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1985

THEORY

tensile load after the tension-column concrete cracks, the wall panel shows the first visual cracking. If there is insufficient tensile-column steel to carry the tension in the tension column when the concrete cracks, the first crack is at the junction of the tension column and foundation. This factor implies that the tension column cracks first with all specimens, but the crack does not open sufficiently that one can observe it because of the steel across the section.

The bending stiffness of the loading beam and columns has no influence on first cracking in shear walls. The larger the wall areas, the more closely a pure shear condition exists in the wall panel, which has been observed in many tests.

From the standpoint of seismic design, the useful range for design is limited to a damage criterion. In general, the walls are cracked so badly at the ultimate load that they are no longer useful even for blast-resistant construction where large cracks are expected. It is questionable if seismic loads should be allowed much beyond the appearance of the first crack. Although the structure may stand and actually be perfectly usable, its appearance may require extensive costly repairs.

The linear-deflection range was studied extensively to discover the influence of reinforcing steel on the load-deflection curve. Careful comparison of test results showed no correlation between the quantity of steel and the rigidity until the wall cracked. Thus, the deflection formula is based on a section of pure unreinforced concrete. Any such study must be based on statistical methods. With shear walls a normal variation in behavior makes the separation of small influences most difficult, if not impossible. Therefore, if a simple formula yields a solution that is as sound statistically as a more complex approach, the writers believe such a formula is best.

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REVIEW OF THE THEORIES FOR SAND DRAINS

By F. E. Richart, Jr., M. ASCE[1]

With Discussion By Messrs. Yoshichika Nishida; S. J. Johnson; and F. E. Richart, Jr.

Synopsis

The existing theories for vertical consolidation of clays by vertical flow of water and by radial flow to a drain well are satisfactory within the limits of the assumptions. The effect of considering void ratio as a variable did not significantly change the consolidation-time characteristics of vertical consolidation by vertical flow. Thus, including the effects of variable void ratio does not contribute toward the explanation of secondary consolidation.

It has been demonstrated that a drain well having a smeared zone at its periphery can be considered as an equivalent "ideal" well of reduced diameter. Diagrams are included for quantitative evaluation of this relation. An example is included to show the effectiveness of even a small diameter ideal well in reducing the time for consolidation.

Numerical procedures were found to be versatile aids for solving the classical consolidation problems as well as for considering consolidation under a variety of conditions. Variable rates of loading, variable soil properties, and layered systems can be readily included in the treatment of consolidation problems by these methods.

Introduction

The purpose of a drain well is to provide an easier path for the excess water to follow as it is squeezed out of a soil layer during consolidation. Thus, an effective well will accelerate the process of consolidation.

In practice, however, such installations of drain wells, usually composed of sand columns and hence called "sand drains," have met with varied success

Note.—Published, essentially as printed here, in July, 1957, in the Journal of the Soil Mechanics and Foundations Division, as *Proceedings Paper 1301*. Positions and titles given are those in effect when the paper or discussion was approved for publication in *Transactions*.

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In order to study the causes of the successes or failures of various installations, it is instructive to review the factors involved in the performance of a drain well, and, if possible, to evaluate the importance of each parameter. The following pages contain a review of the analytical approaches to the problem and a brief discussion of the effects of the more important variables involved.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear and are arranged alphabetically, for convenience of reference, in Appendix II.

Theory of Consolidation

Because the sand drain is merely an auxiliary device to expedite the process of consolidation, it is evident that the consolidation of a soil layer is the fundamental subject to be considered. The theory of consolidation presented by Karl Terzaghi,[2,3,4] Hon. M. ASCE forms the basis of conventional procedures for predicting the time rate of thickness decrease of clay layers under load. The assumptions made in establishing the theory are:

1. The voids in a soil are completely filled with an incompressible fluid, which is water.
2. The solid components of the soil are incompressible.
3. Darcy's law is valid.
4. The coefficient of permeability, k, is a constant.
5. The time lag of consolidation is due entirely to the low permeability of the soil.

Additional assumptions usually adopted unless it is specifically stated otherwise, are that the soil is laterally confined, and that the coefficient of compressibility, a_v is a constant for the range of pressure considered. The assumption of lateral confinement restricts application of the theoretical solutions to conditions in which the lateral deformations in the consolidating material are small with respect to the vertical deformations. The assumptions for the consolidation theory are discussed more completely elsewhere.[4]

Utilizing the given assumptions, as well as the further assumptions that variations in the void ratio, e, are limited to small values so that $(1 + e)$ may be treated as a constant, and the conditions of equilibrium of flow of water through an elemental volume of soil, Mr. Terzaghi established the differential equation of one-dimensional consolidation as

$$\frac{\partial u}{\partial t} = \frac{k\,(1 + e)}{a_v\,\gamma_w}\,\frac{\partial^2 u}{\partial z^2} = c_v\,\frac{\partial^2 u}{\partial z^2} \ldots\ldots\ldots\ldots\ldots (1)$$

In Eq. 1, the excess pore water pressure, u, is expressed as a function of its vertical position in space, z, and time, t. The coefficient of consolidation, c_v, is a constant, because the unit weight of water, γ_w, is a constant and the other terms, k, a_v, and $(1 + e)$ have been assumed to be constants.

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[2] "Erdbaumechanik auf Bodenphsikalischer Grundlage," by K. Terzaghi, Vienna, F. Deuticke, 1925.
[3] "Theorie der Setzung von Tonschichten," by K. Terzaghi and O. K. Fröhlich, Vienna, F. Deuticke, 1936.
[4] "Theoretical Soil Mechanics," by K. Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943.

Eq. 1 defines the vertical consolidation of a loaded clay layer due to a vertical flow of water. By a procedure similar to that used to derive Eq. 1, the equations for vertical consolidation due to a two-dimensional and three-dimensional flow can be expressed in cartesian or cylindrical coordinates. When both radial and vertical flow of water exist so that the resultant flow path is inclined, the consolidation equation can be written in terms of cylindrical coordinates as

$$\frac{\partial u}{\partial t} = c_{vr}\left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\,\frac{\partial u}{\partial r}\right) + c_v\,\frac{\partial^2 u}{\partial z^2} \ldots\ldots\ldots\ldots\ldots (2)$$

in which the coefficient of vertical consolidation due to radial flow (c_{v_r}) has been assumed to be different from the coefficient of vertical consolidation due to vertical flow of water (c_v).

Consolidation Equation Considering Void Ratio as a Variable.—If the change of void ratio, e, is appreciable, such that it is no longer satisfactory to treat $(1 + e)$ as a constant, a one-dimensional equation of consolidation may be derived taking this into account. The expression, derived in Appendix I, is

$$\frac{\partial u}{\partial t} = \frac{k}{a_v\,\gamma_w}\,\frac{1}{(1 + e)}\,\frac{\partial^2 u}{\partial h^2} - \frac{k}{\gamma_w\,(1 + e)^2}\left(\frac{\partial u}{\partial h}\right)^2 \ldots\ldots\ldots\ldots (3)$$

In which e, u, t, and h are variables, and k, a_v, and γ_w are considered to be constants.

The distance increment, h, is based on the equivalent height of the volume of solids in a given volume of soil. That is, for a soil element of unit area and height $d\,z$, the corresponding height of the volume of solids is $d\,h$. The relation between these distance elements, shown in Fig. 13, is

$$(1 + e)\,d\,h = d\,z \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Consolidation by Vertical Flow of Water Only

For a horizontal clay layer of thickness $2\,H$, the top of the layer may be designated as the origin of the coordinate, z, which is measured positive downward.

The excess pore water pressure, u, existing in this layer as a result of an applied vertical load is determined by Eq. 1 or Eq. 3. Since Eq. 1 is simpler, in that it defines u as a function of position, z, and time, t, only, it will be considered first. The manner in which u varies with z and t depends upon the boundary conditions.

For the condition of free drainage at the upper and lower boundaries of the clay layer (that is, at $z = 0$ and $z = 2\,H$) the value of excess pore water pressure at these boundaries will be zero at any time. At the plane of symmetry, $z = H$, no water will flow in a vertical direction. At an infinite time the excess pore water pressure will be zero throughout the entire soil layer. Expressed in terms of symbols, these boundary conditions are

$$u = 0 \quad \text{at} \quad z = 0 \quad \text{and} \quad z = 2\,H \quad \text{for} \quad 0 \leq t \leq \infty \ldots\ldots\ldots (5a)$$

$$\frac{\partial u}{\partial z} = 0 \quad \text{at} \quad z = H \qquad \text{for} \quad 0 \leq t \leq \infty \ldots\ldots\ldots (5b)$$

and

$$u = 0 \quad \text{at} \quad 0 \leq z \leq 2H \qquad \text{for} \quad t = \infty \ldots\ldots\ldots\ldots (5c)$$

In addition, there is a boundary condition defined by the initial excess pressure throughout the layer, which is entirely carried by the pore water. For the case of equal initial excess pore water pressure throughout the soil layer it has the value u_o. This defines a further boundary condition of

$$u = u_o \quad \text{at} \quad 0 \leq z \leq 2H \quad \text{for} \quad t = 0 \ldots\ldots\ldots\ldots (6)$$

If the assumption is satisfied that the total settlement is small compared to the thickness of the clay layer, ($(1 + e) \approx$ constant) then utilizing the prior boundary conditions, the solution of Eq. 1 may be obtained by means of a Fourier series.

$$u = \frac{4}{\pi} u_o \sum_{N=o}^{N=\infty} \frac{1}{2N+1} \sin\left[\frac{(2N+1)\pi z}{2H}\right] \epsilon^{-(2N+1)^2 \frac{\pi^2}{4} T_v} \ldots\ldots (7)$$

in which $\epsilon = 2.718\cdots$ and

$$T_v = \frac{c_v}{H^2} t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The term T_v represents an independent dimensionless variable called the time factor. It is customarily used as the abscissa, with the ratio u/u_o as ordinate to describe graphically the consolidation-time relationship established by Eq. 7.

Solutions of Eq. 1 for a number of different boundary conditions are given elsewhere.[3]

Effect of Variable Void Ratio.—An exact analytical solution for Eq. 3, satisfying the boundary conditions corresponding to Eq. 5 and Eq. 6, would be complicated. However, it was found possible to obtain an approximate solution for several examples through the use of the difference equation procedure. Eq. 3 is expressed in Appendix II in terms of finite differences and the method of solution is indicated.

Three examples were studied for which the initial void ratio, e_o, and the final void ratio, e_2, had the values of (a) $e_o = 0.65$, $e_2 = 0.55$, (b) $e_o = 1.00$, $e_2 = 0.80$, and (c) $e_o = 0.90$, $e_2 = 0.40$. For cases (a), (b), and (c) the final thicknesses of the clay layers were 94%, 90%, and 74%, respectively, of the original thicknesses of these layers.

The consolidation-time curves for the three cases considering void ratio as a variable are shown on Fig. 1. Also shown are two curves obtained by the Terzaghi theory. Curve 1 is obtained using Eq. 7 and curve 2 is obtained by difference equations in which the same space interval (0.25 H) was used as for the solution of the variable void ratio problems.

The general shape of the curves including the variable void ratio is similar to that of the curves obtained by the Terzaghi theory. The most significant effect of taking void ratio changes into consideration is exhibited by the position of these curves below and to the left of the usual curves. This indicates a

slightly greater degree of consolidation at any particular value of time. This might be expected since the term $(1 + e)$ appears in the denominator of the right side of Eq. 3. Thus, the Terzaghi theory gives a conservative estimate of the time required to reach a given degree of consolidation.

Because the difference in consolidation-time relationships by these two methods is unimportant when compared to the errors involved in establishing the soil constants, it is not necessary to introduce the added complication of variations in void ratio even for moderate changes in thickness of the clay layer.

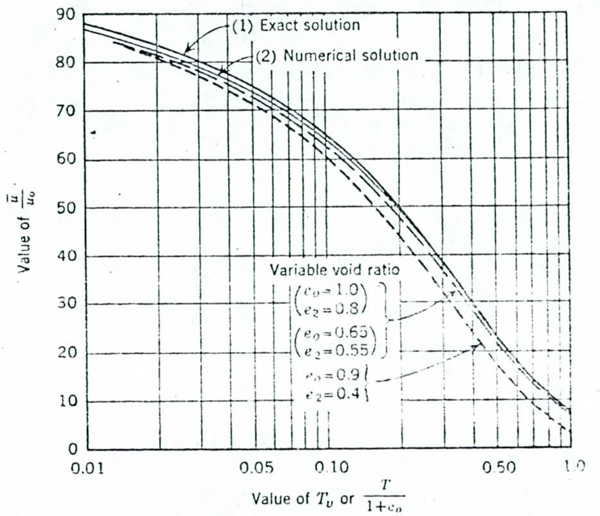


FIG. 1.—PORE WATER PRESSURE VERSUS TIME CURVES FOR VERTICAL WATER FLOW

The curves of Fig. 1 and the discussion included in Appendix I indicate that taking the variable void ratio into account does not contribute to the explanation of secondary consolidation.

SIMPLIFICATION OF ANALYSIS FOR CONSOLIDATION BY TWO OR THREE DIMENSIONAL FLOW

Consolidation by vertical flow alone involves only two variables, time, and depth, z. For consolidation by two-dimensional flow, the variables are x, z, and time, while for three-dimensional flow they are x, y, z, and time. Thus, the general solution for consolidation by three-dimensional flow for a given set of boundary conditions may become mathematically involved.

The method of separation of variables can be applied to this problem, as demonstrated by A. B. Newman[5] and later by N. Carrillo.[6] By use of this procedure, the expression for the resultant excess pore water pressure can be

[5] "The Drying of Porous Solids," by A. B. Newman, *Transactions, Am. Inst. Chem. Eng.*, Vol. 27, 1931, p. 310.

[6] "Simple Two- and Three-Dimensional Cases in the Theory of Consolidation of Soils," by N. Carrillo, *Journal of Mathematics and Physics*, Vol. 21, No. 1, March, 1942.

evaluated in terms of component solutions, which may be combined at the final step in the analysis. For example, in consolidation by two-dimensional flow the solution containing the variables x, z, and t can be determined by first evaluating u_x (which is a function of x and t), then evaluating u_z (which is a function of z and t), and combining these for a particular point in space at a particular time. The relation to be satisfied in the combining procedure is

$$\frac{u}{u_o} = \frac{u_x}{u_o} \cdot \frac{u_z}{u_o} \qquad (9a)$$

or

$$u = \frac{u_x \, u_z}{u_o} \qquad (9b)$$

A similar relation regarding the average values of pore water pressure is

$$\bar{u} = \frac{\bar{u}_x \cdot \bar{u}_z}{\bar{u}_o} \qquad (10)$$

which is the more convenient relationship and can be used for homogeneous layers.

Vertical Consolidation Due to Radial Flow of Water

The treatment of consolidation due to radial flow is an extension of the Terzaghi consolidation theory. A solution was presented in 1935 by L. Rendulic,[7] working under Mr. Terzaghi's direction. A more comprehensive study of the influence of ideal wells on consolidation was made by Reginald A. Barron,[8] A. M. ASCE, during 1940–1942.

The first generally available design information on this topic was given by Mr. Terzaghi[4] and was later supplemented by a paper which he published in 1945.[9] In 1948, Mr. Barron[10] presented a complete summary of the theory of sand drains, including new theories taking into account deviations from the ideal well conditions.

The two papers by Mr. Barron[8,10] constitute the principal analytical studies available at present (1959) of radial flow toward a drain well and the resulting consolidation of the clay. He considered two types of vertical strain which might occur in the clay layer, (1) "free vertical strain" resulting from a uniform distribution of surface load, and (2) "equal vertical strains" resulting from imposing the same vertical deformation at all points on the surface. For both strain conditions he included an analysis of the effect of "smear" of the soil near the well boundary, and the effect of resistance to flow through the well itself.

"Smear" is the term used to define the wiping action provided by the casing or hollow mandrel used to form the well as it is driven down into the soil, and

[7] "Der hydrodynamische Spannungsausgleich in zentral entwässerten Tonzylindern," by L. Rendulic, *Wasserwirtsch. u. Technik.*, Vol. 2, 1935, pp. 250–253, 269–273.

[8] "The Influence of Drain Wells on the Consolidation of Fine Grained Soils," by R. A. Barron, Providence (R. I.) District, U. S. Engrs. Office, 1944.

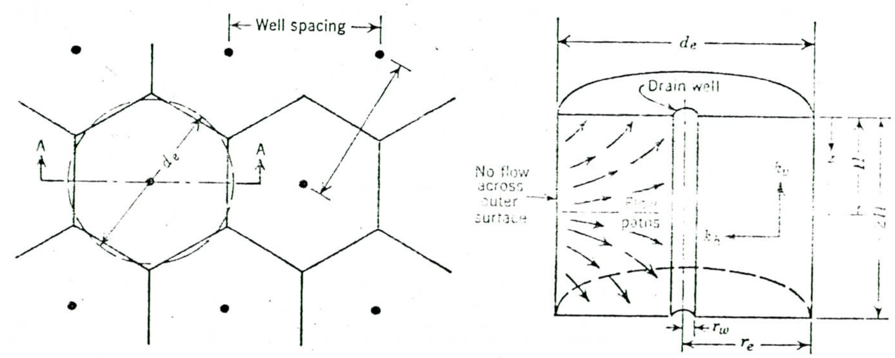
[9] "Drainage of Clay Strata by Filter Wells," by K. Terzaghi, *Civil Engineering*, October, 1945, pp. 463–464.

[10] "Consolidation of Fine-Grained Soils by Drain Wells," by R. A. Barron, *Transactions, ASCE*, Vol. 113, 1948, p. 718.

then pulled out after it has been filled with sand. This action tends to smear the soil at the well periphery. For a soil originally having a greater permeability in the horizontal than in the vertical direction, the smeared zone forms a barrier to the horizontal flow of water, thereby slowing down considerably the process of consolidation.

"Free Strain" Consolidation with No Smear and No Well Resistance.—A regular pattern of vertical drain wells, as shown in Fig. 2, will permit a radial as well as a vertical flow of water from the clay layer if it is subjected to an increase in pressure. Since in a previous section it was shown that the effects due to vertical flow and radial flow can be evaluated separately, and then combined to give the final consolidation behavior, only the radial flow consolidation is considered here.

As indicated on Fig. 2, for a triangular spacing of drain wells, a zone of influence exists having a hexagonal plan form. By approximating the hexagon



(a) PLAN OF DRAIN WELL PATTERN (b) SECTION A–A

Fig. 2.—Drain Wells and Flow Within Zone of Influence of Each Well

by a circle of equivalent diameter, d_e, this can be used as the outer limit of the zone of influence of each drain well. Thus it becomes sufficient to consider the radial flow and resulting consolidation of a soil volume of unit thickness contained between the distances d_e (diameter of well influence) and d_w (diameter of the drain well).

By eliminating the consideration of vertical flow, Eq. 2 becomes

$$\frac{\partial u}{\partial t} = c_{vr} \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r} \right) \qquad (11)$$

which is the equation for consolidation expressed in terms of radial coordinates. The boundary conditions that must be satisfied are:

1. The initial pore water pressure, u_o, is uniform throughout the soil mass when $t = 0$.

2. The excess pore water pressure at the drain well surface (r_w) is zero when $t > 0$.

3. The external radius, r_e, is considered impervious because of symmetry. Thus, $\frac{\partial u}{\partial r} = 0$ when $r = r_e$

The solution of Eq. 11, subject to the boundary conditions indicated above, leads to the following expressions for u_r (the pore water pressure at any location, at any time, t, existing as a result of radial flow only) and $\bar{u}_r$ (the average value of u_r throughout the soil mass at any time, t):

$$u_r = u_o \sum_{\alpha_1, \alpha_2, \alpha_3 \cdots}^{\alpha = \infty} \frac{-2\, U_1(\alpha) \cdot U_o\left(\frac{\alpha r}{r_w}\right)}{\alpha[n^2\, U_o^2(\alpha n) - U_1^2(\alpha)]}\, \epsilon^{-4\alpha^2 n^2 T_h} \quad \ldots\ldots (12)$$

and

$$\bar{u}_r = u_o \sum_{\alpha_1, \alpha_2, \alpha_3 \cdots}^{\alpha = \infty} \frac{4\, U_1^2(\alpha)}{\alpha^2(n^2 - 1)[n^2\, U_o^2(\alpha n) - U_1^2(\alpha)]}\, \epsilon^{-4\alpha^2 n^2 T_h} \quad \ldots (13)$$

in which

$$U_1(\alpha) = J_1(\alpha)\, Y_o(\alpha) - Y_1(\alpha)\, J_o(\alpha) \quad \ldots\ldots (14a)$$

$$U_o(\alpha n) = J_o(\alpha n)\, Y_o(\alpha) - Y_o(\alpha n)\, J_o(\alpha) \quad \ldots\ldots (14b)$$

$$U_o\left(\frac{\alpha r}{r_w}\right) = J_o\left(\frac{\alpha r}{r_w}\right) Y_o(\alpha) - Y_o\left(\frac{\alpha r}{r_w}\right) J_o(\alpha) \quad \ldots\ldots (14c)$$

in which J_o and J_1 are Bessel functions of the first kind, of zero order and first order, respectively; Y_o and Y_1 are Bessel functions of the second kind, of zero order and first order, respectively; α_1, α_2, and $\alpha_3 \cdots$ are roots of the Bessel functions which satisfy $J_1(\alpha n)\, Y_o(\alpha) - Y_1(\alpha n)\, J_o(\alpha) = 0$; and n = ratio equal to $\frac{r_e}{r_w} = \frac{d_e}{d_w}$. The time factor for consolidation by radial flow, $T_h = \frac{k_h(1 + e)\, t}{a_v\, \gamma_w\, d_e^2}$, in which k_h is the coefficient of permeability in the horizontal direction.

Solutions similar to Eqs. 12 and 13 were derived by R. E. Glover[11] for the analogous heat flow problem.

Equal Strain Consolidation with No Smear and No Well Resistance.—Under the condition of free strain it was implied that the settlements at the surface did not change the distribution of the load to the soil. However, in an actual installation, the fact that consolidation proceeds faster near the drain well, thereby causing a greater surface settlement in that region, could very well cause a redistribution of the surface loading. This would be especially true if the loading material had any tendency to arch across such depressions. As an extreme case the arching action could redistribute the loads to the surface in such a fashion that the surface settlement is the same at all points. This is the

[11] "Temperature Movements in Concrete and other Materials, with Special Reference to Conditions at Boulder Dam," by R. E. Glover, *Technical Memorandum No. 158*, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo., 1930.

condition of equal vertical strains for which Mr. Barron has also developed an analytical solution,

$$u_r = \frac{4\,\bar{u}}{d_e^2\, f(n)}\left[r_e^2 \log_e\left(\frac{r}{r_w}\right) - \frac{r^2 - r_w^2}{2}\right] \quad \ldots\ldots (15)$$

in which

$$\bar{u} = u_o\, \epsilon^{\lambda} \quad \ldots\ldots (16)$$

$$\lambda = \frac{-8\, T_h}{f(n)} \quad \ldots\ldots (17)$$

and

$$f(n) = \frac{n^2}{n^2 - 1} \log_e(n) - \frac{3\, n^2 - 1}{4\, n^2} \quad \ldots\ldots (18)$$

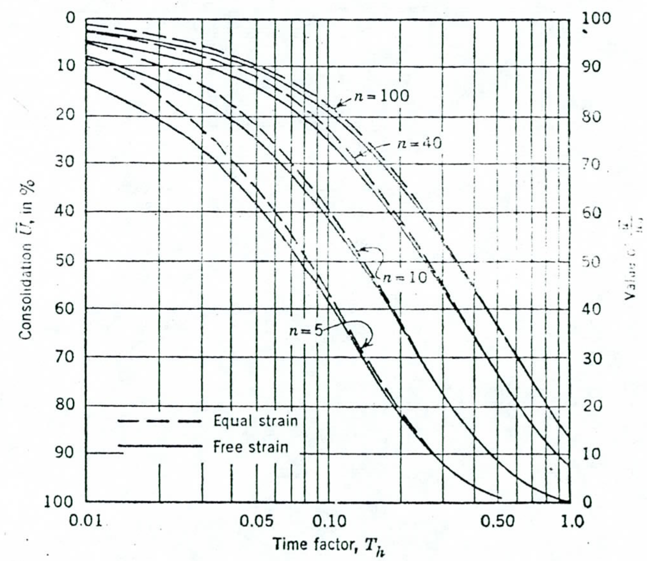


Fig. 3.—Consolidation Versus Time Factor

The initial distribution of hydrostatic excess pressure is not uniform, but may be computed from Eq. 15 for $T_h = \lambda = 0$. Curves showing the relation between the average pore water pressure, $\bar{u}$, and the time factor, T_h, can also be obtained from Eq. 16. Such curves for n = 5, 10, 40, and 100 are shown on Fig. 3, together with the corresponding curves determined by the free strain case.

Comparison of Free Strain and Equal Strain Solutions.—The difference between the results obtained by the two extreme considerations of the process of consolidation is small, particularly for the curves representing values of n greater than approximately 10. For n = 5 the discrepancy is somewhat greater for the first part of consolidation, but above approximately 50% consolidation the curves are almost identical.

Since the results are nearly identical, but the time needed to evaluate Eq. 3 is of the order of ten to fifteen times that needed to evaluate Eq. 16, the qual strain solution is preferable. Both Mr. Barron[10] and W. Kjellman[12] ecommended use of the equal strain solution.

Fig. 3 indicates that the curves representing the equal strain solutions for ifferent values of n have the same shape, but are displaced horizontally. The ›cation of the consolidation-time curve for any particular value of n depends pon the value of λ as determined from Eq. 17.

Effect of Peripheral Smear.—The remolded or smeared zone at the periphery f the drain well creates an additional resistance that must be overcome by the xcess water being expelled. This additional resistance retards the consolida-on process.

The smeared zone will not be uniform or homogeneous with regard to the il properties. It very likely consists of a thin layer of actual smear plus an ljacent region in which the soil has undergone a considerable amount of sturbance. The amount of disturbance decreases with distance away from ie well periphery.

However, in order to include the effects of smear and remolding, Mr. Barron[3] as considered that the smeared zone contains a homogeneous material having il properties different from those in the remaining material in the soil cylinder. he important quantities to be considered in analyzing the effects of this neared region are (*a*) the ratio, s, of the radius of the smeared zone to the well dius ($s = r_s/r_w$) and (*b*) the ratio of the coefficients of horizontal permeability the undisturbed soil (k_h) and in the smeared zone (k_s). For $s = 1$, there is thickness to the smeared ring, and if $k_h/k_s = 1$, then the disturbed zone does ›t change the water flow characteristics of the soil cylinder.

Mr. Barron also assumed that the smeared zone will consolidate very fast, us its consolidation can be ignored and the zone can be treated as an incom-essible material.

By ignoring the consolidation of the smeared zone, he was able to treat this gion as one where flow exists between one boundary value of zero and another oundary value that is time dependent. This is a reasonable approximation of e actual behavior, since the excess pore water pressure within the smeared gion would quickly dissipate into this steady flow condition. For an extreme se in which $n = 5$, $s = 2$, and $k_h/k_s = 2$, and consolidation of the smeared ne was considered, the steady flow condition was reached at approximately $= 0.025$.

Equal Strain with Smear.—Mr. Barron's solution for the excess pore water essure in a soil cylinder undergoing equal vertical strains and containing a neared region around the drain well is

$$u_r = \bar{u}_r \frac{\left[\log_e\left(\frac{r}{r_s}\right) - \frac{r^2 - r_s^2}{2\, r_e^2} + \frac{k_h}{k_s}\left(\frac{n^2 - s^2}{n^2}\right)\log_e(s)\right]}{\nu} \quad \text{......(19)}$$

[12] Discussion by W. Kjellman of "Consolidation of Fine-Grained Soils by Drain Wells," by R. A. Bar-. *Transactions*, ASCE, Vol. 113, 1948.

in which

$$\nu = \left[\frac{n^2}{n^2 - s^2}\log_e\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{s^2}{4\,n^2} + \frac{k_h}{k_s}\left(\frac{n^2 - s^2}{n^2}\right)\log_e(s)\right] \quad \text{...(20)}$$

and $\bar{u}_r$ can be determined from

$$\bar{u}_r = u_o\, \epsilon^{\xi} \quad \text{..........(21)}$$

in which

$$\xi = -\frac{8\,T_h}{\nu} \quad \text{..........(22)}$$

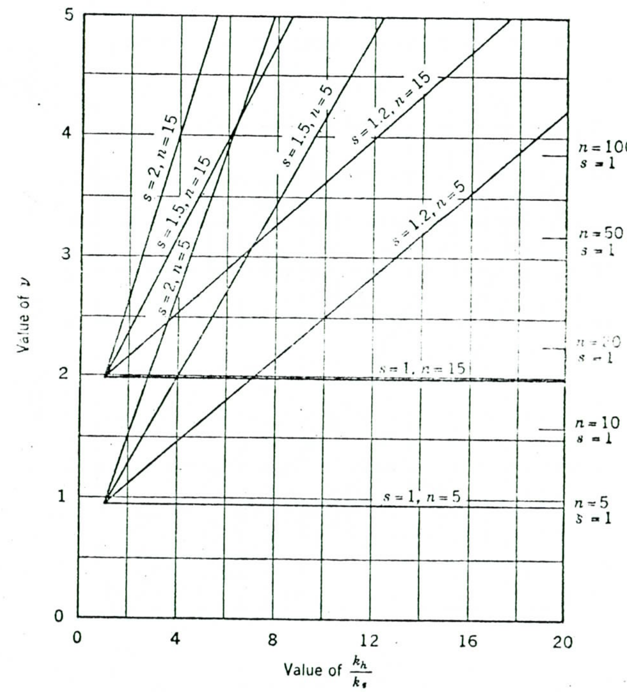


Fig. 4.—Relationship Between $\frac{k_h}{k_s}$, ν, and r, for $n=5$ and $n=15$

Evaluation of Smear Effects as Equivalent Changes of Well Diameter.—By comparing the equations it is seen that Eqs. 19 and 15, 20 and 18, 21 and 16, and 22 and 17 become identical when $s = 1$. Also, Eqs. 16 and 21, and 17 and 22 are similar.

For the ideal wells, the position of the consolidation-time curve for any particular value of n depends on the value of $f(n)$ (Eq. 18) which is a function of n only. For the wells with smear, the position of the resulting consolidation-time curves depends upon ν (Eq. 20) which is a function of n, s, and k_h/k_s.

Thus, it is possible to interpret various combinations of n, s, and k_h/k_s in the treatment of a well with smear as if they define an ideal well having a larger value of n. For a given radius of influence, a larger value of n determines the radius of an equivalent ideal well which is smaller than the radius of the actual well surrounded by a zone of smear. The effect on consolidation of the soil cylinder caused by the introduction of a smeared zone at the well periphery is identical to the effect caused by reducing the size of the ideal well.

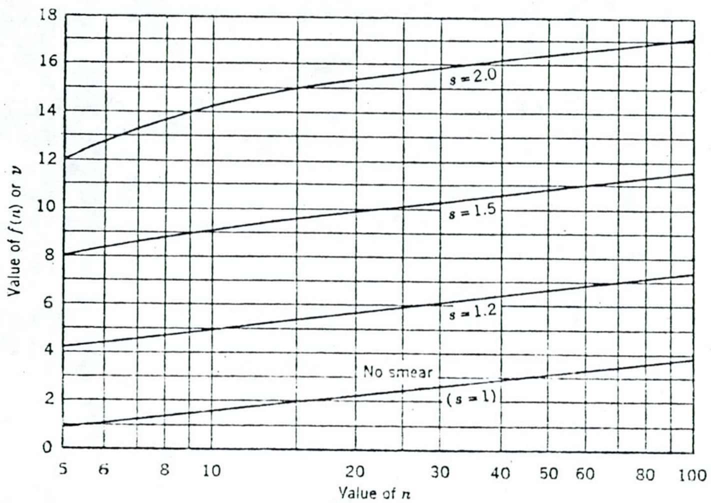


FIG. 5.—RELATIONSHIP BETWEEN n AND $f(n)$ OR ν FOR $\frac{k_h}{k_v} = 20$

Fig. 4 shows the combinations of n, s, and k_h/k_v that may be used to give the same value of ν. This figure, obtained by evaluating Eq. 20, uses the value of ν as ordinate and the ratio k_h/k_v as abscissa. Families of lines for each selected value of n show the influence of variations in s. Only the lines for $n = 5$ and

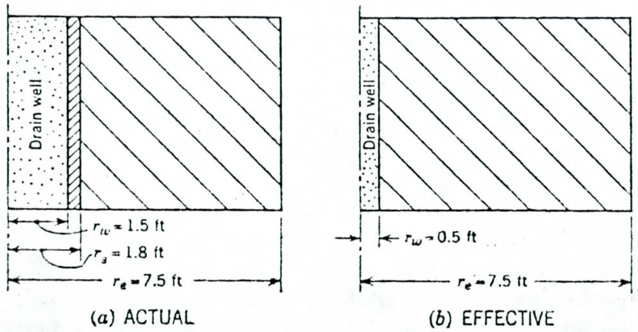


FIG. 6.—ACTUAL AND EQUIVALENT WELL INSTALLATIONS

$n = 15$ are shown on Fig. 4. However, by the use of Fig. 5, diagrams similar to Fig. 4 can be established for a wide range of values of n.

Fig. 6(a) is an example of the use of Fig. 4, considering a drain well for which $n = 5$, $s = 1.2$, and $k_h/k_v = 7$. The value of ν determined from Fig. 4 is 1.97

for Fig. 6(a). This corresponds to $f(n) = 1.97$ (Fig. 5) for a well having no smear ($s = 1$), or it determines $n = 15$ for the equivalent ideal well in Fig. 6(b). Evaluating this in numbers indicates that if the original diameter of well influence was 15 ft, the diameter of the drain well was 3 ft (from $n = 5$), and the outer diameter of the smeared region was 3.6 ft ($s = 1.2$). The effect of the smeared region is to reduce the consolidation-time behavior to one identical to that of a 1 ft diameter well ($n_{eff.} = 15$) for which no smear is present.

Effect of Well Resistance.—The foregoing analyses have considered that there is unrestricted flow of the water through the drain well. Actually, head losses will occur due to the resistance to flow of the well backfill material. The magnitude of the head losses will depend upon the rate of flow, the size of the well, and the permeability of the material filling the well.

Mr. Barron has developed a solution for the case of equal vertical strain, with or without smear, for a material i hich no vertical flow exists due to lack of permeability in the vertical direction, but $\frac{\partial u}{\partial z} \neq 0$.

However, for practical drain well installations for which n is approximately 7 to 15 and for $d_e/H \leq 1.0$, the effect on the consolidation behavior due to resistance of the drain wells should not be significant.

EXAMPLE CONSIDERING IDEAL WELLS

To illustrate the value of this information in selecting the size and spacing of drain wells for a particular installation, the following values were chosen: $a_v = 13 \times 10^{-5}$ cm per gm; $e = 1.50$; $H = 10$ ft; $k_v = 5 \times 10^{-8}$ cm per sec; $k_h = 25 \times 10^{-8}$ cm per sec; and $\Delta e = 0.125$; in which k_v is the coefficient of permeability in the vertical direction.

These values determine $c_v = 9.62 \times 10^{-4}$ cm² per sec, which is of the same order of magnitude as the test results given by K. Terzaghi and R. B. Peck,[13] M.ASCE. The clay layer is considered 20 ft thick if drainage occurs at both top and bottom surfaces, or 10 ft thick if drainage occurs at the top surface only. The horizontal permeability is five times that in the vertical direction.

For the initial conditions, assume that there is no smeared zone and no well resistance, so that the consolidation behavior depends only on the size and spacing of the drain wells.

Constant Well Diameter.—To compare the effects of well spacing a well diameter of 12 in., $\frac{k_h}{k_v} = 5$, and $c_v = 9.6 \times 10^{-4}$ cm² per sec were chosen. The consolidation percentage versus time curves are shown on Fig. 7. Fig. 7(a) shows the consolidation-time curves for radial or vertical drainage only. Fig. 7(b) shows the effect of a one-ft diameter well with a diameter of influence of ten ft when it is introduced into each of the clay layers. The time for consolidation is reduced to a fraction of the time for vertical drainage. The use of closer spacing of drain wells would further reduce this time, as would be demonstrated by using the curve for $d_e = 5$ ft in Fig. 7(b) instead of that for $d_e = 10$ ft.

[13] "Soil Mechanics in Engineering Practice," by K. Terzaghi and R. B. Peck, John Wiley & Sons, Inc., New York, N. Y., 1948, Fig. 29, p. 77.

It should be noted from Fig. 7 (*b*), that the consolidation-time behavior due ı combined radial and vertical drainage is nearly identical to that due to ıdial drainage alone, for $d_e = 10$ ft. This is influenced to a considerable extent y the conditions of $k_h = 5\ k_v$. If $k_h = k_v$, it would be necessary to consider a naller well spacing, or smaller ratio d_e/H, for the radial flow behavior to domi- ıte the consolidation-time behavior of the clay layer due to combined vertical nd radial flow.

Constant Well Spacing.—The consolidation versus time curves shown on ig. 8 are for the conditions involving a constant spacing of the wells, but allow- ıg the diameter of the wells to vary. The chosen well spacing was 10 ft which the same as the thickness of the clay layer, $k_h/k_v = 5$ and $c_v = 9.6 \times 10^{-4}$ cm² er sec.

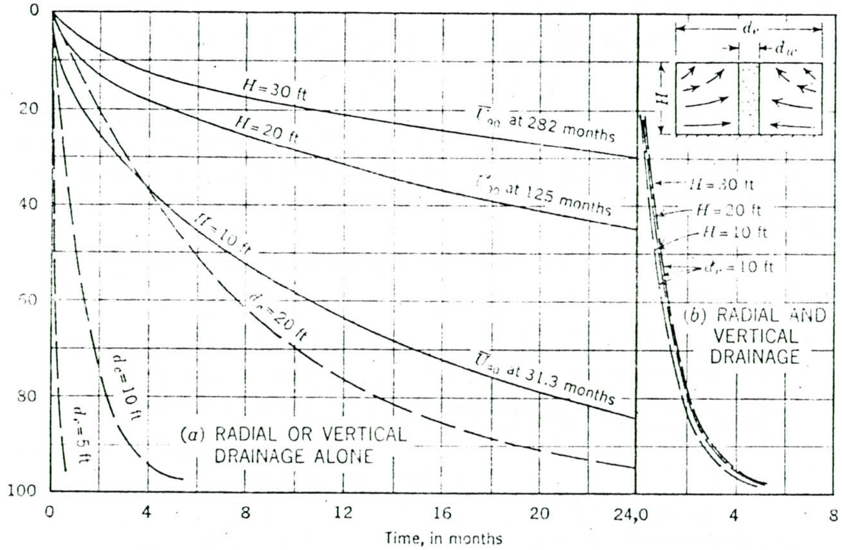


FIG. 7.—EFFECT OF DRAIN WELL ON TIME FOR CONSOLIDATION FOR VARIOUS THICKNESSES OF CLAY LAYER

Fig. 8 (*a*) shows the consolidation versus time curves for the radial or verti- al flow when they act independently. Fig. 8 (*b*) shows the effect on consolida- ion due to combined flow. For 90% consolidation the time amounts to (*a*) 6.0 ıonths (19.2% of time for vertical only) for $d_w = 1.2$ in.; (*b*) 3.9 months 12.5% of time for vertical only) for $d_w = 6$ in.; and (*c*) 1.7 months (5.4% of ime for vertical only) for $d_w = 24$ in. Thus, even the 1.2-in. diameter well in 10-ft soil cylinder cuts the consolidation time to approximately one-fifth of he time required for 90% consolidation by vertical drainage only.

It is of importance to note that doubling of the diameter of well influence d_e), or essentially the well spacing, causes an increase in the time for 90% con- olidation by roughly a factor of 6 (Fig. 7 (*a*)). In Fig. 8 (*a*) it is seen that by educing the diameter of drain well by a factor of 20 only increases the time for

90% consolidation by a factor of approximately 4. Consequently, the effec- tiveness of a given drain well installation is considerably more dependent on the choice of well spacing than it is on the well diameter.

Effect of Horizontal Permeability.—To convert the nondimensional time factor, T_h, into terms of t_h in months, it is necessary to use the equation,

$$t_h = \frac{T_h \cdot \gamma_w \cdot a_v \cdot d_e^2}{k_h\,(1 + e)} \qquad (23)$$

From Eq. 23 it is evident that the time for any given degree of consolidation is inversely proportional to the coefficient of permeability in the horizontal direction.

Fig. 9 illustrates the effect of k_h/k_v on t_{90}, the time required to obtain 90% consolidation by combined vertical and radial flow. Two well systems are used: (*a*) $d_e = 10$ ft, $d_w = 1.2$ in., $n = 100$; and (*b*) $d_e = 10$ ft, $d_w = 12$ in.,

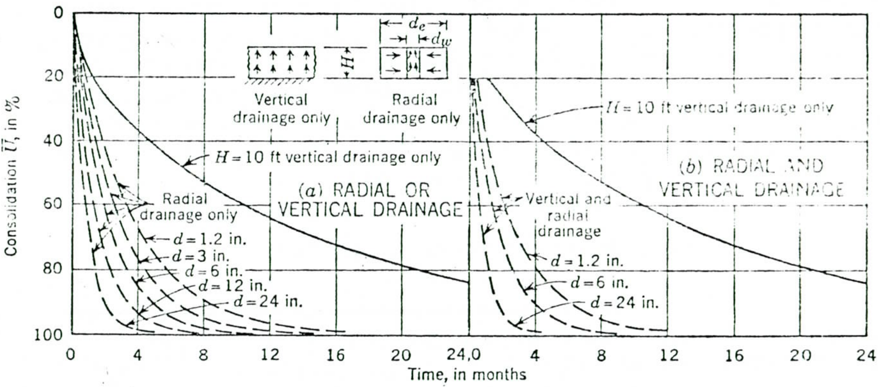


FIG. 8.—EFFECT OF SIZE OF DRAIN WELL ON TIME FOR CONSOLIDATION

and $n = 10$. For both systems, $c_v = 9.6 \times 10^{-4}$ cm² per sec. Fig. 9 demon- strates the pronounced effect of the ratio k_h/k_v for the two well systems. Even for the case of $k_h/k_v = 1.0$, the time for 90% consolidation is reduced to less than 55% of the time required when no wells are present. The 55% time cor- responds to the well system made up of 1.2-in. diameter wells spaced every 10 ft.

Discussion of Example.—The 1.2-in. diameter well considered previously is an "ideal well" for which there is no smear and no well resistance. The sig- nificant point is that such a small well can be so effective. If the well spacing was reduced to three or four ft, the effectiveness of such a 1.2-in. diameter well would be increased many times. Thus, the basic idea of the cardboard wicks spaced at approximately 4 ft, as described by W. Kjellman[12] is based on sound theoretical considerations.

In order to design a system of drain wells, some numerical information is required about the well resistance, the extent or radius of the smear zone, and the permeability of material in the smear zone. Then, using the desired diameter of the ideal well, for a given well spacing, Fig. 4 or Fig. 5 can be used to determine the actual size of well needed. That is, the actual size of well to be constructed is reduced in effectiveness by the smear and well resistance until it behaves similarly to an ideal well of smaller diameter. Then the behavior of the equivalent ideal well may be studied thoroughly with the aid of the many diagrams already prepared by Mr. Barron.[8,10]

Solution of the Consolidation Problem by Numerical Methods

Because the exact solutions for the consolidation problem become unwieldy for rather modest departures from the simplified case, such as when smear is

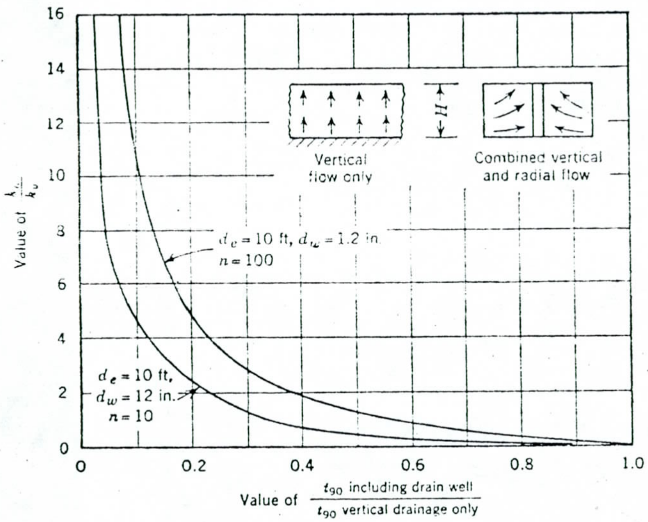


Fig. 9.—Influence of Permeability Ratio on Time for 90% Consolidation

introduced into the problem of consolidation by radial flow, it is desirable to investigate methods that lead to approximate results. In recent years, an appreciable amount of work has been directed toward application of the difference-equation procedure to the problem of transient flow of heat in solids.[14,15,16,17] The results of these studies may be used directly in analogous problems such as consolidation of clays.

[14] "The Numerical Solution of Heat Conduction Problems," by H. W. Emmons, *Transactions*, A.S.M.E., Vol. 65, 1943, pp. 607–615.

[15] "A Practical Method for Numerical Evaluation of Solutions of Partial Differential Equations of the Heat-Conduction Type," by J. Crank and P. Nicolson, *Proceedings, Cambridge Phil. Soc.*, Vol. 43, 1947, pp. 50–67.

[16] "Numerical Analysis of Heat Flow," by G. M. Dusinberre, McGraw-Hill Book Co., Inc., New York, N. Y., 1949.

[17] "A Study of the Numerical Solution of Partial Differential Equations," by G. G. O'Brien, M. A. Hyman, and S. Kaplan, *Journal of Math. and Physics*, Vol. 29, 1950, pp. 223–251.

Explicit Expressions.—One procedure used extensively for numerical solutions of both the heat flow and consolidation[18,19] equations permits a solution for the value of the dependent variable at a given point in space and time by considering only values of this variable at points in space at a previous time. The solution thus amounts to a step-by-step determination of these values at points in space as they vary along the time coordinate.

The expression resulting from replacing Eq. 1, for example, by its differences equivalent, is

$$\frac{u_{o,t+\Delta t} - u_{o,t}}{\Delta t} = c_v \left[\frac{u_{2,t} - 2\,u_{o,t} + u_{4,t}}{(\Delta z)^2} \right] \ldots\ldots\ldots\ldots (24)$$

In Eq. 24 the first subscript for the u-terms denotes the position in space of the point under consideration (also shown on Fig. 11) and the second subscript denotes the time, with Δt equal to the time interval and Δz representing the interval of z.

Eq. 24 is usually rearranged, for convenience in computation, as

$$u_{o,t+\Delta t} = u_{o,t} + A\,\{u_{2,t} - 2\,u_{o,t} + u_{4,t}\} \ldots\ldots\ldots\ldots (25)$$

in which the constant, A, represents

$$A = \frac{c_v\,\Delta t}{\Delta z^2} \ldots (26a)$$

or

$$A = \frac{\Delta T}{(\Delta\zeta)^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26b)$$

if ζ is defined as

$$\zeta = \frac{z}{H} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27a)$$

and

$$\Delta z = H\,(\Delta\zeta) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27b)$$

Previous studies[17] of the heat flow equation have determined that if $A > 0.5$, the values of u determined by Eq. 25 diverge, and for $A \leq 0.5$, they converge. Also, Eq. 26 establishes a definite relation between the increments of time and space for a particular choice of A. Thus, even for a rather crude space network, such as $\Delta\zeta = \frac{1}{4}$, the maximum allowable time increment is $\Delta T_v = 0.03125$, with the result that numerous steps in time are required in the solution. Since this is a step-by-step procedure, in order to determine the excess pore water distribution at a particular time, it is necessary to work up to that time by using equal sized time intervals, starting from $T_v = t = 0$.

Implicit Expressions.—The size of the time interval dictated by the stability consideration of the explicit scheme is often inconveniently small, therefore a method permitting larger time increments is desirable. This has been accom-

[18] "Numerical Solution of Some Problems in the Consolidation of Clay," by R. E. Gibson and P. Lumb, *Proceedings Inst. of Civil Engineers*, London, Vol. 2, Part I, 1953, pp. 182–198.

[19] "Numerical Analysis of Consolidation Problems," by R. F. Scott, thesis presented to Massachusetts Inst. of Technology, at Cambridge, in 1953, in partial fulfilment of the requirements for the degree of Master of Science.

plished by turning around the difference equation, resulting in the form (corresponding to Eq. 1)

$$\frac{u_{o,t+\Delta t} - u_{o,t}}{\Delta t} = c_v \left[\frac{u_{2,t+\Delta t} - 2\,u_{o,t+\Delta t} + u_{4,t+\Delta t}}{(\Delta z)^2} \right] \quad \text{........ (28)}$$

The form of Eq. 28 used for computation[17] considers the average value of the space relations over the time interval as:

$$\frac{u_{o,t+\Delta t} - u_{o,t}}{\Delta t} = \frac{c_v}{2} \left\{ \frac{u_{2,t+\Delta t} - 2\,u_{o,t+\Delta t} + u_{4,t+\Delta t}}{(\Delta z)^2} + \frac{u_{2,t} - 2\,u_{o,t} + u_{4,t}}{(\Delta z)^2} \right\} \quad \text{.. (29)}$$

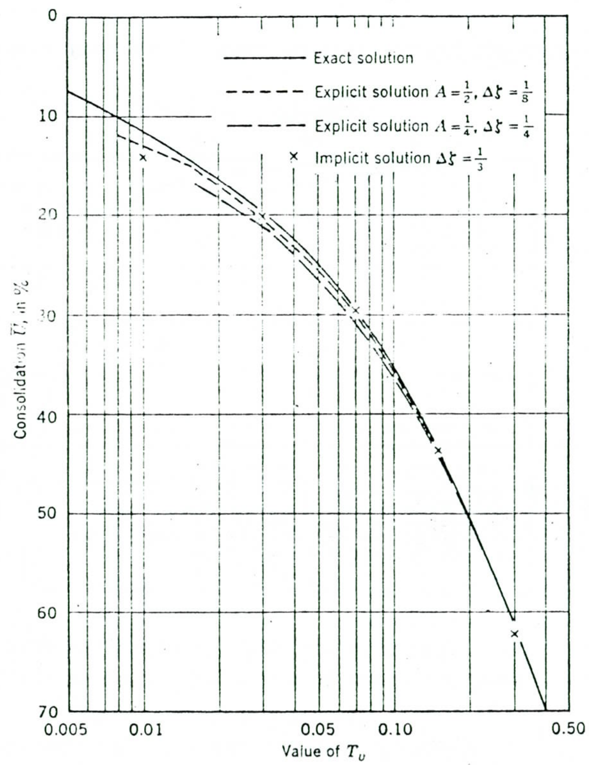
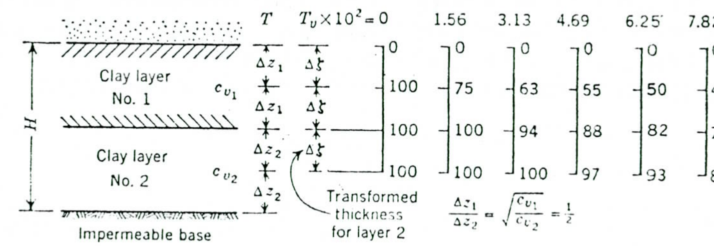


FIG. 10.—CONSOLIDATION VERSUS TIME CURVES FOR VERTICAL FLOW COMPUTED BY EXACT AND NUMERICAL METHODS

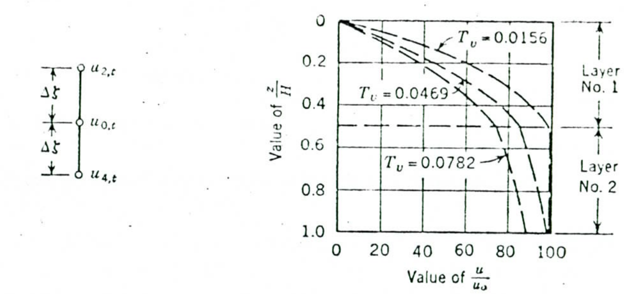
Eq. 28 is an implicit relation for the values of excess pore water pressure at the time $t + \Delta t$. That is, these unknowns occur simultaneously in the equations. This requires the solution of a set of simultaneous equations for each time interval adopted, but the size of the time interval is arbitrary and may be changed during the solution of a problem. The equations are stable and place no restriction on the size of the space or time increments. However, if a small space interval is required in order to gain accuracy, this increases the number

of simultaneous equations. For such cases, the use of digital computing machinery is nearly always required.

Although the implicit scheme does not place restrictions on the size of the time or space increments as a criterion of stability, the fact that the finite differences must be reasonable approximations to the corresponding differentials does prohibit large increments, particularly in time. The size of the allowable time increment depends on the rate of change of pore water pressure within that increment. Criteria for this and for related procedures have been given by T. P. Tung and N. M. Newmark, M. ASCE.[20]



(a) PARTIAL SOLUTION FOR VARIATIONS IN $\frac{u}{u_o}$



(b) ISOCHRONES FOR LAYERED SYSTEM

FIG. 11.—NUMERICAL PROCEDURES FOR CONSOLIDATION OF A TWO-LAYER SYSTEM DUE TO VERTICAL FLOW

Numerical Solutions of Vertical Consolidation by Vertical Flow.—Eq. 24 was evaluated for two cases, $A = 1/4, \zeta = 1/4$ and $A = 1/2, \zeta = 1/8$, for which the resulting time intervals were $\Delta T = 0.015625$ and 0.0078125, respectively. The curves thus obtained are shown on Fig. 10 as well as the curve corresponding to the exact solution. The agreement is good, particularly for values of $T_v > 0.10$.

Eq. 29 was also evaluated for comparison. Using $\zeta = 1/3$ and time increments which doubled in size at each time step, starting with $T_v = 0.010$ (that

[20] "A Method of Numerical Integration for Transient Problems of Heat Conduction," by T. P. Tung and N. M. Newmark, Structural Research Series No. 95, Civil Engineering Studies, Univ. of Illinois, Urbana, Ill., March, 1955.

is, $\Delta T_{v_1} = 0.010$, $\Delta T_{v_2} = 0.020$, $\Delta T_{v_3} = 0.040$) a curve was obtained that agreed more closely with the exact solution than did those computed by the explicit method.

Since the explicit method permits the use of a fine space mesh with little increase in computational effort above that for a coarse mesh, it appears desirable to use this scheme for small values of time for which u is changing rapidly. Then, for larger values of time, it would be convenient to switch to a coarse time mesh and use the implicit scheme.

Vertical Consolidation by Vertical Flow for Layered Systems.—A clay stratum, made up of two or more horizontal layers, may also be treated by numerical procedures. At the interface between any two layers the conditions of equilibrium require that the velocity of flow leaving one layer must be equal to the velocity of flow entering the other. Thus, for two materials having permeability coefficients of k_{v_1}, and k_{v_2}, this condition at the interface requires that

$$k_{v_1}\left(\frac{\partial u}{\partial z_1}\right) = k_{v_2}\left(\frac{\partial u}{\partial z_2}\right) \qquad (30)$$

The two layers also have different coefficients of consolidation, c_v, which are dependent upon a_v as well as k_v. In order to simplify the numerical procedure, it is convenient to adjust the size of the space intervals so that the term A remains the same in both layers for the same Δt. Thus,

$$A = \frac{c_{v_1}\,\Delta t}{(\Delta z_1)^2} = \frac{c_{v_2}\,\Delta t}{(\Delta z_2)^2} \qquad (31)$$

or

$$\frac{\Delta z_1}{\Delta z_2} = \sqrt{\frac{c_{v_1}}{c_{v_2}}} \qquad (32)$$

Fig. 11 illustrates the numerical solution for a two-layered system, consisting of a top layer, 1, resting on a second layer, 2, of equal thickness. Fig. 11(*a*) shows the partial solution for variations in u/u_o with time and depth and $\Delta\zeta$ is the transformed thickness for layer 2. For the two layers, $c_{v_2} = 4\,c_{v_1}$, which determines for a choice of $\Delta\zeta = 0.25$ that the space increments are $0.25\,H$ in layer 1 and $0.50\,H$ in layer 2. In Fig. 11(*b*), the isochrones show a definite change in slope at the interface, which should correspond to Eq. 30.

Numerical Solution of Consolidation by Radial Flow.—Expressing Eq. 11 in terms of finite differences leads to

$$\frac{u_{o,t+\Delta t} - u_{o,t}}{\Delta t} = \frac{c_v}{\Delta r^2}\left[\left(1 + \frac{\Delta r}{2\,r_o}\right)u_{1,t} + \left(1 - \frac{\Delta r}{2\,r_o}\right)u_{3,t} - 2\,u_{o,t}\right] \qquad (33)$$

or

$$u_{o,t+\Delta t} = \frac{c_v\,\Delta t}{\Delta r^2}\left[\left(1 + \frac{\Delta r}{2\,r_o}\right)u_{1,t} + \left(1 - \frac{\Delta r}{2\,r_o}\right)u_{3,t} - 2\,u_{o,t}\right] + u_{o,t} \qquad (34a)$$

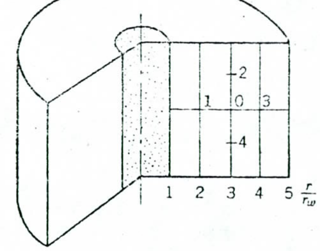
The network points used in Eq. 34 are spaced to the right and to the left of the point, o, for which the value of the excess pore water pressure is being determined.

From Eq. 34(*a*) it is evident that a different equation must be used for each point under consideration, due to the specific value of r at each location. This slows the computations but does not otherwise complicate the solution. Fig. 12 illustrates the procedure for solution of a problem for radial flow in a homogeneous soil cylinder for the case of $n = 5$. As used in Fig. 12,

$$u_{o,t+\Delta t} = u_{o,t} + \frac{4\,r_e^2\,\Delta T}{\Delta r^2}\left[\left(1 + \frac{\Delta r}{2\,r}\right)u_{3,t} + \left(1 - \frac{\Delta r}{2\,r}\right)u_{1,t} - 2\,u_{o,t}\right] \qquad (34b)$$

in which $\Delta r = r_w$, $r_e = 5\,r_w$, and $\Delta T_h = \dfrac{0.5\,(\Delta r)^2}{4\,r_e}$. By taking the average value of the excess pressure at any time, a curve of $\bar{u}_r$ versus T_h can be constructed that corresponds to within a few percent of the values obtained by Mr. Barron's[10] procedure.

Additional Variables Which Can be Treated by the Numerical Procedure.—Because the numerical procedure involves the building up of a solution throughout a series of time intervals, it is ideally suited for consideration of quantities



$\frac{r}{r_w}$ 1	2	3	4	5	$T_h \times 10^2$
0	100	100	100	100	0
0	62.5	100	100	100	0.5
0	62.5	84.4	100	100	1.0
0	52.7	84.4	93.2	100	1.5
0	52.7	76.3	93.2	93.2	2.0
0	43.8	64.3	78.9	78.9	4.0
0	26.4	38.8	47.7	47.7	10.0
0	11.5	16.9	20.7	20.7	20.0

FIG. 12.—NUMERICAL PROCEDURE FOR CONSOLIDATION BY RADIAL FLOW

that vary with time. For cases in which the surface load is varied during consolidation, this can be introduced into the problem simply as an over-all increase (or decrease) of the excess pore water pressure at a particular time. A gradually increasing load can be approximated by using a stairstep variation of load with time.

Variations of fundamental soil properties, such as the coefficients of consolidation or swelling, can be included as they vary both with time and space. That is, the intergranular pressure changes affect the soil properties, which may be adjusted accordingly during the period of consolidation. The soil properties are considered to be constant during an interval of time, Δt, in accordance with the assumptions of the theory of consolidation, but the value of these constants may be changed in successive time intervals. The soil constants may also have different initial values at the various space locations of the network points under consideration.

The flexibility of the numerical procedures also permits studies of two and three-dimensional consolidation to be made without excessive effort. For example, R. E. Gibson and P. Lumb[18] have obtained approximate solutions for the time rate of settlement of circular pervious and impervious footings, resulting from consolidation of the underlying clay.

Use of Theories for Design or Analysis of Sand Drain Installations

Limitations.—The theories for consolidation of clay layers including the effects of sand drains can be expected to give reasonable results only when the clay is comparable to the ideal material assumed as a basis for the theory. This requires the clay layer to be homogeneous if the analytical theories are to be used, or at least homogeneous in horizontal planes, and the variations in soil properties known in vertical direction, if the numerical procedures are to be used.

In addition, it is necessary that the values of the soil properties be established with a reasonable degree of accuracy. The coefficients of consolidation due to vertical and to horizontal flow of water, and the coefficients of permeability in the vertical and horizontal directions as well as for remolded samples, must all be determined from a large enough number of samples that a reliable average value of each is established. Because the need for sand drains and their effectiveness, if installed, is directly dependent on these soil properties, it is evident that serious miscalculations could result if values of the soil properties are inaccurately determined and then used as the basis for design.

Use of Point Pore Pressures.—In addition to settlement measurements which can be compared to theoretical predictions, the time rate of change of excess pore water pressure may be measured at specific locations within the clay layer. These pressure measurements can be compared with diagrams prepared by use of Eq. 7 and Eq. 12 or Eq. 15 which predict the theoretical pressure-time behavior during consolidation. However, unless a good evaluation of the radius of the smeared zone at the well periphery and the coefficient of permeability within this smeared region were obtained, it is likely that a considerable difference may occur between the geometrical value of $n \left(= \frac{d_e}{d_w} \right)$ and the effective value of n which includes the effect of well smear. This will lead to appreciable variation in the predicted excess pore pressure versus time relations.

Difficulties also arise in comparison of theoretical and field results. This is due partially to the fact that the piezometer readings define the behavior of the clay at a point, or at least within a small volume, while the theoretical predictions are based, at best, on representative values of the soil properties. It would be expected that better agreement between actual consolidation behavior and that predicted from theory would be obtained by increasing the number of piezometer installations, thereby minimizing the effects of local variations in soil properties.

Conclusions

In the preceding pages the available theories for vertical consolidation due to vertical flow and due to radial flow of water to a drain well have been reviewed. These theories are entirely satisfactory for the analysis of any situation conforming to the assumptions on which these theories are based.

Including void ratio as a variable did not significantly change the consolidation-time characteristics of vertical consolidations by vertical flow. Thus, a consideration of variable void ratio does not contribute to the explanation of secondary consolidation.

For vertical consolidation due to radial flow toward a drain well, the equal strain solutions given by Mr. Barron are much more convenient to use than are the free strain solutions. Using Mr. Barron's equal strain solutions for ideal wells and for smeared wells, it was shown that the consolidation behavior of the latter is identical to that of an equivalent ideal well of reduced diameter. Diagrams are given for quantitative evaluation of this relation between behavior of smeared and equivalent ideal wells. By interpreting the consolidation behavior due to an actual well in terms of that of an equivalent ideal well, the figures prepared by Mr. Barron for ideal wells can be used directly for design or analysis. An example shows the effectiveness of even a small diameter ideal well.

The numerical procedures for solving consolidation problems were found to be versatile aids for both checking the classical solutions and for evaluating new problems. Variable rates of loading, variable soil properties, and layered systems, can be readily included into the solution of consolidation problems by these methods.

Acknowledgments

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APPENDIX I. DERIVATION OF CONSOLIDATION EQUATIONS INCLUDING VOID RATIO AS A VARIABLE

For a soil element of area A and height dz (Fig. 13) the volume of solids can be represented by a height dh which does not change during the consolidation process. The total height of the element may always be determined by the relation $dz = (1 + e)\, dh$. The change of volume of a soil element is dependent on the change of the volume of voids which is $V_v = A\, e\, dh$. Thus, the change of volume with time is

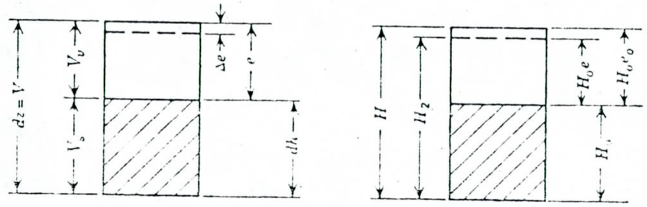
$$\frac{\partial V}{\partial t} = - A\, dh \cdot \frac{\partial_e}{\partial t} \quad \text{......................(35)}$$

Next, considering the upward vertical flow of water through this elemental volume, an amount equal to $- \frac{k}{\gamma_w} \frac{\partial u}{\partial z} A$ flows through the bottom face, and an amount equal to $\frac{kA}{\gamma_w}\left(-\frac{\partial u}{\partial t} - \frac{\partial^2 u}{\partial z^2}dz\right)$ flows through the top face. The difference between flow at these two surfaces determines the rate of loss of water from the soil element as

$$\Delta Q = -\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2}\cdot A\,dz \qquad (36)$$

Since it was assumed that the soil was completely saturated, and that both water and the solid soil particles are incompressible, the rate of water loss (Eq. 36) must equal the rate of volume change of the soil element (Eq. 35) or,

$$-A\,dh\frac{\partial_e}{\partial t} = -\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2}A\,dz \qquad (37)$$



(a) UNIT AREA AND HEIGHT dz (b) UNIT AREA AND HEIGHT H

FIG. 13.—COMPONENTS OF A VOLUME OF SOIL

In order to convert Eq. 37 into terms involving h, the following relations may be used.

$$\frac{\partial u}{\partial z} = \frac{\partial u}{\partial h}\cdot\frac{\partial h}{\partial z} = \frac{\partial u}{\partial h}\cdot\frac{1}{1+e} \qquad (38)$$

$$\frac{\partial^2 u}{\partial z^2} = \frac{\partial h}{\partial z}\cdot\frac{\partial}{\partial h}\left(\frac{\partial u}{\partial h}\cdot\frac{1}{1+e}\right) = \frac{1}{(1+e)^2}\left[\frac{\partial^2 u}{\partial h^2} - \frac{1}{(1+e)}\frac{\partial u}{\partial h}\cdot\frac{\partial e}{\partial h}\right] \qquad (39)$$

$$\frac{\partial\bar{\sigma}}{\partial t} = -\frac{1}{a_v}\cdot\frac{\partial e}{\partial t} = -\frac{\partial u}{\partial t} \qquad (40)$$

and

$$\frac{\partial\bar{\sigma}}{\partial h} = -\frac{1}{a_v}\frac{\partial e}{\partial h} = -\frac{\partial u}{\partial h} \qquad (41)$$

in which $\bar{\sigma}$ is the effective, or intergranular pressure at any point in the soil layer.

By substituting Eqs. 38, 39, 40, and 41 into Eq. 37 the expression is,

$$\frac{\partial u}{\partial t} = \frac{k}{a_v\,\gamma_w}\cdot\frac{1}{(1+e)}\left[\frac{\partial^2 u}{\partial h^2} - \frac{a_v}{1+e}\left(\frac{\partial u}{\partial h}\right)^2\right] \qquad (42)$$

which is the general equation for consolidation with void ratio considered as a variable.

For a solution of Eq. 42 by the explicit numerical scheme, it was found convenient to substitute terms for void ratio in place of those for u, according to Eqs. 40 and 41. The resulting difference equation is

$$e_{o,t+\Delta t} = e_{o,t} + \frac{k\,\Delta t}{a_v\,\gamma_w\,(\Delta h)^2}\left[\frac{e_{2,t} - 2\,e_{o,t} + e_{1,t}}{1 + e_{o,t}} - \frac{(e_{2,t} - e_{1,t})^2}{4\,(1 + e_{o,t})^2}\right] \qquad (43)$$

Let

$$T = \frac{k\cdot t}{a_v\,\gamma_w\,(H_o)^2} \qquad (44a)$$

or

$$\Delta T = \frac{k\,\Delta t}{a_v\,\gamma_w\,(H_o)^2} \qquad (44b)$$

which is similar to T_v (Eq. 8) in appearance and is related by $T = T_v\,(1 + e_o)$ since $H = H_o\,(1 + e_o)$ as shown on Fig. 13.

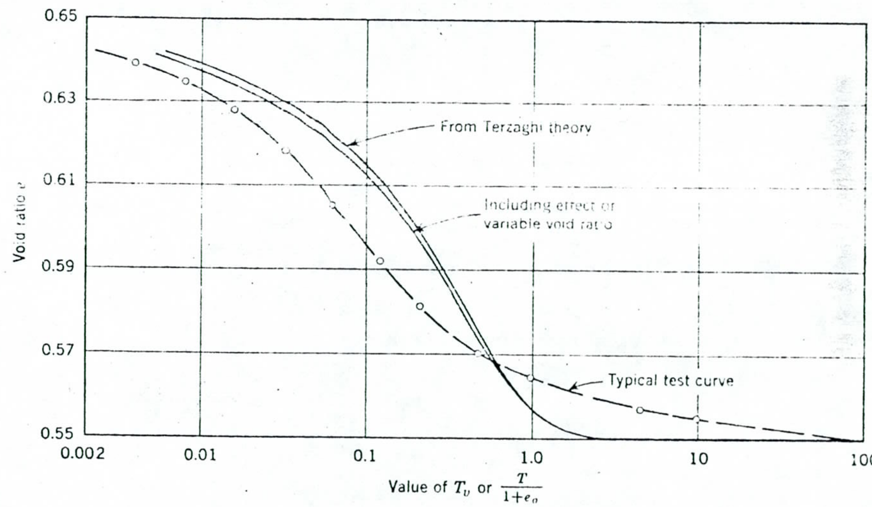


FIG. 14.—VOID RATIO VERSUS TIME FACTOR

Again

$$\zeta = \frac{h}{H_o} = \frac{h\,(1 + e_o)}{H\,(1 + e_o)} = \frac{z}{H} \qquad (45a)$$

or

$$\Delta h = \Delta\zeta\cdot H_o \qquad (45b)$$

and

$$\frac{k\,\Delta t}{a_v\,\gamma_w\,(\Delta h)^2} = \frac{\Delta T}{(\Delta\zeta)^2} = B \qquad (46)$$

The curves shown on Fig. 1 were obtained by use of Eq. 43.

During consolidation, the void ratio changes with time from an initial value of e_o to a final value of e_2 which is reached at an infinite time. Thus, e_o and e_2

are taken as the boundary values, and Eq. 43 determines the manner in which the void ratio changes with time.

The void ratio versus the time factor curve that results from using the boundary values of $e_o = 0.65$, and $e_2 = 0.55$ with Eq. 43 is shown as the solid line on Fig. 14, which is designated as including effect of variable void ratio. The other curve on Fig. 14 drawn as a solid line represents the results obtained from the Terzaghi theory if e_o and e_2 are considered to represent 0% and 100% consolidation, respectively. The dashed curve represents a typical consolidation test, converted into terms of e and T_v, which exhibits the region of "secondary" consolidation.

Secondary consolidation is characterized by a nearly linear relation between decrease in void ratio and logarithm of time, over a considerable period of time during the final part of the test.

From Fig. 14 it is evident that the modification of the Terzaghi theory to include the consideration of variable void ratio does not appreciably change the void ratio-time factor relations during consolidation. The effect of variable void ratio is to throw the curve slightly to the left of the conventional curve. Thus, the effect of including a consideration of variable void ratio in the theory of consolidation of a saturated soil does not contribute to the explanation of secondary consolidation.

By interpreting e_o and e_2 as the 100% and 0% values of excess pore water pressure, the results of the three solutions for variable void ratio can be plotted on the same diagram, as was done on Fig. 1.

APPENDIX II. NOTATION

The following symbols, adopted for use in the paper and for the guidance of discussers, conform essentially with "Glossary of Terms and Definitions in Soil Mechanics," prepared by the Committee on Glossary of Terms and Definitions in Soil Mechanics of the Soil Mechanics and Foundations Division, *Proceedings Paper 1826*, October, 1958:

A = area, or $A = \dfrac{c_v \, \Delta t}{\Delta z^2} = \dfrac{k\,(1+e)\,\Delta t}{a_v\,\gamma_w\,(\Delta z)^2}$;

a_v = coefficient of compressibility;

$B = \dfrac{k\,\Delta t}{a_v\,\gamma_w\,(\Delta h)^2}$ = constant in numerical solution including variable void ratio;

c_v = coefficient of consolidation;

c_{v_r} = coefficient of consolidation due to radial water flow;

d_e = diameter of well influence;

d_w = diameter of drain well;

e = void ratio at any point in the soil layer;

$\bar{e}$ = average value of the void ratio in the soil layer;

$f(n)$ = function of n for equal strain solution (Eq. 18);

H = thickness of clay layer which is drained from one surface only, or half thickness of clay layer drained from top *and* bottom surfaces;

H_o = equivalent thickness of layer of solids in a clay layer of over-all thickness, H (Fig. 13);

h = measure of distance;

$J_o(\ )$ = Bessel function of the first kind, zero order;

$J_1(\ )$ = Bessel function of the first kind, first order;

k = coefficient of permeability (LT^{-1}):

- k_h = coefficient of permeability in the horizontal direction;
- k_v = coefficient of permeability in the vertical direction;
- k_s = coefficient of permeability in the smeared zone;

$n = \dfrac{r_e}{r_w} = \dfrac{d_e}{d_w}$ = ratio of diameter of well influence to diameter of drain well;

$p = \bar{\sigma} + u$ = total pressure at any point in the soil layer;

r = radius:

- r_e = radius of influence of drain well;
- r_s = radius defining boundary of smeared zone;
- r_w = radius of drain well;

$s = \dfrac{r_s}{r_w}$ = ratio of radius of smeared zone to radius of drain well;

$T_h = \dfrac{k_h\,(1+e)\,t}{\gamma_w\,a_v\,d_e^{\,2}}$ = dimensionless time factor for consolidation by radial water flow;

$T_v = \dfrac{k_v\,(1+e)}{\gamma_w\,a_v}\,\dfrac{t}{H^2}$ = dimensionless time factor for consolidation by vertical water flow;

t = time;

Δt = time interval;

t_{90} = time for 90% consolidation;

$\bar{U}$ = average percentage of consolidation;

u = excess pore water pressure at a point in the clay layer;

$\bar{u}$ = average value of pore water pressure in the clay layer;

u_r = excess pore water pressure at a point in the clay layer as a result of radial flow of water;

u_x = excess pore water pressure at a point in the clay layer as a result of water flow in the x-direction;

u_z = excess pore water pressure at a point in the clay layer as a result of vertical flow of water;

$u_{o,t}$ = excess pore water pressure at point o, at time t;

$u_{2,t+\Delta t}$ = excess pore water pressure at point 2, at time $t + \Delta t$;

V_v = volume of voids;

$Y_o(\ )$ = Bessel function of the second kind, zero order;

$Y_1(\ )$ = Bessel function of the second kind, first order;

z = distance below the top surface of a clay layer;

Δz = interval of z;

$\alpha_1, \alpha_2, \alpha_3 \cdots$ = roots of the Bessel function;

γ_w = density of water;

ϵ = base of natural logarithms = $2.718\cdots$;

$\lambda = \frac{-8\,T_h}{f(n)}$ = exponent for the "equal strain" solution;

$\xi = \frac{8\,T_h}{\nu}$ = exponent in solution for equal strain with smear;

ν = factor in the solution for equal strain with smear (Eq. 20);

$\zeta = \frac{z}{H}$ = ratio of depth to a point to the thickness of the clay layer; and

$\bar{\sigma}$ = effective, or integranular pressure at any point in the soil layer.

DISCUSSION

YOSHICHIKA NISHIDA[21].—The author has presented a review that is useful for performing computations on sand drains. Mr. Richart has stated that including the effects of variable void ratio did not contribute to the explanation of secondary consolidation, and this is agreed with by the writer. The secondary consolidation appears to be due mainly to the creep or the flow of clay, and it occupies, in some cases, more than 50% of the consolidation. Mr. Richart checked the influences of well spacing, and his conclusion, that the radial flow dominated the consolidation-time behavior, is easily understood. The fact that doubling of the diameter of well influence, or the well spacing, causes an increase in the time for 90% consolidation by roughly a factor of 6 agrees with the value of 5.65 obtained from studies made in Japan. These studies showed that the time for consolidation is proportional to the 2.5 power of the well spacing. The writer wishes to know the most suitable spacing from the practical point of view, because a spacing that is too small brings the influence of remolding and smearing to the clay.

S. J. JOHNSON,[22] M. ASCE.—A singularly penetrating analysis of the influence of the various factors affecting the theoretical aspects of the design of sand drains has been presented by the author. The review of numerical methods of solution of consolidation phenomena and the means of taking into account variable loading and soil characteristics can be applied to problems of consolidation due to vertical flow and radial flow—of general interest beyond the particular field of sand drains. The effect of the diameter of sand drains pointedly illustrates how easy it would be to over-assess the importance of large diameters. However, the very small diameters referred to are presumably used only for illustration by the author. The writer interprets this presentation as not meaning to imply that such small diameters would be practicable for sand drains installed by methods currently used.

It is necessary for some purposes to know the theoretical excess pore water pressures at specific locations when piezometers are used for field control purposes and for evaluation of the coefficient of consolidation. These can be simply prepared from the property that, for the equal strain theory, the plot of log u/u_o versus the time factor is a straight line, as illustrated on Fig. 15(a). Also, the same is true for average excess pore water pressures as plotted on Fig. 15(b). This is not true for average consolidation due to vertical flow, which is plotted in a similar manner in Fig. 15(c). Fig. 15(a) shows the plots of curves for hydrostatic excess pressure versus time factor T_h and drain spacing, n, at the outer boundary, $r = 1.0\, r_e$ and r_e being equal to the effective radius of influence of the sand drain. The curves in Fig. 15(b) are plotted for the average hydrostatic excess pressure versus time factor, T_h, and average consolidation, $\frac{\bar{u}}{u_0}$, for a varying drain spacing, $n = d_e/d_w$. The curve in Fig. 15(c) was

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[22] Associate, Moran, Proctor, Mueser & Rutledge, New York, N. Y.

otted for the average hydrostatic excess pressure versus the time factor T_r nd average consolidation, $\frac{u}{u_o}$ for consolidation resulting only from vertical rainage, Figs 15(a) and 15(b) are obtained as follows:

Eq. 15 and Eq. 16 can be written, for any specific sand drain installation and ty point, r, from the drain, as:

$$= \frac{4\,\epsilon^\lambda}{d_e^2\,f(n)}\left[r_e^2 \log_e \frac{r}{r_w} - \frac{r^2 - r_w^2}{2}\right]$$

$$= \text{constant} \times \epsilon^\lambda = \text{constant} \times \left\{-\left[\frac{8\,T_h}{f(n)}\right]\right\} \ldots (47)$$

herefore,

$$\log_e \frac{u_r}{u_o} = C_1 + C_2\,T_h \ldots\ldots (48)$$

which

$$C_1 = \log_e\left\{\frac{4}{d_e^2\,f(n)}\left[r_e^2 \log_e \frac{r}{r_w} - \frac{r^2 - r_w^2}{2}\right]\right\} \ldots\ldots (49)$$

nd

$$C_2 = -\frac{8}{f(n)} \ldots\ldots (50)$$

his is the equation of a straight line when u_r/u_o is plotted to a log scale and T_h is plotted to an arithmetic scale. Fig. 15(a) is this relationship at the exrnal boundary where $r = d_e/2$.

The relationship between the average excess pore water pressure from $= r_w$ to $r = r_e$, and the time factor T_h is given by:

$$\frac{\bar{u}}{u_o} = \epsilon^\lambda = \epsilon^{-8T_h/f(n)} \ldots\ldots (51)$$

om which:

$$\log \frac{\bar{u}}{u_o} = -\frac{8}{f(n)}\,T_h \ldots\ldots (52)$$

This is also the equation of a family of straight lines, as plotted on Fig. 5(b). Plots of this type shown on Fig. 15(a) and Fig. 15(b) are readily conructed and are useful in analysis of field observations as well as in initial esign.

F. E. Richart, Jr.,[23] M. ASCE.—The writer wishes to thank Mr. Nishida nd Mr. Johnson for their interest in the paper and for their generous comments.

Mr. Nishida notes that a study made in Japan found that the time for conolidation is proportional to the 2.5 power of the well spacing. By use of q. 16 the time required for a particular degree of consolidation is proportional

[23] Div. of Eng. and Applied Physics, Harvard Univ., Cambridge, Mass.

to the 2.76 power and the 2.38 power of the well spacings when comparing times for 5 ft and 10 ft spacings, and 20 ft and 40 ft spacings, respectively. Thus, the use of the 2.5 power of the well spacing agrees quite well with results obtained from Barron's equal strain condition of consolidation.

With regard to Mr. Nishida's request for information on the most suitable well spacing from the practical point of view, this would necessarily depend on local soil and construction conditions. The terms "suitable" and "practical" imply that the spacings must be feasible and economical in both time and money. From a recent study of existing sand drain installations[24,25] the well spacings most frequently used in 83 installations were 8 ft to 10 ft.

Mr. Johnson is correct in his interpretation of the effectiveness of small diameter ideal wells. The writer was not proposing the use of actual sand drains of 1 in. or 2 in. in diameter. The discussion was intended to point out that even if the smeared zone around a 10 in. or 12 in. diameter actual drain well restricted drainage until the time for consolidation was comparable to that for a 1 in. or 2 in. diameter ideal well, the drain well would still be effective in reducing the time for consolidation.

Fig. 15 showing pore pressure on a log scale versus time factor on an arithmetic scale is most convenient. Values may be readily obtained from the figure, and there is little chance of drafting errors in constructing the diagrams. The writer thanks Mr. Johnson for pointing out this effective method of plotting consolidation-time information.

[24] "Study of Deep Soil Stabilization by Vertical Sand Drains," Report No. [illegible] by Moran, Proctor, Mueser, and Rutledge, Cons. Engrs., New York, N. Y., prepared for the Dept. of the Navy, Bureau of Yards and Docks, June, 1956.

[25] "Review of Uses of Vertical Sand Drains," by P. C. Rutledge and S. J. Johnson, presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January, 1957.

BOWLES: FOUNDATION ANALYSIS & DESIGN

Ponding may be used where the soil in contact with the water is of sufficiently low permeability. A perimeter dike can be constructed and the site ponded to the necessary depth to achieve the desired settlement.

6-4. DRAINAGE USING SAND BLANKETS AND DRAINS

The time for consolidation is computed from rearranging Eq. (2-24) to obtain the time as

$$t = \frac{TH^2}{c_v}$$

The dimensionless factor T depends on the percent consolidation (Table 2-5) and is about 0.848 and 0.197 for 90 and 50 percent consolidation. The coefficient of consolidation c_v is usually back-computed from a consolidation test by solving the above equation for c_v. The coefficient is also

$$c_v = \frac{k}{\gamma_w m_v}$$

where all terms have been defined in Chap. 2. For radial drainage as in sand drains, the coefficient of permeability k in the above equation would be the horizontal value, which is often four or five times as large as the vertical value.

The theory of radial drainage into sand drains, including allowance for "smear" effects on the sides of the holes reducing inflow, has been presented by Barron (1948) and more recently by Richart (1959). Since one is fortunate to determine the order of magnitude of k (the exponent of 10), for practical purposes the time for consolidation of a layer can be computed as:

1. Take $H = \frac{1}{2}$ distance between sand drains.
2. Compute c_v using the horizontal coefficient of permeability.
3. Use T from Table 2-5 for appropriate percent consolidation.

The time will be in some error owing to vertical drainage within the consolidating layer, depending on whether thin sand seams are present, drainage is from one or both faces, how the distance H compares with the clay thickness, etc.

Sand drains are installed by several procedures [see Landau (1966)] in diameters ranging from 6 to 30 in.

1. Mandrel-driven pipes—the pipe is driven with the mandrel closed. Sand is put in the pipe which falls out the bottom as the pile is withdrawn, forming the drain. Air pressure is often used to ensure continuity and densify the sand.
2. Driven pipes—the soil inside is then jetted. Rest of procedure is same as method 1.
3. Rotary drill—then filling boring with sand.
4. Continuous-flight hollow auger.

Figure 6-2 illustrates methods 1 and 3 above.

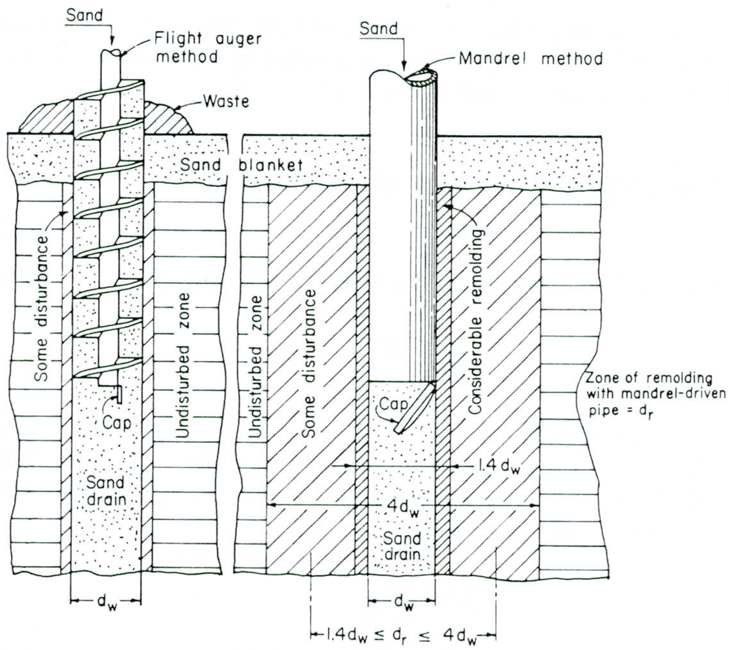


Figure 6-2. Two methods of constructing sand drains. [*Landau (1966).*]

When using earth for the surcharge, the length of the drainage path can be reduced by placing a sand blanket between the soil to be precompressed and the preload. Water can flow vertically through the sand drains to the sand blanket, then laterally to a ditch or other disposal means.

6-5. VIBRATORY METHODS TO INCREASE SOIL DENSITY

The allowable bearing capacity of sands depends heavily on the soil condition. This is reflected in the penetration number or cone resistance value as well as in the angle of internal friction. It is usually not practical to place a footing on loose sand because the allowable bearing capacity (based on settlements) will be too low to be economical. Additionally in earthquake analyses the local building code may not allow construction unless the relative density is above a certain value. Table 6-1 gives liquefaction-potential relationships between magnitude of earthquake and relative density for a water table 1.5 m below ground surface. This table can be used for a water table up to 3 m with slight error. The relative density may be related to penetration testing as in Table 3-2 after making suitable corrections to N for overburden effects.

The methods most commonly used to densify cohesionless deposits of sand and gravel with not over 20 percent silt or 10 percent clay are vibroflotation and insertion

Paper No. 5877

"Numerical Solution of Some Problems in the Consolidation of Clay"

by

Robert Edward Gibson, B.Sc. (Eng.), and Peter Lumb, M.Sc. (Eng.), Studs I.C.E.

(Ordered by the Council to be published with written discussion)†

SYNOPSIS

The rate of consolidation settlement of structures founded above clay strata is usually calculated on the simplifying assumption that the flow of pore-water takes place in one direction only. Although it is widely appreciated that the flow is generally three-dimensional, exact mathematical solutions to problems have been obtained in very few cases which are of importance to the civil engineer.

In this Paper a "step-by-step" method for a solution of the governing equation is presented and used to solve a number of problems. Exact solutions are known to the simpler of these, which fact enables a comparison to be made with the solution as obtained by the numerical procedure. To the more involved problems concerning the rate of settlement of a uniformly loaded circular footing exact solutions are difficult to obtain but the method outlined has enabled a solution, sufficiently accurate for all engineering purposes, to be rapidly evaluated. This solution indicated that the rate of settlement is, in fact, appreciably greater than that found from the simple one-dimensional theory.

Introduction

In the theory of consolidation of homogeneous isotropic clay which was first formulated by Professor Karl von Terzaghi in 1925,[1] the simplifying assumption was made that the flow of pore-water takes place in one direction only. This is a condition which would obtain in practice if a stratum of clay were uniformly loaded over its surface. This one-dimensional consolidation process is governed by an equation ‡ of the form:

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad \qquad (1)$$

The solution of this equation, in any particular case, requires that the position of the drainage surfaces and the stress distribution in the layer should be known. Solutions for a number of cases have been given by Terzaghi and Fröhlich (1936). The validity of applying this simple theory to determine the rate of settlement of a foundation will depend on the extent to which the condition of one-dimensional flow is satisfied. In many cases it is not even approximately satisfied and in recognition of this limitation the consolidation equation is modified to become:

$$\frac{\partial u}{\partial t} = c\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2}\right) \qquad \qquad (2)$$

this equation being assumed to hold so long as the foundation loads do not vary with time.

A general theory of consolidation, which showed that the stress distribution and the consolidation process are interconnected, was developed by Biot.[2] From this theory the Authors have shown (Appendix II) that the consolidation process may be described by the equation:

$$-\frac{1}{3}\frac{\partial \theta}{\partial t} + \frac{\partial u}{\partial t} = c\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2}\right) \qquad \qquad (3)$$

where θ denotes the sum of the normal total stresses $(\sigma_{xx} + \sigma_{yy} + \sigma_{zz})$ at any point. It is seen that equation (2) is a good approximation to this equation, for, when the foundation loads are constant, it may be shown that the variation of θ with time is generally very small, arising only from the changes during consolidation of Poisson's Ratio and other parameters of the clay with respect to the total stresses.

Equations, similar in form to those above, describe the flow of heat in a homogeneous solid, but although exact solutions to a great variety of problems in this field have been published,[3] few are of direct value to foundation engineers. However, two numerical methods of solving such equations have been developed [4,5] which are suitable for application to consolidation problems.

In this Paper a numerical method, which is similar to that given in reference 4, will be used in solving equation (2) for a number of different cases, comparisons being made with exact solutions where they are available. This method is capable of extension to cases where c, the coefficient of consolidation is not constant, and where the boundary conditions are more complicated, but such extensions will not be considered here.

One-Dimensional Consolidation

As an illustration of the method, the simplest of the cases of one-dimensional consolidation will be discussed.

A uniform vertical pressure u_0 is applied to a laterally confined cylinder of clay, of thickness $2H$, which is permitted to drain freely from its horizontal surfaces. Consolidation takes place and after a certain time, largely depending upon the thickness of the sample, it practically ceases and the

† Correspondence on this Paper should be received at the Institution by the 1st July, 1953, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

[1] The references are given on p. 196.

‡ For notation, see Appendix I.

sample can then be considered to be in equilibrium. If the final compression of the sample is denoted by S_∞ (since theoretically an infinite time is required for equilibrium to be reached) and the compression at any time t by S_t, then the average degree of consolidation $\bar{U}$ at this time is defined by the equation :—

$$S_t = \bar{U}\, S_\infty$$

In order to determine the relationship between the degree of consolidation $\bar{U}$ and the time, it is necessary to consider the equation :

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t}$$

which describes the variation with z and t of the pore-water pressure u within the sample.

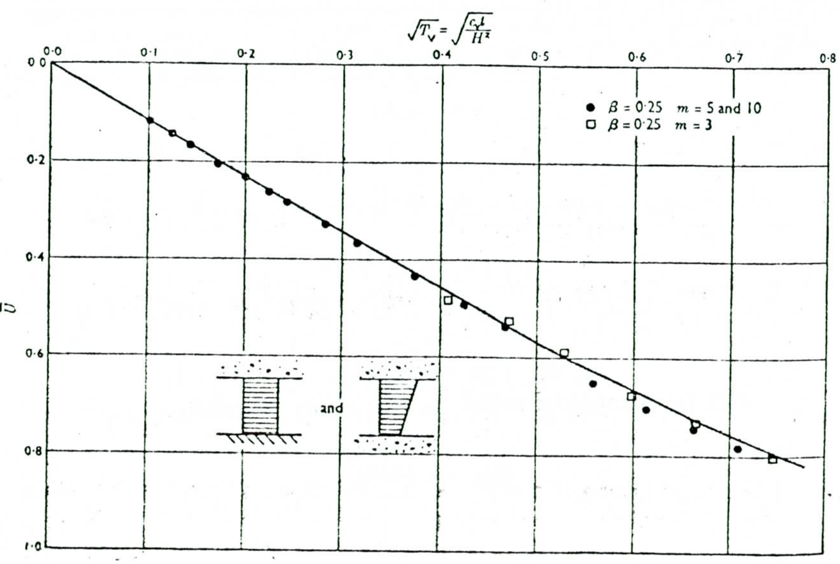
The solution of this equation, with the boundary conditions :

$$u = 0 \text{ at } z = 0 \text{ and } 2H, \text{ when } t > 0$$
$$u = u_o \text{ at } 0 \leq z \leq 2H, \text{ when } t = 0,$$

has been given by Terzaghi and Fröhlich[6] in the form :

$$u = \frac{4}{\pi} u_o \sum_{n=0}^{\infty} \frac{1}{(1+2n)} \sin\left(\frac{(1+2n)\,\pi\, z}{2H}\right) \epsilon^{-(2n+1)^2 \frac{\pi^2}{4} T_v}$$

Fig. 1



ONE-DIMENSIONAL CONSOLIDATION. FINITE LAYER. LINEAR LOADING

where $T_v = \frac{c_v t}{H^2}$ is a dimensionless independent variable called the "time factor."

If it is assumed that the compression depends linearly upon the effective stress change $(u_o - u)$, it follows that the degree of consolidation is given by :

$$\bar{U} = \frac{\int_0^{2H} (u_o - u)\, dz}{\int_0^{2H} u_o dz} \quad \ldots \ldots \quad (4)$$

$$\text{or :} \quad \bar{U} = 1 - \frac{8}{\pi^2} \sum_{n=0}^{\infty} \frac{1}{(1+2n)^2} \epsilon^{-(1+2n)^2 \frac{\pi^2}{4} T_v} \quad \ldots \ldots \quad (4a)$$

This relationship between $\bar{U}$ and T_v is shown in *Fig.* 1.

THE NUMERICAL METHOD

Consider a rectangular region (*Fig.* 2), every point of which is specified by co-ordinates z and t. A rectangular mesh of sides δt and δz, extending over the whole region, is constructed and, instead of considering the values of u at all the points of the region, only those at the mesh points such as 0, 1, 2, 3, and 4, are considered. (See *Fig.* 3.) At the point 0 the pore-water pressure must satisfy equation (1) :

$$\left(\frac{\partial u}{\partial t}\right)_o = c_v \left(\frac{\partial^2 u}{\partial z^2}\right)_o$$

and since* :

$$\left(\frac{\partial^2 u}{\partial z^2}\right)_o \simeq \frac{u_2 + u_4 - 2u_o}{\delta z^2}$$

and :

$$\left(\frac{\partial u}{\partial t}\right)_o \simeq \frac{u_1 - u_o}{\delta t}$$

it follows that :

$$u_1 \simeq \beta\,(u_2 + u_4 - 2u_o) + u_o \quad \ldots \ldots \quad (5)$$

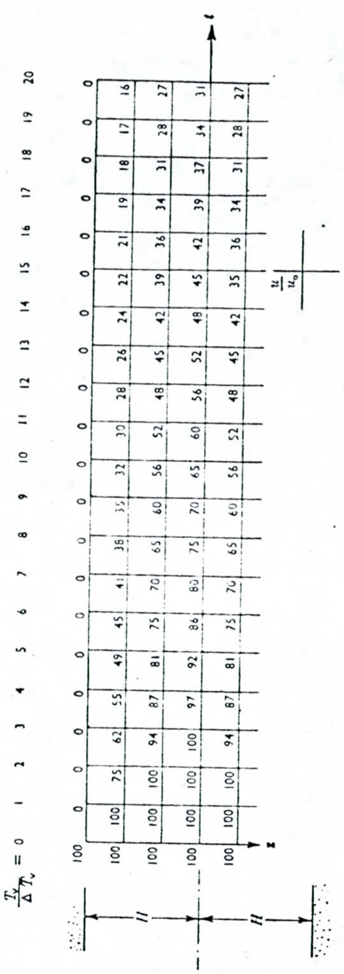
where $\beta = \frac{c_v \delta t}{\delta z^2}$

Since the boundary and initial conditions have been specified it will be seen that a successive application of equation (5) to mesh points proceeding from smaller to larger times will give the value of u at all mesh points (see *Fig.* 2).

The accuracy of the solution will depend upon the number of intervals

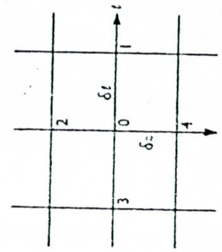
* For a discussion of finite difference approximations to derivatives see reference 7.

Fig. 2



NETWORK SHOWING VARIATION OF u AND T_v IN THE OEDOMETER TEST ($\beta = 0{\cdot}25$). NUMERICAL SOLUTION

Fig. 3



$2m$ ($= 2H/\delta z$) into which the thickness of the sample has been divided, and upon the value chosen for β. Smaller values of β and larger values of m will correspond to more exact solutions, although the work involved will be proportionately increased. If $t = n\delta t$, then the time factor can be evaluated from the expression :

$$T_v = \frac{n}{m^2} \cdot \beta$$

Suppose an initial pressure of 100 units is applied to the sample then during the time interval 0 to δt the pore-water pressure on the boundaries drops from 100 units to zero, whilst the succeeding values of u at all the

internal mesh points are calculated using equation (5) with values of $\beta = 0{\cdot}25$ and $m = 3$. To increase the accuracy of the solution the calculation was repeated with $\beta = 0{\cdot}25$, first with $m = 5$ and then with $m = 10$. The results of these calculations are shown in *Fig. 1* where they are compared with the exact solution for the relation with $\bar{U}$ and T_v as found from equation (4*a*). It will be seen that over the range $0 < \bar{U} < 0{\cdot}8$ the agreement between the numerical method and the exact solution is sufficiently close for practical purposes.

This numerical method has, however, the disadvantage that it is difficult to determine the pore-water pressure-distribution accurately after long periods of time. This is rarely required in practice, but by the use of relaxation methods [5] it could be determined without excessive labour.

CONSOLIDATION OF A CYLINDER OF CLAY

Consideration will now be given to problems which possess axial symmetry. The first cases considered are those relating to the consolidation of a cylinder of clay. Here again, exact solutions exist and the object of applying the numerical method is to examine its reliability in such cases.

If an all-round pressure u_o is applied to a cylindrical sample of clay, of radius R and length $2H$, which is permitted to drain radially, the equation of consolidation is most conveniently written in terms of polar co-ordinates :

$$\frac{\partial u}{\partial t} = c\left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

From a solution given by Carslaw and Jaeger,[3] relating to heat flow, it may easily be shown that the average degree of consolidation is given by

$$\bar{U}w_n{}^2 = 1 - 4\sum_{n=1}^{\infty} \frac{1}{w_n{}^2} \cdot \epsilon^{-\frac{ct}{R^2} w_n{}^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where w_n is the nth root of the equation $J_o(w) = 0$, J_o denoting the Bessel function of order zero.

From this relation, $\bar{U}$ as a function of $T_R = \frac{ct}{p^2}$ has been plotted in *Fig. 4*.

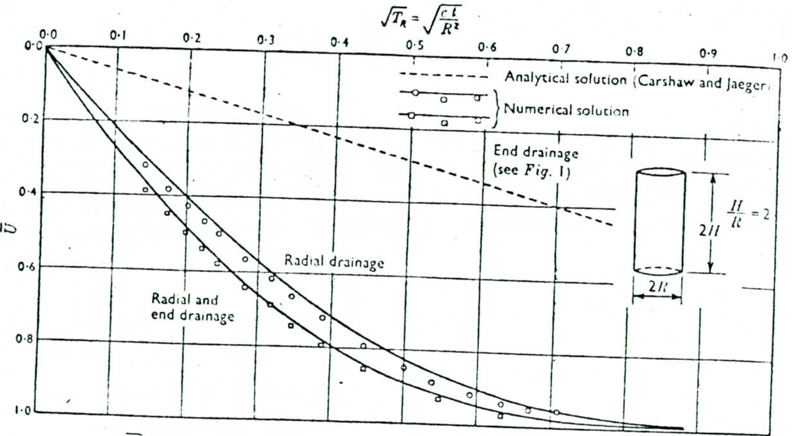
To obtain a numerical solution, equation (6) is written in the form :

$$u_4 \simeq \beta\left[u_1 + u_3 - 2u_o + \frac{(u_1 - u_3)}{2p}\right] + u_o \quad . \quad . \quad (8)$$

$$\text{where } \beta = \frac{c\delta t}{\delta r^2} \text{ and } p = \frac{r}{\delta r},$$

by making use of the usual approximations.

Fig. 4



$\overline{U}$—T_R CURVES FOR CONSOLIDATION OF A CLAY CYLINDER

This equation cannot be used on the axis $r = 0$ where $p = 0$, and it is necessary only to note that as $r \rightarrow 0$:

$$\frac{1}{r}\frac{\partial u}{\partial r} \rightarrow \frac{\partial^2 u}{\partial r^2}$$

so that on the axis $r = 0$:

$$u_4 = 2\beta\,(u_1 + u_3 - 2u_o) + u_o \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

In order to extend the solution to a large range of time with the minimum of labour, two cases were worked out in which:

(1) . $\frac{R}{\delta r} = 5, \beta = 0{\cdot}25$

and (2) $\frac{R}{\delta r} = 2, \beta = 0{\cdot}20$

the values of $\overline{U}$ being calculated from:

$$\overline{U} = \frac{\int_o^R (u_o - u)r.dr}{\int_o^R u_o r.dr}$$

which are shown in *Fig. 4*.

The comparison between this numerical solution and the exact solution is reasonably good, although the values of $\overline{U}$ obtained from the numerical method are consistently rather higher than the exact values, the maximum error in U being about 4 per cent.

A more involved case is that in which a cylinder is permitted to drain

freely from all its surfaces. The equation which must be satisfied may be written in cylindrical co-ordinates:

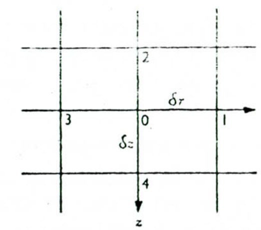
$$\frac{\partial u}{\partial t} = c\left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial z^2}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

It has been shown by Carrillo [8] that this three-dimensional radial flow may be resolved into a plane radial flow (equation (6)) and a linear flow (equation (1)), and if $\overline{U}_R$ and $\overline{U}_v$ respectively are the degrees of consolidation due to these flows, then the degree of consolidation in three-dimensional axially symmetrical flow $\overline{U}$ is given by:

$$\overline{U} = 1 - (1 - \overline{U}_R)\,(1 - \overline{U}_v) \quad . \quad . \quad . \quad . \quad (11)$$

From this equation the variation of $\overline{U}$ with the time factor $T_R = \frac{ct}{R^2}$ has been computed using equations (4*a*) and (7) (see *Fig. 4*) for a cylinder with $H/R = 2$.

Fig. 5



Since three co-ordinates r, z, and t occur in equation (10) it becomes necessary to associate with each node of the network a series of values of the pore-water pressure, showing its variation with time, the co-ordinates r and z occupying the plane of the paper. (See *Fig. 5*.)

The equation (10), written in finite difference form, becomes:

$$u_o\,(t + \delta t) \simeq \beta\left[u_1 + u_2 + u_3 + u_4 - 4u_o + \frac{u_1 - u_3}{2p}\right] + u_o \quad . \quad . \quad (12)$$

where $p = \frac{r}{\delta r}$, $\beta = \frac{ct}{h^2}$, and $\delta r = \delta z = h$

and on the axis it takes the form:

$$u_o\,(t + \delta t) \simeq \beta\,(2u_1 + u_2 + 2u_3 + u_4 - 6u_o) + u_o \quad . \quad . \quad . \quad (13)$$

For a cylinder in which the ratio of the length $2H$ to the diameter $2R$ was 2, two cases were worked out in which:

(1) $\frac{R}{\delta r} = 5, \quad \beta = 0{\cdot}05$ and $0{\cdot}25$

$$\text{and} \quad (2)\ \frac{R}{\delta r} = 2, \quad \beta = 0{\cdot}2 \text{ and } 0{\cdot}4$$

the values of $\bar{U}$ being calculated from the equation :

$$\bar{U} = \frac{\int_o^{2H} \int_o^R (u_o - u)\, r.dr.dz}{\int_o^{2H} \int_o^R u_o r.dr.dz}$$

and compared with the exact solution in *Fig. 4* as obtained from Terzaghi's linear flow solution and Carslaw and Jaeger's radial flow solution, together with equation (11). The agreement is seen to be good.

Consolidation in Clay Beneath a Circular Footing

Having shown in three cases, involving one- and two-dimensional consolidation processes, that the numerical method yields results in close agreement with the exact solutions, the numerical method may now be applied to problems for which no exact solution exists, and for which an exact solution would be difficult to obtain and evaluate.

Two cases of some practical interest will be considered, namely, the consolidation of a clay layer beneath a uniformly loaded circular flexible footing ; first, with a footing very permeable in comparison with the clay, and secondly with an impermeable footing.

In order to estimate the time/settlement curve in such cases it has been usual to employ the one-dimensional consolidation equation (1), but it is evident that appreciable error may thereby be introduced. The three-dimensional analytical solution of such problems presents great difficulties, whilst the numerical solution involves little more labour than that required in the previous example.

Considering now the case of the permeable footing, the pore-water pressure u must satisfy equations (12) and (13) together with the boundary conditions :

$$z = H, \quad \frac{\partial u}{\partial z} = 0,\ t \geqslant 0$$

$$z = 0, \quad u = 0,\ t > 0$$

the pore-water being free to drain both through the surface of the clay and through the footing.

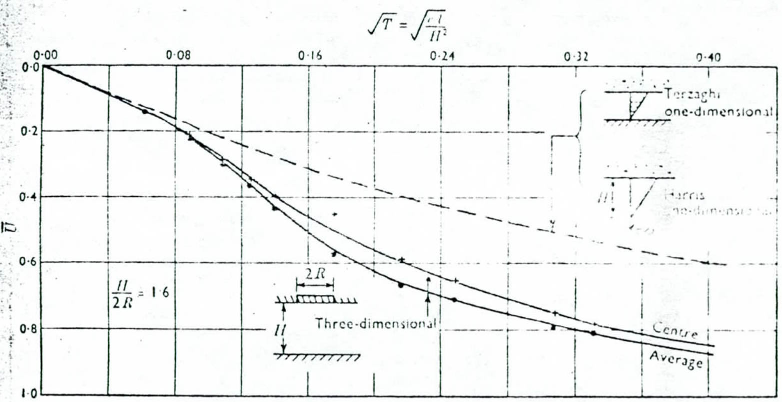
The initial condition, that is, the value of the pore-water pressure immediately after the load has been applied, has been assumed to be equal to the major principal stress. This assumption is in accord with the limited evidence available, and is moreover a consequence of Skempton's λ-theory,[9] which predicts that the initial pore-water pressure, before any drainage has had opportunity to occur, is given by :

$$u_o = \frac{\sigma_1 + \lambda(\sigma_2 + \sigma_3)}{1 + 2\lambda}$$

where σ_1, σ_2, and σ_3 denote the major, intermediate, and minor principal stresses at a point and λ is a parameter characterizing the behaviour of the clay. For most clays it appears that λ has small values (about 0·05 to 0·30) and the assumption made above is equivalent to taking λ as being zero.

The rate of consolidation of the centre of the loaded area was evaluated in a case where the ratio of the depth of the clay stratum to the diameter of the loaded area was 1·6, the solution being shown in *Fig. 6*. The initial pore-water pressure, assumed to be equal to the major principal stress, was evaluated from Tables prepared by Jürgenson[10] from Boussinesq's solution. This solution is compared with solutions of the one-dimensional

Fig. 6



$\bar{U}$—T Curves for Consolidation of Clay Layer. Circular Uniform Load (Permeable Surface). Numerical Solution

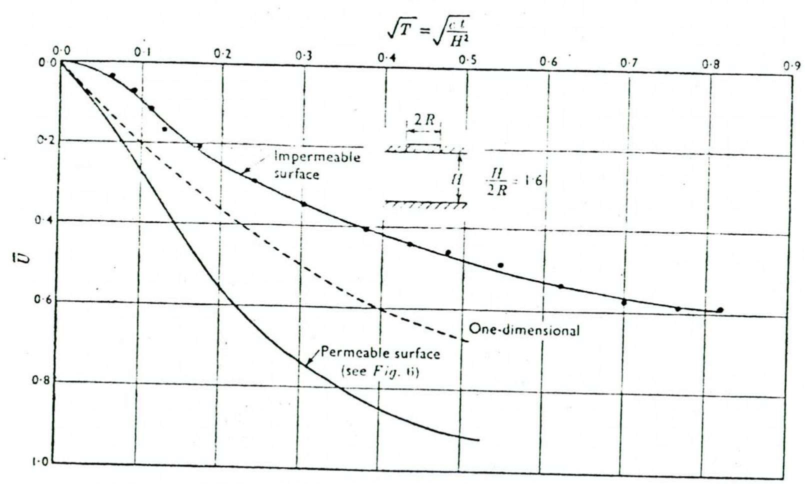
equation derived by Terzaghi and Fröhlich[6] and by Harris,* where the initial pressure in the pore-water is distributed triangularly with depth, the depths of the stratum being assumed to be finite and infinite respectively in these two cases. For about the first half of the consolidation process ($\bar{U} < 0{\cdot}6$) these two theoretical solutions do not differ greatly and thus it may be inferred that the numerical solution of the problem which includes radial flow would hold, at least for short times even if the thickness of the clay stratum were somewhat greater than the $1{\cdot}6 \times 2R$ adopted.

Turning now to the comparison between this numerical solution and the one-dimensional solution (see *Fig. 6*), it will be seen that, except for small times ($\bar{U} < 0{\cdot}2$) the consolidation with three-dimensional flow is decidedly more rapid, as would be expected.

* A private communication from A. J. Harris. The work was carried out at the Building Research Station (Department of Scientific and Industrial Research) and the results are given with permission from the Director of Building Research.

A further problem of interest is that of determining the rate of consolidation of the clay beneath an impermeable circular footing founded at considerable depth. An approximate analysis of this problem has been carried out by replacing the permeable upper surface of the clay in the previous case by an impermeable surface. Since the pore-water is not permitted to escape, a redistribution takes place in the stratum, the pore-water flowing away from regions of high stress intensity around the buried footing. As a consequence of this redistribution, swelling takes place in regions away from the footing and a coefficient of rate of swelling, somewhat larger than c, should be used. This factor could be incorporated into the numerical solution of the problem but it was not considered worth while, since the errors involved would probably be small.

Fig. 7



$\overline{U}$—T CURVES FOR CONSOLIDATION OF CLAY LAYER. CIRCULAR UNIFORM LOAD (IMPERMEABLE SURFACE)

The solution, based on the assumption that the coefficient of swelling equals the coefficient of consolidation, is given in *Fig. 7*. A comparison is made with the permeable-footing case and the one-dimensional solution and, although the rate of consolidation beneath the impermeable circular footing is definitely less than that of the permeable footing, it is interesting to note that the degree of consolidation beneath the impermeable footing is rather more than two-thirds that in the one-dimensional case even with a permeable footing, for all time-factors greater than about 0·1.

It is probable that the relation between $\overline{U}$ and T given in *Fig. 7* can be applied as a rough approximation to impermeable footings underlain by a clay stratum of thickness other than $1{\cdot}6 \times 2R$.

CONSOLIDATION OF CLAY CORES IN EARTH DAMS

In engineering practice an important process of two-dimensional consolidation occurs in the case of hydraulic-fill dams. An analysis of this problem has been carried out by G. Gilboy[11] for a triangular core with a base angle of 45 degrees resting upon an impermeable stratum, which yields a solution :

$$\overline{U} = 1 - \frac{24}{\pi^4}\left[\sum_{m=0}^{\infty}\sum_{n=0}^{\infty}\frac{2}{(2n+1)^2\,(2m+1)^2}\,\epsilon^{-[(2n+1)^2+(2m+1)^2]\frac{\pi^2 T}{2}} + \sum_{n=0}^{\infty}\frac{1}{(2n+1)^4}\,\epsilon^{-(2n+1)^2\pi^2 T}\right]$$

where $T = \frac{ct}{b^2}$ and $2b$ denotes the core base width. It may be shown that when the base angle of the core is 90 degrees the solution is that of the one-dimensional case (see equation (4*a*)).

As a means of predicting the rate of consolidation of cores with intermediate base angles, Gilboy proposed an interpolation formula :

$$T'_{\alpha} = T_{90} \cot \alpha \,(T_{90} - T_{45})$$

where α denotes the base angle of the core.

In order to examine the validity of this formula and to extend the above solution, numerical analyses were applied to cores with base angles of 15, 60, and 75 degrees. The equation controlling the pore-water pressure is :

$$\frac{\partial u}{\partial t} = c\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial z^2}\right) \quad . \quad . \quad . \quad . \quad . \quad (14)$$

If a square mesh in the x,z plane is used for the latter two cases the nodes of the mesh do not lie on the inclined faces of the core. This is a disadvantage from a computational point of view and in order to overcome this difficulty it is convenient to transform the above equation by the substitution :

$$z = w \tan \alpha$$

where w and z denote co-ordinates measured vertically from the centre of the base. Then :

$$\frac{\partial u}{\partial t} = c\left(\frac{\partial^2 u}{\partial x^2} + \cot^2\alpha\,\frac{\partial^2 u}{\partial w^2}\right)$$

Thus, in the x,w plane all cores are transformed into triangles with a base angle of 45 degrees, and therefore the difficulty mentioned above does not arise.

The boundary conditions in the x,w plane are :—

(1) Along the inclined faces $u = 0$, $t > 0$

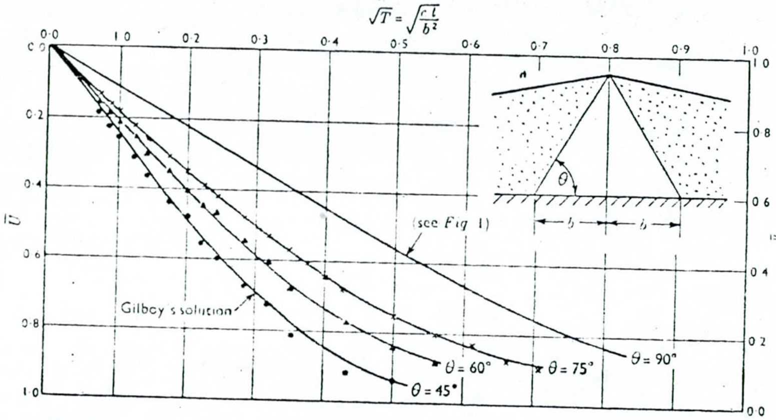
(2) Where $w = 0$, $\frac{\partial u}{\partial w} = 0$ $t \geqslant 0$

In order to effect a comparison with Gilboy's solution his assumption with regard to the initial values of the pore-water pressure was made, namely, that the pore-water pressure at a point is proportional to its vertical depth below the apex of the core. This question has been investigated in some detail by A. W. Bishop[12] and his conclusions indicate that the assumption is sufficiently good for all engineering purposes.

A solution has been obtained for the three cases mentioned above, and the average rates of consolidation determined (*Fig. 8*), the average

Fig. 8

$\sqrt{T} = \sqrt{\frac{ct}{b^2}}$



$\bar{U}$—T CURVES FOR CONSOLIDATION OF A HYDRAULIC-FILL CORE IN A DAM

degree of consolidation in the core being connected with the average pore-water pressure $\bar{u}$ by the equation :

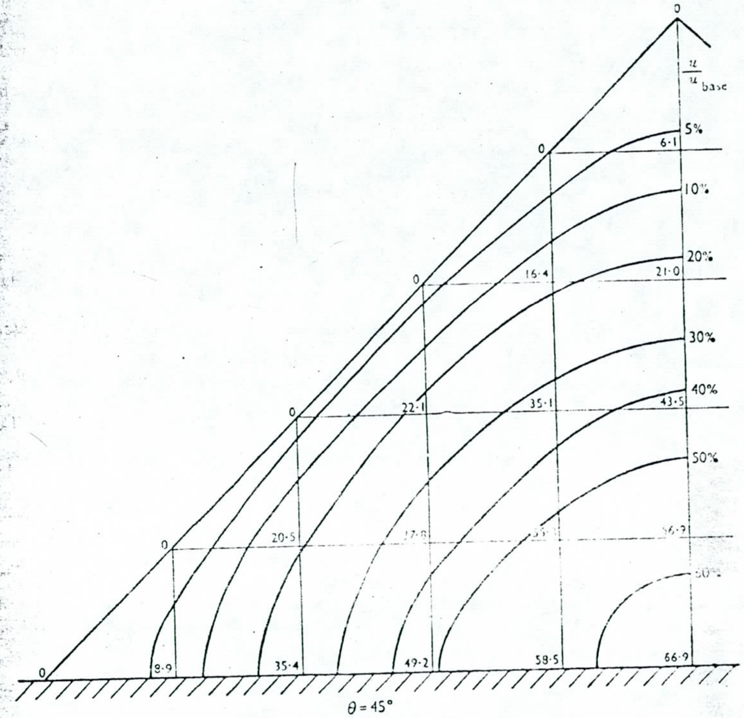
$$\bar{U} = 1 - \frac{\bar{u}}{\bar{u}_o}$$

In stability analyses it may be necessary to know the distribution of pore-water pressure in the core, and a typical distribution is shown in *Fig. 9* for the case where $\alpha = 45$ degrees, at a time factor $T = 0{\cdot}06$ (the initial pore-water pressure at the base of the core being equal to 100 units).

It will be seen that good agreement with Gilboy's solution has been obtained for the 45-degree core. The interpolation formula proposed by Gilboy was not, however, found to be valid.

It must be emphasized, however, that the case considered above is rather artificial, since in practice the greater proportion of the consolidation occurs during the construction period. In this case the consolidation equation (14) must be modified by the inclusion of a term involving the rate of construction, and account must be taken of the fact that the disposition of the drainage surfaces is changing with time. A numerical

Fig. 9



CONTOURS OF EQUI-PORE-PRESSURE FOR HYDRAULIC FILL OF A DAM AT A TIME FACTOR OF $T = 0{\cdot}06$

solution could be obtained to this problem where an exact solution would be virtually impossible.

CONCLUSIONS

It has been shown that results in good agreement with existing exact solutions of consolidation problems may be found by using numerical methods. The advantage of such methods lies in their ability to take into account features in the problem which would have to be excluded or over-simplified in an analytical solution. The accuracy obtainable is quite sufficient for all engineering purposes.

ACKNOWLEDGEMENTS

The work described in this Paper was carried out in the Civil Engineering Department, Imperial College, London.

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The Paper is accompanied by six sheets of diagrams, from which the Figures in the text have been prepared, and by the following Appendices.

APPENDIX I

Notation

x, y, z	denotes	Cartesian co-ordinates.
$\delta x, \delta y, \delta z$	"	length of mesh sides.
z, r, θ	"	cylindrical co-ordinates.
$\delta z, \delta r, \delta\theta, h$	"	length of mesh sides.
t	"	time.
δt	"	time interval.
u	"	excess hydrostatic pressure in the pore-water.
c_v	"	coefficient of one-dimensional consolidation.
c	"	coefficient of two- or three-dimensional consolidation.
σ_{xx}, σ_{yy}, etc.	"	normal total stresses.
$\sigma_{xx}', \sigma_{yy}'$, etc.	"	normal effective stresses.
$\sigma_1, \sigma_2, \sigma_3$	"	principal total stresses.
θ	"	sum of normal total stresses $(\sigma_{xx} + \sigma_{yy} + \sigma_{zz})$.
θ'	"	sum of normal effective stresses $(\sigma'_{xx} + \sigma'_{yy} + \sigma'_{zz})$.
e_{xx}, e_{yy}, etc.	"	normal strains.
$\triangle$	"	sum of normal strains $(e_{xx} + e_{yy} + e_{zz})$.
k	"	permeability coefficient of the soil.
μ	"	Poisson's Ratio for the soil structure.
E	"	Young's Modulus for the soil structure.

$\bar{U}$	"	average degree of consolidation.
T	"	time factor.
$2H$	"	thickness of consolidation sample.
u_0	"	initial pore-water pressure.
β	=	$\frac{c.\delta t}{h^2}$
m	=	$H/\delta z$
n	=	$t/\delta t$
p	=	$\frac{r}{\delta r}$
R	denotes	radius of cylinder.
λ	"	ratio of compressibility to expansibility of soil structure.
$2b$	"	base width of hydraulic-fill dam core.
α	"	base angle of core.
w	=	$z/\tan\alpha$.

APPENDIX II

General Equations of Consolidation

In the general theory of consolidation as developed by Biot,[2] three differential equations are derived relating the displacements ρ_x, ρ_y, and ρ_z to the pore-water pressure u. The connexion between these equations and the familiar Terzaghi equation is not apparent. In the following the Biot equations are cast into a form that makes this connexion clear. The strains are assumed to be connected with the effective stress changes by equations such as:

$$e_{xx} = \frac{(1+\mu)}{E}\,\sigma'_{xx} - \frac{\mu}{E}\,\theta' \quad \ldots \ldots \quad (15)$$

where the effective and total stress changes are related by equations such as:

$$\sigma'_{xx} = \sigma_{xx} - u \quad \ldots \ldots \quad (16)$$

Adding equations (15):

$$(e_{xx} + e_{yy} + e_{zz}) = \frac{(1-2\mu)}{E}\,\theta'$$

Adding equations (16):

$$\theta' = \theta - 3u$$

Thus:

$$\frac{E\triangle}{3(1-2\mu)} = \frac{1}{3}\theta - u$$

Further, by assuming d'Arcy's law to be valid, it may be shown that:

$$k\nabla^2 u + \frac{\partial \triangle}{\partial t} = 0$$

where

$$\nabla^2 \equiv \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$$

Thus:

$$-\frac{1}{3}\frac{\partial\theta}{\partial t} + \frac{\partial u}{\partial t} = c\nabla^2 u \quad \ldots \ldots \quad (17)$$

where

$$c = \frac{kE}{3(1-2\mu)}$$

VERTICAL DRAINS

THOMAS TELFORD LTD
LONDON 1982

Consolidation of soil using vertical drains

M. S. ATKINSON* and P. J. L. ELDRED*

The use of vertical drains is examined critically in view of the fact that in half a century of use there have been a large number of occasions when vertical drains have failed to promote a more rapid consolidation of the soil under load. The mode of operation of the drain is discussed and it is shown that an understanding of this has affected the design of the more recent band-shaped drains. The effects of smear and drain resistance are also discussed. Reference is made to a three-dimensional finite difference computer program, modelling the flow of excess pore water in the drain and the surrounding soil which is used to aid in the drain design. Specific cases which can also be analysed by published techniques have been used to check the validity of the model. Consideration is also given to the methods of site investigation which should be employed to obtain the appropriate soil parameters for design. Cases are cited where a lack of data on the soil parameters has restricted the design of the vertical drainage schemes.

L'utilisation de drains verticaux fait l'objet d'une analyse critique car au cours d'un demi-siècle de service, les drains verticaux se sont révélés, à maintes reprises, incapables de permettre une consolidation plus rapide du sol sous chargement. L'article décrit le mode de fonctionnement d'un drain et montre comment la compréhension de ce mode a influencé la conception des drains récents en forme de bande. Il étudie également les effects de l'étalement et de la résistance des drains.

L'article se réfère à un programme informatique à différences finies tridimensionnel qui modèl isele débit de l'eau interstitielle excédentaire dans le drain et le sol environnant; celui-ci sert à faciliter la conception des drains. Des cas spécifiques qui peuvent également être analysés à l'aide de techniques existantes déjà publiées ont été utilisés pour vérifier la validité du modèle.

L'article traite ègalement des méthodes d'exploration in situ qui doivent être utilisé eb pour obtenir des paramètres du sol appropriés en vue de la conc[illegible]ion. Il présente des cas où un manque de données concernant les paramètres du sol a considérablement limité la conception de systèmes de drainage vertical.

NOTATION

c_h	horizontal coefficient of consolidation
c_v	vertical coefficient of consolidation
D	drain spacing
D_E	diameter of equivalent cylinder of drained soil
D_W	diameter of drain
d	equivalent diameter of a band drain
H	depth of soil from a free draining horizontal surface to an impervious one
k_h	average horizontal coefficient of permeability
k_v	average vertical coefficient of permeability
k_x	horizontal coefficient of permeability in the x direction
k_y	horizontal coefficient of permeability in the y direction
k_z	vertical coefficient of permeability in the z direction
m_v	coefficient of volume compressibility
N	integer varying between zero and infinity
n	D_E/D_W
t	time
Δt	elemental increment of time
T_h	time factor for radial flow $= c_h t/D_E^2$
T_v	time factor for vertical flow $= c_h t/H^2$
u	excess pore pressure
Δ_u	remaining excess pore pressure
Δu_0	initial excess of pore pressure
U_r	average degree of consolidation due to radial drainage only
U_v	average degree of consolidation due to vertical drainage only
U_{rv}	average degree of consolidation due to both radial and vertical drainage
x	a co-ordinate in the rectangular system
Δx	an elemental length in the x direction
y	a co-ordinate in the rectangular system
Δy	an elemental length in the y direction
z	a co-ordinate in the rectangular system
Δz	an elemental length in the z direction

* Soil Mechanics Ltd.

INTRODUCTION

Vertical drains have been employed for almost half a century to promote more rapid consolidation of relatively thick deposits of soft fine grained soils. The first installation appears to have been in California in 1934 (Porter, 1936) using 20 in. dia. sand drains at 10 ft centres. Until the early 1970s the majority of the vertical drains also used large-diameter sand drains. In the United States of America these were installed mainly by means of closed-ended mandrels which caused a considerable thickness of smear around the drain. The smear problem was overcome in the Netherlands during the 1950s with the development of jetted sand drains. This method had its own problems, notably the additional cost of large jetting pumps and the difficulties of disposing of large quantities of water; it is not nowadays frequently used. The principal alternative to large-diameter sand drains was the much smaller band-shaped cardboard wicks first employed by Kjellman (1948). These early band drains proved to be susceptible to rotting, particularly in acidic groundwaters, and the use of band drains was not common until the last decade. In the 1970s the rising costs of providing large quantities of suitable sand for sand drains and the great technical advances in the manufacture of man-made fabrics led to the development and use of increasing numbers of band drains produced from polyethylenes, PVC, polypropylenes and polyesters etc. Currently there are a large number of these manufactured drains available and a cursory examination suggests that there is much similarity between the majority of them. Generally they consist of a central core, whose function is primarily to act as a free-draining water channel, surrounded by a thin filter jacket, which prevents the soil surrounding the drain from entering the central core but allows free entry to the core of the excess pore water.

In order to assess the viability of installing vertical drains in general, and band drains in particular, it is necessary to consider the potential gains from the use of drains, the mode of operation of the drain and how these can be affected by the ground conditions, the design of the drain and the method of installation.

POTENTIAL ADVANTAGES OF VERTICAL DRAINS

In general terms the installation of vertical drains into a relatively thick stratum of clay before the application of a load should increase the rate of consolidation of the clay under the load by shortening the drainage path. In addition, in non-uniform soils the horizontal permeability may be considerably greater than the vertical permeability; this anisotropy confers an additional advantage on the use of drains. The advantages can be threefold.

(a) The increased rate of gain in shear strength of the clay enables the load to be applied more rapidly than would otherwise have been possible, often allowing a better utilization of construction plant. Furthermore, in the case of embankments, steeper side slopes and the avoidance of the use of berms may also be possible when vertical drains are employed. Thus the total volume of fill required may be reduced and the rate of construction increased. The consequent savings in cost may be appreciably more than the outlay on the vertical drain installation.

(b) The increased rate of consolidation of the clay results in a reduction of the time required for primary settlement to take place. Consequently structures can be built or embankments can be put into commission far earlier than would otherwise have been possible, or the subsequent maintenance costs can be greatly reduced. Again there can be a considerable cost saving.

(c) Many soft clay strata contain thin bands, or partings, of silt or sand. Instability of embankments or tankage built on such strata is sometimes due primarily to the horizontal spread of excess pore pressure along these bands or partings (Terzaghi, 1943b). Vertical drains relieve these excess pore pressures and thus avoid the occurrence of instability.

While the potential advantages of using vertical drains are considerable, these can only be realized if the drains perform as designed.

THEORETICAL CONSIDERATIONS

The problem of designing a vertical drain scheme is to determine the drain spacing which will give the required degree of consolidation in a specified time for any given drain type and size in the ground conditions that prevail. In practice, drainage will take place in both the vertical and horizontal planes and therefore any design method should take this into account if it is to model the real situation properly.

The evaluation of the vertical consolidation due to vertical drainage only is based on the one-dimensional consolidation theory set out by Terzaghi (1943a); the average degree of consolidation in a homogeneous layer is expressed in the form

$$U_{\mathrm{v}} = 1 - \frac{8}{2\pi} \sum_{N=0}^{N=\infty} \frac{1}{(2N+1)^2} \times \exp\left[-(2N+1)^2 \pi^2 \frac{c_{\mathrm{v}} t}{4H}\right] \qquad (1)$$

The assessment of the average degree of consolida-

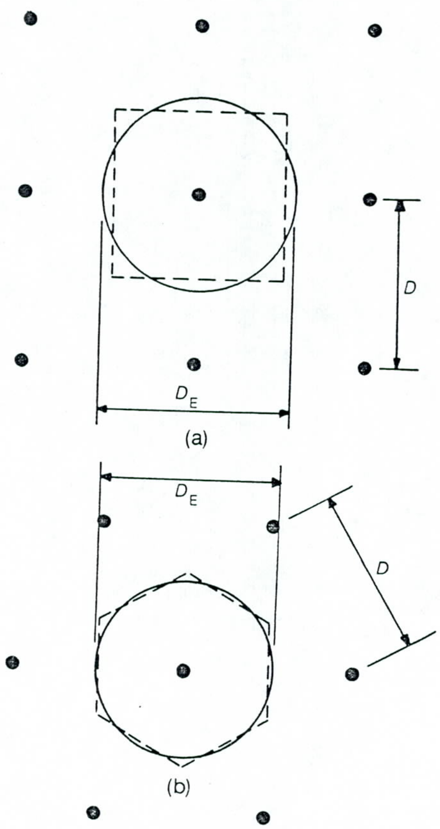

Fig. 1. Equivalent diameters for drains installed in (a) square and (b) triangular grid patterns

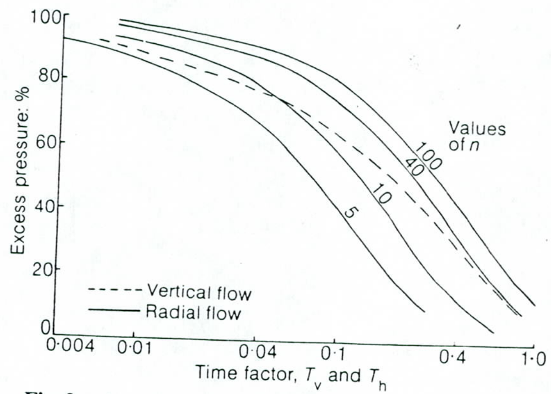

Fig. 2. Average consolidation rates; after Barron (1948): for vertical flow in a clay stratum of thickness 2H drained on both upper and lower surfaces; and for radial flow to axial drains in clay cylinders having various values of n; percentage consolidation (U_v and U_r) = 100 − excess pressure

tion due to horizontal drainage to the drain is more difficult. From a practical viewpoint the drains must be installed in some rectangular or triangular grid pattern and therefore the problem is not axisymmetric. No analytical solutions exist for these real situations and it is usual to approximate the problem to that of a cylindrical drain placed at the centre of a cylinder of consolidating soil. An analytical solution for this truly axisymmetric case was given by Rendulic (1935) in the form:

$$U_r = 1 - \exp[-8c_h t / D_E{}^2 \alpha] \qquad (2)$$

where

$$\alpha = \left[\frac{n^2}{n^2-1}\ln(n)\right] - \left[\frac{3n^2-1}{4n^2}\right]$$

$$n = D_E/D_W$$

The diameter of the equivalent cylinder of soil surrounding each drain D_E is calculated on the basis of equivalent cross-sectional areas (Fig. 1), i.e. for drains on a square grid pattern with a drain spacing of D

$$\pi D_E{}^2/4 = D^2$$

$$D_E \simeq 1{\cdot}13D$$

For a triangular grid this becomes

$$D_E{}^2 = \frac{2\sqrt{3}}{\pi}D^2$$

$$D_E \simeq 1{\cdot}05D$$

The combination of the two solutions, given by equations (1) and (2) respectively, to give the total average degree of consolidation was presented by Carillo (1942) in the form

$$1 - U_{rv} = (1 - U_r)(1 - U_v) \qquad (3)$$

While a combination of equations (1)–(3) enables the total average consolidation to be calculated, Kjellman (1948) proposed that the spacing of vertical drains be fixed by considering only radial drainage (equation (2)); the Authors' impression is that this method is still used at present in Western Europe, but not in the United Kingdom. The calculations tend to be conservative although, unless very short drains are employed, the error in ignoring the natural vertical consolidation probably rarely exceeds 10%.

The complete problem has been presented by Barron (1948) who gave the results in the form of curves for purely radial flow and purely vertical flow against the time factors T_r and T_v respectively (Fig. 2). The use of the curves is straightforward and allows an estimate to be made of drain spacing necessary to achieve a given percentage average consolidation in a specified time for any particular drain diameter and given soil conditions. However, when considered in detail, it is apparent that there are a number of assumptions implicit in the basic Barron analysis which are not valid in practice.

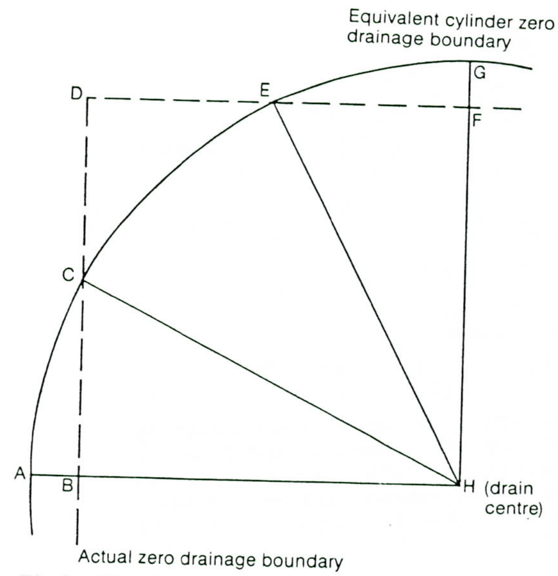

Fig. 3. Plan view of a quadrant of the equivalent cylinder for one drain from a square grid pattern

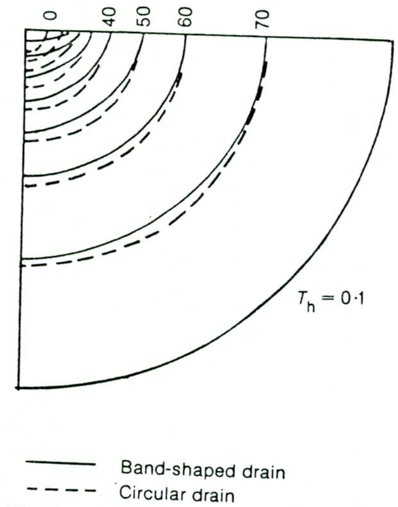

Fig. 4. Comparison of consolidation effects (remaining excess pore pressure Δu in % of initial excess pore pressure Δu_0) caused by 100 mm × 4 mm band-shaped drain and a circular drain with equivalent circumference (d = 66 mm); D_E = 1 m (after Hansbo, 1979)

With regard to the horizontal drainage, the actual shape of the volume of soil drained by each drain is replaced by an equivalent cylinder of soil. It follows that (Fig. 3) the average consolidation for the cross-sectional areas enclosed by points BCH and EGH will be underestimated by considering areas ACH and EFH.

Conversely the average consolidation for the area CDEH will be overestimated by considering the area CEH. Since the rate of consolidation is proportional to the square of the drainage path length, the overall effect is that the average consolidation calculated by means of an equivalent cylinder overestimates the true consolidation. The error is likely to be small, probably less than 5%, but it is on the unsafe side. For a triangular grid, the error is smaller as this grid pattern is considerably nearer to the equivalent cylinder model.

A further possible source of error in the analysis associated with shape is related to the drain itself. Where sand drains, or their modern derivatives such as sand wicks or plastic tube drains, are employed, the cross-sectional shape of the drain is the same as that used in the analytical model and there should be no shape error. This is not the case for band drains where the flow pattern around the drain is considerably different from the cylindrical case, as indicated in Fig. 4. The problem is usually dealt with by modelling the drain as an equivalent cylindrical drain. Kjellman (1948) suggested that the equivalent diameter could be estimated from a consideration of the drain surface area. Thus for a typical band drain 100 mm wide by 4 mm thick the equivalent diameter d would be given by

$$d = 2(100+4)/\pi$$

i.e. approximately 66 mm.

Such an estimate takes no account of the throttle which must occur close to the drain corner where the flow lines converge rapidly. More recently (Van den Elzen & Atkinson, 1980) the Delft Laboratory of Soil Mechanics in the Netherlands has proposed that a factor of $\pi/4$ should be applied to this estimate thereby reducing the equivalent diameter to about 52 mm. A back-calculated equivalent diameter from field installations of 100 mm band drains (Humpheson & Davies, 1981) was about 50 mm. Even with the large variation in the estimation of the drain equivalent diameter, it may easily be shown that for typical band drain installations this would generally only change the estimated degree of consolidation by less than 2%.

Barron (1948) also considered the two extreme cases of free strain and equal vertical strain. He showed that at the degrees of consolidation normally of interest, i.e. greater than 50%, the difference between the analyses was negligible.

Whilst the probable errors directly associated with the analytical technique may appear small (probably less than 5%), this is the error expressed in terms of the degree of consolidation. Since the consolidation curve becomes asymptotic to the time axis at full consolidation, a 5% error in estimating the degree of consolidation is equivalent to a somewhat larger error of about 17% in the estimate of the time required to achieve, say, 90%

consolidation which is often a stated requirement for particular installations. Although seemingly larger, when expressed in terms of an error in time it is shown subsequently that this may well be considered as insignificant when compared with the errors arising from the effects of smear, well resistance and incorrect assessment of the soil drainage parameters. It is concluded that provided smear, well resistance and the soil drainage parameters are correctly assessed and the drain geometry matches that assumed by the theories, both the Barron and Kjellman methods of assessing appropriate vertical drain spacings will give reasonable results.

SMEAR

The effects of various thicknesses of smear zones were considered by Barron (1948) who derived analytical expressions to take this factor into account and the solutions appear in graphical form in a report by Moran *et al.* (1958). Barron assumed for the analysis that the ratio of permeabilities in the undisturbed and smeared zones was 10 and showed that if the thickness of the smeared zone was 1/6 of the drain radius, the time to achieve a particular degree of consolidation would be increased by about 20%. If the thickness of the smeared zone was increased to twice the drain radius, then the effect would be approximately to double the consolidation time. The problem was also considered by Casagrande & Poulos (1969) who concluded that the permeability of the smeared zone could be considerably less than 1/10 of that of the undisturbed soil, and possibly as little as 1/1000. They noted that the area of the severely disturbed or remoulded ground around a driven closed-ended casing would be approximately equal to the cross-sectional area of the casing.

In the case of an 18 in. (457 mm) dia. driven casing, a typical size for many sand drain installations, the thickness of the remoulded zone would be about 90 mm. The effect of such a thick smeared zone of greatly reduced permeability would be to negate any potential beneficial effects of such a drain. Casagrande & Poulos (1969) therefore concluded that drains installed by displacement methods were generally uneconomic and cited a number of installations where such drains had not only failed to produce any beneficial effects, but probably caused additional problems due to the disturbance during installation. Despite this, the majority of the vertical drains installed in recent years, which have been band drains, have been installed by displacement methods and on many of the sites the drains appear to have been successful in accelerating the natural rate of consolidation. It is important to examine the reasons for this apparent anomaly and to this end it is necessary to consider the area of the soil/drain interface after installation

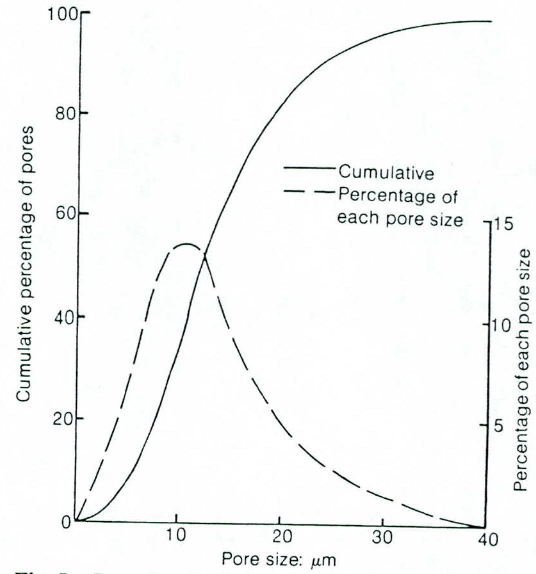

Fig. 5. Pore size distribution for the filter fabric used in Colbond CX1000 drains

of the drain.

The modern band drains are generally 100 mm in width, about 4 mm thick and are installed using a lance about 140 mm wide by 30–40 mm thick. Using the same approach as Casagrande & Poulos, a smeared zone about 10 mm thick would be expected along the wall of the hole made by the lance. After a period of time when the hole had closed the smear zone would lie against the filter layer of the drain. Research by McGown & Sweetland (1973) and Marks (1975) showed that a fabric filter initially allows the finer soil particles to pass through the filter, i.e. piping occurs. However, as these smaller particles pass through the fabric, a bridging network of the larger soil particles builds up adjacent to the drain, thus forming a natural graded filter within the soil, the thickness of which was found to be several millimetres. The effect of the piping is to remove the clay particles from the smeared zone immediately adjacent to the drain. In the case of the band drains installed by the typical size of lance used at present, the thickness of the smear zone is similar to the thickness of the natural soil filter created by piping. Consequently it appears probable that the majority of the smear caused by the installation process is removed by formation of the naturally formed graded soil filter. Obviously this can only be true if the drain fabric has the correct pore sizes to produce the natural graded soil filter. Too large a pore size will permit continuous piping of the soil leading eventually to a significant loss of ground, and clogging of the drain if the upward velocity of the water is less than the

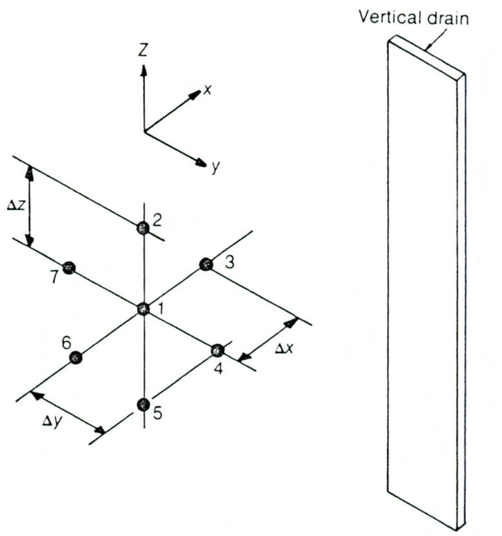

Fig. 6. Three-dimensional arrangement of nodes in relation to a drain used in the finite difference computer model

settling velocity of the heavier particles. Too small a pore size will produce a finer soil filter than desired. This will not only reduce the thickness of the soil filter and therefore restrict the amount of smear removed, but will also produce a soil filter of low permeability, significantly affecting the efficiency of the drain installation. Field and laboratory experience by fabric manufacturers has shown that the optimum filter for drains used in clayey soils has an average pore size of about 10–20 μm. An example of the appropriate pore size distribution for the drain filter material is given as Fig. 5.

DRAIN RESISTANCE

The effect of the internal resistance of a vertical drain to the flow of the collected water has been considered by a number of other authors, notably Barron (1948), Richart (1957) and Bhide (1979). In general, consideration has been given to the case where the drain spacing is comparable to the half-depth of the soil layer. For this case the effect of the well resistance is not excessive, increasing the time required to obtain a particular degree of consolidation by about 25% by comparison with the ideal case of an infinitely permeable drain. However, in the majority of the cases examined by Casagrande & Poulos (1969), the half-depth of the drained stratum was considerably in excess of the drain spacing and the internal resistance of the drains may well have made a significant contribution to the lack of acceleration of the consolidation process. The recent trend towards using band drains, of considerably smaller cross-section than sand drains, and with drain lengths of up to 50 m in some cases, makes it imperative to consider the effect of the drain internal resistance.

In order to study the effect of the drain resistance in particular, and to aid in design of band drain installations in general, the Authors have developed a three-dimensional finite difference computer program to model the flow of water in a single drain and the surrounding soil. The program is based on the solution of the partial differential consolidation of the following form

$$k_x \frac{\partial^2 u}{\partial x^2} + k_y \frac{\partial^2 u}{\partial y^2} + k_z \frac{\partial^2 u}{\partial z^2} = \frac{k_z}{c_v} \frac{\partial u}{\partial t}$$

In the program a large number of discrete points or nodes are examined such that the excess pore water pressure at any particular node (node 1 in Fig. 6) at some time $t + \Delta t$ is determined from the excess pore water pressures at the node considered and the six adjacent nodes (nodes 2–7 in Fig. 6) at the time, t. The process is continued for all the nodes in the matrix until a three-dimensional picture of the excess pore water pressures is built up. The program calculates the average excess pore water pressure at any given time and compares this with the initial conditions to obtain the degree of consolidation. The solution is output in the form of time, degree of consolidation and the rate of discharge from the drain. To obtain flexibility the program has been designed so that the drain size, shape and spacing are defined in input parameters. In addition, more than one soil layer may be considered, each with differing horizontal and vertical drainage parameters and, if considered necessary, these drainage parameters can be varied as the consolidation proceeds. The principal method used to check the correct operation of the program has been to reproduce the theoretical results obtained previously by Barron (1948) for a range of drain sizes and spacings and soil conditions. The results are comparable with those obtained theoretically, bearing in mind the errors which are implicit in the theoretical studies.

Because the nodes are located both in the drain and in the surrounding soil, any restriction to water flow within the drain is automatically taken into account. In order to assess the effect of internal drain resistance it is only necessary to adjust the drain permeability value and rerun the program. An example of the results illustrating the significant throttling effect which can occur is given in Fig. 7 where it may be seen that for 30 m long drains placed at, for example, 1·5 m centres, the time to achieve 70% consolidation using a relatively high internal resistance drain, such as the original Colbond KF650, is about ten times as long as that for the recently developed Colbond CX1000 which has an open channel form of central core. Even with an extremely low internal resistance drain such as

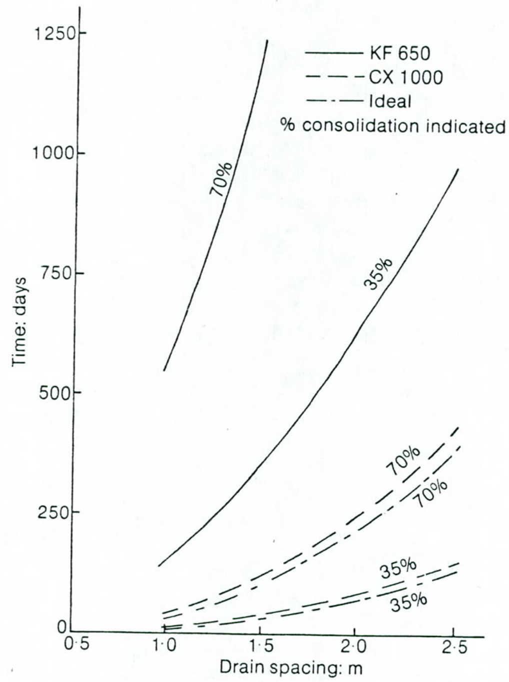

Fig. 7. Comparison of the consolidation times for Colbond KF650, Colbond CX1000 and ideal drains for drain spacings between 1·0 m and 2·5 m; degrees of consolidation = 35% and 70%; drain length = 30 m

the Colbond CX1000, there is still a measurable increase in the consolidation time by comparison with the ideal infinitely permeable drain.

In considering the resistance to water flow within the drain, an important consideration is the potential for clogging. If the drain functions correctly in the very early stages of the consolidation, the excess pore water will contain a small proportion of soil particles which may collect within the drain and cause it to clog. Laboratory tests by the Dutch company Enka (1980) have shown that some drains are particularly susceptible to this. In the main these are drains which have no open channels in the central core. Although the majority of the band drains available at present do have a channel structure within the drain core, there are differences between them which may be significant. In most cases the channels are formed by a continuous extrusion of a plastic sheet so that there is no interconnection between the individual channels. Consequently, if a blockage does occur in an individual channel, only that part of the channel which is on the discharge side of the blockage remains useful. This potential problem has been overcome in the Colbond CX1000 drain by forming a channel-shaped core from a mesh of polyester monofilaments fused together at the points of intersection which allows interconnection between the channels.

DRAINAGE BLANKET

In any vertical drain installation it is normal to provide a granular drainage blanket over the complete area where the drains are to be installed. While this provides a suitable working platform for heavy plant, which may otherwise be unable to gain access to the area, its primary function is to provide a free-draining outlet for the water discharged from the drains. In certain cases, where a large volume of soil is being drained, considerable quantities of water can be discharged into the drainage blanket, particularly in the early stages of consolidation. Under these conditions the drainage blanket itself can be a throttle on the consolidation process, although an implicit assumption in all the design methods considered in this Paper is that the drains discharge at about atmospheric pressure. A rudimentary assessment of the problem can be made using the discharge rates output from the computer program referred to in the previous section. When the drain spacing has been finalized, the total quantities of water being discharged into the drainage blanket can be calculated at any degree of consolidation. An assessment can then be made of the adequacy or otherwise of the drainage blanket and its design can be adjusted accordingly.

A minor modification to the program allows the vertical water flow rates between nodes within the drain to be output. This facility assists in assessing whether the larger soil particles passing into the drain during the initial piping are likely to settle out within the drain and cause clogging.

SOIL DRAINAGE PARAMETERS

Although it is important to consider such aspects as drain resistance, smear etc., and their effect on a vertical drain design, the accuracy of any design method must obviously be limited by the accuracy with which the soil drainage parameters can be assessed. It is self-evident that the correct methods of site investigation carried out with strict supervision will enable a more appropriate design to be undertaken. In the Authors' experience there are many cases where drain designs have been called for with no appropriate site investigations. On one site in the Far East the only data which were available were an inadequate visual description of the soils to be drained which gave no information on the soil fabric, together with undrained shear strength data. Furthermore, there were no permeability or consolidation test data. A less extreme case concerns a site within the United Kingdom where there were apparently quite good data, including laboratory consolidation tests and in situ permeability measurements. However, when examined closely it was apparent that the in situ tests gave a permeability three to four orders greater than that obtained from laboratory oedometer tests. Part of the discrepancy was considered to be attributable to the normally accepted differences between field and laboratory measurements, but there was evidence that the in situ permeability tests had been carried out in such a way that hydraulic cracking could not be ruled out and thus the permeability data were considered questionable. It is imperative to review all data critically to reveal such anomalies.

The number of soil parameters required for the design depends upon the design method chosen. That proposed by Kjellman (1948) is the simplest, requiring only a knowledge of c_h, but no account is taken of the drain resistance or the vertical drainage in the soil. The analyses proposed by Barron (1948) vary in their requirements. When the drain resistance is ignored, both horizontal and vertical drainage in the soil are considered and c_h and c_v are required. When the drain resistance is to be taken into account the solution was developed assuming no vertical flow in the soil and the required soil parameters are c_h and k_h. The computer solution used by the Authors allows for the drain resistance and both vertical and horizontal drainage in the soil. This method of analysis requires any three of the four parameters c_v, c_h, k_v and k_h to be defined.

The horizontal and vertical coefficients of consolidation can both be measured directly in the laboratory by carefully controlled testing of selected samples. The testing can be carried out either in 76 mm dia. oedometers or in the larger Rowe cell type of oedometer described by Rowe & Barden (1966); in most cases a more representative value will be obtained from the Rowe cell tests owing to the larger volume of soil tested. However, general experience has shown that laboratory measurements can seriously underestimate the field values of the coefficients of consolidation, but give a reasonable estimate of the coefficient of volume decrease m_v. A better method of determining c_h and c_v is the indirect approach of combining laboratory m_v values with field permeability measurements. Implicit in the latter approach is a knowledge of the ratio of the horizontal to vertical permeability. Individual laboratory measurements of vertical or horizontal permeability are not likely to give representative field values, but the ratio of the two can be satisfactorily assessed if the laboratory results are considered in conjunction with a visual assessment of high quality soil samples where particular attention has been paid to a description of the soil fabric. Since the majority of the soils will be soft, perhaps ranging up to firm, and may be sensitive to disturbance, they should be obtained by Delft continuous samplers or piston samplers.

With regard to the field permeability determinations, there seems little doubt that constant head tests are likely to yield the most representative values. The tests must be carefully controlled to avoid hydraulic cracking of the soil due to the use of too high a pressure head. Furthermore, the tests should not be carried out until the natural groundwater level has been established in the piezometers. The method of interpretation will depend on the ratio of horizontal to vertical permeability, the determination of which has been referred to above. A full discussion of in situ permeability testing is not appropriate here (refer to Barden & Perry, 1965 and Wilkinson, 1968).

While the extent of any site investigation must be related to the size and cost of the construction, an investigation for a vertical drain installation should include high quality Delft or piston sampling, field permeability testing and laboratory consolidation and permeability testing. It should not exclude index property or strength measurements as these are essential elements in assessing the overall character of the soils to be drained. Consideration should also be given to pore pressure soundings described by Torstensson (1975) as an additional means of assessing the soil structure and the in situ values of the coefficients of consolidations, although at the present time experience of their use is limited in the United Kingdom. The site investigation should be planned, controlled and interpreted by experienced engineers or engineering geologists who are familiar with the requirements of the design of vertical drain schemes.

ASSESSMENT OF THE DRAIN SPACING

In the section on theoretical considerations it was noted that, provided smear, drain resistance and the soil drainage parameters could be correctly assessed and the system geometry matches that assumed by the Barron and Kjellman methods, then these methods would provide a reasonable assessment of the necessary drain spacing. In practice the situation is often far from that ideal. A case dealt with recently by the Authors provides an example of this and illustrates the use of the computer program referred to earlier in the Paper.

The site in question is situated in the Far East and generally consists of an upper and lower soft clay horizon approximately 4 m and 8 m thick respectively, overlying a stiff clay. The assessed soil parameters for the two soft clay horizons were: for the upper layer $c_v = 1{\cdot}1\ \mathrm{m^2/year}$, $k_v = 5{\cdot}8 \times 10^{-10}$ m/s and a ratio $c_h/c_v = 1{\cdot}0$; and for the lower layer $c_v = 7{\cdot}0\ \mathrm{m^2/year}$, $k_v = 3{\cdot}0 \times 10^{-9}$ m/s, again with a ratio $c_h/c_v = 1{\cdot}0$. Based on these parameters and the strata thicknesses only a very small part of the total consolidation would occur within a specified period of 2 months as a result of natural vertical drainage. Consequently vertical drains were proposed. The work described subsequently formed part of a preliminary assessment aimed at determining the necessary drain spacings for a number of combinations of drain size, type and length that would meet the Consulting Engineer's consolidation requirements. The overall requirement was that approximately 60% dissipation of the excess pore pressures due to the applied load should be achieved in 2 months. In addition it was considered that there could be a stability problem towards the edge of the proposed fill area. An additional constraint which was imposed was that the excess pore pressure in the specified time period at the base of the upper layer would also approach 60%; this depth was most critical with regard to potential slip circles.

The effect of the greater permeability of the lower layer meant that a drain spacing capable of giving an overall dissipation of 60% gave a considerably smaller dissipation at the base of the upper layer. To increase the dissipation in the upper layer relative to the lower layer, it was proposed to install vertical drains to the base of the lower layer (12 m long drains), with shorter drains (4 m long), interspersed between the deep drains. Consequently the problem was complex with two distinctly different soil layers and two lengths of vertical drains.

A problem of this type could not be adequately assessed by means of a Barron or Kjellman method and the three-dimensional computer program was employed. However, to model the combination of drain lengths correctly requires at least four drains to be considered simultaneously with complex boundary conditions. The computing capacity necessary for such a problem was not available and an alternative approach was adopted. Two models were set up; one for the short drain and one for the long drain. The short drain model consisted of a 4 m long drain within a 4 m thick layer of soil using the given soil drainage parameters. The model for the long drains considered a single deep drain at twice the short drain spacing, but with the given soil parameters for the upper soil layer adjusted to account for the effect of the adjacent shorter drains. The adjustment was carried out on the basis that equal volumes of the upper soil horizon were drained by short and long drains and the water flows to the drains were equal. The two models could then be considered consistent with each other provided both gave essentially the same degree of consolidation at the given time.

The computer models were run for a variety of types and sizes of drain at different drain spacings. An extract from the results is presented as Table 1. Quite obviously one combination of long and short drains considered is inconsistent in that the models suggest that a considerably greater degree of consolidation is achieved in the upper soil layer by the short drains. In practice this would not occur and means that in fact the short drains would drain a greater, and the deep drains a lesser, volume of ground than has been modelled. The other two combinations of different length drains are more consistent and suggest that either of these would produce the required performance from the vertical drain system. The effect of internal drain resistance is also exhibited, the predicted consolidation for Colbond CX1000 deep drains at 2·0 m spacing being considerably greater than that obtained with a Colbond KF650 drain at the same spacing.

While the computer modelling is less than ideal, for complex problems it provides a suitable alternative method of assessing drain spacings to the use of Barron and Kjellman methods with associated additional simplifying assumptions.

CONCLUSIONS

Vertical drains can be successful in accelerating the rate of consolidation of soft fine grained soils. This can lead to reductions in construction and maintenance costs which are considerably greater than the costs of investigating, designing and installing a vertical drain scheme. The large number of cases where vertical drain schemes have not been successful shows that a full understanding of how the drains operate is essential to obtain an economic design.

A consideration of the theories available to determine the required drain spacing shows that the errors which are implicit in the theories by virtue of invalid assumptions are small by comparison with errors arising from other sources.

The effects of smear and drain internal resistance have been considered. The effects of smear caused by installing the drains using displacement methods can be largely overcome by the correct choice of the drain filter fabric and the size of the installation lance. Internal resistance of the drain can have a large effect on the consolidation process, in certain circumstances increasing the time to achieve a particular degree of consolidation by about one order compared with that theoretically obtainable with an infinitely permeable drain.

The three-dimensional computer program developed by the Authors to aid in the drain design models the true shape of the drain and the drained volume of soil. It is also capable of considering more than one soil layer and simultaneously considering horizontal and vertical drainage within the soil and the internal resistance of the drain. The output from the program also includes water discharge rates from the drain enabling an assessment to be made of the adequacy of the design of the drainage blanket.

Consideration has been given to the site in-

Table 1. Extract from results of computer models; specified soil parameters: at 0–4 m depth, $c_v = 1{\cdot}10\ \mathrm{m^2/year}$, $m_v = 1{\cdot}70\ \mathrm{m^2/MN}$, $k_v = 5{\cdot}80 \times 10^{-10}\ \mathrm{m/s}$, $c_h/c_v = 1{\cdot}00$; at 4–12 m depth, $c_v = 7{\cdot}00\ \mathrm{m^2/year}$, $m_v = 1{\cdot}40\ \mathrm{m^2/MN}$, $k_v = 3{\cdot}03 \times 10^{-9}\ \mathrm{m/s}$, $c_h/c_v = 1{\cdot}00$

Colbond deep drains (assumed length = 12 m)	Drain width: mm	Drain spacing: m	Overall consolidation at 2 months: %	Time for 60% overall consolidation: days	Consolidation at 2 months and depth of 4 m: %	Colbond short drain (assumed length = 4 m)	Drain width: mm	Drain spacing: m	Consolidation at 4 m depth: %
KF650	300	2·00	33	146	29				
KF650	300	1·70	44	97	44	KF650	300	0·85	77*
KF650	300	1·50	53	74	54				
CX1000	300	2·05	56	69	50				
CX1000	300	2·00	58	65	52				
CX1000	300	1·95	61	60	55	KF650	300	0·975	63
CX1000	150	2·40	32	154	26				
CX1000	150	2·20	38	123	31				
CX1000	150	2·00	46	92	40				
CX1000	150	1·80	55	71	48				
CX1000	150	1·70	60	61	54	KF650	150	0·85	58

* Model inconsistency suggesting computed consolidation for deep drains considerably underestimated.

vestigations which should be carried out to enable an appropriate drain design to be achieved. A combination of laboratory and field testing techniques are recommended.

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Consolidation by vertical drains

S. HANSBO,* M. JAMIOLKOWSKI† and L. KOK‡

A theory is described which can be used in designing a vertical drainage system. This theory incorporates important parameters such as vertical discharge capacity, remoulding effects during installation and filter resistance. The importance of these parameters is widely recognized, but very seldom taken into account in the design, by engineers. Six well-documented case records, related to different drain types are briefly presented. Experience gained from these case records shows that in many cases, even if the overall performance appears successful, there are aspects of soil/drain behaviour that cannot be explained on the basis of existing theories.

L'article décrit une théorie que l'on peut appliquer à la conception d'un système de drainage vertical. Cette théorie renferme d'importants paramètres tels que la capacité de décharge verticale, les effets du remaniement pendant l'installation et la résistance de filtration. L'importance de ces paramètres est reconnue par bien des ingènieurs, mais ils en tiennent rarement compte au stade de la conception. L'article donne une description sommaire de six cas bien documentés dans lesquels divers types de drains sont utilisés. L'expérience acquise grâce à cette étude de cas montre que, très souvent, même si l'efficacité globale semble satisfaisante, il existe certains aspects du comportement sol/drains que l'on ne peut expliquer sur la base des théories existantes.

NOTATION

A_w	cross-sectional area of the drain well
a'	effective cohesion
b	width of band-shape drain
C_c	compression index
C_h	coefficient of consolidation in the horizontal direction
c_u	unconfined compressive strength
C_v	coefficient of consolidation in the vertical direction
D	diameter of soil cylinder
d	diameter of drain
d_e	equivalent diameter
d_s	diameter of zone of smear
e_o	original void ratio
E_{u50}	undrained modules of deformation at 50% failure stress
I_p	plasticity index
k_c	permeability of undisturbed soil
k_c'	permeability of soil in zone of smear
k_w	axial permeability of drain
l	half length of drain
n	D/d
q_w	discharge capacity of the drain well
r_u	pore pressure coefficient
s	d_s/d
t	thickness of band-shape drain
T_h	time factor in radial consolidation
u	excess porewater pressure
U_h	average degree of consolidation
w	natural moisture content
w_L	liquid limit
w_P	plastic limit
z	depth of soil
γ	bulk density
Δu	change in porewater pressure
σ_{1c}	principle consolidation stress
σ_{vc}'	vertical preconsolidation pressure
σ_{vo}'	original effective stress

THEORETICAL CONSIDERATIONS

The design of a vertical sand drain system is generally based on the classical theoretical solution developed by Barron (1948) in which the drains are assumed to be functioning as ideal wells, i.e. their permeability is considered infinitely high as compared with that of the soil in which the drains are placed. This assumption is justified when the drain sand fulfils the requirements of an ideal filter material. However, in practice it is doubtful whether such an ideal condition can be achieved. If the permeability of the sand is in the order of magnitude of 500–1000 m/year, the effect of well resistance cannot be ignored.

In the case of prefabricated drains the effect of well resistance must be taken into account, particularly when the drains are long. A simple theoretical solution to this problem is needed. Solutions which take into account the effect of well resistance have existed for some time (Barron, 1948; Yoshikuni & Nakanodo, 1974) but although these solutions are based on the simplified assumption of equal strain they are still very complicated to use in practice. For example, Barron's solution in respect

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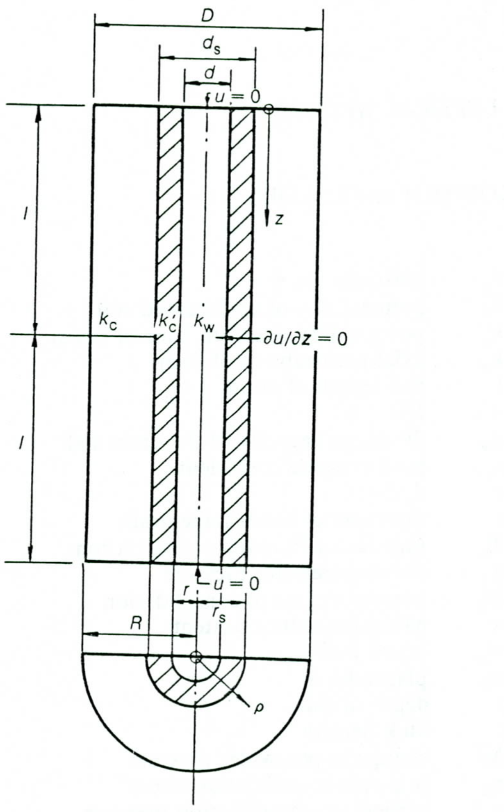

Fig. 1. Schematic picture of soil cylinder dewatered by vertical drain

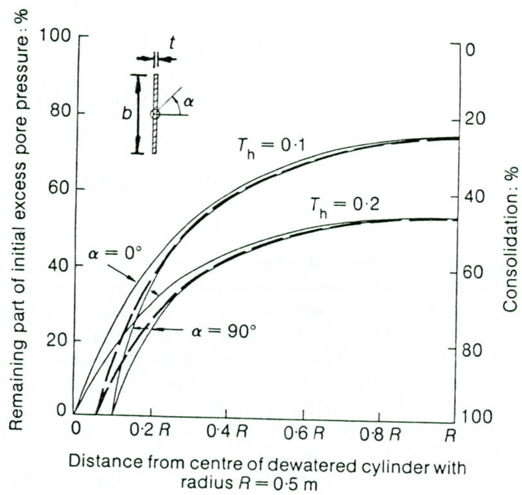

Fig. 2 Comparison of consolidation effects caused by a 100 mm × 4 mm band-shaped drain and a circular drain with equivalent circumference; $d = 66$ mm, $D = 1$ m, time factor $T_h = c_h t/D^2$

of well resistance is given in implicit form and the diagrams presented are not correct (Fig. 10 of Barron, 1948). Yoshikuni & Nakanodo's solution includes Bessel functions of the zeroth and first order and thus requires the use of mathematical tables or advanced computers. However, a simple solution to the problem of smear and well resistance was presented by Hansbo (1979), giving results almost identical with those presented by Barron and Yoshikuni & Nakanodo.

Thus for a water-saturated soil the average degree of consolidation ($\bar{U}_h$) at a depth z due to the effect of radial drainage only (Fig. 1) can be expressed as

$$\bar{U}_h = 1 - \exp(-8T_h/\mu_s) \qquad (1)$$

where

$T_h = c_h t/D^2$ is the time factor in radial consolidation

$\mu_s = \ln(n/s) + (k_c/k_c') \ln(s) - 3/4 + \pi z(2l - z) k_c/q_w$

$n = D/d$

$s = d_s/d$

$q_w = k_w A_w = k_w \pi d^2/4$ is the discharge capacity of the drain well.

The derivation of this formula is presented by Hansbo, 1981). Equation (1) is deduced for a drain well with circular cross-section. A band-shaped drain has to be converted into a circular cylindrical drain producing the same consolidation effect as the band-shaped drain. The equivalent diameter d_e of this drain (cf Fig. 2) is

$$d_e = \frac{2(b+t)}{\pi}$$

where b is the width of and t is the thickness of the band-shaped drain (Hansbo, 1979).

This discharge capacity of the prefabricated band-shaped drains will be a function of the effective lateral earth pressure against the drain sleeve. In the majority of the cases the filter will be partly squeezed into the channel system of the core by the pressure of the surrounding soil and this will reduce the channel volume and consequently the discharge capacity. The discharge capacity may also decrease with time. For example, fines in the pore water may not be retained by the filter and may cause a gradual decrease of discharge capacity or even clogging. The filter permeability should not be higher than required with respect to the discharge capacity. The filter permeability of the existing prefabricated drains (which according to the results of laboratory tests have short-term discharge capacities of maximum 10–25 m^3/year) need not be higher than 0·01–0·05 m/year (Hansbo, 1981).

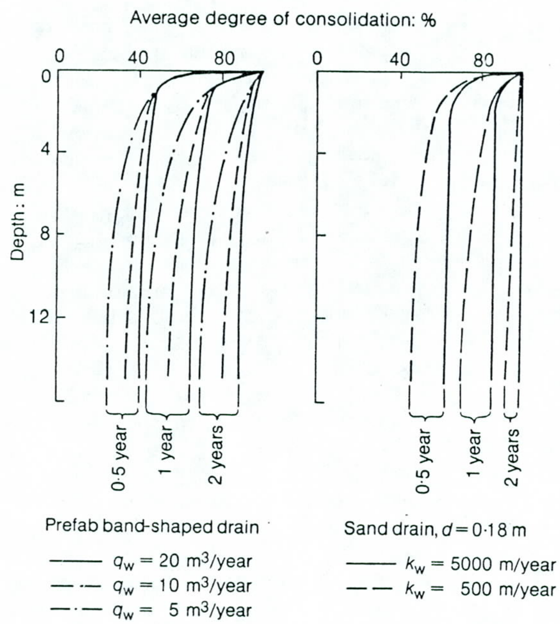

Fig. 3 Influence of well resistance on average consolidation at different depths in clay with $c_v = 0{\cdot}15\,\mathrm{m^2/year}$, $c_h = 0{\cdot}5\,\mathrm{m^2/year}$ and $k_c = 0{\cdot}03\,\mathrm{m/year}$; drain spacing 0·9 m ($D = 0{\cdot}95$ m); discharge capacity $q_w = k_w A_w$

Drains with low filter permeability ought to be cut as close as possible to the groundwater level in pervious soil or in the drainage blanket. Otherwise the water level inside the filter, in the drain core, may cause excess back-pressure in the wells, thereby reducing their efficiency.

For a certain fixed value of s, the influence of smear will increase with increasing diameter of the drain. However, the s value may be greatly influenced by the installation method. Equation (1) offers the possibility of easily making a systematic study of the influence of smear and well resistance.

Take as an example two typical cases with 30 m long fully penetrating drains (draining at top and bottom). In one case prefabricated and band-shaped drains are used; these have an equivalent diameter of 0·062 m and discharge capacities of 5, 10 and 20 m^3/year. In the other case displacement-type sand drains are used; these are 0·18 m in diameter, with discharge capacities of 13 m^3/year (permeability of drain sand = 500 m/year = $1{\cdot}6 \times 10^{-4}$ m/s) and 130 m^3/year. In both cases the zone of smear is assumed to have a diameter $d_s = 2d$ and a permeability $k_c' = 1/3k_c$. The clay in which the drains are placed has a permeability $k_c = 0{\cdot}03$ m/year in the horizontal direction. The coefficients of consolidation are $c_h = 0{\cdot}5\,\mathrm{m^2/year}$ and $c_v = 0{\cdot}15\,\mathrm{m^2/year}$. The result of the analysis in respect of two different spacings is given in Table 1 and Fig. 3.

Whether the drains are open or closed at the bottom has an important influence on well resistance. Well resistance of penetrating drains 30 m in length would have the same negative effect as that of drains 15 m in length which are not penetrating. Moreover, the tip of the band-shaped drains is generally folded round an anchor rod during installation. Therefore even penetrating drains may have the bottom outlet in clay and thus be more or less closed unless they are driven to a certain depth in pervious soil.

A more detailed theoretical parametric study has been made by Hansbo (1981). However, the best and safest basis for a comparative study is to use data from a large number of full-scale tests in which the effect of different drain types can be directly compared. As yet the Authors have not enough data to provide a statistical basis for such a comparison.

COMPARATIVE CASE RECORDS

Despite the previously outlined theoretical developments vertical drain design is still subject to many uncertainties; these will be overcome when more experience is gained in respect of field performance related to various soils all over the world. The results of six geotechnically well-documented case records are presented here, with the aim of evaluating how factors such as soil parameters, drain types, spacing and installation procedures

Table 1. Average consolidation (%) at 15 m depth without and with regard to smear and well resistance; drain spacing = 1·5 m (D = 1·58 m); discharge capacity of prefabricated drain = 20 m^3/year and of sand drain = 13 m^3/year

Time of consolidation: years	Sand drains: No smear ideal wells	Sand drains: Smear ideal wells	Sand drains: Smear well resistance	Prefabricated drains: No smear ideal wells	Prefabricated drains: Smea[r]
0·5	42	25	17	27	
1	67	44	31	47	
2	89	68	52	72	5
4	99	90	77	92	81

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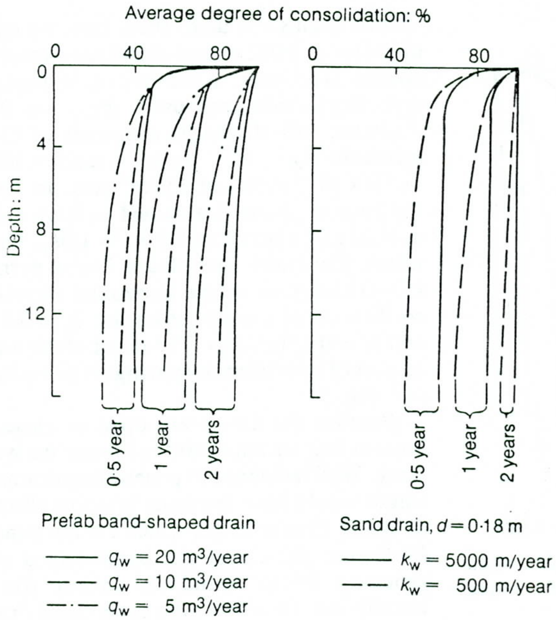

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	Sand drains			Prefabricated drains		
Time of consolidation: years	No smear ideal wells	Smear ideal wells	Smear well resistance	No smear ideal wells	Smear ideal wells	Smear well resistance
0·5	42	25	17	27	19	15
1	67	44	31	47	34	28
2	89	68	52	72	56	48
4	99	90	77	92	81	73

Table 2. Geological profile and geotechnical characteristics of soil in test areas I and V at Skå-Edeby; ground water level at 0·5 to 1 m depth; hydrostatic condition; studied layer = 2·5 to 7·5 m

Depth: m	Soil profile: Area V Geodrains		Soil profile: Area I Sand drains		γ: t/m³ V	γ: t/m³ I	w: % V	w: % I	w_L: % V	w_L: % I	w_P: % V	w_P: % I	*CF* I	†c_u/σ_{vo} V	†c_u/σ_{vo} I	OCR V	OCR I
	Post glacial	Dry crust	Post glacial	Dry crust													
1																	
	Post glacial	Green-grey slightly organic	Post glacial	Grey-green		1·45	95	100	106	132	47	—	28		2·2		
2					1·55	1·28	77	128	61	150	20	62		0·50	0·93	2·0	1·5
3	Post glacial	Grey coloured by iron sulphide	Post glacial	Grey coloured by iron sulphide	1·48	1·46	90	89	78	84	25	—	63	0·35	0·44	1·1	1·2
4	Glacial	Brown grey varved Bands of iron sulphide	Post glacial		1·48	1·53	92	82	76	72	27	25	64	0·34	0·32	1	1
5					1·58	1·62	64	68	58	63	22	—		0·34	0·30	1	1
6					1·62	1·43	63	103	50	85	22	28	75 70	0·32	0·32	1·2	1
7			Glacial	Grey varved Bands of iron sulphide	1·60	1·59	70	72	50	58	20	—		0·31	0·26	1·1	1·2
8					1·66	1·62	65	68	48	65	16	23		0·30	0·22	1·1	1
9					1·66	1·60	60	79	79	67	21	—	65	0·30	0·23		
10					1·61	1·68	66	67	—	53	—	22	60	0·33	0·21		

† c_u according to field vane test.

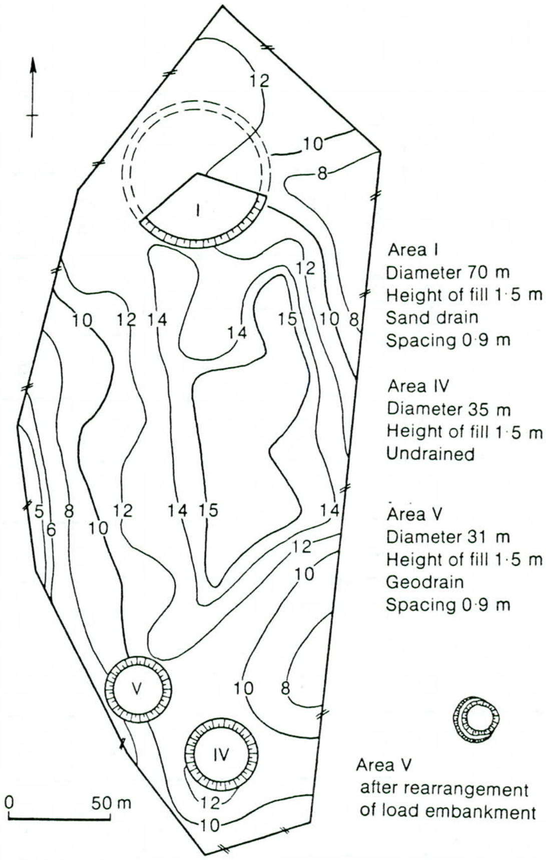


Fig. 4. Plan of test field at Skå-Edeby showing the location of test area I, IV and V and depth of clay layer

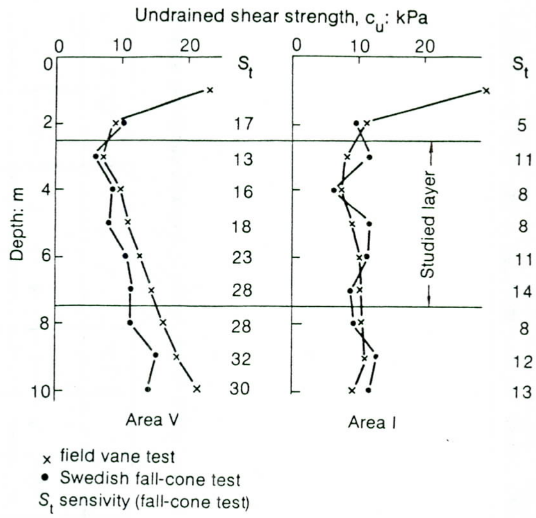


Fig. 5. Original undrained shear strength in test area I and V at Skå-Edeby

influence the performance of a vertical drainage system. Behaviour of drainage systems in the field which is apparently in disagreement with existing theory or commonly adopted design rules is also reported.

Skå-Edeby case record

At Skå-Edeby, located about 25 km west of Stockholm, a test field was constructed in 1957 consisting of four circular test areas, 35–70 m in diameter. In three of these areas displacement-type sand drains, 0·18 m in diameter, were installed in an equilateral triangular pattern with 0·9 m, 1·5 m and 2·2 m spacing. The fourth area was undrained.

The main object with the test field was to study the influence on the consolidation process of displacement-type sand drains with various spacing on the one hand and of load intensity on the other. The result of this study has been reported by Hansbo (1960) and by Holtz & Broms (1972). In 1972 a new circular test area, 31 m in diameter, was constructed for the purpose of studying the efficiency of the then recently developed Geodrain. These drains were installed with the aid of a sounding rod in an equilateral triangular pattern with 0·9 m spacing. Since the same spacing was used as in one of the old test areas, the consolidation effects of equally spaced sand drains and Geodrains could be compared with one another and with the consolidation achieved in the undrained area.

The geotechnical data of the soil in the Skå-Edeby test field are well documented (Hansbo, 1960; Holtz & Broms, 1972). In this Paper geotechnical data of the soil only in the two test areas provided with drains with 0·9 m spacing (Fig. 4) are discussed (Fig. 5 and Table 2).

Although the distance between the two areas is small there are marked differences in soil characteristics. In order to avoid the influence of variations in thickness of the dry crust and variations in depth, the layer 2·5–7·5 m deep was studied. For this layer the estimated average virgin compression ratio according to oedometer tests is about the same in the two areas, being about 0·35 (compression ratio (CR) = $C_c/(1+e_0)$). The estimated average coefficient of consolidation from oedometer tests is $c_v = 0{\cdot}015$–$0{\cdot}020\ \mathrm{m^2/year}$.

The initial loading was the same for both the studied test areas: 1·5 m of sand fill corresponding to a load of 27 kN/m^2. After about $3\frac{1}{2}$ years the load on Area V with Geodrains was increased by rearrangement of the original sand fill. In this way an additional load of 21 kN/m^2 on the area was obtained as shown in Fig. 4. This action was taken for the purpose of studying the long-term efficiency

† c_u according to field vane test.

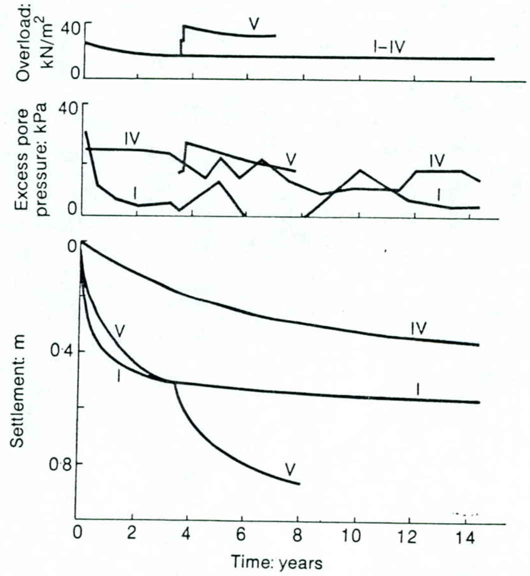


Fig. 6 Excess porewater pressure (at 5 m depth) and settlement (compression of clay layer between 2·5 and 7·5 m depth) vs time in the centre of test areas I, IV and V at Skå-Edeby

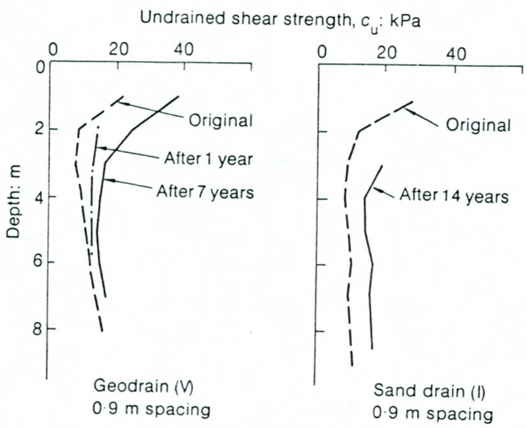


Fig. 7. Increase in undrained shear strength due to consolidation; test areas I and V at Skå-Edeby

of the Geodrain. Samples of Geodrain were taken in test pits to depths below the groundwater level before the additional load was applied.

The results of the settlement and pore pressure observations are given in Fig. 6. The rate of settlement in Area I with sand drains was greater than in Area V with Geodrain. The efficiency of Geodrain was no less with time although the filter paper, collected from the test pit after $3\frac{1}{2}$ years, was partly degraded. The back-calculated coefficient of consolidation, giving the best fit with the observed rate of settlement, has the same value, both after

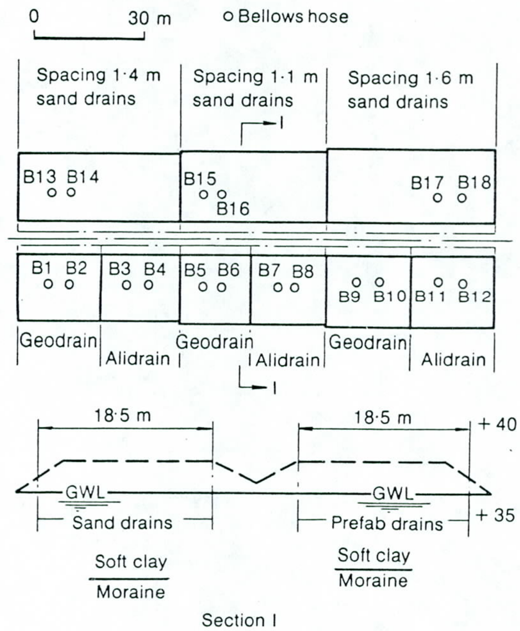


Fig. 8. Plan and cross-section of test field at Örebro; depth of clay layer varying between about 8 and 9·5 m

and before the additional loading. Comparing the rate of settlement for the two areas the effect of smear seems more pronounced for the sand drains than for the Geodrains. A good correspondence between theory and practice would be obtained assuming the values $c_h = 0{\cdot}5\ \mathrm{m^2/year}$, $s = 2$ and $k_c/k_c' = 3$ for the sand drains and $s = 2$ and $k_c/k_c' = 2$ for the Geodrains. The increase in undrained shear strength caused by consolidation in the drained areas is presented in Fig. 7. In Area V the increase is less pronounced at greater depth indicating the influence of well resistance.

The result of pore pressure measurements in Area I shows how difficult it may be to judge, from pore pressure readings, the degree of consolidation achieved. In Area V there still remains an excess pore pressure of about 15 kPa in the centre of the studied layer.

Örebro case record

In connection with the construction of a new motorway section between Örebro and Gothenburg a test field, 125 m by 45 m, was arranged just outside Örebro for the purpose of comparing the efficiency of the two types of prefabricated drains manufactured in Sweden—Geodrain and Alidrain—with that of displacement-type sand drains, 0·18 m in diameter. The test field was divided into three sections of equal size with drain spacings 1·4 m, 1·1 m and 1·6 m in an equilateral triangular pattern (Fig. 8).

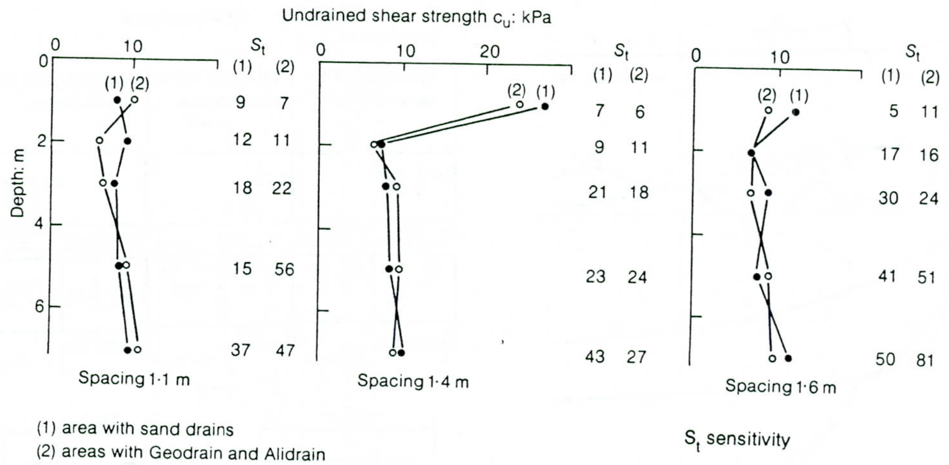


Fig. 9. Original undrained shear strength in the Örebro test field determined by the Swedish fall-cone test

The geotechnical data of the soil are presented in Fig. 9 and Table 3. As in the case of Skå-Edeby, the soil characteristics vary between the three studied sections. The consolidation characteristics according to oedometer tests are also subjected to variations which make the interpretation difficult.

The average virgin compression modulus increases with depth from an average of about CR = 0·4 just below the dry crust to an average of about CR = 0·65 in the lower part of the clay layer. In a few cases values of up to CR ≈ 1 have been measured, indicating some kind of structural collapse when the preconsolidation pressure is exceeded.

The coefficient of consolidation in the vertical direction c_v according to oedometer tests varies from a minimum of 0·06 m^2/year to a maximum of 1 m^2/year. An estimated average is $c_v = 0{\cdot}2$ m^2/year. The results of the settlement observations are presented in Fig. 10.

Excess pore pressure readings were also taken but the variation in remaining excess pore pressures (ranging, after nearly 2 years of consolidation, from 15 to 25 kPa in the middle of the clay layer) does not allow a reliable interpretation. The effectiveness of Geodrain and Alidrain is very nearly the same. The difference in effect between prefabricated drains and sand drains with a much larger diameter is also negligible. Only in the case of 0·9 m spacing can there be found, as expected, a clearly better consolidation effect for the sand drains than for the prefabricated drains. A good explanation of this result is, as in the case of Skå-Edeby, that the effect of smear is more pronounced in the case of the sand drains than in the case of the prefabricated drains. Taking into account the variation in compression

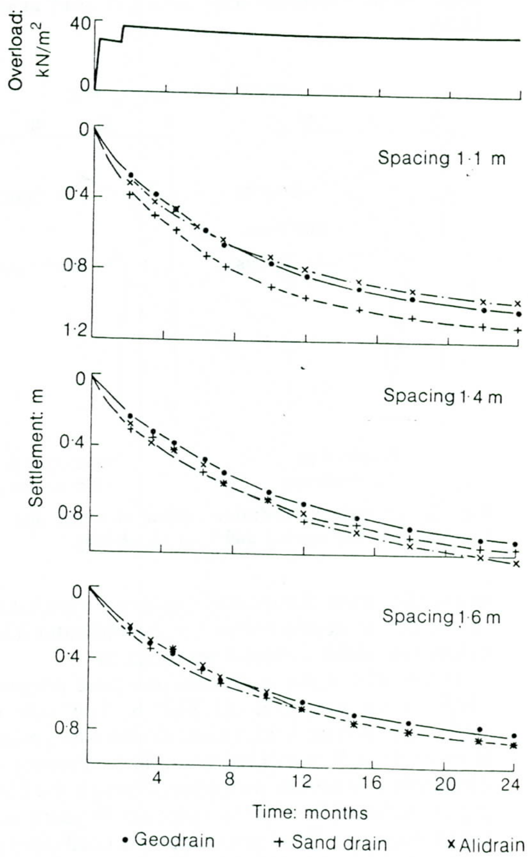


Fig. 10. Settlement of ground surface vs time observed at the Örebro test field

Table 3. Geological profile and geotechnical characteristics of soil in the Örebro test field; groundwater level at about 1 m depth; hydrostatic condition

Depth: m		Soil profile (1)*	Soil profile (2)*	γ: t/m^3 (1)	γ: t/m^3 (2)	w: % (1)	w: % (2)	w_L: % (1)	w_L: % (2)	w_P: %	†c_u/σ_{vo}' (1)	†c_u/σ_{vo}' (2)	OCR (1)	OCR (2)
1	Drain spacing 1·1 m	Grey, slightly organic	Grey, slightly organic	1·41	1·32	116	92	104	91	Not determined	0·64	0·89	2·8	2·1
2		The same, with shells	The same	1·42	1·40	115	105	101	81		0·59	0·38	1·0	1·2
3		Grey, coloured by sulphide	Grey, coloured by sulphide	1·48	1·58	93	67	69	45		0·39	0·31	1·5	1·1
5		Grey, varved	Grey, varved th. seams silt	1·64	1·63	69	64	53	44		0·27	0·30	1·3	1·4
7		Brown, varved coloured by sulphide	Brown, varved coloured by sulphide	1·64	1·71	60	48	43	32		0·22	0·24	1·2	1·2
1	Drain spacing 1·4 m	Grey, slightly organic	Grey, slightly organic	1·26	1·38	91	102	103	113		2·5	2·0	2·3	1·3
2		The same, with shells	The same	1·44	1·43	106	107	94	83		0·49	0·42	1·6	—
3		Grey, coloured by sulphide	Grey, coloured by sulphide	1·52	1·53	91	82	65	63		0·42	0·42	1·0	1·3
5		Grey, varved th. seams silt	Grey, varved th. seams silt	1·50	1·52	97	93	66	65		0·30	0·31	1·2	1·3
7		Brown, varved, coloured by sulphide	Brown, varved coloured by sulphide	1·62	1·64	68	60	46	41		0·24	0·22	1·0	1·5
1	Drain spacing 1·6 m	Grey, slightly organic	Grey, slightly organic	1·45	1·50	88	94	95	83		0·96	0·69	—	1·1
2		Grey, coloured by sulphide	Grey, coloured by sulphide	1·53	1·55	82	90	59	66		0·41	0·39	1·4	—
3		The same	The same	1·56	1·55	79	73	54	52		0·38	0·28	1·8	—
5		The same	Grey-brown, varved coloured by sulphide	1·53	1·67	69	58	46	39		0·24	0·25	1·2	1·0
7		Grey-brown, varved coloured by sulphide	Brown, varved th. seams silt	1·64	1·71	67	54	43	36		0·22 0·28	0·20	1·2	1·0

*(1) Area with sand drains; (2) areas with Geodrain and Alidrain.
† According to fall-cone test.

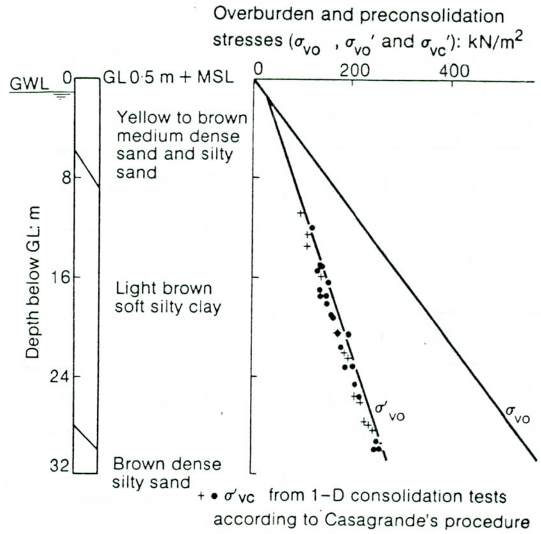


Fig. 11. Porto Tolle typical soil profile

characteristics and depth in the test field (and consequently variations in final settlement) a satisfactory correlation between theory and practice will be obtained assuming that $c_h = 0{\cdot}8\ \mathrm{m^2/year}$, $s = 2$ and $k_c/k_c' = 3$ for the sand drains, and $s = 2$ and $k_c/k_c' = 2$ for the prefabricated drains.

Porto Tolle case record

During the construction of a large (2400 MW) Thermal Power Plant located in Porto Tolle at the extreme eastern point of the Po river delta, extensive controlled water loading and other preloading techniques for large steel tanks and many secondary structures were used (Garassino *et al.*, 1979). A large number ($\approx$1 700 000 m) of vertical drains of various types (jetted drains, Geodrain, Soildrain, Sandwick) 27–30 m in length were installed with the aim of accelerating the consolidation of a thick stratum of soft young normally consolidated (NC) silty clay (cf. Fig. 11). The geotechnical properties of the clay are presented in Table 4 (see also Jamiolkowski, Lancellotta & Tordella, 1980). One of the preloading embankments (340 m by 65 m) was used as a trial embankment with the purpose of comparing the efficiency of the different drain types installed. In this Paper the observed behaviour of this embankment (Fig. 12) is reported. The trial embankment was divided into four sections of nearly equal length (about 85m) each one provided with different drain types. The spacing of the drains was chosen so that the process of consolidation according to the conventional Terzaghi consolidation theory should be the same. As a basis for the design the following values were used: $c_v = 9\ \mathrm{m^2/year}$, determined by back-analysis of the behaviour of a preloading embankment without vertical drains and representing the upper limit of the laboratory values determined on small-size, good quality, undisturbed specimens; and $c_h = 15\ \mathrm{m^2/year}$, representing the upper limit of the laboratory values obtained from tests on small-size specimens.

Further in situ testing such as constant head permeability, piezometer probe dissipation

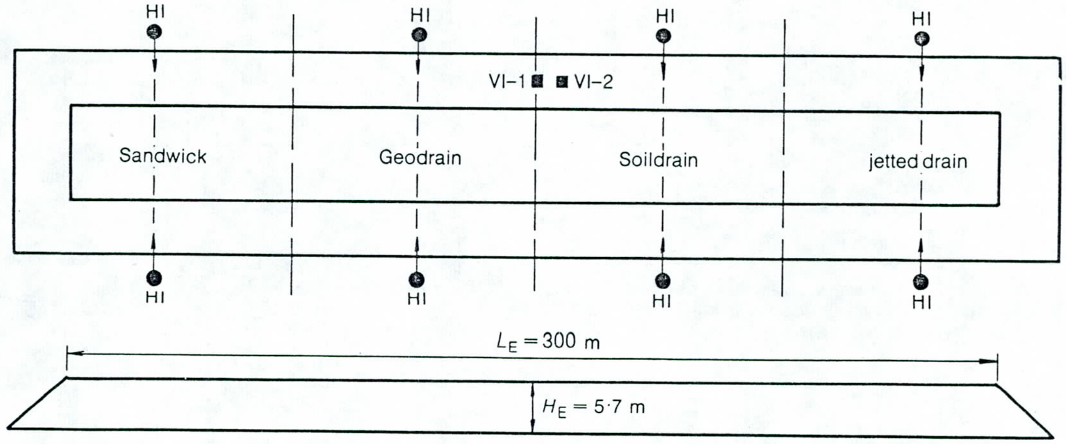
Table 4. Porto Tolle, soil properties; FVT = field vane test; TC = triaxial compression; TE = triaxial extension; DSS = direct simple shear; $\mathrm{Ck_0U}$ = consolidated k_0 undrained

Index properties	γ: t/m³	w: %	w_L: %	I_P: %	CF: %
	1·85±0·05	36·4±4·9	52·6±6·9	30·9±6·6	33·9±4·2

Undrained behaviour	c_u/σ_{1c}'				E_{u50}/σ_{1c}'		
	FVT	$\mathrm{TC-Ck_0U}$	$\mathrm{TE-Ck_0U}$	$\mathrm{DSS-Ck_0U}$	$\mathrm{TC-Ck_0U}$	$\mathrm{TE-Ck_0U}$	$\mathrm{DSS-Ck_0U}$
	0·29*	0·31	0·19	0·26	89	82	87

Drained behaviour	$\mathrm{TC-Ck_0U}$		Oedometer tests	
	ϕ': degree	c': t/m²	CR	c_v: m²/year
	29	0	0·20	3–9

* Sensitivity as obtained from the FVT = 2–3.



Type of drain	Diameter: mm	Spacing:* m
Sandwick	120	4·20
Geodrain	62†	3·80
Soildrain	55	3·80
jetted drain	300	5·00

* Triangular equilateral array
† Equivalent diameter

● Horizontal inclinometer
■ Vertical inclinometer

Fig. 12 Plan and cross-section of Porto Tolle experimental embankment

(Ghionna, Jamiolkowski & Lancellotta, 1978; Lacasse, Ladd & Baligh, 1979) and tests with a self-boring pressuremeter (Jezequel & Mieussens, 1975) permitted an in situ assessment of the c_h leading to the values given in Table 5. In Fig. 13 the penetration pore pressure measured by means of the piezometer probe is also reported.

After the embankment was placed, its behaviour was monitored by means of 45 piezometers of various types, settlement plates, deep settlement sensors of the bellows-hose type, and vertical and horizontal inclinometer tubes which allowed measurement of surface settlement (Fig. 14), vertical strain, pore pressure (Fig. 15) and horizontal displacement (Fig. 16).

From Figs 14–16 and from the experience gained at the site (see also Garassino *et al.*, 1979) the performance of the four types of drain used here is found to be substantially similar in practice. Differences in settlement under different areas cannot be attributed to the variations of soil properties and/or the thickness of the soft silty clay stratum. The higher settlement of the area with Sandwick drains may indicate a greater efficiency of these drains or a higher disturbance caused by their installation than in the case of other types of drain.

Table 5. Porto Tolle, summary of consolidation properties (from Garassino *et al.*, 1979)

Source	Consolidation coefficient c_v: m^2/year	Consolidation coefficient c_h: m^2/year
1. Laboratory tests*	3–9	6–16
2. Field permeability plus laboratory compressibility		
dissipation tests/piezometer probe		22
field permeability tests in piezometer†		30
self-boring permeameter		25
3. Back-analyses		
experimental embankment without drains on the basis of pore pressure	9	
preloading embankment with drains on the basis of pore pressure		19–28
preloading embankment with drains, from strain and deformation measurements		44–66

* These data available when drainage system has been designed.
† Bilotta & Viggiani (1975).

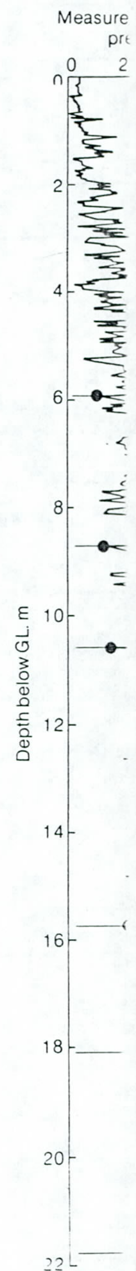


Fig. 13. Pie
experimental

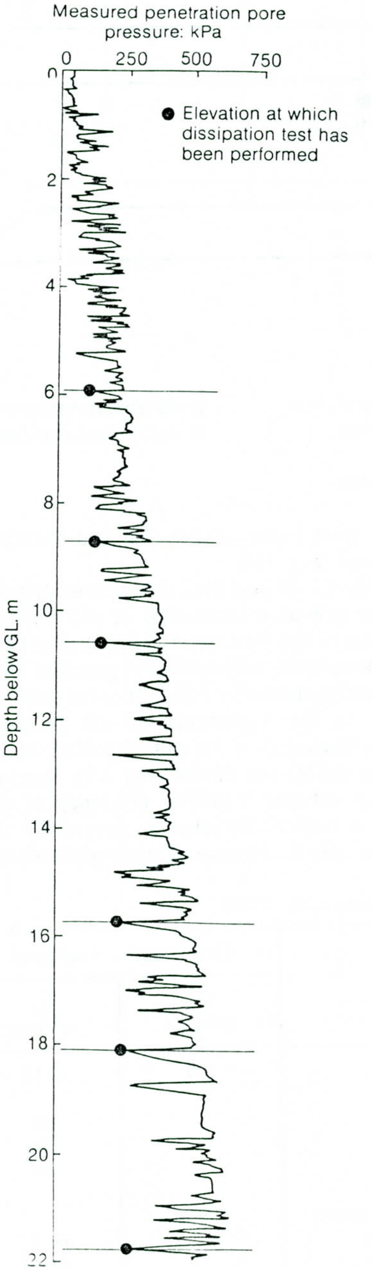


Fig. 13. Piezometer probe penetration test: Porto Tolle experimental embankment

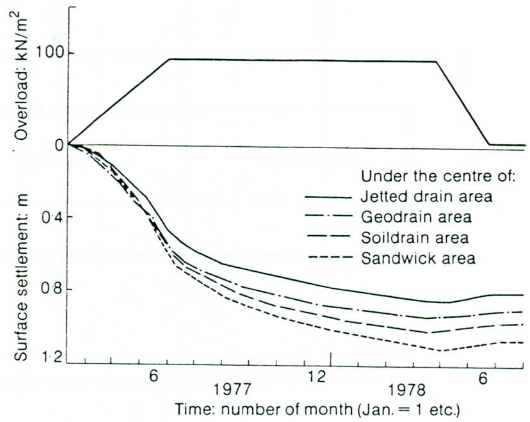


Fig. 14. Observed surface settlement: Porto Tolle experimental embankment

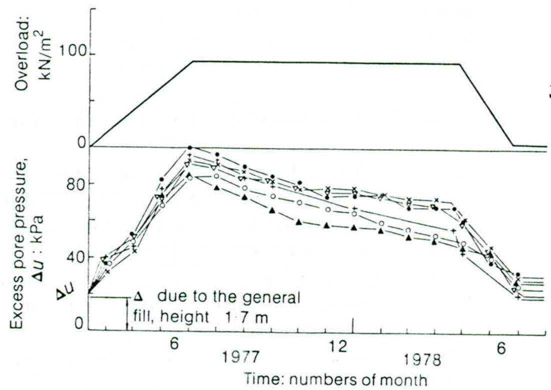


Fig. 15. Observed excess pore pressure: Porto Tolle experimental embankment (see also Table 6 for piezometers and additional information.)

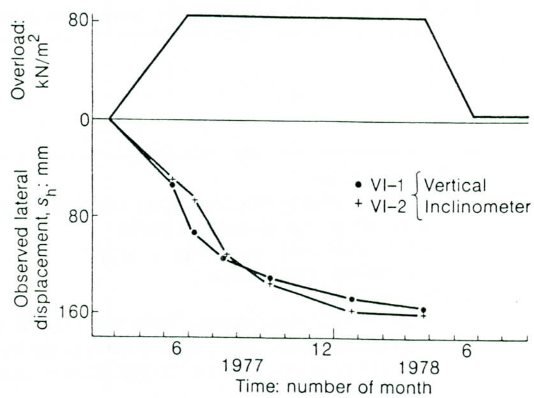


Fig. 16. Porto Tolle experimental embankment, maximum (depth ≈ 10 m below GL) observed horizontal displacement vs time

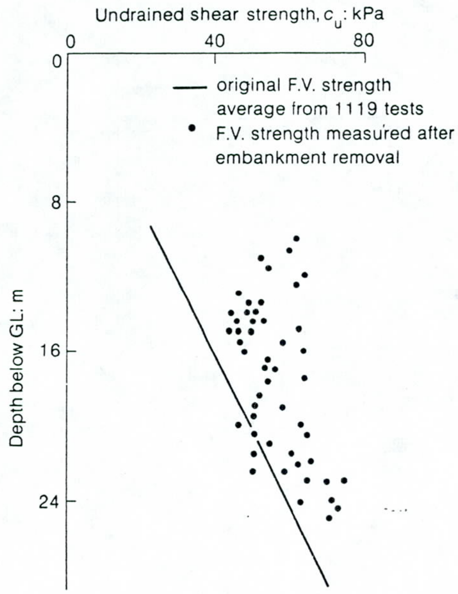


Fig. 17. Porto Tolle experimental embankment, field vane strength before and after preloading

Finding the true reason would require a reliable knowledge of the achieved degree of consolidation deduced from the pore pressure readings. In this case such a deduction is not possible because of problems with the measurement of Δu against time.

The back-analysis of observed settlements against time, measured within the soft silty clay stratum using different velocity methods (Ellstein, 1971, 1972; Scott, 1961), leads to the c_h values shown in Table 5 (for more details see Garassino *et al.*, 1979), which tends to indicate that when the embankment load was removed at least 80% of the consolidation had been achieved. Such a statement is supported by the following facts.

(a) Observed vertical settlements are in the range 75–85% of consolidation settlements evaluated using compressibility properties back-calculated from previously available case records of embankments with similar characteristics of loading and geometry.

(b) The inclinometer readings indicate only very small lateral soil displacement during the period in which large vertical settlement was observed (Fig. 16).

(c) The field vane tests carried out after the removal of the embankment show an increase of undrained shear strength (Fig. 17). This may be explained only on the basis of the achieved consolidation.

The Δu–time trend observed in almost all piezometers[1] of different types is in contradiction with what has been deduced from strain and settlement observations. All these piezometers exhibit a very slow pore pressure dissipation. Those shown in Fig. 15 lead to a consolidation at points in which piezometers are located in the range of 25–40%. Such low values cannot be justified even if making reference to the lowest laboratory-determined consolidation properties (Table 6). In addition, in many piezometers, even a very long time after the load application when settlement had almost ceased, significant residual excess pore pressure remains, showing no tendency to further dissipation.

This case record, representing only a small part of the large experience gained with vertical drainage problems in Porto Tolle, showed the following.

(a) Vertical drains have performed correctly. With the type of preloading technique utilized they solved, in an economical way, many important foundation problems.

(b) The observed consolidation speed deduced from the settlement observations is appreciably higher than that which can be predicted on the basis of the consolidation properties determined by means of in situ tests given in Table 5. This may be explained by the trace of the penetration pore pressure curve in Fig. 13, which indicates frequent occurrence of thin coarse seams (probably silt horizons). When these seams are interconnected by vertical drains, there is on average a very short vertical drainage path. The presence of seams may thus make a substantial contribution to the consolidation by vertical flow.

(c) The delayed excess pore pressure dissipation,[2] or even lack of it (with the 'trapped' values of Δu, 30–40 kPa, in excess of existing static pore pressure), which is in contrast with all other information gained from the monitoring programme, cannot be explained at the present stage of the Authors' knowledge.

Trieste case record

In early 1969 an area of about 1·5 km², reserved for the extension of an existing refinery, was filled up under uncontrolled geotechnical conditions with regard to fill thickness and relevant soil

[1] The discussed piezometer performance is typical for the majority of about 150 piezometers of different types installed in Porte Tolle on areas with vertical drains.

[2] On the other areas with vertical drains few piezometers showed a reasonable Δu dissipation leading to the back-calculated c_h value given in Table 5 and in agreement with other in situ tests.

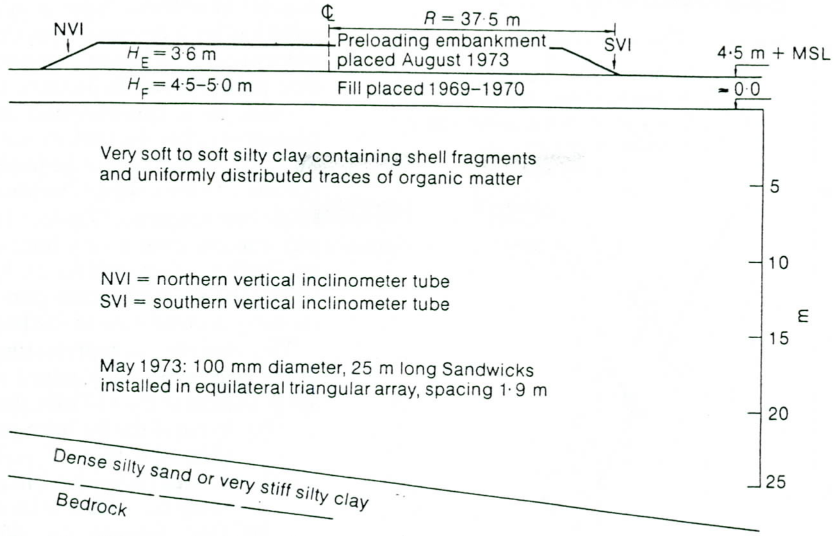


Fig. 18. Trieste preloading embankment, cross-section

characteristics (Fig. 18). Extensive in situ and laboratory investigations were later carried out, leading to the geotechnical characterization of the soft NC clay stratum beneath the fill, summarized in Table 7.

In March 1973, after it had been decided to build two 50 000 m^3 crude oil steel tanks on the examined area, displacement-type sand drains (Sandwicks), 0·10 m in diameter, were installed with 2 m spacing in an equilateral triangular pattern at the sites of the tanks to be built. The drainage system was designed, assuming $c_v = 0{\cdot}6\ m^2/year$ and $c_h = 1{\cdot}2\ m^2/year$, representing the upper limit of the laboratory values determined on good quality, small-volume, undisturbed specimens (see Fig. 19 and Jamiolkowski, 1974).

In June 1973, two embankments, 4·0 m and 3·6 m in height, were placed on the drained area, representing in the intention of the designer a first lift of a two-stage preloading embankment with the aim of anticipating the consolidation of the soft clay stratum before the construction of the steel tanks.

The lower embankment (3·6 m) which is considered here was instrumented as shown in Fig. 20. The pore pressure measurements, made before the placement of the fill, revealed that in June 1973 the soft clay stratum was practically unconsolidated under the weight of the general fill dumped over the area in 1969.

During and immediately after the placement of the first preloading lift, large vertical settlement and horizontal displacement (Fig. 21) took place within

Table 6. Porto Tolle, piezometer types used, as shown in Fig. 15

Type of piezometer	Depth:* m	Drain type/ area†	Observed $\Delta u/\Delta u_{max}$		Predicted‡ $\Delta u/\Delta u_{max}$	
			Time in days			
			175	345	175	345
+ Hydraulic, Bishop type	24·0	Jetted drain	0·807	0·594	0·510	0·260
● Electro-pneumatic	19·4	Soildrain	0·812	0·673	0·534	0·290
× Vibrating wire, Maihak	19·3	Geodrain	0·893	0·767	0·534	0·290
○ Hydraulic, Bishop type	24·0	Sandwick	0·824	0·577	0·520	0·276
▲ Electrical, BAT type	12·2	Geodrain	0·718	0·565	0·534	0·290
▽ Electrical, BAT type	20·2	Jetted drain	0·893	0·699	0·538	0·293

* Below ground level at depth 0·10 m below MSL.
† Under the centre of each area and halfway from the drains.
‡ Terzaghi consolidation theory with no allowance for smear and well resistance assuming minimum possible consolidation properties: $c_v = c_h = 9\ m^2/year$.

the soft clay stratum, with effectively no excess pore pressure (Δu) dissipation recorded, indicating pronounced confined plastic flow within the clay formation (estimated safety factor against undrained bearing capacity failure $\approx 1{\cdot}15$–$1{\cdot}30$).

In the following 9 months large vertical settlement and increasing lateral displacements were observed, together with very slow pore pressure decay (Fig. 22). This decay is in accordance with the conventional Terzaghi consolidation theory adopting $c_v = c_h = 0{\cdot}3\ \mathrm{m^2/year}$ corresponding to the lower limit of the laboratory determined values of c_v.

In April 1974, because of the observed very slow rate of consolidation, the idea of using the preloading technique was abandoned and the embank-

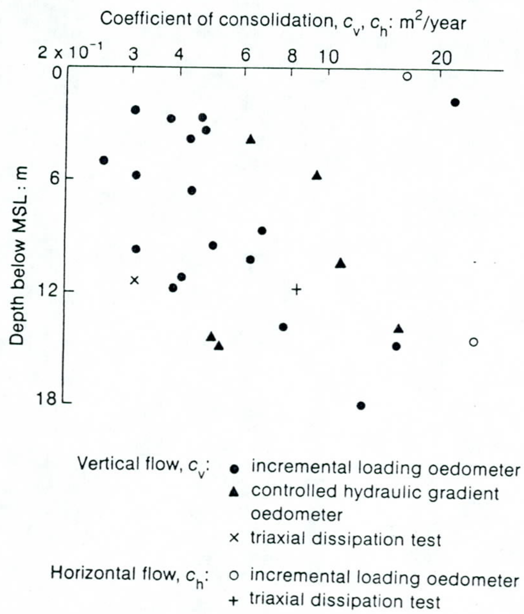


Fig. 19. Trieste preloading embankment, laboratory determined coefficients of consolidation

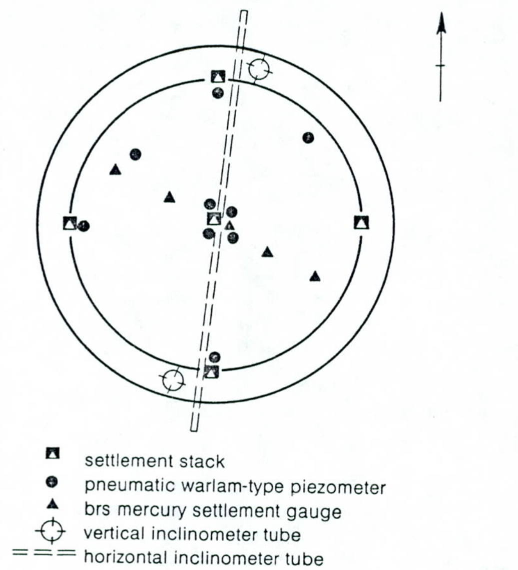


Fig. 20. Trieste preloading embankment, instrumentation

Table 7. Trieste, soil properties

Index properties	γ: t/m³	w: %	w_L: %	I_p: %	CF: %
	1·72 ± 0·04	49·9 ± 4·0	70·7 ± 11·5	47·4 ± 10·0	45·8 ± 6·7

Undrained behaviour	c_u/σ_{1c}'				E_{u59}/σ_{1c}'		
	FVT	TC–Ck$_0$U	TE–Ck$_0$U	DSS–Ck$_0$U	TC–Ck$_0$U	TE–Ck$_0$U	DSS–Ck$_0$U
	0·35*	0·32	0·26	0·28	90	77	144

Drained behaviour	TC–Ck$_0$U		Oedometer tests	
	ϕ': degrees	c': t/m²	CR	c_v: m²/year
	26	0	0·24	0·3–0·9

* Sensitivity as obtained from the FVT = 2–4.

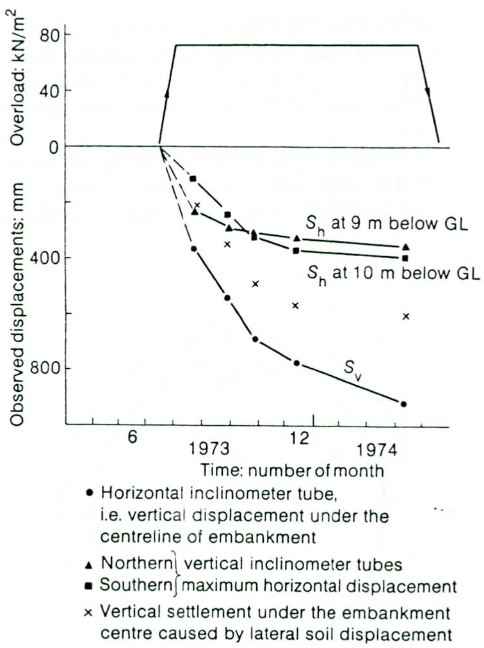


Fig. 21. Trieste preloading embankment, observed vertical and horizontal displacements

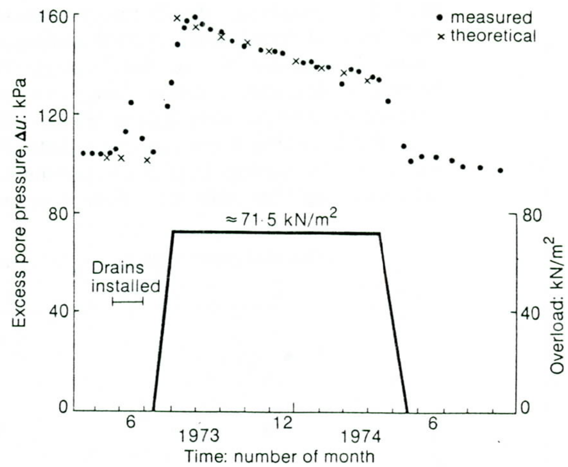


Fig. 22. Trieste preloading embankment, typical piezometer record

ments were therefore removed. In 1975, two floating roof tanks were constructed on the drained areas. These tanks were subjected to storage restrictions, controlled by the measurements of consolidation of the soft clay stratum.

Finally, in November 1978, it was possible to allow unrestricted use of the two tanks under examination. In July 1980 the observed settlement of the tanks, except that which occurred under the preloading embankment, reached values of 1·2–1·4 m under the centre and 0·8–1·0 m under the perimeter, with the primary consolidation process almost totally completed.

During the period 1975–80 (Ghionna *et al.*, 1980) additional in situ and laboratory soil investigations were carried out with the aim of understanding better the geotechnical behaviour of the Trieste clay and obtaining a deeper insight into the examined case record. The investigations using the piezometer probe and the self-boring pressuremeter obtained the following information relevant to the vertical drainage problem.

(a) Penetration pore pressure, not reported here, revealed a complete lack of any more permeable layers of lenses within the clay mass.
(b) Dissipation tests with the piezometer probe indicated a range of c_v from 1·2 to 1·4 m²/year.
(c) The equilibrium pore pressure deduced from piezometer probe dissipation tests and from relaxation tests run with the self-boring device gave as a result the July 1980 distribution of Δu beneath the area without drains that is shown in Fig. 23. This confirms very low clay consolidation properties with respect to the vertical flow.

The examination of the Trieste case record allows the following comments.

(a) The vertical drainage design and performance must be considered as unsatisfactory. The consolidation process unexpectedly coincides with the lower limit values $c_v = c_h = 0{\cdot}3\ \mathrm{m^2/year}$ determined in the laboratory on small-size undisturbed clay specimens.
(b) In respect to this, why, at the considered site, is the field consolidation so slow? What is the influence on the consolidation process of the measured large, virtually undrained, lateral and vertical displacements?

The change in soil structure that would follow upon large undrained deformations may lead to a reduction of the coefficient of consolidation; and the conventional consolidation theory may be inadequate when applied to high plasticity clays with pronounced viscous response, particularly at high stress level. These theories may at least partially explain the peculiar behaviour of the Trieste embankment.

Aziëhaven case record

For quick and efficient installation of prefabricated drains, contractors offer in principle two methods.

In the static type of installation a heavy machine

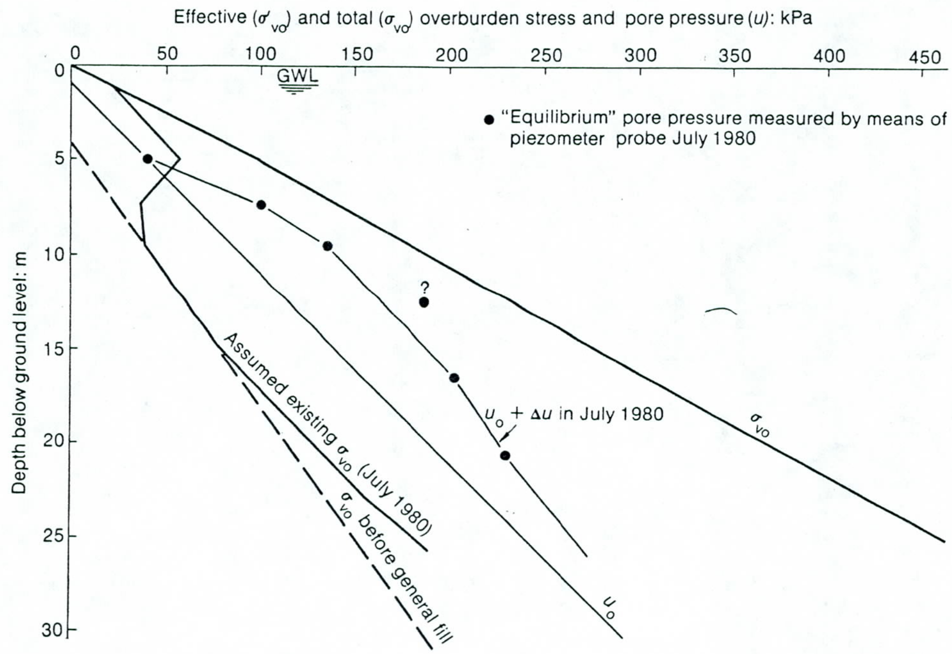


Fig. 23. Trieste preloading embankment, overburden stresses and pore pressure in the area without vertical drains

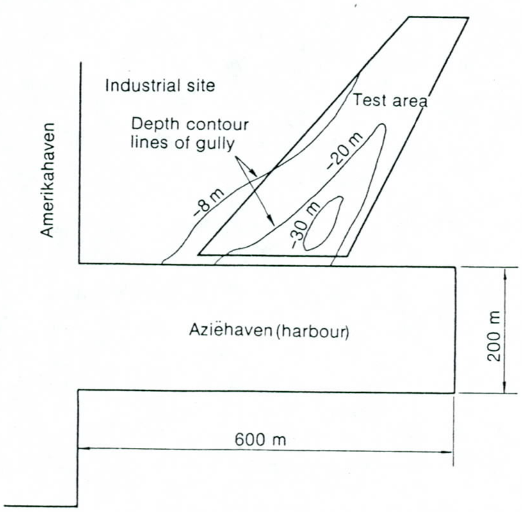


Fig. 24. Aziëhaven, location of test area and depth of clay/peat layer

with sufficient thrust forces a mandrel down to the required depth. This machinery has a high production rate, but has to be assembled on the site.

In the dynamic type of installation the mandrel is either hammered down (as in piling) or the driving force is delivered by a vibrating hammer. These vibrating hammers, with working frequencies of

Table 8. Characteristics of the installation techniques (Aziëhaven and Hemspoor case records)

Characteristics	Static	Dynamic
Penetration velocity (mandrel)	≈0·5 m/s	≈1 m/s
Frequency (vibrator)	—	25 Hz
Cross-sectional area of mandrel	52/70 cm²	50 cm²

8–50 Hz, can be electrically or electro-hydraulically operated. As the vibrating mandrel eliminates an important part of the friction along the shaft, a light crane can handle the combination of vibrating hammer, guidemast and mandrel (Table 8).

To study the effect of the two installation techniques on pore pressure and consolidation response, a new industrial site in the western harbour area of Amsterdam was chosen. The actual site was intersected by an old gully, which was completely filled with naturally sedimented layers of clay and peat. The geographical layout of the area and some depth contour lines are given in Fig. 24. This area needed improvement as far as soil conditions were concerned. It was decided to accelerate the consolidation of the compressible layers by means of vertical drains and preloading.

In view of the techniques the area was divided into two sub-areas. In each sub-area 150 000 m of prefabicated drain (Geodrain) were installed, static-

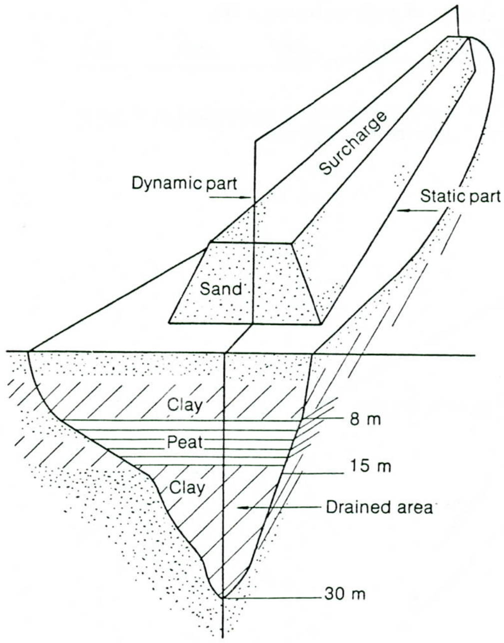


Fig. 25. **Aziëhaven, schematic view of test area**

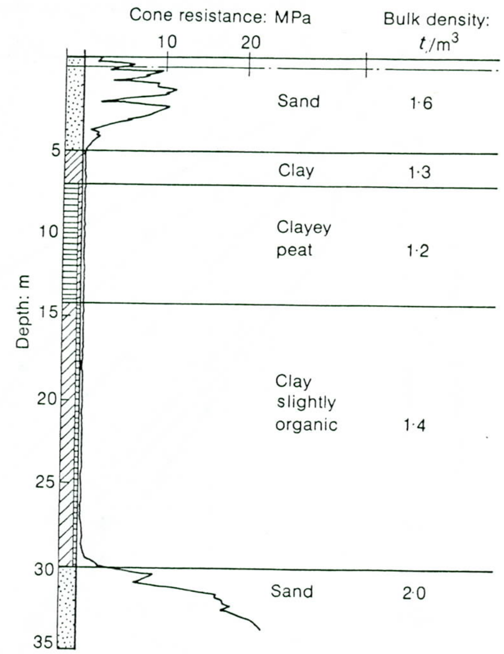


Fig. 26. **Aziëhaven, soil characteristics**

ally as well as dynamically. A schematic view is given in Fig. 25.

Additional subdivision was provided to detect differences in the drain spacing. Triangular grids with drain spacings of 1·2 and 1·8 m were used. The characteristics of the soil strata encountered are shown in Fig. 26.

Direct excess porewater pressure response. Electrical pore pressure devices were installed beforehand in the anticipated triangular grids. A general picture of the excess pore pressure behaviour during and after installation of the prefabricated drains is given in Fig. 27. These results were obtained from the clay layer at depths of 14–15 m below datum. In the static case, relatively small excess and underpressure with respect to hydrostatic pore pressure is generated. In the dynamic case the pore pressure is significantly higher. If it is assumed that the porewater pressure generation in the static case can be calculated according to the theory of an expanding cavity, an estimate can be made of the relation between excess porewater pressure and distance from the mandrel. The result of this calculation, made according to Vesić (1972), is indicated in Fig. 28. The magnitude of the calculated pore pressure is dependent on the unconfined compressive strength (cohesion), but independent of the cross-sectional area of the mandrel.

The pore pressure distribution in soil space is dependent on the cross-sectional area of the

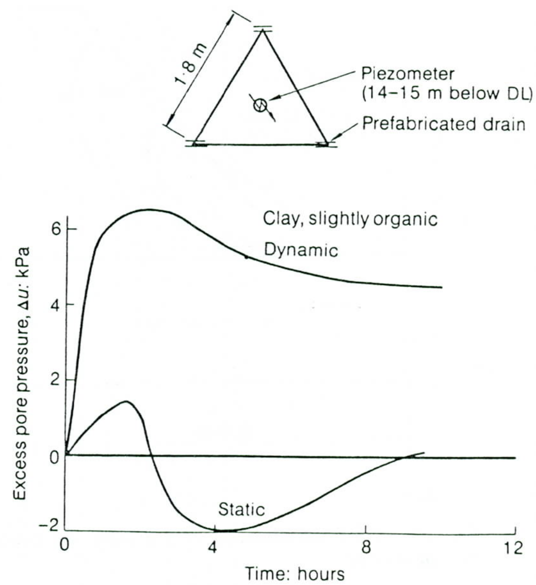


Fig. 27. **Aziëhaven, excess pore pressure after static and dynamic installation**

mandrel. If this is so then high pore pressures are set up in the direct neighbourhood of the mandrel during penetration, and in the plastic zone very close to the mandrel the generated excess pore pressure may exceed the original effective stress

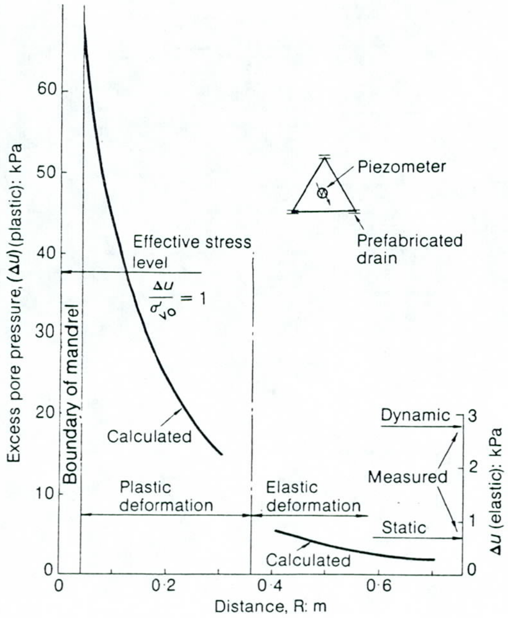


Fig. 28. Aziëhaven, excess pore pressure vs distance from the mandrel

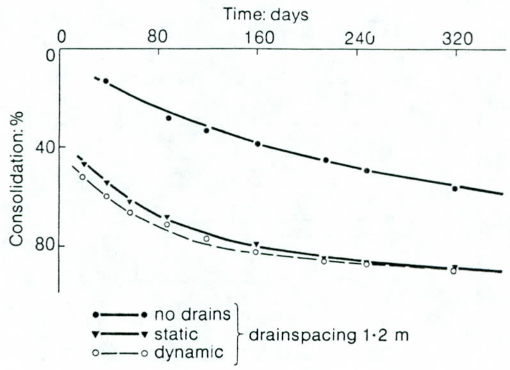


Fig. 29. Aziëhaven, rate of consolidation

($\Delta u/\sigma'_{vo} \geq 1$). This criterion may be used to determine a zone of disturbance (remoulding) around the mandrel. Arbitrarily, $\Delta u/\sigma'_{vo} = 0{\cdot}8$ might be chosen as a criterion for the extension of the zone of smear. From the pore pressure coefficient the distance R can be determined and chosen as d_s in the dimensionless form $d_s/d = s$. This criterion would be invalidated if permeable layers in stratified soils are blocked horizontally due to a dragdown of clay by the mandrel. In certain soils mandrel penetration can cause additional fracturing of the clay layer and improve the permeability of the surrounding soil of the installed prefabricated drain. A substantial effect on drain performance can be achieved if the cross-sectional areas of the prefabricated drain and mandrel, including the anchor plate, are well adapted to the characteristics of the soil to be consolidated. Where prefabricated drains are used to improve the stability of soil structures account must be taken of the generated excess porewater pressure during installation. For that purpose static installation is to be preferred.

Direct consolidation response. The sub-areas were equipped with instruments to record various events like pore pressure, settlements and horizontal displacement. From the settlement records in both areas the consolidation velocity was calculated with respect to time. No distinction is made here with regard to drain spacing (1·2 and 1·8 m). The excess porewater pressure was not taken into account in the presentation of the consolidation process because of the inexplicably high excess pore pressures compared with recorded settlement. No significant difference between the two methods of installation can be made with regard to the rate of consolidation (Fig. 29).

Hemspoor case record

This record shows a pseudo-consolidation effect with respect to piercing the soil layers to be consolidated.

A new elevated railway track had to be built very close to the city of Amsterdam and the alignment of the embankment had to cross an old gully, naturally filled with very soft clay (Fig. 30). The depth of this very compressible layer is approximately 25 m. The design of the elevated embankment shows a maximum height of 5 m above datum and it was assumed that about 13 m of sand was needed to reach this design height (Fig. 30).

To obtain sufficient stability during the construction, and a minimum of end settlement in the long term, vertical drainage by means of prefabricated drains was envisaged. Prefabricated drains were chosen because of successful applications in other projects (Aziëhaven case record) and the hazards to be expected during the installation of sand drains over 25 m length.

Characteristics of the soil, which consists of soft clay, except just below original ground level, are shown in Fig. 31. Prefabricated drains were installed in a triangular grid with spacing of 1·2 m by dynamic means. Piezometers were installed at various depths in the soft clay and the first fill was placed. The filling operations and the appropriate pore pressure readings can be seen in Figs 32 and 33. When the prefabricated Geodrains were installed, an immediate pore pressure response was obtained. The excess pore pressure dropped dramatically, but there was concern when a new load (dry sand fill) was placed and there was no sign

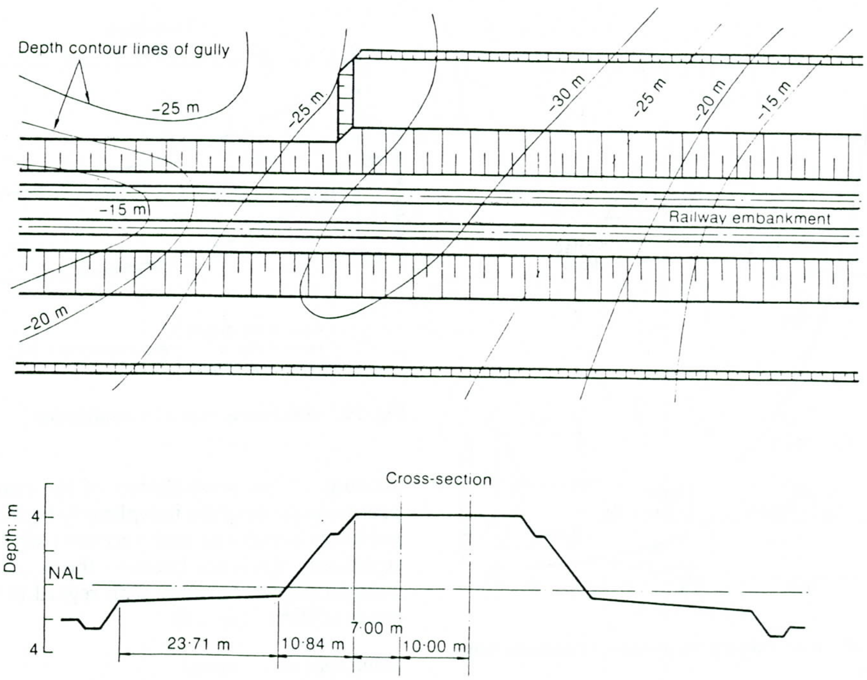


Fig. 30. Hemspoor, view and cross-section of test area with depth of soft layer

of a quick release of excess porewater pressure. Additional prefabricated drains were installed to obtain a spacing of 0·6 m. The excess porewater pressure responded immediately. Again filling the site resulted in a load-dependent increase in pore pressure, without a significant drainage effect. The results are given schematically in Fig. 34.

No satisfactory answer for this odd drainage behaviour has been given yet. This effect could be initiated by the installation of the drain itself. The large cross-sectional area ratio between mandrel and prefabricated drain may cause the release of excess pore pressure. The pore pressure coefficient r_u recorded during the normal filling procedures with dry sand, varied between 0·6 and 0·7. An exceptional decrease of the pore pressure coefficient was observed directly and for three months after installing the prefabricated drains.

CONCLUSIONS AND RECOMMENDATIONS

Installation effects

There is a remarkable difference in the excess porewater pressure generation between the static and the dynamic installation techniques.

There is no significant difference, it seems, between dynamic and static installation techniques in respect of consolidation.

A remarkable excess pore pressure release is recorded immediately after the installation of drains; this release has no direct connection with consolidation by drain wells in itself.

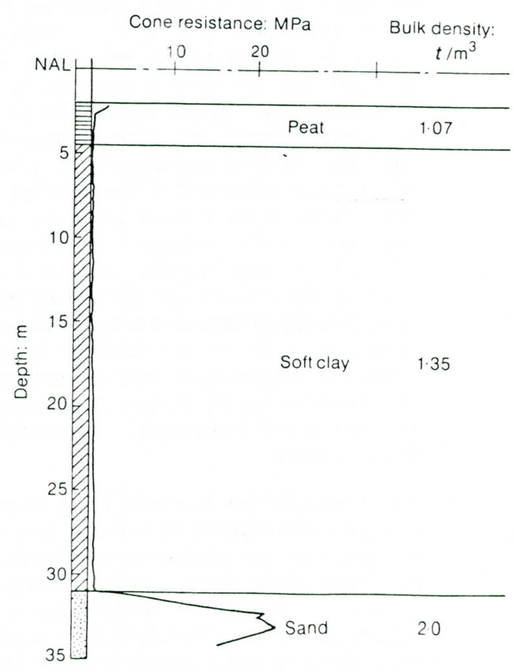


Fig. 31. Hemspoor, soil characteristics

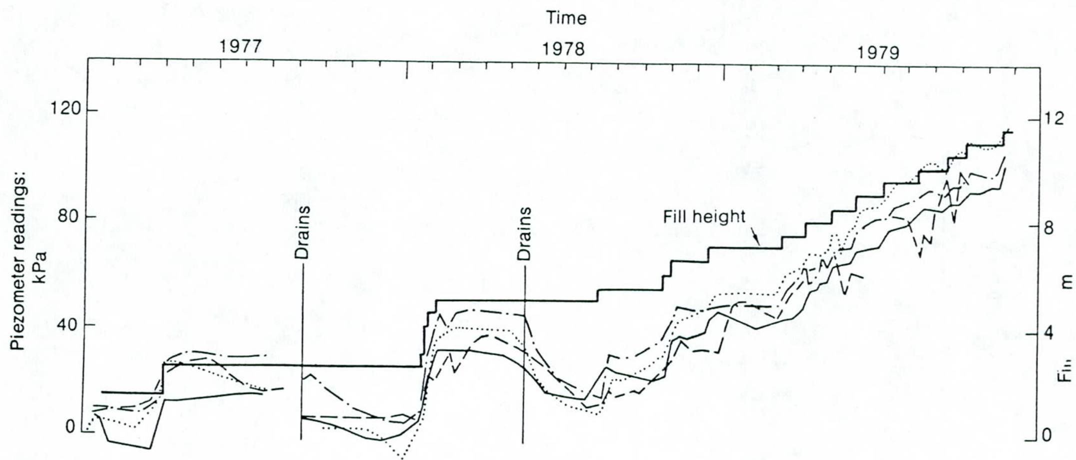


Fig. 32. Hemspoor, pore pressure and fill height vs time

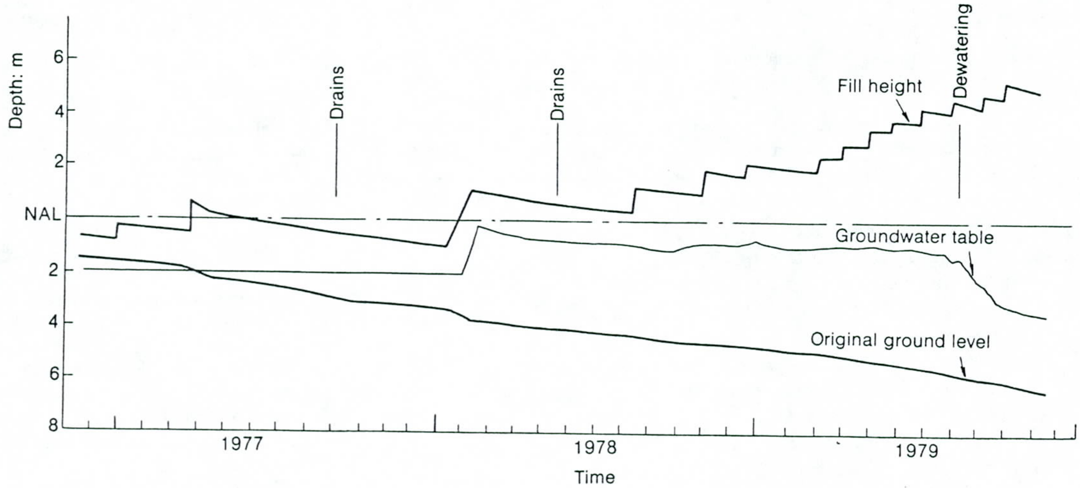


Fig. 33. Hemspoor, depth of original ground level and fill height vs time

When prefabricated drains are used to improve the stability of, for instance, slopes of foundations, the dynamic installation technique should be avoided. Otherwise the installation of drains may occasionally affect the existing stability condition.

Excess pore pressures are generated by the insertion of the mandrel. This can lead to an extended zone of disturbance. The ratio of cross-sectional area of drain and mandrel (anchor plate) should be carefully adapted to the soil. More attention should therefore be given to the installation equipment. The adopted installation procedure and related modifications of properties of the soil surrounding the drain have a great influence on the drainage system performance and therefore this factor has to be considered in the design. The fact that drains with different diameters but equal spacing may give the same consolidation effects can be explained by different values of s or k_c/k_c'.

Process of consolidation

Notwithstanding good progress made in laboratory and field techniques allowing determination of k_h or c_h, the reliable assessment of these critical design parameters remains one of the most difficult tasks within the area of geotechnical engineering. Even if in the majority of the examined case records $c_h \gg c_v$, an uncritical and generalized assumption that this is always the case is not justified. For long prefabricated drains and small-diameter sand drains, well resistance has to be considered in the

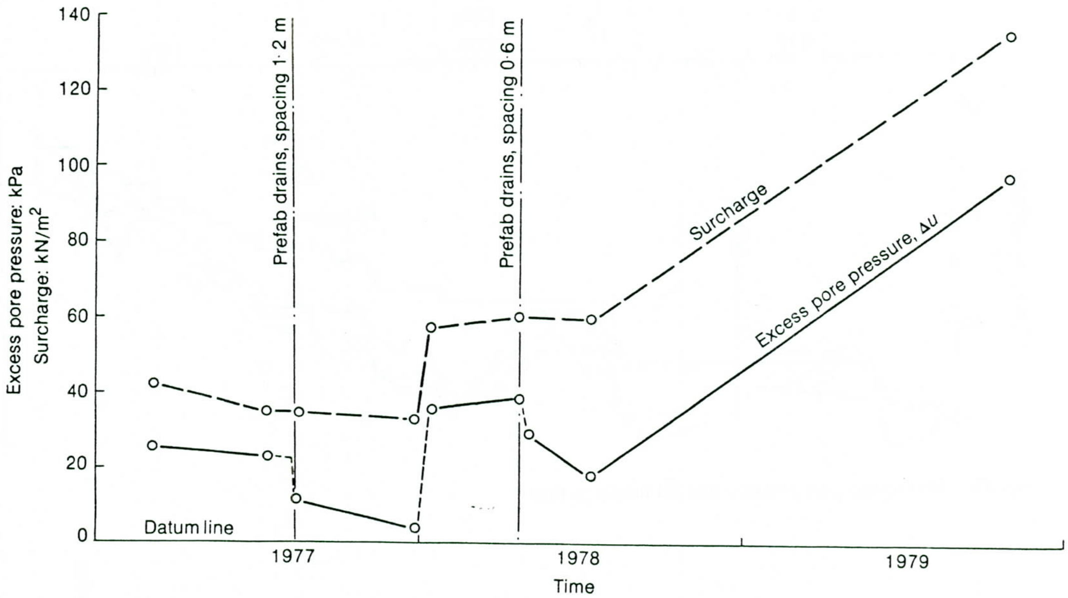


Fig. 34. Hemspoor, schematic result of excess pore pressure and surcharge vs time

design of the drainage system. Well resistance can explain why the relative increase in shear strength due to consolidation gets smaller with depth.

In many cases the rate of consolidation estimated from vertical settlement observations is higher than that estimated from excess pore pressure dissipation, although almost no lateral movements have occurred. The fact that an increase in undrained shear strength has been observed in spite of an almost unchanged excess pore pressure is a reason to rely more on settlement than on pore pressure dissipation. Since in some cases excess pore pressure remains although further long-term settlement is negligible, the Authors recommend the use of settlement as a measure of the process of consolidation.

Experienced engineering judgement is very necessary with respect to several uncertainties in parameter determination, design and drain performance, to avoid disappointing results.

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AMERDRAIN™

DESIGNING WITH SOIL DEWATERING WICKS

BACKGROUND

In foundation engineering, consolidation settlement of clay and mud often creates serious problems. Significant consolidation settlement occurs when, for some reason, the preconsolidation pressure of the subsoil, representing the past maximum effective stress, is exceeded. When the stresses in the subsoil exceed the preconsolidation pressure, the soil skeleton will break down (internal shear failure) and the stress increment will have to be carried by excess porewater pressure (in water saturated soils) or by a combination of excess porewater and excess pore gas pressures (in non-saturated soils). Porewater (and dissolved pore gas) will thereby be squeezed out of the soil until the soil skeleton is again able to carry the load.

This consolidation process is governed by the rate of excess pore pressure dissipation, i.e. by the coefficient of consolidation of the soil and of the thickness of the consolidating layer. In a case where the clay or mud layer is homogeneous (having no horizontal continuous highly permeable seams or layers) and the width of the load placed on the layer is large in comparison with the thickness of the layer, the porewater is squeezed out mainly on the vertical direction. In such a case, the time during which consolidation settlement will occur is often very long—for a 10m (33 ft) homogeneous clay layer drained at the top and bottom, some 50 to 100 years depending upon the magnitude of the coefficient of consolidation of the soil. If the thickness of the homogeneous layer is doubled, consolidation time will be increased four-fold.

To reduce consolidation time, it is obviously necessary to shorten the length of the flow paths. One way of doing this is to install vertical drains of high permeability. Thereby porewater can also escape in the horizontal direction toward the drains and flow freely along the drains vertically to a drainage blanket placed on the soil surface or to other highly permeable layers deeper down in the soil.

DRAIN TYPES

The best-known type of drain in foundation engineering is the sand drain. It is probably less well known that prefabricated drains were introduced into the field in 1937, almost simultaneously with sand drains.

Early prefabricated drains were constructed of cardboard which proved to be an inferior material due to its poor permeability and inadequate strength. The new AMERDRAIN wick drain takes advantage of recent advantages in geotextiles to provide a highly permeable, extremely strong wick with high water discharge capacity. AMERDRAIN soil drainage wick can be installed in a fraction of the time and at a fraction of the cost of sand drains.

Engineers who have worked only with vertical cylindrical drains of conventional type, i.e. sand drains, may be unfamiliar with prefabricated drains and be unaware of how to apply the knowledge obtained from their previous experience of vertical drain installations. It therefore seems necessary to discuss the design of a vertical drain installation with prefab drains and to clarify the difference between these drains and the more well-known cylindrical sand drains.

DESIGN CONSIDERATIONS

Basic Approach

The principle underlying vertical drainage is simple, but the theoretical description of the operating mechanism is fairly complex. Basically, the task is to determine the drain spacing which will give the required settlement in the desired time given the soil conditions at the construction site.

Theoretical calculation (Hansbo, 1979)

The theoretical calculation of the maximum drain spacing required to obtain a desired result is based on the classical assumption that each drain has a zone of influence represented by a circular cylindrical soil column of the same length as the drain and containing that volume of soil from which water can be assumed to be squeezed (or sucked) into the drain in question. It can then be readily found that the diameter D of the dewatered cylinder varies from 1.05 times the spacing when the drains are placed in an equilateral triangle grid to 1.15 times the drain spacing when they are placed in a square grid.

Another basic assumption which considerably facilitates the theoretical calculation is that, during consolidation, horizontal sections remain horizontal (equal strain theory). The difference between the results thus obtained and the result of—as is often believed—the more correct assumption of a free development of strains in the clay between the drains (free strain theory) is negligible (Barron, 1944). Moreover, settlement observations in the field strongly support the assumption of equal vertical stains (cf. Holtz & Holm, 1972). In the classical solution it is further assumed that the permeability of the drain is infinite in comparison with that of the clay and that Darcy's law is valid.

As has been shown in several investigations (e.g. Hansbo, 1960), Darcy's law is sometimes invalidated at small hydraulic gradients prevailing in practice in drained areas. Thus, the results of permeability tests in the laboratory and of fullscale consolidation tests at Ska-Edeby, Sweden, showed that the relation between porewater flow v and hydraulic gradient i in this case followed the exponential law $v = Ki^n$ where K = the coefficient of permeability in non-Darcian flow. A new solution to the "equal stain" consolidation theory based on this exponential law was presented (Hansbo, 1960).

The best agreement between the fullscale test results at Ska-Edeby and this new theory was obtained for the exponent value n = 1.5 (Hansbo, 1960; Holtz & Broms, 1972). For this value, the new theory gives:

$$t = \frac{\alpha}{\lambda} D^2 \sqrt{Dg\rho_\omega/\Delta\bar{u}_o} \left(\frac{1}{\sqrt{1-\bar{U}_h}} - 1 \right) \quad (1)$$

where

t = time of consolidation,

α = function of D / d (see below).

D = diameter of dewatered soil cylinder,

d = diameter of drain,

$\lambda = \frac{KM}{g\rho_\omega}$ = coefficient of consolidation in horizontal non-Darcian porewater flow,

$M = 1/m_v$ = compression modulus,

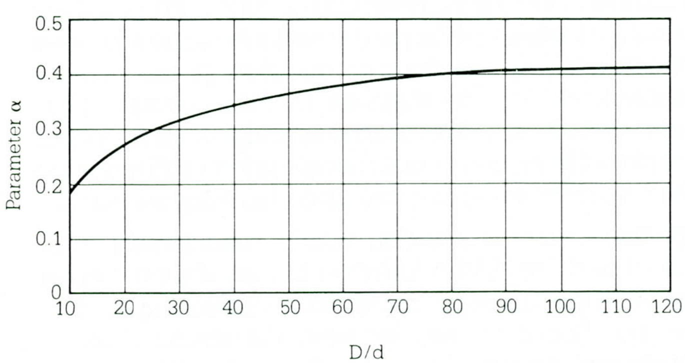
K = permeability in horizontal direction,

ρ_ω = density of water,

g = acceleration of gravity,

$\Delta\bar{u}_o$ = average excess pore pressure at t = 0, and

$\bar{U}_h$ = average degree of consolidation taking into account only the effect of the vertical drains.



The coefficient of consolidation λ can be assumed to be approximately equal to c_v as determined by an oedometer test. The magnitude of the consolidation load or surcharge (i.e. the instantaneous excess pore pressure $\Delta\bar{u}_o$ due to loading) has an influence on the time of consolidation: the higher the load, the shorter the time of consolidation. This is encountered in many cases in practice. Furthermore, the process of consolidation is more rapid at the beginning, a fact which also agrees in many cases with practical experience, particularly on a site with soft highly plastic clay.

In the theoretical calculation, it is presumed that the drain is a circular cylinder with the diameter d. When dealing with a band-shaped drain, we therefore have to assume a d value that will produce the same effect as the band-shaped prefabricated drain in question. This question was treated by Kjellman (1948) who stated that "the draining effect of a drain depends to a great extent upon the circumference of its cross-section, but very little upon its cross-sectional area" and that "certain considerations show that the cardboard wick is as effective as a circular drain with a 1 inch radius". Kjellman's assumption has been verified by finite element analysis (Runesson, Tagnfors & Wiberg, 1977). The result of this analysis shows that the equivalent diameter of a band-shaped drain with width b and thickness t can be expressed by:

$$d = \frac{2(b+t)}{\pi} \quad (2)$$

yielding an equivalent diameter for AMERDRAIN (b = 100mm, t = 3mm) of 50mm.

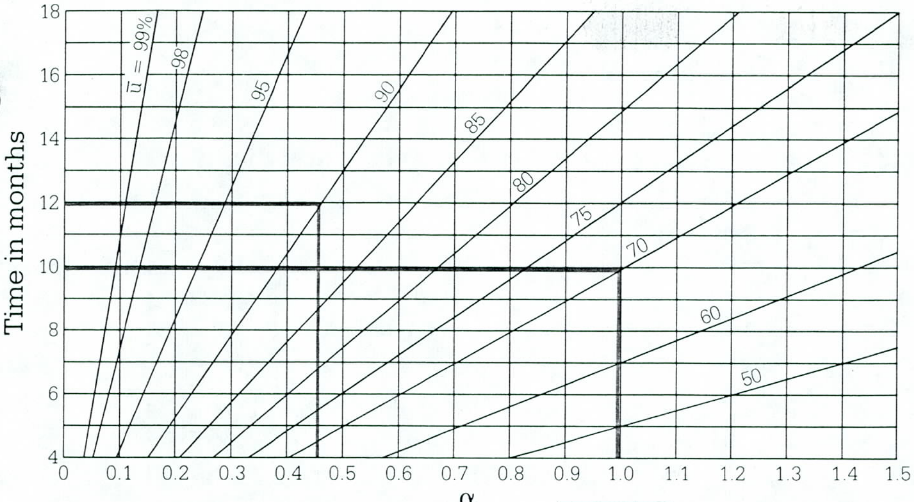
Drain Layout

The most common drain layout is with the drains placed in a triangular pattern, i.e. the drains in the corner points of equilateral triangles. This is the most efficient placement of drains. For a triangular spacing, the drain spacing is D (the diameter of the dewatered cylinder as determined by equation 1) divided by 1.05. If a square pattern is used, the drain spacing is equal to D divided by 1.15. In either case, the number of drains will be the same, as the spacing must be closer with the square pattern to compensate for a less efficient layout.

Drain Spacing

Equation 1 determines the diameter of the dewatered cylinder D and therefore the drain spacing required to achieve the desired consolidation. The equation is presented graphically in Figures 1-5 so that drain spacing may be determined easily. Using Figure 1, with the required degree of consolidation $\bar{U}_h$ plus the desired time of consolidation t yields an intermediate factor f_2 expressed in years. This factor is used with Figures 2 & 3 for metric units or Figures 4 & 5 for English units to determine drain spacing.

FIGURE 1



f_2 in years Parameter $f_2 = \frac{\alpha}{\lambda} D^2 \sqrt{Dg\rho_\omega/\Delta\bar{u}_o}$

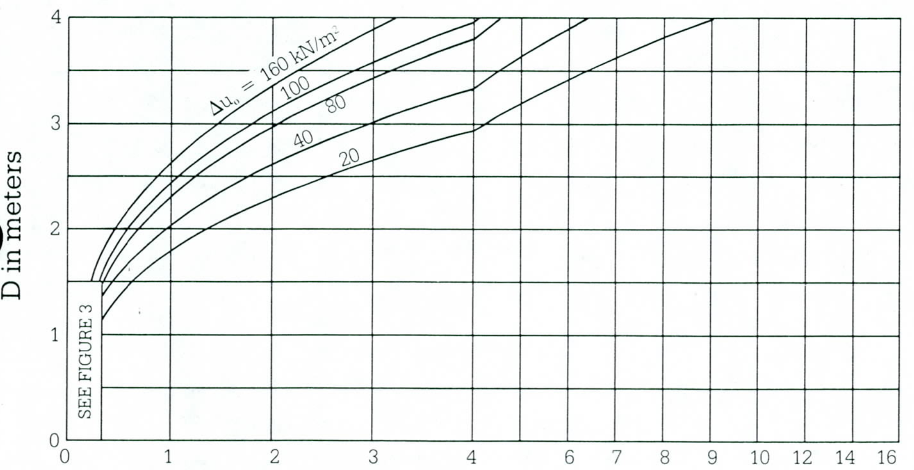
Example (metric units)

What drain spacing is required using a triangular pattern to achieve a 90% average consolidation in 12 months in a homogeneous clay with $c_v = 0.2 m^2/year$ and with a surcharge of $80 kN/m^2$.

For a time of 12 months and an average consolidation of 90%, Figure 1 provides $f_2 = 0.46$ year therefore $f_2\lambda = 0.092 \text{ m}^2$.

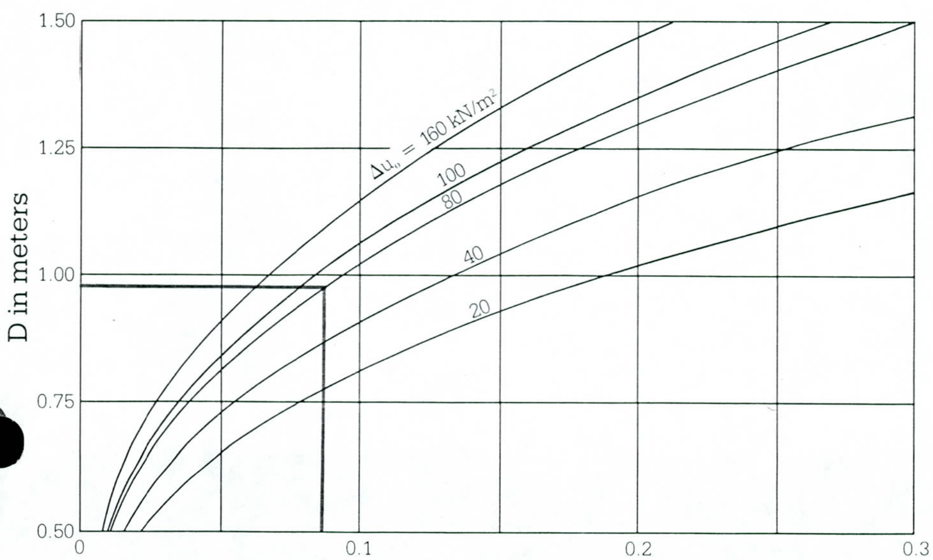
For $f_2\lambda = 0.092$ and a loading of 80 kN/m^2, Figure 3 yields D = 1.0 meter. For a triangular pattern, drain spacing is 0.97 divided by 1.05 or .9 meter.

FIGURE 2



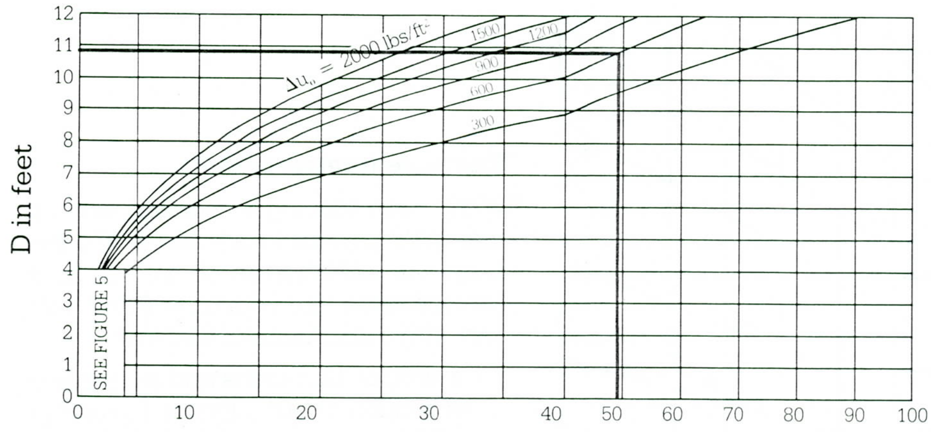
$f_2 \lambda$ in m^2

FIGURE 3



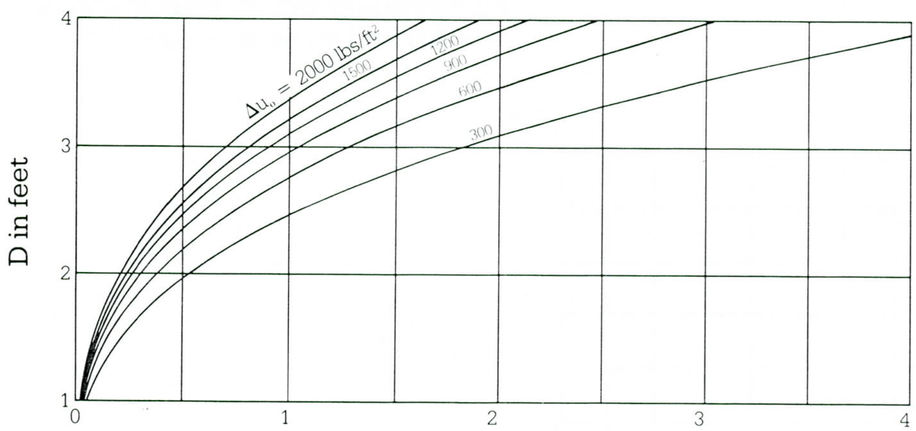
$f_2 \lambda$ in m^2

FIGURE 4



$f_2 \lambda$ in ft ^{2}

FIGURE 5



$f_2 \lambda$ in ft^2

Example (English units)

What drain spacing using a square pattern is required to reach a 70% consolidation after 10 months in a homogeneous silt with c_v = 50 ft^2/year and with a surcharge of 600 lbs/ft^2?

For a time of 10 months and an average consolidation of 70%, Figure 1 provides f_2 = 1.0 year. There $f_2\lambda$ = 50 ft^2.

For $f_2\lambda$ = 50 and a loading of 600 lbs/ft^2, Figure 4 yields D = 10.8 feet. On a square pattern, the drain spacing would be 10 divided by 1.15 or 9.4 feet.

INSTALLATION CONSIDERATIONS

Installation Methods

Two methods are used for installing wick drains. Commonly, they are referred to as the "static" and "dynamic" methods. Both methods utilize a crane or back-hoe together with a mast which contains a hollow mandrel or lance through which the drain material is threaded. An anchor plate or device is attached to the end of the drain and serves not only to keep the soil out of the inside of the mandrel but, primarily, to keep the drain anchored in the soil at the proper installation depth when the mandrel is removed.

In the static method, hydraulic pressure is used to push the mandrel into the soil and to extract it. In the dynamic method, vibration is used to drive and extract the mandrel. Both methods have merits. Recently in Holland, at Aziehanen, both methods were employed on the same large scale project according to Hansbo, Jamiolowski and Kok (1981). Comparison results showed no difference in the performance of drains installed by either method.

Checking the Consolidation Process

The methods used for checking the degree of consolidation generally consist of measuring the excess pore pressure dissipation and/or the amount of settlement (Hansbo, 1979 & 1981).

Some case records indicate that measuring pore pressure dissipation is not fully reliable. Excess pore pressure may remain although settlement has ceased entirely. This may be caused by a change in the ground water level due to the placement of the surcharge. Another reason can be that the drainage blanket is not pervious enough and that back pressure exists in the drains.

The most reliable way of checking the rate of consolidation is usually to measure the rate of settlement and the increase in undrained shear strength.

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October 17, 1985

BGB 10/23

Flood Control District of Maricopa County
3335 West Durango Street
Phoenix, Arizona 85009

Attn: Chief, Construction and Contracting Branch

Re: Aqua Fria River Improvement Project
Thomas Road to north of Indian School Road
Construction Contract 85-10
Consultant Contract 85-12

Job No. 2155J020-E

This report contains the results of our geotechnical investigation for a portion of the proposed RID canal embankment located between Stations 20+50 to 24+00 on the above referenced project. The purpose of these services is to provide information relative to foundation conditions and the anticipated performance of the proposed embankment within this portion of the alignment.

Prior to the time of our exploration, this portion of the proposed embankment had been covered by 15 to 20 foot high aggregate stockpiles. We were informed by JSJ personnel that these stockpiles had been in-place approximately 18 months. During removal of the stockpiles, soft subsurface material was encountered which would not support large earthmoving equipment. At this point, removal of the aggregate was discontinued and several test pits were excavated in the area. Observation of the pits indicated that 3 to 8 feet of coarse aggregate remained over the very soft, saturated clay. JSJ personnel later confirmed that this area had been mined as an aggregate source, and then used as a settling pond for wash water from the nearby aggregate and ready-mix plant.

Three borings were drilled to depths of 25 to 39 feet at the stations shown on the Log of Boring sheets. In addition, continuous penetration resistance was measured at 10 other locations in an attempt to better define the western limits of the old pit. During exploration, subsurface materials were examined visually and sampled at selected intervals. As presented on Logs of Borings, 3.5 to 7.5 feet of coarse aggregate remained at the locations drilled. The underlying clay material exhibited low density and very high plasticity. The clay was soft to very soft and at or near saturation throughout the entire depth. Indications of organic material were encountered in all samples of the clay. Borings 1 and 2, drilled to depths of 25 and 27 feet, did not penetrate the clay. Dense sand, gravel and cobbles were encountered at a depth of 30.5 feet in Boring 3. Auger refusal occurred on cobbles at a depth of 39 feet in Boring 3. A static groundwater table did not develop in any of the borings at the time of this exploration.

Selected samples were tested for water content, density, gradation, plasticity, unconfined compressive strength, and consolidation characteristics. Laboratory test results indicate that the clay material is highly compressible and exhibits relatively low unconfined compressive strength. The water content of the clay was well above the plastic limit and nearing the liquid limit in all of the samples tested. Laboratory test results are included hereinafter.

During the field exploration, a shallow backhoe trench was excavated to locate the south side of the old pit. The edge was located approximately 5 feet south of the proposed canal centerline. This correlated well with the south edge of the existing pit near Sta. 13+00. Based on the continuous penetration measurements, the west end of the old pit was somewhat variable, but appeared to occur around Sta. 23+75. The side slope angles of the old pit are still unknown.

Based on the data developed during the field exploration and laboratory testing, the following conclusions and recommendations are presented:

- The clay sediment material contained within the old settling pond area is highly compressible and exhibits relatively low shear strength. It is anticipated that construction of the embankment, as designed, would be extremely difficult. Calculations indicate that continual progressive bearing capacity failure would occur in the pit foundation area during construction.

- Time rate of settlement analysis indicates that the clay deposit would require 12 to 15 years to reach 90 percent of primary consolidation. In addition, the moderate organic content of the clay would result in significant long-term secondary compression.

- Total settlements at the top of the north side-slope of the proposed embankment are estimated to be within a range of 3 to 4 feet. Native subsoils in the area consist of medium dense sand and gravel which will exhibit only minor settlements under embankment loadings. Since the edge of the old pit occurs within the canal prism, severe differential settlements will occur at this location resulting in cracking and displacement of the canal lining.

- The canal embankment, as designed, will be extremely difficult to construct over this portion of the proposed alignment. In addition, continual long-term maintenance problems should be anticipated as the clay sediment material consolidates with time.

It is recommended that the clay sediment material be removed if the canal embankment, as designed, is constructed over this portion of the alignment. Consideration should be given to redesigning and/or realigning the canal in this area. One such redesign might be a structural flume founded on the native subsoils south of the old pit. A reinforced fill, isolated from the flume, could then be constructed for required access adjacent to the north side. Movements would still be experienced in the fill areas underlain by the clay, but the flume would remain intact and maintenance problems would be greatly reduced. A large diameter pipeline, constructed in the native soil area, may also be a feasible alternative.

The conclusions and recommendations presented in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed by the borings. If realignment or redesign is performed, additional exploration should be conducted to supplement this information and provide detailed geotechnical recommendations.

If you have any questions concerning this information, or if we may be of any additional service, please call us.

Sincerely,
WESTERN TECHNOLOGIES INC.
Geotechnical Services

Registered Professional Engineer
11860
CRAIG P.
WIEDEMAN
Date Signed 6/7/85
ARIZONA, U.S.A.

Craig P. Wiedeman, P.E. Reviewed by: Kenneth L. Ricker, P.E.

jh

Copies to: Addressee (3)

ALLOWABLE SOIL BEARING CAPACITY ALLOWABLE FOUNDATION PRESSURE	The recommended maximum contact stress developed at the interface of the foundation element and the supporting material.
BACKFILL	A specified material placed and compacted in a confined area.
BASE COURSE	A layer of specified material placed on a subgrade or subbase.
BASE COURSE GRADE	Top of base course.
BENCH	A horizontal surface in a sloped deposit.
CAISSON	A concrete foundation element cast in a circular excavation which may have an enlarged base. Sometimes referred to as a cast-in-place pier.
CONCRETE SLABS-ON-GRADE	A concrete surface layer cast directly upon a base, subbase or subgrade.
CRUSHED ROCK BASE COURSE	A base course composed of crushed rock of a specified gradation.
DIFFERENTIAL SETTLEMENT	Unequal settlement between or within foundation elements of a structure.
ENGINEERED FILL	Specified material placed and compacted to specified density and/or moisture conditions under observation of a representative of a soil engineer.
EXISTING FILL	Materials deposited through the action of man prior to exploration of the site.
EXISTING GRADE	The ground surface at the time of field exploration.
EXPANSIVE POTENTIAL	The potential of a soil to expand (increase in volume) due to the absorption of moisture.
FILL	Materials deposited by the action of man.
FINISHED GRADE	The final grade created as a part of the project.
GRAVEL BASE COURSE	A base course composed of naturally occurring gravel with a specified gradation.
HEAVE	Upward movement.
NATIVE GRADE	The naturally occurring ground surface.
NATIVE SOIL	Naturally occurring on-site soil.
ROCK	A natural aggregate of mineral grains connected by strong and permanent cohesive forces. Usually requires drilling, wedging, blasting or other methods of extraordinary force for excavation.
SAND AND GRAVEL BASE	A base course of sand and gravel of a specified gradation.
SAND BASE COURSE	A base course composed primarily of sand of a specified gradation.
SCARIFY	To mechanically loosen soil or break down existing soil structure.
SETTLEMENT	Downward movement.
SOIL	Any unconsolidated material composed of discrete solid particles, derived from the physical and/or chemical disintegration of vegetable or mineral matter, which can be separated by gentle mechanical means such as agitation in water.
STRIP	To remove from present location.
SUBBASE	A layer of specified material placed to form a layer between the subgrade and base course.
SUBBASE GRADE	Top of subbase.
SUBGRADE	Prepared native soil surface.

METHOD OF SOIL CLASSIFICATION
(ASTM D 2487)

COARSE-GRAINED SOILS

LESS THAN 50% FINES*

GROUP SYMBOLS	DESCRIPTION	MAJOR DIVISIONS
GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% FINES	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% FINES	
GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, MORE THAN 12% FINES	
GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, MORE THAN 12% FINES	
SW	WELL-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% FINES	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% FINES	
SM	SILTY SANDS, SAND - SILT MIXTURES, MORE THAN 12% FINES	
SC	CLAYEY SANDS, SAND-CLAY MIXTURES, MORE THAN 12% FINES	

NOTE:
Coarse grained soils receive dual symbols if they contain 5 to 12% fines (e.g. SW-SM, GP-GC, etc.)

FINE-GRAINED SOILS

MORE THAN 50% FINES*

GROUP SYMBOLS	DESCRIPTION	MAJOR DIVISIONS
ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS	SILTS AND CLAYS Liquid limit less than 50
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
OL	ORGANIC SILTS OR ORGANIC SILTY-CLAYS OF LOW PLASTICITY	
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS	SILTS AND CLAYS Liquid Limit more than 50
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY	
PT	PEAT, MUCK, AND OTHER HIGHLY ORGANIC SOILS	HIGHLY ORGANIC SOILS

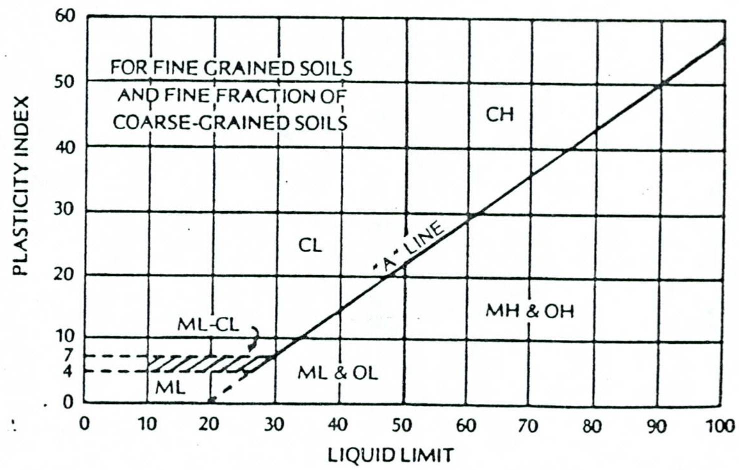
NOTE:
Fine grained soils receive dual symbols if their limits plot in the hatched zone on the Plasticity Chart (ML-CL)

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	ABOVE 12 in.
COBBLES	3 in. to 12 in.
GRAVEL	No. 4 to 3 in.
Coarse	¾ in. to 3 in.
Fine	No. 4 to ¾ in.
SAND	No. 200 to No. 4
Coarse	No. 10 to No. 4
Medium	No. 40 to No. 10
Fine	No. 200 to No. 40
*FINES (Silt or Clay)	BELOW No. 200

NOTE:
Only sizes smaller than three inches are used to classify soils.

PLASTICITY CHART



SOIL PROPERTIES

Job No. 2155J020-E

Boring No.	Depth, ft.	Soil Class.	Expansion/Compression: Initial Dry Density pcf	Initial Moisture Content, %	Surcharge KSF	+ Expan. - Comp. %	Max. Swell Pressure KSF	Water Soluble Matter, %: Salts	Sulfates	Shear Strength: Test Method	Initial Moisture Content, %	Dry Density pcf	C KSF	Ø Deg.	Consolidation: Initial Void Ratio	Surcharge KSF	Consol. %
1	5 - 8	CH								UC	85.1	94.5	0.21				
2	14 - 17	CH								UC	61.3 (4)	64.2 (1)	0.18				
3	5 - 8	CH								UC	58.1 (4)	63.6 (1)	0.60				

Boring No.	Depth, ft.	Remarks
1	5 - 8	Possible sample disturbance during test preparation.

LEGEND

Shear Strength Test Method
- DS Direct Shear
- DS Direct Shear (saturated)
- UC Unconfined Compression
- UU Unconsolidated Undrained
- CU Consolidated Undrained w/pore press
- CU Consolidated Undrained
- CD Consolidated Drained
- CR Cyclic Consolidated Undrained w/pore press

REMARKS

1. In-situ density.
2. Compacted density (Approx. 95% of ASTM:D698 max. density at moisture content slightly below optimum).
3. Compacted density (Approx. 95% of ASTM:D1557 max. density at moisture content slightly below optimum).
4. In-situ moisture.
5. Submerged to approximate saturation.
6. Consolidation % upon saturation.
7.

PHYSICAL PROPERTIES

Job No. 2155J020-E

Boring No.	Depth, ft.	Soil Class.	Particle Size Distribution, % Passing by wt.					Atterberg Limits		Moisture - Density Rel.			Specific Gravity	Permeability		'R' Value	Remarks
			3"	#4	#10	#40	#200	LL	PI	Dry Density pcf	Optimum Moisture %	Meth.		Dry Density pcf	K Cm/Sec	Corrected 'R'	
1	5 - 8	CH					100	70	52								2
1	8 - 9	CH					96	85	63								2
1	17 -	CH					100	81	63								2
2	14 - 17	CH					98	77	57								2
2	24 - 27	CH					100	78	57								2
3	5 - 8	CH					100	65	48								2
3	13 - 16	CH					90	72	55								2
																	2

Boring No.	Depth, ft.	Comments

REMARKS

Classification/Particle Size
1. Visual
2. Laboratory Tested
3. Minus #200 Only

Moisture Density Relationship
4. Tested ASTM D-689/AASHTO T-99
5. Tested ASTM D-1557/AASHTO T-180
6. Other______

Specific Gravity
7. Minus #4
8. Plus #4

Permeability
9. Constant Head
10. Falling Head

'R' Value
11. Expansion Pressure______psf
12. Exudation Pressure______psi

Note: NP = nonplastic

CONSOLIDATION PROPERTIES OF SOIL

Job No. 2155J020-E

Lab/Invoice No.

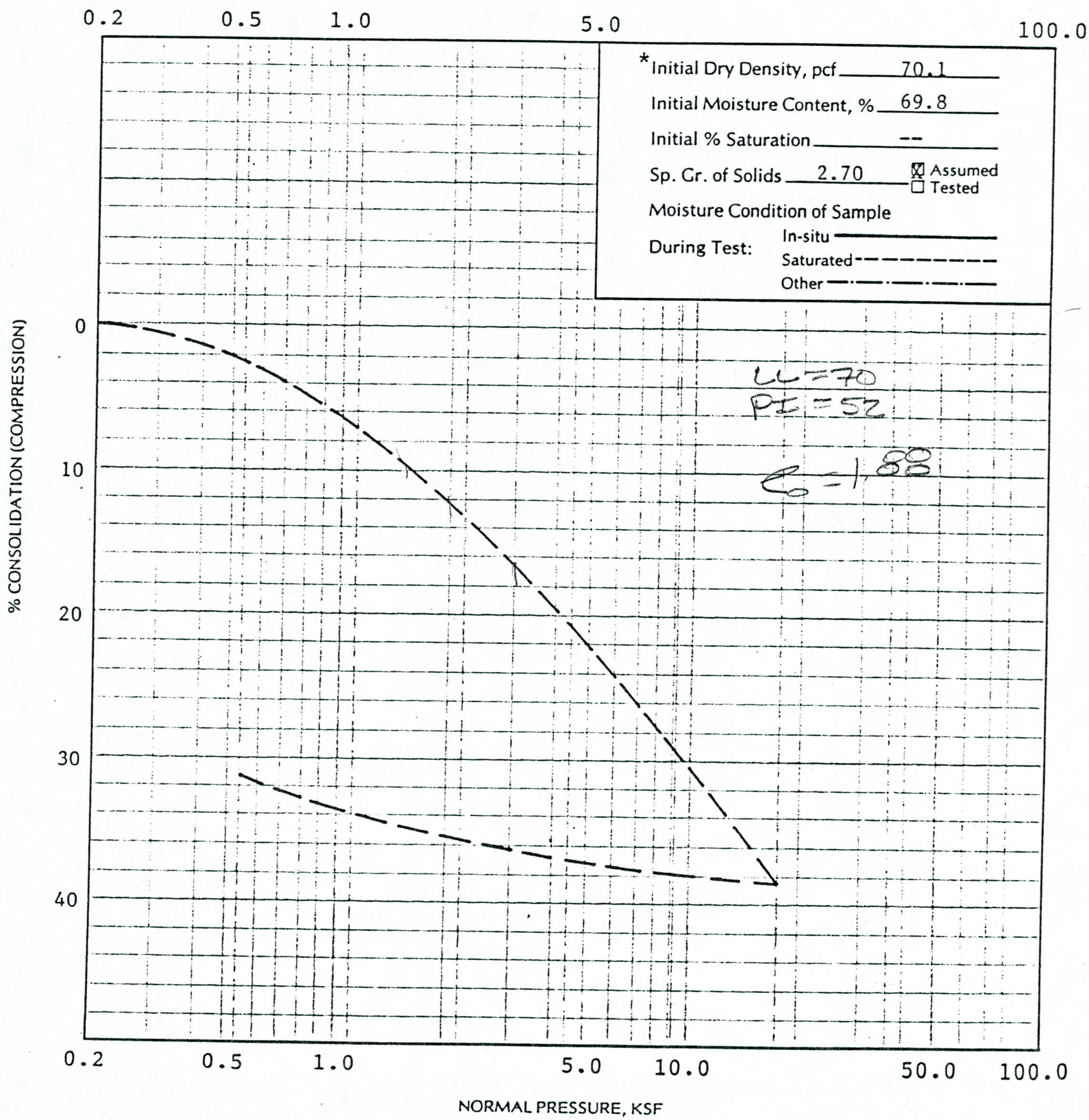
Date of Report

Reviewed By CPW

Type of Material: Clay (CH)

Source of Material: Shelby Tube Sample — ☒ Undisturbed ☐ Remolded ☐ Compacted

Boring: 1 — Depth: 5 - 8 — Test Procedure: ASTM D2435-



*Possible sample disturbance.

CONSOLIDATION PROPERTIES OF SOIL

Job No. 2155J020-E

Lab/Invoice No.

Date of Report

Reviewed By CPW

Type of Material Clay (CH)

Source of Material Shelby Tube Sample ☒ Undisturbed ☐ Remolded ☐ Compacted

Boring 2 Depth 14-17 Test Procedure ASTM D2435-

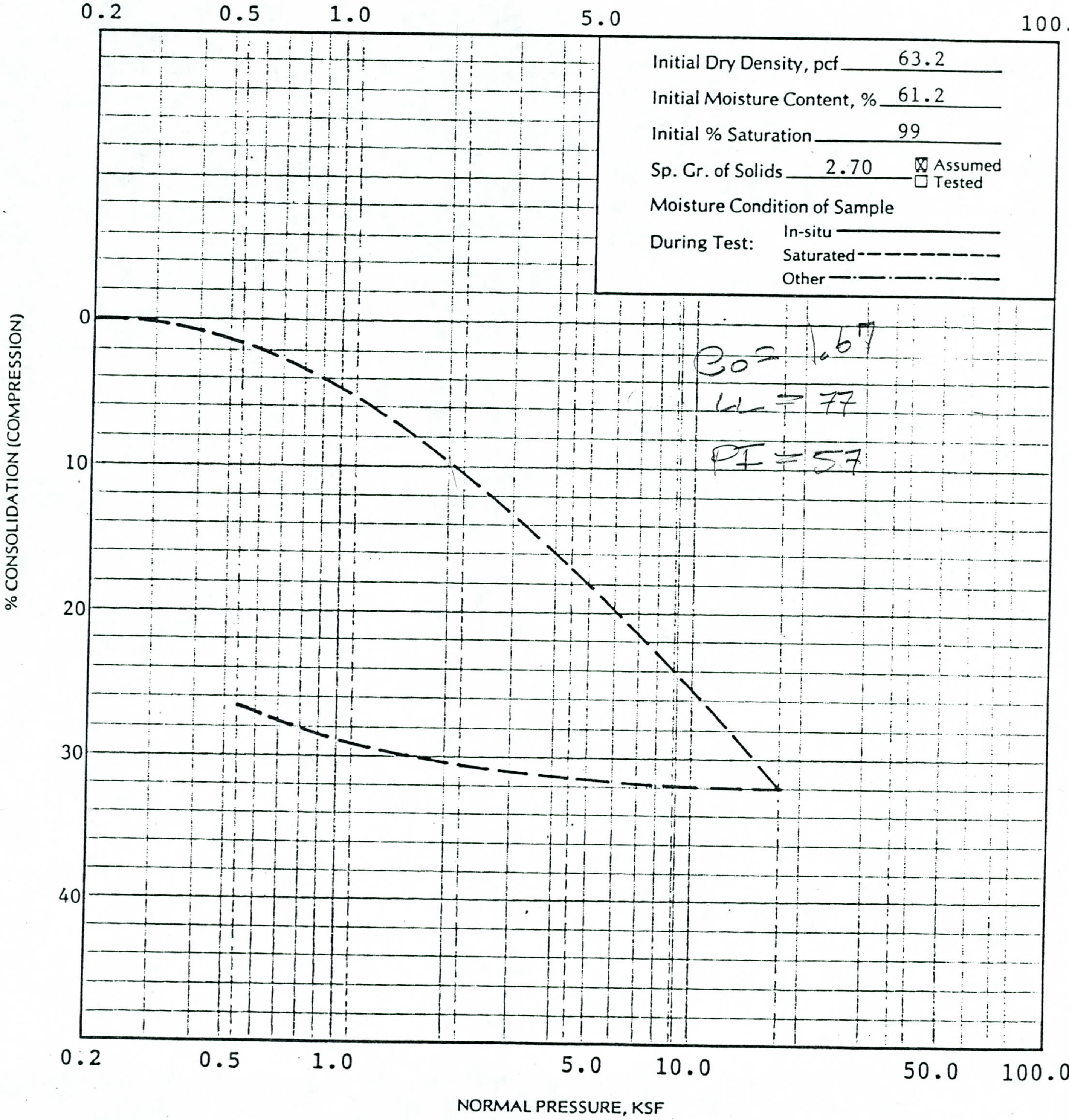
Initial Dry Density, pcf 63.2

Initial Moisture Content, % 61.2

Initial % Saturation 99

Sp. Gr. of Solids 2.70 ☒ Assumed ☐ Tested

Moisture Condition of Sample During Test: In-situ ——— Saturated - - - - Other —·—·—



CONSOLIDATION PROPERTIES OF SOIL

Job No. 2155J020-E

Lab/Invoice No.

Date of Report

Reviewed By CPW

Type of Material Clay (CH)

Source of Material Shelby Tube Sample ☒ Undisturbed ☐ Remolded ☐ Compacted

Boring 3 Depth 5-8 Test Procedure ASTM D2435-

Initial Dry Density, pcf 67.7

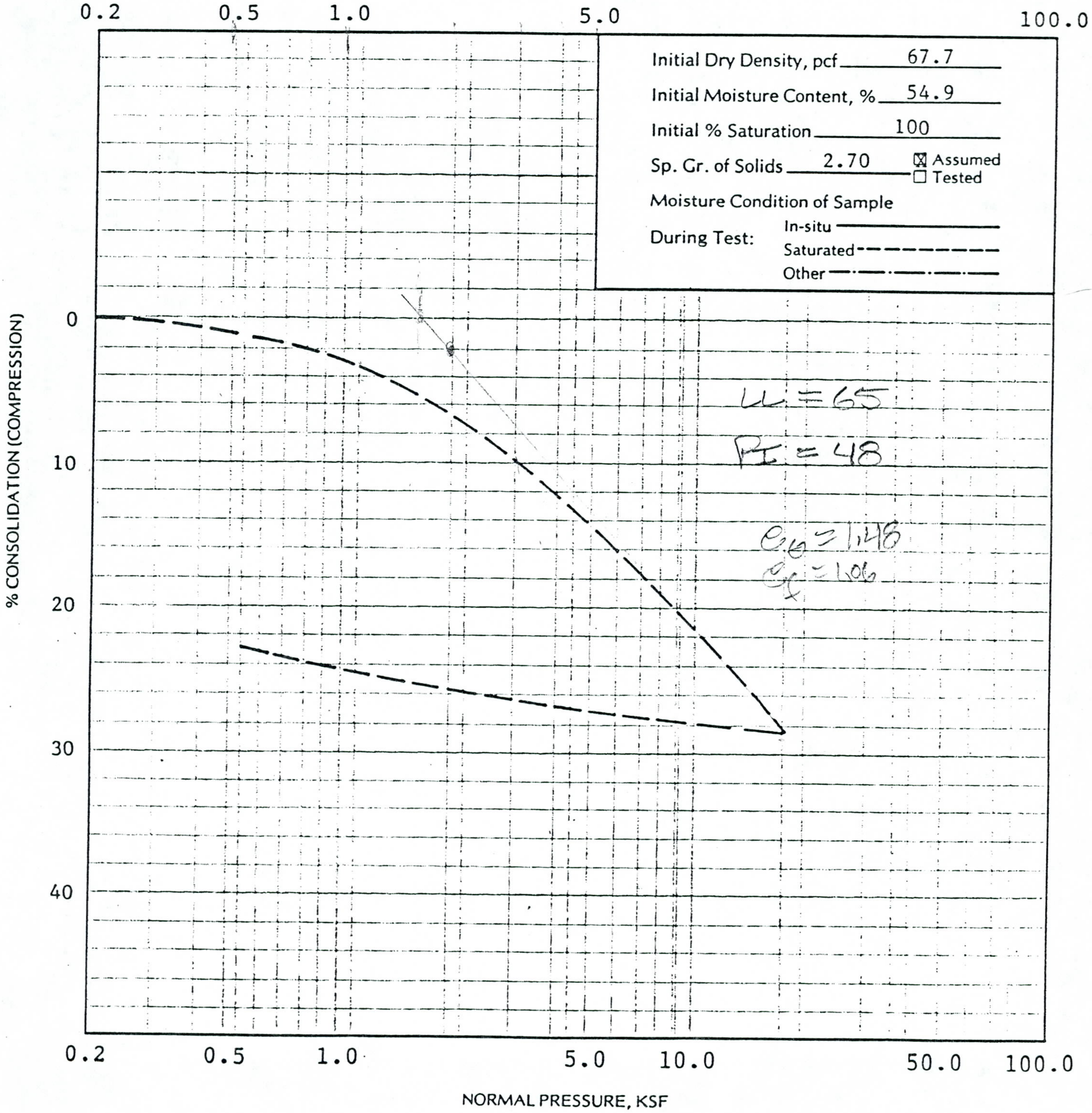
Initial Moisture Content, % 54.9

Initial % Saturation 100

Sp. Gr. of Solids 2.70 ☒ Assumed ☐ Tested

Moisture Condition of Sample

During Test: In-situ ———— Saturated - - - - Other —·—·—



BORING LOG NOTES

"STA" refers to the approximate stationing of the boring along the proposed canal alignment according to centerline control points.

"R" or "L" refers to the approximate lateral offset, right or left respectively, from the centerline of the proposed canal alignment.

"ELEVATION" refers to ground surface elevation at the boring location relative to the indicated "DATUM" established by measurements with a hand level from a project bench mark.

"TYPE/SIZE BORING" refers to the exploratory equipment used in the boring wherein HSA = hollow-stem auger.

"C" in "Blows/Foot" refers to the number of blows of a 140-pound weight, dropped 30 inches, required to advance an AW rod tipped with a two-inch-outside-diameter disk a distance of 1 foot. Refusal to penetration is considered more than 100 blows per foot.

"N" in "Blows/Foot" refers to the number of blows of a 140-pound weight, dropped 30 inches, required to advance a two-inch-outside-diameter split-barrel sampler a distance of 1 foot, Standard Penetration Test (ASTM D1586). Refusal to penetration is defined as more than 100 blows per foot.

"R" in "Blows/Foot" refers to the number of blows of a 140-pound weight, dropped 30 inches, required to advance a 2.42-inch-inside-diameter ring sampler a distance of 1 foot. Refusal to penetration is considered more than 50 blows per foot.

"Sample Type" refers to the form of sample recovery, in which N = Split-barrel sample, R = Ring sample, and T = Thin-walled tube sample.

"Dry Density, pcf" refers to the laboratory-determined dry density in pounds per cubic foot. The symbol "NR" indicates that no sample was recovered. The symbol "*" indicates that determination of dry density was not possible.

"Water Content, %" refers to the laboratory-determined moisture content in percent (ASTM D2216).

"Unified Classification" refers to the soil type as defined by "Method of Soil Classification". The soils were classified visually in the field and, where appropriate, classifications were modified by visual examination of samples in the laboratory and/or by appropriate tests.

BORING LOG NOTES (Cont'd)

These notes and boring logs are intended for use in conjunction with the purposes of our services defined in the text. Boring log data should not be construed as part of the construction plans nor as defining construction conditions.

Boring logs depict our interpretations of subsurface conditions at the locations and on the date(s) noted. Variations in subsurface conditions and soil characteristics may occur between borings. Groundwater levels may fluctuate due to seasonal variations and other factors.

In general, terms and symbols on the boring logs conform with "Standard Definitions of Terms and Symbols Relating to Soil and Rock Mechanics" (ASTM D653).

LOG OF BORING NO. 1 Sta. 23+60 - 34' Rt.

Project: RID Canal Embankment Job No. 2155J020-E

Elevation: 1009' Datum: Centerline Control

Type/Size Boring: HSA/7" Rig Type: CME 45

Groundwater Conditions: None Encountered Date: 9/16/85

	Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
1009							GP	SANDY GRAVEL AND COBBLES; some silt, trace clay, remnants of aggregate stockpile, brown to grey, medium dense, moist to wet
10055	5						CH	CLAY; some silt, trace fine sand, reddish brown to grey, soft to very soft, contains plant fibers, near saturation
				T	94.5	85.1		
			3	R	*			
	10							
	15		6	R	64.6	53.5		
				T				
	20							
984	25							Stopped @ 25.0 feet
	30							

LOG OF BORING NO. 2 Sta. 22+60 - 23' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1007.5' Datum Centerline Control

Type/Size Boring HSA/7" Rig Type CME 45

Groundwater Conditions None Encountered Date 9/17/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
1007.5 0–7.5						GP	SANDY GRAVEL AND COBBLES; some silt, trace clay, remnants of aggregate stockpile, brown to grey, medium dense, moist to wet
7.5–27						CH	CLAY; some silt, trace fine sand, reddish brown to grey, soft to very soft, contains plant fibers, near saturation
~11			T	62.6	47.5		
~16			T	64.2	61.3		
~26			T	56.4	63.2		
27 (980.5)							Stopped @ 27.0 feet

189

LOG OF BORING NO. 3 Sta. 21+57 - 19' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1005.5' Datum Centerline Control

Type/Size Boring HSA/7" Rig Type CME 75

Groundwater Conditions None Encountered Date 9/17/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
1005.5						GP	SANDY GRAVEL AND COBBLES; some silt, trace clay, remnants of aggregate stockpile, brown to grey, medium dense, moist to wet
5 (1000.5)						CH	CLAY; some silt, trace fine sand, reddish brown to grey, soft to very soft, contains plant fibers, near saturation
			T	63.6	58.1		
10							
		4	N	NR			
15			T	62.7	62.4		
20			T	63.0	55.9		
25							
30							

LOG OF BORING NO. 3 CONTINUED

Project RID Canal Embankment Job No. 2155J020-E

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Moisture Content, %	Unified Classification	Description
975 31		39	N	*		CH	CLAY; (Cont'd)
35						GW	SAND, GRAVEL AND COBBLES; trace silt and clay, brown, dense, wet
966.5 40							Refusal @ 39.0 feet on cobbles
45							
50							
55							
60							

LOG OF BORING NO. 4 Sta. 24+00 - 12' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1011.0' Datum Centerline Control

Type/Size Boring -- Rig Type --

Groundwater Conditions None Encountered Date 10/1/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	41						CONTINUOUS PENETRATION ONLY
	86						
	118						
	78						
5	44						
	33						
	30						
	17						
	6						
10	7						
	7						
	12						
	26						
	38						
15	39						
	30						
	37						
	47						
20							
25							
30							

LOG OF BORING NO. 5 Sta. 24+00 - 37' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1010.7' Datum Centerline Control

Type/Size Boring -- Rig Type --

Groundwater Conditions None Encountered Date 10/1/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	10						CONTINUOUS PENETRATION ONLY
	20						
	26						
	28						
5	84						
	75						
	245						
	137						
	103						
10	90						
	80						
	55						
	55						
	50						
15	30						
	35						
	22						
	22						
	21						
20	22						
	24						
	13						
	9						
	12						
25	11						
30							

LOG OF BORING NO. 6 Sta. 23+70 - 33' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1008.6' Datum Centerline Control

Type/Size Boring -- Rig Type --

Groundwater Conditions None Encountered Date 10/1/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	9						CONTINUOUS PENETRATION ONLY
	17						
	14						
	14						
5	20						
	17						
	16						
	17						
	19						
10	15						
	6						
	6						
	7						
	7						
15	6						
	6						
	5						
	6						
	6						
20	6						
	6						
	7						
	6						
	8						
25	8						
30							

LOG OF BORING NO. 7 Sta. 23+76 - 34' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1009.5' Datum Centerline Control

Type/Size Boring -- Rig Type --

Groundwater Conditions None Encountered Date 10/1/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	2						CONTINUOUS PENETRATION ONLY
	5						
	16						
	20						
5	152						
	240						
	64						
	45						
	51						
10	34						
	24						
	19						
	7						
	6						
15	8						
	8						
	9						
	9						
	9						
20	9						
25							
30							

LOG OF BORING NO. 8 Sta. 23+90 - 34' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1010.0' Datum Centerline Control

Type/Size Boring -- Rig Type --

Groundwater Conditions None Encountered Date 10/1/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	3						CONTINUOUS PENETRATION ONLY
	7						
	17						
	27						
5	252						
10							
15							
20							
25							
30							

LOG OF BORING NO. 8A Sta. 23+88 - 34' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1010.0' Datum Centerline Control

Type/Size Boring -- Rig Type --

Groundwater Conditions None Encountered Date 10/1/85

Depth, feet	Blows/Foot		Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	C	N/R					
	4						CONTINUOUS PENETRATION ONLY
	10						
	22						
	55						
5	203						
	400						
10							
15							
20							
25							
30							

LOG OF BORING NO. 9 Sta. 23+70 - 5' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1008.4' Datum Centerline Control

Type/Size Boring -- Rig Type --

Groundwater Conditions None Encountered Date 10/1/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	30						CONTINUOUS PENETRATION ONLY
	64						
	100						
	85						
5	39						
	8						
	5						
	6						
	8						
10	9						
	11						
	23						
	27						
	29						
15	31						
20							
25							
30							

LOG OF BORING NO. 10 Sta. 23+76 - 5' Rt.

Project RID Canal Embankment Job No. 2155J020-E

Elevation 1008.9' Datum Centerline Control

Type/Size Boring -- Rig Type --

Groundwater Conditions None Encountered Date 10/1/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	24						CONTINUOUS PENETRATION ONLY
	75						
	240						
5							
10							
15							
20							
25							
30							

LOG OF BORING NO. 11 Sta. 23+50 - 20' Rt.

Project: RID Canal Embankment Job No. 2155J020-E

Elevation: 1007.7' Datum: Centerline Control

Type/Size Boring: -- Rig Type: --

Groundwater Conditions: None Encountered Date: 10/1/85

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	8						CONTINUOUS PENETRATION ONLY
	27						
	20						
	20						
5	19						
	27						
	13						
	8						
	3						
10	3						
	4						
	4						
	5						
	4						
15	3						
	5						
	6						
	6						
	8						
20	7						
	9						
	9						
	10						
	9						
25	10						
	11						
	13						
	14						
	19						
30	40						

LOG OF BORING NO. 12 Sta. 23+50 - Centerline

Project: RID Canal Embankment Job No. 2155J020-E

Elevation: 1007.5' Datum: Centerline Control

Type/Size Boring: -- Rig Type: --

Groundwater Conditions: None Encountered Date: 10/1/85

Depth, feet	Blows/Foot		Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	C	N/R					
	34						CONTINUOUS PENETRATION ONLY
	55						
	105						
	64						
5	28						
	19						
	16						
	13						
	8						
10	9						
	16						
	33						
	21						
	17						
15	34						
	41						
	33						
	25						
	75						
20	48						
25							
30							

GEOTECHNICAL ENGINEERING INVESTIGATION

on

Roosevelt Irrigation District Canal
Station 20+50 to Station 24+00
Phoenix, Arizona

for

Simons, Li & Associates
120 West Broadway, Suite 170
Tucson, Arizona 85701

524 North Sixth Avenue
Tucson, Arizona 85705
(602) 623-7774

DESERT EARTH ENGINEERING

November 22, 1985
84-210B

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10296
ROBERT L.
SOGGE
Date Signed 11/22
ARIZONA, U.S.A.

CONSULTING GEOTECHNICAL ENGINEERS

November 22, 1985
84-210B

Simons, Li & Associates
120 West Broadway, Suite 170
Tucson, Arizona 85701

ATTN: John Lynch

RE: Geotechnical Engineering Design Investigation Relating to Options for Roosevelt Irrigation District Canal Station 20+50 to Station 24+00 Phoenix, Arizona

Gentlemen:

At your request Desert Earth Engineering has reviewed several proposed solutions to the problem of placing the proposed Roosevelt Irrigation District canal over the clay filled waste pit near the Agua Fria River in Phoenix. The options reviewed include the following: 1) realigning the canal, 2) removal of the clay and replacing it with engineered fill, 3) a pier and grade beam system to support the canal, 4) a pier system to support an elevated flume over the stretch of the alignment affected by the waste pit, 5) stablizing the soil by use of geotechnical fabric or sand drains, 6) and a combination of partial realignment of the canal and the constructuion of a retaining wall to limit the width of the canal and the attendant removal of the clay material.

On November 20, 1985 Desert Earth Engineering drilled six boreholes on the site. These borings were used to establish the depth of the clay layer and the approximate configuration of the waste pit side slopes, as shown in Fig. 1. These boring confirm the findings of Western Technologies Inc. report #21555020-E. Boring logs and results of further lab testing will be supplied at a later date.

DESERT EARTH ENGINEERING

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Based on the results of our borings and analysis, it is the opinion of our soils engineers that extreme difficulty may be anticipated for the proposed solutions which involve removal of the clay material from the pit area below the canal. This is the result of the extremely weak nature of the material, the depth of the deposit, and the large lateral extent of excavation required to produce stable conditions both during construction and afterwards. It will also be extremely difficult to operate heavy equipment in this weak, saturated material. Therefore, it is the recommendation of Desert Earth Engineering that the options which would allow the material to remain in place be given priority over those which would require removal of the clay.

It has been a pleasure being associated with you on this project. If we may be of further assistance to you, please call.

Submitted By:

Donald Tharp, B.S.C.E.

Ralph Pattison, B.S.C.E.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
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ROBERT L.
SOGGE
Date Signed 11/22
ARIZONA, U.S.A.

R. L. Sogge, P.E.

RLS/ajt

Copies: (2) Addressee

1. REALIGN CANAL

Due to the presence of the clay material in this pit, the canal may be realigned so as to avoid the area. Based upon the current configuration this alternative would require moving the centerline of the channel approximately 80 to 90 feet south of the currently envisioned alignment in the area of the clay-filled waste pit.

Due to the highly variable soil conditions encountered across this entire project, if the realignment option is selected, it would be inadvisable to generalize the soil conditions under any new alignment from those encountered in our test borings. Any new alignment which may be selected should be drilled prior to design and construction in order to accurately ascertain soil conditions prevalent along the route chosen.

2. REMOVE CLAY AND REPLACE WITH ENGINEERED FILL

A second alternative is to remove the saturated clay and replace it with engineered fill. If this alternative is selected extreme difficulty may be anticipated during excavation. Due to the extremely weak nature of the clay the angle of excavation should not exceed 1 vertical to 4 horizontal. The anticipated depth of excavation is in excess of 30 feet in some areas of this project. The splay angle below the toe of the slope will be dependent on the material selected as fill; however a 1 horizontal to 1 vertical splay slope may be used for preliminary design Based on these dimensions, and the currently proposed canal cross section and alignment the lateral extent of removal would be approximately 265 feet at the top and 145 feet at the bottom.

Our analysis of this option was based on an average value for cohesion obtained from unconfined compressive strength testing performed by Western Technologies Inc. The results of these tests also indicate that weaker spots are present in some areas of the pit. In these weaker areas even greater excavation difficulties may be anticipated. The stable excavation slope in these areas is 6 horizontal to 1 vertical which translates to a top width of excavation equal to 325 feet.

3. CONSTRUCT THE CANAL ALONG THE CURRENTLY PROPOSED ALIGNMENT WITH THE FLUME INDEPENDENTLY SUPPORTED

A third alternative is to construct the canal along the currently proposed alignment while supporting the flume independently on drilled, cast-in-place, concrete shafts. Assuming a system of two shafts per pier groups with each group spaced 10 feet on center, the required depth of embedment for a 2 foot diameter shaft would be 20 feet, (bottom elevation = 993) for the shafts on the south side of the flume and 23 feet, (bottom elevation = 969) for the shafts on the north side of the flume. These shafts could then be connected by a grade beam to support the flume (see Fig. 2A).

Alternatively a single 3-foot-diameter shaft may be used directly beneath the center of the flume. Assuming a 10-foot, on-center spacing, the required depth of embedment would be 22 feet, corresponding to a bottom elevation of 967 (see Fig. 2B). This alternative would not require a grade beam. All embedment depths are measured from the lowest adjacent granular material grade. The clay material is not to be used in determining embedment depth.

It should be noted that if alternative 3 is chosen the flume itself will need to be designed so as to be structurally sound without soil support since considerable settlement will occur in the clay material supporting the canal fill.

4. CONSTRUCT AN ELEVATED FLUME SUPPORTED BY SHAFTS ALONG THE CURRENTLY PROPOSED ALIGNMENT

A fourth alternative is to construct the flume along this stretch. This would result in an elevated flume along this section. The flume could then be supported on single, 3 foot diameter, drilled, cast-in-place, concrete shafts (see Fig. 3). Based upon 10-foot, on-center spacing for these shafts the required depth of embedment would be 24 feet which corresponds to a bottom elevation of approximately 969. Once again if alternative 4 is chosen the depth of embedment is to be measured from the lowest adjacent granular material grade. The depth of embedment for this option reflects vertical capacity only. A lateral load analysis should be performed prior to final design.

5. STABILIZATION OF SOIL USING GEOTEXTILE FABRIC OR SAND DRAINS

Geotextile fabric by itself will not be sufficient to stabilize the clay. This method was recommended for the portion of the alignment passing over the waste pits between Station 11+80 to Station 20+60 because it was beneficial to maintain separation between the embankment material and the quick-settling waste pit material. For the portion of the alignment from Station 20+60 to Station 24+00, the waste pit material is a slow settling clay; geotextile fabric will not prevent settlement of this clay. Nevertheless, fabric should be used with any alternative that involves placement of fill over the clay layer since it will be beneficial to maintain separation between the layers.

Geotextile reinforcement could also be used in soil below the embankment if the clay were removed. If clay removal is a preferred option, then use of reinforcing fabric can be analyzed further. However, for reasons given elsewhere in this report, clay removal is not recommended.

Stabilizing the soil through placement of sand drains is an alternative worthy of consideration. If these drains were constructed as soon as possible, and a sufficient overburden placed above the clay layer, 90% consolidation or better could be achieved within a year. This would enable construction of the embanked channel to proceed as originally intended, without modification of the cross-section or the alignment.

Tentatively, sand drains could be spaced 10 feet apart over that portion of the clay that will support the embanked channel. It would be feasible to use 1-foot-diameter or even 6 5/8-inch-diameter drilled shafts. The latter diameter has the advantage of being easily drilled using standard hollow-stem, continuous-flight auger, which would make drilling and placement of the sand easily manageable.

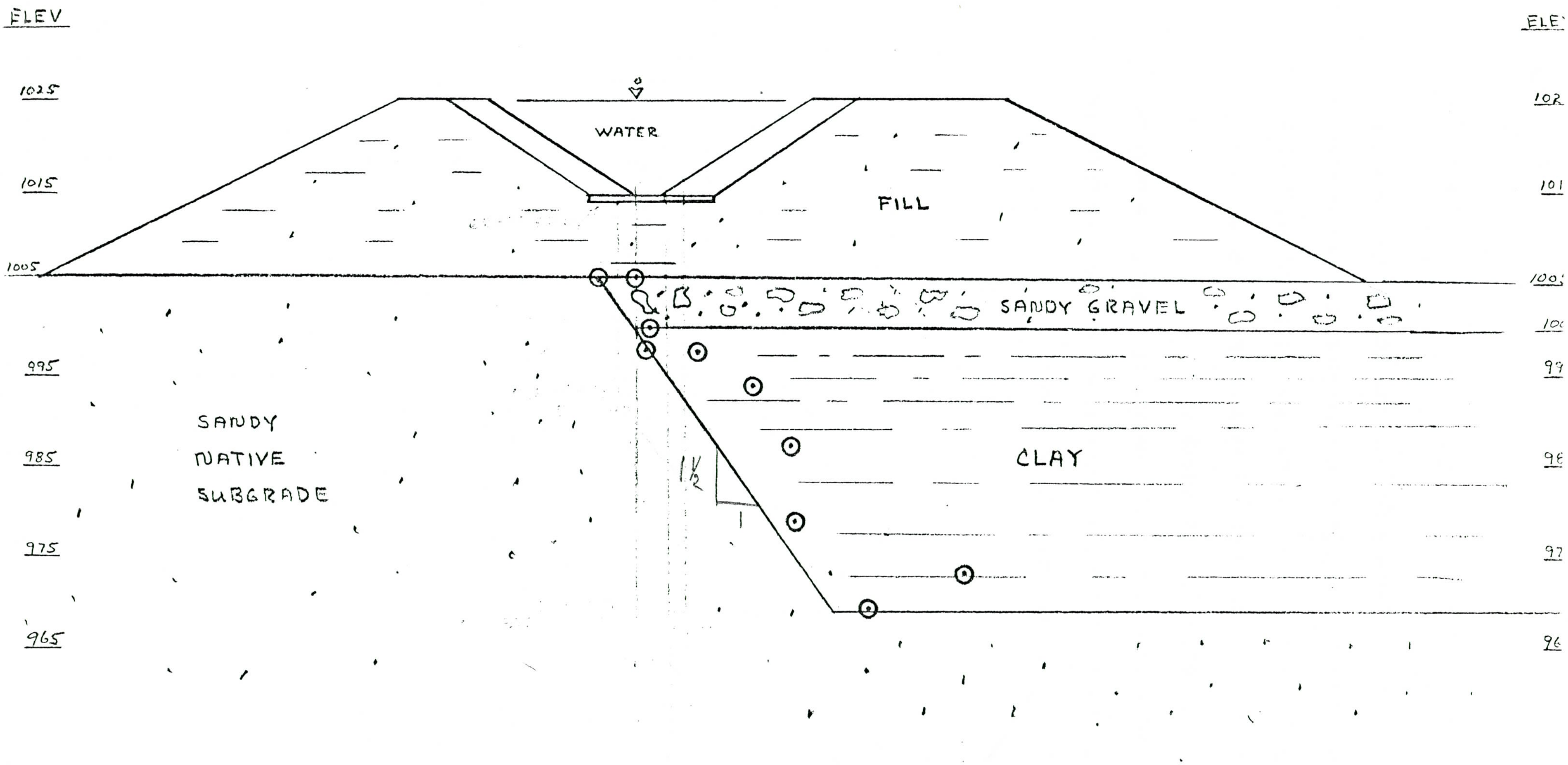
Actual spacing of sand drains would depend on measurable settlement taking place during early stages of the operation. If this settlement were occurring too slowly, it could be accelerated through placement of additional drains. These could be readily drilled with the overburden already in place.

6. PARTIALLY REALIGN CHANNEL AND CONSTRUCT RETAINING WALLS ALONG THE SIDES OF THE CANAL

A sixth alternative is to partially realign the channel by shifting the centerline approximately 26 feet to the south and construct retaining walls along the sides of the canal to restrict the canal width and subsequent need for clay removal.

The splay angle of the foundation for this retaining wall would depend on the material selected for fill but a 1:1 horizontal to vertical slope may be assumed for preliminary design. Due to the weak nature of the clay an excavation slope not to exceed 1 vertical to 4 horizontal is recommended during the clay removal. Based on an embedment depth of 5 feet for this wall and a 30 foot depth of excavation to remove the clay, this alternative would result in a top width of 33 feet.

It should be noted that if this alternative is chosen, the problems attendant with the removal of the clay material which have been outlined previously would also be encountered for this option.



⊙ BOTTOM OF CLAY LAYER
PER BORINGS & TRENCHES

FIGURE 1
LOCATION OF BOTTOM
OF CLAY LAYER ENCOUNTERE
IN BORINGS AND TRENCHES

Desert Earth Engineering
consulting geotechnical engineers

Drawn by: DT	Date 11/22/85	Checked by: RYS	Date 11/22/85
Sheet 1 of	Job No. 85-210b	Figure No. 1	

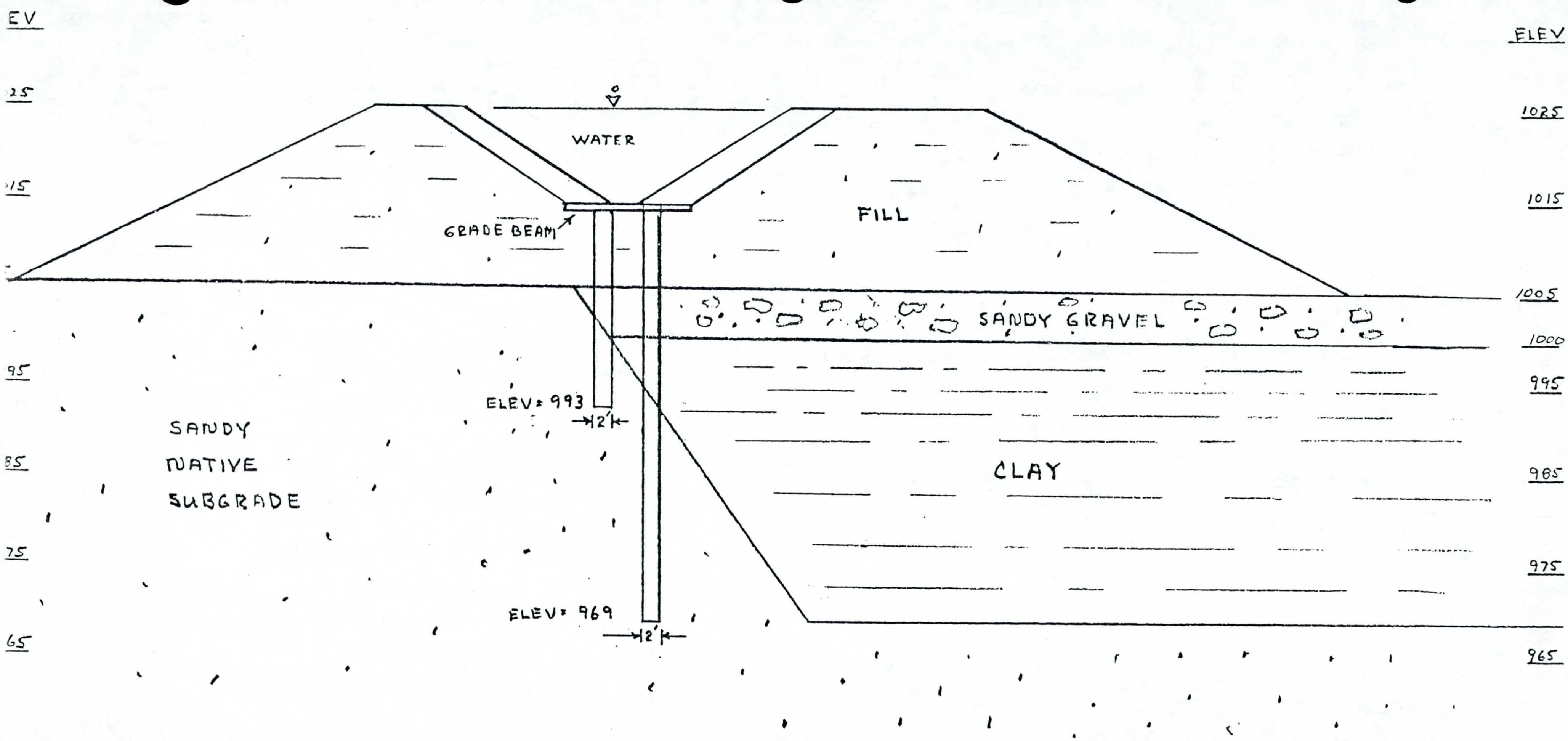


FIGURE 2A
(OPTION 3)
2 FOOT DIAMETER
CAST-IN-PLACE
SHAFTS w/ GRADE BEAM

Desert Earth Engineering consulting geotechnical engineers			
Drawn by: DT	Date 11/22/85	Checked by: RYS	Date 11/22/85
Sheet 1 of 1	Job No. 85-210b	Figure No. 2A	

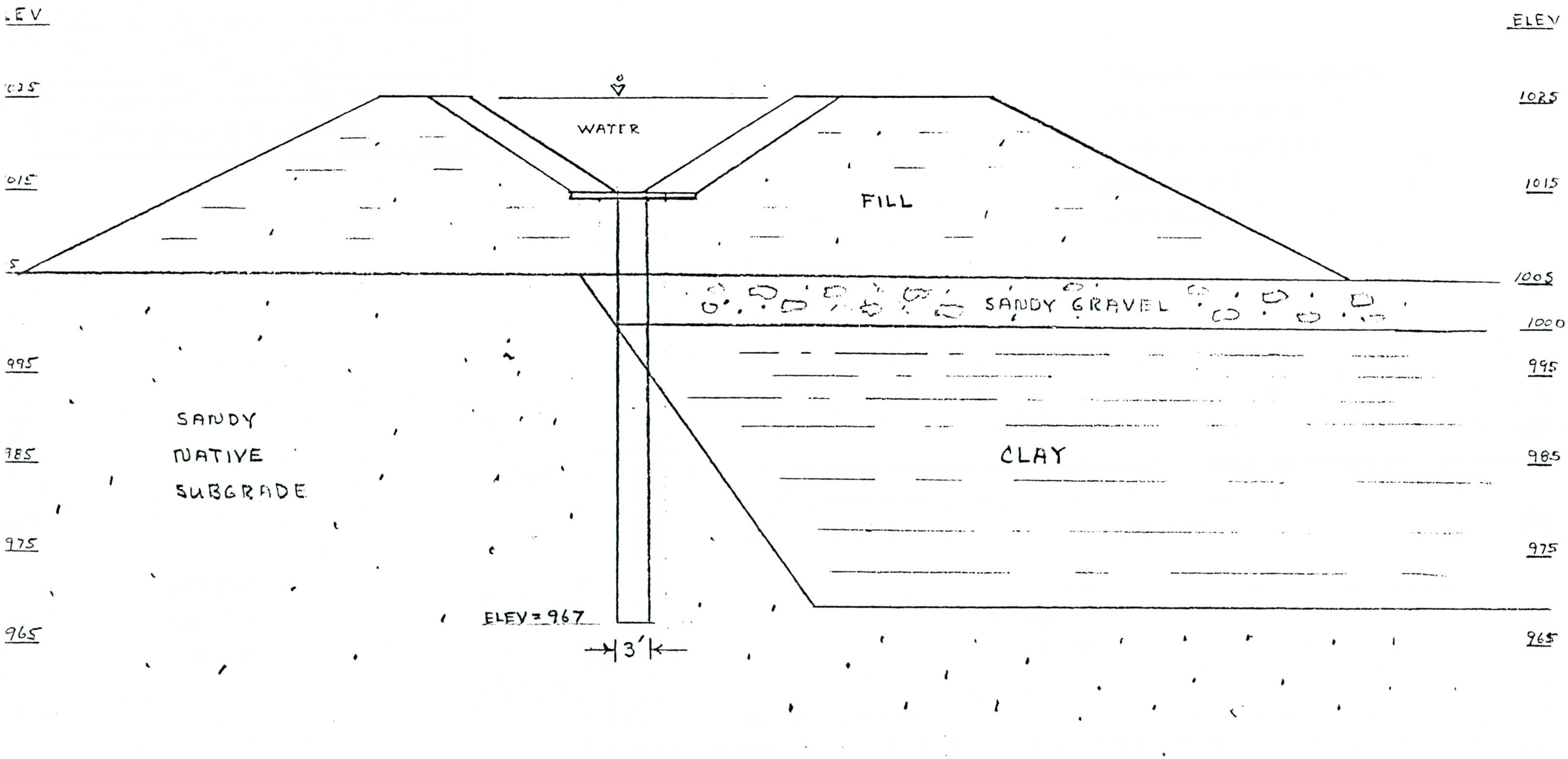


FIGURE 2B
(OPTION 3)
3 FOOT DIAMETER
CAST-IN-PLACE
SHAFTS w/o GRADE BEAM

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Drawn by: DT Date 11/22/85	Checked by: KYL Date 11/22/85	
Sheet 1 of 1	Job No. 85-210b	Figure No. 2B

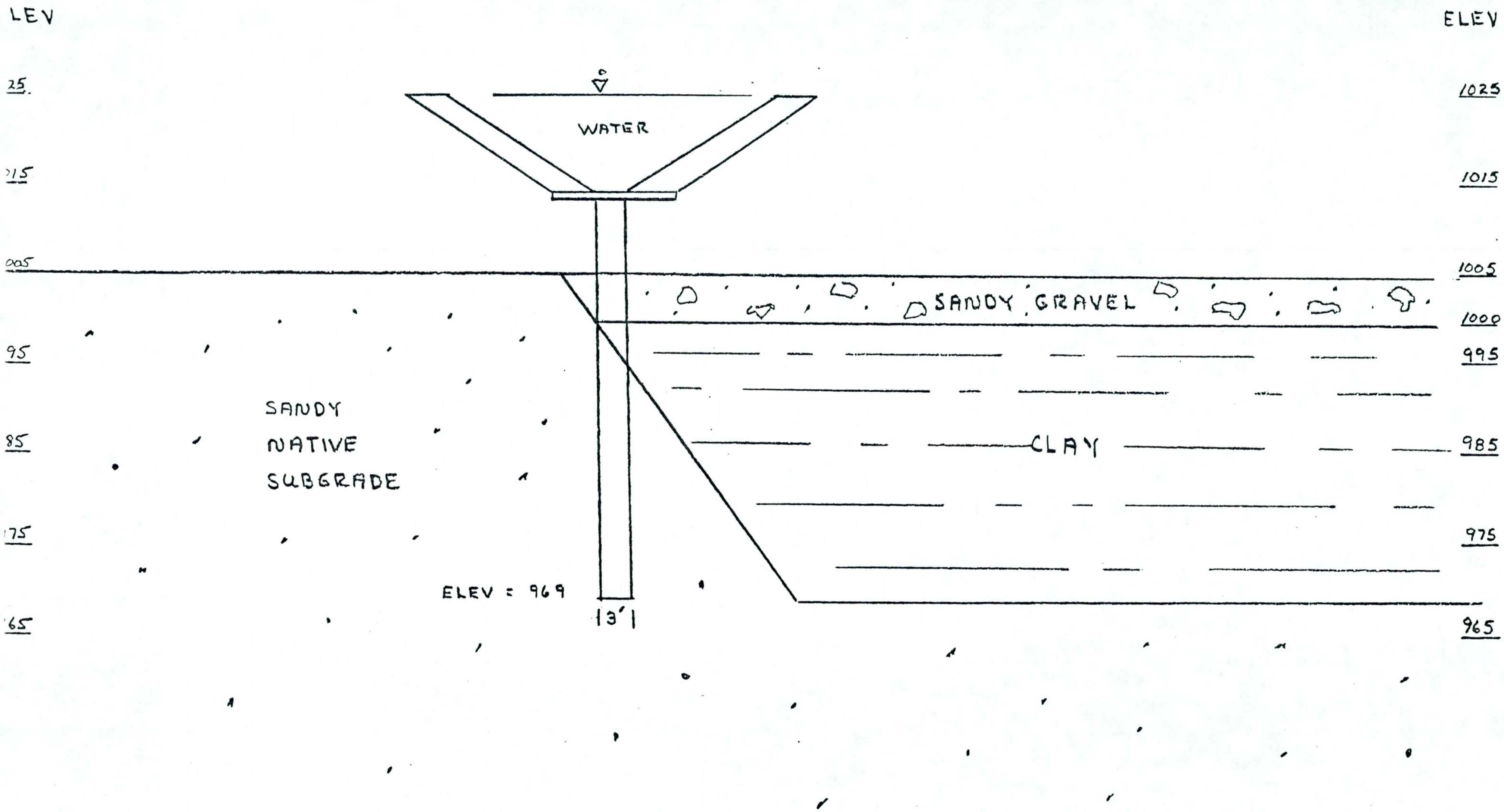

FIGURE 3:
(OPTION 4)
ELEVATED FLUME
SUPPORTED BY 3 FOOT
DIAMETER SHAFTS

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consulting geotechnical engineers

Drawn by: DT	Date 11/22/85	Checked by: RYA	Date 11/22/85
Sheet 1 of 1	Job No. 85-[illegible]	Figure No. 3	

PAZ-MC-Ø8(4)

December 12, 1985
84-210B

Simons, Li & Associates
P.O. Box 1816
3555 Stanford Road
Ft. Collins, Colorado 80522

ATTN: Noel Borman

RE: Sand Drain Spacing Design Parameters
Roosevelt Irrigation District Flume
Phoenix, Arizona

Dear Noel:

The parameters given below are derived from laboratory testing of tailing-pond clays found between Station 21+00 and Station 24+00 along the Roosevelt Irrigation District flume alignment. These parameters may be used for calculation of sand-drain spacing as may be required to accelerate consolidation of the clay strata. Coefficients of Consolidation and Permeability were assumed identical horizontally and vertically. Because of the homogeneity of this clay layer no testing was done to provide an accurate horizontal coefficient of permeability.

A sample calculation is also shown. To calculate the required spacing for different time periods of consolidation, simply substitute the time period (t) and calculate H. H is the drainage path to the sand drain, and should be doubled to indicate spacing between between drains.

DESERT EARTH ENGINEERING

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Compression Index C_c = .66

Coefficient of Consolidation C_v = 0.043 ft^2/day JSW ← USE THIS VALUE FROM CONSOLIDATION [illegible] AND [illegible], DESIGN [illegible], 1:45 PM

Coefficient of Permeability k = 4×10^{-5} ft/day

Time Factor ~~H~~ T JSW, for 90% of primary consolidation = .848

Sample Calculation of Sand-Drain Spacing for 100 Days

$$H^2 = \frac{tC_v}{T} = \frac{100 \text{ days } \cancel{0.022}\ .043 \text{ ft}^2/\text{day}}{.848} = 5.1 \text{ ft}^2$$

$$H = \sqrt{5.1 \text{ ft}^2} = 2.25 \therefore \text{ Spacing is } 4.5 \text{ ft}$$

If you have any questions, please call.

Submitted By:

Ralph Pattison, B.S.C.E.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10296
ROBERT L.
SOGGE
Date Signed 12/12/[illegible]
ARIZONA, U.S.A.

R. L. Sogge, P.E.

RLS/ajt

Copies: (1) Addressee

GEOTECHNICAL ENGINEERING INVESTIGATION

on

Roosevelt Irrigation District Canal
Station 20+50 to Station 24+00
Phoenix, Arizona

for

Simons, Li & Associates
P.O. Box 1816
3555 Stanford Road
Ft. Collins, Colorado

524 North Sixth Avenue
Tucson, Arizona 85705
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DESERT
EARTH
ENGINEERING

Registered Professional Engineer (Civil)
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Sogge

December 18, 1985
84-210B

CONSULTING GEOTECHNICAL ENGINEERS

TABLE OF CONTENTS

Page

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December 18, 1985
84-210B

Simons, Li & Associates
P.O. Box 1816
3555 Stanford Road
Ft. Collins, Colorado 80522

ATTN: Noel Borman

RE: Geotechnical Engineering Report
RID Canal Station 11+00 to Station 24+00
Phoenix, Arizona

Introduction

This report is supplementary to Desert Earth Engineering reports on the Roosevelt Irrigation District Canal dated October 23, 1984 and November 22, 1985. This report primarily discusses the use of vertical drains to accelerate consolidation of the clay-filled waste pit between Stations 21+00 and 24+00. Consideration is also given to problems in the water-filled waste pit between Stations 11+00 and 18+00.

DESERT EARTH ENGINEERING
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Field Investigation

Number of Test Borings: 5

Location: See Site Plan, Fig. 1

Date Drilled: November 20, 1985

Subsurface soils were sampled using a CME-55 drill rig equipped with split spoon and ring samplers. Boring logs containing descriptions of the materials encountered in the subsurface investigation of the site are presented in Appendix A. Continuous penetrometer testing was used to locate the bottom of the waste pit. The penetration resistance values are included on the boring logs.

A soil profile for the project site is presented in Fig. 2. This soil profile is transverse to the canal alignment and shows the approximate side slope of the filled waste pit at Station 21+40. All evidence indicates that this profile is consistent from Station 21+00 to Station 24+00.

Laboratory Test Results

Selected samples taken in the field investigation were tested in the laboratory for grain size, Atterberg Limits, consolidation, and permeability. No test values for cohesion were taken, since the use of vertical drains will preclude the requirement of excavating clay. If clay excavation is needed, cohesion values and slope stability analyses can be provided on request.

Summary of Laboratory Testing Results (Complete Results are given in Appendix B)

Gradation and Plastic Index Tests

Boring No.	Depth (ft)	% Passing #200 Sieve	LL	PI	USCS Symbol
B-13	14-15.5	99	77	51	CH
B-13	34-35	99	80	53	CH

Consolidation Characteristic of Soil Subjected to Saturation

Boring No.	Depth (ft)	Surcharge Pressure (psf)	Consolidation %
B-13	24-25	2800	19.1
B-18	14-15	1400	9.8

Consolidation Parameters

Compression Index $C_c = .66$

Coefficient of Consolidation $C_v = 0.043\ ft^2/day$

Coefficient of Permeability $k = 4 \times 10^{-5}$ ft/day

Time Factor H, for

90% of primary consolidation .848

Conclusions and Recommendations

Our field and laboratory testing shows the waste pit to be filled with a uniform, very plastic, saturated clay. This clay is unconsolidated and, under the expected loading, has a settlement potential of up to 20%. This could lead to total settlements of up to 5 feet for overburden placed above the clay layer.

Because of the low permeability of the clay, settlement will be dependent on dissipation of pore-water pressure over time. Laboratory testing indicates that 90% of primary consolidation will occur 8-12 years after placement of the overburden.

Consolidation of the clay can be greatly accelerated through the use of vertical drains. Effectiveness of these drains will be largely determined by the spacing between them. A graph of percent consolidation versus time for 3 different spacings is given in Fig. 3. This graph is based on radial drainage theory and accounts for both vertical and radical drainage.

Properly spaced vertical drains can reduce the time required for 90 percent of primary consolidation to months or less. Some degree of secondary consolidation will occur simultaneously with primary consolidation. Remaining secondary consolidation may take place over many years, but should not results in significant settlement. If greater than 90% of primary consolidation is achieved, differential settlement of the embankment will be limited to less than 2 inches.

The application of fabric wick drains is especially suitable to this site. The existing sand-gravel-cobble overburden, 5 to 7 feet thick, can be graded out to provide a smooth surface for construction equipment. Wicks should be hammered through this layer and all the way through the clay layer to depths as great as 45 feet below the surface, depending on where firm native soil is encountered.

A system of adequate drainage must also be provided to conduct water from the drains horizontally away from the site. This may be accomplished by the use of French drains filled with permeable soil or with filter fabric. Once this system of drainage is installed, placement of the overburden may begin.

Further problems at the site have been encountered in the area of the water-filled waste pond between Stations 11+00 and 18+00. These problems were discussed in an informal meeting between Ralph Pattison of Desert Earth Engineering and Kent Hamm of Western Technologies Incorporated on December 6, 1985.

Construction problems in the area of the water-filled waste pit seem to be primarily related to the inability to provide an adequate dewatering system for the pond. Furthermore, a soft clay or silt layer at the bottom of the pond, 2-9 feet thick, aggravates the difficulty of bringing construction equipment onto the site. The depth of this layer was determined by WTI personnel by pushing a probe down from a boat in the pond. These probes indicate that, at its maximum depth about 110 feet north of the alignment centerline, this layer starts about 4 feet below the water surface (Elevation 1001.5) and is about 9 feet thick.

According to Mr. Hamm it has been the experience of contractors that, during placement of backfill in the waste pond, this extremely soft layer forms a mud wave which impinges on the property of the gravel yard and poses a danger to their pumps. This is one of the factors that makes the construction of a dike necessary for dewatering the pond difficult.

As an alternative to dewatering we propose that fill be pushed out slowly from the south end of the pond, so that construction equipment may be supported. If necessary, clay pushed out ahead of this fill may then be removed with equipment supported on the fill. When the entire canal right-of-way has been so covered several random test borings should be made to assess the condition of waste material below the pad. If no problems are exposed by these borings treatment of the site may continue as planned and specified in our report of October 23, 1984. If a problem is encountered, the worst case will be the presence of clay. If necessary this can be meliorated through the use of vertical drains as planned for the clay pit between Stations 21+00 and 24+00.

We thank you for working with us on this project. If we can be of further help, please call.

Submitted By:

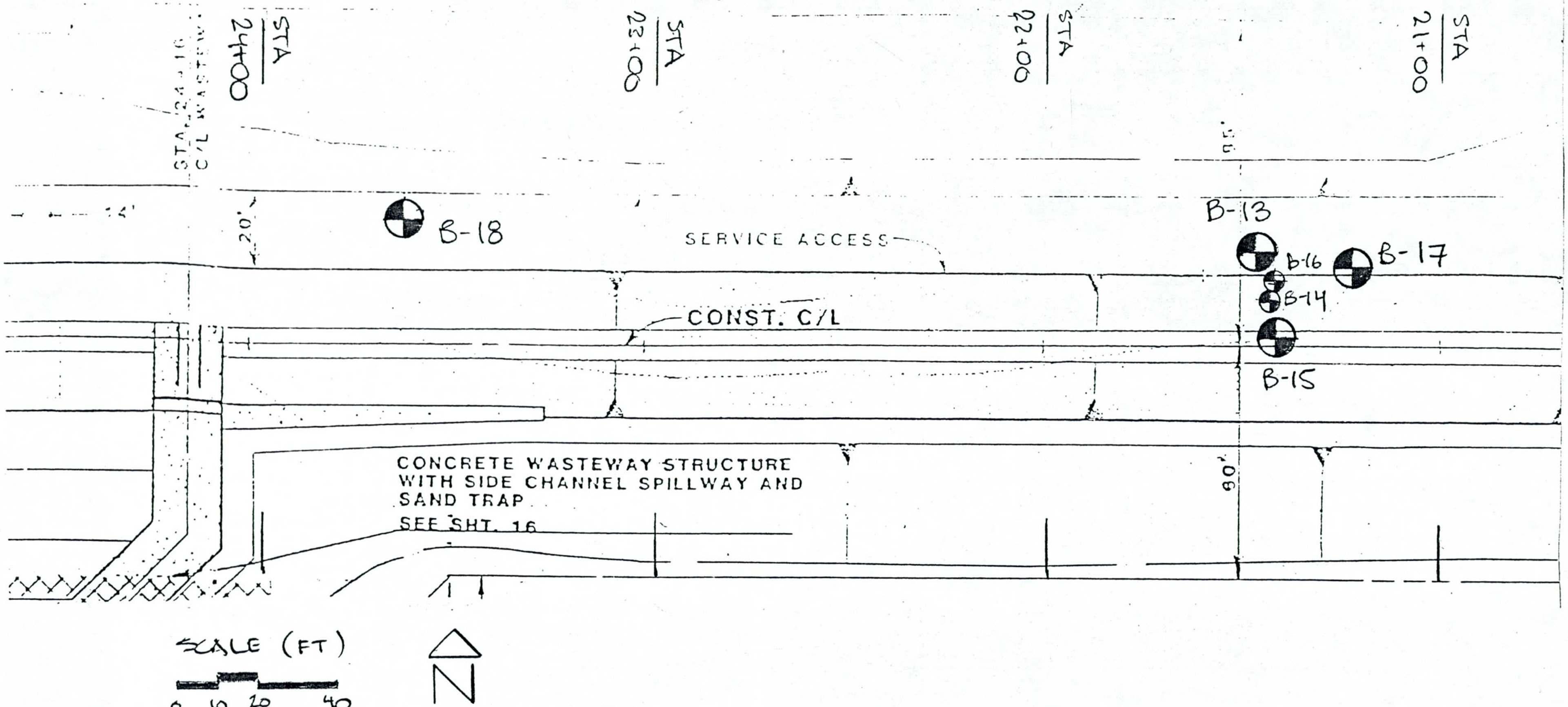
Ralph Pattison, B.S.C.E.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10296
ROBERT L.
SOGGE
Date Signed 12/1/8_
ARIZONA, U.S.A.

R. L. Sogge, P.E.

RLS/ajt

Copies: (1) Addressee
(1) Simons, Li & Associates
Tucson, Arizona
ATTN: John Lynch



BOREHOLE

PENETROMETER PROBE

BOREHOLE LOCATIONS

ROOSEVELT IRRIGATION DISTRICT CANAL

STATION 21+00 TO STATION 24+00

Desert Earth Engineering
consulting geotechnical engineers

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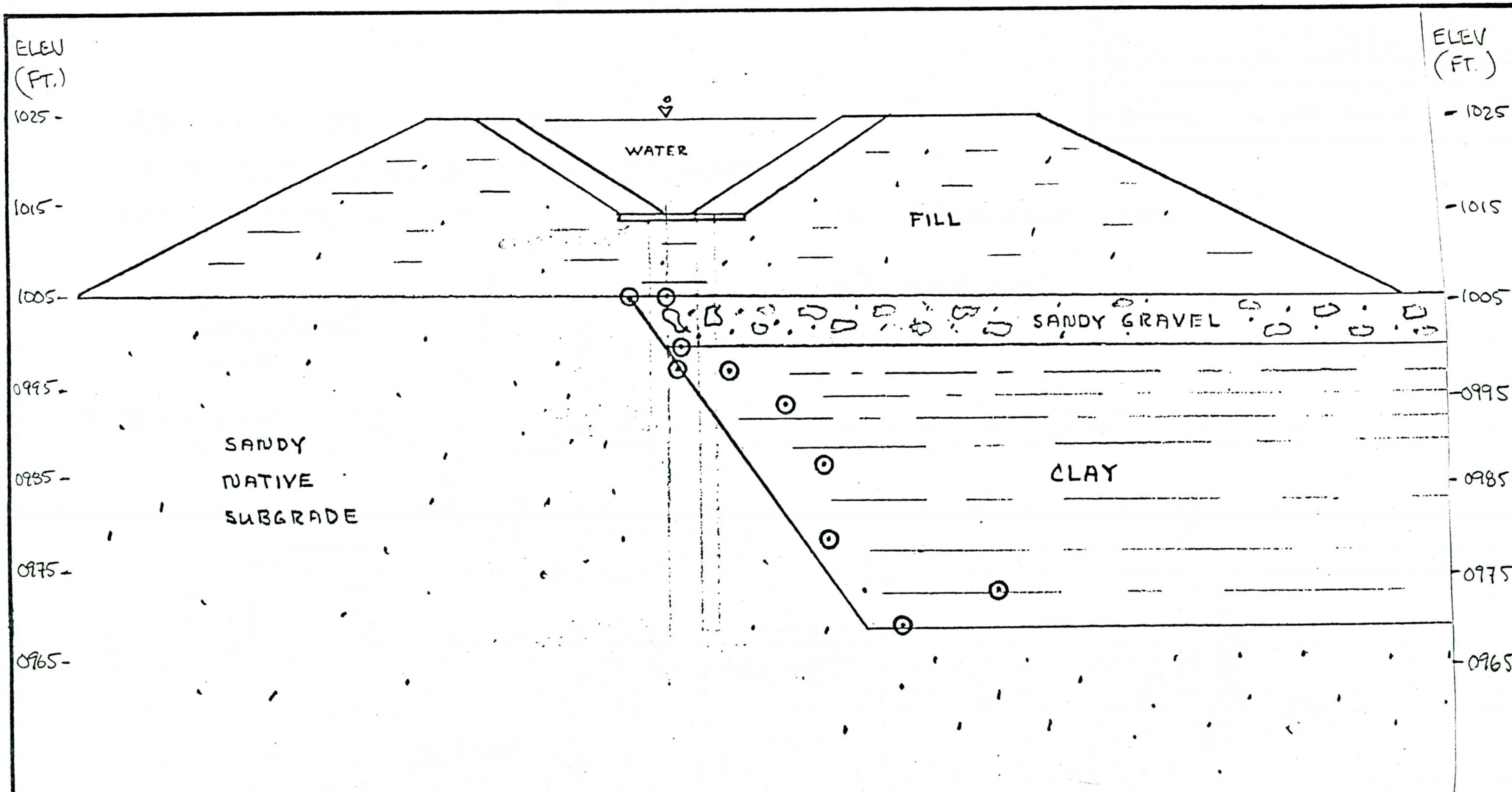
Sheet of	Job No. 84-210B	Figure No. 1	



⊙ BOTTOM OF CLAY LAYER PER BORINGS & TRENCHES

SOIL PROFILE

ROOSEVELT IRRIGATION DISTRICT CANAL

STA 21+00 TO STA 24+00

Desert Earth Engineering
consulting geotechnical engineers

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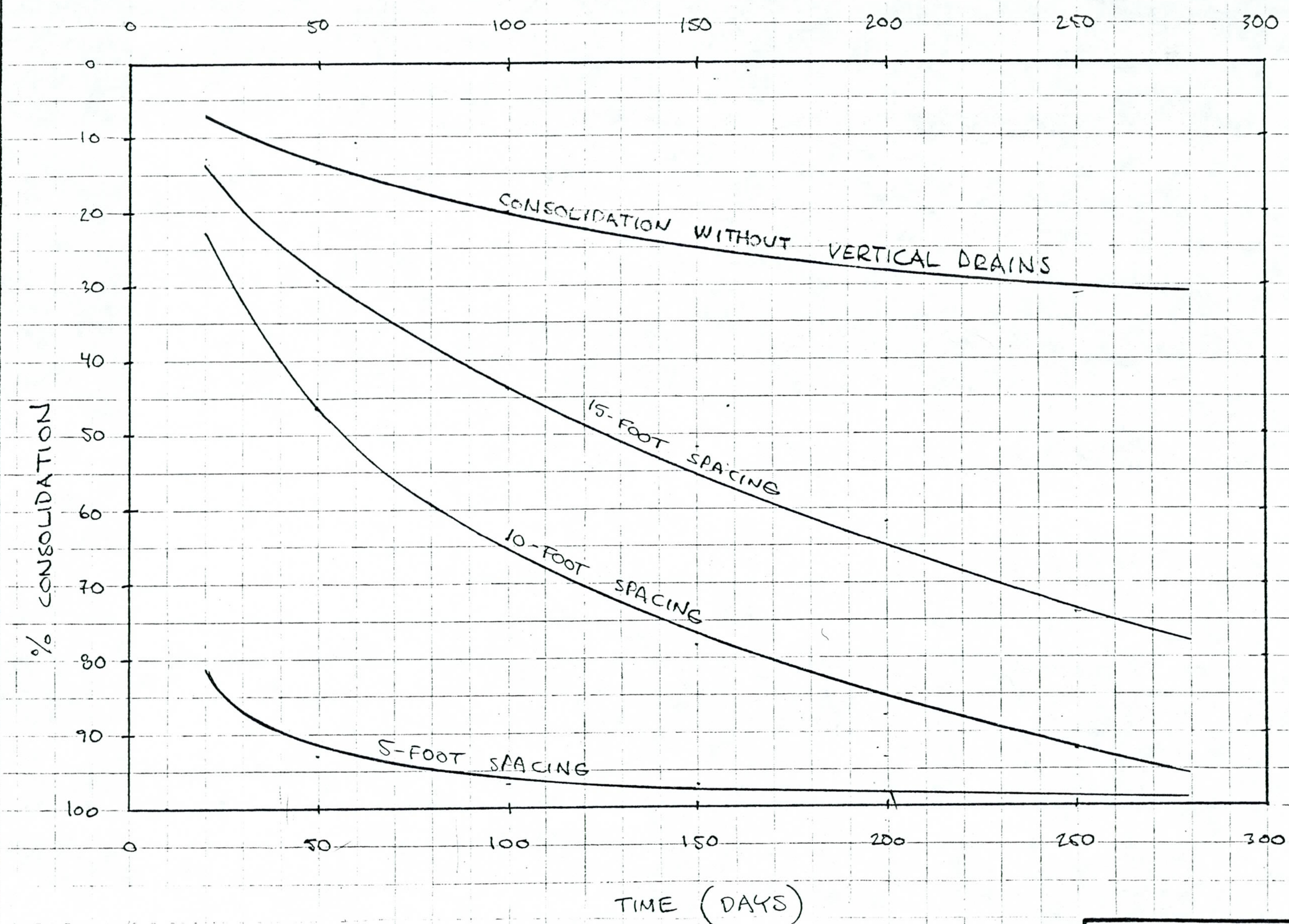
Sheet of	Job No.	Figure No.
	84-210B	2



EFFECT OF VERTICAL DRAINS ON CLAY PIT

ROOSEVELT IRRIGATION DISTRICT CANAL

Desert Earth Engineering
consulting geotechnical engineers

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Sheet of		Job No. 85-210B	Figure No. 3

APPENDIX A

Soil Boring Logs

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION	BORING NO.
84-210B	Simons, Li & Assoc.	RID	B-13

LOCATION OF BORING: Sta. 21+47 24 ft N of CL

DRILLING METHOD & EQUIPMENT: CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers

SAMPLING METHOD: Split-spoon Penetrometer and Ring Sampler

CASING DEPTH:

SHEET: 1 OF 2

ENGINEER: RMP

TIME:

DATE: 20 Nov 85

DATUM:

ELEVATION: 1005

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	SURFACE CONDITIONS
			0			Gray and Brown SAND, ROCK and COBBLES
			1			
			2			
			3			Some clay
			4			
			5			
			6			(CH) Brown CLAY with trace sand; saturated, very plastic, soft
			7			
SS	18/0	9/8/8	8			
			9			
			10			
			11			
			12			
			13			
			14			(CH) CLAY
SS	18/18	1/1/1	15			
			16			
			17			
			18			
			19			
RS	12/0	3/3	20			

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION
84-210B	Simons, Li & Assoc.	RID

LOCATION OF BORING

DRILLING METHOD & EQUIPMENT	BORING NO.
CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers	B-13
	SHEET 2 OF 2
SAMPLING METHOD: Split-spoon Penetrometer and Ring Sampler	ENGINEER: RMP
	TIME
CASING DEPTH	DATE: 20 Nov 85

DATUM

ELEVATION

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	SURFACE CONDITIONS
			20			(CH) Brown CLAY; saturated, very plastic, soft
			21			
			22			
			23			
			24			
RS	12/6	2/1	25			
			26			
			27			
			28			
			29			
SS	18/24	2/2/2	30			
			31			
			32			
			33			
			34			
SS	18/18	4/12/16	35			(SM) Brown SAND with trace silt; moist, medium dense, nonplastic
			36			
			37			Bottom of hole at 35.5 ft
			38			
			39			
			40			

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION
84-210B	Simons, Li & Assoc.	RID

LOCATION OF BORING

Sta. 21+44 12 ft N of CL

DRILLING METHOD & EQUIPMENT

CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers

SAMPLING METHOD

Split-spoon Penetrometer and Ring Sampler

BORING NO.	SHEET	ENGINEER	TIME	DATE
B-14	1 OF 1	RMP		20 Nov 85

DATUM

ELEVATION 1004

CASING DEPTH

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	SURFACE CONDITIONS
			0			Gray and Brown SAND, GRAVEL and COBBLES
			1			
			2			
			3			
			4			
			5			Bottom of overburden layer 5.0 ft
			6	5		Bullnose penetration only, 6 ft to 14 ft
			7	6		
			8	9		
			9	9		
			10	11		
			11	22		
			12	37		
			13	44		
			14			
			15			Bottom of hole at 14.0 ft
			16			
			17			
			18			
			19			
			20			

DESERT EARTH ENGINEERING

JOB NO. 84-210B

CLIENT Simons, Li & Assoc.

LOCATION RID

LOCATION OF BORING

Sta. 21+42 2 ft N of CL

DRILLING METHOD & EQUIPMENT

CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers

SAMPLING METHOD

Split-spoon Penetrometer and Ring Sampler

CASING DEPTH

BORING NO. B-15

SHEET 1 OF 1

ENGINEER RMP

TIME

DATE 20 Nov 85

DATUM

ELEVATION 1003

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	SURFACE CONDITIONS
			0			Gray and Brown SAND, GRAVEL and COBBLES
			1			
			2			
			3			Clean Brown SAND with trace gravel; moist, medium dense, clasts are rounded
			4			
SS	18/18	4/5/7	5			
			6			
			7			
			8			
			9			
SS	18/18	12/15/14	10			
			11			
			12			Bottom of hole at 10.5 ft
			13			
			14			
			15			
			16			
			17			
			18			
			19			
			20			

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION
84-210B	Simons, Li & Assoc.	RID

LOCATION OF BORING

Sta. 21+43 15 ft N of CL

DRILLING METHOD & EQUIPMENT

CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers

SAMPLING METHOD

Split-spoon Penetrometer and Ring Sampler

CASING DEPTH

BORING NO. B-16

SHEET 1 OF 1

ENGINEER RMP

TIME

DATE 20 Nov 85

DATUM

ELEVATION

SURFACE CONDITIONS

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	
			0			Gray and Brown SAND, GRAVEL and COBBLES
			1			
			2			
			3			
			4			
			5			
			6			
			7			(CH) Brown CLAY; saturated, very plastic, soft
			8			
			9			
RS	12 / 0	5/6	10			Bullnose penetration only, 10 ft to 18 ft
				8		
			11			
				8		
			12			
				6		
			13			
				9		
			14			
				11		
			15			
				16		
			16			
				17		
			17			
				24		
			18			
			19			Bottom of hole at 18.0 ft
			20			

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION
84-210B	Simons, Li & Assoc.	RID

LOCATION OF BORING: Sta. 21+21 20 ft N of CL

DRILLING METHOD & EQUIPMENT: CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers

SAMPLING METHOD: Split-spoon Penetrometer and Ring Sampler

CASING DEPTH:

BORING NO. B-17

SHEET 1 OF 2

ENGINEER RMP

TIME

DATE 20 Nov 85

DATUM

ELEVATION 1003

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	SURFACE CONDITIONS
			0			Gray and Brown SAND, GRAVEL and COBBLES
			1			
			2			
			3			Some clay
			4			
			5			
			6			
			7			(CH) Brown CLAY; saturated, very plastic, soft
			8			
			9			
SS	18 / 12	3/2/4	10			(SM) Brown SAND with some silt
			11			
			12			
			13			(CH) CLAY interlayered with (SM) SAND with some silt; saturated, soft
			14			
RS	12 / 12	2/3	15			(CH) CLAY
			16			
			17			
			18			
			19			Some layers with trace organics
SS	18 / 18	½/½/½	20			

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION
84-210B	Simons, Li & Assoc.	RID

LOCATION OF BORING

DRILLING METHOD & EQUIPMENT	BORING NO.
CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers	B-17
	SHEET 2 OF 2
SAMPLING METHOD	ENGINEER RMP
Split-spoon Penetrometer and Ring Sampler	TIME
CASING DEPTH	DATE 20 Nov 85

DATUM　　ELEVATION

SURFACE CONDITIONS

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	DESCRIPTION
			20			(CH) CLAY
			21			
			22			
			23			
RS	12/12	3/3	24			
			25			
			26			GRAVELLY
			27			
			28			
			29			(GW) Brown SANDY GRAVEL with some clay
SS	18/18	9/8/5	30			
			31			
			32			
			33			
			34			(GW) Brown SANDY GRAVEL with trace clay; very dense, angular to subangular
SS	12/12	33/47	35			
			36			Bottom of hole at 35.0 ft
			37			
			38			
			39			
			40			

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION
84-210B	Simons, Li & Assoc.	RID

LOCATION OF BORING: Sta. 23+60 35 ft N of CL

DRILLING METHOD & EQUIPMENT: CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers

SAMPLING METHOD: Split-spoon Penetrometer and Ring Sampler

CASING DEPTH:

BORING NO.: B-18

SHEET: 1 OF 3

ENGINEER: RMP

TIME:

DATE: 20 Nov 85

DATUM:

ELEVATION: 1009

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	SURFACE CONDITIONS
			0			Gray and Brown SAND, GRAVEL and COBBLES
			1			
			2			
			3			
			4			
			5			
			6			
			7			(CH) CLAY
			8			
			9			Some cobbles, 7 ft to 14 ft
			10			
			11			
			12			
			13			
			14			(CH) CLAY
RS	12 / 12	2/4	15			
			16			
			17			
			18			
			19			
			20			

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION
84-210B	Simons, Li & Assoc.	RID

LOCATION OF BORING	
DRILLING METHOD & EQUIPMENT	CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers
BORING NO.	B-18
SHEET	2 OF 3
ENGINEER	RMP
SAMPLING METHOD	Split-spoon Penetrometer and Ring Sampler
TIME	
DATE	20 Nov 85
DATUM	
ELEVATION	
CASING DEPTH	

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	SURFACE CONDITIONS
			20			
			21			
			22			
			23			
			24	1		Bullnose penetration only, 24 ft to 45 ft
			25	1		
			26	1		
			27	2		
			28	2		
			29	2		
			30	2		
			31	3		
			32	3		
			33	4		
			34	5		
			35	5		
			36	6		
			37	7		
			38	7		
			39	8		
			40			

DESERT EARTH ENGINEERING

JOB NO.	CLIENT	LOCATION
84-210B	Simons, Li & Assoc.	RID

LOCATION OF BORING

DRILLING METHOD & EQUIPMENT
CME-55 Drill Rig equipped with 6 5/8" OD 3 1/4" ID hollow-stem continuous flight augers

BORING NO. B-18

SHEET 3 OF 3

ENGINEER RMP

SAMPLING METHOD
Split-spoon Penetrometer and Ring Sampler

TIME

DATE 20 Nov 85

DATUM

ELEVATION

CASING DEPTH

SAMPLER TYPE	INCHES DRIVEN / INCHES RECOVERED	BLOWS/6" SAMPLER	DEPTH IN FEET	BULL-NOSE BLOWS/FT	WATER LEVEL	SURFACE CONDITIONS
			40			
				8		
			41			
				10		
			42			
				10		
			43			
				11		
			44			
				32		
			45			
			46			Bottom of hole at 45.0 ft
			47			
			48			
			49			
			50			
			51			
			52			
			53			
			54			
			55			
			56			
			57			
			58			
			59			
			60			

APPENDIX B

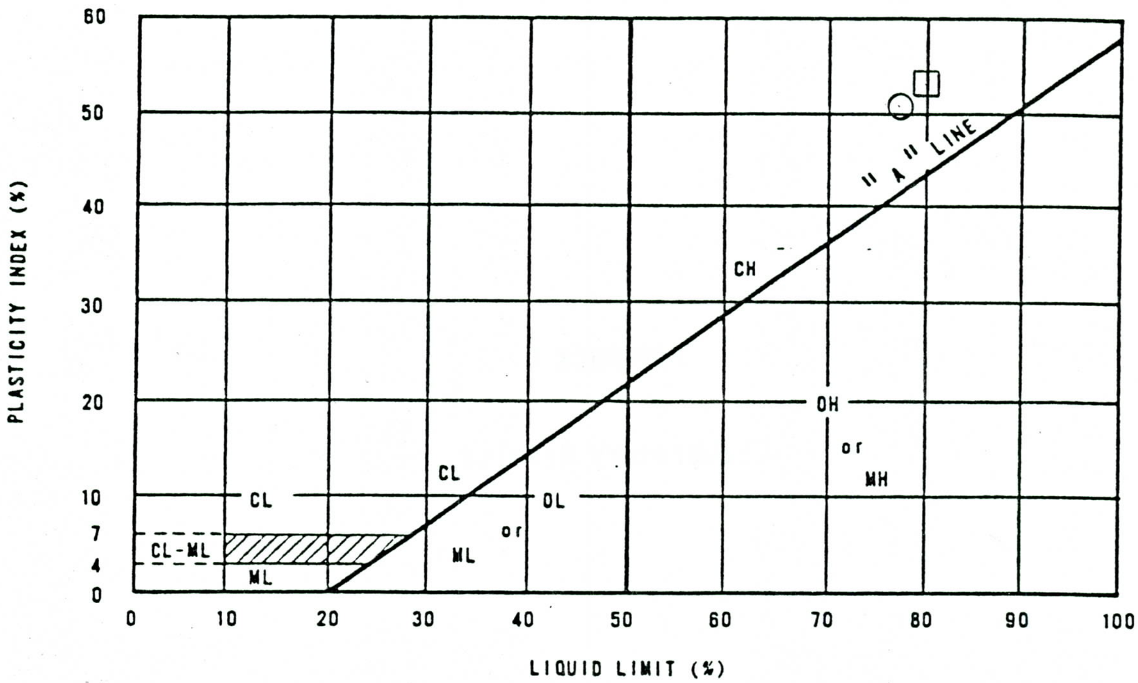
Laboratory Results

JOB 84-210B

DATE 18 Dec 85

BY RMP

PLASTICITY CHART

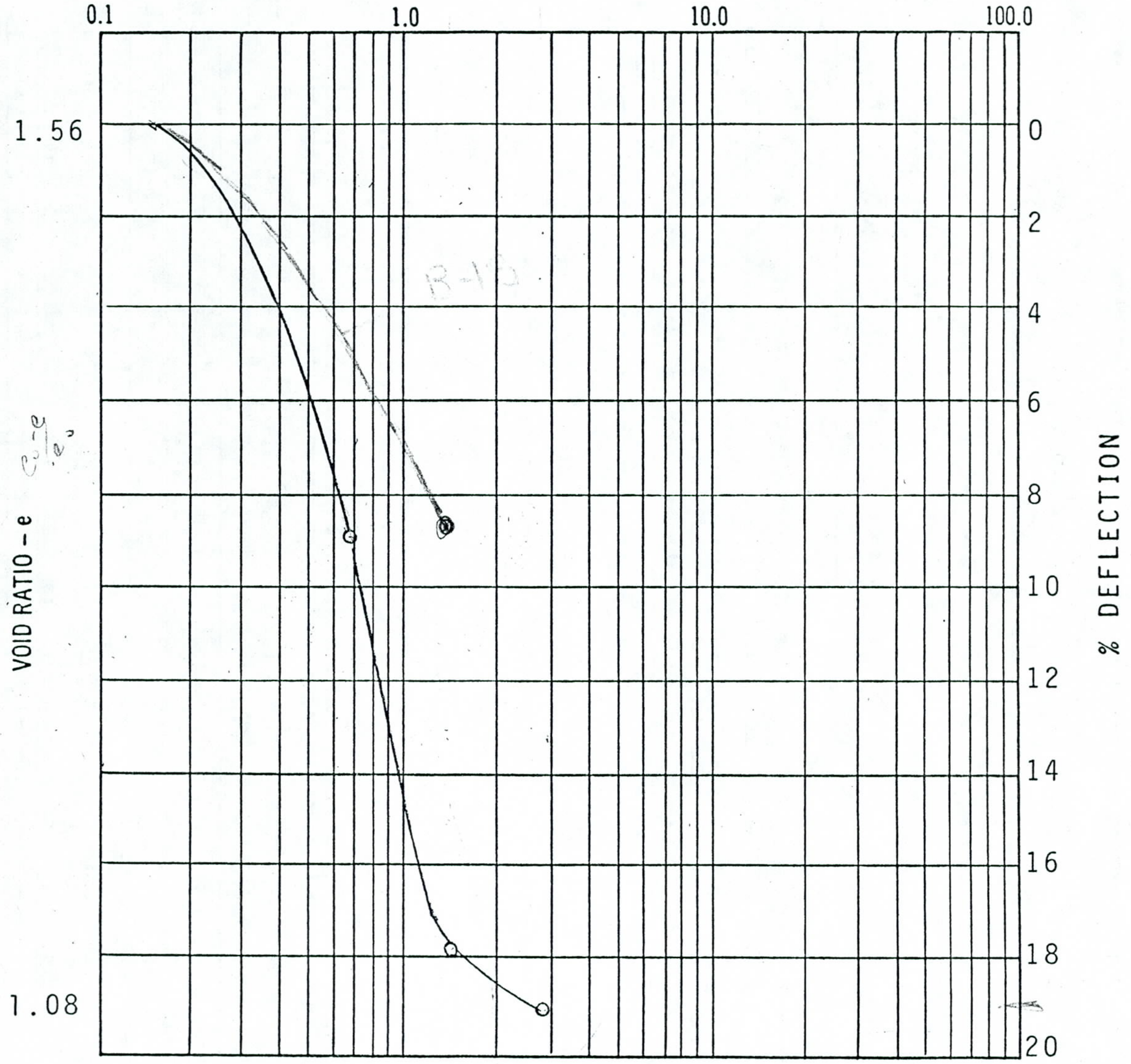


PLASTICITY DATA

KEY SYMBOL	HOLE NO	DEPTH (feet)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	UNIFIED SOIL CLASSI-FICATION SYMBOL	% PASSING # 200
○	B-13	14-15.5	77	51	CH	99
□	B-13	34-35	80	53	CH	99

desert earth engineering

CONSOLIDATION TEST DATA



JOB NO. 84-210B BY RMP DATE 11 Dec 85

DESCRIPTION OF SAMPLE:	(CH) Brown CLAY				
BORING: B-13		DEPTH: 24-25		ELEVATION: 0981	
MOST PROBABLE PRECONSOLIDATION STRESS (KSF) SHOWN THUS: P_c↓			EXISTING OVERBURDEN STRESS (KSF) SHOWN THUS: P_o↓		
COMPRESSION INDEX (C_c)		.676	SWELLING INDEX (C_s)		
MOISTURE CONTENT %	INITIAL	62.5	VOID RATIO	INITIAL - e_o	1.56
	FINAL	39.8		FINAL - e_F	1.08
DRY DENSITY (PCF.)	INITIAL	65.7	DEGREE OF SATURATION %	INITIAL	SAT
	FINAL	81.2		FINAL	SAT

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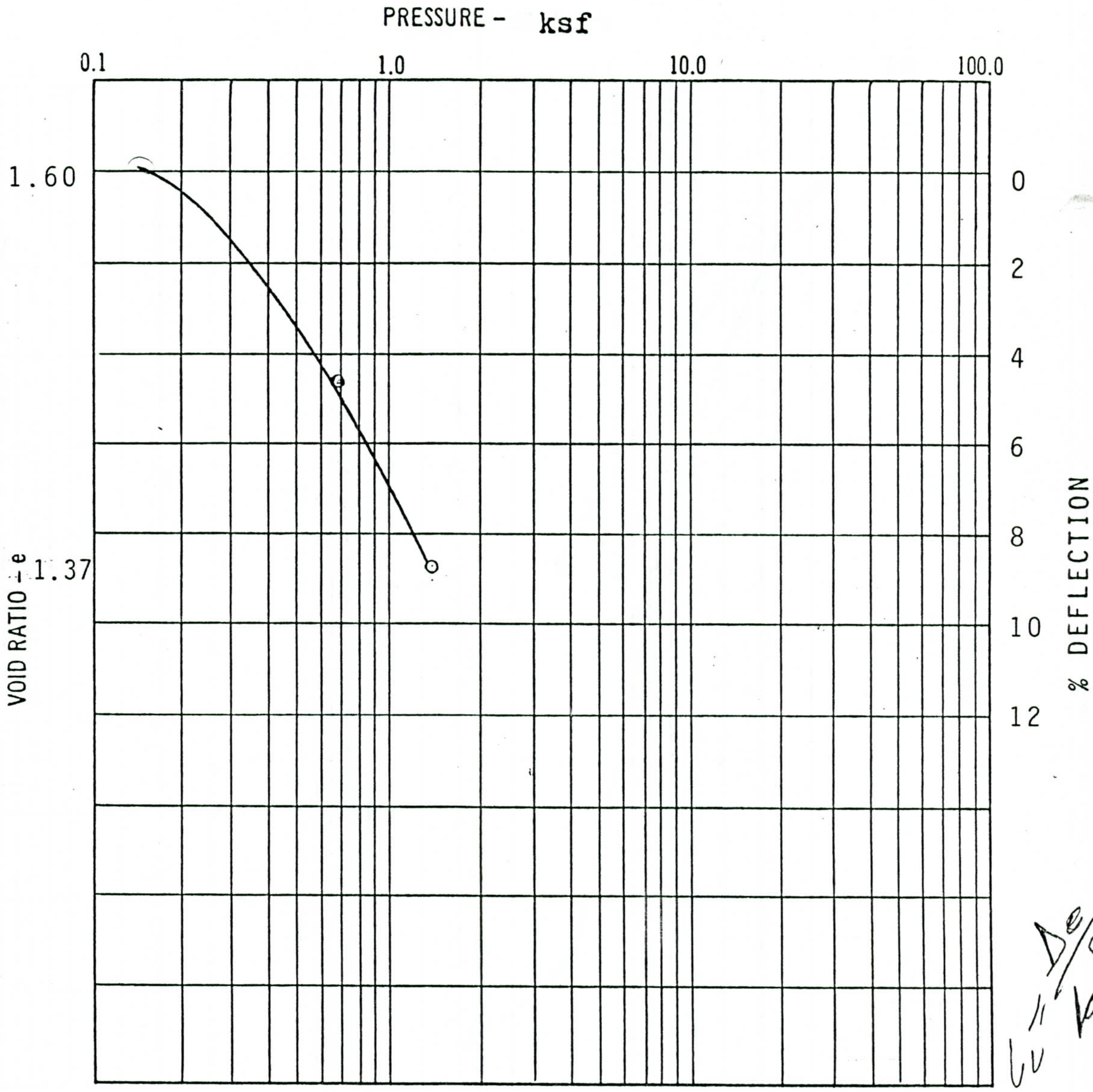
CONSOLIDATION TEST DATA



$C_c = \frac{\Delta e}{\log P_2 - \log P_1}$

JOB NO. 84-210B BY RMP DATE 18 Dec 85

DESCRIPTION OF SAMPLE:	(CH) Brown CLAY				
BORING: B-18		DEPTH: 14-15		ELEVATION: 0995	
MOST PROBABLE PRECONSOLIDATION STRESS (KSF) SHOWN THUS: P_c↓			EXISTING OVERBURDEN STRESS (KSF) SHOWN THUS: P_o↓		
COMPRESSION INDEX (C_c)			SWELLING INDEX (C_s)		
MOISTURE CONTENT %	INITIAL	54.2	VOID RATIO	INITIAL - e_o	1.60
	FINAL	50.7		FINAL - e_F	1.37
DRY DENSITY (PCF.)	INITIAL	64.9	DEGREE OF SATURATION %	INITIAL	91.7
	FINAL	71.1		FINAL	SAT

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CALCULATIONS

SIMONS, LI & ASSOCIATES, INC.

CLIENT: MCFCD — JOB No. AAZ-MC-08 — PAGE:
PROJECT: BID CANAL — DATE CHECKED: 12/17/85 — DATE: 12.13.85
DETAIL: DRAINS — CHECKED BY: NEB — COMPUTED BY: JSW

CONSOLIDATION DETERMINED BY RADIAL DRAINAGE FORMULA
— SEE ATTACHED MATERIAL

DRAIN TYPE	PATTERN	D	D_E (FT)	D_W (FT)	n	α	U_r 200 DAYS	t_{90} (DAYS)
WICK	RECT	5	5.65	.2	28.25	2.5956	.564	355 DAYS
	TRIANGULAR	5	5.25	.2	26.25	2.52278	.628	466 DAYS
	RECT	4	4.52	.2	22.60	2.3746	.758	325 DAYS
	TRIANGULAR	4	4.20	.2	21.00	2.3020	.816	272 DAYS
SAND	RECT	5	5.65	.504'	11.21	1.688	.721	362 DAYS
	TRIANGULAR	5	5.25	.504'	10.41	1.6175	.786	299 DAYS
	R	6	6.78	.504'	13.45	1.8650	.552	575 DAYS
	T	6	6.30	.504'	12.50	1.7936	.620	478 DAYS

COSTS

SPACING	PATTERN	# HOLES*	L.F.**	UNIT COST	TOTAL
4	R	88 * 25	83,600	2.00	167,200
	T 3.46	88 * 28	93,632		187,264
5	R	71 * 20	53,960		107,920
	T 4.33	71 * 23	62,054		124,108
6	R	59 * 17	38,114		76,228
	T 5.180	59 * 20	44,840		89,680

* AREA = 350 x 95
** USE AVG. 38' DEEP

sla
SIMONS, LI & ASSOCIATES, INC.

CLIENT MCFCD	JOB NO. PAZ-MC-08
PROJECT RID CANAL	DATE CHECKED 12/17/85
DETAIL DRAINS	CHECKED BY AEB
PAGE	DATE 12.13.85
	COMPUTED BY SW

SOIL CONSOLIDATION USING VERTICAL DRAINS

$$U_r = 1 - \exp\left[-8C_h t / D_e^2 \alpha\right]$$

FROM
M.S. ATKINSON & P.J.L. ELDRED
VERTICAL DRAINS
THOMAS TELFORD LTD
LONDON 1982

WHERE

$$\alpha = \left[\frac{n^2}{n^2-1} \ln(n)\right] - \frac{3n^2-1}{4n^2}$$

$$n = \frac{D_e}{D_W}$$

FOR RECTANGULAR GRID $\quad \pi \frac{D_E^2}{4} = D^2 \Rightarrow D_E \cong 1.13D$

FOR TRIANGULAR GRID $\quad D_E^2 = \frac{2\sqrt{3}}{\pi} D^2$

$$D_E \cong 1.05\,D$$

FOR ANALYSIS

USE $D_W = 52\,mm \cong .2'$

DRAIN SPACINGS 4', 5' & 6' CENTERS IN RECT & TRIANGULAR CONFIGURATION

$C_h = C_v = .043\ ft^2/DAY$

CHECK U_r @ 200 DAYS

SOLVE t FOR $U_{90\%}$ PRIMARY CONSOLIDATION

SLA
SIMONS, LI & ASSOCIATES, INC.

CLIENT McFCD
PROJECT RID CANAL
DETAIL CONSOLIDATION

JOB NO. PAZ-MC-08
DATE CHECKED 12/17/85
CHECKED BY NEB

PAGE
DATE 12.13.85
COMPUTED BY SKL

PROGRAM "RADIAL DRAINAGE" FOR HP-15C

REGISTERS

STO 1 D_e
STO 2 D_w
STO 3 C_h
STO 4 t

PROGRAM

STO 4
RCL 01 RCL 02 ÷ STO 5 PSE PSE → n

[RCL 5 ln RCL 5 x^2 * RCL 5 x^2 1 − ÷] RCL 5 x^2 3 * 1 −

RCL 5 x^2 4 * ÷ − STO 6 PSE PSE → α

8 CHS RCL 3 * RCL 4 * RCL 1 x^2 ÷ RCL 6 ÷ e^x CHS 1 + → U_r

g RTN

SIMONS, LI & ASSOCIATES, INC.

CLIENT MCFCD JOB No. PAZ-MC-02(4) PAGE
PROJECT DATE CHECKED 12/17/85 DATE
DETAIL CHECKED BY NEB COMPUTED BY

CHECK CONSOLIDATION USING SIMPLIFIED CONSOLIDATION FORMULA PRESENTED BY DESERT EARTH ENGINEERING, LETTER REPORT 12.12.85

$C_V = .043\ FT^2/DAY$

$t = 200\ DAYS$

$T = .848$ {FROM D.E.E. & FOUNDATION AND DESIGN, BOWLES}

$H = \sqrt{\frac{t C_V}{T}}$ WHERE $H = 1/2$ DRAIN SPACING

$H = \sqrt{\frac{(200)(.043)}{.848}}$

$H = 3.18\ FT$

$D = 2H = 6.4\ FT$

USE $2H = 5'$, $H = 2.23$

$t = \frac{H^2 T}{C_V}$

$= (2.23)^2(.848) / .043\ FT^2/DAY$

$= 98\ DAYS$

sla SIMONS, LI & ASSOCIATES, INC.

CLIENT: MCFCD
PROJECT: RID CANAL
DETAIL: DRAINS
JOB NO. PAZ-MC-08(4)
DATE CHECKED: 12/17/85
CHECKED BY: NEB
PAGE: 1/1
DATE: 12·13·85
COMPUTED BY: SW

VALUES FROM RALPH PATTISON TO NOEL BORMANN, BY PHONE P.M. 12·11·85

$C_v = 2.5 \times 10^{-3}\ in^2/min$

$C_c = 0.67$

$K = 2 \times 10^{-8}\ cm/sec$

$e_o = 1.5$

SIMONS, LI & ASSOCIATES, INC.

CLIENT MCFCD
PROJECT RID CANAL
DETAIL DEE LETTER 12.12.85

JOB NO. PAZ-MG-Ø8
DATE CHECKED 12/17/85
CHECKED BY NEB

PAGE 1/2
DATE 12.13.85
COMPUTED BY SW

GIVEN IN PHONE CONVERSATION NEB & RALPH

$$C_V = 2.5 \times 10^{-3} \frac{in^2}{MIN}$$

$$C_V = \frac{T H^2}{t}$$

T = .848 FOR CASE I, 90% CONSOLIDATION
H = 1/2 DRAIN SPACING, = 5'

$$t_{90} = \frac{.848 \left(2.5 \cancel{FT} \frac{12 in}{\cancel{FT}}\right)^2}{2.5 \times 10^{-3} \frac{in^2}{MIN}} = 305,280 \text{ MIN.} = \underline{\underline{212 \text{ DAYS}}}$$

USING C_V GIVEN IN D.E.E. LETTER 12.12.85

$$C_V = 0.043 \text{ FT}^2/\text{DAY}$$

$$t_{90} = \frac{.848 (2.5 \text{ FT})^2}{.043 \text{ FT}^2/\text{DAY}} = \underline{\underline{123.3 \text{ DAYS}}}$$

USING C_V USED IN FORMULA, D.E.E. LETTER 12.12.85

$$C_V = .022 \text{ FT}^2/\text{DAY}$$

$$t_{90} = \frac{.848 (2.5 \text{ FT})^2}{.022 \text{ FT}^2/\text{DAY}} = 240.9 \text{ DAYS}$$

CONVERT $C_V = 2.5 \times 10^{-3}$ IN^2/MIN TO FT^2/DAY

$$2.5 \times 10^{-3} \frac{in^2}{MIN} \times 60 \frac{MIN}{HR} \times \frac{24 HR}{DAY} \times \frac{FT^2}{144 in^2} = \underline{\underline{.025 \text{ FT}^2/\text{DAY}}}$$

sla
SIMONS, LI & ASSOCIATES, INC.

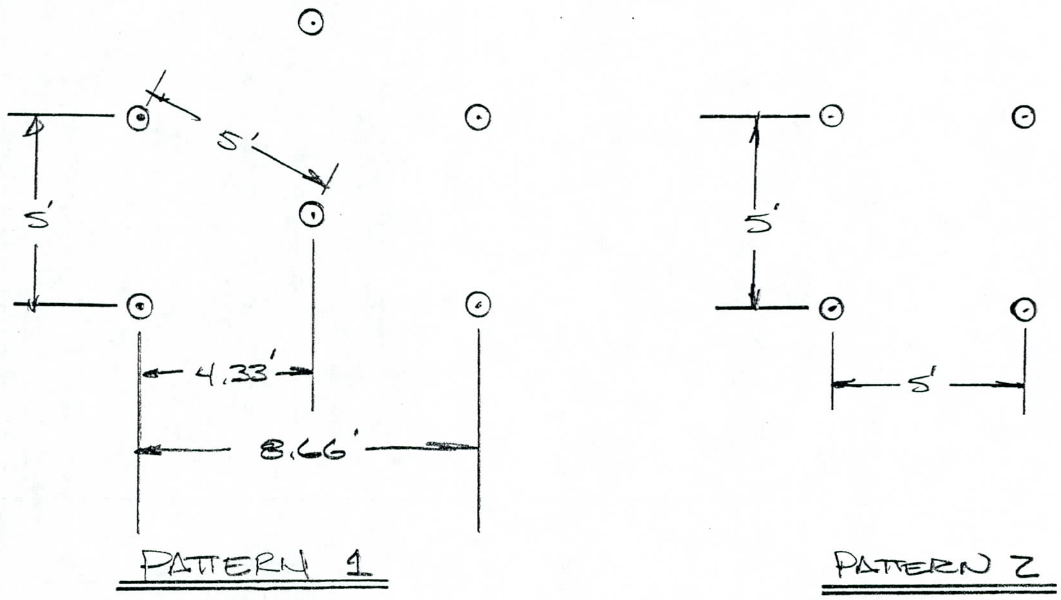
CLIENT MCFCD
PROJECT RID CANAL
DETAIL D.E.E. LETTER 12.12.85
JOB NO. AZ.MC.08
DATE CHECKED 12/17/85
CHECKED BY NEB
PAGE 2/2
DATE 12.13.85
COMPUTED BY KN

CONVERT $K = 2\times10^{-8} \frac{cm}{sec}$ TO Ft/DAY

$$2\times10^{-8}\frac{cm}{sec} \times \frac{60s}{m} \times \frac{60m}{Hr} \times \frac{24Hr}{DAY} \times \frac{1in}{2.54cm} \times \frac{Ft}{12in} = \underline{\underline{5.67\times10^{-5}\frac{Ft}{DAY}}}$$

CLIENT: MCFCD	JOB NO. PAZ-MC-08(4)	PAGE:
PROJECT: RID CANAL & FLUME	DATE CHECKED:	DATE: 12-16-85
DETAIL: DRAIN SPACING	CHECKED BY:	COMPUTED BY: JSW

SIMONS, LI & ASSOCIATES, INC.



AREA = 115' x 330'

$\# \text{HOLES} = \dfrac{115'}{5'} \times \dfrac{330'}{4.33'} = 23 \times 76 = 1748$ HOLES

LINEAR FOOTAGE = #HOLES * DEPTH

$= 1748 \times [1005 - 970] = 61{,}180$ L.F.

PATTERN 2

$\# \text{HOLES} = \dfrac{115'}{5'} \times \dfrac{330'}{5'} = 23 \times 66 = 1518$

LINEAR FOOTAGE $= 1518 \times 35'/\text{HOLE} = 53{,}130$ L.F.

$\Delta = 61{,}180 - 53{,}130 = 8{,}050$ L.F.

CHECK CONSOLIDATION USING "AMERDRAIN: DESIGNING WITH SOIL DEWATERING WICKS"

FIND DIAMETER TO ACHIEVE $\bar{U} = 90\%$ IN 200 DAYS

$C_V = .043\ FT^2/DAY = 15.7\ FT^2/YEAR = \lambda$
$\Delta U_o = 14\ FT\ (\times 100 pcf) = 1400\ psf$
$t = 200\ DAYS \cong 6.5\ MONTHS$

FROM FIG. 1

$f_2 = .25\ YEARS$

$f_2 \lambda = .25\ \cancel{YEARS}\ (15.7\ FT^2/\cancel{YR}) = 3.9\ FT^2$

FROM FIG. 5

$D\ (IN\ FEET) \cong 5.5\ FEET$

CASE EXAMPLES

f., Antwerp.
Baligh, M. M. (1978).
cone penetrometer and
Rept, Dept Civ. Engng,

of consolidation coef-
. Fdns Div. Am. Soc.

avities in infinite soil
Am. Soc. Civ. Engrs.

74). Consolidation of
ls with finite perme-
n Engn 14, No. 2.

Performance of vertical drains at Queenborough bypass

D. P. NICHOLSON* and R. J. JARDINE†

The bypass round the town of Queenborough on the Isle of Sheppey, Kent is constructed on soft alluvial clay up to 10 m deep. Where the bypass bridges an existing railway line the approach embankments rise to a height of 7 m above the existing marshland. Embankment construction was started in 1976, but instrument readings indicated that the initial stage of filling could not safely be constructed higher than 3·0 m. Further site investigations were made which included in situ constant head permeability tests performed by reducing the pore pressure at the piezometer cell using recently developed equipment. Analysis showed that vertical drains would be required to accelerate the consolidation and enable the embankment to be completed within the remaining construction programme. Subsequently, two types of vertical drains were assessed during trials in 1978: a 65 mm dia. Sandwick drain and a 300 mm wide AV Colbond fabric strip drain. The Paper compares the consolidation parameters and settlements predicted from laboratory and in situ tests with the field performance of the embankment with and without the vertical drains. Good agreement was found between these comparisons which confirm the predicted decrease in coefficient of consolidation as effective stress exceeded the preconsolidation pressure.

La Route de Déviation autour de la ville de Queenborough sur l'Ile Sheppey, Kent, a été construite sur de l'argile alluvionnaire molle d'une profondeur atteignant 10 mètres. Là où la route de déviation passe au-dessus d'une ligne de chemin de fer existante, les remblais d'approche s'élèvent à une hauteur de 7 m au-dessus du terrain marécageux existant. La construction des remblais a commencé en 1976 mais les valeurs obtenues à l'aide d'instruments de mesure indiquaient que l'on ne pouvait dépasser sans danger 3·0 m pour le remblai initial. Une analyse ultérieure a révélé qu'il faudrait prévoir des drains verticaux pour accélérer la cadence de consolidation et pour permettre de terminer les remblais dans les limites du programme de construction portant sur trois ans. Des essais de perméabilité in situ à charge constante ont été effectués en réduisant la pression interstitielle à l'extrémité du piézomètre à l'aide d'un matériel récemment mis au point. Deux types de drains verticaux ont été testés par la suite lors des essais effectués en 1978; il s'agissait du drain du type 'Sandwick' de 65 mm de diamètre, et du drain du type à bande de toile 'AV Colbond', de 300 mm de large. L'article compare les paramètre de consolidation et de tassement, deduits des essais de consolidation en laboratoire et des essais de perméabilité in situ, à la performance sur place du remblai avec et sans drains verticaux. Les comparaisons effectuées révèlent une réduction analogue du coefficient de consolidation lorsque la contrainte efficace dépasse la pression de préconsolidation.

* Ove Arup & Partners.
† Kent County Council.

INTRODUCTION

Queenborough bypass is being constructed by Kent County Council to improve access to Sheerness Docks and to relieve traffic congestion in Queenborough. The bypass alignment crosses an area of former tidal marshland and an existing railway track that will be bridged by the new road (Fig. 1). A section through the northern bridge approach embankment, which has to be constructed to +8·5 m OD, approximately 7 m above the original ground level, is shown in Fig. 2.

The initial site investigation was undertaken by Kent County Council in 1973 and revealed about 10 m of soft alluvial clays which would necessitate a staged construction programme for the approach embankment. An examination of consecutive piston samples indicated the presence of some laminations and layers of permeable peat and silt. It was considered that these might be sufficient to provide adequate horizontal drainage to enable the embankment to be constructed with a 4 year programme.

The first construction stage was placed in 1976, with fully instrumented sections to study the in situ behaviour and rate of consolidation of the foundation soils so that the placing of subsequent stages could be determined. The measured rates of consolidation were found to be too slow to enable the programmed completion date in August 1981 to be met by natural drainage alone. At this point, in August 1977, Kent County Council requested Ove Arup and Partners to give advice on alternative ways of completing the embankment.

Further site investigations were made during the winter of 1977–78 together with an analysis of existing instrumentation readings. Preliminary studies showed that vertical drains would be required to accelerate consolidation and that savings in cost could be made by the use of lightweight fill coupled with a lengthening of the viaduct and a revision of the vertical alignment.

In order to arrive at the most efficient design, studies of the in situ permeability were carried out to evaluate the drainage properties of the soil. In

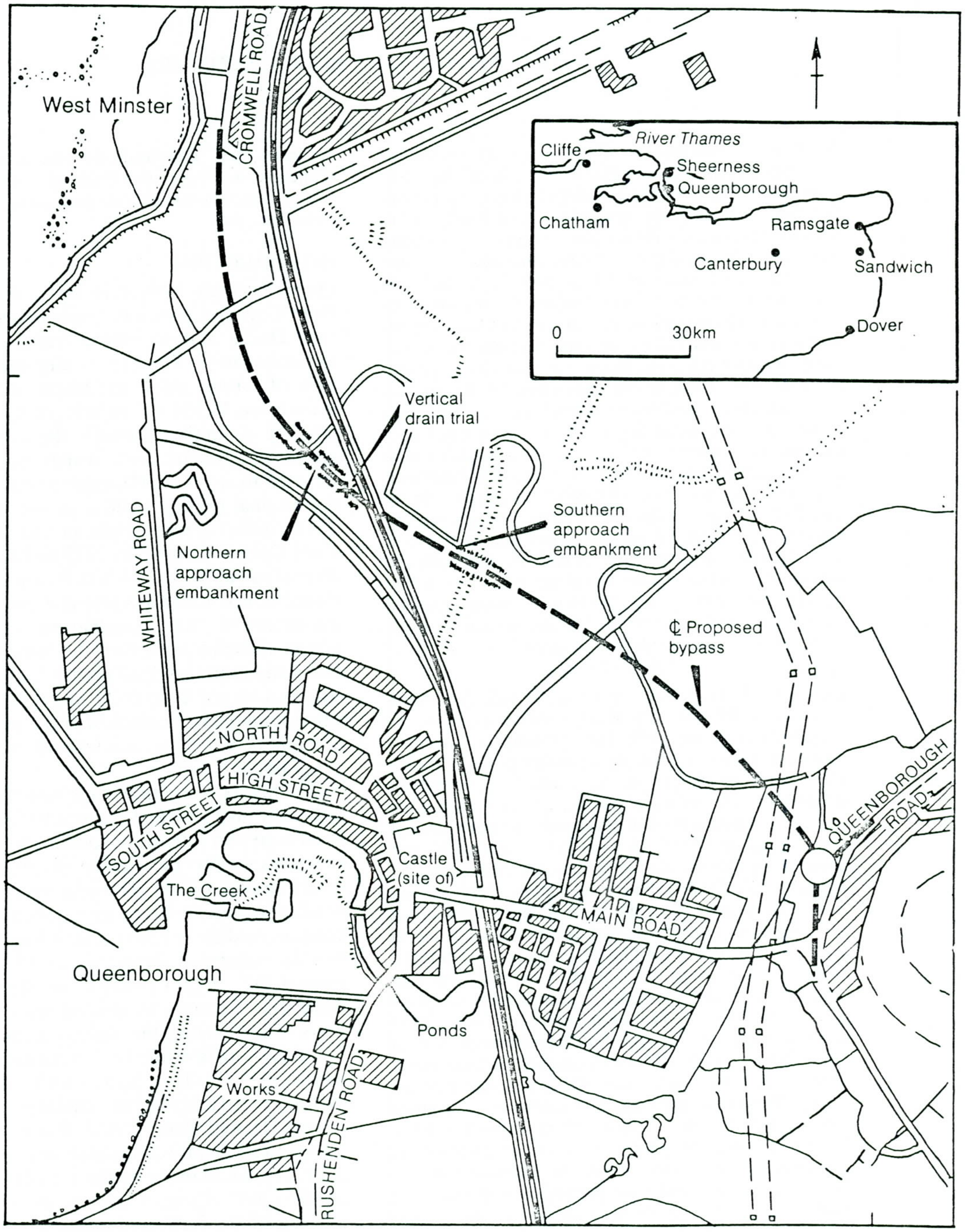
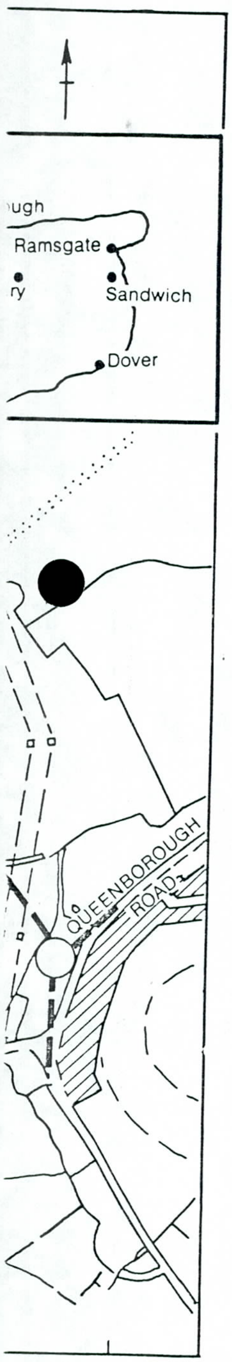


Fig. 1. Site plan



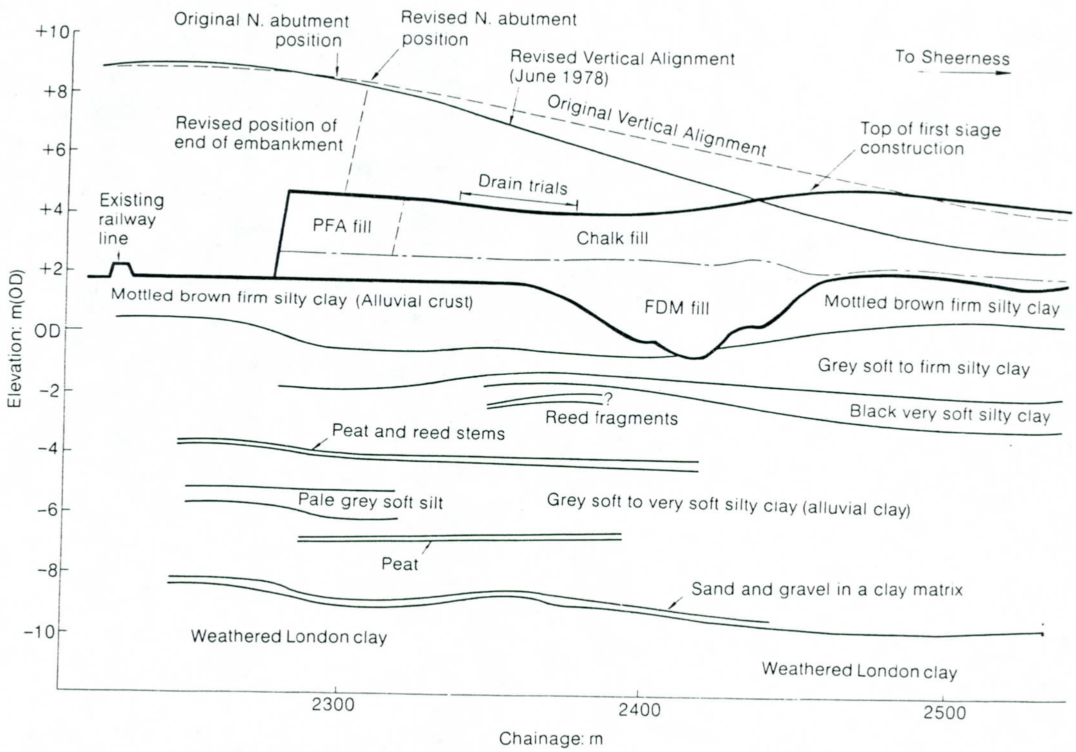


Fig. 2. Section along centreline of northern approach embankment

addition, the field performance of two commercial vertical drain systems were evaluated in full-scale trials using the 65 mm dia. Sandwick drain installed by Cementation Ground Engineering Ltd and the 300 mm wide AV Colbond strip drain installed by Soil Mechanics Ltd. These trials were carried out over the summer of 1978.

Based on these studies, a drain layout was designed to provide sufficient dissipation to ensure stability during construction and to limit the settlement remaining at the programmed completion of the viaduct and road. A contract was subsequently let to Cementation Ground Engineering to install 7500 Sandwick drains in the approach embankments. This work was carried out between October and December 1978. Embankment construction is now nearing completion and it is possible to compare the predicted and measured performance of the vertical drains.

GROUND CONDITIONS AND SITE

At the approach embankment ground levels vary from about OD to +1·8 m OD, the lower areas being associated with drainage ditches and channels. The ground surface is covered by rough marsh grasses. The groundwater level is subject to seasonal variations; the level rises to the ground surface during winter months when the low lying areas are prone to flooding, and falls to about 1 m below ground level during the summer. The soil profile and properties are summarized in Figs 2 and 3. These indicate very soft to soft recent alluvial clays to about −8·5 m OD below a firm desiccated crust which decreases in strength with depth to −0·5 m OD, with firm brown London clay below the alluvial clays. The London clay becomes very stiff with depth and was proved by boring to about −30 m OD.

During the 1978 site investigation, borings were made in sets of three. Consecutive 1 m long by 102 mm dia. piston samples were taken from the first borehole of the set. Hand vane tests were made as these samples were extruded and split for visual examination and index tests (Fig. 3). The samples were then photographed at various stages of drying to examine the soil fabric. This revealed numerous vertical 1–2 mm dia. hollow root holes in the alluvial clays, with occasional 5–8 mm dia. peat-filled holes. On drying, the soils in the crust showed signs of silty laminations but these were not observed in the underlying alluvium. Peaty layers and thin beds of reed stems were occasionally found (Fig. 2). At the base of the alluvium a layer of gravel

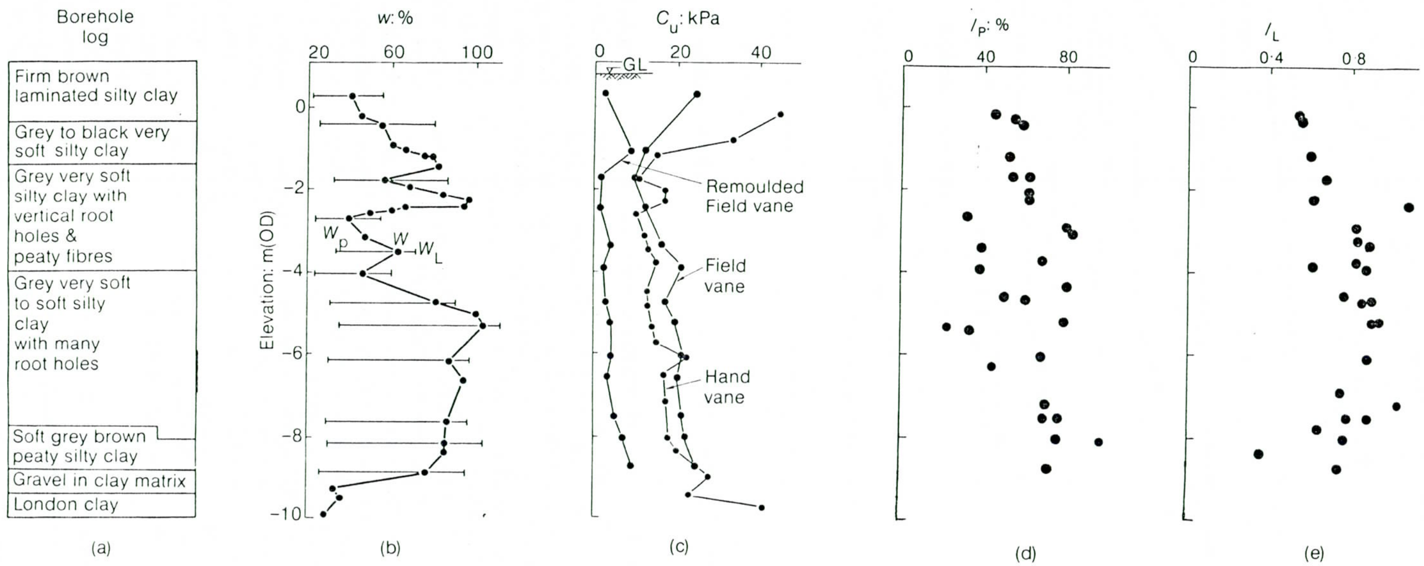


Fig. 3. Borehole profile and index properties: (a) simplified borehole log; (b) water content, w; (c) vane strength, c_u; (d) plasticity index, I_P; (e) liquidity index, I_L

Predicted primary settlement: m

Depth: m(OD)

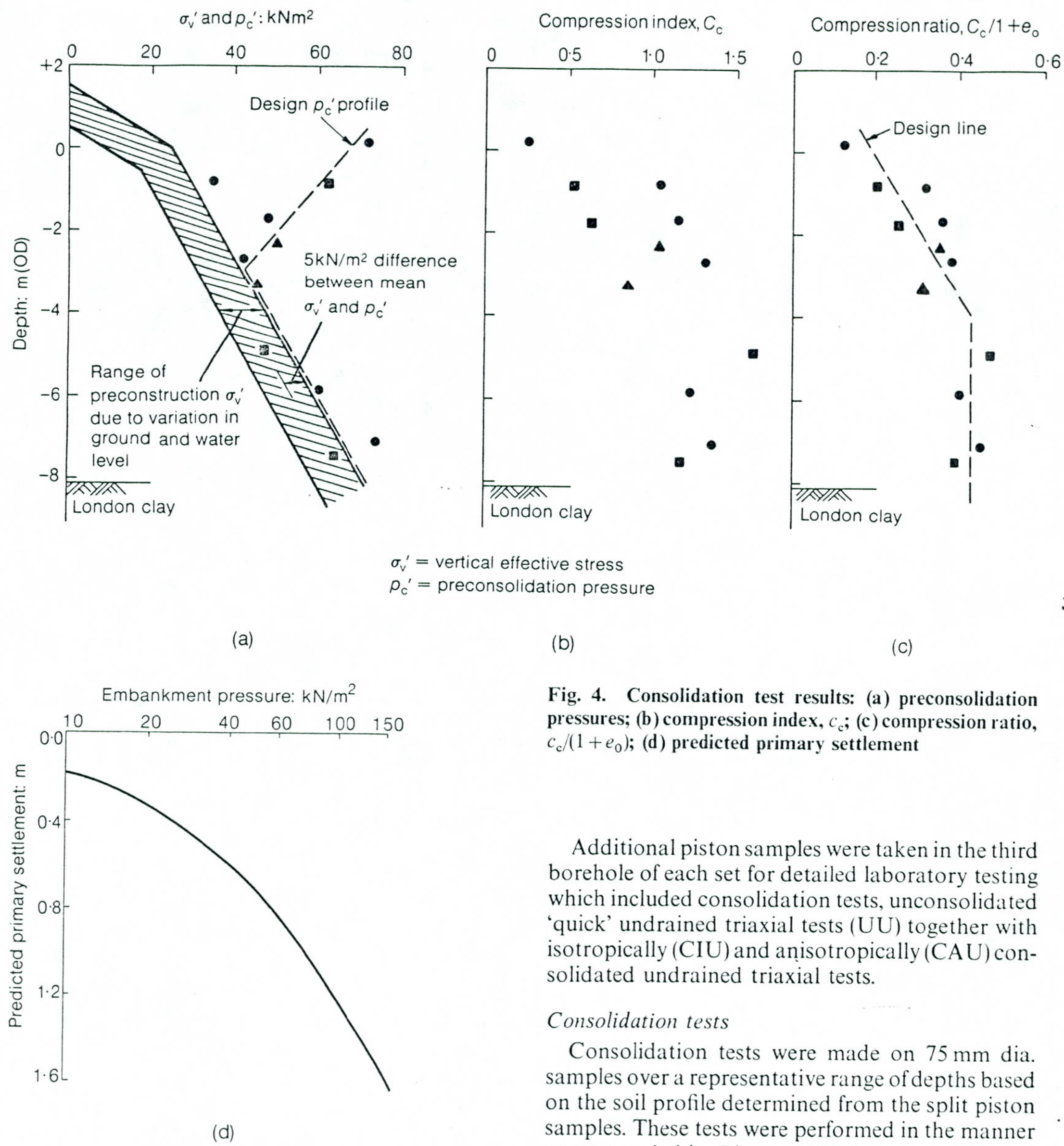


Fig. 4. Consolidation test results: (a) preconsolidation pressures; (b) compression index, c_c; (c) compression ratio, $c_c/(1+e_0)$; (d) predicted primary settlement

in a clay matrix was identified. The large scatter of plasticity index values, which generally indicated high plasticity clays, may be seen in Fig. 3(d). The liquidity index profile (Fig. 3(e)) showed less scatter, with an average value of about 0·8 below −2 m OD.

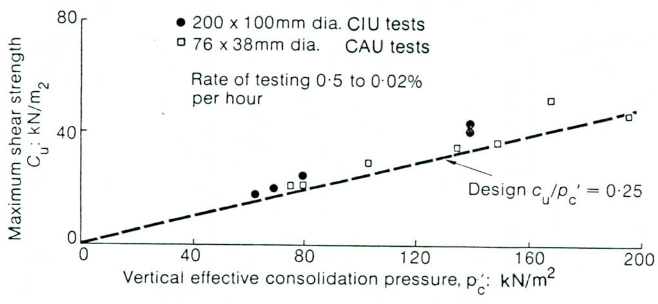
Field vane tests were made in the second borehole of each set of three and undisturbed samples were taken over the same depths to examine the soil tested by the vane and provide material for plasticity index tests. The vane shear strengths are shown in Fig. 3(c).

Additional piston samples were taken in the third borehole of each set for detailed laboratory testing which included consolidation tests, unconsolidated 'quick' undrained triaxial tests (UU) together with isotropically (CIU) and anisotropically (CAU) consolidated undrained triaxial tests.

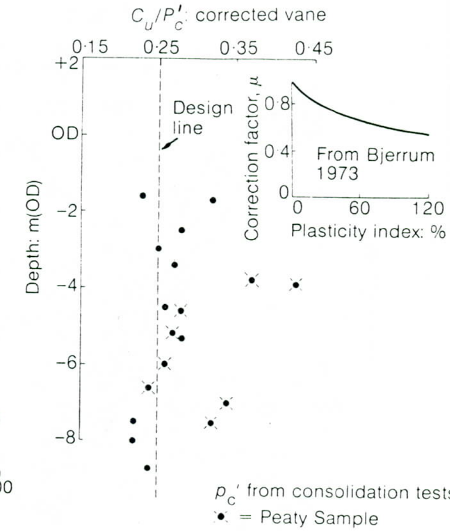
Consolidation tests

Consolidation tests were made on 75 mm dia. samples over a representative range of depths based on the soil profile determined from the split piston samples. These tests were performed in the manner recommended by Bjerrum (1973) with small increments of load up to preconsolidation pressure. Each successive increment was added when 90% of the primary settlement had been defined on the plot of settlement versus the square root of time $\sqrt{t}$. Further increments above preconsolidation pressure were added at approximately 24 h intervals by generally doubling the applied load to minimize the effects of creep. A profile of the preconsolidation pressures p_c' based on the Casagrande construction method is shown in Fig. 4(a). Also shown are the compression index and compression ratio profiles (Fig. 4(b), (c)) on which the primary settlement calculations shown in Fig. 4(d) were based.

Fig. 5. Shear strength parameters: (a) triaxial test results; (b) corrected vane c_u/p_c' profile

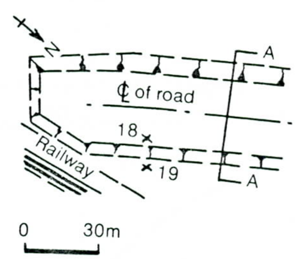


(a)

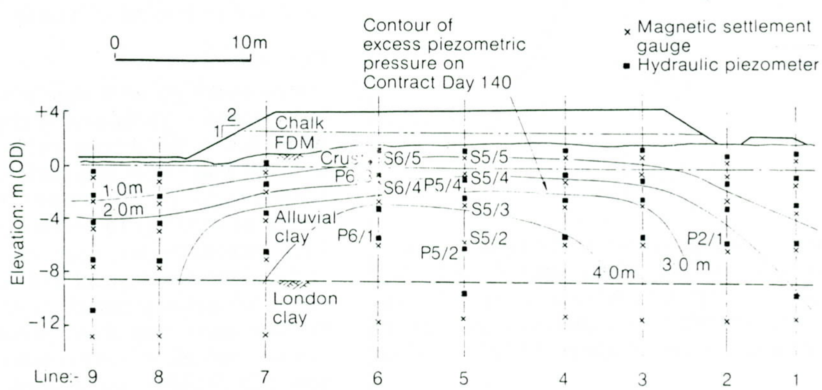


(b)

Fig. 6. (Below and facing page.) Initial stage of construction: (a) plan; (b) instrumentation section (section AA—contours of excess piezometric pressure at day 140); (c) typical instrumentation records; (d) centreline settlement profile; (e) lateral movement profile; (f) piezometric level profile



(a)



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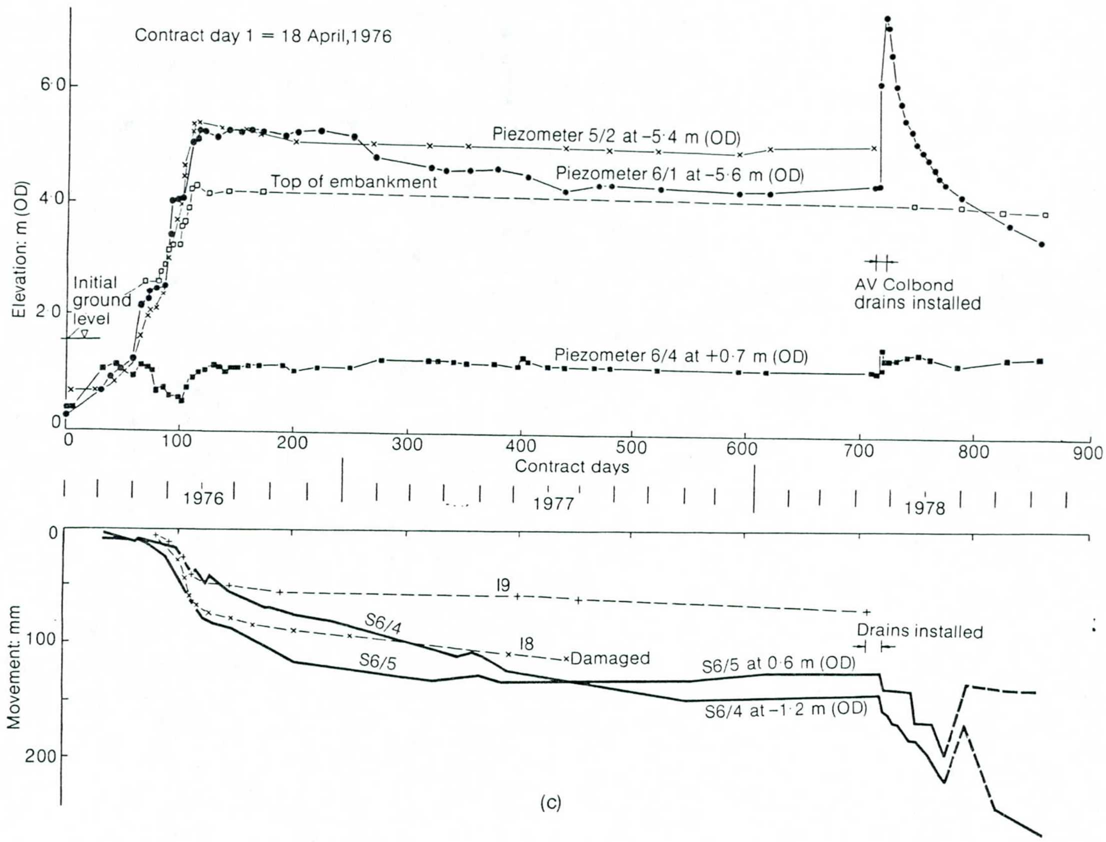
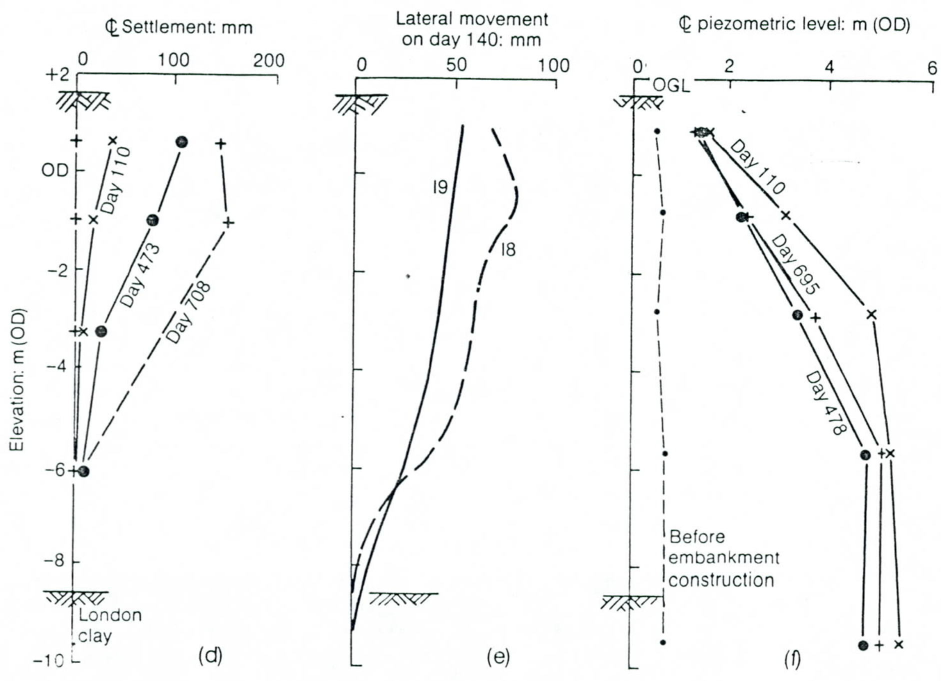


Contract day 1 = 18 April,1976
Piezometer 5/2 at −5·4 m (OD)
Piezometer 6/1 at −5·6 m (OD)
Top of embankment
Initial ground level
AV Colbond drains installed
Piezometer 6/4 at +0·7 m (OD)
Elevation: m (OD)
Contract days
1976
1977
1978
Movement: mm
I9
I8
S6/4
S6/5
Damaged
Drains installed
S6/5 at 0·6 m (OD)
S6/4 at −1·2 m (OD)

(c)



℄ Settlement: mm
Lateral movement on day 140: mm
℄ piezometric level: m (OD)
Elevation: m (OD)
OD
Day 110
Day 473
Day 708
London clay
I9
I8
OGL
Day 695
Day 478
Before embankment construction

(d) (e) (f)

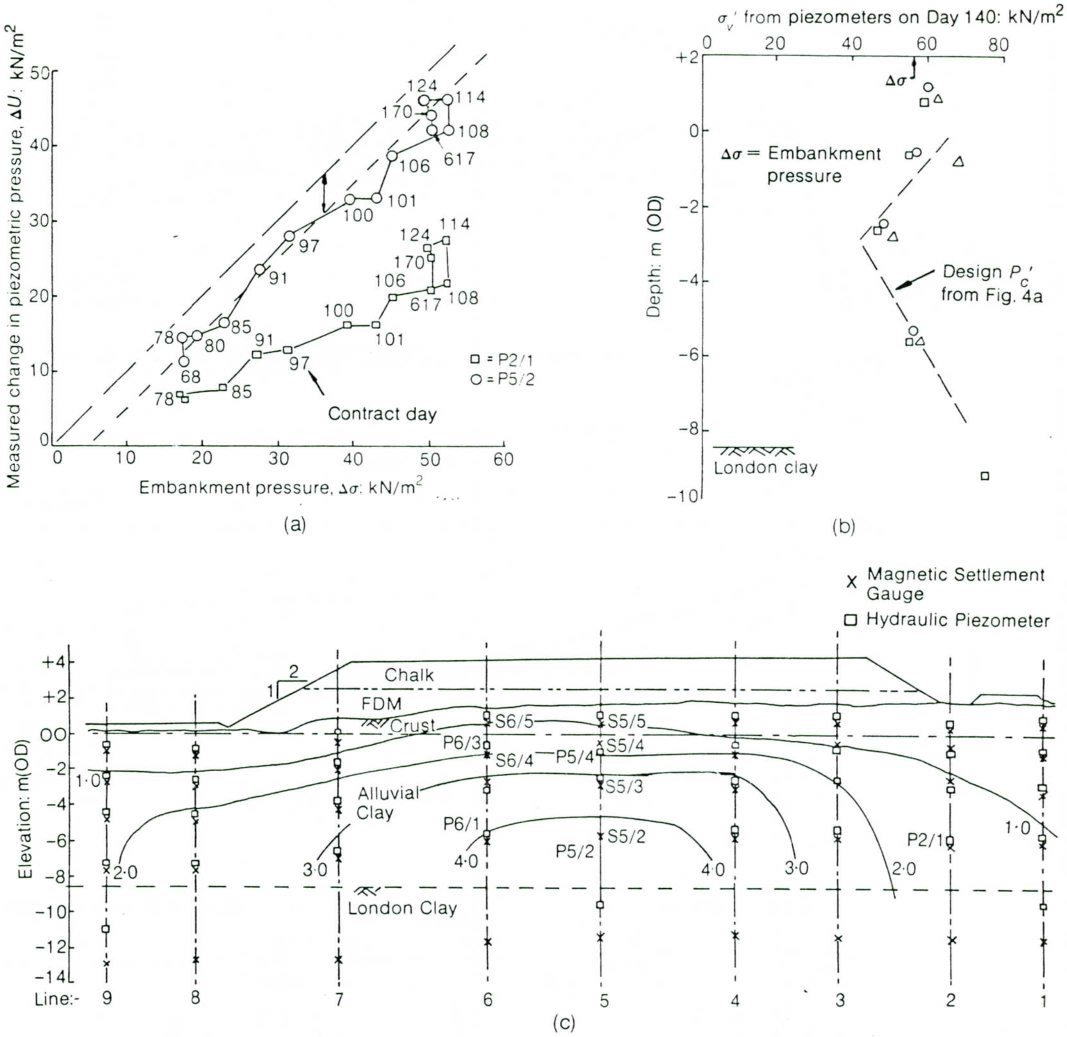


Fig. 7. Piezometer response to embankment loading; (a) piezometric pressure change with centreline embankment loading; (b) effective vertical stress profile on day 140; (c) section AA—contours of excess piezometric pressure on day 708

Shear strengths

The stability of the first stage of construction was governed by the original undrained shear strengths illustrated in Fig. 3(c). The field vane strength differs from the overall strength mobilized in embankment failures because of strain rate, anisotropy and progressive failure effects; the empirical correction factors proposed by Bjerrum (1973) were applied to the data in order to produce design parameters. One of the case histories used by Bjerrum to develop the vane correction factors was that of Scrapsgate, approximately 5 km north east of the Queenborough bypass site (Golder & Palmer, 1955).

Using these corrected strengths, undrained stability analysis showed that a maximum thickness of ground of about 3 m of fill could be placed in the first undrained loading stage. Construction to higher levels required a multi-stage construction technique with consolidation pause periods between lifts so that each increment of load could be balanced by a corresponding gain in strength. The multi-stage analysis was based on the assumption that consolidated undrained strengths c_u would grow in proportion to their maximum vertical preconsolidation stress p_c', and the calculations were therefore dominated by the choice of c_u/p_c' ratio (where c_u is undrained shear strength). Both isotropically consolidated undrained (CIU) and anisotropically consolidated (CAU) undrained triaxial compression tests were carried out over the range of effective stress anticipated under the embankment to assess the correct choice of the parameter c_u/p_c' as shown in Fig. 5(a).

The corrected vane test results were also normalized by the preconsolidation pressure given in Fig. 4(a) and are shown in Fig. 5(b). An overall ratio of $c_u/p_c' = 0{\cdot}25$ was selected after considering the likely effects of the change to plane strain conditions, anisotropy and strain rate dependence for the multi-stage case. Reference was made to the SHANSEP procedures of Ladd & Foott (1974), and to the work of Wesley (1975) on the Thames alluvial clays.

FIRST STAGE OF EMBANKMENT CONSTRUCTION

The first stage of embankment construction was placed during the summer of 1976 to the levels shown in Figs 2 and 6(b), at a construction rate shown in Fig. 6(c). The embankment consisted of a blanket of free-draining material comprising silty sand and gravel to a level of +2·5 m OD with chalk fill being placed above this level. The bulk density of the free-drainage material is about 22 kN/m^3, while the bulk density of the chalk is about 18 kN/m^3.

The positions of the instruments used to monitor the embankment are shown in Figs 6(a) and (b). Piezometric pressures were recorded by push-in twin tube hydraulic piezometers, while the vertical settlements were measured by magnetic settlement gauges and lateral movements were monitored by inclinometers. The examination of the first stage loading behaviour may be divided chronologically into the construction period up to contract day 140 (19 September, 1976) and the post-construction pause period from contract day 140 to contract day 708 (10 April, 1978).

Construction period

The construction period was characterized by a rapid buildup in piezometric pressures in the alluvial soils, some initial settlements and substantial horizontal movements. Some cracking of the chalk fill occurred along the centreline on placing the final lift of the first construction stage. To ensure stability the maximum thickness of fill was reduced slightly, with this material being used to form a 0·5 m thick toe surcharge in the critical areas.

The development of the vertical and horizontal displacements is best illustrated by the records of settlement gauges S6/5 and S6/4 and readings from inclinometers I8 and I9 measured 2 m below ground level (Fig. 6(c)). Typical displacement profiles for contract day 140 are shown in Fig. 6(d) and (e). Tavenas, Miessens & Bourges (1979) proposed that the non-dimensional ratios y_m/s (i.e. the maximum inclinometer movement at the toe divided by the centreline settlement) and its differential $\Delta y_m/\Delta s$ may be used to compare the displacement patterns beneath different embankments. The ratio $\Delta y_m/\Delta s$ interpolated for the first loading at Queenborough was approximately 0·9 after the soil reached preconsolidation pressures, and shows close agreement with Tavenas's study of 21 embankments from sites around the world.

The variation of piezometric level with time is shown in Fig. 6(c) for a shallow piezometer (P6/4 at +0·7 m OD) and two deep piezometers (P6/1 at −5·6 m OD and P5/2 at −5·4 m OD). Typical profiles of piezometric level are given on Fig. 6(f) for different contract days. Assuming an initial increase in pore pressure equal to the embankment pressure, the profile on contract day 110 indicates that a substantial drop in pore pressure occurred above −2 m OD, whereas below this level very little dissipation occurred. This effect is confirmed by the overall distribution of excess piezometric pressure at the completion of the first construction period (shown on Fig. 6(b) for contract day 140).

The linear response of piezometer P5/2 to embankment pressure at the centreline is shown in Fig. 7(a), together with that of P2/1 at −5·5 m OD below the embankment toe. The net effect of this dissipation is to increase the in situ vertical effective stresses in the foundation soils up to their preconsolidation pressures. This may be seen in Fig. 7(b), which compares the vertical effective stress data calculated from the piezometer readings for contract day 140, with the original preconsolidation pressure profile from Fig. 4(a). The largest changes in vertical effective stress, above −2 m OD, corresponded to the area with greatest vertical settlements as shown on Fig. 6(d). This process of rapid reconsolidation to p_c' followed by very slow dissipation above this stress level is also reflected in the 5 kN/m^2 difference between the centreline piezometer response and the applied embankment pressure for piezometer P5/2. This is similar to the difference between the original in situ vertical stress and the preconsolidation pressure for that depth (Figs 7(a) and 4(a)). The effects noted at Queenborough are in accordance with the findings of Rowe (1972), Marsland & Powell (1977) and Tavenas & Leroueil (1980).

Construction pause period

No additional construction took place from contract day 140 to contract day 708 and the period was therefore one of undisturbed natural drainage. The contours of excess piezometric pressure are shown on Fig. 7(c) for contract day 708 and these may be compared with the contours for contract day 140 shown on Fig. 6(b). These indicate that very little lateral drainage was occurring under the centre of the embankment.

The profiles of piezometric pressure (Fig. 6(f)) showed significant dissipation between -3 and -6 m OD which was also demonstrated by further settlement between these levels. However, the instrumentation below -6 m OD showed that very little consolidation was taking place below this level.

The changes in the profile of piezometric pressure during this period were modelled by a finite difference method after Gibson & Lumb (1953) assuming vertical drainage and a reasonable agreement could be demonstrated with a coefficient of consolidation (vertical) c_{vv} of $9\,m^2$/year above $-2{\cdot}5$ m OD, and $1{\cdot}0\,m^2$/year below this level.

VERTICAL DRAIN DESIGN CONSIDERATIONS

The analysis of the first stage of loading clearly indicated that natural drainage would be insufficiently rapid to allow subsequent stages of filling to be placed within the four year construction programme. The use of large loading berms to assist construction was restricted by the presence of the existing railway track and availability of land. Consolidation assisted by vertical drains was considered to be the most efficient means of completing the embankment within these constraints.

The design requirements for the vertical drains were based on the following three criteria. Firstly, the rate of consolidation and increase in strength of the alluvium had to be sufficiently rapid to allow completion of the remaining stages of embankment construction while maintaining a minimum factor of safety of 1·25. Secondly, the primary consolidation at the abutments had to be at least 95% complete before bridge construction work could commence to prevent excessive ground movements near the piled bridge foundations. Thirdly, the remaining settlement on completion of the road surface had to be such that significant differential settlements would not occur in the approach embankment, particularly adjacent to the bridge abutment. A design differential settlement limit of 1:250 was used in the calculations with the provision that, at road construction stage, the vertical alignment would be finally adjusted to account for remaining primary and secondary consolidation.

As the first stage of loading had shown the Queenborough clays to be relatively impermeable, it was considered that a vertical drain system would generate a nearly one-dimensional radial consolidation pattern and that Barron's (1948) simple equal strain theory could be employed for design calculations. The main design problems then centred on the selection of the following parameters:

(a) the range of the in situ 'undisturbed' coefficient of vertical consolidation due to horizontal radial drainage c_{vh} which, being a combined permeability and compressibility parameter, was expected to vary with effective stress, since

$$c_{vh} = \frac{k_h}{m_v \gamma_w}$$

where k_h = horizontal permeability; m_v = modulus of volume change; and γ_w = unit weight of water; c_{vv} and k_v similarly apply to vertical drainage conditions

(b) the equivalent cylindrical drain diameter for non-circular strip drains

(c) the effect that remoulding and smearing produced during installation has on the drain's performance

With reference to parameter (c), Richart (1959) and Johnson (1970) have shown that reductions in the drain's effective diameter could be made to model such effects, but the magnitude of any reductions was not known. Alternatively, the uncorrected cylindrical drain diameter could be used and the smear and remoulding effects incorporated into a reduced 'bulk' value of c_v.

To assess these factors a study of laboratory and in situ coefficients of consolidation and an examination of the performance of two drain systems in a full-scale trial were carried out.

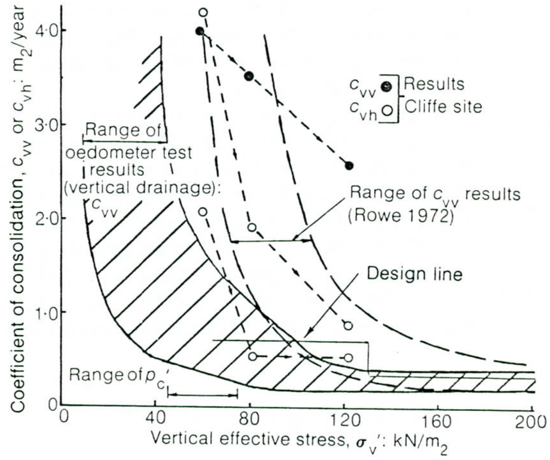


Fig. 8. Laboratory consolidation parameters

Laboratory consolidation parameters

The range of c_{vv} from 75 mm consolidation tests is shown on Fig. 8 plotted against vertical effective stress. Also shown is the band of results given by Rowe (1972) from 250 mm hydraulic oedometer tests for his D-type clay into which group the Queenborough clays broadly fall. The results of both vertically and horizontally draining 250 mm Rowe cell tests on similar soils from a nearby site at

Cliffe are also given. All the tests showed the same overall trend for the coefficient of consolidation to fall steeply with increasing effective stress. The value of c_{vv} of 1 m^2/year back-calculated from the dissipation of pore pressures after the first stage of embankment loading are seen to be in general agreement with the laboratory data. The vertical effective stresses under the centreline increased to 60–70 kN/m^2 by contract day 708. On completion of the embankment vertical effective stresses of up to 200 kN/m^2 are anticipated.

IN SITU PERMEABILITY TESTS

Two different types of constant head permeability tests were made. The simpler method made use of the artesian pressures acting in the alluvium under the embankment (Fig. 9(a)). A constant head reduction was created at the cells of a number of twin tube hydraulic piezometers by disconnecting one of the tubes and clamping it at a fixed level in the gauge house. The flow rate from this tube was monitored with a measuring cylinder, and the precise head change at the cell was found from the mercury manometer connected to the remaining piezometer limb.

The second method of testing was a miniature pumping test carried out using Casagrande piezometers. The standpipe water levels were drawn down using the permeability apparatus described by Jardine (1979). The test equipment (Fig. 9(b)) used a 12 V small diaphragm vacuum pump to achieve a constant head condition at the base of the suction pipe. The rate of groundwater flow into the piezometer was monitored by the rate of collection in the graduated separator traps at the ground surface. Further details of the equipment may be obtained from the manufacturer's data sheet (Jaybury Ltd, 1978).

Calculations of permeability and test requirements

Field permeability is usually found from a steady state flow calculation. However, in clayey soils, the test involves a consolidation process and the steady state flow rate q_∞' is only approached after a considerable interval of time. This consolidation process is dependent on the initial effective stress and pore pressure regime induced in the ground surrounding the piezometer cell (Gibson, 1970). This in turn is a function of the magnitude and sign of the initial change in head in the piezometer, the rigidity of the piezometer cell itself, and the soil's undrained pore pressure behaviour.

Where a rigid piezometer cell is used, such as the hydraulic push-in piezometer, a draw-down test will cause no appreciable change in cell volume. As a result there are no changes in total stress around the cell and the initial effective stress distributions in the soil will remain unchanged. This corresponds to $A = 1/3$ in Gibson's solution (where A is the pore pressure parameter). However, when a loosely deposited sand cell is used the draw-down will lead to rapid consolidation of the sand and causing a change in total stress and porewater pressures in the surrounding soils.

Further study of Gibson's work indicates that the consolidation process becomes independent of the initial boundary conditions once the time factor $T > 1$, where $T = c_{vh}\, t/a_e^2$ and t is the time after the start of the test on a piezometer of equivalent spherical radius a_e. At this stage in the test it is possible to extrapolate linearly the data on a q versus $1/\sqrt{t}$ plot to q_∞ at $1/\sqrt{t} = 0$. In the case of the hydraulic piezometers ($a_e = 40$ mm) this condition was always met. However, the larger Casagrande standpipe piezometer tests did not achieve this time factor requirement. In these cases the value of c_v was obtained by fitting Gibson's curves (Gibson, 1970, Fig. 1) relating c_v, q, t and the pore pressure parameter A to piezometer data plotted as q versus $1/\sqrt{t}$. Figure 9(c) shows a range of curves which have been fitted through a specific data point at the end of an 8·25 h test that satisfy Gibson's solution for $m_v = 1{\cdot}5$ m^2/MN and three values of A.

The plot of field data falls between the theoretical limits of a perfectly rigid cell, and a perfectly compressible cell with $A = 1$. Within these bounds the value of q_∞ (and hence c_v) may be roughly estimated with reasonable confidence from the results of a single day's testing. It is preferable to extend the test until the time factor reaches a value of about 1. For $c_v = 0{\cdot}3$ m^2/year such a time factor is reached after approximately two weeks.

Recent testing at various sites in the UK and Ireland using commercial versions of the apparatus has shown that, when the permeability is too low for a suitable value of T to be reached in a single day's testing, the flow into the standpipe usually becomes low after the first 10 h of pumping. With these low flow rates the withdrawal of water may be carried out intermittently; pumping can often be stopped overnight without seriously affecting the constant head condition, thus enabling the test to be extended indefinitely. Alternatively, a smaller diameter of sand cell may be used to achieve higher values of T within a given period. At the completion of the test further confirmation of the permeability may be obtained by monitoring the recovery curve as a rising head test.

Results

The coefficients of consolidation c_{vh} were calculated from the permeability data by using the coefficient of volume compressibility m_v obtained from the laboratory consolidation tests over the relevant effective stress range. The calculated coefficients of consolidation are plotted against the

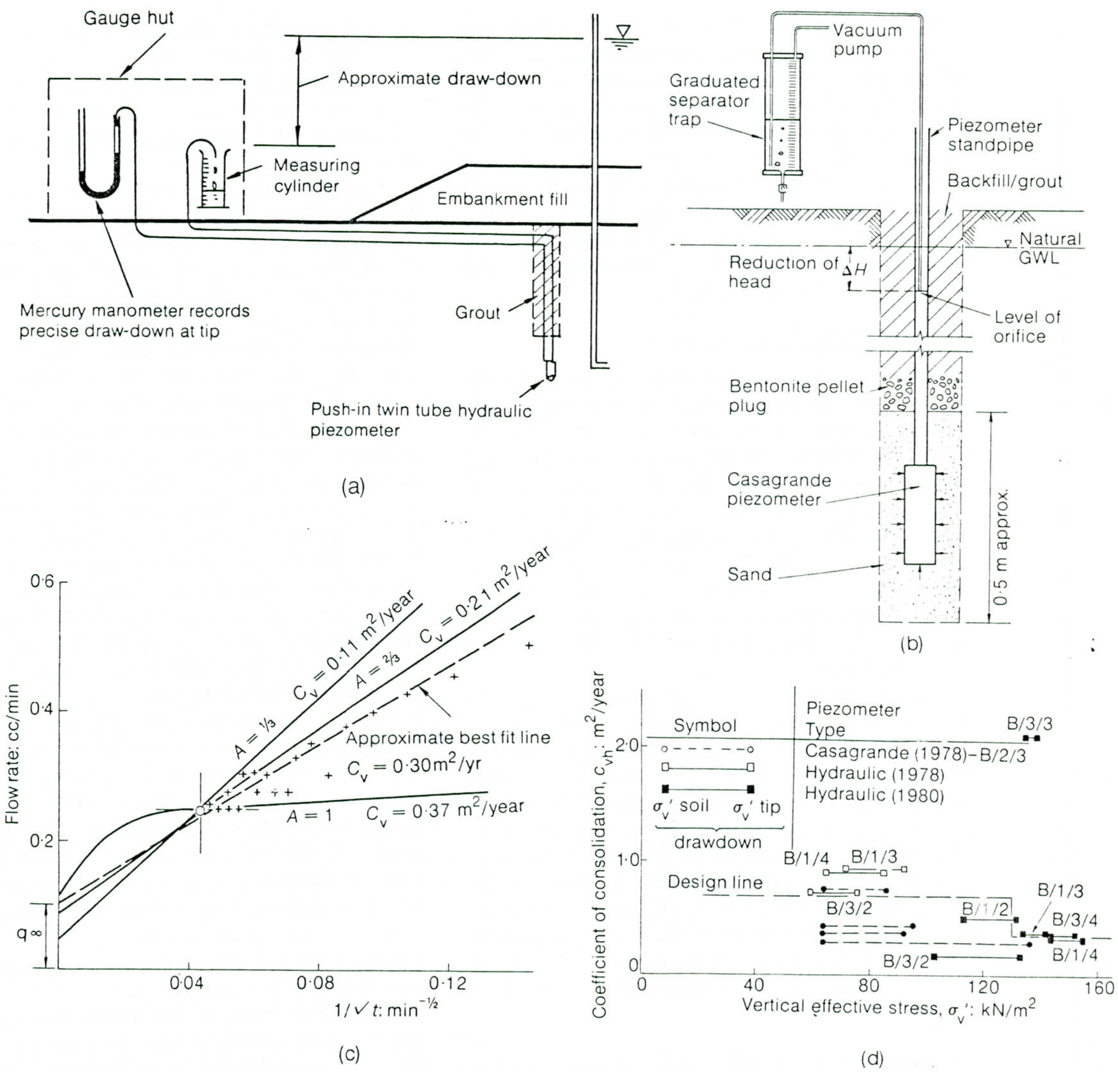


Fig. 9. In-situ permeability tests; (a) hydraulic piezometer system; (b) standpipe piezometer system; (c) test results; solid lines represent 'theoretical' curves caused to pass through point of intersection for the following assumptions: in spherical piezometer, 0·14 m radius; compressibility from oedometers, $m_v = 1{\cdot}5\,m^2/mN$; piezometer draw-down = 7·2 m; (d) coefficients of consolidation

effective stress range induced in the soil during the test in Fig. 9(d). The reduction in c_{vh} caused by increasing the draw-down on the Casagrande standpipe piezometer and therefore the effective stress around the cell may be seen in Fig. 9(d).

In 1980 some additional permeability tests were made on the same hydraulic peizometers tested in 1978. During these two years consolidation had occurred in the soil under the embankment and the in situ vertical effective stresses had increased by about $70\,kN/m^2$. The coefficient of consolidation has decreased by a factor of about 3 as the vertical effective stress has increased.

VERTICAL DRAIN TRIALS

During the preliminary assessment of alternative ways of completing the embankment, several vertical drain systems were considered including driven, augered and jetted sand drains as well as the various band and wick drain systems. However, the need to install the drains through the existing embankment without impairing its integrity limited the trials to the Sandwick and AV Colbond systems.

Sandwick drains

The Sandwick drain was developed in India (Dastidor, Gupta & Ghosh, 1969; Hughes & Chalmers, 1972). It consists of a 65 mm dia. porous woven fabric sock filled with medium sand. For the trials the drains were installed by predrilling holes through the embankment fill using rotary percussive air flush drilling rigs. A 95 mm outside dia. hollow mandrel, with a disposable shoe at its base, was then driven, with the assistance of a vibratory hammer mounted on the top of the mandrel, to a depth of 13·5 m below the top of the embankment. A 12 m long Sandwick drain was then lowered into the mandrel which was subsequently removed leaving the drain in place. These drains had been used previously under similar circumstances at Sandwich bypass (Cole & Garrett, 1981). The Sandwick drain was provisionally assumed for preliminary purposes to have an equivalent theoretical diameter of 65 mm.

AV Colbond drains

The AV Colbond drain is a more recent development (van der Elzen, Risseeuw & Beyer, 1977). It comprises a strip, 300 mm wide by 4 mm thick, of polyester fabric material. The drains were installed by initially preboring a 0·3 m dia. hole through the 1·7 m of chalk and loosening the upper 0·5 m of free-draining material using a crane-mounted auger (Fig. 10(a)). A 165 mm dia. lance was then driven, using vibration, to a depth of 13·5 m. A disposable anchor plate retained the fabric in position in the soil when the lance was retracted. To induce the fabric to leave the lance and remain in the ground as a 300 mm wide strip, the end of the lance has been splayed out (Fig. 10(b)).

Reports in the literature by Kjellman (1948) and Hansbo & Torstensson (1977) suggest that 100 mm strip drains gave performances equivalent to cylindrical sand drains of similar peripheral length. For the 300 mm wide AV Colbond strip drain such a relationship gives an equivalent diameter of 194 mm. Risseeuw & van der Elzen (1977), however, suggested that the AV Colbond drain was equivalent to a 250–300 mm dia. sand drain. An equivalent diameter of 220 mm was assumed in the design of the drainage trial.

Layout of trials

Two trial areas were prepared; area A consisted of 256 AV Colbond drains, and area B consisted of 406 Sandwick drains, as shown on Figs 1 and 10(c). The Sandwick drains were set out on a 1 m square grid which allowed a direct comparison with a previous trial at Sandwich bypass, and the AV Colbond drains were set out on a 1·35 m square grid to give theoretically similar rates of consolidation based on the equivalent drain diameters discussed previously. These drains were installed between 12 and 20 April, 1978.

Instrumentation of trials

Use was made of the existing instrumentation at trial area A to monitor the AV Colbond drains. Some additional instruments were added to replace those damaged during the installation of the drains. Details of the operational instruments are given in Fig. 11(a). Additional instrumentation was installed to monitor the Sandwick drains in trial area B (Fig. 11(b)). Every effort was made to ensure the grid of drains was placed squarely round the instruments.

To monitor the hydraulic head loss between the base of the drain and the drainage blanket, standpipe piezometers were attached to some of the vertical drains (Fig. 11(c)). These indicated a maximum of 0·5 m head loss between the base of the drains and the drainage blanket for both types of vertical drains.

A number of measurements were made of the volume of water flowing from instrumented vertical drains. These showed the volume flowing agreed with the settlement multiplied by the area of influence of the individual drain over the same period of time.

Results of trials

Typical piezometer and settlement gauge records for both trial areas are shown in Fig. 12(a) and 6(c). Increases in piezometric levels of up to 3 m were observed in both areas as the drains were installed; these were caused by large increases in horizontal stress induced by the driving process. The piezometric levels then decayed rapidly to their pre-installation values. These large changes in piezometric pressure were achieved with small settlements of around 45 mm in both areas, indicating the relatively low compressibility of the soil during this phase of consolidation. The piezometric level and settlement profiles shown in Fig. 12(b) illustrate the relatively uniform nature of the dissipation throughout the full depth of the alluvium as water flowed from the soil into the drains and then to the surface.

A comparison of the rates of dissipation of excess head, taking the time origin from when the piezometric level returned to its pre-installation value, is given in Fig. 13. This shows that dissipation was more rapid in the Sandwick trial area B than in the AV Colbond trial area (Fig. 12).

For comparison the dissipation curve for a 1 m square grid of drains of 65 mm equivalent diameter is also shown in Fig. 13(a) computed from Barron's theory with a coefficient of consolidation c_v of 1 m^2 year. The curve fits the field data reasonably well between day 20 and 60. However, for the AV Colbond trial area the same theoretical curve was

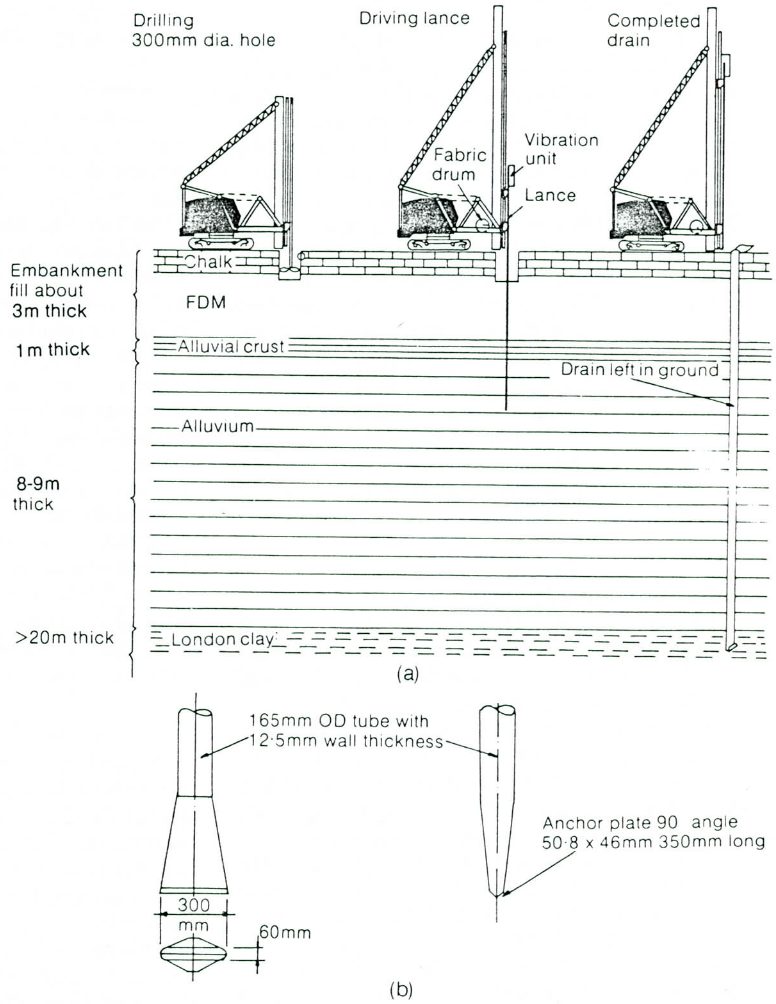


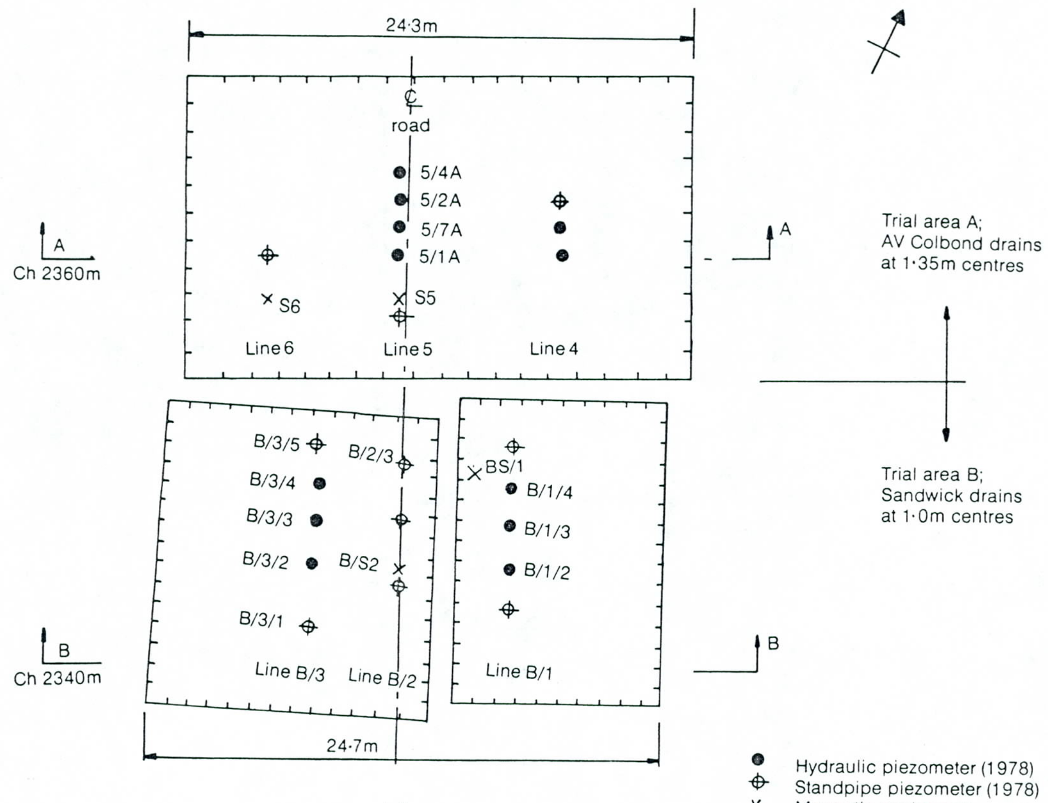
Fig. 10. (Above and facing page.) Vertical drain trials: (a) AV Colbond drain installation method; (b) details of AV Colbond installation lance; (c) layout of drain trial areas

only found to fit well if the equivalent drain diameter was reduced to 120 mm, which was considerably less than the 220 mm diameter used for the provisional design. A possible explanation for the anomaly may be that the edges of the strip were folded inwards or became compressed during the installation process. On the basis of the trial results the AV Colbond drains would be required at 1·15 m centres to achieve a similar rate of dissipation to the Sandwick drains. In both cases the recorded behaviour suggests a continuous slowing of dissipation associated with a reduction in the coefficient of consolidation with increasing vertical effective stress.

PERFORMANCE OF SUBSEQUENT STAGES OF EMBANKMENT CONSTRUCTION

The vertical drain trial confirmed the tendency for c_{vh} to fall with increasing effective stress and also gave reliable equivalent drain diameters for the full-scale design. The design coefficient of consolidation was taken as 0·7 m^2/year for effective stresses below 130 kN/m^2, and 0·35 m^2/year above that value after considering the field permeability tests (Fig. 9(d)), the laboratory consolidation data (Fig. 8) and the results of the drainage trial. Alternative designs were worked out for each system and separate tenders were invited.

On this basis the Sandwick system proved to be



(c)

the more economical and 7500 Sandwick drains were installed in the remaining embankments between October and December 1978. Details of the Sandwick drain layout are shown in Fig. 14. The drains were installed across the full width of the embankment, the square grid spacings in the blocks being progressively increased from 0·9 m at the abutment to a maximum of 3 m away from the viaduct (Fig. 14).

The remaining stages of embankment construction were made using pulverized fuel ash fill (PFA) with a bulk density of about 13 kN/m^3. Details of when this fill was placed and the response of the instruments in the Sandwick and AV Colbond trial areas are shown in Figs 15 and 16. Also shown are the stability control charts which give the upper piezometric level for various embankment elevations to maintain the minimum factor of safety. These were used as the primary construction control method. The predicted settlement required before a further 0·5 m lift of PFA could be added is also shown. This was based on the laboratory consolidation tests and used as a secondary control to check the overall consolidation of the alluvial clay. Good agreement can be seen between the predicted and actual settlements shown in Figs 15 and 16 immediately before placing additional lifts. By contract day 1580 (September 1980) the Sandwick trial area had settled 1·2 m compared with the 1·0 m in the AV Colbond trial area. Profiles of piezometric level and total settlement are shown in Fig. 17. The settlements were seen to develop uniformly through the alluvial clays and the piezometric profiles were seen to be similarly uniform with the exception of instrument B/3/3 which was apparently installed in a more permeable peaty layer found at −4·7 m OD. The permeability test on piezometer B/3/3 showed the soil around the tip to be up to 10 times more permeable than at other piezometers. The development of lateral displacements may also be seen from Fig. 15 which shows the plot for the maximum movements at 3 m depth for Inclinometer 19.

From contract day 1060 to contract day 1600 the lateral movements occurred at a fairly uniform rate. The ratio of changes in the lateral movements to

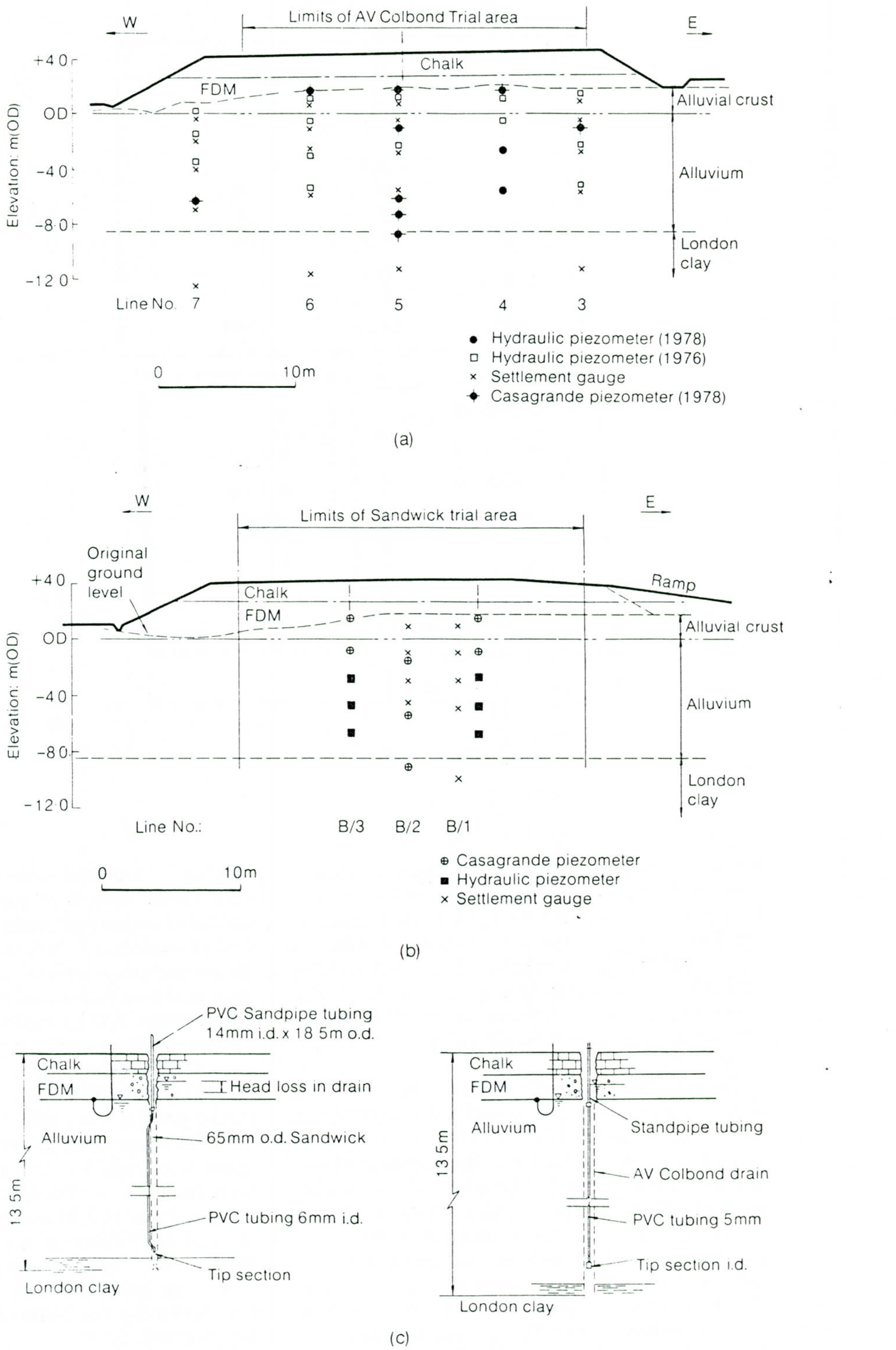


Fig. 11. Trial instrumentation: (a) AV Colbond area section A–A on Fig. 10(c); (b) Sandwick area section B–B on Fig. 10(c); (c) standpipe piezometers attached to vertical drains

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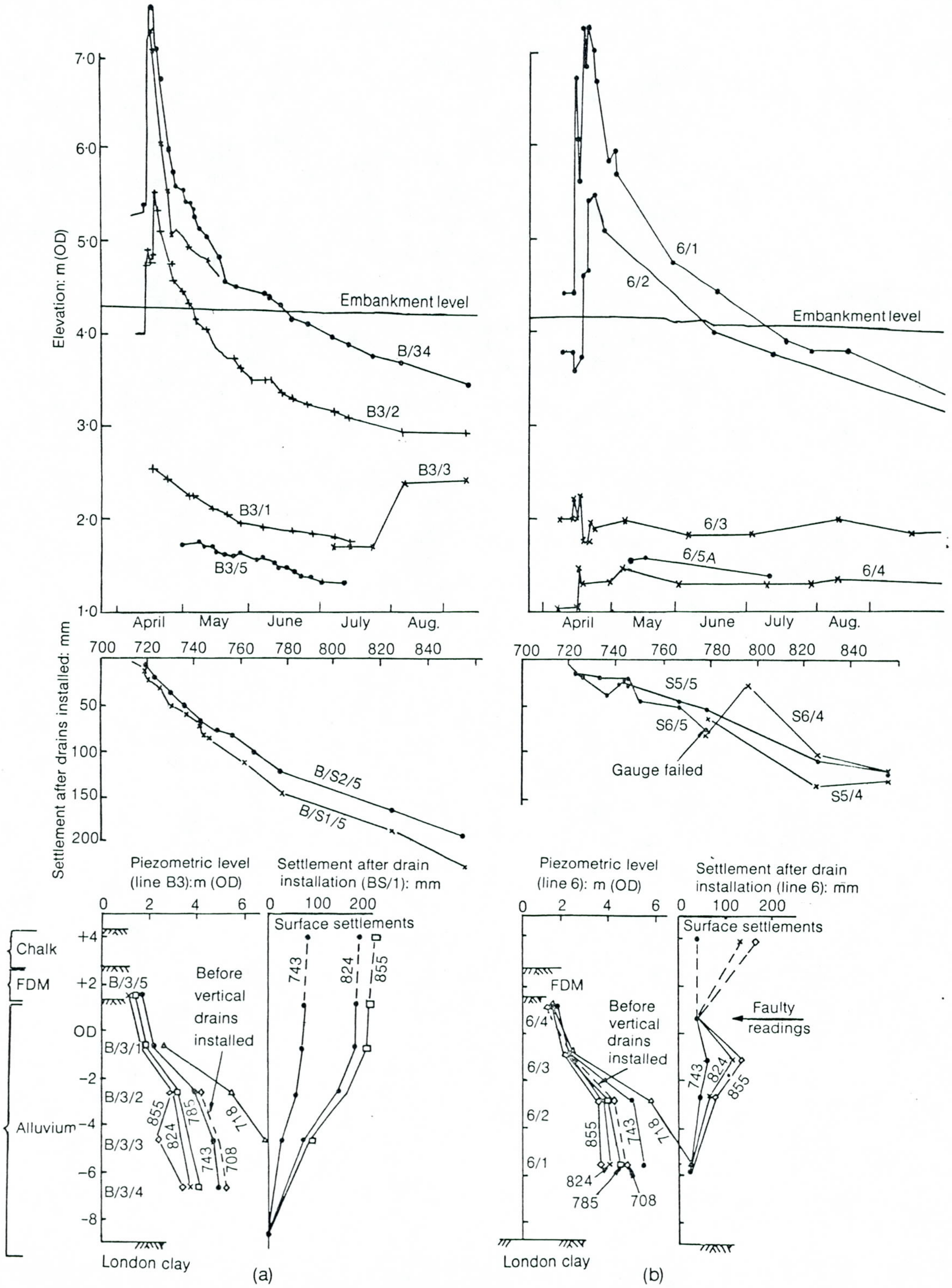

Fig. 12. Trial results: (a) Sandwick area (piezometer line B3); (b) AV Colbond area (piezometer line 6)

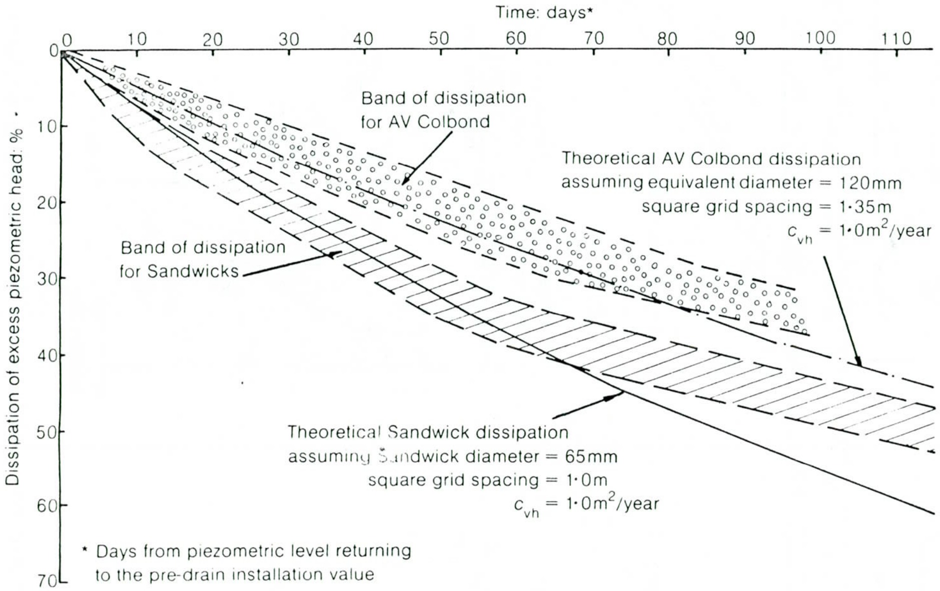

Fig. 13. Comparison of trial area dissipation rates

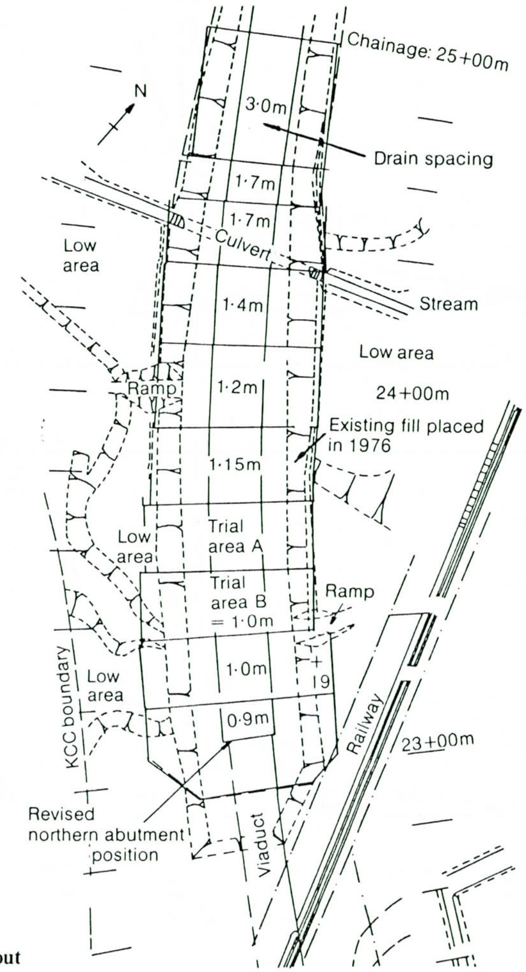

Fig. 14. Plan of northern approach embankment with drain layout

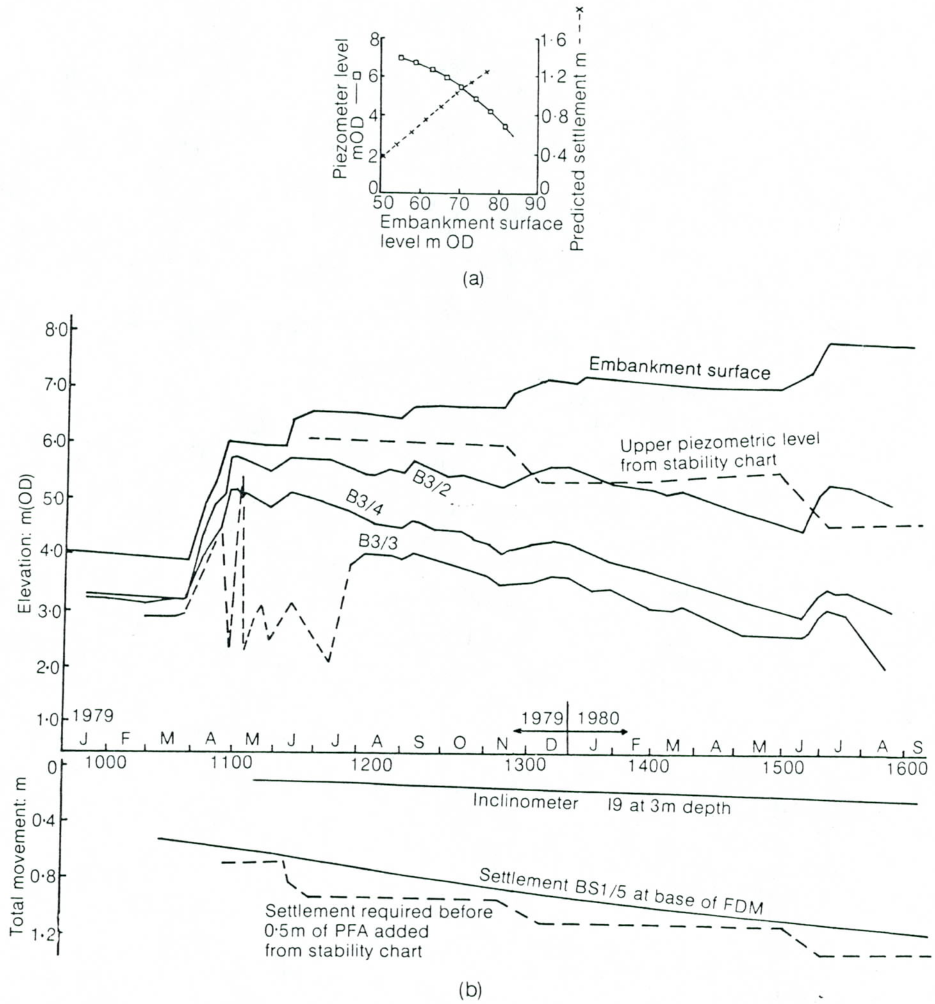

Fig. 15. Sandwick performance; contract days 1000 to 1600

centreline vertical settlement was about 0·20. From an analysis of case histories, Tavanas & Leroueil (1980) suggest a ratio of 0·09 for the 1 in 2·3 side slopes for consolidating single stage embankments. The placing of subsequent lifts has not been accompanied by large increases in lateral and vertical movement.

DISCUSSION OF RESULTS

Sufficient field dissipation has now occurred to enable the variation of coefficient of consolidation c_{vh} with effective stress to be calculated from the piezometer records between contract day 700 and 1600. This has been done by assuming only radial drainage and applying Barron's solution which gives

$$u_r = \frac{4\bar{u}}{d_e^2 F_n}\left[r_e^2 \ln\left(\frac{r}{r_w}\right) - \frac{r^2 - r_w^2}{2}\right] \qquad (1)$$

u_r = excess pore pressure at radius r

$\bar{u}$ = average excess pore pressure

d_e = equivalent diameter of drain influence

r_e = equivalent radius of drain influence

r_w = equivalent radius of drain

$$F_n = \frac{n^2}{n^2 - 1}\ln(n) - \frac{3n^2 - 1}{4n^2}$$

$$n = \frac{r_e}{r_w}$$

Fig. 14. Plan of northern approach embankment with drain layout

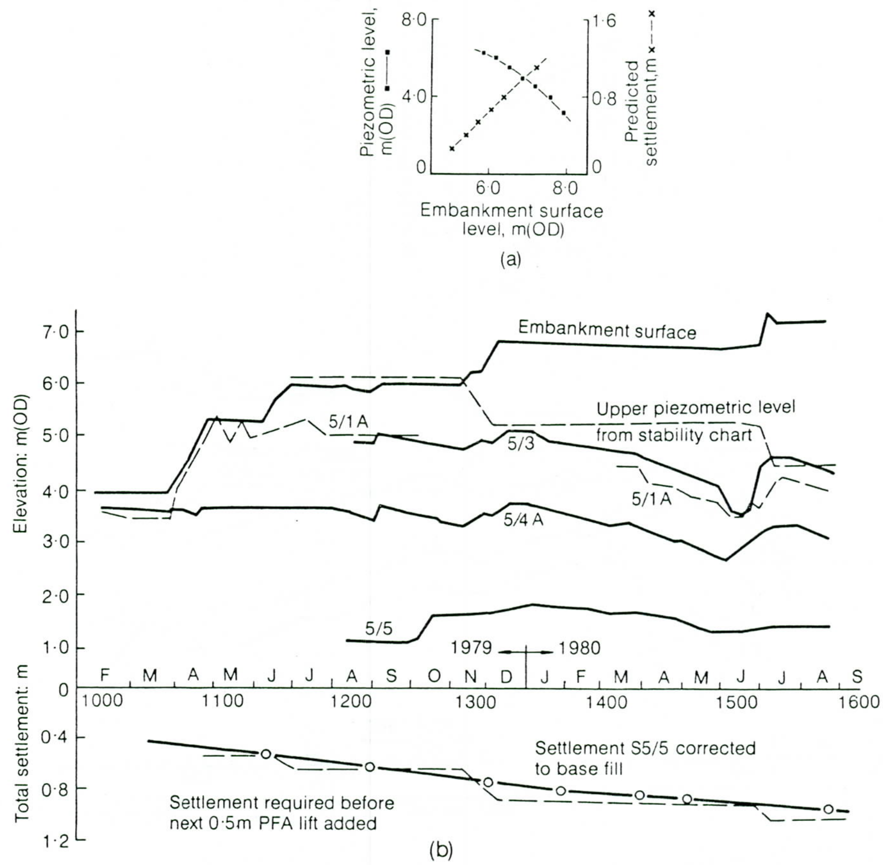

Fig. 16. AV Colbond performance; contract days 1000 to 1600

The relationship between u_r and $\bar{u}$ expressed in equation (1) is a function of the geometry and independent of time. A result of this equation is that a piezometer placed near the centre of the grid should express a constant proportion of the average pore pressure.

The variation of $\bar{u}$ with time is

$$\bar{u} = u_0 e^{\lambda} \tag{2}$$

u_0 = initial average pore pressure

$$\lambda = \frac{-8T_h}{F_n} = \frac{-8c_{vh}t}{F_n d_e^2}$$

Differentiating equation (2) with respect to time and substituting equation (1)

$$\frac{d\bar{u}}{dt} = \frac{-8c_{vh}}{F_n d_e^2}\bar{u} \tag{3}$$

or

$$\frac{du_r}{dt} = \frac{-8c_{vh}}{F_n d_e^2}u_r \tag{4}$$

The bulk coefficient of consolidation (horizontal) c_{vh} has been back-calculated using equation (4), taking the excess pore pressure and rate of dissipation from the piezometer records over increments of time when the embankment load remained constant. The coefficients of consolidation c_{vh} calculated by applying Barron's theory in this manner are shown in Fig. 18 for six piezometers in the Sandwick trial area and four in the AV Colbond area. A similar variation of c_{vh} with effective stress may be seen.

The in situ permeability tests which were also performed on the same piezometers in the Sandwick trial area (Fig. 9(d)) show similar values of c_{vh} to those obtained from Barron's theory shown in Fig. 18. Furthermore, in this case the c_{vh} values are similar to those obtained from the laboratory consolidation tests shown in Fig. 8.

The permeabilities calculated from the in situ tests described in the Paper do not necessarily correspond to the undisturbed field horizontal permeability because the piezometer installation

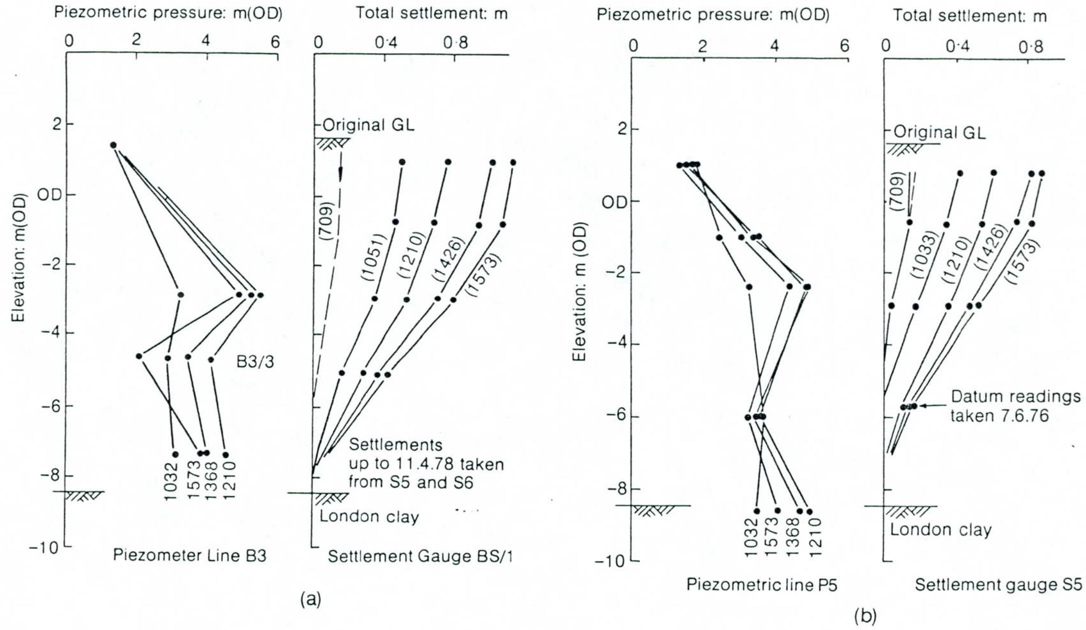

Fig. 17. Comparison of piezometric and settlement profiles; contract days 700 to 1600: (a) Sandwick trial area; (b) AV Colbond trial area

process may have caused local remoulding and smearing of the surrounding soil. This effect has been demonstrated by Jezequel & Mieussen (1975). However, when the cell installation method is similar to that proposed for the vertical drain system the draw-down in situ test provides a method of obtaining the bulk permeability which incorporates these local effects. The in situ test also subjects the soil to hydraulic gradients similar to those encountered around the vertical drain.

The exact correspondence between the laboratory consolidation, in situ permeability and full-scale performance is not easy to analyse. The theory developed for all three cases is based on a constant coefficient of consolidation. In practice, this parameter varies with effective stress. This limitation can be overcome by analysing the dissipation over small increments of effective stress, i.e. in the manner adopted for the full-scale test results.

In constant head permeability tests where large variations of draw-down are considered, the distribution of piezometric pressure and total stress between the piezometer boundary and some distant radius is not known. However, by incrementally increasing the draw-down the sensitivity of the bulk horizontal permeability k and hence bulk c_{vh} can be found.

At Queenborough the similarity of the results obtained from the small driven hydraulic piezometer and the large augered standpipe piezometer cells indicates the uniformity of the alluvial clay and its insensitivity to smear and remoulding caused by installation. The uniformity of the clay also explains the unusual agreement between the laboratory consolidation tests and the field performance. Draw-down tests at other sites have yielded horizontal permeabilities in excess of the 76 mm oedometer laboratory (vertical) values when the soils have contained important macro-fabric systems, such as silty laminations.

A problem encountered with the interpretation of the in situ permeability test was that of identifying the appropriate vertical effective stress to which the bulk c_{vh} derived from that test applied. The test is made over an effective stress range which varies with the distance from the cell wall as shown by the bars on Fig. 9(d). It it tentatively suggested that the average stress be adopted from the experience to date.

The excess piezometric pressures set up by the drain installation process dissipated rapidly (Fig. 12). Back-calculated c_{vh} values for this stage of dissipation, where the vertical effective stresses were less than the values before drain installation, are also shown in Fig. 18. These initial c_{vh} values were often 10–20 times the values calculated for effective stress increments in excess of the preconsolidation pressure. This reflects the rapid variations in compressibility and permeability which should be considered in the interpretation of

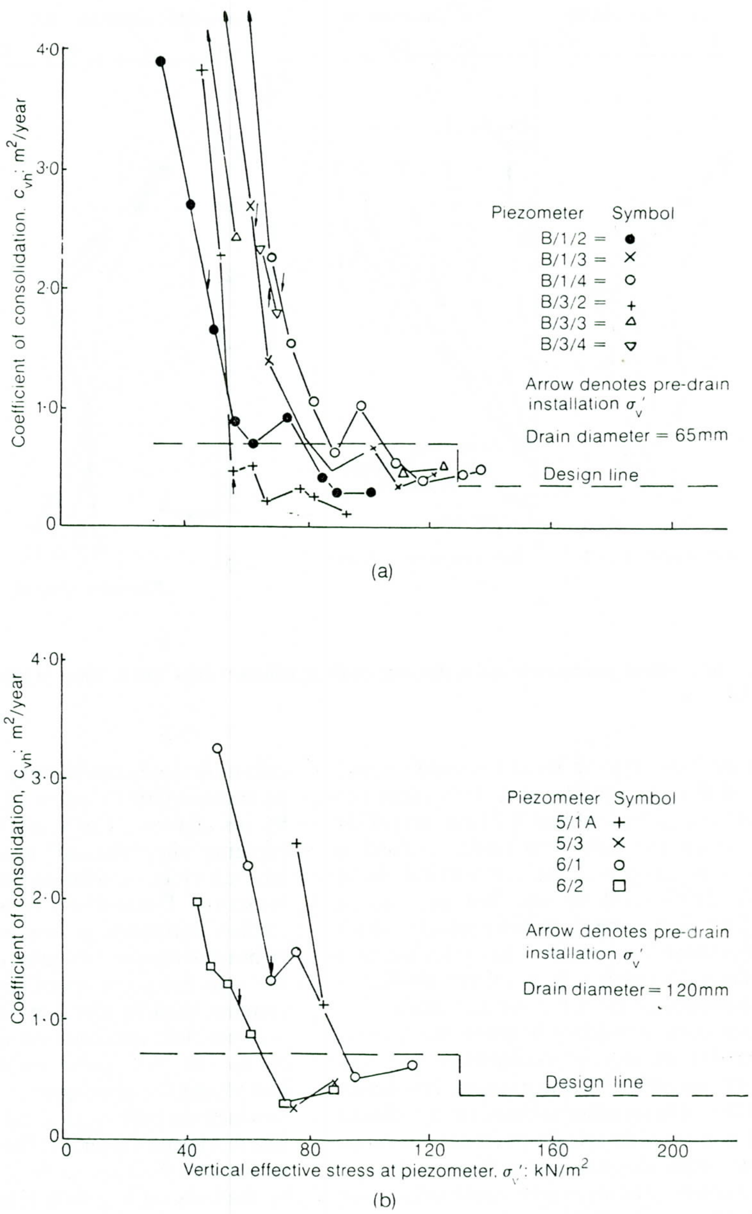

Fig. 18. Comparisons of c_{vh} from piezometer dissipation; (a) Sandwick drains trial area; (b) AV Colbond drains trial area

the vertical drain trial results.

Other in situ methods of obtaining a field coefficient of consolidation have recently been developed. They include the pressuremeter test (Clarke, Carter & Wroth, 1979) and the pore pressure sounding probe (Torstensson, 1975; Hanbro & Torstensson, 1977). These methods are based on monitoring the decay of excess pore pressures caused by expansion of the pressuremeter or driving the sounding probe. From the information set out in this Paper it would be expected that the coefficients determined by these new methods would be appropriate to problems of drive piles rather than embankment construction on soft clays where the preconsolidation pressure is often exceeded.

It is therefore recommended that the appropriate in situ test which models the proposed construction work should be selected. In the case of embankment construction where consolidation is assisted by vertical drains the draw-down in situ permeability test on a piezometer installed in the appropriate manner is suggested. This may be combined with the m_v obtained from laboratory consolidation tests to obtain a bulk c_{vh} value.

CONCLUSIONS

This case history highlights the problems involved in determining the rate of dissipation when reliance is placed on the continuity of horizontal permeable drainage layers within the alluvium. At Queenborough, vertical drains have been used successfully to improve the rate of dissipation enabling the bypass approach embankments to be completed within the construction programme. Barron's equal strain theory provided an adequate model of field performance once allowance had been made for the variation of the coefficient of consolidation with effective stress, and field trials had been made in order to assess the equivalent diameters of the vertical drain systems.

The assessment of the coefficient of consolidation c_{vh} is of paramount importance for the successful design of any vertical drain scheme. This parameter is shown to vary with the type of loading process and the effective stress level, particularly around the preconsolidation pressure. The use of draw-down in situ constant head permeability tests in conjunction with laboratory consolidation tests is discussed and the results obtained are found to agree with those back-calculated from the field performance of the drains.

Full-scale trials were made on 65 mm dia. Sandwick drains and 300 mm wide AV Colbond drains to assess their field performance and compare their theoretical equivalent diameters. The field dissipation was found to fit equal strain theory when an equivalent drain diameter of 65 mm was used for the Sandwick drain and 120 mm for the AV Colbond drain. This latter figure was less than the equivalent diameter of 220 mm on which the drain trial design was based. A possible explanation may be that edges of the fabric strip had folded inwards or had become compressed. Hydraulic head losses measured in both types of drains did not exceed 0·5 m of water.

Settlements predicted from one-dimensional consolidation tests agreed well with those observed in the field. Changes in lateral movement at the toe during the first stage of construction amounted to about 90% of the centreline settlement. During subsequent construction stages after drain installation this figure dropped to 20%.

ACKNOWLEDGEMENTS

Queenborough bypass forms part of Kent County Council's road schemes and the Authors are grateful to the County Surveyor, Mr A. D. W. Smith, for his support in this work and for permission to publish this Paper. Thanks are also due to the staff of Ove Arup and Partners and the KCC Highways Laboratory who jointly have carried out the work described in this Paper.

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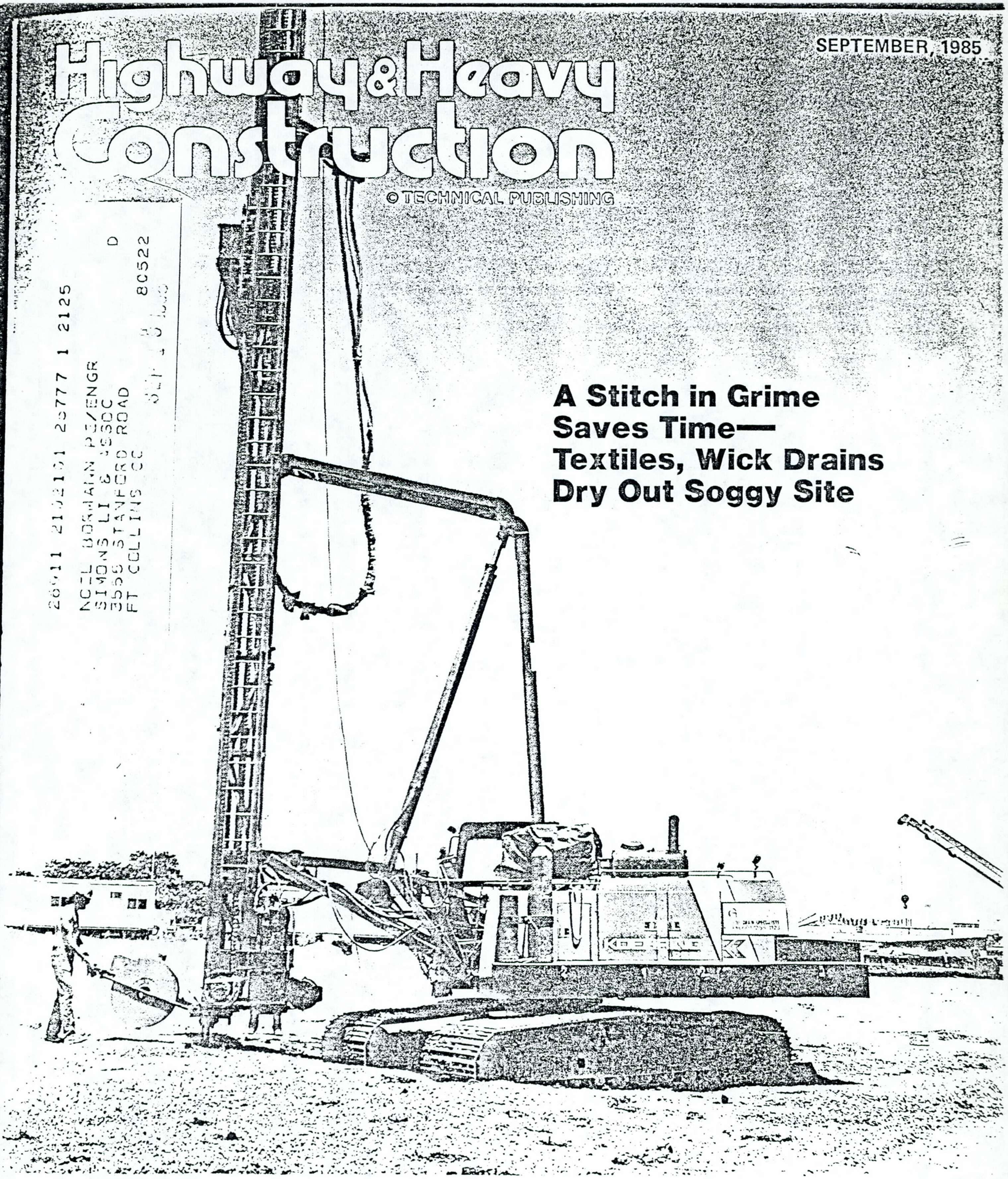
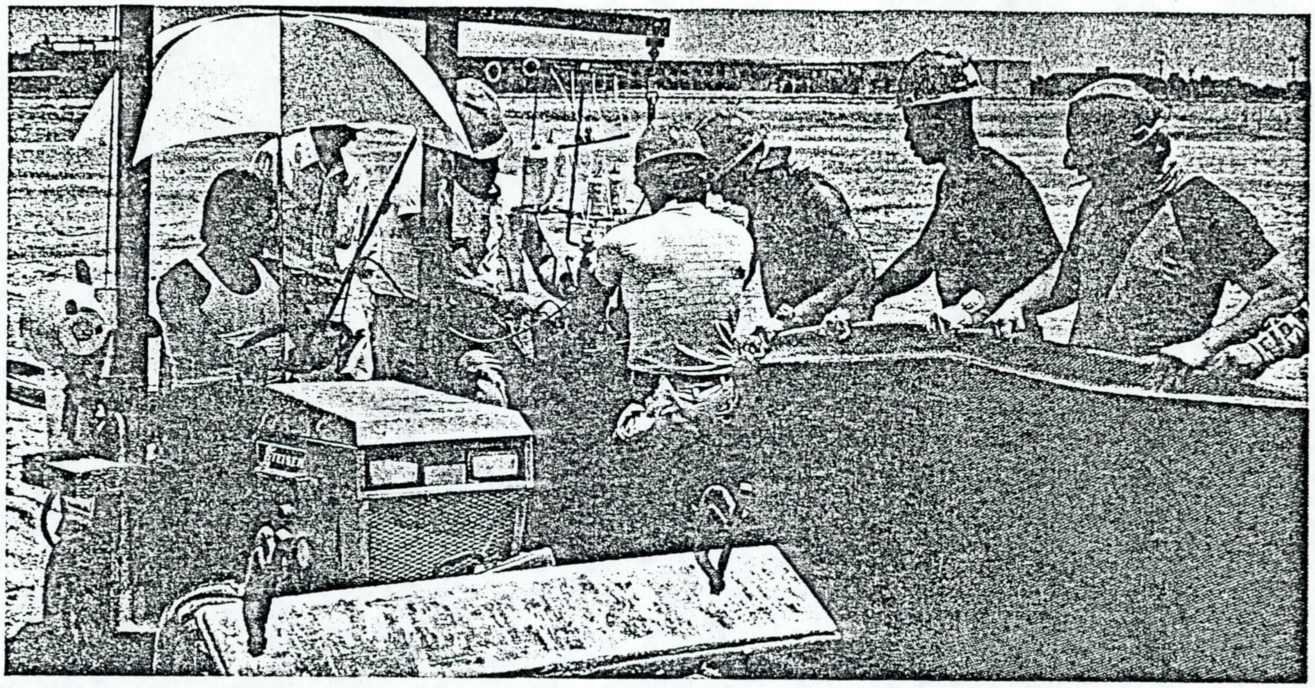


A Stitch in Grime Saves Time— Textiles, Wick Drains Dry Out Soggy Site

Spotlight on Geotextiles



A tractor-supported sewing machine stitches seams along heavy, hand-spread geotextile used to support workers and equipment constructing new marine terminal on soft, wet dredgings in Baltimore Harbor.

Cover Story

A Stitch in Grime Saves Time

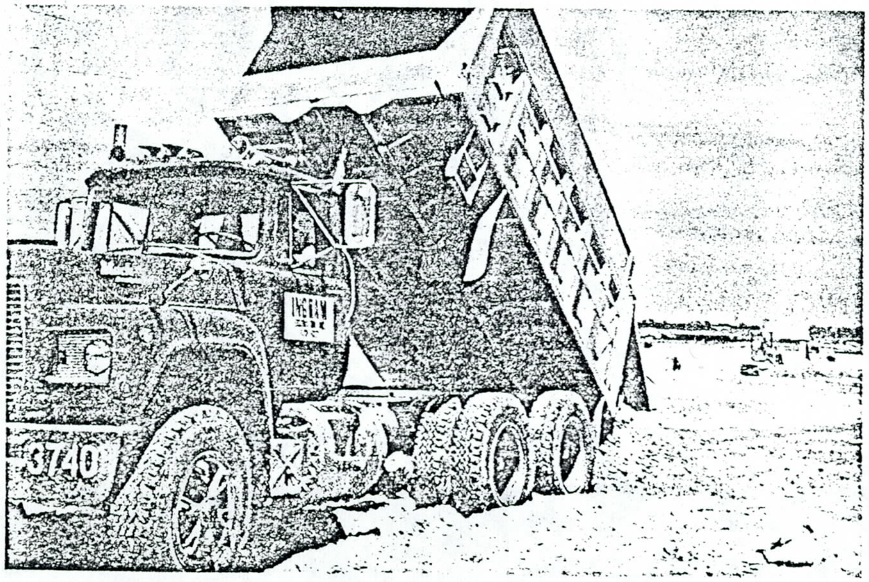
More than 3,000,000 l.f. of vertical wick drain—the largest such installation ever in this country—and 300,000 sq. yd. of an extra heavy geotextile field stitched in the grime are saving time on a new marine terminal in Baltimore.

The 113-acre disposal area for spoil from the I-95 tunnel under Baltimore harbor is now being consolidated for development as a container facility. C.J. Langenfelder & Son, Inc., Baltimore, is the general contractor on the $10.9-million, 600-calendar-day project for the Maryland Port Administration.

"It is all on a fast track schedule," said Ron Lange, project engineer for the Administration. "The heavy duty geotextile is essential to support the equipment installing the wicks. If we didn't use the wicks, the surcharge would have to remain for a settlement period of at least an additional year to adequately consolidate the spoil."

The pumped spoil is almost liquid. In most areas, the surface crust will support a man, but not construction machinery. Some areas will not even support a man, and equipment is safe only in a few areas along the old shoreline.

The extra heavy woven geotextile on top of the spoil distributes loads to the point that a man is safe, even in the softest areas, where movement feels like walking on a water bed. A 2½ ft. blanket of sand must be placed over the geotextile and capped with a six in. layer of crushed slag to provide barely adequate support for heavy equipment.



A 2½-ft. layer of sand and 6 in. of slag are spread over geotextile by wide-track dozers.

Equipment works on slag surface

Subcontractor Geotechnics, Inc., Bay St. Louis, Miss., works off the slag surface while installing the wicks with a modified Koehring 266 hydraulic excavator. It has 42-in.-wide crawler tracks to reduce its ground pressure to 6 psi, and its bucket and boom have been replaced with a 56-ft.-high mast. A mandrel in the mast, actually a fixed lead, acts like a needle while inserting the wick drain from 16 to 50 ft. down into the soft goo.

The toughest part is punching through the heavy geotextile, which is usually done with the rig's 15-ton static force. On occasions, a short burst of vibratory power helps. Once the pointed mandrel tip pierces the geotextile, the mandrel is pushed almost effortlessly to the bottom of the spoil and withdrawn leaving the wick behind.

After the mandrel is withdrawn, a laborer cuts the wick material with handheld hedge trimmers, doubles the loose end back into the eye of the mandrel and inserts a 9-in. length of ½-in. rebar to hold the wick snuggly in position for the next insertion.

One wick every 30 seconds

John Singleman, the foreman for Geotechnic, regularly installs one wick every 30 seconds as the rig rotates from side to side inserting two or three rows of wicks from a single setting. The wicks are spaced about 5 ft. apart in a diamond-shaped pattern. Daily production, including downtime to change reels of wick material or for major movement of the machine, ranges from 10,000 to 18,000 l.f. installed per 10-hour day.

Another modification which Geotechnic's Russell Joiner made on the rig is an additional hydraulic system and cylinder to raise or lower the mast. By extending and pinning that cylinder to the mast, the entire 56-ft.-high fixed lead can be rotated to or from the horizontal position by the operator in the cab without the need for any support equipment.

With the mast lowered, the 70,000-lb., 11½-ft.-wide rig can be walked onto a special four-axle, hydraulic beam trailer, jacked up to provide 6-in. of clearance under its tracks, and moved over the highway as a 112,000-lb., special-permit load when moving from job to job.

The Amerdrain vertical wick drain used on the job is manufactured by ICE. Each reel consists of 1000 ft. of a flat, 4-in.-wide sleeve of nonwoven filter fabric surrounding a ribbon of longitudinally corrugated plastic.

Water is squeezed out

Once the wicks are installed and a heavy, 7- to 9-ft. earth surcharge is placed, water will be squeezed out of the spoil through the filter fabric and up the wicks for discharge into the sand blanket. An underdrain system in the sand blanket will collect that water and convey it to a sump for pumping into a settling basin prior to discharge in the bay.

The ADS underdrain is a 6-in. corrugated, perforated polypropylene pipe installed within the 2½-ft. sand blanket by one of Langenfelder's crews using a Vermeer V-430 trencher.

The extra heavy geotextile spread over the surface of the spoil is Nicolon's 62809 woven fabric consisting of cords of polypropylene in one direction and polyester in the other. The fabric weighs nearly two lb. per sq. yd. and has a tensile strength of more than 1000 lb. per lineal inch in each direction.

The fabric, which comes in 1200 lb. rolls about 16½-ft. wide and 270 ft. long, is towed to the site by wide track dozers, unrolled and hand-spread by laborers. Seaming is done with a special heavy duty electric sewing machine hung on a small Steiner farm tractor. The sewing machine is powered by a Homelite HG 1400 portable generator mounted on the rear of the tractor.

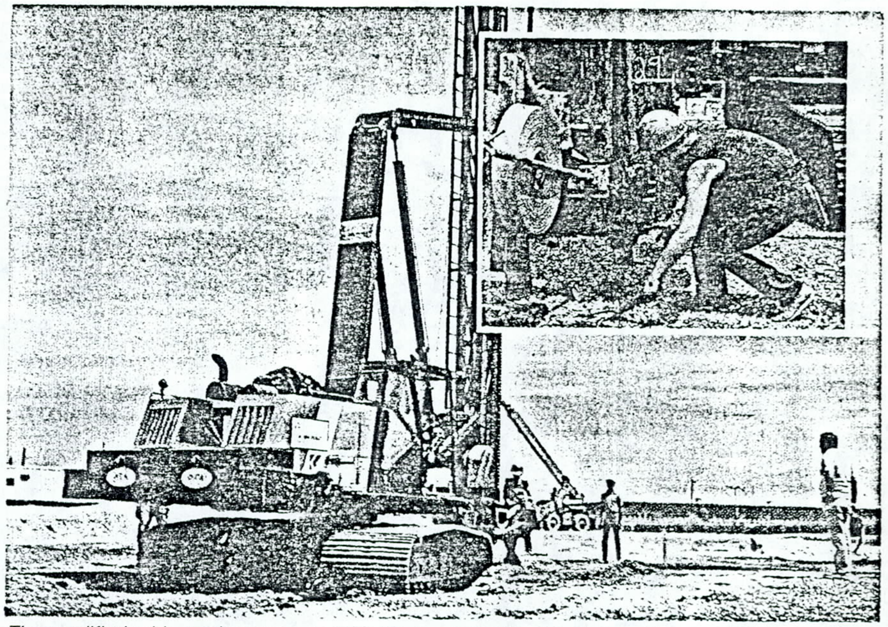
The longitudinal joints are double J-stitched in which two layers of geotextile are lapped, folded over, and the four thicknesses of material (about a third of an inch) stitched together with a heavy polyester "thread." In practice, this takes a crew of six to eight laborers to support and shape the fabric seams for the sewing machine operator.

Sand spread over geotextile

About 250,000 cu. yd. of sand is being spread over the seamed geotextile to a 2½ ft. depth. The sand is hauled in from off-site locations by Ingram Trucking, Co., Baltimore, and spread by two small Caterpillar D3B and one Komatsu D31P wide-track dozers.

Slag is placed in a similar manner after the sand course has been leveled by the small dozers dragging their blades. After the wicks and underdrains are installed, the 210,000 cu. yd. surcharge is built up in one ft. lifts to provide the load to squeeze water out of the subsoils.

"Sud" Cockey is project manager for Langenfelder coordinating a work force of 30 to 35, plus truck drivers, on the project. Crews typically work five 10-hour days a week. □



The modified wide-track excavator with 56 ft. mast still sinks into soft ground while installing more than three million l.f. of vertical wicks to drain the wet dredgings under new port site. Mandrel is withdrawn from soft goo and top of wick is cut from reel with hand shears (inset). Loose end from reel is then doubled back into tip of mandrel around a piece of rebar, and pulled snug by laborer at left before next wick is installed.

Horizontal drains are installed in 2½-ft. sand blanket to collect water to be discharged from vertical wicks after a heavy, earth surcharge is placed.

More information on equipment used is available by circling the appropriate Reader Service Numbers in this issue.
176 Hydraulic excavator
177 Vertical wick drains
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Highway & Heavy Construction

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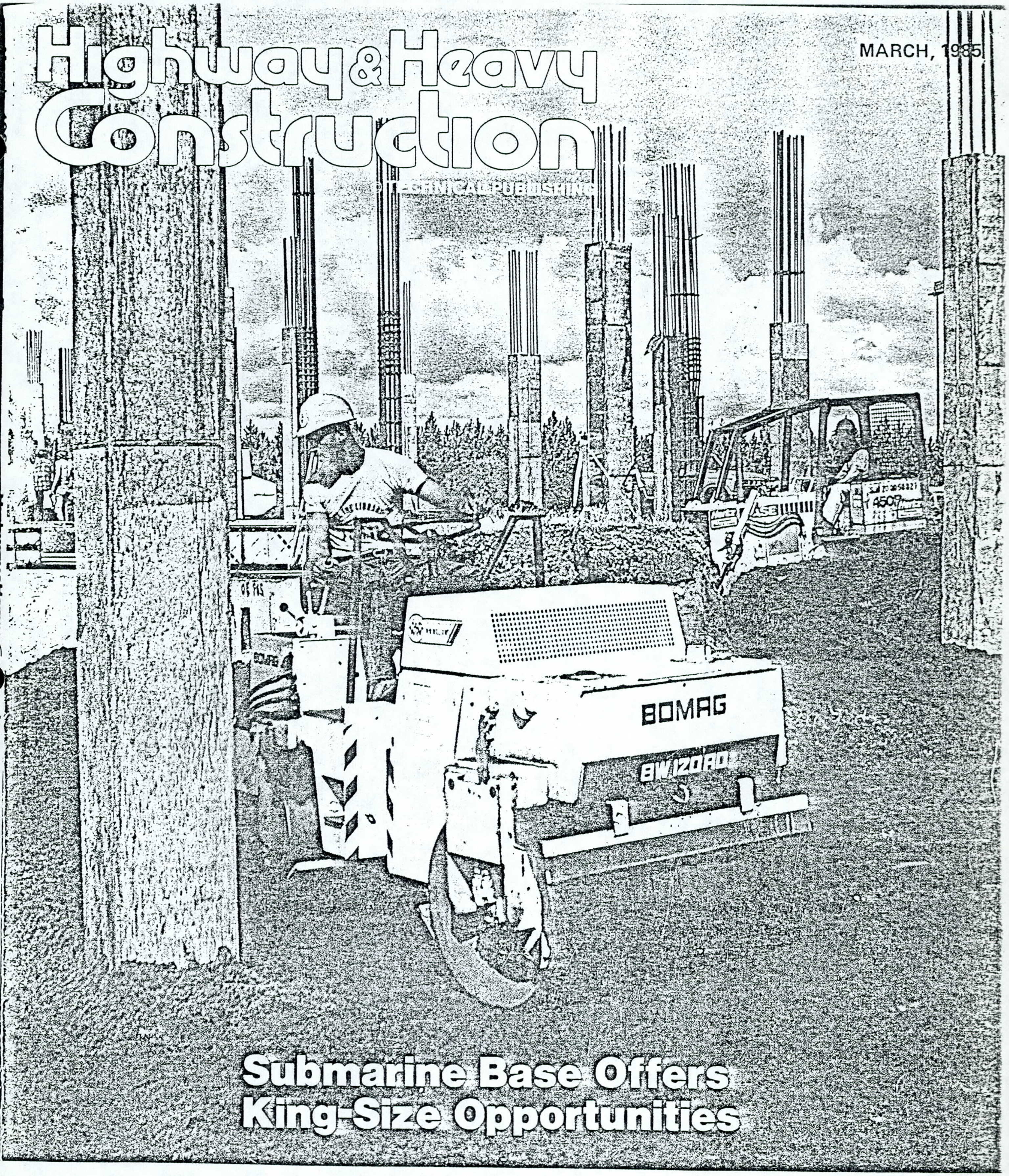
MARCH, 1985



Submarine Base Offers King-Size Opportunities

New Horizons — Drainage

New Horizons In Drainage

What's new and different in drainage . . . for foundations . . . for surface runoff . . . for area drainage . . . and for construction sites.

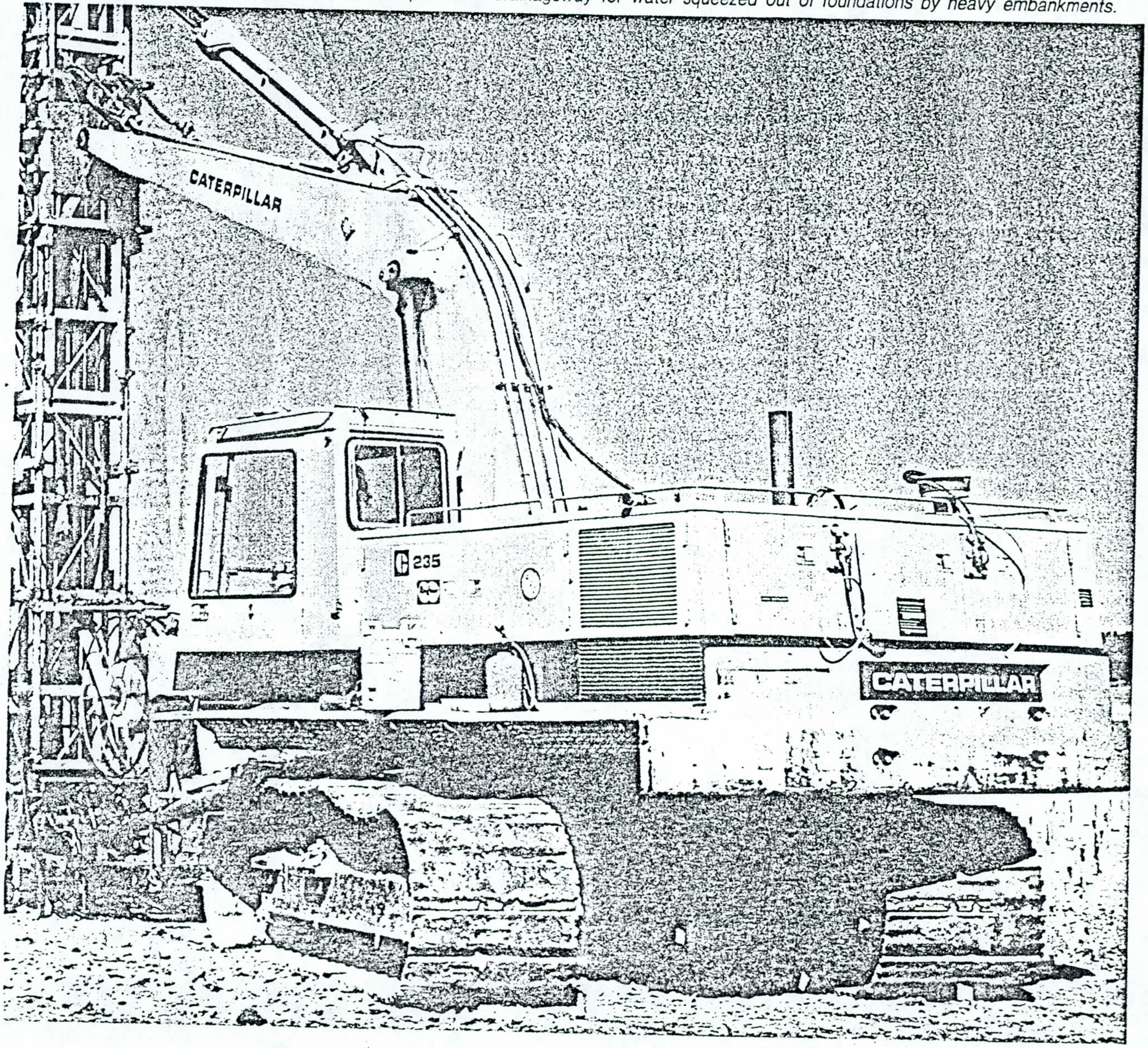
The three most important factors in highway design—according to an old engineering adage—are drainage, drainage, drainage. Good drainage of the subsoils and embankments provides the stable foundation needed to carry the loads. Good surface drainage avoids ponding on pavements and allows traffic to move safely. Good area drainage—properly positioned and adequately-sized culverts and bridges—maintains stream flows and protects the highway from flooding and wash out.

Those same principals apply to most construction projects, not just to highways. But there is a fourth often-neglected factor, too. Drainage of the construction site so the contractor can build the project. That's our outline—What's New and Different in Drainage . . . for foundations . . . for surface runoff . . . for area drainage . . . and for construction sites.

Foundations

Excavation and replacement, replacement by rolling surcharge, and gradual consolidation under the weight of an oversized embankment are tried and true methods of coping with wet or compressible foundation conditions. In some cases, sand drains or rock drains (vertical columns of sand or rock) support an embankment and also provide a drainageway through im-

Wick drains inserted vertically in soft, wet soils provide a drainageway for water squeezed out of foundations by heavy embankments.



permeable soils into an underlying water bearing strata.

But dynamic compaction and wick drains are relatively new methods of stabilizing foundations. Dynamic compaction—the dropping of multi-ton weights from heights of 50 to 100 ft.—is faster than building a heavy embankment surcharge to compact compressible soils, but it does little to drain foundations. Several companies which specialize in this kind of work (such as GKN Hayward Baker, Inc., and Geopac, Inc.) will provide further background on request.

Wick drains are the newest method for draining wet foundations. The wick is little more than a continuous sleeve of porous construction fabric surrounding an irregular plastic core. It is inserted vertically—usually to a depth of 20 ft. or more—in soft, wet soils to provide a drainageway for water squeezed out of the foundations by an oversized embankment. Companies such as Vibroflotation Foundation Co., Geotechnics and others should be consulted for further information on installing wicks.

Surface runoff

Adequate pavement cross slopes and curb and gutter sections having adequate longitudinal slopes are the standard in municipal paving. Stringlines and sensors have greatly improved grade control during paving, and modern slipformers can economically extrude a variety of curb and gutter sections to collect and carry runoff to designated points of discharge. But plugged inlets or heavy rainfalls can still cause ponding of runoff on the pavement, frequently extending well into the travelled lanes where it slows traffic and creates a hazard.

In some cases, particularly where the terrain is very flat or the paved areas are very large, slotted drains can be used to store that runoff and reduce ponding on the paved surface. Armco and ACO Polymer Products are two manufacturers of this type of drain which is being used increasingly in parking lots, driveways and municipal streets. It is probably being used on runways and other large paved areas at airports, although we're not aware of any such installations yet.

Most paving contractors are now familar with open graded friction courses, a thin asphalt surface placed on many highways these days. It improves highway safety through increased friction and better braking for vehicles, and by virtual elimination of hydroplaning. Both occur because the lack of fines in the paving mix creates a porous, sponge-like series of openings in the surface to carry runoff to the edges and to provide a means of escape for water which could otherwise be trapped between the tire and a perfectly smooth pavement.

Other contractors specialize in cutting grooves in pavement surfaces (usually concrete) to drain runoff and reduce the possibilities of hydroplaning. A series of diamond-tipped circular saws operating longitudinally on highway pavements and transversely on runways are the norm for this work.

Similar diamond-tipped circular saws eliminate rutting (and ponding in the wheel ruts) on worn concrete pavements. Milling machines with carbide-tipped cutting teeth perform similarly along asphalt highways. But the millers also remove the worn surfaces, which can then be recycled, and restore flow lines along curbs on municipal streets.

Much of the surface runoff on older concrete highway pavements drains through cracks or joints in the slabs into the underlying base. Many highway departments now specify longitudinal underdrains along each edge of a roadway.

Frequently, the underdrain trench is lined with a filter fabric to keep fine soils out of the perforated underdrain pipe. In other cases, coils of perforated plastic pipe already encased in a filter fabric sleeve are used.

In some areas, saw-like trenching machines are being used to retrofit old highways with underdrains. But at least one state—Georgia—prefers to invest in effective sealing of cracks and joints in the pavement to prevent intrusion of surface drainage, rather than investing in underdrains to remove that water.

Area drainage

Urban development invariably converts large areas of rain-absorbing soils to water-shedding rooftops and pavements. That increases the amount of runoff after storms and temporarily overloads existing storm sewer systems and increases the likelihood of downstream flooding.

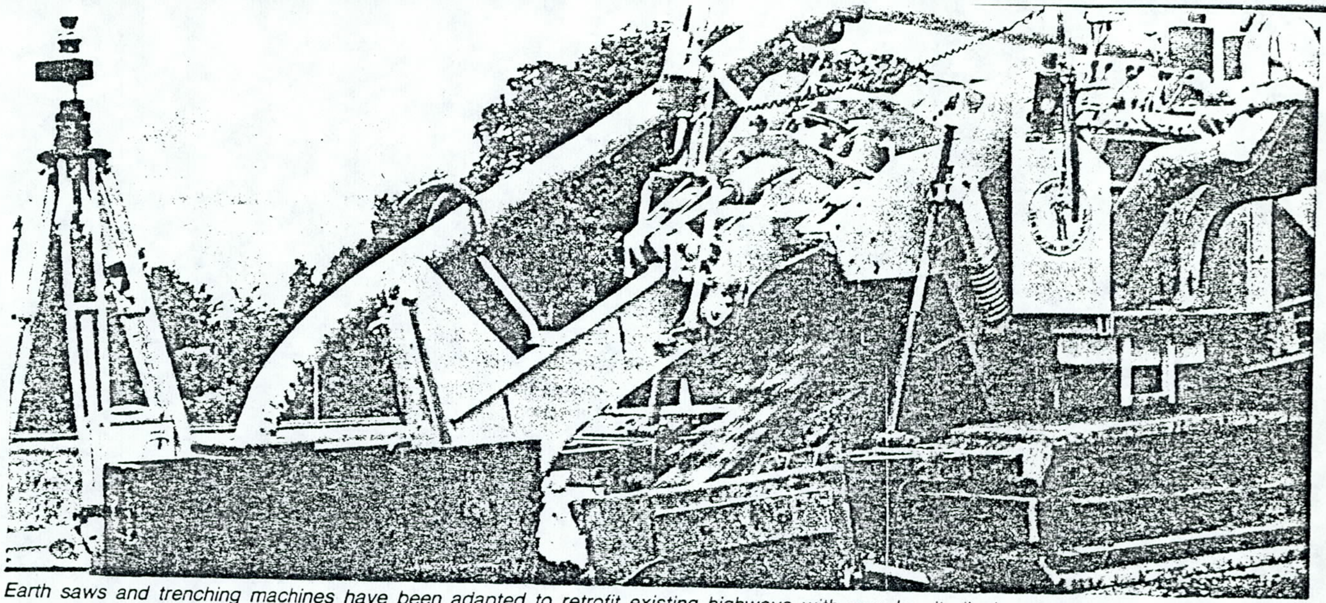
Many zoning codes now carry restrictions limiting peak runoffs *after construction* to the same flows that existed before development. The result is to force temporary on-site storage of peak runoff, with a gradual and controlled release when downstream facilities are able to cope with the flow.

Construction of pre-planned, on-site retention basins as an initial part of site development now occurs with increasing regularity. Earthmovers are finding a new market in reshaping flat lands to create storage lakes and ponds. Culverts limit the outflows through dams, which are designed to detain peak flows exceeding the capacity of the outlet. Earthfill detention dams are most frequent, but Roller Compacted Concrete dams are being used in some cases (see following story).

This same stormwater storage concept is the basis for the deep tunnel systems now being constructed in the Chicago and Milwaukee Metropolitan Areas. Both areas have combined sewers which collect and convey sanitary *and* storm flows to treatment plants. In both cases, the treatment plants would be flooded by peak runoffs after rainstorms if those flows weren't discharged untreated into Lake Michigan. In both cases, those runoffs will eventually be stored underground and pumped back to the surface for treatment during off-peak periods.

In a similar vein, porous concrete pavements—which, like the open graded asphalt friction courses, contain little in the way of fines—are now being used in some parking lots to *store*

Slotted drains store runoff and reduce ponding on pavements after rainstorms.



Earth saws and trenching machines have been adapted to retrofit existing highways with new longitudinal underdrains.

as much as an inch of runoff within the pavement. That stored runoff later evaporates if the base is impermeable, or soaks into the subsoil if the porous concrete is used for the full depth of the pavement.

The latter concept is carried a step further in other cases where an open, precast concrete grid is placed over the subsoils, backfilled with topsoil and seeded. Further data on this approach is available through Armortec and other companies.

Water quality—rather than water quantity—also affects the design and construction of drainage facilities. Many power plants and other similar facilities are in enviromental hot water because of the high discharge temperature of their cooling waters into streams and waterways. New reservoir projects to further cool those waters are underway in several cases.

The Soil Conservation Service and other federal agencies are deeply concerned about windborne and waterborne erosion of agricultural lands. Land leveling, contour farming, check dams in drainage ways and settling basins are current methods of controlling soil erosion.

Construction sites

Most grading contractors are already well-versed in the use of check dams and brush barriers in drainage ways, and in settling basins and filter systems used to curtail soil erosion at or near a construction site. But water pumps have been greatly improved in recent years. Many pumps can be submerged, while others can be fitted with trash guards. Impeller pumps can "lift" (actually push) water for greater distances.

Wellpoints, actually small submersible pumps connected to a surface level storm drain, are a traditional dewatering answer to contractors working in sites below ground water levels. Several companies, including Stang Hydronics and Moretrench American, specialize in this type of dewatering.

Concrete cutoff walls constructed in Bentonite slurry-supported trenches are also used to protect many foundation excavations from unwanted inflows of water. ICOS and Case International are two of the companies active in this kind of work.

But the newest technique for temporarily protecting a construction site from ground water involves freezing—rather than draining—the surrounding soils. Firms like Geofreeze Corp. and Geo Systems, Inc., specialize in installing the coils and refrigeration equipment to permit excavation to proceed in areas which would otherwise defy drainage and construction.

References and training

Entire libraries of theoretical and technical data are available on the topic of drainage, and we won't bore you with any such listings. But there are two sources which may be of particular interest to designers and contractors involved in that type of business.

The American Public Works Assn.'s 1981 Special Report No. 49, titled *Urban Stormwater Management*, is one of the most comprehensive documents on that topic we've seen. It provides legal and operational perspective for managing urban stormwaters, and detailed guidelines for planning, designing and constructing stormwater management facilities. This 15-chapter, 285-page document is available at a cost of $30 plus postage through APWA, 1313 E. 60th St., Chicago, IL 60637.

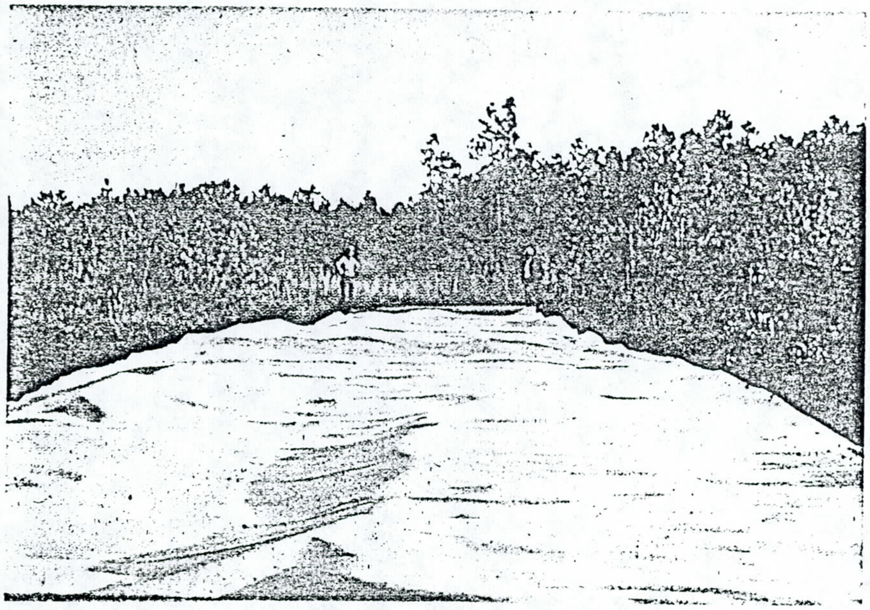
The Federal Highway Administration's (FHWA) Demonstration Projects Div. is developing a new demonstration project which builds on an earlier, now completed series of workshops. It is expected to start late this year and to consist of three-day workshops to provide design guidance and procedures for hydrologic analysis, culvert design, channel design and bridge waterways. It will include demonstrations on the use of microcomputers in hydrolgic and hydraulic analyses. A portable hydraulic flume will be used. Further information on this demonstration project can be obtained through: Douglas A. Bernard, Chief; FHWA Demonstration Projects Div.; HH-O, Nassif Building; 400 7th St., SW, Washington, DC 20590.

Reference manuals for these workshops are already available. They include:

- HEC 12—Drainage of Highway Pavements (1984);
- HEC 13—Hydraulic Design of Improved Inlets for Culverts (1972);
- HEC 14—Hydraulic Design of Energy Dissipators for Culverts and Channels (1983);
- HEC 15—Design of Stable Channels with Flexible Linings (1975); and,
- HEC 19—Hydrology (1984).

Reference manuals HEC 12 and 19 are available from the Government Printing Office. The others may be obtained, in limited quantities, from the FHWA Office of Engineering; Bridge Div., HNG-31; 400 7th St. SW, Washington, DC 20590. □

Geofabric Floats Road On Swamp For Easy Access



Geotextile fabric is placed prior to two ft. of sand rock fill being spread and compacted to create this access road over a swamp in Pennsylvania.

An access road was recently built quickly and inexpensively over a swamp. A high strength geotextile placed first on the surface of the swamp effectively prevented the loss of fill to the swamp.

D&F Coal, Penn Run, Pa., needed a new access to the coal mine. But the site was surrounded by swamp and the project appeared too costly and difficult. The use of a strong geotextile made the job feasible.

Phillips Supac 5WS(UV) was unrolled over the swampy material. Two ft. of sand rock fill was spread and compacted by a Cat D6 dozer and the road was ready to use. Encouraged by the success with the road, D&F placed fabric and fill over another couple of acres to increase their stockpile area. □

More information on equipment used is available by circling the appropriate Reader Service Numbers in this issue.
195 Geotextile
196 Dozer

PRODUCTS

AMERICAN WICK DRAIN CORPORATION

301 WAREHOUSE DRIVE • MATTHEWS, NORTH CAROLINA 28105 • 800 - 438-9281 or 704 - 821-7681 / TELEX 572385

December 6, 1985

DEC 1985

Simons, Li & Associates
P. O. Box 1816
Fort Collins, CO 80522

Attention: Mr. Jim Wailes

As per your request, I have enclosed literature and samples of the AMERDRAIN Type 407 wick drain and Type 360 sheet drain.

If you have questions, please contact me at 704-821-7681.

Thank you,

Tom Cunningham

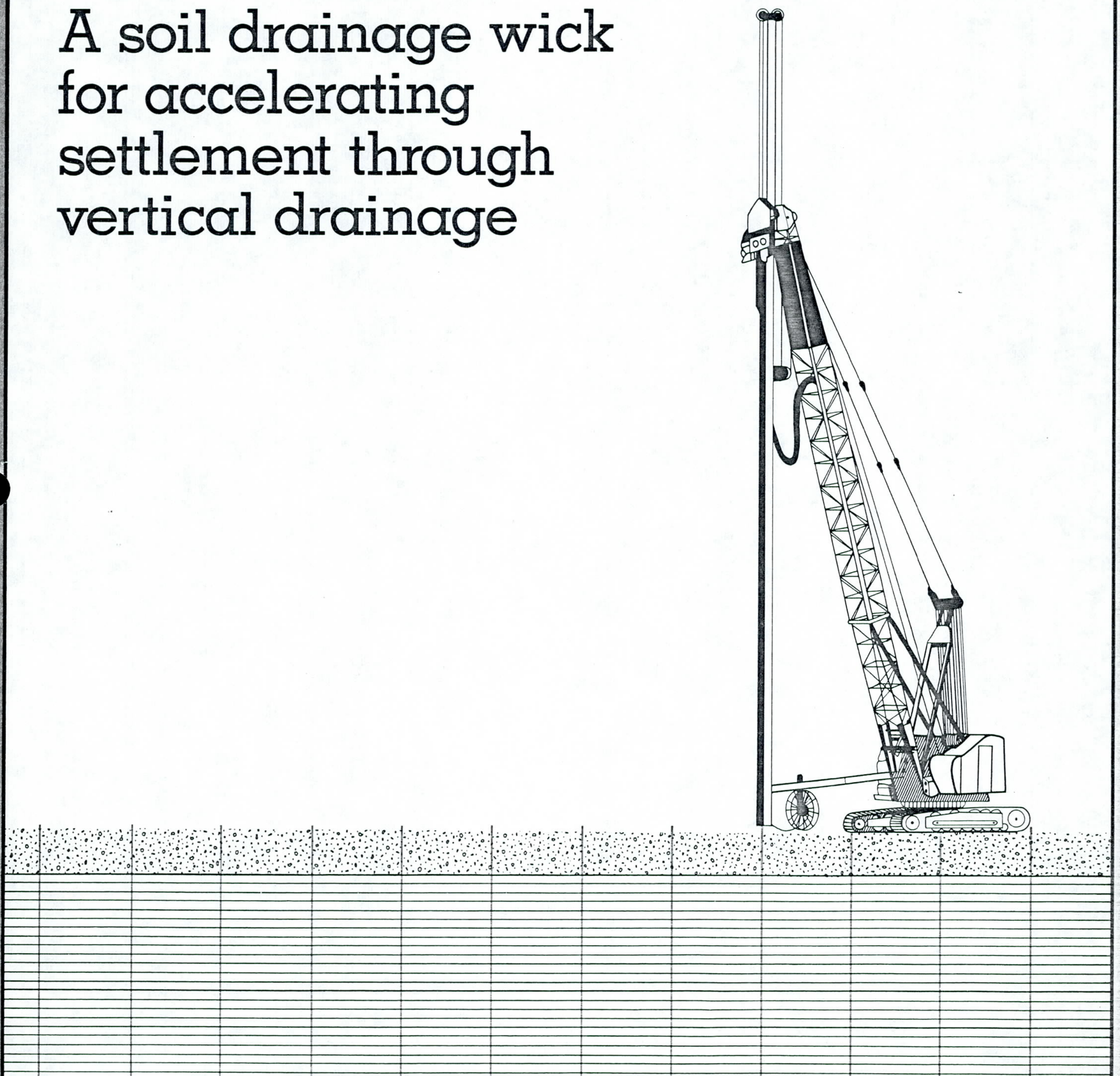
Tom Cunningham

TC:lb
Encls:

AMERDRAIN™

SOIL DEWATERING WICKS

A soil drainage wick for accelerating settlement through vertical drainage



AMERDRAIN VERSUS SAND DRAINS

AMERDRAIN soil drainage wick can be installed in a fraction of the time and at a fraction of the cost of sand drains. A comparison of AMERDRAIN with sand drains reveals a number of advantages in using AMERDRAIN.

1. AMERDRAIN wicks work in all types of soils.
2. AMERDRAIN wicks are constant in quality. Sand drains suffer from variations in sand quality. The ideal quality of sand is often not available nearby.
3. There is much less soil disturbance during placement of the drain due to the small size of the AMERDRAIN wick. There is minimum reduction in soil permeability and minimum increase in pore pressure.
4. Installation is less expensive as installation equipment is simpler and fewer people are required.
5. Installation speed is high — 2000 to 4000 meters (6500-13,000 feet) per day is not uncommon.
6. No water is usually required during installation eliminating environmental nuisance and cold weather problems.
7. Transportation costs are much lower. One truck can handle 120,000 meters (360,000 feet) of wick. Sand for the same amount of sand drains would require over a thousand 12-ton trucks.
8. The design and high strength of the AMERDRAIN assures drain operation despite large lateral soil movement or high soil pressures.
9. Installation is feasible to depths of over 50 meters (164 feet).

SPACING OF AMERDRAIN WICKS VERSUS SAND DRAINS

Some engineers call for replacement of sand drains with wick drains on a one-to-one basis. However, a 1½ or 2-to-one ratio is more common. To obtain optimum results, a complete geotechnical analysis should be completed to finalize drain spacing.

PHYSICAL PROPERTIES

AMERDRAIN 407	VALUES	TEST METHODS
Drain Core	Polypropylene	
Filter Fabric	Polypropylene	
Weight	93 gm./m. (1.0 oz./ft.)	
Width	100mm. (4 in.)	
Thickness	3mm. (1/8 in.)	
Grab Tensile*	61.3 kg. (135 lbs.)	ASTM D1682-64 (1975)
Elongation at Break*	62%	ASTM D1682-64 (1975)
Modulus*	544 kg. (1200 lbs.)	ASTM D1682-64 (1975)
Trapezoidal Tear*	33.6 kg. (74 lbs.)	ASTM D2263-68
Puncture Strength*	22.7 kg. (50 lbs.)	ASTM D751-73
Mullen Burst*	14 kg/cm.^{2} (200 psi)	ASTM D774-46
Abrasion Resistance*	19.1 kg. (42 lbs.)	ASTM D1175-71
Specific Gravity	0.95	
Flux*	3238 lit./m.^{2}/min. (230 gal./ft.^{2}/min.)	EURM-100 (DuPont)
Coefficient of H_2O Permeability (K)*	2 X 10^{-2}cm./sec. (7.9 X 10^{-3} in./sec.)	EURM-100 (DuPont)

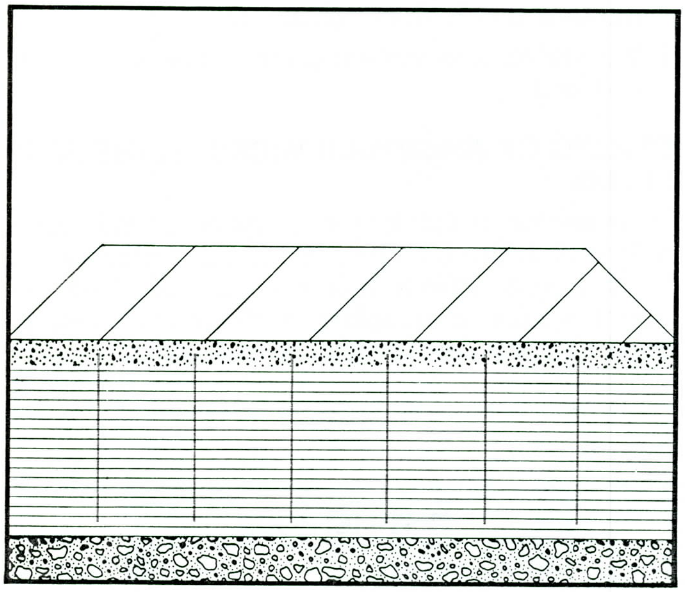
*Data for filter fabric only

The facts stated and the recommendations made herein, based on our research and research of others, are offered free of charge, and are believed to be accurate. No guarantee of this accuracy is made, however, and the products discussed are distributed without warranty, expressed or implied, and upon condition that recipients shall make their own tests to determine the suitability of such products for their particular purposes. Likewise, statements concerning the possible uses of our product are not intended as a recommendation to use in the infringement of any patent, whether owned by American Wick Drain Co. or by others.

TYPICAL APPLICATIONS

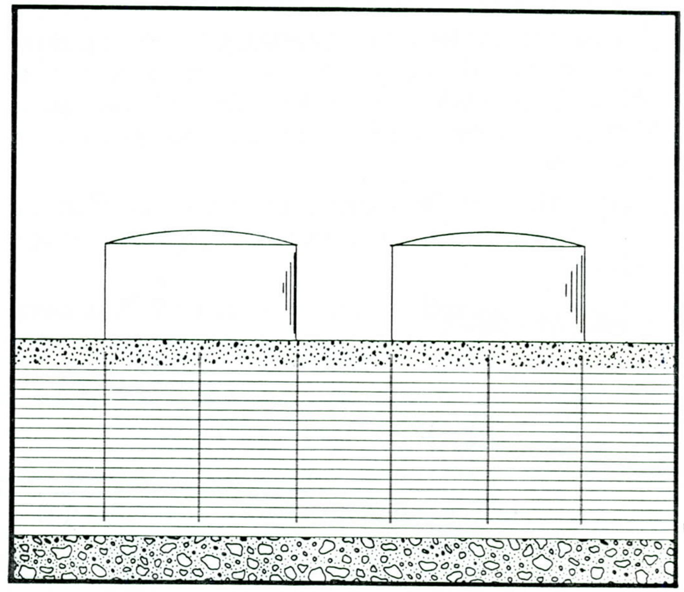
EMBANKMENT CONSTRUCTION

Drain wicks may be used to accelerate settlement of embankments for roadways, railroad tracks, runways, or bridge approaches which must be put into operation very soon after construction is completed. Presettlement can greatly reduce long-term maintenance costs that would result from extended periods of settlement during the life of the project.



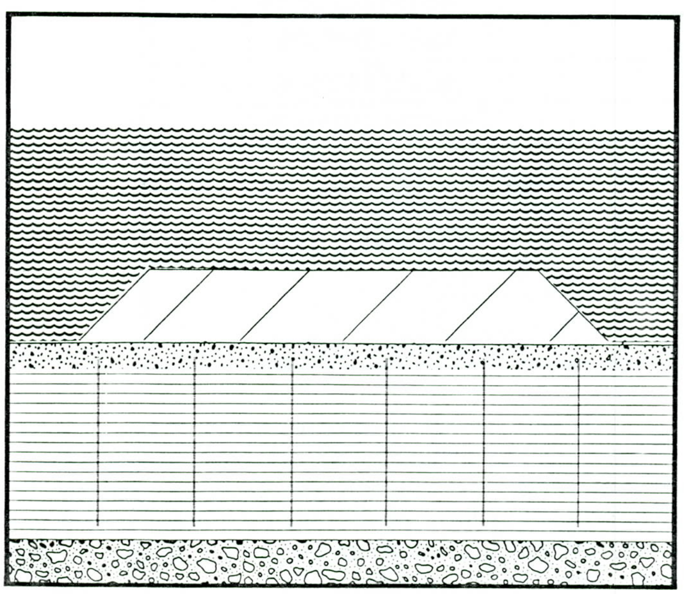
TANK FARM FOUNDATIONS & MATERIAL STORAGE AREAS

Because of high unit loadings, liquid storage tanks are subject to settlement in soft soils. Vertical drain wicks used in conjunction with a sand surcharge can provide rapid soil consolidation prior to construction. Storage sites for solid materials — coal, ore, paper — also can benefit from vertical drainage prior to use.



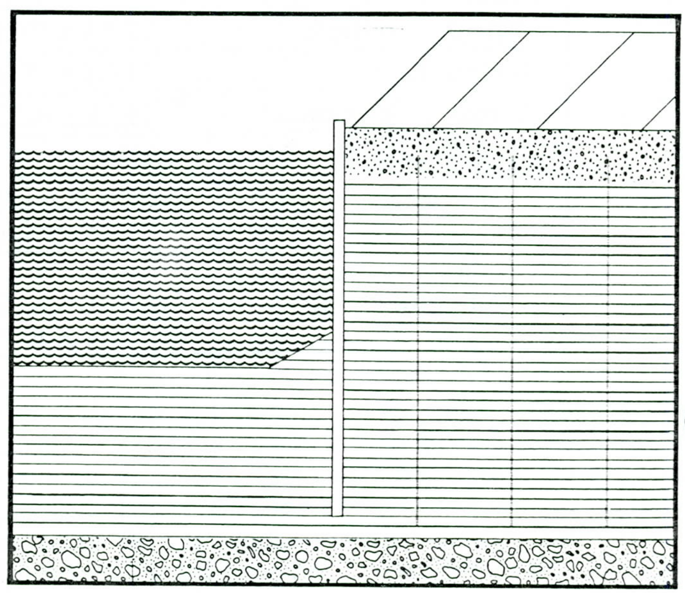
UNDERWATER CONSOLIDATION

Vertical wicks may be used to accelerate settlement of soil below water level. The differential pressure created by the surcharge is as effective under water as on land. This technique can be used in preparation for placing tunnel sections in a river bed, for example.



LANDFILL AREAS

Fill is often placed behind sheet piling walls or cofferdams for use as docks or industrial sites. Vertical drainage with wicks is an effective method to accelerate settlement thereby making the site available for use in the shortest possible time.



HOW AMERDRAIN FUNCTIONS

The function of the AMERDRAIN has two aspects — 1) The capacity of the drain to accept water through the filter jacket and 2) the capacity to discharge the water vertically.

The AMERDRAIN filter material uses a strong, tough, permeable nonwoven filter fabric of 100% polypropylene specifically designed for drainage. Its continuous filaments are arranged preferentially in the length and width directions of the sheet and thermally bonded. AMERDRAIN filter fabric resists mildew, rotting, insects and chemicals normally encountered in a subsurface drainage system. It is dimensionally stable, wet or dry, has good tear and puncture resistance and will not shrink, grow or unravel.

AMERDRAIN has a unique structure (Figure 3) that enhances its function as a filter fabric. It has a large number of openings with a range of opening sizes throughout its structure instead of a few openings of fixed size as in woven fabrics. Its bonded fibers create a tortuous pathway resembling that of a well graded aggregate filter rather than a simple, straight line exit for soil particles. Because of its unique structure, the filter fabric has both high permeability and the ability to restrict the movement of most soil particles, while allowing the very fine silts to flow into and out of the drain. The initial removal of very fine silts is beneficial because this leaves the larger particles to form a highly permeable soil network (Figure 4) against the fabric. The soil network restricts the further movement of fine soil particles and helps to develop a graded soil filter. This soil filter effectively stops piping of soil and prevents other fine particles from entering the drain. The fabric filter, being more permeable than the soil filter and the natural soil, does not restrict the flow of water into the drain.

The effectiveness of the filter fabric has been proven in government and commercial projects in a wide range of soil types. Its effectiveness has also been confirmed by extensive laboratory tests at the Colorado State University Engineering Research Center. Tests were run with soil mixtures of fine sand, silts and clays which simulated actual drainage systems. Under these conditions, the tests confirmed that, after the initial passage of fine silts through the filter there was:

(a) No measurable migration of soil fines within the soil filter or into the drain, and
(b) No measurable pressure drop across the filter indicating no reduction of water flow through the fabric. Scanning electron microscopy confirmed that there was no clogging of the fabric.

The AMERDRAIN core (Figure 5) is a strong, tough structural member extended from 100% polypropylene specifically designed for drain wick systems. A total of 38 longitudinal grooves distributed on both sides of the core provides discharge passages for water flowing to the surface. The core is dimensionally stable when wet, has good puncture and collapse resistance and will not shrink or rot.

Figure 3

50x Magnification of AMERDRAIN Filter Fabric

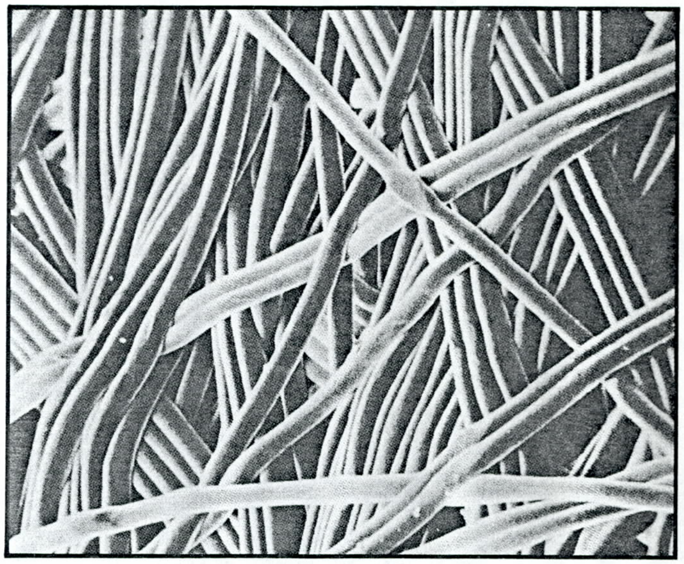


Figure 4

Graded Soil Filter Formed Against AMERDRAIN

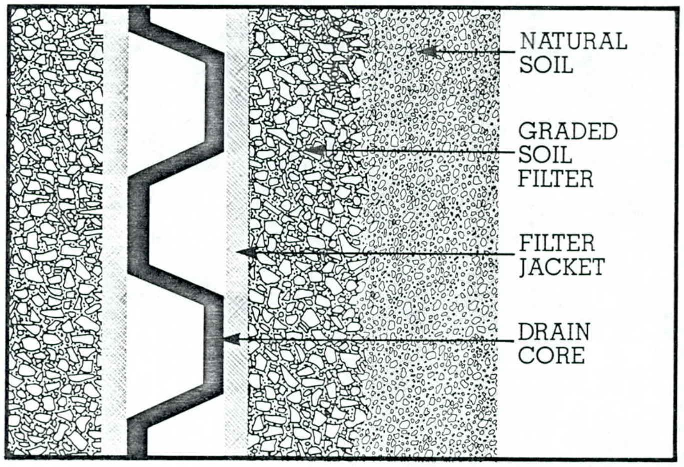
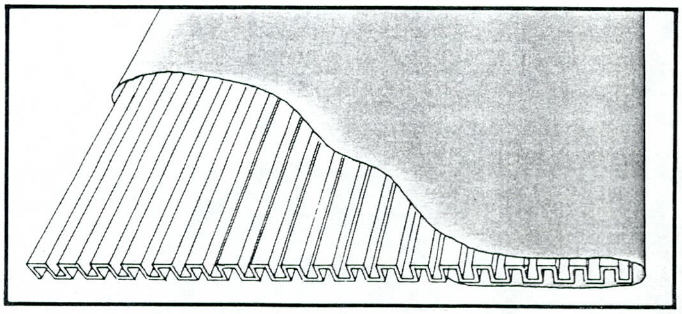


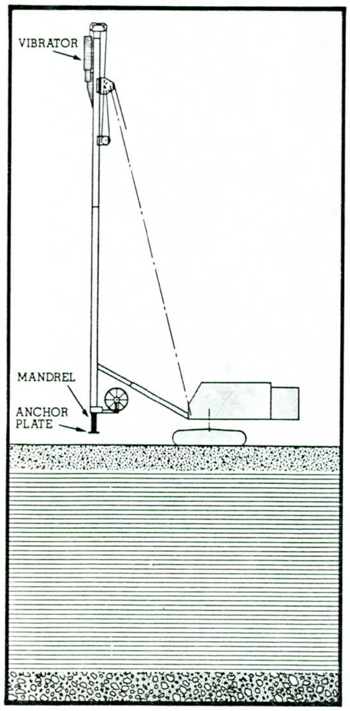
Figure 5

AMERDRAIN Wick Core

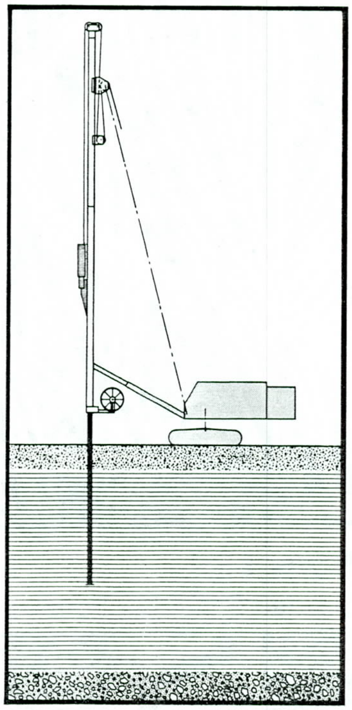


INSTALLING AMERDRAIN

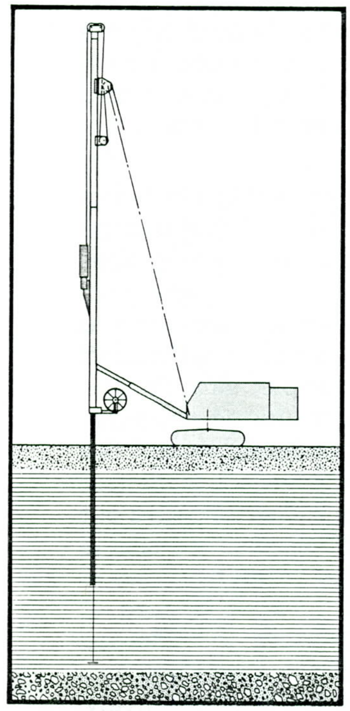
AMERDRAIN may be installed employing either vibratory or static methods. In either case, the wick is enclosed in a tubular steel mandrel of small cross-sectional area (usually 2 X 5 inches). A small steel anchor plate is attached to the wick at the bottom of the mandrel. The mandrel is then driven into the soil either with a static or vibratory rig. When the depth is reached, the mandrel is extracted. The anchor plate retains the wick in the soil. When the mandrel is fully extracted, the wick is cut off, a new anchor plate is installed and the process begins again.



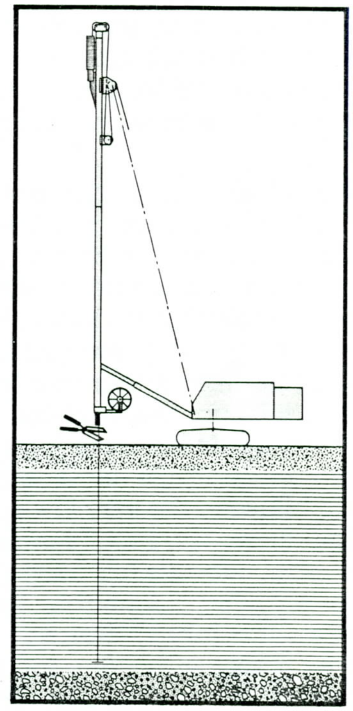
INSTALLATION EQUIPMENT



DRIVING MANDREL



EXTRACTING MANDREL



CUTTING AMERDRAIN

AMERICAN WICK DRAIN CORP.
301 Warehouse Drive Matthews, NC 28105
Phones 800 438-9281 & 704 821-7681 Telex 572385

GEOTECHNICS AMERICA, INC.
INSTALLERS OF SOIL DRAINAGE WICKS

Geotechnics America Inc. is a nationwide construction firm specializing in the supply and installation of AMER-DRAIN soil drainage wicks. Geotechnics America employs both vibratory and static installation equipment which can install drains to depths of 125 feet (40 meters). The vibratory method has been proved during the installation of over 40 million feet in Europe and over five million feet in the United States to be the most reliable and predictable installation technique. For unusual job sites or soil conditions, the static technique may be used.

During installation, the drainage wick is completely protected by the steel mandrel from damage from rocks or other materials. An economical anchor plate holds the wick in place during extraction of the mandrel.

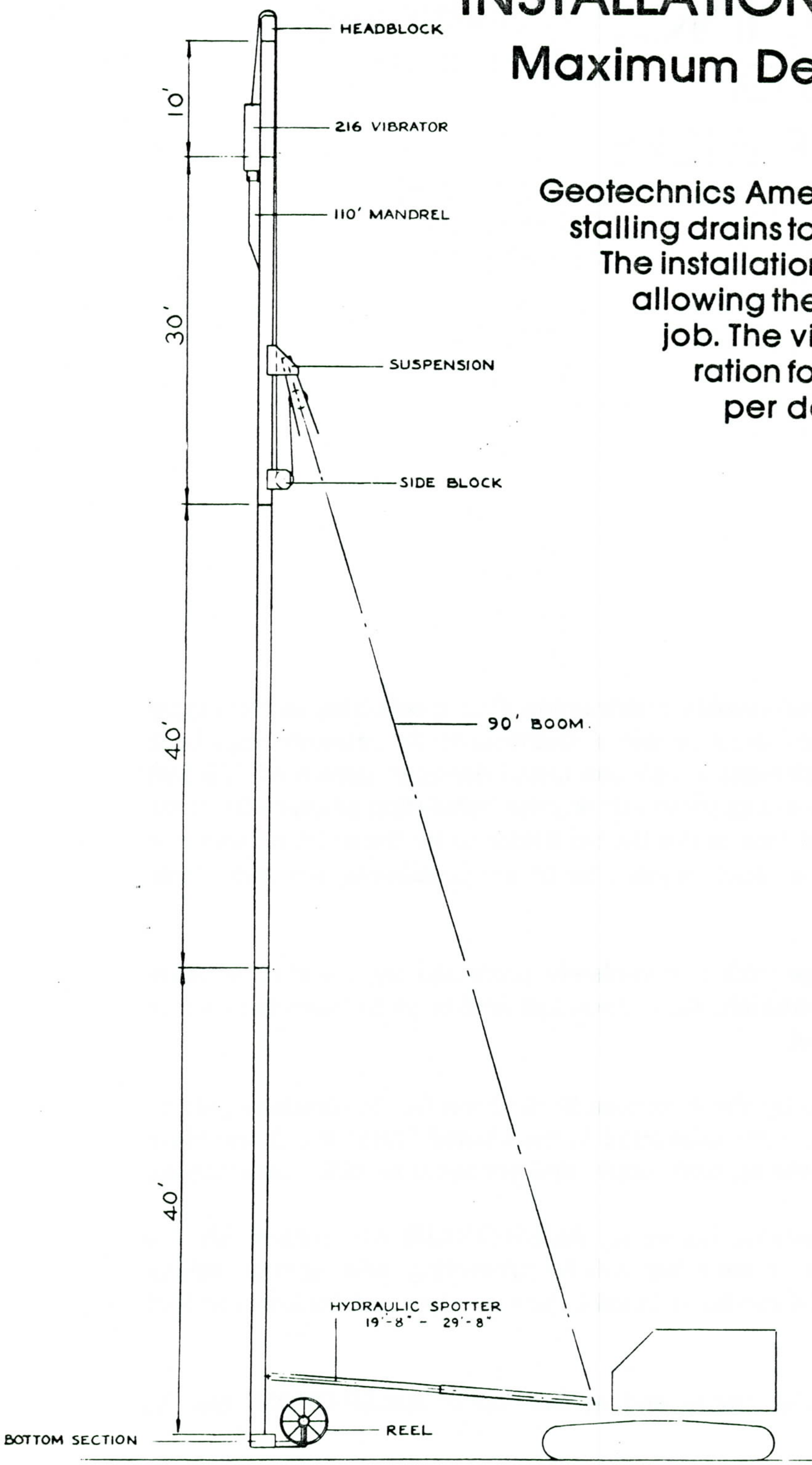
AMER-DRAIN is manufactured by the American Wick Drain Co. in Matthews, North Carolina. It is the only soil wick drain manufactured in the United States and is therefore the only wick drain that can offer the assured supply and economy of U.S. manufacture.

As illustrated in the accompanying literature, AMER-DRAIN can accomplish the stabilization and strengthening of water saturated soils by promoting their rapid consolidation through drainage. AMER-DRAIN can be installed in half the time and for less than half the cost of conventional sand drains.

We welcome inquiries about the supply and installation of AMER-DRAIN and its pricing for specific projects.

INSTALLATION EQUIPMENT CAPABILITY

Maximum Depth 125' (40 meter



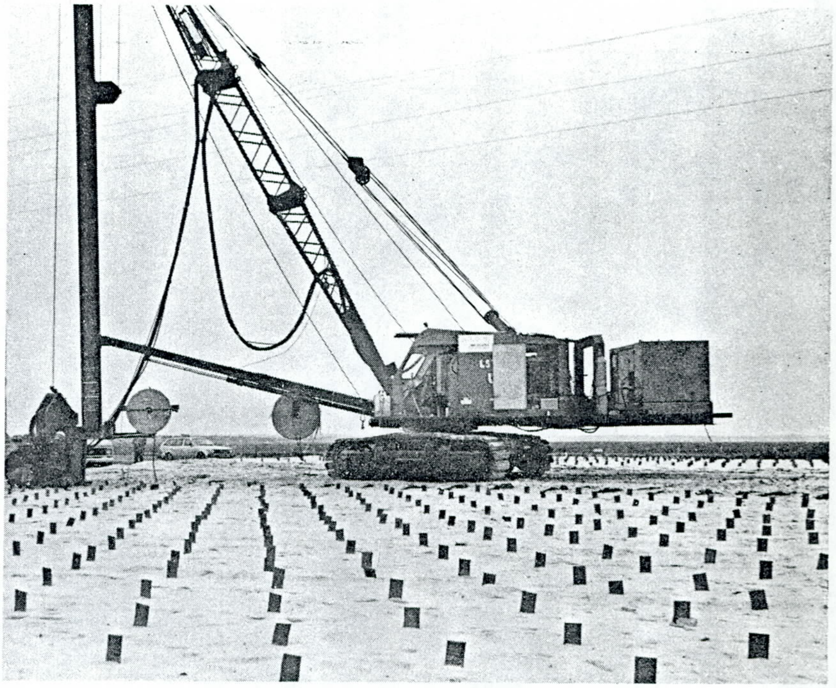
Geotechnics America's drain equipment is capable of installing drains to a maximum depth of 125 feet (40 meters). The installation mast and mandrel are modular thereby allowing the most efficient length to be used on every job. The vibratory driver provides 36 tons of penetration force. Production rates of 8000 - 10,000 feet per day are typical.

GEOTECHNICS AMERICA, INC.

P. O. BOX 552
BAY ST. LOUIS, MS 39520
(601) 255-3123

Printed in U.S.A.

New materials are speeding construction and cutting overall costs

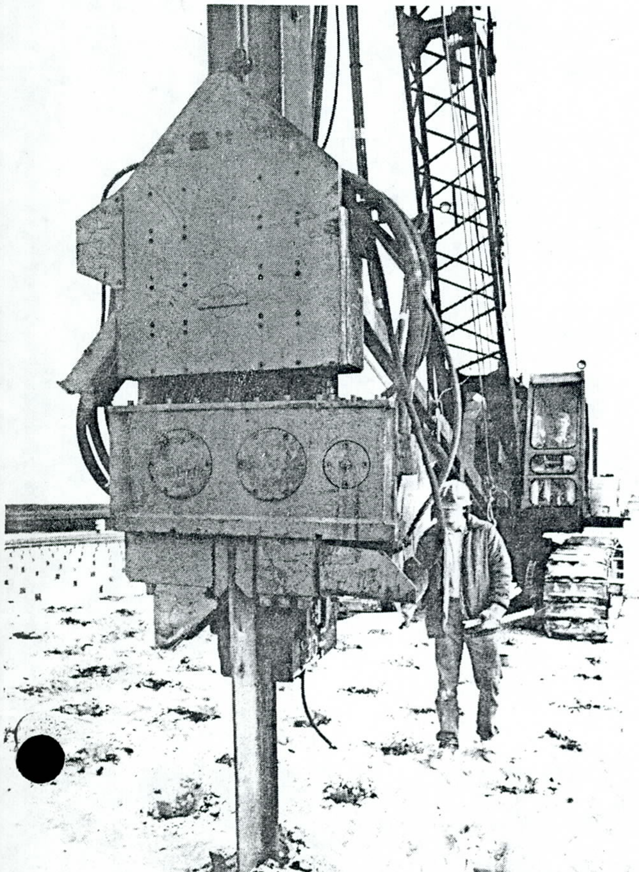


A Link Belt LS78 crane moves slowly through a *field of wick drains being installed to stabilize soil on a highway project.*

Construction of the Torras Causeway in Georgia marked the first time in that state that the vertical wick drain system was used to obtain soil consolidation on a major highway project. (See page 17 for additional photos and information.)

The $20 million effort to replace four outdated bridges and 2.95 miles of roadway was hampered by an almost uniform 20 ft. thick layer of loose water-saturated organic material at the location of the approaches to the bridges. Before any equipment could even be brought on to the site a layer of polyester woven fabric, Pro Pex 2006 by Amoco Fabrics Co., had to be layed down to form a stabilization blanket.

Southeastern Highway Contracting Co. of Gainesville, Ga., won the contract for grading and drainage which included site preparation of the bridge approach sections. The soil consolidation part of the work was subcontracted to Geotechnics America, Inc. Preassembled drain wicks were chosen to save both time and money.



An ICE 216 vibratory *driver is being used to place the wick drains.*

Nearly 640,000 linear ft. of wicks were used in the state of the art soil consolidation project. By incorporating the use of a vibratory installation rig the contractor was able to install the wicks without resorting to predrilling.

According to Russell Joiner, general manager of Geotechnics America, "Our initial estimate was to get between five and six thousand (linear) feet (of wicks) installed per day. Our actual production," he added, "averaged eight to ten thousand feet a day."

The company used an ICE 216 vibratory driver to push down the porous wicks to a depth of 25 ft. Geotechnics America used a Link Belt LS 78 "high walker" 20 ton capacity crane as the support platform for the ICE vibratory driver and 7,800 lb. power pack.

The general contractor's superintendent, John Cox, said that about three weeks after the vertical wick drains were installed in one area there was a measured seven and one-half inch ground settlement. The vertical wicks have enabled the state of Georgia to go ahead rapidly with a much needed project and save considerable money.

GEOTECHNICS AMERICA, INC.

P. O. Box 552
Bay Saint Louis, MS 39520
Telephone 601-255-3123

ADVANCED DRAINAGE SYSTEMS, INC.

3300 RIVERSIDE DRIVE P.O. BOX 21307 COLUMBUS, OHIO 43221 (614) 457-3051 TELEX NO. 245-461

June 13, 1985

Mr. Jim Wailes
Engr.
Simons Li & Assoc.
Box 1816
Ft. Collins CO 80522

Dear Mr. Wailes:

Enclosed is the literature you requested on ADS corrugated plastic drainage tubing and fittings.

In order to determine how we might address your specific drainage application, please take a moment to fill out the enclosed reply card and return it to me at your earliest convenience.

We appreciate your interest in our products and look forward to working with you in the near future.

Sincerely,

R. E. Slicker

R.E. Slicker
Market Manager

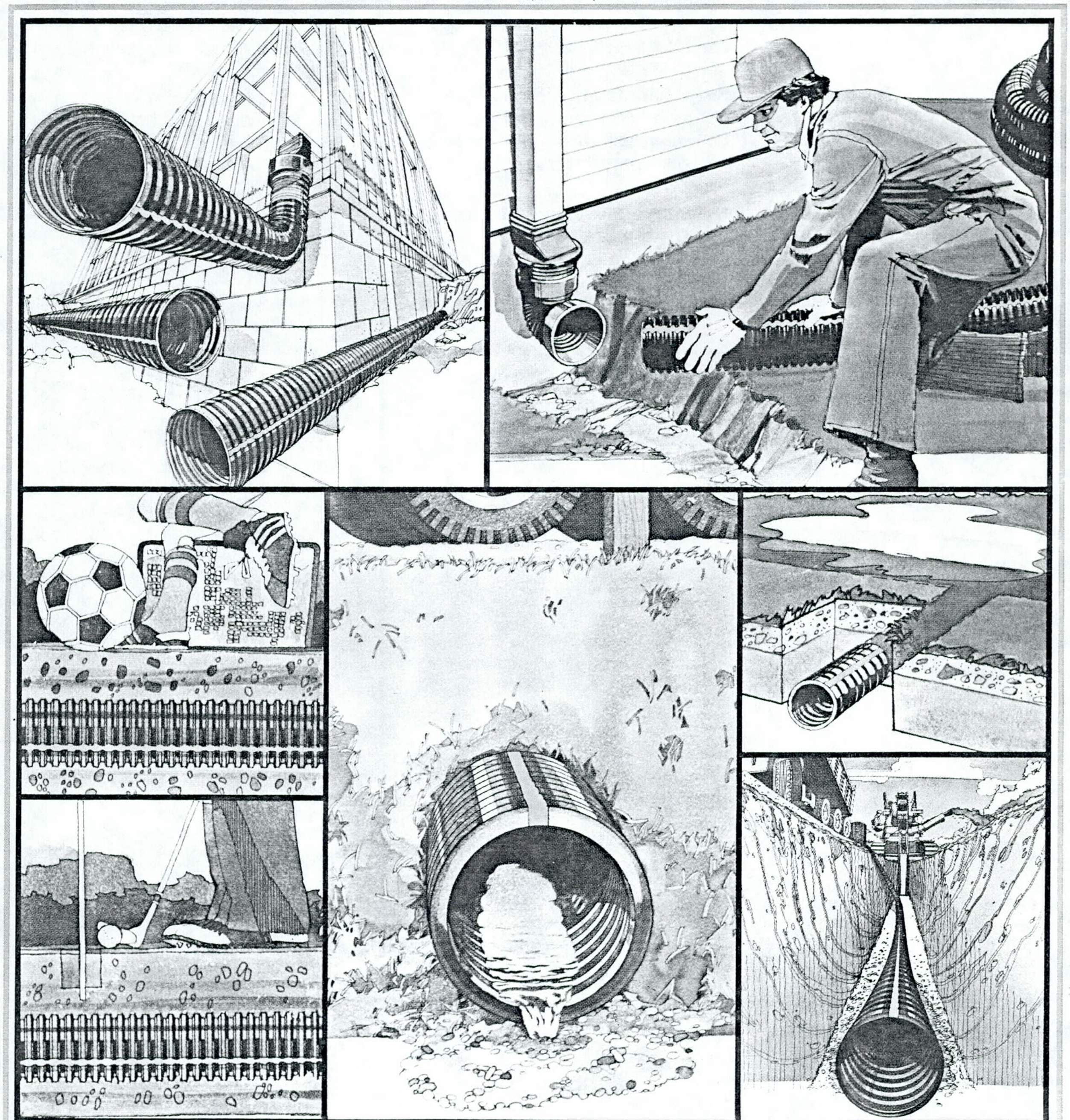
FJP/ti

Enclosure

0951

2.7a/Ad

ADS corrugated polyethylene tubing



We'll show you dozens of reasons why it's #1 in the land.

ADS®

ADVANCED DRAINAGE SYSTEMS, INC.

Choose ADS tubing

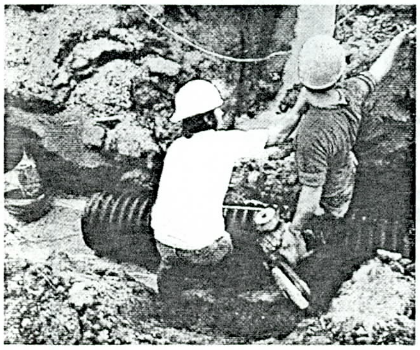
Corrugated polyethylene tubing from Advanced Drainage Systems, Inc. provides years of trouble-free drainage in a wide variety of applications, at a cost of just pennies per foot. Lightweight ADS tubing is available in continuous coiled lengths, or in straight lengths, and is flexible and easy to install, requiring less labor than traditional drainage materials. ADS tubing is manufactured with high density polyethylene resin, a virtually chemically inert material, so it resists corrosion and abrasion, and won't rot, rust or break down during handling.

ADS grain aeration pipe contains specially designed perforations with a durable, knitted polyester "sock" wrap to maximize air flow and prevent restriction of the perforations.

A full line of accessory fittings and couplings help simplify even the most challenging installations.

Residential and Commercial Construction

Ideal for all homesite and commercial drainage, ADS tubing is lighter, easier to handle, and requires less time and equipment to install than PVC, clay, concrete or corrugated metal. ADS snap-on fittings and couplings keep installation time and labor to a minimum.



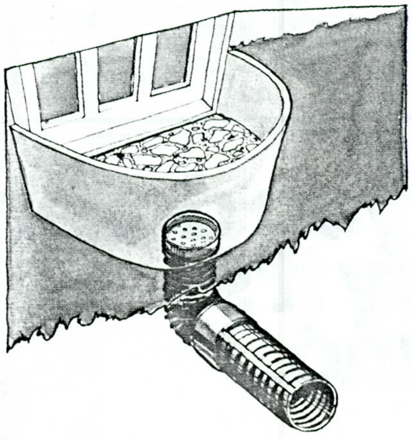
Exterior foundation drains are necessary for both residential and industrial buildings, below the level of the lowest floor, where high water tables and rainwater result in wet basements. These drains are placed to collect and channel water away from footers and basement walls to a suitable outlet.

Interior foundation drains where ground water is a problem. These drains intercept water that otherwise would gain entry through the basement walls or floor.

Downspout run-off drains using corrugated plastic pipe are used to channel water collected in the roof gutters to areas away from the building. These can be discharged into storm sewers, into the curb at the edge of the street, or into other suitable outlets.

Low-spot drainage in lawns or yards can be accomplished using surface inlets and corrugated polyethylene tubing to collect and carry the water to a storm sewer or other disposal area.

Basement window well drainage prevents rainwater from seeping down the foundation wall and entering the basement. This is accomplished by running a length of non-perforated tubing from the drain in the bottom of the well to the disposal area.

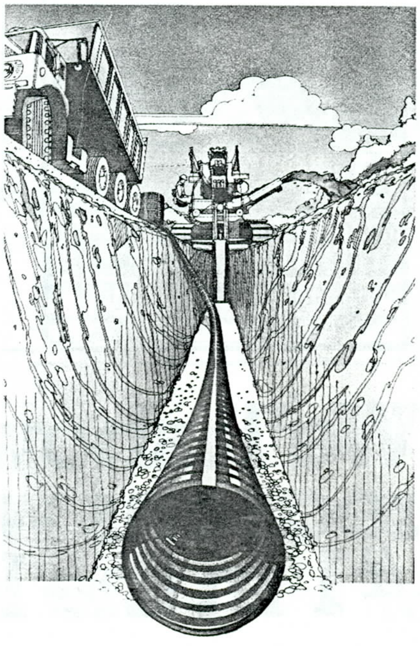


Driveway and sidewalk underdrainage is used to prevent frost damage or pavement deterioration due to unstable bases. Installation of perforated drainage tubing in a bed of gravel allows the water to drain out of the base course and be channeled away from the pavement.

SB2 gravel-less septic system, a recent innovation, is constructed of 8" and 10" tubing encased in a spun/bonded nylon mesh material, Drain Guard protective wrap and eliminates the need for gravel. Alternately, of the most common methods for home wastewater disposal are septic tank leach fields utilizing 4" ADS tubing, which features virtual immunity to the corrosive environment found in septic tank leach fields.

Highways and Roads

Excess water in the subbase of highway pavements is the leading cause of pavement failures; the adverse effects of inadequate drainage are evident in highways which begin to deteriorate after only two to three years. ADS corrugated polyethylene berm and underdrains collect and remove excess subbase water and reduce pavement damage. Continuous lengths require fewer fittings and connections and less labor to install.



for a variety of applications.

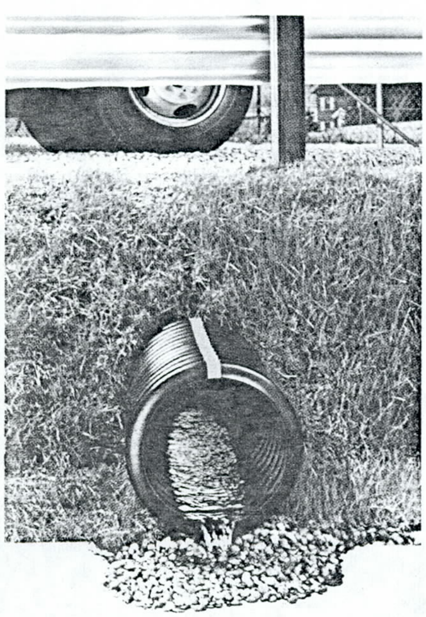
Culverts are easily drained with ADS culvert pipe; it is lighter and easier to handle and install than clay, concrete or corrugated metal, resulting in reduced labor costs. ADS culvert pipe has excellent load bearing strength, meeting the toughest requirements. It is approved by most state departments of transportation, and many county and local regulatory agencies. ADS culvert pipe is available in 10", 12" and 15" diameters, in 20' standard lengths.

Parking Lots

The durability of ADS tubing makes it especially suitable for parking lot applications. Excellent deflection qualities enable ADS tubing to resist critical loading conditions without damage.

Airport Runways

Airport runways suffer from the same water-related problems as highways and roads. Whereas corrugated steel, concrete or clay pipe have been widely used in the past, corrugated polyethylene tubing has recently been approved by the Federal Aviation Administration for use as collector systems, culverts and runway underdrains.



Golf Courses

Golf courses are kept lush, green and playable with ADS tubing. ADS tubing resists rot and is flexible, so it follows ground contours and adapts to underground obstacles. It's adaptable to a wide range of soil conditions, including sand traps that collect water or are subject to erosion. Proper installation is an important factor, and ADS provides detailed installation recommendations for every type of soil and topographical condition.

Athletic/Recreational

Strong and durable ADS tubing provides year after year of reliable drainage with minimal maintenance, to keep landscapes as hardy as they are beautiful. Slope drainage is easy with ADS. In sandy or other problem soils, ADS Drain Guard keeps drains flowing. In the case of athletic fields and other places where it is desirable to use the areas as soon as possible after a downpour, the ADS drainage system provides runoff that keeps up with rainfall.

Utility Companies

Public utilities and manufacturing companies have experienced problems with excess water in coal handling and storage operation. In the winter, freezing of wet coal is a problem, while during the warmer months, coal piles often must be sprayed with water to reduce coal dust and eliminate spontaneous combustion.

The ADS drainage system utilizing corrugated polyethylene tubing under the coal pile is an effective means of removing excess water. Filter protective wrap is required to prevent particles of soil or coal from entering the tubing.

Mining

ADS tubing offers low cost of installation plus excellent performance in corrosive and abrasive environments, solving mine-related water problems. These include drainage of coal piles, hollow-fills, earth dams, dam overflows, air ducts, deep shaft mines, sedimentation ponds, and roads.

Railroads

Poor railbed drainage often results in an unstable subbase and unsafe conditions. ADS tubing is used on new projects as well as to correct problems caused by excess water in existing railbeds. ADS performs under severe loading applications, making it ideal for railroad bed drainage.

Grain/Commodity Aeration

ADS aeration pipe can be easily adapted to all types of grain storage facilities (temporary as well as permanent), including metal buildings, round silos and wooden bins, resulting in uniform air flow.

Technical Notes

ADS corrugated polyethylene tubing is structurally designed to be used as culvert pipe and for other heavy duty drainage applications. This corrugated pipe may also be adapted to other drainage needs.

Applicable Specifications and Installation Guidelines

1. ASTM F 405, Standard Specification for Corrugated Polyethylene Tubing and Fittings.
2. ASTM F 667, Standard Specification for 10", 12" and 15" Corrugated Polyethylene Tubing.
3. AASHTO M 252, Standard Specification for Polyethylene Corrugated Drainage Tubing.
4. ADS Installation Guidelines for Culvert and Other Heavy-Duty Drainage Applications.

Look for the ADS green stripe.

It's your sign of quality— #1 in the land.

ADS®

ADVANCED DRAINAGE SYSTEMS, INC.

Easy-to-use heavy-duty ADS fittings

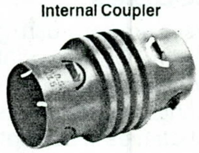
Split Coupling

3"–311 · 4"–411 · 5"–511 · 6"–611 · 8"– 811 · 10"–1011 · 12"–1211 · 15"–1511

Snap Coupling

3"–312 · 4"–412 · 5"–512 · 6"–612 · 8"–812 · 10"–1012

Internal Coupler



4"–415 · 5"–515 · 6"–615

Internal Reducing Coupler

5"–4"–425 · 6"–5"–526

Reducing Coupler

4" x 3"–314 · 5" x 4"–514 · 6" x 4"–614 · 6" x 5"–516 · 8" x 6"–816 · 10" x 8"–1018 · 12" x 10"–1210 · 15" x 12"–1512

Snap Tee

3"–321 · 4"–421 · 5"–525 · 6"–626

Saddle Tee

4"–443 · 5"–545 · 6"–646 · 8"–843

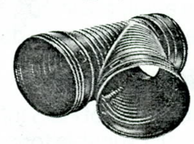
Blind Tee

3"–341 · 4"–441 · 5"–541 · 6"–641

Downspout Adapter

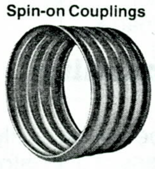
3" — 364 (3¼ × 2½)
4" — 464 (3¼ × 2½)
4" — 465 (3 × 4¼)

45° "Y"



3"–322 · 4"–422 · 5"–522 · 6"–622

Spin-on Couplings



18"–1811 · 24"–2411

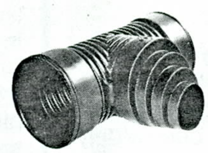
Split End Cap

3"–331 · 4"–431 · 5"–531 · 6"–631 · 8"– 831 · 10"–1031 · 12"–1231 · 15"–1531

Snap Adapter

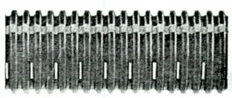
3"–362 · 4"–462 · 5"–562 · 6"–662 · 8"–862

Reducing Tee (Multiple)



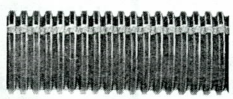
644–6" to 6"/6" to 5"
6" to 4"/6" to 3"
844–8" to 8"/8" to 6"
8" to 5"/8" to 4"
1044–10" to 10"/10" to 8"
10" to 6"
1244–12" to 12"/12" to 10"
12" to 8"
1544–15" to 15"/15" to 12"
15" to 10"

Perforated Tubing



3"–301–300' Coils
4"–401–250' Coils
5"–501–165' Coils
6"–601–100' Coils
8"–801– 20' Lengths
10"–1001–20' Lengths
12"–1201–20' Lengths
15"–1501–20' Lengths
18"–1801–20' Lengths
(301 and 401 also available in bundles of ten–10' lengths.)

Non-Perforated Tubing



3"–351–300' Coils
4"–451–250' Coils
5"–551–165' Coils
6"–651–100' Coils
8"–851–20' Lengths
10"–1051–20' Lengths
12"–1251–20' Lengths
15"–1551–20' Lengths
18"–1851–20' Lengths
(351 and 451 also available in bundles of ten–10' lengths.)

Protective Wrap

Drain Guard®	Sock
3"–372	373–300' Coils
4"–472	473–250' Coils
5"–572	573–165' Coils
6"–672	673–100' Coils
8"–872	873–20' Lengths
10"–1072	1073–20' Lengths
12"–1272	12"–1273–20' Lengths
	15"–1573–20' Lengths

Heavy Duty Tubing

Perf.	Non-Perf.
8"–801	851–20' Lengths
10"–1001	1051–20' Lengths
12"–1201	1251–20' Lengths
15"–1501	1551–20' Lengths
18"–1801	1851–20' Lengths
24"–2401	2451–20' Lengths

45° ELL

4"–445

90° ELL

3"–390
4"–490

Advanced Drainage Systems, Inc., is America's leading manufacturer of quality corrugated polyethylene pipe. Manufactured of selected polyethylene resins, ADS pipe meets the strictest product quality standards and industry specifications.

In addition, ADS manufactures a complete line of fittings and couplings, simplifying installations for highway and construction drainage applications.

From coast to coast, ADS tubing is available through the industry's most extensive distribution network. For the name of your local distributor, contact the nearest ADS sales office.

Nationwide Sales and Manufacturing Network

CALIFORNIA
Madera (209) 674-0054, (209) 674-0903*

GEORGIA
Atlanta (404) 393-0602*
Montezuma (912) 472-7556

ILLINOIS
Harvard (815) 943-5477
Monticello (217) 762-9448

IOWA
Cresco (319) 547-3105
Creston (515) 782-8565
Eagle Grove (515) 448-5101
Iowa City (319) 338-9448, (319) 338-3689*

KENTUCKY
Livermore (502) 733-4324
Versailles (606) 873-8046

MASSACHUSETTS
Palmer (413) 283-9797

MICHIGAN
Bad Axe (517) 269-9506
Owosso (517) 723-5208

NORTH CAROLINA
Rowland (919) 422-3303

OHIO
London (614) 852-9554
Napoleon (419) 599-9565, (419) 599-0585*
Wooster (216) 264-4949

TEXAS
Ennis (214) 875-6591

VIRGINIA
Buena Vista (703) 261-6131

WASHINGTON
Bellevue (206) 643-2770*
Washougal (206) 835-8522

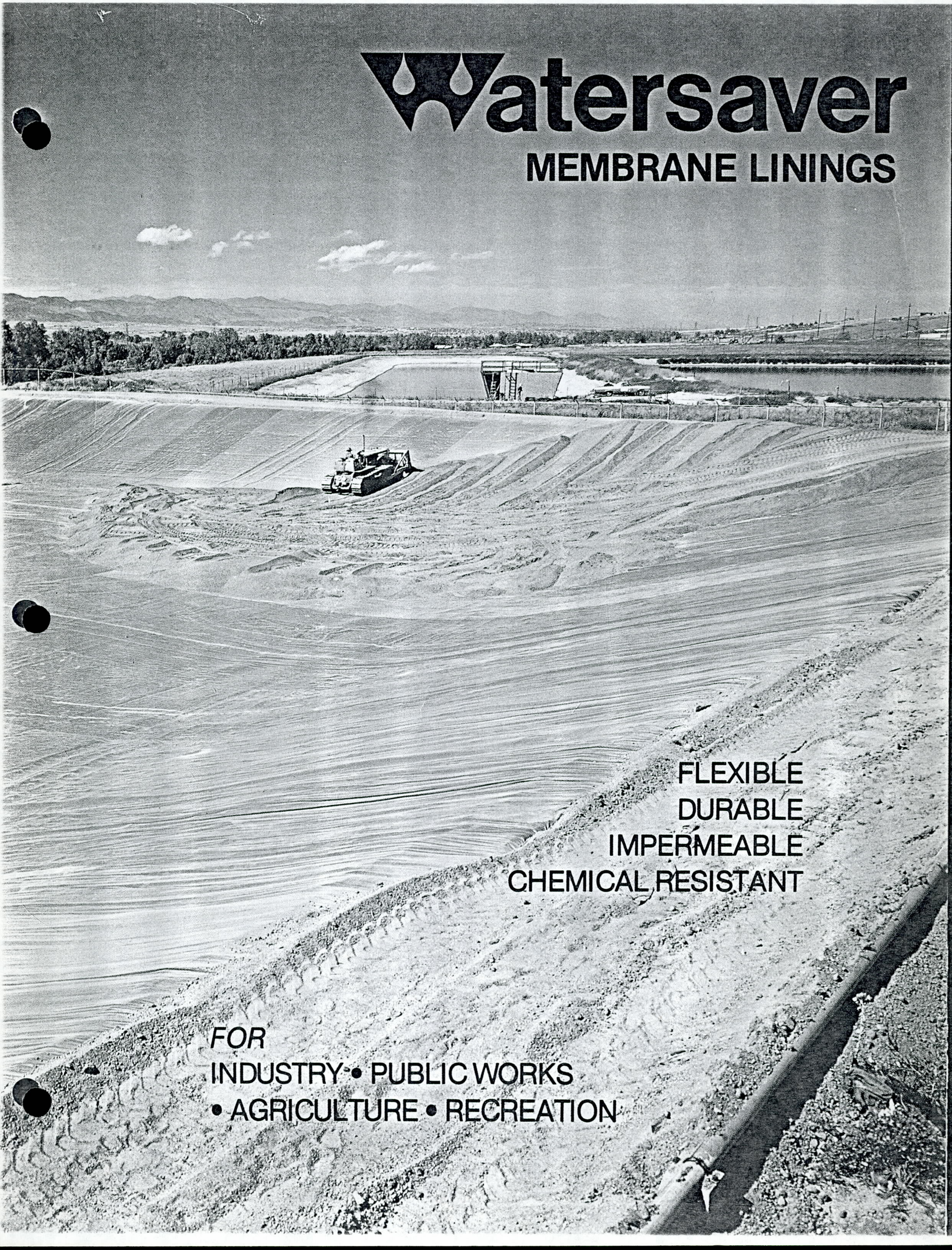
*Sales Office Only

Insist on the ADS green stripe.

It's your sign of quality—#1 in the land.

Corporate Office
3300 Riverside Drive
Columbus, Ohio 43221
(614) 457-3051

ADVANCED DRAINAGE SYSTEMS, INC.


Watersaver
MEMBRANE LININGS
FLEXIBLE
DURABLE
IMPERMEABLE
CHEMICAL RESISTANT
FOR
INDUSTRY • PUBLIC WORKS
• AGRICULTURE • RECREATION

WATERSAVER COMPANY, INC.

A company dedicated to service! For over twenty-five years we have been supplying membrane linings for installations throughout the world.

Watersaver Company growth and progress parallels the history of flexible membrane liner as used for liquid containment. We continue to develop lining systems and techniques for controlling fluids, water pollution and preventing see page.

Our experienced personnel have provided engineers with the professional know-how to assist many companies in solving their particular containment problems.

Watersaver membrane lining systems are in wide use throughout the world in water reservoirs, industrial waste ponds, chemical and brine storage, sewage lagoons and canals.

Watersaver Company, Inc. is the world's foremost fabricator of flexible membrane linings.

We have furnished liners for more installations than anyone in the world.

VATERSAVER INTERNATIONAL, LTD.

WATER POLLUTION CONTROL

Water quality awareness has developed a keen interest in pollution control in the world and particularly in the United States.

Congress has passed the Federal Water Pollution Control Act with the goal of eliminating discharge of pollutants by 1985. Under the Safe Drinking Water Act of 1974, States must now set their own standards for protecting ground water.

To meet this challenge, Watersaver Company adapted the use of its membrane lining systems to the problems of pollution control.

SEEPAGE PREVENTION

Federal and State pollution control agencies are demanding the control of seepage that pollutes ground water endangering life.

Watersaver Company works with engineers, architects, technicians, farmers, ranchers, industrialists and governmental agencies throughout the world, demonstrating how seepage problems are solved quickly - economically - and permanently with the proper installation and use of a membrane lining system.

Watersaver

THE PROFESSIONALS

Saving water is our business...and that's exactly what we do! This informative brochure introduces you to our **Membrane Lining Systems**. If you have a liquid containment problem, for action CONTACT THE PROFESSIONALS!

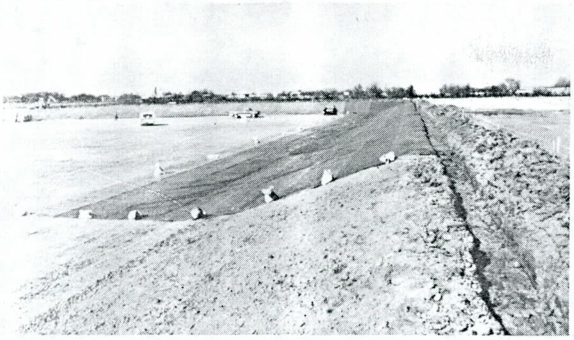
PRODUCT FEATURES

FLEXIBLE • DURABLE • IMPERMEABLE • CHEMICAL RESISTANT • WIDE PANELS • LATEST FABRICATION TECHNOLOGY • ECONOMICAL

USES

INDUSTRIAL PONDS • BRINE PITS • IRRIGATION RESERVOIRS • FIRE WATER STORAGE • SOLAR EVAPORATION PONDS • CANAL LININGS • FLY ASH & SOLID WASTE LEACHATE CONTROL • LANDSCAPE LAKES • SEWAGE LAGOONS • COOLING PONDS • SLUDGE DRYING BEDS • OIL SPILL CONTAINMENT • POTABLE WATER RESERVOIR LININGS & COVERS • AND MANY OTHERS

COVER PHOTO
Photo shows a protective earth cover being placed on a Watersaver Liner. This is the third industrial waste pond lined by Watersaver in five years at this location.



SITE PREPARATION

Excavation is completed by the earthmoving contractor. The base upon which the liner will be placed must be smooth, compacted and free of sharp rocks, roots and other foreign material, meeting the Engineer's specifications.

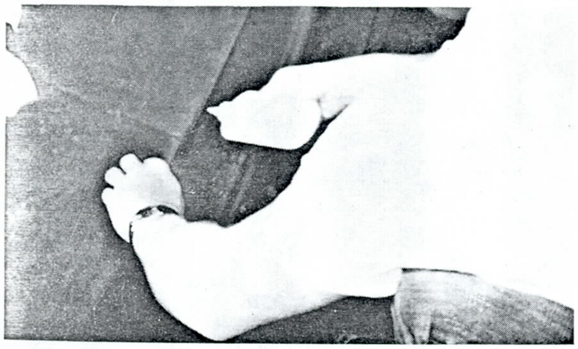
Structures including pipes, splash pads, inlets, outlets, and headwalls should be finished prior to placement of the liner.

POSITIONING THE PANEL

Liner panels may weigh as much as 2000 kg (4500 lbs.), therefore a large front end loader or forklift will assist in positioning the lining panels which are accordion folded in cartons on pallets.

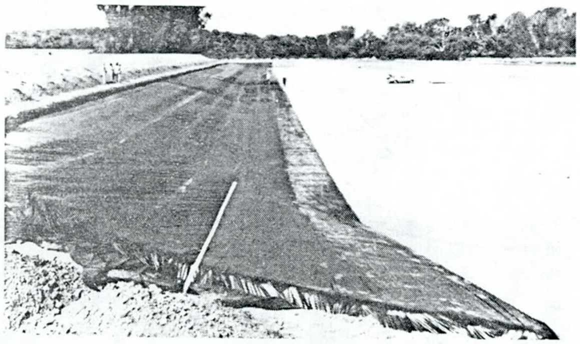
SPREADING THE PANEL

A crew of eight to ten men is needed to spread the panels to their full width. This crew installed 1 hectare (2.5 acres) in 8 hours.



FIELD SPLICING OF LARGE PANELS

Large factory fabricated panels are easily spliced together using specially formulated cements and adhesives developed by Watersaver Company.



ANCHORING TRENCH

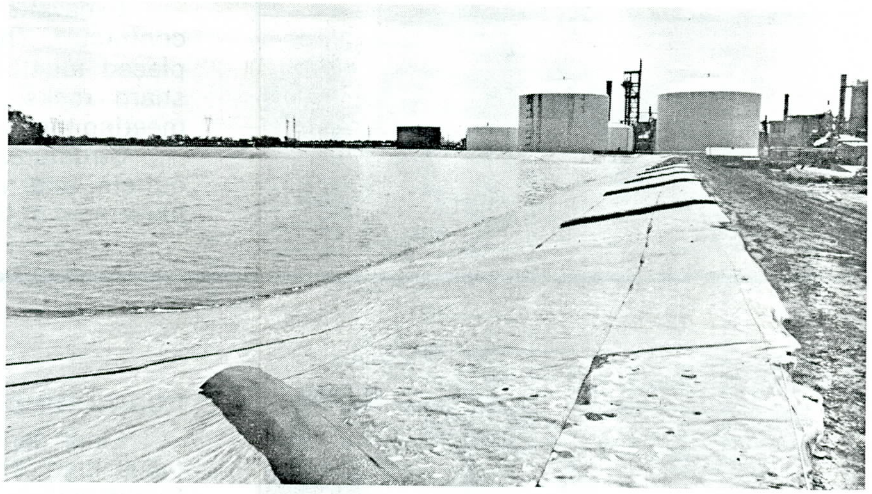
Liner panels are anchored in a trench at the top of the slope. The flat sheet conforms to the substrate in the corner.

GENERAL MEMBRANE LINER INFORMATION

FACTORY FABRICATION

Just as you know there is no one liner that meets all containment problems - there is no one system of fabrication that best serves each liner to be fabricated.

Watersaver has all fabrication systems and specifically uses the best sealing method for each membrane. Todays state of the art demands it - and Watersaver leads the way - Rigid Quality Control is your assurance of the Worlds finest membrane linings from Watersaver Company.



FACTORY FABRICATED ACCESSORIES

Watersaver Company takes pride in its fabrication facility. Factory fabricated accessories such as pipe seals, corners, berm vents, ballast tubes, pressure relief vents and sump liners are available to complete the Watersaver Membrane Lining System.

MEMBRANE CHARACTERISTICS

Watersaver Company membrane linings meet the most rigid specifications in the industry. Reinforced or non-reinforced lining materials may be selected in a variety of thicknesses ranging from .25 mm to 1.5 mm (10 to 60 mils). Material selection is based on specific project requirements.

PANEL SIZES

Watersaver panels are custom fabricated to specific project requirements to minimize field splices and installation time. Panel widths to 45 meters (150 ft.) are available.

INSTALLATION

Watersaver membrane lining systems are easily installed. A technical service representative is available from Watersaver Company to instruct the installation contractor in recommended procedures.

ECONOMY

Watersaver membrane lining systems are economical and long lasting. We stock all lining materials and match the specific project requirements with the particular membrane lining system that most economically meets those requirements.

	POLYVINYL CHLORIDE (PVC)	CHLORINATED POLYETHYLENE (CPE)	CHLOROSULFONATED POLYETHYLENE (HYPALON) (CSM)	ETHYLENE PROPYLENE DIENE MONOMER (EPDM)	POLYCHLOROPRENE (NEOPRENE) (CR)	ISOBUTYLENE ISOPRENE (BUTYL) (IIR)	OIL RESISTANT POLYVINYL CHLORIDE (ORPVC)
EXPOSED LINER	NR	R	RR	R	R	R	NR
EXPOSED SIDE SLOPE LINER	NR	RR	RR	RR	RR	RR	NR
BURIED LINERS	R	R	RR	R	R	R	R
ACID RESISTANCE pH 2 to 7	R	R	RR	R	R	R	R
ALKALINE RESISTANCE pH above 8	NR	R	RR	R	R	R	NR
PETROLEUM PRODUCTS	NR	R	NR	NR	R	NR	R
POTABLE WATER	NR	R	RR	R	NR	R	NR
DOMESTIC WASTE	R	R	RR	R	R	R	R
ROOFING MEMBRANE	NR	R	NR	R	R	NR	NR

R - RECOMMENDED　　RR - RECOMMENDED ONLY WITH REINFORCING　　NR - NOT RECOMMENDED

THE ABOVE ARE GENERAL GUIDELINES ONLY. MATERIAL SELECTION SHOULD BE BASED ON SPECIFIC PROJECT REQUIREMENTS. CONTACT WATERSAVER FOR RECOMMENDATIONS.

PIONEERING MANUFACTURING TECHNIQUES

Plants and equipment are only as effective as the people who run them. At Watersaver Company we employ experienced personnel for efficient design, engineering and product construction concepts. All system planning, engineering, design and fabrication are worked out in advance through modern technology and machinery. Watersaver Company makes certain that quality is maintained by responsible factory trained employees. We provide assistance to Engineers and Contractors in the design and installation of lining materials.

Watersaver MEMBRANE LININGS

HYPALON®
(Chlorosulfonated Polyethylene)

...provides excellent resistance to weathering and chemical attack. Hypalon is available only as a reinforced membrane and does not require a protective cover for most applications. Hypalon is approved for potable water containment.

PVC (Polyvinyl Chloride)

...membrane offers good chemical resistance, sealability, and serviceability in unexposed applications. It has performed satisfactorily as a liner for recreational lakes, canals, evaporation ponds, sewage lagoons, brine ponds, etc. It is recommended that an earthen cover be provided for PVC to maximize its service life as a fluid barrier.

OR CPE (Chlorinated Polyethylene)

...specifically formulated for resistance to oils. Membrane features excellent weatherability, sealability, chemical resistance and long term durability. CPE does not require a cover material for most applications.

OR CPE R
(Reinforced Chlorinated Polyethylene)

...specifically formulated for resistance to oils. Offers all of the desirable characteristics of Watersaver CPE and in addition, provides greater strength and resistance to creep, sagging, and puncture where conditions of use are severe, such as steep slopes or other high stress applications.

EPDM
(Ethylene Propylene Diene Monomer)

...has been used for roofing and lining applications for many years. Superior weathering and elongation characteristics have made EPDM the most widely used single ply roofing membrane in the U.S.A.

EPDM R (Reinforced EPDM)

...has the superior weathering characteristics of the non-reinforced EPDM with additional strength and tear resistance required by some applications. Many potable water reservoirs are rehabilitated with EPDM R or Hypalon.

NOTE: Product information is of a general nature. Specific application may vary.

PROBLEM SOLVERS

Watersaver has probably solved a problem similar to yours - for someone - somewhere! Our long experience in the field enables us to evaluate the situation and arrive at the correct solution without wasting time or money. We know the proper applications for flexible linings, and our recommendations are based on facts.

DESIGN ASSISTANCE

After we receive the information on your particular project from you, we can study your needs and make initial evaluations and recommendations. We provide drawings showing membrane panel layout for approval.

ACCEPTABILITY

Watersaver has achieved prominence in its field for the development of membrane linings to meet current State and Federal requirements for water quality and pollution control practices. New liners are being field tested continually. Thus, we bridge the important gap between laboratory technology and job-site requirements with liners of proven capability.

WARRANTY

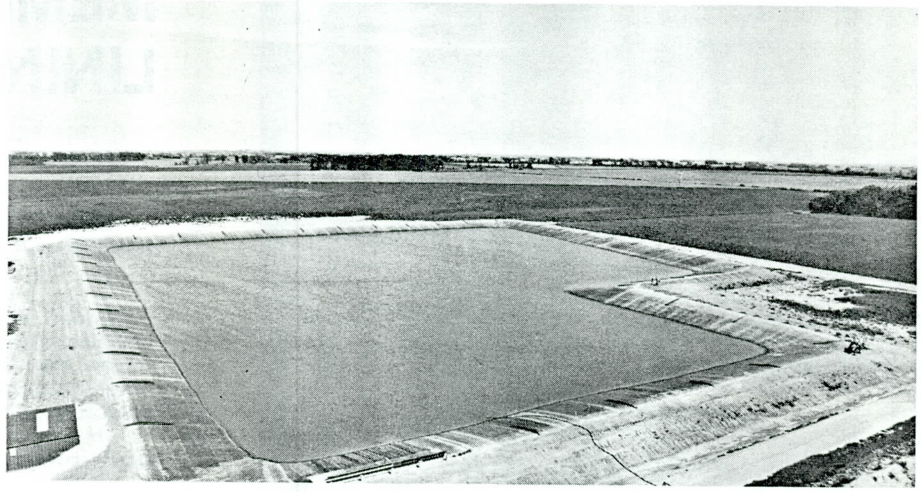
Our warranty and agreements policy reads in part that fabrication of the roll goods into panels by the Watersaver Company is warranted to be free from defects in workmanship under normal use and service.

ADDITIONAL INFORMATION

We can provide you with information pertaining to your specific requirements.

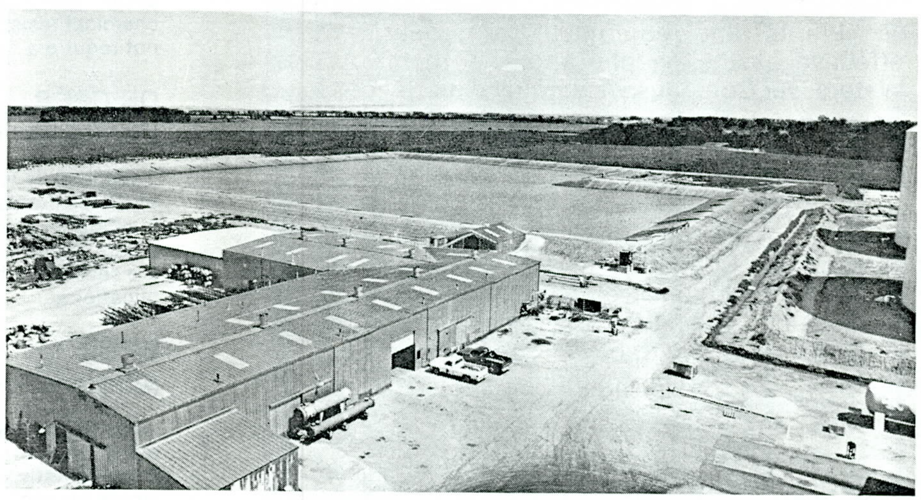
AVAILABILITY

Watersaver Company maintains the largest inventory in the industry. Standard panels are available for immediate shipment.



WATERSAVER MEMBRANE LININGS

STOP SEEPAGE ECONOMICALLY...



LITHO IN U.S.A.

CATALOG NO. GB 82

WATERSAVER DENVER

WATERSAVER COMPANY, INC.

P.O. BOX 16465, DENVER, COLORADO USA 80216

303-623-4111 (TWX)910-931-0433

WATERSAVER INTERNATIONAL, LTD.

Have a question about liners? Call Toll Free 800-525-2424

COTTER CORP.'S NEW TAILINGS POND HAS A SYNTHETIC LINER

Reprinted from July 1981

E&Mj ENGINEERING AND MINING JOURNAL

COTTER CORP.'S NEW TAILINGS POND HAS A SYNTHETIC LINER

About 450 panels of Hypalon weighing about 4,000 lb each were used. Field seaming of the panels was done with a solvent-type adhesive.

Raymond Thorpe, National sales manager
J.P. Stevens Co. Inc., Elastomeric Products Dept.

When Cotter Corp., a large vanadium and uranium processor in Canon City, Colo., decided to build a new tailings pond, it was required to make special provisions to prevent seepage. Cotter had recently built a new mill that handles up to 1,200 st/d of ore. A two-cell impoundment was designed to hold both acidic tailings from the new mill and reprocessed alkaline tailings from older ponds. It will also receive runoff from the mill site.

It has taken two years and $21 million to finish Stage 1 of the impoundment, which is lined with a membrane of Du Pont's Hypalon synthetic resin. When a Stage 2 expansion is completed, the impoundment will have a surface area of 175 acres and a maximum depth of 90 ft, making it the largest two-cell uranium tailings impoundment in the world.

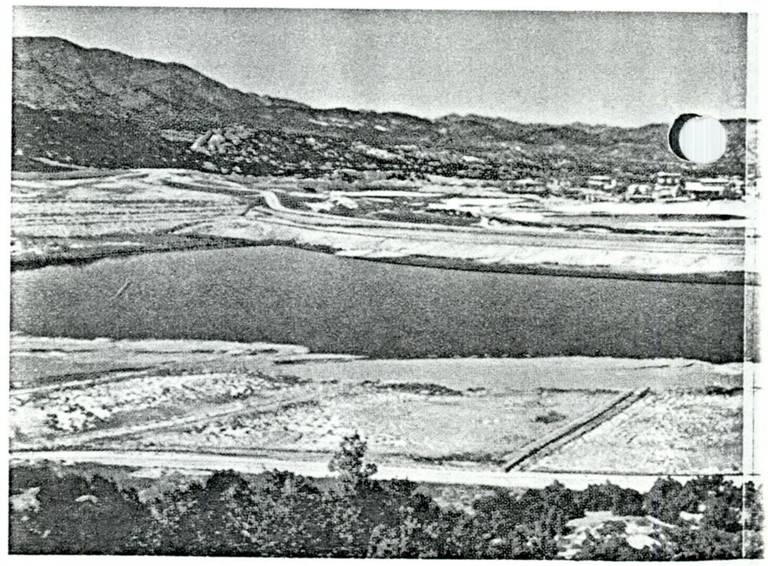
"After a year in use, we haven't experienced any leakage," says Joseph McCluskey, Cotter's executive vice president. The pond has a 20-yr (life of the mill) storage capacity of 7,500 acre-ft.

The primary design objectives for the lining were:

- To prevent leakage.
- To withstand tailings and water pressure in the deepest part of the pond (the tailings are 80% water).
- Because of the long-term radioactivity of the tailings, to provide a lining that would be durable and effective well beyond the life of the mill.
- To contain both alkaline and acid wastes.

Single linings, like clay, asphalt, and concrete, were carefully evaluated. Portland cement was rejected because of its rigidity, incompatibility with acid wastes, and the high cost of installation. Hydraulic asphalt concrete was ruled out because of quality control problems during construction. Clay alone, normally a good seepage barrier, would not be reliable enough in a uranium tailings pond without additional protection.

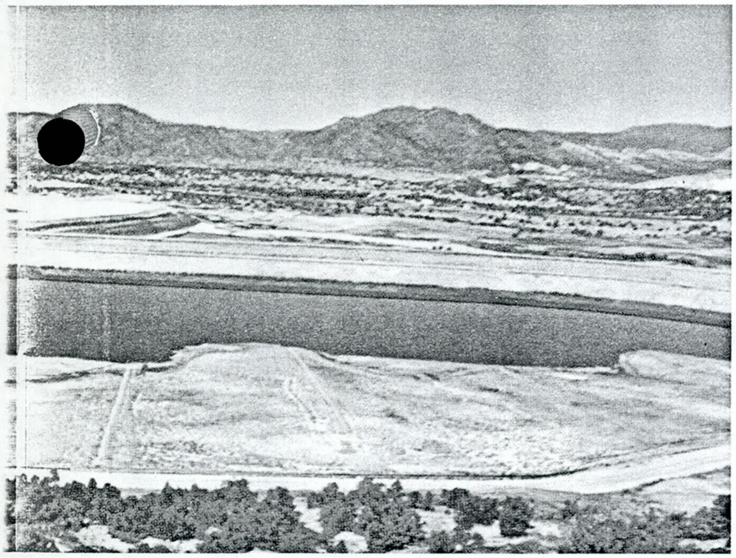
Hovater-Way Engineers Inc., a Laguna Hills, Calif., consulting engineering firm, recommended a multi-component lining system using an impervious membrane as the primary seepage barrier. But membranes require a smooth underlying surface to protect against puncture and tearing. Although clay was not chosen as a single lining, it does qualify as a supporting surface, because it is smooth and can act as a secondary seepage barrier if the membrane is damaged.



The Cotter Corp. uranium tailings impoundment has a Stage 2 depth of 90 ft. The total weight of the liner is 1.8 million lb.

Several kinds of rubber and synthetic materials, such as EPDM (ethylene-propylene-diene-monomer), PVC (polyvinyl chloride), and Hypalon, were considered for the membrane. The Elastomeric Products Dept. of J. P. Stevens Co. Inc., Easthampton, Mass., a manufacturer of synthetic rubber sheeting made with Hypalon, was one of several companies that tested various materials for compatibility with both the alkaline and acid wastes involved in this job. Steven's

Each pond lining panel was spread to its full width by trapping air under it to float it into position.



testing led to the development of an industrial-grade Hypalon sheeting consisting of reinforcing fabric sandwiched between two sheets of Hypalon. The Hypalon is nearly 50% of the total liner weight. The reinforcing fabric is a polyester whose open weave allows the synthetic rubber to penetrate the fabric, resulting in excellent adhesion between layers.

The industrial-grade sheet has improved weight and volume change properties compared with "potable water"-grade Hypalon synthetic rubber. The sheeting produced by Stevens was made into various sized large panels by Watersaver Co. Inc., Denver, fabricator of flexible membrane linings for the Cotter tailings pond.

The Cotter job was the first use of industrial-grade sheeting containing Hypalon. "Since then, it's become very popular," says Bill Slifer, vice president of Watersaver.

SURROUNDED BY CLAY AND EARTH

The first installation step at Cotter was covering the pond bottom with a smooth layer of compacted clay at least 18 in. thick. To install the rubber membrane lining, a crew of 10 men placed a panel of Hypalon on the clay, laying it out to its full length, then spreading the panel to its full width by trapping enough air under the panel to float it into position. A crew of three men seamed the panels together with a solvent-type adhesive that chemically welds the two matching surfaces of the field seam.

The job required roughly 450 panels of Hypalon weighing about 4,000 lb each, for an installed weight of 1.8 million lb and a surface area of 6.5 million ft^2. Three different thicknesses of Hypalon were used in the pond: 36 mils (about 1/32 in.) for the shallow area; 45 mils (about 3/64 in.) for mid-depths; 60 mils (about 1/16 in.) for the deepest parts.

The membrane was secured at the pond edge in an 18-in.-deep, 24-in.-wide anchor trench at the berm. The upper edge of the lining on the side slope was anchored in this trench by backfilling and compacting. This increased the stability of the lining on the side slope. A continuous strip of Hypalon was attached to the lining near the top of the impoundment slope. During Stage 2 expansion, the strip will be removed, exposing the unweathered lining and allowing the seaming of additional, new membrane lining to the old lining.

Twelve inches of earth was spread on top of the entire lining to protect it from possible tearing or puncture. There are also subdrains beneath the clay sub-lining that relieve reverse hydrostatic pressure and prevent damage to the lining by up-lifting. The grade of the ponds ranges from a minimum slope of 20:1 to a maximum of 3:1. Any gas forming under the membrane can drift upward and outward to the perimeter of the lining and escape through vents.

After one year, the impoundment contains 420 acre-ft of liquid. Currently, two-thirds of this liquid is runoff, while the rest is tailings and liquids from the new mill. Once the impoundment is filled with tailings, it will be covered with earth.

ENGINEERING SPECIFICATION GUIDE
.036 INDUSTRIAL GRADE HYPALON®
SUPPORTED WITH 10 × 10 1000d SCRIM

WATERSAVER CO., INC. • P.O. Box 16465 Denver, CO 80216 • Plant/Gen. Office 5870 E. 56th Ave., Commerce City, CO 80022 •
Phone 303-623-4111 • Colo. WATS 800-332-1971 • Interstate WATS 800-525-2424 • TWX 910-931-0433 (Watersaver Dvl) •

1. SCOPE

1.1 The scope covered by these specifications covers the furnishing and installation of a fabric-reinforced industrial grade Hypalon lining. All work shall be done in strict accordance with the engineers drawings and specifications.

2. CONTRACTOR'S EXPERIENCE

2.1 Any contractor proposing to perform the work hereunder shall have demonstrated his ability to do the work by having successfully installed at least two million square feet of reinforced membrane lining.

3. LINING MATERIAL

3.1 The membrane lining material shall be fabric-reinforced Hypalon of new, first-quality products designed and manufactured specifically for the purpose of this work, and shall have been satisfactorily demonstrated by prior use to be suitable and durable for such purposes. The manufacturer shall have produced, and have in service in similar applications for a period of not less than one (1) year, at least five (5) million square feet of fabric-reinforced industrial grade Hypalon material utilizing the same scrim specified for use under these specifications.

3.2 Hypalon utilized for encapsulation of the scrim shall be manufactured from a composition of high quality ingredients, suitably compounded, of which Hypalon 45 synthetic rubber resin is the sole elastomer. Zinc compounds of any kind, including zinc oxide, zinc stearate and zinc dusting agents, are prohibited. Dusting agents of any kind of prohibited on the finished product.

3.3 Scrim used in the membrane shall be 10 × 10 1000d polyester of an open type weave that permits strike-through of the Hypalon through the fabric to facilitate adhesion between the plies of Hypalon. The fill yarn must have 2.5 turns per inch maximum and 2.0 turns per inch minimum. All selvage edges must be trimmed prior to applying the Hypalon coating.

3.4 The composite membrane material shall consist of a thoroughly bonded, fabric-reinforced Hypalon rubber sheeting. It shall be manufactured by the calendering process and shall be uniform in color, thickness, size, and surface texture. The fabric shall be totally encapsulated between plies of Hypalon and shall not extend closer than 1/8 inch to the edge of the Hypalon coating either side of the fabric. Exposed fabric along longitudinal edges of roll stock and indications of delamination will not be permitted. The composite material shall be a flexible, durable, watertight product free of pinholes, blisters, holes, and contaminants and shall not delaminate in a water environment.

The composite membrane material shall be fabric-reinforced Hypalon consisting of one ply of scrim and two plies of Hypalon.

Property	Specification	Test Method
Tensile Strength, psi, min.	1500	ASTM D-412
Elongation, @ Break % min.	300	ASTM D-412
Water Absorption, (max. wt. gain), %		ASTM D-471
7 days @ 70°F	1.0	
14 days @ 70°F	1.0	
30 days @ 70°F	1.0	
14 days @ 158°F	30.0	
30 days @ 158°F	30.0	
Low Temperature, Cold Bend, 1/8" mandrel for 4 hrs., °F	-45	ASTM D-2136
Ozone Resistance (3 ppm @ 30% strain @ 104°F, 72 hrs.)	Pass	ASTM D-1149
Heat Aging, (14 days @ 212°F)		
Tensile Strength, psi, min.	1500	ASTM D-412
Elongation, % min.	150	

ENGINEERING SPECIFICATION GUIDE
.036 INDUSTRIAL GRADE HYPALON® SUPPORTED WITH 10 × 10 1000d SCRIM

Property	Specification	Test Method
Thickness	.036 and not less than .033	ASTM D-751
Breaking Strength, lbs., min.	200	ASTM D-751 Grad Method
Tongue Tear, lbs., min.	80	ASTM D-413
Ply Adhesion, Machine Method 180° peel, lbs./2" width, min.	8	ASTM D-413 Method A

3.5 The fabricator shall be an experienced firm customarily engaged in factory-fabricating individual widths of fabric-reinforced Hypalon roll stock into large sheets. Factory seams shall have a minimum of 1-½" scrim to scrim overlap when made by the solvent seaming method, and 5/8 inch scrim to scrim overlap when made by the heat welded method.

Each factory-fabricated sheet shall be given prominent, unique indelible identifying markings indicating proper direction of unrolling and/or unfolding to facilitate layout and positioning in the field. Each factory-fabricated sheet shall be individually packaged in a heavy cardboard or wooden crate fully enclosed and protected to prevent damage to it during shipment, prominently identified in the same fashion as the sheet within and showing the date of shipment. Until installed, factory-fabricated sheets shall be stored in their original unopened crates; if outdoors, they shall be stored on pallet and shall be potected from the direct rays of the sun under a light-colored heat-reflective opague cover in a manner that provides a free-flowing air space between the crate and cover.

4. OTHER MATERIALS

4.1 Solvent for cleaning contact surfaces of field joints and for other required uses shall be as recommended by the manufacturer or approved fabricator of the fabric-reinforced Hypalon

4.2 All seaming, sealing and high-solids adhesives shall be of a type or types recommended by the manufacturer or approved fabricator of the fabric-reinforced Hypalon and shall be delivered in original sealed containers.

5. INSTALLATION

5.1 Prior to ordering fabric-reinforced Hypalon material, the contractor may submit, for the engineer's approval, shop drawings showing lining sheet layout with proposed size, number, position, of all factory-fabricated sheets and indicating the location of all field joints. Shop drawings may also show complete details and/or methods for anchoring the lining at top of slope, making field joints, seals at structures, etc.

5.2 Lap joints shall be used to seal factory-fabricated sheets of fabric-reinforced Hypalon together in the field. All field joints between sheets of fabric-reinforced Hypalon shall be made on a supporting smooth surface and, unless the weather is sufficiently warm, heat guns shall be used to make the sealing temperature at least 90°F. The lap joints shall be formed by lapping the edges of sheets a minimum of 3" scrim-to-scrim. The contact surfaces of the sheets shall be wiped clean to remove all dirt, dust, moisture, or other foreign materials then wiped clean. Sufficient Hypalon-to-Hypalon bonding adhesive shall be applied to both contact surfaces in the joint area and the two surfaces pressed together while wet and immediately rolled. Any wrinkles shall be smoothed out and any cut edges of the fabric-reinforced Hypalon shall be sealed with a Hypalon adhesive to prevent wicking.

5.3 Any necessary repairs to the Hypalon membrane shall be patched with a piece of the membrane material itself and Hypalon-to-Hypalon adhesive. The adhesive shall be applied to the contact surfaces of both the patch and lining to be repaired, the two surfaces pressed together immediately and rolled, and any wrinkles smoothed out, all in accordance with Paragraph 5.2 hereof.

5.4 All joints, on completion of the work, shall be tightly bonded. Any membrane surface showing injury due to scuffing, penetration by foreign objects, or distress from other causes shall, as directed by the engineer, be replaced or repaired with an additional piece of fabric-reinforced Hypalon membrane of the proper size.

5.5 On completion of installation, the contractor shall dispose of all trash, waste, material and equipment used in connection with the work hereunder, and shall leave the premises in a neat and acceptable condition.

6. SEAM STRENGTH

6.1 All factory and field seams (joints) shall, after 12 days, have a seam strength of 200 pounds when tested in accordance with ASTM D-751, Grab Method (using 4" wide specimens having a length of 10" plus the seam width). The distance between the jaws of the testing apparatus at the start of the test must be 8" plus the seam width and shall have sufficient strength in peel that they fail by delamination from the scrim rather than in the plane of the seam.

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