PREFABRICATED VERTICAL-DRAIN TEST SECTION IN CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

By Timothy D. Stark,[1] Member, ASCE, Thomas A. Williamson,[2] Jack Fowler,[3] David Pezza,[4], and Yvonne Gibbons[5]

ABSTRACT: A 183 m by 122 m prefabricated vertical-drain test section was completed in February, 1993, in the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated vertical drains in consolidating the dredged fill and underlying foundation clay, thus increasing the storage capacity of the facility. The feasibility of installing prefabricated vertical drains was questionable because vertical drains had never been installed in an active dredged material management area; a drain length of approximately 50 m was close to the longest vertical drain ever installed, and the installation equipment could exert a ground pressure of only 10.3 kPa to operate on the surface of the soft dredged material. Results show that the dredged fill and foundation clay underwent substantial consolidation settlement (1.8 to 2.7 m in 30 months) after drain installation. In summary, prefabricated vertical drains appear to be an economical technique for increasing the storage capacity of active and inactive dredged material management areas.

INTRODUCTION

The Craney Island Dredged Material Management Area is a man-made 10 km^2 site with a storage area of approximately 8.9 km^2. Planned in the early 1940s, construction of Craney Island began in August 1954 and was completed in January 1957. Craney Island, located in Portsmouth, Virginia, near Norfolk, is the placement area for material dredged from the channels and ports in Virginia's Hampton Roads area.

The original design was for an initial capacity of about 76,400,000 m^3 at an annual dredging rate of 3,100,000–5,400,000 m^3. Based on an annual dredging rate of 3,800,000 m^3, Craney Island was designed for a service life of approximately 20 years (1957–1977). Continued dredging in the Norfolk channel has required the capacity of Craney Island to be increased through three major dike raising efforts. However, the dike setbacks used to prevent foundation instability have resulted in approximately 0.1–0.2 km^2 of lost storage capacity during each dike raising. After the third raising in 1992, the perimeter dikes were at their maximum height without inducing foundation instability.

Palermo and Schaefer (1980) conducted an extensive consolidation and desiccation analysis to predict the remaining service life of Craney Island. This study utilized the finite-strain consolidation microcomputer program PCDDF89 (Stark 1991) and concluded that the current capacity of Craney Island would be exhausted around the year 2000. As a result, the U.S. Army Engineer District in Norfolk began investigating new techniques for increasing the storage capacity of Craney Island.

One alternative was to reduce the volume of dredged material previously placed in Craney Island. Piezometers were installed in the perimeter dikes at Craney Island to investigate the pore-water pressures and degree of consolidation of the dredged material and underlying marine clay (Stark 1995). The piezometers revealed that large excess pore-water pressures existed in the marine clay. In some locations the total hydraulic head exceeded the ground surface level by 7.5 m. The dissipation of these excess pore-water pressures would result in substantial consolidation settlement, and thus increased storage capacity. In addition, consolidation of the marine clay and dredged fill would cause an increase in the undrained shear strength of these materials. This would allow the perimeter dikes to be constructed to higher elevations without setbacks or stability berms.

USE OF PREFABRICATED VERTICAL DRAINS TO INCREASE STORAGE CAPACITY

Fig. 1 shows a north-south cross section at the prefabricated vertical drain (PVD) test section (described subsequently) in the north compartment of Craney Island. It can be seen that the installation of vertical drains will result in radial flow as well as some vertical flow. Vertical drains reduce the maximum drainage path to one-half of the drain spacing (2 m) instead of one-half of the compressible layer thickness (46 m). This reduction in drainage path is extremely significant since the time rate of consolidation is approximately a function of the length of drainage path squared. This will yield a rapid increase in consolidation of the dredged fill and marine clay,

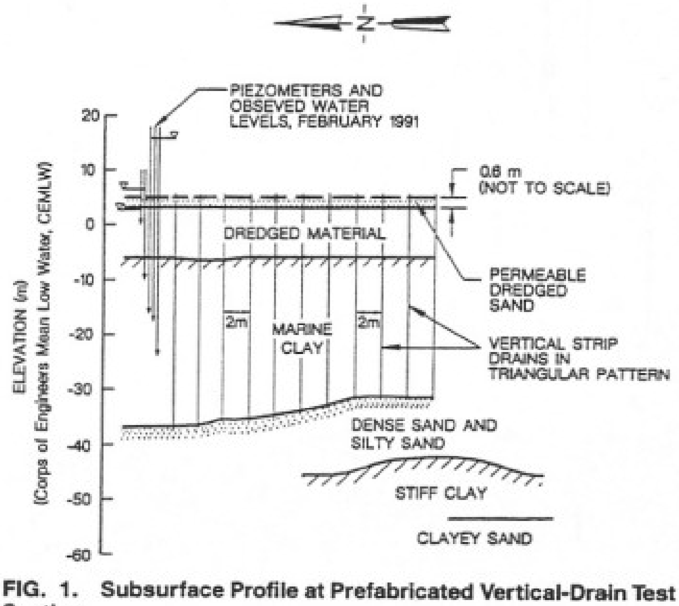

FIG. 1. Subsurface Profile at Prefabricated Vertical-Drain Test Section

[1]Assoc. Prof., Newmark Civ. Engrg. Lab, Univ. of Illinois, Urbana, IL 61801.
[2]Proj. Engr., GeoSyntec Consultants, Huntington Beach, CA 92648.
[3]GEOTEC Associates, Vicksburg, MS 39180.
[4]Geotech. Engrg. Sect., U.S. Army Corps of Engrs., Norfolk, VA 23510.
[5]Geotech. Engrg. Sect., U.S. Army Corps of Engrs., Norfolk, VA.

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which will increase the storage capacity of Craney Island. Additionally, the undrained shear strength of the dredged fill and marine clay will increase, allowing construction of higher dikes and therefore increased storage capacity.

PREFABRICATED-VERTICAL-DRAIN THEORIES

The design of PVDs is generally based on the theoretical solution for radial consolidation developed by Barron (1948) in which the drains are assumed to be of infinite permeability. Hansbo (1979 and 1981) simplified Barron's solution and accounted for well resistance and the effects of smear caused by drain installation. Yoshikuni and Nakanodo (1974), Onoue (1988), and Onoue et al. (1991) have presented rigorous solutions to the radial flow problem, ones that also account for the effects of smear and well resistance. However, these solutions are complicated and thus difficult to use in practice. Lo (1991) simplified the rigorous solutions, and the solution that follows is the result:

$$U_h = 1 - \exp\left[-\left(\frac{8 * C_h}{(d_e)^2 * [F(n, s) + 2.5 * G]} + \frac{4C_v}{H_{dr}^2}\right) t\right] \quad (1)$$

$$F(n, s) = \left(\frac{n^2}{n^2 - 1}\right) * \left[\ln\left(\frac{n}{s}\right) + \left(\frac{K_h}{K_s}\right)\ln(s) - 0.75\right] + \left(\frac{s^2}{n^2 - 1}\right)\left[1 - \left(\frac{s^2}{4n^2}\right)\right] + \frac{K_h}{K_s}\left(\frac{1}{n^2 - 1}\right) * \left[\frac{(s^4 - 1)}{4n^2} - s^2 + 1\right] \quad (2)$$

$$G = \left(\frac{K_h}{K_w}\right)\left(\frac{l_m}{d_w}\right)^2 = \left(\frac{K_h l_m^2}{\frac{4}{\pi} q_w}\right) = \left(\frac{\pi * K_h l_m^2}{4q_w}\right) \quad (3)$$

where U_h = average degree of consolidation for radial and vertical flow; C_h = horizontal coefficient of consolidation; C_v = vertical coefficient of consolidation; d_e = diameter of influence of vertical drain (triangular pattern = 1.05S where S = vertical drain spacing); H_{dr} = maximum length of vertical drainage path; t = time; $F(n,s)$ = term describing smear zones; n = ratio of drain diameters = d_e/d_w; s = ratio of smear-zone diameter to drain diameter = d_s/d_w; d_s = outer diameter of smear zone; K_h = horizontal coefficient of permeability of undisturbed soil; K_s = horizontal coefficient of permeability of smeared soil; G = term describing well resistance; K_w = coefficient of permeability of vertical drain; l_m = maximum drainage length of vertical drain; d_w = equivalent vertical-drain diameter = $[2(b + l_w)]/\pi$; b = width of vertical drain (typically 0.305–0.328 ft, used 0.31 ft); l_w = thickness of vertical drain (typically 0.01–0.013 ft, used 0.0115 ft); and q_w = discharge capacity of vertical drain = $(\pi/4)d_w^2 K_w$.

The difference between the solutions presented by Lo (1991) and Hansbo (1981) are the expressions for G and $F(n, s)$ and the effect of vertical flow on the rate of consolidation. It should be noted that Zeng and Xie (1989) also developed a simplified solution for the effect of well resistance that has a slightly different expression than Lo (1991). Review of several case histories (Mesri and Lo 1991; Mesri et al. 1994) has shown that the modifications presented by Lo (1991) provide excellent agreement with field case histories. The case histories also revealed that the importance of vertical drainage increases with increased spacing of the vertical drains.

FIELD TEST SECTION OBJECTIVES AND LAYOUT

A 183 m × 122 m field test section was constructed, instrumented, and monitored to evaluate the effectiveness of prefabricated vertical drains in consolidating the dredged fill and underlying marine clay at Craney Island. The test section was constructed in the north compartment of Craney Island, as shown in Figure 2. The north compartment was chosen for construction of the test section because of the presence of a well-developed desiccated crust. The north compartment also required the longest drains, which provided a good evaluation of the vertical drain equipment and a comparison between measured and predicted effects of smear zone and well resistance. The vertical-drain test section consists of two areas. The main area is 152 m × 122 m and was covered with a 0.6 m thick sand blanket to promote surface drainage and support the installation equipment. The prefabricated vertical drains were installed in a triangular pattern and pushed through the sand blanket to the underlying dense sands. It can be seen that the bottom of the marine clay is located at approximately El. -36 m CEMLW (see Fig. 1) because of the presence of an old river channel. CEMLW refers to the Corps of Engineers Mean Low Water, which is 0.6 m below national geodetic vertical datum and 0.2 m below MLW (National Ocean Survey). Typically the bottom of the marine clay is located at approximately El. -31 m CEMLW.

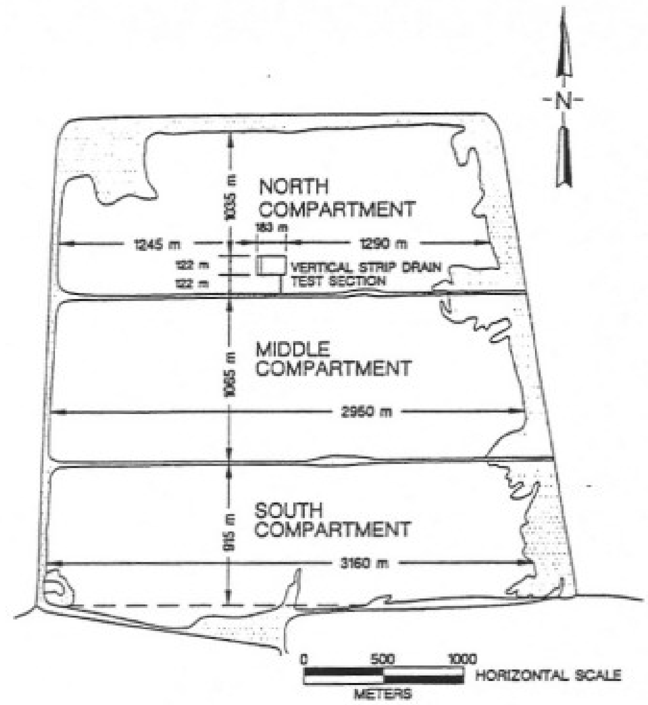

FIG. 2. Plan View of Craney Island and Location of Vertical-Drain Test Section

The mobility test section was 30 m × 122 m and used prefabricated horizontal drains to promote surface drainage. The main objective of the adjacent mobility section was to determine whether or not a sand blanket was required to install vertical drains throughout the remainder of the management area. As a result, the 15 cm to 30 cm thick desiccated crust in this area had to support the installation equipment. Bearing capacity calculations indicated that a maximum ground pressure less than or equal to 10.3 kPa would be required to operate on the crust. The drains were installed using a triangular pattern and in an east-west direction.

Initial excess pore-water pressures were estimated from the installed piezometers, piezocone dissipation tests, and preconsolidation pressure prior to drain installation. The distribution of excess pore-water pressure indicated that the marine clay was under-consolidated and the underlying dense sand is freely draining (Fig. 1). The measured pore-water pressures and cone penetration data indicated that the dredged fill was at least partially drained because of the presence of sand seams, as described subsequently.

INITIAL UNDRAINED SHEAR STRENGTH

The existing undrained shear strength, Su, profile in the test section was estimated using a number of techniques. The first technique described utilizes the tip resistance from cone penetration tests and the following equation:

$$Su = \frac{q_c - \sigma_{vo}}{N_k} \tag{4}$$

where q_c = cone tip resistance; σ_{vo}= total vertical overburden pressure; and the denominator or empirical cone factor can be based on field vane shear tests, N_k, (Lunne and Kleven 1981; Meigh 1987) or on unconsolidated-undrained (UU) triaxial compression tests, N_{kUU}, (Stark and Delashaw 1990). The empirical correlations of N_k and N_{kUU} utilize the plasticity index (PI) to estimate values of cone factor.

Table 1 presents the index properties of the marine clay at Craney Island. The statistical values of the index properties were determined from the results of 135 tests (Ishibashi et al. 1993). Since the dredged material is similar to the foundation clay the same index properties were used for both deposits.

The value of N_k for a PI of 41 ranges from 10 to 15, while the value of N_{kUU} ranges from 8 to 14. Since field vane shear test data was not available to estimate a site-specific N_k value, an average value of N_k equal to 12 was used in the analysis for comparison purposes. In addition, this average value of N_k is the same as the average value of N_{kUU} (12) for the 18 sites considered by Stark and Delashaw (1990). Fig. 3 presents the variation of undrained shear strength with depth using N_k equal to 12. Each data point corresponds to a calculation of Su using (4), the appropriate total stress, and a value of N_k equal to 12.

TABLE 1. Summary of Index Properties of Marine Clay (after Ishibashi et al. 1993)

Variable (1)	Natural water content (%) (2)	Liquid limit (3)	Plastic limit (4)	Plasticity index (5)	Clay size fraction (%) (6)	Specific gravity of solids (7)
Average	70.2	70.7	29.3	41.4	94.4	2.71
Standard deviation	12.4	14.7	4.88	12.3	7.25	0.04
Coefficient of variation	0.17	0.21	0.17	0.3	0.04	0.02

From Fig. 3 several interesting facts can be ascertained concerning the undrained shear strength. First, variability exists in tip resistance measurements in the dredged material; this indicates that the dredged material contains many sand/silt seams. This explains the lack of excess pore-water pressures measured in the piezocone dissipation tests and piezometers in the dredged fill, as shown in Fig. 1. The dredged fill is probably undergoing self-weight consolidation and the excess pore-water pressures are being dissipated by the sand/silt seams. Based on this conclusion, the majority of the settlement measured in the test section was attributed to consolidation of the marine clay. The dredged fill appears to be undergoing self-weight consolidation and acting as a surcharge for the marine clay.

Secondly, the marine clay appears to be under- or normally consolidated. This is evident by the smoothness of the Su profile and slight increase in Su with depth. In addition, it appears that the sand underlying the marine clay is free-draining, because the values of Su increase near the bottom of the marine clay. In fact, the value of Su near the bottom of the marine clay corresponds to the effective stress at 100% consolidation.

UNDRAINED SHEAR STRENGTH RATIO

The undrained strength ratio (Su/σ'_p where σ'_p is preconsolidation pressure) of the marine clay was estimated from historic field vane shear (FV), unconsolidated-undrained triaxial compression (UU), unconfined compression (UC), and isotropically consolidated-undrained triaxial compression (CU) test results collected since 1948 (U.S. Army 1949, 1986). Table 2 summarizes the values of undrained strength ratio esti-

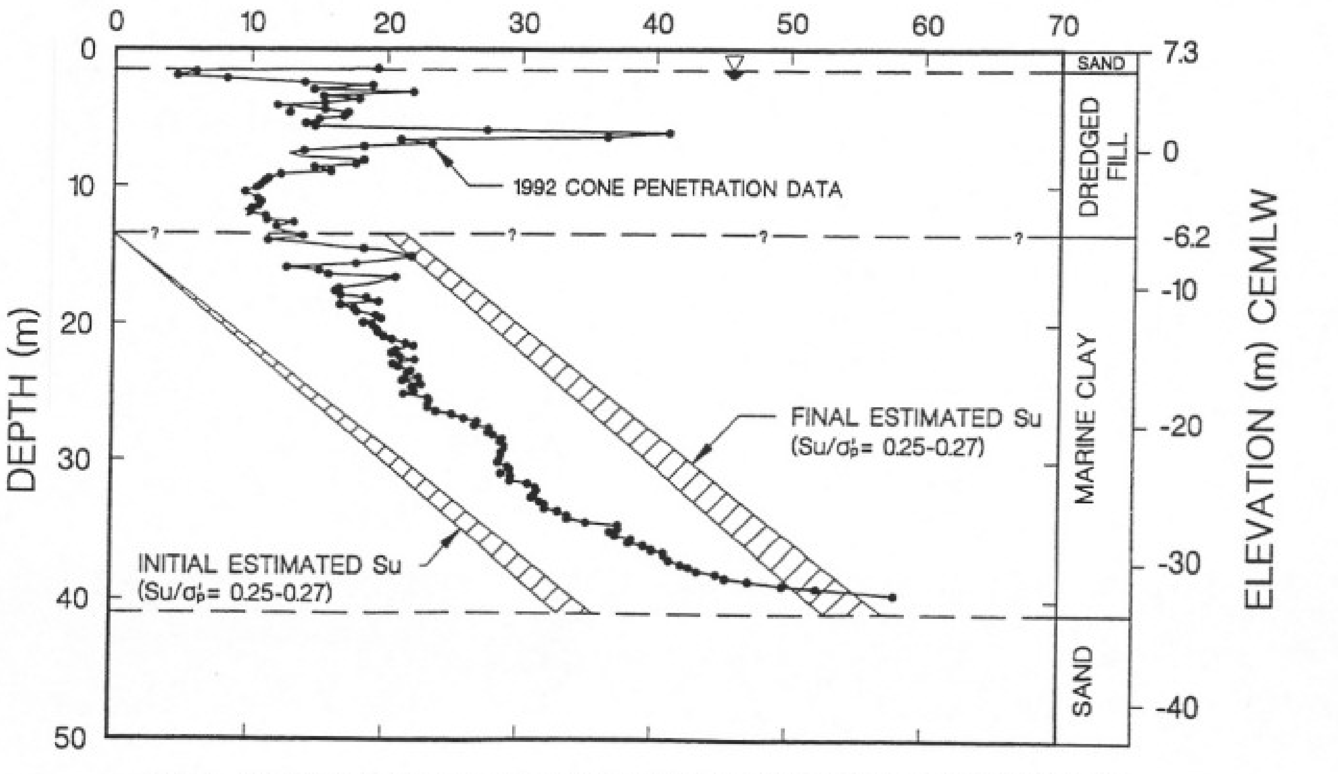

FIG. 3. Undrained Shear Strength versus Depth in Craney Island Vertical-Drain Test Section

TABLE 2. Undrained Strength Ratios for Marine Clay from Various Test Methods (after Ishibashi et al. 1993)

Test method (1)	Number of measurements (2)	Average Su/σ'_p (3)	Standard deviation (4)	Coefficient of variation (5)
FV	102	0.26	0.04	0.16
UU	55	0.24[a]	0.13	0.46
UC	56	0.28	0.16	0.55
CU	10	0.27	0.05	0.17

[a]Five values of undrained strength ratio higher than 0.7 were omitted.

mated from the tests reported in the General Design Memorandums for Craney Island. Table 2 shows that the undrained strength ratio ranges from 0.24 to 0.28 with an average of 0.26.

For comparison purposes, the undrained strength ratio also was estimated using published correlations, an average plasticity index of 41, and the field vane mode of shear and the ratio ranges from 0.25 to 0.27 with an average of approximately 0.26 (Mesri 1989). Since this report is describing the increase in undrained strength below the test section, a range of undrained strength ratio of 0.25–0.27 was used in Fig. 3 and an average ratio of 0.26 was used in the analysis described herein.

The undrained strength ratio was used to estimate the Su profile using a measured unit weight of 14.6 kN/m^3 (Ishibashi et al. 1993) and by assuming that the marine clay is saturated and normally consolidated. Fig. 3 shows the initial estimated Su profile, which corresponds to the Su profile prior to dredged fill disposal (i.e., 1956) and thus starts at the original mudline of El. -6.2 m CEMLW. Undrained strength ratios of 0.25 and 0.27 were also used to estimate the increase in Su that will result from installation of prefabricated vertical drains, and thus 100% consolidation of the marine clay (Fig. 3). A unit weight of 15.4 kN/m^3 was used to estimate the undrained shear strength after 100% consolidation. A comparison of the final estimated profile and the profile estimated from the 1992 cone penetration tests suggests that the marine clay for depths ranging from 20 m to 35 m is under-consolidated.

It can be seen that the marine clay will probably undergo a substantial increase in undrained shear strength, especially between depths of 20 m and 35 m, due to consolidation. Between 1956 and 1992 a strength gain of only 30–35% occurred in the marine clay because of the slow rate of consolidation. As a result, an increase in Su of 65–70% is expected when the marine clay achieves a degree of consolidation of approximately 100%.

INITIAL WATER CONTENT PROFILE

One boring was drilled at the center of the test section in March 1993. Samples were obtained with a piston tube sampler from the boring every 3 m to a depth of approximately 37 m. Natural water contents were determined for the recovered samples and are compared with the plastic and liquid limits of the samples. The water contents of the dredged fill and marine clay were at or near the liquid limit. In addition, the pre-1956 (U.S. Army 1949) and 1993 water content profiles are similar. This also indicates that minimal consolidation had occurred in the marine clay between 1956 and 1993.

Void ratios were determined from the water content data by assuming a degree of saturation of 100% and a specific gravity of soil mass equal to 2.71. The dredged fill exhibited void ratios of 2 to 3 and considerably more scatter than the marine clay, which had a void ratio of approximately 2.5.

TABLE 3. Estimated Values of C_v and C_h for Dredged Fill and Marine Clay

Source of data (1)	C_h (m^2/day) (2)	C_v (m^2/day) (3)
Dredge fill data		
Field piezometers (Stark 1995)	3.7×10^{-2}	3.0×10^{-2}
Cargill (1983)	1.1×10^{-2}	8.8×10^{-3}
Marine clay data		
Field piezometers (Stark 1995)	1.4×10^{-2}	1.1×10^{-2}
Design memorandums (U.S. Army 1949, 1986)	1.9×10^{-3}	1.5×10^{-3}
Empirical correlations (U.S. Navy 1982)	6.5×10^{-3}	5.2×10^{-3}
Design parameters	1.1×10^{-2}	8.6×10^{-3}

COMPRESSION INDEX

Two separate methods for estimating the compression index (C_c) for the marine clay were considered. The compression index was used to estimate the magnitude of consolidation settlement after drain installation. Headquarters (1990) presents the following empirical correlation for clay of medium to low sensitivity:

$$C_c = 0.01 * (\text{LL} - 13\%) \quad (5)$$

This equation and a liquid limit (LL) of 71 (Table 1) were used to estimate a value of C_c equal to 0.58. Data from oedometer tests on the marine clay showed C_c ranges from 0.41 to 0.79 for the majority of the data, with an average or representative value of 0.58. This range of C_c is used in a subsequent section to estimate the consolidation settlement induced by installation of vertical drains.

COEFFICIENT OF CONSOLIDATION

Vertical-drain spacing is governed by the horizontal (C_h) and vertical (C_v) coefficients of consolidation. It can be seen from Fig. 1 that vertical drains will penetrate the dredged fill and marine clay which have different hydraulic conductivities. These soil types are similar, but the void ratio of the dredged fill is larger than the marine clay. This results in a higher hydraulic conductivity and coefficient of consolidation for the dredged fill than the marine clay. The results of the subsurface investigation were used to estimate design values of C_h and C_v for the dredged fill and marine clay.

Based on the data presented by Mesri and Lo (1991), it was assumed that C_v could be estimated by dividing C_h by an average ratio of 1.25 for the soft marine clay and dredged fill. From Table 3 it can be seen that the values of C_h and C_v are uncertain. To facilitate the design of the test section, it was decided to treat the dredged fill and marine clay as a single homogeneous layer and to use an average value of C_h and C_v. For design purposes, it was decided to use a weighted average value of C_h and C_v based on the thickness of the dredged fill and marine clay. The estimated average values of C_h and C_v were equal to 1.1×10^{-2} and 8.6×10^{-3} m^2/day, respectively, and were used to determine the preliminary spacing of the vertical drains.

VERTICAL-DRAIN DESIGN PARAMETERS

The other major parameters required to develop an estimate of prefabricated-vertical-drain spacing are the well resistance and the extent of the smear zone. It can be seen from (1)–(3) that the well resistance is governed by the ratio of K_h/K_w or K_h/q_w. Using field case histories, Lo (1991) showed that the effect of well resistance can be neglected if the parameter G is less than 0.2. Typical values of vertical drain discharge capacity, q_w, range from 5.7 to 11.3 m^3/day (Koerner 1994).

Since the consolidating clay is doubly drained, the maximum drainage length of the vertical drain in the test section area (l_m) is equal to 22 m. This value of l_m also equals the maximum length of vertical drainage path (H_{dr}) in the clay. Using these parameters, an average value of q_w equal to 8.5 m^3/day, and the average horizontal hydraulic conductivity measured in the field piezometers, the value of G ranges from 0.06 to 0.03. Therefore, well resistance may be neglected if the field discharge capacity of the vertical drains is greater than 8.5 m^3/day.

The radial extent of the smear zone was studied using laboratory model tests by Onoue et al. (1991) and experience from pile driving and sand drain installations. This study revealed that the ratio of smear zone diameter to strip drain diameter, d_s/d_w, varies from 1.6 to 4.0. For design purposes the ratio of d_s/d_w was assumed to be 2. In addition, the horizontal hydraulic conductivity in the smear zone, K_s, was assumed to be one-half of the undisturbed hydraulic conductivity, K_h. This assumption is based on data presented by Onoue et al. (1991) that showed the ratio of K_s/K_h ranged from 0.2 to 1.0 in the smear zone.

DESIGN OF VERTICAL-DRAIN TEST SECTION

The major design constraints for the test section were cost and the time required for 90% consolidation. Using the design theory presented by Lo (1991) and the design parameters previously described, and presented in Table 4, a diameter of influence of a vertical drain, d_e, equal to 2.3 m was required to obtain a degree of consolidation of 90% in the dredged fill and foundation clay within one year. The value of d_e is obtained by an iterative process in which values of d_e are selected until (1) yields a degree of consolidation of 90%. A preliminary vertical-drain spacing for a triangular pattern was calculated to be 2.2 m by dividing the diameter of influence of the vertical drain, d_e, by 1.05.

TABLE 4. Prefabricated Vertical-Drain Test Section Design Parameters

Parameter (1)	Value (2)
Degree of consolidation	90%
Time	1 year
K_h	7.3×10^{-4} m/day
C_v	8.6×10^{-3} m^2/day
C_h	1.1×10^{-2} m^2/day
H_{dr}	22 m
q_w	8.5 m^3/day
l_m	22 m
d_s/d_w	2.0
K_h/K_s	2.0

VERTICAL-DRAIN INSTALLATION EQUIPMENT

Prefabricated vertical drains were installed in the test section using a novel piece of equipment. The equipment minimized disturbance to the sand blanket, confined dredged material, and the underlying marine clay during the installation operation. The vertical-drain installation equipment was placed on track-mounted pontoons (2.1 m wide and 10.7 m long) to reduce the maximum contact pressure to less than or equal to 10.3 kPa and to minimize disturbance. This would enable the equipment to operate on the 15–30 cm thick desiccated crust in the mobility test section. The ground pressure exerted by this equipment was only 9.7 kPa, which resulted in the equipment experiencing little, if any, difficulty operating on the desiccated crust. The cross-sectional area of the mandrel was restricted to 6.5×10^{-3} m^2 to reduce soil disturbance during drain installation. However, it should be noted that the cross-sectional area of the mandrel was still considerably larger than the cross-sectional area of the strip (6.0×10^{-4} m^2).

Vertical drain installation in the test section began on December 21, 1992, and was completed on February 26, 1993. The total number of drains installed in the main and mobility test sections was 5,557. Approximately 193,820 lineal meters of vertical drain were installed in the main test section and 40,755 lineal meters of vertical drain were installed in the mobility test section. In the mobility section 2,181 lineal meters of horizontal drain were installed.

TEST SECTION PERFORMANCE

Immediately following installation of the prefabricated vertical drains, water could be seen rising along the drainage core

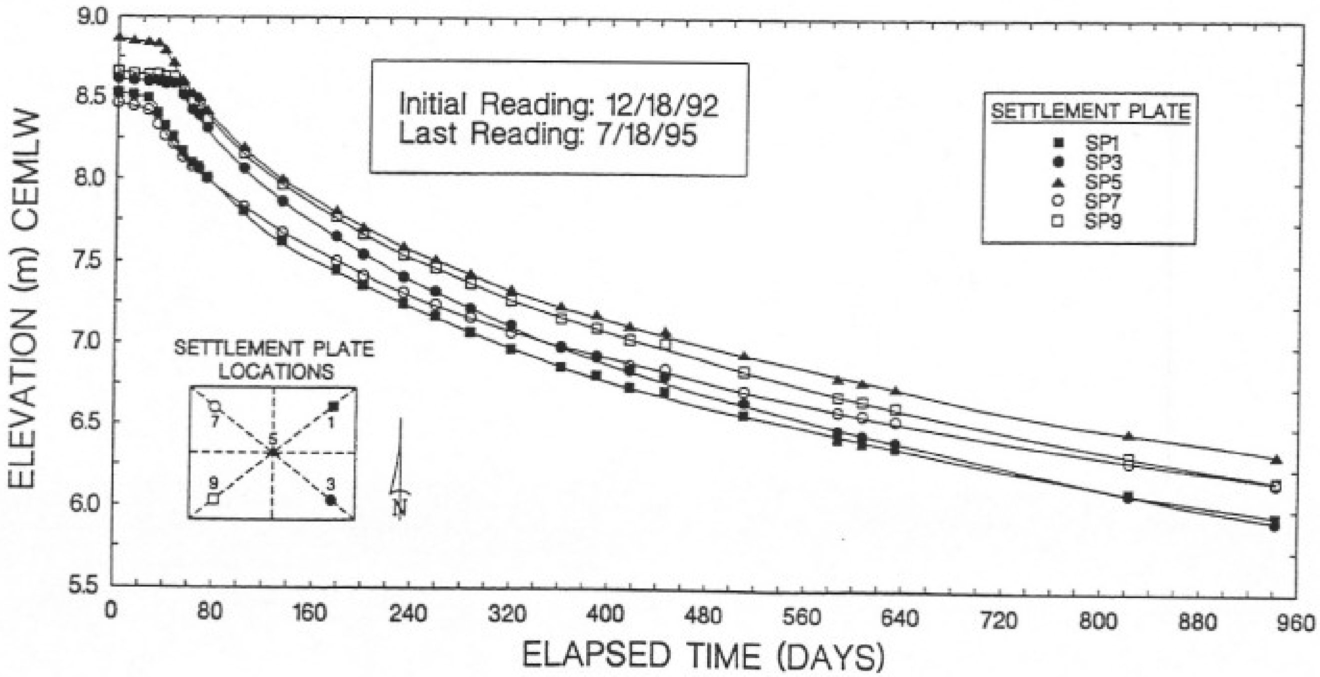

FIG. 4. Settlement Plate Measurements in Main Test Section

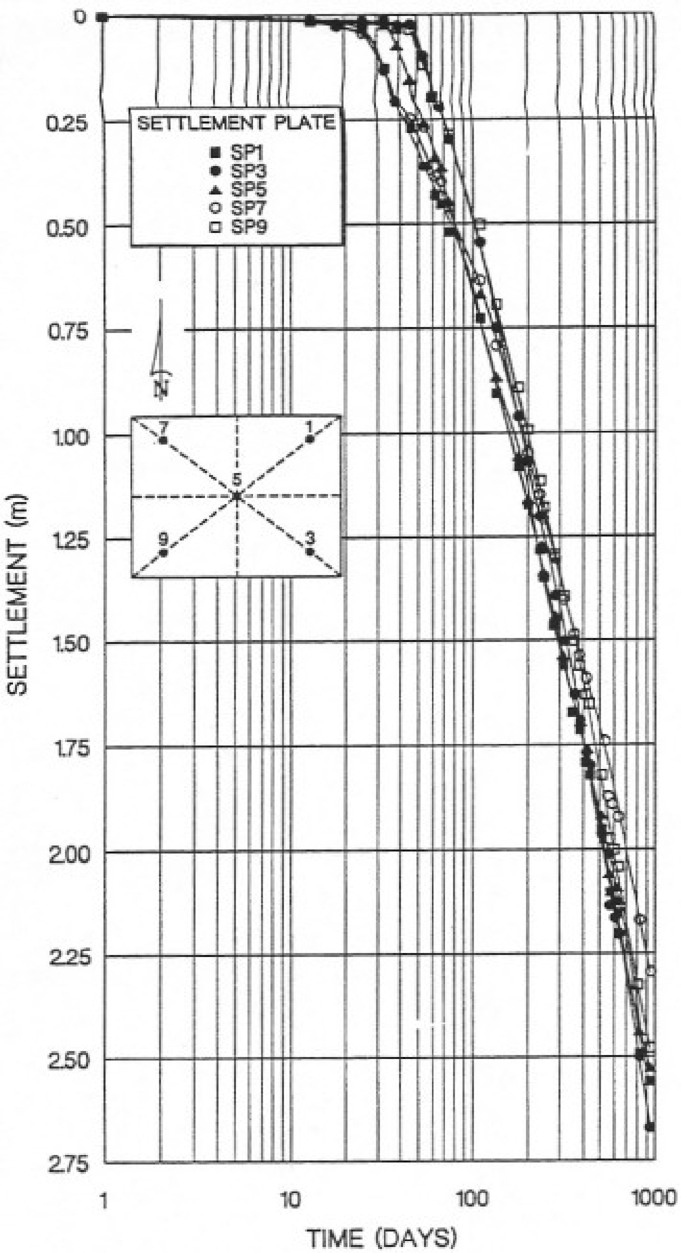

FIG. 5. Semilogarithmic Presentation of Settlement Plate Measurements in Main Test Section

and around the outside of the vertical drain. The water rising around the drain was caused by the void left by the mandrel after retraction. Within 10–15 minutes after installation, water rose 0.05–0.15 m above the ground surface inside the drain.

Measured Settlements in Main Test Section

Settlement plate readings for the main test section are presented in Fig. 4. Installation of the vertical drains in the test section was completed on February 26, 1993. On July 18, 1995, approximately 30 months after drain installation, the maximum consolidation settlement in the main test section ranged from 2.3 to 2.7 m. This was the last settlement reading because new dredged material was pumped into the north compartment from August 21, 1995, through December 31, 1995. The new dredged material covered the settlement plates, which precluded further measurements. The measured settlements are in agreement with the predicted value of 2.7–2.9 m, which is based on an average of C_c of 0.58 and a degree of consolidation of 100%. The measured settlements are also within the predicted range (1.5–3.0 m) for values of C_c equal to 0.41 and 0.79.

Fig. 5 presents the settlement plate data from the main test section using a semilogarithmic scale. It can be seen that none of the settlement plates indicate that primary consolidation was completed. However, it appears that the estimated settlements are in agreement with field measurements. A final conclusion on the accuracy of the estimated settlements will probably not be known because new dredged material was pumped into the north compartment on August 21, 1995.

For design purposes, it can be assumed that the north compartment would settle at least between 2.7 and 2.9 m if a sand blanket and strip drains are installed and approximately 100% consolidation is allowed to occur. If consolidation settlements are to be estimated for the center and south compartments or the perimeter dikes, it is recommended that a value of C_c equal to 0.71 be used for estimating the final consolidation settlement. This value of C_c was back-calculated from measured settlements.

Measured Settlements in Mobility Test Section

The mobility test section was developed to demonstrate that a sand blanket was not required to support the vertical drain equipment. A comparison of Figs. 4 and 6 provides an insight into the effect of the sand blanket on the consolidation settlement of the dredged fill and marine clay. It can be seen that settlement plate SP-10 is located at the northern end of the adjacent mobility section and can be compared with settlement plates SP-1 and SP-7 at the northern end of the main section. Settlement plates SP-1 and SP-7 have settled 2.5 m to 2.3 m, respectively, while settlement plate SP-10 has settled only 1.85 m. Therefore, it may be concluded that the additional surcharge provided by the sand blanket results in a significant increase in consolidation settlement (0.45–0.65 m). It is anticipated that the additional consolidation primarily occurred in the dredged fill because of the compressible nature of the dredged material and the limited extent of the sand blanket.

In summary, the storage capacity lost by the installation of a sand blanket can probably be recouped by the subsequent consolidation of the underlying dredged fill. However, the cost of the sand blanket and the ability of the vertical drain equipment to operate without the sand blanket may preclude the use of a sand blanket throughout the remainder of the placement area.

Fig. 7 presents the settlement plate data from the mobility test section using a semilogarithmic scale. It can be seen that none of the settlement plates indicate that primary consolidation was completed, and the measured settlements range from 1.75 to approximately 1.85 m. The measured settlements agree with the estimated settlements. The average settlement was estimated to be 1.7 m for 100% consolidation and an average C_c value of 0.58. The estimated range of consolidation settlement is 0.9–2.4 m for values of C_c equal to 0.41 and 0.79, respectively. The measured consolidation settlement was also used to back-calculate the value of C_c for the dredged material and marine clay in the mobility test section. It was found that the back-calculated value of C_c equal to 0.71 from the main test section yields a settlement of 2.3 m, which is in agreement with field measurements in the mobility section.

In summary, the mobility test section will probably settle between 2.1 and 2.3 m, which indicates that this section would have undergone some additional settlement if dredged material was not pumped into the area in August, 1995. However, it can be assumed that the north compartment will settle between 2.1 to 2.3 m without a sand blanket after vertical drains are installed. For comparison purposes, the measured settlement rates in the north compartment prior to vertical-drain installation ranged from 0.11 to 0.12 m/year from 1991 to 1994.

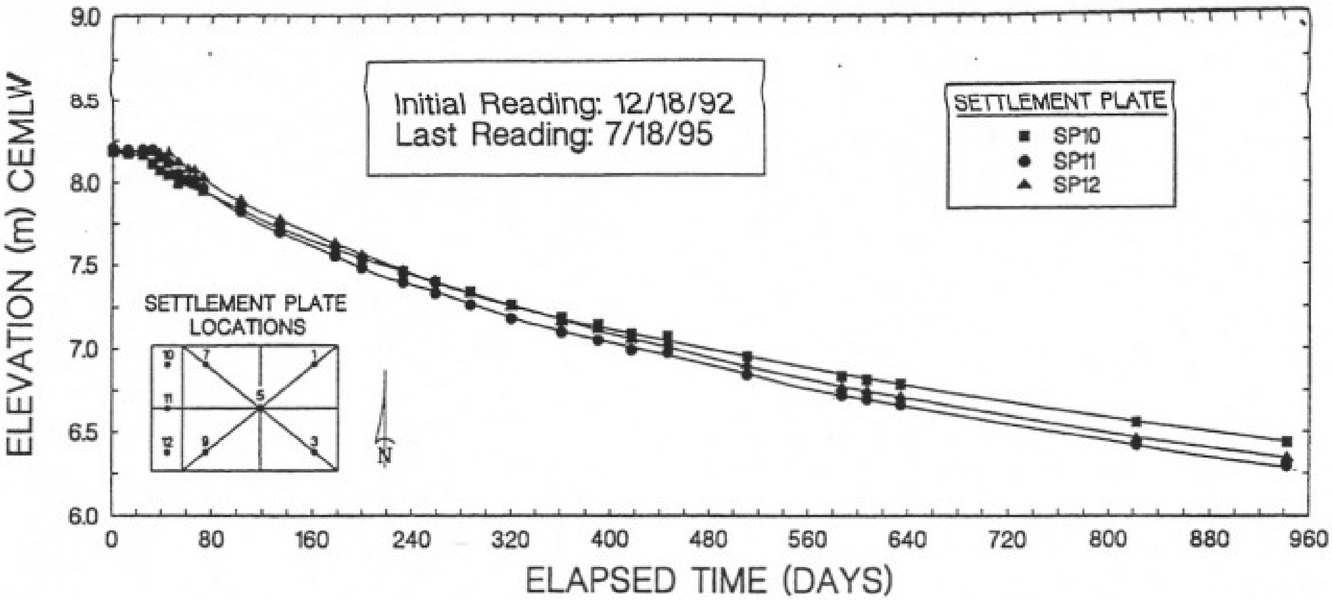

FIG. 6. Settlement Plate Measurements in Mobility Test Section

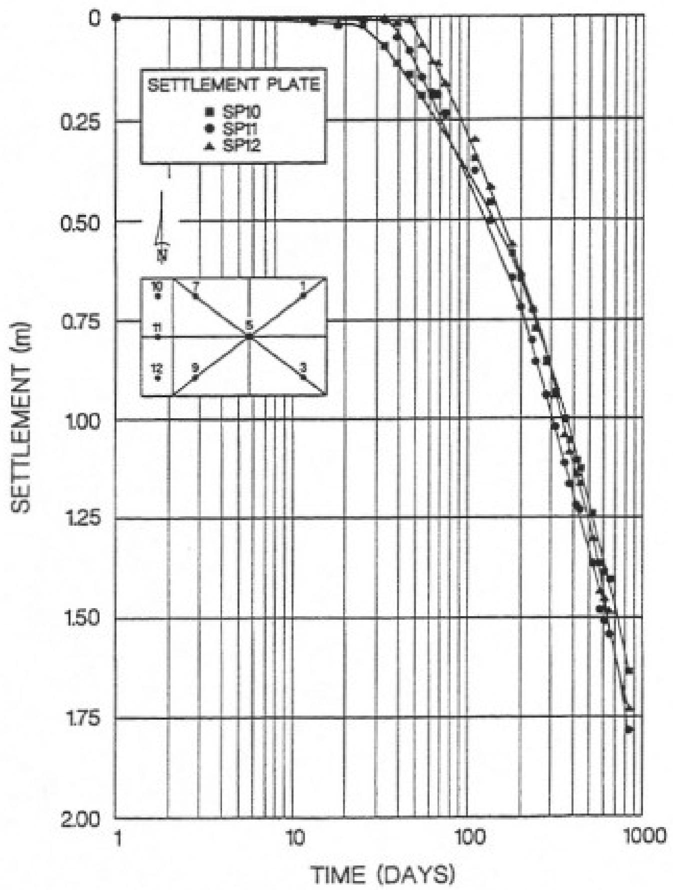

FIG. 7. Semilogarithmic Presentation of Settlement Plate Measurements in Mobility Section

Thus, the installation of prefabricated vertical drains significantly accelerated the rate of consolidation.

Time Rate of Consolidation

A vertical-drain spacing of 2.2 m should have resulted in 90% consolidation in 12–13 months, based on the vertical-drain design parameters in Table 4. Since the consolidation settlement was still occurring about 30 months after drain installation, one or more of the design parameters did not model the field conditions. Several possible explanations for this discrepancy are:

1. The prefabricated vertical drain, Amerdrain 407, did not exhibit a field discharge capacity of greater than or equal to 8.5 m^3/day.
2. The mandrel insertion created a larger smear zone. It should be noted again that the cross-sectional area of the vertical drain and mandrel are 6.0×10^{-4} m^2 and 6.5×10^{-3} m^2, respectively. The ratio of smear zone diameter (d_s) to vertical drain diameter (d_w) usually varies from 1.6 to 4.0. For design purposes this ratio was assumed to be 2.0.
3. The horizontal coefficient of consolidation is lower than 1.1×10^{-2} m^2/day. This could be caused by a larger smear zone and/or variability in the field piezometer data, oedometer test results in the Design Memorandums, or the empirical correlation presented in the Navy Design Manual DM-7.1 (U.S. Navy 1982).
4. The vertical drain is not acting as doubly drained. This could be caused by the drain not being anchored into the underlying sand or by the large earth pressure at a depth of approximately 50 m significantly reducing the discharge capacity of the drain. If the drain is not doubly drained, the time required for 90% consolidation will increase slightly because the maximum drainage length l_m in (1) and (3) will double.
5. Some of the settlement after 30 months was occurring due to secondary compression and not consolidation.

Fig. 8 presents the measured and estimated consolidation settlement versus time for the main test section. The estimated relationships were obtained using the design parameters in Table 4 and the vertical-drain theory presented in (1)–(3). The range in time rate of settlement was estimated using the degree of consolidation calculated using (1)–(3) and final consolidation settlements (1.5 and 3.0 m) that correspond to C_c equal to 0.41 and 0.79, respectively. The measured settlements correspond to settlement plate SP-5, which is located at the center of the main test section. It was decided that the center of the main test section and the accompanying measured settlements (Fig. 4) are representative of the time rate of consolidation of

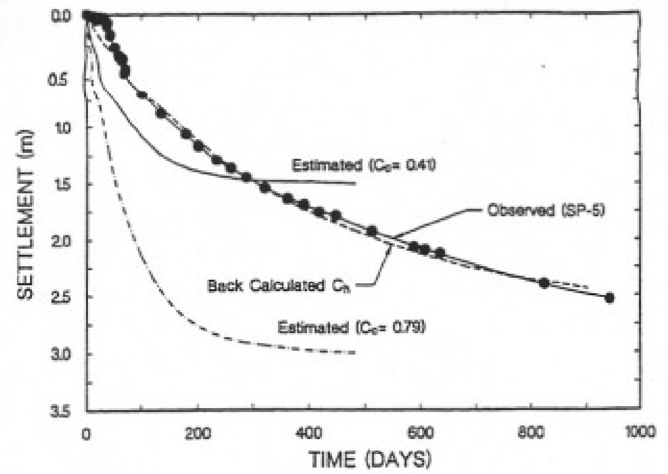

FIG. 8. Measured and Estimated Time Rate of Consolidation Settlement for Main Test Section

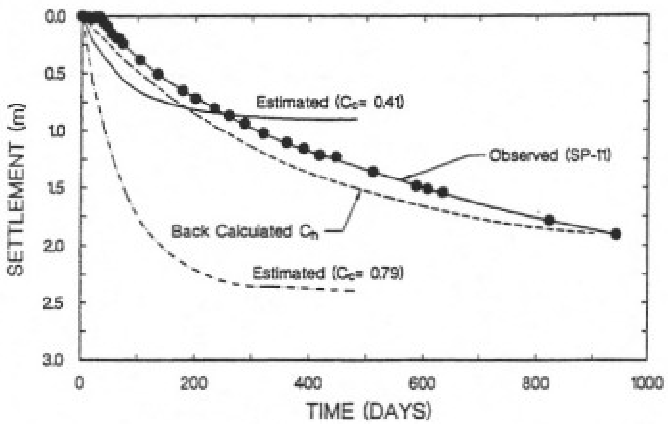

FIG. 9. Measured and Estimated Time Rate of Consolidation Settlement for Mobility Test Section

the main test section, and were thus compared to the estimated rates of consolidation.

It can be seen in Fig. 8 that the estimated time rates of settlement are not in agreement with the measured values. This was expected because the vertical-drain spacing of 2.2 m was designed to achieve 90% consolidation in 12–13 months and the test section was still settling 30 months after vertical-drain installation.

Fig. 9 presents the measured and estimated consolidation settlement versus time for the mobility test section. The estimated relationships were obtained using the design parameters in Table 4 and the vertical-drain theory presented in (1)–(3). The measured settlements correspond to settlement plate SP-11, which is located at the center of the mobility test section. It can be seen that the estimated time rates of settlement are also not in agreement with the measured values in the mobility test section.

The measured time rate of settlements in Figs. 8 and 9 were used to back-calculate vertical-drain design parameters to aid future time rate of consolidation predictions at Craney Island. Eqs. (1)–(3) show that the degree of consolidation for radial flow depends on a number of parameters. A parametric study revealed that the degree of consolidation is significantly influenced by the value of C_h. As a result, the parameters in Table 4 were used to back-calculate the mobilized or field value of C_h using the theory in (1)–(3). A mobilized value of C_h equal to 1.3×10^{-3} m^2/day was calculated for the main and mobility sections. This value is significantly lower than the design value of 1.1×10^{-2} m^2/day (Table 4). This helps to explain why the test section did not reach 100% consolidation after 12–13 months as designed.

In summary, it is recommended that a value of C_h equal to 1.3×10^{-3} m^2/day be used for future vertical-drain design at Craney Island. However, it should be noted that this mobilized value of C_h reflects uncertainties in all of the design parameters in Table 4, e.g., drain discharge capacity, single versus double drainage, and extent of the smear zone. Therefore, this mobilized value of C_h represents the field value for the vertical-drain equipment, installation procedure, and type of drain used in this test section. If the same or similar equipment, installation procedure, and prefabricated drain are used, this value of C_h and the remaining values in Table 4 can be used for design purposes.

Excess Pore-Water Pressures

The piezometric data did not show a significant decrease in pore-water pressure even though substantial settlement had occurred at the test section. This trend has been noted by other researchers. For example, Hansbo et al. (1982) showed that an increase in undrained shear strength was observed in several case histories with a negligible change in excess pore-water pressure. Mesri and Choi (1979) showed that when the effective vertical stress approaches the preconsolidation pressure, settlement continues at a nearly constant value of excess pore-water pressure. The dredged material and marine clay are under- or normally consolidated so the effective vertical stress is the preconsolidation pressure. This is probably the cause of the small decrease observed in the piezometric data.

Based on these results and the data presented by Mesri and Choi (1979), it is recommended that subsequent prefabricated vertical-drain test sections in dredged material and normally consolidated clay rely more on settlement plate measurements, settlement points with depth, changes in water content or void ratio, and/or changes in cone penetration resistance than on pore-water pressure measurements, to evaluate the effectiveness of vertical drains. However, the cone penetrometer must be able to measure small changes in tip resistance to illustrate small increases in tip resistance.

CONCLUSIONS

A 183 m × 122 m prefabricated vertical-drain test section was completed in February, 1993, in the north compartment of the Craney Island Dredged Material Management Area near Norfolk, Virginia. The test section was constructed to evaluate the effectiveness of prefabricated vertical drains in consolidating the dredged fill and underlying marine clay, thereby increasing the storage capacity of the facility. The feasibility of installing prefabricated vertical drains was questionable because drains had never been installed in an active dredged material management area; a drain length of approximately 50 m was close to the longest vertical drain ever installed, and the installation equipment had to operate directly on the surface of the soft dredged material.

Settlement plates installed in the main test section settled approximately 2.3–2.7 m in 30 months (0.9–1.1 m/year). The mobility test section settled 1.75–1.85 m in 30 months (0.7–0.8 m/year). These consolidation settlements are in agreement with the estimated values. For comparison purposes, the measured settlement rates in the north compartment prior to vertical-drain placement ranged from 0.11 to 0.12 m/year from 1991 to 1994. Thus, the installation of prefabricated vertical drains significantly accelerated the rate of consolidation.

The measured settlements were also used to estimate mobilized or field values of C_c and C_h. These mobilized values (C_c = 0.71 and C_h = 1.3×10^{-3} m^2/day) should be used to design future vertical-drain installations at Craney Island that utilize similar equipment, installation procedure, and prefabricated vertical drains.

In summary, the Craney Island test section showed that prefabricated vertical drains are an effective technique for increasing the storage capacity, and thus service life, of confined dredged material management areas. This technique appears to be applicable to many management areas around the country.

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