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# Exit Gradients Into Subsurface Drains

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The exit gradients into drain pipes of various diameters and at various depths below the ponded soil water surface and with an impermeable barrier at varying depths below the drain were investigated by programming Kirkham's equation for flow into drains under ponded conditions. It is shown that a quick condition or a weightless condition of the soil can result only under the drain lines. It is only at this point that the upward hydrodynamic seepage force is greater than the downward gravity force of the soil particle. The seepage forces result in an exit gradient that is higher than the usual critical exit gradient of one for a fine sand material without surcharge load. In almost every case, the critical exit gradient is exceeded with drain lines located 2, 3, and 5 feet below the water table and for drain diameters of 0.2–1.2 feet. These conditions will occur under irrigated agriculture during leaching operations and also may occur during the irrigation season. The effect of a surcharge load and the effect of the cohesive forces among the soil particles is discussed. As a result of the analysis, it is tentatively recommended that the gravel envelope be located beneath the drain rather than all around the drain as is currently practiced. In addition, the top of the drain can be covered with an impermeable material to prevent piping of soil into the drain during the settling of the soil in the trench. These findings are confirmed by the field observations of Pillsbury, 1967.

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# Exit Gradients Into Subsurface Drains[1]

## INTRODUCTION

THE DESIGN OF GRAVEL ENVELOPES based on a consideration of the particle size distribution curves for the envelope and the natural soil is well known. However, the location of the gravel envelope and the need for gravel envelopes have not been specified in terms of the hydrodynamic seepage forces causing movement of the soil. This paper examines the seepage forces and hence the exit gradients on the periphery of drain lines without gravel envelopes and on the periphery of a gravel envelope.

Where soils are stable and the movement of sediments into the drain is not a problem, a gravel envelope is not needed. Stable soils are commonly found in the Middle West of the United States and other areas where the natural soil structure prevents movement of soil into the drains. Many soils which are high in clay and organic matter have sufficient cohesiveness to resist disruption of the larger aggregates and movement of the individual particles. Where soils are unstable or consist of easily erodible sediments—usually structureless fine sands and silts but also fine-grained soils—gravel envelopes must be installed around the drain line to prevent the inflow of sediments into the drains.

The movement of sediments into a drain line is caused by a quick condition of the cohesionless soil materials. This quick condition is induced by the inflowing water and causes a weightless situation for the soil materials. It occurs when the water moves upward to exit from the soil. The materials most susceptible to a quick condition are very fine sand and coarse silt. Such materials lack cohesion, and the particles are sufficiently small to move through the cracks between successive pipes or through holes provided in the pipe for water entry. In the structureless soils of the western United States, clay particles may also move into the tile drains.

## CAUSES OF DRAIN LINE FAILURES

Failure of drainage lines may be caused by a number of factors, including:

1. Excessive crack width between successive sections or excessively large perforations.
2. Improper alignment of successive sections.
3. Grade reversals during construction.
4. Failure or collapse of drain pipe because of excessive loads or inadequate strength of pipe.
5. Settlement of sections of line because foundation conditions are unstable.

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6. Erosion of backfill soil into drain lines during compaction procedures ("subbing in").

7. Improper design of gravel envelope.

8. Improper location of gravel envelope.

Quality control can reduce failures caused by 1, 2, 3, and 4 above. Settlement (5) may be reduced if the problem is recognized and adequate bedding or cradle support for the drain pipes is provided. Erosion of the loose backfill into the pipe occurs when water is added to the backfill in order to settle it. A strip of impermeable material on top of the drain may be used to protect the pipe during this operation.

# THE QUICK CONDITION

Water percolating through a soil mass has a residual force acting along its path and in the direction of flow. This force is proportional to the prevailing hydraulic gradient at each point.

When upward moving water emerges from the soil, the prevailing force acts in an upward direction and tends to lift the soil particles. Once the surface particles are disturbed, the resistance to the upward pressure of the percolating water is further reduced, tending to give progressive disruption of the subsoil mass.

This action may also be described as a flotation process in which the upward force exceeds the downward force of gravity of the soil mass. The solid materials of the composite soil appear to be weightless and are entirely lacking in cohesion and bearing capacity, and a quick condition results (Jumikis, 1962).

From Bernouilli's equation we know that the force per unit volume exerted by the seeping water is given by

$$F = \rho g i = \gamma i \qquad (1)$$

where $i$ = the hydraulic gradient and is the space rate of energy dissipation (Jumikis, 1962).

$\rho$ = the density of water
$g$ = gravitational constant
$\gamma = \rho g$ = specific weight of water

The hydrodynamic seepage force $F$ has the dimensions of a force per unit volume. It acts in a direction normal to the equipotential lines in isotropic soil.

Equation (1) shows that the seepage force is independent of the hydraulic conductivity, $k$, and the velocity of flow. This means that the hydrodynamic seepage force is the same for clayey soils of low hydraulic conductivity and sandy soils of high hydraulic conductivity.

# FORCES OPPOSING THE HYDRODYNAMIC SEEPAGE FORCE

The hydrodynamic seepage force acting on the soil particles will cause them to move if not opposed by other forces having greater magnitude and acting in the opposite direction. The stability of the soil will depend on the relationship amongst all of the forces acting on it.

## Gravity force

If the unit weight of the soil particles is $W$ and $n$ is the soil porosity, then the gravity force per unit volume of submerged soil particles will be

$$W_{sub} = (W - \gamma)(1 - n)\gamma \qquad (2)$$

where $\gamma$ is the specific weight of water. The gravity force acts in a downward direction.

If $G$ is the specific gravity of the solid soil particles, then

$$W_{sub} = (G - 1)\ (1 - n)\gamma \qquad (3)$$

Because the void ratio $e$ is related to the porosity by the relationship

$$n = \frac{e}{1+e}$$

we have

$$W_{\text{sub}} = \frac{G-1}{1+e}\,\gamma$$

## Surcharge load

A surcharge load may be placed on the soil. This load may be the result of the weight of the gravel filter; it may be caused by the weight of the backfill in the trench; or a combination of these two. For this load to effectively oppose the upward hydrodynamic seepage force, it must be transmitted to all the soil particles at the interface between the soil and the surcharge material. An intimate grain contact is necessary.

This is expressed by

$$W_{\text{sur}} = \text{surcharge load}$$

## Soil cohesion

The cohesive forces of soil can be substantial in soils high in clay and organic matter. This is the reason why movement of the soil into drain lines is not a problem in the well-structured soils of the middle western United States. In California, cohesive forces are high in the clayey soils and in certain soils high in organic matter. These soils do not require a gravel envelope or filter to prevent soil movement. The hydrodynamic seepage forces do not exceed the soil cohesive forces. However many alluvial soils contain cohesionless sands and silts and clays. These soils are particularly susceptible to movement by the seepage forces.

## Resultant force

The direction of flow of the water through the soil determines the direction and magnitude of the resultant force because the gravity force always acts vertically downward, the surcharge load acts vertically, and the cohesion force acts in all directions.

Consider flow in a vertical downward direction. The seepage force now acts in the same direction as the gravity force. The resultant force is the sum of the gravity force and the seepage force and hence soil densification results. It is impossible to have a quick condition occur for this flow situation. See figure 1a. The resultant body force is obtained by adding the seepage force to the gravity force so that the resultant, $R$, is given by

$$\bar{R} = \bar{W}_{\text{sub}} \downarrow + \bar{F} \downarrow$$

in which $W_{\text{sub}}$ is the submerged weight of the soil mass acting vertically downward and $F$ is the seepage force per unit volume of soil mass acting tangentially to the streamline.

For vertically upward flow the seepage force is opposite in direction to the gravity force, and a quick condition or weightless condition of the soil may result. The cohesive forces and the surcharge load oppose the seepage force and tend to prevent a quick condition.

In the absence of surcharge and cohesion the resultant force $R$ is given by (see figure 1b)

$$R = W_{\text{sub}} \downarrow - F \uparrow$$

When the seepage force becomes equal to the submerged weight of the soil, then the soil appears to be weightless and instability of the soil mass is impending. At this point the hydraulic gradient causing the seepage force is called the "critical gradient" and

$$F = \gamma i_{\text{cr}} = W_{\text{sub}}$$

Because we know that

$$W_{\text{sub}} = \frac{G-1}{1+e}\,\gamma$$

then

$$i_{\text{cr}} = \frac{G-1}{1+e}$$

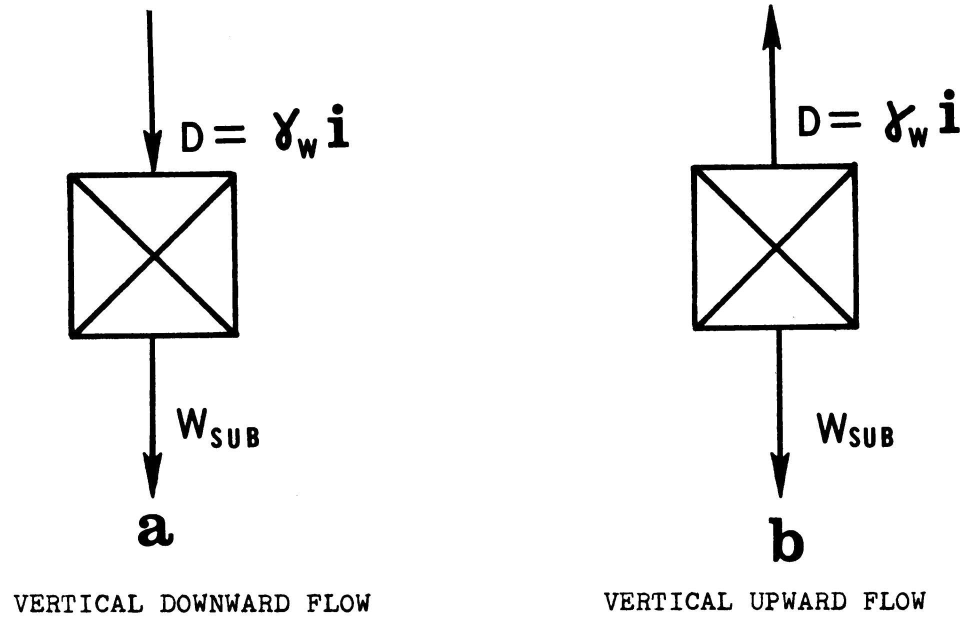


Fig. 1. Force diagrams for seepage force, D, and gravity force $W_{sub}$. Figure 1a is for vertical downward flow and figure 1b is for vertical upward flow.

which is the basic formula for the critical gradient in the absence of surcharge or cohesion. In the presence of surcharge and cohesion the formula becomes

$$i_{cr} = \frac{G-1}{1+e} + W_{sur} + W_{coh}$$

For sand having a specific gravity, $G$, of 2.65, and a void ratio, $e$, of about 0.65, the critical gradient is 1 in the absence of surcharge load and cohesion. Note that the critical gradient is independent of the soil hydraulic conductivity.

## THEORETICAL EXIT GRADIENTS

The hydraulic gradient at the place where the seeping water emerges from the soil is called the exit gradient.

To calculate the exit gradients into drain pipes, Kirkham's equation for hydraulic head potential and stream function (Luthin, 1957, pp. 160–61) were programmed for the computer. Basically Kirkham's equation is the solution for flow into a series of parallel drain tubes located beneath the soil surface at specified depths and above an impermeable layer. Factors such as drain outer diameter, depth to impermeable layer, depths of ponded water on the soil surface, depths of drain lines, and drain spacing are all taken into account in the equation. It is assumed that the soil is homogeneous and that the drain is running full with no back pressure. The origin of coordinates and reference plane for hydraulic head is taken at the center of the drains. The hydraulic head over the periphery of the drain is then equal to the drain radius. The walls of the drain are considered completely permeable. A sample flow net in the vicinity of a drain is shown in figure 2.

For the purposes of the computations presented here, it is assumed that the depth of ponded water is equal to 0; however, the water table is taken to

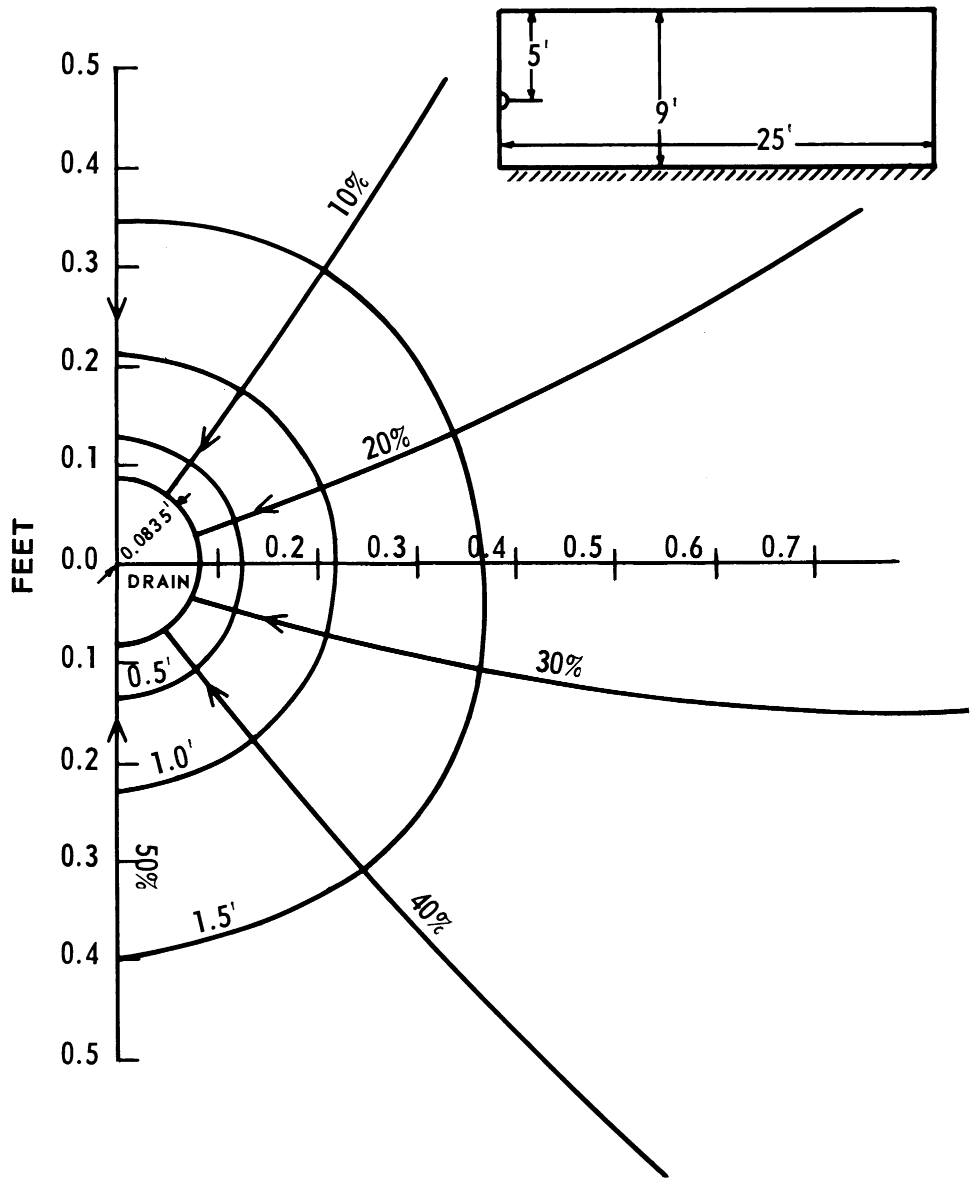


Fig. 2. Flow net in the vicinity of a drain having a diameter of 0.167 feet. The drain spacing is 50 feet, drain depth is 5 feet and depth of impermeable barrier is 9 feet.

coincide with the ground surface. It is recognized that this is a special condition; however, it corresponds to certain field conditions. As an example, the solution for a drain depth of 2 feet would correspond approximately to a situation in the field where the water table is approximately 2 feet above the drain lines even though the drains may be installed at a depth of 5 feet below the ground surface. The solutions presented here are for a flat water table. Under actual conditions the water table will have an elliptic shape. Hence the solutions presented here contain higher exit gradients than will be encountered

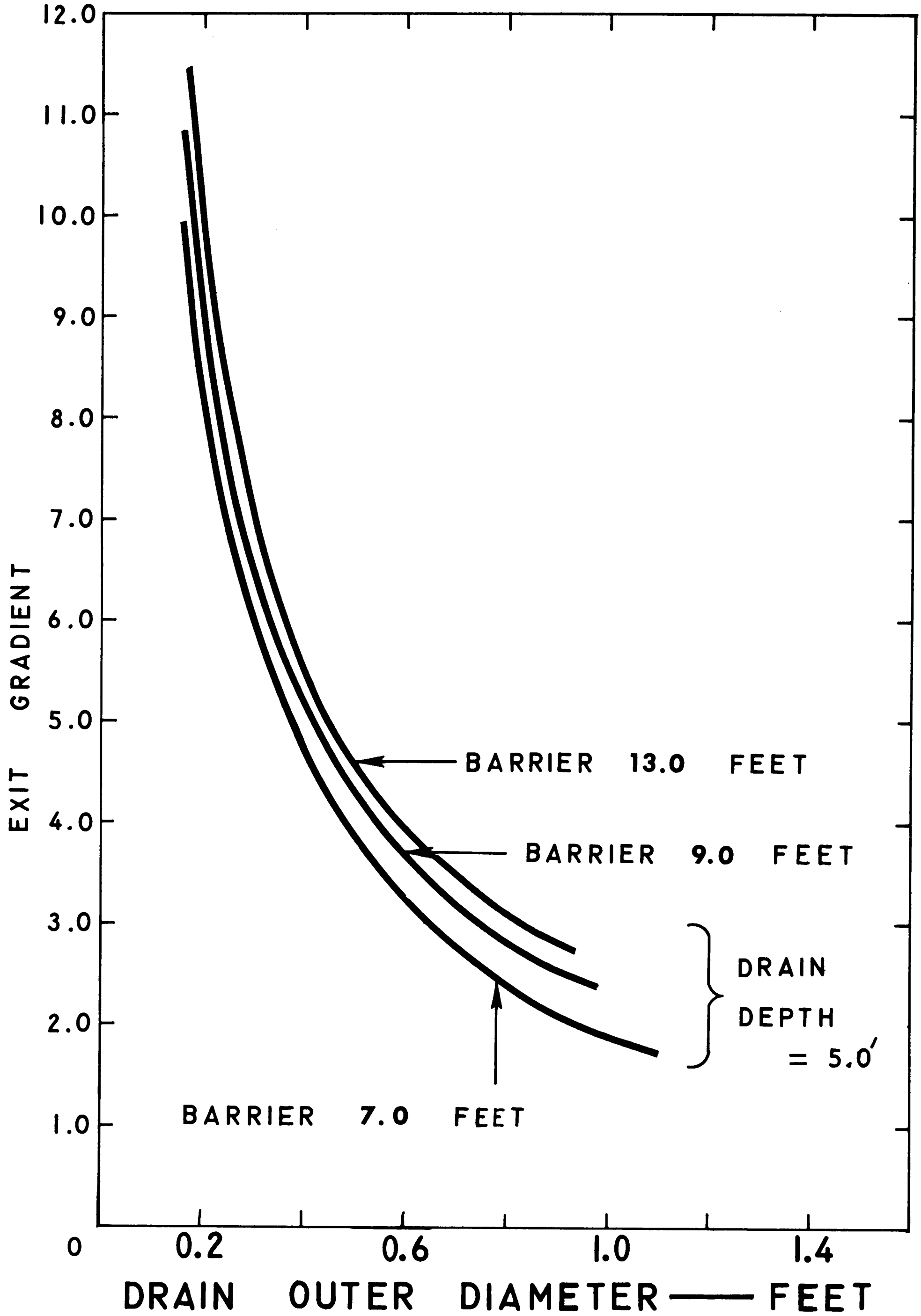


Fig. 3. Exit gradients plotted as a function of drain diameter for drains located 5 feet below the ground surface and having a barrier layer at 7, 9, and 13 feet below the soil surface.

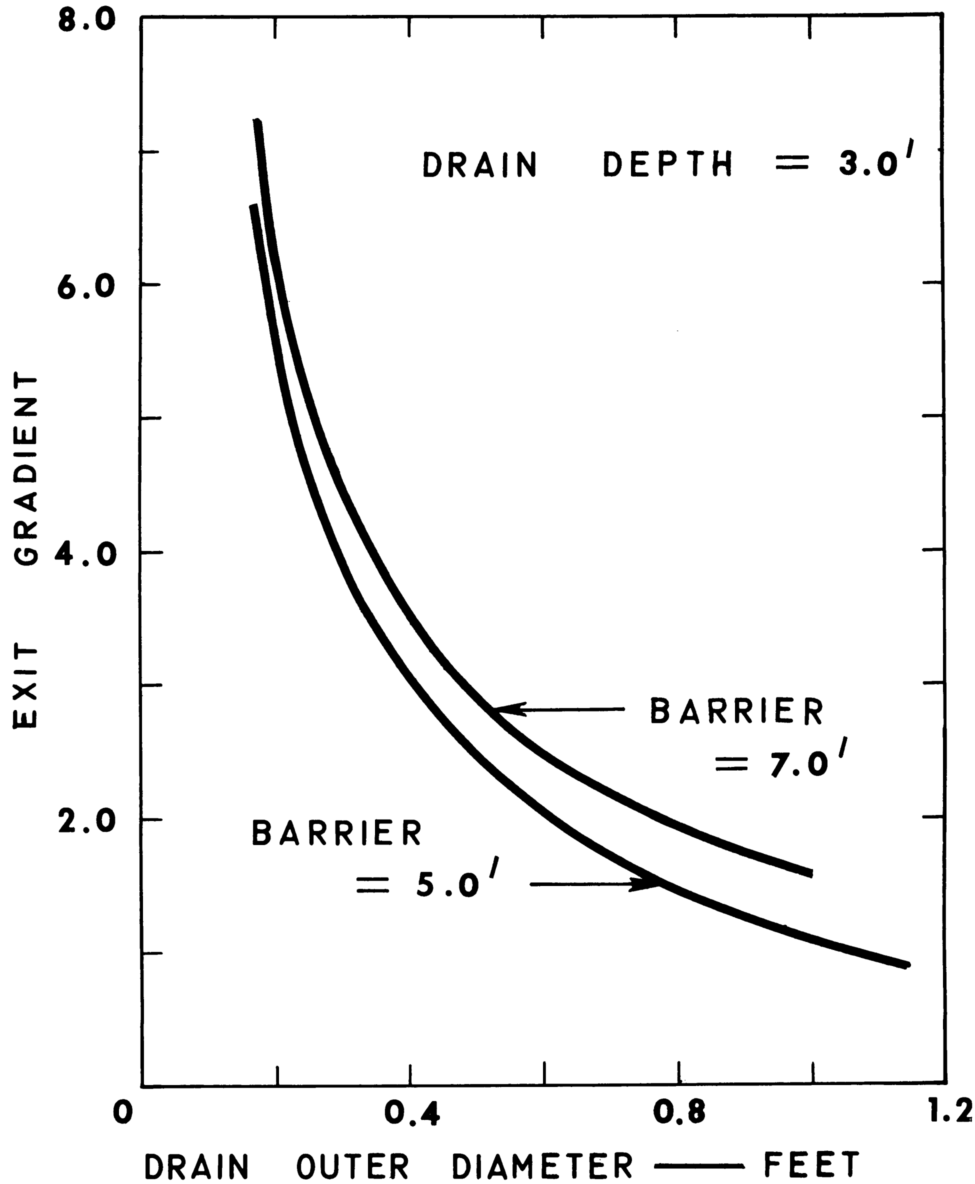


Fig. 4. Exit gradients plotted as a function of drain diameter for drains located 2 feet below the ground surface.

with an elliptic water table. The exit gradients were computed from a solution of Kirkham's equation for the ponded water case. As Kirkham has pointed out, his solution does not give accurate figures for a circular drain. The solution is for a drain which corresponds to an equipotential. When calculating exit gradients into the drain, the error inherent in Kirkham's solution can be significant. We adjusted the solutions in the following manner:

The hydraulic heads along the center line passing through the drain are

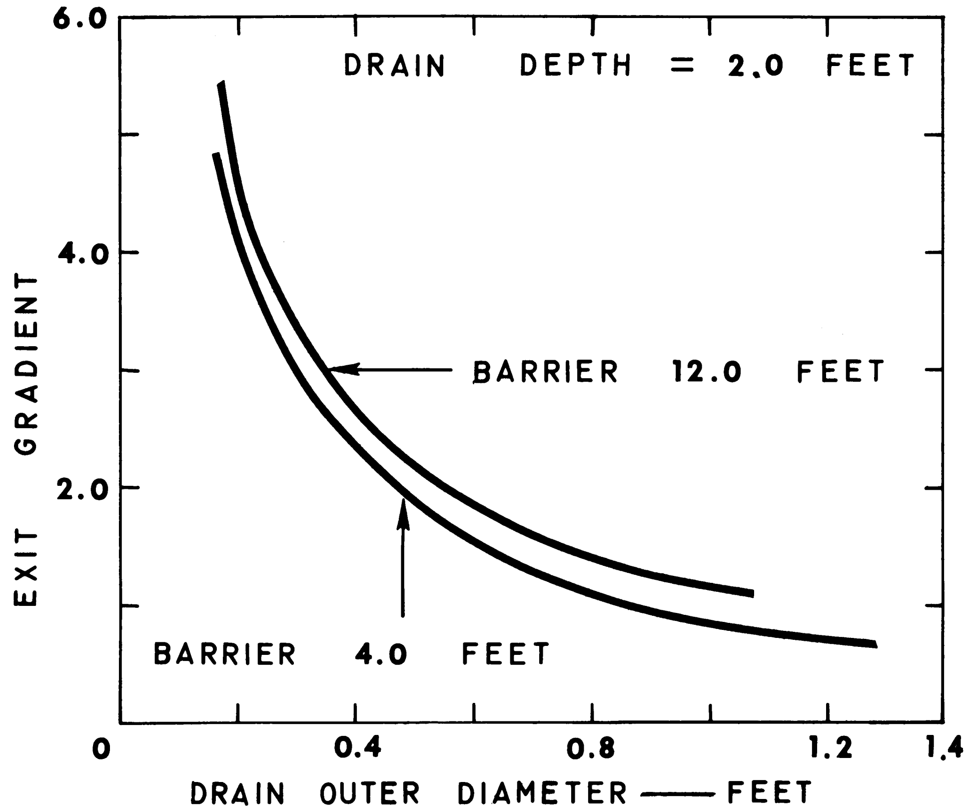


Fig. 5. Exit gradients plotted as a function of drain diameter for drains located 3 feet below the ground surface.

plotted on semilog paper for the situation below the drain. The results are extrapolated to a point where the hydraulic head is equal to the nominal radius of the drain. In other words, if the drain has a radius of 0.25 foot, then the hydraulic head in the drain is assumed to be 0.25 foot. A plot of hydraulic head versus radial distance from the origin enables us to extrapolate the radius through the point at which the hydraulic head is equal to 0.25 foot. In all cases, the actual drain radius is greater than the hydraulic head.

The procedure was repeated for every solution that was obtained. All of the radii were corrected for this discrepancy. The plot of exit gradients as a function of drain radius and drain depth shown is a result of plotting the actual radius as calculated in the above manner.

Figure 3 shows a plot of exit gradients as a function of drain diameter for drains installed at a depth of 5 feet and with a barrier layer at 7, 9, and 13 feet below the soil surface. For drain diameters of the order 0.2 foot the exit gradients are greater than 10 and hence far greater than the critical gradient for sand. Increasing the drain diameter by a factor of 5 to 1.0 foot reduces the exit gradient by a factor of about 5 to approximately 2.0, still above the critical gradient for sand. The exit gradient is increased by an increase in the depth of the barrier layer.

With drains at a depth of 3 feet below a flat water table the exit gradient is

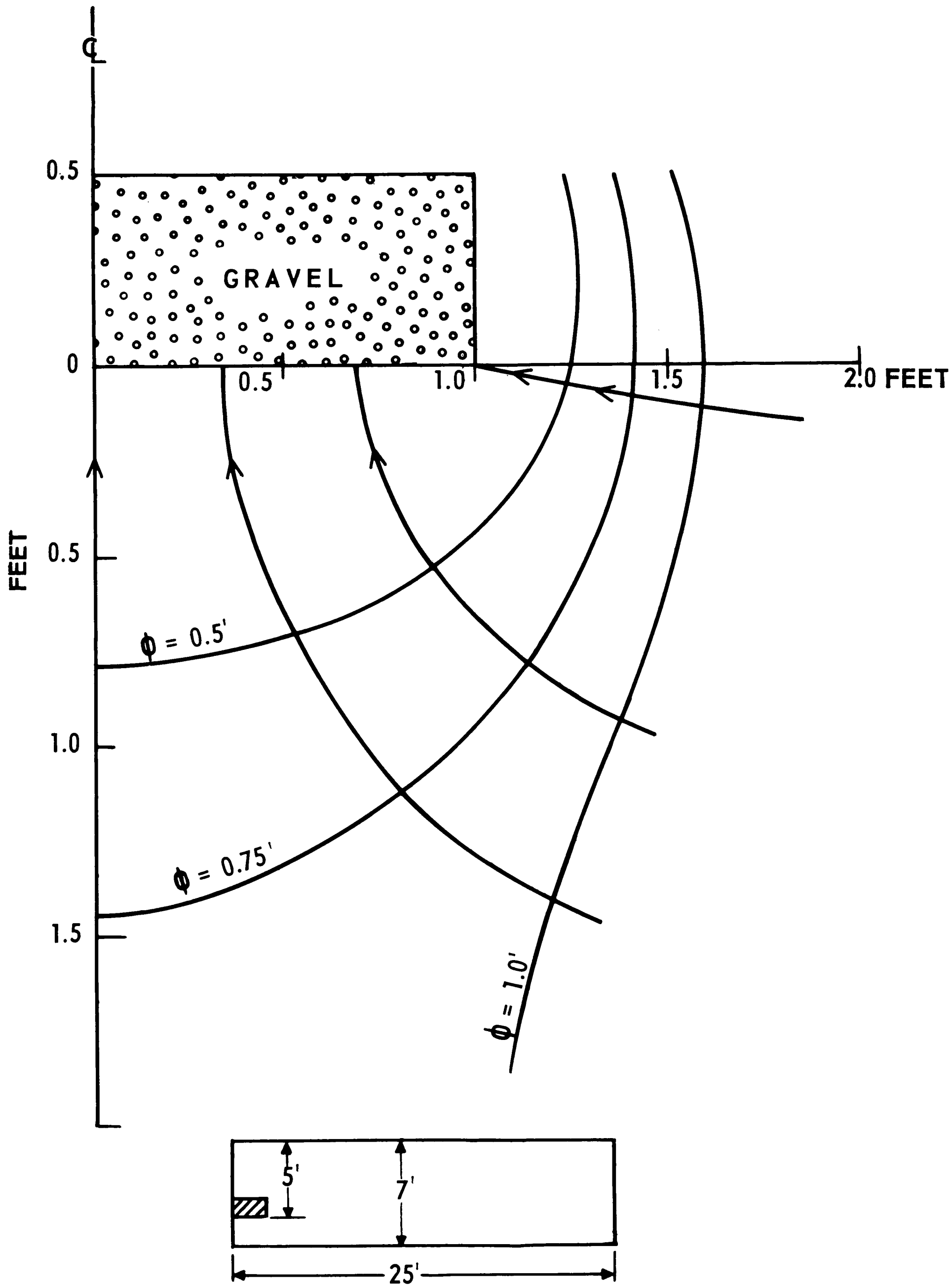


Fig. 6. Flow net in the vicinity of a gravel envelope. The gravel envelope is considered to be completely permeable and is 2 feet wide and 0.5 feet high. The streamlines, sketched in, have no quantitative significance.

reduced to a range of 7 for a drain diameter of 0.2 foot to about 1.5 for the larger drain diameter of 1.0 foot. The presence of a barrier layer close to the drain can cause a significant reduction in the exit gradient (see figure 4).

For drains 2 feet below a flat water table, figure 5, the exit gradients are reduced to about 5 for a drain diameter of 0.2 foot and about 1 for a drain diameter of 1.0 foot. Once again a barrier close to the drains can lower the exit gradient substantially.

One case involving a gravel envelope was investigated. It was assumed that the bottom of a 24-inch-wide trench was filled to a depth of 6 inches with gravel. While this situation is not of immediate practical significance the results are interesting. The solution shown in figure 6 was obtained by use of the electrical resistance network. The exit gradient into the gravel was below 1 at the center line of the envelope but increased to maximum of about 2 at the outer edge. The results show that enlarging the effective diameter of the drain by a gravel envelope will result in lowering the exit gradient.

## SUMMARY

Exit gradients were investigated for flow into circular drain pipes of various outer diameters. Kirkham's equation for the distribution of hydraulic head and the stream function around buried drain pipes in soil under ponded conditions was programmed for the digital computer and solutions were obtained for a number of situations of interest. The program simultaneously gives the hydraulic head function, the stream function, and the flow rate into the drain. From these flow nets the exit gradients were calculated as a function of drain depths, drain outer diameter, and depths to the impermeable barrier. In all cases the drain spacing was taken constant as 50 feet. A comparison was then made with one case of a gravel envelope 6 inches high and 24 inches wide. The results of the analysis indicate that exit gradients in excess of critical gradients for fine sand are encountered in almost every case. The exit gradient increases as the drain diameter decreases. Depths to the impermeable barrier is a factor in that the exit gradient decreases as the impermeable barrier approaches the bottom of the drain pipe. The use of a gravel envelope substantially decreases the exit gradient. The effect of perforation spacing was not investigated. The drains are considered to be completely permeable.

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