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A Gravel Envelope for a Tile Drain in a Coarse Silt Base Material

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A GRAVEL ENVELOPE FOR A TILE DRAIN

IN A COARSE SILT BASE MATERIAL

BY

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This thesis is approved as a creditable and independent investigation by a candidate for the degree, Master of Science, and is acceptable as meeting the thesis requirements for this degree, but without implying that the conclusions reached by the candidate are necessarily the conclusions of the major department.

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INTRODUCTION

In recent years drainage has been recognized as essential to the development of a large-scale irrigation project. South Dakota is now planning for the irrigation of approximately one-half million acres of land through the proposed Oahe Irrigation Unit located in the northeastern part of the state. There are two main bodies of land in the Oahe Unit--the Lake Plain Area which is the postglacial "Lake Dakota" and the larger portion of the project, and the Missouri Slope Area which is part of the Great Plains province lying east of the Missouri River. A large percentage of this project's total developmental cost will be for tile drainage.

The principal function of tile drainage is to control water table levels. The Bureau of Reclamation in the 1965 Oahe Unit Report (21) recommends that tile drains be placed from 6 to 12 feet in depth with spacings from 400 to 900 feet in the Lake Plain Area to control water table levels. These tile drains will range from 4 to 6 inches in diameter. The soil textures prevailing at the 6 to 12 feet depth in the Lake Plain Area are silt loam, silty clay loam, sandy loam and silt. Silt is the most predominant base material for the tile drains.

To ensure a longer life for the tile-drainage system, it is often necessary to place a more permeable backfill material than the base material around the tile drain. This material, placed on either the top, bottom or sides of the drain, singularly or in combination, is called an envelope. The three-fold purpose of an envelope is as follows:

1. To exclude fine soil particles from moving into the drain and resulting in clogging.
2. To increase the effective drain diameter by providing a highly permeable zone around the drain.
3. To serve as a stabilizing foundation for the drain.

REVIEW OF LITERATURE

This review of literature will be divided into four areas of discussion: South Dakota envelope research, gravel envelope design, mechanics of tile drainage and an analog model of tile drainage.

South Dakota Envelope Research

Research on envelopes for the drainage of the Oahe Irrigation Unit in South Dakota has been done only by the Bureau of Reclamation and the South Dakota State University Agricultural Experiment Station. The Bureau of Reclamation in the 1960 Oahe Unit Report (21) recommended a 6-inch gravel envelope above and below the tile with a 6-inch minimum thickness on the sides. Preliminary studies indicated that approximately 75 percent of the envelope material may be pit-run gravel and can be obtained within a haul distance of 20 miles from the irrigation project. Then, in 1965 the Bureau of Reclamation (22) recommended that a 4-inch gravel envelope be provided around the drain and graded to give satisfactory performance for tile drains.

In 1963, a field drainage plot was constructed at the Redfield Irrigation Farm, Redfield, South Dakota, within the proposed Oahe Irrigation Unit to compare predicted tile outflow rates measured under a controlled condition. Lembke (10) summarized two years of study after the drainage plot was installed: "It was also found that a tile drain embedded in a gravel filter under a ponded condition increased in flow rate during the first two years after construction.

This increase was attributed to removal and rearrangement of fine particles in and around the gravel filter."

Plot Description

The drainage plot was constructed 75 feet by 150 feet with a 20-foot border surrounding the plot. A double thickness of polyethylene to a depth of 8.5 feet separates the plot from the border. The center of the plot is drained by a 6-inch drain at a depth of 8 feet. The tile drains are all 4-foot lengths of bell and spigot concrete tile, loosely connected, and embedded in an envelope of at least 6 inches of gravel separating the drain from the surrounding soil.

Soil Description

The soil on the drainage plot is classified as Beotia silt loam. The surface texture is silt loam; but at a depth of from 4 to 7 feet, it consists of stratifications of lake bed sediment. At a depth of 7 feet the soil changes to a loose silt which continues to a depth of 13 feet where there appears a heavy glacial till which was deposited prior to the lake bed development. Table 1 summarizes the average hydraulic conductivity measurements, $\bar{K}$, made with auger holes in the soil profile. Lembke (10) recommends the auger hole method for measuring hydraulic conductivity in this soil.

Table I. Auger Hole Hydraulic Conductivity Measurements at Redfield Irrigation Farm

Average Depth of Water Table (inches)	Average Depth of Auger Holes (inches)	Number of Holes	$\bar{K}$ (in/hr)	Range $\bar{K}$
18	40	10	0.72	0.43 - 1.43
48	68	4	0.48	0.26 - 0.66
70	96	2	0.80	0.78 - 0.82
40	(backfill) 54	3	0.52	0.36 - 0.68

Tile Outflow

Tile outflow was measured from the drainage plot for the severe case of a ponded water surface. There was a large increase in tile outflow in the second year in comparison to the first year, which was attributed to a removal or rearrangement of fine particles in and around the gravel filter. However, after the third year of tile installation, the outflow started at a higher level than the second year, but then decreased to a lower value than the second year. The pattern of decrease was similar to that which occurred in 1964. Figure I (9) shows the hydrograph of tile outflow for the first three years of installation. Also, the predicted tile outflows from hydraulic conductivity measurements by auger holes and 3-inch cores using an IBM 1620 computer are shown on the hydrograph. Lembke found a considerable amount of sediment in the tile outflow; however, this amount of sediment has not yet been analyzed from the sediment samples. This sediment came during the early part of the tile outflow period.

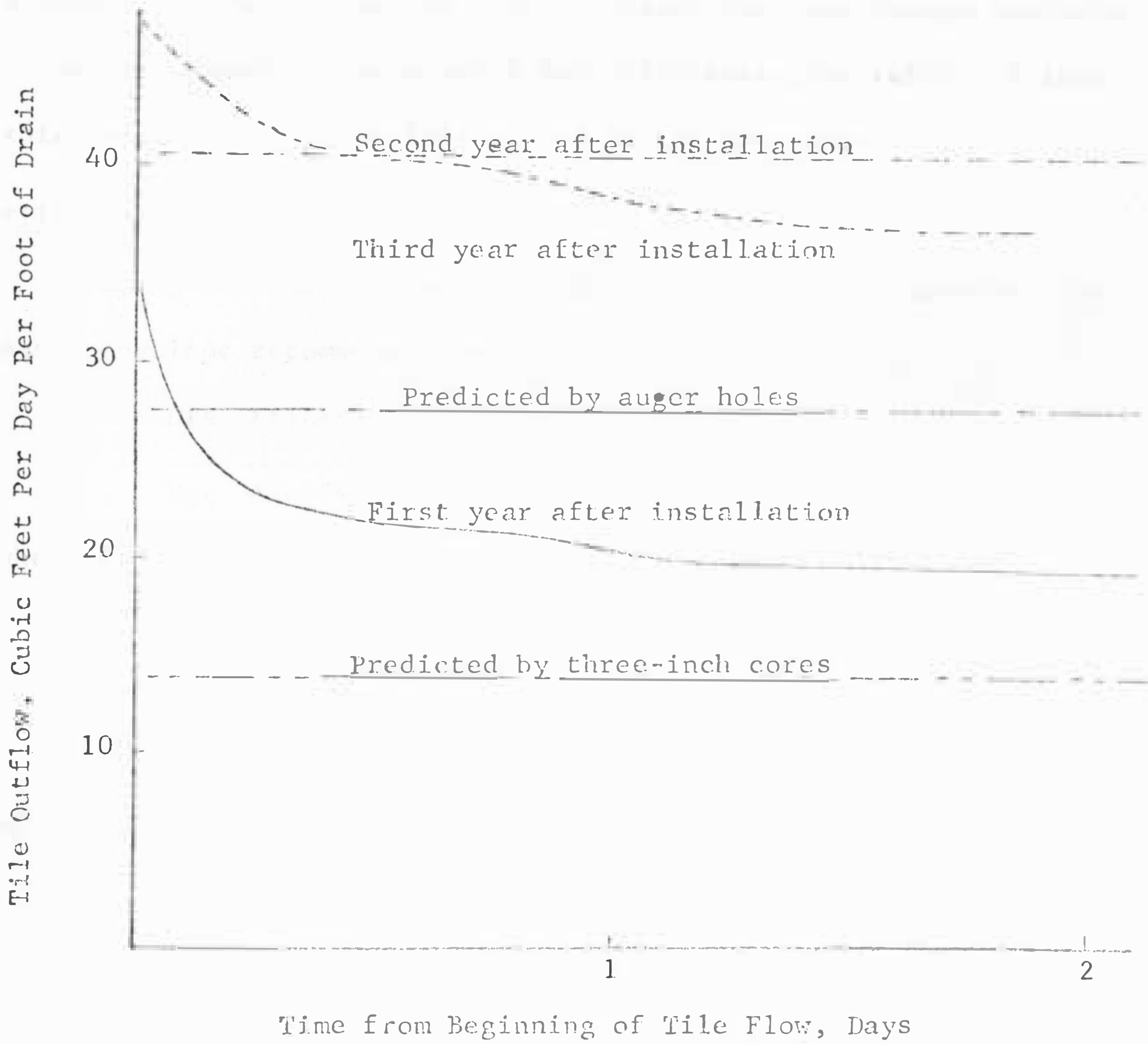

Figure I. Hydrograph of Tile Outflow from Redfield Drainage Plot

Gravel Envelope Design

A gravel envelope is required to facilitate groundwater entry into the drain, prevent erosion of the base material, and serve as a stabilizing foundation for the drain. Gravel envelope design characteristics can generally be divided into filter-aquifer ratios, filter thickness, filter permeability, and filter placement.

Filter-Aquifer Ratios

During the 1920's, Karl Terzaghi (17) of Austria made the first basic envelope recommendation:

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{Aquifer}} < 4 < \frac{D_{15}\ \text{Filter}}{D_{15}\ \text{Aquifer}}$$

where D_{15} and D_{85} are the particle sizes at which 15% and 85% of the particle weight is smaller. The terms Filter and Aquifer refer to the gravel envelope and base material.

Several agencies have since developed design criteria. Principal of these are the Corps of Engineers at the U.S. Army Waterways Experiment Station (20) and the U.S. Department of Interior, Bureau of Reclamation (23). A composite of these investigations by the U.S.D.A. Soil Conservation Service (19) has resulted in the following recommendations as shown in Figure II:

The first step is to determine by mechanical analysis and gradation curve of the base material. From this gradation curve a filter material of known gradation is selected to meet the design requirements. Multiplying the 50% grain size of the base material by 12 and

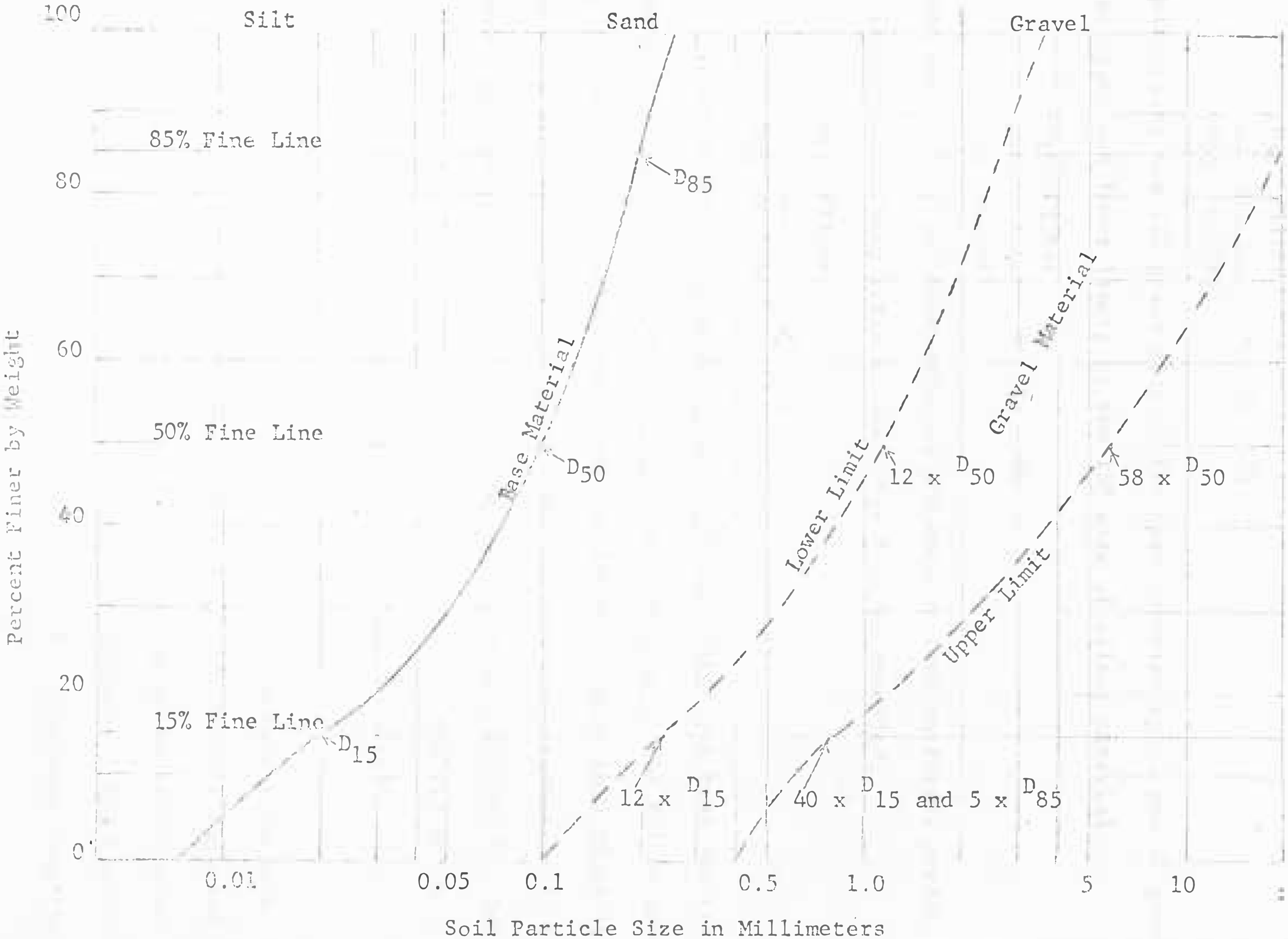

Figure II. A Typical Grain-Size Distribution Curve Showing Criteria for Gravel Envelope Selection

58 will give upper and lower limits of the 50% size of filter material.

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{Aquifer}} = 12 - 58$$

Multiplying the 15% grain size of the base material by 12 and 40 gives the upper and lower limits of the 15% size of filter material.

$$\frac{D_{15}\ \text{Filter}}{D_{15}\ \text{Aquifer}} = 12 - 40$$

When both filter and base material are more or less uniformly graded, a filter-stabilizing ratio of less than 5 is recommended.

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{Aquifer}} < 5$$

In addition, the gradation curves of the filter and base material should be approximately parallel. The maximum size of the filter material can be about 1½ inches, and there should be not more than 5% of filter material passing the No. 200 sieve. The maximum size limitation is to ensure against too much segregation during placement, and the No. 200 sieve limitation is to prevent an excess of fines in the filter which are more easily carried by water percolation into the drain tile.

Filter Thickness

Some of the recommendations on filter thickness have been made by the U.S.D.A. Soil Conservation Service (19), des Bouvrie (5), and Edward E. Johnson Company (7). The Soil Conservation Service recommends three inches as a minimum gravel filter thickness. The Edward E. Johnson Company, manufacturer of well screens, recommends that the gravel pack have a minimum wall thickness of three inches and a maximum of nine inches. Des Bouvrie concluded the following concerning filter thickness and Filter-Aquifer ratios at the 50% grain size:

1. A Filter-Aquifer ratio of around 12.0 allows the use of a filter 0.5 - 1.0 inches thick.
2. Gravel filters with successful combinations of Filter-Aquifer ratios and standard deviations permit a filter thickness of:
   a. 3 inches for Filter-Aquifer ratios = 12.0 - 24.0
   b. 6 inches for Filter-Aquifer ratios = 24.0 - 28.0
   c. 9 inches for Filter-Aquifer ratios = 28.0 - 40.0
3. Gravel filters with Filter-Aquifer ratios between 40 and 52.0 can only be useful when having a thickness of at least 12 inches.

Filter Permeability

For a given hydraulic gradient, Quazi, Lockman, and Halderman (1) found that an increase in Filter-Aquifer ratio at the 50% grain size produces an increase in discharge. Leatherwood (1) found that for Filter-Aquifer ratios at the 50% grain size ≤ 5 the interface head loss varies linearly with velocity and head loss generally increases as filter mean diameter decreases. Des Bouvrie (6) found the permeability of the interface zone to generally decrease with increasing Filter-Aquifer ratios at the 50% grain size. Des Bouvrie states:

1. In general, filters with low Filter-Aquifer ratios retain high hydraulic conductivities, because sand does not penetrate in large amounts and alter its hydraulic properties.

2. The hydraulic conductivity of the successful filters is not affected significantly by sand movement beyond 2 inches into the gravel from the interface, which separates the sand and gravel.

Filter Placement

Sisson (16) explained why it is sometimes recommended that envelope materials be placed only on the top and sides of the tile if the primary function of the envelope is to prevent clogging of the drain with sediment. He states, "Soil particles under the drain will enter the drain from the bottom only if the upward force from water moving into the drain is greater than the opposing gravitational force from the weight of the soil particles." As soil particles such as fine sand and silt become smaller, these particles will move at slower velocities; thus, the forces caused by flowing water can easily move the smaller particles. Instability or piping tendencies become progressively worse as soil particles become smaller. However, as soil particles become quite small, as in fine silt and clays, the soils become cohesive and individual soil particles bind together to provide stability. Nelson (15) described the group of soils that are the most unstable as those lying within the size range of 0.05 to 1.0 millimeters in diameter.

Mechanics of Tile Drainage

The purpose here is to discuss the flow of water through the soil and into a subsurface drain. The fundamentals of ground water are summarized as pertaining to this author's thesis (see, for example, Luthin (12) and Harr (6) for complete discussion of ground water flow).

Darcy's Law

In 1856, Henry Darcy discovered an empirical law which is regarded as the fundamental law concerning flow of water through soils. Expressed in words, the law states that the flow of water through a porous medium is proportional to the hydraulic gradient and to a factor known as the hydraulic conductivity, which is characteristic of the porous media. In mathematical symbols Darcy's law is as follows:

$$Q = KiA$$

where

Q = volume of water per unit time (l^3t^{-1})

i = hydraulic gradient (dimensionless)

A = cross section of flow area (l^2)

K = hydraulic conductivity (lt^{-1})

The hydraulic gradient, i, represents the total head loss of the fluid over a given distance. The hydraulic gradient can be evaluated by dropping the velocity head terms from the Bernoulli equation, since for flow of water through soils the kinetic energy due to velocity is negligible, leaving only the pressure head and gravitational or positional head to supply the driving force. The sum of these two heads is called the hydraulic head or potential head and can be written as follows:

$$\phi = \frac{p}{\rho g} + h$$

where

ϕ = hydraulic head at a particular point (l)

p = pressure (fl^{-2})

ρ = density ($fl^{-4}t^{2}$)

g = gravitational constant (lt^{-2})

h = elevation measured from a reference plane (l)

Laplace's Equation

The linear flow of water through a column of soil is easily analyzed using Darcy's law. However, the two- or three-dimensional flow which occurs in land drainage requires the derivation of an equation describing the distribution of hydraulic head.

Laplace's equation is developed by examining the flow through a small rectangular parallelepiped. Here, the net mass of water gained or lost within the parallelepiped is set equal to the time rate of change or water mass. Next, the following assumptions are made:

1. The soil is isotropic.
2. The voids are completely filled with water.
3. No consolidation or expansion of the soil takes place.
4. The soil and water are incompressible.
5. Flow is laminar, and Darcy's law is valid.

Laplace's equation for three-dimensional flow, applying the previous assumptions, is obtained as

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0$$

where ϕ is the hydraulic head or potential; and x, y and z are cartesian coordinates. In two dimensions, Laplace's equation has the form

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0$$

The Flow Net

A flow net is a representation of a family of streamlines and equipotentials in a two-dimensional plane. A streamline is a path of flow of an individual particle through the soil. An equipotential is a line drawn through all points in the soil having the same potential. Streamlines and equipotential lines are related in that the potential function and stream function are connected by means of the Cauchy-Riemann differential equations:

$$\frac{\partial \phi}{\partial x} = \frac{\partial \psi}{\partial y} \qquad\qquad \frac{\partial \phi}{\partial y} = -\frac{\partial \psi}{\partial x}$$

where ϕ is the hydraulic head and ψ is the stream function. Hence, the stream function and hydraulic head function are both solutions of Laplace's equation for two-dimensional flow, and the streamlines and equipotential lines representing the flow net are orthogonal in an isotropic soil.

Flow into Drains

Prior to 1911, there seemed to be different views on how water flowed into the drain tile. Waring (25) states that "There seems to remain in the minds of many writers on drainage a glimmering of the old fallacy that underdrains, like open drains, receive their water from above, and it is too commonly recommended that porous substances be placed above the tile. If, as is universally conceded, the water rises into the tile from below, this is unnecessary." There is now a general agreement that, under a saturated condition, water flows to an underdrain approach from around the entire perimeter, as shown in Figure III. Monke (14) described in detail the flow pattern around a partially filled drain. He called this flow pattern a surface of seepage. Water approaching the top and sides of a drain does not drop into the drain but enters the surface of seepage because the force into the drain is less than the opposing force of surface tension. The flow follows along the surface of seepage downward because of the gravity force and breaks away to enter the stream of flow in the drain when the surface of seepage intersects the water level in the drain.

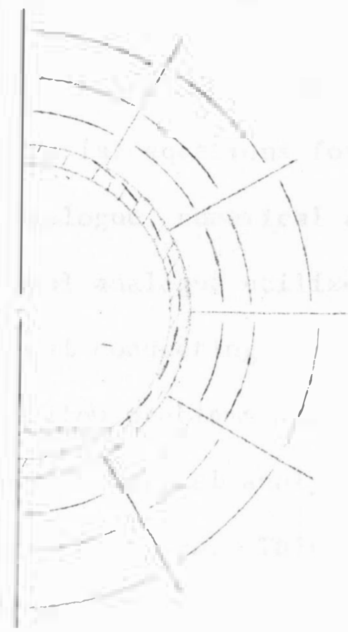

Figure III. Surface of Seepage Around the Periphery of a Nearly Empty Tile Drain

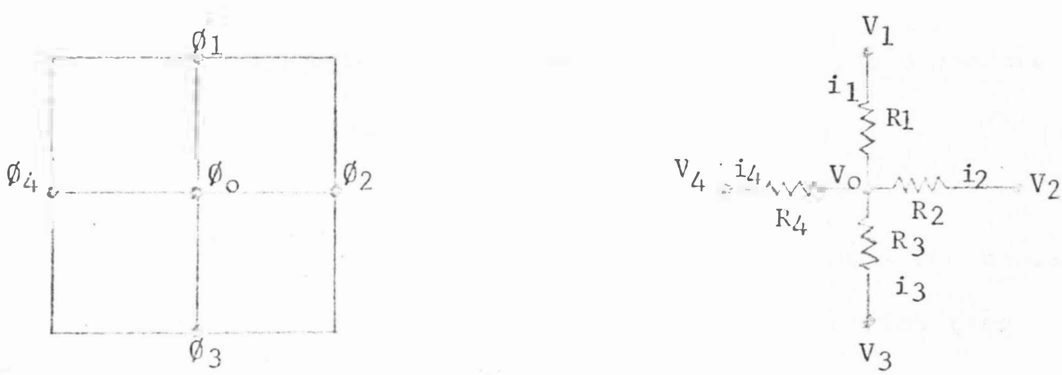

Figure IV. Hydraulic Potentials at Corners of a Square Grid Being Represented by a Portion of the Resistance Network

Analog Model of Tile Drainage

There are three analog methods (12) besides a tedious mathematical analysis of differential equations for obtaining a flow net. These are the electrical analogue, numerical analysis, and the resistance network. The electrical analogue utilizes the flow of electricity through a sheet of electrical conducting paper or fluid. This method is limited to steady-state flow problems where soils are saturated, isotropic, and homogeneous. Numerical analysis utilizes an iterative procedure to solve Laplace's equation. This method can solve flow problems involving nonuniform hydraulic conductivities as well as irregular boundaries. Numerical analysis is particularly adaptable to the digital computer. The resistance network combines the principles of numerical analysis and the electrical analogue to solve a wide variety of flow problems for the steady-state condition as well as the transient condition. An important advantage of the resistance network is the instantaneous relaxing of the entire net, a procedure requiring many hours of work using numerical analysis.

Resistance Network

The similarity between Ohm's law and Darcy's law forms the basis for the analogy between electrical flow and ground water flow (see Luthin (12) for detailed discussion). Ohm's law can be expressed as

$$I = V/R$$

where I is current in amps, V is voltage in volts, and R is resistance in ohms. Since in Ohm's law the conductance K^1 is the reciprocal of resistance, the law can be written as

$$I = K^1 V$$

Conductance K^1 varies directly as the specific conductivity k^1 and the area, A, and inversely as the length, L, then $K^1 = k^1 A/L$ from which one can write

$$I = k^1 \frac{V}{L} A$$

This is similar to Darcy's law which can be written as

$$Q = K \frac{\emptyset}{L} A$$

where Q is flow, K is hydraulic conductivity, $\emptyset/L$ is hydraulic gradient, and A is cross-sectional flow area. The hydraulic conductivity is related to the reciprocal of resistance, and the voltage can be used to represent hydraulic head or potential.

The resistance network uses a network of resistors having a finite resistance to represent a soil profile, as shown in Figure V. Looking at a node of four equal resistances shown in Figure IV and applying Kirchoff's law, the algebraic sum of the currents at a junction equals zero, the current at point V_0 is

$$i_1+i_2+i_3+i_4 = 0$$

Using Ohm's law to write the current in terms of resistance and voltage, the equation becomes

$$\frac{V_1 - V_0}{R_1} + \frac{V_2 - V_0}{R_2} + \frac{V_3 - V_0}{R_3} + \frac{V_4 - V_0}{R_4} = 0$$

Where all values of resistances are equal (representing a homogeneous soil mass), the voltage V_0 is the following:

$$V_0 = \tfrac{1}{4}(V_1+V_2+V_3+V_4)$$

This equation is exactly analogous to the potential formula, $\emptyset_0$, used in the numerical analysis solution of Laplace's equation for two-dimensional flow (see Luthin (13)). This basic formula for the numerical analysis method is

$$\emptyset_0 = \frac{1}{4}(\emptyset_1+\emptyset_2+\emptyset_3+\emptyset_4)$$

In a network study, there are three types of solutions which can be obtained: (1) equipotential lines, (2) streamlines, and (3) flow rate; or, in the case of drainage, the amount of water that can be removed during a given time. A voltmeter is used to measure potential at each node, and the equipotential lines can be drawn by interpolating between potentials. To obtain streamlines, the boundaries are now reversed; then the potentials are again measured. To determine the flow rate, the amount of current passing through the circuit is measured.

Boundary Conditions

Vimoke and Taylor (24) describe the calculating and assembling of a resistance network by the "building block method". Detailed procedures are given in this report on representing a rectangular block of soil, representing a circular drain, and representing a stratified soil condition. The boundary conditions which apply to this thesis will be discussed. These are representing an impermeable boundary, a ponded water condition, and a surface of seepage around a drain.

Luthin (11) described how points on an impermeable boundary can be represented. On an impermeable boundary the first derivative of the hydraulic head taken normal to the impermeable boundary is equal to zero. The average cross-sectional area of the flow section on an impermeable boundary is one-half that of the flow section in the interior. Since the cross-sectional area of the flow section is one-half and resistance is inversely proportional to the area of flow, the resistance on the boundary must be twice the resistance of the interior. Therefore, the resistances on the impermeable boundaries are twice the value of the interior resistances.

A ponded water condition is essentially a water table at the ground surface. Luthin (12) defines a water table as "the locus of points of atmospheric pressure." In ground water flow, atmospheric pressure is taken as the datum point where pressure is zero. Since hydraulic head or potential is the sum of the pressure head and the gravitational head, the horizontal water table represents an equipotential line where the hydraulic potential, $\emptyset_n$, at the ground surface is equal to the elevation above the reference plane, h_n

$$\emptyset_n = h_n$$

Figure V shows how the voltage is applied to the resistance network for a ponded water condition.

Monke (14) described how water flows as a surface of seepage around the drain until this surface of seepage intersects the water level in the drain. Luthin (11) describes how this surface of seepage above the water level in the drain can be taken as the interface between the saturated soil and the free atmosphere. Again the hydraulic head, which is the sum of the pressure head and the gravitational head for the saturated case, would be equal to the gravitational head over the

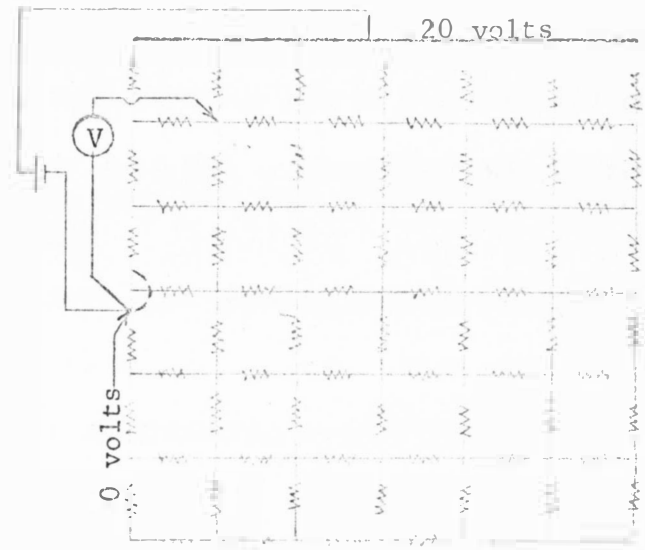

Figure V. A Resistance Network for a Ponded Water Condition to Determine Equipotentials

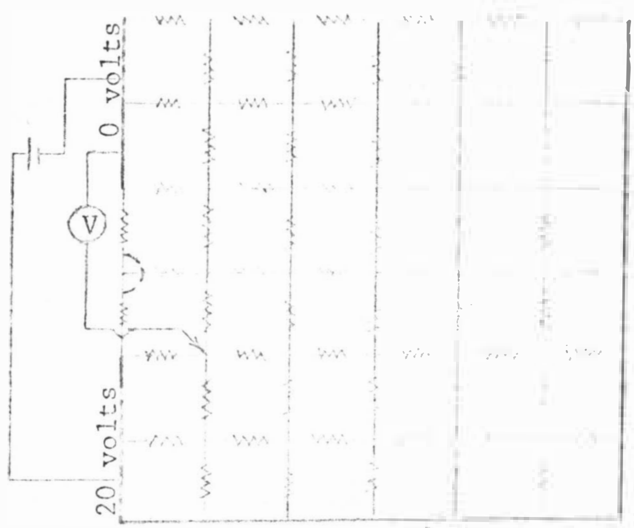

Figure VI. A Resistance Network for a Ponded Water Condition to Determine Streamlines surface of seepage. Here, the numerical value for the hydraulic head or potential head at any point on the surface of seepage would be equal to the vertical distance of the point around the drain above the reference plane from which hydraulic head is calculated.

Accuracy of Results

Thiel (18) lists four primary sources of error in solving a field problem with the resistance network. They are:

1. Errors in representing an electrical equivalent of the field problem.
2. Errors resulting from inaccurate boundary definitions.
3. Electrical errors, due primarily to deviations of the individual resistor values from their specified magnitudes and to measuring errors.
4. Possibility of error in interpolating and drawing the flow net.

The first and second sources of error are probably the most limiting since it is extremely difficult to obtain accurate definitions of parameters and boundaries of a complex prototype field. Errors in representing an electrical equivalent can depend upon the net spacing. In the resistance network as well as the numerical analysis, the assumption was made that the potential is a linear function in two dimensions. This assumption is accurate for the parts of the profile distant from the tile drain but becomes less accurate close to the drain. Therefore, a finer grid system is employed near the drain.

Electrical errors are usually small, probably less than 1% for network resistors of $\pm$ 1%. Measuring errors also are usually small. Luthin (11) states that over-all accuracies of 1% to 2% are not unreasonable to expect.

OBJECTIVE OF THE RESEARCH

A gravel envelope is required to facilitate ground water entry into the drain, prevent erosion of the base material, and serve as a stabilizing foundation for the drain. The typical gravel envelope for a tile drain in the Oahe Irrigation Unit will be embedded in a course silt base material which could prove to be a major hazard to the life of the tile-drainage system. The immediate objective of this research is to study in detail the design criteria for a nonuniform (pit-run) gravel envelope as proposed in the drainage of the Oahe Irrigation Unit. Some of the questions needed to be answered are as follows:

1. Is a gravel envelope necessary for adequate protection of the tile drain in this case?
2. Does the proposed thickness of gravel envelope meet the requirements of a successful envelope?
3. Would a larger gravel envelope thickness effectively increase the flow rate and provide more protection for the tile drain?
4. What is the maximum physically noticed penetration and concentration of base material into the gravel envelope?
5. What effect would time have on the flow rate and sediment discharge from the tile drain?
6. Would the gravel envelope remain completely saturated at all times in a field situation?

EXPERIMENTAL APPARATUS AND PROCEDURES

This section will be divided into four areas of discussion: plan of experiment, testing of materials, resistance network analog, and tile drain model. The problem is limited and examined in the plan of experiment section. The experimental apparatus and procedures used in this investigation are described in the testing of materials, resistance network analog, and tile drain model sections.

Plan of Experiment

The problem, as presented in the section entitled "Objective of the Research" is to study in detail the design criteria for a non-uniform gravel envelope as proposed in the drainage of the Oahe Irrigation Unit.

From the review of literature it is evident that design recommendations for gravel envelopes have been made by many investigators. However, most of the gravel envelope research has been with a cylinder model and fine sand as the base material. Using a cylinder model restricts the study to only vertical flow of water through the pores of the gravel envelope; whereby, questions concerning the hydraulics or flow convergence near the tile drain cannot be answered. It is the author's opinion that flow patterns near the drain are important to the design of a gravel envelope for a loose, coarse silt base material.

The prototype field selected for this investigation was the field drainage plot at the Redfield Irrigation Farm, Redfield, South Dakota. The drainage plot was selected because of its well-defined parameters and boundary conditions. Hydraulic conductivity measurements have been made by a number of methods and compared to tile outflow from the drainage plot for the severe case of a ponded water surface. A plastic barrier separating the drainage plot from the border and a well-defined, impervious boundary at a depth of 13 feet provide ideal boundary conditions. Figure VII shows the stratified soil layers in a tile trench resembling the soil profile found on the drainage plot; the bottom layer of coarse silt is where the tile drain is embedded in the field. The rather shallow soil profile and the coarse silt base material give cause for a hazardous drainage problem.

After considering the problem and the selected prototype field, this investigation was limited to a study of the following situations:

1. A single nonuniform (pit-run) gravel material, similar to the envelope originally used for the drainage plot.
2. A single base material of coarse silt, as found below the 7-foot depth in the drainage plot.
3. A ponded water surface condition, providing a severe test case for the envelope in the field.

The experimental plan called for study in three areas: preliminary testing of materials, resistance network analog, and a tile drain model. The preliminary testing of materials included a mechanical analysis of the base material and the gravel envelope material, a permeability test of the base material and the gravel envelope material, and a bulk density test of the base material in the field. The resistance network analog predicted the flow net as well as tile outflow for five methods of installation--no gravel

Figure VII. Tile Trench at Redfield Irrigation Farm Showing Stratified Soil Layers in the Field envelope and gravel envelopes of 3, 6, 9 and 12-inch thicknesses. The tile drain model was constructed in the laboratory permitting inquiry into the necessity of the gravel envelope, the penetration and concentration of sediment in the gravel envelope, and the hydraulic properties of the gravel envelope.

Testing of Materials

One and one-half tons of coarse silt and a total of 50 gallons of gravel were procured from the Redfield Irrigation Farm, Redfield, South Dakota. The coarse silt material was obtained from a pile where the silt had been removed earlier at a depth of 7 to 13 feet below the surface. The gravel material was obtained from two other piles, which came from the same pit south of Redfield from which the original gravel material for the drainage plot was obtained. A mechanical analysis was made of both materials to determine the particle size relationship existing between the materials. A disturbed permeability test was made for the gravel material to determine a saturated hydraulic conductivity value that could be used in the resistance network analog, and a disturbed permeability test was made for the coarse silt material to determine what hydraulic conductivity could be anticipated in the tile drain model. Also, a bulk density test was made of the coarse silt in the field to determine the necessary compaction in the tile drain model. Care was taken in all these tests to use only clean, well-mixed material.

Mechanical Analysis

A gravel envelope is usually designed on the basis of determining the mechanical analysis gradation curve of the base material and selecting a gravel material to meet the design gradation requirements. In this investigation there was no attempt to alter or grade the pit-run gravel material to give satisfactory performance, for indications were that the pit-run gravel material would fall within design requirements. To determine the gradation curves for the two materials, a sieve analysis was made for the sand and gravel; and a combined sieve and hydrometer analysis was made for the coarse silt material. The grain-size analysis test procedure was based on recommendations in "Soil Testing for Engineers" by Lambe (8). Figure VIII shows the nest of sieves, the motor-driven shaker, and the torsion balance used in the sieve analysis.

Sieve Analysis Procedure for Sand and Gravel

1. Two or three kilograms of the material were oven-dried.
2. Each sieve, thoroughly cleaned, was weighed to 0.1 gram.
3. A dried sample between 500 to 1000 grams was weighed to 0.1 gram.
4. The sample was sieved through a nest of sieves on a motor-driven shaker for 3 to 5 minutes.
5. The material retained on each sieve and the pan was weighed to 0.1 gram.
6. The weight of the soil retained on each sieve was determined by subtracting the weights in step 2 from step 5.
7. The percentage retained on each sieve, the cumulative percentage retained on each sieve, and the percent finer by weight for each sieve were calculated by the following:

   a. Percent retained = $\frac{\text{weight of soil retained}}{\text{total soil weight}} \times 100\%$

   b. Cumulative percentage retained = sum of percentages retained on all coarser sieves.

   c. Percent finer = 100% - cumulative percentage retained.

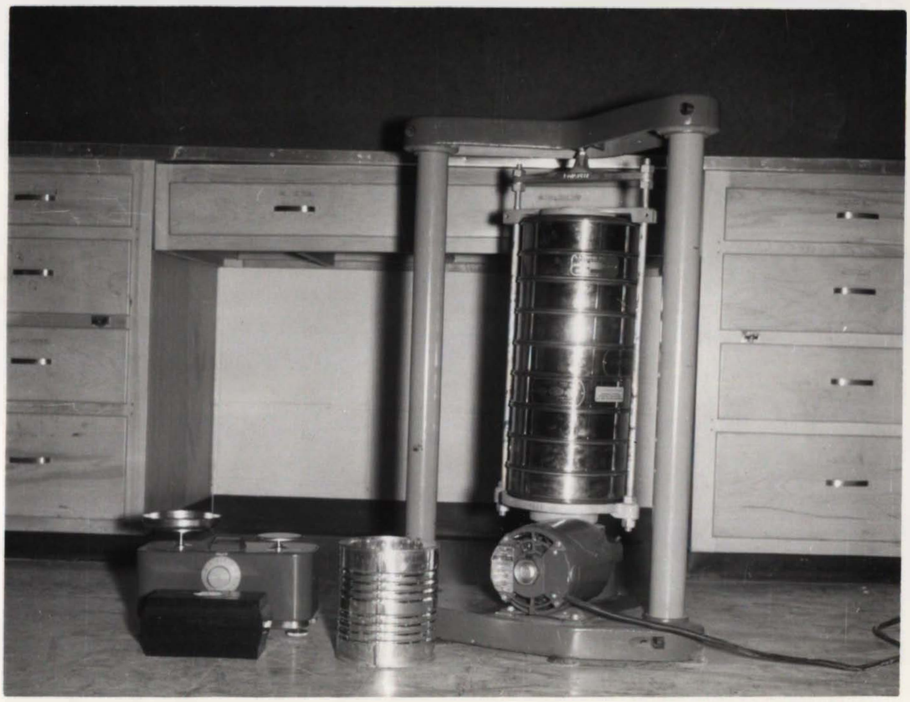

Figure VIII. Sieve Analysis Equipment

Combined Analysis Procedure for Coarse Silt

1. A sieve analysis was taken as outlined for the sand and gravel material.
2. The No. 200 sieve was washed to obtain the total amount of soil finer than the No. 200 sieve.
3. A suspension of one liter was made by adding distilled water to approximately 50 grams of the dry soil retained in the pan.
4. After the suspension was shaken for 30 seconds, hydrometer readings were taken at ¼, ½, 1 and 2 minutes without removing the hydrometer.
5. The suspension was remixed and readings were taken at time intervals of 2, 5, 10, 20 minutes, etc., inserting the hydrometer at each of these times.
6. Temperature observations and meniscus corrections for the hydrometer were taken periodically.
7. The percent finer for the hydrometer, the effective diameter for the hydrometer readings and the corrected percent finer for the combined analysis were calculated by the following:

(a.) $$N = \frac{G_s}{G_s - 1} \frac{\bar{V}}{W_s} \gamma_c (r - r_w) \times 100\%$$

where:

N = percent finer by weight for the hydrometer
G_s = specific gravity of solids
$\bar{V}$ = volume of suspension (1000 cm^3)
W_s = weight of dry soil
γ_c = unit weight of water at 20°C
r = hydrometer reading in suspension
r_w = hydrometer reading in distilled water

(b.) $$D = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w}} \sqrt{\frac{Zr}{t}}$$

where:

D = effective diameter of the hydrometer readings
μ = viscosity of water at test temperature
γ_s = unit weight of soil grains
γ_w = unit weight of water at test temperature
Zr = distance from surface of suspension to center of hydrometer volume obtained from the calibration chart
t = total elapsed time

(c.) $N^1 = N \times \frac{W^1}{W_s}$

where:

N^1 = percent finer than No. 200 sieve
N = percent finer obtained for hydrometer
W^1 = weight of dry soil passing No. 200 sieve
W_s = total weight of dry soil used for sieve analysis

Permeability Tests

A plexiglass, constant head, cylindrical permeameter was used to determine the disturbed hydraulic conductivities of the coarse silt base material and the gravel envelope material. The test procedures used for the two materials were somewhat different because of the application of their results.

A representative hydraulic conductivity measurement of the gravel envelope material was obtained using the apparatus shown in Figure IX. A completely saturated hydraulic conductivity value was needed to represent the gravel envelope in the resistance network analog; therefore, the entrapped air in the pores of the material need be removed. Entrapped air can be removed from the porous material over a period of time by the passage of de-aired water through the sample. This requires considerable time to accomplish (3). Carbon dioxide can also be used to remove soil air. Carbon dioxide is slowly introduced before wetting the sample; then upon percolating water through the soil, the carbon dioxide will be removed being readily soluble in water. The initial permeability of a carbon dioxide treated soil will be approximately equal to the maximum permeability of an untreated sample (4).

Determining a saturated hydraulic conductivity value for the gravel material was accomplished in the following described manner. A

Figure IX. Constant Head, Cylindrical Permeameter Used to Determine Permeability of Gravel Material vacuum pump was connected to the top of the permeameter, closing all other valves and clamps. After the vacuum was applied for 10 to 15 minutes, carbon dioxide gas was introduced very slowly at the bottom of the permeameter while the column was still evacuated of air. When a sufficient volume of carbon dioxide gas had reduced the vacuum gauge pressure close to zero, both the vacuum line and the carbon dioxide line were clamped off. Next de-aired, distilled water from a Mariotte bottle of sufficient head was introduced by capillary action into the bottom of the permeameter to saturate the sample and to dissolve the remaining carbon dioxide. Finally de-aired, distilled water was started from the constant head tank on the top of the permeameter, and the amount of water flowing through the sample was measured by timing the volume of flow into a graduated cylinder. Observations were taken every 10 minutes until a gradual decrease in the flow was noticed. The hydraulic conductivity was calculated by rearranging Darcy's law and correcting for temperature.

$$K = \frac{Q\,L}{A\,\emptyset}\;\frac{\mu}{\mu_c}$$

where

K = hydraulic conductivity (lt^{-1})

Q = volume of water per unit time (l^3t^{-1})

A = cross section of flow area (l^2)

L = sample length (l)

$\emptyset$ = head loss or hydraulic potential difference (l)

μ = viscosity of water at temperature of test

μ_c = viscosity of water at temperature 20°C

A hydraulic conductivity measurement of the coarse silt base material was obtained using the apparatus shown in Figure X. The purpose of this measurement was to determine what hydraulic conductivity value could be expected from the disturbed silt material in the tile drain model. Two trials were made--an unchanged silt sample and an oven-dried and ground silt sample. Since conditions were to represent those in the tile drain model, there was no need of a vacuum, carbon dioxide or de-aired water supply. Instead, the permeameter sample was saturated by only capillary flow from the bottom of the permeameter. A Mariotte bottle was adjusted to supply a constant head of water below the sample surface in the permeameter eliminating the problem of smearing the soil pores on the sample surface. This saturation process was continued overnight. Then, flow measurements and hydraulic conductivity calculations were made as described for the gravel material.

Field Density

A bulk density test was made for the coarse silt base material in the field to determine the necessary compaction in the tile drain model. Three such measurements were taken in a tile trench at the Redfield Irrigation Farm, Redfield, South Dakota; two measurements were taken on the bottom of the trench, and one measurement was made on the side of the trench. The simplified field procedure was: level an area in the trench, auger a small hole, measure by volume the amount of coarse gravel needed to fill the hole, and oven-dry the soil obtained from the hole. The bulk density is the dry weight of soil divided by the volume

Figure X. Constant Head, Cylindrical Permeameter Used to Determine Permeability of Base Material of soil for the small auger hole.

Resistance Network Analog

The resistance network analog uses a network of resistors having a finite resistance to represent a soil profile. The prototype field selected for this investigation was the field drainage plot at the Redfield Irrigation Farm, Redfield, South Dakota. Figure XI shows the resistance network analog constructed on three tag boards in the laboratory to predict the flow net and tile outflow for five methods of installation--no gravel envelope and gravel envelopes of 3, 6, 9 and 12-inch thicknesses. The analog model represents a one-half cross section of the drainage plot, since flow patterns on one side of a plane through the center of the tile drain would be symmetrical to the other side.

As previously mentioned, Vimoke and Taylor (24) described calculating and assembling of a resistance network by the "building block method". These procedures were followed in this investigation. Figure XII shows the one-half cross section of the drainage plot with the various soil layers of different hydraulic conductivity and the resistance values calculated to represent the hydraulic conductivities. A resistance of 12,400 ohms was chosen as the characteristic resistance value for the network and the base material. This resistance value was selected for three reasons: the availability of resistors, a value considerably smaller than the internal resistance of the measuring equipment, and a value considerably larger than the contact resistance in the many wires connecting the jack and plug assemblies. Figures

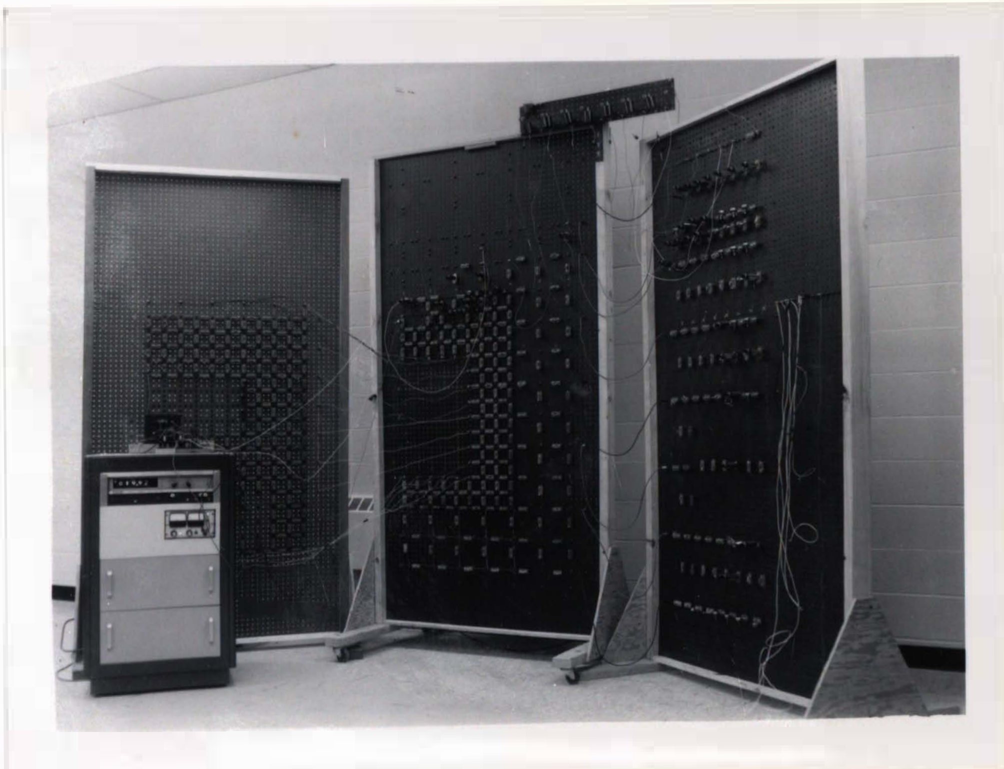

Figure XI. Resistance Network Analog Simulating Water Flow on Redfield Drainage Plot

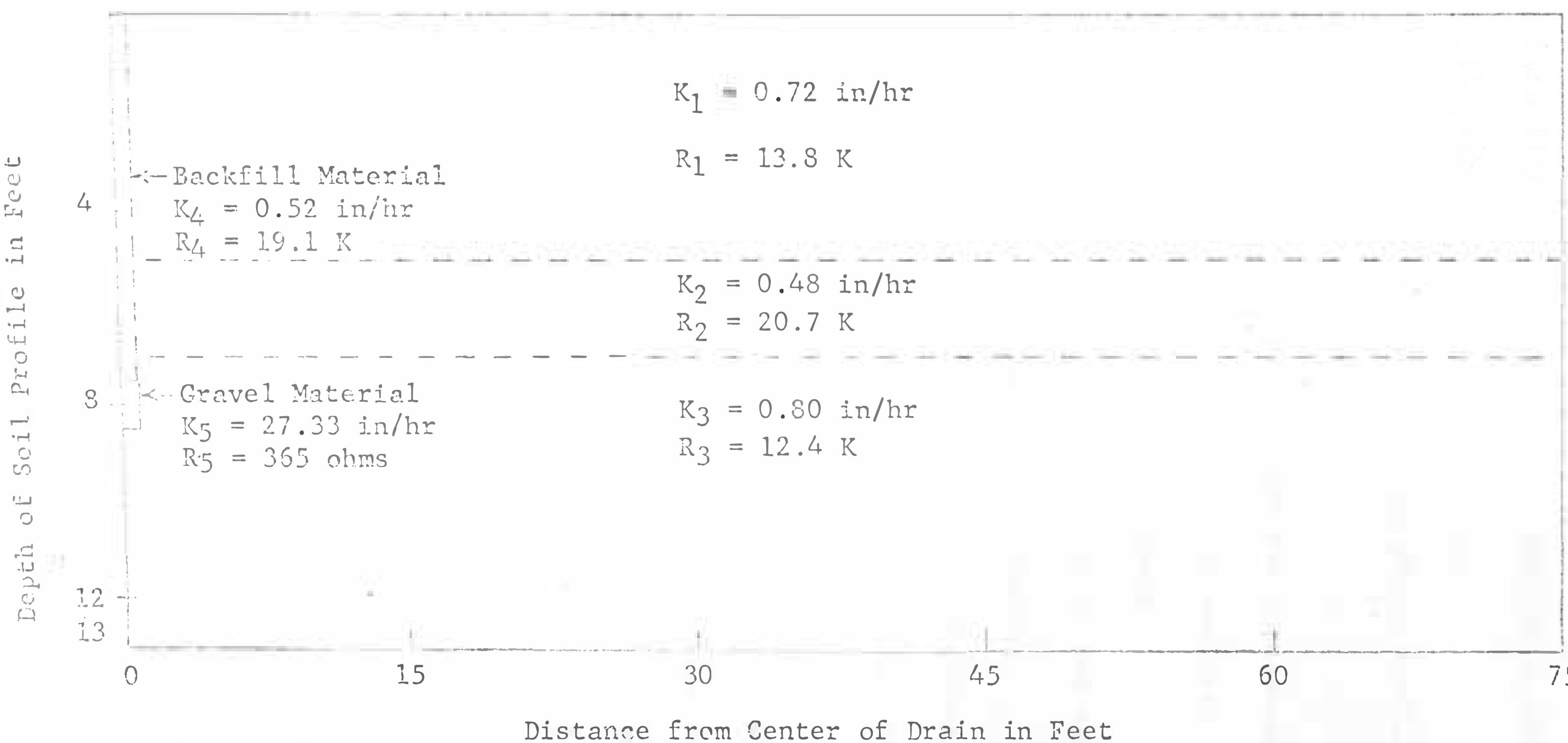

Figure XII. One-Half Cross Section of the Redfield Drainage Plot Showing Resistance Values Representing Various Soil Layers of Different Hydraulic Conductivity

XX-XXII in Appendix B show the assembled resistors representing the soil profile; each figure corresponds to a resistance network board constructed in the laboratory.

The following assumptions concerning boundary conditions for the resistance network analog were made:

1. No contribution to the flow beyond the impermeable boundaries.
2. A ponded water condition on the ground surface.
3. An empty drain with a surface of seepage.
4. A completely saturated gravel envelope of the same thickness on the top, bottom and sides of the drain.

Employing these boundary conditions, three solutions were obtained for the five methods of installation studied: (1) equipotential lines, (2) streamlines, and (3) tile outflow. The power supply used to establish the boundary conditions had an output voltage of 0-30 volts, a current regulation of $\pm$ 0.15%, and a voltage regulation of $\pm$ (0.01% + 1 mv).

To obtain equipotential lines, 20 volts were applied to the top boundary. The bottom of the drain was connected to the ground as shown in previous Figure V. Voltages from a voltage potentiometer were inserted around the periphery of the drain to represent a surface of seepage. The unconnected boundaries represented impermeable boundaries. A digital voltmeter with a range, $\pm$ 1099.9 volts, and a voltage accuracy, $\pm$ 0.01% of the reading and $\pm$ 0.01% of the full scale reading was used to measure the voltage at each node in the resistance network. Then, the voltages obtained at each of the nodes were interpreted in terms of hydraulic potentials, and the equipotential lines were drawn by interpolating between the potentials.

To obtain streamlines, the boundaries were "reversed" as shown in previous Figure VI. This time 20 volts were applied to the previously unconnected boundaries. The voltages were again measured using the digital voltmeter. The voltages obtained at each of the nodes were interpreted in terms of percentage of flow, and the streamlines were drawn by interpolating between these percentages. The outlined procedure for obtaining streamlines was actually for a full drain instead of an empty drain. In order to have obtained streamlines for an empty drain, the procedure would have been to sketch orthogonal lines to the equipotentials after knowing the current flow distribution on the ground surface and around the drain. Making current measurements around the drain would require an extremely fine net to obtain a detailed description of the current flow. Monke (14) stated that there was no particular difference in flow patterns for an electrical or physical model when the drain was allowed to empty. As a check on using streamlines for a full drain rather than an empty drain, the equipotential lines were obtained for the full drain as well as the empty drain. The potential for a drain running full with no back pressure would be equal to the radius of the drain where the reference plane is through the center of the drain parallel to the soil surface. The potential for an empty drain would be equal to the vertical distance of any point around the drain above the reference plane. The results showed little difference in the potential pattern for a full drain in comparison to an empty drain.

To determine the flow rate per foot of tile drain, the amount of current passing through the network was measured. This was accomplished by plugging a high-precision shunt across the digital voltmeter in series with the resistance network. The shunt used for all the measurements was a 10-ohm resistor with an accuracy of $\pm$ 0.01% plus the digital voltmeter. The selected shunt measured millivolts on the 100 millivolt scale having a numerical value equal to a current flow of 10 milliamps. The amount of current flow was measured for both an empty and full drain for the five methods of installation studied. Using the following equations (24) tile outflow was calculated:

$$C_o = \frac{V}{\phi_n - \phi_d}$$

where

C_o = conversion coefficient (volts/foot)

V = applied voltage (20 volts)

ϕ_n = potential at top boundary (feet)

ϕ_d = potential at the drain (feet)

$$Q' = \frac{2I\ K\ R_o}{C_o}$$

where

Q' = flow rate per foot of drain (cubic feet/day/foot)

I = current through the network (amps)

K = hydraulic conductivity represented by R_o (1.6 ft/day)

R_o = characteristic resistance for the network (12,400 ohms)

C_o = conversion factor (volts/foot)

Tile Drain Model

A tile drain model was constructed in the laboratory permitting inquiry into the necessity of the gravel envelope, the penetration and concentration of sediment in the gravel envelope, and the hydraulic properties of the gravel envelope. A unique aspect of this model was the inserting of a single predicted equipotential line to study the flow of water through the pores of the gravel envelope in all directions. Previous investigators such as des Bouvrie (5) and Sisson (16) used a rectangular box or cylinder model where the water was ponded on the surface in the model. In their studies, the flow was not represented from all directions in the proportions found in the field. Considering the exploratory nature of the model constructed in the laboratory for this thesis, only two methods of installation, no gravel envelope and a six-inch gravel envelope, were studied. The six-inch gravel envelope was chosen because this thickness was placed on the field drainage plot and recommended in the 1960 Oahe Unit Report.

The tile drain model, as sketched in Figure XIII, has outside dimensions of 4 feet by 4 feet and a depth of 2 feet. The model has two compartments, a front compartment for soil placement and a back compartment for water control. A 3/4-inch plexiglass plate, with 12 piezometer taps arranged in a rectangular pattern, was bolted to the front of the model. Six of the piezometer taps on one side of the plexiglass plate were placed flush with the soil; the other six taps on the opposite side of the plate were attached to ceramic cups inserted into the soil. Soldered in the divider separating the compartments were

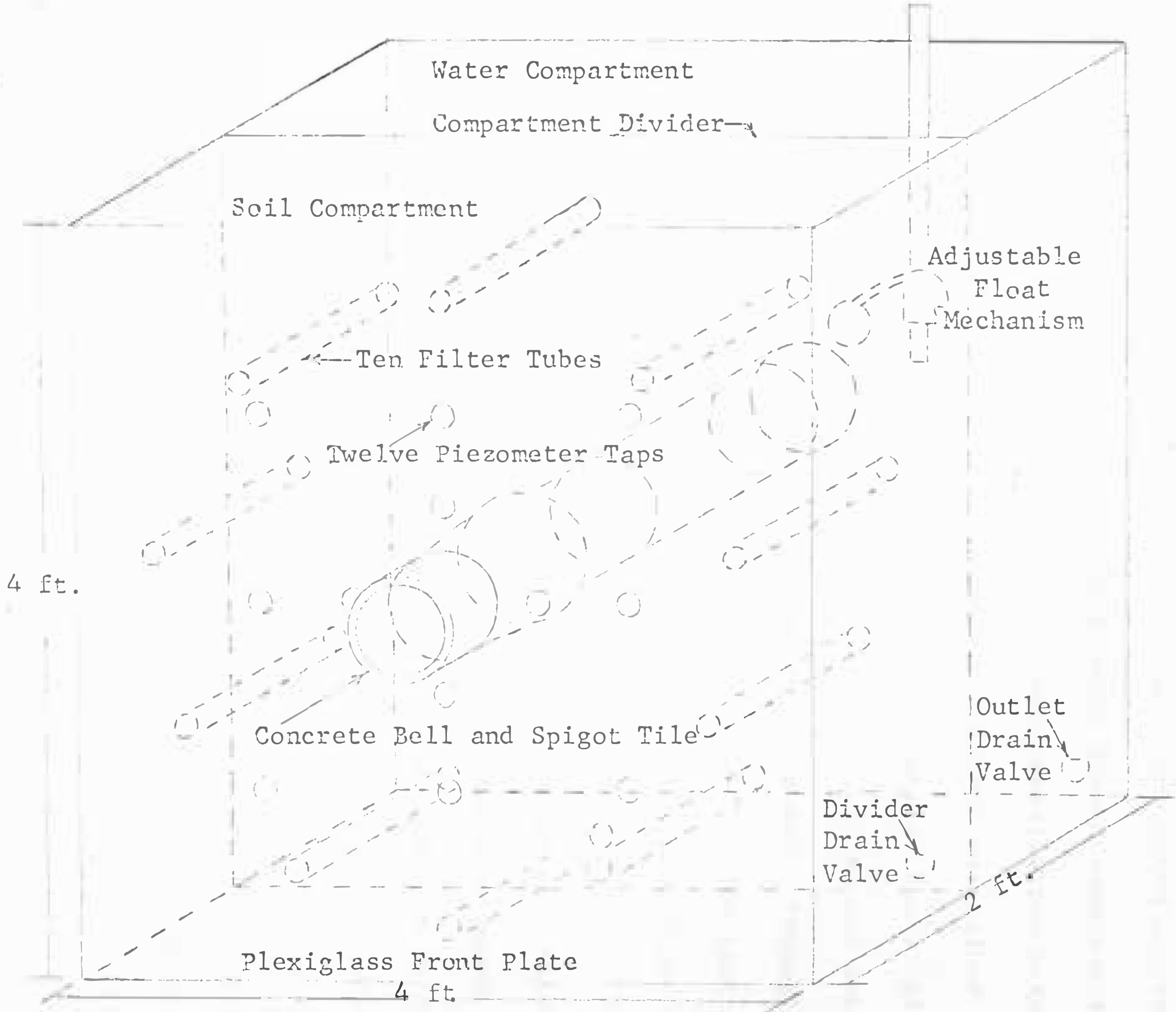

Figure XIII. Schematic Drawing of Tile Drain Model

$\frac{1}{4}$-inch, brass couplings. These brass couplings were spaced according to the streamline pattern on a single equipotential line predicted by the resistance network analog. A different equipotential line was used for each of the two installations studied. Connected to these couplings were filter tubes through which water flowed into the soil compartment. Ten filter tubes represented each installation studied. An adjustable float mechanism in the back compartment maintained the required head of water in the filter tubes to establish the single, predicted equipotential line in the soil. Plugs could be put into the couplings whenever the filter tubes were not in use. Through the center of the tile drain model a 6-inch diameter bell and spigot concrete tile was permanently fixed leaving a 1/8 to 1/4-inch joint spacing centered in the soil compartment.

The original test procedure for placement of materials, saturation of the porous media, and maintenance of flow resulted in a number of problems. First, the materials were placed and saturated in 3-inch layers. It was expected that the coarse silt material would compact itself to field density when saturated in small layers. As the 3-inch soil layers were placed in the front compartment of the model, water from the back compartment, through a valve in the bottom of the compartment divider, was allowed to rise in the soil layers. This process continued until saturation reached the tile drain. Then, with plexiglass covers over the concrete tile ends, water from a 25-foot head tank was forced back through the tile opening in order to saturate the soil layers above the tile drain. The saturating of the soil in layers caused smearing of the soil pores between the coarse silt layers. Also, the forcing of water into the model with a high head caused a surging which moved the smaller particles into the gravel envelope as the head of water was removed.

An additional problem was the method for supplying water into the soil compartment. Originally, ceramic filter candles were connected to the brass couplings to supply water into the soil. However, these ceramic candles were not sufficiently permeable to maintain the required flow of water. To replace the filter candles, filter tubes were designed. These filter tubes were made from plastic PVC pipe. The PVC pipe was longitudinally slit three times around the circumference of the pipe. Then, the slotted pipe was wrapped with glass-fiber sheets ("Tileguard") to prevent sediment clogging. These filter tubes efficiently maintained the required flow of water.

The corrected test procedure began with the placement of materials in the tile drain model. A nearly dry sample of coarse silt was weighed and compacted in 3-inch layers to the previously determined field density. Very little packing was necessary to obtain the required bulk density for the coarse silt base material. When the gravel envelope was compacted in the model, the final bulk density of the permeability test of the gravel was duplicated. The ten filter tubes, spaced on a single predicted equipotential line for the installation being studied, were placed as the soil was compacted. After the placement of materials, flow was started in the filter tubes leaving the plexiglass covers over the concrete tile ends. The soil model was allowed to saturate for 10 to 12 hours. The plexiglass covers were then removed and flow commenced from the tile drain model.

The corrected test procedure was repeated for the two methods of installation--no gravel envelope and a 6-inch gravel envelope. Figures XXIII and XXV in Appendix C show with a double line the equipotential line predicted by the resistance network analog and replicated in the tile drain model for each of the installations studied.

Three observations were ascertained from the tile drain model: water discharge, sediment discharge, and hydraulic head or potential head distribution. Water discharge was determined by two volumetric methods. First, the volume of flow into a graduated cylinder was timed periodically. Secondly, the flow rate was determined using a tipping bucket where an Esterline Angus event recorder indicated the number of times the bucket tipped for a given time. Sediment discharge samples were taken whenever a noticeable amount of sediment appeared in the tile outflow. Then an additional discharge sample was taken whenever the sediment ceased in the tile outflow. These discharge samples were taken at the same time as the graduated cylinder flow measurements. To determine the amount of sediment, a total solids analysis was performed on the discharge samples containing sediment; and a dissolved solids analysis was performed on discharge samples containing no sediment. The solids analysis was performed with a 100-milliliter sample of the discharge by weighing the residue upon evaporation over a water bath and drying at 105° C in a porcelain dish. The rate of sediment discharge was calculated by multiplying the amount of sediment, total solids minus dissolved solids, times the flow rate at the time of sampling. To determine the hydraulic potential representation in the model, the twelve piezometer taps on the plexiglass front were connected by clear vinyl tubing to an inverted manometer board. The adjustable, inverted manometer measured the head of water for both positive and negative pressures. These hydraulic head or piezometric head readings were made periodically along with the graduate cylinder flow measurements.

RESULTS OF TESTS

The primary objective of this investigation was to study in detail the design criteria for a nonuniform (pit-run) gravel envelope as proposed in the drainage of the Oahe Irrigation Unit. Some questions were asked concerning the necessity, required thickness, sediment movement, and hydraulic properties of the proposed gravel envelope.

Data, analysis, and discussion are found in the following sections.

Properties of Test Materials

A mechanical analysis was made of the base and gravel materials. Figure XIV presents the grain-size distribution curves for the materials. Shown in dashed lines on the same figure are upper and lower limit curves based on design criteria recommendations by the U.S.D.A. Soil Conservation Service (19, and on page 9 in this thesis). The following Filter-Aquifer ratios were calculated for the test materials:

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{Aquifer}} = 78$$

$$\frac{D_{15}\ \text{Filter}}{D_{15}\ \text{Aquifer}} = 42$$

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{Aquifer}} = 9.1$$

where D_{15}, D_{50}, and D_{85} are the respective 15%, 50%, and 85% finer grain size from the gradation curves of the materials. It is worth noting that the grain-size distribution curves are often only approximate. Lambe (8) discusses some of the reasons why the mechanical

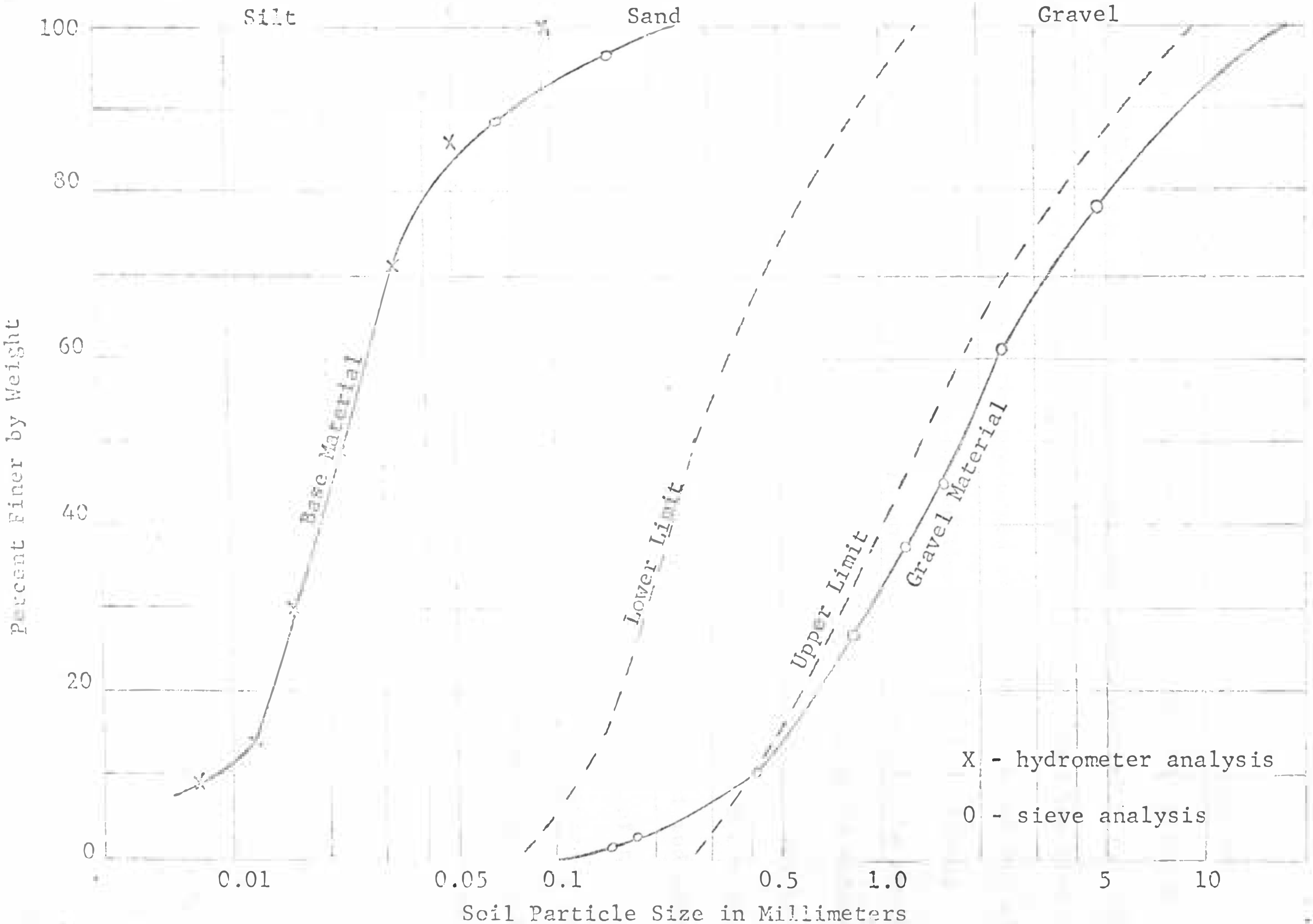

Figure XIV. Grain-Size Distribution Curves of Base and Gravel Material Along with Upper and Lower Limit Curves for Gravel Selection analysis measurements in the laboratory are more questionable for the fine-grained soils than for the coarser materials.

Comparing the mechanical analysis gradation curves with the design criteria recommendations, it was concluded that the pit-run gravel material may have been coarse for the base material. The 15% size (D_{15}) of an envelope material should not be greater than five times the 85% size (D_{85}) of the protected soil. The ratio of D_{15} of the filter to D_{85} of the soil is called the piping ratio. Piping occurs when a large amount of soil is washed in or through the envelope. Cedergren (2) indicates that the piping ratio may be up to 10 before appreciable amounts of soil will move through the envelope. But with a piping ratio above 10, erosion is very likely to occur. The piping ratio for the envelope material in this investigation would indicate the possibility of an erosional failure. The ratio of D_{15} of the filter to D_{15} of the soil is to guarantee sufficient permeability to prevent the buildup of large seepage forces and hydrostatic pressures in filter and drains. The ratio of D_{50} of the filter to D_{50} of the soil is to ensure that grain-size curve for the envelope material is somewhat parallel to the base material. In all three criteria, the Filter-Aquifer ratios were somewhat larger than design recommendations.

A disturbed permeability test was made for the gravel material to determine a saturated hydraulic conductivity value that could be used in the resistance network analog, and a disturbed permeability test was made for the base material to determine what hydraulic conductivity could be anticipated in the tile drain model. Also, a bulk density test was made of the base material in the field to determine the required packing in the tile drain model. Table 2 shows the average values and ranges of hydraulic conductivity and bulk density of the gravel and base material for the preliminary tests.

Table 2. Average Values and Ranges of Hydraulic Conductivity and Bulk Density of Gravel and Base Material for Preliminary Tests

Material Tested	Hydraulic Conductivity			Bulk Density	
	Time of Test (hours)	Average (in/hr)	Range (in/hr)	Average (lbs/ft^3)	Range (lbs/ft^3)
Nonuniform Gravel	1	27.33	25.51-28.74	110*	______**
Unaltered Silt	5	0.19	0.15-0.25	80***	71-84***
Dried and Ground Silt	3	0.27	0.20-0.40		

***Bulk density of silt from field test.
**Data not obtained.
*Bulk density of gravel from cylindrical permeameter in laboratory.

The disturbed hydraulic conductivity values for the nonuniform (pit-run) gravel material were quite high. Comparing the hydraulic conductivity of the gravel material to the base material, there is a tremendous increase in the ability to conduct air or water through the macropores of the gravel envelope. This explains why the envelope is often assumed to be completely permeable and to represent the actual or effective diameter of the tile drain. This assumption is only approximately true and there would be some resistance or head loss attributed to the envelope as water flows into the tile drain. Thus, the assumption was made for the ponded water condition on the ground surface that the gravel envelope would be completely saturated at all times. The average hydraulic conductivity value was used to represent the gravel envelope for the resistance network analog.

In contrast to the nonuniform gravel material, the disturbed hydraulic conductivity values for the base material were quite low. Two trials were made for the base material--an unchanged silt sample, and an oven-dried and ground silt sample. The unchanged silt sample, as procured from the prototype field, was initially around 10% moisture. The hydraulic conductivity obtained in the laboratory cylinder permeameter was approximately one-quarter the hydraulic conductivity obtained in the field by the auger hole method. It was hoped that by oven-drying and grinding the silt, a hydraulic conductivity near field value could be reached. Table 2 shows the hydraulic conductivity of the coarse silt to have increased after being dried and ground. However, it was decided that the small increase was nonbeneficial since a huge quantity of oven-dried and ground material would be required to fill the tile drain model. The average bulk densities shown in Table 2 for gravel and unchanged silt material were employed in packing the materials in the tile drain model. The bulk density of the gravel material was the final density which the water compacted the gravel in the cylindrical permeameter. The bulk density of the coarse silt material was determined by field measurement and replicated in the permeameter and the tile drain model.

Predicted Flow Nets and Tile Outflow

A resistance network analog was constructed to predict the flow net and tile outflow for five methods of installation--no gravel envelope and gravel envelopes of 3, 6, 9 and 12-inch thicknesses. The assumptions listed on page 40 made concerning the boundary conditions were adhered to in the resistance network analog.

Figures XXIII - XXXII in Appendix C show the complete flow net and an expanded flow net near the drain for all five installations. When the equipotential lines for the various installations are examined, it appears that the potential heads near the drain are continually moving outward as the thickness of gravel material increases. However, there is no particular difference in the potential pattern further out in the soil profile as the thickness of gravel material increases. When the streamlines for the various installations are examined, it appears that the percentage of flow from the bottom of the drain is continually increasing as the gravel envelope thickness increases.

There are two possible advantages to a larger gravel envelope thickness if the primary function of the envelope is to prevent clogging of the drain with sediment. These two advantages are additional protection from sediment movement into the envelope and additional water flow into the bottom of the drain. On the basis of this study, the additional water flow into the bottom of the drain has been predicted by the resistance network analog for a ponded water case. A soil particle entering from the top of the drain has both a downward force from the water and a gravitational force from the weight of soil particles moving into the drain; whereas, a soil particle entering the drain from the bottom will enter only if the upward force from the water is greater than the gravitational force from the weight of soil particles. A greater percentage of a given water flow from the bottom of the drain could feasibly decrease the sediment movement into the drain.

Table 3 gives the predicted tile outflow for an empty and full drain from the resistance network analog at different envelope thicknesses. The predicted tile outflow was calculated to three significant figures, since the given parameters were only to three significant figures. The largest percentage increase in tile outflow occurred when the gravel envelope was first added. The subsequent increases in tile outflow, as the gravel thickness was increased, were of approximately the same magnitude. The difference between the predicted tile outflow for an empty and full drain was almost the same for each gravel envelope thickness. This difference, although the empty drain had a larger predicted tile outflow, may not be considered physically significant in the field.

Monke (14) concluded that the percentage increase in discharge would be about equal to the corresponding maximum change in hydraulic head which occurs when the water level in the drain subsides. The conclusion pertains to a flow region which is constantly saturated and extends at least a small distance below the drain. In this investigation, the full drain had a 93-inch hydraulic head or potential head; the empty drain had a 99-inch hydraulic head or potential head. Therefore, there was a 6.5% increase in hydraulic head as the drain emptied. Looking at the percentage increase in discharge from the full to empty drain shown in Table 3, this study appears to verify Monke's conclusion.

Table 3. Predicted Tile Outflow for Empty and Full Drains from Resistance Network at Different Envelope Thicknesses

Gravel Envelope Thickness	Empty Drain		Full Drain		Difference	
	Tile Outflow (ft^3/day/ft)	Percent Increase*	Tile Outflow (ft^3/day/ft)	Percent Increase*	Tile Outflow (ft^3/day/ft)	Percent Increase**
none	15.10	——	14.10	——	1.00	7.1
3	18.30	21.2	17.10	21.3	1.20	7.0
6	20.30	10.9	18.80	9.9	1.50	8.0
9	21.80	7.4	20.30	8.8	1.50	7.4
12	23.40	7.3	22.10	8.8	1.30	5.9

* An incremental percent increase in tile outflow from previous gravel envelope thickness.
** A percent increase in tile outflow from a full to an empty drain.

Thus, the tile outflow at various water levels in the drain can be adequately predicted knowing the outflow for a given water level.

No Gravel Envelope Model

The no gravel envelope tile drain model resulted in almost immediate piping. Piping occurs when erosion channels are formed by the flow of water through the soil moving soil particles into the drain. A piping failure in the field would be exhibited by a sink hole or cavity in the soil. Figure XV shows the tile drain model failure in the laboratory. The conditions replicated in the model were adverse, since a ponded water case was represented and only a shallow profile of coarse silt was represented. The extreme ponded water condition established a high hydraulic head or potential head near the drain permanently. Whereas, in the field the ponded water condition would occur for only a short duration of time. Also, the shallow soil profile gave immediate access to piping where the hydraulic gradient could easily move the coarse silt material into the drain. Whereas, once a cavity had formed in the field, more stable layers of silty clay loam and silt loam would pack into the cavity where the prevailing hydraulic gradient near the drain could less likely move the soil particles. Even though the outcome without a gravel envelope in the field would be questionable, an erosional failure would probably still occur.

Table 4 compares the tile outflow and rate of sediment discharge of the no gravel envelope model with the 6-inch gravel envelope model. Care was taken in both models to see that the porous media was satu-

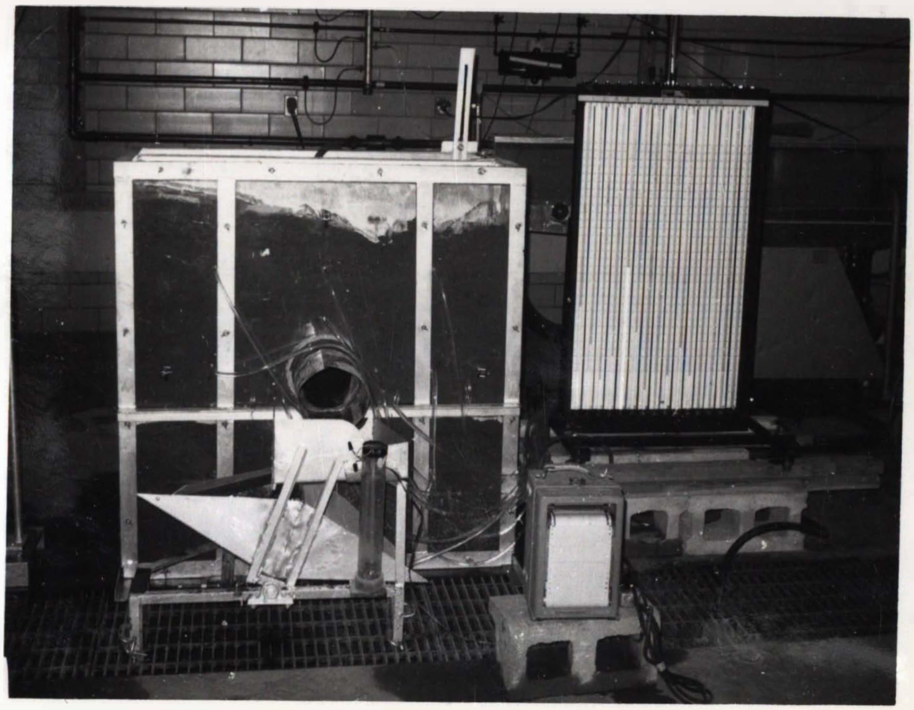

Figure XV. No Gravel Envelope Tile Drain
Model Piping Failure

Table 4. Comparison of Water Discharge and Sediment Discharge for No Gravel Envelope and Six-Inch Gravel Envelope Models

Treatment	Time of Test	Water Discharge*		Sediment Discharge**	
		Average (ft^3/day/ft)	Range (ft^3/day/ft)	Average (lbs/hr/ft)	Range (lbs/hr/ft)
No gravel	15 minutes	1440	906 - 2820	420	218 - 622
6-inch gravel	4 days	4.95	7.26 - 3.52	none	0 - .08

* No gravel measured by tipping bucket, 6-inch gravel measured by graduate cylinder.
** Six-inch gravel sediment only during first 2 hours of test.

rated and compacted to field density before tile outflow commenced. The no gravel envelope trial lasted for only 15 minutes. Yet, in 15 minutes over one hundred pounds of soil was lost from the model. An overwhelming flow rate was developed in the model. In the author's opinion, chances for clogging the tile drain without a gravel envelope are quite probable.

Six-Inch Gravel Envelope Model

The 6-inch gravel envelope model resulted in protection for the tile drain. Table 4 compares the tile outflow and rate of sediment discharge of the 6-inch gravel envelope model with no gravel envelope model. Both water discharge and sediment discharge were sizeably reduced. Only a trace of fine soil particles moved into the drain. The average tile outflow was approximately one-quarter the predicted tile outflow into an empty drain for a 6-inch gravel thickness from the resistance network analog, as shown in Table 3.

Water Discharge

Figure XVI presents the hydrograph of tile outflow from the 6-inch gravel envelope model for a four-day trial. The tile outflow started at a higher rate and decreased as time progressed. The pattern of decrease was similar to what occurred on the field drainage plot at the Redfield Irrigation Farm, Redfield, South Dakota, as shown in previous Figure I. The overall tile outflow from the tile drain model was considerably less than the outflow from the drainage plot and the predicted outflow from the resistance network using auger hole hydraulic conductivity measurements in the field.

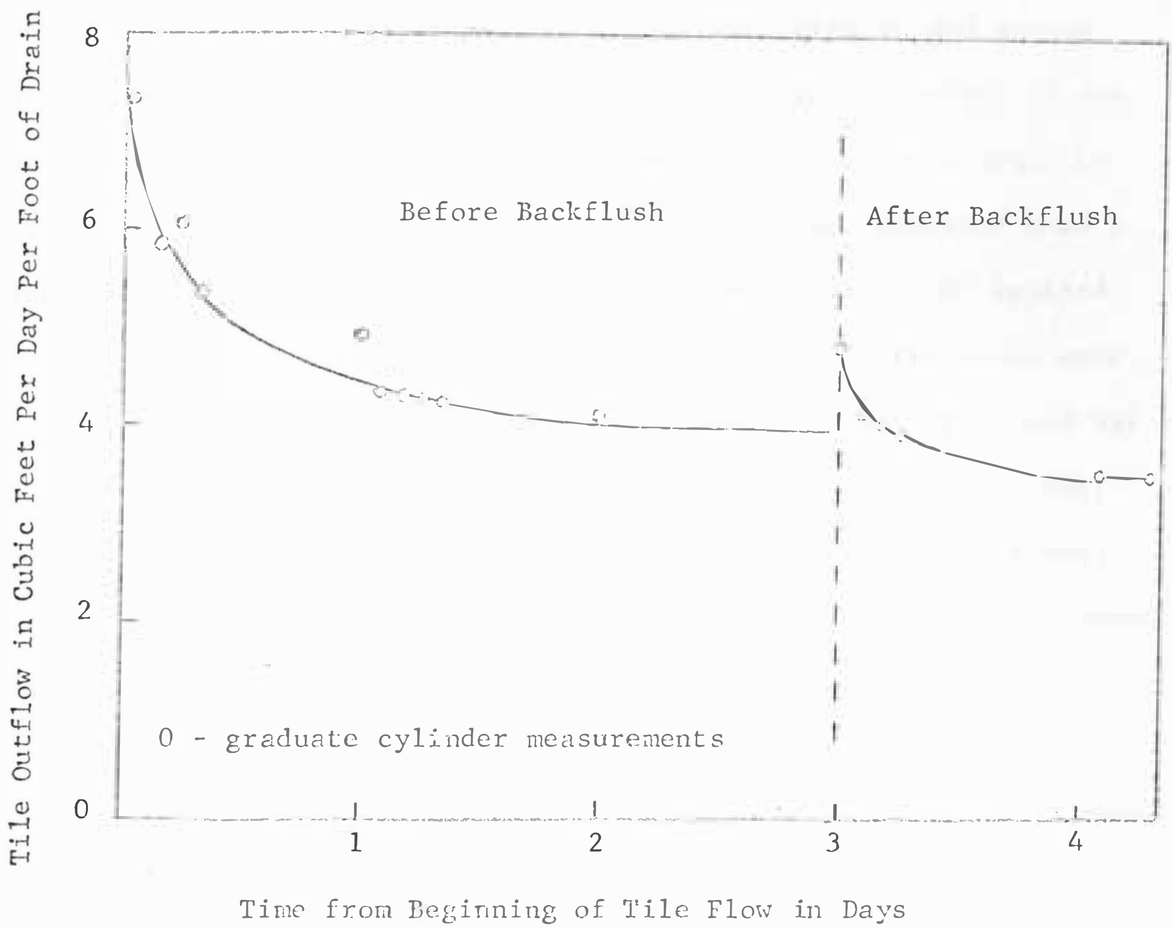

Figure XVI. Hydrograph of Tile Outflow for Six-Inch Gravel Envelope Tile Drain Model

A possible reason for the previously stated lower tile outflow from the tile drain model was bridging of soil particles on the interface, which separates the base material and the gravel envelope, or within the gravel envelope itself. Lembke (10) attributed an increase in flow rate for the second year after construction of the drainage plot to a removal or rearrangement of fine particles in and around the gravel envelope. If bridging was occurring in the model, it was decided to force water at a low head back through the tile drain to develop the gravel envelope. Figure XVII shows the apparatus used to backflush the gravel envelope. A 1½ foot head of water was applied to the tile drain with the plexiglass cover over the tile drain ends. Here, the filter tubes, which injected the water at a 1½ foot hydraulic potential during the model trial, became multiple drains on a single equipotential line. The backflush process was continued for a day. A reversed potential distribution was indicated on the manometer board concluding that the backflush process was a success.

After backflushing the tile drain model, the plexiglass covers were removed, flow started in the filter tubes, and the flow commenced from the tile drain model. Again the tile outflow started at a higher rate, as shown on the hydrograph on Figure XVI, but soon decreased to about the same flow rate as before backflushing. It appears that the lower tile outflow was not caused by restrictions in the gravel envelope.

Model Permeability

As previously discussed, the lower than predicted tile outflow from the tile drain model was not caused by restrictions within the

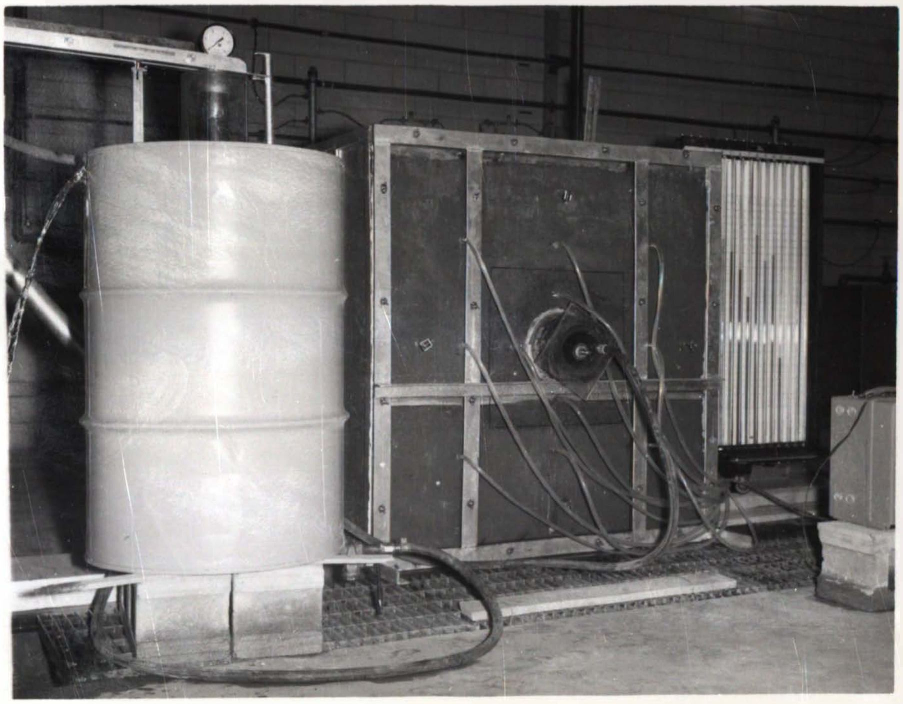

Figure XVII. Backflushing Six-Inch Gravel Envelope Tile Drain Model gravel envelope. Rather, the lower tile outflow may be attributed to a reduction in the permeability of the disturbed, coarse silt base material. The higher tile outflow was predicted using the 1964 auger hole measurements for the permeability of the base material in the field. Table 5 compares the average values and ranges of hydraulic conductivity in the field and laboratory. The field permeability of the base material was approximately four times the permeability obtained in the laboratory.

Table 5. Comparison of Average Values and Ranges of Hydraulic Conductivity for the Base Material in the Field and Laboratory

Source of Data	Hydraulic Conductivity*	
	Average (in/hr)	Range (in/hr)
1964 Auger Hole Measurements	0.80	0.78-0.82
Unaltered Silt Sample Using Cylindrical Permeameter	0.19	0.15-0.25
Six-Inch Gravel Envelope Tile Drain Model	0.18	0.13-0.27

*Corrected to 20° C.

To calculate the hydraulic conductivity values of the base material in the tile drain model, the resistance network tile outflow equation, on page 42 in this thesis, was rearranged with hydraulic conductivity, K, term as the unknown. The actual tile outflow, Q', was then inserted along with the same characteristic resistance value, R, representing the base material and amount of current flow, I, for the empty drain, 6-inch gravel envelope resistance network analog. The two assumptions being made were: (1) the ratio between the hydraulic conductivity of the base and gravel material for the resistance network analog would be the same in the tile drain model, and (2) the flow pattern for the resistance network analog would be identical in the tile drain model.

Although hydraulic conductivity for a given soil is often considered constant, it can vary widely for given material, depending on a number of factors. Cedergren (2) lists the following factors which can affect the ease at which water can travel through the soil:

1. The viscosity of the flowing fluid (water)
2. The size and continuity of the pore spaces or joints through which the fluid flows, which depends upon:
    a. The size and shape of soil particles
    b. The density
    c. The detailed arrangement of the individual soil grains, called the structure
3. The presence of discontinuities

The influence of particle arrangement and of discontinuities are possibly the more important reasons for the reduction in permeability of the base material in the tile drain model. Natural soil deposits are usually nonuniform in structure. Water-deposited soils are constructed in horizontal layers and usually more permeable in a horizontal than in a vertical direction; whereas, windblown soils are often more permeable vertically than horizontally. There is a possibility that the base material would have been water-deposited or set under water for a long period of time. Dispersion of fine particles often affects particle arrangement in the laboratory. Soils compacted in a relatively dry state can result in a fairly high permeability; on the other hand, if a liberal amount of moisture is present, the particles tend to slide over each other resulting in a relatively impermeable structure. In the permeability test of materials, a sample of ground and dried material was tried; the permeability was increased but not to the hydraulic conductivity value found in the field. Discontinuities, such as undetected joints or seams, can easily develop with time in the field to substantially increase permeability. Any one of these reasons could explain the reduced permeability of the base material.

Sediment Discharge

There was little sediment discharge into the tile drain for the 6-inch gravel envelope model. The small amount of sediment that did occur came during the early part of the tile outflow period. The sediment discharge rate from the model was initially less than one-tenth pound per hour per foot of drain and decreased as the time progressed. This was similar to the sediment discharge pattern from the field drainage plot where Lembke (10) stated that sediment occurred only during the early part of the tile outflow.

Looking at Figure XVII showing the backflushing of the gravel envelope, it can be seen that the gravel envelope thickness on top of the tile drain model has been reduced, where the original gravel envelope thickness is shown by a black square around the tile drain in the figure. A mechanical analysis was made of a 2-inch vertical core on top and bottom of the tile drain after completion of the model trial. The grain-size distribution curves of the top and bottom gravel cores along with the original gravel samples for the 6-inch gravel envelope are shown in Figure XVIII. It appears that the gradation curve for the gravel material on top of the tile drain has more fine particles than the gravel material on the bottom of the tile drain or the original gravel material. It can be noticed that the original gravel material used in the 6-inch envelope model was somewhat coarser than the gravel tested in the preliminary test, the reason being that the original sample was depleted and the second sample obtained proved to be coarser. It was concluded that a larger physical penetration and concentration of coarse silt could be expected into the top half than into the bottom half of the gravel envelope.

Model Representation

Duplicating the predicted flow net in the tile drain model, filter tubes were spaced according to the predicted streamline pattern on a single equipotential from the resistance network analog. Figure XXV in the Appendix C shows the predicted, 1½ foot equipotential line and streamline pattern which was replicated in the tile drain model for the 6-inch gravel envelope trial. Figure XIX shows the typical equipotential patterns existing in the model over the four-day trial. These equipotential lines were obtained by interpolating between the hydraulic or piezometric head readings on the inverted manometer board.

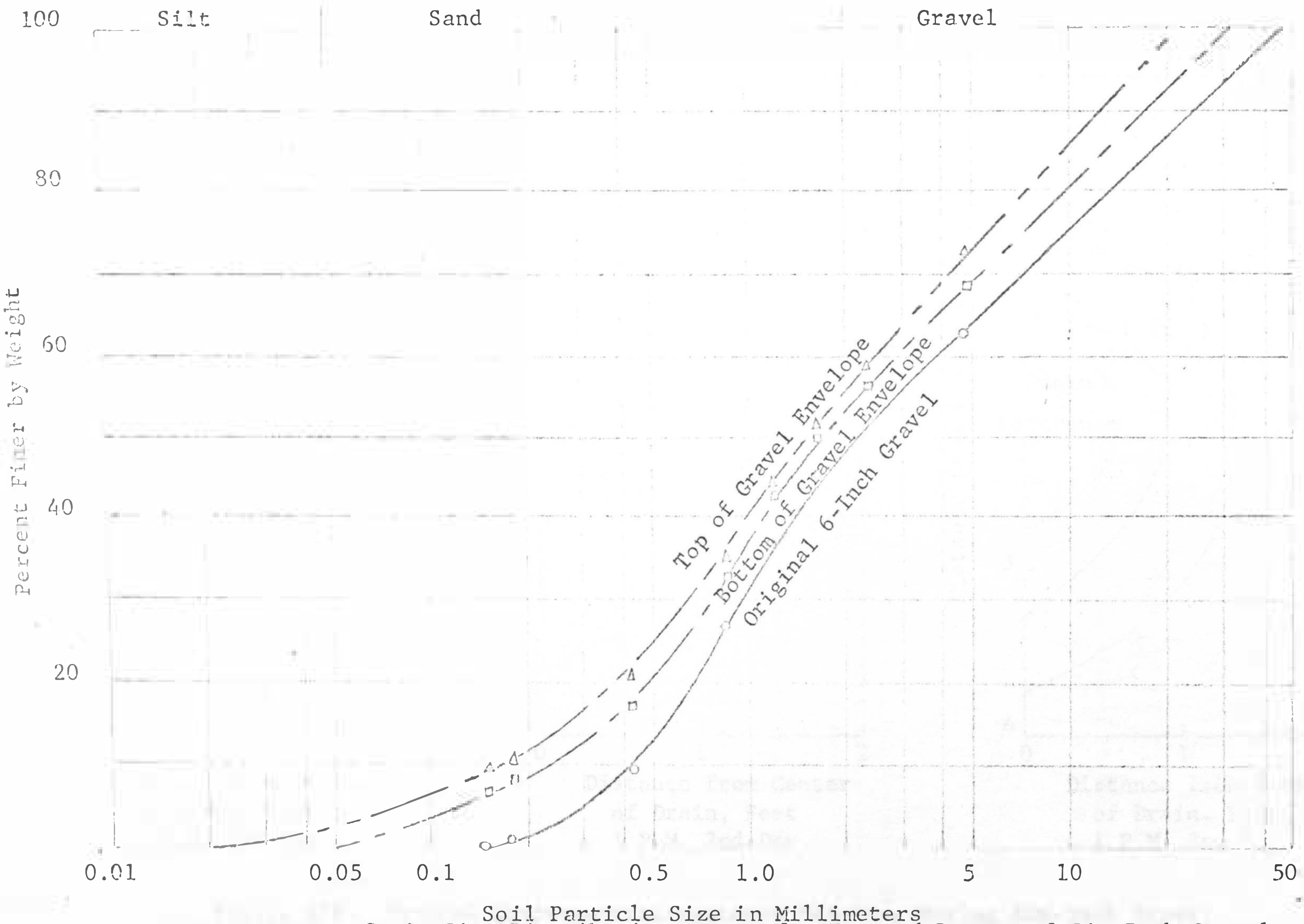

Figure XVIII. Grain-Size Distribution Curves for Top and Bottom of Six-Inch Gravel Envelope After Completion of Tile Drain Model Trial

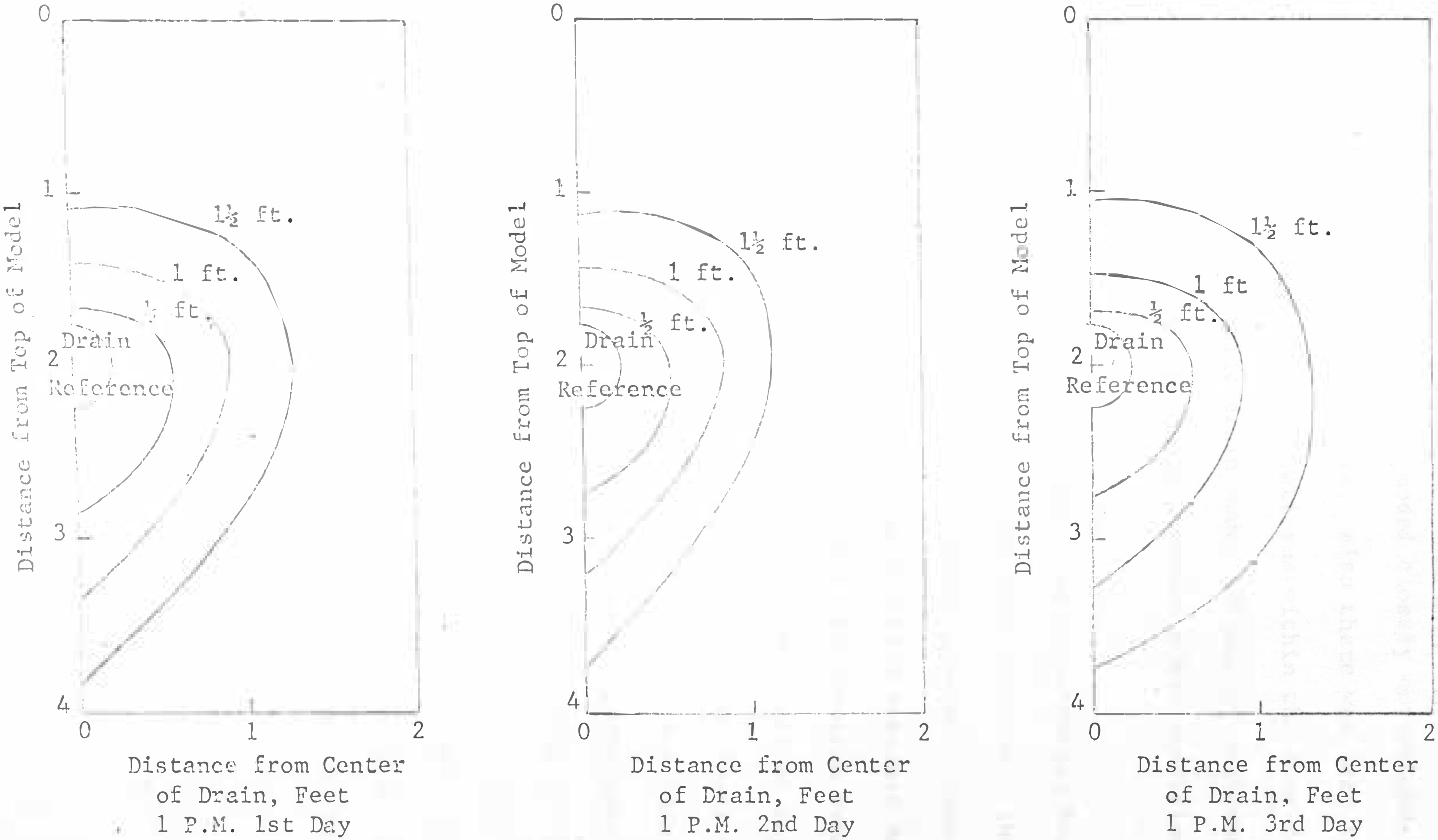

Figure XIX. Typical Equipotential Patterns Existing During Six-Inch Gravel Envelope Tile Drain Model Trial

The 1½ foot equipotential line in the model closely corresponds to the applied 1½ foot equipotential curve. Also there was very little variation of the hydraulic potential patterns within the four-day trial. Thus, evidently the tile drain model adequately replicated the equipotential pattern predicted for a ponded water condition in the field.

An outcome accrued from the hydraulic head or potential head readings was that all pressures in the model were positive. The resistance network analog had predicted negative pressures above the tile drain within the gravel envelope. The positive pressure readings within the gravel envelope would indicate that the envelope remained saturated at all times in the tile drain model. The lack of negative pressures above the tile drain can be explained from the lower hydraulic conductivity value of the coarse silt base material in the tile drain model than found in the field and represented in the resistance network. The lower hydraulic conductivity caused a reduction in the head loss through the base material establishing a higher equipotential pattern within the gravel envelope for the tile drain model. This higher equipotential pattern near the drain can be seen by comparing Figure XIX with the predicted equipotential pattern from the resistance network analog shown in Figure XXV in Appendix C.

RECOMMENDED SPECIFICATIONS

Although nonuniform (pit-run) gravels and sands sometimes can be used for envelopes, most natural deposits are highly variable in grading from point to point in the borrow area. Also, many aggregates break down and develop a greater proportion of fine particles during handling, placement, and compaction. The materials may meet grading specifications when they arrive at the job but may fail if tested after compaction. Hence, samples for testing should be taken of the materials after they have been compacted. The need for a high-quality gravel for tile drains cannot be overemphasized. Among prerequisites for good-quality construction are well-established specifications for the gravel material.

Based on the results of this investigation, no change is proposed in the use of a nonuniform (pit-run) gravel envelope in the drainage of the Oahe Irrigation Unit. Also, no change in the gradation requirements in the selection of a gravel envelope for the coarse silt base material, as shown in the previous Figure XIV, will be made. Possible specifications for the placement of a satisfactory envelope are the following adapted from the form used by Cedergren (2):

> The aggregates used shall be composed of hard durable sand and gravel particles free from organic matter, clay balls, soft particles, and other impurities or foreign matter. After being compacted in the tile trench, the material shall conform to the following grading requirements:

Sieve No. or Size	Percent Passing by Weight
3/4 inch	100
No. 4	85 to 100
No. 16	50 to 95
No. 50	5 to 50
No. 100	0 to 10

The trench will be excavated 6 inches below the bottom grade of the tile to accommodate a 6-inch gravel envelope. A 6-inch gravel envelope will be provided above and below the tile drain with a 6-inch minimum thickness on the sides. The maximum thickness on the sides can vary, depending on the method of installation and the contractor. At no place shall a tile drain be embedded in a gravel envelope of dimensions smaller than those prescribed.

The gravel material as placed and compacted in the tile trench shall be free of segregation and contamination. If the gravel envelope material fails to meet the specified requirements, the material will be considered unacceptable and shall be removed.

SUMMARY AND CONCLUSIONS

Summary

To ensure a longer life for the tile-drainage system, a more permeable backfill material than the base material is often placed around the drain. This material, placed on either the top, bottom, or sides of the drain, singularly or in combination, is called an envelope. The three-fold purpose is as follows: (1) to exclude fine soil particles from moving into the drain and resulting in clogging, (2) to increase the effective diameter by providing a highly permeable zone around the drain, and (3) to serve as a stabilizing foundation for the drain.

The principle objective of the investigation was to study in detail the design criteria for a nonuniform (pit-run) gravel envelope as proposed in the drainage of the Oahe Irrigation Unit. The prototype field selected for this investigation was the field drainage plot at the Redfield Irrigation Farm, Redfield, South Dakota. This study was limited in the following ways: (1) a single nonuniform (pit-run) gravel material, (2) a single base material of coarse silt, and (3) a ponded water condition.

The experimental plan called for study in three areas--preliminary testing of materials, a resistance network analog, and a tile drain model. The preliminary testing of materials included a mechanical analysis, permeability test, and bulk density test for the materials. The resistance network analog was constructed to predict the flow net and tile outflow for five methods of installation--no gravel envelope and gravel envelopes of 3, 6, 9 and 12-inch thicknesses. The assumptions made concerning boundary conditions for the resistance network analog were (1) no contribution to flow beyond the impermeable boundaries, (2) a ponded water condition on the ground surface, (3) an empty drain with a surface of seepage, and (4) a completely saturated gravel envelope of the same thickness on all sides. A tile drain model was designed to study two installations, no gravel envelope and a 6-inch gravel envelope. A unique aspect of the model was the replication of the streamline pattern on a single equipotential near the drain predicted by the resistance network analog. Water discharge, sediment discharge, and hydraulic head or potential head data were obtained from the tile drain model.

Conclusions

The following conclusions are offered as a result of this investigation:

1. A gravel envelope is essential to the protection of the tile drain embedded in a coarse silt base material.

2. The proposed six-inch gravel envelope for the coarse silt base material can successfully exclude sediment from the tile drain.

3. The nonuniform (pit-run) gravel envelope material selected in this investigation may have been somewhat coarse for the base material according to established design criteria recommendations.

4. While a larger physical penetration and concentration of sediment may be expected in the top half than into the bottom half of the gravel envelope, the difference may not be significant.

5. The addition of a gravel envelope, no matter how small, could substantially increase the predicted tile outflow. Subsequent increases in gravel envelope thicknesses may provide still a larger outflow; however, the additional outflow is not in direct proportion to the increase in envelope thickness.

6. The possible advantages of a thicker gravel envelope are additional protection from sediment movement into the envelope and additional water flow into the bottom of the tile drain. A greater percentage of a given water flow from the bottom of the drain could feasibly decrease the sediment movement into the tile drain.

7. The predicted tile outflow was greater for an empty tile drain than for a full tile drain. The difference, which was less than 10 percent for all installations studied, may not be considered physically significant in the field.

8. Tile outflow and sediment discharge decreased with time in the tile drain model.

9. The six-inch gravel envelope remained completely saturated at all times in the tile drain model.

10. The tile drain model in this investigation could be used to answer questions concerning the hydraulics near the drain in a tile-drainage system; however, for questions of a more practical nature regarding a particular aquifer and envelope, a simple cylinder model would be satisfactory.

The following suggestions are offered for further study on this subject:

1. A graded gravel envelope material which is more nearly within established design criteria recommendations could be tested in the tile drain model.

2. Additional gravel envelope thicknesses could be tested in the tile drain model.

3. Additional study could be made into the explanation for the reduction in permeability of the base material in the laboratory.

4. A falling water-table condition could be studied for various gravel envelopes.

5. A tile drain model and resistance network analog study could be initiated using only a gravel envelope on the top half of the tile drain with a glass-fiber sheet or mat on the bottom of the tile drain.

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APPENDICES

APPENDIX A. DEFINITION OF SYMBOLS

Definition of Symbols

A - cross section of flow area

C_o - conversion coefficient

D - effective diameter for hydrometer reading

D_{15}-, D_{50}-, D_{85}- particle size at which 15%, 50%, and 85% of the particle weight is smaller

G_s - specific gravity of solids

g - gravitational constant

h - elevation measured from reference plane

h_n - elevation of ground surface from reference plane

I - current

i - hydraulic gradient

K - hydraulic conductivity

L - length

N - percent finer for the hydrometer

N' - percent finer than No. 200 sieve

P - pressure

Q - volume of water per unit time

Q' - flow rate per foot of drain

R - resistance

R_o - characteristic resistance

r - hydrometer reading in suspension

r_w - hydrometer reading in distilled water

t - elapsed time

V - voltage

$\overline{V}$ - volume of suspension

V_0 - voltage at a specific node

W_s - weight of dry soil

W' - weight of dry soil passing No. 200 sieve

x, y, z - cartesian coordinates

Z_r - distance from surface of suspension to center of hydrometer

ϕ - hydraulic head or potential

ϕ_d - hydraulic head or potential at drain

ϕ_n - hydraulic head or potential at ground surface

ϕ_0 - hydraulic potential at a specific node

μ - viscosity of water at test temperature

μ_c - viscosity of water at 20° C

γ_c - unit weight of water at 20° C

γ_s - unit weight of soil grains

γ_w - unit weight of water at test temperature

ρ - fluid density

APPENDIX B. ASSEMBLED RESISTANCE NETWORK ANALOG

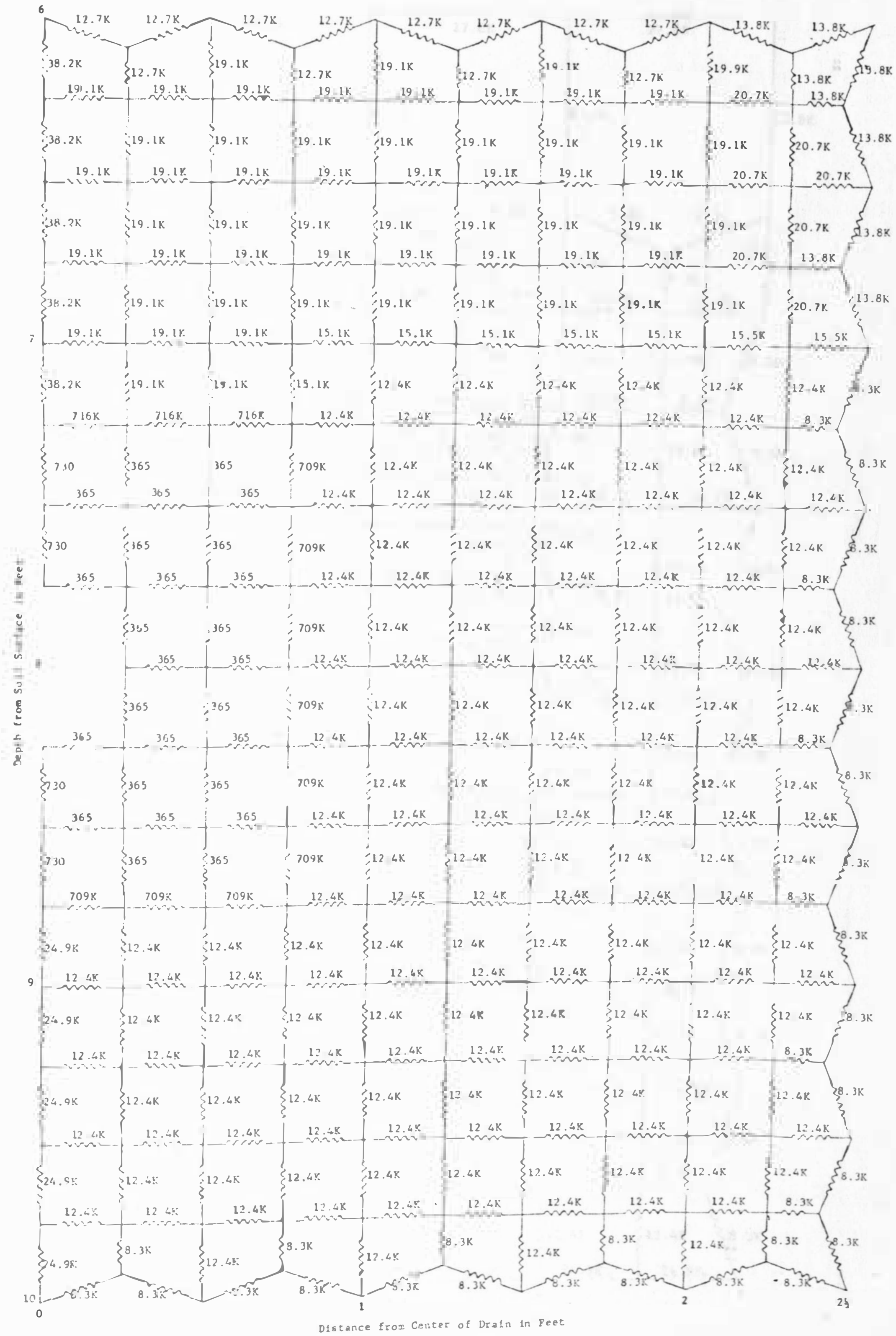

Figure XX. Assembled Resistance Network
Analog Board Number One

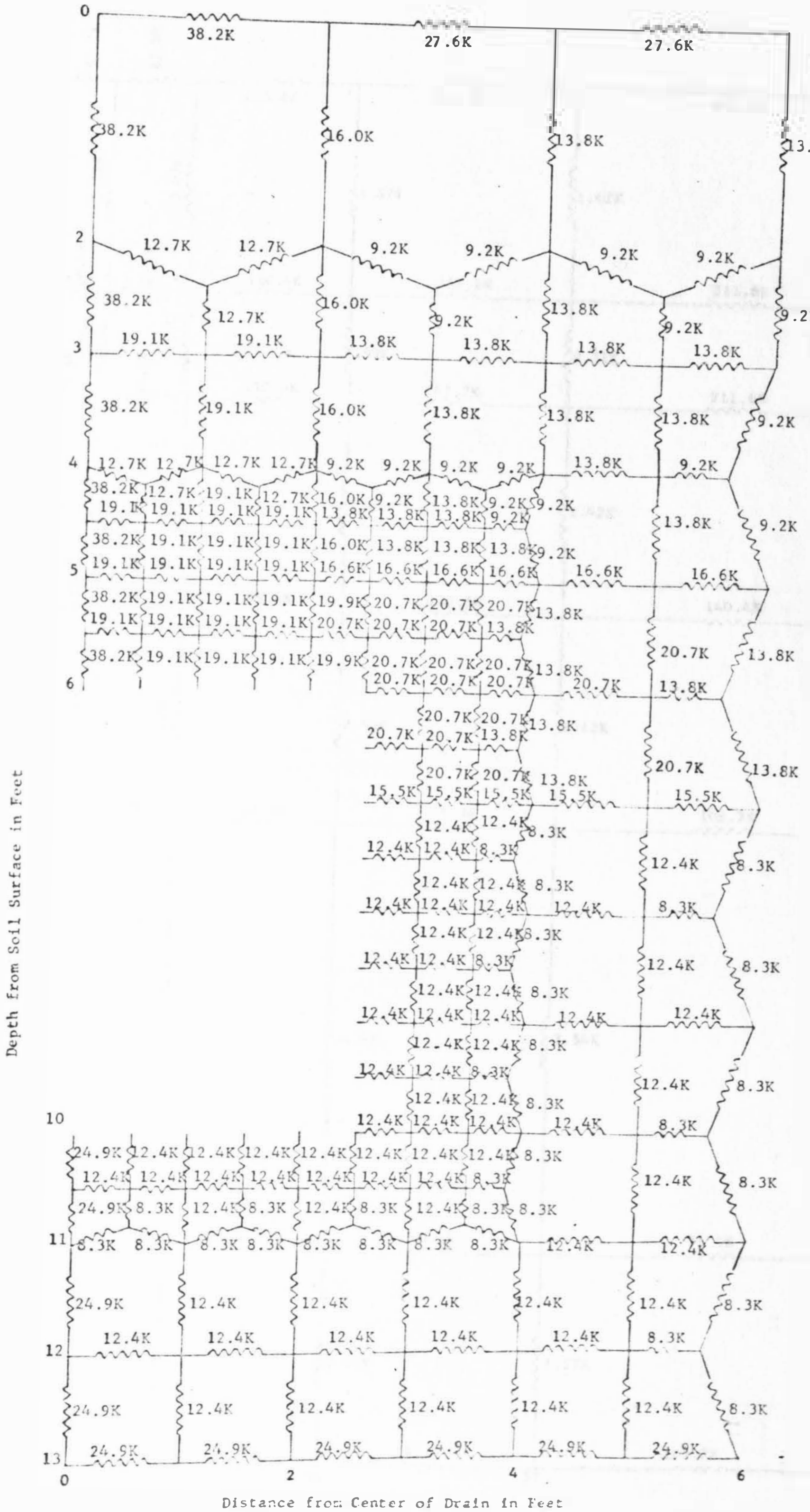

Figure XXI. Assembled Resistance Network Analog Board Number Two

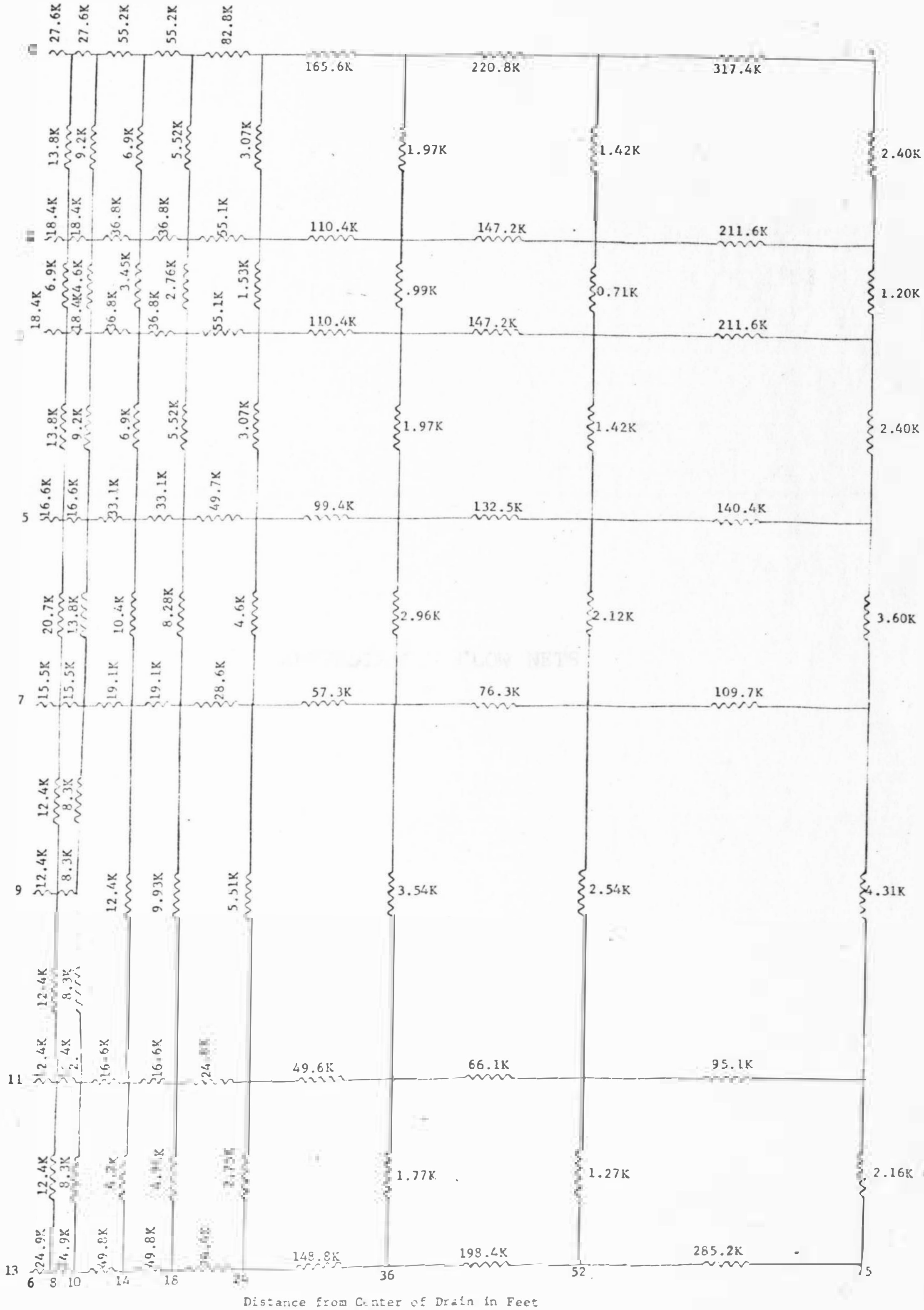

Figure XXII. Assembled Resistance Network Analog Board Number Three

APPENDIX C. FLOW NETS

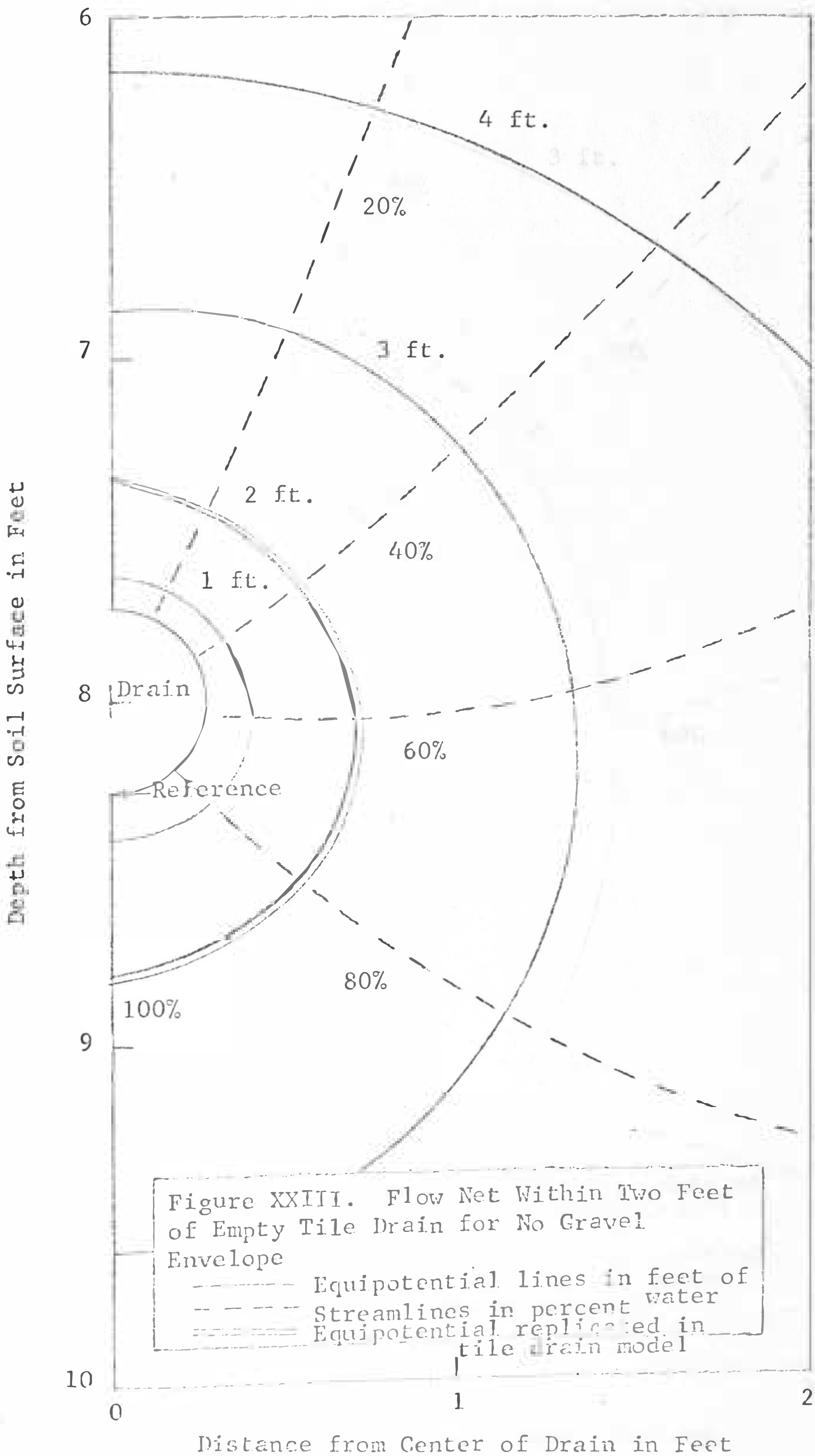

Figure XXIII. Flow Net Within Two Feet of Empty Tile Drain for No Gravel Envelope

Equipotential lines in feet of water
Streamlines in percent
Equipotential replicated in tile drain model

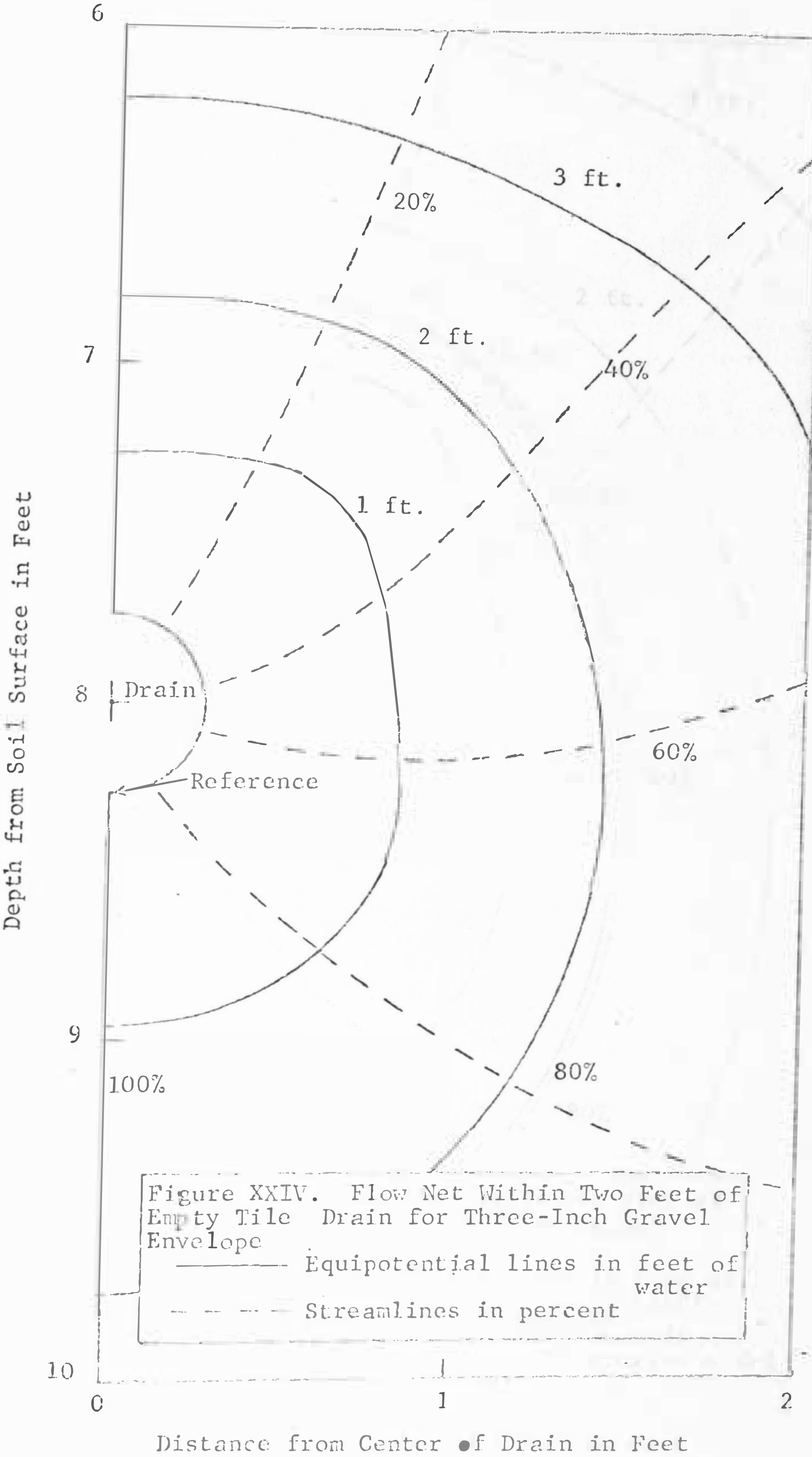

Figure XXIV. Flow Net Within Two Feet of Empty Tile Drain for Three-Inch Gravel Envelope

———— Equipotential lines in feet of water

– – – – Streamlines in percent

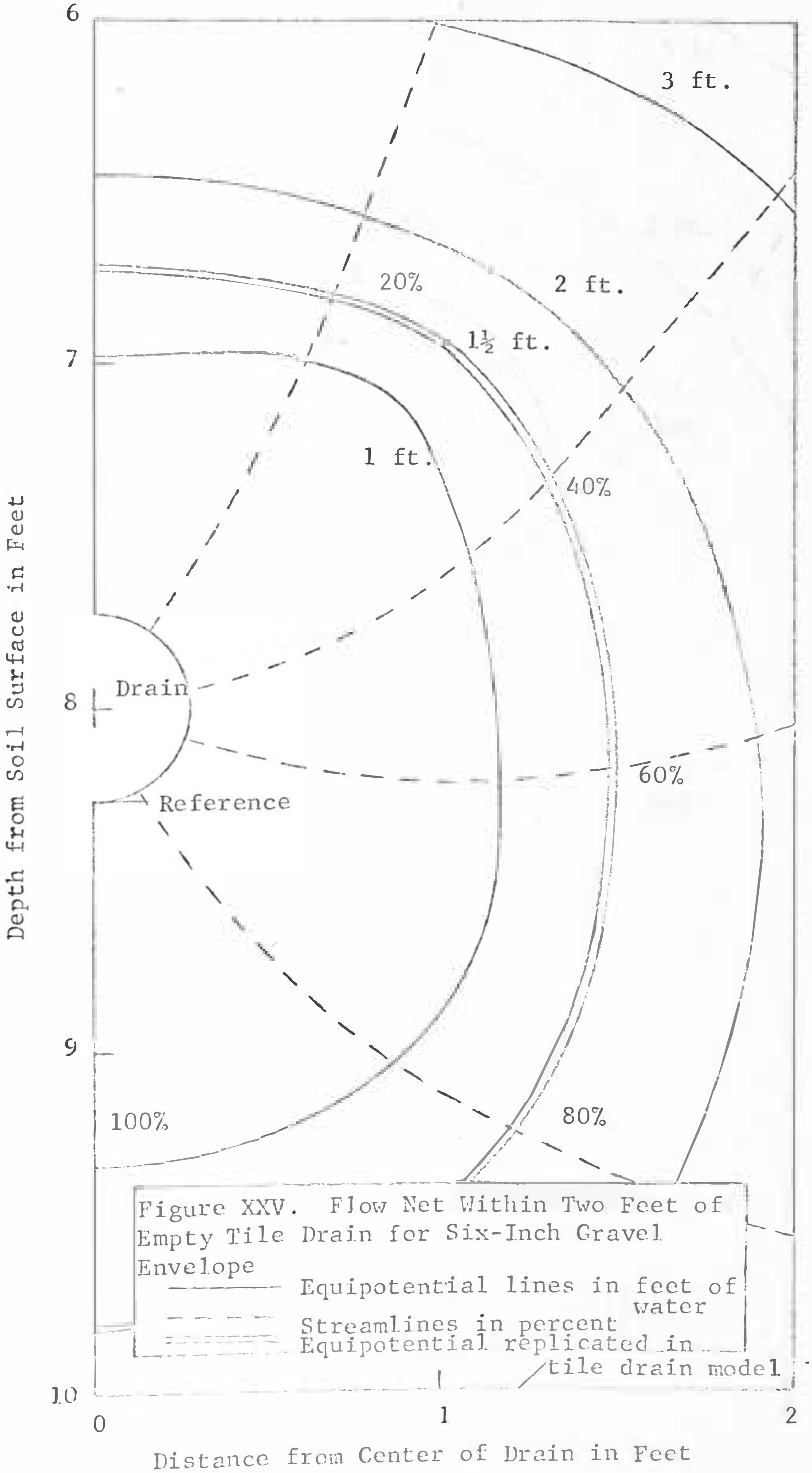

Figure XXV. Flow Net Within Two Feet of Empty Tile Drain for Six-Inch Gravel Envelope

Equipotential lines in feet of water
Streamlines in percent
Equipotential replicated in tile drain model

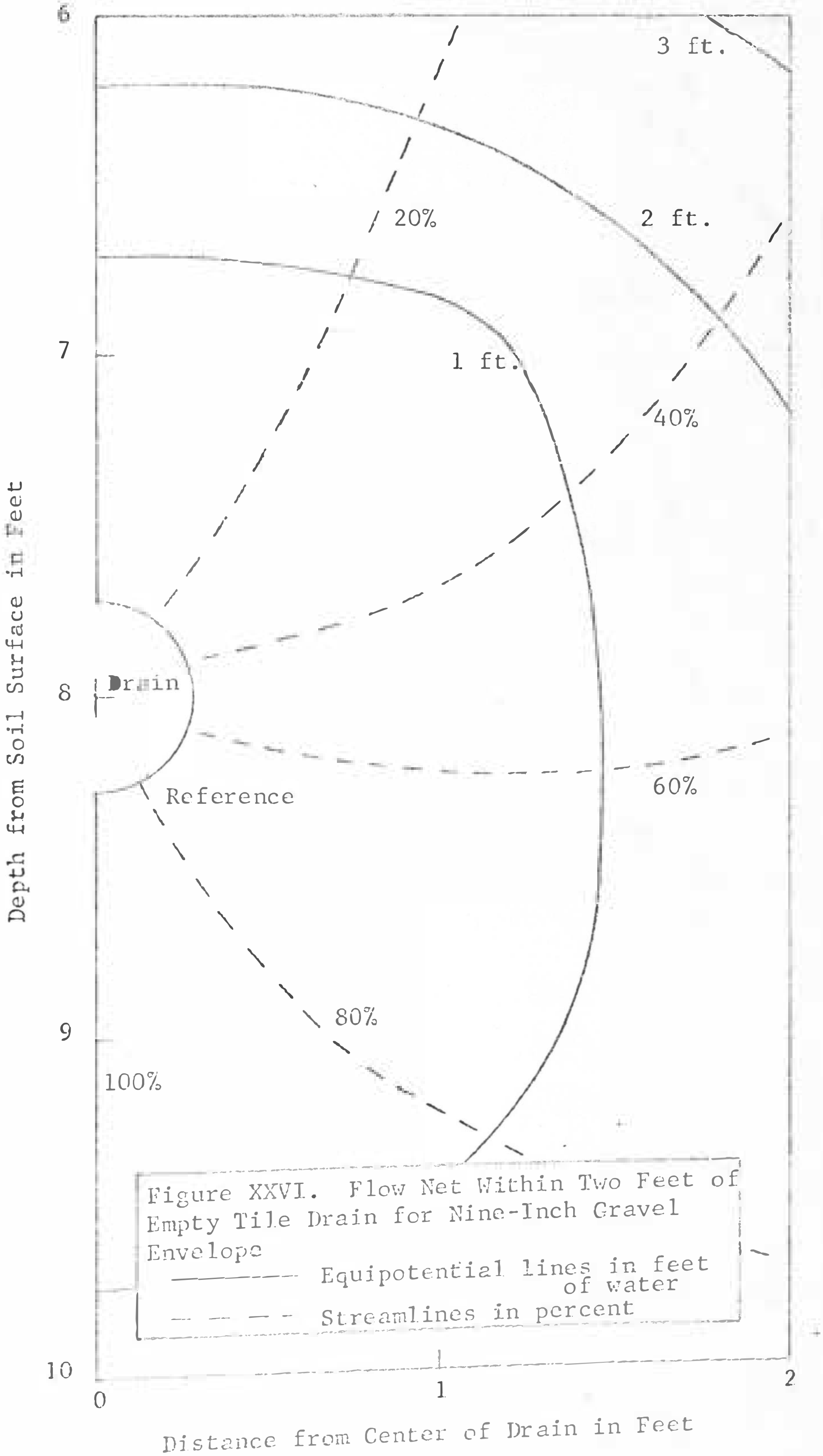

Figure XXVI. Flow Net Within Two Feet of Empty Tile Drain for Nine-Inch Gravel Envelope

——— Equipotential lines in feet of water

— — — Streamlines in percent

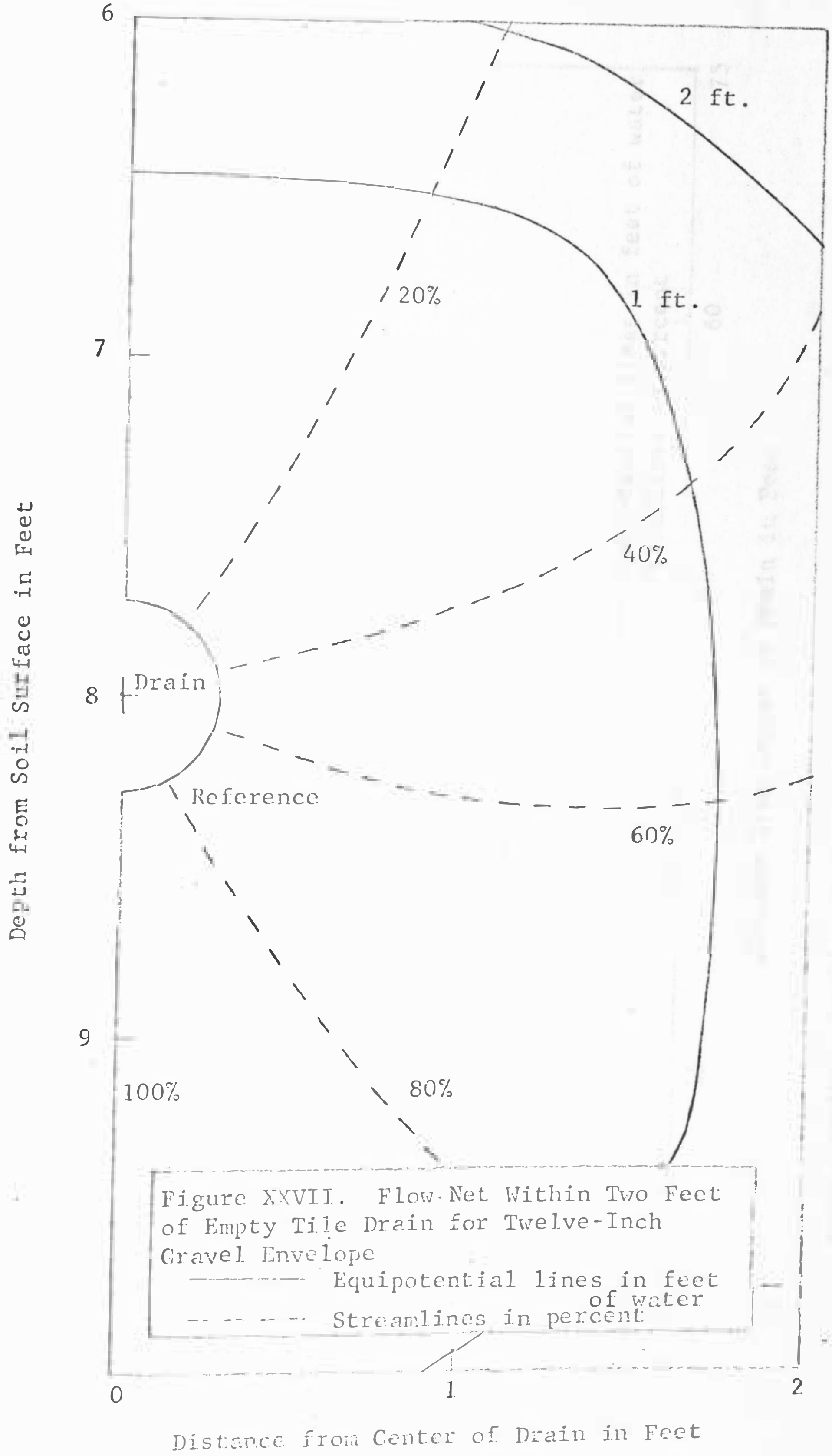

Figure XXVII. Flow-Net Within Two Feet of Empty Tile Drain for Twelve-Inch Gravel Envelope

——— Equipotential lines in feet of water

– – – – Streamlines in percent

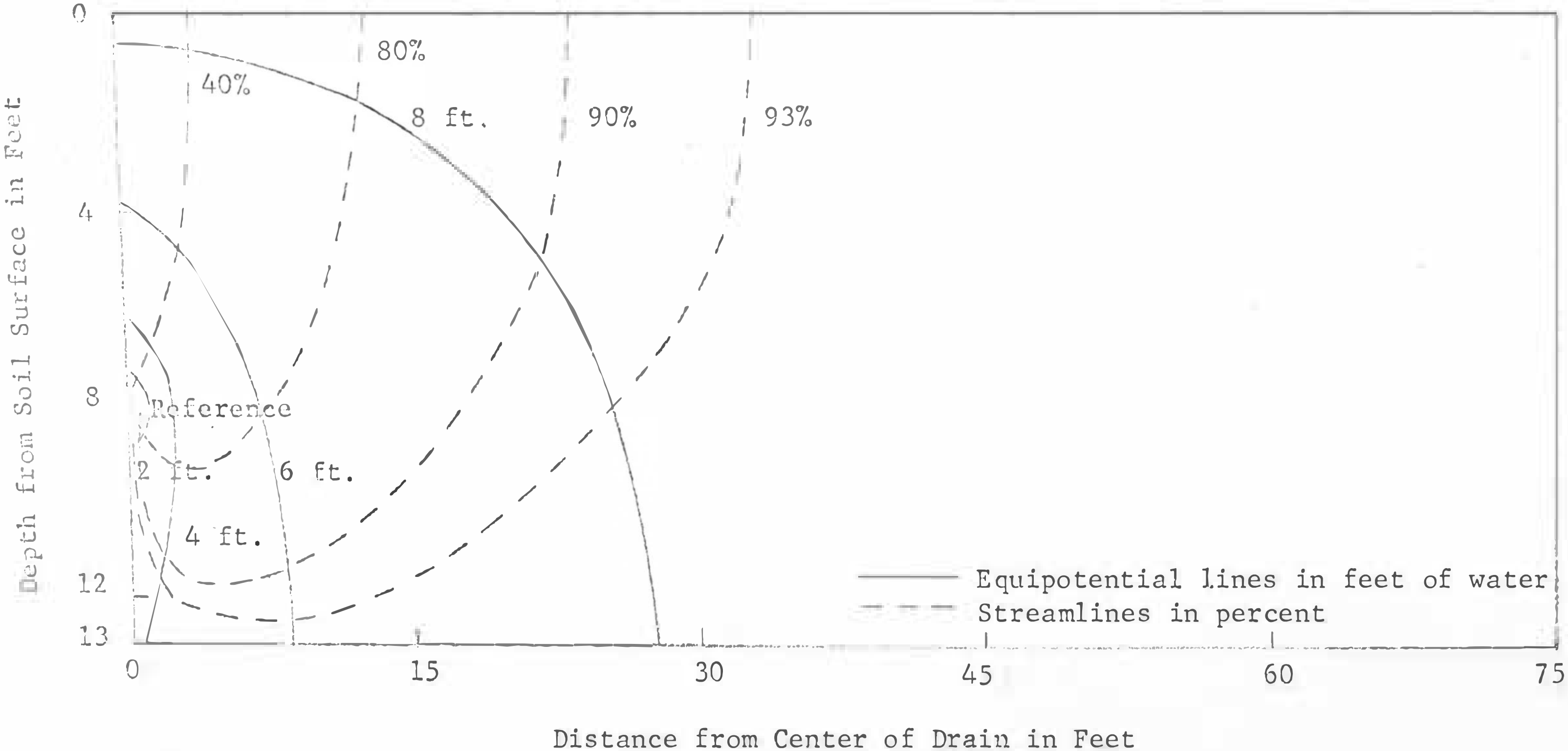

Figure XXVIII. Complete Flow Net During Ponded Water Flow into Empty Tile Drain for No Gravel Envelope

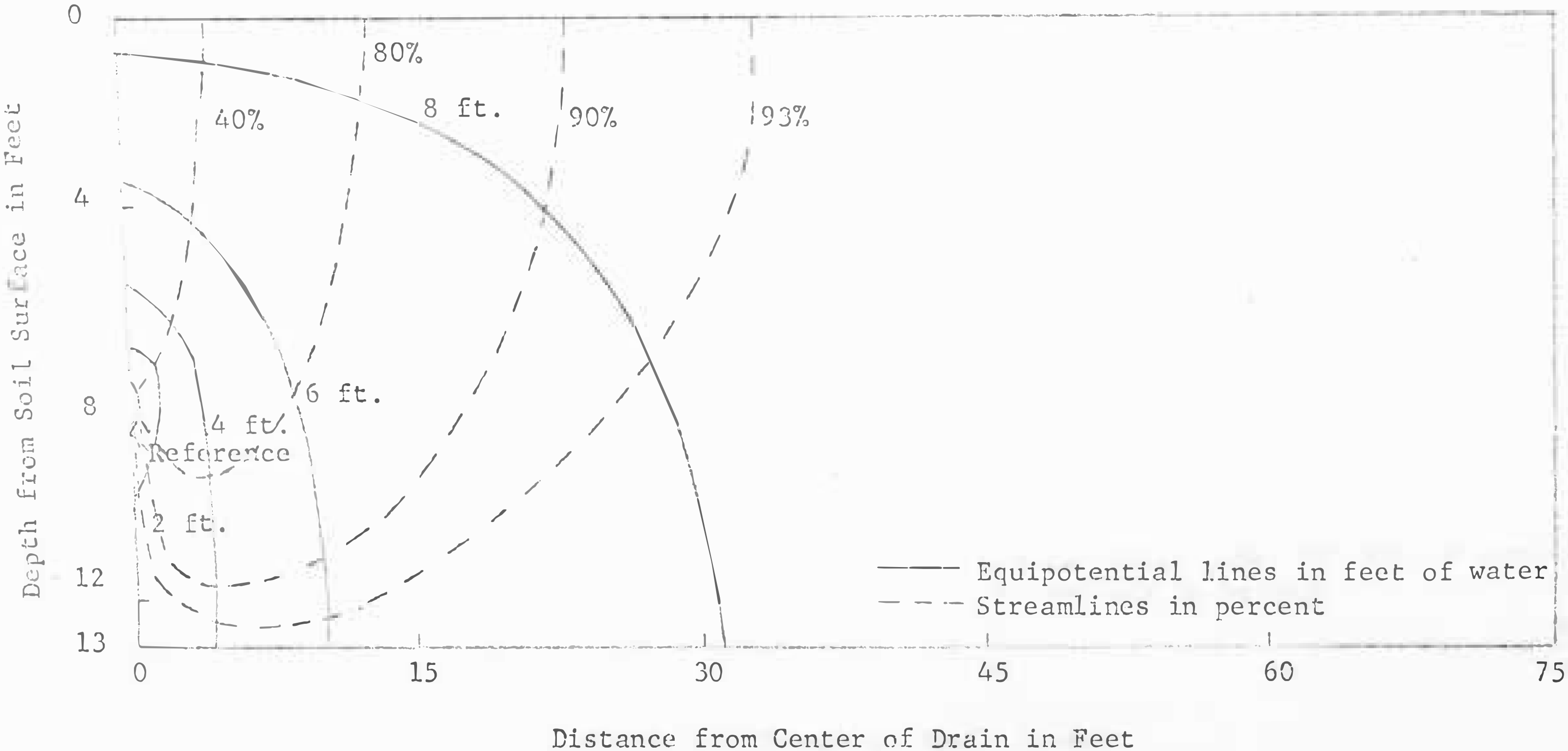

Figure XXIX. Complete Flow Net During Ponded Water Flow into Empty Tile Drain for Three-Inch Gravel Envelope

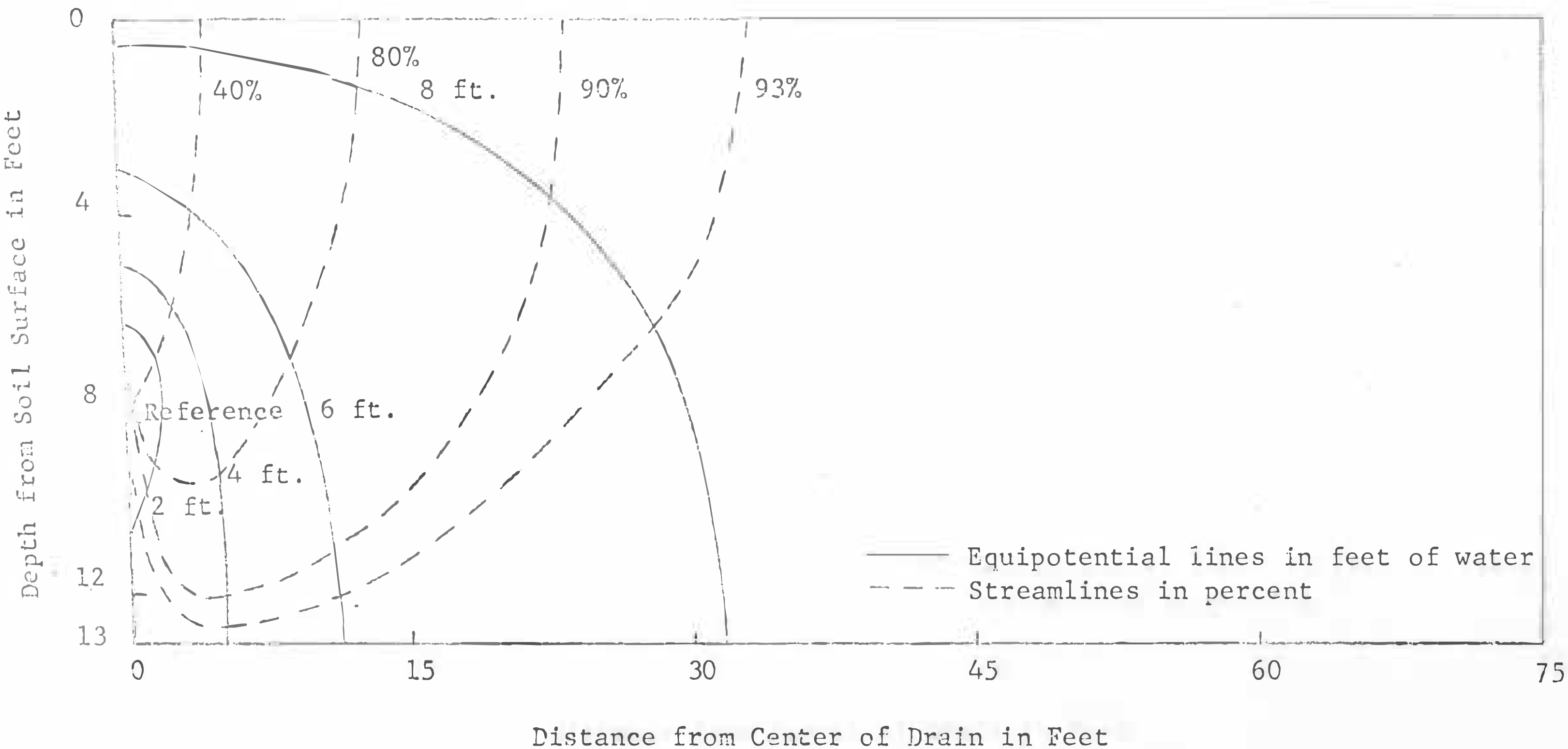

Figure XXX. Complete Flow Net During Ponded Water Flow into Empty Tile Drain for Six-Inch Gravel Envelope

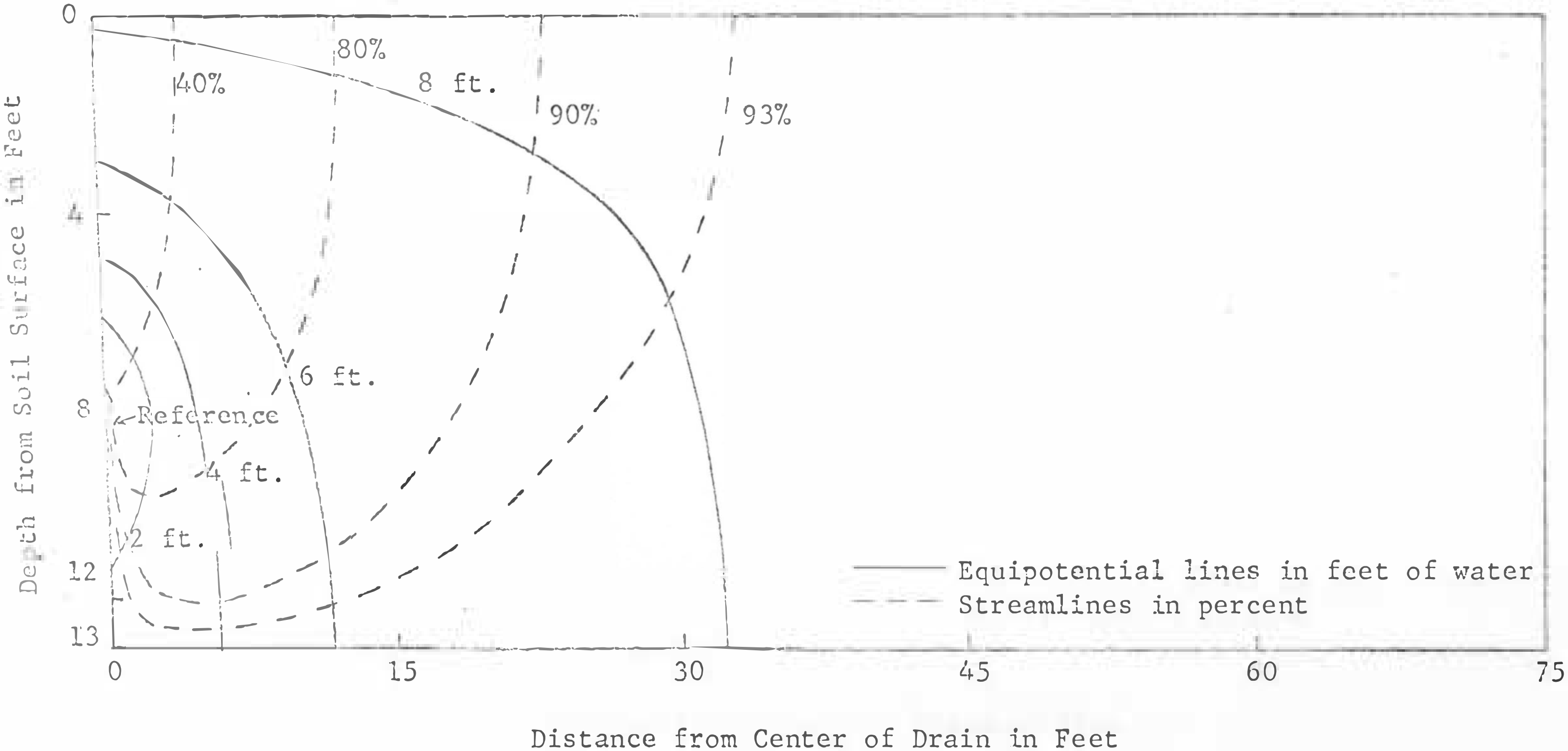

Figure XXXI. Complete Flow Net During Ponded Water Flow into Empty Tile Drain for Nine-Inch Gravel Envelope

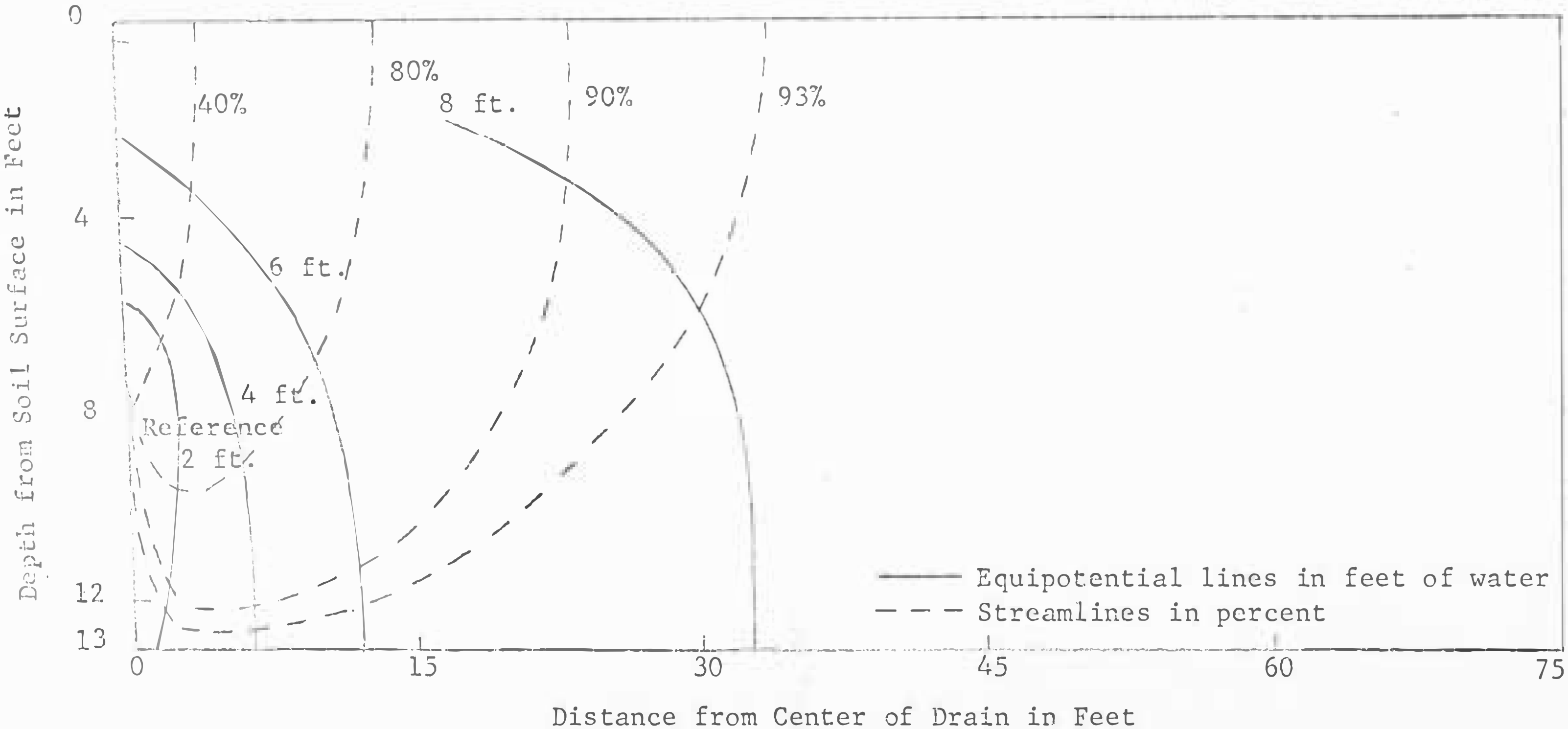

Figure XXXII. Complete Flow Net During Ponded Water Flow into Empty Tile Drain for Twelve-Inch Gravel Envelope

APPENDIX D. SIX-INCH GRAVEL ENVELOPE TILE DRAIN MODEL DATA

Table 6. Six-Inch Gravel Envelope Tile Drain Model Data
Bulk Density of Base Material: 75.4 lbs/ft^3
Bulk Density of Gravel: 102 lbs/ft^3
Temperature of Water: 15°C, initial; 17°C, discharge

Date	Time	Piezometer Readings inches of water*												Tile Outflow ml/min				Sediment gm/l
		1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	Avg.	
11/6/67	10A.M.	14.7	14.6	11.0	3.4	18.6	12.2	-3.0	9.1	4.8	20.5	14.4	19.0	150	140	140	143	4.26(9A.M.)
	1P.M.	17.3	14.1	11.8	6.3	20.9	10.2	-0.6	8.4	3.4	19.8	13.7	19.8	115	116	114	115	(no silt)
	3P.M.	18.4	14.0	13.1	6.1	20.1	10.3	-0.6	8.4	3.0	19.6	13.4	19.8	110	106	110	119	
	5P.M.	18.7	13.9	13.1	6.0	19.5	10.1	-0.6	8.1	2.7	19.1	13.1	19.6	105	106	105	105	
11/7/67	9A.M.	19.8	14.7	15.1	5.7	20.3	10.8	-0.5	9.5	2.9	19.5	14.4	20.4	97	97	97	97	
	11A.M.	19.8	15.4	15.4	7.1	20.2	Air	1.6	10.8	5.1	19.5	15.0	20.4	84	87	88	86	
	1P.M.	19.7	15.4	15.4	7.1	20.2	Air	1.5	10.8	4.5	19.5	15.1	20.4	86	86	84	85	
	3P.M.	19.6	15.4	15.4	7.0	20.3	11.5	1.5	10.8	4.5	19.4	15.1	20.4	85	84	85	85	
	5P.M.	19.5	15.4	15.4	7.0	20.4	11.5	1.5	10.8	4.5	19.4	15.2	20.4	83	84	84	84	
11/8/67	9A.M.	19.0	15.5	15.5	6.9	19.1	11.4	1.4	10.6	4.5	19.0	15.6	20.4	82	83	82	82	
Backflushed tile drain model at 9:30 A.M. - 1½ ft. head																		
11/9/67	9A.M.	18.1	16.5	13.6	5.6	18.4	13.6	2.2	10.0	4.0	19.9	14.0	16.5	96	95	95	95	0.07
	11A.M.	19.2	15.2	14.4	5.8	18.5	13.5	2.2	9.6	4.0	19.9	13.6	16.3	82	82	81	82	(no silt)
	1P.M.	19.3	15.2	14.4	5.7	18.5	13.5	2.2	9.6	3.9	19.8	13.6	16.2	80	79	79	79	
	3P.M.	19.4	15.2	14.4	5.7	18.4	13.5	2.2	9.5	3.9	19.8	13.7	16.2	77	77	77	77	
11/10/67	11A.M.	19.0	15.2	14.3	5.6	18.0	13.2	2.2	9.3	3.9	18.9	13.5	15.6	69	70	69	69	
	4P.M.	18.9	15.2	14.3	5.6	17.9	12.2	2.2	9.3	3.9	18.7	13.5	15.5	70	69	69	69	

* Reference plane bottom of tile drain