EMBANKMENT TEST SECTIONS TO EVALUATE FIELD PERFORMANCE OF VERTICAL SAND DRAINS FOR INTERSTATE 295 IN PORTLAND, MAINE

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A field test program was conducted in 1967-69 to evaluate the in situ performance of vertical sand drains. The work was sponsored by the Maine State Highway Commission in order to provide pertinent design data for the highway embankment of Interstate 295 in Portland, Maine. Test drains were installed to average depths of 60 to 70 ft at one location and 30 to 40 ft at another. The main objectives were to evaluate the relative effects of drain installation method (driven, jetted, and augered), drain spacing, and influence of soil type. Each drain type was installed at 10- and 14-ft spacing in a triangular pattern, and the test areas were surcharged with at least 20 ft of embankment fill above original grade. Backfigured values of the coefficient of consolidation for horizontal drainage, c_h, were compared with values interpreted from laboratory tests, assuming that there was no disturbance. The ratio was approximately one-half for the augered and jetted drains and considerably less for the driven drains. There was less evidence of disturbance for the augered and jetted drains; the driven drains indicated a significant reduction in c_h at the closer spacing. The test program illustrates that the method of installation is indeed important and that there is a need for development of improved methods of drain installation to further reduce the effects of remolding in sensitive clay soils.

•THE ALIGNMENT of Interstate 295, referred to as the Portland Loop, consists of a complex of expressways and feeder routes connecting with the Maine Turnpike in south Portland and terminating at Tukey Bridge located north of Portland. The selected route traverses 2 separate tidal-flat areas, each approximately 1 mile in length, at the Fore River crossing and along the easterly edge of the Back Cove area.

Initial studies by the Maine State Highway Commission (MSHC) and its consultants reached the conclusion that the most feasible highway design in these areas would be earth embankments, rather than viaduct structures, with the exception of required bridge crossings at the Fore River channel and at the interchanges.

The design pavement grades range from 15 to 50 ft above the existing tidal mud flats. Underlying each of the areas are extensive deposits of soft to medium consistency, sensitive, gray silty clays to depths of 100 ft or more. Overlying the clay in tidal areas are soft, weak organic clays, with depths from 10 to 40 ft.

Embankment settlements of up to approximately 7 ft were predicted, which would normally require several years to complete. Also there were serious embankment stability problems. Therefore, it was decided that the installation of vertical sand drains would be appropriate to increase the rate of consolidation and the gain in shear strength of the soft foundation soils. The magnitude of the project indicated that ap-

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proximately 2 to 3 million linear feet of sand drains would be needed to achieve the desired results.

In view of the many unknown factors and variables associated with the design of sand-drain stabilization systems and the prediction of their performance, realistic cost estimates and construction scheduling were highly indeterminate. A number of reports (1, 5) discuss the many complexities involved with the design and installation of vertical sand-drain projects. The MSHC decided that an extensive field test program to evaluate at least some of the unknowns was warranted and would provide a much more realistic basis for final project design. Because of the differences in soil profiles, 2 separate test areas at the Back Cove and the Fore River crossings were necessary to provide criteria for final design of sand drains based on in situ performance records. For this purpose, 3 methods of sand-drain installation, which had been used in construction elsewhere, were selected. The standard driven-drain method, with closed-end mandrel, was included in order to measure the probable adverse effects of soil displacement, as compared to 2 other available installation methods that had been developed to minimize displacement in soft cohesive soils. The relative effects of drain spacing were evaluated by installing each drain type at 2 different center-to-center spacings on a triangular pattern.

In addition, the MSHC gained considerable experience with respect to design and construction of embankments placed on soft organic clay soils in the presence of a tidal range of approximately 9 ft.

SITE AND SUBSURFACE SOIL CONDITIONS

The Long Creek-Fore River area is a transverse river crossing of approximately 4,200 ft between shorelines, of which all but approximately 300 ft is exposed mud flats at periods of low tide. Access for embankment construction equipment in this vicinity could be gained from either shoreline only by advancing out over completed fills.

The Back Cove area covers a distance of approximately 4,500 ft along the easterly edge of a broad tidal cove, most of which is exposed at periods of low tide. Access for embankment construction was available at several points along the shoreline from the existing parallel roadway, Marginal Way. Outboard of the proposed embankment, an existing dredged channel had to be maintained.

At each of these areas, the tidal range is approximately 4.5 ft above and below USCGS mean sea level, which is referenced here as el. 100.

The soil conditions throughout the project limits are described in MSHC reports (6, 7) based on borings and laboratory tests performed by the MSHC prior to 1967. Subsequently, additional borings were made in connection with the Back Cove test (BCT) site and the Fore River test (FRT) site and with final design.

An earlier report (8) contains a complete summary and interpretation of soil data and analyses of soil engineering properties, based on MSHC laboratory data, as well as additional testing performed by Haley and Aldrich, Inc., and at the Massachusetts Institute of Technology.

The major subsoils encountered in the project areas are given in Table 1. For convenience in referencing, they are designated as layers A, B, C, and D, with subdivisions as noted.

A general soil profile across the BCT site is shown in Figure 1. Depth profiles at each of the 6 test areas within the BCT site are shown in Figure 2. Underlying the clay, layer D is considered to be a free-draining sand and gravel generally 10 to 20 ft thick, extending to the bedrock surface. During the original borings in the vicinity, artesian pressures on the order of 5 ft were encountered in this sand layer.

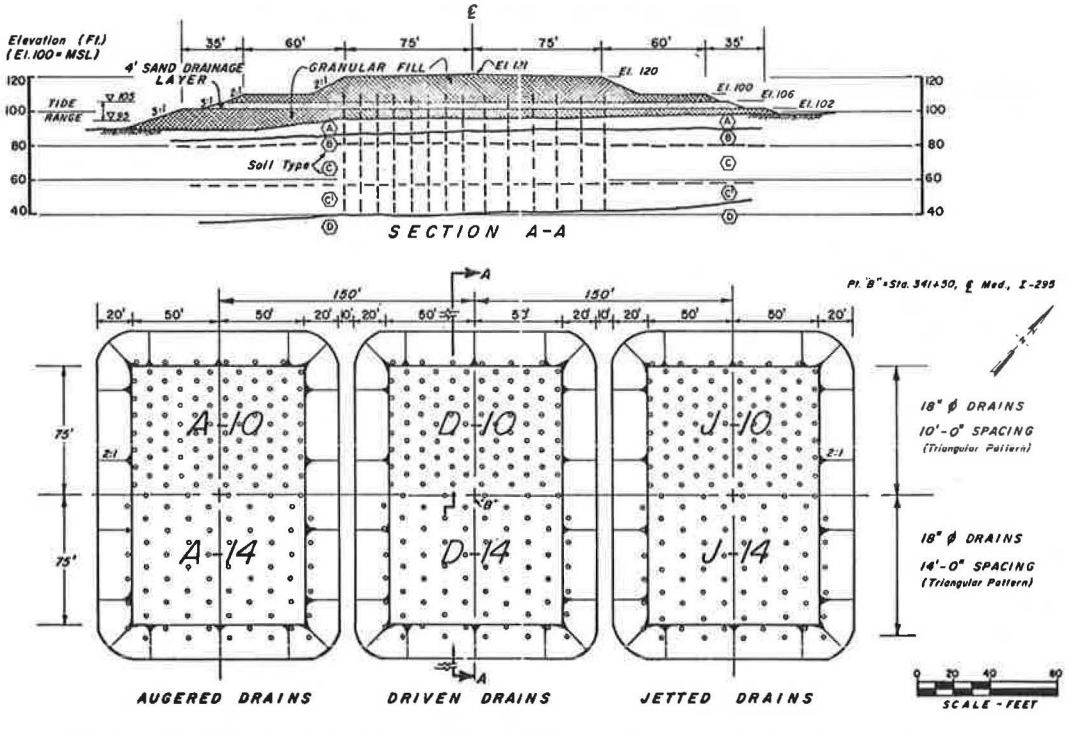
Typical properties of the cohesive materials at the site are given in Table 2, including the approximate compressibility factors based on a very thorough evaluation of all available test data. For layer C, the silty clay is medium to soft in consistency, only slightly precompressed, with an average liquidity index of about 1.2. The sensitivity of this material is estimated to range from 10 to 20.

General soil profiles at the FRT site are shown in Figure 3; detailed depth profiles at each of the 4 test areas within the FRT site are shown in Figure 2. At this site the

Table 1. Foundation soil types.

Layer	Soil	Description	Remarks
A	Organic clay	Medium to soft slightly organic gray silty clay with many broken shells, bits of wood chips, frequent sandy zones, and occasional peaty zones	Usually the surface layer in tidal mud-flat areas
A_s	Organic sand	Loose, slightly organic, gray silty sand with broken shells	Generally encountered at bottom of layer A at Fore River
B	Stiff clay crust	Medium to very stiff, weathered, gray or brown silty clay with occasional sand layers	Often forms the crust of layer C
C	Gray silty clay with black specks	Gray silty clay with black specks or bands and occasional shells; medium to soft and very sensitive at Back Cove, and medium to stiff at Fore River	Illitic marine clay that has been leached
C′ or C″	Varved clay with black bands	Same as above, with lenses to alternate layers of silt or sand or both; soil frequently becomes more varved or more sandy or both with depth	Frequently forms the lower portion of layer C
C_s	Silty sand	Loose to dense gray silty sand (usually poorly defined)	Sometimes encountered at top of layer C at Fore River
D	Silty sand and gravel	Medium to loose gray silty sand and gravel, with artesian water pressure	Generally overlies bedrock; free draining

Figure 1. Back Cove test section.



organic clay is underlain by granular soil (layer D) approximately 20 ft in thickness, extending to probable bedrock surface at approximately el. 50.

Summarized soil properties and compressibility values are given in Table 2. The organic clay (layer A) is slightly precompressed. The Atterberg limits fall close to the A-line on the plasticity chart, with the plasticity index averaging about 34 percent; the liquidity index is usually very near unity. The sensitivity of this material is on the order of 5 to 10. Ladd, Aldrich, and Johnson (10) give further information on the strength and stress history characteristics of this soil.

DESIGN OF TEST SECTIONS

The selection of suitable sites for the 2 test sections included the following general considerations: (a) They should be within the limits of future highway construction in order to salvage the fills and sand drains after completion of the tests; (b) the subsoil conditions should be representative of a major portion of each project area; (c) the subsoil conditions within the test areas must be reasonably uniform for purposes of the controlled tests; and (d) the sites must be readily accessible from the shore and be within property limits that the MSHC could gain rights to at the time.

Specific considerations at each site selected were as follows:

1. At the BCT site, the outer limits of fill were restricted in part by the existing channel, the inboard side was designed to maintain an existing storm sewer discharge, and the center of the fill area was located to coincide with the centerline of the I-295 median; and
2. At the FRT site, it would have been too costly to place the fill in the river proper, and, therefore, it was necessary to locate within the Long Creek area where space was limited by the presence of an important pipeline crossing, which could have been damaged by the fill construction (the test area was subdivided into 4 sections rather than 6 as at BCT).

A supplementary benefit realized from the test project was to be gained in the selection of suitable earthwork materials for the embankments and development of feasible methods of placement. There was a degree of uncertainty regarding difficulties that might be encountered in the placement of fills out over the existing soft organic clay (mud flats)—coping with a tidal range of approximately 9 ft, losses of material due to erosion, fill stability, trafficability of construction equipment, and other related problems.

Granular borrow for underwater fill and for embankment fill were specified in accordance with general MSHC standards for such materials, for which a wide gradation range is acceptable.

A uniform, 4-ft thick sand-drainage layer was specified to be placed above el. 102. to provide unrestricted drainage. Material of the same gradation was also specified for the sand-drain backfill (free-draining sand with less than 2 percent passing a No. 200 sieve).

The horizontal drainage layer was placed at the lowest practical level for efficiency. Therefore, it was necessary to penetrate this layer completely with the drains, which were installed from a stable working level at el. 110 (BCT) and el. 108 (FRT).

Back Cove Test Site

The 3-section fill area, as shown in Figure 1, was designed to provide for testing of 3 types of drains. Alternate layouts that might have achieved better symmetry were considered but ruled out because of variations in subsoil conditions, existing channel, property limits, and budget. Although there are some variations in soil profiles beneath each area, they were carefully observed and accounted for in the analyses. The theoretical stress distribution below each surcharge fill is such that the overlapping effects are minimal, and reasonably uniform stress conditions apply for each of the 6 areas.

The dimensions of the initial stage of fill, which was placed to el. 110, is 520 by 270 ft. At this level the sand drains were installed. Each sand-drain area was subse-

Figure 2. Soil profiles and instrument locations at test sections.

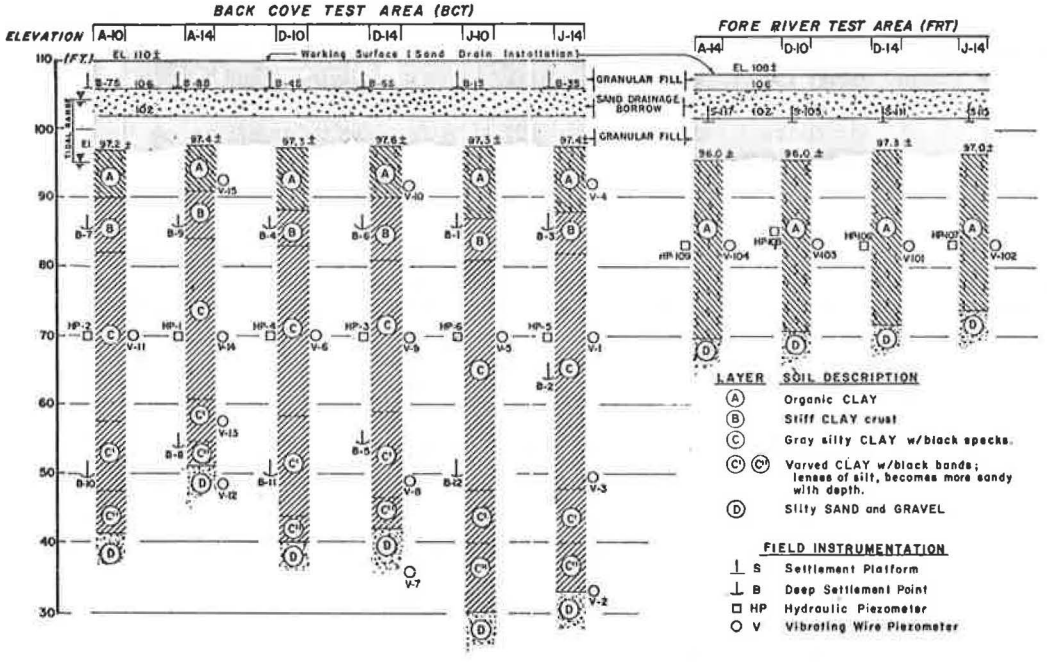


Table 2. Properties of cohesive foundation soils.

Layer	Site	Natural Water Content (percent)	Atterberg Limits		Specific Gravity	Total Unit Weight (pcf)	Average Field Vane Strength (psf)	Approximate Sensitivity	One-Dimensional Consolidation			Stress History
			W_L	PI					CR[a]	CR[b]	RR[c]	
A	BCT	—	—	—	2.66	102.5	400 ± 100	5 to 10	0.22	0.25	0.022	Slightly precompressed
	FRT	65 ± 15	65 ± 15	34 ± 12	2.66	101	600 ± 100	5 to 10	0.24	0.25	0.022	Slightly precompressed
B	BCT and FRT	—	45 ± 5	22 ± 3	2.78	118	Over 2,000	—	—	—	0.020	Very highly precompressed
C and C′	BCT	47 ± 7	44 ± 5	20 ± 4	2.78	111	650 ± 150	10 to 20	0.24	0.28	0.020	Slightly precompressed
	FRT	40 ± 15	42 ± 7	20 ± 4	2.78	115	1,500 ± 500	10 to 20	0.21	0.25	0.022	Moderately precompressed
C″	BCT	—	—	—	2.78	115	—	—	0.20	0.20	0.020	Slightly precompressed

[a]Virgin compression ratio = $C_c/1 + e_o$ measured from odometer tests.
[b]Virgin compression ratio value, modified for use in analyses, to account for sample disturbance or 3-dimensional effects or both.
[c]Recompression ratio = $C_r/1 + e_o$, taken along the rebound curve; this may be low, and values of RR as high as 0.05 ± 0.02 should be considered.

quently surcharged by 10 ft of additional fill, placed in 3 separate pads measuring 100 by 150 ft each. The center of each pad was crowned to 1 ft higher for purposes of drainage and to allow for "dishing" with settlement. A 60-ft wide berm extends in all directions beyond the sand-drain test areas for stability and vehicle access.

Fore River Test Site

At the FRT site, space and budget limitations restricted the size of the test area. Figure 3 shows that a square fill area permitted the installation of drains in 4 quadrants for symmetry. The location was further restricted by the presence of existing 12- and 18-in. oil pipelines buried in the soft tidal flats. These could not be disturbed until such time as a stabilized alternate crossing route was prepared.

The design dimensions of the initial fill to el. 110 were approximately 240 by 240 ft, and the limits of an additional 10-ft surcharge fill were 120 by 120 ft, which was subdivided equally into 4 test sections. A 40-ft wide level berm extends around the perimeter for stability and access.

VERTICAL SAND DRAINS

In view of budget limitations, the scope of this program included only those factors that could be more readily observed and be of most direct benefit to this particular project. The test program included the following considerations:

1. A comparison of various drain diameters was not attempted (in this case, all drains were to be 18 in., a size that is commonly used in the United States);
2. A triangular pattern of drain layout was assumed to be most efficient, based on theoretical considerations (3);
3. Maximum and minimum center-to-center drain spacings of 14 and 10 ft respectively were selected to "bracket" the probable range in final design, with the anticipation that performance for intermediate spacings could be interpolated from the field results;
4. Methods of installation were limited to 3 types, each type to utilize equipment and procedures generally accepted and used elsewhere on previous work;
5. Generally accepted gradation requirements for sand backfill materials were adopted; and
6. Rate and magnitude of consolidation of the compressible soils and pore pressure dissipation were observed by field instrumentation (as necessary).

Selection of the 3 drain types was based on several considerations.

The driven with closed-end mandrel method was included because it was the most commonly used in the past (2) and was generally found to be the least expensive to install. However, there was serious doubt as to the effectiveness of this method in soft, sensitive soils, for the disturbance associated with installation would probably increase total settlements and reduce in situ shear strengths where potential stability problems exist (1). This method was included, therefore, to provide a measure of comparison.

Of the several methods utilizing jetting procedures available in 1967, the jetted with open-end mandrel method, which had been in use for several years, deserved consideration. It was believed that there would be essentially no lateral displacement of in situ soils during installation because all soil is theoretically cut and removed by internal jetting action within the casing.

The augered with continuous hollow shaft auger method, which had been developed and patented in recent years (11), was considered by many to satisfy the requirements of a "nondisplacement" installation method. There were specific problems at this site to be evaluated such as the difficulties in augering through 10 to 15 ft of granular fill, the penetration of a stiff clay crust at Back Cove, and possible equipment limitations when advancing to depths of approximately 80 ft.

CONSTRUCTION OF TEST SECTIONS

A contract for construction of the embankment stabilization test project was awarded by the MSHC in April 1967. The placement of underwater fill started initially at the BCT site during April and at the FRT site in May. The fill was brought up to working

level for installation of sand drains by July 1967. The settlement platforms, deep settlement points, and piezometers (vibrating wire type) were installed as soon as possible. The platforms were placed by the contractor, and observation readings were taken by MSHC personnel. All other instrumentation was installed and observed by commission personnel. The vertical sand drains were installed after filling had been brought to a stable working level (el. 110 at BCT and el. 108 at FRT). The augered and jetted drains were installed in both areas during July and August, and the driven drains were placed during September and October 1967. The placement of the remainder of the test embankment fills proceeded thereafter and was essentially completed by late November 1967.

Augered Drains

A design total of 183 augered sand drains were installed by the contractor (11). The 18-in. diameter hollow-stem auger was advanced to the required depths by lowering the assembly under its own weight while being rotated by the electric drive motor through the assembly at the top of the shaft. A guide, close to the base of the leads, controlled plumbness. In general, the rate of advance was controlled at approximately 1 pitch length per revolution. However, at the BCT site, the auger often met high resistance in the stiff clay crust, resulting in slower penetration rates; at the FRT site, faster rates occurred in the soft organic clay soils. When the maximum depth was reached, the shaft was given one complete revolution in the reverse direction. The specified sand backfill was placed from a loading skip, filling the 8-in. inside diameter hollow stem and the feed tank at the top. With air pressure (up to 75 psi) applied to the internal system, the unit was pulled out of the ground without further rotation. Sand was expelled through the bottom of the shaft as the steel cover plate fell free of the end.

Driven Drains

A design total of 235 driven drains were installed by the contractor. The 16-in. OD mandrel, with an 18-in. built-up end section, was driven by means of an air-operated McKiernan-Terry Model 11-B-3 hammer (rated energy = 19,000 ft-lb). Plumbness was maintained by guides attached to fixed leads. The initial penetration through the sand fill proved to be very difficult. At the BCT site, it was found necessary to assist the penetration of the mandrel by means of portable jet pipes. Also, the contractor experimented by using a vibratory hammer for the last 12 drains but with limited success. Upon advancing to maximum depth, the mandrel was filled with specified sand by means of a skip that traveled up the leads. Air pressure, up to 100 psi, was applied, and the mandrel was extracted. By trial, the most suitable air pressures for various depths were determined.

Jetted Drains

A design total of 183 jetted sand drains were installed by a subcontractor. The top of the 18-in. OD casing was suspended from a bridle and cable arrangement attached to the crane rig (without guides near the bottom). The "holepuncher" consisted of a 12-in. OD internal jetting pipe, fitted with connections for water hoses at the upper end, which could be lifted independently within the outer casing. Its travel with respect to the casing was limited by a built-up flange that would stop against the built-up driving head at the top of the casing. Thus, the length of the internal pipe determined the penetration depth of the holepuncher with respect to the outer casing. Generally, the pipe was maintained approximately 12 in. short of the casing tip. The holepuncher was raised and lowered such that the whole assembly penetrated the soil by the combined washing and chopping action. When the holepuncher had advanced to full depth, the flow of water continued until solid materials within the casing were removed and the amount of suspended solids in the wash water was acceptable (2 percent specified). The holepuncher assembly was then removed, and the casing was backfilled with specified sand, through the water, whereupon the casing was pulled and the drain was completed.

INSTRUMENTS AND FIELD MEASUREMENTS

A major design objective had been the creation of essentially similar soil stress conditions below the interior portions of the surcharge fill at each of the 6 sand-drain test areas at the BCT site and at each of the 4 quadrants at the FRT site so that drain performance could be evaluated. Therefore, the major concentration of instrumentation was within and below the central portion of each of these areas. Figure 2 shows the location and identification of instrumentation.

Settlement measurements were obtained at original ground surface by means of standard settlement platforms (SP) and at intermediate depths (to obtain relative consolidation within layers A and C) by means of deep settlement points (DSP). For the latter, Borros points were used (modification of anchor posts, manufactured by the Borros Company, Ltd., Sweden). All piezometers (vibrating wire type supplied by Geonor, Ltd.) were located as closely as possible to the midpoint of the equilateral triangle formed by 3 adjacent sand drains. Observations at the BCT site were made at middepth in soils of layers A, C, and D, whereas at the FRT site they were required only within layer A. For purposes of comparison and supplementary data, a number of porous-tube hydraulic piezometers were also installed.

All measurements were made and recorded by MSHC field personnel.

EVALUATION OF PERFORMANCE OF TEST SECTIONS

Analysis of Field Data

To evaluate the field results, we realized that direct comparative plots of settlement versus time for each test group would not be sufficient because the depth of compressible soils varied at each section, and the method of drain installation might have affected the rate and amount of consolidation settlement. Therefore, the following approach was adopted.

1. After the observed settlement curves were adjusted for possible instrument errors, for movements due to soil heave, and for initial settlements due to undrained shear, semilog plots of consolidation settlement versus time were prepared. Because the soil thicknesses for each area varied, the data were then replotted in terms of percentage of vertical strain versus time. However, these plots indicated that primary settlement was not complete (as of November 1969); therefore, it was necessary to predict the magnitudes of final consolidation settlements, from which the estimated average degree of consolidation, U percent, was plotted for each test area. Also, semilog plots of the excess pore-pressure ratio, u/u_o, versus time were prepared. From either of these 2 plots, backfigured values of c_h were computed.
2. The cost of a sand-drain installation is strongly influenced by the adopted design value of the coefficient of consolidation, c_h. Therefore, relative "efficiencies" of the sand-drain test groups at the BCT and FRT sites were compared on the basis of the resultant backfigured values of c_h. These values serve as a basis for determining expected rates of settlement for each group, whereas the ratios of the backfigured field values to the laboratory values, determined for relatively undisturbed soil samples, serve as a basis for evaluating the apparent degree of disturbance that might have taken place in the field.
3. Another important consideration is the effect of the particular type of drain installation on the final total amount of consolidation settlement, for soils that are disturbed or remolded in situ become more compressible.
4. To arrive at meaningful conclusions with respect to the relative or absolute performance of the sand drains required that a considerable amount of judgment be applied to the analysis of the field data. This was largely attributed to the following: (a) The primary consolidation of the compressible soils (layer C at the BCT site and layer A at the FRT site) was not complete as of November 1969 (approximately 2 years after completion of surcharging), and (b) the observed pore-pressure readings were quite erratic, especially at the BCT site. Therefore, the conclusions presented here were not based entirely on factual observations inasmuch as it was necessary to adopt

Figure 3. Fore River test section.

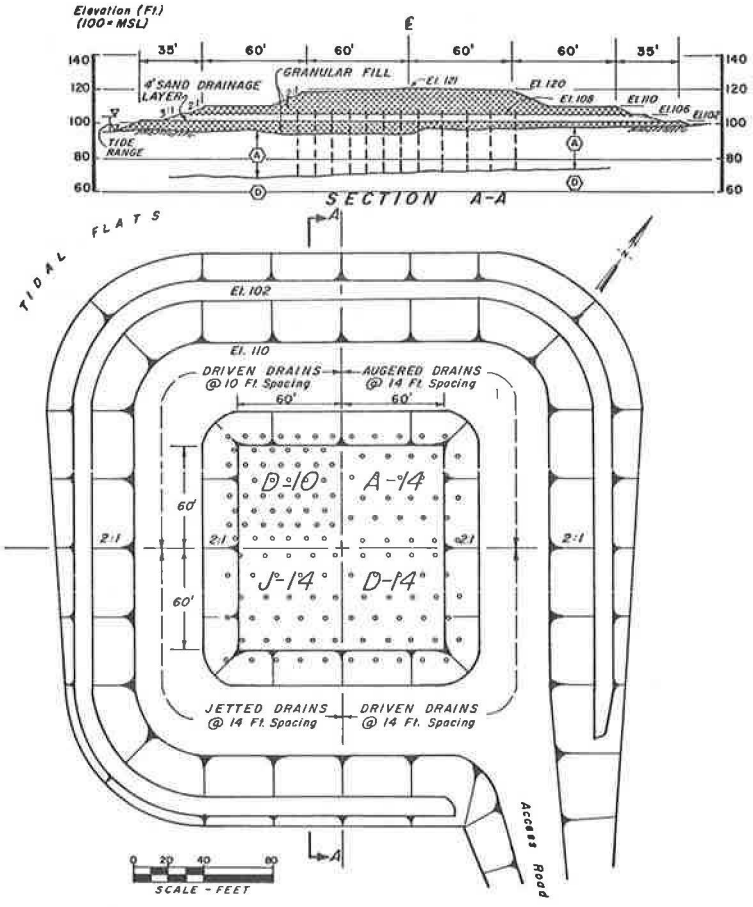


Table 3. Summary of results.

Site	Sand-Drain Installation Method	Sand-Drain Spacing (ft)	Coefficient of Consolidation: Backfigured From Field Tests (ft^2/day)	Coefficient of Consolidation: Field: Laboratory	Predicted Total Primary Consolidation: Laboratory Tests (ft)	Predicted Total Primary Consolidation: Field Data[a] (ft)	Predicted Total Primary Consolidation: Field: Laboratory
BCT	Driven	10	0.040	0.27[b]	1.89[c]	2.95[c]	1.56
		14	0.040	0.27	1.58	3.53	2.23
	Augered	10	0.080	0.53	1.86	1.70	0.91
		14	0.100	0.67	1.63	1.86	1.14
	Jetted	10	0.065	0.43	1.84	2.84	1.54
		14	0.085	0.57	0.97	1.97	2.03
FRT	Driven	10	0.030	0.38[d]	3.0	4.0	1.33
		14	0.055	0.69	3.0	4.0	1.33
	Augered	14	0.065	0.81	3.2	3.7	1.15
	Jetted	14	0.065	0.81	2.7	3.2	1.18

[a]Predicted from field data as of Nov. 1969 (primary settlement not complete).
[b]Average value adopted from laboratory odometer tests, $c_h = 0.15 \pm 0.07$ ft^2/day.
[c]For layer C, between shallow and deep settlement points.
[d]Average value adopted from laboratory odometer tests, $c_h = 0.08 \pm 0.04$ ft^2/day.

many assumptions during the course of these analyses. Where feasible, averaging methods were employed to minimize the effect of any erroneous assumptions. A complete report on the test project is contained in another publication (9).

Principal Field Results

The predicted and field-measured results of major significance for each test area are given in Table 3.

For the BCT section, the principal results are as follows.

1. All 3 drain types caused a significant reduction in the effective values of c_h. The augered and jetted types were, however, relatively more efficient than the driven type by a factor of 1.5 to 2.5.
2. For the augered and jetted drains, the closer spacing (10 ft on center) resulted in 20 to 25 percent lower values of c_h than the larger spacing (14 ft); for the driven drains, no appreciable difference was indicated.
3. The magnitudes of the predicted total final field consolidation settlements for the areas of driven drains were 1.5 to 2.2 times greater than values predicted from laboratory tests. The jetted areas also showed a considerable increase, whereas the augered areas agreed within approximately 15 percent.

For the FRT section, the principal results are as follows.

1. All 3 drain types caused a reduction of the effective values of c_h but not so large as for the soils at the BCT site. The driven type was definitely less efficient than the other types at equivalent 14-ft spacing.
2. For the driven drains, the closer spacing (10 ft) resulted in a further reduction of c_h of approximately 45 percent. (The spacing effect was not tested for the other 2 drain types.)
3. The magnitudes of the predicted total final field consolidation settlements for driven drains were 1.3 times greater than values predicted from laboratory tests. The augered and jetted types had closer agreement, within 15 to 20 percent.

PRINCIPAL CONCLUSIONS

1. The method of installation is very important in the very sensitive (S_t = 10 to 20), slightly layered, inorganic clay (layer C, BCT site). In spite of the relatively smaller soil displacements associated with the augered and jetted methods of installation used here, the effective backfigured in situ coefficient of consolidation for radial drainage, c_h, was only 45 to 65 percent of the value based on laboratory tests. An even further reduction was observed for driven drains.
2. The method of installation is not quite so important for the soft, less sensitive (S_t = 5 to 10), slightly organic clay (layer A, FRT site). The driven drains did show, however, that there are significant reductions in c_h with closer drain spacing.
3. These tests clearly indicate that further improvements in the methods of sand-drain installation are needed to increase the relative efficiencies in sand-drain performance. Some of the anticipated advantages or benefits would be in terms of (a) further reduction in sand-drain footage requirements, (b) reduction in time required to achieve desired consolidation, and (c) reduction in the magnitude of total vertical settlement in treated areas.

RECOMMENDED GUIDELINES FOR FUTURE FIELD TEST PROGRAMS

There are many obvious benefits in the performance of a full-scale field test and evaluation program. In this way, realistic design parameters can be observed and measured. Based on the experiences gained from this project, the following comments and guidelines are offered:

1. Select test locations at which subsoil conditions are known to be typical of the major portions of the anticipated project;
2. Make very detailed studies of the soils directly below the test area (include a number of laboratory tests to determine compressibility, stress history, permeability,

and sensitivity and to determine by an adequate number of borings and field tests the limits and possible variations in soil strata, field vane shear strength, field permeability, and existing hydrostatic conditions);

3. Make conservative estimates of time required to achieve primary consolidation and establish a time schedule to permit this and to allow for contingencies (it is extremely difficult to evaluate results unless primary consolidation is completed in the field);

4. Select a drain type or types that appear to promise a minimum of soil disturbance if the soils are sensitive (in any event, include, for comparison, an area of driven drains and also include, if possible, a fully instrumented nondrain area for comparison of pore pressure and strain settlement behavior under similar loading conditions);

5. Include sufficient field instrumentation to provide reliable piezometric and settlement data throughout the anticipated time period (careful attention must be given to accurate location of instruments with respect to drain locations, installation of instruments at several depths within the compressible layer, and provision of an adequate number of duplicate instruments, particularly piezometers, to serve as backup units in case of malfunction);

6. Install sufficient settlement units at an early stage of filling to record initial strain and early consolidation movements prior to, and immediately after, drain installation; and

7. Install the various types of sand drains within as short a time period as possible and apply the surcharge load as uniformly and rapidly as possible thereafter to minimize the effects of time delays and to provide a reasonable basis for comparative performance of drain types.

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DISCUSSION

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As an interested party (consultant to Haley and Aldrich, Inc., for review of test section plans for Back Cove and holder of U. S. Patent 3096622 and others for sand-drain installation by means of augers) to the investigation of the performance of sand drains installed by the mandrel, jetting, and auger methods, the writer was disappointed at the inability of the authors to reach substantive conclusions.

It is the opinion of the writer that the inability of the authors to develop more cogent results relates to their questionable use of laboratory-derived c_h values, which have a range (Table 3) varying by 300 percent for each soil type involved, e.g., from 0.08 to 0.22 ft^2/day at Back Cove and from 0.04 to 0.12 ft^2/day at Fore River. The use of an "average" value of c_h in each instance as a common denominator for backfigured field values developed for each method of sand-drain installation (in order to produce a comparative measure of field performance of each method) results in numerical values having no significance. Considering that the mandrel method of installation has the characteristic of 100 percent displacement of the subsoil in the cavity formation process, the writer has developed data given in Table 4 to show the effectiveness of each method of sand-drain installation using the c_h values obtained from the mandrel-stabilized areas as the basis for comparison. It is evident from this presentation that the most effective method of sand-drain installation is consistently the auger method.

Inasmuch as subsoil disturbance results in altering the consolidation characteristics of subsoil in a manner that increases the magnitude of settlement as compared to that obtained for undisturbed soil, the ratio of field to theoretical settlement given by the authors in Table 3 is given again in Table 5.

A review of the settlement ratios given in Table 5 indicates that the auger method of sand-drain installation produces the least disturbance of the 3 methods tested. Fur-

Table 4. Sand-drain effectiveness.

Site	Soil	Sand-Drain Spacing (f)	Effectiveness Ratio of Method[a] ($c_h/c_{h,\ mandrel}$)		
			Mandrel	Jetting	Auger
BCT	Silty clay	10	1.0	1.6	2.0
BCT	Silty clay	14	1.0	2.1	2.5
FRT	Organic clay	14	1.0	1.2	1.2

[a]The greater the effectiveness ratio is, the greater is the efficiency of the sand-drain installation method. The increase in efficiency with an increase in sand-drain spacing is to be expected (11).

Table 5. Settlement ratio.

Site	Soil	Sand-Drain Spacing (ft)	Settlement Ratio of Method[a] (field value/theoretical)		
			Mandrel	Jetting	Auger
BCT	Silty clay	10	1.56	1.54	0.91
BCT	Silty clay	14	2.23	2.03	1.14
FRT	Organic clay	14	1.33	1.18	1.15

[a]The lower the settlement ratio is, the less is the disturbance induced by the sand-drain installation method (11, pp. 82ff).

thermore, the range of results in the field for the auger method varies from the theoretical by a consistent value of approximately 15 percent, while the settlement ratio results obtained by both the jetting and mandrel methods are inconsistent and range up to 100 percent or more in excess of theoretical. The indications that the mandrel method shows its best results when applied to the stabilization of organic clay at the FRT site cannot be taken as an indication that displacement methods are best applied in such soil types, for this is contrary to the results obtained for the mandrel method as applied to the organic clay of Flushing Meadows in New York (11, p. 85). In contrast to this, the relatively mediocre results obtained at the FRT site for the nondisplacement methods are believed to be related to the fact that the specifications applied for the test installations did not provide for close control of the axial deviation of the cavity-forming apparatus during the sand-drain installation process. Such control is considered essential, particularly when applied to soft soils, as was encountered at Fore River, which is in contrast to the stiffer soils encountered at Back Cove. Since the construction of the test section, the criterion evolved for such control requires that the axis of the cavity-forming apparatus be maintained within a tolerance of 1 in. in 15 ft at all times during the sand-drain installation process. More effective results can be expected by requiring that the apparatus be guided at its upper end and at a point within 10 ft of the ground surface during at least the first 25 ft of penetration of the apparatus into the subsoil.

Although the writer concurs with the authors that additional test sections would be desirable to establish sand-drain design criteria throughout the country, the guidelines for such work as presented by the authors are considered to be much too general for implementation by interested agencies. In this vein, it may be of interest to the reader to know that the HRB Committee on Embankments and Earth Slopes is sponsoring the development of a formal approach to the design and testing of sand-drain installations in an effort to fill this need.

AUTHORS' CLOSURE

Landau candidly admits that he is an "interested party" in view of his direct beneficial interest in promoting the use of the hollow-stem auger method of drain installation.

The authors emphasize that this test program was planned, executed, and evaluated on a strictly impartial basis. The objective of this work was to establish realistic performance data for specific application to the design of the planned sand-drain installation rather than to serve as a "proving ground" for the selection of the most superior type of drain. It was demonstrated that this field test program was fully justified, for the design criteria that probably would have been adopted on the basis of previously available laboratory odometer test data alone would not have been adequate (i.e., the desired soil stabilization would not have been achieved within the available construction time limits).

As stated in the paper, each of 3 types of sand drains were installed at 2 spacings, and their performances were observed in terms of the settlement achieved and the pore-pressure dissipation within the compressible soil layers. The observation and evaluation of these data might have been relatively straightforward, except that a number of complicating factors and conditions had to be considered in the analysis of the results. Some of these are discussed below.

1. The time available (2.5 years) was insufficient to achieve completion of "primary" consolidation for the drain spacings selected.
2. Therefore, the backfigured values of the coefficient of consolidation, c_h, based on settlement data, had to be based on estimated degrees of consolidation for each test area as best interpreted from available pore-pressure and settlement data.
3. The pore-pressure data were influenced by the initial pore pressures developed during drain installation (including the augered type, which produced excess heads up to 7 ft at the BCT site). It is difficult to handle this situation adequately from a theoretical viewpoint.

4. The pore-pressure data were, or could have been, influenced by the relative positions of piezometers with respect to the centerline between adjacent drains, inherent instrument errors, and potential long-term deterioration of instrument accuracy after several feet of settlement occurred.

5. The settlement data were generally considered to be reliable, but considerable judgment was required to take into account the influence of lateral movements due to shear deformations that occurred during initial fill placement and movements associated with the installation of the drains per se, such as the heave that occurred with the driven drains.

6. A major problem arose from the fact that in several instances the total measured settlements within given increments of soil depth actually exceeded the magnitudes anticipated from theoretical predictions based on laboratory data. Plots of percentage of strain versus log time generally yielded essentially straight lines, even after nearly 2.5 years since the middle of the loading period, which strongly suggested that primary consolidation was not yet completed. The piezometers also generally showed that significant excess pore pressure still existed after 2.5 years. Therefore, considerable judgment had to be used to develop "predictions" of the final consolidation settlements, especially at the BCT site because of the sensitive nature of the clay. At the BCT site, these predictions were made as follows: (a) Plots of percentage of strain versus the average effective stress (to a log scale) were developed based on the average degree of consolidation from measured piezometer data between the deep settlement points; and (b) these plots were then extrapolated, assuming a linear relation between strain and log effective stress, to the final computed effective stress that would exist after all excess pore pressures had dissipated (9, Appendix H). In many cases, these extrapolations yielded a final consolidation settlement that was 1.25 to 2.0 times the measured settlement after 2.5 years. Moreover, the shape of the "field" compression curves was sometimes contrary to that which would be expected (based on what is known about the effects of disturbance on the compression characteristics of sensitive clays). Consequently, the values of predicted total field consolidation settlements given in Table 3 are subject to considerable uncertainty in some cases.

Specific comments on Landau's statements are offered as follows.

1. The admittedly wide range of laboratory values of c_h reflected the extreme limits of all data collected. The odometer tests were performed by 3 different agencies, using samples obtained from various elevations and locations, with a variety of laboratory testing equipment and techniques. The selected values of c_h, however, took into account all of these factors, plus others such as the typical variation of c_h with applied load, to arrive at the "best" laboratory value.

2. The use of the best average laboratory value of c_h was believed to be the most logical basis for a common denominator for backfigured field values. We believe the resulting ratios do have numerical significance. They illustrate the relative degrees of efficiency among the 3 drain types (which Landau simply gives in a different format in Table 4), and, of more importance, they illustrate the relative efficiency of each drain type with respect to average laboratory values of c_h for these soils, which might have been adopted for design without the benefit of field observations. In addition, the effects of spacing can be observed for each drain type as well as the degree of disturbance for the 2 soil types.

3. It is readily agreed that the drains installed by the augered method do appear to have a somehwat higher relative efficiency than those installed by the jetted method as given in either Table 3 or Table 4. However, in view of the clearly inferior relative performance of the driven drains, the recommendation was given to the MSHC that this latter method be excluded from the specifications and the design value of c_h be based on the average field performance of the jetted and augered methods.

4. The footnote to Table 4 states that an increase in efficiency with an increase in sand-drain spacing is to be expected. This statement is contrary to theoretical relations that indicate that the "effective" c_h value for any compressible soil is independent of sand-drain spacing (if the effects of remolding in the immediate vicinity of individual drains are ignored). We believe that the increases in efficiency computed for the larger

drain spacings at the BCT site indicate that these methods did, in fact, cause some disturbance close to these drains and that the overall influence on the effective c_h value becomes less with larger spacings. Moreover, field data obtained from the Dutch "jet-bailer" method of drain installation in Portsmouth, New Hampshire, in a very sensitive clay, clearly showed no effect of drain spacing (for spacings varying from 9.0 to 16.2 ft on center with 12-in. diameter drains) on the computed in situ values of c_h (12).

5. The authors did not intend to give the impression that the driven method is given suitable than other methods in organic clay soils. The fact that the efficiency ratio was higher for all 3 drain types (at the 14-ft spacing) at the FRT site simply indicates that these organic clay soils are apparently less sensitive to the disturbance associated with drain installation at these drain spacings than the soils at the BCT site.

6. We judged the performance of the nondisplacement drains at the FRT site to be very good rather than "relatively mediocre," inasmuch as the efficiency ratio was 0.81 for both the augered and jetted methods.

7. No quantitative criteria for drain plumbness were given in the specifications for installation inasmuch as there is no practical, feasible way to measure the alignment of the in situ sand drain. It was specified, however, that the drains be located within a tolerance of 4 in. from design position at the ground surface and that the equipment be maintained in a plumb position to install "vertical" sand drains. The MSHC inspection personnel did, in fact, maintain very close checks on plumbness of the equipment during installation. The driven and augered drains were installed by using a crane with fixed leads, whereas the jetted type was not. The major reason for insisting on plumbness was, in this instance, the fact that subsequently piezometers were to be installed at locations that were supposed to be midway between adjacent drains.

8. The criterion for plumbness referred to by Landau (1 in. per 15 ft or 0.55 percent) is believed to be excessively strict and one that cannot be verified in a practical manner. Although it is certainly agreed that close tolerance with regard to plumbness is important, it is likely that there will always be an inherent random pattern of out-of-plumb drains within an overall area. That, we feel, is not nearly so important as the quality of installation of individual drains.

9. The guidelines and comments given at the end of the paper were obviously intended to be general in nature. Any specific test program would have to be developed with a great deal of study of the specific local conditions.

10. The authors are extremely pleased to learn that the HRB Committee on Embankments and Earth Slopes is sponsoring the development of a formal approach to the problem. We hope that the studies and comments presented here, plus other unpublished information that is available, will contribute in some way to such an undertaking.

Reference

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