2. Acquisition and Use of Geotechnical Information. NCHRP, Synthesis of Highway Practice 33, 1976, 40 pp.
3. G. D. Goldberg. Development of the Computerized Geotechnical Data Bank for the State of Indiana. Purdue Univ., West Lafayette, IN, M.S.C.E. thesis; Joint Highway Research Project, Rept. 78-6, May 1978, 163 pp.
4. Soil Survey Manuals. Soil Conservation Service, U.S. Department of Agriculture, 1958-present.
5. D. Spradling. A Soils Data System for Kentucky. Bureau of Highways, Kentucky Department of Transportation, Frankfort, Res. Rept. 441, 1976, 58 pp.
6. T. K. Liu and T. H. Thornburn. Engineering Index Properties of Some Surficial Soils in Illinois. College of Engineering, Univ. of Illinois at Urbana, Bull. 477, 1965.
7. P. Lumb. The Variability of Natural Soils. Canadian Geotechnical Journal, Vol. 3, No. 2, May 1966, pp. 74-97.
8. P. Lumb. Application of Statistics in Soil Mechanics. In Soil Mechanics—New Horizons (I. K. Lee, ed.), Elsevier, New York, 1974, pp. 44-111.
9. D. J. A. van Zyl. Storage, Retrieval, and Statistical Analysis of Indiana Shale Data. Joint Highway Research Project, Purdue Univ., West Lafayette, IN, 140 pp.
10. V. Rizkallah and E. El Nimr. Applicability of Regression Analysis in Soil Mechanics with the Help of Data Banks. Proc., 2nd International Conference on the Application of Statistics and Probability in Soil and Structural Engineering, Aachen, Federal Republic of Germany, Vol. 2, 1975, pp. 423-438.
11. M. Van Rooyen. Intercorrelation of Engineering Properties of Soil. Proc., 4th Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Cape Town, South Africa, 1967, pp. 203-208.
12. J. Neter and W. Wasserman. Applied Linear Statistical Models. Irwin, Homewood, IL, 1974, 842 pp.

Publication of this paper sponsored by Committee on Engineering Geology.

Design of Subsurface Drainage Systems for Control of Groundwater

Lyle K. Moulton, Department of Civil Engineering, West Virginia University, Morgantown

In recent years, awareness has grown of the need for subsurface drainage systems that can drain water from the pavement structural system. Much of the emphasis associated with studies of this subject has been on the removal of the moisture that infiltrates through the surface of the pavement, but it has also been recognized that the control of groundwater is an essential part of any effective highway subsurface drainage system. In this paper, rational analytical methods for the design of subsurface drainage systems for the control of groundwater are developed and presented. Although these methods are, in general, approximate in nature, they are soundly based on fundamental seepage theory. The resulting solutions have been used to prepare graphical design aids that can be readily applied by the highway designer. The use of these design aids is illustrated by a series of examples, and the results are compared with more-exact flow-net solutions obtained by the use of electric analogs. On the basis of this comparison, it concluded that the proposed design procedures, although approximate, do permit the development of good practical designs for subsurface drainage systems for the removal and/or control of groundwater in highway applications.

In recent years, there has been a growing awareness of the need for subsurface drainage systems that can drain water from a pavement structural system and thus minimize detrimental effects. Workshops dealing with water in pavements (1) have been conducted, and guidelines for the design of subsurface drainage systems for pavement structural sections have been published (2, 3). Although much of the emphasis of these activities has been on the removal of the moisture that infiltrates through the surface of the pavement, it has also been recognized (3) that the control of groundwater is an essential part of any effective highway subsurface drainage system.

Commonly, the design of groundwater drainage systems is based on empirical rules of thumb that have been developed by trial and error over a period of years or on rather tedious graphical techniques involving the use of flow nets (4). The purpose of this paper is to present some rational, approximate analytical methods for the design of groundwater control systems such as the interceptor drains shown in Figures 1 and 2 and the symmetrical drawdown drains shown in Figure 3. Although, at present, it is not possible to eliminate all elements of empiricism, the methods presented are based on fundamental seepage theory.

LONGITUDINAL INTERCEPTOR DRAINS

Calculation Method

Let us consider the case of the unconfined flow of groundwater over a sloping impervious boundary toward a single interceptor drain, as illustrated in Figure 4. A solution for the shape of the drawdown curve for this situation, which was developed by R. E. Glover of the U.S. Bureau of Reclamation, is given by Donnan (5). This solution, which is based on an adaptation (6) of Dupuit theory, has the form

$$x = \{H \ell n[(H - H_0)/(H - y)] - (y - H_0)\}/S \qquad (1)$$

where

x and y = coordinates of a point on the drawdown

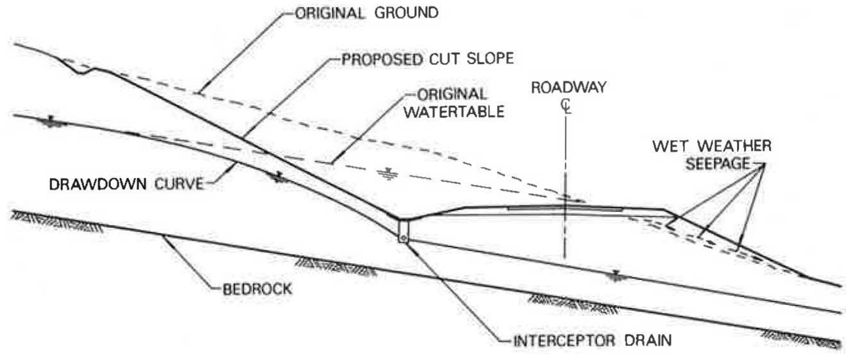


Figure 1. Longitudinal interceptor drain used to cut off seepage and lower the groundwater table.

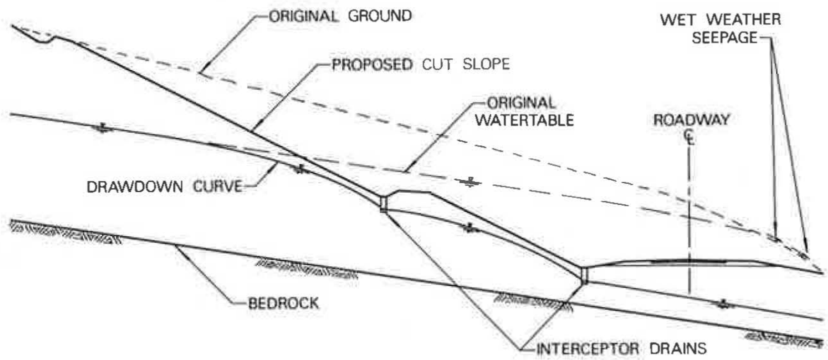


Figure 2. Multiple interceptor drain.

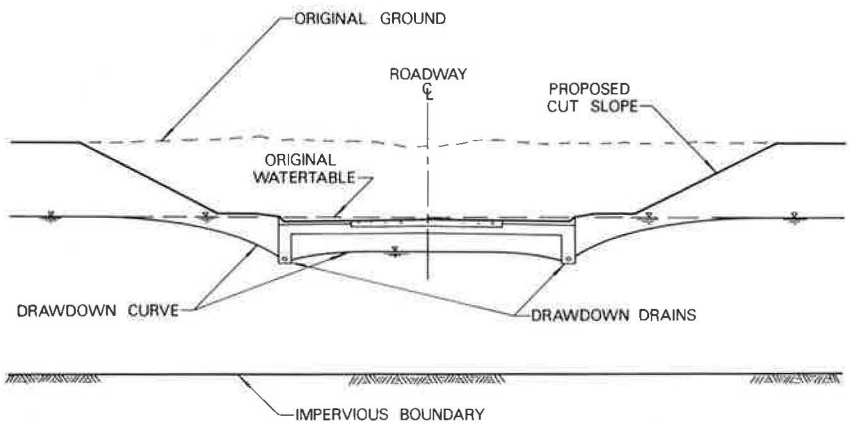


Figure 3. Symmetrical longitudinal drains used to lower the water table.

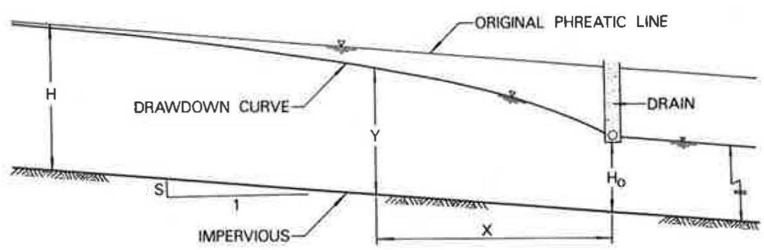


Figure 4. Flow toward a single interceptor drain.

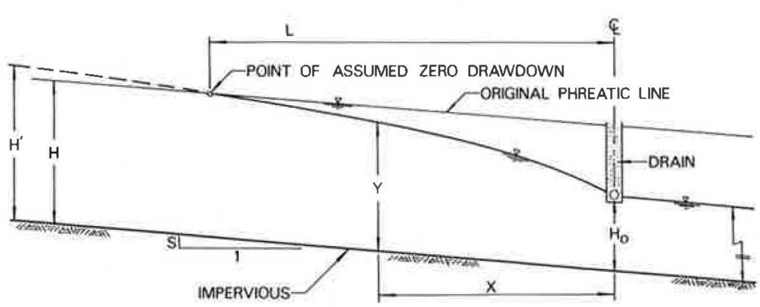


Figure 5. Flow toward a single interceptor drain when the drawdown can be considered to be insignificant at distance L from the drain.

curve, as shown in Figure 4;

H = height of the original groundwater table above an impervious boundary of slope S; and

H_0 = height of the drain above the impervious boundary.

Examination of Figure 4 and Equation 1 shows that the drawdown curve becomes asymtotic to the original free-water surface (phreatic line) at infinity. Dealing with this boundary condition in practical problems is awkward and, consequently, most solutions to gravity-flow problems of this type have assumed that there is a finite distance (L) from the drain at which the drawdown can be considered to be insignificant and at which, for practical purposes, $y = H$, as shown in Figure 5. In well theory, L is generally referred to as the radius of influence.

In an investigation of interceptor drains of this type, Keller and Robinson (7) conducted a laboratory study in which, for practical purposes, the conditions shown in Figure 5 were duplicated by the use of a finite source of seepage located at distance L from the drain. They found that Glover's equation, i.e., Equation 1, checked the experimental data when modified into the form

$$Sx = H' \ln[(H' - H_0)/(H' - y)] - (y - H_0) \quad (2)$$

where H' = a point on a fictitious extension of the drawdown curve, as shown in Figure 5. Then, because $y = H$ when $x = L$,

$$SL = H' \ln[(H' - H_0)/(H' - H)] - (H - H_0) \quad (3)$$

and H' can be determined for known values of S, L, H, and H_0.

Keller and Robinson also found that the quantity of flow into the drain (q_d) could be determined from the relationship

$$q_d = q_0 (H' - H_0)/H \quad (4)$$

where q_0 = magnitude of the approach flow and is given by

$$q_0 = kHS \quad (5)$$

where k = coefficient of permeability of the porous medium. A complete solution to the problem can thus be obtained by using Equations 2, 3, 4, and 5. For convenience, Equations 3 and 4 have been combined in dimensionless form and solved by computer to prepare Figure 6, from which q_d/kHS and H'/H can be determined in terms of known values of SL/H and H_0/H. The same computations also provided the data by which, through a change of variables, Figure 7 was prepared. Figure 7 permits determination of the location of the drawdown curve by giving values of Sx/y for known values of H_0/y and $(H' - H_0)/y$. In practice, a series of values of y (between H_0 and H) are assumed, and Figure 7 is used to assist in the determination of the corresponding values of x.

In order to use Figures 6 and 7 for any highway drainage problem, it is necessary to have an estimate of the value of L. One method for estimating this value might be through the use of the Sichardt (8) equation,

$$L = C(H - H_0)\sqrt{k} \quad (6)$$

which has been widely accepted and used in connection with pumped wells and dewatering systems (9). The value of the coefficient C in Equation 6 is dependent in part on the units of H, H_0, and k. For example, the value proposed by Sichardt (8) would be approximately 6 if H and H_0 were in feet and k were in feet per day. However, a series of experimental flow nets for typical interceptor drain problems constructed by use of an electric analog, suggests that the value of L is independent of k and dependent only on the geometry of the problem. Although this study is at present incomplete, it suggests that, for the range of drawdowns and slopes commonly encountered in interceptor and drawdown-drain problems, the value of L can be estimated, for practical purposes, from the relationship

$$L = 3.8(H - H_0) \quad (7)$$

For the purposes of this paper, Equation 7 has been adopted as the method for estimating the value of L. However, it is anticipated that, on completion of the experimental flow-net analyses, some refinement to this relationship might be forthcoming.

Example 1

Let us consider the proposed construction shown in Figure 1 and, for this problem, (a) compute the reduced flow rate (q_d/k) into the drain and (b) plot the location of the drawdown curve (free-water surface). The detailed dimensions of the problem are given in Figure 8. To keep the left branch of the free-water surface from breaking out through the cut slope and to lower the right branch of the free-water surface well below the pavement structural system, the underdrain was set below the ditch line at a depth of 1.5 m (5 ft). It is proposed to pave the ditch over the drain to avoid infiltration and clogging.

From Equation 7, $L = 3.8(H - H_0) = 3.8(4.27) = 16.2$ m (53.2 ft).
From Figure 6, if $SL/H = 0.15(16.2)/6.1 = 0.398$ and $H_0/H = 1.83/6.1 = 0.3$, $q_d/kHS = 1.57$ and $H'/H = 1.84$ [therefore, $H' = 1.84(6.1) = 11.22$ m (36.8 ft)].

Reduced Flow Rate

Thus, $q_d/k = 1.57HS = 1.57(6.1 \times 0.15) = 1.44$ m (4.71 ft). The reduced flow rate could also be computed from the flow net (see Figure 8), i.e., $q_d/k = \Delta H N_f/N_d = 6.4(6)/28 = 1.37$ m (4.50 ft).

Drawdown Curve

From Figure 7, if $H' = 11.22$ m and the following values are assumed for the y coordinates, the x coordinates of the drawdown curve can be determined as follows (1 m = 3.28 ft):

y (m)	H_0/y	$(H' - H_0)/y$	Sx/y	x (m)
2.26	0.811	4.16	0.041	0.60
2.68	0.682	3.48	0.080	1.43
3.11	0.588	3.02	0.117	2.43
3.54	0.517	2.66	0.149	3.52
3.96	0.462	2.37	0.190	5.02
4.39	0.417	2.14	0.226	6.61
4.82	0.380	1.95	0.265	8.52
5.24	0.349	1.79	0.310	10.83
5.67	0.323	1.66	0.350	13.23

This drawdown curve is shown as the dashed curve in Figure 8; it is only approximate, but can be used as a starting point for constructing the flow net that ultimately yields a more accurate location of the free-water surface.

MULTIPLE INTERCEPTOR DRAINS

Calculation Method

A subsurface drainage system consisting of multiple interceptor drains (such as that shown in Figure 2) can be designed by using the principles outlined above and considering each drain separately. However, to properly define the boundary conditions for each of the upper drains correctly, it is necessary to establish the location of the limiting streamline above which the flow pattern is essentially that of a single drain installed in the flow domain above a sloping impervious boundary. In essence, this establishes an impervious boundary for each upper drain roughly parallel to the lower sloping impervious boundary. Flow-net studies conducted by using an electric analog have shown that boundaries of this type can be established by drawing a line parallel to the sloping impervious boundary and located at a depth below the drain equal to $\frac{1}{10}$ to $\frac{1}{12}$ of the drain spacing. This is an adaptation of the generalized method of fragments, which, according to Aravin and Numerov (10), was first proposed by Pavlovsky in Russia in 1935 and was introduced into the United States, for fragments in series, by Harr (6) in 1962. In this instance, the flow fragments are considered to be in parallel.

Figure 6. Chart for determining flow rate in interceptor drains.

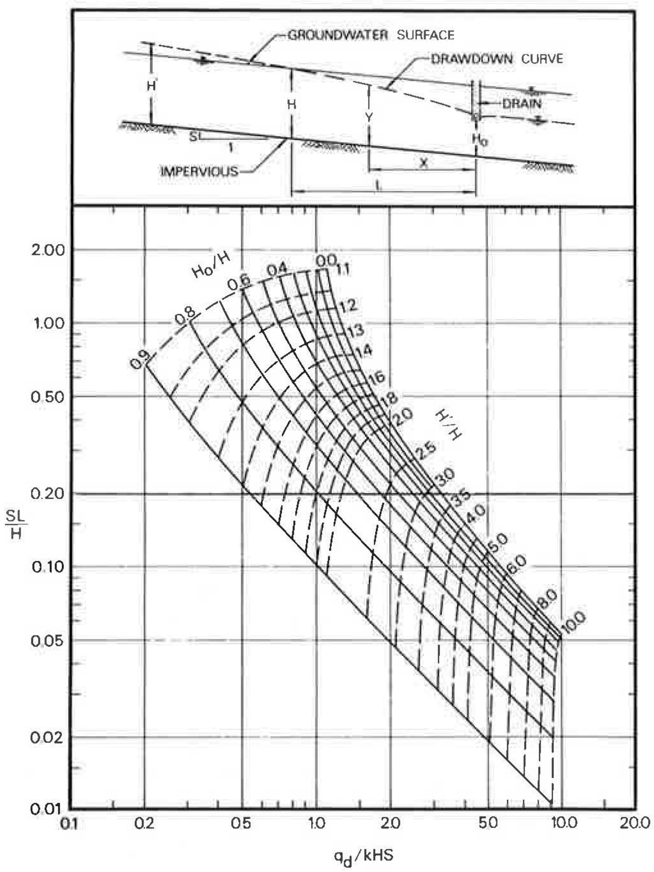


Figure 7. Chart for determining drawdown curves for interceptor drains.

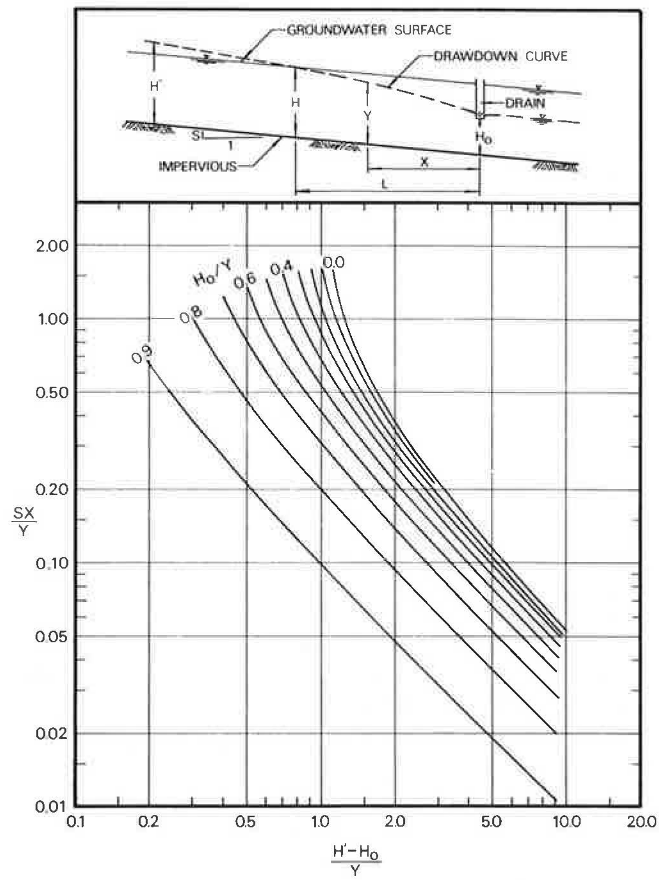


Figure 8. Example 1: flow net, dimensions, and details.

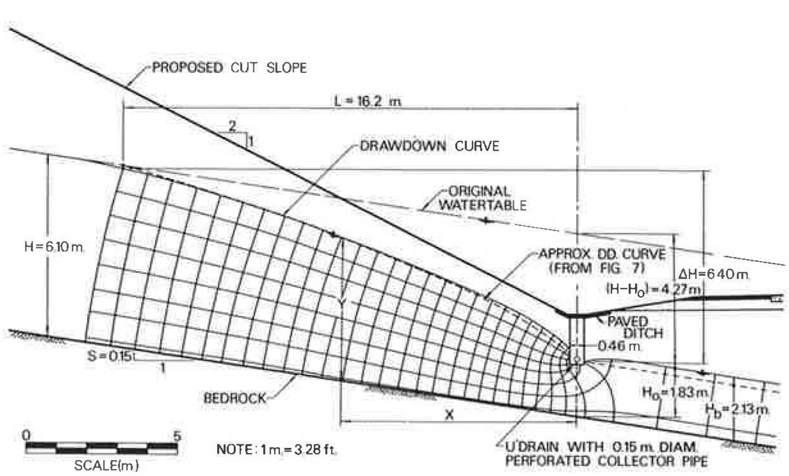


Figure 9. Example 2: dimensions and details required for the use of Figures 6 and 7.

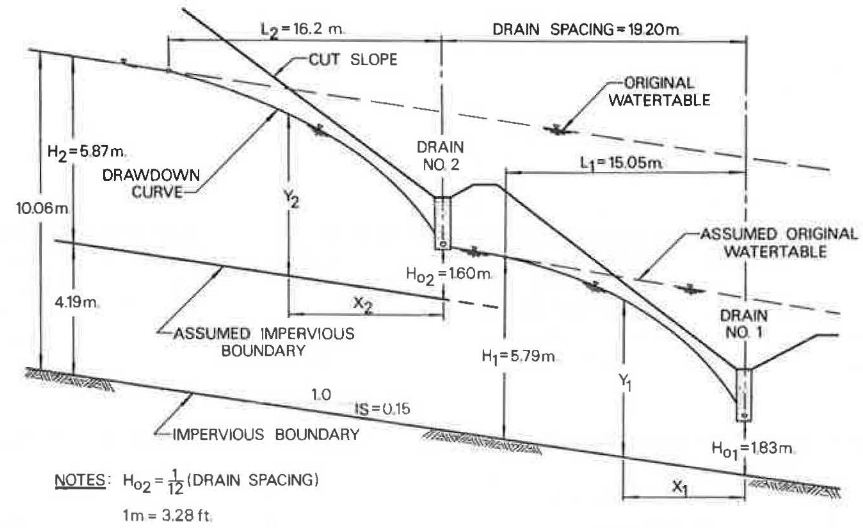
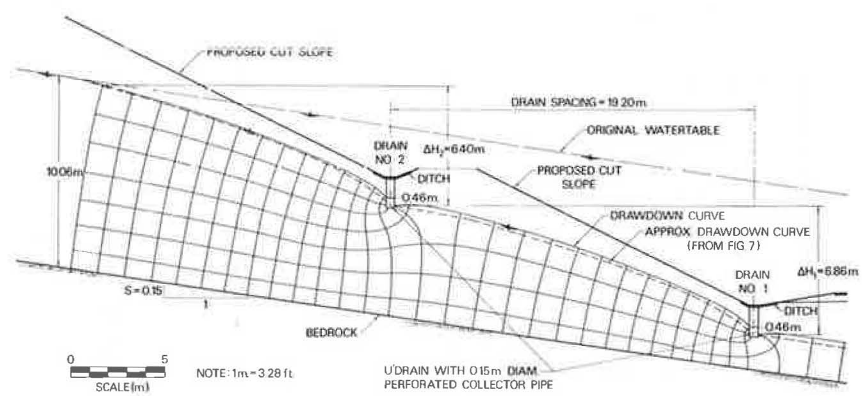


Figure 10. Example 2: flow net, dimensions, and details.



Example 2

Let us consider the proposed construction situation shown in Figure 2, which represents a deeper portion of the cut shown in Figure 1. This situation requires two drains to cut off and drawdown the water table to prevent it from breaking out through the slope and to keep water from this source out of the pavement structure. The detailed dimensions of the problem are shown in Figures 9 and 10. The locations and depths of the drains were established by trial, taking into consideration the desirability of maintaining the free-water surface below the cut slope. The dimensions given in Figure 9 are those required to solve the problem by using the method of fragments and Figures 6 and 7.

From Equation 7, $L_1 = 3.8(H_1 - H_{01}) = 3.8(5.79 - 1.83) = 15.05$ m (49.4 ft) and $L_2 = 3.8(H_2 - H_{02}) = 3.8(5.87 - 1.60) = 16.2$ m (53.2 ft).

For drain 1, Figure 6 shows that, for $SL_1/H_1 = 0.15(15.05/5.79) = 0.389$ and $H_{01}/H_1 = 1.83/5.79 = 0.316$, $q_{d1}/kH_1S = 1.58$ and $H_1'/H_1 = 1.90$ [therefore $H_1' = 1.90(5.79) = 11.0$ m (36.1 ft)].

Similarly, for drain 2, $q_{d2} = kH_2S = 1.57$ and $H_2' = 10.85$ m (35.6 ft).

Reduced Flow Rate

Thus, $q_{d1}/k = 1.57H_1S = 1.57(5.87 \times 0.15) = 1.37$ m and $q_{d2} = 1.57H_2S = 1.57(5.87 \times 0.15) = 1.38$ m (4.53 ft).

Or, for comparison purposes (see Figure 10), based on the flow net, $q_{d1} = H_1N_{f1}/N_{d1} = 6.86(3)/15 = 1.37$ m and $q_{d2}/k = H_2N_{f2}/N_{d2} = 6.3(3)/14 = 1.37$ m.

Drawdown Curves

The method illustrated in example 1 was used with the data shown in Figure 9 and the chart shown in Figure 7 to determine the locations of the x_1, y_1, and x_2, y_2 coordinates of the drawdown curve. The resulting curve was then plotted as the dashed line in Figure 10. It can be seen that the agreement between this approximate curve and the more exact free-water surface generated by the flow-net solution is quite good.

SYMMETRICAL DRAWDOWN DRAINS

Calculation Method

To solve a problem such as that shown in Figure 3, the method of fragments can be used by breaking the flow domain into fragments, as shown in Figure 11. Basically, this amounts to assuming that there is a horizontal streamline existing at the level of the drain. Flow-net analyses have shown that this is not an unreasonable assumption.

The quantity of flow into the drain from fragment 1 (q_1) can be estimated by using Dupuit theory (6) to be

$$q_1 = k(H - H_0)^2/2(L - b) \tag{8}$$

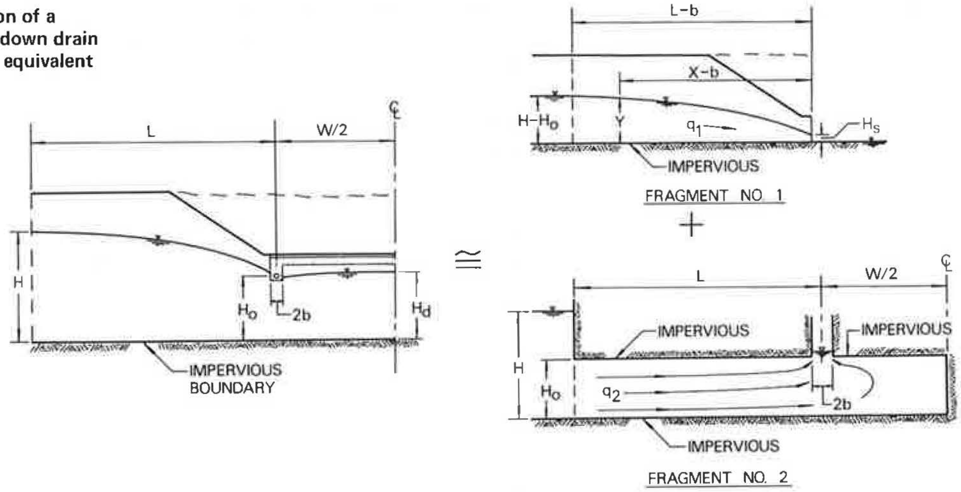


Figure 11. Division of a symmetrical drawdown drain problem into two equivalent fragments.

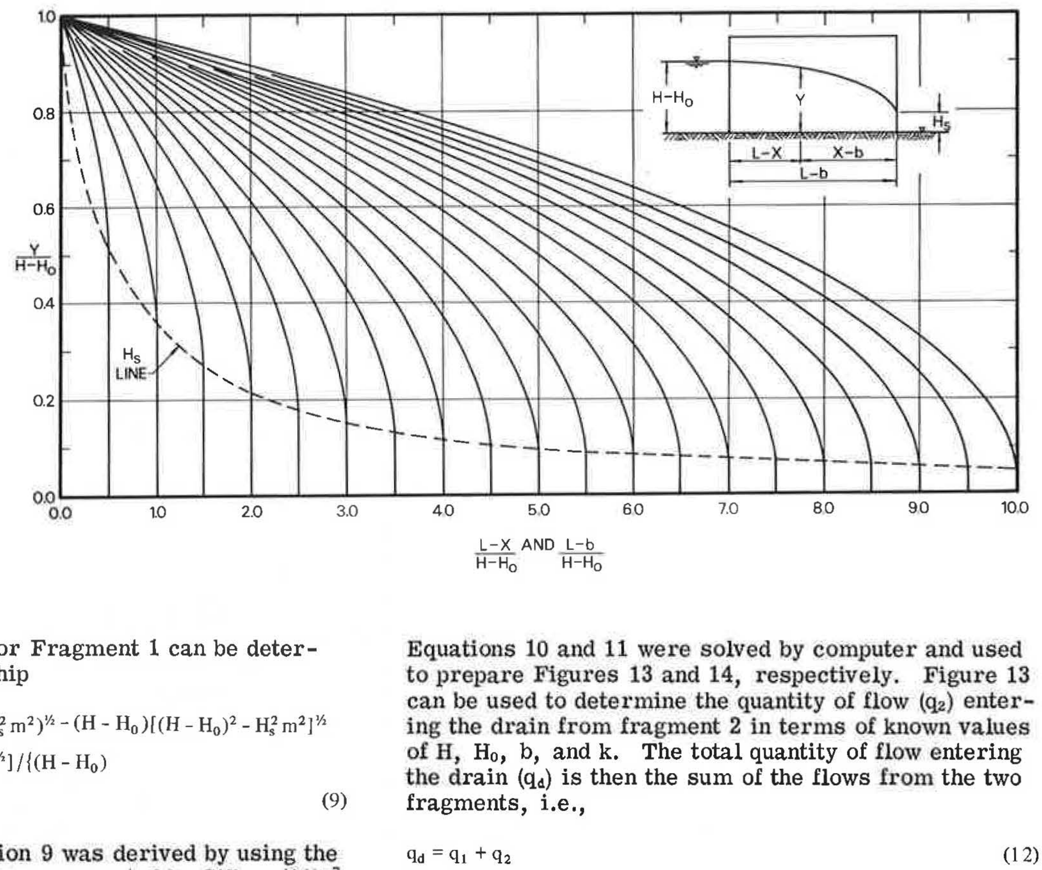


Figure 12. Free-water surfaces based on Gilboy modification of Dupuit theory.

The drawdown curve for Fragment 1 can be determined from the relationship

$$x = (L - b) + (1/2H_s m)\Big[y(y^2 - H_s^2 m^2)^{1/2} - (H - H_0)[(H - H_0)^2 - H_s^2 m^2]^{1/2} - H_s^2 m^2 \ln\big([y + (y^2 - H_s^2 m^2)^{1/2}]/\{(H - H_0) + [(H - H_0)^2 - H_s^2 m^2]^{1/2}\}\big)\Big] \quad (9)$$

where $m = 0.43\pi$. [Equation 9 was derived by using the modification of Dupuit theory suggested by Gilboy (11).] For convenience, Equation 9 has been put into dimensionless form and solved by computer to prepare Figure 12, which can be used to determine the x and y coordinates of the drawdown curve.

The solution to the problem represented by fragment 2 in Figure 11 has been given by Aravin and Numerov (10), who showed that the flow rate (q_2) for this situation can be computed from the relationship

$$q_2 = k(H - H_0)/\{(L/H_0) - (1/\pi)\ln[(1/2)\sinh(\pi b/H_0)]\} \quad (10)$$

and that the value of the piezometric head at the roadway centerline ($H_d - H_0$) can be determined from the relationship

$$(H_d - H_0) = (q_2/\pi k)\ln[\coth(\pi b/2H_0)] \quad (11)$$

Equations 10 and 11 were solved by computer and used to prepare Figures 13 and 14, respectively. Figure 13 can be used to determine the quantity of flow (q_2) entering the drain from fragment 2 in terms of known values of H, H_0, b, and k. The total quantity of flow entering the drain (q_d) is then the sum of the flows from the two fragments, i.e.,

$$q_d = q_1 + q_2 \quad (12)$$

In the method of solution proposed here, it is assumed that the right branch of the drawdown curve can be approximated by the piezometric level along the upper boundary of fragment 2. Thus, Figure 14 can be used to estimate the location of the drawdown curve between the drain and the roadway centerline.

Example 3

Let us consider the proposed construction of a two-lane depressed roadway in an urban area, as shown in Figure 3. In connection with this proposed construction, it is desired to design a system of symmetrical longitudinal underdrains to draw the groundwater down as far as possible below the bottom of the granular base course. The detailed dimensions of the problem are

Figure 13. Chart for determining flow rate in symmetrical underdrains.

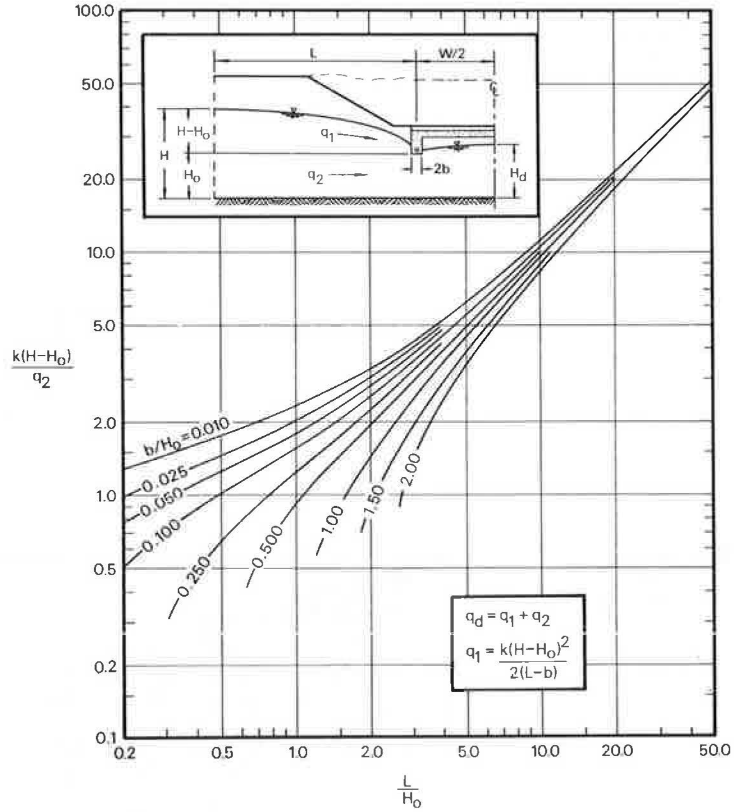
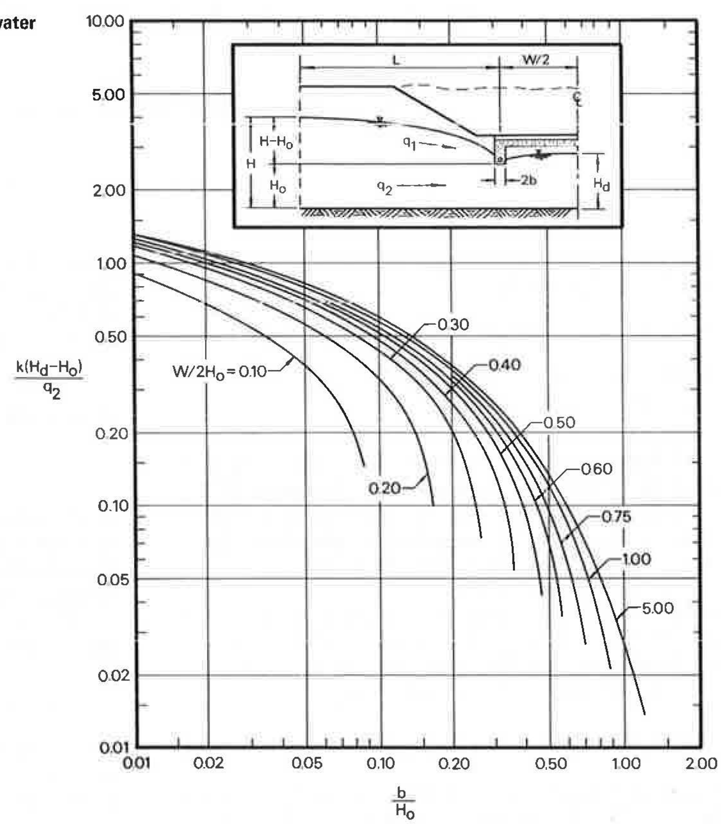


Figure 14. Chart for determining maximum height of free-water surface between symmetrical underdrains.



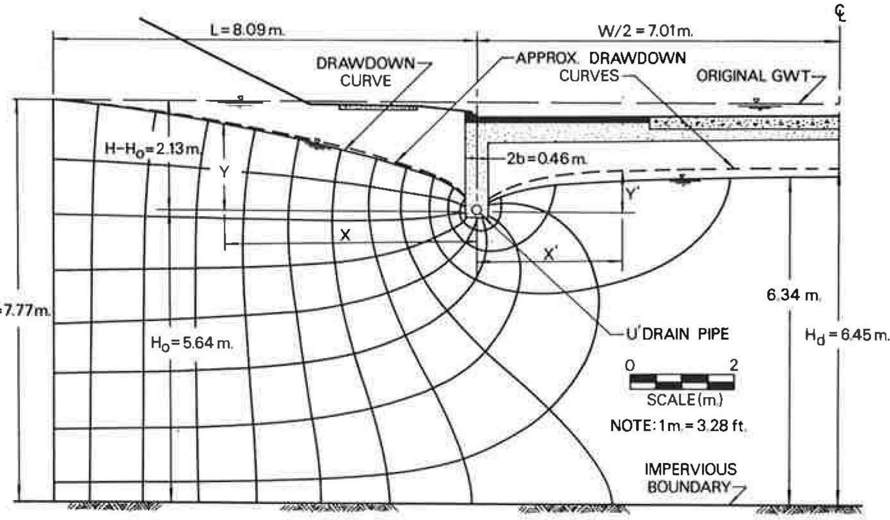


Figure 15. Example 3: flow net, dimensions, and details.

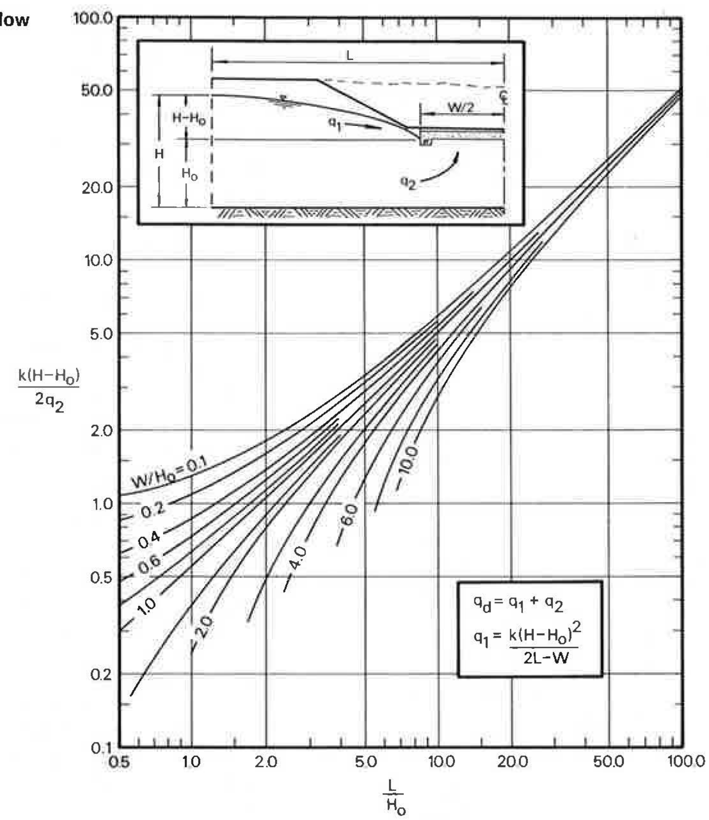


Figure 16. Chart for determining flow rate in horizontal drainage blanket.

shown in Figure 15. The depth of the drains was established by trial, taking into consideration the desirability of producing the maximum drawdown without requiring excessively deep excavation (the trench depth below the bottom of roadway excavation was limited to 1.5 m).

From Equation 7, $L = 3.8(H - H_0) = 3.8(2.13) =$ 8.09 m (26.5 ft).

Reduced Flow Rate

From Figure 13, for $b/H_0 = 0.23/5.64 = 0.041$ and $L/H_0 = 8.09/5.64 = 1.43$, it is found that $k(H - H_0)/q_2 = 2.30$. Thus, $q_2/k = (H - H_0)/2.30 = 2.13/2.30 = 0.926$ m (3.04 ft).

Then, from Equation 8, $q_1/k = (H - H_0)^2/2(L - b) = 2.13^2/2(8.09 - 0.23) = 0.289$ m (0.945 ft).

Therefore, the total reduced flow rate to the drain becomes, from Equation 12, $q_d/k = q_1/k + q_2/k = 0.926 + 0.289 = 1.28$ m (4.20 ft).

Or (for comparison), based on the flow net shown in Figure 15, if $\Delta H = (H - H_0) = 2.13$ m (7.0 ft), $q_d/k = \Delta H N_f/N_d = 2.13\ (7.4)/11.8 = 1.33$ m (4.38 ft).

Drawdown Curves

The right branch of the drawdown curve can be determined by taking various values of x' in Figure 15 as W/2 in Figure 14 and considering y' in Figure 15 as $(H_d - H_0)$ in Figure 14 as follows (noting that b/H_0 =

0.041 and q_2/k = 0.926 m) (1 m = 3.28 ft):

$x' = W/2$ (m)	$W/2H_0$	$k(H - H_0)/q_2$	$y' = (H_d - H_0)$ (m)
0.61	0.108	0.47	0.44
1.22	0.216	0.64	0.59
1.83	0.324	0.73	0.68
2.44	0.432	0.78	0.72
4.57	0.811	0.85	0.79
7.01	1.243	0.87	0.81

Then, from Figure 15, the left branch of the drawdown curve can be determined from Figure 12 by noting that $(L - b)/(H - H_0) = (8.09 - 0.23)/2.13 = 3.69$. Thus, for various values of y, the values of x can be determined by using Figure 12 as follows:

$y/(H - H_0)$	$(L - x)/(H - H_0)$	(L - x) (m)	y (m)	x (m)
0.13	3.69	7.86	0.28	0.23
0.20	3.65	7.77	0.43	0.32
0.40	3.27	6.97	0.85	1.12
0.60	2.50	5.33	1.28	2.76
0.80	1.38	2.94	1.70	5.15
1.00	0	0	2.13	8.09

These approximate drawdown curves are shown dashed in Figure 15. It can be seen that, although this method produces a free-water surface that is slightly high, the agreement between it and the more exact free-water surface produced by flow-net analysis is reasonable.

For the special case where the underdrain cannot be placed sufficiently deep to draw down the groundwater table below the granular drainage blanket of the pavement system, the flow rate to this layer can be estimated by using Figure 16. Figure 16 was prepared by using Equation 10 with L as defined in the figure and b = W/2.

CONCLUSIONS

On the basis of the comparison between the solutions obtained by the approximate rational methods presented in this paper and those obtained by the use of the more-exact flow nets, it can be concluded that the proposed methods do permit the development of reasonably good practical designs for the removal and/or control of groundwater in highway applications. However, a few limitations of the proposed methods should be noted.

The solutions are based on the assumption that the soil is homogeneous and isotropic. The problems offered by layered or by anisotropic systems are difficult, although, in many instances, they can be treated approximately by the use of appropriate transformations of coordinates (6, 12).

It has been assumed that there is negligible head loss in the underdrains and that they are designed to have sufficient capacity to carry all the water that could theoretically flow into them. It should be noted in this regard that underdrains should be very carefully designed and have an appropriate filter system if their long-term performance is to be ensured.

Finally, it is necessary to know the coefficient of permeability of the soil in order to translate the reduced quantity of seepage into a meaningful flow rate that can be used in designing underdrain collection pipes and checking on capacity of the underdrain system. In many instances, this coefficient of permeability may be difficult to estimate without reliable field measurements.

ACKNOWLEDGMENT

I wish to express my gratitude to Sameh A. Mitry, whose computer analyses made the various design charts possible, and to Randy L. Moulton, who prepared the figures.

REFERENCES

1. Water in Pavements. Offices of Research, Development, Engineering, and Highway Operations, Federal Highway Administration, 1973.
2. H. R. Cedergren, J. A. Armon, and K. H. O'Brien. Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Sections. Federal Highway Administration, Rept. FHWA-RD-73-14, 1973. NTIS: PB 231 173/AS.
3. H. R. Cedergren. Drainage of Highway and Airfield Pavements. Wiley, New York, 1974.
4. B. McClelland and L. E. Gregg. Methods of Analysis of Flow Problems for Highway Subdrainage. Proc., HRB, 1944, pp. 364-376.
5. W. W. Donnan. Drainage of Agricultural Lands Using Interceptor Lines. Journal of the Irrigation and Drainage Division, Proc., ASCE, Vol. 85, No. IR1, March 1959, pp. 13-23.
6. M. E. Harr. Groundwater and Seepage. McGraw-Hill, New York, 1962.
7. J. Keller and A. R. Robinson. Laboratory Research on Interceptor Drains. Journal of the Irrigation and Drainage Division, Proc., ASCE, Vol. 85, No. IR3, Sept. 1959, pp. 25-40.
8. W. Sichardt and W. Kyrieleis. Grundwasserabsenkungen bei Fundierungsarbeiten. J. Springer, Berlin, 1930.
9. C. I. Mansur and R. I. Kaufman. Dewatering. In Foundation Engineering (G. A. Leonards, ed.). McGraw-Hill, New York, 1962.
10. V. I. Aravin and S. N. Numerov. Theory of Fluid Flow in Undeformable Porous Media (A. Moscona, tr. and ed.). Israel Program for Scientific Translations, Jerusalem, 1965, 511 pp.
11. G. Gilboy. Hydraulic-Fill Dams. Proc., International Committee on Large Dams, Stockholm, Sweden, 1933.
12. H. R. Cedergren. Seepage, Drainage and Flow Nets. Wiley, New York, 1967.

Publication of this paper sponsored by Committee on Subsurface Drainage.