Performance of Wick Drains at Windsor, Connecticut

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Reconstruction of highways in Windsor, Connecticut, required fills to separate Route I-91 from I-291. To keep this project on schedule, the consolidation and settlement of the underlying varved clay was accelerated with vertical drains. The soil profile and properties are described and the predicted behavior is illustrated. The field data included settlement observations, piezometer readings, and inclinometer measurements. The settlement observations and the piezometer readings are analyzed, independent of laboratory data, for coefficient of consolidation and total settlement. The analytical techniques are presented. The analyzed results compare with predicted values.

Reconstruction of the highways in Windsor, Connecticut, required several new features. A limited access highway replaces the existing roadway to develop Route I-291. The original interchange between I-91 and I-291 was inadequate for the new arrangement. The new alignment elevates the grade of I-291, to cross over I-91 and local roads, with a series of structures and fills. The separation of these roadways required many large fills over deep clay deposits. To complete the project in the specified 4-year period, vertical drains were needed to accelerate the consolidation process so primary compression settlements and some secondary compression settlement could occur before paving began. The work presented here analyzes the field data associated with these settlements and compares them with design parameters and predictions.

SUBSURFACE PROFILE

The earth materials in this area occur in five distinct layers as shown in Figure 1. The site is underlain by sedimentary bedrock that supports a dense glacial till. The major deposit affecting settlements is the varved clay above the till, which is covered with a layer of dense silt. A layer of sand rests on top of the silt at most locations.

The most important geological feature in the history of the area is Lake Hitchcock. This lake formed behind a natural dam at the present location of Rocky Hill, Connecticut, during the glacial age of the late Pleistocene era. The glacier's advance carved deeply into the present Connecticut River Valley, and the shrinking ice sheet formed a reservoir behind the dam. The yearly cycle of sediment deposition started in the warming periods of spring, when the annual snowmelt increased the flow of water bringing new sediment. During the high flow periods the coarser particles settled out. Later in each year, as the average daily temperature dropped, the flow reduced, allowing the finer particles to settle. The varves are generally 0.1 to 1.5 in. thick. The coarse varves contain silt with some fine sand and usually have a relatively high horizontal permeability.

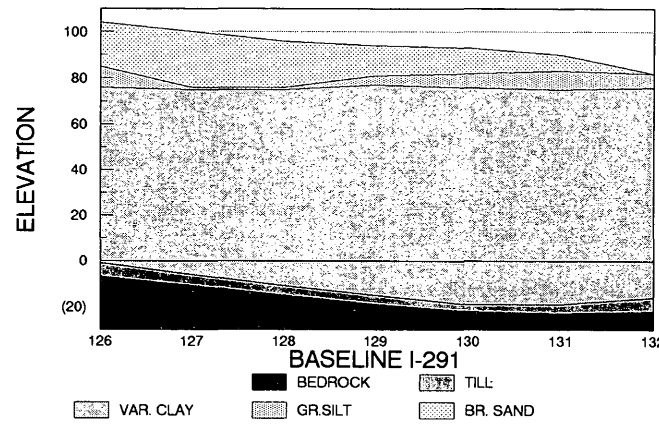

FIGURE 1 Soil profile, I-291, Windsor, Connecticut.

Lake Hitchcock extended as far north as Lyme, New Hampshire, and existed for 2,000 to 3,000 years, allowing a clay stratum between 50 and 150 ft thick to form in some parts of the deposit. The natural dam was breached about 11,000 years ago, draining the lake. Since the draining of Lake Hitchcock, the Connecticut River has been the dominant geological factor in the area.

ENGINEERING CHARACTERISTICS OF THE VARVED CLAY

Some properties for the varved clay in this area, including moisture content, stress history, and undrained strength, are shown in Figure 2 as reported from several geotechnical investigations (*1,2*). Predictions of rate and amount of settlement for this area were made from these studies. The combined subsurface investigations included 339 borings and laboratory testing of the soil properties. The upper fine sand layer varies in thickness between 0 and 15 ft; the silt layer varies between the same thicknesses and is dense. The maximum thickness of the clay in this area is 95 ft. The laboratory testing measured Atterburg limits, consolidation characteristics, and strength properties. The varves varied in thickness between 0.1 and 0.8 in. The coefficient of consolidation in the vertical direction varied from a value of 0.1 ft^2/day in the

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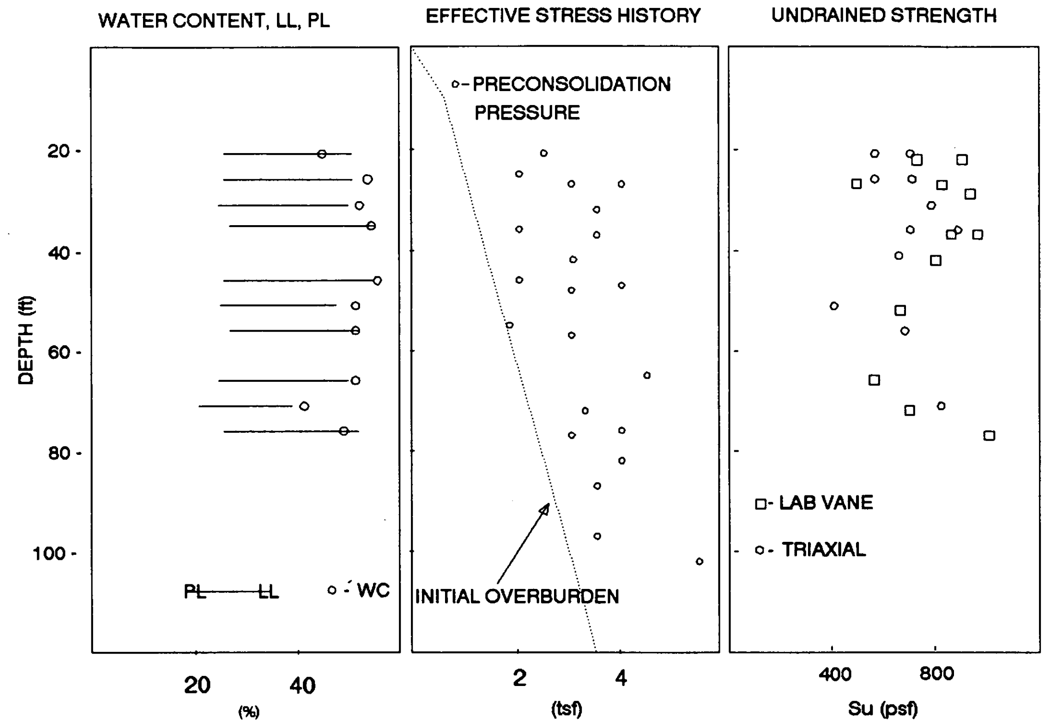

FIGURE 2 Varved clay properties, I-291, Windsor, Connecticut.

normally consolidated range to 0.9 ft^2/day in the overconsolidated range. The coefficient of secondary compression for the samples tested fell into the range 0.0025 to 0.0082, measured as strain per log cycle. The undrained strength of the samples varied between 400 and 1,000 lb/ft^2. Permeability of the clay was not measured in the laboratory. Values of horizontal to vertical permeability reported in the literature range between 4.5 and 14 for Connecticut Valley varved clays (*3*). However, the apparent coefficients of consolidation backfigured from field data indicate significantly higher values of horizontal permeability (*4*).

INSTRUMENTATION OF THE FOUNDATION AND CONSTRUCTION OF THE FILL

Instrumentation

The work reported here is primarily from the approach fill at the western abutment shown in Figure 3. The fill was built to a height of 30 ft including 3 ft of surcharge in approximately 40 days. The fill lies east of Pine Lane, north of Wolcott Ave., and west of the proposed abutment as shown in Figure 3. This figure is a plan of the settlement platforms, inclinometers, and piezometers. Two inclinometers, of the Sinco type, were installed on the eastern and western edges of the fill to monitor movements of the underlying soil. Groundwater levels were monitored through the inclinometer casings. Two clusters of pneumatic piezometers were used. The western group, labeled PZ-8a, b, and c, were installed at depths of 28.5, 53.3, and 78.5 ft, respectively; the eastern group, called PZ-9a, b, c, and d, were placed at depths of 25, 40, 55, and 70 ft. The settlements were monitored through observations on seven platforms: SP-31, 33, 34, 35, 36, 37, and 39. Each settlement platform consisted of a 2.5-in.-diameter steel pipe attached to a 3 × 3 ft wood base.

Drain Specification and Installation

Wick drains were installed on a triangular pattern with spacings varying from 6 to 12 ft under the fill. The closer spacing was used under the footprint of the proposed structure including retaining walls. A brand of wick drain was not spec-

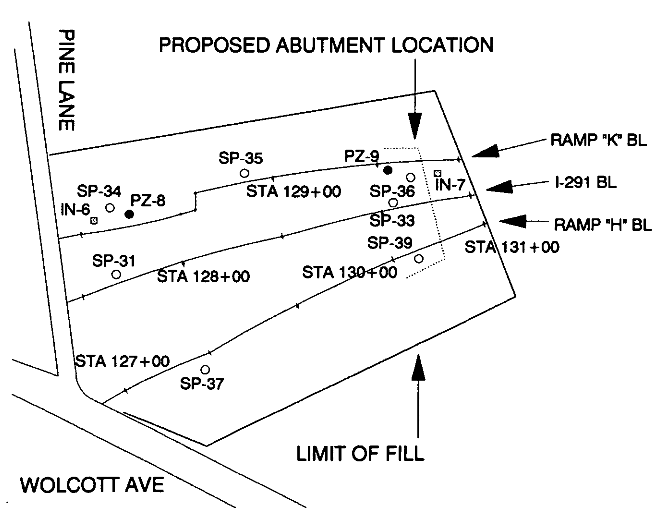

FIGURE 3 Plan of instrumentation.

ified. The generic specifications, included in special provisions of the contract, stated that the drain had to consist of a plastic core wrapped in a geotextile having a permeability of at least 6.0×10^{-4} ft/min, a free surface of 72 in.^{2}/ft of length, and a free volume greater than 7.56 in.^{3}/ft of length. The cross-sectional area of the mandrel of installation had to be less than 7 in.^{2}. The type of wick drain chosen by the contractor was Alidrain.

Each wick was to be within 6 in. of its plan location, and a vertical tolerance of 1 in. in 1 ft was specified. The wicks could not be installed by impact driving of the mandrel and were assumed to reach the bottom of the clay when the mandrel, pushing the wick through the clay, could penetrate no further.

A 2-ft-thick sand blanket was placed to handle the drainage from the wicks. The sand blanket had an undulated surface. The contractor's machinery could not hold the specified vertical tolerance when working on the steeper portions of the surface. The area was leveled with additional fill to facilitate installation of the drains. The additional fill increased the linear feet of wick installed. The location for each wick was determined by survey after the fill was leveled and marked with a small flag. A second problem was the resistance of the dense silt about the clay of the penetration of the mandrel. Each location of wick drain was first penetrated by a rod that was vibrated to top of clay. Once the silt was thus loosened, the wicks were easily installed.

The bottom of each wick was secured to the level of deepest mandrel penetration with a steel pin. On the average, about 100 wicks were installed each day. The rate was not affected by the depth of the clay. A total of 1.4 million linear ft of drain was installed.

Fill Placement

To ensure enough time for the settlement without causing construction delays, the area analyzed in this study was filled first. When the fill reached a height of 20 ft, the stability was monitored, as is CONNDOT practice, by comparing each increase of excess pore pressure with the corresponding increase of fill height. Lateral movements were monitored with the inclinometer readings.

SETTLEMENT CHARACTERISTICS—EXPECTED AND OBSERVED

Design Predictions

Design calculations, assuming one-dimensional compression, indicated settlements between 12 and 18 in. for the fill on this project. To have construction proceed smoothly, the waiting period to limit postconstruction settlements was not to exceed 18 months. Vertical drains were used because the time to achieve the necessary settlement without them was estimated to be 5 to 6 years on the basis of a vertical coefficient of consolidation of 1 ft^2/day and double drainage. Stability concerns, the proximity of existing roadways, and interference with proposed construction next to the site negated the use of large surcharges to reduce the waiting period.

The best estimates of the amount and rate of settlement for this fill are shown in Figure 4. The solid lines show the predicted behavior with vertical drains. The predicted curves shown in Figure 4 include only consolidation settlements. Initial settlements were not computed specifically, because experience has shown that the one-dimensional compression computations yield a settlement that is as great as the field values of initial plus consolidation settlement. The time axis in Figure 4 does not extend into the period of secondary compression.

Observations

Piezometers

No prediction of initial excess pore pressures was made before construction began. The piezometers were read daily from the time the fill reached a height of 20 ft until it reached its surcharged height. After the fill was completed, readings were taken weekly until the changes between successive readings were negligible. Piezometer readings with depth at five dif-

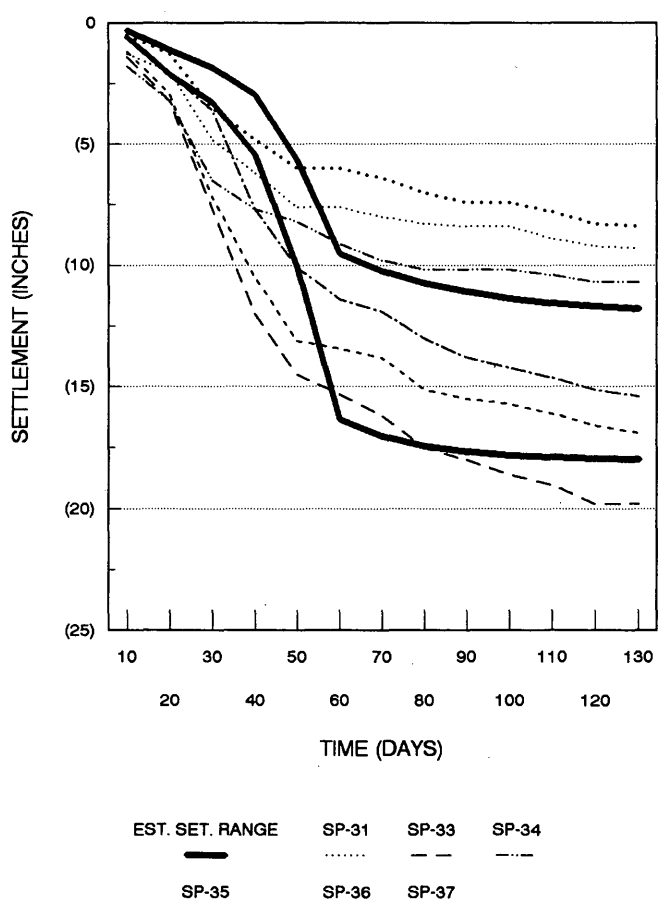

FIGURE 4 Predicted and observed settlement versus time.

ferent times are plotted in Figures 5 and 6. Figure 5 shows the pore pressures at PZ-8. The pore pressures at this location show little influence of vertical drainage. The pore pressures from PZ-9, shown in Figure 6, appear to show more influence of vertical drainage. This may be due to a difference in clay thickness. Both sets of piezometers showed little change in pore pressure after 130 days, which was used as an estimate of the end of primary compression.

Settlements

Figure 4 shows both predicted and observed settlements. The predictions were made using radial dissipation of pore pressures neglecting any vertical drainage. The waiting period allowed settlement observations to be made during both primary and secondary compression. The observed settlements up to 130 days are shown in Figure 4. From Figure 4, the observed settlements occur somewhat faster than predicted and are mostly within the predicted range.

Plots such as Figure 4 indicate general agreement between prediction and observation. However, specific consolidation

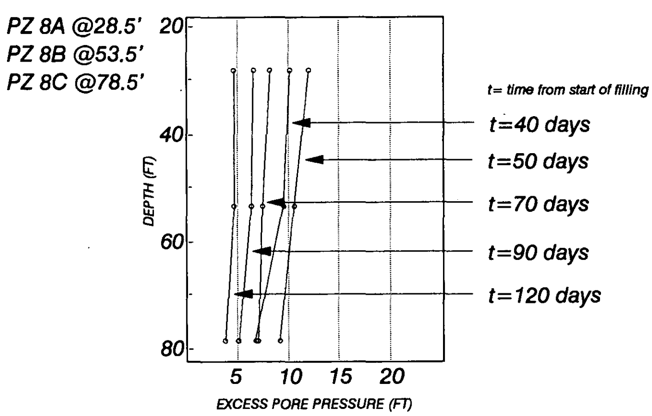

FIGURE 5 Plot of excess pore pressures with depth at various times for PZ-8.

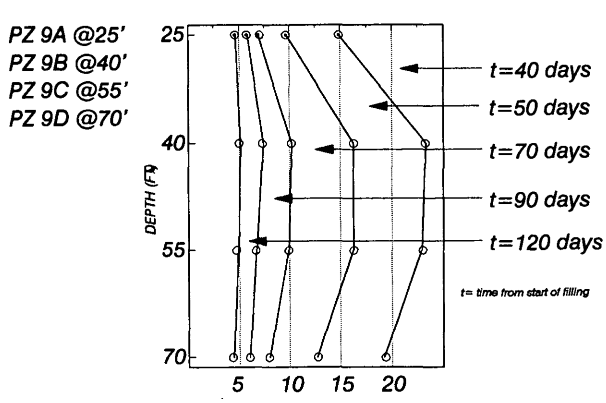

FIGURE 6 Plot of excess pore pressures with depth at various times for PZ-9.

properties from the field data are required to improve the accuracy of predictions on future projects involving the varved clays along the Connecticut River Valley.

DATA ANALYSIS

General

The techniques to analyze settlement and piezometer data have been published in detail elsewhere (*5*). A brief description of the procedures will be given here. These techniques are independent of laboratory data. The consolidation and settlement properties backfigured from the field data can be used to check the accuracy of the properties measured in the laboratory and their application in conventional design methods.

During filling, the load is changing, and the observed settlements may be due to increases in both initial and consolidation settlements. After filling is complete, the changes in observed settlements are due to changes in the percent consolidation, and the data can be analyzed for an apparent coefficient of consolidation and the total settlement. The total settlement is defined as the initial plus consolidation settlement. The backfigured apparent coefficient of consolidation in the radial direction includes any effects of pore pressure dissipation in the vertical direction. Knowing the ratio of horizontal to vertical permeability, the consolidation coefficients in the vertical and radial directions can be estimated (*6*). The analysis usually begins by assuming constant soil properties and using the equal strain theory of Barron (*7*). If the properties vary with level of effective stress, there are techniques for handling these cases when the settlement data are accurate enough to define the variation (*8,9*).

Piezometer data can be analyzed for coefficient of consolidation using the same basic theory (*7*). A manipulation of the basic equation for pore pressure dissipation with time derives a relation that can be used to analyze for cases showing a decrease in the coefficient of consolidation.

Pore Pressures

The dissipation of excess pore pressures at a point, for a soil having constant properties, can be described using the following equation (*7*):

$$\frac{u}{u_o} = A \exp(-Lt) \tag{1}$$

which can be written

$$\ln(u) = \ln(B) - (Lt) \tag{2}$$

where

u = the excess pore pressure at time t,
u_o = the initial excess pore pressure, and
A and B = constants.

Or

$$L = \frac{2C_r}{F(n)\, R_e^2} \tag{3}$$

$$F(n) = \frac{n^2}{n^2 - 1} \ln(n) - \frac{3n^2 - 1}{4n^2} \tag{4}$$

$$F(n) \approx \ln(n) - 0.74 \tag{5}$$

and

$$n = \frac{R_e}{R_w} \tag{6}$$

where R_e is the effective radius of the area serviced by the vertical drain and R_w is the equivalent radius of the vertical drain.

Equation 2 indicates that a plot of the natural logarithm of the excess pore pressure versus time yields a straight line for a clay that has a constant coefficient of consolidation and radial drainage. If the plot is concave upward, the coefficient of consolidation may be decreasing with time or the dissipation of pore pressures in the vertical direction may be affecting the readings. When conditions indicate that the curvature is due to a decreasing coefficient of consolidation, the following equation can be used to compute the instantaneous C_r (*10*):

$$\frac{du}{dt} = -Lu \tag{7}$$

To use Equation 7, the excess pore pressure versus time curve must be analyzed for slope at a selected time, which is then used with the values of the pore pressure at the same time to compute L. The value of the coefficient of consolidation at that time can then be computed from L with Equation 3. By repeating this process throughout the range of interest, the variation of the coefficient of consolidation can be determined.

The radius serviced by each drain was taken as 0.53 times the triangular spacing of the drains (*5*). The equivalent radius of the drain requires some consideration. The theory was derived on the basis of a circular drain. The wick drain is closer to a rectangular shape; therefore an approximation is required. Several methods have been suggested, but the method that appears to give the closest approximation is the following (*11*):

$$R_w = \frac{w + b}{4} \tag{8}$$

where w is the width of the wick drain and b is its thickness.

The excess pore pressures and their times of observation were first fitted with a polynomial curve using the program POLYMATH (*12*). Values of du/dt and u were computed from the best fit equation and used according to Equations 2 and 7. Piezometer data plotted according to Equation 2 showed a slight curvature upward but were fitted with the best fit straight line to get an average C_r during the observation period. Analysis according to Equation 7 showed the variation in the coefficient of consolidation.

The results of the analysis by Equation 2 are given in Table 1. The results of the analysis by Equation 7 are given in Table 2. These tables indicate that the values from Equation 2 are approximate average values of the results by Equation 7.

Settlement

The settlement data are plotted in Figure 4. Filling of the area required about 38 days. The observed settlements, after the filling and assuming that the properties are constant, can be described by the following equation (*5*,*6*):

$$S = S_t + S_c \exp(-Lt) \tag{9}$$

where

S = observed settlements,
S_t = total settlement = $S_i + S_c$,
S_i = initial settlement, and
S_c = consolidation settlement.

Equation 9 can be used in several ways (*5*). When the soil properties are known to be constant, the data can be fitted to Equation 9 directly using a computer program for nonlinear equations (*11*). Alternatively, taking the derivative of Equa-

TABLE 1 Coefficients of Consolidation Analyzed from Piezometer Data (Assuming Constant Properties)

Piez. No.	Elev. Clay Bound (ft) Top	Bott	Elev. of Piez. (ft)	C_r Piez. (ft^2/day)	Drain Spacing (ft)	Closest Set. Pl No.	C_r Set. (ft^2/day)
8b	-9	+75	-45	0.21	6	34	0.45
9b	-20	+75	-55	0.25	6	36	0.34
9c	-20	+75	-40	0.24	6	36	0.34

TABLE 2 Coefficients of Consolidation Analyzed from Piezometer Data (Assuming Varying Properties)

Time days	C_r (ft²/day) Piez. No. PZ-8b	PZ-9b	PZ-9c
50	0.36	0.41	0.39
60	0.30	0.35	0.32
70	0.23	0.28	0.25
80	0.20	0.23	0.21
90	0.17	0.18	0.21
100	0.15	0.18	0.21
110	0.14	0.18	0.21
120	0.08	0.12	0.15

tion 9 results in an equation that can be written

$$\ln\left(\frac{dS}{dt}\right) = C - (Lt) \quad (10)$$

where C is a constant.

Plots of ln (dS/dt) versus t are a straight line when the soil properties are constant. If the plot represented by Equation 10 is concave upward due to a decrease of C_r with time, there is a possibility of determining the range of C_r from the settlement data (*8,9,12*). This method was tried, but the data did not yield a consistently converging solution. A best fit straight line was made using Equation 10 and the slope of the line used to compute C_r.

Another method of analyzing settlement data is to plot successive settlements, as P_n versus P_{n-1}, observed at equal time intervals (*5,13*). It can be shown that a plot of these observations follows the equation

$$P_n = D + mP_{n-1} \quad (11)$$

where

$$\ln(m) = \frac{2C_r}{F(n)\,R_e^2} \quad (12)$$

Settlement data were analyzed by Equations 9, 10, and 11. The results are given in Table 3, and the results by the three methods compare.

Secondary Compression

Observations continued for more than 1 year, but primary compression was essentially complete at the end of 150 days. The observations beyond primary were used to compute the coefficient of secondary compression according to the expression

$$\frac{\Delta P}{h} = C_s \log\left(\frac{t_2}{t_1}\right) \quad (13)$$

where h is the thickness of the clay layer and C_s is the coefficient of secondary compression.

The field values of C_s are given in Table 4.

TABLE 3 Results of Analysis of Data from Settlement Platforms

Set. Plat. No.	Drain Spacing	C_r (ft²/day) (Eq.8)	(Eq. 9)	(Eq.10)	Total Set.	Obs.Set. @ 150 d.
	(ft)				(ft)	(ft)
31	12	1.37	1.19	1.29	0.81	0.78
33	6	0.28	0.29	0.30	1.78	1.70
34	6	0.34	0.56	0.41	0.91	0.91
35	6	0.35	0.28	0.34	1.35	1.31
36	6	0.25	0.26	0.25	1.51	1.45
37	10	0.45	0.39	0.61	0.87	0.72

TABLE 4 Coefficients of Secondary Compression

Set Pl No.	Drain Spacing	C_s
	(ft)	(strain/log cycle)
31	12	0.006
33	6	0.007
34	6	0.003
35	6	0.007
36	6	0.006
37	10	0.006
39	6	0.005

COMPARISON OF RESULTS AND PREDICTIONS

The results given in Tables 1, 2, and 3 show that the analyses yield consistent results for both the piezometer and the settlement data. The C_r values from the settlement platforms are slightly larger than from the piezometers because the settlement observations are more strongly influenced by consolidation in the vertical direction than the piezometers near the center of the clay layer. The C_r values are toward the low end of the values expected from the laboratory program. This may be due to smear around the drains at the time of installation.

The settlements are consistent with the location of the platforms in the fill. The magnitudes of settlement found by the approaches used here are consistent with the observed values at the end of primary consolidation. Most of the observed primary settlements fell within the estimates of 12 to 18 in.

The values of C_s are given in Table 4 and are within the range predicted from laboratory testing.

CONCLUSIONS

1. The coefficient of consolidation decreased somewhat during the time of observations, but the analysis techniques yielded reasonable values of the consolidation properties of the varved clay.
2. The laboratory test program yielded good design parameters for estimating total settlements.
3. The effectiveness of the wick drains in varved clay is somewhat reduced by smear.
4. The values of the consolidation and settlement properties, backfigured from the field data, will allow better predictions of field behavior on future projects with similar stratigraphy.

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