STRIP DRAIN TEST SECTION IN CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

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ABSTRACT: A 183 m by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and underlying foundation clay, and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since strip drains had never been installed in an active dredged material management area, a drain length of approximately 50 m was close to the longest strip drain ever installed, and the installation equipment could only exert a ground pressure of 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 2.6 m in 30 months). In summary, prefabricated strip drains appear to be an economical technique for increasing the storage capacity of active and inactive dredged material management areas. It is anticipated that the strip drains will continue to function as additional dredged material is placed in the management area, and thus increase storage capacity in the future.

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 1940's, construction of Craney Island began in August 1954 and was completed in January 1957. Craney Island is the long-term management area for material dredged from the channels and ports in the Hampton Roads area located near Norfolk, Virginia, in Portsmouth, Virginia.

Dredged material has been placed in the management area almost continuously since it was completed in 1957. The original design was for an initial capacity of about 76.4 million m^3 at an annual dredging rate of 3.1 to 5.4 million m^3. Based on an annual dredging rate of 3.8 million m^3, Craney Island was designed for a service life of approximately 20-years (1957 to 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 km^2 to 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.

The Environmental Laboratory at the US Army Engineer Waterways Experiment Station in Vicksburg, Mississippi conducted an extensive consolidation and desiccation analysis to predict the remaining service life of Craney Island (Palermo and Schaefer 1990). This study utilized the finite strain consolidation microcomputer program PCDDF (Stark 1991; Stark and O'Meara 1991) and revealed that the current capacity of Craney Island will be exhausted around the year 2000. As a result, the US Army Engineer District, Norfolk (CENAO) began investigating new techniques for increasing the storage capacity of Craney Island.

One alternative was expanding Craney Island. Six expansion configurations were considered but in 1991 the Virginia State Legislature ruled that Craney Island could not be expanded or replaced at that time. Therefore, the feasibility of restricting the usage of Craney Island to placement of unsuitable dredged material and ocean placing the suitable material was investigated (Myers et al. 1993). The cost difference between placement at Craney Island and ocean placement is approximately $44.8 million per year. In addition, the environmental impact of ocean placing approximately 3 million m^3 of dredged material per year would require substantial study. As a result, other alternatives for increasing the storage capacity of Craney Island were sought. 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 marine clay and dredged material (Stark 1995). The piezometers revealed that large excess pore-water pressures, some exceeding the ground surface by 7.5 m, existed in the marine clay. The dissipation of these excess pore-water pressures would result in substantial consolidation settlement, and thus increase 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 Strip Drains to Increase Storage Capacity

In the last 10 to 15 years, vertical strip drains have been used to accelerate consolidation of soft cohesive soils because the drains allow radial flow as well as some vertical flow. Figure 1 shows a north-south cross-section under the strip drain test section (described subsequently) in the north compartment of Craney Island. It can be seen that the installation of vertical strip drains will result in radial flow as well as some vertical flow. Strip drains reduce the maximum drainage path to one-half of the strip 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 a function of the length of drainage path squared. This will yield a rapid increase in consolidation of the dredged fill and marine clay 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.

Vertical Strip Drain Theories

The design of vertical strip drains 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 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, which resulted in the solution shown below:

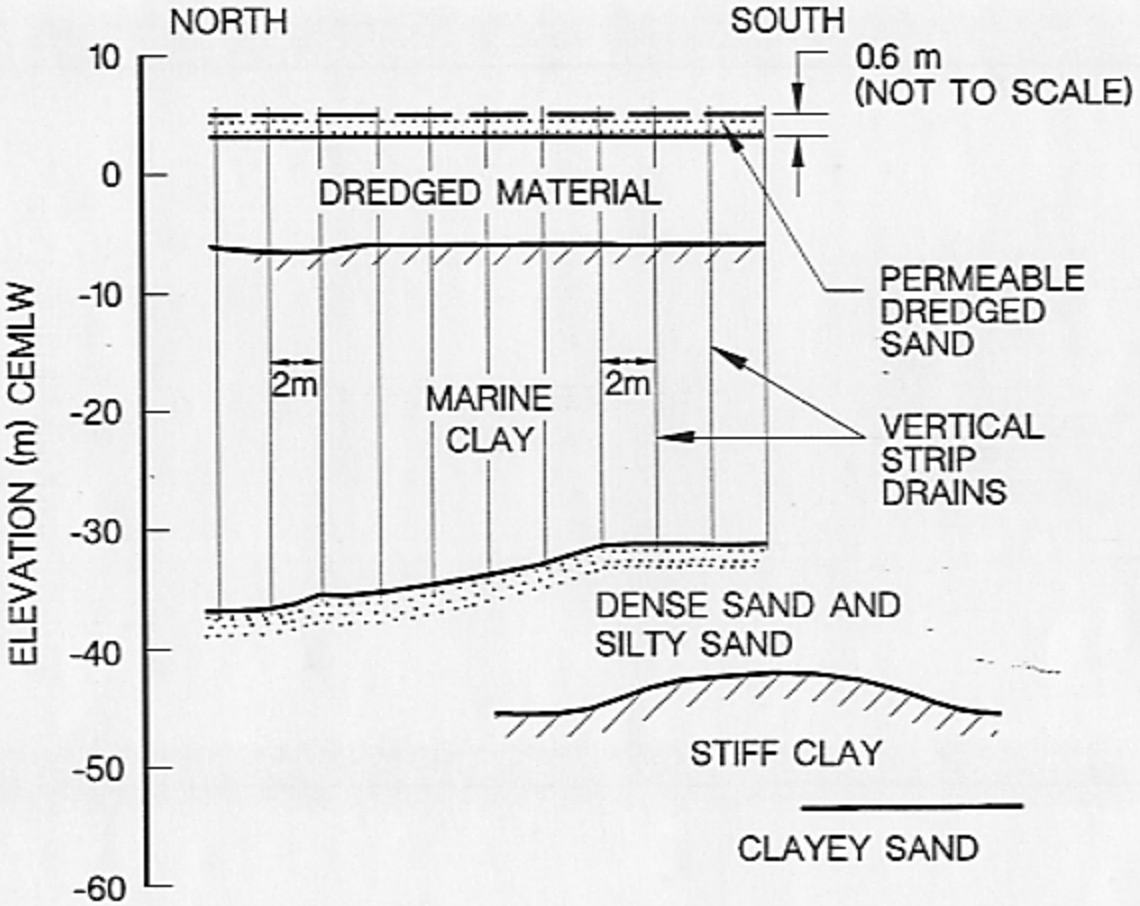

Figure 1. Subsurface Profile at Vertical Strip Drain Test Section

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

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

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

where U_h = average degree of consolidation for radial flow;
C_h = horizontal coefficient of consolidation;
C_v = vertical coefficient of consolidation;
d_e = sphere of influence of the strip drain (triangular pattern = 1.05S where S = strip drain spacing);
H_{dr} = maximum drainage length of vertical drainage path;
t = time;
$F(n,s)$ = term describing smear zones;
n = ratio of drain diameters $= \frac{d_e}{d_w}$;
s = ratio of smear zone diameter to drain diameter $= \frac{d_s}{d_w}$;
d_s = outer radius of the smear zone;
K_h = horizontal coefficient of permeability of the undisturbed soil;
K_s = horizontal coefficient of permeability of the smeared soil;
G = term describing well resistance;
K_w = coefficient of permeability of the strip drain;
l_m = maximum drainage length of strip drain;
d_w = equivalent strip drain diameter $= \frac{2(b+1)}{\pi}$;
b = width of strip drain (typically 0.305 - 0.328 ft, used 0.31 ft);

l = thickness of strip drain (typically 0.01 - 0.013 ft, used 0.0115 ft);

q_w = discharge capacity of strip drain = $\frac{\pi}{4} d_w^2 K_w$.

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 that has a slightly different expression for the effect of well resistance than Lo (1991). Review of several case histories (Mesri et al. 1994 and Mesri and Lo 1991) 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 strip drains.

Field Test Section Objectives and Layout

A 183 m by 122 m field test section was constructed, instrumented, and monitored to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and marine clay in 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 strip drain equipment and a comparison between measured and predicted effects of smear zone and well resistance. The vertical strip drain test section consists of two areas. The main area is 152 m by 122 m and was covered with a 0.6 meter thick sand blanket to promote surface drainage and support the installation equipment. The vertical strip drains were 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 (Figure 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 approximately El. -31 m CEMLW.

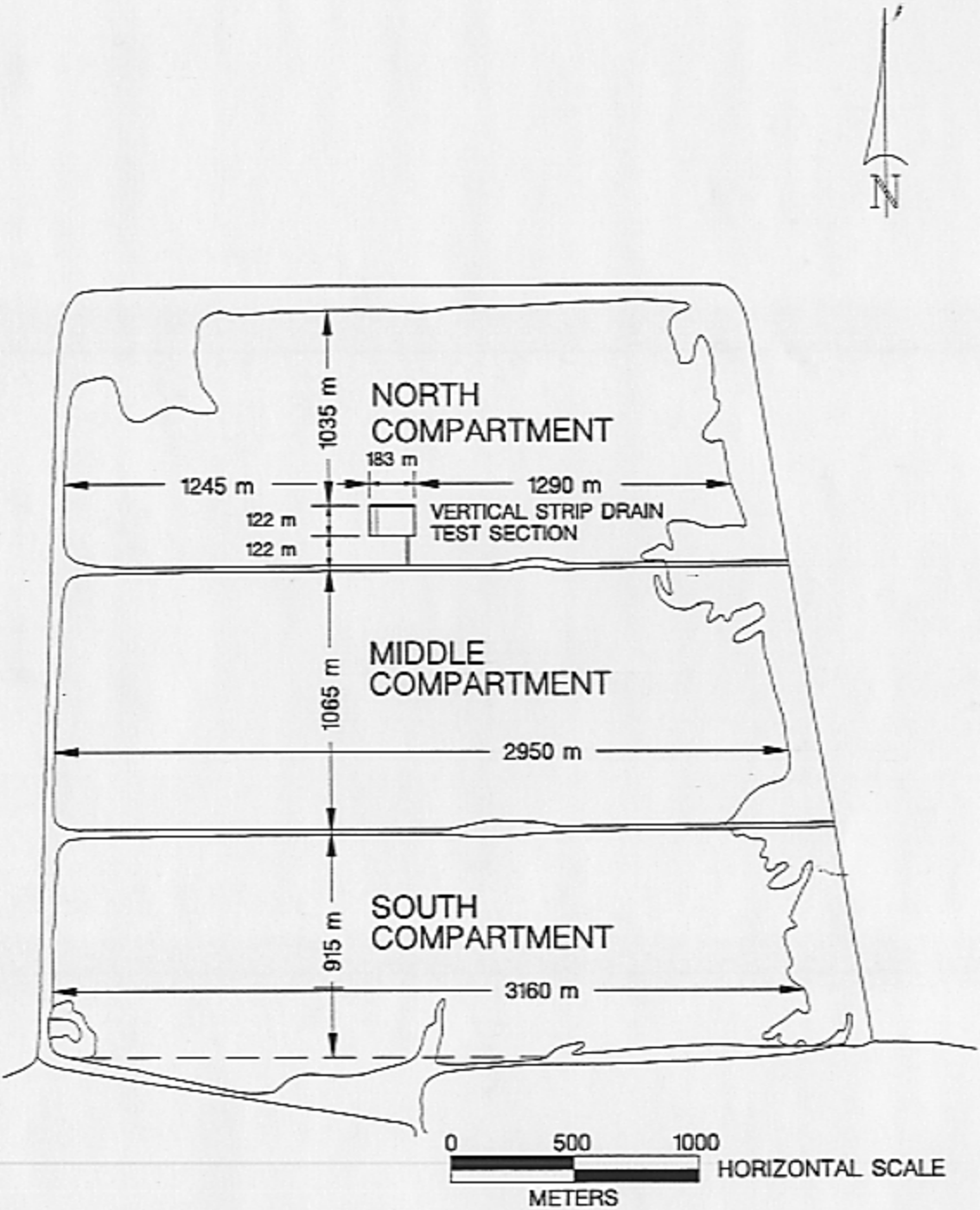

Figure 2. Plan View of Craney Island and Location of Vertical Strip Drain Test Section

The mobility test section was 30 m by 122 m and utilized 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 strip 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. The vertical strip drain equipment was required to exert a ground pressure that would enable the equipment to operate on the desiccated crust. It was anticipated 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 in an east to west direction. After the last drain was installed at the west end of the mobility section, the equipment turned-around and returned to the east end of the sand blanket installing strip drains along the way. The equipment repeated this east-west and west-east movement until all of the strip drains were installed.

The vertical strip drains in the mobility section were connected to horizontal strip drains on the ground surface. The horizontal strip drains were 30 cm wide and 2.5 cm thick. Each vertical strip drain was connected to a horizontal strip drain to promote drainage to the surrounding perimeter ditch. Horizontal strip drains were used to evaluate their effectiveness in conveying water from the test section to the surrounding perimeter trenches and their ease of installation. If the horizontal drains were effective and the desiccated crust can support the installation equipment, a sand blanket would not be required over the remainder of the site. In addition, the horizontal drains will promote drainage as future dredged material is placed at Craney Island. Horizontal strip drains were not installed in the main test section because the sand blanket would act as a drainage layer for future dredged fill.

Initial excess pore-water pressures were estimated from the installed piezometers, piezocone dissipation tests, and the preconsolidation pressure prior to drain installation. The distribution of excess pore-water pressure indicate that the marine clay was under-consolidated and the underlying dense sand is freely draining. 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

It was anticipated that consolidation of the dredged fill and marine clay foundation would result in a rapid increase in storage capacity and soil shear strength. This increase in undrained shear strength would result in substantial settlement, and thus increase in storage capacity. This also would probably allow the perimeter dikes to be raised again. The existing undrained shear strength 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 equations:

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

$$S_u = \frac{q_c - \sigma_{vo}}{N_{kuu}} \tag{5}$$

where q_c is the cone tip resistance, σ_{vo} is the total vertical overburden pressure, and N_k is an empirical cone factor based on field vane shear tests (Lunne and Kleven 1981 and Meigh 1987) and N_{kuu} is based on unconsolidated - undrained (UU) triaxial compression tests (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 utilized in the analysis to facilitate comparison purposes. In addition, this average value of N_k is the same as the average value of N_{kuu} (12) for the eighteen sites considered by Stark and Delashaw (1990). Figure 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 Equation (4), the appropriate total stress, and a value of N_k equal to 12.

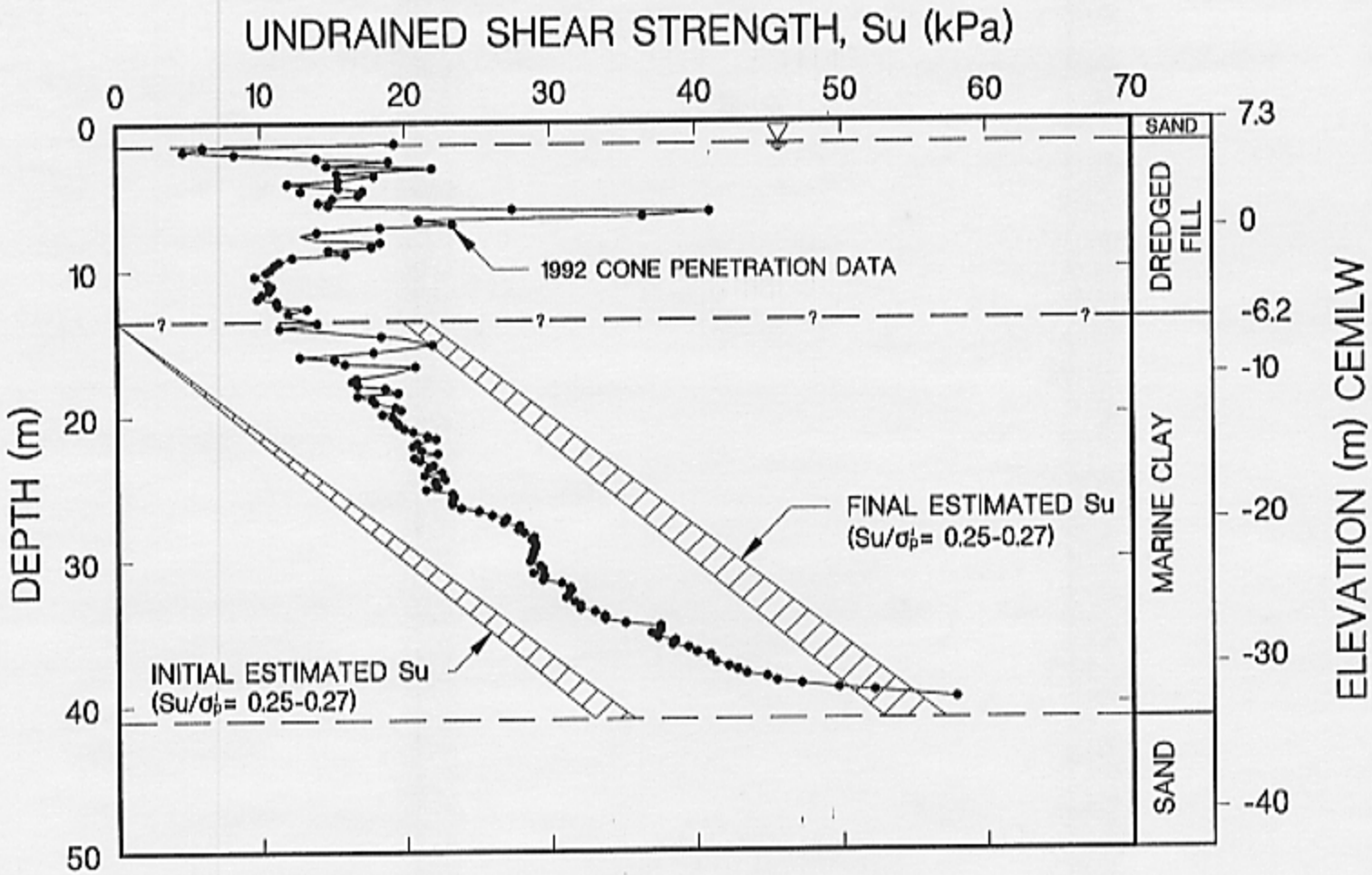

Figure 3. Undrained Shear Strength versus Depth in Craney Island Strip Drain Test Section

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

	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Clay Size Fraction (%)	Specific Gravity of Solids
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 Figure 3 several interesting facts can be ascertained concerning the undrained shear strength. First, variability exists in tip resistance measurements in the dredged material which 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. 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 is 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 percent consolidation.

Thirdly, the interface between the dredged fill and marine clay appears to be located at a depth of approximately 13.5 m or El. -6.2 m CEMLW. Craney Island was constructed in approximately 3 to 4 m of water (U.S. Army 1949). Therefore, it appears that the dredged fill and marine clay interface has subsided 2.2 m to 3.2 m since 1957.

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 (UU), unconfined compression (UC), and isotropically consolidated-undrained (CU) triaxial test results collected since 1948. Table 2 summarizes the values of undrained strength ratio estimated from the tests reported in the General Design Memorandums (U.S. Army 1949 and 1986) for Craney Island. Table 2 reveals that the undrained strength ratio ranges from 0.24 to 0.28. It can be seen that the historic field vane shear data yield an average undrained strength ratio of 0.26.

For comparison purposes, the undrained strength ratio was estimated from published correlations. The undrained strength ratio for an average plasticity index of 41 and the field vane mode of shear 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 to 0.27 was used in Figure 3 and an average ratio of 0.26 was used in the analysis described herein.

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

TEST METHOD	NO. OF MEASUREMENTS	AVERAGE Su/s_p'	STANDARD DEVIATION	COEFFICIENT OF VARIATION
FV	102	0.26	0.04	0.16
UU	55	0.24 *	0.13	0.46
UC	56	0.28	0.16	0.55
CU	10	0.27	0.05	0.17

* Five values of undrained strength ratio higher than 0.7 were omitted

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. Figure 3 shows the initial estimated Su profile, which corresponds to the Su profile before Craney Island was constructed, that is, prior to dredged fill disposal (i.e. 1956). As a result, the range of Su was estimated using an Su/σ_p'

equal to 0.25 to 0.27 and a normally consolidated marine clay starting at an 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 strip drains, and thus 100 percent consolidation of the marine clay. Figure 3 shows the final estimated Su profile, which was estimated assuming the dredged fill and marine clay are normally consolidated and the dredged fill surface is at El. +7.3 m CEMLW. A unit weight of 15.4 kN/m^3 was used to estimate the undrained shear strength after 100 percent consolidation. A comparison of the final estimated profile and the profile estimated from the 1992 cone penetration tests revealed 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. For example, at a depth of 30 m the initial, 1992, and 100 percent consolidation values of Su are 21, 28, and 41 kPa, respectively. Therefore, between 1956 and 1992 a strength gain of only 30 to 35 percent occurred in the marine clay because of the slow rate of consolidation. The slow rate of consolidation is attributed to the large vertical drainage path in the marine clay. The use of strip drains decreases the drainage path and accelerates consolidation. As a result, an increase in Su of 65 to 70 percent is expected when the marine clay achieves a degree of consolidation of approximately 100 percent.

In summary, installation of vertical strip drains will cause consolidation resulting in an increase in undrained shear strength and settlement. This increase in shear strength should allow the perimeter dikes to be constructed to higher elevations without dike setbacks or stability berms.

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 three meters to a depth of approximately 37 meters. Natural water contents (W_N) were determined for the recovered

samples and are compared to the plastic (W_P) and liquid limits (W_L) of the samples (Figure 4). It can be seen that the water contents of the dredged fill and marine clay are at or near the liquid limit. A water content near the liquid limit indicates a soft to liquid consistency, and thus a normally consolidated soil. Also shown in Figure 4 are water contents measured prior to construction of Craney Island (U.S. Army 1949). It can be seen that the pre-1956 and 1993 profiles are similar. This also indicates that minimal consolidation has occurred in the marine clay between 1956 and 1993.

Void ratios (e) can be determined from the water content data by assuming a degree of saturation (S) of 100 percent, a specific gravity of soil mass (G_s) equal to 2.71 (Table 1), and the following equation:

$$S * e = G_s * W_N \tag{6}$$

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.

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:

$$C_c = 0.01 * (LL - 13\%) \tag{7}$$

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 are presented in Figure 5. It can be seen that C_c ranges from 0.41 to 0.79 for the majority of the data with an average or representative value of 0.58.

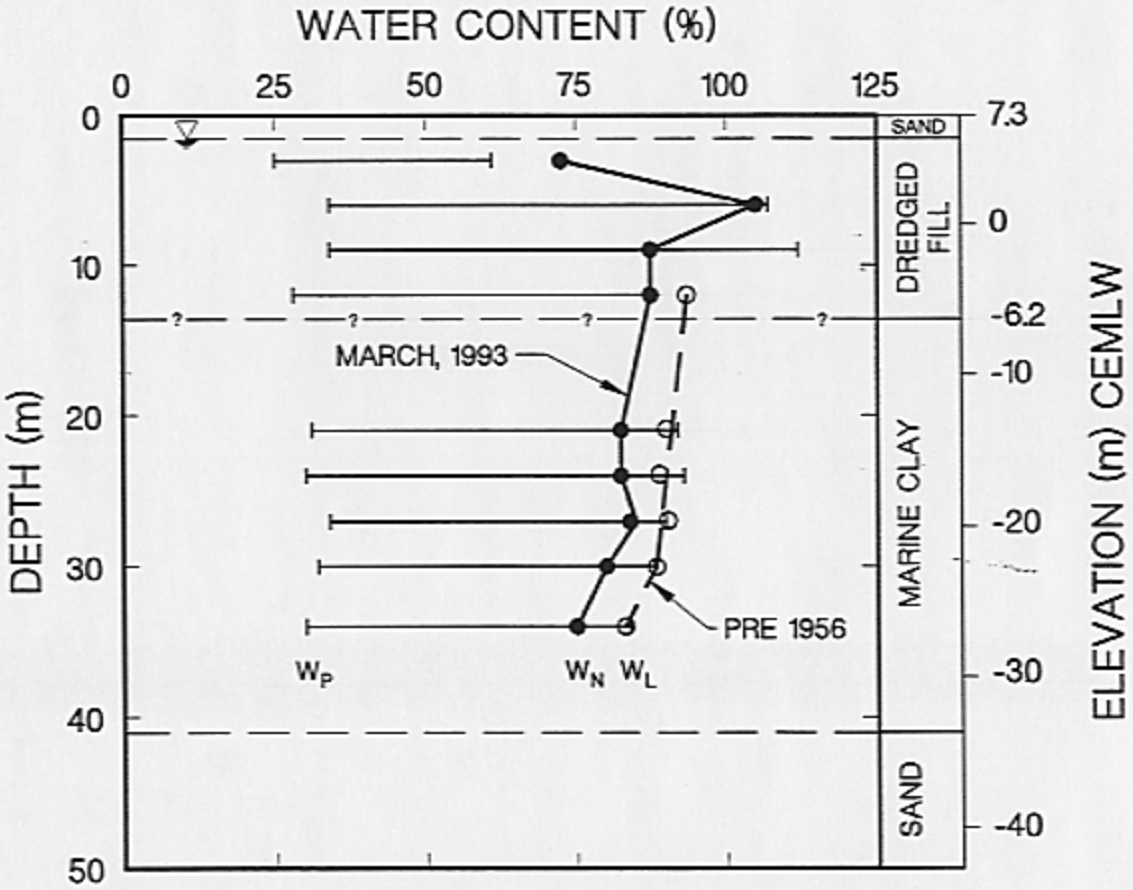

Figure 4. Water Content versus Depth in the Craney Island Strip Drain Test Section

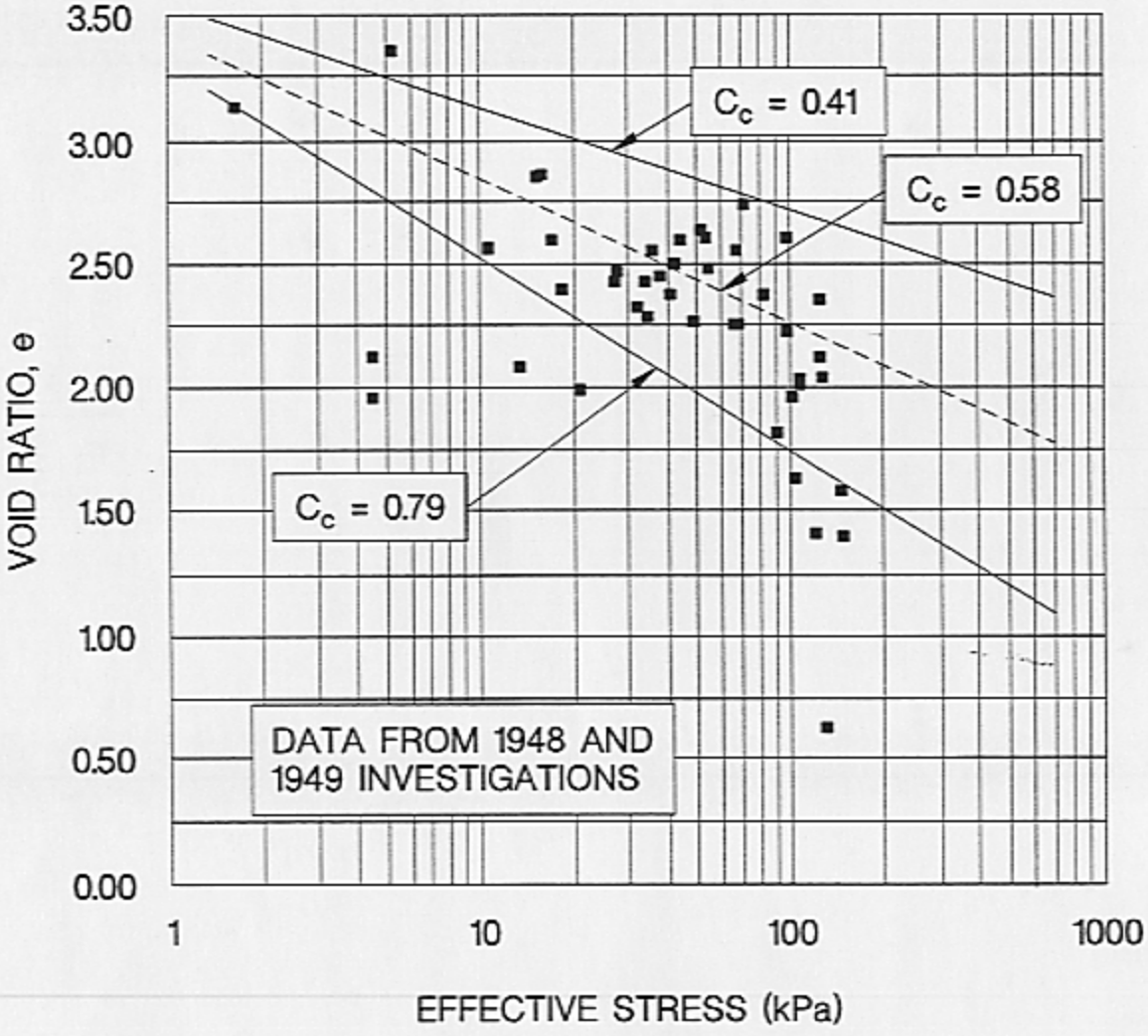

Figure 5. Void Ratio-Effective Stress Relationship for Marine Clay

This range of C_c is used in a subsequent section to estimate the consolidation settlement induced by installation of strip drains.

Coefficient of Consolidation

Strip drain spacing is governed by the horizontal (C_h) and vertical (C_v) coefficients of consolidation. It can be seen from Figure 1 that strip 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.

The results of hydraulic conductivity tests in piezometers installed in the perimeter dikes were used to estimate C_h and C_v. The flow around the piezometer tip is probably a combination of vertical and horizontal flow. However, for simplicity the flow was assumed to be horizontal and thus the hydraulic conductivity tests were assumed to be measuring the horizontal hydraulic conductivity.

The value of horizontal permeability was calculated from these hydraulic conductivity tests using the following expression (British Standards Institution 1981):

$$K_h = \left[\frac{1.45 * A}{(t_2 - t_1)}\right] \log\left(\frac{H_1}{H_2}\right) \tag{8}$$

where K_h is horizontal hydraulic conductivity, t is time, A is the cross sectional area of the standpipe, and H_1 and H_2 are the variable hydraulic heads at times t_1 and t_2, respectively. The values of C_h were calculated for the dredged fill using the horizontal hydraulic conductivity from Equation (8), an initial void ratio, e_o, of 3, a unit weight of water equal to 9.8 kN/m^3, a horizontal coefficient of compressibility, a_h, of 9.1×10^{-3} $(kPa)^{-1}$, and the following equation:

$$C_h = \left[\frac{K_h * (1 + e_o)}{a_h * \gamma_w} \right] \tag{9}$$

The value of a_h was obtained from oedometer and self weight consolidation test results (Cargill 1983) in the proper range of effective stress. The average horizontal hydraulic conductivity from three hydraulic conductivity tests in piezometers located in the dredged fill was estimated to be 2.4×10^{-3} m/day. This hydraulic conductivity corresponds to an average value of C_h of 3.7×10^{-2} m^2/day.

The values of C_h were calculated for the marine clay using the average horizontal hydraulic conductivity from field hydraulic conductivity tests in piezometers, an initial void ratio of 2.5, and a coefficient of compressibility of 1.9×10^{-2} $(kPa)^{-1}$ (Cargill 1983). The average horizontal hydraulic conductivity from three hydraulic conductivity tests in the marine clay was estimated to be 7.2×10^{-4} m/day, which corresponds to an average C_h of 1.4×10^{-2} m^2 /day. As expected, the marine clay exhibited a lower horizontal hydraulic conductivity and C_h because of the smaller initial void ratio.

Based on the data presented by Mesri and Lo (1991), a value of C_v was estimated by dividing C_h by an average ratio of 1.25. Therefore, the values of C_v for the dredged fill and marine clay were calculated to be 3.0×10^{-2} and 1.1×10^{-2} m^2 /day, respectively (Table 3).

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

Source of Data	C_h (m^2/day)	C_v (m^2/day)
Dredge Fill Data		
Field Piezometers (Stark 1995)	3.70×10^{-2}	3.00×10^{-2}
Cargill (1983)	1.10×10^{-2}	8.80×10^{-3}
Marine Clay Data		
Field Piezometers (Stark 1995)	1.40×10^{-2}	1.10×10^{-2}
Design Memorandums (U.S. Army, 1949 & 1986)	1.90×10^{-3}	1.50×10^{-3}
Empirical Correlations (U.S. Navy, 1982)	9.90×10^{-3}	7.90×10^{-3}
Design Parameters	1.16×10^{-2}	9.29×10^{-3}

Oedometer test results from General Design Memorandums (U.S. Army 1949 and 1986) were also used to estimate C_v. Thirty-two time curves corresponding to the average effective stress in the marine clay (approximately 60 kPa) were used to estimate an average value of C_v for the test section area. The resulting average value of C_v for the marine clay is 1.5×10^{-3} m^2 /day. An average value of C_h was estimated to be 1.9×10^{-3} m^2/day by multiplying C_v by 1.25.

It can be seen from Table 3 that the oedometer test results from the General Design Memorandums (U.S. Army 1949 and 1986) yield values of C_h and C_v that are lower than the field hydraulic conductivity test values. The difference is attributed to sample disturbance, the lack of a representative sample, the accuracy of evaluating hydraulic conductivity without flow quantity or pore-water pressure measurements in oedometer tests, and the combined vertical and horizontal flow that probably occurred around the piezometers during the field hydraulic conductivity tests.

Laboratory consolidation data on the dredged fill reported by Cargill (1983) were also used to obtain values of C_v and C_h equal to 8.8×10^{-3} and 1.1×10^{-2} m^2/day, respectively, for a void ratio of 3. Since these values of C_v and C_h were obtained from oedometer tests on the dredged fill at void ratios of approximately 3 (Cargill 1983), these values are similar to the field hydraulic conductivity tests. The slight discrepancy may be related to soil disturbance, differences in soil type, combined vertical and horizontal flow around the field piezometers, and the presence of thin drainage layers around the piezometers.

Empirical correlations of C_v presented in the Navy Design Manual DM-7.1 (U.S. Navy 1982) and a liquid limit of 71 were also used to estimate a value of C_v equal to 7.9×10^{-3} m^2/day for the normally consolidated marine clay. This value of C_v corresponds to a value of C_h equal to 9.9×10^{-3} m^2/day. Since the dredged fill is under going self-weight consolidation, values of C_v and C_h could not be estimated from this correlation. The values of C_v and C_h reported in this correlation correspond to effective stresses greater than those present in the dredged fill. Therefore, the dredged fill values of C_v and C_h are probably higher than those reported in the Navy Design Manual DM-7.1 (U.S. Navy 1982) correlation.

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 layer and 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 1.16×10^{-2} and 9.29×10^{-3} m^2/day, respectively, and were used to determine the preliminary spacing of the strip drains.

Strip Drain Design Parameters

The other major parameters required to develop an estimate of strip drain spacing are the well resistance and the extent of the smear zone. It can be seen from Equations (1) through (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 strip 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 strip drain in the test section area (l_m) is equal to 22 m. This value of l_m also equals the maximum drainage 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 strip 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 Strip Drain Test Section

The major design constraints for the test section were cost and the time required for 90 percent consolidation. Using the design theory presented by Lo (1991) and the design parameters previously described and presented in Table 4, a sphere of influence of a strip drain, d_e, equal to 2.3 m is required to obtain a degree of consolidation of 90 percent 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 Equation (1) yields a degree of consolidation of 90 percent.

The sphere influenced by each vertical strip drain is calculated using Equation (1) for both square and triangular drain patterns. A preliminary strip drain spacing for a triangular pattern was calculated to be 2.2 m by dividing the sphere of influence on the strip drain, d_e, by 1.05. The value of 1.05 is determined by geometry of three spheres creating a triangle.

Table 4. Strip Drain Test Section Design Parameters

Parameter	Value
Degree of Consolidation	90%
Time	1 year
K_h	7.3×10^{-4} m/day
C_v	9.3×10^{-3} m^2/day
C_h	1.2×10^{-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

Strip Drain Installation Equipment

Vertical strip 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 strip drain installation equipment had to be mounted on pontoons to reduce the maximum contact pressure to less than or equal to 10.4 kPa. This would enable the equipment to operate on the 15 to 30 cm thick desiccated crust in the mobility test section. The contractor mounted the installation equipment between two 2.1 m wide and 10.7 m long pontoons. The entire equipment weighs approximately 440 kN (99,120 lbs). Since the area of the pontoons was 45.6 m^2, the ground pressure exerted by this equipment was only 9.7 kPa. As a result, the equipment encountered little difficulty operating on the desiccated crust. The 50 m long mandrel had to be stabilized using guy wires. The cross-sectional area of the mandrel was restricted to 6.5×10^{-3} m^2 to reduce soil disturbance during 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).

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

were installed in the mobility test section. In the mobility section 2,181 lineal meters of horizontal strip drain were installed.

Test Section Performance

Immediately following installation of the vertical strip drains, water could be seen rising along the drainage core and around the strip drain. The water rising around the strip drain was caused by the void left by the mandrel after retraction. Within 10 to 15 minutes after installation, water rose 0.05 to 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 Figure 6. Installation of the vertical strip drains in the test section was completed on 26 February 1993. On 18 July 1995, approximately 25 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 21 August 1995 through December 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 to 2.9 m, which is based on an average C_c of 0.58 and a degree of consolidation of 100 percent. 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 (Figure 5).

Figure 7 presents the settlement plate data from the main test section using a semi-logarithmic scale. It can be seen that none of the settlement plates indicate that primary consolidation has been 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 21 August 1995.

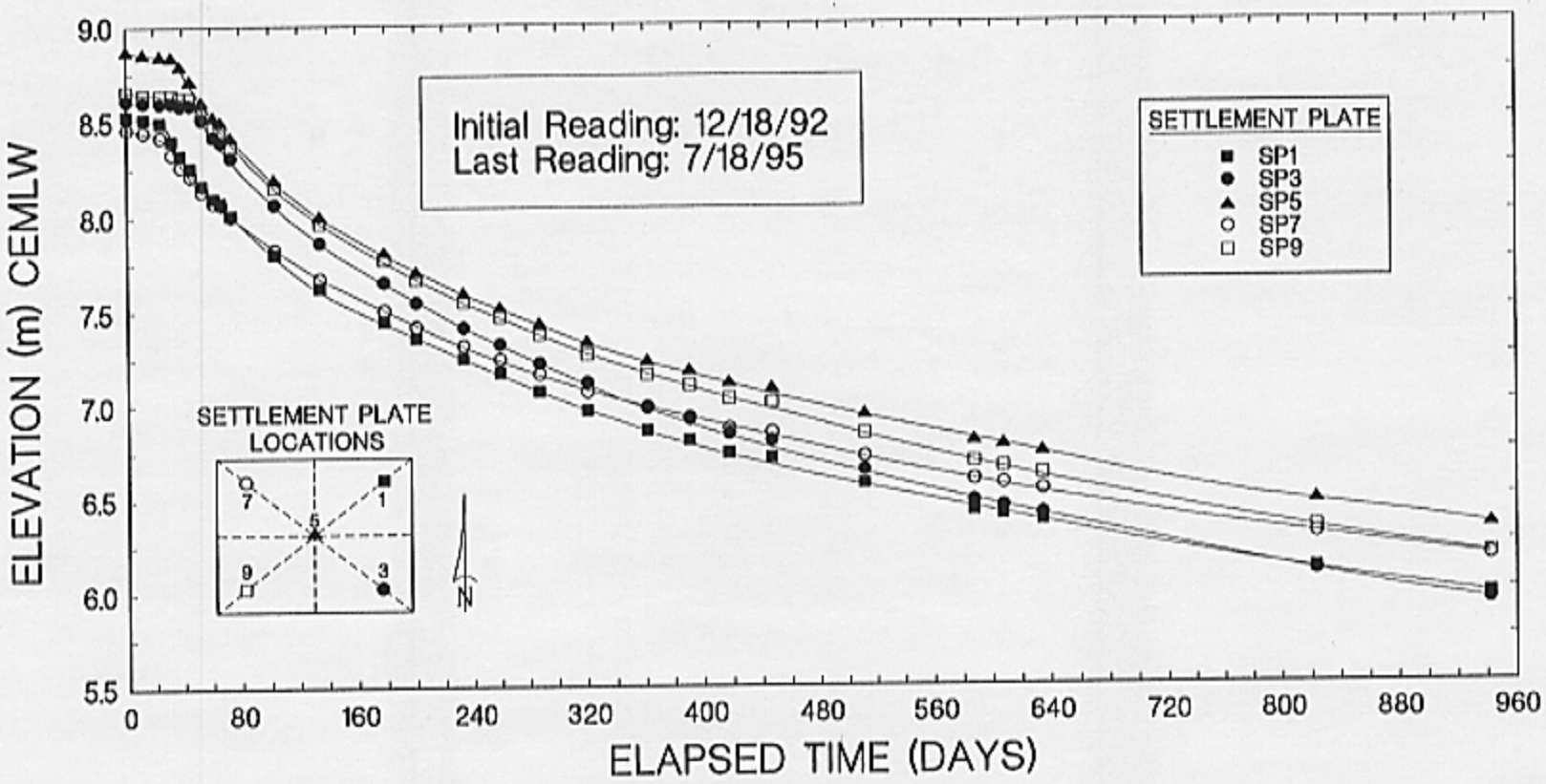

Figure 6. Settlement Plate Measurements in Main Test Section

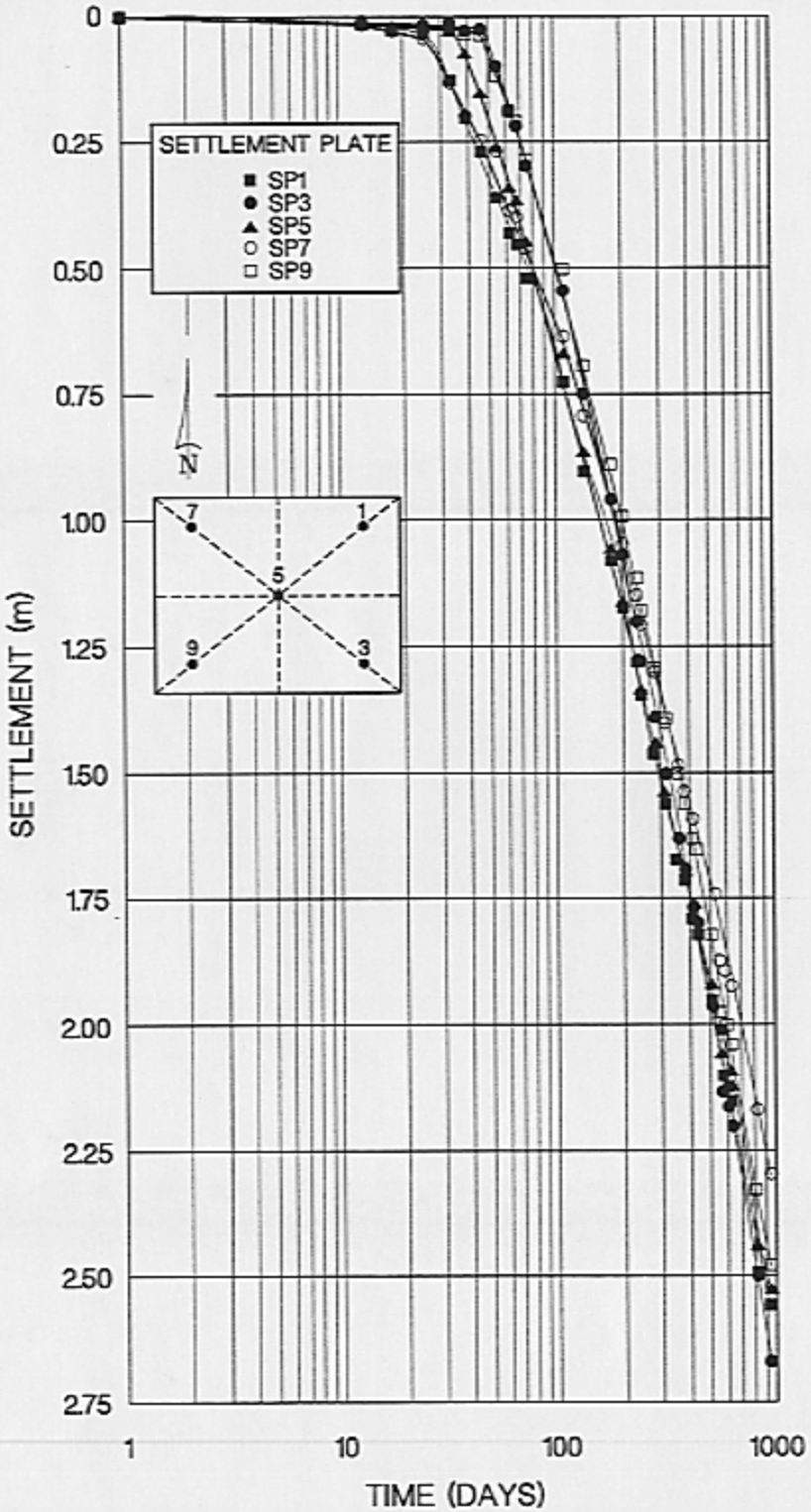

Figure 7. Semi-Logarithmic Presentation of Settlement Plate Measurements in Main Test Section

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 percent 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 The void ratio after 100 percent consolidation can also be estimated using a range of C_c (0.41 to 0.79) and the final effective overburden stress to predict a range of consolidation settlement.

Measured Settlements in Mobility Test Section

The mobility section was developed to demonstrate that a sand blanket was not required to support the strip drain equipment. Since the equipment exerted a ground pressure of only 9.7 kPa and was able to operate with little or no difficulty on the dredged material surface, a sand blanket is not required for future strip drain installations. A comparison of Figures 6 and 8 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 to 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 strip drain equipment to operate without the

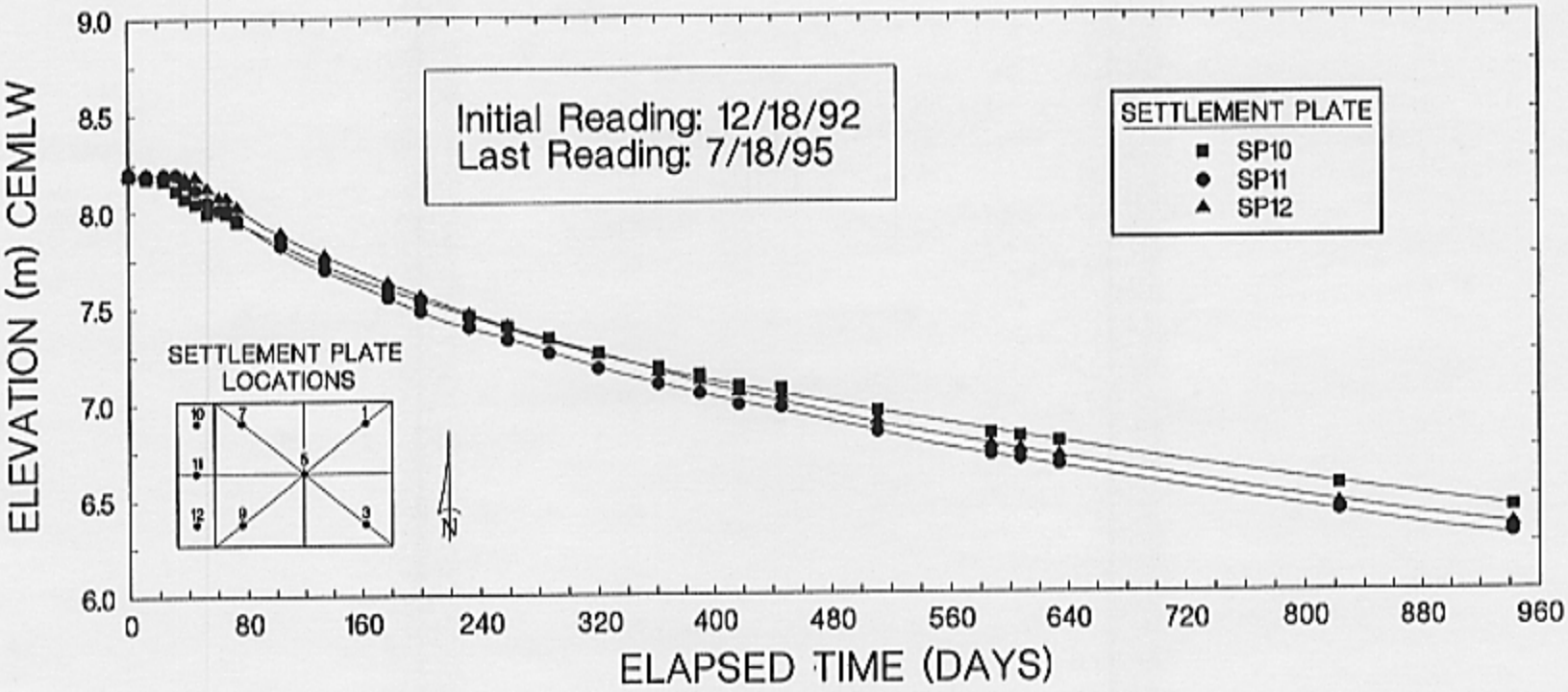

Figure 8. Settlement Plate Measurements in Mobility Test Section

sand blanket will probably preclude the use of a sand blanket throughout the remainder of the placement area.

Figure 9 presents the settlement plate data from the mobility test section using a semi-logarithmic scale. It can be seen that none of the settlement plates indicate that primary consolidation has been 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 percent consolidation and an average C_c value of 0.58. The estimated range consolidation settlement is 0.9 to 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 a value of C_c equal to 0.71 yields a settlement of 2.3 m, which is in agreement with field measurements..

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. A final conclusion on the accuracy of the estimated settlement will probably not be known because new dredged material was pumped into the north compartment between August 1995 and December 1995. However, it can be assumed that the north compartment will settle between 2.1 to 2.3 m without a sand blanket after strip drains are installed. This information was used to determine whether strip drains should be installed in the placement area, the perimeter dikes, or both to extend the service life of Craney Island.

Time Rate of Consolidation

A strip drain spacing of 2.2 m should have resulted in 90% consolidation in twelve to thirteen months based on the strip drain design parameters in Table 4. Since the consolidation settlement was still occurring 30 months after drain installation, one or more of the design parameters does not model the field conditions. Several possible explanations for this discrepancy are:

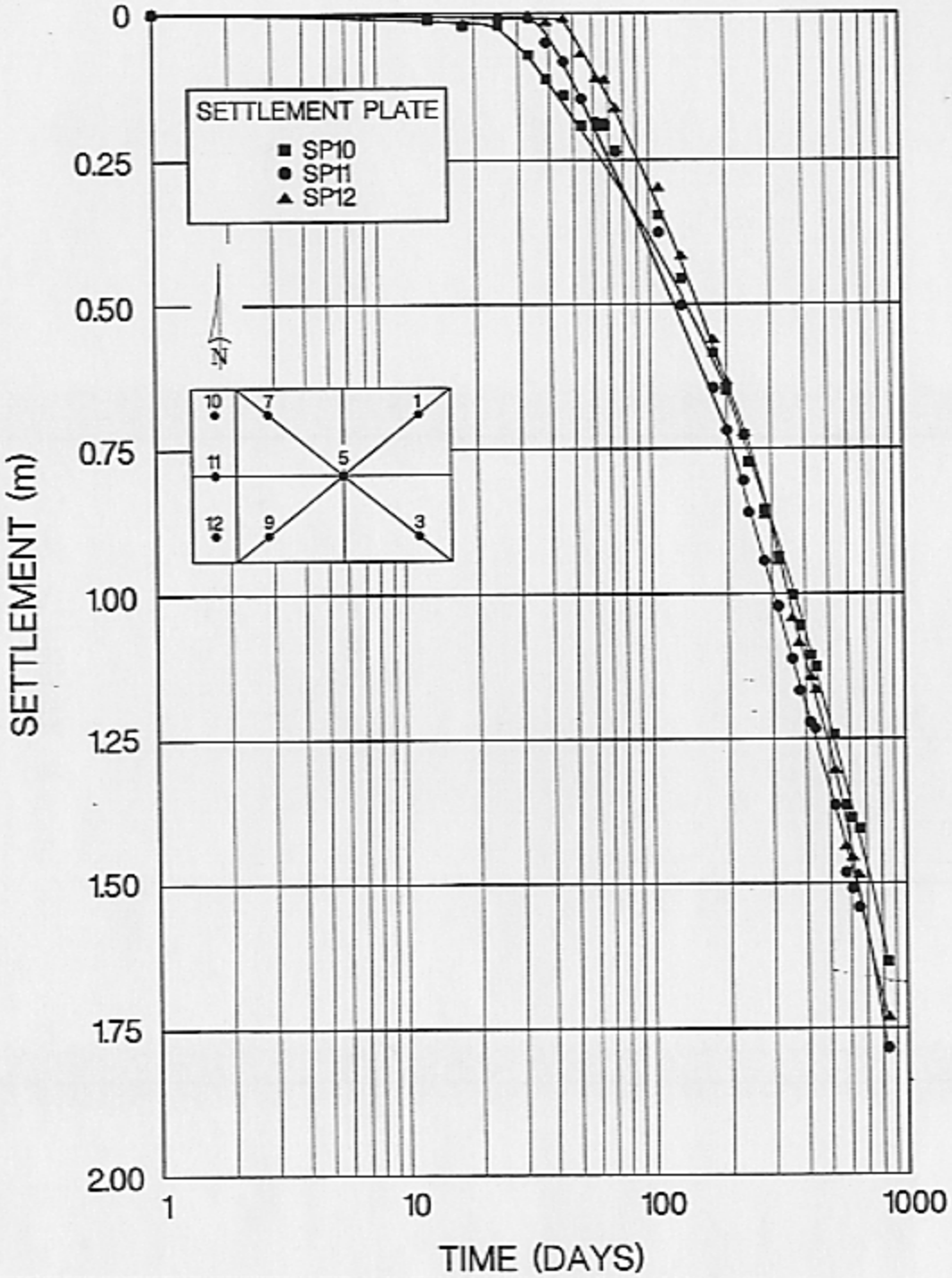

Figure 9. Semi-Logarithmic Presentation of Settlement Plate Measurements in Mobility Section

1.) The strip 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 strip 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 strip 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 strip drain is not acting as doubly drained. This could be caused by the drain not being anchored into the underlying sand or 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 approximately quadruple. If so, at least three to four years would be required to achieve 90% consolidation.

4.) The horizontal coefficient of consolidation is lower than 1.16×10^{-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).

Figure 10 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 strip drain theory presented in Equations (1) through (3). The range in time rate of settlement was estimated using the degree of consolidation calculated using Equations (1) through (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

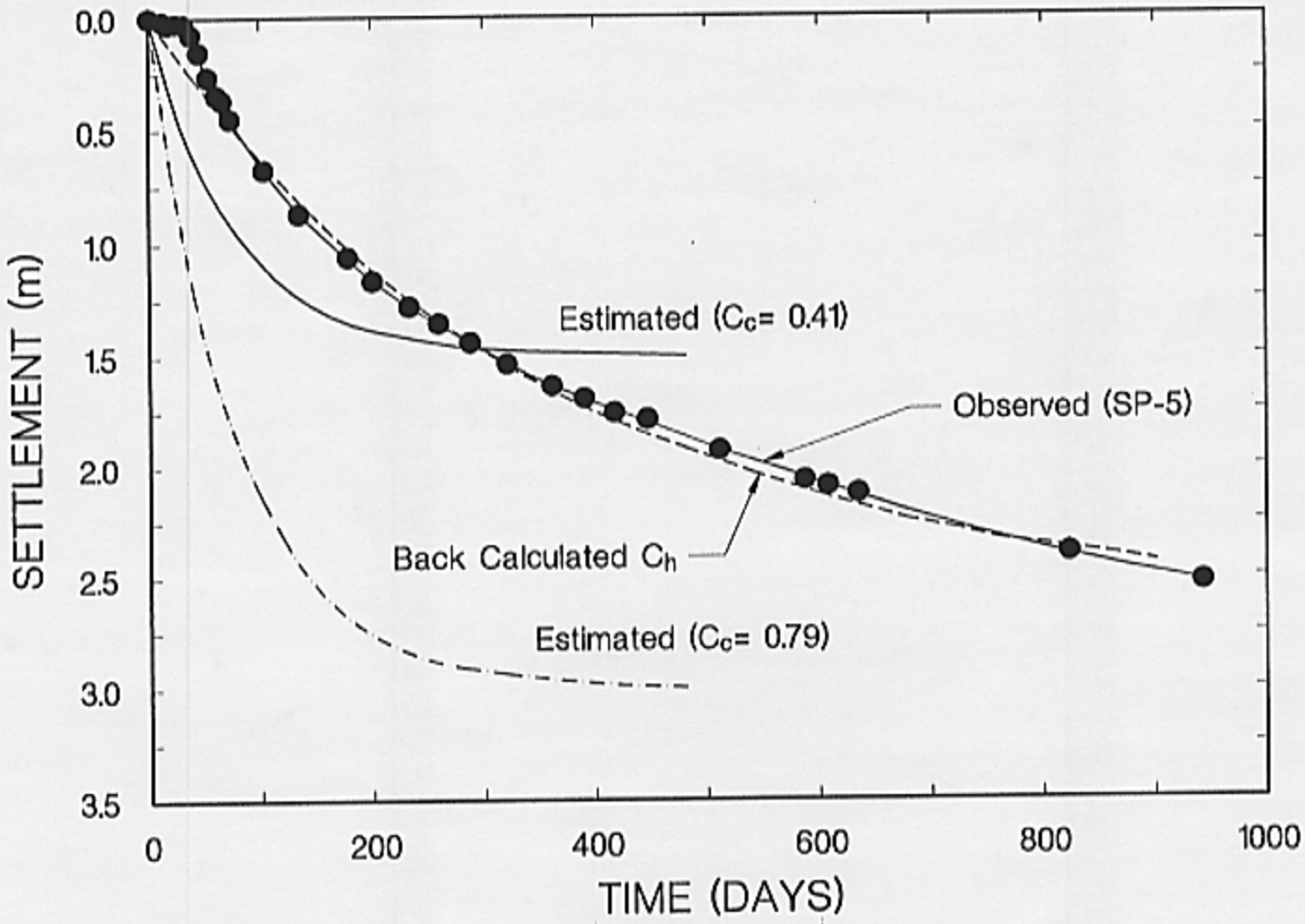

Figure 10. Measured and Estimated Time Rate of Consolidation Settlement for Main Test Section

measured settlements (Figure 6) are representative of the time rate of consolidation of the main test section, and were thus compared to the estimated rates of consolidation.

It can be seen in Figure 10 that the estimated time rates of settlement are not in agreement with the measured values. This was expected because the strip drain spacing of 2.2 m was designed to achieve 90 percent consolidation in twelve to thirteen months and the test section was still settling 30 months after strip drain installation.

Figure 11 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 strip drain theory presented in Equations (1) through (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 Figures 10 and 11 were used to back-calculate strip drain design parameters to aid future time rate of consolidation predictions at Craney Island. Equations (1) through (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 Equations (1) through (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 percent consolidation after twelve to thirteen 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 strip 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. In particular, this lower value of C_h probably reflects a number of uncertainties including 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 strip drain equipment, installation procedure, and type of drain used in this test section. If the same or similar equipment,

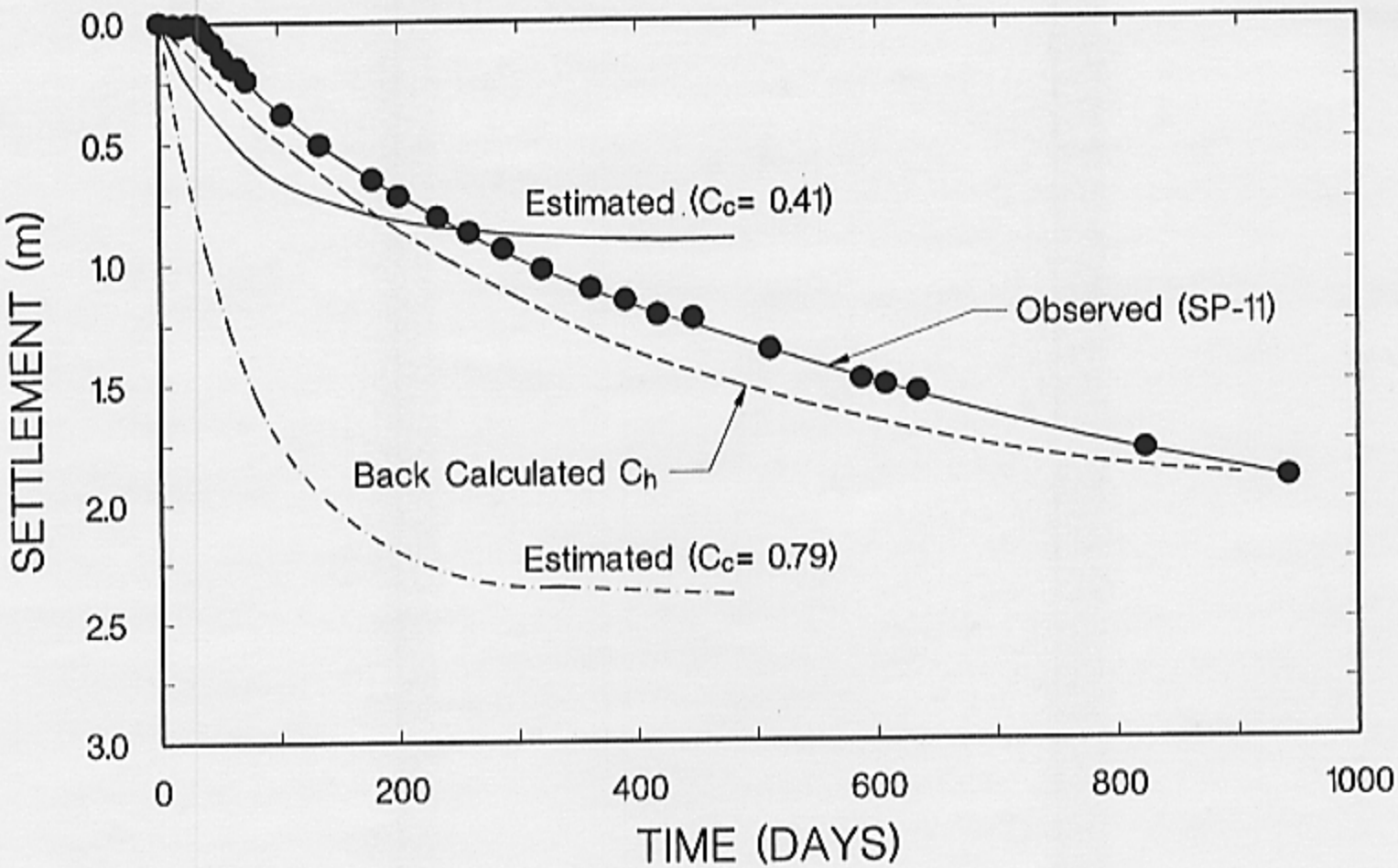

Figure 11. Measured and Estimated Time Rate of Consolidation Settlement for Mobility Test Section

installation procedure, and strip 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. 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. In other words, near the preconsolidation pressure settlement takes place at an almost constant value of effective vertical stress with a significant amount of excess pore-water pressure still remaining in the clay. 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 strip 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 strip 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 by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in

consolidating the dredged fill and underlying marine clay, and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since drains had never been installed in an active dredged material management area, a drain length of approximately 50 m was close to the longest drain ever installed, and the installation equipment had to operate directly on the surface of the soft dredged material.

The subsurface investigation conducted prior to strip drain installation revealed that large excess pore-water pressures exist in the underlying marine clay. The excess pore-water pressures in the dredged fill are smaller than the marine clay. This is probably due to the sand and silt seams in the dredged fill dissipating the excess pore-water pressures induced by the self-weight consolidation of the dredged fill. As a result, the large consolidation settlements that occurred at the test section are primarily attributed to consolidation of the marine clay.

In the main test section a sand working pad was constructed while no sand pad was constructed in the mobility test area. The main objective of the mobility test section was to determine if the installation equipment could operate directly on the soft dredged material. The pontoon mounted equipment was successful and efficient in operating in the mobility section. Comparison of the settlements in the mobility and main test sections revealed that the sand blanket caused an increase in settlement by surcharging the dredged material. Therefore, surcharging confined dredged material can lead to substantial consolidation and increased in storage capacity. However, the cost of the sand blanket and the ability of the equipment to operate without the sand blanket will probably preclude the use of a sand blanket for future strip drain installations in the placement area.

Settlement plates installed in the main test section settled approximately 2.3 to 2.7 m in 25 months. The mobility test section settled 1.75 to 1.85 m in 25 months. These consolidation settlements are in agreement with the estimated values. 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 $C_h = 1.3 \times 10^{-3}$ m^2/day) should be used to design future strip drain installations at Craney Island that utilize similar equipment, installation procedure, and strip drains.

In summary, the Craney Island test section showed that strip 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.

Acknowledgments

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STRIP DRAIN TEST SECTION IN CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

By: Timothy D. Stark, Thomas A. Williamson, Jack Fowler, David Pezza, and Yvonne Gibbons.

Information Retrieval Abstract:

A 183 m by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and underlying foundation clay, and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since strip drains had never been installed in an active dredged material management area, a drain length of approximately 50 m was close to the longest strip drain ever installed, and the installation equipment could only exert a ground pressure of 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 2.6 m in 30 months). In summary, prefabricated strip drains appear to be an economical technique for increasing the storage capacity of active and inactive dredged material management areas. It is anticipated that the strip drains will continue to function as additional dredged material is placed in the management area, and thus increase storage capacity in the future.

Reprint Sales Summary:

A 183 m by 122 m vertical strip drain test section was completed in a dredged material placement area. Results show that the dredged fill and foundation clay underwent substantial consolidation settlement, which suggests strip drains may be an economical technique for increasing storage capacity.

Keywords:

Soil mechanics, Cohesive Soil, Consolidation, Strip Drains, Dredged Fill

STRIP DRAIN TEST SECTION IN CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

By: Timothy D. Stark, Thomas A. Williamson, Jack Fowler, David Pezza, and Yvonne Gibbons.

Figure Captions:

Fig. 1. Subsurface profile at vertical strip drain test section
Fig. 2. Plan view of Craney Island and location of vertical strip drain test section
Fig. 3. Undrained shear strength versus depth in Craney Island strip drain test section
Fig. 4. Water content versus depth in the Craney Island strip drain test section
Fig. 5. Void ratio-effective stress relationship for marine clay
Fig. 6. Settlement plate measurements in main test section
Fig. 7. Semi-Logarithmic presentation of settlement plate measurements in main test section
Fig. 8. Settlement plate measurements in mobility test section
Fig. 9. Semi-Logarithmic presentation of settlement plate measurements in mobility section
Fig. 10. Measured and estimated time rate of consolidation settlement for main test section
Fig. 11. Measured and estimated time rate of consolidation settlement for mobility test section

Table Captions:

Table 1. Summary of index properties of clay.
Table 2. Undrained strength ratios for marine clay form various test methods.
Table 3. Estimated values of C_v and C_h for the dredged fill and marine clay.
Table 4. Strip drain test section design parameters.