THE UNIVERSITY OF ALBERTA

THE EFFECT OF SURFACE LOADS ON CORRUGATED PLASTIC DRAIN TUBES

by

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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE

DEPARTMENT OF AGRICULTURAL ENGINEERING
EDMONTON, ALBERTA

SPRING, 1974

ABSTRACT

Static surface loads were applied to two types of corrugated plastic drain tubes installed in two types of Edmonton soils (sandy loam and silt loam) at two probable depths of installation (95 cm and 125 cm). The tubes were installed in mole-channels (trenchless installation) and in trenches. The surface loads were increased in 200 to 250 lbs increments and corresponding deflections were recorded with a strain gage 'measuring mouse'. Effects on the load-bearing capacity of corrugated plastic drain tubes due to soil type, depths of installation and method of installation were evaluated statistically by an analysis of variance.

Conclusions from this study show that corrugated plastic tube drains installed at the 125 cm depth of installation can support significantly greater surface loads than when installed at the 95 cm depth of installation. The strength requirements for corrugated plastic tubes installed in mole-channels (trenchless installation) are significantly less than when installed in trenches. Both types of corrugated tube, that is, 'ADS', manufactured by the Big 'O' Drain Tile Co. and that manufactured by the Daymond Co., were equal in load carrying capacities. Surface loads of 101.1 and 98.2 lbs/in.^{2} for mole and 98.5 lbs/in.^{2} and 90.3 lbs/in.^{2} for trench-installed plastic tubes at the 95 cm depth in sandy loam and silt loam soils respectively could be applied to the soil surface where corrugated plastic tube drains are installed. The values of these loads for plastic tubes installed at the 125 cm depth in sandy loam and silt loam soil

respectively were 121.3 and 105.1 lbs/in.^{2} for mole and 114.7 and 87.6 lbs/in.^{2} for trench installation respectively. Corrugated plastic tube drains installed at the 95 cm depth in sandy loam and silt loam soil would fail at 105.0 and 99.0 lbs/in.^{2}, and 103.6 and 95.0 lbs/in.^{2} surface loads when installed in mole-channels and trenches respectively. At the 125 cm depth, the values were 127.5 and 105.7 lbs/in.^{2} and 117.1 and 98.1 lbs/in.^{2} for mole and trench installation in sandy loam and silt loam soil respectively.

ACKNOWLEDGEMENT

I would like to express my sincerest appreciation and gratitude to my supervisor Professor Egon Rapp for his invaluable advice, constructive criticism, and generosity of his time in research and preparation of this manuscript.

Cordial gratitude is extended to Dr. J.A. Toogood, Chairman, Department of Soil Science, for his sincere advice and suggestions.

High thanks and sincere appreciations go to Dr. R.T. Hardin, Professor, Department of Animal Science for his help in statistical analysis.

Appreciation is expressed for financial support provided by The National Research Council of Canada. Thanks to Big 'O' Drain Tile Co. Ltd., Hensall, Ontario and Daymond Ltd., Rexdale, Ontario who provided corrugated plastic tubes for this research.

Sincere thanks are due to Miss E., Symons, for her assistance in typing. To all those, who helped so willingly, I express my deep gratitude.

Finally to my parents for a great deal of understanding and encouragement.

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CHAPTER 1

INTRODUCTION

Subsurface drainage has been practised on irrigated lands for a long time. Irrigation has been the means of coping with the increased demands for food and fibre of an increasing world population. When the productive capacity of land decreases due to excessive application of water and accumulation of soluble salts, the need for subsurface drainage becomes evident. A number of references (39) indicate that the Romans were fully aware of the need to remove excess water from agricultural lands.

The economic feasibility of subsurface drainage has always been associated with a profitable agriculture. Open ditches, closed drains and relief wells have been the principal methods used for removing excess water from the plant root zone of the soil. Open ditches take productive land out of cultivation and require continual maintenance. Relief wells are used only in a few special areas. Closed drains have become the best practice for subsurface drainage. They do not take valuable land out of cultivation and do not interfere with farming operations. Also, the problems of weed control are reduced.

Mole drains and tile drains are used for subsurface drainage. Mole drains are formed by pulling a pointed cylinder through the soil. Drain tiles are hollow cylinders from one to four feet in length and are usually made of concrete or fired clay.

Mole drains have not proved successful for subsurface drainage due to rapid deterioration. Failures have been (6,8) mainly due to the soil falling through the slit left by the plow standard, erosion of soil from the sides of the mole channel, small rodents living in the mole channel loosening the soil and surface loads applied above the mole channel may close the drain.

The relatively high cost and the low speed of installation for clay or concrete tiles led scientists and engineers to improve the installation equipment. However, the introduction of plastics in the 1940's resulted in fundamental changes in drainage techniques. Plastic liners and smooth-walled plastic pipes were used in land drainage to improve the strength of mole drains. In the mid-1960's, research with corrugated plastic tubing was started. It offered obvious advantages over lined mole drains, and also clay and concrete tiles. Coiling the tubing into large rolls does not reduce its strength. Due to the lighter weight (a 250-ft coil weighs only 70 lbs) less labour is needed for loading and transportation. With the longer lengths of rolls (200 to 900 ft), few joints are needed and there are no problems of alignment as found with the installation of clay or concrete tiles. The tubing can be installed around relatively small-radius bends and high-speed equipment can be used resulting in a lower cost of installation.

With these advantages, corrugated plastic tubes have found wide use in industry, building construction, water supply, sewage disposal, septic tanks, and laying of electric cables but still the major use is for farm drainage. Hauck (18) predicted that by 1975, 70 percent of the market for corrugated plastic tubing would be for agricultural drainage. Of the 25 million feet of subsurface drains installed in

1972 on farms in Quebec, 18 million were plastic drains. About 55 million feet of subsurface drain are installed each year on Ontario farms of which 25 million are plastic drain tubes (4).

With the anticipated increase in the use of corrugated plastic tubing for land drainage, it is imperative that resear h be done on the durability of these tubes. Advances in technology have made farm operations increasingly mechanized. Heavy farm machinery and equipment is used on such farms. The use of modern installation equipment has resulted in an increase in the depth at which drainage tubes can be installed. Consequently, drain tubes are likely to be subjected to greater loads now than was common in the past. Loads on these drain tubes include those caused by the weight of the soil and by concentrated loads due to the passage of vehicles or equipment.

Unfortunately, not much research has been carried out to prove that these newly-introduced corrugated plastic tubes are a successful means for land drainage. Doubts expressed by farmers and contractors of the probability of collapsing of these light-weight, thin-walled corrugated plastic tubes in the field have not yet been answered satisfactorily. The values of concentrated surface loads that will render the drain tubes beyond serviceability have not been established specifically. The effect of soil texture and depth of installation on the load carrying capacity of these tubes have not been investigated. The strength requirements for a drain tube installed in a trench with a certain bottom groove angle and one installed in a mole-channel have not been compared.

A laboratory study simulating field conditions was conducted in an attempt to answer some of these questions. The specific objectives of this study were:

1. To establish the surface load values at which there will be no deflection in 4-in. corrugated plastic drain tubes installed at two depths (95 cm and 125 cm) in two types of soil (sandy loam and silt loam).
2. To determine the magnitude of surface loads that produces a 30 percent deflection, and the loads that cause total failure in drain tubes. Deflections up to 30 percent of the original diameter do not influence the flow capacity of the drain tube (26).
3. To compare the trench and mole method of installation on the basis of the load carrying capacity of the drain tubes.
4. To compare the durability of two types of 4-in. corrugated plastic drain tubes and to determine the deflection due to backfill loads only on drain tubes installed in trenches.

CHAPTER 2

REVIEW OF LITERATURE

The effect of loads on underground pipes due to earthfill and the passage of equipment and vehicles has been of great concern to agricultural, civil, and highway engineers during the past half century. Watkins (41) cites research carried out in Iowa where farmers had to move vehicles to the fields and the roads, due to insufficient gravel, became lagoons of mud. Dean Marston set to work with the slogan, "Get Iowa out of the mud". He suggested drainage to control the mud and in 1913, published the Marston theory of soil loads on drainage pipes.

2.1 Analysis of Loads on Subdrains.

Underground conduits derive their ability to support the earth above them from two sources; from the inherent strength of the pipe to resist external pressures, and the lateral pressure of the soil at the sides of the pipe. The latter produces stresses in the pipe ring in an opposite direction to those produced by vertical loads and thereby assist the pipe in supporting the vertical loads.

For purposes of load analysis (32) the underground conduits, depending upon their degree of rigidity, are classified into:

1. Rigid conduits such as those made of concrete or burned clay. The inherent strength of the pipe is the predominant source of supporting strength and it fails by rupture of the pipe walls.
2. Flexible conduits, such as corrugated-metal culverts, thin-walled steel pipes and plastic pipes. The inherent strength

is relatively less and a large part of its ability to support vertical loads is derived from the passive pressure induced as the sides move outward against the soil.

Based on construction or environmental conditions under which the field installations are made, pipe conduits are classified as ditch conduits or projecting conduits. Ditch conduits are those which are installed in a relatively narrow trench dug in undisturbed soil and then covered with earth backfill. Projecting conduit conditions occur when a drain is installed in shallow bedding with its top either projecting above the natural ground surface or at an elevation below the natural ground surface and then covered with an embankment. For tile drains, projecting conduit conditions occur when the trench is wider than 2 or 3 times the outside diameter of the tile.

2.2 Loads Due to Earthfill Materials

Marston analysed the load due to earthfill materials on underground conduits and published his theory under the heading "Marston theory of soil loads on drainage pipe". Spangler and Handy (32) describe this theory in detail. The theory is based upon a prism of soil which imposes a load W on the pipe and is given by the formula:

$$W_c = CwB^2$$

where W_c = load on conduit, in lbs per lin ft,

w = unit weight (wet density) of fill material, in lbs per cu ft,

B = width of the trench or conduit, in ft and

C = load coefficient for which values are listed by Marston.

In the case of ditch conduit conditions, B denoted by B_d, is the horizontal width of ditch at the top of the conduit whereas in the case of projecting conditions, denoted by B_c, this is the horizonta breadth (outside) of the conduit. Similarly, Marston presented curves for values of C and denoted them as C_d and C_c for ditch conduit and projecting conduit conditions respectively.

For a flexible pipe installed in a narrow trench with the fill thoroughly tamped on the sides to give the same degree of stiffness as the pipe itself, Marston gave the following equation;

$$W_c = C_d w B_c B_d$$

Spangler (34), a student of Dean Marston, found that these concepts were insufficient for the design of flexible pipes. His experiments showed that with the application of soil loads, the flexible pipe was flattened down into an approximately elliptical cross-section with a decrease in the vertical diameter and an increase in the horizontal diameter. As a result of lateral expansion, lateral support from the soil on the sides of pipe was developed so that the flexible pipe could sustain more loads with less deflection when buried than it could sustain in a three-edge bearing test. Consequently in 1941, Spangler arrived at the following equation for predicting the deflection of buried flexible pipe and published it under the name "Iowa Formula"..

$$\Delta x = D_l \frac{K W_c r^3}{EI + 0.061\, e r^4}$$

where Δx = horizontal deflection of the pipe, in inches,

D_l = deflection lag factor,

K = a constant depending upon the width of bedding of the pipe,

W_c = vertical load per unit length of pipe, in lbs per linear in.,

r = mean radius of the pipe, in inches,

E = modulus of elasticity of the pipe material, in lbs per sq in.,

I = moment of inertia per unit length of cross-section of the pipe wall, in inches^4 per in.,

e = modulus of passive resistance of the enveloping soil, in lbs per sq in. per in.

van Schilfgaarde, Frevert and Schlick (38) prepared load nomographs to facilitate the solution of Marston's equations. The three load nomographs prepared were for installations in thoroughly wet clay, saturated sand, and saturated top soil and can be used for computing loads for all rigid conduits as well as on conduits in narrow ditches. The lower value found from the nomographs is the one to be used in design.

Armco engineers, Schafer and Kelley (1), were among the first engineers who used and rewrote the Iowa formula to calculate the allowable fill height in terms of pipe diameter for maximum allowable ring deflection.

A similar relationship was found by Klimko and Kostikov (21) in Russia. The following formula was used to determine the deformation in a plastic pipe;

$$\Delta_o = \frac{\bar{\Delta}.q.r^3}{EI}$$

where Δ_o = pipe deformation in the plane of the vertical diameter at overload coefficient = 1,

g = equivalent of the vertical load,

r = average pipe radius,

E = modulus of elasticity,

I = moment of inertia of a longitudinal section of pipe wall,

$\bar{\Delta}$ = a coefficient equal to 0.016 at zero slope.

The equivalent of vertical load g was determined by the equation:

$$g = K_h W_{fill} . H . d$$

where H = earth cover over the pipe in meters,

d = pipe diameter in meters,

W = unit weight of backfill in kg/m^3,

K_h = a coefficient; for plastic pipe K_h equals 1.0.

Klimko and Kostikov (21) verified their results using the Marston equation and found that loads calculated by their formulae and by the Marston equation were in close agreement.

2.3 Surface Loads.

Surface loads due to moving farm vehicles and equipment, loads on railways, highways and other type of loads applied at the surface of the fill and transmitted to the underground drains were investigated by Marston. Spangler (32) mentions that experiments on both ditch and projecting conduits indicated that static concentrated surface loads, such as that of a truck wheel transmitted to an underground conduit, followed the Boussinesq solution for stress distribution in a semi-infinite elastic solid. The magnitude of impact load produced by moving wheel loads was also determined. From these facts, Marston arrived at the following formula for calculating the live loads on underground conduits;

$$W_t = \frac{1}{A} I_c C_t P$$

where W_t = average load per unit length of conduit, due to wheel load,

A = length of conduit section on which the load is computed,

I_c = impact factor,

C_t = load coefficient and

P = concentrated wheel load on the surface of the fill.

Marston found that the value of the impact factor I_c was equal to unity when the surface load was static and varied from 1.5 to 2.0 when the load was moving. Also, the value of the impact factor was independent of the depth of cover.

Investigations by Marston (24) and Spangler et al (33) showed that the value of the coefficient C_t depended on the length and width of the conduit section and on the depth of cover over the conduit. Graphs were prepared to find the value of C_t for different diameter of conduits installed at different depths.

2.4 Plastic Tubings for Subsurface Drainage.

Research on improving the stability of mole drains by lining the channel with plastic started when polyethelene was introduced in the 1940's. Prior to this, sheet metal was tried as a liner for the mole channels. A machine developed by Sack (28) consisted of a mole plow and a tube-forming mechanism which installed the sheet metal in the mole drains in a continuous oval tube.

Ede (13) used dry-mix concrete as a mole liner. Concrete was placed around a core using a vibrating vertical chute. As the core moved forward with the machine, a circular channel left in the concrete mix formed a tube. This method was satisfactory only for 1 1/2 in. and 2 in. core sizes. The U.S. Corps of Engineers (36) investigated several methods of stabilizing mole channels and concluded that placing the perforated plastic tubing by cable-laying machines appeared to be the most promising and economical method.

Schwab (29) conducted experiments at Iowa State College to test the feasibility of polyethylene tubing. Perforated polyethylene tubes, 1 to 4 in. in diameter and in 20 ft lengths, were attached to a mole plough and pulled into the mole channels from the outlet ends at a depth of 30 in. From results of the five-year study, he concluded that the deformations for 1 1/2 in. and 2 in. diameter tubes with a 0.040 in. and 0.050 in. wall thickness respectively, were less than 20 percent of the original diameters. The greatest amount of deformation occurred during the first two years after installation. The stability of the perforated polyethylene tubes in mole drains decreased as the diameter of the tubes increased and as the wall thickness decreased and hence drain tubes 2 in. or less in diameter were the most practicable considering stability, capacity, and cost. A minimum slope of 0.5 percent was suggested for these tubes.

Busch (6) conducted experiments at Cornell University and succeeded in forming a plastic arch to prevent the fall of soil into the mole channel. Rigid vinyl, 0.015 in. thick by 6 in. wide, was formed into a tight "U" and fed down through a chute on the channel lining machine into the mole channel. The arch, approximately 2 in. wide by 2 1/2 in. high, was formed as the material left the machine.

Fouss and Donnan (16) tried five types of plastic liners made from 15-mil polyvinylchloride semi-rigid plastic sheet. After nine months of field testing, the zipper-type mole liner had maintained its cross-sectional size and shape better than the overlap types.

Field and laboratory loading tests on plastic-lined mole drains were conducted by Manley (23). The laboratory experimentation consisted of applying loads to the drains installed at depths ranging from 10 to 22 inches in three different soils: fine sandy loam, silt loam, and silty clay loam. Loads were applied in 250 lb increments using a steel plate attached to the ram of a hydraulic jack. He found that the "bridging" phenomenon of soil tends to transfer internal soil pressure caused by surface loads to the soil on the side of the mole channel. The correlation between the bridging action of soil and the bulk density showed that the tendency of a soil to "bridge" increases as the bulk density of the soil increases. Manley also found that settlement of the loading plate preceded failure of the drain. A highly significant correlation coefficient was obtained between the maximum load that could be applied to the ground surface without drain failure and the bulk density of soil. He concluded that lined or unlined mole channels were collapsed by surface loading only after considerable depression on the soil surface.

Vaigneur (37) investigated the loading characteristics of a 3 in. diameter plastic liner by the three-point bearing test and hydraulic jacket test in sandy soil. The mole liner used was made from vinyl plastic 0.015 in. thick and 11 in. wide. The conclusion was that mole drains lined with plastic could be expected to fail when the surface pressures exceeded 40 psi for drains 30 in. deep, 30 psi for drains 24 in. deep, 20 psi for drains 18 in. deep, and 10 psi for drains 12 in. deep. The surface displacement varied from 1.23 in. with a load of 13.78 psi to 8.04 with a load of 44.58 psi. He reported that plastic-lined mole drains would be distorted beyond serviceability (vertical deflection more than 33%) when exposed to a stress of about 0.50 psi and suggested that stress in the soil near the mole drain should not exceed 0.25 psi.

Willardson and others (44) investigated seven types of conduits for load carrying capacity. Among these types were 3 in. diameter polyvinyl chloride plastic (PVC) mole liner, 4 in. diameter semi-rigid polyethylene tubing with 0.078 in. walls, and 3 in. diameter steel-reinforced fiberglass wrapped plastic. Test sections of conduits, 15 1/2 in. long, were placed in the test cell which was a rubber cylinder surrounded by constraining concentric rings. Vertical loads were applied to the test cell in the Universal Testing machine. The 3 in. diameter PVC mole drain liner failed by longitudinal buckling after the reduction in diameter by shearing of the zippers. Failure was reached at 8,000 lbs. The 3 in. diameter steel-reinforced fiberglass-wrapped plastic took an oval shape with the 80,000-lb limit of the test cell without reaching failure.

Myers, Rektorik and Wolfe (25) conducted laboratory experiments with 4 in. ID polyethylene drainage pipe. The test sections of pipe were installed in a steel tank, 2 ft by 5 ft based and 10 ft high, filled with backfill soil such that the depth of cover was 8 ft. Irrigation water was applied from the top of the steel tank and the deflections developed were measured through strain gages bonded and waterproofed on the pipe. The hydraulically loaded test equipment was also used for loading tests. The investigations showed that a 5 in. diameter cradle excavated in undisturbed soil to support 160 degree sections of the pipe should provide support for loadings up to 300 lbs per lineal ft. In order to minimize the load on plastic pipes installed in trenches, 10 in. width of trench was recommended.

Rektorik and Myers (27) also conducted experiments with the same pipe and concluded that the pipe had sufficient strength to withstand soil overburden loads at the 8 ft depth.

In the early 1960's, corrugated plastic pipe was introduced. These tubes provided greater strength, better longitudinal flexibility for ease of handling, lighter weight and much lower cost than comparable smooth-walled plastic pipe. In order to have the same structural strength in a smooth-walled plastic pipe of given diameter as in a corrugated wall tube, the plastic material in the smooth-walled pipe needs to be much thicker and hence the pipe will be much heavier and costly. Fouss (15) in 1968 reported that installation of corrugated plastic tubes with the drain tube plow was promising as a means for more effective and more economic field drainage. The plow eliminated the need for ditching and backfilling, and installation with the laser-beam automatic grade control was efficient and quick.

Klimko and Kostikov (21) conducted experiments on smooth-walled and corrugated plastic drain tubes. The laboratory experiments showed that the drainage pipes made from polyvinyl chloride were extremely strong when subjected to a load of 150 kg per lineal meter of pipe and the corresponding deflection did not exceed 20% of the pipe diameter. The corrugated pipe, made of the same material as that for the smooth-walled pipe was exceptionally strong in crushing strength. The recommendation was that the wall thickness of corrugated PVC drainage pipes could be reduced two or three times in comparison to smooth pipe of corresponding diameters. From published data and experimental results from tests conducted by the Byelorussian Experimental Institute of Land Reclamation and Water Resources in Russia, it was reported that deformation of plastic pipe occurred only during the back-filling of trenches and that loads due to moving machinery had little effect on deformation of the pipe.

Drablos and Schwab (12) made field and laboratory investigations on 4 in. corrugated plastic drain tubes which had been installed under different field conditions for at least one year. Field investigations were made at 42 sites where 4 in. corrugated plastic tubes had been installed. The factors investigated at each site were soil type, period of time that the tubing had been installed, method of blinding, method of back-filling, stretch of tubing, size of trench-bottom groove, tube deflection and strength of tubing. Insignificant differences were found in the amount of deflection that occurred for different soil groupings, the reason given being the small number of sites in each soil grouping. The length of time that the tubing had been installed had some effect on deflection. It was suggested that the deflection would be less after the initial 2-year or perhaps 4-year period of installation. Less deflection was found where care had been taken in properly placing the blinding and backfill soil. Investigations relating to stretch produced in tubing while placing showed that the average amount of stretch was 5.3% with a range between 0 and 11.4%.

Negi and Broughton (26) made field loading tests and laboratory investigations on corrugated plastic tubings. The drains were installed in sandy loam soil at the Macdonald College Farm in 1968 and field loading tests were performed in 1969. The field loading tests consisted of passing loaded wagons and trucks over the plastic drain tubes and some clay tiles installed with a trencher or trenchless plow at 2 and 3 ft depths. The loads and the number of passes along three vehicle routes over each of the thirty subdrains were noted. The deformation in tubes up to 20 and 30% of the pipe diameter was measured using wooden plugs. Some of the plastic drain tubes installed at a 2 ft depth were deformed beyond serviceability when the rear axle load exceeded 20 tons. The plastic drain tubes were crushed by repeated passes of surface loads only after considerable depression on the soil surface and the formation of ruts.

The laboratory tests on samples taken from the field installations showed that the load bearing capacity of the plastic tubes continued to increase to deflections of more than 40% of the original diameter (26). The investigations showed that field deflections up to 30% of the original inside diameter of the plastic tubes could be allowed for carrying the earth and live loads. No difficulty was reported with plastic drain tubes installed at 2.5 ft depths or more with normal backfilling, including 6 in. deep blinding with loose soil. For the drain tubes passing under the laneways which carry heavy loads, a depth of installation of more than 3 ft was recommended or the portion of drain tube under the laneway should have a steel pipe section.

2.5 Deflection Measuring Devices.

Deflection of plastic tubes has been measured by several research workers using various devices. Schwab (29) measured the inside diameter of plastic tubings using various sized eyebolts attached to 1/2 in. steel tubing. An electrical resistance inside caliper was also used to measure the inside diameter for 3 and 4 in. plastic tubes.

Busch (7) developed a "mechanical mouse" which was pulled through the drain tube by a small wire. The instrument had six spring fingers equally spaced on an aluminum chassis such that when the unit was pulled through the tube the fingers acted as feelers when moved along the inside wall of the tube. Strain gages attached to the fingers permitted the measurement of changes in electrical resistance as caused by variations in the shape of the tube walls. This gave a complete measurement of tube cross-section. Manley (23) used the same "measuring mouse" developed by Busch to measure the deformations in the plastic lined mole drains. The instrument has been reproduced and used later on with few modifications. Vaigneur (37) installed mercury switches with eight contacts, one for each finger to properly orientate the fingers.

A self-propelled drain-line camera system developed by Shull (30) gives a picture of the inside conditions of the drain. The system was limited only to pipe diameters of 4 ins. or more and water in the pipe to a maximum depth of 1/2 in.

Strain gages secured and waterproofed to the exterior of plastic tubes were used by Myers et al (25) to obtain the strains and deflections produced as a result of loading the tubes.

Negi and Broughton (26) measured 20 and 30% deflections produced in corrugated plastic tubes as a result of field loading, using hard, wooden, oblong 'Go-NoGo' plugs attached to 50 ft long, 1/2 in. semi-rigid plastic pipes which were inserted in the drain tubes from access trenches.

2.6. Bedding Angle.

Load bearing capacity of flexible conduit is increased by the bottom groove angle. The exact size of this angle in the trench bottom has been a matter of conflict.

Vaigneur (37) investigated three groove sizes, 120 degrees, 180 degrees and 240 degrees. The conclusion was that the 120 degree bottom support was the most representative of field conditions.

Myers and others (25) recommended that, to provide lateral support to 4 in. diameter semi-rigid pipe, a 4 1/2 or 5 in. diameter cradle should be provided to support 160 degrees of the pipe bottom circumference.

Davis and Gieselman (11) have mentioned that groove angle has recently been changed from 180 degrees to 60 degrees. A similar expression is mentioned by Galeshouse (17). Accordingly, the groove specification has been reduced to 60 degrees. The depth of a 60-degree groover for 4 in. tubing is 5/16 in., for 5 in. tubing 3/8 in., and for 6 in. tubing 7/16 in. He mentioned that the groove was so small in cross-section that a change was not necessary for 4,5 or 6 in. tubes in the same trench as the amount of deformation to conform to the groove was only about 1/16 to 5/32 in.

Drablos and Schwab (12) measured groove angles for corrugated plastic tubes installed in the state of Ohio. The groove angle ranged from 0 to 157 degrees with an average of 90.4 degrees. The average deflection was greater for those installations with a measured groove less than 60 degrees as compared with those installations with a groove angle greater than 60 degrees.

Negi and Broughton (26) made field investigations with a 90 degree 'V' groove and a 150 degree circular arc cradle in the trench bottom. The conclusion was that a 150 degree circular arc cradle was not significantly better than a 90 degree 'V' groove.

CHAPTER 3

MATERIALS AND PROCEDURE

3.1 Selection and Description of Soils.

The major soil types in the Edmonton area vary from sandy loam to clay loam (2,3). To investigate whether soil type is related to the load bearing capacity of corrugated plastic drains installed at different depths by different installation methods, two soil types having different textures and representative of the Edmonton soils were used in the experiment. Following the soil survey classification of the Edmonton and Peace Hills sheets, samples were collected from locations at Devon, Ellerslie, Millet, and Ponoka. Samples were taken from each 35 cm depth starting from the top 25 cm cultivated horizon ('A' horizon) to the 140 cm subsoil ('C' horizon). Mechanical analysis was done by the hydrometer method on each of the soil samples on the basis of the United States Department of Agriculture soil classification (5). Soils from Ponoka and the Ellerslie Agricultural Engineering Farm were found to be sandy loam and silt loam texture respectively. These soils were then selected for study.

The particle size distribution curves for the soil from the 'A' horizon of both soils are shown in figure 1. Analysis of the subsoil from 'C' horizon is reported in Appendices 1 and 2. The physical properties such as texture and Atterberg limits (22) were found and are reported in tables 1 and 2. The general description (2,3) of both soils is as follows:

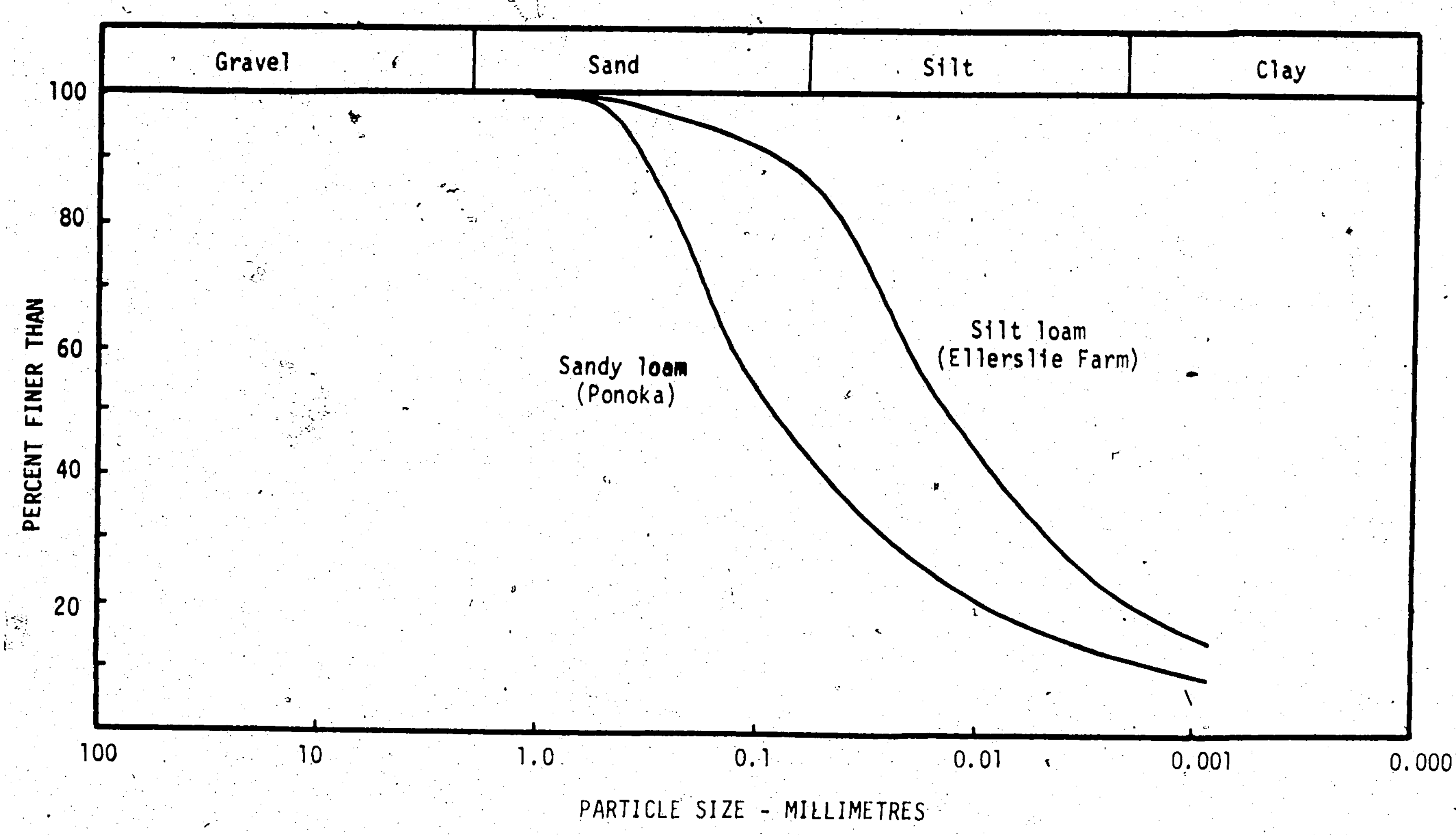


Figure 1. Particle size distribution curves for top soils from Ponoka and Ellerslie Farm locations.

TABLE 1: MECHANICAL ANALYSIS OF SOIL, PONOKA LOCATION.

Depth, cm	Sand %	Silt %	Clay %	Texture
0 - 25	60	29	11	Sandy loam
25 - 70	58	23	19	Sandy loam
70 - 105	69	16	15	Sandy loam
105 - 140	52	29	19	Sandy loam to loam

TABLE 2: MECHANICAL ANALYSIS AND OTHER PHYSICAL PROPERTIES OF SOIL, ELLERSLIE LOCATION.

Depth, cm	Sand %	Silt %	Clay %	Liquid Limit	Plastic Limit	Plasticity Index	Flow Index	Toughness Index	Texture
0-25	16	64	20	49	33	16	15.5	1.0	Silt loam
25-70	14	54	32	47	30	17	12.3	1.4	Silt loam
70-105	26	35	39	42	22	20	14.3	1.4	Clay loam
105-140	27	36	37	39	20	19	12.3	1.5	Clay loam

	Sandy Loam	Silt Loam
Location	Ponoka SW 20-42-25-W4	Agricultural Engineering Farm Ellerslie, $S\frac{1}{2}$ -24 -51-25-W4
Soil Series	Peace Hills Sandy Loam (Ph.SL)	Ponoka Silt Loam (Pk. SiL)
Classification	Orthic Black Chernozem	Eluviated Black Chernozem
Parent Material	Alluvial aeolian material	Alluvial lacustrine, medium textured.

Both soils were also classified according to the Unified Engineering Soil Classification. The sandy loam soil from the Ponoka location falls within the SM to SC soil groups and the silt loam soil from the Ellerslie Farm location falls within the ML to OL soil groups.

3.2 Description of the Corrugated Plastic Tubes.

The two types of corrugated plastic drainage tubes investigated were the "ADS" manufactured by the Big "O" Drain Tile Co. and that manufactured by the Daymond Co. These are shown in figure 2. Both drain tubes are 4 in. inside diameter. The major difference lies in the number of water entrance slots and the pattern of corrugations.

The shape of the corrugations are flat and square in the ADS tube whereas they are round and spiralled in the Daymond tube. The ADS tube has three rows of water entrance slots spaced equally around the tube circumference providing a total water inlet area of 4.752 sq. ins. per linear foot. The Daymond tube has eight rows of water inlet slots which are bigger than in the ADS tube. The total inlet area was not specified for the Daymond tube. The average depth of corrugation is 0.225 inches in the ADS tube and 0.205 inches in the Daymond tube.

3.3 Experimental Design.

The factorial design (19) was used for the experiment. The factors and the levels chosen for the study are as follows:

Factor	Code	Levels
Soil type	S	Sandy loam, silt loam
Corrugated plastic tubing	P	Big "O", Daymond
Depths of installation	D	95 cm, 125 cm
Method of installation	I	Mole, Trench

There were two replicates for each treatment and all 32 treatments were randomized. The model for this factorial design is:

$$Y_{ijklm} = \mu + I_i + D_j + ID_{ij} + P_k + IP_{ik} + DP_{jk} + IDP_{ijk} + S_l + IS_{il} + DS_{jl} + IDS_{ijk} + PS_{kl} + IPS_{ikl} + DPS_{jkl} + IDPS_{ijkl} + e_{m(ijkl)}$$

where Y_{ijklm} represents the observed loads,

μ represents population mean,

I_i represents installations: $i = 1,2$

D_j represents depths: $j = 1,2$

P_k represents plastic tubes: $k = 1,2$

S_l represents soils: $l = 1,2$

$e_m(ijkl)$ represents random error (replication): $m = 1,2$

All sources of variation except replications were considered fixed.

3.4 Experimental Procedures.

Two soil test boxes 1.2 m x 0.75 m and 1.8 m high made from 3/4 in. plywood were used (see figure 3). Two holes, 42 cm above the bottom of the test boxes, were made on the 0.75 m side in order to

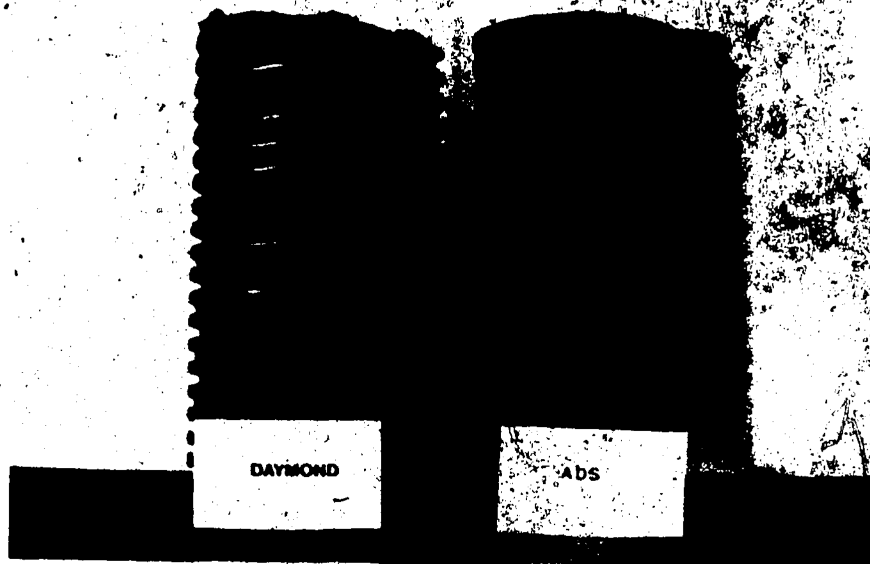


Figure 2. Corrugated plastic tubes used in the experiment.

Figure 3. Soil test box and experimental set-up.

place the test sections of 4 in. corrugated plastic tubing. The test boxes were set on concrete block supports.

The soil was placed in the test boxes in 10 cm layers and packed with a standard 10 lb tamping rod. The tamping was done on a 6 in. x 6 in. steel plate placed and moved over each soil layer. The plastic tubings were installed in mole channels as well as in trenches.

The mole channels were made by laying aluminum pipe having an outside diameter slightly larger than the outside diameter of the plastic tubes. After filling the soil box to a depth about half the diameter of the holes on the sides of the boxes, the aluminum pipe was pushed horizontally through the soil from one hole to the other. Then the soil was packed carefully around and above the aluminum pipe until the desired depth of installation was reached. After filling the box with soil to the 95 cm or 125 cm depth, the aluminum pipe was pulled out smoothly. Then the corrugated plastic tube was pulled into the mole channel thus formed.

Plastic tubes were installed in 10 in. wide trenches which were made by holding two plywood sheets 8 1/2 ins. apart in a soil box. After the soil was packed along the sides, the plywood sheets were removed thus leaving a trench. Investigations by Negi and Broughton (26) showed that a bedding angle of 140 to 160 degrees gave a maximum load bearing capacity to the plastic tube. Rektorik et al (27) found that the bedding angle in the range of 120 to 160 degrees was the most desired bedding angle for plastic tubes. From the above investigations, a decision was made to use a 150 degree bedding angle. The bedding angle was provided by placing cut sections of 5 in. diameter aluminum pipe, shown in figure 4. After filling the soil box to the centre of the side-holes, the aluminum cradle was pushed horizontally through the side hole. Great care was taken to place this aluminum cradle in a horizontal and central position in the soil box. The plastic tube then was laid on this cradle and the soil was carefully packed around it.

While packing the soil in the soil boxes, the bulk density of each layer was measured using a hand-held Gamma Ray soil bulk density meter shown in figure 5. The two probes, 2.17 cm in diameter, are 14 cm apart (14). The probe, marked in 5 cm depths, contains one millicurie of caesium-137. The time taken for the gamma rays to reach the second probe is recorded by the scaler. The scaler is operated by a 6-volt rechargeable storage battery and the counts are set by a preset count selector. The probes were lowered in the holes made by a soil core sampler and then made to the exact diameter by pushing the dummy soil probes into the holes. The probes were lowered and held at two depths in each 10 cm layer of packed soil. The time in seconds was recorded by a watch on the scaler. The preset count was kept at 5×10^3 counts per minute throughout the experiment. Moisture content samples from each layer were taken with a soil core sampler while making holes for the soil probes. The moisture content was determined by the gravimetric method.

The soil bulk density was measured at two sites in the soil box for the mole installation but for the trench installation, measurements were made in the trench and also outside the trench. From

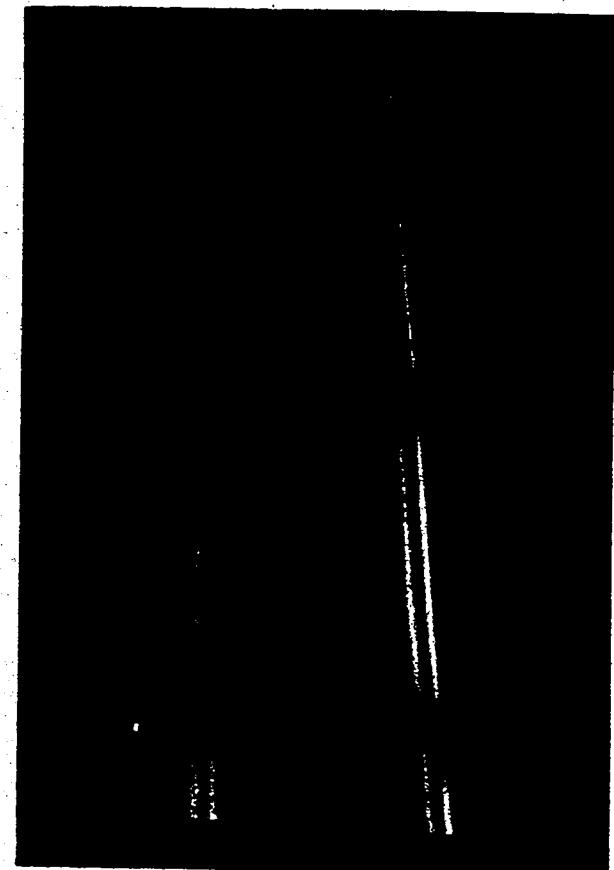


Figure 4. Aluminum sections used for bedding angle.

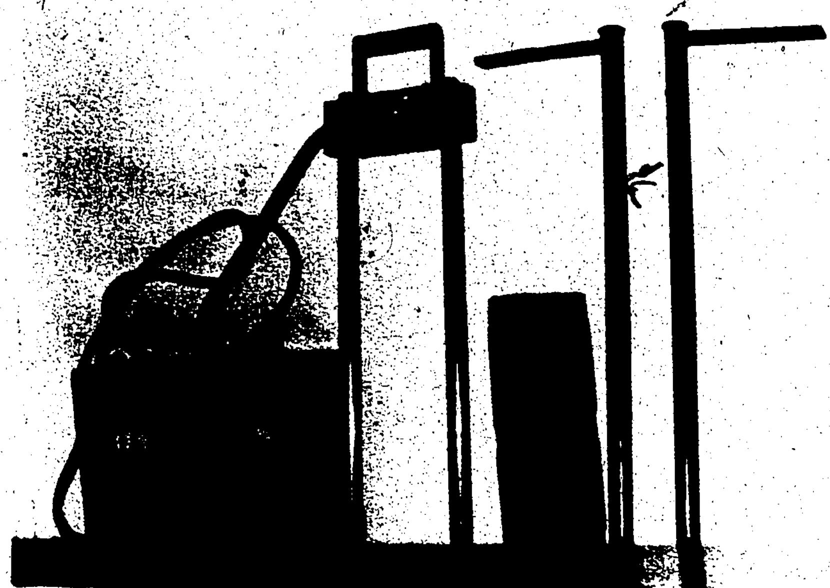


Figure 5. Hand-held Gamma Ray soil bulk density meter.

the time recorded and the moisture contents, the dry soil bulk density was determined from the following equation (14).

$$DD = 1.3 \pm 5.46 \log_{10} T_t/T_s - W$$

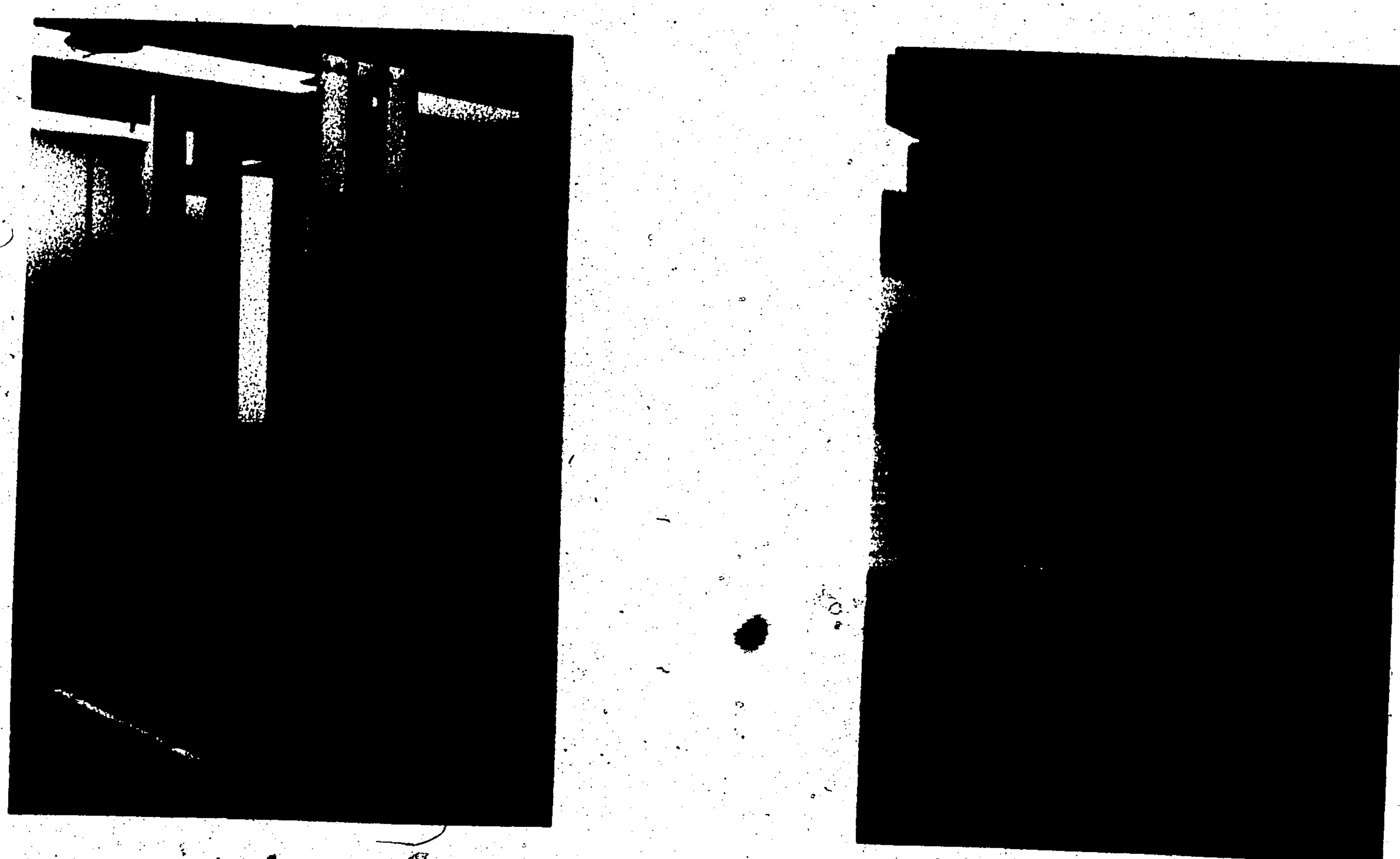
where DD = dry bulk density, g/cm^3

T_t/Ts = the ratio of the time to collect 5,000 counts in the test position to the time to collect the same number of counts for a standard exposure,

W = the moisture content in percent on an oven dry basis.

A Cassette computer programme No. 1, Block 3, verify No. 2422 for this equation, available in the Department of Agricultural Engineering, University of Alberta, was used to calculate the dry bulk density. Computations were made using this programme on a Wang Programmable calculator Model 600 connected to a Teletype. The values of T_t, T_s and W were inserted on teletype to give dry bulk density results.

For the application of loads from the surface of the soil in the test box a steel frame, shown in figure 6, was designed. All loads applied to the soil surface were taken by the steel frame and no load was transmitted to the floor except the dead load of the soil test box and the steel frame itself. It was anticipated that loads of up to 20,000 lbs would be reached during the experiment so that the steel frame was designed (9) for a 20,000 lb ultimate load with a factor of safety of 1.5. The frame essentially consists of a top WF-beam, side channel beams, bottom side I-beams and free bottom I-beams. The sizes and cross-sections of the members are given in Appendix 3.



(a) (b)

Figure 6: (a) Steel frame, (b) Top joint detail.

The bottom I-beams were free and could be removed easily. During the experiment, the two soil test boxes remained stationary on supports while the steel frame was moved for load application to the soil surface of each soil test box.

The loads were applied to the soil surface in each soil test box through an 8 in. x 10 in. loading plate made of 3/4 in. steel plate. The size of this loading plate was selected by taking the mean of the tire contact areas of a variety of farm machinery. The loading plate was shaped to fit the base of the hydraulic jack when setting on the soil surface with the ram facing the top I-beam of the frame as shown in figure 3.

A 50-ton hand-operated hydraulic jack was used in the experiment. The jack was calibrated for actual loads and indicated loads on the gauge. The calibration was done at the Civil Engineering Department Laboratories, University of Alberta. The calibration curve is shown in figure 7.

The load was applied in 200 to 250 lb increments and was held constant for about 2 minutes to record the corresponding deflections. A strain gage mouse, similar to the "mechanical mouse" previously mentioned, was constructed to record the deflection. There were two spring finger units connected as shown in figure 8. The fingers for the alignment set were 27 cm long and for the rear set 34 cm long. They were made of 1.2 cm wide beryllium-bronze material with a thickness of 0.020 in. The front set helped to align the instrument whereas the rear set measured the deflections developed in the plastic tube. Each finger on the rear set had two strain gages;

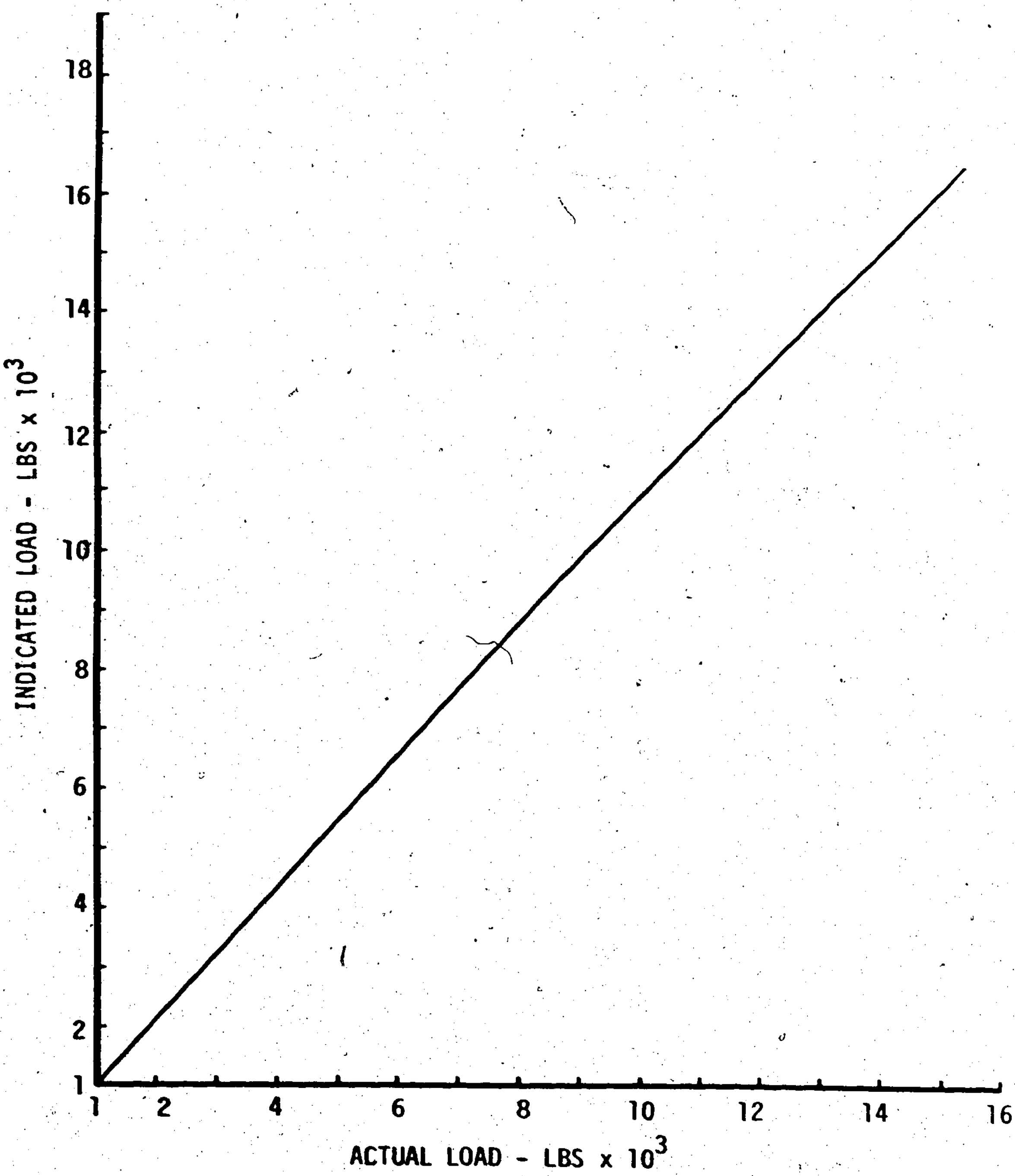


Figure 7. Calibration curve for the 50-ton hydraulic jack.

active and compensating of the type EA-06-250BG-120 (figure 10). The two opposite fingers were considered to make one channel and hence channel No. 1 measured the vertical deflection and channel No. 2 measured the horizontal deflection. The signal from each channel was amplified through a DC amplifier Accudata 104 and recorded on direct print paper spec. 98, in ultra-violet recorder type SE 2005 (see figure 9). Specifications are given in Appendix 3.

The strain gage mouse was calibrated with a constant gain on the amplifier for both channels. The calibration was done by holding the two fingers in a channel made of aluminum pieces, shown in figure 9, which had clearances of 2,4,5,6,7,8,10,12 cm including the thickness of the strain gage mouse fingers. The corresponding deflections were recorded on direct print paper. The calibration curves are shown in figures 11 and 12.

For each load increment applied, the deflection was recorded on direct print paper by pulling the strain gage mouse attached to the 5 ft long copper rod, through the plastic tube. Care was taken to keep channel No. 1 in a vertical position by keeping the handle attached to the pulling rod facing vertically downward.

The stroke of the hydraulic jack was limited to 7 ins. After reaching that limit, the pressure in the hydraulic jack was released and wedges made from steel plate were placed in between the hydraulic jack base and loading plate to fill the gap. After the pipe collapsed, the hydraulic jack was removed and the total settlement of the plate was measured.

After completing a load test, the soil box was placed on supports

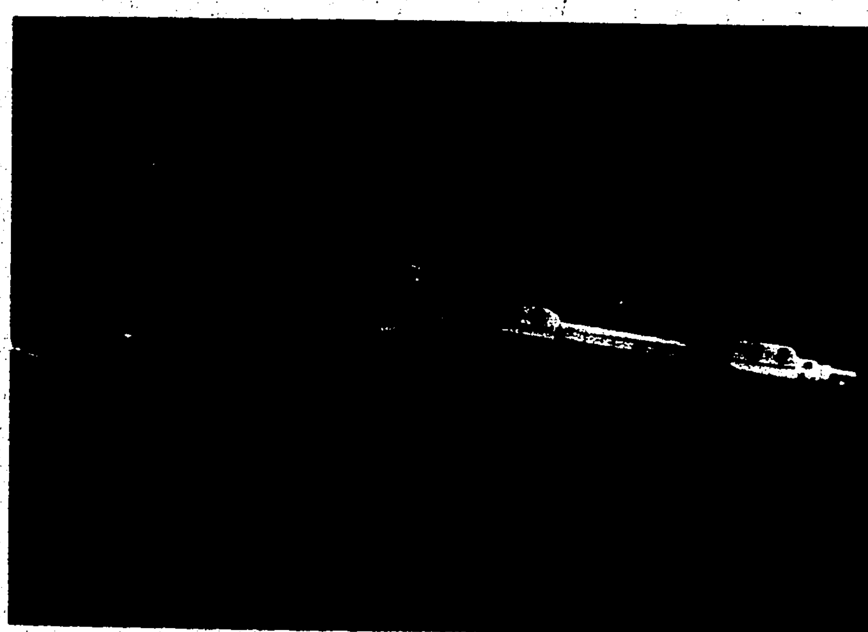


Figure 8. Strain-gage measuring mouse.

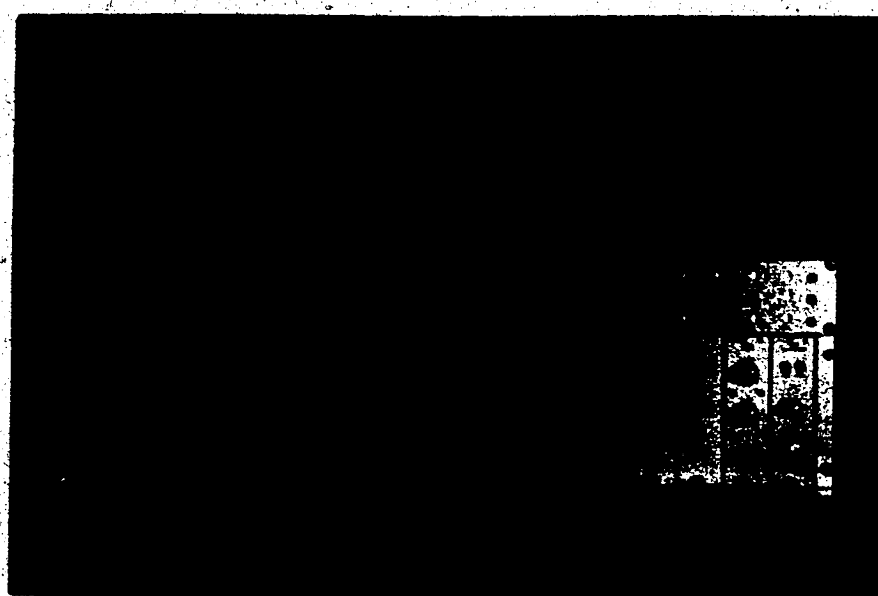
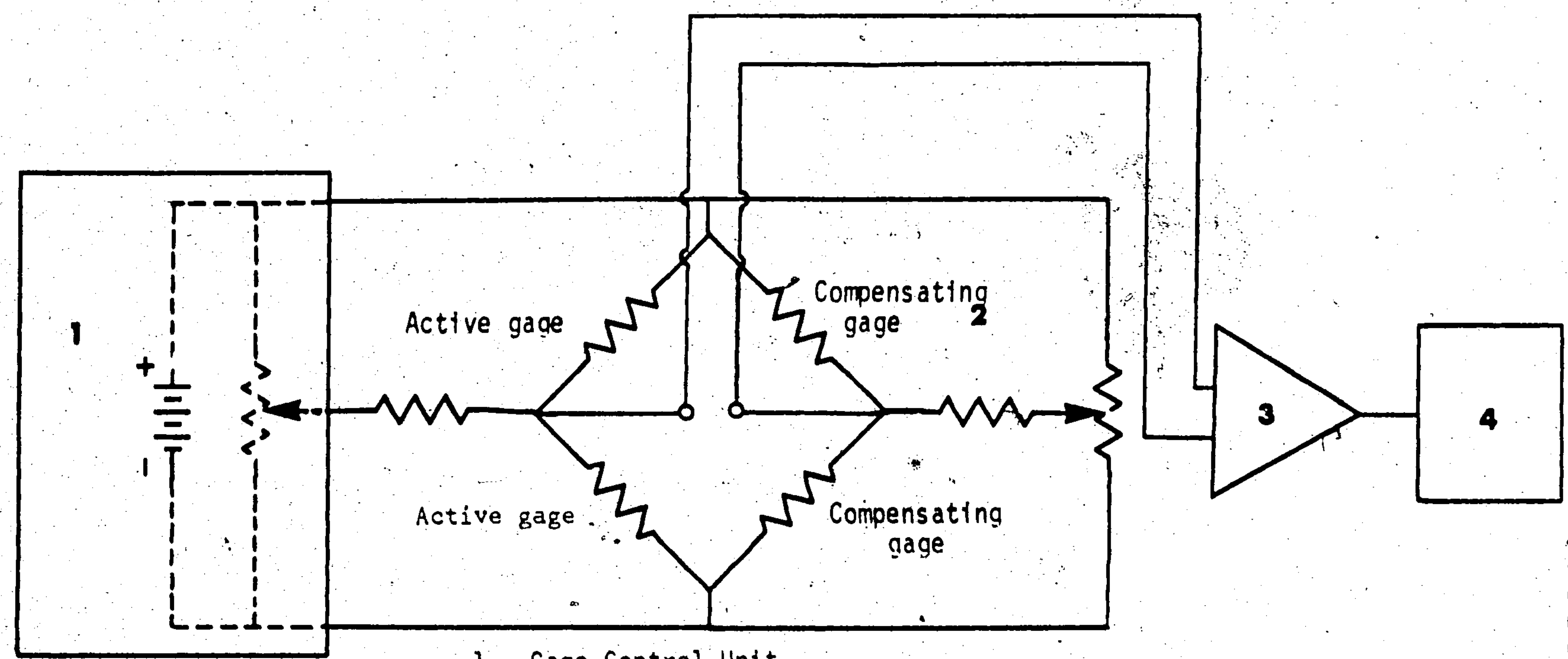


Figure 9. Aluminum pieces, D.C. amplifier and U.V. Recorder.



1. Gage Control Unit.
2. Additional Calibrator
3. D.C. Amplifier.
4. U.V. Recorder

Figure 10. Circuit diagrams and general set-up for deflection measuring equipment.

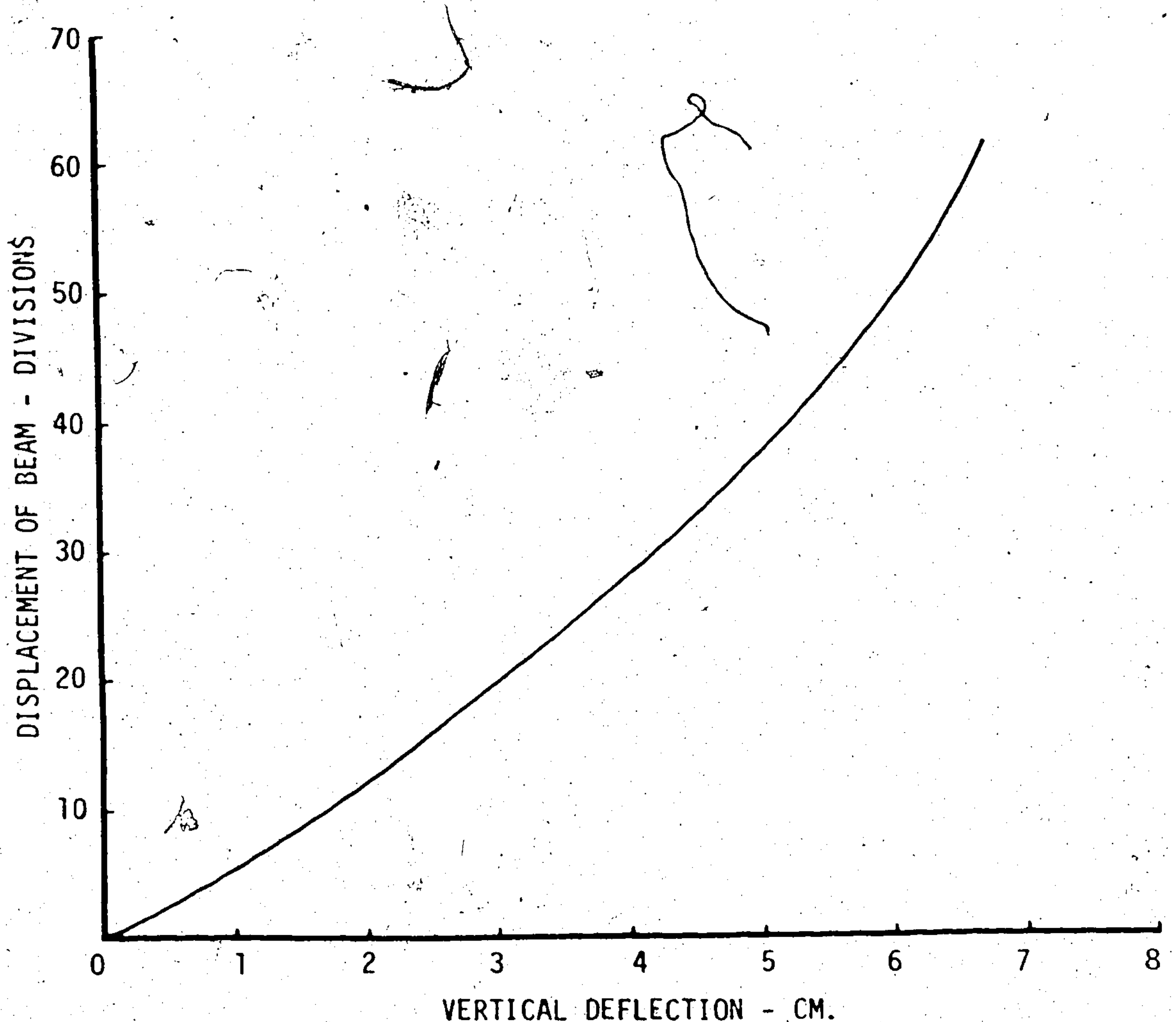


Figure 11. Calibration curve for Channel No. 1 of strain gage measuring mouse.

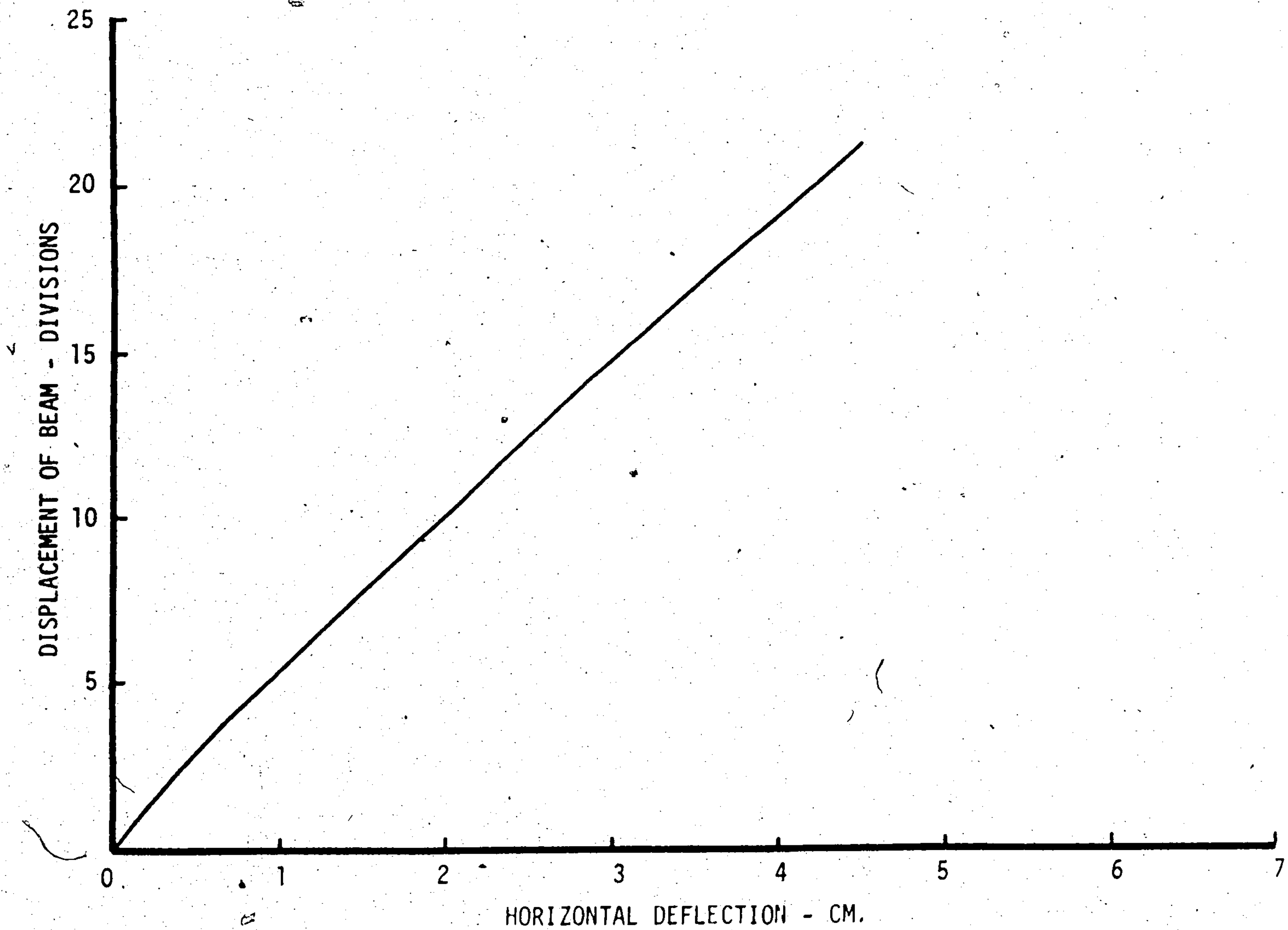


Figure 12. Calibration curve for Channel No. 2 of strain gage measuring mouse.

using the 1 1/2 ton hydraulic jack. Then the free bottom I-beams were taken out and the steel frame was moved to the second soil test box ready for a load test. After moving the steel frame to the second soil test box, the soil was taken out of this box and covered with a polyethylene sheet in order to reduce the loss of moisture. While emptying the soil box, the bulk density of the compressed soil underneath the loading plate and 10 cm above the top of the plastic tube was measured with the gamma ray soil bulk density meter previously mentioned.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Soil Bulk Density.

The average soil* bulk density gradient maintained in the experiment and the field soil bulk density gradient of soils in the Edmonton area as reported by Verma (40), are shown in figure 13. An average soil bulk density of 1.4 was maintained in both soils used in the experiment.

Irwin (20) found that the dry bulk density was increased about 5% over a thickness of 8 to 10 in. below the drainage tubings and at the sides after the tubings had been laid with the Badger drain plow. This was achieved in the experiment by giving more tampings to the soil underneath and around the aluminum pipe which was then pulled out to leave a mole-channel. The bulk density of the soil measured around the mole-channel in the experiment was 3 to 6% greater than the surrounding soil.

Although a comparison between the bulk densities of earthfill in trenches and that of the surrounding undisturbed soil has never been made, expectations are that after 2 or 3 years of drain installation the bulk density of earthfill will still be less than in an undisturbed condition. To simulate this condition, the bulk density of the earthfill in the experiment was kept at 7 to 12% less than the surrounding soil.

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* The term soil bulk density refers to the dry soil bulk density.

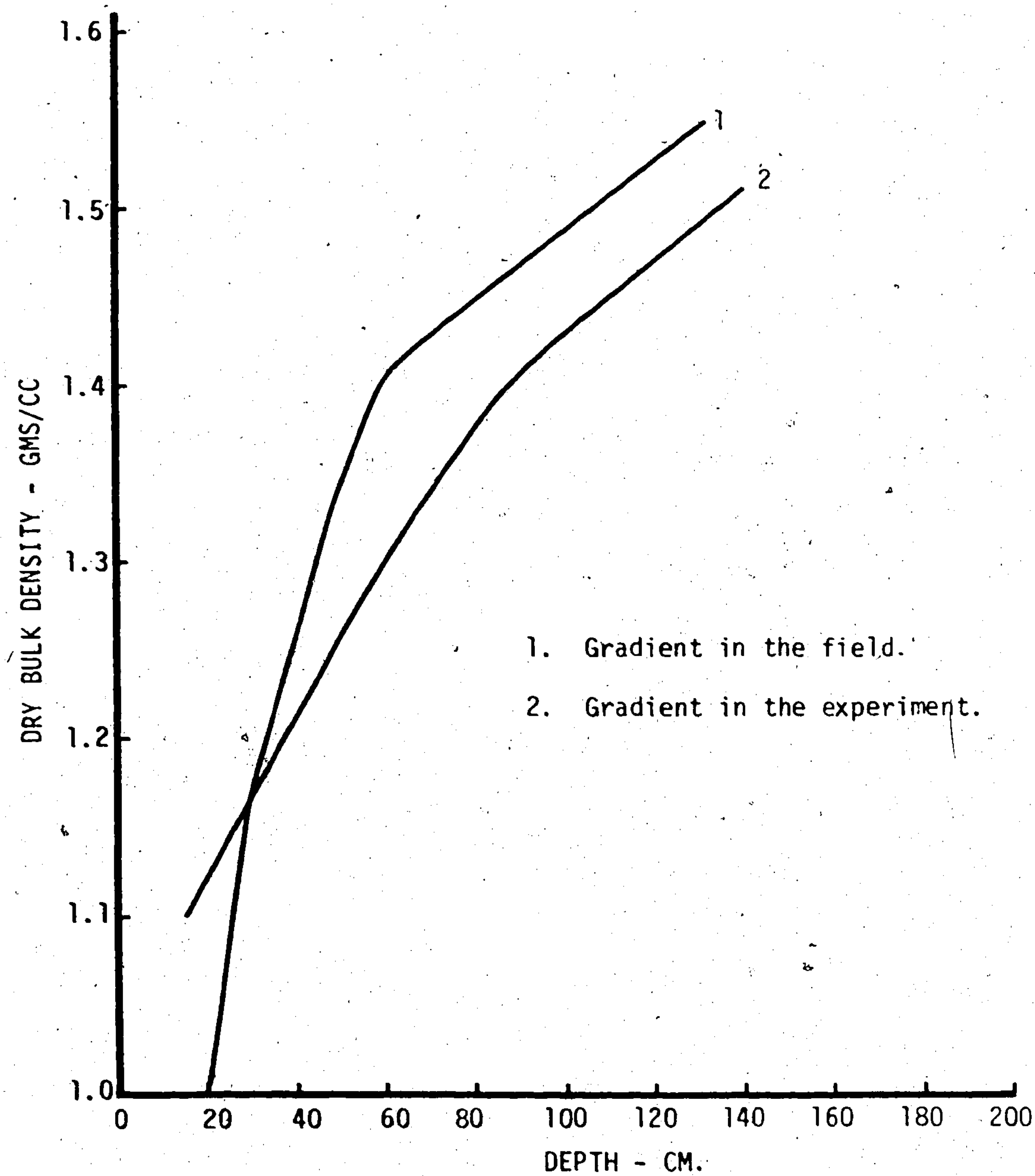

Figure 13. Soil bulk density gradient in the field and in the experiment.

Analysis of the bulk density data obtained in each test run showed that the average variation of bulk density in the experiment was 9.13%. This variation seems to be close to that expected under field conditions.

4.2 Analysis of Variance.

Data was obtained on load vs deflection in the corrugated plastic pipe. Both vertical and horizontal deflections were recorded but only the vertical deflection is taken as a basis for testing and comparing the durability of corrugated plastic drain tubing and other factors such as soil type, installations and depths. The graphs of load vs vertical deflection, as a percent of the original diameter of the tube, were drawn for each treatment combination. Some of these graphs are given in Appendix 4. The data used in the analysis of variance are the surface loads (lbs) that produced 0,5,10,15 ...70,75% deflection+, and also the loads at failure. The failure load is defined as the maximum load reached after whith the load starts decreasing and the increase in deflection is rapid.

The analysis of variance was computed using a library computer programme (43). Analysis was made for each deflection percentage, that is, 0,5,10,15 .... 70,75%, and for failure loads. The significant factors, with the F-values for each analysis, are reported in Appendix 5. The analysis of variance for all deflections was also computed. The results are reported in table 3. This analysis of variance shows the variation over means of percent deflections and interactions of factors with percent deflections. These results are not interpreted as they have little practical value. However, they are reported in Appendix 6.

+ These deflections do not include deflections due to earthfill loads.

TABLE 3. ANALYSIS OF VARIANCE OF LOADS (LBS) ON THE LOAD PLATE.

Source of Variation	Degrees of Freedom	Mean Squares	F-Values
I	1	0.38862E08 +	14.2206**
D	1	0.81360E08	29.7713***
ID	1	0.18581E08	6.7993*
S	1	0.15605E09	57.1017***
IS	1	0.15247E07	0.5579
DS	1	0.34492E08	12.6213**
P	1	0.32811E06	0.1201
IP	1	0.35524E07	1.2999
DP	1	0.44621E06	0.1633
SP	1	0.78311E06	0.2866
Error	16	0.27328E07	

* Significant at .05 probability level.

** Significant at .01 probability level.

*** Significant at .001 probability level.

\+ 0.38862E08 means 0.38862×10^8

4.2.1 Main Effects.

The computed F-values (table 3) show that the main effects of installations, depths and soils, were highly significant.

The surface loads which the corrugated plastic tube drains can support are significantly greater when installed in sandy loam soil than when installed in silt loam soil (tables 7,8). The reason seems to be the higher clay contents at the depths of installation in silt loam soil than in sandy loam soil. The moisture contents around the plastic tubes ranged from 26 to 30% in silt loam, which were greater than the plastic limit, and hence the soil was in a relative ow condition. Soehne (31) reports that the compressive stress in the soil has a tendency to concentrate around the load axis. This tendency becomes greater when soil becomes more plastic due to increased moisture content and when the soil is less cohesive. Although the silt loam soil is more cohesive than the sandy loam soil, the higher moisture contents in silt loam contributed to the greater concentration of equal pressure lines towards the load axis. Also the equal pressure distribution lines become narrower due to flow of soil and extend deeper thus transmitting greater pressure. This flow of soil also resulted in greater settlement of the load plate as compared to the settlement in sandy loam soil causing the plastic tubes installed in silt loam soil to fail at lesser surface loads as compared to the tubes installed in sandy loam soil.

Another factor which contributes to significant differences in load values for the two soil types seems to be the difference in bulk density. This may be due to the greater tendency of the silt loam soil to increase in moisture content with increasing depth and thus to form lumps more readily. Although more compactive effort was given to the silt loam soil as compared to the sandy loam, the higher moisture contents and tendency to form clods contributed to a relatively low bulk density compared to the sandy loam soil. As mentioned in the review of literature, Manley (23) conducted investigations on three different types of soil and concluded that the greater percentage of load variation in different soil types was due to the variation in bulk density.

The surface loads which the corrugated plastic tubes can support when installed at the 125 cm depth are significantly higher than those installed at the 95 cm depth. The conclusion is that drain tubes installed at greater depths are affected less by surface loads than those installed at shallower depths. The settlement of the load plate was 67.5 cm in the case of the 125 cm depth of installation as compared to 46.7 cm in the case of the 95 cm depth of installation. This may be explained by Soehne's pressure distribution in soil (31). At greater depths of installation the bulb-shaped equal pressure distribution lines would transmit lesser pressure to the plastic tubes installed at greater depths as compared to the plastic tubes installed at shallower depths when the same amount of surface load is applied. Consequently, greater loads and greater settlements of the load plate are needed for failure of plastic pipes at greater depths of installations as compared to shallower depths. Most farm machinery and equipment used on farms probably would not settle [illegible]ch depths and cause drain tube failures.

The F-values in table 3 show that the differences in effect due to installation are highly significant. The mean load value of the mole-installation was 8089 lbs as compared to 7554 lbs for the trench installation. This shows that the strength requirements for corrugated plastic tubes installed in mole-channels is significantly lower than for plastic tubes installed in trenches. This is due to additional support provided by the arching effect of the soil in the mole-channels. Arching is defined (35) as a transfer of pressure from a yielding mass of soil onto adjoining stationary parts. This pressure transfer takes place through the mobilization of the shearing resistance of the material which tends to oppose the relative movement with the soil mass. Although this arching effect is also produced in trenches when soil settles, this seems to be more pronounced in mole-channels. This probably is due to a more uniform bulk density circular area around the mole-channel giving a better bridging effect for the soil. Also the curved bottom of the mole-channel gives better bedding conditions to plastic tubes and hence lessens the strength requirements when installed in mole-channels as compared to those installed in trenches. Although the bulk density around the mole-channel was maintained at 3 to 6% more than that of the surrounding soil to simulate field conditions, the increase in bulk density due to pulling of the mole plug in the natural soil conditions in the field may be expected to give a more uniform bridging effect and hence even less strength requirements for drain tubes installed in mole-channels.

The trench width [illegible] 10 in. as compared to the conventional 18 in. trench in the fie[illegible] Newer trenching machines have recently reduced this width from 18 in. to 10 in. As the overburden weight on a buried pipe is a function of the square of the trench width, so the comparison of mole-channel installation with trench installation with respect to the strength requirements of corrugated plastic drain tubes is on the safe side. No gravel was used on the trench installed drains for blinding purposes as the gravel enevelope does not improve the load carrying capacity of the subsurface drains (26).

The computed F-values in table 3 and in Appendix 5 show that the differences in the two types of corrugated plastic drain tubes are not significant. Both types of tubes are about equal in load-bearing capacity.

4.2.2 Interactions.

The interaction between soils and depth is highly significant and between depth and installation is significant. The results in Appendix 5 show that the interaction between depth and installation (ID) is significant only for deflections greater than 35%. However, this discussion is based on the results obtained from the analysis of variance over all the deflections.

Depth and Soils (DS).

The mean load values for both soils and depths are shown in the interaction table 4. The means of the interaction are compared using Tukey's procedure as outlined by Cicchetti (10). The table shows that the mean surface loads which the corrugated plastic tube drains installed at 125 cm and 95 cm depths in sandy loam soil are significantly greater than when installed in silt loam soil. The response curves for this interaction are shown in figure 14. The trend between the two soils is diverging with an increase in depth of installation showing

TABLE 4: INTERACTION TABLE MEAN LOAD (LBS) FOR DEPTH AND SOIL.

Description	125 cm depth (D_1)	95 cm depth (D_2)	Columns difference
Sandy loam soil (S_1)	8996.0	7719.0	1277.0*
Silt loam soil (S_2)	7421.3	7151.4	269.9
Rows difference	1574.7*	567.6*	

The smallest mear difference at the .05 level calculated by Tukey's procedure is 513.63.

* Significant mean differences.

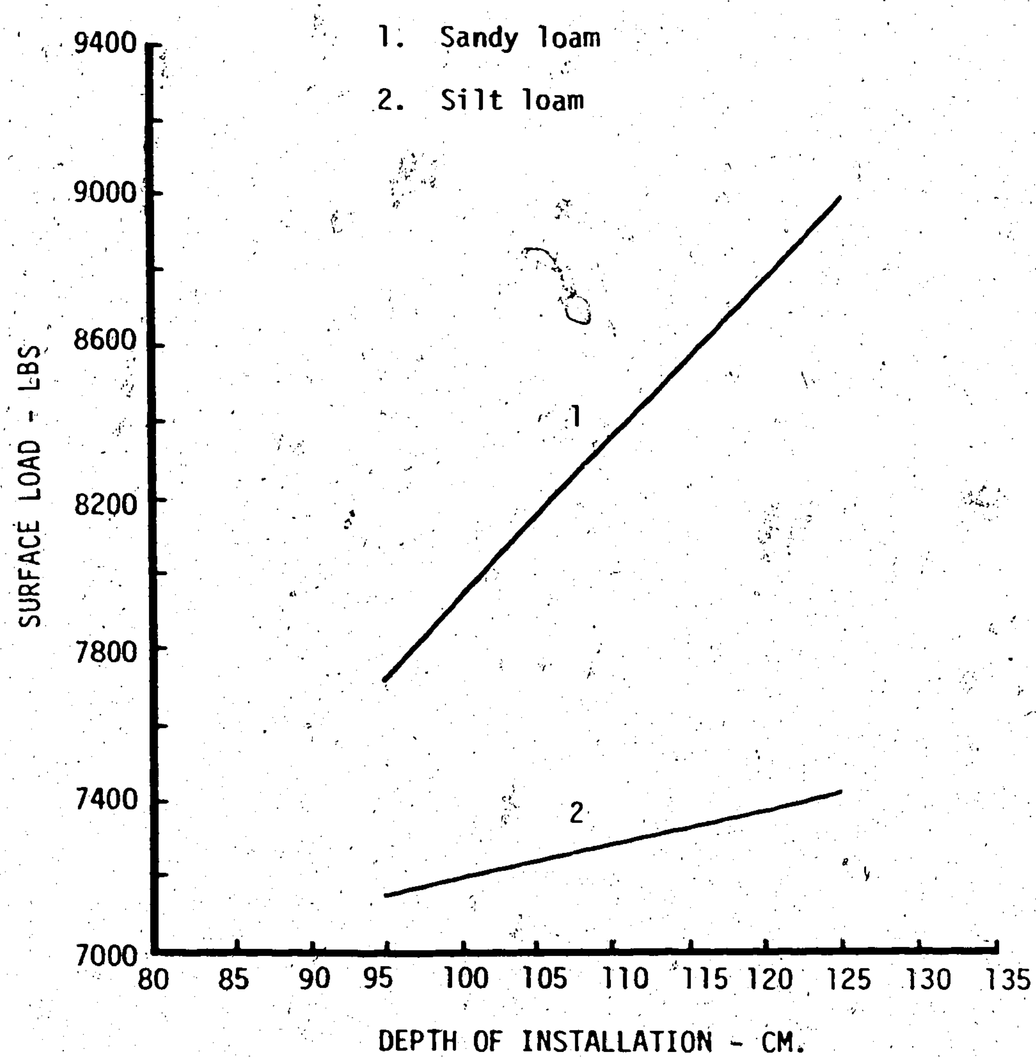

Figure 14. Depth - Soil Interaction.

the marked contrast in load bearing capacity of plastic tubes installed in sandy loam soil than in silt loam soil. The slope of the interaction line for the sandy loam soil is steeper than that of the silt loam soil showing a relatively greater change in load carrying capacity for plastic tubes when the depth of installation changes from 95 cm to 125 cm. This trend also shows that with the increase in depth of installation, the effect of surface loads becomes less prominent and becomes relatively less effective in sandy loam soil than in the case of silt loam soil.

Installations and Depth (ID).

The mean load values of the interaction shown in the interaction table 5, when compared by Tukey's procedure, indicate that the mean load values for the mole installation at the 125 cm depth are significantly higher than the mean load values at the 95 cm depth. This is not the case for the trench installation. Also, the load values for the mole installation at the 125 cm depth is significantly greater than the load value for the same depth for the trench installation. Response curves for this interaction in figure 15 show a diverging trend for the mole installation. This indicates that with the increase in depth of installation, mole installation becomes a more efficient way of installation for reducing the strength requirements of corrugated plastic drain tubes. The reason is the bridging effect of soil around the mole channel, which has been discussed previously.

4.3 Surface Loads.

The surface loads for deflections of 0, 30 and more than 30% (failure loads) are given in tables 6, 7, and 8 respectively.

TABLE 5: INTERACTION TABLE MEAN LOAD (LBS) FOR INSTALLATION AND DEPTH.

Description	Mole Installation I_1(lbs)	Trench Installation I_2(lbs)	Columns difference
125 cm depth (D_1)	8660.7	7756.5	905.2*
95 cm depth (D_2)	7517.6	7352.7	164.9
Rows difference	1143.1*	403.8	

The smallest mean difference at the .05 level calculated by Tukey's procedure is 513.63.

* Significant mean differences.

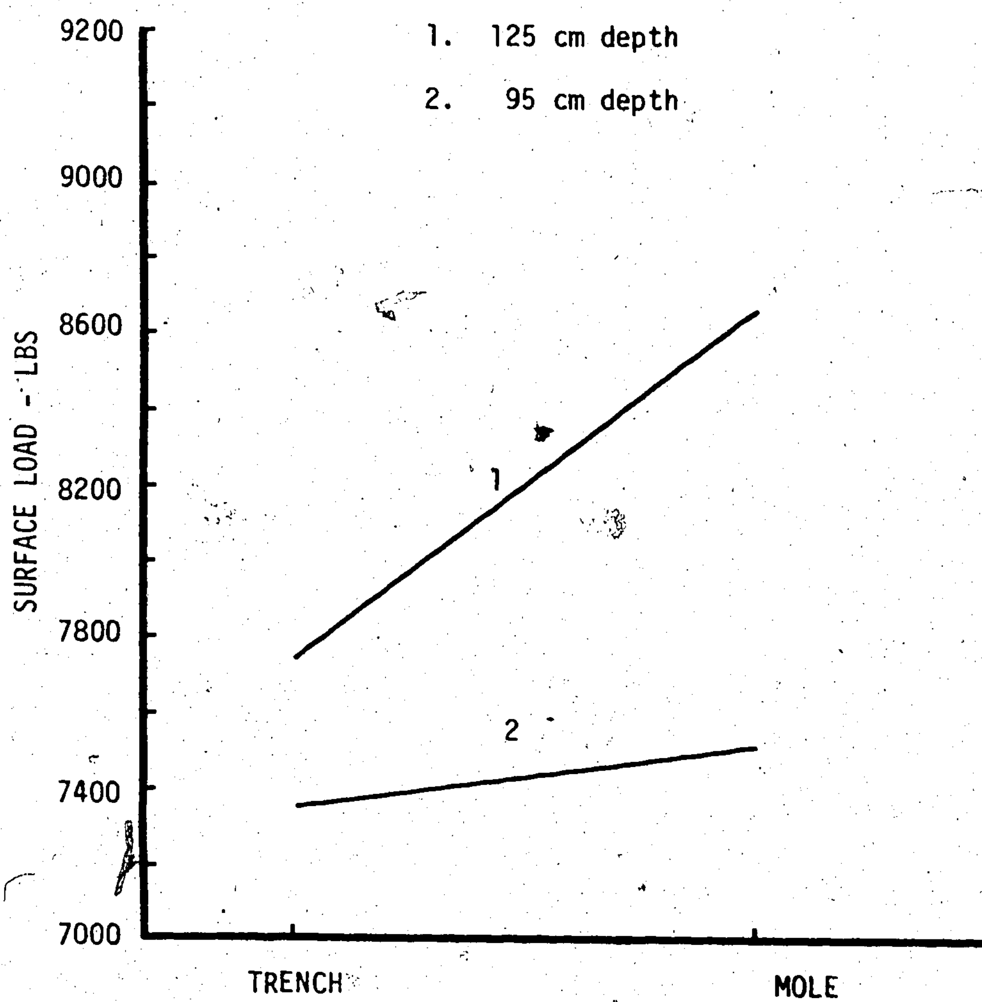

Figure 15. Installation - Depth Interaction.

The analysis of variance for 0% deflection (Appendix 5) shows that only the effect due to installations and depths are highly significant. The effect due to soil type is not significant. The mean loads for both the soils are reported in table 6. A minimum average surface load of 62.7 psi will produce no deflection in corrugated plastic tubes installed in mole-channels at a depth of 95 cm. This value (58.8 psi) is lower for plastic tubes installed in trenches. The surface loads are evidently higher for the 125 cm depth being 67.3 psi for mole installations and 63.8 psi for trench-installed plastic tubes.

As reported in Appendix 5, for the case of the 30% deflection, installations, depths, soils and the interaction between depth and soil (DS) are highly significant. The surface load values for these significant factors are shown in table [illegible] The average surface load values (in psi) are higher for the mole installation than the trench-installed tubes. These values are higher in sandy loam soil than for silt loam soil and also higher for greater depths of installation. These conclusions add to the confidence of interpreting the results obtained from an analysis of variance of overall data rather than depending only on the results from an analysis of variance of single deflection percentages. These load values are the safe limits for the surface loads which can be applied to the fields having corrugated tubes installed in the respective soils and depths in moles or in trenches.

The surface loads which will cause failure in corrugated plastic tubes are shown in table 8. The deflections for these failure

TABLE 6: SURFACE LOADS ($LBS/IN.^2$) FOR 0% DEFLECTION.

Depth of Installation	Sandy Loam Soil Mole	Sandy Loam Soil Trench	Silt Loam Soil Mole	Silt Loam Soil Trench
95 cm	62.9	58.8	62.8	61.6
125 cm	68.4	63.8	67.3	65.0

TABLE 7: SAFE LOADS (30% DEFLECTION) IN $LBS/IN.^2$.

Depth of Installation	Sandy Loam Soil Mole	Sandy Loam Soil Trench	Silt Loam Soil Mole	Silt Loam Soil Trench
95 cm	101.1	98.5	98.2	90.3
125 cm	121.3	114.7	105.1	87.6

TABLE 8: FAILURE LOADS ($LBS/IN.^2$)

Depth of Installation	Sand Loam Soil Mole	Sand Loam Soil Trench	Silt Loam Soil Mole	Silt Loam Soil Trench
95 cm	105.0	103.6	99.0	95.0
125 cm	127.5	117.1	105.7	98.1

loads ranged from 35 to 50% of the original diameters of the plastic tubes.

The analysis of variance for the failure loads is also given in Appendix 5. The effects due to methods of installation and the interactions between depth and soil (DS) are significant. Also, the effects due to depth and soils are highly significant. These are the same factors which were significant for 30% deflections and for the overall data. These failure load values are not likely to be exceeded in the field due to present day farm machinery and equipment. However, these values are a good guide for heavy machines or equipment which may be used in the future.

4.4 Deflection Due to Earthfill.

The average deflection produced in corrugated plastic tubes due to load of earthfill in trenches was 0.4 cm in the case of the 125 cm depth of installation and 0.22 cm in the case of the 95 cm depth of installation.

These deflections do not lessen the strength of corrugated plastic tube drains but according to Watkins (42), a mutual contribution in strength of soil to tube and tube to soil is created in a highly successful interaction. He explains this in the following statement "The decrease in vertical diameter of plastic tubes relieves the tube of high vertical pressure concentration and forces the soil to support part of the load in arching action over the tube." This condition is attained in the experiment with these small amounts of deflections.

The deflections due to earthfill loads are expected to be higher in the field while laying plastic tubes with conventional trench-laying machines. The reason may be the great care taken during placing earthfill on the plastic drain tubes in the experiment which is not expected in the field. Watkins (42) also reports that ring deflection is determined primarily by settlement of the sidefill and is approximately equal to the settlement of the sidefill. He mentions that problems in ring deflection are solved if density of the sidefill is greater than the critical void ratio. Although the critical void ratio of the sidefill was not determined, it can be said that due to the care taken in placing and compacting the sidefill, the density of the sidefill was greater than the density at the critical void ratio.

CHAPTER 5

SUMMARY AND CONCLUSIONS

The cost and labour for installing subsurface drainage has been reduced by using corrugated plastic tube drains. Research is needed to assure that performance is adequate under specific soil and surface loads due to farm machinery and equipment or other concentrated loads. Specific values of surface loads which may cause failure in two types of corrugated plastic tube drains installed in two Edmonton soils at two most probable depths of installation in trenches and moles were established. Effects due to soil types, change in depth and comparison between trench and mole installation, and two types of corrugated plastic tubes were also investigated.

Test sections of 4-in. corrugated plastic tubes were laid horizontally in two types of soils (sandy loam and silt loam). The soil was packed in soil test boxes and bulk density was measured. The two types of tubes were installed at 95 cm and 125 cm depths of installation in trenches and as mole drains. Static surface loads were applied by a hydraulic jack in 200 to 250 lbs. increments and corresponding deflections were recorded by a strain gage measuring mouse. The following conclusions were drawn:

1. Strength requirements for corrugated plastic tube drains were less when installed in sandy loam soil than when installed in silt loam soil.

2. The surface loads which the corrugated plastic tubes could support at the 125 cm depth of installation were significantly

higher than at the 95 cm depth of installation.

3. The corrugated plastic tubes installed in mole-channels (trenchless method) supported significantly greater loads than when installed in trenches.

4. There was no significant difference in load carrying capacities of the 'ADS' corrugated plastic tube drains manufactured by the Big 'O' Drain Tile Co. and that manufactured by the Daymond Co.

5. Minimum surface loads of 62.7 and 58.8 lbs/ $in.^2$ at the 95 cm depth of installation caused no deflection (0% deflection) in corrugated plastic tube drains installed in mole-channels and in trenches respectively. For the drain tubes installed ~~at~~ the 125 cm depth in mole-channels and in trenches, the loads were 67.3 and 63.8 lbs/$in.^2$ respectively.

6. Surface loads of 101.1 and 98.2 lbs/$in.^2$ for mole and 98.5 and 90.3 lbs/$in.^2$ for trench-installed plastic tubes at the 95 cm depth in sandy loam and silt loam soils respectively were safe loads (30% deflection). The values of these loads for plastic tubes installed at 125 cm in sandy loam and silt loam soils respectively were 121.3 and 105.1 lbs/$in.^2$ for mole and 114.7 and 87.6 lbs/$in.^2$ for trench installation respectively.

7. Corrugated plastic tube drains installed at the 95 cm depth of installation in sandy loam and silt loam soils failed at 105.0 and 99.0 lbs/$in.^2$ and 103.6 and 95.0 lbs/$in.^2$ when installed in mole-channels and trenches respectively. The values of these failure loads for the 125 cm depth of installation were significantly higher, 127.5 and 105.7 lbs/$in.^2$, and 117.1 and

98.1 lbs/in.^{2} for mole and trench installation in sandy loam and silt loam soils respectively, than when installed at the 95 cm depth.

8. Deflection due to earthfill loads was very small (less than 1/2 cm). Careful blinding and backfilling while laying the corrugated plastic tubes in trenches prevented any damage.

9. With the increase in depth of installation, trenchless (mole) installation becomes more efficient than trench installation by reducing the strength requirements for corrugated plastic tubes.

10. The corrugated plastic tubes failed after considerable settlement of the load plate; 46.7 and 67.5 cm for 95 and 125 cm depths of installation respectively.

11. Test sections of corrugated plastic tubes regained their original shape after they were taken out of the soil.

CHAPTER 6

SUGGESTIONS FOR FURTHER WORK

1. Similar research should also include investigations of soil pressure distribution.
2. Investigation of surface loads should be made at different moisture levels.
3. Research is needed for larger diameter corrugated plastic drain tubes in the 6 to 12 in. diameter range.
4. Deflections of corrugated plastic drain tubes in the field over long periods of 10 to 15 years should be investigated.

CHAPTER 7

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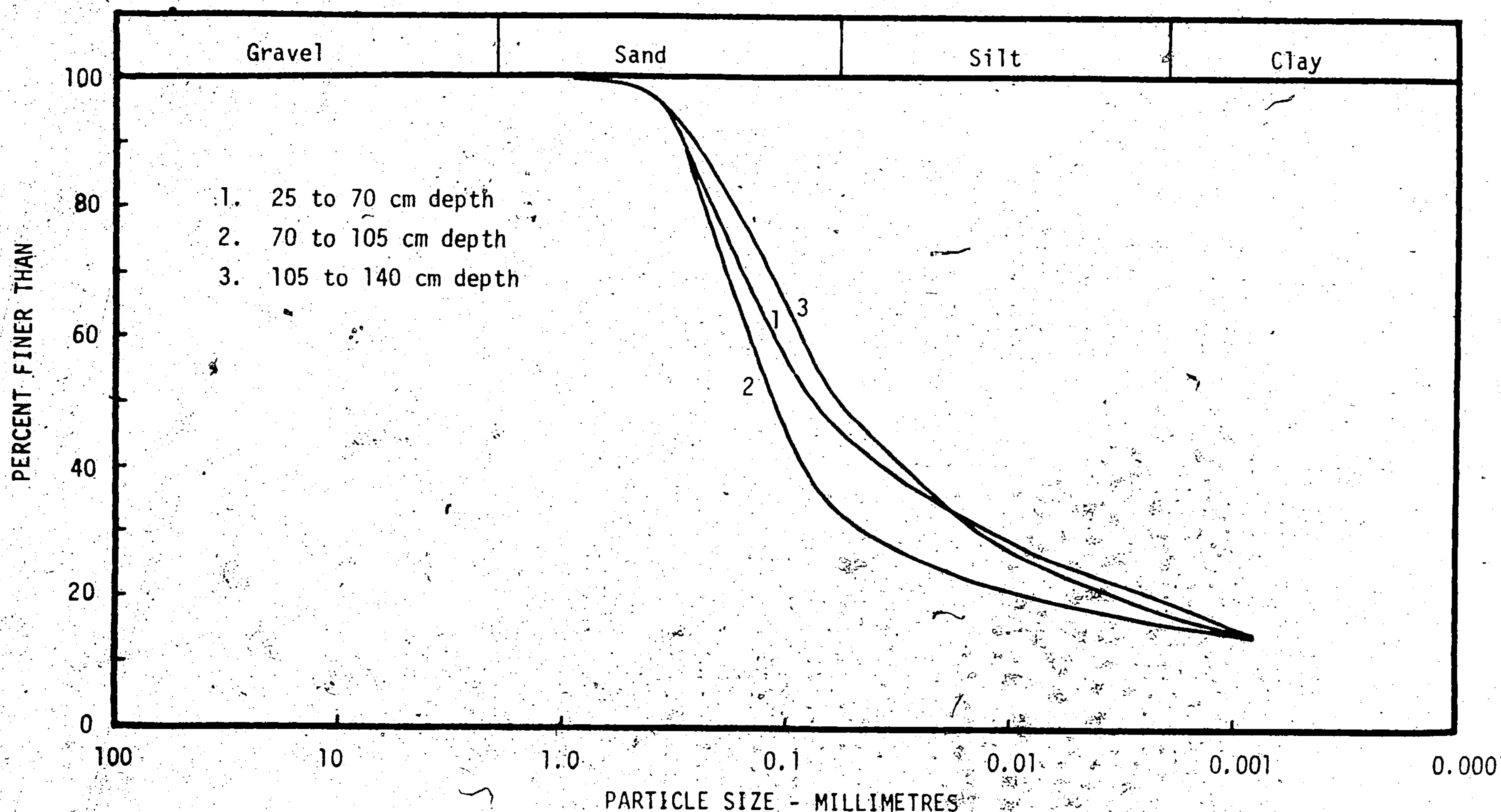
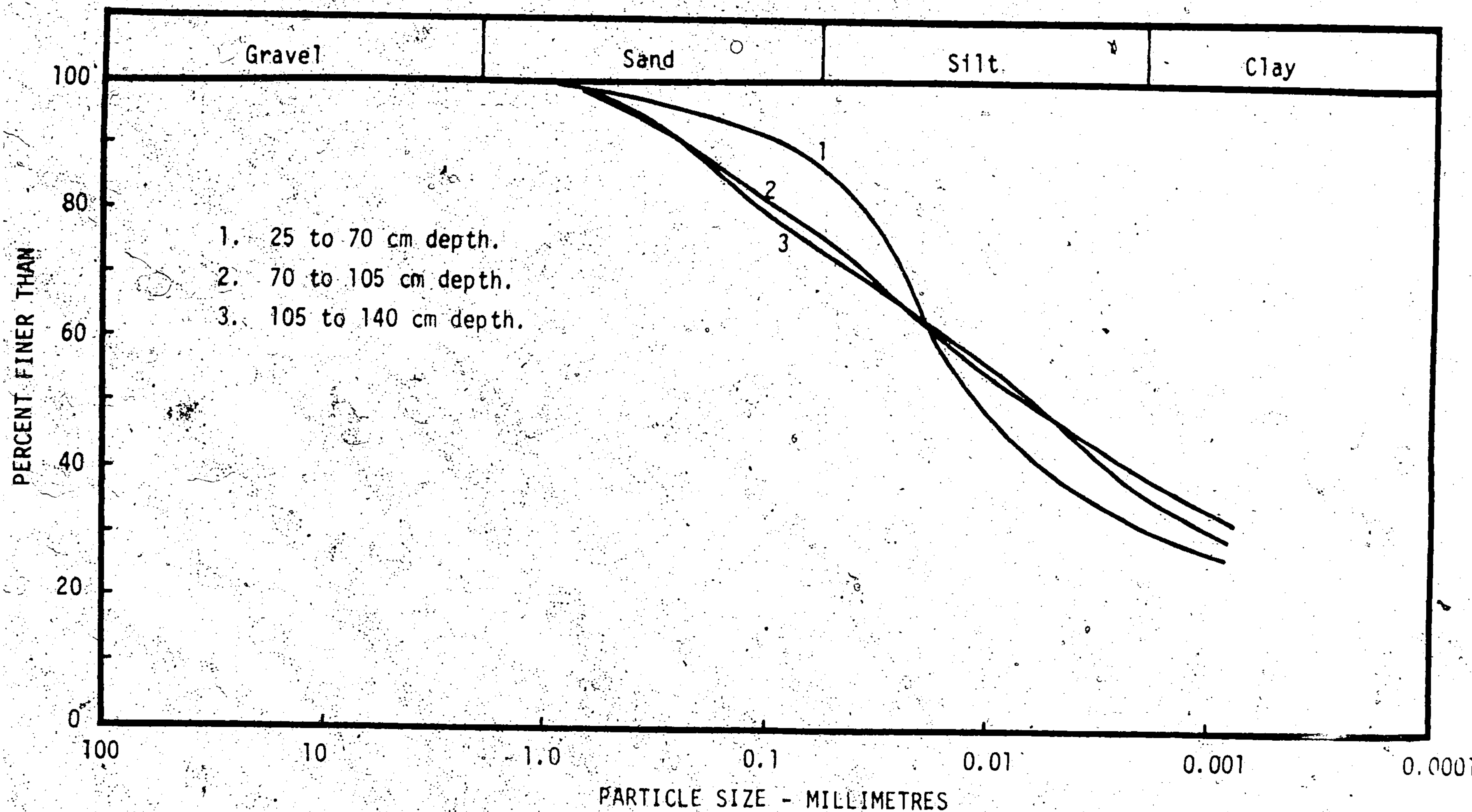
APPENDICES



APPENDIX 1: PARTICLE SIZE DISTRIBUTION CURVES FOR SUBSOIL FROM PONOKA LOCATION.



APPENDIX 2: PARTICLE SIZE DISTRIBUTION CURVES FOR SUBSOIL FROM ELLERSLIE FARM LOCATION.

APPENDIX 3: EQUIPMENT SPECIFICATIONS.

1. Strain Gauges
   Type EA-06-250 BG-120
   120.0 ± 0.15% ohms
   Gauge Factor 2.095 ± 0.5% at 75°F

2. Accudata 104 DC amplifier (Honeywell)
   gain: 0-250 continuously variable
   DC gain accuracy: better than ± 0.5%
   linearity: better than ± 0.1% of full scale output

3. Accudata 105 Gage Control Unit (Honeywell)
   excitation: 3.5 - 11.5 V for 350 Ω
   1.5 - 5 V for 120 Ω
   compensates for ± 5% unbalance.

4. Ultra violet recorder type SE 2005 (SE Laboratories, England)
   direct writing oscillograph
   galvanometers cover frequency ranges 0-30 c/s to 0-8000 c/s
   linearities of ± 1% over 6 cm deflection can be achieved.
   Paper speeds 1.25 - 2000 mm/sec.

5. Steel Frame

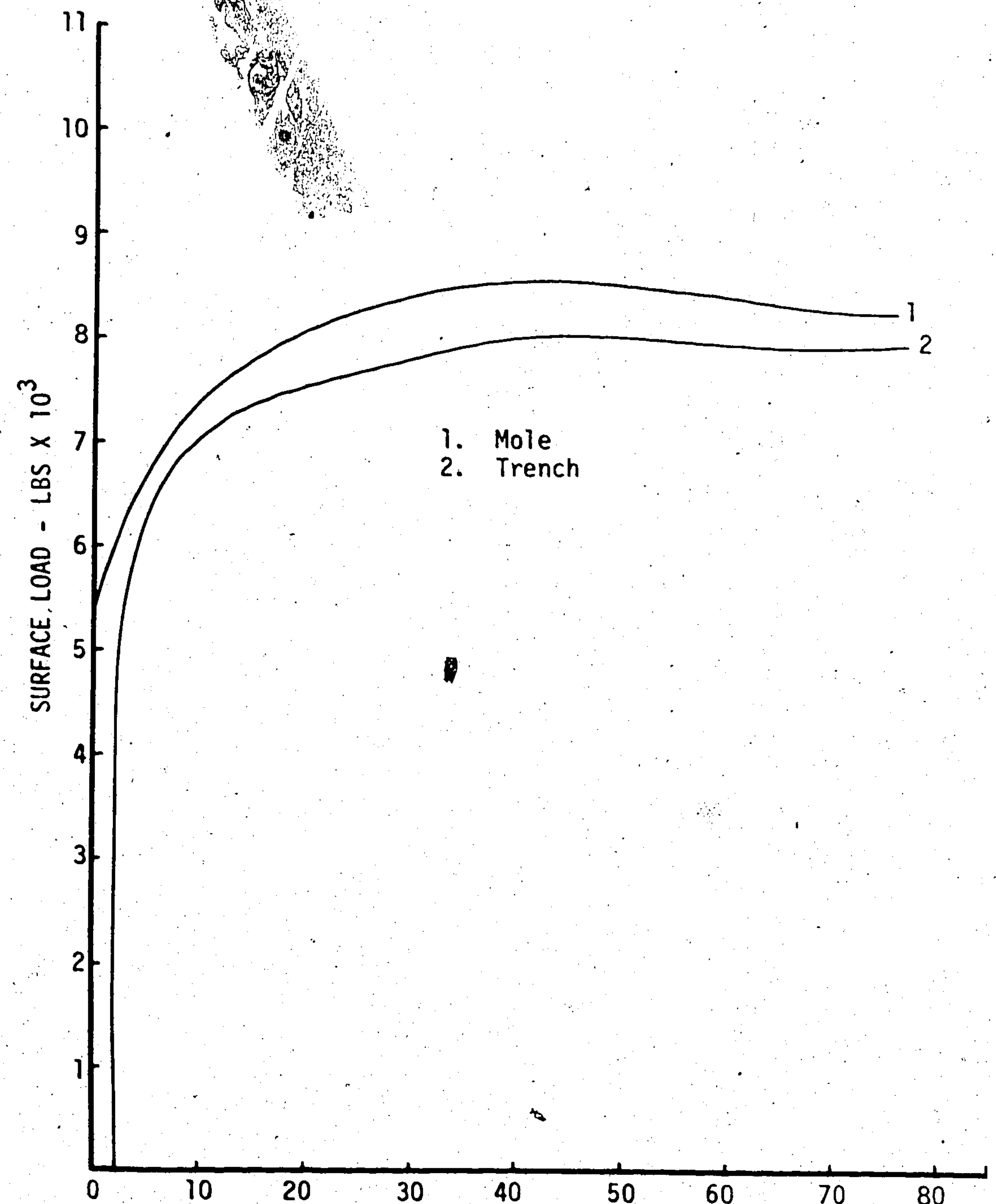
Description	Size	Nos.	X-section
Bottom Free I-beams	4 ft	5	4I7.7
Bottom Side I-beams	$4\frac{1}{2}$ ft	2	5I10
Side Channel-beams	$8\frac{1}{2}$ ft	4	5[11.5
Top Wide Flange beam	4 ft	1	8WF20
Channel-braces	$3\frac{1}{2}$ ft	4	3[5.0
Bolts A325	Dia. $\frac{3}{4}$ in	16	
Cross-beam supports (equal leg angles)		4	L4 x 4 x $\frac{3}{4}$

Lower cross-beam supports welded to channels

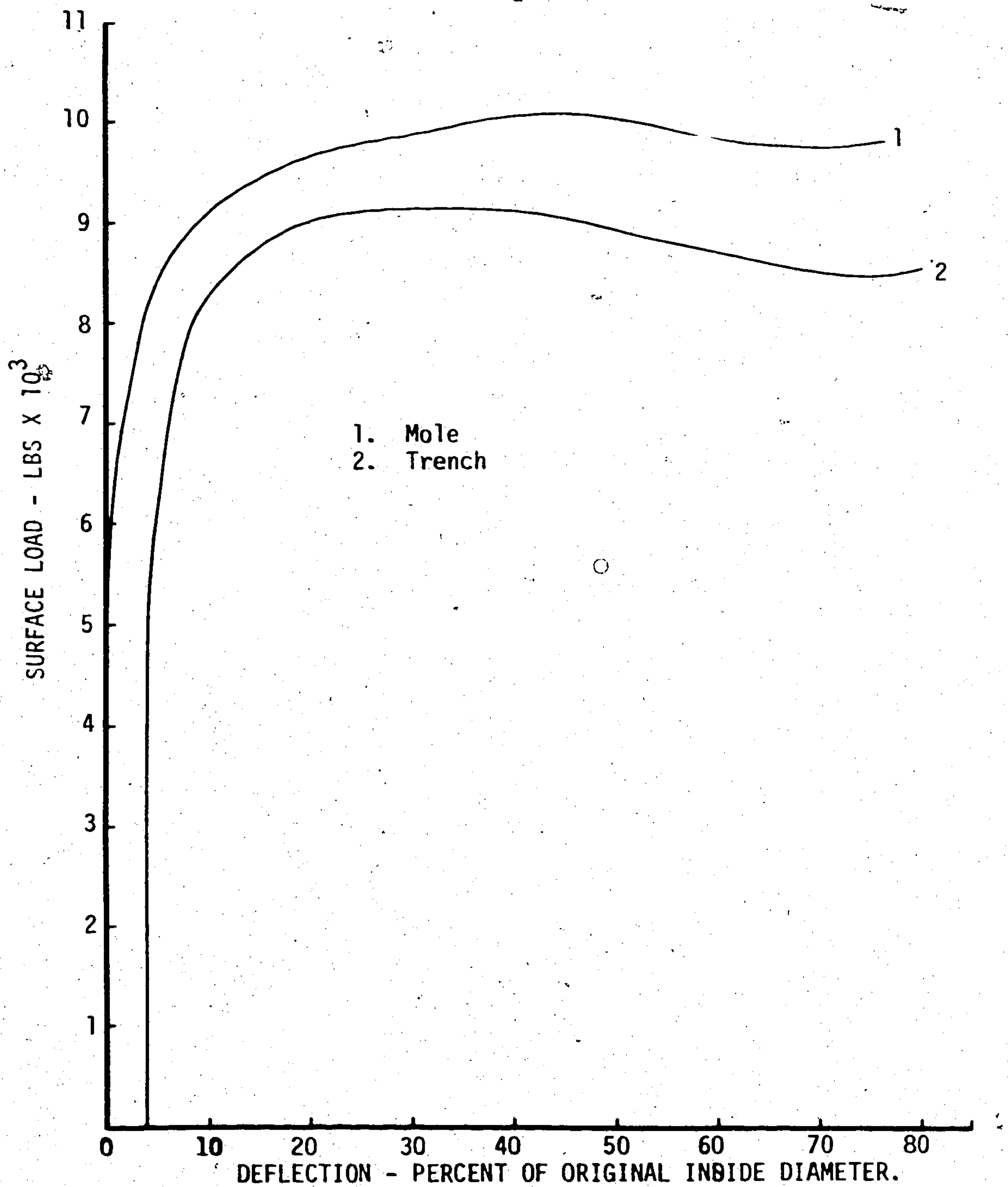
Upper cross-beam supports bolted to channels

Channel braces welded on top and bottom.

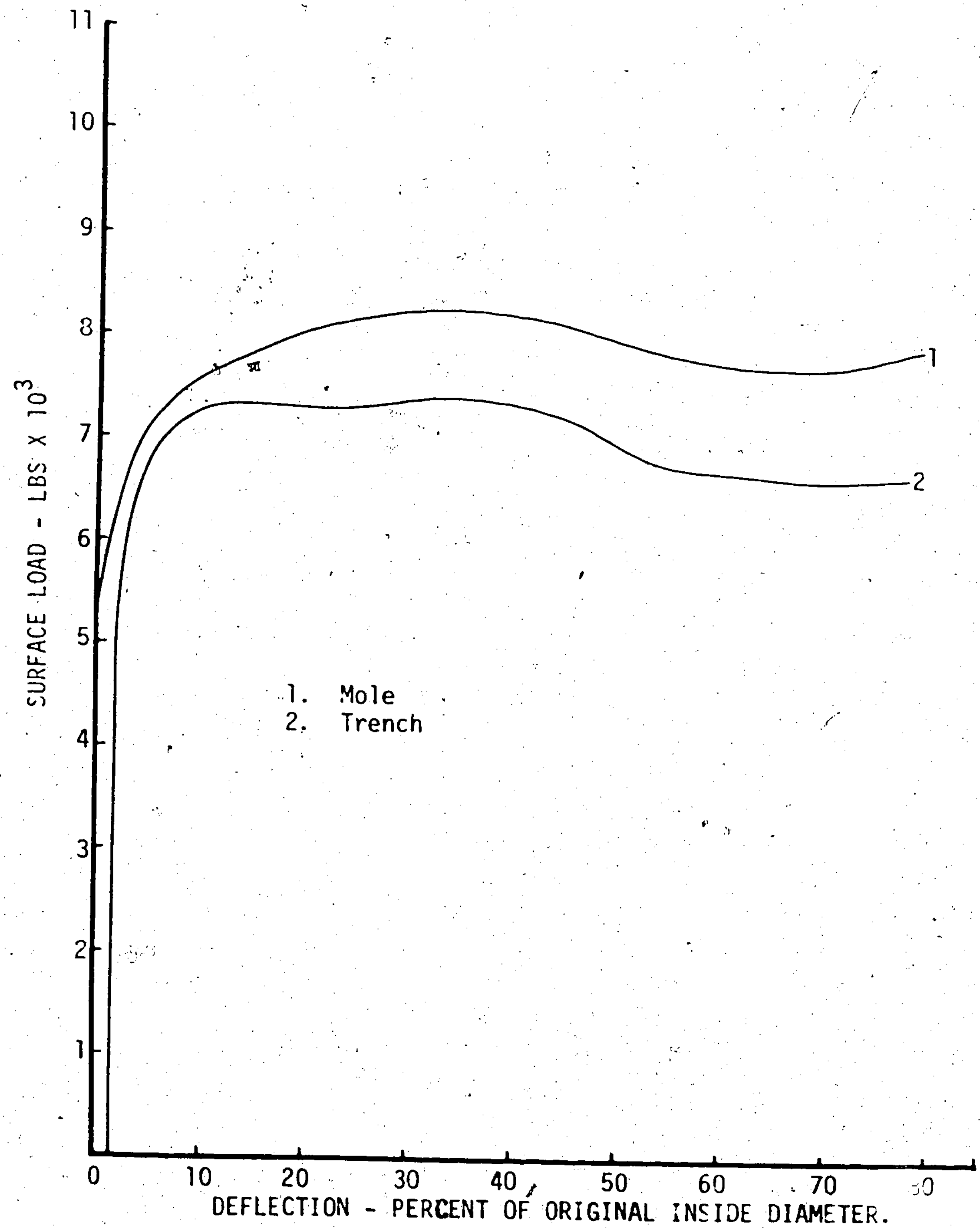
APPENDIX 4: SURFACE LOADS VS DEFLECTION CURVES. (Each curve represents average of two replicates).



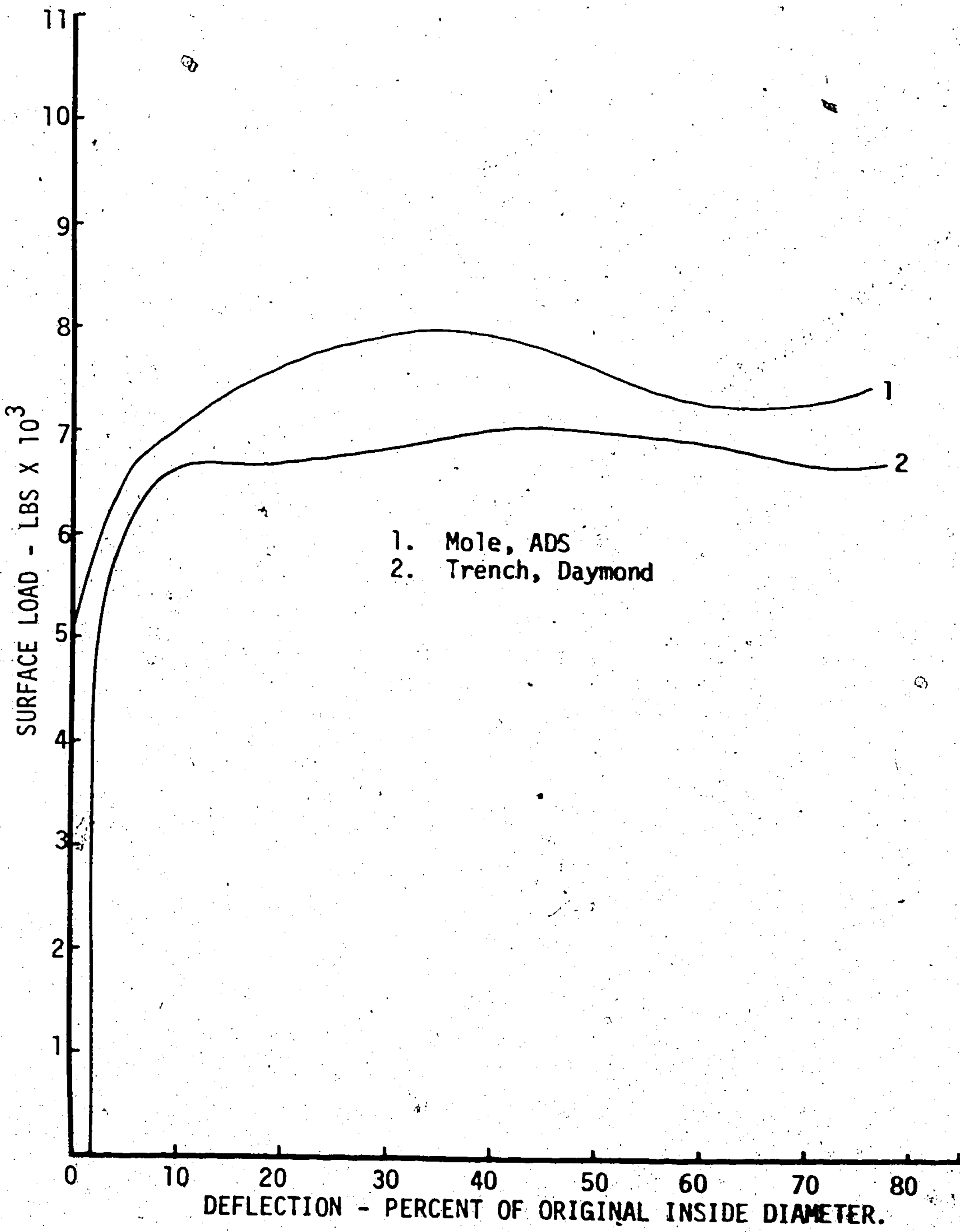
Sandy loam - Daymond - 95 cm Depth of Installation



Sandy loam - ADS - 125 cm Depth of Installation.



Silt loam - ADS - 125 cm Depth of Installation



Silt loam - 95 cm Depth of Installation.

APPENDIX 5: ANALYSIS OF VARIANCE FOR VARIOUS PERCENTAGES OF TUBE DEFLECTION.

Percent Deflections	Installations (I)	Depths (D)	Soils (S)	Interaction Installations x Depth (I x D)	Interaction Depths x Soils (D x S)
0	9.1903**	21.7523***	-	Note (S x P) 4.7372* (Soils x Pipes)	-
5	-	36.62***	-	-	-
10	29.82***		[illegible]		7.30*
15	-	28.31***	[illegible]	-	10.10**
20	4.89*	25.43***	[illegible]	-	13.37**
25	9.37**	24.75***	31.08***	-	13.82**
30	15.56**	21.49***	38.32***	-	13.39**
35	17.47***	19.63***	43.50***	-	12.69**
40	21.64***	22.03***	70.67***	5.64*	13.73**
45	30.59***	26.68***	121.83***	15.03**	16.93***
50	40.21***	24.82***	155.20***	21.31***	20.83***

Continued.

APPENDIX 5: CONTINUED

Percent Deflections	Installations (I)	Depths (D)	Soils (S)	Interaction Installations x Depth (I x D)	Interaction Depths x Soils (D x S)
55	33.53***	20.98***	140.94***	20.80***	13.29**
60	30.38***	16.44***	117.06***	16.65***	9.99**
65	23.80***	13.25**	89.79***	14.96**	7.32*
70	20.83***	12.82**	70.76***	14.44**	6.14*
75	19.98***	12.90**	62.46***	14.37**	5.18*
Failure	6.72*	25.80***	37.76***	-	8.44*

* Significant at .05 probability level.
** Significant at .01 probability level.
*** Significant at .001 probability level.

APPENDIX 6: ANALYSIS OF VARIANCE SHOWING VARIATION OVER MEANS OF PERCENT DEFLECTIONS (OBSERVATIONS).

Source of Variation	Degrees of Freedom	Mean Square	F-value
O	16	0.17964E08	276.9626***
OI	16	0.78383E06	12.0847***
OD	16	0.44301E06	6.8300***
OID	16	0.36072E06	5.5613***
OS	16	0.19756E07	30.4585***
OIS	16	0.26670E06	4.118***
ODS	16	0.18978E06	2.9258***
ODP	16	0.25469E06	3.92266***
ERROR	256	64862	

*** Significant at .001 probability level.
Third and higher order interactions are not included.