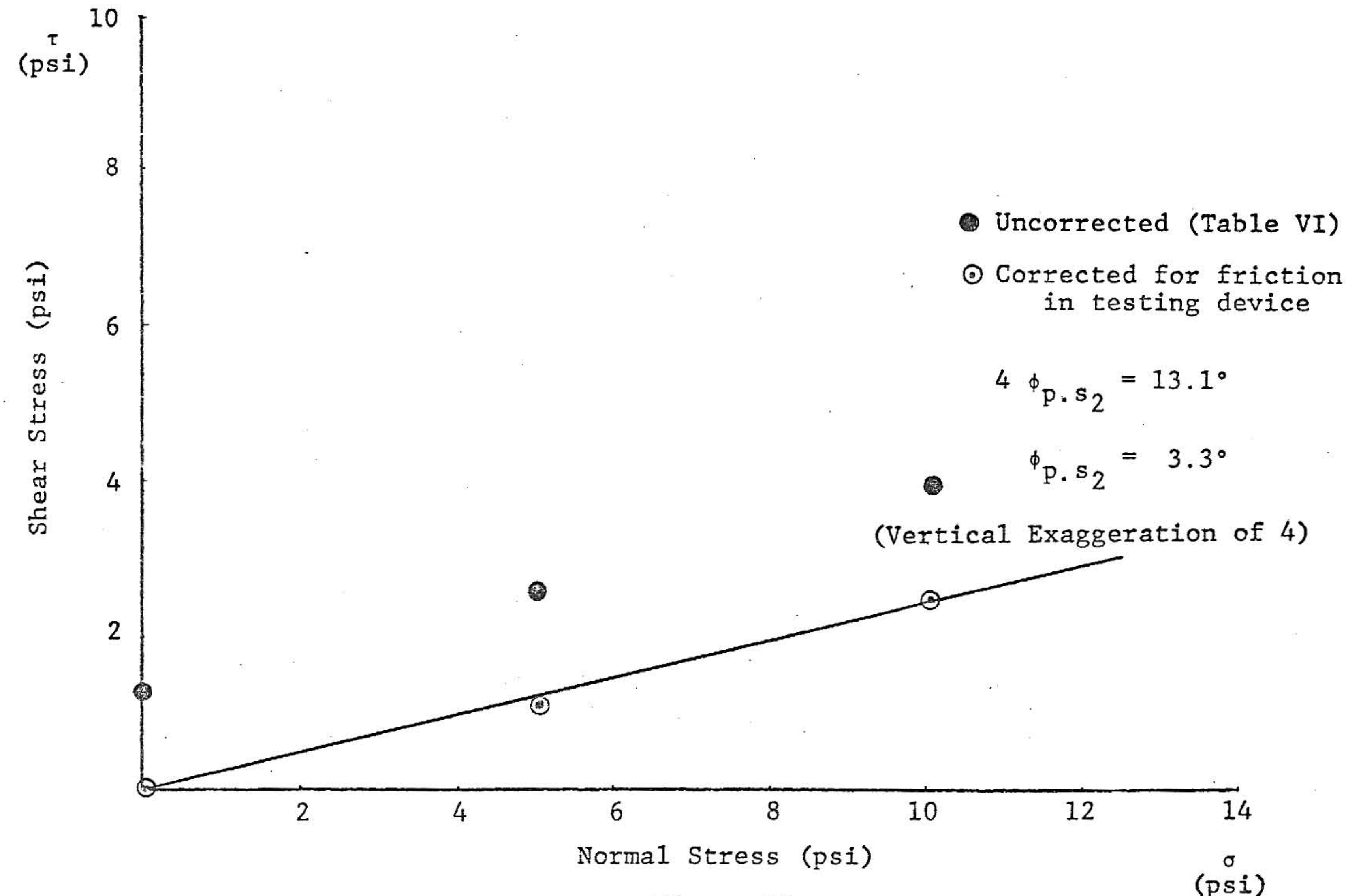


Figure 19

Radial Shear
2nd Determination

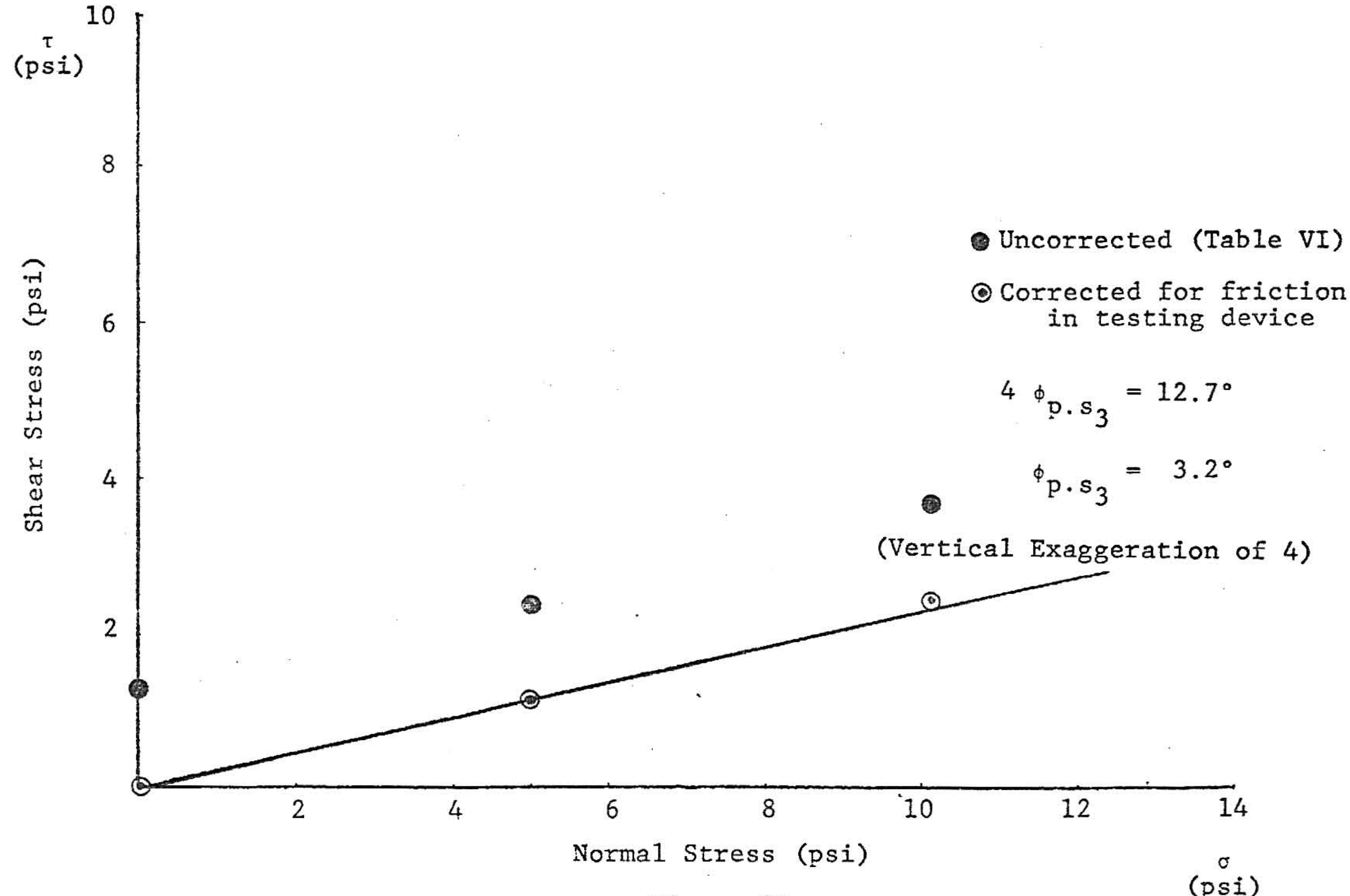


Figure 20

Radial Shear
3rd Determination

DISCUSSION OF RESULTS

All results which are discussed in this section are enumerated in prior sections and were obtained through procedures previously described.

As may be noted from Table IV, the internal angle of friction between individual soil particles in the sample of sand was 40.5°. Since the sand did not contain any cohesive material, its cohesion was equal to zero. In contrast to these values, the angle of friction measured between the soil and the surface of the P.V.C. pipe while shearing the soil-pipe bond in the longitudinal and radial directions were 8.9° and 3.4°, respectively (Tables V and VI). The cohesion was assigned zero, although the data for longitudinal and radial shear indicated from .21 to .33 psi. These readings were regarded as friction inherent to the testing device and they were subtracted from the subsequent readings when their respective graphs were plotted.

These results are not really very surprising since the surface of P.V.C. pipe is very smooth. It has no surface roughness detectable by the unaided human touch or sight. And obviously, for any appreciable amount of adhesion to develop between the surface of a buried conduit and the surrounding soil, a surface must be rough and have indentations to allow some interlocking with the soil particles. Such surfaces as concrete, corregated steel, and vitrified clay are much rougher than P.V.C. pipe and thus would develop more adhesion with the soil.

As previously noted, the longitudinal and radial shearing data show some cohesion between the P.V.C. pipe and sand (Figures 15-20 are drawn with zero cohesion values). This fact illustrates the possible inaccuracies intrinsic to the testing device used. However, the author possesses significant confidence in results in relative terms. That is, if there is error included in the results, it is a consistent error, thus the angle of friction determined from the research is correct. However in absolute terms, the results may lack total accuracy. In other words, the longitudinal shearing stress at 10 psi normal stress may not necessarily be equal to 2.09 psi as the 1st determination in Table V indicates.

Further, the results obtained in the longitudinal shearing test are probably slightly more accurate than those determined in the radial shearing test. This is due to the relative sophistication of the methods of applying and measuring the shearing force in the two testing procedures. In the longitudinal shearing test, the shearing force was applied by a hydraulic jack and measured by a load ring. Whereas, the shearing force in the radial shearing test was applied by a system of weights and a lever.

Finally, in both cases, another factor contributing to producing inexact results is the rate of applying the shearing load. Ideally, the rate should be uniform and non-varying; however that was difficult to achieve in this research due to the use of a hydraulic hand pump (in the longitudinal shearing test) and the addition of weights (in the radial shearing test).

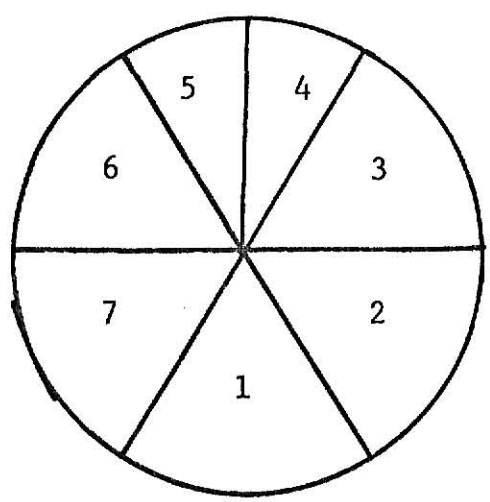
When considering the frictional forces previously described in the literature review, it is important to realize that these forces will have a significant effect on the performance of the conduit. For example, to illustrate the difference in magnitude of these frictional forces for different soil-to-pipe friction angles, a hypothetical flexible ditch conduit with the following characteristics will be analyzed with $\phi_{p.s}$ equal to 4° and 30°.

H = Depth of Pipe = 20 feet

D = Diameter of Pipe = 16 inches

γ = Unit Weight of Soil = 100 pounds/cubic foot

W_c = Marston's Load on Conduit = 58.3 pounds/linear inch



SECTION	FRICTIONAL FORCES (p/in)		
	$\phi_{p.s} = 4°$	$\phi_{p.s} = 30°$	Difference
1	0.0	0.0	0.0
2	7.816	64.536	56.720
3	7.266	59.992	52.726
4	0.163	1.346	1.183
5	0.163	1.346	1.183
6	7.266	59.992	52.726
7	7.816	64.536	56.720
Total	30.490	251.748	221.258

As may be noted from the previous table, the magnitude of the frictional forces varies significantly with different pipe-to-soil friction angles. However, it is beyond the scope of this research to completely describe this effect. This is due to the lack of knowledge concerning the amount and direction of displacement of the soil particles directly adjacent to the conduit's surface. Only, when these movements are completely determined, can the benefits, or liabilities of the frictional forces be fully evaluated.

CONCLUSIONS

This research has yielded five major conclusions concerning adhesion at the sand and PVC pipe interface. These conclusions are:

1) That the angle of friction between the sand and the PVC pipe was 8.9° when the soil was sheared in the longitudinal direction.

2) That the angle of friction between the sand and the PVC pipe was 3.4° when the soil was sheared in the radial direction.

3) That the angle of friction between the PVC pipe and sand was equal to 8.4 to 23.3 percent of the internal angle of friction of the sand. The cohesion remained zero, the same as the sand normally possesses.

4) That the frictional forces between the pipe and sand vary considerably with differing wall friction angles as illustrated in the example calculations contained in the prior section.

5) That although these forces have been defined in terms of magnitude, their exact effect on the conduit relative to its performance and strength is still unknown. More information must be gained concerning the actual movements of the soil particles

adjacent to the pipe. Thus, it is unfeasible at this time to relate in any practical terms the contribution of these frictional forces to the strength of buried conduits.

AREAS FOR FURTHER RESEARCH

In general, there are four major areas in which further research would be useful. The first area would involve the further definition and methods of measurement of the modulus of soil reaction. Secondly, the study of adhesion between PVC pipe and other types of soils would be of significant interest. The third area would involve research to further varify the importance of pipe to soil friction to the ultimate strength of the flexible conduit. And lastly, a study involving the description of soil particle displacement adjacent to a buried conduit when that conduit is deformed.

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A STUDY OF ADHESION
BETWEEN
P.V.C. PIPE AND SAND

by

Forrest E. Erickson

B.S., Kansas State University

1972

AN ABSTRACT OF A MASTER'S THESIS

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1975

ABSTRACT

It is readily apparent that P.V.C. conduits have drastically smoother skin textures than do the steel and concrete conduits on which early theories were based. This difference in texture was determined by embedding a P.V.C. pipe in sand and shearing the pipe-sand interface bond. By shearing these bonds in the longitudinal and radial direction at various normal pressures, it was possible to measure and define the textural differences in terms of angle of friction and cohesion. After these differences were defined, it was possible to relate them in practical terms to existing theories concerning predicted deflection and behavior of conduits under loading.

The angle of friction between the pipe and the sand (wall friction angle) was found to be 8.9° and 3.4° in the longitudinal and radial directions, respectively. This is a significant reduction from the sand's internal angle of friction of 40.5°. This reduction will ultimately affect the strength of the conduit. However, further research is necessary to define exactly what this effect may be.