THE PERIPHERAL MOVEMENT OF SOIL AROUND A BURIED FLEXIBLE PIPE

by

Wayne A. Duryee

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Approved by:

Wayne W. Williams
Major Professor

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INTRODUCTION

STATEMENT OF PROBLEM

The services provided by buried conduits in our everyday lives are generally taken for granted by most of us. Sewer lines, water mains, culverts, gas lines, storm sewer pipe, and buried irrigation pipe are among the types of buried conduits which aid in maintaining and improving our standard of living. Although the use of buried conduits was not unheard of in early times, the developments within the past few decades have led to their actual widespread usage today.

Studies are being conducted in order to make the design of buried conduits more safe and economical. Model studies have been the most popular in recent years. The results of some of these model studies are being recommended for use in design of actual buried conduit installations. In general, it is felt that there are certain aspects of the model studies which have so far been neglected. One of these is the extent of the peripheral movement of the soil around the pipe and how this movement might be affected by the proximity of the test chamber walls, thus possibly affecting the results of the model tests.

PURPOSE OF THE STUDY

The purpose of this study was to determine the peripheral movement of the soil around a buried PVC pipe as it deflected under loading to a point beyond the performance limit. After the zone of visible movement was known, the relationship of deflection of the pipe and disturbance of the soil in the pipe-soil system was determined.

SCOPE OF THE STUDY

The study initially consisted of a comprehensive review of the literature pertaining to buried conduits. Based upon the findings in the literature review, an experiment was designed to model test four different sizes of PVC pipe buried in sand in two series of test conditions; the sand in the loosest possible state and in a highly compacted state. The movement of the soil around the pipe was determined visually.

LITERATURE REVIEW

INTRODUCTION

Little was known about the forces on a buried conduit and the strength of the conduit itself before the turn of this century. This lack of understanding accounted for the low success rate of early buried conduits. A study undertaken in 1911 at Iowa State University provided theoretical and experimental conclusions which opened the way for the design of successful buried conduit systems. In this effort, Marston and Anderson (1) studied actual ditch conduit installations, both good and bad, and performed experiments on rigid cement and clay pipe. Some of their general conclusions as to the previous failures of pipe in ditches were:

1. The large number of failures (cracks) are caused principally by the fact that presently manufactured pipes are too weak to carry the weight resting upon them from more than a few feet depth of fill.

2. Very few failures are due to poor installation or poor estimates of loading.

3. Thus, in some instances it is impossible to prevent cracking even though a reasonable amount of care is taken in bedding and laying the pipe and refilling the ditches. This does improve the carrying power though.

4. Bedding in concrete is the only sure way to prevent cracking.

5. The bottom of the ditch should be shaped to fit the lowest 90° of the pipe. This material should be sand or some other granular soil.

6. Tamping the sides of the ditch around the pipe does not prevent the rigid pipe from cracking, but does prevent collapse of the pipe after it is cracked.

Along with these conclusions, Marston and Anderson (1) introduced the mathematical theory of loads on rigid pipes in ditches. The basic concept of the theory is that the load due to the weight of the soil column above a buried pipe is modified by arch action in which a part of its weight is transferred to the adjacent side prisms, with the result that in some cases the load on the pipe may be less than the weight of the overlying column of soil (2). This theory and others will be discussed in greater detail later in the review.

Using the principles of mechanics to analyze buried conduits has led to classifying them according to types of pipe and types of installation techniques. Buried conduits are generally of two types, rigid and flexible. Rigid conduits, those usually made from concrete, cast iron, and clay, are designed to maintain their original circular cross section under loading. A small percentage of deflection will generally result in failure. Flexible pipe, those usually made from corrugated metal, thin-walled steel, and plastic, are designed to deflect readily under loading and still retain their structural integrity.

Installation of buried conduits falls into two major classes, ditch conduits and projecting conduits. Projecting conduits are further subdivided into positive projecting conduits and negative projecting conduits. Also there are several special cases having characteristics which are similar to those of both of the major classes (3). The primary emphasis will be placed on the major classes in this review.

A ditch conduit is one which is installed in a relatively narrow ditch dug in passive or undisturbed soil and which is then covered with earth backfill. Examples of this class of conduits are sewers, drains, water mains, and gas mains. A positive projecting conduit is one which is installed in shallow bedding with its top projecting above the surface of the natural ground and which is then covered with an embankment. Railway and highway culverts are frequently installed in this manner. A negative projecting conduit is one which is installed in a relatively narrow and shallow ditch with its top at an elevation below the natural ground surface and which is then covered with an embankment. This is a very favorable method of installing a railway or highway culvert, since the load produced by a given height of fill is generally less than it would be in the case of a positive projecting conduit. This method of construction is most effective in minimizing the load if the ditch between the top of the conduit and the natural ground surface is backfilled with loose uncompacted soil (2). Figure 1 illustrates the various classes of conduit installations described.

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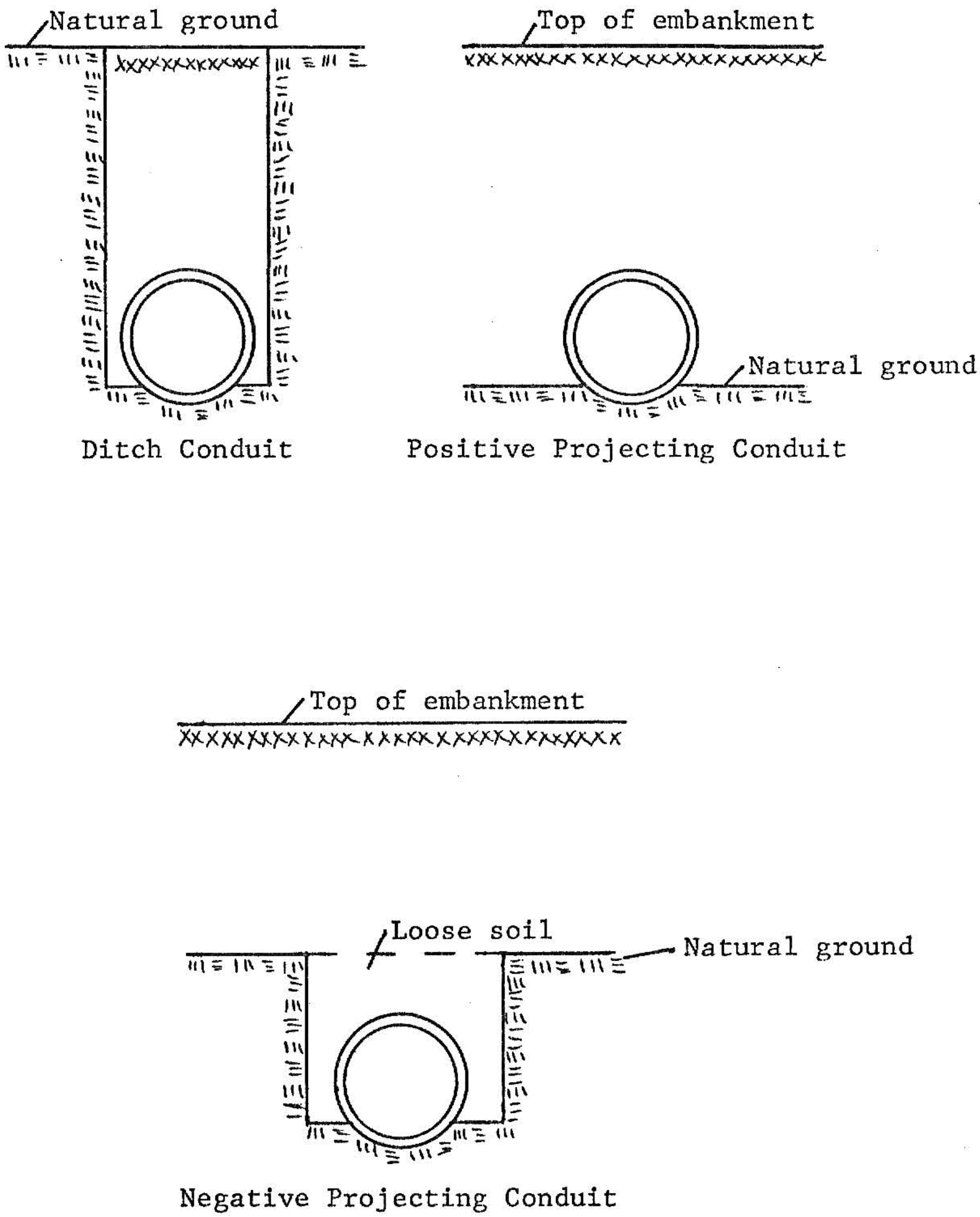

Figure 1

Various Classes of Conduit Installations

LOADS ON BURIED CONDUITS

The theory of loads on pipes in ditches, which was introduced by Marston and Anderson (1), pertained to rigid pipes in ditches. The theory states that the side pressure of the fill against the sides of the ditch develops a frictional resistance which aids in carrying part of the weight. This resistance relieves part of the vertical pressure near the sides of the ditch so that at the level of the top of the pipe the vertical pressure of the filling material is much heavier in the middle of the ditch than at the sides. Moreover, there is some arching effect at about the 45° point on each side, and the comparatively level part of the top of the pipe is much more solid and unyielding than the side filling material. For these reasons, the ditch filling above the top of the pipe receives only a negligible support, in ditches of ordinary width, from the filling at the sides of the ditches. Imperfections in the side filling and tamping add to the exactness of this principle.

Hence, the pipe must be strong enough to carry safely the entire weight of the ditch filling materials above the top of the pipe less the frictional force of the filling against the sides of the ditch.

Cohesion between the backfill material and the sides of the ditch is assumed negligible for several reasons. A cohesionless backfill develops no cohesion with time. A cohesive backfill requires time for the cohesion between the backfill and the sides of the ditch to develop. The cohesion that does eventually develop is subject to environmental effects and may be destroyed or altered at any time. Thus, the maximum load can only be determined by assuming no cohesion between the backfill and the sides of the ditch (1).

From this theory came the equation for calculating the load on a rigid ditch conduit.

$$W_c = C_d \gamma B_d^2 \qquad (1)$$

where

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

and

W_c = load on conduit, plf

C_d = a coefficient of loads on pipes in ditches

γ = unit weight (wet density) of fill material, pcf

B_d = horizontal width of ditch at top of conduit, ft.

H = height of ditch filling above top of conduit, ft.

K_a = coefficient of active earth pressure

μ' = coefficient of friction of ditch filling against the sides of the ditch = $\tan \phi'$

e = base of Naperian (natural) logarithms = 2.7182818

Figure 2 gives the values of C_d to use in the load equation.

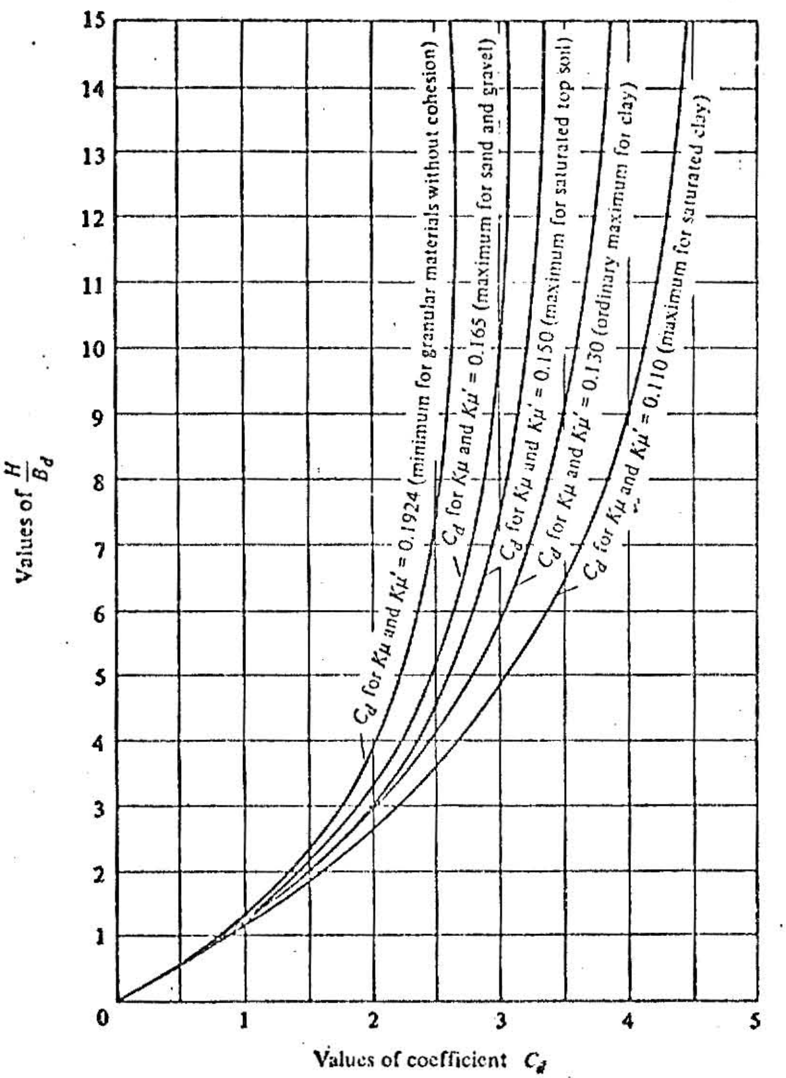

Figure 2

Diagram for Coefficient C_d Ditch Conduits

If the conduit is flexible and the side fills have been carefully compacted to approximately the same degree of stiffness as the pipe, then equation 1 becomes

$$W_c = C_d \gamma B_d B_c \qquad (2)$$

where

B_c = horizontal breadth of conduit, ft.

Spangler and Handy (2) point out that for equation 2 to be applicable, the side fills must be compacted a sufficient amount to have the same resistance to deformation under vertical load as the pipe itself. It seems then, that as the side fills for flexible pipe become less compact, the load on the pipe increases from equation 2 to a maximum value given by equation 1. Thus, the load is dependent upon the relative rigidity of the pipe and the density of the sidefill.

Another important aspect, which was first pointed out by Marston and Anderson (1), and which can be seen in Figure 2, is that there is very little increase in C_d and therefore the load on pipe in ditches for any increase in depth of fill beyond 10 times the width of the ditch. Tschebotarioff (4) tends to agree with this conclusion by comparing it to results of measurements of silos.

The value of K_a, which is attributed to Rankine, is explained very well by Terzaghi (5) and can be computed by the equation

$$K_a = \tan^2 (45° - \phi/2) \qquad (3)$$

where

ϕ = angle of internal friction of the fill material

Many times it is convenient or necessary to make a ditch with sloping sides in order to install a buried conduit, as in the case of caving soils. This makes the ditch much wider at the top than at the bottom. Studies by Marston and Anderson (1) revealed that in this case the proper value for B_d to use in the load equations is the width of ditch at a height of the 45° points on the pipe circumference, just a little below the top of the pipe.

Marston (6) extended his work to include positive projecting conduits. His experiments led to the equation for the load applied to a positive projecting conduit.

$$W_c = C_c \gamma B_c^2 \qquad (4)$$

where

W_c = load on conduit, plf

C_c = load coefficient for positive projecting conduits

γ = unit weight (wet density) of embankment soil, pcf

B_c = horizontal breadth of conduit, ft.

Spangler (7) later confirmed this equation for use on flexible conduits. C_c is a function of the ratio of the height of fill to the width of the conduit, H/B_c, and of the product of the settlement ratio and the projection ratio, as well as of the friction characteristics of the soil. A good explanation of loads on positive projecting conduits and a graph for easy access to C_c is available in Spangler (3), and Spangler and Handy (2).

Additional equations for calculating the loads on buried conduits due to super loads were developed by Marston (6). Super loads are of two types, 1) concentrated super loads such as wheel loads; and 2) uniformly distributed super loads.

Schlick (8) studied both the ditch conduit and the positive projecting conduit equations in order to determine their relationship, if any. The results on rigid conduits show that as the width of the ditch increases, other conditions remaining constant, the load upon the conduit increases in accordance with the ditch conduit load theory until it equals that by the projecting conduit load theory, and then remains constant for all greater widths. Thus, it is possible to calculate the transition-width ratio, B_d/B_c, for any given set of conditions by determining the width of ditch for which the ditch conduit load equals that by the projecting conduit load theory.

It has been observed that the transition width varies with H/B_c, settlement of the fill and conduit, and the projection ratios as used in the projecting conduit theory.

Schlick (8) stated that it is improbable that this transition width is instantaneous, however data indicated that for practical purposes any intermediate zone of widths that may exist is so narrow that it may be neglected.

The preceding formulas give ultimate limiting loads. Actual long-term load tests (1,6,7) showed that the loads vary greatly with variations in properties of the materials, as to weight, settlement, moisture, temperature, internal friction, and cohesion. A given conduit may escape for a long time this ultimate load or perhaps may never reach it. A period of several years may be required to impose the remaining load, which can be as much as 20-25% of the total load.

SUPPORTING STRENGTH OF BURIED CONDUITS

Although the load imposed on a buried conduit has been shown to be dependent upon the type of pipe and the soil characteristics, this pipe-soil relationship has an even greater effect on the supporting strength of the conduit. Research has shown that the field supporting strength of a buried conduit is dependent upon three major factors:

1) the inherent strength of the pipe;
2) the quality of the bedding as it affects the distribution of the bottom reaction; and
3) the magnitude and distribution of active lateral pressures which may act on the sides of the pipe (2).

In a rigid pipe the inherent strength is the predominant source of supporting strength. The inherent strength depends primarily on the materials used to make the pipe. The lateral pressure of the earth at the sides of the pipe causes stresses in the pipe ring in opposite directions to those produced by vertical loads and therefore assists the pipe in supporting the vertical loads. The only lateral pressure which can be relied upon to aid the load-carrying capacity of a rigid pipe is the pressure at rest, since the pipe deforms very little under vertical load and the sides do not move outward enough to develop any appreciable passive pressure in the enveloping earth (7).

The shape and quality of the bedding influences the distribution of the vertical reaction on the bottom of the pipe and the bending moments in the pipe wall which in turn influences the load-carrying capacity of the pipe.

Rigid pipe has been tested extensively in order to be able to predict the field supporting strength under any stated condition of installation. Early work by Marston and Stewart (9) resulted in the specifications for making rigid pipe and the standardization of tests on their supporting strength. Marston, Schlick, and Clemmer (10) reported on the supporting strength for pipe under different pipe laying and bedding conditions.

Spangler (11) came up with the working values of the load factor for use in the design of actual pipe systems. This load factor is the ratio of the supporting strength of a pipe under any stated condition of loading in the field to the supporting strength of similar pipe as determined with the three-edge bearing test. These load factors are associated with the classes of bedding which are generally used in the field. Schlick (12) expanded this work by determining supporting strengths for rigid pipe under pressure. More recently, methods have been introduced for bedding sewer pipe which require the use of selected granular materials, but much less hand labor. Examples of these and their corresponding load factors can be found in Spangler and Handy (2).

In a flexible pipe the pipe itself has relatively little inherent strength, and thus only a small part of the load is actually carried by the pipe. A large part of its ability to support vertical loads must be derived from the passive pressures induced as the sides move outward against the earth. The ability of a flexible pipe to deform readily and thus utilize the passive pressure of the soil on each side of the pipe is its principal distinguishing structural characteristic and accounts for the fact that such a relatively light-weight pipe can support earth fills of considerable height. Since so much of the total supporting strength depends upon the sidefill material, any attempt to analyze the structural behavior of the flexible conduits must consider the soil at the sides to be an integral part of the structure (7).

Flexible pipes usually fail by deflection rather than by rupture of the pipe walls, as do rigid pipe. A flexible pipe, installed in the ordinary manner without vertical struts or other prestressing devices, will deflect under the vertical earth load, the vertical diameter becoming less and the horizontal diameter becoming greater by appreciable amounts. The outward movement of the sides of the pipe against the enveloping fill material brings into play the passive resistance of the soil, which acts horizontally against the pipe and keeps the actual deflection of the pipe considerably below the amount the pipe would deflect if acted upon by the vertical earth loads alone.

This action continues, as the load is increased, until the top of the pipe becomes almost flat. Additional load may cause the top to become concave upward. When this occurs, the pipe will pull inward; and the side supports of the pipe will be eliminated. This is because the passive forces cannot follow the inward movement. The earth above the pipe continues to follow the downward movement of the pipe and exert pressure on the structure causing more deflection. Finally, complete collapse and failure may result (2). This sequence of the development of pipe deflection is shown in Figure 3.

The magnitude and distribution of the various forces to which a flexible pipe is subjected when installed as a projecting conduit has been researched extensively. Spangler (7) summarizes this information as follows:

1) The vertical load on a pipe may be determined by Marston's theory of loads on conduits and is distributed approximately uniformly over the breadth of the pipe.

2) The vertical reaction on the bottom of a pipe is equal to the vertical load and is distributed approximately

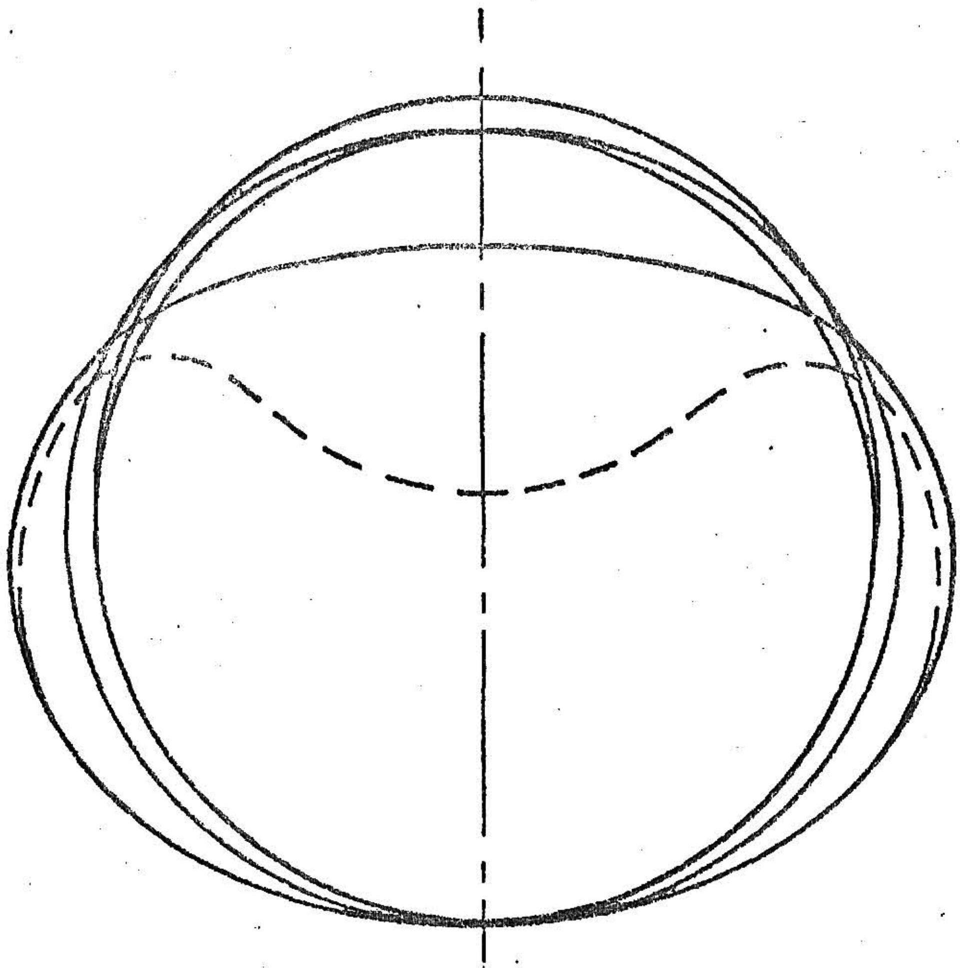

Figure 3

Stages of Deflection of a Flexible Pipe

uniformly over the width of bedding of the pipe.

3) The passive horizontal pressure on the sides of a pipe are distributed parabolically over the middle 100° of the pipe and the maximum unit pressure, which occurs at the ends of the horizontal diameter of the pipe, is equal to the modulus of passive resistance of the sidefill material multiplied by one half the horizontal deflection of the pipe. The distribution of pressure around a flexible pipe under an earth fill is shown in Figure 4.

Using this information, it is possible to develop mathematical expressions for the moments, thrusts, shears, and deflections of a pipe in terms of the properties of the pipe and of the soil of which the sidefills are constructed. These expressions can in turn be used to aid in the design of flexible pipe conduit systems. A very commonly used equation for design is one which determines the ultimate horizontal deflection of a flexible pipe culvert under a fill. Spangler (7) introduced this equation in 1941. It is commonly called the Iowa Formula, and is

$$\Delta x = \frac{D_1 \, K \, W_c \, r^3}{EI + 0.061 \, er^4} \qquad (5)$$

in which

Δx = ultimate horizontal deflection of the pipe

D_1 = deflection lag factor

K = a bedding constant, its value depending on the bedding angle, α, in Figure 4.

W_c = Marston's vertical load on the pipe

r = mean radius of the pipe

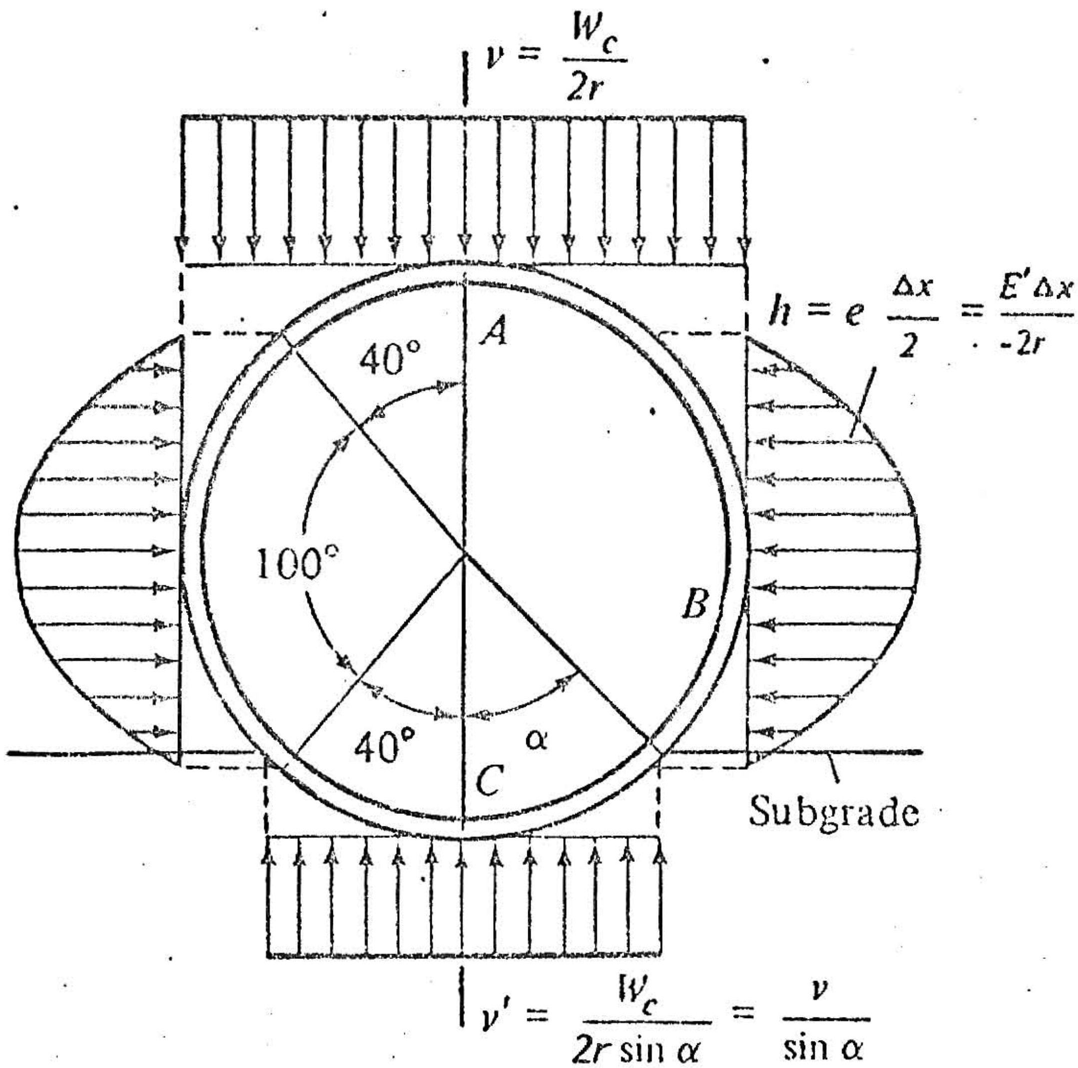

Figure 4

Distribution of Pressure Around a Flexible Pipe

E = modulus of elasticity of the pipe material

I = moment of inertia of the pipe wall

e = modulus of passive resistance of the enveloping soil

The accuracy of this equation is very good if the necessary properties of the soil and pipe are properly determined. Some of these properties are easy to determine while one in particular, e, is not. This has put limitations on the use of the Iowa Formula and has led to extensive testing since the formula was first introduced. Discussion of each of the properties, or terms, will help in the understanding of the problems surrounding the use of the Iowa Formula.

The horizontal deflection of the pipe, Δx, has been found to be nearly the same as the vertical deflection of the pipe, Δy, under most field conditions. Knowing this, design of a flexible pipe culvert is usually done by using the equation to predict Δx and then comparing this to a performance limit for the pipe. The performance limit is the maximum allowable ring deflection beyond which the pipe-soil system cannot adequately perform the purpose for which it was designed. For example, extreme ring deflection might conceivably reduce flow capacity of the pipe to below the minimum acceptable. Or it might cause a hump, dip or crack in the soil surface above the pipe. Or, if the pipe has a brittle lining or coating, excessive ring deflection could cause cracking or spalling of the coating or lining. Such a ring deflection would be a performance limit (13). For flexible pipe, such as PVC, the performance limit is usually set at $0.05 = \Delta y/D$ (decrease in vertical diameter/pipe diameter). This value must be greater than or equal to the predicted ring deflection of the buried pipe, $\Delta x/D$. If it is not, then a different pipe-soil system must be used.

It has been observed that flexible pipe conduits continue to deflect slowly for a period of time after the maximum vertical load has developed. This is caused by the continued yielding of the soil at the sides of the pipe in response to the horizontal pressures exerted over a long period of time. This yielding results in a continuation of the pipe deformation to a value beyond that which is primarily attributable to the vertical load. Thus, in order to predict the ultimate horizontal quantity called the deflection lag factor, D_1, must be introduced. The deflection lag factor cannot be less than 1.0 and has been observed to range upward toward a value of 2.0. A normal range of values from 1.25 to 1.50 is suggested for design purposes (2).

The values of K, a bedding constant, apply for all widths of bedding since the reaction is assumed to be uniformly distributed over the full width of bedding. K is usually found by first determining the bedding angle, α, one half the angle subtended by the arc of the pipe ring which is in contact with the pipe bedding, and then taking the corresponding value of K from a graph such as Figure 5. The bedding angle, α, is shown in Figure 4.

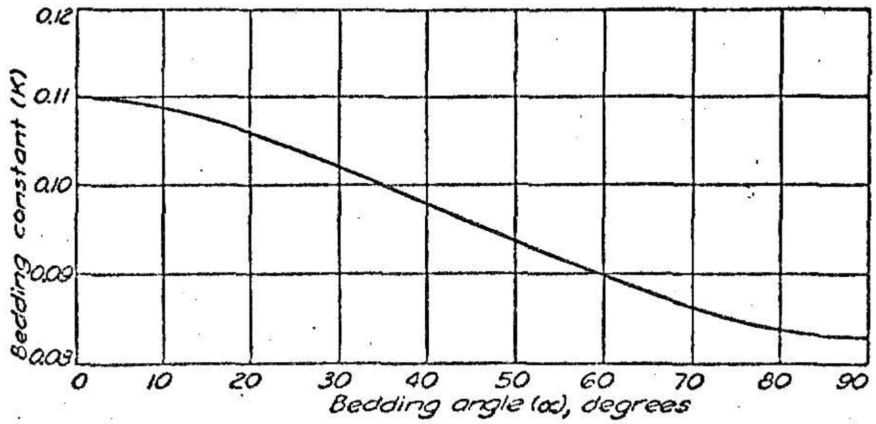

Figure 5

Graph for Determining Bedding Constant (K)

The vertical load on the pipe, W_c, can be predicted by Marston's load theory for certain stated conditions. If the pipe is relatively flexible, such as some steel and PVC pipe, then the load becomes more difficult to evaluate. The actual load depends on the soil properties and the relative rigidity of the conduit compared with the side fills (14). Since this is difficult to arrive at at times, one must design for the maximum load, thus greatly overdesigning much of the time.

The stiffness factor, EI, can easily be determined by several methods. The pipe material can be tested to determine E and the shape of the cross section of the pipe wall can be used to determine I. The stiffness factor, EI, can also be determined from the parallel plate test or a three-edge bearing test (15).

All of the factors in the deflection equation are fairly easily determined for a proposed flexible pipe installation, except the modulus of passive resistance, e. The evaluation of this modulus is a major stumbling block to the use of the Iowa Formula. The modulus is a measure of the lateral bearing resistance or support contributed by the adjacent soil as the sides of the pipe move outward (16). Terzaghi (5) defines a term similar to e, which he calls the coefficient of horizontal subgrade reaction. Earlier work by Westergaard (17) dealing with the analysis of concrete pavements resulted in a term referred to as the modulus of subgrade reaction. Cummings (18) defined a quantity the modulus of foundation in his analysis of foundation piles which is also similar to e.

Spangler (7), in introducing the modulus of passive resistance, e, indicated that it is a function of the density of the sidefill material. Tests showed that the greater the density, the greater was the modulus of passive resistance, e. The modulus appeared also to be a function of other properties of the soil, but to a much lesser degree than the soil density. Another interesting conclusion is that e was found to be independent of the height of fill over the pipe.

In an attempt to better understand the modulus of passive resistance, e, Watkins and Spangler (16) employed the principles of engineering similitude and the Buckingham pi-theorem in a study of the true nature of e and its influence on the deflection of flexible pipe culverts. The results indicate that the modulus of passive resistance, e, is not a function of the

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soil properties alone, but is materially influenced by the diameter of the pipe. The product, er, appears to remain constant in pipe-soil systems which have the same soil characteristics. This would indicate that e varies inversely as the pipe radius.

Watkins (19) studied the product, er, which has been given the symbol E' and is called the modulus of soil reaction, to determine if it is a function of any of the commonly recognized and easily measured soil properties. His tests show that E' is highly dependent on soil type and density, while only showing a comparatively small dependence on the wall stiffness, EI. Since E' relates to the soil, it is dependent on C_d and C_c, Marston's load coefficients, which in turn ~~is~~ are dependent on bedding angle, ditch conditions, projection condition, etc. The modulus E' was found to also relate to the compression index and the water content, but only to a minor degree. Watkins and Nielson (20) constructed a device, called the Modpares device, to measure E'. This device appears to give satisfactory results, but the complexity of the test and the time required to perform the test would certainly limit its usefulness. Nielson (21) established a relationship between E' and E by using the theory of elasticity.

$$E' = \frac{1.5\ E\ (1-\mu)}{(1+\mu)(1-2\mu)}$$

where

μ = Poisson's ratio for soil

E = modulus of elasticity of soil

This relationship shows that E' depends on E and μ for a given soil. Both of these values can be determined from the triaxial shear test, thus E' can be determined also. E and μ are assumed constant in this analysis, which is not valid for soils. The error introduced by this assumption is thought to be very small. E' is very sensitive to μ, but for most design work a value of $\mu = 0.25$ can be used. More work is needed to further this investigation.

A more practical means for determining E' for design purposes might be the one presented by Nielson, Bhandhausavee, and Yeb (22). They worked to find a correlation between E' and the results obtained from the CBR test, Hveem's stabilometer test, and standard soil properties. Lab tests show that the relationships obtained yield results satisfactory for most design situations. Some of the restrictions on their use appear to be that not all soils work or fit the design values recommended and only low (around 3%) deflection values work well.

Soil density was considered to be the most important variable in determining E', affecting its value as much as 35 times. Consolidation was found to work to get E' only for deflections less than 3%. Optimum moisture has little effect and plasticity index cannot be depended upon to predict E'.

After all the work done to try and understand E' and relate it to known soil tests and soil properties, it still appears that E' is a very difficult quantity to determine.

Realizing this, several studies have been based on incorporating E' in other properties or have come up with other means of predicting flexible pipe performance. The most noteworthy of these have been the ones which involve testing model pipe or actual pipe installations and then determining the total performance of the pipe-soil system. Results from these types of tests have led to new and easier design procedures.

Watkins and Smith (23) developed a method based on ring deflection for the structural design of CML and CML-CMC steel pipe. They discovered that pipe ring stiffness, EI/D^3, is more important in loose soil than in dense soil. Also, if the soil is well compacted, the ring stiffness is less important and has little influence on ring deflection, $\Delta y/D$. A number of graphs of pipe and soil stiffnesses have been designed to predict ring deflection. Later this method was extended to include all types of flexible pipe conduits by Watkins and Smith (13). For this purpose the Iowa Formula is written in terms of dimensionless parameters. Prediction of ring deflection is based on the earlier study with the values being placed in the modified Iowa Formula.

Extensive field tests by Watkins and Moser (24) resulted in performance curves for corrugated steel pipes in backfills of varying density. These curves are used as a guideline in the design of actual pipe installations. The curves emphasize the importance of proper compaction in the installation of corrugated steel pipe, with a recommendation to specify backfill to at least 90% standard density to avoid trouble.

Bishop and Moser (14) presented a table predicting the maximum long term deflections of PVC conduit in various ASTM Standard D2321-72 bedding classifications and AASHO T-99 proctor density specifications for various heights of cover over the pipe.

Moser, Watkins, and Bishop (25) tested 12 to 24 inch PVC pipe in a test cell to determine the structural performance of the PVC pipe when subjected to external soil pressure. The soil used was a typical sand which was placed at varying densities. The results included a load-deflection chart which includes soil density zones. Tests performed relating ring deflection, $\Delta y/D$, to ring flexibility, D/t, for a constant vertical soil pressure revealed interesting results. They show that there are definite relative minimum points at a ring flexibility of about $D/t = 37$ to 38. These minimum points are at optimum ring flexibility. They indicate the least vertical ring deflection as a function of ring flexibility with all other variables remaining constant.

In contrast to this study, Howard and Selander (15) reported that for flexible steel pipe, the ring stiffness has less effect on pipe deflection with a high soil modulus than with a low soil modulus of the same soil type. In fact, they state that with a very high soil modulus, ring stiffness has negligible effect on the pipe deflection; the deflection being controlled by the soil modulus value.

Increasing the density of a clay from 90% to 100% proctor reduced the pipe deflection about 50% while a high modulus sand reduced the deflection about 95%. They also concluded that PVC pipe deflects similarly to steel pipe of the same stiffness, EI/r^3. Tests on RPM, FRP, and polyethylene pipe gave results which are confusing and not in line with those of steel and PVC pipe.

When an engineer looks at these various studies and decides to use a particular one to design an actual flexible pipe installation, he must be absolutely sure that the pipe in the field will perform exactly as the model pipe performed.

Williams (26) states that perhaps in the efforts to model test flexible pipe, a couple of significant factors have been overlooked. The pipe-soil adhesion is generally ignored and the confines of the test chamber may significantly influence the test results. The pipe-soil adhesion for PVC pipe is currently being studied by F. E. Erickson in a Master's thesis.

Several studies have been done to try and measure the extent of the zone of deformation of the sidefill material as the pipe is loaded and deflects. Spangler and Donovan (27) placed metal plates in vertical positions in fill around a pipe in a test box. After the tests, the positions of these plates relative to their beginning was measured. They were successful in two attempts in uncompacted fill and unsuccessful in three attempts in compacted fill. They concluded that the influence of the size and shape of the test box could not be determined.

Nielson (28) tells of a study performed by R. K. Watkins in which lead shot was placed in a grid pattern in the soil mass around a model pipe. As the model was loaded, the movement of the lead shot was followed by taking a series of x-ray pictures. Among the conclusions was that it is not known what effect the boundaries of the cell had on the displacement pattern observed. The cell wall appeared to be approximately 2D from the pipe.

Watkins and Smith (23), in predicting the ring deflection of buried pipe, concluded that in a pipe trench installation, if the walls of the trench are vertical and rigid, the ring deflection is less by a ring deflection factor which is in terms of the ratio of trench width B, to the pipe diameter, D.

B/D	Ring Deflection Factor
1.5	0.86
2	0.92
3	0.98
∞	1.00

For most trench installations, the chart shows that the width of the trench can be conservatively neglected.

Howard (29) placed pressure cells in the soil container walls to measure the horizontal soil pressures on the wall. In the tests on steel and RPM pipe, the cells opposite the pipe showed about the same pressures as the cells above the influence of the pipe for 18-inch diameter pipes. For 24 and 30-inch pipe, the cells opposite the pipe showed definitely higher pressures than the upper cells. The container width was seven feet.

Telescoping tubes with small plates on the ends were buried in the soil in line with the horizontal diameter of the pipe. The ends of the tubes extended through the soil container wall so horizontal soil movements during the loading could be measured. The data show that 50% of the soil compression between the pipe and the container wall occurred in the nine inches of soil adjacent to the pipe for 18-inch pipe. Yet even at small vertical loads, the soil near the container wall was displaced horizontally.

It appears from the limited results that the extent of the zone of deformation of the sidefill material is not known. Therefore, no one can say which model studies are affected by the container and which are not. Since model studies are becoming the popular method of design of flexible pipe systems, this presents an interesting situation. Thus, it seems that the zone of deformation, how it relates to the size of the pipe, and how it is affected by soil properties is a subject which should be investigated.

DESIGN OF EXPERIMENT

A testing program was set up to model test sections of four diameters of flexible pipe in a metal chamber. PVC pipe was chosen because it was easily acquired, it is growing in popularity for use as a buried conduit, and it was thought to be easier to work with in the experiments performed. Model testing refers to simulating actual buried conduit systems in terms of placement and loading, but on a smaller scale. The chamber used for these tests is shown in Figure 6. The test sections of pipe were 12 inches in length and included four sizes having diameters of 1.00, 1.30, 1.65, and 2.40 inches. These sizes of pipe were chosen in hopes that the effects of the chamber walls on the testing results would be minimal and so that any movement of the soil could be observed through the transparent plexiglass front.

The backfill soil used was a clean river sand obtained locally which was easy to work with and of the type commonly encountered in conduit construction. Results of tests performed by F. E. Erickson on the sand to determine some significant properties are included in the presentation of data. Two densities of backfill were used for purposes of comparison. The soil was either placed as loosely as possible around and above the pipe or it was placed as dense as possible. These two densities represent the extremes of backfill placement.

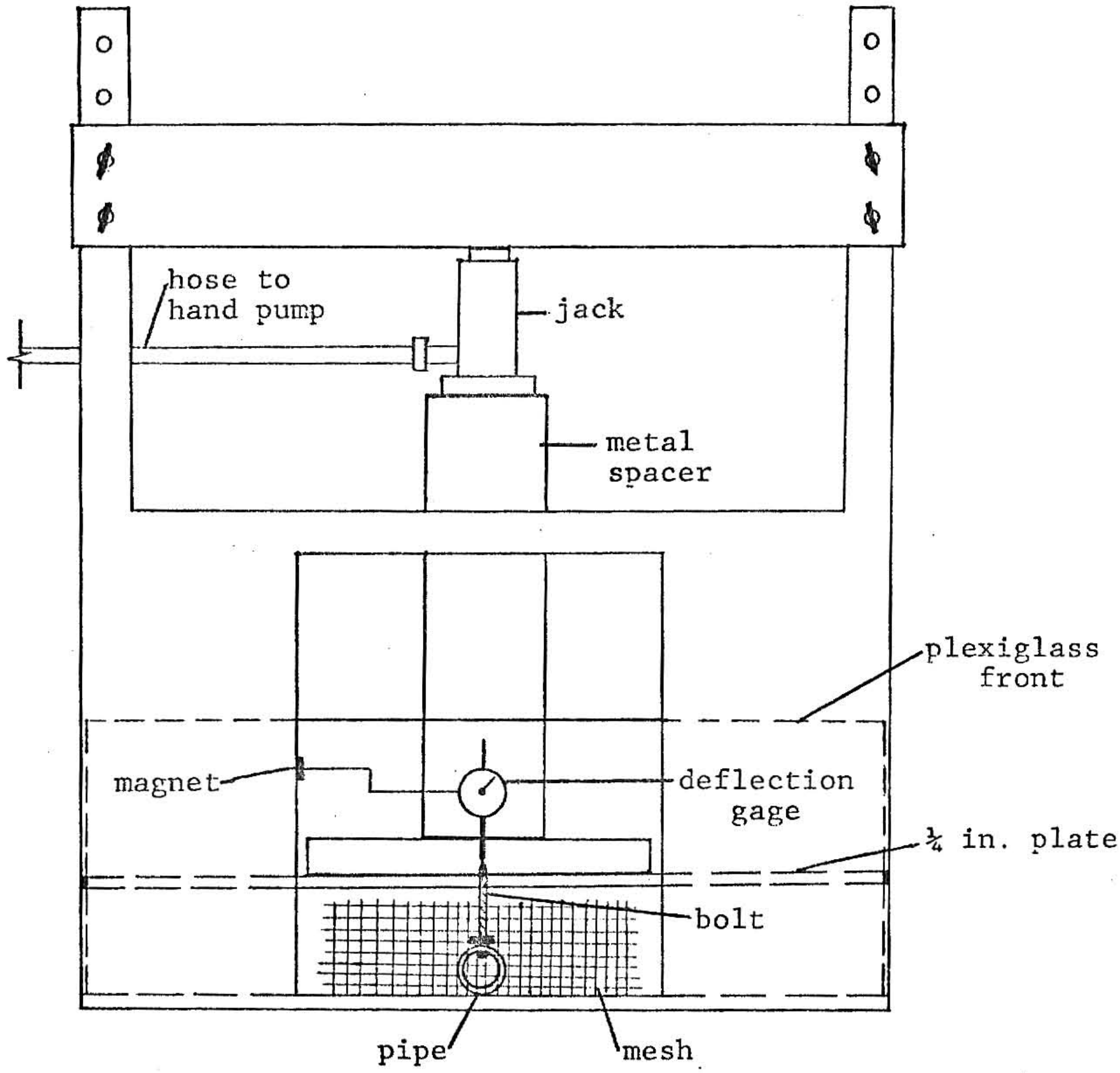
The soil was placed to a height of at least one diameter above the top of the pipe, which is in accordance with other model studies. A 1/4-inch thick plate covered the soil entirely and a hydraulic jack placed on the plate exerted the vertical pressure. The plate and jack together simulate the building of a fill over the buried conduit as the load is applied by a hand pump.

A nylon mesh was placed in between the plexiglass front of the test chamber and the sand fill to aid in seeing the peripheral movement of the side fill.

The pipe was placed on the bottom of the chamber without allowing for any type of bedding as in an actual installation. This type of placement made it easy to measure the vertical deflection of the pipe. A 3/16-inch bolt was placed through the top of the pipe and secured. The bolt, as shown in Figure 6, extended vertically upward through the fill and a hole in the plate. A deflection gage was mounted on top of the bolt to measure the vertical pipe deflection.

Each of the four diameters of pipe were tested through for series of loadings for a total of 16 load tests. Of the four series on each diameter pipe, two were with loose backfill and two were with dense backfill. During each test the pipe was loaded to various deflections, and the visible movement zone was observed and measured.

The visible movement zone, V_z, is the visible peripheral movement of the sidefill material caused by deflection of the pipe. This measurement was taken in all cases from the original position of the pipe to the furthermost extent of the visible movement of the soil. The movement was measured



Scale: 1 in. = 6 in.

Figure 6

Test Chamber With Pipe in Place

on only one side of the pipe, but was observed to be symmetrical about both sides. V_z is shown in Figure 7.

The pipe sections were generally deflected vertically to at least 5% of their diameters, which is usually considered the performance limit for most flexible pipe in actual service.

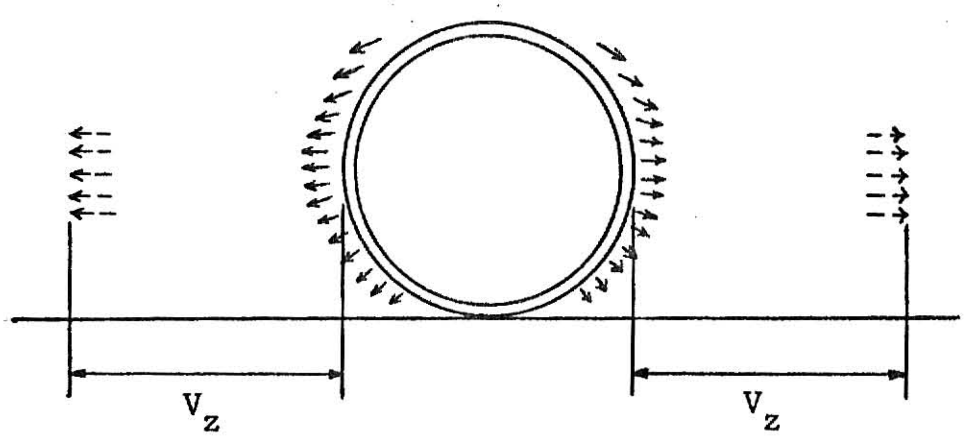


Figure 7

Peripheral Soil Movement and V_z

PROCEDURE

The same procedure was followed for the 16 load tests performed. First the chamber was cleaned, and the plexiglass front was installed, allowing for the pipe to be placed on the bottom of the chamber. Then placement of the nylon mesh on the inside face of the plexiglass was next and the soil was carefully placed around the pipe. The same backfilling procedure was used for all of the loose backfill tests in order to achieve identical conditions for each test. Soil was poured in from a height of six inches and leveled, thus handling the soil as little as possible and hopefully achieving a very loose and uniform backfill. For the dense fill, soil was placed in the box in one inch layers that were carefully tamped by a 2 x 2-inch wooden block with the tamping covering the entire surface three times for each lift. Tapping the sides of the chamber after completion of the fill also aided in densifying the soil.

The plate was lowered onto the fill and over the bolt. The jack assembly was then set up and centered over the pipe. The dial gage for measuring the vertical deflection of the pipe was mounted over the bolt. A small strip of divisioned paper was pasted on the plexiglass to be used in measuring the visible movement zone. The pipe was loaded to a set deflection, and the visible movement zone was observed and measured. The pipe was then loaded to the next deflection and the zone was measured again. This was repeated until the pipe had been deflected vertically to at least 5% of its diameter.

In addition to measuring V_z as the pipe was loaded, the peripheral movement of the soil as a whole was watched to see if the affected zone had an identifiable shape.

PRESENTATION OF DATA

The results of the tests on the sand are presented in Figures 8-14 and Tables I-IV. The data from the load tests is presented in Tables V-VIII, showing the extent of the visible movement zone, V_z, for set vertical pipe deflections of the four pipe diameters. This data is shown in graphical form in Figures 15-18. Figures 19-22 depict the percent deflection, $\Delta y/D$, versus the visible movement zone, V_z, divided by the pipe diameter, D, for the four diameters of pipe tested.

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

Figure 8

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

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

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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.

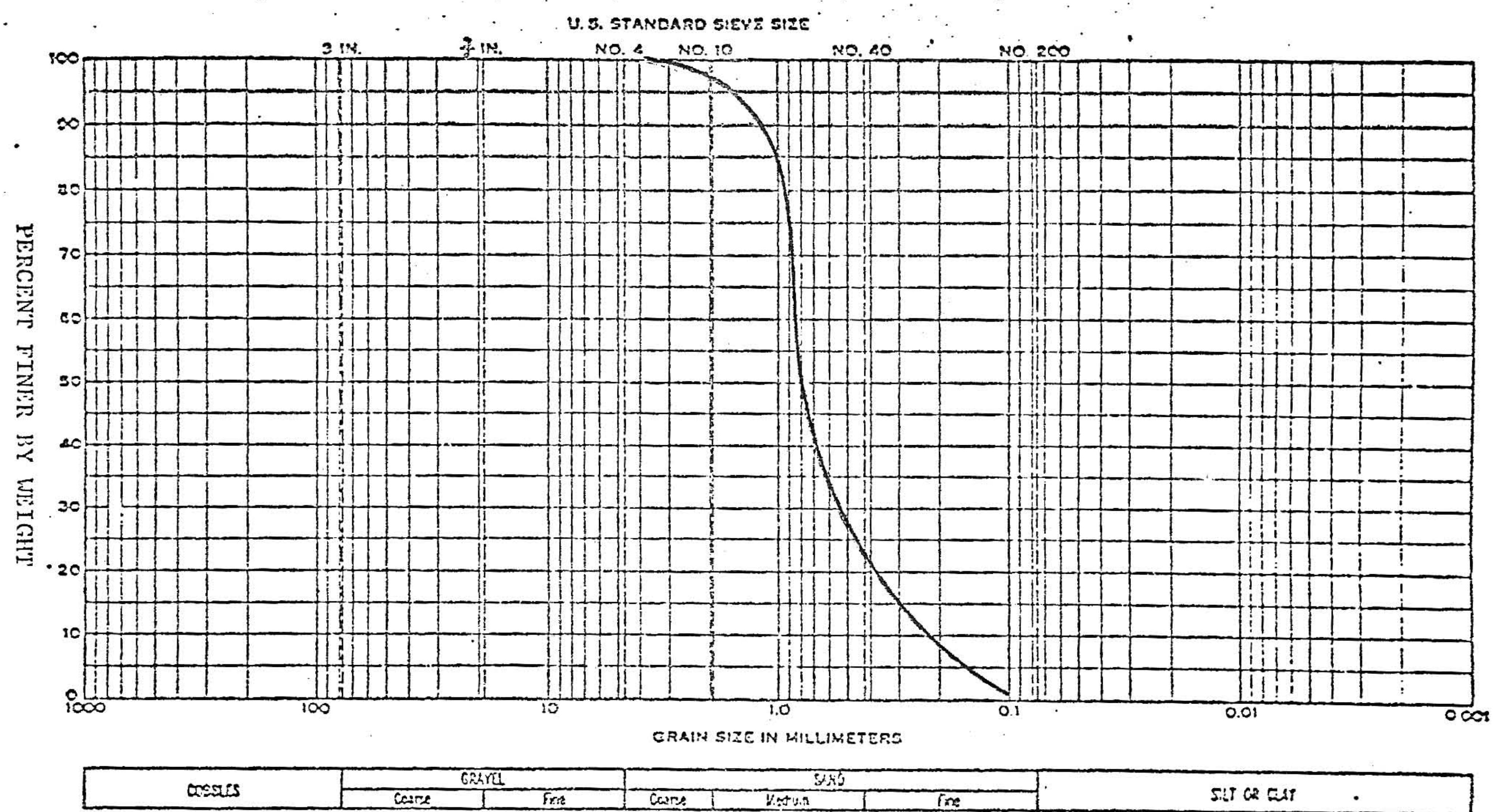

Figure 9

Sieve Analysis

1st Determination

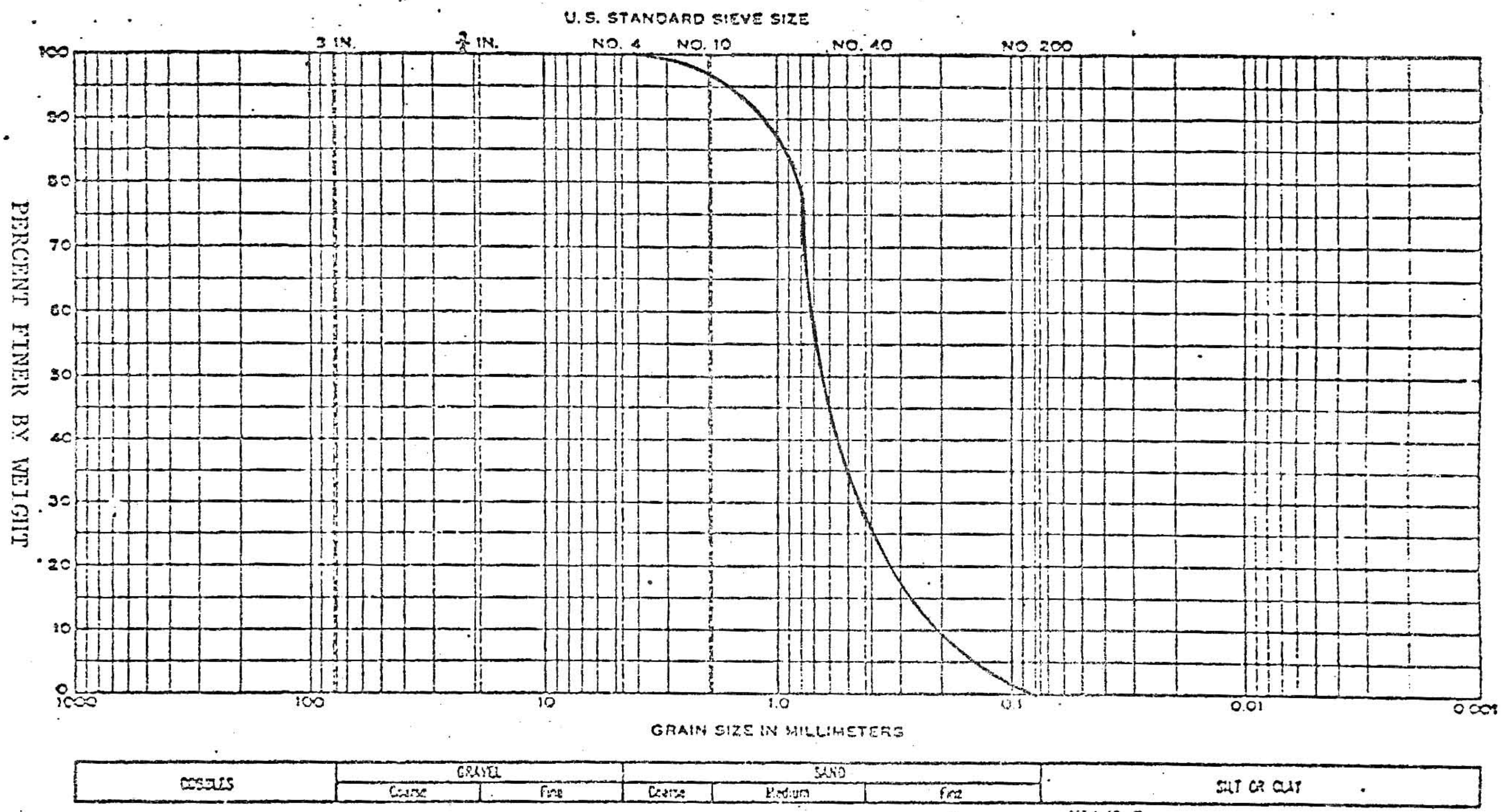

Figure 10

Sieve Analysis

2nd Determination

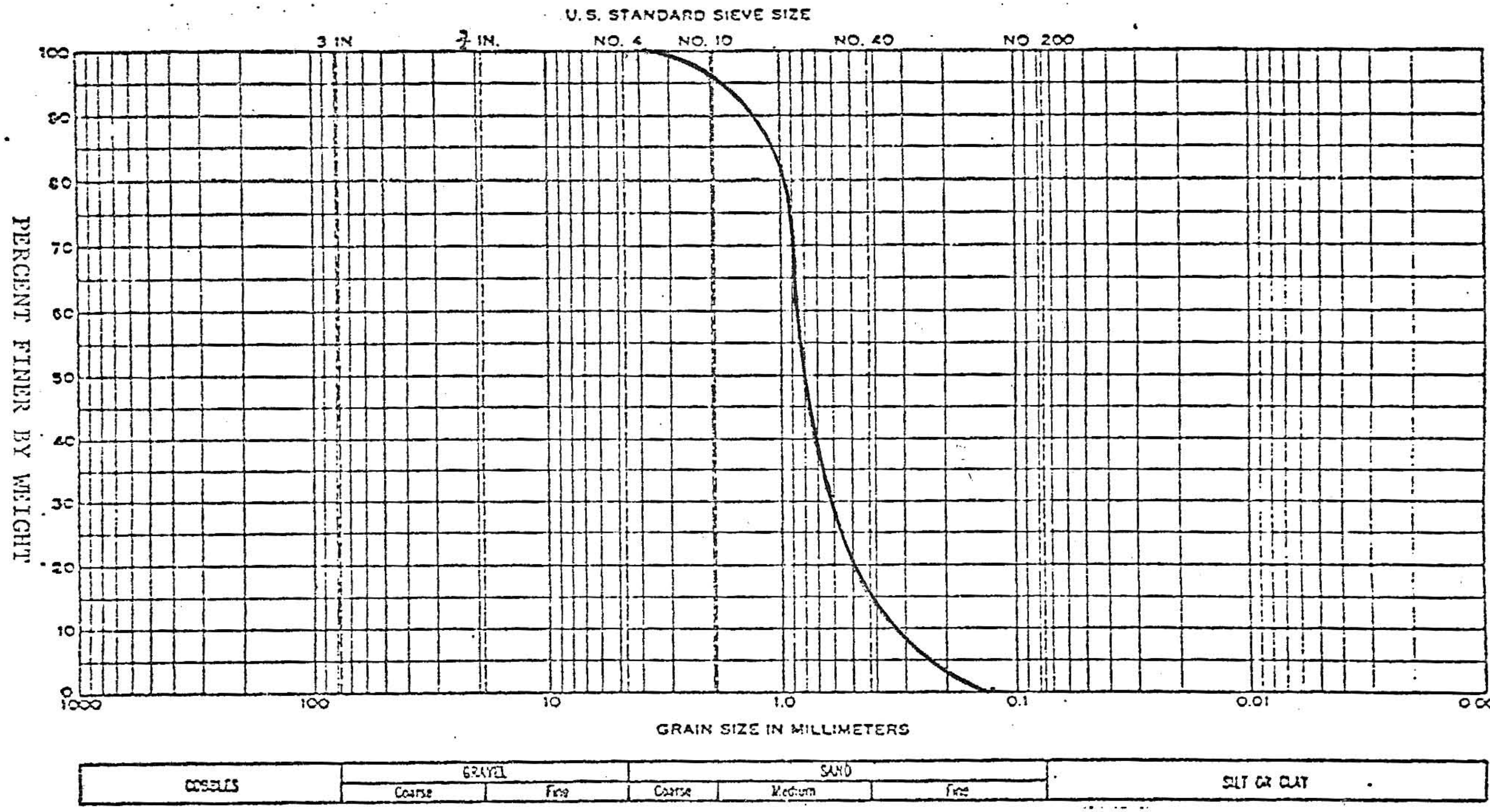

Figure 11

Sieve Analysis

3rd Determination

TABLE IV

DIRECT SHEAR

Shear Specimen Data:

Diameter: 2.5" Volume: 6.14 in^3

Area: 4.91 in^2 Weight of Sample: .414 pound

Thickness: 1.25"

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

$\phi_1 = 40.5°$

$\phi_2 = 40.0°$ ϕ average $= 40.5°$

$\phi_3 = 41.0°$

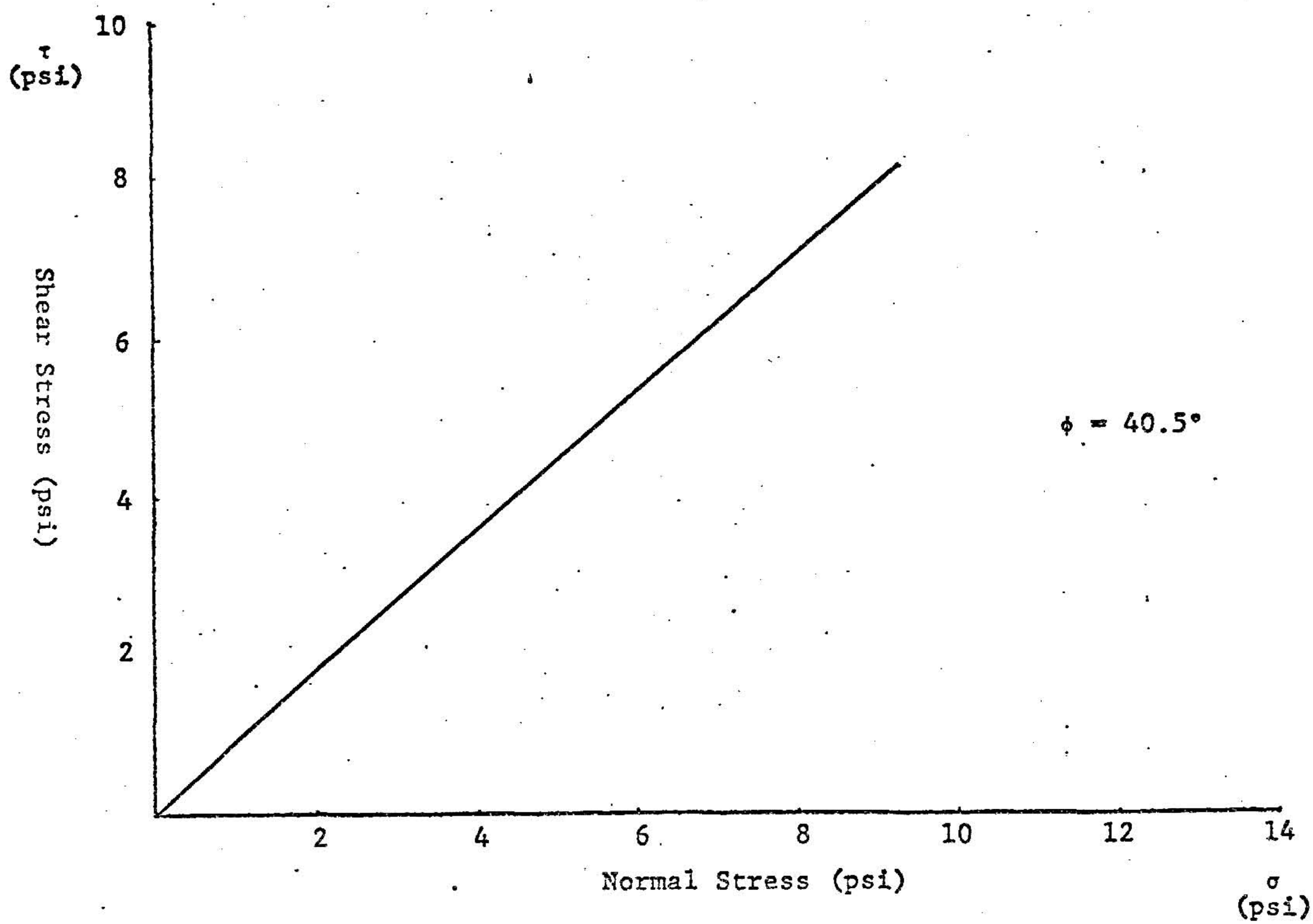

Figure 12

Direct Shear
1st Determination

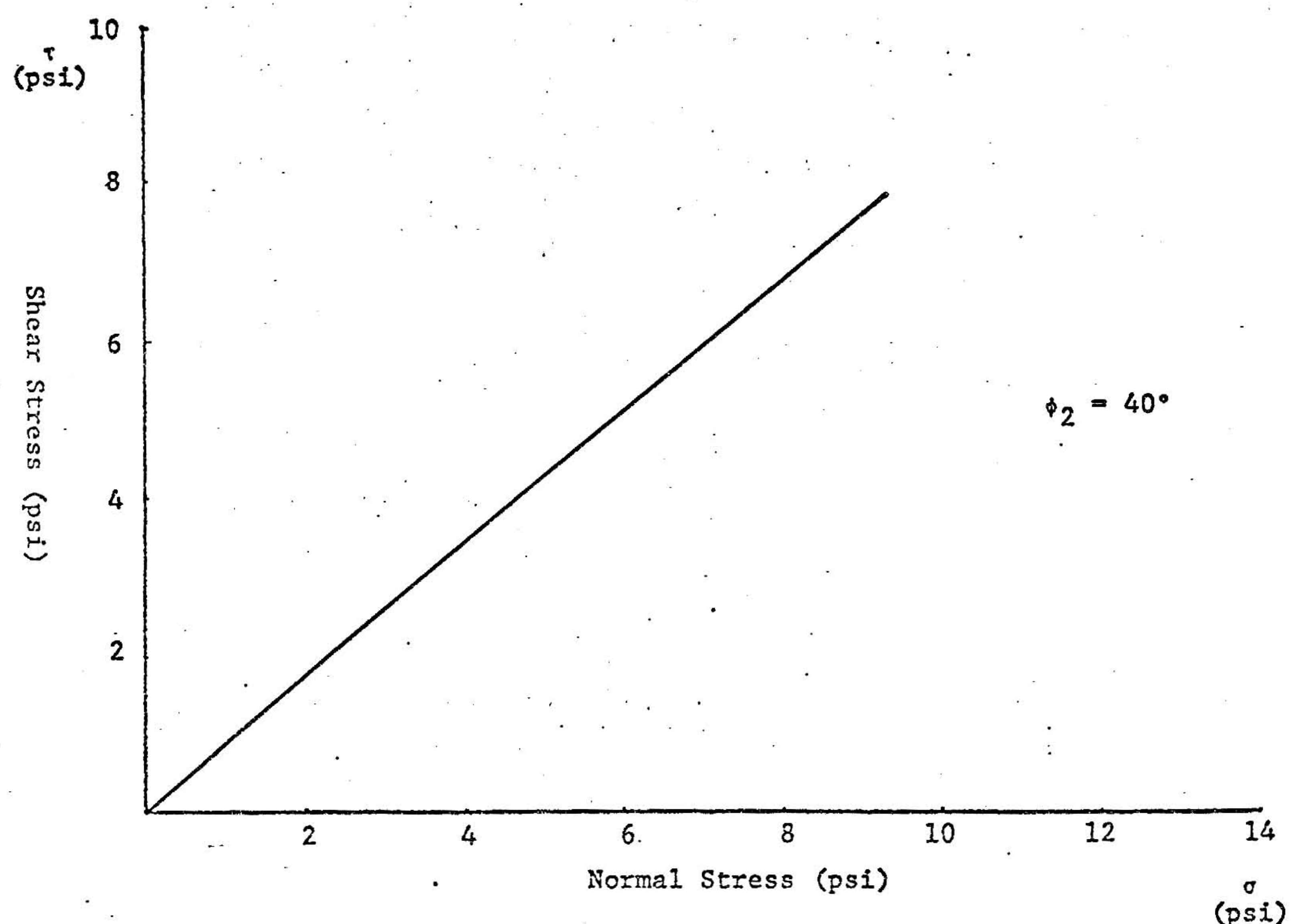

Figure 13

Direct Shear
2nd Determination

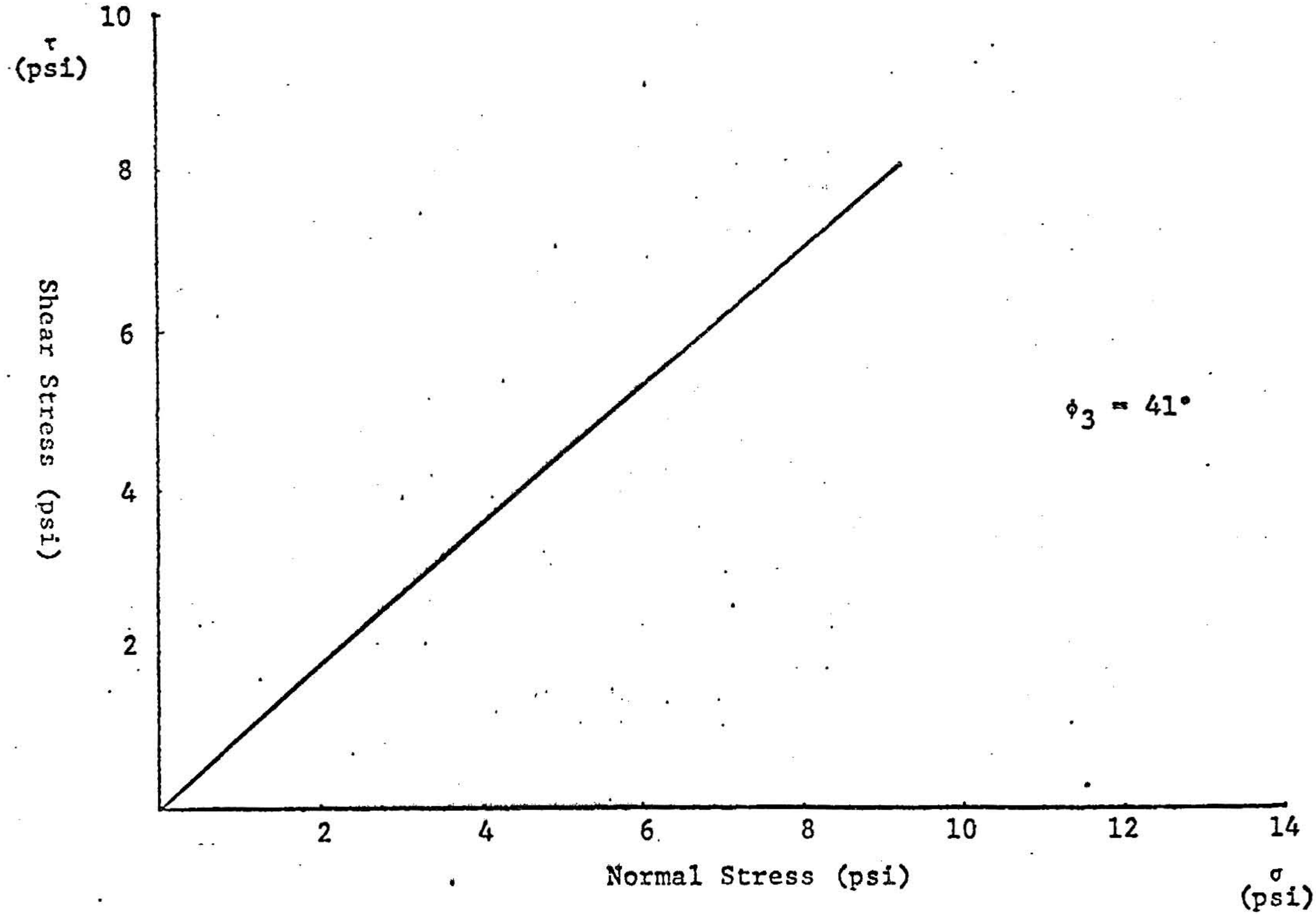

Figure 14

Direct Shear
3rd Determination

TABLE V

Trial #1 Loose Fill

Pipe Diameter D (inches)	Visible Movement Zone - V_Z (inches)						
	Vertical Pipe Deflection - Δy (inches)						
	0.02	0.04	0.06	0.08	0.10	0.12	0.14
1.00	0.15	0.50	1.05				
1.30	0.15	0.45	1.15	2.10			
1.65	0.25	0.40	0.70	1.40	2.55		
2.40	0.20	0.35	0.50	1.20	1.90	2.70	3.35

TABLE VI

Trial #2 Loose Fill

Pipe Diameter D (inches)	Visible Movement Zone - V_Z (inches)						
	Vertical Pipe Deflection - Δy (inches)						
	0.02	0.04	0.06	0.08	0.10	0.12	0.14
1.00	0.10	0.45	1.25				
1.30	0.25	0.60	0.90	1.55			
1.65	0.20	0.45	0.85	1.60	2.25		
2.40	0.15	0.30	0.60	1.45	2.05	2.45	3.10

TABLE VII

Trial #1 Dense

Pipe Diameter D (inches)	Visible Movement Zone - V_Z (inches)						
	Vertical Pipe Deflection - Δy (inches)						
	0.02	0.04	0.06	0.08	0.10	0.12	0.14
1.00	0.45	1.20	2.70				
1.30	0.45	1.05	2.50	--			
1.65	0.40	1.30	2.35	3.15	--		
2.40	0.50	1.00	1.95	3.25	3.90	5.15	--

TABLE VIII

Trial #2 Dense

Pipe Diameter D (inches)	Visible Movement Zone - V_Z (inches)						
	Vertical Pipe Deflection - Δy (inches)						
	0.02	0.04	0.06	0.08	0.10	0.12	0.14
1.00	0.35	1.20	2.45				
1.30	0.50	0.95	2.20	--			
1.65	0.60	1.25	2.15	3.30	4.00		
2.40	0.45	1.25	1.75	3.50	4.40	5.25	--

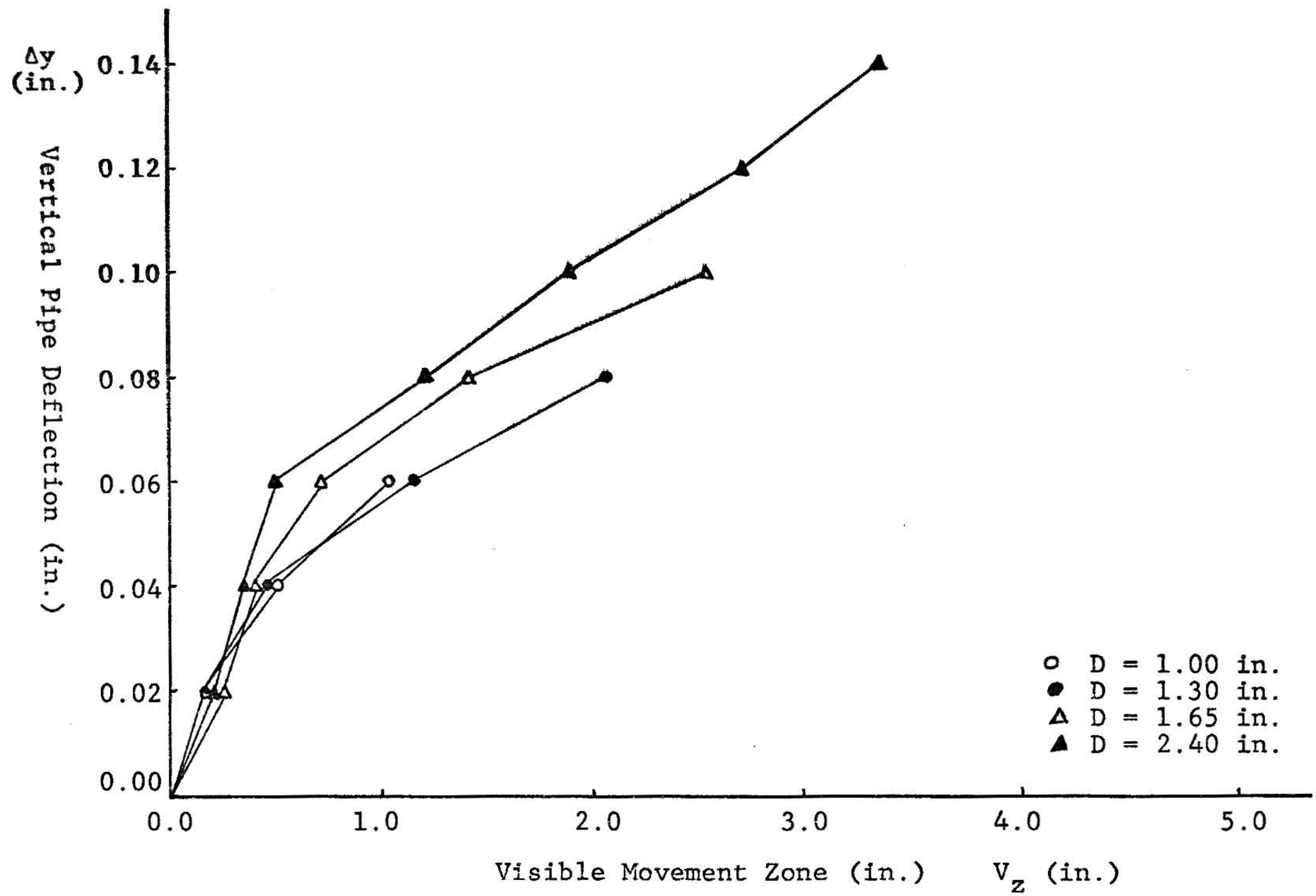

Figure 15

Trial 1 Loose

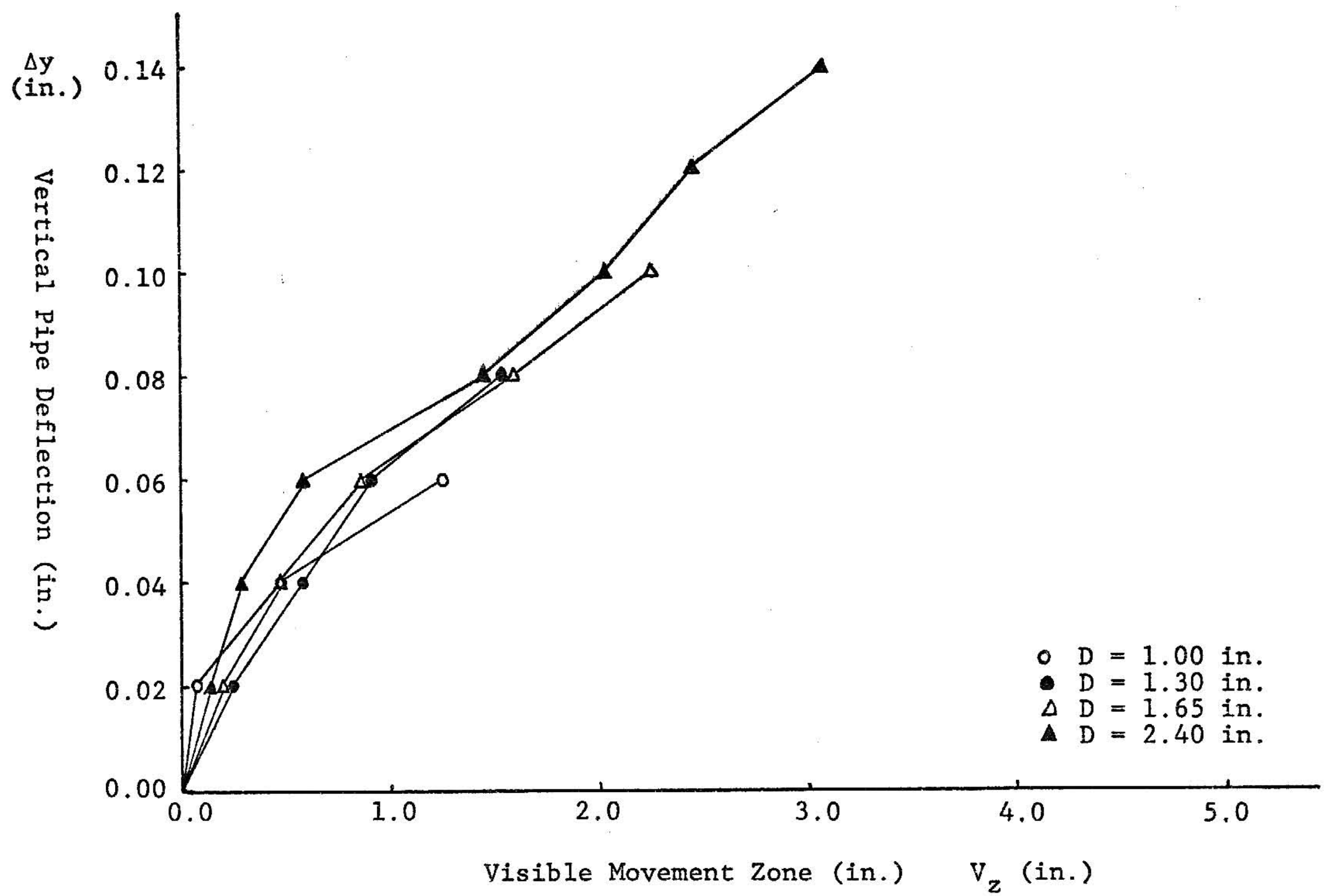

Figure 16

Trial 2 Loose

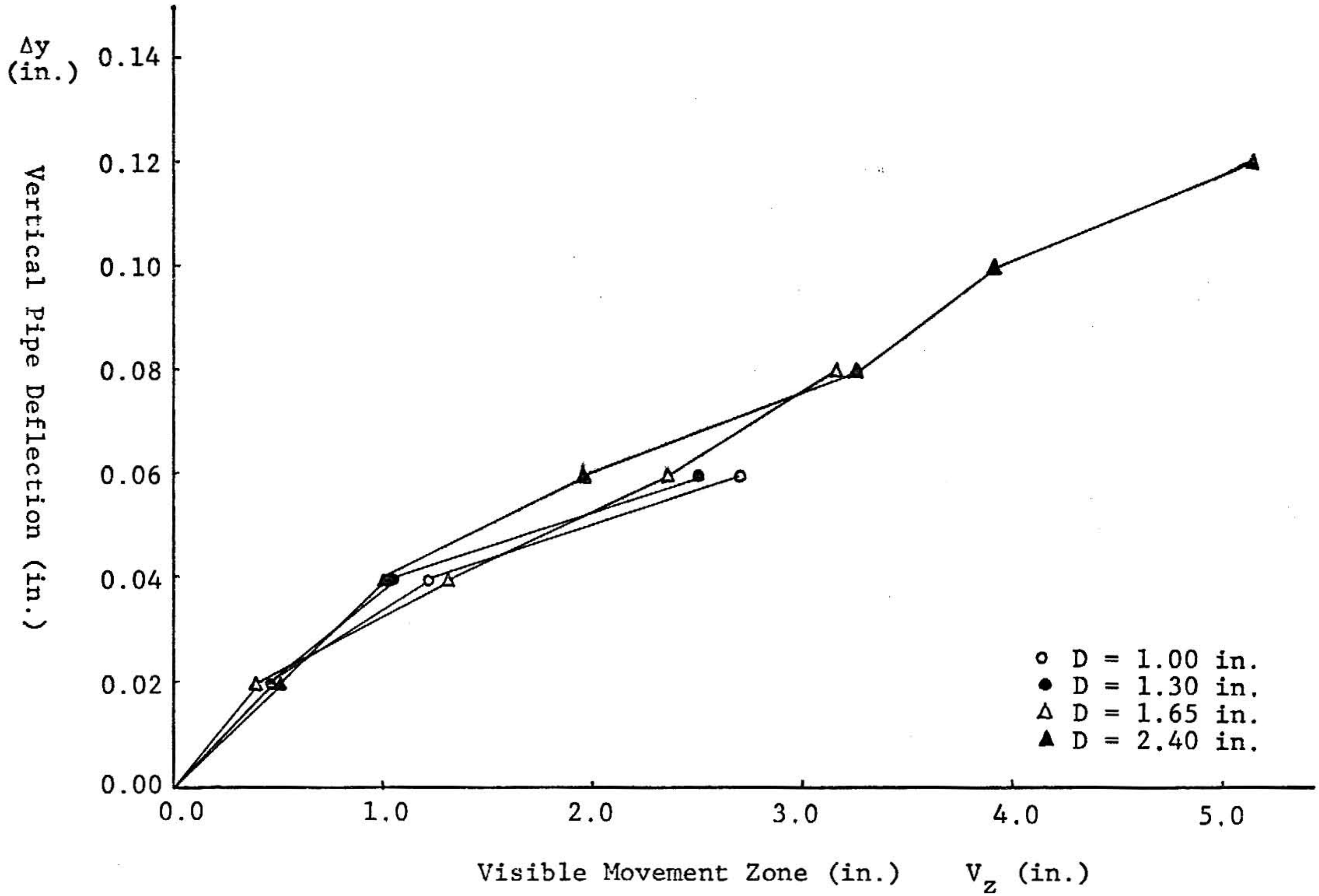

Figure 17

Trial 1 Dense

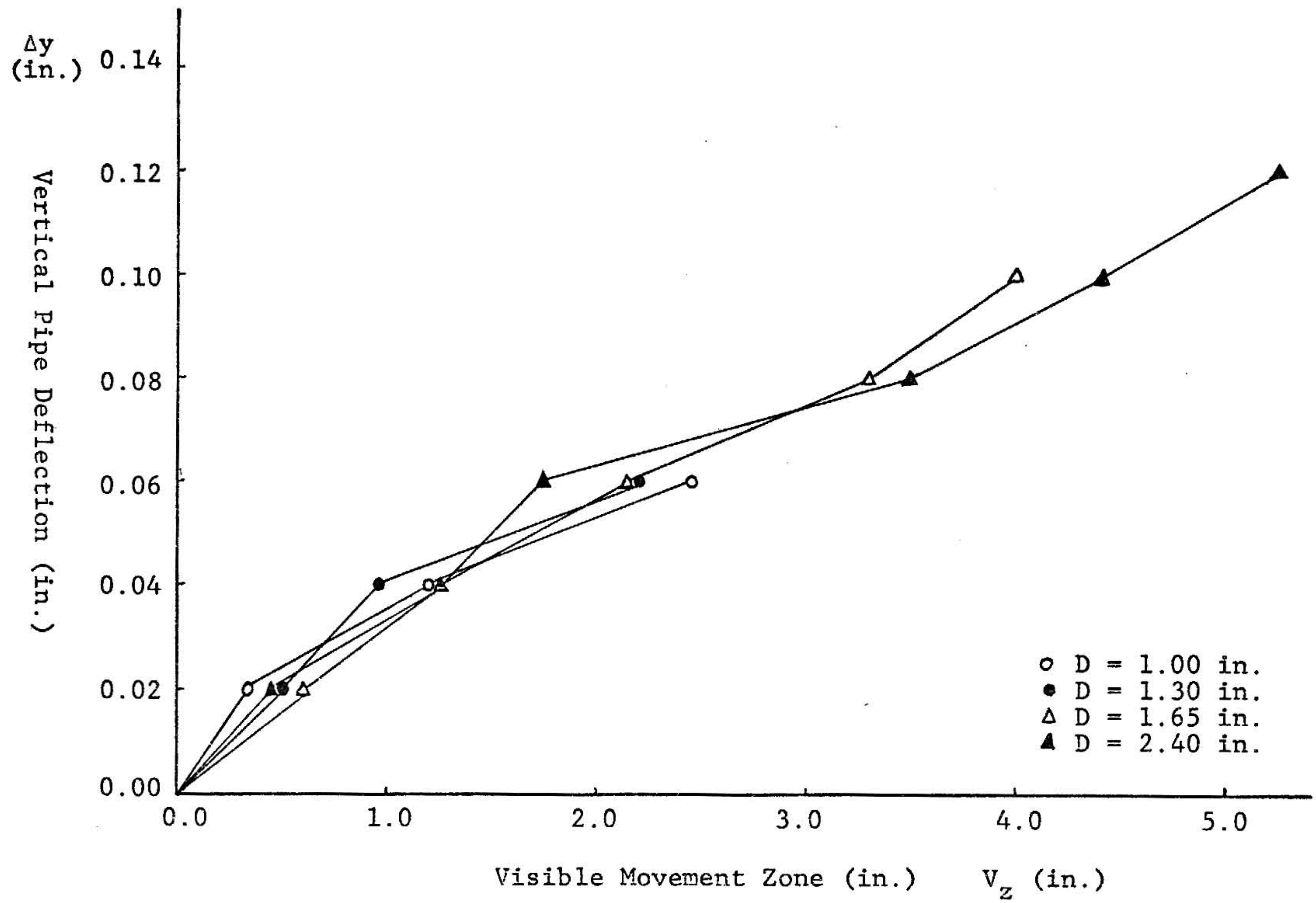

Figure 18

Trial 2 Dense

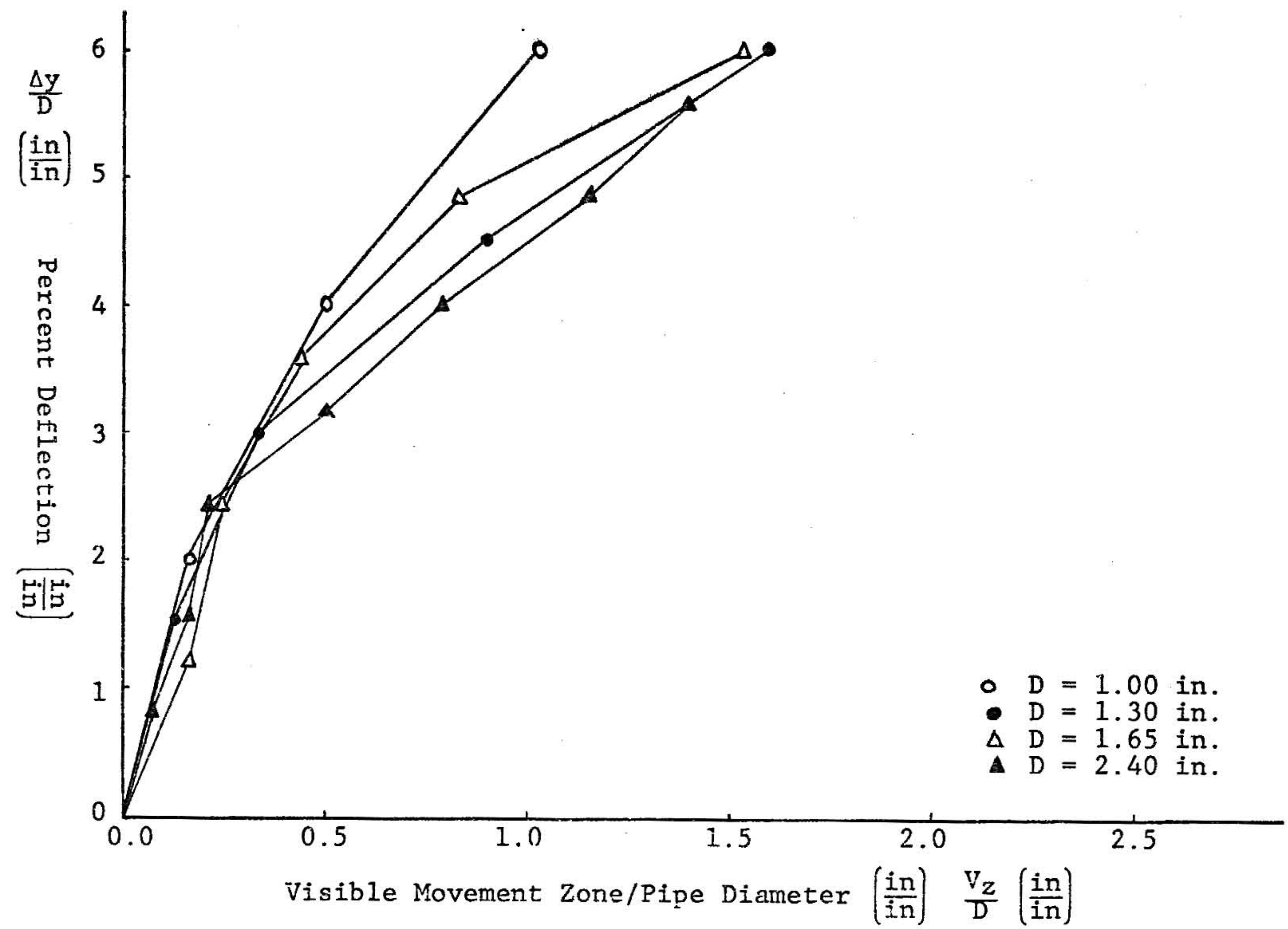

Figure 19

Trial 1 Loose

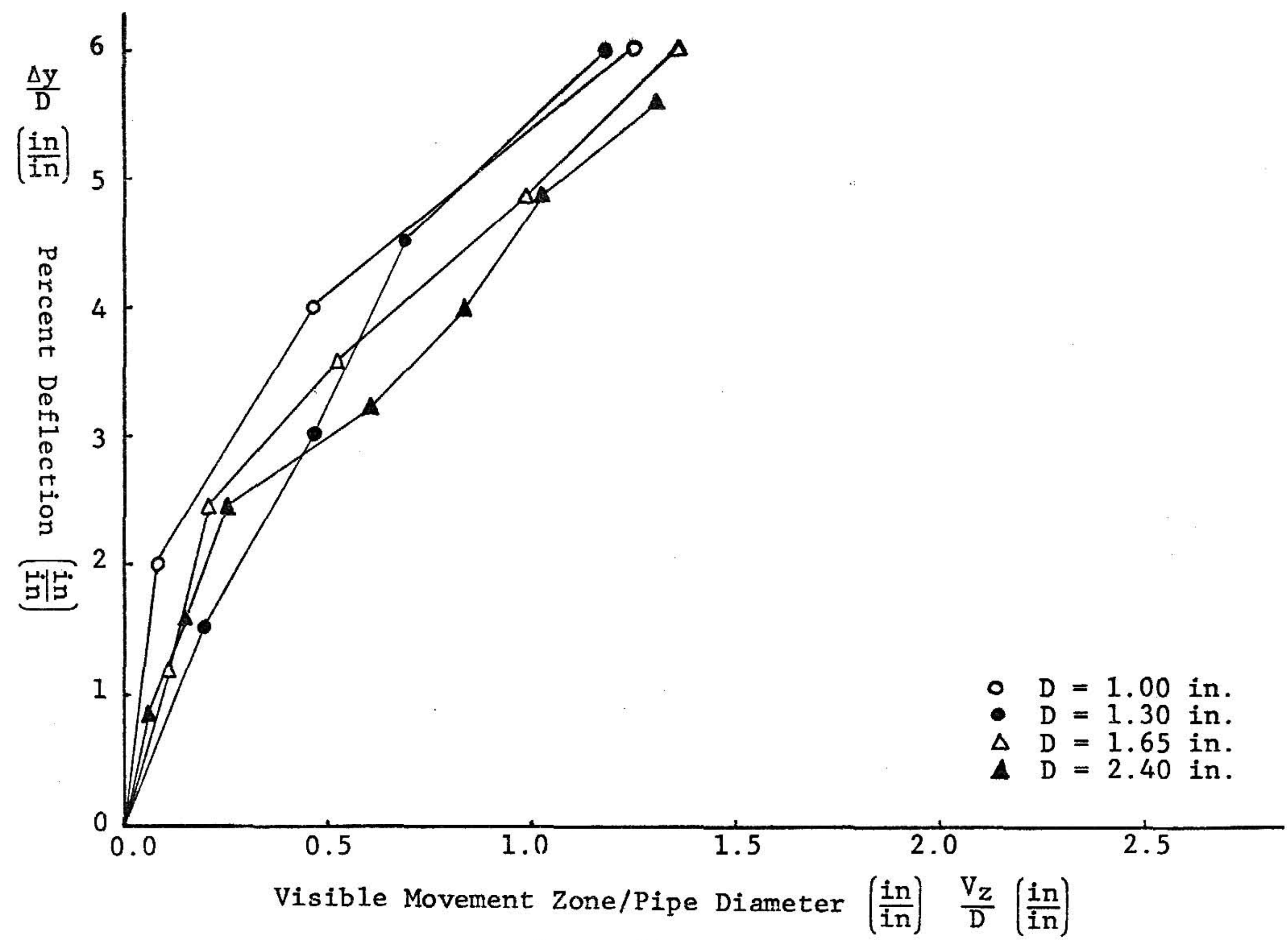

Figure 20

Trial 2 Loose

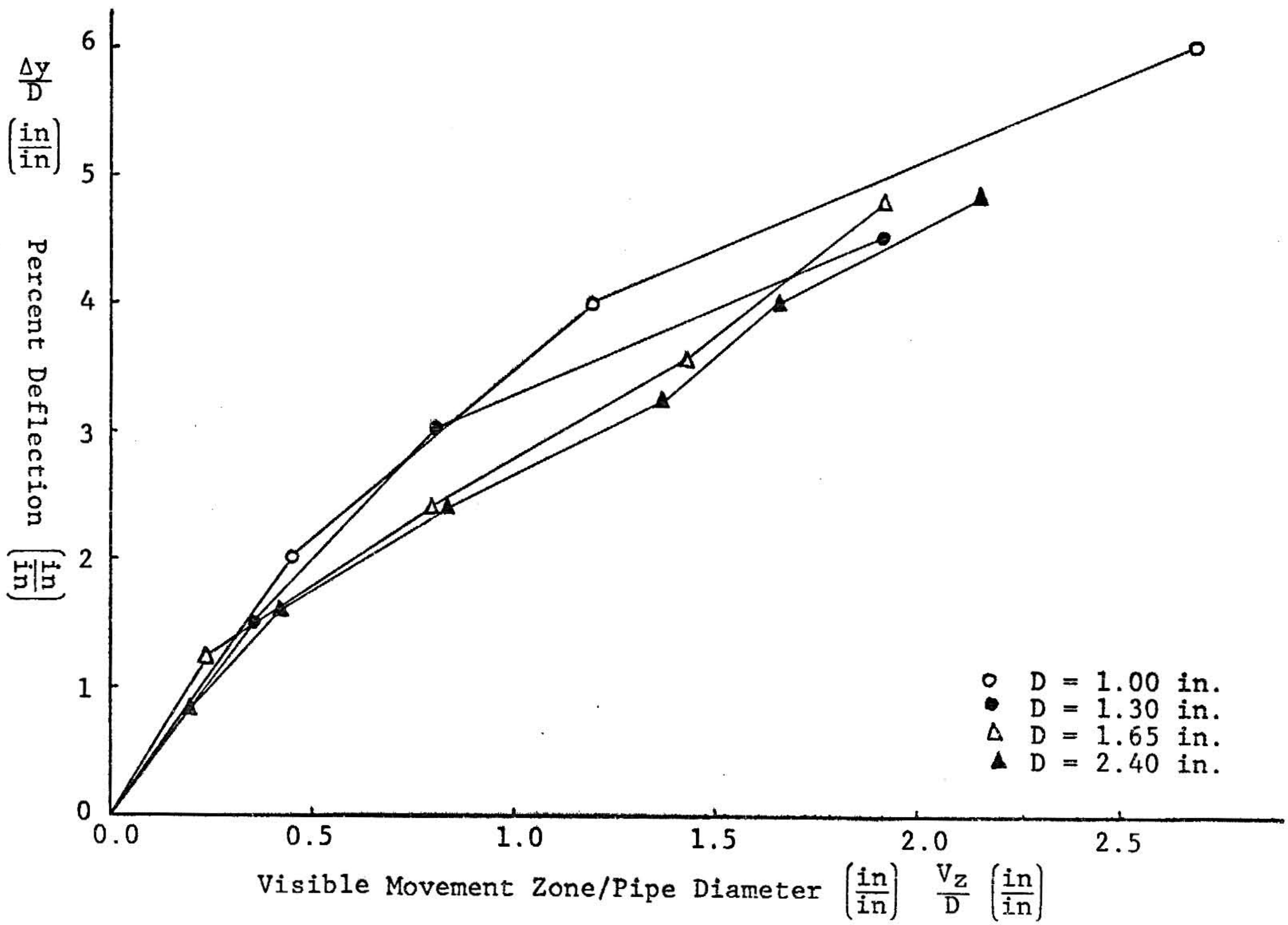

Figure 21

Trial 1 Dense

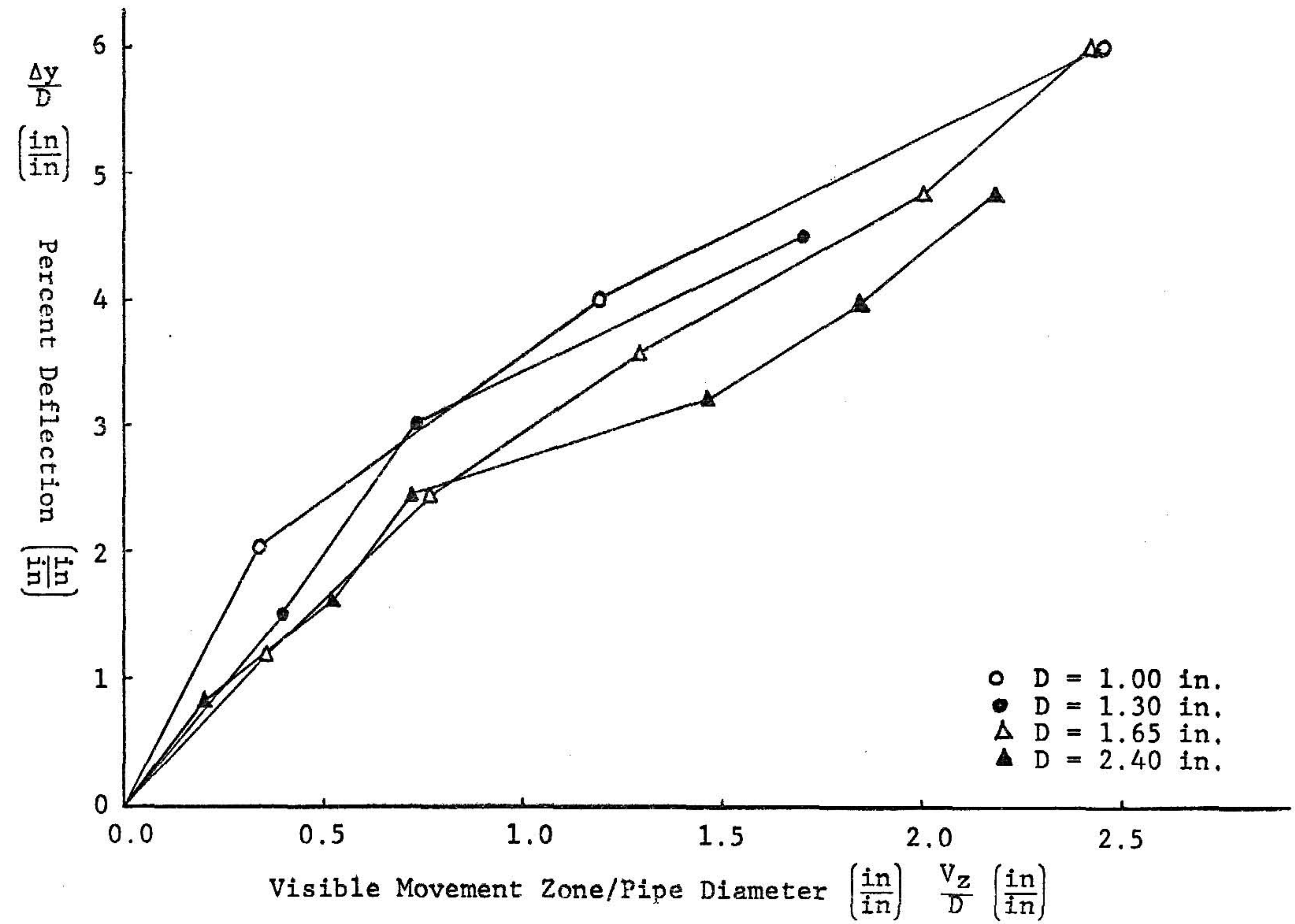

Figure 22

Trial 2 Dense

DISCUSSION OF RESULTS

There are a number of things which should be brought to the attention of the reader concerning the results of the experiments.

1) The visible movement zone, V_z, is only the extent of visible peripheral movement of the soil caused by the deflection of the pipe. It is felt that the actual peripheral movement zone is greater than what can be seen by the eye, but that the visible movement zone is still representative of the actual zone.

2) To see the movement of the soil required close observation, and so only the movement on one side of the pipe was recorded.

3) The pipe was placed on the bottom of the test chamber without allowing for any type of bedding in order to magnify the soil reaction at the sides of the pipe.

4) The pipe sizes used were chosen because of the limitations of the size of the test chamber.

5) The results for each of the test runs were found to be fairly consistent.

6) A single soil type (sand) at two different densities was used in the tests.

7) Previous to the recorded tests, much testing was done in order to perfect testing techniques.

8) Although the tests were performed on small diameter pipes, there is little reason to doubt that the results can be applied to larger scale model tests.

9) The peripheral movement of the soil observed during the tests had no distinct shape. When the pipe was first beginning to deflect under loading, the peripheral movement of the soil generally followed the directions as shown by the solid arrows in Figure 7. As the deflection continued and approached 5% of the pipe diameter, the peripheral movement was primarily horizontal as shown by the hatched arrows in Figure 7. Overall there was no distinct shape to the peripheral movement that was observed during the tests.

CONCLUSIONS

The results from the tests indicate that the visible peripheral movement of the soil due to the deflection of the pipe is dependent upon the amount of deflection of the pipe, the size or diameter of the pipe, and the density of the soil. Several factors have led to the conclusions. They are:

1) As the deflection of the pipe increased, the visible movement zone, V_z, increased also.

2) As the size of the pipe was increased for the same amount of deflection and the same original soil density, the value of V_z generally was found to be smaller.

3) As the size of the pipe was changed for the same percent deflection, $\Delta y/D$, and the same original soil density, the values of V_z/D could be seen to fall in the same range.

4) As the original density of the fill was changed from loose to dense, V_z/D increased for all sizes of pipe at a constant percent deflection. For the loose fill the range of values of V_z/D for 5% deflection averages to about 1 and for the dense fill the range of values of V_z/D for 5% deflection averages to about 2. Intermediate densities should fall between these extremes.

Thus, it has been shown that the visible peripheral movement of the soil is a function of the pipe-soil system and can extend to a considerable distance under certain conditions. It is not known what the side effects would be if the chamber walls of a model study interfered in any way with this movement, but it is felt that it could lead to misleading results about the performance of the pipe.

RECOMMENDATIONS OF FURTHER RESEARCH

This study has just touched upon a small aspect of the peripheral movement of the soil around a flexible buried conduit. Further study in this area could bring to light some of the answers concerning pipe-soil interaction. Some of the projects which should be considered are:

1) A more sophisticated study on the same subject just presented, with perhaps pressure cells or photoelastic stress analysis being used to measure the affected soil zone and its shape.

2) The placement of rigid walls within the affected soil zone to determine the effect on the strength of the buried conduit. This would require knowing the loads imposed on the pipe.

3) Mathematical modeling of the pipe-soil system and establishment of design criteria for flexible buried conduits.

4) Full scale testing of pipe sections in actual buried conduit installations to verify the model study results.

REFERENCES CITED

1. Marston, A. and Anderson, A. O., "The Theory of Loads on Pipes in Ditches, and Tests of Cement and Clay Drain Tile and Sewer Pipe," Iowa Engineering Experiment Station Bulletin 31, 1913.

2. Spangler, M. G. and Handy, R. L., Soil Engineering, Intext Press, Inc., 3rd Ed., 1973.

3. Spangler, M. G., Soil Engineering, International Textbook Co., 1st Ed., 1951.

4. Tschebotarioff, G. B., Foundations, Retaining, and Earth Structures, McGraw-Hill, 2nd Ed., 1973.

5. Terzaghi, K., Theoretical Soil Mechanics, John Wiley and Sons, New York, 1943.

6. Marston, A., "The Theory of External Loads on Closed Conduits in the Light of the Latest Experiments," Iowa Engineering Experiment Station Bulletin 96, 1930.

7. Spangler, M. G., "The Structural Design of Flexible Pipe Culverts," Iowa Engineering Experiment Station Bulletin 153, 1941.

8. Schlick, W. J., "Loads on Pipe in Wide Ditches," Iowa Engineering Experiment Station Bulletin 108, 1932.

9. Marston, A., Chr., Stewart, J. T., Sec., Report of the Investigation on Drain Tile of Committee C-6 on Standard Tests and Specifications for Drain Tile, American Society for Testing Materials, Iowa Engineering Experiment Station Bulletin 36, 1914.

10. Marston, A., Schlick, W. J., and Clemmer, H. F., "The Supporting Strength of Sewer Pipe in Ditches and Methods of Testing Sewer Pipe in Laboratories to Determine Their Ordinary Supporting Strength," Iowa Engineering Experiment Station Bulletin 47, 1917.

11. Spangler, M. G., "The Supporting Strength of Rigid Pipe Culverts," Iowa Engineering Experiment Station Bulletin 112, 1933.

12. Schlick, W. J., "Supporting Strengths of Cast-Iron Pipe for Water and Gas Service," Iowa Engineering Experiment Station Bulletin 146, 1940.

13. Watkins, R. K. and Smith, A. B., "Deflection of Buried Pipes, Journal of AWWA, Vol. 65, September, 1973.

14. Bishop, R. R. and Moser, A. P., "Design of Flexible Conduits," unpublished study for Johns-Manville.

15. Howard, A. K. and Selander, C. E., "Laboratory Load Tests on Buried Reinforced Thermosetting, Thermoplastic and Steel Pipe," 28th Technical Conference, Reinforced Plastics/Composites Institute, 1973.

16. Watkins, R. K. and Spangler, M. G., "Some Characteristics of the Modulus of Passive Resistance of Soil: A Study in Similitude," HRB, Proceedings, Vol. 37, 1958.

17. Westergaard, H. M., "Computation of Stresses in Concrete Roads," HRB, Proceedings, Vol. 5, 1925.

18. Cummings, A. E., "The Stability of Foundation Piles Against Buckling Under Axial Load," HRB, Proceedings, Vol. 18, 1938.

19. Watkins, R. K., "Influence of Soil Characteristics on Deformation of Embedded Flexible Pipe Culverts," HRB, Bulletin 223, 1959.

20. Watkins, R. K. and Nielson, D. F., "Development and Use of the Modpares Device," Journal Pipeline Division, ASCE Proc., 1964.

21. Nielson, D. F., "Modulus of Soil Reaction as Determined from Triaxial Shear Test," HR Record, Vol. 185, 1967.

22. Nielson, D. F., Brandhausavee, C., and Yeb, Ko-Shing, "Determination of Modulus of Soil Reaction from Standard Soil Tests," HR Record, Vol. 284, 1969.

23. Watkins, R. K. and Smith, A. B., "Ring Deflection of Buried Pipe," Journal of AWWA, Vol. 59, 1967.

24. Watkins, R. K. and Moser, A. P., "New Design Method for Buried Corrugated Steel Pipe," ASCE Civil Engineering, June, 1972.

25. Moser, A. P., Watkins, R. K., and Bishop, R. R., "Structural Response of Buried PVC Pipe," Modern Plastics, November, 1973.

26. Williams, W. W., from a conversation at Kansas State University, 1975.

27. Spangler, M. G. and Donovan, J. C., "Application of the Modulus of Passive Resistance of Soil in the Design of Flexible Pipe Culverts," HRB, Proceedings, Vol. 36, 1957.

28. Nielson, D. F., "Soil Structure Arching Analysis of Buried Flexible Structures," HR Record, Vol. 185, 1967.

29. Howard, A. K., "Laboratory Load Tests on Buried Flexible Pipe Progress Report No. 5, FRP, PE, and PVC Pipe," Bureau of Reclamation Report No. 5, REC-ERC-73-16, July, 1973.

THE PERIPHERAL MOVEMENT OF SOIL
AROUND A BURIED FLEXIBLE PIPE

by

Wayne A. Duryee

B.S., Kansas State University, 1974

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

As the vertical earth load is imposed on a buried flexible conduit, the sides of the conduit move outward against the surrounding fill material. This movement causes the soil around the pipe to be displaced in some fashion.

The peripheral movement of the soil around a buried flexible pipe was determined by load testing sections of PVC pipe buried in sand and observing the movement visually. Relationships of the deflection of the pipe and the disturbance of the soil to the pipe-soil system were determined.

The results show that the visible movement zone of the soil increases as the deflection of the pipe increases. Also the visible movement zone is dependent upon the soil density and the size of the pipe. These relationships are important in understanding the pipe-soil interaction and with further research into this same area should aid in the design of flexible conduit systems.