Sand Drain Theory and Practice

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The theoretical approach to the design of sand drain installations has often proved inadequate in the prediction of field performance. The divergence of field performance from designs based on data obtained from tests of undisturbed samples has been found to be greatest where displacement methods of sand drain installation are employed and least where nondisplacement techniques are used. The nondisplacement methods most commonly employed involve controlled jetting and augering systems. Nondisplacement methods are not equivalent with respect to the avoidance of subsoil disturbance; therefore, some divergence between designs based on undisturbed sample test data and field performance is still encountered. This paper reviews the basis for using sand drains and the background of the development of nondisplacement techniques and presents a systematic approach applicable to the evaluation of all installation methods. This approach is essential if the designer is to be provided with all tools necessary for the development of sand drain designs that have a reliable factor of safety when applied in construction, as the nondisplacement methods in use today do not produce comparable results in the field. The results of the Maine test section are reviewed to demonstrate how equivalent designs can be developed for specific methods of installation and specific types of soil.

Where stability problems or excessive residual primary settlements are anticipated, the use of sand drain stabilization should be considered either as an alternative to or in conjunction with preloading in the development of a feasible design. Other methods of construction should be considered and priced, and the final selection made on the basis of cost as well as on reliability of the design in producing the required result. Such intangibles as aesthetic and environmental effects during and after construction should be given due consideration.

The inconsistent performance of sand drain projects has often been related to an incomplete evaluation of factors involved in design (1). However, even where design theory (2) has been properly applied, it has been found that the effects of the installation procedure employed can be a determining factor in performance. This paper describes the use of empirically derived parameters in design to compensate for aspects of the installation procedure and discusses standardization of sand drain installation procedures as a means of minimizing the introduction of construction-related variables.

BACKGROUND

The history of sand drain stabilization is well documented (1) and need not be recounted. Typical displacement methods available are listed below. (1 cm = 0.39 in. Paper, Sandwick, and Fabridrains may be protected by U.S. patents.)

Descriptive Name	Usual Diameter (cm)	Backfill
Driven mandrel (closed end)	30 to 50	Placed at time of mandrel withdrawal
Paper drains (Kjellman method)	10 x 0.3	Drainage strip placed at time of mandrel withdrawal
Sandwick drains	Up to 15	Prepacked fabric filled with sand placed after hole is made by mandrel
Fabridrain	Up to 15	Fabric liner filled with sand placed at time of mandrel withdrawal

The first sand drain project specifically designed for installation of drains by nondisplacement methods that was successfully completed involved stabilization of sensitive varved silt and clay deposits, which supported the approach embankment of the State Street Bridge in East Hartford, Connecticut. This project, which involved use of the flight auger method, was the forerunner of many successful projects constructed in Connecticut (3) that involved similar soil conditions. The concept of nondisplacement as an element in sand drain stabilization design was later adopted by the New York State Department of Transportation for stabilizing highway embankment foundations on sensitive organic clay deposits in the borough of Queens in New York City (4, 5). [A previous project in the same soil deposit involving the use of the mandrel method proved unsatisfactory because of excessive settlements caused by remolding of the soil structure (4).]

After the success of the auger method used in sensitive soils in Connecticut and New York, other states adopted the use of augered sand drains. As a consequence, other nondisplacement methods, as listed below, were introduced (6, 7, 8, 9) in the United States. (1 cm = 0.39 in. Jet-bailer and Sandwick drains, jet augers, and the auger method may be protected by U.S. patents.)

Descriptive Name	Usual Diameter (cm)	Backfill
Pressed casing	Up to 45	Placed at time of casing withdrawal
Jet-bailer drains	30	Placed after hole is formed
Jet augers and jet casing	Up to 45	Placed at time of drill or casing withdrawal
Jetted mandrel	30 to 50	Placed at time of mandrel withdrawal
Rotary jet	Up to 50	Placed after hole is formed
Auger method	Up to 45	Placed during or after auger withdrawal
Sandwick drains	Up to 15	Fabric-filled wick placed in any suitable hole formed by nondisplacement techniques

The improvement in the performance of nondisplacement installations over displacement methods has been related to a reduction in smear. Smear (1) is defined as the ratio of the diameter of the remolded zone immediately adjacent to the cavity periphery to the diameter of the cavity itself. Inasmuch as the formation of the remolded zone can only be produced by lateral soil displacement (5), the simple rubbing of a cavity-forming tool over the periphery of cavities formed by nondisplacement techniques will not result in smear when the rate of tool advance is controlled to ensure full cavity excavation. Any differences in performance that may be observed between various displacement techniques as well as between various nondisplacement installation methods must be related to disturbance effects associated with field as well as operating conditions. As such, various techniques may be substantially superior in one type of soil than in another (10) for reasons that are not definable by purely theoretical considerations.

SAND DRAIN DESIGN

The principal purpose of a sand drain installation is to accelerate the primary consolidation of compressible subsoils during the construction period and to limit the magnitude and rate of postconstruction settlements to acceptable values. Slope stability is improved as a result of a concurrent increase in soil strength. Most often, this involves achievement of about 85 percent of primary consolidation during construction and substantial limitations on postconstruction settlements to secondary values. Where settlements approach 100 percent of primary consolidation during construction, a surcharge load is needed. Where differential rather than total settlement is the controlling factor, surcharge loading may be avoided when the thickness of the compressible subsoil does not vary sharply and finished grade requirements are not critical.

The theory of consolidation is well known (11). Where slope stability can be developed, the feasibility of using sand drain stabilization depends on the magnitude and rate of residual primary and secondary consolidation falling within limits acceptable for the proposed construction. The magnitude of secondary consolidation is established by the relationship:

$$H_{sec} = Lc_{sec} \log(t/t_c) \qquad (1)$$

where

H_{sec} = secondary settlement at time interval t,
L = thickness of compressible stratum,
c_{sec} = coefficient of secondary consolidation,
t_c = time interval to reach 85 percent of primary consolidation,
t = time interval (must be greater than t_c).

The rate of secondary settlement (Δh_{sec}) will be essentially equal to the rate of primary settlement at the time of substantial completion of primary consolidation and is approximately expressed as

$$\Delta h_{sec} = 0.435L(c_{sec}/t_c) \qquad (2)$$

If the foregoing values as determined from Equations 1 and 2 meet design requirements, then sand drain stabilization can be considered for the project. Stability and settlement analyses are performed to determine the need for subsoil strength increase or berm stabilization to ensure safe completion of the proposed construction, and to estimate the total values of settlement incurred under the design loading. Consideration of soil strength increase with consolidation related to nondisturbance installation techniques can be handled by correlating strength with moisture content or by an evaluation of effective stresses as consolidation occurs (11). In general, for normally consolidated soils, analyses are based on average values of test data obtained from undisturbed samples. For precompressed soil the values used would distinguish between characteristics above and below the preconsolidation loading.

The basic equations involved in the design of sand drain installations, as concerns the determination of the rate at which primary consolidation can be expected to occur, are as follows for consolidation related to vertical drainage,

$$t_p = [T_v(L/f_v)^2]/c_v \qquad (3)$$

where

t_p = time interval during primary consolidation,
T_v = time factor for specific degree of consolidation,
f_v = vertical drainage factor [1 (single drainage) or 2 (double drainage)], and
c_v^c = coefficient of vertical consolidation.

For consolidation related only to radial drainage,

$$t_h = [T_h(f_h S)^2]/c_h = (T_h D_e^2)/c_h \qquad (4)$$

where

T_h = time factor for horizontal consolidation,
f_h = radial drainage grid factor [1.13 (square grid) or 1.07 triangular grid)],
S = spacing of sand drains in grid pattern,
c_h = coefficient of horizontal consolidation, and
D_e = diameter of sand drain influence.

Time factor curves for vertical and radial drainage are presented in Figure 1. Approximate values for the indicated ranges of consolidation can be obtained by using Equations 5 and 6 (approximately):

$$T_v \overset{\times}{=} 0.8\, u_v^2 \text{ and } 0 \leqslant u_v \leqslant 0.5 \qquad (5)$$

where u_v = vertical degree of consolidation.

$$T_h \overset{\times}{=} 0.8\, u_h^{2.5} \log_{10}(n/2) \text{ and } \begin{matrix} 0.5 \leqslant n \leqslant 20 \\ 0.7 \leqslant u_h \leqslant 0.9 \end{matrix} \qquad (6)$$

where

u_h = horizontal degree of consolidation and
n = ratio of D_e to d_w.

In the foregoing equations, as in later expressions, no distinction is made between free strain and equal strain settlement, as the difference is negligible as compared to many other uncertainties in design (2).

Where vertical consolidation is found to exceed 5 percent ($u_v \geq 0.05$) for the assumed construction period, the vertical consolidation is often considered in determining total consolidation. Where the total percentile magnitude of desired consolidation is established and the corresponding value of vertical consolidation is known, the settlement contribution required from the sand drain installation can be found by

$$U_h = 100\% - (100 - U_c)/(1 - U_v/100) \qquad (7)$$

where

U_h = percent of consolidation, horizontal;
U_c = percent of consolidation of construction; and
U_v = percent of consolidation, vertical.

An alternative expression relating the terms is

$$u_h = 1 - (1 - u_c)/(1 - u_v) \qquad (8)$$

Where u_c = degree of consolidation of construction.

The geometry of the san drain pattern to attain the desired consolidation can be developed in the following manner:

1. By experience, assume a value for D_e, with d_w as 45 cm (1.5 ft) and develop a trial value of n. (The value of d_w can be increased or decreased at will based on theoretical considerations; however, the initial design should be based on a specified value of d_w.)

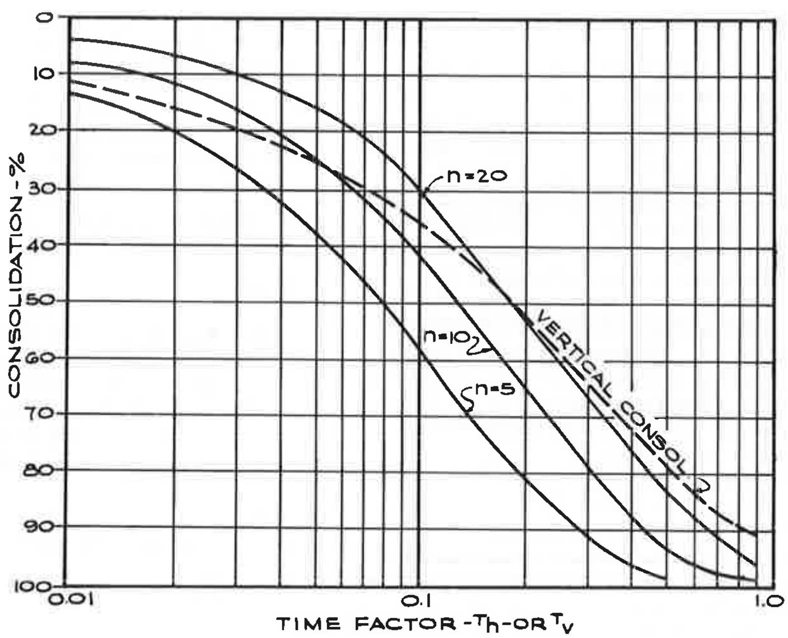


Figure 1. Time factor versus percent consolidation.

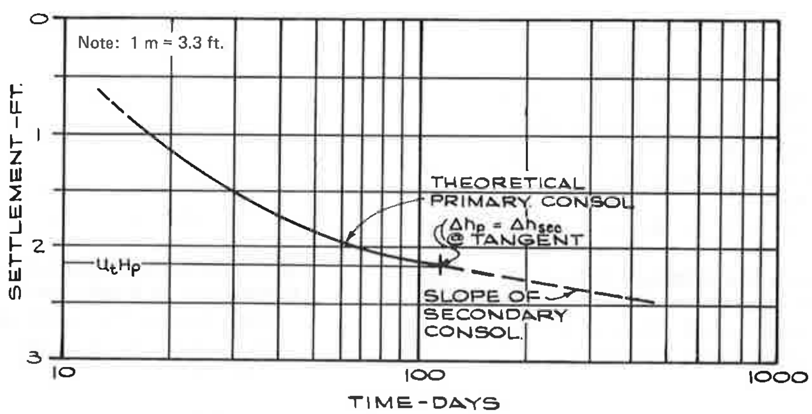


Figure 2. Time-settlement (typical).

2. With the values of t_h (= t_c), D_e, and c_h, enter Equation 4 to find the required value of T_h.
3. Use Figure 1 or Equation 6 with T_h and n to find the degree of consolidation involved ($U_h = 100u_h$).
4. If U_h is other than the desired value, alter the assumption for D_e and repeat the foregoing steps until U_h meets design requirements.

Secondary Consolidation

Although surcharge has been used to reduce the rate of secondary settlement, postconstruction settlements cannot be less or slower than values related to secondary consolidation. As such, the optimum sand drain design will be one that develops a rate of primary consolidation equal to that of secondary at the time of completion of construction. Inasmuch as the occurrence of secondary consolidation relates only to the thickness of the compressible stratum and is presumed to start toward the end of primary consolidation, it is evident from the expression for secondary consolidation shown in Equation 1 that the graph of consolidation versus time will be a straight line on a semilogarithmic scale.

By superimposition of the secondary consolidation curve on the primary consolidation curve so that the two are tangent, the optimum end point for primary consolidation (u_c) can be established (Figure 2). (This model was designed for U.S. customary units only; therefore values in Figure 2 are not given in SI units.) Inasmuch as U_v at t_c is fixed, u_h can be established by means of Equation 7. Returning to Equation 4 using any desired value of d_w, the value of D_e can be established by trial and error.

Once a workable set of values is established for D_e and n, Equation 9 can be used to approximate an equivalent set of values should it be desired to alter either D_e or n.

$$(D_e/E_w^{1/2})\,[\mathrm{Log}_{10}(n/2)]^{1/2} = M \qquad (9)$$

where

E_w = efficiency of sand drain,
t_f = field or final time,
t_d = design or theoretical time,
u_f = field or final degree of consolidation at t_f,
u_d = design or theoretical degree of consolidation, at t_d,
M = sand drain grid equivalence factor, and

$$t_f = t_d \begin{cases} E_w = T_{hf}/T_{hd}, \text{ or} \\ E_w \approx (u_f/u_d)^{2.5} \\ \text{when } 0.7 \le u_h \le 0.9. \end{cases}$$

The value M is a constant for any set of field conditions. The foregoing relationship also permits applying the sand drain efficiency (E_w) to each sand drain installation method contemplated. A theoretical set of efficiency curves for the mandrel method is shown in Figure 3. Where d_w is a constant, use $n = n_e/E_w^{1/2}$ in Equation 9.

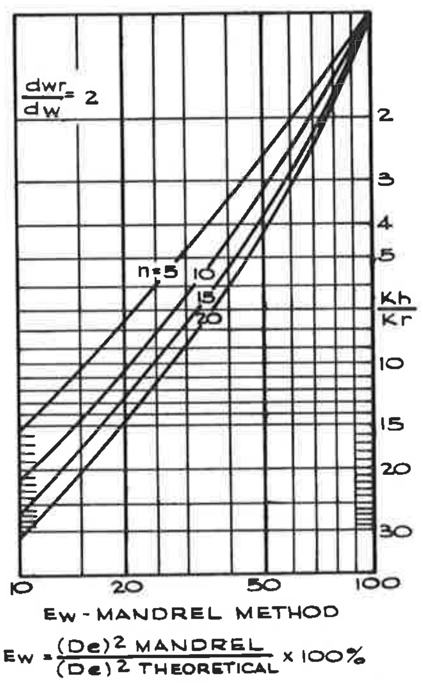
Sand Drain and Sand Blanket Material

Equivalent sand drain designs can be developed for a given set of soil conditions and time parameters by varying the sand drain diameter (d_w) and its diameter of influence (D_e) in accordance with Equation 3. It can be shown mathematically that equivalent designs based on a large percentile change in d_w will reflect only as a small change in D_e^2 and the capacity of the sand drain to carry water varies with d_w^2, then for equivalent designs a smaller-sized drain diameter will necessitate the use of higher permeability sand backfill material to avoid introducing excessive head losses within the sand drain itself. The effect of such head losses reflects as backpressure, which affects sand drain performance as described by Barron (2).

Where hydrostatic backpressure is permitted, the backfill permeability may be approximated by Equation 10:

$$k_w = \Delta hn^2 (L+B)/2(P_t - P_0 - P_b) \tag{10}$$

Figure 3. Grid efficiency versus permeability ratio for mandrel method.



where

k_w = permeability of sand drain backfill;
Δh = settlement rate;
B = sand blanket thickness;
P_t = total load expressed as hydrostatic head;
P_o = initial ground level above natural water table, expressed as hydrostatic head; and
P_b = sand blanket or lateral drainage channel load or hydrostatic backpressure.

Thus, the larger the sand drain diameter for equal settlement rates (Equation 9), the lower the permissible permeability of the backfill material, which would reflect as a cost differential for the installation.

Similarly (as shown in Figure 4) an approximation can be derived for the permeability of the sand blanket material, as follows:

$$k_b = \Delta hW^2/[2B(B + P_b)] \tag{11}$$

If, instead of using a uniform sand blanket, French drains (height, B) are used to interconnect the sand drains, an expression similar to that above can be developed:

$$k_b = \Delta hW^2S/[2A_b(B + P_b)] \tag{12}$$

where

k_b = sand blanket permeability,
W = half width of embankment (stabilized), and
A_b = sand blanket cross-section area.

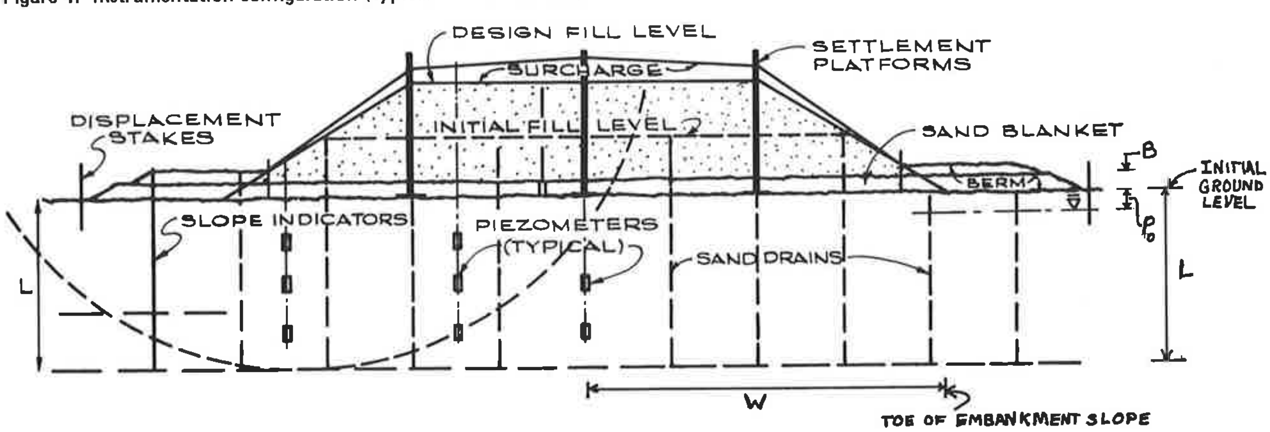
As in the case of the sand drain backfill, the sand blanket permeability presumes an acceptable hydrostatic backpressure of P_b. The consideration of sand blanket permeability is important inasmuch as coarse sand material is often scarce and, therefore, can be costly to obtain. Equations 11 and 12 permit an economic evaluation of the best means to provide a drainage blanket for the sand drain installation. A substantial factor of safety (such as 10 or more) should be applied to K_b where feasible.

Surcharge

The use of surcharge in conjunction with the design of sand drain installations may be desirable if:

1. Postconstruction total settlements without surcharge exceed maximum tolerable limits for the type of construction involved;
2. The cost of surcharge material (including placement and removal) is less than the cost of using a closer

Figure 4. Instrumentation configuration (typical for test sections).



spaced sand drain grid to accomplish substantially the same end result; and

3. The addition of surcharge does not result in embankment instability.

Determination of the surcharge required to achieve a desired degree of primary consolidation (u_t) based on attaining an effective value (u_e) by sand drain stabilization can be accomplished by Equation 13:

$$\log_{10}(P_f + P_{sur})/P_p = [a_v(P_p - P_0)(u_t - u_e) + u_t C_c \log_{10}(P_f/P_p)]\, u_e C_c \quad (13)$$

where

P_f = field or design load,
P_{sur} = required surcharge load,
P_p = preconsolidation load of subsoil,
a_v = coefficient of compressibility,
u_t = degree of consolidation required for load P_f,
u_e = degree of consolidation to be achieved with surcharge added, and
C_c = compression index.

Effects of Installation Methods

It has often been demonstrated that subsoil disturbance will have an adverse effect on the performance of sand drain installations by virtue of changes effected in subsoil characteristics of sensitive soils (12, 13). The principal changes in performance of installations involving disturbance to sensitive soils are expected to relate to the disturbance ratio (R_z) as defined in Equation 14 (14).

$$R_z = (c_{vu} - c_{vz})/(c_{vu} - c_{vr}) = (q_u - q_z)/(q_u - q_r) \quad (14)$$

where

R_z = disturbance ratio,
c_{vu} = undisturbed vertical coefficient of consolidation,
c_{vz} = disturbed vertical coefficient of consolidation,
c_{vr} = remolded vertical coefficient of consolidation,
q_u = undisturbed compression strength,
q_z = disturbed compression strength, and
q_r = remolded compression strength

and exhibits the following effects:

1. The rate of occurrence of primary consolidation is decreased.
2. The rate of occurrence of secondary consolidation is increased.
3. The magnitude of primary settlement is increased.
4. In situ shear strength characteristics are decreased (at least during the early stages of stabilization).

Other adverse effects include an increase in pore pressure not related to construction loading; destruction of the continuity of varves and partings; thus impeding horizontal drainage; and lateral displacements, which may result in the shearing of previously installed drains. These effects must be compensated for by the selection of conservative soil characteristics, as well as by high factors of safety, in the development of designs involving displacement methods applied in sensitive soils. Conversely, the use of disturbed soils characteristics might be costly in designing installations involving nondisplacement techniques as the field results could exceed the expectations of the engineer. It is necessary, therefore, that more accurate determination of subsoil design characteristics be made where nondisplacement installation methods are to be employed in order to take full advantage of the potential efficiency of such methods.

To permit a more accurate means of designing for nondisplacement sand drain installations, in situ soil characteristics must be developed. It is equally important to determine the efficiency of each method based on its performance in the field (Equation 9) so that in situ characteristics may be used in design. On major projects, prototype performance may be determined in advance of design by means of test sections. In addition to being an aid to design, such field tests would also permit development of efficiency data for methods of sand drain installation used. Where test sections cannot be implemented, selected areas of the construction can be staged and closely monitored as a means for verifying the design and obtaining data for evaluation of sand drain performance.

EVALUATION OF SAND DRAIN INSTALLATIONS

In order to evaluate each of the various methods of installation (particularly if a comparative test section is not utilized), it is important to develop a reproducible body of soil characteristics based on the use of laboratory test data. Samples are often at least partially disturbed as a result of normal soil sampling and handling methods as well as in laboratory test preparation and work; therefore, it would seem appropriate to use only maximum test results for design purposes. Conversely, if a reproducible basis for comparison is to be developed, perhaps all results should be compared to the most conservative design, involving the use of average values of remolded soil data and parameters.

In order to advance the present state of knowledge, it would be desirable to compare performance in the field with designs established on paper for maximum values from test results (after discarding inconsistent values), as well as paper designs based on remolded test values, with settlement estimates based on initial void ratios (e_o) derived from in situ moisture contents. So that there be no misunderstanding concerning the paper designs, these need not be the designs on which the installations would actually be constructed. The engineer would continue to prepare construction plans and specifications in accordance with current knowledge and experience. However, inasmuch as the selection of design values from test data is almost entirely subjective, the success of field installations would not result in any improvement in the working knowledge concerning various methods of installation. Inasmuch as average values of remolded test data are more likely to be reproducible for any soil type, and the maximum range of test values might also be reproducible, the subjective aspects involved in the paper designs would be largely eliminated. By using these limiting values and minimizing the use of subjectively derived data, a degree of uniformity in the classification of field performance will ultimately be developed.

Comparisons of field results to remolded design values are best reported as specific improvement factors, but field results and field-derived values are best compared to maximum laboratory values as efficiency factors. On this basis, the following factors are suggested for describing field performance relative to design performance (I = improvement factor, E = efficiency factor, v = vertical, h = horizontal, sec = secondary, p = primary, t = total, s = strength, and w = sand drain).

Relevant Item	Improvement	Efficiency
Vertical consolidation	I_v	E_v
Horizontal consolidation	I_h	E_h

Relevant Item	Improvement	Efficiency
Secondary consolidation	I_{sec}	E_{sec}
Settlement factor, primary	I_p	E_p
Settlement factor, total (20 years by extrapolated data)	I_t	E_t
Shear strength or cohesion	I_s	E_s
Sand drain grid	I_w	E_w

The consolidation factors refer to ratios of field values to laboratory values using coefficients of primary and secondary consolidation. The settlement factor (primary) is a ratio of design estimates to field settlements under each loading condition (including surcharge loading where used). Strength and sand drain grid factors are based on tests of undisturbed samples from borings obtained after stabilization to values obtained from design boring data as well as from field data.

The most difficult element to establish in the laboratory is the value of the horizontal coefficient of consolidation (c_h) for use in determining the horizontal consolidation factors. In isotropic silty or clayey soils, it is recommended that c_h be taken arbitrarily as ten times the laboratory c_v (undisturbed) value for the efficiency factor, and as being equal to c_v for the improvement factor. In silty and clayey varved soils, the c_h value should be taken as ten times the c_v (undisturbed) value of the more permeable of the varves involved. Where substantially continuous sand partings and varves are involved, then c_h for use in determining the consolidation efficiency factor should be taken as equal to c_v derived from the permeability of the sand.

SAND DRAIN INSTALLATIONS

To ensure uniformity in construction of sand drain installations, development of a set of specifications applicable to nondisplacement methods is required. The following key points are suggested to be covered for all nondisplacement techniques. These recommendations are based on experience involving sand drain installations at the Maine and East St. Louis test section sites.

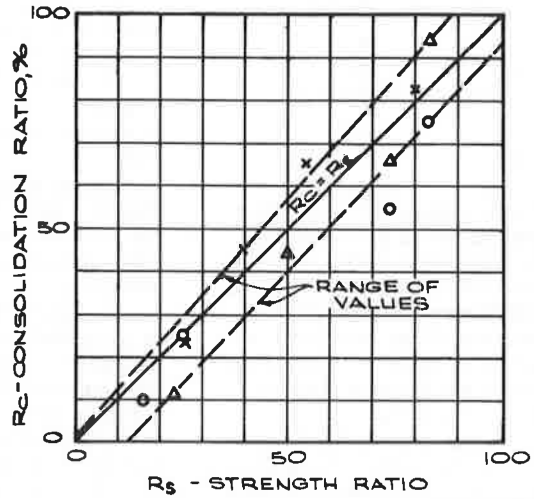
1. No alternate raising and lowering or free fall of the cavity-forming tool is permitted. A maximum 30-cm (12-in) free fall of the jetting tool is permitted if it is shown that the jetting tool forms a cavity at least 30 cm ahead of the tool.
2. The vertical alignment of the cavity-forming tool shall be maintained to a plumbness and axial linearity within a maximum deviation of 1 percent at all times during the sand drain cavity formation and held to within 7.5 cm (0.25 ft) of plan location.
3. The maximum rate of tool advance shall be limited to one pitch length per revolution for augers but is to be maintained at a lower rate to ensure excavation of the subsoil. Thus consideration is given to the physical volume of the auger. Reverse auger rotation is not permitted.
4. The maximum rate of advances of the jetting tool shall be 3 m/min (10 ft/min), and the actual rate increased or decreased as required in the field to ensure nondisplacement cavity excavation (zero pore-pressure increase).
5. Fluid pressures used in excavating or backfilling the sand drain cavity shall not exceed twice the existing hydrostatic pressure in the subsoil at the level of the bottom of the cutting tool as the apparatus is progressed through the compressible soil; however, fluid pressure of 275.8 kPa (40 lb/in^2) is permissible at all depths in the compressible soil during backfill. Higher pressures may be allowed where it is ascertained that jetting-induced excess hydrostatic pressure dissipates within 24 h.
6. Effluent from jetting installations must be disposed of in a manner that will not affect environmental conditions adversely.
7. At the discretion of the engineer, where jetting is performed in the vicinity of waterways, a casing may be desirable for use with the jetting apparatus, and the contractor shall do all that is necessary to ensure return of all jetting water and effluent to the top of the casing. Such return water shall be accumulated or disposed of off the site as required by the engineer to ensure against the inadvertent pollution of the adjacent waterway.
8. In all methods used, rigid cavity support shall be provided at all times for the portion of the sand drain that passes through the sand blanket and any soft or granular subsoil stratum encountered to prevent yielding or collapse of the formed cavity. The sand drain shall be backfilled simultaneously with the removal of cavity support in a manner to ensure columnar continuity by applying 206.8 kPa (30 lb/in^2) (min) air pressure to the sand during backfill. Such air pressure should not exceed twice the in situ hydrostatic head at the depth of backfill placement.
9. Where sand drain cavities are not rigidly supported, measure each for size and depth. For rigidly supported cavities, control the rate of support removal to reflect the rate of backfill. Check the volume of backfill used, as needed for proper control.
10. Develop backfill permeability requirements for the range of sand drain sizes permitted.

CONSTRUCTION SEQUENCE FOR SAND DRAIN SECTIONS

The following procedure is proposed for construction of sand drain installations. Although this is most appropriately applicable to test sections where extensive monitoring can be implemented without impeding project completion, the procedure can be tailored to meet the varying control requirements for specific construction projects that utilize sand drain stabilization.

1. Install construction control devices as early as possible in the construction (preferably in advance of any fill placement). Such devices include settlement platforms and piezometers; deep settlement points, displacement stakes, and slope indicators are used as appropriate. Settlement platforms should be located at points of change in the loading shape as well as at the center of the construction. Piezometers should be placed to permit development of longitudinal as well as vertical pore-pressure profiles (Figure 4). Add displacement stakes and slope indicators where lateral movements (creep and potential slope stability problems) are anticipated.
2. In test sections and where economically feasible in construction, place sand blanket and fill material to highest possible level (or to the top of berm level, where contemplated) but no higher than 50 percent of the design load for full width of the embankment. Displacement stakes are best located outside the toes of slope, but the slope indicators are best located so as to pass through the anticipated critical failure plane (Figure 4). Install piezometers within 60 cm (2 ft) of the first few sand drains as a means of establishing a maximum rate of cavity-forming-tool advance consistent with complete dissipation of induced pore pressure within 24 h.
3. In test sections, and where feasible in actual construction, allow the fill to remain in place as long as possible without delay to the construction schedules, to permit development of settlement data for use in establishing c_v.

Figure 5. Strength ratio versus consolidation ratio.



(After Tazawa Wakame)

○ 90%
△ 80%
× 70%
} SILT + CLAY CONTENT OF SOIL

R% EQUIVALENTS

$$R_c = \frac{C_{vu} - C_{vo}}{C_{vu} - C_{vr}} \times 100\% \quad (C_v \text{ @ } t_{90})$$

$$R_s = \frac{q_u - q_o}{q_u - q_r} \times 100\% \quad (q \text{ Values @ Strain for } q_u)$$

4. Install sand drains as required. For test sections, install drains in more than one grid pattern if feasible.

5. Additional piezometers are to be installed after sand drain work is completed. For test sections, piezometers are also to be placed within typical drains as well as within the sand blanket.

6. Bring fill up to final design level plus surcharge at a uniform rate of filling and allow to remain in place for as long as possible (for test sections until at least 85 percent consolidation is achieved).

7. After review of field data, establish the amount of surcharge to be removed so that the degree of consolidation (u_t in Equation 13) is substantially 1.0.

8. Where feasible, maintain at least a portion of the instrumentation in place for a sufficient time to develop the secondary consolidation characteristic of the subsoil.

9. Take borings and undisturbed samples for testing to redefine soil characteristics developed for the design as well as to evaluate disturbance (Figure 5 and Equation 14).

10. Test sections should include a control area without stabilization, using berms as needed for stability. Sand drains should be installed by displacement as well as by nondisplacement methods at equivalent spacings in accordance with Equation 9.

EVALUATION OF FIELD DATA

The rate of occurrence and value of total primary settlement may be determined in the field by using settlement platform and instrumentation data. Incremental settlements at intermediate levels within strata may be determined by means of earth anchor devices installed at specific depths through bore holes. Piezometer data permit an estimation of the degree of consolidation at any time during construction. By employing the suggested construction sequence, the evaluation of the sand drain installations can be accomplished in the following manner:

1. If fill is left in place prior to installing sand drains, sufficient data are developed to permit interim evaluation of c_v. A final evaluation of c_v is made after the value for any control area is established by using total primary settlement from field data after sand drain installation and substantial completion (85 percent) of primary settlement.

2. The value of t_o applicable to installation of sand drains for the initial fill level is closely approximated by the actual date that sand drains are installed.

3. Theories developed to evaluate consolidation of sand drain installations under uniform rates of fill can be checked by using data obtained for loads placed after sand drain installation (15) and consolidation factors.

4. The coefficient of consolidation can be checked for the final loading condition by approximating a new t_o taken at the midpoint of the loading cycle, which will permit evaluation of consolidation factors and sand drain efficiency.

5. By removing the surcharge portion of the fill, a check can be made of the coefficient of secondary consolidation and settlement factors since in situ fill produces a loading that is consistent with achievement in the field of 100 percent of primary settlement.

6. Piezometer data in the drains and in the sand blanket will indicate differences in backpressure, which will provide needed information on the effects of sand drain backfill and sand blanket permeability.

7. The evaluation of effective stresses as well as testing of undisturbed samples from final borings will permit a determination of the strength or cohesion factors. Moisture-content profile changes determined from final borings can be used to check observed settlement data and to compute settlement factors.

COMPARISON OF INSTALLATION METHODS

The results of the test sections as well as other instrumented construction projects can be evaluated on a quantitative basis, which can ultimately be used to reflect cost differences relative to each of the methods of sand drain installation available. Improvement and efficiency factors determined for each installation over the predicted paper design performance will permit tying field performance to reproducible laboratory test values. In this manner, it will be possible to determine variations in quantities relative to size and spacing of sand drains and fill requirements due to differences in primary and secondary settlements, surcharge, and berms. It will also be possible to evaluate the need for specific sand blanket and sand drain backfill materials.

When a test section is involved, it will become possible to compare directly the various methods of installation utilized. Such comparisons may be made in a manner similar to that for the Maine test section (6, 7, 8), based on ratios of performance to design values. The term settlement ratio (R_y) is the ratio of field settlement (H_f) to theoretical settlement (H_d). Effectiveness ratio (R_e) is the relationship between the backfigured value of c_h for each method to that of the base method selected. These results for the Maine test section are presented in Tables 1 and 2. The mandrel method was the base method for Table 1 (9).

The following can be used to establish true costs of sand drain installations:

1. If the use of 45-cm (1.5-ft) diameter sand drains in a 3-m (10-ft) triangular pattern of mandrel sand drains for the back cove site is assumed, an equivalent set of data for 30-cm (1-ft) diameter drains as well as corresponding values of influence area [fS^2 ($=n^2A_w$)] may be established for the jetting and auger methods, as shown in Table 3.

2. Using the settlement ratio (such as in Table 3) and corresponding values of influence area, a fill quantity increase can be established for each method and drain size, which will reflect as a unit cost increment (F_c) per foot of sand drain installed:

$$F_c = F_f\, Hn^2 A_w (Ry - 1)/L_w \qquad (15)$$

where

F_c = unit cost increment,
F_f = field or final unit cost,
H = settlement,
n = ratio of D_e to d_w,
A_w = sand drain cross-section area,
R_y = settlement ratio, and
L_w = average length of sand drain.

3. The volume of fill used in berms (and surcharge) (B_v) for equivalent stability and consolidation, as well as any credit (where applicable) for reduction in fill (f_vA_w) due to ground heave (displacement methods) or the reuse of spoil developed in excavating sand drain cavities by augering (Figure 6) are factors in establishing a supplemental cost increment (or credit) (ΔF_c) per foot of sand drain installed:

$$\Delta F_c = F_f(B_v/\Sigma L_w) - f_v A_w \qquad (16)$$

where

ΔF_c = supplement cost increment,
B_v = total volume of berm and surchange, and
Σ = number of sand drains for total project.

4. To compare the costs of various methods and alternative designs, the unit price (F_w) actually bid for each method of installation is increased by F_c and ΔF_c and the total divided by the applicable effectiveness ratio (R_e) to obtain the effective unit cost (F_e) per foot of sand drain installed.

$$F_e = (F_w + F_c + \Delta F_c)/R_e \qquad (17)$$

where

F_e = effective unit cost,
F_w = sand drain unit cost, and
R_e = effectiveness ratio.

A typical computation format for determining the pattern of sand drains applicable to each method considered is presented in Table 3. Also included in Table 3 is the incremental fill quantity reflecting induced soil consolidation characteristics altered by disturbance effects of each method.

In developing true comparisons, certain misconceptions must be avoided. Because of the tendency to employ specifications developed by others, certain methods have become standardized as to size. As an illustration, the jetting method has been specified as permitting the use of 30-cm (1-ft) and 45-cm (1.5-ft) sand drains; for the auger method only 45-cm diameter drains are usually specified. The larger diameter drains are often required in an effort to make certain of sand drain continuity where a degree of lateral yield or creep may need to be accommodated. Where lateral creep is not anticipated, and where the stratification overlying the soft soil requiring treatment is 10 m (30 ft) or more of stiff (desiccated or preloaded) clay, the sand drain diameter should be fixed for bidding purposes, with the construction contractor permitted the option to vary the diameter used within a specified range and theoretical spacing based on the efficiency of the installation method. Whereas jetted drains can be penetrated to great depths with relatively small changes in equipment size, the size of the auger and mandrel equipment generally increase in power and weight substantially in direct proportion to the drain length. It is noted that it takes approximately one-half the weight and power to install a 30-cm drain by the auger method as compared to a 45-cm diameter drain. Whereas there may be approximately 20 percent more drains required for a 30-cm diameter, the saving in equipment and sand backfill may more than compensate for the extra number of drains to be installed. The same would undoubtedly be true for the mandrel method.

The designer is also cautioned that there may be circumstances when specific sand drain installation methods are not appropriate. Jetted drains should be avoided where adjacent structures are in place and migration of water used in jetting may affect its foundation adversely, as in the case of piers and bulkheads. In urban areas where noise codes are in effect, the use of driven sand drains may be prohibitive in cost if special equipment is required to limit noise. Care must also be taken to ensure cavity support during backfill of each sand drain cavity. Although limits are suggested in drain sizes for specific methods of installation, such limits only reflect common usage; there is no reason to assume that equipment does not exist or cannot be made to install sand drains of greater or smaller size in each instance. However, it is desirable to establish a realistic range limit on drain sizes, e.g., 10-cm (4-in) minimum for sand drains installed in supported cavities, 30-cm (1-ft) minimum for those installed in unsupported cavities, and a maximum 60-cm (2-ft) diameter allowed. Sand drain spacing should be limited to 7.6 m (25 ft), while jetted and driven drains should not be spaced closer than 2.4 m (8 ft) to avoid any adverse effects on previously completed drains.

CONCLUSIONS AND RECOMMENDATIONS

1. Available theory of sand drain design is adequate to approximate field performance provided that information is available to permit a determination of the effects of the available methods of sand drain installation on subsoil characteristics.

2. In view of the subjective aspects involved in the selection of laboratory test data for use in design, a reproducible standard should be developed to serve as a basis for evaluation of field performance of sand drain installations. Such values selected without subjectivity would be used to produce paper designs without any factor of safety being applied or the need to be conservative

Table 1. Effectiveness ratio.

		Effectiveness Ratio Values[a]		
Soil Type	Drain Spacing (m)	Mandrel Method	Jetting Method	Auger Method
Silty clay	3	1.0	1.6	2.0
Silty clay	4.3	1.0	2.1	2.5
Organic clay	4.3	1.0	1.2	1.2

Notes: 1 m = 3.3 ft.
The greater the effectiveness, the greater the efficiency of the method of installation. The increase in efficiency with with sand drain spacing is predictable, Figure 3.

[a] Use as minimum value $R_{e2} : R_{e1} \sim I_{w2} : I_{w1} = E_{w2} : E_{w1}$.

Table 2. Settlement ratio.

		Settlement Ratio Values[a]		
Soil Type	Drain Spacing (m)	Mandrel Method	Jetting Method	Auger Method
Silty clay	3	1.56	1.54	0.91
Silty clay	4.3	1.23	2.03	1.14
Organic clay	4.3	1.33	1.18	1.15

Notes: 1 m = 3.3 ft.
The lower the settlement ratio, the less the disturbance developed by the method of installation. For a given diameter of sand drain and method, R_y should decrease with an increase in drain spacing.

[a] Use as minimum value $R_y = 1/E_t > 1.0$, $R_{y2} : R_{y1} \sim E_{t1} : E_{t2}$.

Table 3. Settlement volume increment.

	d_w = 45 cm			d_w = 30 cm			
Sand Drain Installation	Auger Method	Jetted Casing Method	Mandrel Method	Auger Method	Jetted Casing Method	Mandrel Method	Comments
Effectiveness ratio, R_e (S = 3.0 m)	2.0	1.6	1.0	1.63	1.31	0.82	$R_{e2} = R_{e1}\,(d_{w2}/d_{w1})^{0.5}$
Drain spacing when equal E_w, S, m	4.3	3.8	3.0	3.88	3.44	2.72	$S_2 = S_1(R_{e2}/R_{e1})^{0.5}$
Corrected effectiveness ratio, R_e	2.5	1.84	1.0	2.04	1.50	0.82	Interpolated-Table 1
Corrected sand drain spacing, S	4.74	4.07	3.0	4.29	3.68	2.72	Using corrected R_e
Influence area, $f_w S^2$, m^2	20.22	14.91	8.1	16.56	12.19	6.66	f_w = 0.9, triangular grid
Settlement ratio, R_y	0.91[a]	1.54	1.56	0.91[a]	1.54	1.56	From Table 2, Use 1.0[a]
Settlement factor, f_y, m^3/m	0	8.05	4.54	0	6.58	3.73	Added fill/settlement/ sand drain installed

Notes: 1 m = 3.3 ft.
R_e values use the mandrel method performance as the base.

[a] R_y is defined as field settlement/theoretical settlement. Use R_y = 1 as minimum. The volume of fill required to compensate for settlement per sand drain installed is $H(f_y + 1)$. The settlement volume increment related to the method of sand drain installation, expressed per sand drain installed is Hf_y.

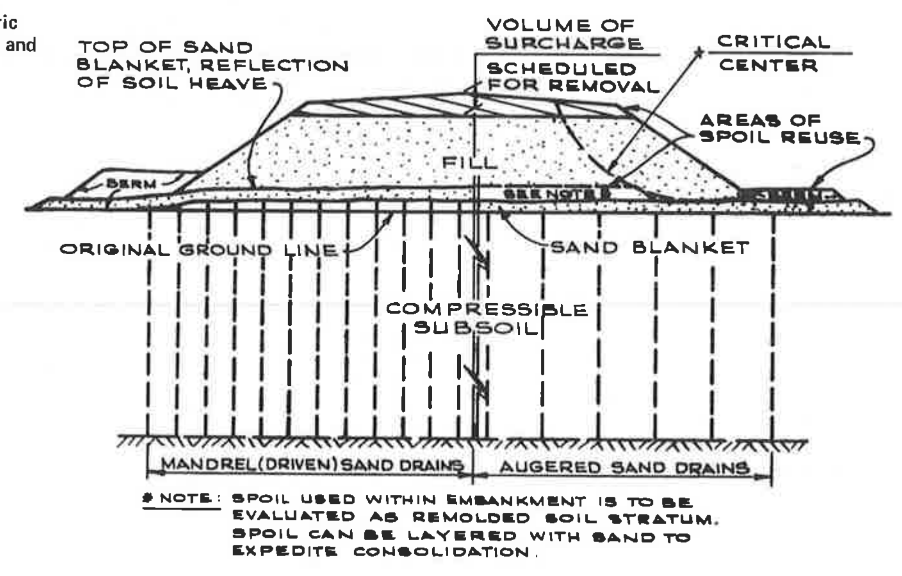


Figure 6. Volumetric effects of soil heave and spoil reuse.

as concerns field performance.

3. Fully instrumented sand drain test sections should be planned under state and federal research funds to permit accumulation of comparable field data, at least for the available nondisplacement methods of installation, so as to ultimately provide a basis for developing equal designs. In so doing, the state of the art will be advanced and the degree of conservatism now exercised in current sand drain design practice can be reduced, for substantial savings in construction cost.

4. A uniform set of key specification requirements should be established for auger and jetting methods of sand drain installations, as an important first step toward ensuring reproducibility of field results. With uniformity in construction, including environmental safeguards, it will be possible to develop means of establishing meaningful efficiency and improvement characteristics for available methods of sand drain installation. Of importance in such specifications are controls on: cavity verticality, rates of tool advance and withdrawal, fluid pressure in forming and backfilling cavities, and pore pressure increase during sand drain installation.

5. Evaluation of the performance of sand drain installations is not limited to the determination of the rate of occurrence of settlement and environmental effects of each method but must also include an economic evaluation.

6. The design of sand drain installations may involve drain diameters ranging from 7.5 to 65 cm (0.25 to 2 ft) depending on the methods of installation considered. Where more than one method is specified as acceptable, alternative designs (size and spacing of drains) for each method must reflect performance efficiency by taking into account the permeability of the available backfill material. Where lateral yielding or creep is a problem, the larger diameter drains are preferable.

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Discussion

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The primary objective of this discussion will be to request additional information and clarification of points that I find unclear. Furthermore, the validity and importance of many worthwhile ideas are often obscured by what unfortunately appears to be an attempt by Landau to promote a particular drain installation system in which he has a strong commercial interest. Therefore, a second objective will be to present additional sources of information and alternative viewpoints on some of the more controversial aspects of the paper. For easy reference, the discussion will follow the main headings of the paper.

BACKGROUND

In the excellent, comprehensive paper on sand drains, Johnson (16, pp. 156-157) discusses the controversy regarding the relative merits of various drain installation techniques. He points out that

> Any installation method must cause some disturbing effects, just as disturbance results when obtaining even the best undisturbed soil samples. It is obvious, therefore, that primary interest must be focused upon assessing the severity of these effects on results obtained, rather than upon the question of whether disturbance does or does not exist.

The first drains presented are all installed with a mandrel. The drains in the second in-text table may be installed with less displacement than conventional large-diameter, closed-end mandrel sand drains, but there will still be some soil displacement, remolding, smear, or distortion of thin sand layers (16, Table 2). Perhaps these methods should be termed minimum displacement drains. Even though the Kjellman paper drains are mandrel driven, their size and spacing may actually cause less disturbance in certain soil conditions than the other methods.

The first two in-text tables should also include some recent European and Japanese developments. The Geodrain and Alidrain, both invented by O. Wager of the Swedish Geotechnical Institute, are improvements of the Kjellman paper drains. They are also band-shaped, about 100 mm wide by 4 mm thick, and they have a plastic core surrounded by a paper or nonwoven fabric filter (17). They can be pushed, vibrated, or jetted into the soil by a mandrel slightly larger than the drain itself (18, 19). Japanese engineers have developed a similar band-shaped drain. A 100-mm wide corrugated plastic core is covered by a fabric filter (20). Another recent European development is the AV-Colbond drain. It is composed entirely of a nonwoven polyester fabric 300 mm wide by 4 mm thick (21, 22). A hollow lance with the strip of fabric attached is jetted into the ground to the desired depth. The fabric acts as both a filter and a conduit for the water. From several field tests, the drain seems to be as effective as ordinary sand drains (19, 21, 22). Moran (1) and Richart (23, p. 723) have shown that even small-diameter drains can be effective in dissipating excess pore-water pressure. Finally, Johnson (16, pp. 160-161) gives an excellent summary of the major considerations involved in the selection of sand drain installation procedures.

SAND DRAIN DESIGN

The author suggests that the feasibility of using sand drains depends on the magnitude and rates of residual primary and secondary consolidation. It would be helpful if he would tell us how to obtain these rates, especially c_{sec}. Does he recommend laboratory test or field observations? Laboratory tests sometimes overestimate rates of secondary compression (24, p. 458). A distinctly better design approach would be to follow Moran (1) or Johnson (16). I am not sure about the practicality of the suggestion that strength increase with consolidation can be correlated with water contents. The wide range (scatter) of natural water contents observed in most deposits of soft clay would make such correlations difficult to conduct in practice.

Some minor points: The c_{sec} in Equations 1 and 2 is not the same as C_α defined in the literature (1, 13, 16); rather it is $c_\alpha/1 + e$, where e is some reference void ratio, usually the end of primary e_p. There seems to be a typographical error in Equation 2. Figure 1 is essentially as presented by Moran and Barron (1, 2). Equations 7 and 8 may be more familiar as

$$U = U_h + U_v - U_h U_v = 1 - (1 - U_h)(1 - U_v) \qquad \text{(7a and 8a)}$$

as developed by Carrillo (25) and others. Under Item 3, U_h does not equal 100 u_h (26, p. 233).

Of course the real design problem is to determine the soil properties necessary to compute U_h. The four-step design procedure proposed by Landau is apparently only an outline of the procedure given by NAVFAC Manual DM-7, Chapter 6 (11). The designer is also advised to consider carefully some of the recent developments in deep drainage mentioned previously.

Secondary Consolidation

Several important points in this section are not immediately obvious. Why is the optimum sand drain design the one in which the rate of primary consolidation is equal to the rate of secondary at the end of construction? Landau states that "secondary compression relates only to the thickness of the compressible stratum." This is an oversimplification; many other factors also affect secondary compression, such as time, consolidation pressure, precompression, duration of the previous load increment, remolding, and rate of increase of effective stress (27). How is the superposition of the two curves, primary and secondary, so that their slopes are the same (Figure 2) different from the common Casagrande construction for determining t_{100} (26, p. 241)?

For the design of sand drains wherein some consideration of secondary compression rates is to be made, it is probably easier to use the procedures suggested in the literature (1, 11, 13), especially if a surcharge is to be utilized in conjunction with deep drainage.

The design equation (Equation 9) proposed by Landau is not easy to understand. Perhaps Landau could give the derivation or at least a reference to the derivation of this equation. Figure 3 is also not very clear. How were the theoretical efficiency curves determined? Should the definition of E_w given in Figure 3 be inverted to be consistent with the definitions given in Equation 9? Even if, as stated, the value of M is a constant for a given set of field conditions, it would be helpful to know how to obtain these factors for typical design situations. How is $n = n_e/E_w^{1/2}$ obtained when d_w is a constant? Is not d_w always a constant for a particular installation? A numerical example showing the reader how to use Equation 9 and Figure 3 would help considerably in following Landau's suggested design procedure.

Sand Drain and Sand Blanket Material

Landau's contention that backpressure in the drain itself is important is apparently not shared by others. For example, Moran (1, p. 35) and Richart (23, p. 721) state that for practical values of n and reasonable geometries, the resistance of the drain wells should be insignificant. The dissipation of excess hydrostatic pressure in the clay is not really a function of the coefficient of permeability of the drain (k_w), but rather it is a function of the ratio of the permeabilities of the two materials. This is the classic case of impeded drainage from the theory of consolidation. Bishop and Gibson (28) show that as long as the ratio of the permeabilities is at least 100, the drain could be considered for practical purposes to be infinitely permeable. Perhaps Landau can present some data to show why he thinks the resistance of the drains is significant.

After Equation 10, Landau states: "Thus, the larger the sand drain diameter for equal settlement rates (Equation 9), the lower the permissible permeability of the backfill material, which would reflect as a cost differential for the installation." It should be recognized that cost differential (presumably a lower cost in this case) may not be immediately realized, since the larger the drain diameter, the larger volume of sand required, which might offset any savings from using poorer quality backfill materials. Where on Figure 4 is it shown how the approximation of Equation 11 is developed? Equations 10 through 12 seem to be related to Darcy's law—it would be helpful if Landau would give the source, or better, the derivation of these equations.

Landau states that Equations 11 and 12, "permit an economic evaluation of the best means to provide a drainage blanket. . . ." Without being facetious, it might occur to the design engineer that the best means to provide a drainage blanket probably would be with conventional construction and hauling equipment, and no way to evaluate these costs is indicated in Equations 11 and 12. Finally, would Landau please explain how to apply a factor of safety to a coefficient of permeability?

Surcharge

One of the important possible objectives for using a surcharge is to reduce secondary compressions, as was suggested by Landau. Thus, it is indeed strange that procedures for surcharge design to consider secondary compression are not mentioned in this section.

In regard to Equation 13: The coefficient of compressibility a_v ($= -de/d\sigma'$) cannot be equal to zero for a normally consolidated clay. Since the derivation and source of Equation 13 is not immediately apparent, the procedure given by Johnson (13, pp. 122-133) or Moran (1, pp. 79-85) should be followed for the details of precompression design.

Effects of Installation Methods

Not all of the effects indicated by Landau as resulting from disturbance due to installation of displacement sand drains occur in all soils. For example, Johnson (13, Figure 7, and 16, p. 157) indicates that the rate of secondary compression decreases rather than increases, as stated by Landau (effect 2). In some cases, the magnitude of primary settlement may increase with increasing disturbance, but the amount of the effect will depend on the relative stress increase due to the fill or surcharge load. In a well-documented field test series in Sweden, Holtz and Broms (24, p. 462) found no significant increase in primary settlement due to driving of closed-end mandrel drains.

I would suggest that accurate determination of in situ soil properties and subsurface drainage characteristics is required for all deep drainage methods and not just for nondisplacement techniques.

Evaluation of Sand Drain Installations

I strongly disagree with the suggested design evaluation procedure. The comparison of designs utilizing soil properties backcalculated from field observations with designs based on soil properties determined on (a) completely remolded soil samples and (b) maximum values of soil properties determined on (presumably) relatively good undisturbed samples seems highly dubious.

It is difficult to see how soil properties determined on completely remolded samples have any relation to soil properties in situ. Are all pertinent soil properties comparable when tests are conducted on such samples? Presumably these tests would be carried out at natural water contents. Anyone who has tried to remold even a soft clay to determine its sensitivity knows that the procedure is not that simple. Not only are some soils difficult to remold thoroughly at field water contents, but the remolded properties of some highly sensitive soils (e.g., Leda clay) depend strongly on how much energy or effort is applied to remold them. Once such soils are thoroughly remolded, however, they may have the consistency of a viscous liquid. Consequently, such a procedure for all pertinent soil properties tests seems at best impractical.

Appropos the suggestion that, because of sample disturbance, maximum rather than average values of soil properties determined on undisturbed samples be used, one might ask if all pertinent soil properties are reduced because of mechanical disturbance? How would the natural variability in soil properties from point to point within the same site be considered in the procedure?

A much more straightforward evaluation procedure has been suggested by, among others, Moran (1) and Taylor (26). This is to compare field observations with predictions from existing theory (2, 23) using soil properties obtained on the best possible undisturbed samples and utilizing the highest quality laboratory testing techniques. Grid factors mentioned in this section were defined in an earlier paper by Landau (5).

In the last paragraph of this section, Landau makes the rather astounding suggestion that for isotropic silty or clayey soils the c_h be taken arbitrarily as 10 times the undisturbed laboratory c_v for comparison of the efficiencies of various installation methods. Even for varved clays, a factor of 10 may be too large. Careful laboratory and field investigations of a varved clay by Chan and Kenney (29, 30) found the ratio k_h/k_v to be less than 5. Hansbo (31) found c_h/c_v to be between 3 and 5. Thus, the suggestion that c_h/c_v be taken as 10 seems arbitrary and without foundation, even for obviously varved clays.

SAND DRAIN INSTALLATIONS

It is in this section that Landau's commercial interest in the hollow-stem auger technique is evident. Some of the points suggested will either preclude competitive nondisplacement techniques or require expensive (and probably unnecessary) alterations in equipment and procedures. I would prefer that the specifications be more general and include all the minimum displacements methods listed in the second in-text table as well as the newer band-shaped drains described earlier. Perhaps Landau would consider altering his suggested points to make them more generally applicable and, therefore, more useful to the profession. The following comments are offered to assist him in the task.

1. By not allowing the alternate raising and lowering of the cavity-forming tool, Landau effectively precludes one of his strongest competitors, the Dutch Jet-Bailer method. Other jetting techniques may also utilize an up-and-down action of the jetting tool. The important item here is that the tool should be operated in such a manner as to minimize displacement and disturbance of the soil. It would also seem that different techniques might be applicable to different soil types and geologic conditions.

2. The vertical alignment of the formed drain is probably less critical to its function than either the continuity or the disturbance factor. It is interesting to note that in 1972, Landau proposed vertical alignment limitations of 0.55 percent (9, p. 72); Aldrich and Johnson (9, p. 74) pointed out that such a specification was excessively strict and difficult to verify in practice. Apparently, Landau has followed their suggestion and relaxed his plumbness criterion somewhat. It seems that

this specification is relatively unimportant and furthermore is difficult to verify in practice, especially after the drain is installed.

3. This item is applicable to augers only.

4. Landau's suggestion to adjust the rate of advance of the jetting tool as required in the field is better than specifying an exact maximum. Some jetting specifications have permitted tool advance rates twice as fast as he recommends [up to 0.102 km/s (20 ft/min)] without apparent excessive disturbance. Field tests or past experience could be used to determine the appropriate rate.

One hopes that Landau does not really mean to equate nondisplacement with zero pore-pressure increase. (a) An increase in excess pore pressure results from an increase in total stress and does not necessarily mean soil disturbance has occurred. (b) If a pore-pressure increase is indicative of disturbance, then the auger method apparently caused some disturbance to the clays in the Maine test section he later refers to. Aldrich and Johnson (9, p. 72) report excess pore pressures up to 2.13 m (7 ft) of water head as a result of installation of drains by the hollow-stem auger method. It should also be mentioned that this excess pore pressure was the least of the three methods tested (32, p. 46).

One wonders if it is reasonable to apply specific requirements under the rubric jetting to all the jetting installation techniques in the second text table.

5. Where does Landau recommend that limiting jetting pressures be measured? What is the purpose of limiting the pressure (however determined) to twice the existing hydrostatic pressure in the soil at that depth? A better approach would be to utilize jetting pressures that are adequate to do the job. Actual pressures would be determined by experience or by preliminary field trials. They must be somewhat soil and site dependent (i.e., not all pressures will work satisfactorily for all sites). If jetting-induced pore pressures dissipate within 24 h the soil should be rather permeable. Is deep drainage really required at such sites?

6. This item should also include a statement that spoil from all auger methods must be disposed of properly.

7. This is covered by item 6 if protection of the environment is the objective.

8. Why is it necessary that a rigid cavity support be provided when penetrating the granular working platform or sand blanket under the fill? It would be better to simply require that the cavity be maintained or at least the drain should have continuity with the drainage blanket. As long as the cavity is filled with water, there should be little problem with collapse of the hole. Also is it really necessary to backfill the hole simultaneously with the removal of the cavity support? As long as the hole stays open until the hole is completely filled, that should be sufficient. Careful inspection during backfilling operations would ensure that collapse or excessive squeeze has not occurred. In fact, an added advantage accrues with some jetting methods in that the hole can be inspected for depth, diameter, and plumbness prior to backfilling. Finally, why is the recommended 206.84 kPa (30 lb/in^2) (minimum) air pressure limited to twice the in situ hydrostatic pressure? How is one assured that arching in the pipe or hollow stem and, therefore, a void in the sand drain has not occurred?

9. Landau might want to point out that sometimes sand bulks and there might be some difficulty in knowing the volume of the sand backfilled, i.e., volumetric measurements must be at the same relative densities.

CONSTRUCTION SEQUENCE FOR SAND DRAIN SECTIONS

The procedure suggested in item 2 would rarely, if ever, be followed in actual practice for several reasons. Potential stability problems at very soft sites would dictate that sand drain installations be carried out from low working platforms of free-draining granular materials (this platform would later serve as the drainage blanket). The advantage is that there would be less distance to drive, push, auger, or jet the drains, and drain lengths would thereby be minimized.

Installation of piezometers within 60 cm (2 ft) of the first few sand drains is not so easily accomplished. As pointed out by Hansbo (31, p. 90) and from personal experience, it is difficult to know the exact location of the tip after installation, especially if the piezometers are slowly pushed into the ground as is common in very soft deposits. Anyway, why not attempt to install the piezometer tip halfway between two drains? What is the significance of the 24-h dissipation time? Is there some valid reason for recommending 85 percent instead of the more common 90 percent for nearly complete consolidation?

Under item 9, Landau recommends use of Equation 14 and Figure 5 for the evaluations of disturbance (presumably for comparison of different spacings and installation techniques). Can the range of values indicated on Figure 5 be extrapolated to other sites and soil conditions? What values of the two disturbance parameters should be allowable in practice? Finally, is strength loss due to drain installation really one of the properties we want to use to compare different methods? As mentioned before, it is doubtful that such an approach is practical in many sedimentary deposits due to the natural variation of undrained strength with depth and across the site.

EVALUATION OF FIELD DATA

Backcalculation of soil properties, especially consolidation properties, from field measurements of settlement and pore pressures is not always so easy (33). For example, in the previous section the problem of knowing the exact location of the piezometer tip was mentioned. Calculations of c_v from pore-pressure dissipation data require a precise knowledge of the distance from the piezometer tip to the drainage surface of the sand drain. Another problem in evaluating field data occurs in slightly overconsolidated soils if the stress increase due to the fill does not substantially exceed the preconsolidation stress. Leaving the fill in place prior to installation of the drains may be a worthwhile suggestion, but it is probably only practical for test sections. Then one has the problem of c_h, which is required in the design calculations. What will be the relation between the c_v (as backcalculated from field observations) and the c_h?

The suggestion in item 4 seems unnecessarily complicated. It would be simpler to use the piezometer observations to check for dissipation of excess pore-water pressures. Under item 5, perhaps Landau can recommend a practical method for estimating the completion of primary in advance of complete dissipation of excess pore-water pressure. Does he recommend extrapolation of the surface settlement-time curves, or would he utilize the compressibilities from conventional laboratory consolidation tests?

While the suggestion in item 7 to use changes in water contents to check settlements is attractive, I have not found it to be particularly successful (24, pp. 460-462), even in relatively (by U.S. standards) homogeneous clays.

COMPARISON OF INSTALLATION METHODS

It is a pity the results and observations of the East St. Louis test section, so often referred to by Landau, have never been published. Thus he has unfortunately only one case history to illustrate his suggested procedure for comparing methods of installation. Tables 1 and 2 appear to be identical to those presented by him previously in his discussion of Aldrich and Johnson (9). A detailed critique of these tables and Landau's procedures has already been effectively done by Aldrich and Johnson in their closure (9, pp. 72-74).

The suggestion that spoil from augered sand drains can be utilized in the embankment fill itself (Figure 6) is somewhat doubtful, due to the generally poor quality soils to be sand drained. They are often silty and organic, have high water contents, and are difficult to compact properly. Such materials should only be used in the berms.

To Landau's comments on developing true comparisons must be added differences in site conditions, geology, availability and cost of backfill materials, depth of stratum to be drained, labor (including local union regulations), weather, and a myriad of other factors that must be considered for any cost comparison to be meaningful. Finally, I believe that within a few years some of the newer European and Japanese drainage techniques described earlier will make much of the controversy about sand drain installation techniques irrelevant.

COMMENTS ON DRIVEN SAND DRAINS

As noted by Johnson (16), depending on choice of c_v and magnitude of additional load, mandrel-driven sand drains have been satisfactory at a large number of sites. Disadvantages of increased settlement and decreased c_v due to smear and disturbance may be negligible and of little practical importance (16, p. 160). A hypothesis will now be offered as to why this has often been the case.

In 1972, Holtz and Holm (34) excavated more than 2 m around some sand drains at the Skå-Edeby test field in Sweden and examined carefully the surrounding soft clays for evidence of disturbance and remolding. We were astonished to find vertical cracks in this very soft clay ($s_u < 10$ kPa, $S_t \sim 15$) filled with sand as far as 200 mm away from the drain face. In other words, the operating diameter of the sand drain was not 180 mm as originally installed, but up to 380 mm in places. Drain spacing at this site (test area no. 1) was 0.9 m (i.e., the actual $n \sim 2.4$, or about half the design n of 5). Thus the time for consolidation would have been about four times faster than calculated if no allowance was made for disturbance (c_h versus c_v). The cracks probably resulted from hydraulic fracturing. Recent theoretical work and field observations during pile driving by Massarsch and Broms (35) and Massarsch (36) show that fractures in soft clays will tend to form in the vertical direction, which verifies the field observations by Holtz and Holm (34). Thus, hydraulic fracturing with the associated formation of sand-filled vertical cracks may be another plausible explanation why closed-end mandrel-driven sand drains, with all their disturbance and smear, still have worked reasonably well at so many sites.

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Analysis of Settlement Data From Sand-Drained Areas

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Rate of field consolidation is usually calculated from changes with time of piezometer readings. Presented here is a technique for analysis of field settlement observations to determine the field rate of consolidation and total settlement for sand-drained areas. This technique is developed from equal strain consolidation theory. The approach is demonstrated and verified using field data from three construction sites. For each site, the settlement data were analyzed for rate of consolidation and total settlement. The coefficient of consolidation values extracted from the settlement data are compared to those calculated from changes in pore pressures. Total settlement indicated by the analysis is compared to the maximum settlement observed at each platform. Piezometers are important for controlling construction. However, by use of this technique the complete analysis of field data can be achieved independent of piezometer readings.

Analysis of field data for rate and amount of settlement provides a check on design parameters and assumptions. A review of field data from previous projects in similar soil deposits can be a valuable guide to the most economic design. This is particularly important when considering vertical sand drains, since the expense of drain installation must be offset by faster consolidation.

Vertical sand drains have been used for nearly 50 years to shorten the time to achieve settlements in clay layers (1). There is, however, some question about sand drain effectiveness in sensitive clays when displacement methods are used to form the drains (2, 3, 4, 5, 6, 7). A judgment on the effectiveness of sand drains usually requires analysis of field data for rate and amount of settlement and the comparison of these field values to the parameters predicted from laboratory tests. The analysis of field data also provides information on soil disturbance due to the method of drain installation, since disturbance tends to decrease the rate of consolidation and increase the amount of settlement.

Field data often include information on rate of filling, piezometer readings, and settlement platform observations. Field values of rate of consolidation are normally computed from the change with time of the excess pore pressures as indicated by piezometers. The amount of ultimate consolidation settlement the fill will experience is usually computed from the change in piezometer readings and settlement platform observations. Steps in the analysis using piezometer readings have been outlined by Johnson (1) and Moran and others (8).

Settlement platforms are less expensive to install and easier to maintain than piezometers. The determination of rate of consolidation from settlement data alone has been considered difficult or impossible (1). Presented here, however, is a simple technique for using settlement data only to analyze for rate of consolidation and total settlement. The technique is based on equal strain consolidation theory for vertical sand drains and is as easily applied as the conventional analysis of pore pressures. The values of rate, as indicated by the coefficient of consolidation, analyzed by this settlement method are compared to values determined from piezometer readings. Computed total settlements are compared to the maximum observed settlement. This technique allows more extensive analysis of field data from sand-drained areas.

THEORETICAL BASIS

Consolidation in a sand-drained area can be envisaged as dissipation of excess pore pressures in the vertical and radial directions with reasonably well-defined boundary conditions. The average consolidation reflects the dissipation in both directions and can be written (9)

$$1 - U_c = (1 - U_R)(1 - U_v) \quad (1)$$

where

U_c = average consolidation of the clay layer,
U_R = average consolidation if only radial flow to the sand drains occurs, and
U_v = average consolidation if only vertical drainage occurs.

The expression for average consolidation due to radial drainage, assuming equal strain, is (10)

$$U_R = 1 - \exp[-2T_R/F(n)] \quad (2)$$

$$F(n) = \{[n^2/(n^2 - 1)]\ln(n) - (3n^2 - 1)/4n^2\} \quad (3)$$