HIGHWAY RESEARCH RECORD

Number 133

Utilization of Sites of Soft Foundations

5 Reports

Subject Classification

62 Foundations (Soils)
33 Construction
22 Highway Design

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Washington, D. C., 1966 Publication 1377

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Foreword

This symposium was the outgrowth of a conference on the utilization for construction purposes of sites having soft foundations. Throughout the nation highway locations over soft foundations are becoming more and more common. The sprawl of urban development, zoning regulations, high land values, and high construction costs, along with the economic necessity for the shortest and most direct routing, have made it more difficult to avoid locating highways and appurtenances over sites of soft foundations.

This RECORD provides information that is especially valuable to design and construction engineers as well as to soils and location engineers. Theoretical aspects are discussed and methods of analyses are presented. Of utmost importance to any consideration of construction on soft foundation sites is the necessity of figuring into the total cost an estimate based on a predicted time of construction. In most of these cases the added time to obtain stability of the fill and foundation is a major item. Therefore an estimate which missed the actual settlement time required by an appreciable amount will add materially to the cost of the project. In one case this appears to be 2 to $2\frac{1}{2}$ times the theoretical estimate.

Smear of sand drains due to construction methods is one of the contributing causes to unrealistic estimates of settlement time. The use of the flight auger method in place of the closed mandrel method is proposed to alleviate the smear of the side walls. It will require good construction technique and close control but should result in settlement times more closely in agreement with the theoretical.

An "observational method" of evaluating the effects of construction and handling the unpredicted variations in the foundation is advanced as a contribution to more economical construction. This takes an orderly procedure of instrumentation and observation to produce the quantitative measurements necessary for interpretation by competent engineers.

The case histories described in several papers illustrate the economies of design and construction which result when thoughtful consideration of the variety of problems inherent in soft foundations is given prior to the design. Good exploration and analysis programs are essential. These histories also point up the need for additional study for the prediction of times and amounts of settlement arising from secondary compression. Several of them can serve as guidelines for those highway engineers who are more recently facing these problems due to the impact of the Interstate program.

This RECORD is a valuable contribution to the working knowledge so urgently needed by engineers across the nation so that they can design and construct the most economical yet highly durable and serviceable highways.

Contents

Construction on Marshland Deposits: Treatments and Results

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For the past 17 years, The Port of New York Authority has been reclaiming compressible marshland areas by filling and surcharging with and without the aid of sand drains. Responsibility for the maintenance of these areas has enabled the Authority to construct buildings and paved areas over marshlands, allowing for tolerable post-construction settlements. Four field cases are examined of post-construction settlements of structures built over compressible organic soil deposits. Laboratory tests show that these soils exhibit substantial rates of secondary compression. All sites were stabilized by surcharge fills, one site involving the use of sand drains. Two adjoining buildings are compared, one with sand drains and one without. Surcharge settlement records and piezometer data are evaluated to estimate the effective consolidation pressures which were achieved due to surcharging.

•CONSTRUCTION OF transportation facilities by The Port of New York Authority offers an unusual opportunity not normally available to the consulting engineer. Being both the owner and designer of these facilities affords a chance to completely follow through investigations from the conception of the project to many years after the structure has been completed. Therefore, post-construction settlement records can be kept, analyzed and compared with predictions made during the design period. In addition, being both designer and owner (responsible for maintenance), liberal engineering techniques can be used, improvised, and developed. For example, at the present time we are designing warehouse structures that will settle many inches in the future, but the large monetary savings in initial costs are well worth the small differential settlements and their small accompanying maintenance costs.

One reason for construction over marshland deposits is that most tracts of land available and large enough for airport and harbor development are in areas of marshland, which consists of soils of the weakest and most compressible nature (Fig. 1). These site conditions require filling operations to bring the land to a usable elevation and some means of supporting structures and pavements to prevent unacceptable large settlements.

Although piling has always been a tool for supporting structures in soft soil areas, advances in the art of soil mechanics and foundations have created new techniques for treating these areas. For the past 17 years, the Port Authority has been using the surcharging technique, on many diverse projects, to stabilize underlying compressible soils. Runways, buildings, roadways, bridges, open-paved areas, wharfs and water storage tanks rest over treated soils at many facilities. Just recently an area was treated inside an existing building, which shows the flexibility of this technique (1).

Paper sponsored by Committee on Embankments and Earth Slopes and presented at the 45th Annual Meeting.

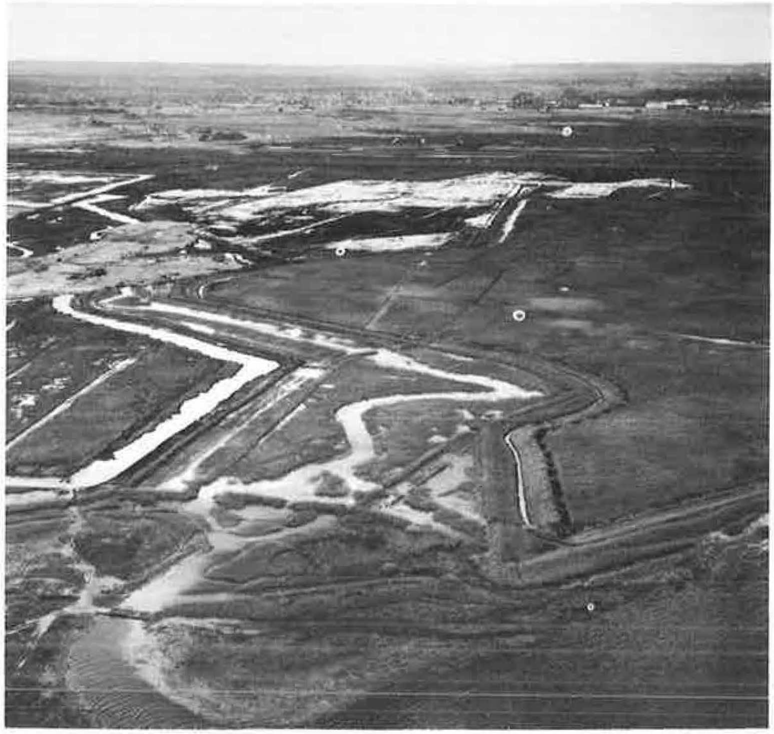

Figure 1. Virgin marshland at Elizabeth-Port Authority Marine Terminal.

Figure 2. Completed first phase of the Elizabeth-Port Authority Marine Terminal.

In this paper, field records are presented of post-construction settlements for several structures where techniques were used to stabilize the underlying marshland soils. The principal reason for writing this paper is that very little such field data have been published (2, 3).

Figure 2 shows more than 100 acres of completed area for the first phase of the Elizabeth-Port Authority Marine Terminal. This area, originally a wasted marsh, has been reclaimed and made into a successful operating marine terminal.

The leasing of Port Newark and Newark Airport in 1948, and purchase of the land for the Elizabeth-Port Authority Marine Terminal in 1955 (Fig. 3), gave the Port Authority a veritable proving ground for the surcharging technique. These facilities are built over a tidal marsh deposit consisting of weak and compressible organic silts and peats extending from about mean high water to a depth of 10 to 30 ft. The moisture contents range from an average of 100 percent for organic silts to as high as 600 percent for some unconsolidated peats. The organic deposits are generally underlain by a medium-dense, fine-grained sand or silty sand with a thickness of 5 ft or more. Below the sand are glacial lake deposits and bedrock. The glacial lake deposits consist of reddish-brown silts and clays, frequently varved, and preconsolidated to pressures of 3 to 4 tons/ft^2. Bedrock is a soft red shale and occurs at depths ranging from 40 to 100 ft.

Reclaiming this land by filling, both dry and hydraulically, has been in process since 1913. In general, the first fill placed on the tidal marsh has been dredged spoil excavated in constructing the Port Newark and the Elizabeth channels. In certain areas, however, miscellaneous fill has been placed by dumping and spreading directly on the virgin marsh. The spoil material is relatively weak and impermeable, and usually it is necessary to provide a granular subgrade under pavements. This land reclamation is still continuing with some 14 million cu yd of sand fill presently being pumped into adjacent Newark Airport, and another 6 million cu yd to be pumped into Port Newark and Elizabeth-Port Authority Marine Terminal in 1966 and 1967.

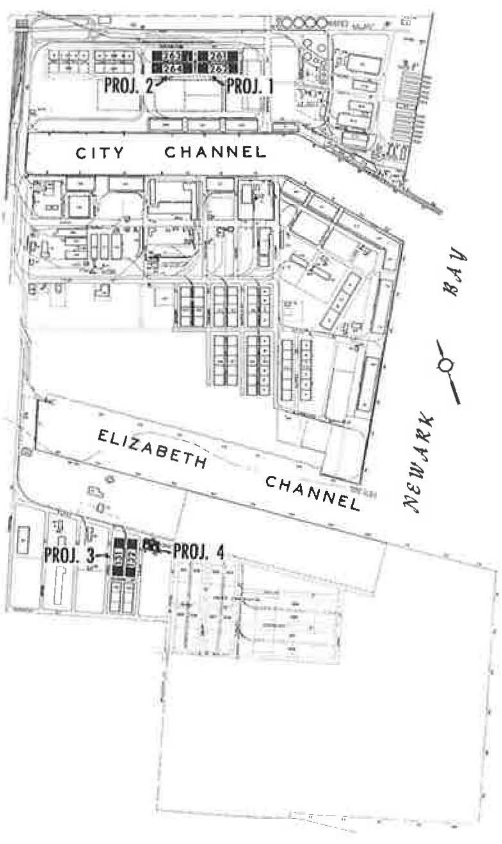

Figure 3. Location plan of Port Newark—Elizabeth-Port Authority Marine Terminal.

In building over the tidal marsh deposit the approach has been to use surcharge fills, both with and without sand drains, in order to consolidate the compressible organic soils. In the first buildings constructed by the Port Authority (1949) this technique was used only to support floors on fill, while the superstructure and firewalls were pile-supported (4). Over the years, this design has evolved so that now the entire structure is supported on fill without the use of piling. The buildings are single-floor, lightweight structures designed to provide flexibility in the superstructure.

METHOD OF ANALYSIS

In this report the settlements (ΔH) under surcharge loading (P_s) and design loading (P_d) are given by

$$\Delta H_d = \frac{C_c}{1 + e_o}\left[\log_{10}\frac{P_o + P_d}{P_o}\right] \quad (1a)$$

and

$$\Delta H_s = \frac{C_c}{1 + e_o}\left[\log_{10}\frac{P_o + P_d + P_s}{P_o}\right] \tag{1b}$$

The time-rate of settlement for primary consolidation is computed according to the well-known Terzaghi theory for one-dimensional consolidation (5), and according to Barron's analysis for radial flow to drain wells (6). Pore pressure dissipation, according to a nonlinear theory of consolidation proposed by Davis and Raymond (7), is also presented. This theory is described below and results are compared to the Terzaghi theory.

Settlements due to secondary compression are computed according to the log-time relation as given by

$$\Delta H_{sec} = C_{\alpha} H_o \left[\log_{10}\frac{t_{sec}}{t_{primary}}\right] \tag{2}$$

The above procedures seem to be in accord with current practice (8).

Since the Port Authority is continually building over the tidal marsh, it has been useful to develop correlations for the compression index C_c, the coefficient of consolidation C_v and the coefficient of secondary compression C_{α}. Laboratory data relating these parameters to the initial void ratio w_oG_s are shown in Figures 4, 5 and 6. The

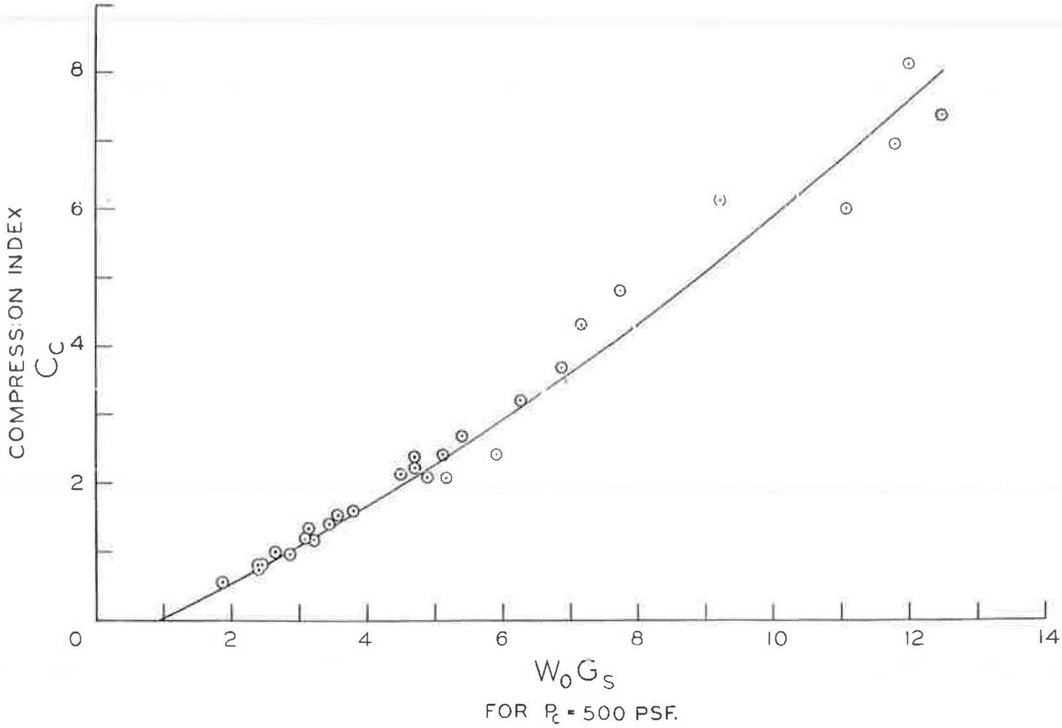

Figure 4. Relationship of compression index to initial void rates.

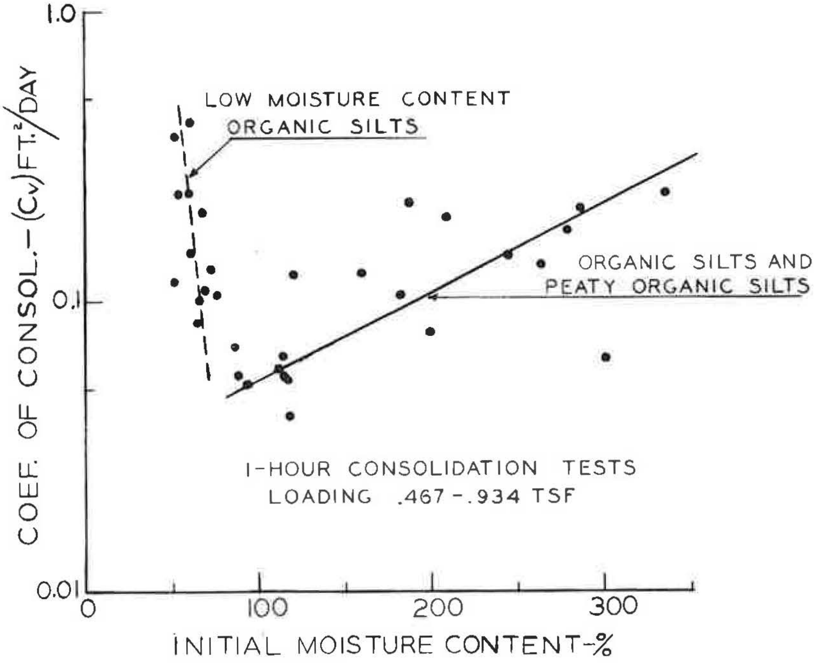

Figure 5. Relationship of coefficient of consolidation to initial moisture content.

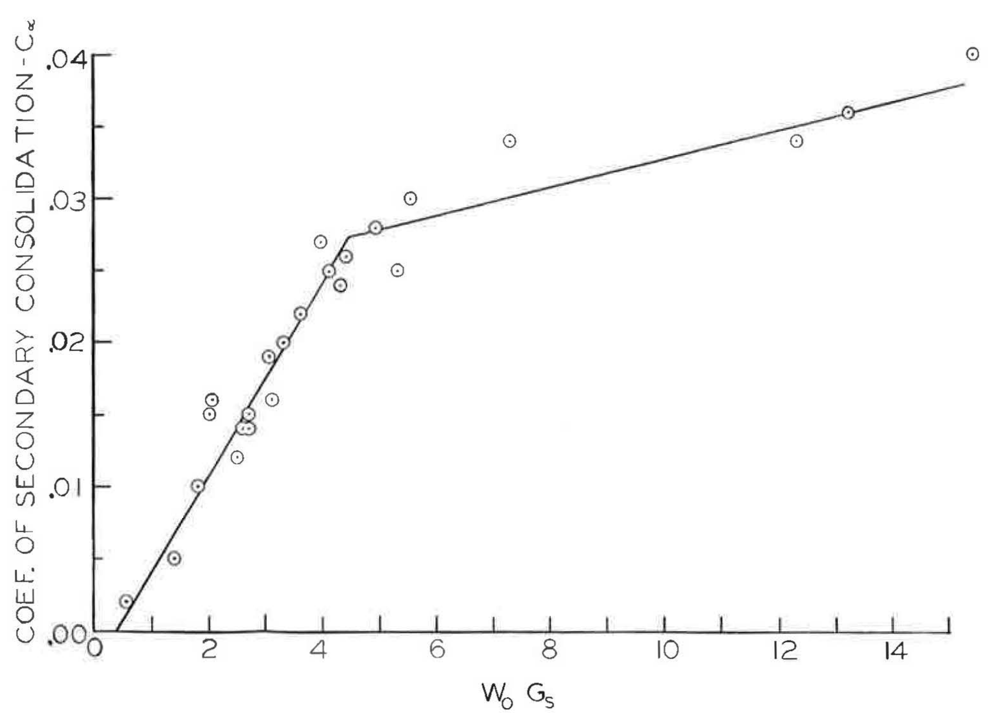

Figure 6. Relationship of coefficient of secondary compression to initial void ratio.

data shown in Figure 4, relating the compression index to the initial void ratio, have been corrected to a preconsolidation pressure (P_c) of 500 psf by

$$\Delta e = C_c \left[\log_{10} \frac{P_c + \Delta P}{P_c} \right] \tag{3}$$

This correction is necessary because the initial void ratio is a function of the preconsolidation pressure. With this curve for a preconsolidation pressure of 500 psf, a family of curves for various preconsolidation pressures can easily be constructed.

Because the moisture contents of the tidal marsh deposit are variable and erratic, it is usual to estimate the average soil properties for design purposes, from the average moisture content of all boring samples within a particular area. Strata of organic silt with moisture contents less than 80 percent are treated separately. Except where sand drains are used, the coefficient of consolidation is computed on the basis of the initial sample height (no shortening of the drainage path is made for reduction in sample height), and the length of the drainage path in the field is based on the initial stratum thickness. The results are essentially the same as when using corrected heights, but it facilitates computations to use initial heights.

Project 1—Port Newark Buildings 261 and 262

These buildings are single-floor, cargo distribution buildings with a truck-height floor designed for a cargo load of 500 psf. They are lightweight structures with siding and roofing of corrugated aluminum (Fig. 7). Except for the firewalls, which are pile-

Figure 7. Cargo distribution buildings at Port Newark buildings 261 & 262.

supported, the foundations rest on footings supported in fill overlying the tidal marsh deposit.

The buildings measure 160 × 640 ft in plan with 20- × 40-ft bay spacing. The roof framing consists of rafters supported by knee braces from the columns. These particular buildings are framed in timber; however, most recent structures of this type are framed in lightweight steel. Depending on the grade of steel used, the steel-framed structures will tolerate a maximum differential settlement of adjacent columns ranging from $1\frac{3}{4}$ to $2\frac{1}{2}$ in. Newer structures utilize stronger steels (A441), and this results in more slender sections and greater allowable differential settlements. Provisions have been made for jacking the columns, but this has not been necessary on any buildings to date.

A typical soil profile at the site (Fig. 8) shows a tidal marsh deposit consisting of 3 ft of organic silt and fine to very-fine sand over 13 ft of organic silt. Above the tidal marsh is 3 ft of miscellaneous fill. The results of odeometer tests show the tidal marsh deposit to be preconsolidated to pressures of 300 to 700 psf in excess of existing overburden. A value of 500 psf was assumed for the surcharge design.

To stabilize this site, 20-in. diameter sand drains were driven (plugged-end mandrel) on 11-ft centers in a square pattern, and a 13-ft fill (P_S = 190 psf) was placed for 7. 7 months. Settlement plates were installed at locations shown in Figure 9, and single-tube, Casagrande-type piezometers were installed adjacent to settlement plates 2, 5, 8 and 9. Settlement plate and piezometer data are presented in Figures 10 and 11, together with theoretical curves. Although the location of sand drains was not precisely known, the piezometers are assumed to be at the midpoint of the compressible layer and midway between drains. Settlements, as measured by the settlement plates, ranged from 0. 77 to 1. 33 ft; however, only the two center plates are presented due to the possibility that reduced edge-loading and shear settlement may have influenced the other plates.

There is generally good agreement between the theoretical curves and the measured data. The piezometer readings show somewhat higher pore water pressures than the theoretical. However, this may be due to either trapped gas in the piezometer tubes or to excess pore water pressure created by driving the sand drains. More recently we have been using double-tube piezometers, which are flushed out occasionally to evacuate any trapped gases. We have also been experimenting with air-operated pore pressure piezometers (air-water pressure transducers) designed by Dr. A. A. Warlam.

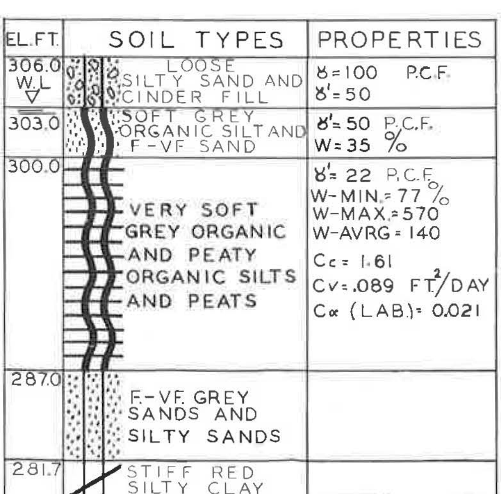

Figure 8. Typical boring at buildings 261 & 262.

The settlement plate readings indicate that the surcharge fill consolidated the compressible stratum to a higher effective pressure (U = 87%) than those imposed by the service loading (250 psf). This is true for both the average percent consolidation and for the consolidation at the midpoint. The pore water pressure measurements show lower effective consolidation pressures. Based on the highest piezometer reading, the effective consolidation pressure at the piezometer point is 100 psf less than the imposed service load.

After the removal of the surcharge fill (which was placed on Project No. 2) and the construction of the buildings, 78 settlement points were established in each building. The buildings have been in use since May 1960, and sufficient readings to establish rates of post-construction settlement have been obtained on 25 settlement points in building 261 and 18 settlement points in building 262. These readings show a mean settlement rate of 0. 284 ft/log cycle for

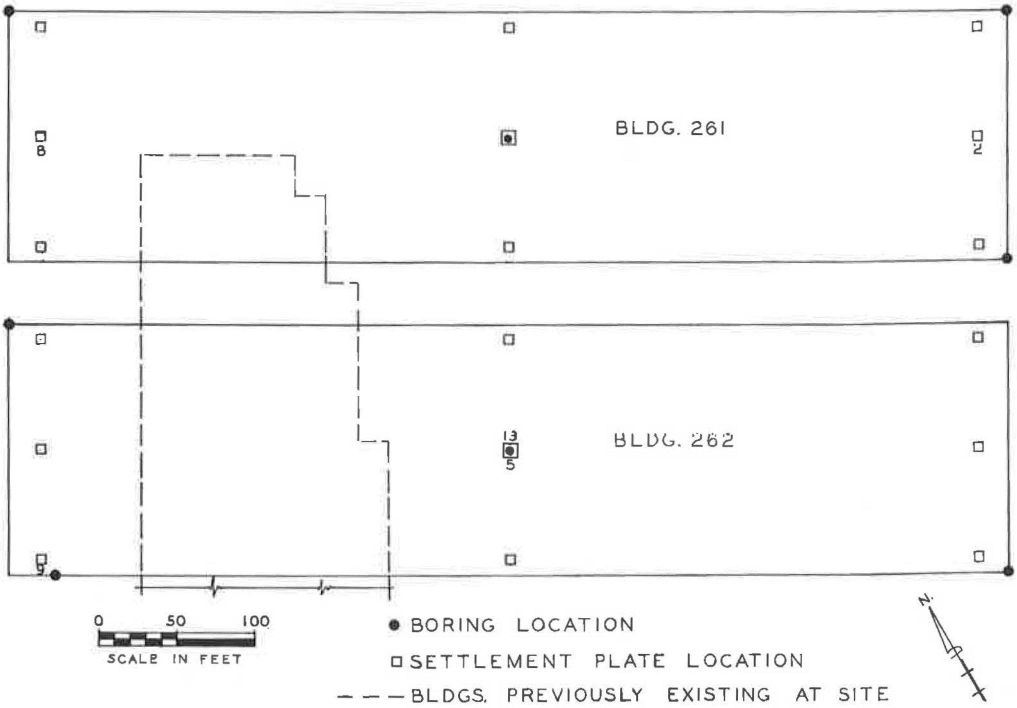

Figure 9. Location of borings and settlement plates.

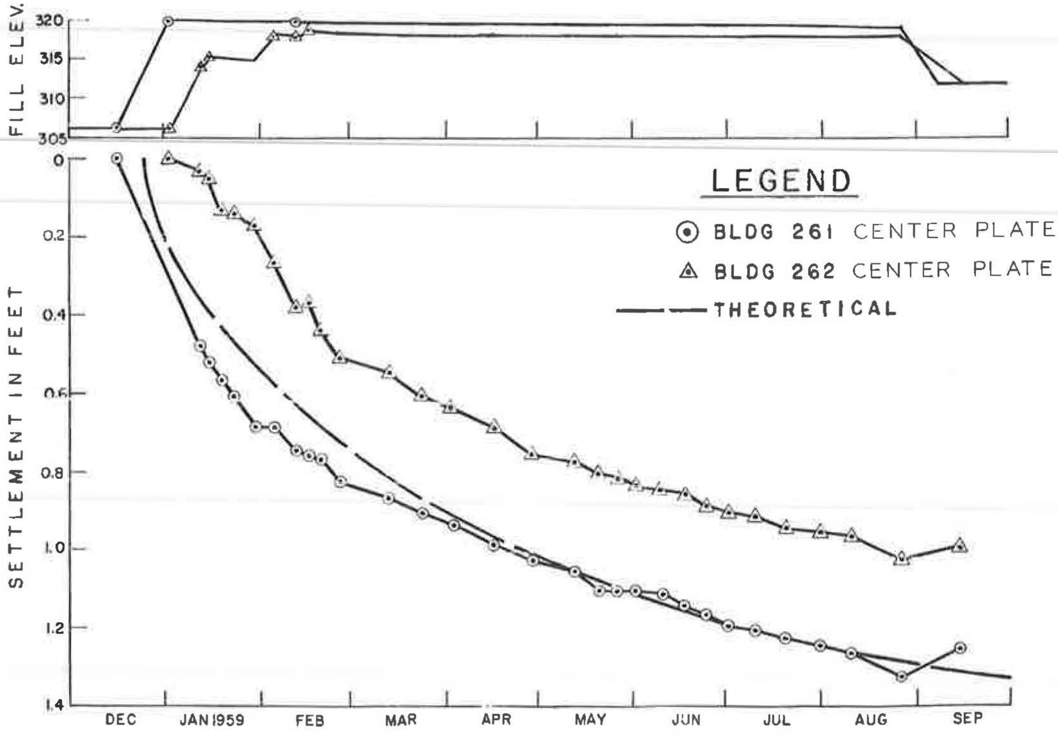

Figure 10. Settlements under surcharge for buildings 261 & 262.

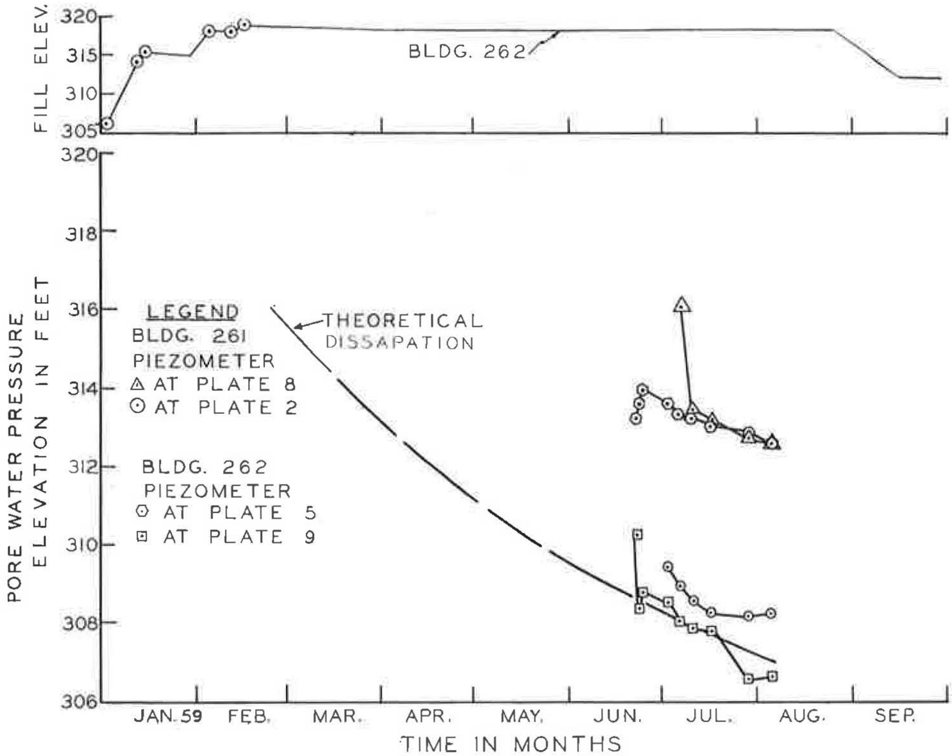

Figure 11. Piezometer readings for buildings 261 & 262.

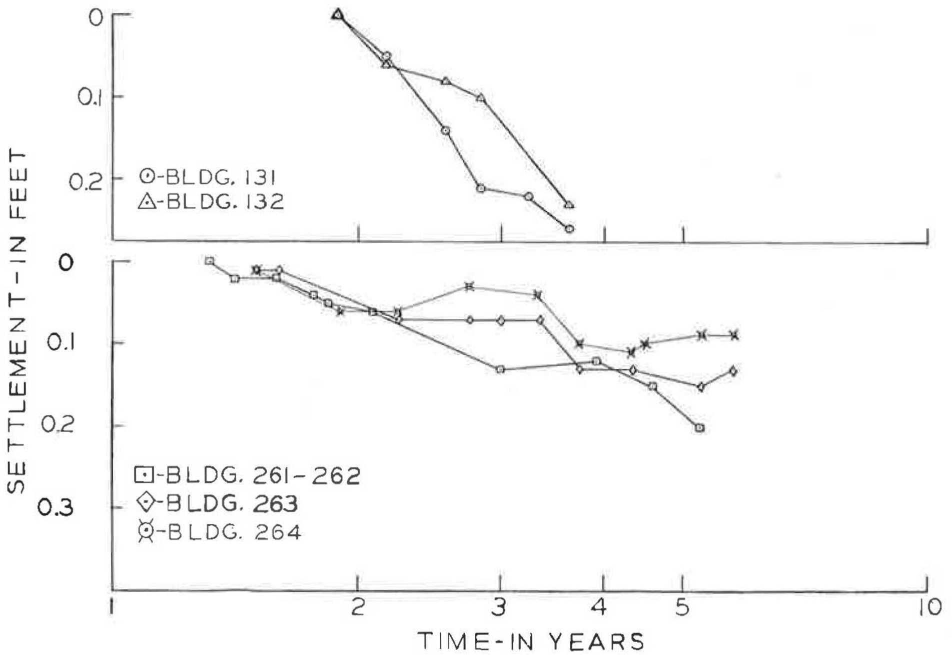

Figure 12. Post-construction settlements for buildings 261 & 262, buildings 263 & 264 and buildings 131 & 132.

building 261 and 0.310 ft/log cycle for building 262. This corresponds to an average coefficient of secondary compression of 0.023 strain/log cycle, and compares very well with the laboratory value of 0.020 strain/log cycle. The post-construction settlement of a typical settlement point is shown in Figure 12.

Project 2—Port Newark Buildings 263 and 264

A second pair of cargo distribution buildings were built on a site adjoining Project 1, where the subsurface conditions were quite similar. A typical boring (Fig. 13) shows a tidal marsh deposit 11.5 ft thick underlying 4.5 ft of loose, miscellaneous fill. As was found in Project 1, odeometer tests showed preconsolidation pressures on the order of 500 psf.

As a result of the experience gained from a test fill at the site, it was decided to surcharge without sand drains. The field results from this project and several later projects showed that sand drains could be eliminated by using higher surcharge fills and somewhat longer surcharge periods. At the time, however, it seemed to be a daring departure from usual practice. In part, our pessimism was due to the use of the log-time method to determine the coefficient of consolidation (C_V). This method yields slower C_V-values on organic soil deposits than Taylor's "square root of time" method, and field results seem to indicate that for these soils, the Taylor method is more nearly correct. Also, there is considerable scatter in the C_V-values, and for this reason it is preferable to develop a statistical correlation for the C_V-value based on many consolidation tests, as shown in Figure 5.

This site was filled and surcharged by placing 18 ft of fill (P_S = 625 psf) for a surcharge period of 8.2 months. The surcharge removal from another project was placed first and covered the site with 6 ft of fill. It was approximately two months later before fill removal from Project 1 could be released to bring the surcharge fill to its final elevation. As shown in Figure 14 the settlement of the surcharge fill under the initial lift of fill (Δp = 675 psf) follows quite closely the predicted curve based on an assumed preconsolidation load of 500 psf. There is generally good agreement between the theoretical settlement curve and the measured data, particularly for building 263.

A single-tube piezometer was installed at the location of boring 4 (Fig. 15). The pore pressure dissipation plot (Fig. 16) shows readings only for the final three months of the surcharge period. Actually the piezometer was installed about two months after the surcharge fill reached its final elevation. However, the earlier readings were erratic due to the use of an awkward measuring device, and have not been plotted. Also shown in Figure 16 are two theoretical pore pressure dissipation curves, one based on the conventional Terzaghi theory and the other as predicted by a nonlinear theory of consolidation proposed by Davis and Raymond (7). The nonlinear theory assumes a soil skeleton which compresses in accord with the log-pressure relationship (Eq. 1), whereas the Terzaghi theory assumes a linear coefficient of volume change (m_V) to act over the loading increment. The pore pressures predicted by either theory are in close agreement when the load-pressure increment $\left(\frac{P + \Delta P}{P}\right)$ is close to unity, but the nonlinear theory predicts slower dissipation of pore pressure for larger load-pressure increments. For this particular case the load-pressure increment is approximately 2.5, but a ratio on the order of 4 to 5 is not uncommon for surcharge fills over tidal marsh deposits.

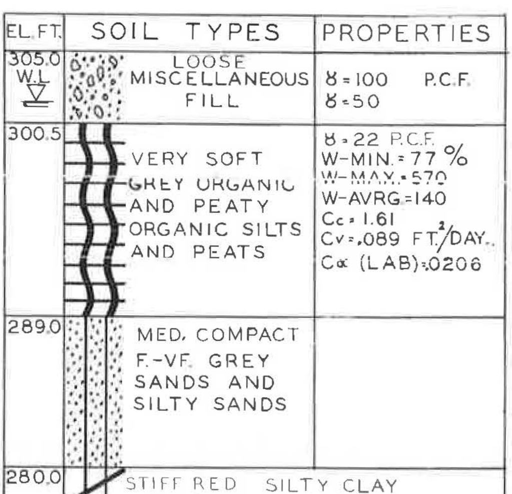

Figure 13. Typical boring at buildings 263 & 264.

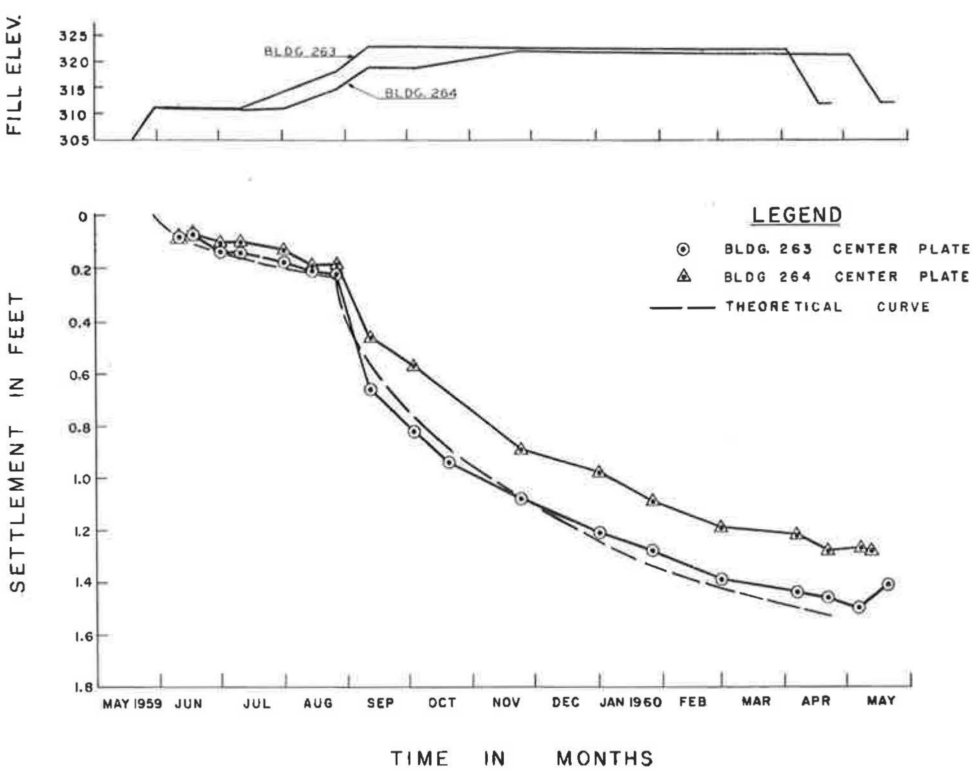

Figure 14. Settlements under surcharge for buildings 263 & 264.

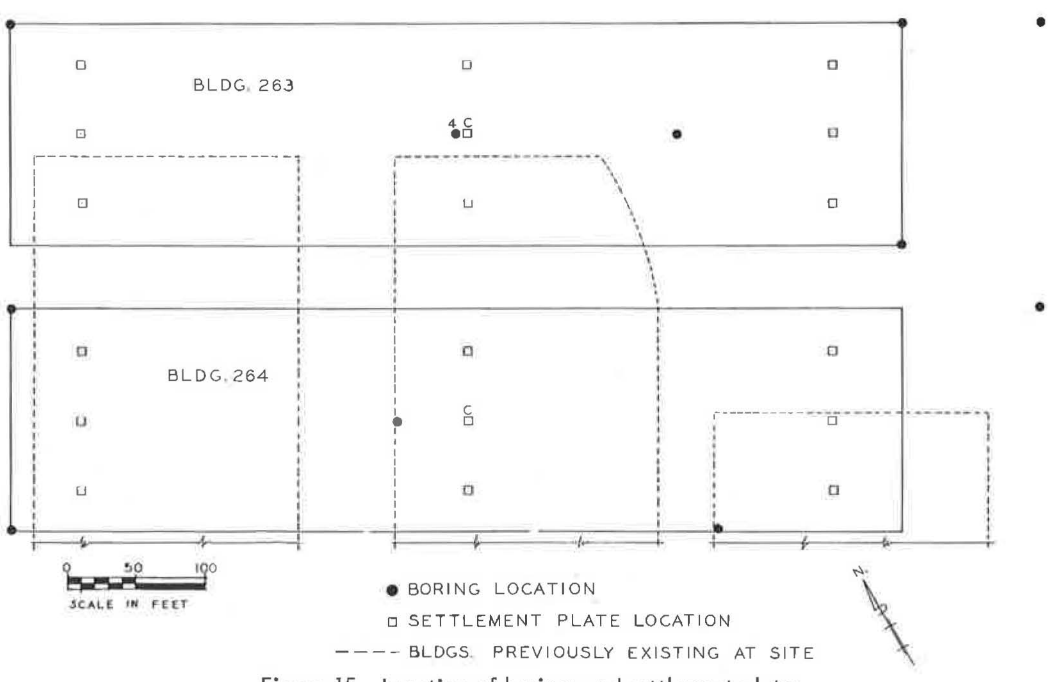

Figure 15. Location of borings and settlement plates.

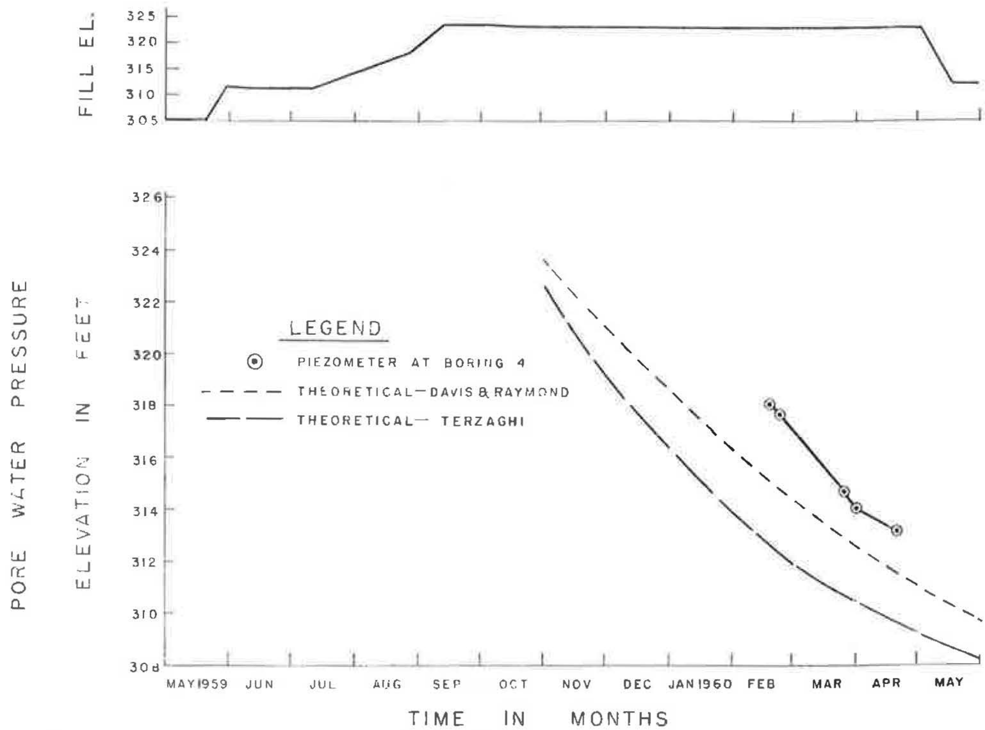

Figure 16. Piezometer readings for buildings 263 & 264.

The measured data are in better agreement with the pore pressure dissipation curve as predicted by the nonlinear theory; however, the differences are relatively minor and the measured readings could be high due to trapped gas or other causes.

Both the settlement plate readings and the piezometer readings indicate that the surcharge fill consolidated the tidal marsh to a higher effective pressure (U = 86%) than that imposed by the service loading (200 psf). As with Project 1, this was also true for the consolidation at the midpoint of the compressible layer.

These buildings were completed about January 1961, and readings on the permanent settlement points show a mean rate of post-construction settlement of 0.229 ft/log cycle for building 263 and 0.148 ft/log cycle for building 264 (Fig. 12). These rates correspond to a coefficient of secondary compression of 0.020 strain/log cycle for building 263 and 0.013 for building 264. The field value agrees exactly with the laboratory $C_{\alpha c}$-value for building 263, but the field value for building 264 is only 65 percent of the laboratory value.

Project 3—Elizabeth-Port Authority Marine Terminal Buildings 131 and 132

These two cargo distribution buildings are supported over subsurface soil conditions similar to Projects 1 and 2, except that the first fill placed on the virgin marsh was spoil from dredging the Elizabeth Channel in the summer of 1959. At this site the fill consists of loose sand underlain by a very soft, clayey silt. A typical boring, together with soil properties, is shown in Figure 17. Analysis of the settlement due to this initial fill indicates that the tidal marsh deposit was not fully consolidated when the surcharge fill was placed in early 1962. On the basis of settlement plate data and the results of consolidation tests, it was assumed that the tidal marsh was consolidated to an average pressure of 120 psf less than the existing overburden.

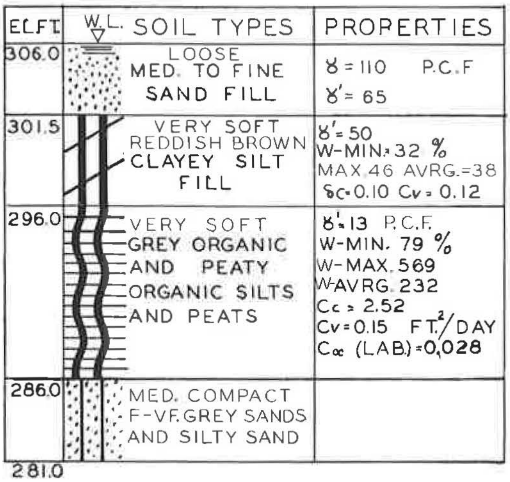

Figure 17. Typical boring at buildings 131 & 132.

In addition to 7 one-hour consolidation tests performed on the organic soils and the clayey silt fill, 3 consolidation tests were performed that duplicated the proposed field loading (Fig. 18). These tests consisted of loading the samples to the existing overburden pressure 0.31 tsf, and then applying the full surcharge load as a single increment ($\Delta P = 1.25$ tsf). When 90-percent consolidation was achieved, the load was reduced by an amount equal to the fill removal ($\Delta P = -0.66$ tsf) and secondary compression was observed for 6 days. To observe the effects of overconsolidation the loading sequence was then repeated.

One test, on a sample of gray organic silt, is shown in Figure 19. When the surcharge was unloaded the first time the sample showed a slight rebound; however, after a few hours it assumed the usual secondary compression plot with a coefficient of 0.0065 strain/log cycle. This is roughly one-half the virgin rate as shown in Figure 6. On reloading, the C_α-value was 0.012 strain/log cycle, which agrees reasonably well with the virgin rate. When the sample was unloaded a second time it continually expanded over a period of 6 days. These results show that overconsolidation has the effect of lowering the coefficient of secondary compression. This conclusion is in agreement with the findings of other investigators (2, 9).

This site was filled and surcharged by placing 24 ft of hydraulic sand fill ($P_s = 600$ psf), which was left in place for 15 months. As shown in Figure 20, the measured settlement ranged from 3.3 to 4.5 ft, somewhat greater than expected.

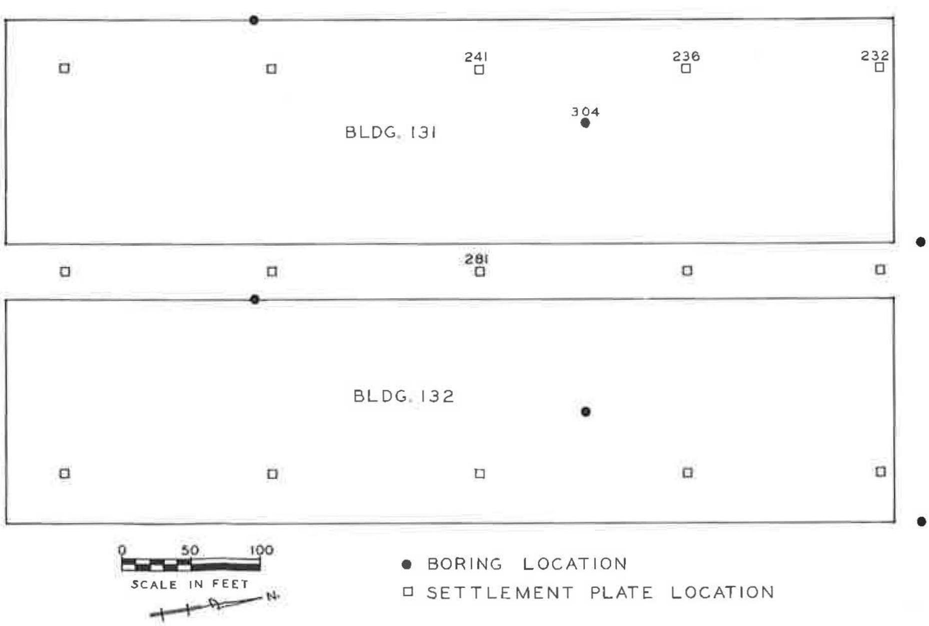

Figure 18. Location of borings and settlement plates.

Single-tube piezometers were installed in the clayey silt fill and the tidal marsh deposit, together with water level pipes in the upper sand fill and the lower drainage stratum. The two piezometers in the tidal marsh deposit showed lower excess pore

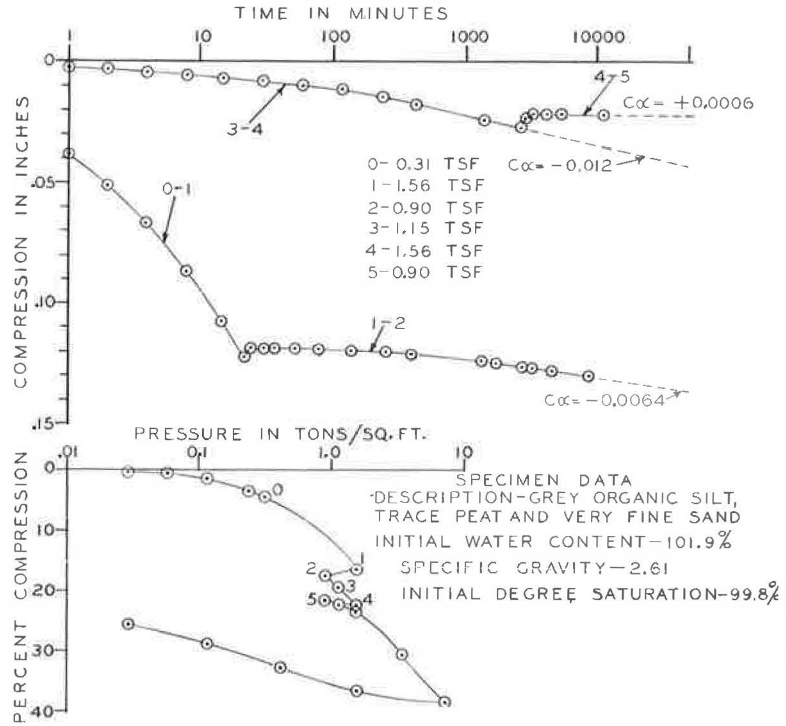

Figure 19. Consolidation test duplicating field loading.

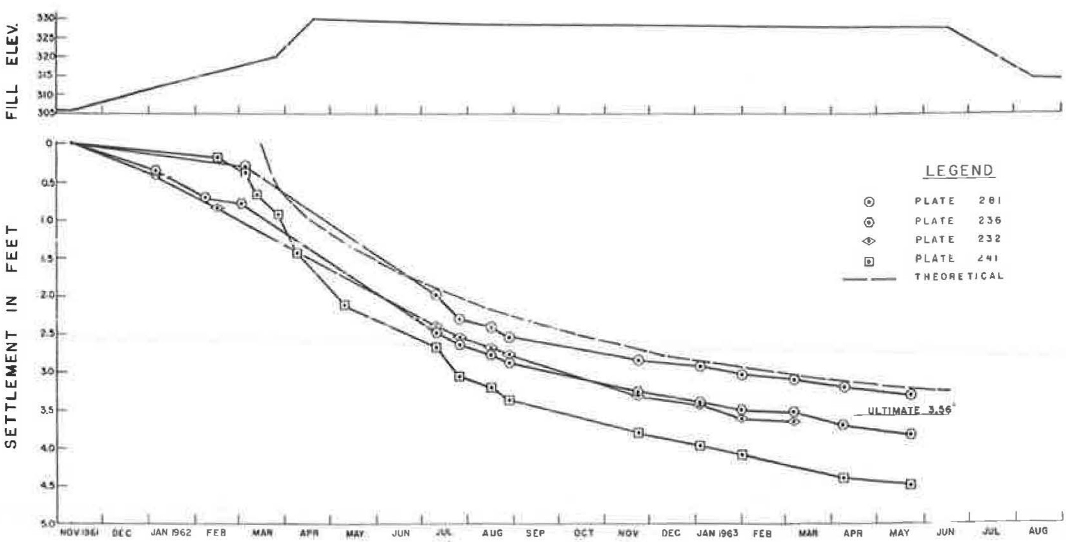

Figure 20. Settlement readings for buildings 131 & 132.

pressures than the equivalent loading increment, and little pore pressure dissipation. If the piezometers were functioning properly, the measurements would indicate that the tidal marsh deposit contains a significant amount of gas. The presence of gas in a soil causes the coefficient of consolidation to decrease during the loading increment (2). The settlement plate readings indicate that the surcharge fill consolidated the compressible strata to a higher effective pressure (U = 93%) than that imposed by the 500 psf service load. However, if the C_v-values were decreasing during the surcharge period the average percent consolidation would be less.

Both the water level pipe in the hydraulic sand fill and the pipe in the underlying drainage stratum showed initial excess pore pressures, which dissipated during the surcharge period. The lower drainage stratum was sandwiched between the tidal marsh deposit and an underlying stratum of relatively impermeable clayey silts. As a result, there was considerable lateral drainage in the lower drainage stratum. Since a large area was being surcharged at this time, the lateral drainage path was several hundred feet long, and higher excess pore pressures occurred than would have if only a small area was under surcharge.

The decrease in excess pore water pressures in the drainage strata is treated as a linear increase in the effective consolidating pressure with time. This condition is analyzed by using Schiffman's method for time-dependent loading (10).

The buildings were completed in March 1964. Readings on the permanent settlement points show mean rates of post-construction settlements of 1.08 ft/log cycle for building 131 and 0.78 ft/log cycle for building 132 (Fig. 12). This corresponds to a C-value of 0.109 strain/log cycle for building 131 and 0.090 strain/log cycle for building 132. This is three to four times greater than the laboratory rate of 0.028 strain/log cycle; however, it is possible that the post-construction settlements result from some combination of secondary compression and remaining primary consolidation. Similar rates of post-construction settlement have been reported by others (2).

These buildings have experienced 0.2 to 0.4 ft of settlement since construction. However, no structural distress or noticeable unevenness of the floor has occurred. Locally the settlements are very uniform. The firewalls in these buildings rest on a continuous footing supported in the sand fill. To provide maximum flexibility, firewalls are constructed in panels which frame into vertical steel columns. Lightweight waylite blocks are used to reduce dead load. No cracking of the firewalls has occurred, and as shown in a recent photograph (Fig. 21) of one of these firewalls, the floor is receiving every pound of the 500-psf cargo load for which it was designed.

Project 4—Port Elizabeth Water Storage Tanks

Two water storage tanks of one-million gallon capacity were required for the Elizabeth-Port Authority Marine Terminal. Various alternate designs were considered, ranging from small-diameter tanks supported on piles to preloading the site to support large-diameter tanks on grade with a small ring-wall around each tank. The results of this comparison demonstrated that the tanks supported on grade would cost approximately one-half the amount it would cost to support the tanks on a pile foundation (a saving of more than $100,000). However, it was appreciated that post-construction settlements would occur which would require future releveling of the tanks. The bottoms of the tanks were cambered 4 in. in order to postpone the eventual releveling. Jacking points were provided in the ring-wall to facilitate releveling of the tanks.

To increase the bearing capacity of the soil and to minimize the post-construction settlements, a preload fill was designed using stabilizing berms so that the safety factor against failure, using the Swedish Circle method of analysis, would be approximately 1.1.

The geologic profile at this site is similar to the profile described for Project 3. The thickness of the red silty clay dredged spoil material varied from 0 to 11 ft and the organic soils below varied in thickness from 7.5 to 14.0 ft. The variable soil profile beneath the tanks is shown in Figure 22. The moisture content of the organic soils varied from 50 percent to 570 percent, and this range might be encountered in any one boring. Detailed investigations were performed for the center and for several critical

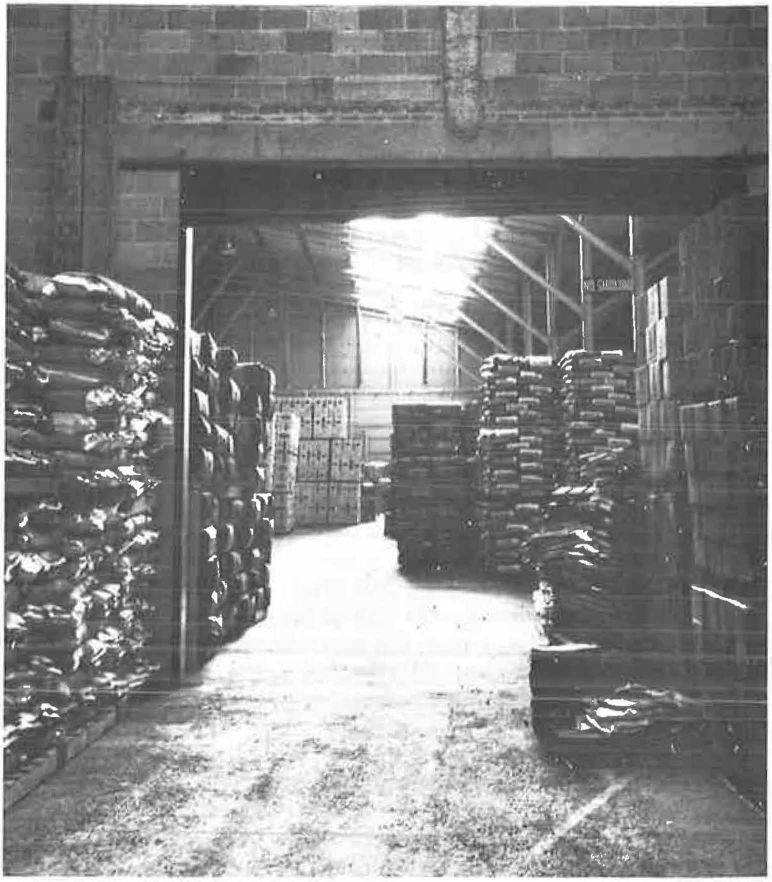

Figure 21. Interior of building 131 showing firewall and floor loading.

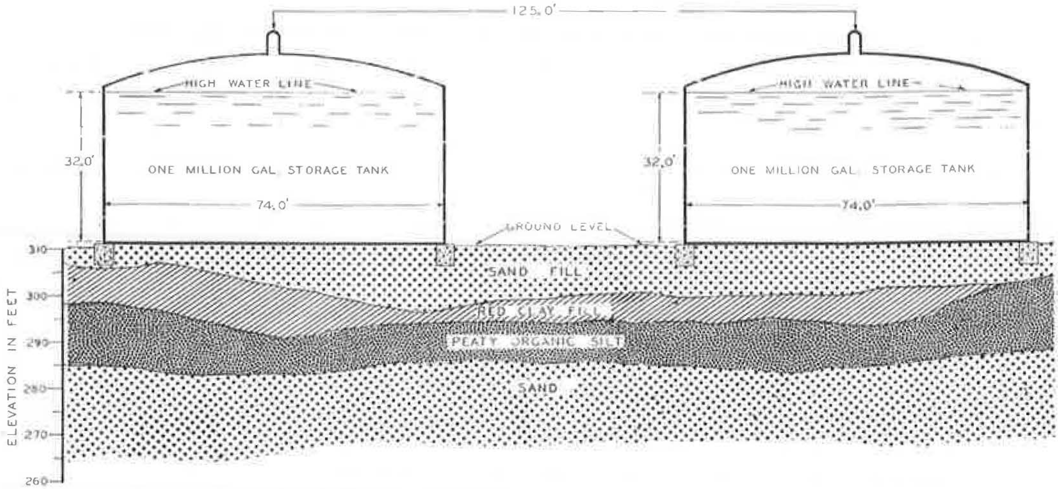

Figure 22. Geologic profile for water storage tanks.

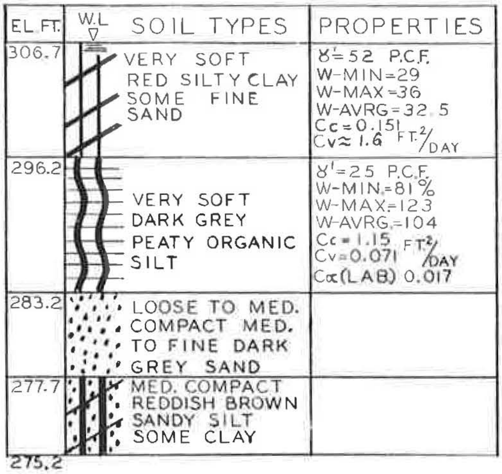

Figure 23. Boring 256 at water storage tanks.

points around the perimeter of the west tank. Boring 256 was considered representative of the soil profile at the center of the tank. The properties of the various layers in this boring can be seen in Figure 23. The coefficient of consolidation (C_V) of the upper stratum could not be reliably determined from lab test data because the range in values was so great. In estimating a C_V-value for this stratum, careful visual identification seemed to indicate that the coefficient of consolidation would be a large number, so a value of 1.6 ft^2 per day was assumed for the initial design. To understand better the behavior of these soils, and to determine the required length of time that the preload would have to remain, the site of the tanks was instrumented with settlement plates and piezometers at the future center of the tank (Fig. 24). Triaxial, consolidation and other tests were performed on samples from borings in the vicinity of this project.

Piezometers 3 and 4 showed that the dredged fill behaved similarly to what was initially estimated but was much more permeable. The observed rate of settlement for the most part was similar to what was anticipated; however, the total settlement was approximately 2.2 ft greater than initially calculated (Fig. 25). The initial investigation, prior to the installation of the tanks, considered the settlement resulting only from consolidation and did not consider settlement due to triaxial stresses. Part of the difference between the observed and theoretical settlement was due to the conservative selection of unit weights of the preload fill (Fig. 26); however, the major difference was attributed to strains due to triaxial stresses.

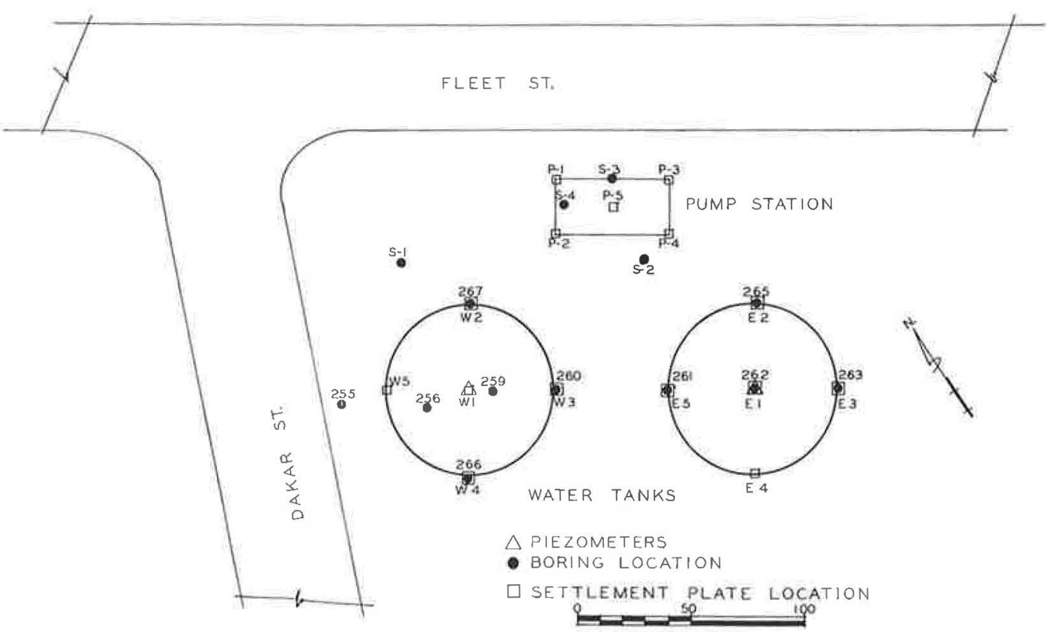

Figure 24. Location of borings and settlement plates.

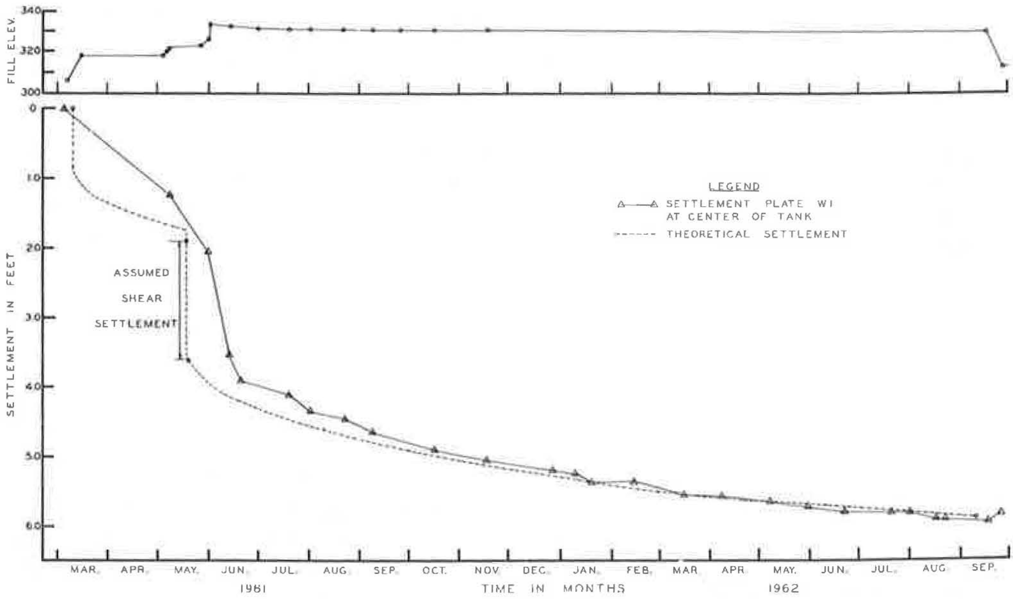

Figure 25. Settlement data for water storage tanks.

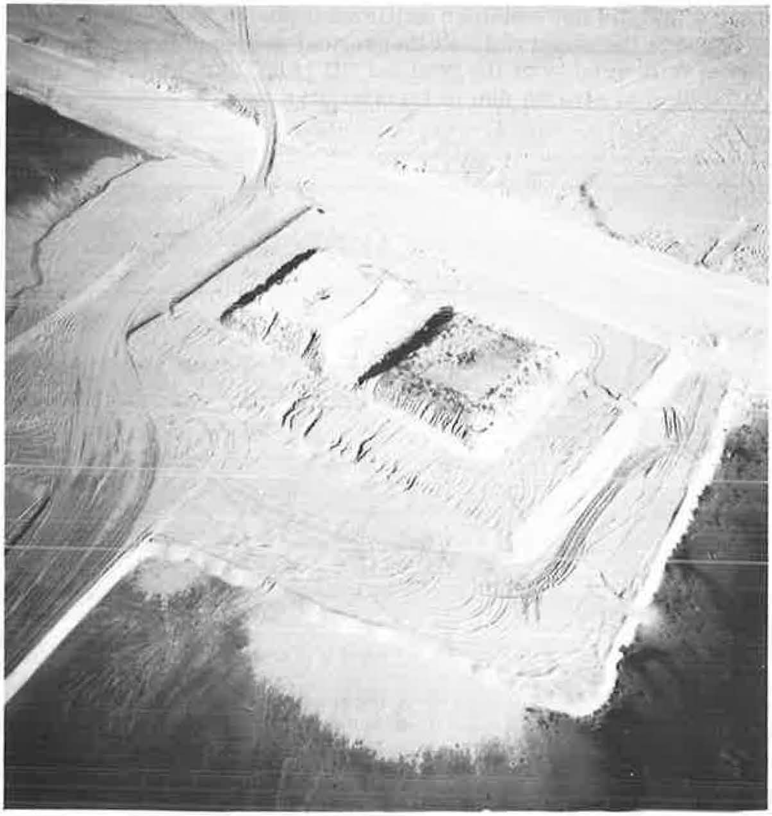

Figure 26. Preload for water storage tanks.

An attempt was made to estimate shear settlements by methods similar to those described by Skempton and Bjerrum (15) as well as Lambe (14). Triaxial tests showed that the A-value close to failure ranged from 0.75 to 1.17. The field value was observed to be very close to 1.0. In analyzing the strains due to the shear stresses, and considering the increase in strength due to the initial lift (a blanket load), it appeared that the middle 9 ft of the peaty organic silt layer under the center of the tank was stressed beyond its ultimate strength. The stress distribution used to estimate the stresses was based on the Boussinesq theory (12, 13). Since shear failure did not occur, either an increase in lateral stresses must have occurred when the soil was strained beyond its ultimate strength, or the soil was stronger than the results of triaxial tests seemed to demonstrate. Another obvious possibility was that the stresses were not best described by this theory. Considering only strains due to stresses below the ultimate strength of the soil, all but 0.28 ft of the observed settlement was accounted for. An additional average strain of 3.1 percent for the middle 9 ft of the layer would be required to account for the total observed settlement. The observed rate of settlement at the center of the tank corresponds to a degree of consolidation of approximately 90 percent, indicating that approximately 0.3 ft of primary settlement would occur as post-construction settlement due to the tank loading. The edge plates' observed rates of settlement indicated that the perimeter of the tank was overconsolidated, relative to the tank loading. It was also observed that when the preload was removed, the center settlement plates rebounded a minimum of 0.1-0.2 ft.

The piezometer in the underlying sand stratum measured excess pore pressure relative to the upper piezometer, indicating that the lower stratum was not acting as a perfect drainage layer. By using Terzaghi's theory, theoretical dissipation curves were fitted for piezometers in the peaty organic silt layer. The C_V-value used for the peaty organic silt layer was obtained from the regional relationship (Fig. 5). The C_V of the surface stratum that made the best fit was 300 ft^2 per day. Dissipation curves were also determined for the nonlinear theory. The pore pressures as predicted by this theory are higher than the pressures indicated by the Terzaghi theory. Piezometer No. 4 follows both theories relatively closely, falling between the two curves, but it is perhaps closer to Terzaghi's theory. Piezometer No. 2 readings are somewhat erratic, generally falling between the two theories. The piezometers used were single-

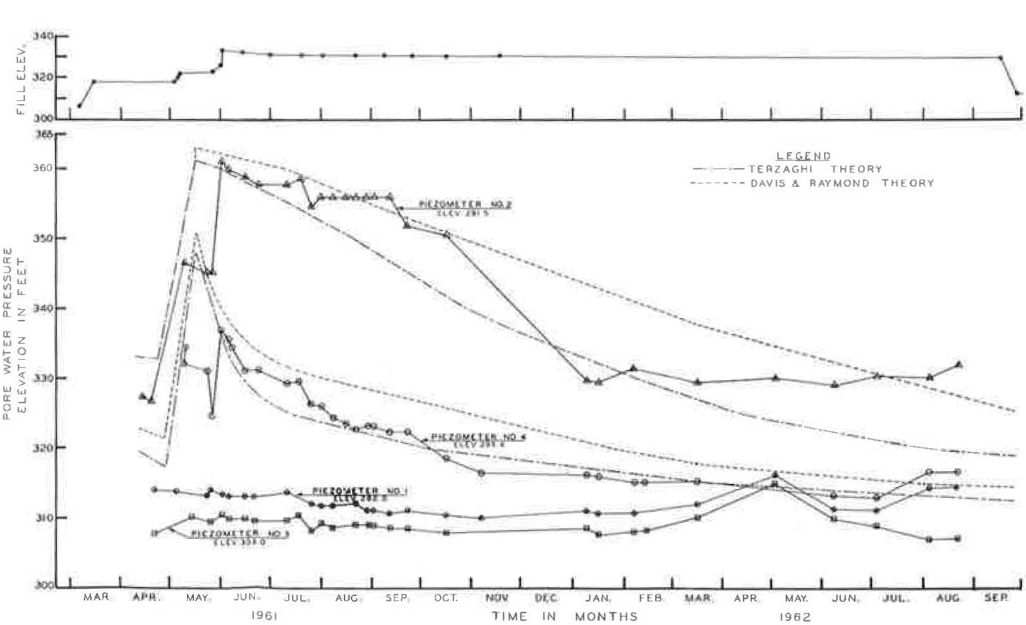

Figure 27. Piezometer data for water storage tanks.

tube Casagrandes piezometers that could not be flushed out; they were bled or filled with water to check their performance. It should also be noted that when adjacent areas were hydraulically filled, the piezometers at this site were observed to reflect a rise in pore water pressure (Fig. 27) starting in March 1962.

At the end of the preload period the preload fill was removed and the tanks were constructed. A load test was performed by filling the tank with water. The water test was performed by filling the tank in three increments to a height of 12 ft, 22 ft and finally to 32 ft. The total time to fill the tank was 14 days and the full load was left in for 85 days. Elevations at points on the ringwall were periodically obtained throughout the loading period. The tanks were emptied, painted and only at this time was the piping, with molox ball joints, connected to the tank. The post-construction settlements including the load test are shown in Figure 28. The maximum settlement to date is approximately 0.61 ft in 31½ months for one edge and perhaps 4 in. more for the center.

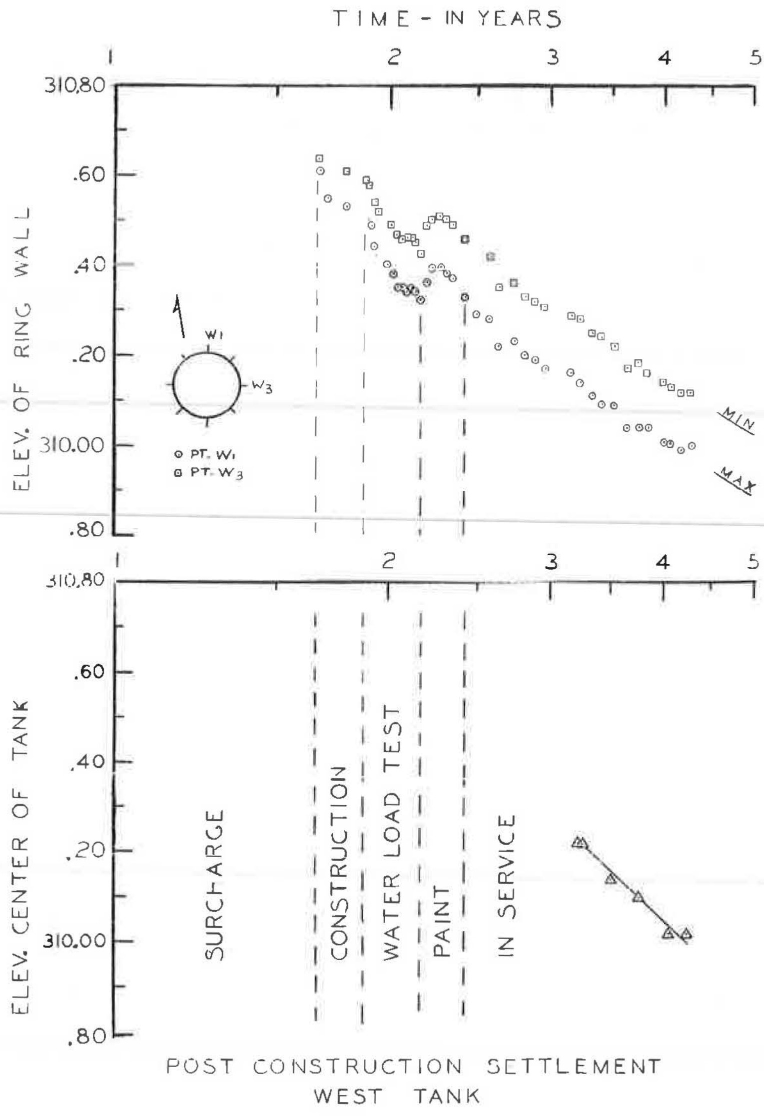

Figure 28. Post-construction settlements for water storage tanks.

The observed settlement is attributed to a combination of rebound recompression remaining primary (not eliminated by the preload), secondary compression, and creep due to triaxial stresses. During the early stages the post-construction settlement is considered to be primarily due to recompression, and perhaps also due to some remaining primary and creep due to triaxial stresses. The long-term settlements should be primarily due to secondary compression.

Burmister (11) performed creep tests on peats and organic silts, with the same range in moisture content and from the same vicinity as the soils reported in this paper. His tests demonstrated that the undrained shear strength of these soils is strain-rate dependent, the slower the rate of strain the lower the shear strength. The shear strength of peats and organic silts remained relatively constant when tested at a strain rate of 0.00001 of the normal strain rate of 0.55 percent/min. At this very low strain rate, the lowest shear strength obtained was 66 percent of the value obtained when performing the tests at the normal strain rate for the organic silts. However, it was somewhat higher for the peats. Burmister mentions in his paper that "The lowest equilibrium-sustained creep strengths of such soils have seldom dropped below 60 percent of the maximum obtained under 'closed system' conditions." The ratio of imposed shear stress to the ultimate shear strength beneath the center of the tank in the middle of the tidal marsh deposit was approximately 0.72, but lower throughout the rest of the stratum, at the start of construction. It will be 0.54 by the time the pore water pressures are fully dissipated. The ratio of the ultimate bearing capacity to the imposed load was greater than 2.5 at the start of construction.

Considering only the more current observations, the rates of post-construction settlement range from a minimum of 0.115 strain per log cycle for one edge to 0.145 for the center. These rates are 4 to 5 times the laboratory coefficient of 0.028 secondary compression, based on the average moisture content of all samples in the vicinity of the west tank. To date the tanks have not been releveled and have been performing well. It is anticipated that one releveling of the tanks will be required sometime in the future.

CONCLUSIONS

The four projects described show that by preconsolidating with the use of surcharge fills, it is feasible to support flexible structures over compressible soils which exhibit secondary compression. Since post-construction settlements will occur, it is desirable to make provision for future releveling.

Post-construction settlements generally followed a log-time relation (Eq. 2). However, the rates of post-construction settlements varied from 65 percent to 500 percent of the corresponding rates of secondary compression as measured in odeometer tests. Rates of post-construction settlements less than the laboratory rates of secondary compression are probably due to the effects of over-consolidation. At the present time (1966), there does not appear to be any satisfactory explanation of rates of post-construction settlements which are several times greater than the laboratory rates. However, it is possible that these post-construction settlements result from some combination of secondary compression together with some remaining primary consolidation. In cases where the compressible soil is being subjected to appreciable shearing stresses, it is also reasonable to expect that a portion of the post-construction settlement may result from creep.

There is a need for post-construction pore pressure measurements to establish if the dissipation of pore pressure is influencing post-construction settlements. Currently, installation of double-tube, Casagrande-type piezometers is being undertaken in many completed structures. However, it will be a few years before significant field data are obtained.

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Experimental Sand Drain Fill at Napa River

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The use of sand drains to accelerate the consolidation of soft, fine-grain soils has been widely used in California. Difficulty has been encountered in predicting the action of sand drains during the design of these projects. Accordingly, the experimental fill at Napa River was constructed to obtain information to better understand the action of sand drains. It was found that sand drains did not accelerate the consolidation of the soft foundation soil as rapidly as theory would indicate. It appears that the method of placing the sand drains with a closed mandrel was one of the main causes of reduction in the expected rate of consolidation.

•THE USE of sand drains to stabilize soft foundation soils has been widely used by the California Division of Highways and other organizations since 1934. There have been many successful projects and some others that did not perform as theory indicated. In 1957, a review of all previous sand drain projects in the Division of Highways was conducted to determine why some of the projects were not performing as expected. It was noted that the questionable projects were principally in the soft silty clays of recent geological origin. The decision was made to conduct an extensive test program at a sand drain installation constructed in an area where soft silty clay foundation soil exsisted. The rate of consolidation and strength increase of the soft foundation soil could then be studied and compared to the theoretical solutions. A step-type fill was to be constructed where sand drain and non-sand drain areas could be compared with the same fill loading, and where the fill height was sufficient that the stability would be questionable. It was thus hoped to diminish some of the uncertainties in the design of sand drain installations in soft silty clay foundation soils.

A program was planned to determine why the sand drains have not accelerated the consolidation of the soft foundation soil in all of the projects, as anticipated by theory. A comparison of areas with and without sand drains at constant height of fill was desired to determine the amount that sand drains accelerated the consolidation of the foundation soil. The effect of sand drains upon the stability of the fill was also to be studied. The west approach to the Napa River Bridge on Route 37 in Solano County provided the desired location for a test section.

SITE DESCRIPTION

The foundation soil at the west approach to the Napa River consisted of soft bay mud to a depth of 60 to 70 ft, reasonably homogeneous, with a firm silty clay underlying the soft bay mud. The natural water contents average above 90 percent and the in-place strength varied from about 100 psf at a depth of 10 ft to near 500 psf at a depth of 40 ft. The average soil properties are shown in Table 1. Without any special treatment this soil will support fills about 6 ft in height. With the use of struts it is possible to build somewhat higher fills.

The height of fill to produce the planned profile grade at the west approach to the Napa River Bridge varied from less than 5 ft to over 50 ft above the mud flats, depending

Paper sponsored by Committee on Embankments and Earth Slopes and presented at the 45th Annual Meeting.

TABLE 1

PROPERTIES OF SOFT BAY MUD
AT NAPA RIVER

Properties	Elevation -10 ft	Elevation -50 ft
Moisture, % dry wt	110	65
Compressive strength, psf	225	885
Wet density, pcf	85	95
Composition:		
Sand, %	0	
Silt, %	40	
Clay, %	60	
Plastic index, %	38	
Liquid limit, %	87	

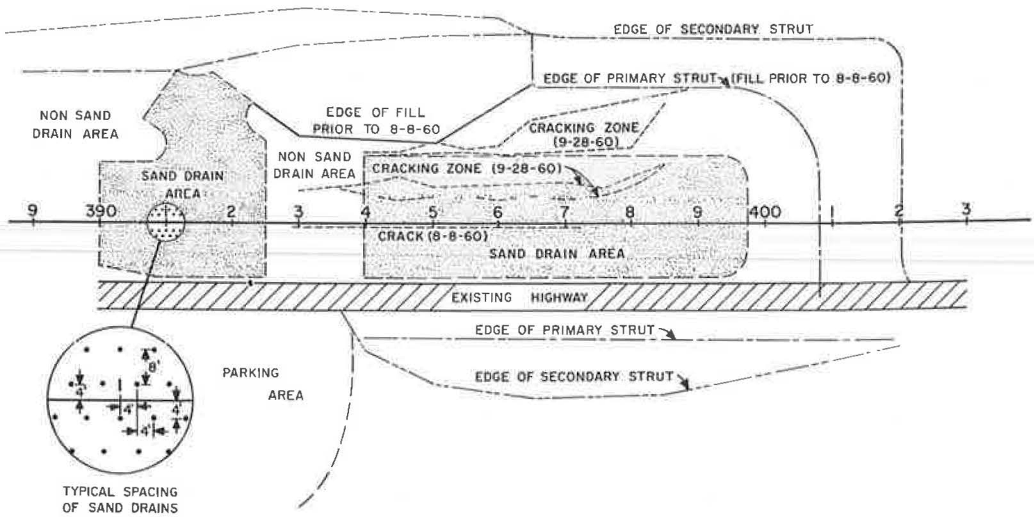

Figure 1. Plan of test site.

on where the structure ended. Construction of the fill would end, and the structure begin, when no special stabilization was used where the profile grade was about 5 ft above the mud flats. The cost of sand drain treatment and fill construction was estimated at about $120, 000 per station less than a structure.

The new alignment was adjacent to the existing roadway, thus avoiding interference with existing traffic. The west approach fill could be constructed prior to the structure under a separate contract, allowing sufficient time to construct and observe the fill (Fig. 1).

The soft bay mud was very impervious, having a permeability of 10^{-5} ft per hour. Theoretical calculations (1) indicated that sand drains on an 8-ft spacing would consolidate the soil at a rate which would increase the strength sufficiently during construction to allow loading of $1\frac{1}{2}$ ft of fill per week, with waiting periods required for the higher fill heights. In previous sand drain construction (2) on similar soils, theoretical

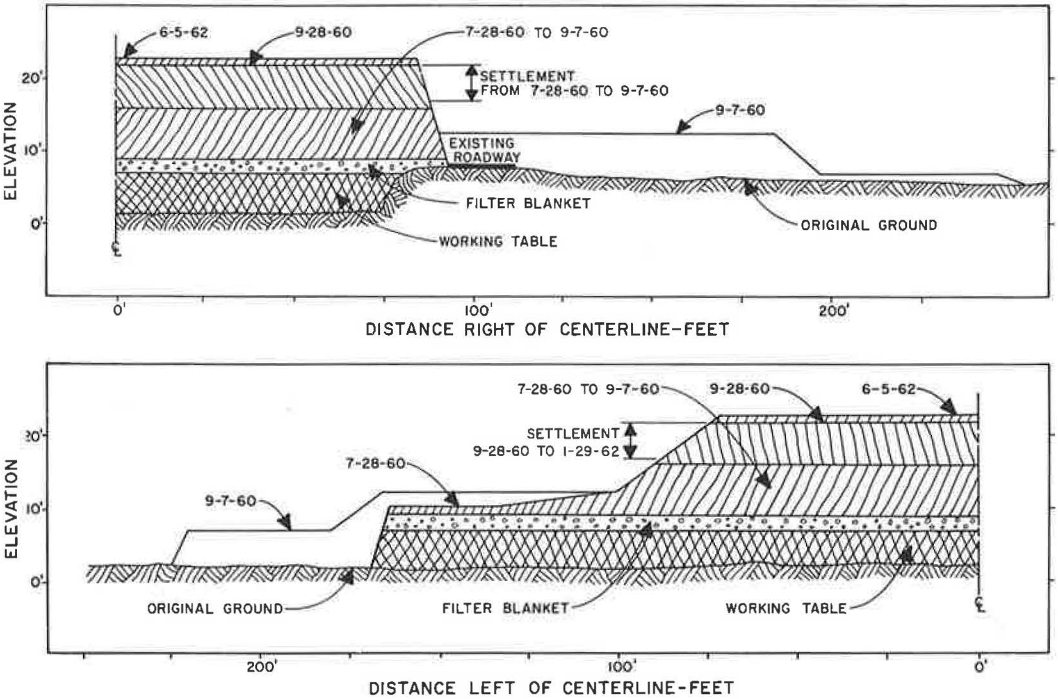

Figure 2. Typical cross sections.

TABLE 2

GRADING OF MATERIAL

Sieve Size	Percent Passing	
	Sand Drain	Permeable Blanket
1 in.		100
$^3/_4$ in.		97
$^3/_8$ in.	100	68
No. 4	97	52
No. 8	73	40
No. 16	53	29
No. 30	36	20
No. 50	16	11
No. 100	5	6
No. 200	3	5

solutions indicated that a spacing of 10 ft with a rate of loading of 3 ft of fill per week was required. The difference between Napa River and the previous sand drain projects was that the upper 10 ft of the foundation soil at Napa River had a very low shearing strength (2). With fills about 15 ft in height, berms 100 ft wide and 8 ft high were required on the left side and the existing roadway acted as a berm on the right side of the fill. Sand drains were installed under the main fill only (Figs. 1 and 2).

CONSTRUCTION

In February 1960, the construction of 1,200 ft of fill as a bridge approach on the west side of the Napa River was begun. This was to be an experimental project, a step-type surcharge fill with struts. After the designed height was reached, a waiting period would begin to allow observation of the fill.

Because of the extremely swampy nature of the foundation soil, a working platform was constructed over the tules and swamp grass. With the exception of a few large timbers, no clearing was performed. Care was taken to keep the working platform as thin as possible; it generally was about 5 ft thick. An occasional mud wave developed and when this occurred the mud wave was trapped by placing fill around it and covering it.

Approximately 2,500 sand drains were driven, varying in depth from 42 to 72 ft, using an 18-in. hollow mandrel with a hinged bottom. After the mandrel had been driven and the sand placed inside, compressed air was applied at the top forcing the sand out of the bottom as the steel mandrel was withdrawn. When the drains were completed, a 2- to 3-ft blanket of filter material was placed over the working table. The grading of the sand used in the sand drains and the filter material used in the blanket are shown in Table 2. To aid drainage of water from the filter blanket an 8-in. perforated metal pipe (PMP) was placed along the centerline. Construction of the fill was then started at a rate of one and one-half ft per week. After the main fill reached a height of about 17 ft cracks appeared, with the right side tending to move excessively (Fig. 2).

The placing of embankment on the main fill was discontinued and a strut constructed on the right side. Traffic was temporarily detoured to the right until additional fill could be placed on top of the existing pavement. After this fill was brought to an elevation of 14 ft, traffic was rerouted to the original location with the right strut reinforced as shown in Figures 1 and 2. As there were some indications that the left side was moving, the struts were widened to 160 ft on the left and their height near the main fill increased.

On September 27, with the main fill at an elevation of 22 ft and about 22 ft thickness of fill, the left side of the fill showed excessive movement. Numerous cracks from 2 to 7 ft in width appeared. The placing of embankment on the main fill was discontinued and a waiting period begun. This contract was closed on January 15, 1961.

Between March and June of 1962, the elevation of the main fill from Station 394+00 to Station 396+80 was increased to elevation 24 ft. The fill was at about elevation 17 ft at the start of this loading, and was placed at a rate of 1 ft per week. Material was obtained from the existing fill between Station 396+50 and 400+00. The structure would then begin at Station 396+00.

FAILURES

The movement when the main fill was at elevation +17 was minor in nature. The piezometers in the soft foundation soil to the right of the existing roadway showed an increase in pressure of 0.2 ton/ft^2 in about 1 to 2 hours' time. The slope indicator to the right of the roadway indicated that the upper 15 to 20 ft of soft mud below the roadway had moved slightly outward. The heave line on the existing roadway indicated an acceleration in the movement of the roadway surface to the right. Minor cracking occurred near the centerline of the fill with cracks opening less than one inch. Loading of the main fill was immediately stopped. Readings the next day indicated that the movement of the roadway was continuing so additional berm was then placed on the right side of the fill.

A cross section of the fill is shown in Figure 3 along with the minimum failure arc. Stability analysis indicated that no significant increase in strength of the underlying foundation soil had occurred. Two minimum failure arcs with factors of safety just below unity were located, indicating that heave of the roadway or existing fill was occurring. This agreed with the observations that no visible movement of the mud surface to the right of the existing fill occurred, and that the roadway fill was rising.

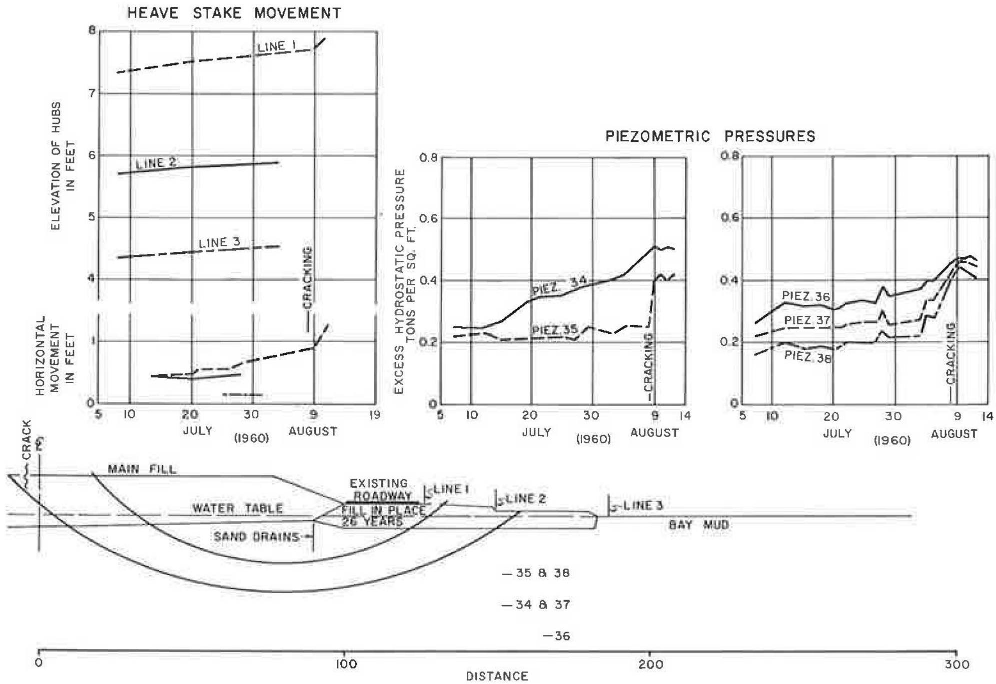

Figure 3. Movement on right side.

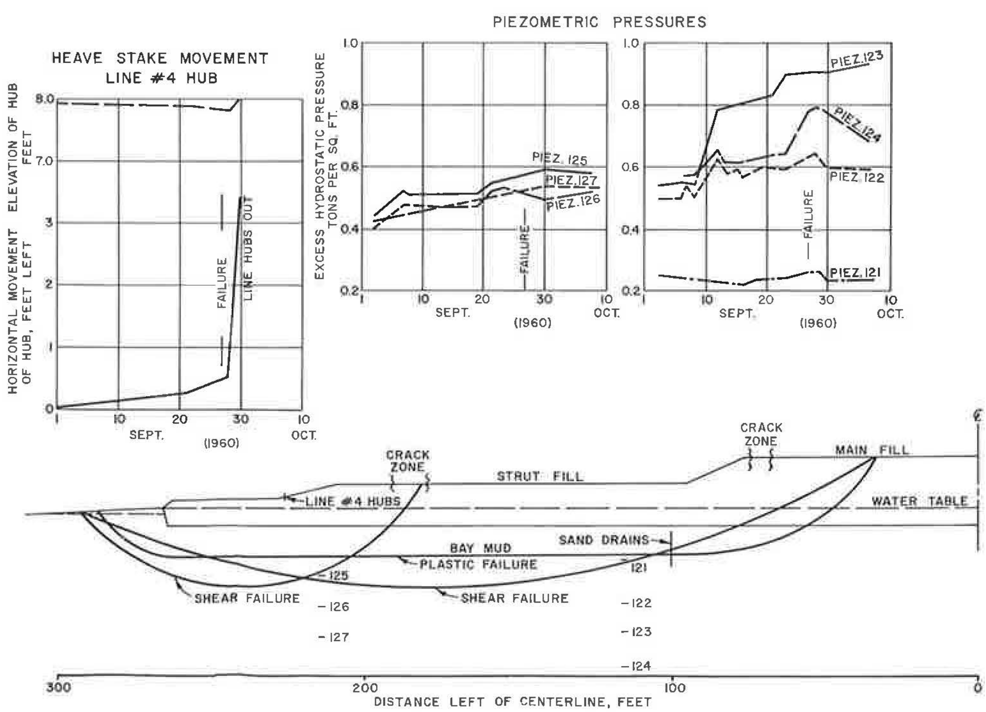

Figure 4. Failure on left side.

The movement on the right raised questions as to the ability of the berm on the left to support the main fill. To increase the stability of the main fill the primary berm on the left was widened 60 ft and the existing 100-ft berm increased in height to elevation 14 ft. To provide drainage of the filter blanket, 8-in. PMPs were installed through the widened berm. The increase in the width of the struts is shown in Figure 2. On September 8, 1960, the loading of the main fill was resumed.

On September 27, 1960, a major movement occurred on the left side as shown in Figures 1 and 4. The piezometers at the toe of the main fill had been indicating a slow, steady rise in excess hydrostatic pressure prior to the failure. The heave stakes had indicated a small steady outward movement, and the slope indicators showed that a minor flow of the soft bay mud was occurring at about elevation -10 ft. This movement occurred very rapidly with about a 4-ft drop in the left side of the fill.

The stability analyses of this movement are shown in Figure 4. The shear circle analysis indicated a factor of safety of unity existed. Using the stability analysis for plastic type movements developed from analysis of the failures at Candlestick Cove (3), the results indicated a factor of safety of 0.7. The rapid development of a large mud wave and the general appearance of the area indicated that this plastic type of failure had occurred. There was no indication that a significant increase in strength of the soft foundation soil had occurred.

This shallow shear failure through the upper portions of the soft foundation soil was not expected. It had been expected that the upper portions of the soft foundation soil would consolidate rapidly, increasing the shearing strength of the soil immediately below the berm. This would have resulted in a deeper failure surface with an increased length of the failure plane and in a higher strength soil. This failure surface in the upper portion of the soft foundation soil indicated very wide berms would be required to increase the length of the failure surface so that a stable fill could be constructed. This wide berm would have been uneconomical to construct. An alternative to such a wide berm would be the stabilization of the foundation soil under the berm by means of sand drains.

INSTALLATIONS

More than 200 installations such as settlement platforms, piezometers, borings, and other instrumentations were made at the west approach to the Napa River Bridge. Prior to construction of the fill, 19 sample borings were made. During and after construction 40 sample borings have been made. These borings were primarily to determine the strength and moisture content of the foundation soil. The sample borings were supplemented with 13 vane borings (4).

A total of 25 settlement platforms were installed on the surface of the working table. Four settlement platforms were also installed at the bottom of the soft foundation soil. The settlement platforms were for measuring the vertical movement of the surface on which they were placed.

To measure the excess hydrostatic pressures developed, 106 piezometers were placed at various locations. Seven piezometers were placed in the filter material to measure the excess hydrostatic pressure being developed on the filter system. Forty piezometers were installed for stability studies at the edges of the fill, 15 were installed in the soft foundation soil under the berms to study their rates of consolidation, and 44 were placed in the soft foundation soils below the main fill to measure the rate of consolidation in the sand drain and non-sand drain areas. All of the piezometers were of the nonmetallic type similar to those developed for use at Logan Airport (5).

The movement of the soft foundation soil was measured using 9 slope indicators (6). The movement of the fill surfaces was measured by the use of 6 heave stake lines totaling 4,700 linear feet.

Water tables were determined by means of 12 water table tubes in the fill area and in the foundation soil outside the fill area. These water table tubes consisted of perforated $\frac{3}{4}$-in. pipe placed in an augered 3-in. diameter hole and back-filled with sand.

The changes in the moisture and density of the soft foundation soil below the fill were measured by a nuclear subsurface gage and 8 nuclear access tubes consisting of steel tubing with $\frac{1}{16}$-in. wall thickness. The Nuclear Chicago subsurface nuclear probes were used to measure the changes in soil moisture and density (7).

SETTLEMENT

The settlement of the main fill along the centerline is shown in Figure 5. Settlement in the sand drain areas was about twice that in the non-sand drain areas during construction. After completion of construction, the sand drain areas have continued to settle at this same rate. The settlement in the sand drain areas is, in 1965, approaching the estimated ultimate settlement under the 3 heights of fill involved.

Typical time settlement plots are shown in Figures 6, 7 and 8. The estimated theoretical rate of settlement for the non-sand drain areas was calculated using Terzaghi's equations for double drainage (8). The theoretical rate of settlement for the sand drain areas was calculated using the equations developed by Barron, with no correction for well resistance, smear zone and soil disturbance (1).

The settlements in the non-sand drain areas have occurred about twice as fast as was estimated. For example, it was estimated that about 750 days would be required for 2 ft of settlement to occur under 15 ft of fill, whereas about 450 days were actually required (Fig. 7). There is a tendency now, in 1965, for the settlement in the non-sand drain areas to indicate a decreased rate on a logarithmic plot.

There are 3 possible reasons for the non-sand drain areas showing an accelerated rate of settlement: (a) the adjacent sand drain areas provided some horizontal drainage that would accelerate the consolidation in the non-sand drain areas; (b) the increased rate of settlement of the adjacent sand drain areas produced a drag down on the fill in the non-sand drain areas increasing the effective loading; and (c) settlement caused by plastic flow of the soft foundation soil. Considering these factors and the normal uncertainties in the calculation of the rate of settlement, reasonable agreement has been obtained between the measured and theoretical rates of settlement.

The settlements in the sand drain areas required about twice the time that was estimated from the application of Barron's equations (1). This resulted in a much smaller increase in strength of the foundation soil during construction than had been anticipated. There are several possible reasons for this reduced settlement. The resistance of the flow of water through the filter system would reduce the rate of settlement. This will be discussed later, in connection with excess hydrostatic pressure. The reduction in permeability of the foundation soil due to remolding when the sand drains are driven could slow settlement. By using the piezometers as permeameters (9) the permeability of the soil near the sand drain was measured and appeared to decrease greatly. This will be discussed in detail later. The peripheral smear produced by the drag of the mandrel could form a highly impervious zone. There was no way to measure this factor directly in this test section. The indications are that when the equations developed by Barron are used with soil and installation conditions similar to those at the west approach to the Napa River, the time for consolidation should be increased by a factor of 2 to $2\frac{1}{2}$.

An attempt was made to evaluate the above factors by using the equation developed by Schiffman (10). In his paper various factors affecting the rate of consolidation in sand drain areas are mathematically analyzed. The effect of various horizontal and vertical permeabilities and coefficients of consolidation can be evaluated, and the effect of peripheral smear can be studied. The ratio between the horizontal and vertical permeabilities averaged 2 and the ratio between the horizontal and vertical coefficients of consolidation varied from 2 to 3. Using these values and the measured settlement, the zone of peripheral smear was indicated to be about 3 ft from the center of the sand drain. This would indicate that placing the sand drains by the closed mandrel method remolded the soil for an area of about 2 ft around the sand drain.

The settlement platforms placed at the bottom of the soft mud indicated that less than 1 ft of settlement of the underlying stiff muds had occurred as of 1965. This would indicate that no appreciable error is involved in assuming that the measured settlement of the top of the working table represents the consolidation of the soft foundation soil.

EXCESS HYDROSTATIC PRESSURE

Piezometers were installed for two primary purposes. One was to detect instability of the fill during construction. These piezometers were placed near the toe of the main fill and at the toe of the berm, generally in the upper 20 to 30 ft of the foundation soil.

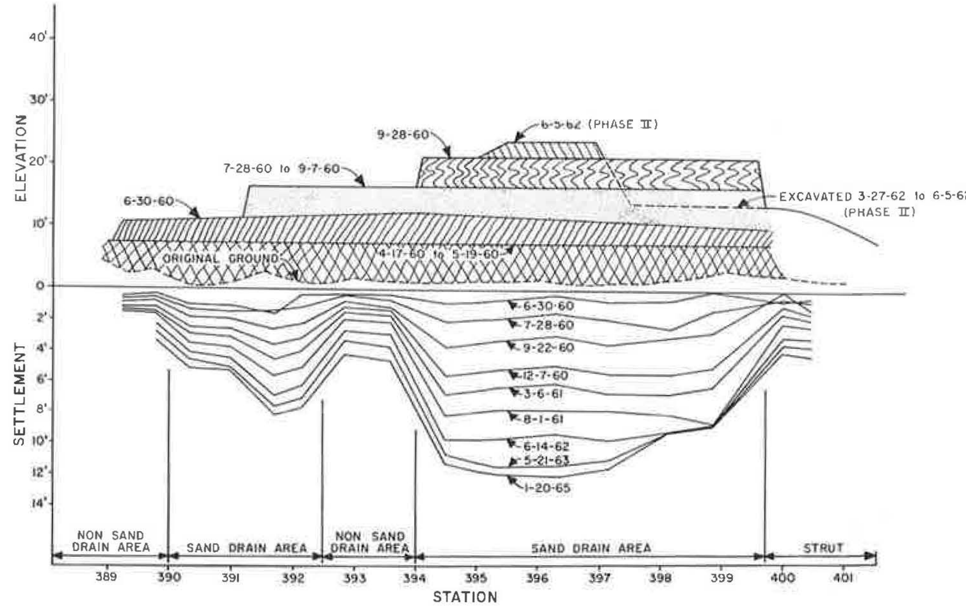

Figure 5. Settlement profile in sand drain and non-sand drain areas.

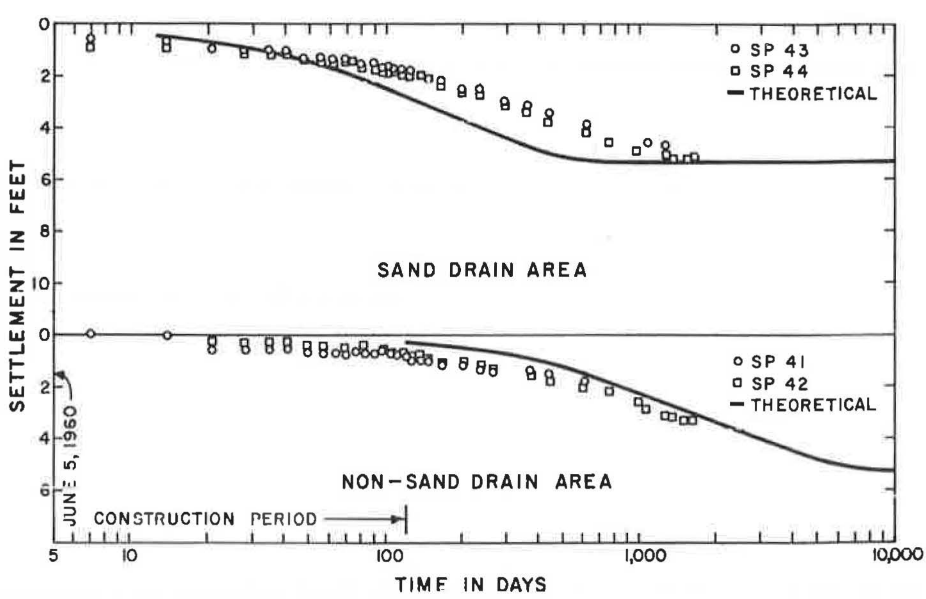

Figure 6. Comparison of theoretical and measured settlement, 10 ft of fill.

The other purpose was to measure the rate of consolidation of the foundation soil. To measure vertical drainage, piezometers were placed near the centerline of the main fill and in the center of the struts at about every ten feet of depth of the soft foundation soil in both the sand drain and non-sand drain areas. To measure the horizontal drainage in the sand drain areas, piezometers were also placed at elevation -15 in the cross section between 2 sand drains. Thus, vertical drainage in the sand drain and non-sand drain areas and horizontal drainage in the sand drain areas were measured.

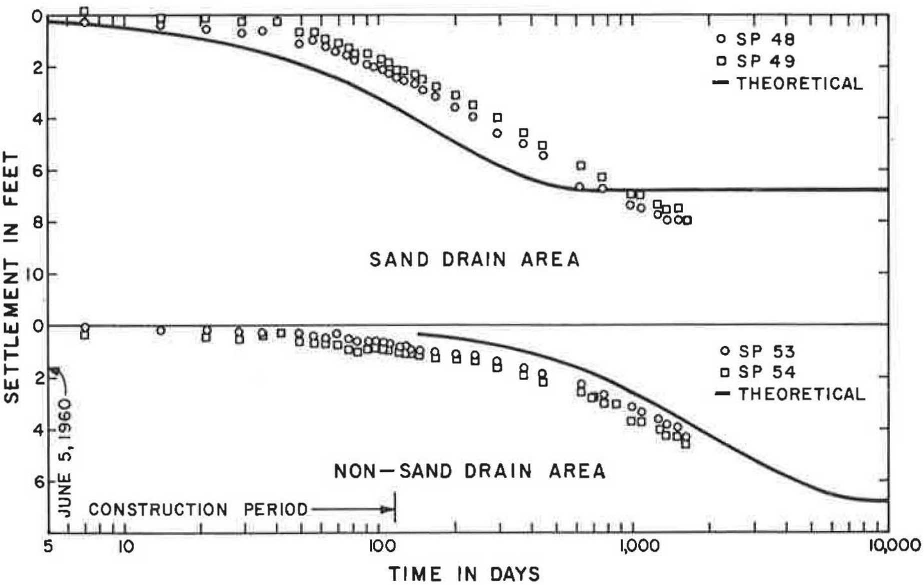

Figure 7. Comparison of theoretical and measured settlement, 15 ft of fill.

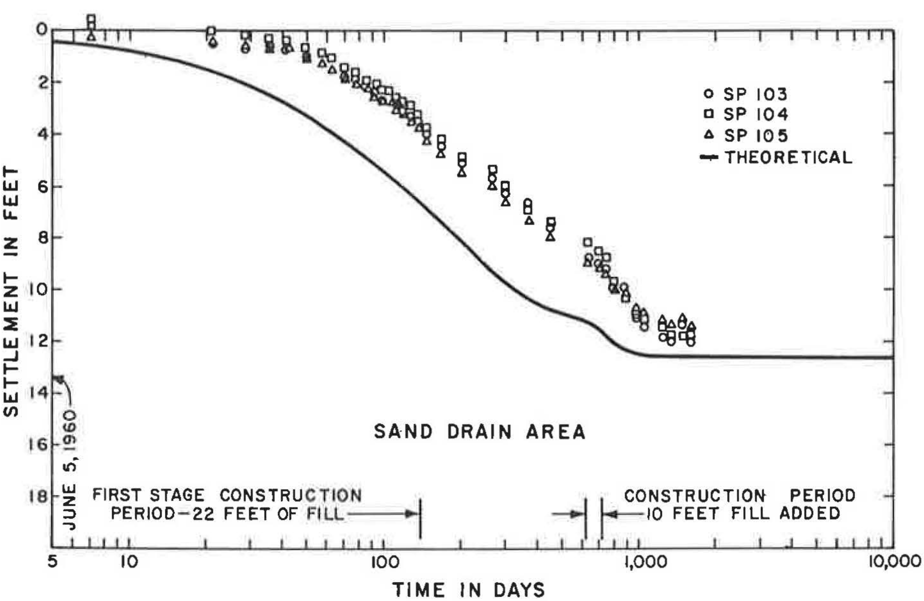

Figure 8. Comparison of theoretical and measured settlement, 32 ft of fill, 2 stages.

The piezometers placed at the toe of the fills for the predication of unstable conditions were partially successful in predicting failures. The shear type movement that occurred on the right caused an increase in excess hydrostatic pressure (Fig. 3). These increases in excess hydrostatic pressure were about 0.2 ton/ft^2. The warning given by these piezometers prevented a serious failure from occurring. However, with the plastic type failure that occurred on the left side, the piezometers did not show an excessive rise in excess hydrostatic pressure (Fig. 4). As the main fill was loaded, the

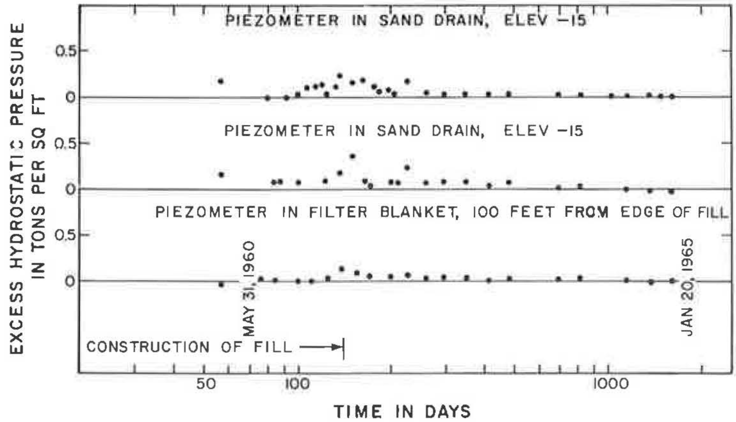

Figure 9. Water pressure in filter system.

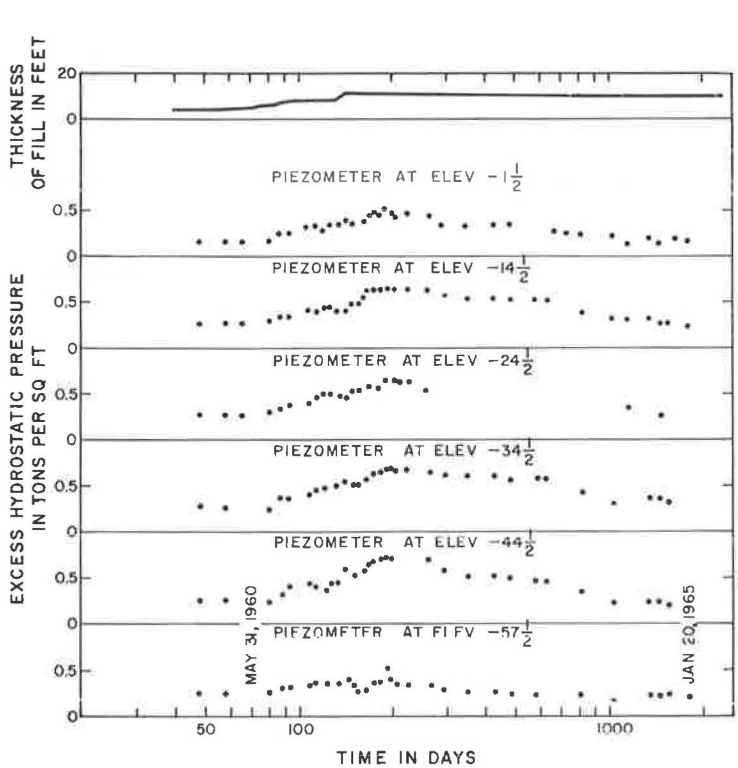

Figure 10. Variation of excess hydrostatic pressure with depth, non-sand drain area.

piezometers at the toe of the main fill indicated a slight rise in pressure as would be expected due to the spreading of the fill load in the foundation soil. These piezometers were being read 3 times a day—morning, noon, and evening. At the time of the movement of the fill on the left side a rise of about 0.1 ton/ft^2 or less was noted in the excess hydrostatic pressure. This pressure then returned in a few days to the pressure prior to the movement on the left side of the fill. The plastic flow type of movement thus appears to occur without a major rise in pore pressure prior to the movement. This will limit the value of the piezometers when this type of movement may occur.

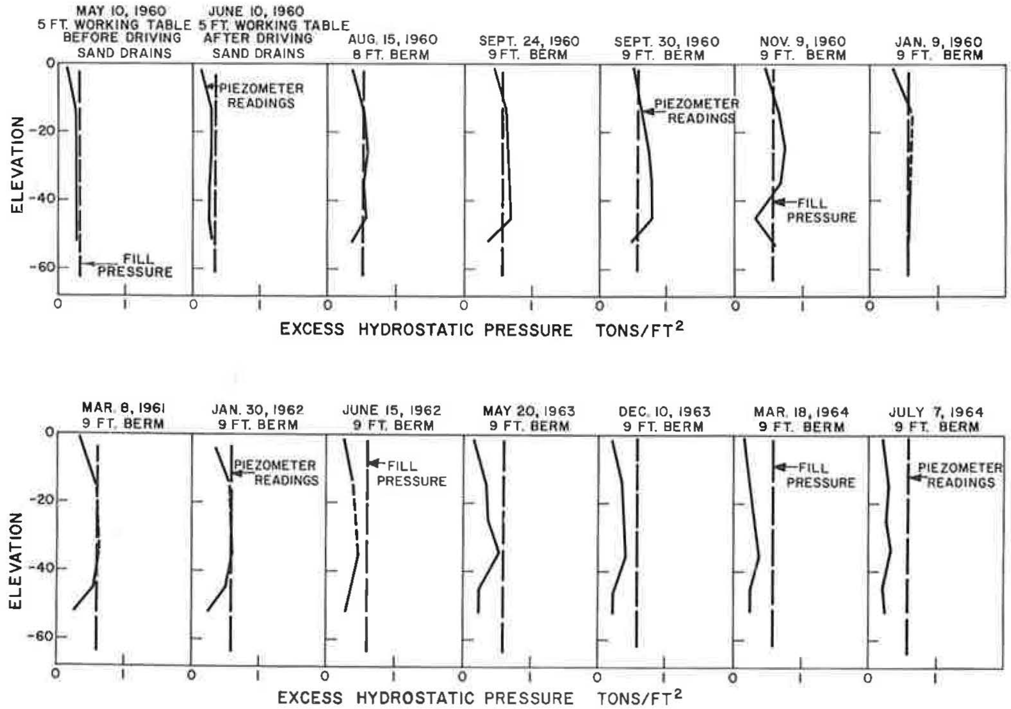

Figure 11. Variation of excess hydrostatic pressure with depth for various stages of construction, non-sand drain area.

Limited piezometers were placed in the filter material. These piezometers indicated that generally about 0.1 to 0.2 tons/ft^2 excess hydrostatic pressure was required to cause the water to flow through the filter material (Fig. 9). As the quantity of flow decreased after completion of the fill the pressure rapidly approached zero. The pressure of 0.2 tons/ft^2 is in reasonable agreement with the calculated pressure, assuming that the volume of water passing through the filter material is represented by the settlement of the fill. These piezometers indicate the value of placing PMP in the filter blanket to reduce the drainage path through the filter blanket and the need for high permeability in the filter blanket. Occasionally a pressure of up to 0.5 ton/ft^2 was recorded in the sand drains. However, this high pressure occurred only infrequently and did not continue for a long period of time.

Piezometers were placed at various depths in the foundation soil in the non-sand drain areas below the main fill and the berm, to measure the rate of consolidation of the foundation soil without sand drains. A typical set of data is shown in Figure 10, which is in the berm area to the left of the main fill. As the piezometers were read every other day during construction, only representative data are shown during this period. The excess hydrostatic pressures rose during the loading of the foundation soil and decreased when the load remained constant. The excess hydrostatic pressure reached a maximum of 0.65 tons/ft^2 under a fill loading of 0.70 tons/ft^2. After completion of the fill the pressure has slowly dissipated until in 1965 about 60 percent consolidation has occurred. This is in agreement with the settlement data.

In Figure 11 the same data have been plotted with excess hydrostatic pressure as a function of depth at given times. The data indicate that at the time of the failure on the left side no significant consolidation of the foundation soil had occurred, which would indicate that no increase in strength occurred at that time under the berms. The piezometers near the top and bottom of the layer started showing a decrease in excess hydrostatic pressure by 1961, indicating that drainage had been occurring. By 1962 a

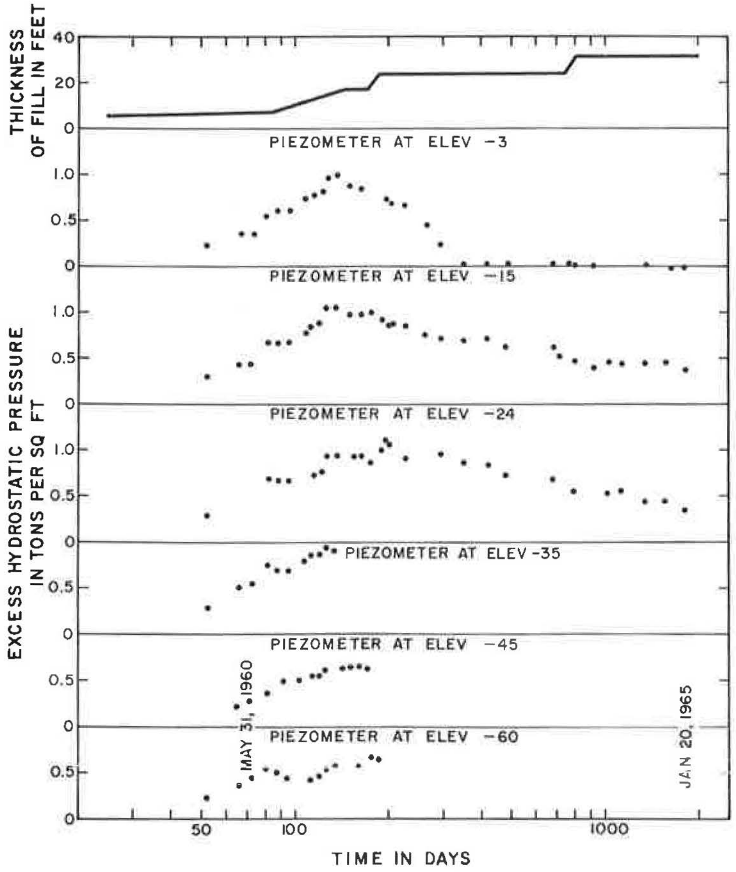

Figure 12. Variation of excess hydrostatic pressure with depth, sand drain area.

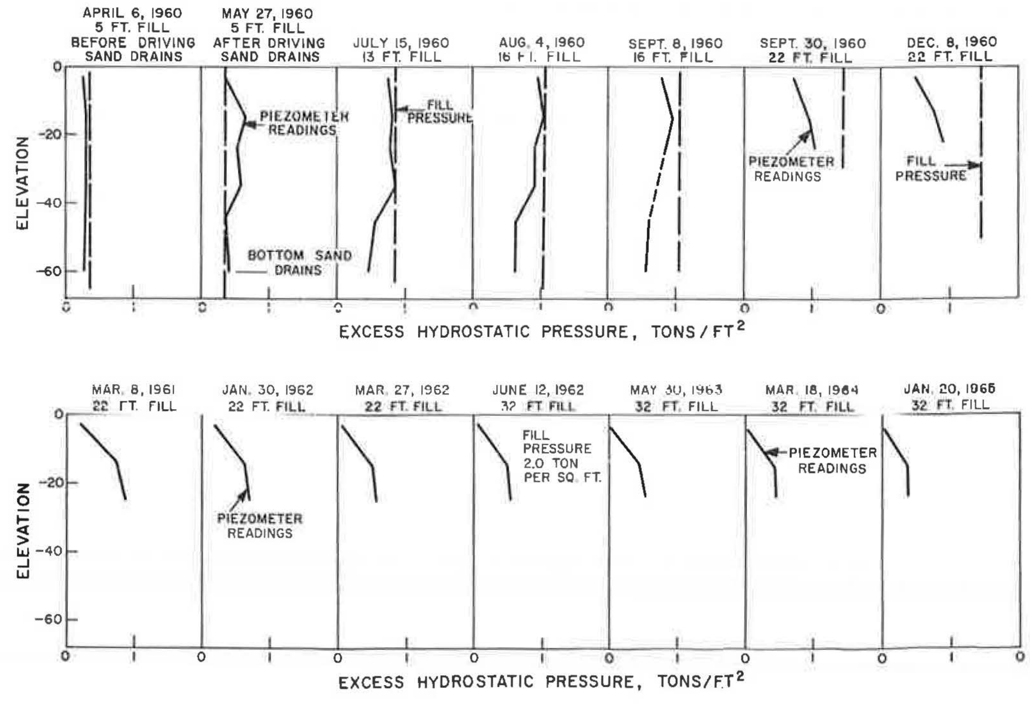

Figure 13. Variation of excess hydrostatic pressure with depth for various stages of construction, sand drain area.

significant decrease in excess hydrostatic pressure in the center of the layer was recorded. At the present time the excess hydrostatic pressure is decreasing as was indicated by the Terzaghi theory of consolidation. The piezometer near the bottom of the layer indicates that the sandy clay and/or peat layer is providing drainage for the soft clay layer. The above data are similar to the data at 4 other locations in non-sand drain areas and are in agreement with the settlement data at these locations.

Piezometers were also placed at various depths in the sand drain area and within 1 ft of equal distance from 4 sand drains. A typical set of this data is shown in Figure 12. The excess hydrostatic pressure increased as loading occurred and then decreased as the load was maintained constant. The maximum excess hydrostatic pressure was 1.0 ton/ft^2 at the end of loading under a load of about 1.4 ton/ft^2. In Figure 13 the same data have been plotted with excess hydrostatic pressure as a function of depth at given times. The data indicate that at the time of the failure on the left side about 40 percent consolidation had occurred. The settlement curves at the location of these piezometers are shown in Figure 8, and the settlement data are in reasonable agreement with the excess hydrostatic data. The piezometers below elevation -35 failed to function properly after loading was completed so that the data are incomplete for this period. These malfunctions occurred for most piezometers at greater depth after loading was completed and the lines appeared to be pinched, that is, a wire would not go through the plastic tubing to the porous stone, but would be stopped above it (5). Prior to placing the additional load in 1962 the piezometers indicated about 70 percent consolidation had occurred, which is in agreement with the settlement data. At the present time the piezometer data indicate that about 90 percent consolidation has occurred, whereas the settlement data indicate that the settlement is completed. This minor discrepancy is not considered serious. The data are typical of the data obtained at 5 other locations in various sand drain areas.

On previous sand drain projects, abnormally high excess hydrostatic pressures had been noted after driving the sand drains with a closed mandrel. Since the piezometers had been placed after driving the sand drains, the reason for this high excess hydrostatic pressure was uncertain. At Napa River the piezometers in the center of the area of 4 sand drains were placed before the sand drains were driven. In Figure 13, the piezometer readings before and after driving the sand drains are shown. The excess hydrostatic pressures before driving the sand drains were about equal to the load of the working table (data on April 6, 1960). After driving the sand drains the excess hydrostatic pressure was about 45 percent in excess of the loading due to the working table (data on May 22, 1960). The driving of the sand drains thus produced an abnormally high excess hydrostatic pressure. This high excess hydrostatic pressure could result in failures of working tables that had previously been stable. It is not known whether the displacement of the soil during the driving of the sand drains or the use of air pressure during the withdrawal of the mandrel produced this high excess hydrostatic pressure. The change in excess hydrostatic pressure at the locations where data are available is shown in Table 3.

At three locations in the sand drain area piezometers were placed 1, 2 and 3 ft from the outer edge of a sand drain after the sand drains were driven. A piezometer was also placed in the sand drain. These piezometers were to gain an understanding of how drainage was occurring into the sand drain. One set of these data is shown in Figure 14, and is shown as excess hydrostatic pressure variation with time. The pressures in the native soil were constant with distance from the sand drain. The excess hydrostatic pressures as a function of the distance from the sand drain at any given time are shown in Figure 15. The data indicate that the expected hyperbolic pressure distribution did not develop, even after a period of several years. This condition existed at all three test locations. The blockage of drainage at the peripheral smear zone appears to be the reason for this condition. As a routine operation, the permeability test was performed on each piezometer after it was installed (9). A review of these data indicated that the permeability indicated by the piezometers 1 ft from the sand drain was $\frac{1}{5}$ to $\frac{1}{10}$ the permeability 2 to 3 ft from the sand drain. It would thus appear that the smear zone extends outward from the sand drain 1 to 2 ft. The permeability 2 to 3 ft from the sand drain was slightly lower than the permeabilities indicated by other piezometers

TABLE 3

EXCESS HYDROSTATIC PRESSURE BEFORE AND AFTER DRIVING SAND DRAINS
(Fill Loading 0.33 Tons Per Square Foot)

Piezometer No.	Elevation	Excess Hydrostatic Pressure (ton/ft^2)		
		Before	After	Increase
45	- 4	0.29	0.38	0.09
46	-12	0.15	0.41	0.26
71	- 4	0.32	0.37	0.05
72	-16	0.29	0.59	0.30
73	-23	0.30	0.58	0.28
74	-36	0.35	0.42	0.07
83	- 4	0.29	0.36	0.07
84	-16	0.30	0.46	0.16
85	-25	0.30	0.52	0.22
86	-36	0.30	0.67	0.37
88	- 4	0.32	0.42	0.10
89	-15	0.38	0.47	0.09
90	-25	0.32	0.54	0.22
91	-35	0.33	0.56	0.23
92	- 4	0.28	0.44	0.16
95	-16	0.28	0.46	0.16
96	-26	0.32	0.48	0.16
97	-36	0.34	0.56	0.26

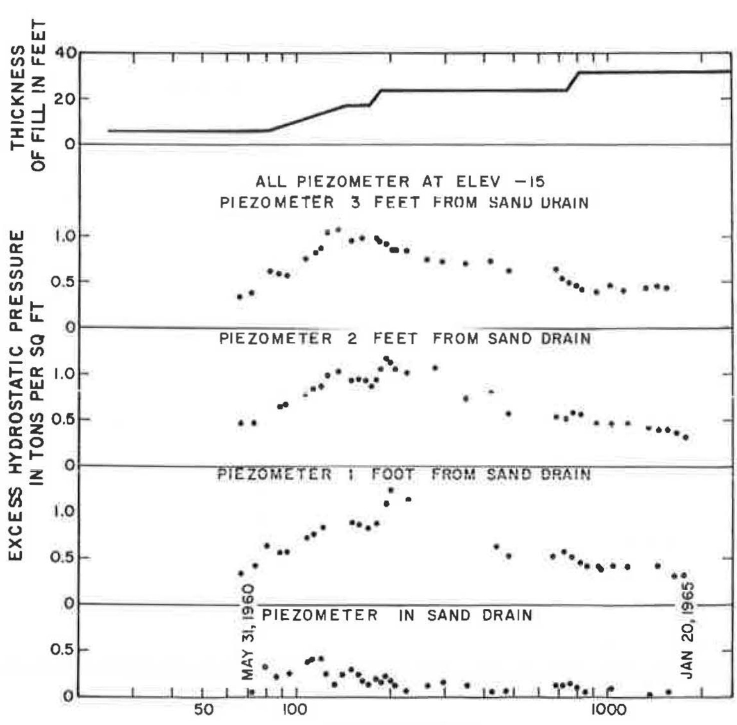

Figure 14. Variation of excess hydrostatic pressure with distance from sand drain.

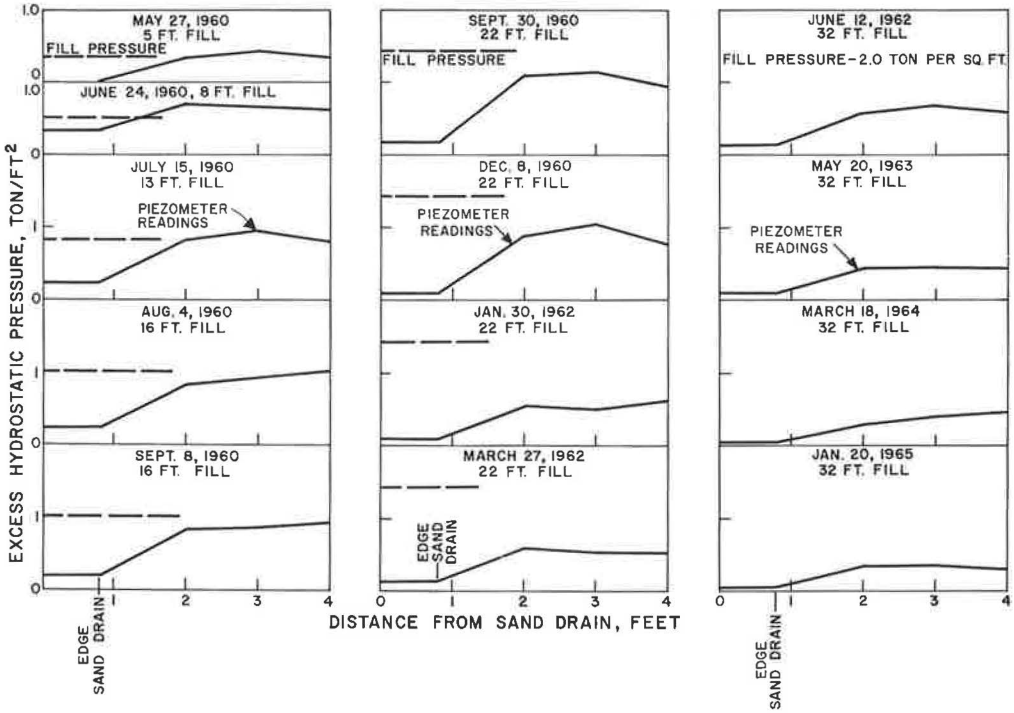

Figure 15. Variation of excess hydrostatic pressure with distance from sand drain.

prior to the driving of the sand drains. No data were obtained at the same piezometer location before and after driving sand drains. However, the data are fairly consistent at the three test locations.

STRENGTH OF THE FOUNDATION SOIL

An important consideration in the design of sand drain installations is the increase in strength of the foundation soil under load. Where sand drains are used to increase the stability of the fill, the increase in strength expected will determine the rate of loading that could be utilized. To measure this increase in strength of the foundation soil an extensive boring program was used at the Napa River test section.

The sample borings were made with a 2-in. inside diameter fixed piston type sampler. A thin-wall extension point was used to reduce disturbance. The sampler was forced into the foundation soil by means of a hydraulic pressure device on a drill rig. When sampling was performed through the fill, an 8-in. auger was used to make a hole through the fill. The strength of the foundation soil was then determined by means of the unconfined compression test. At various times the vane borer was used to measure the in situ shearing strength of the foundation soil. The vane borer used was constructed by the Materials and Research Department of the California Division of Highways, and is modeled after the Swedish vane borer (4). The shear strength as determined by the unconfined compression test and the vane borer were in reasonable agreement.

The soft foundation soil at the west approach to the Napa River showed an increase in shearing strength with depth. The strength at the surface approached zero psf and increased about 10 psf for each foot of depth. At about elevation -50 to -60, layers of sandy silt and peat existed. The strength became erratic at this depth. Underlying this layer was a stiff silty clay layer with a shearing strength of 600 to 1,000 psf. This stiff silty clay layer extended to a depth of about 200 ft and had numerous sand layers in it.

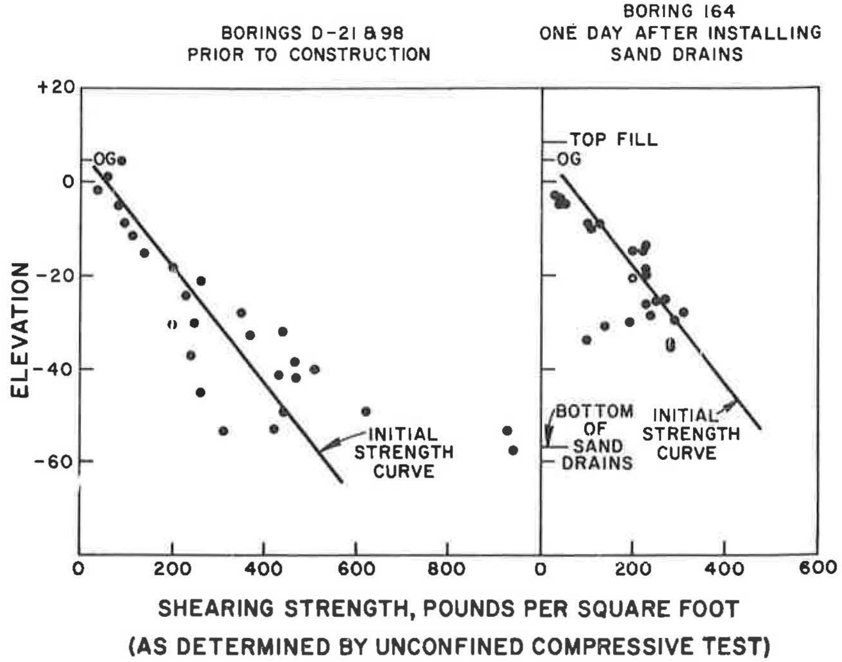

Figure 16. Comparison of shearing strength with depth—sand drains on 8-ft spacing.

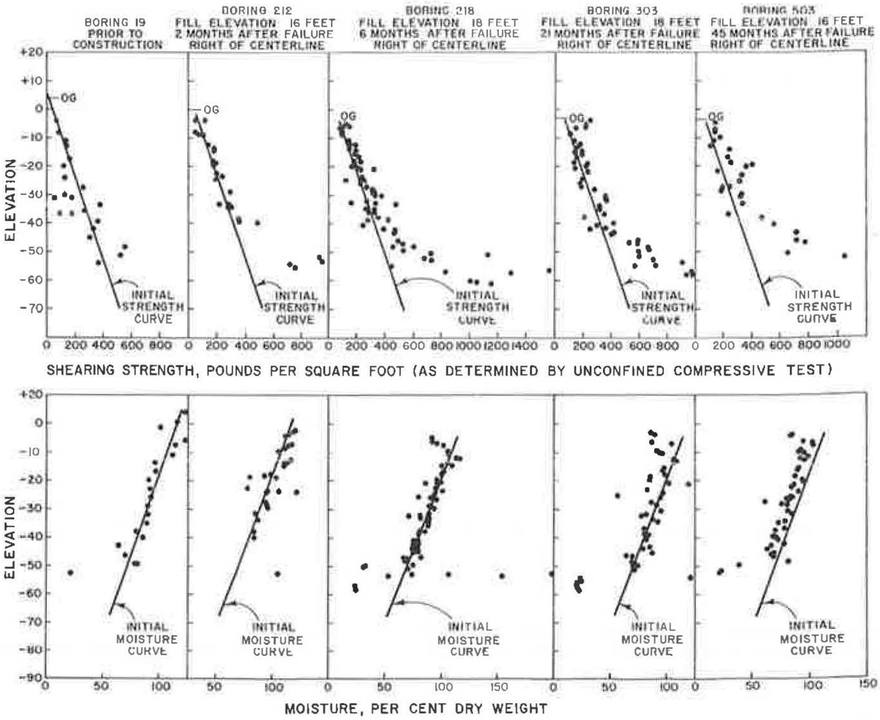

Figure 17. Comparison of shearing strength and moisture with depth—non-sand drain area.

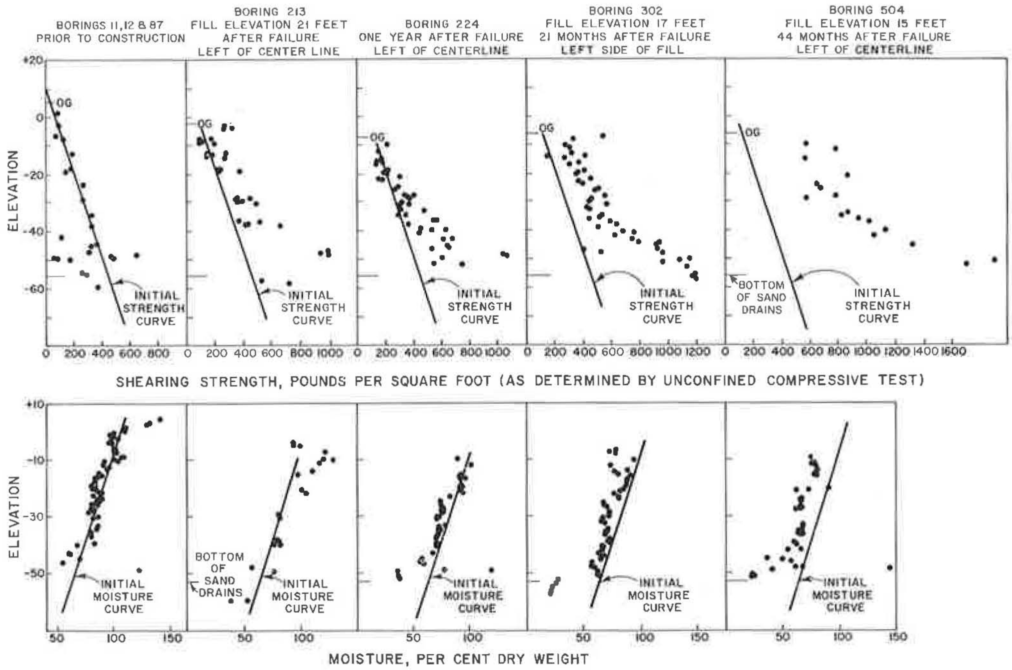

Figure 18. Comparison of shearing strength and moisture with depth—sand drain area.

To determine the effect of the driving of the sand drains on the strength of the foundation soil, 3 borings were made 1 day after the sand drains in 3 different areas were driven. A typical set of data is shown in Figure 16. The borings that were made after the sand drains were driven were an equal distance from 4 sand drains. The data indicate that no significant change in shearing strength of the foundation soil occurred when the sand drains were driven.

Two areas were designated as strength test sections in the non-sand drain areas where borings were to be made at various times to determine the strength of the foundation soil. Areas were chosen where sample borings prior to construction were available, where uniform fill conditions were planned, and where settlement and excess hydrostatic pressure data were available. One was on centerline under 15 ft of fill and one in the berm left of centerline. The data obtained from the area on centerline are shown in Figure 17. This area is near the settlement indicated in Figure 7. There was a small increase in shearing strength indicated by the boring data 6 months after completion of the fill. The boring data indicate that the strength of the foundation soil has continued to increase slowly. The data indicate a 50 percent increase in strength has occurred to the present time in 1965. This is a relatively small increase in shearing strength for a 4-yr period. The other strength test area in the non-sand drain area indicated that about the same strength increase occurred at that location.

Three areas were designated as strength test sections in the sand drain areas where borings were to be made at various times to determine the strength of the foundation soil. These areas were chosen as in the non-sand drain areas.

A set of representative data at one of the test locations is shown in Figure 18. This test area is near the settlements shown in Figure 8. All borings in the sand drain area were made equal distance from 4 sand drains. At the time of failure on the left side there was about a 25 percent increase in shearing strength of the foundation soil in the sand drain area. About 2 years after the completion of the fill the shearing strength of the foundation soils had increased by 50 to 100 percent, increasing about 200 psf at all depths. Four years after completion of the fill the strength of the foundation soil had increased to 600 to 1,000 psf, which is about what the ultimate shearing strength was estimated to be under the loading of 24 ft of fill.

The shearing strength of the soft foundation soil did not increase as rapidly as was estimated prior to construction. It was estimated that the upper 20 ft of the foundation soil would increase about 50 to 75 psf in the non-sand drain area during the loading of the fill. It was estimated that the foundation soil in the sand drain area would increase in shearing strength about 100 psf during loading. Neither of these strength increases occurred. This lack of strength increase resulted in the failure of the fill.

MOISTURE DECREASE

One measure of the rate of consolidation of a foundation soil is the decrease in moisture content. The moisture content is also related to the strength of a given soil. Two methods of following the moisture change in the soft silty clay at the Napa River experimental sand drain section were used. The moisture content of the soil was measured from the samples obtained from the borings. The other method of measuring moisture changes was by the use of nuclear subsurface gages.

The moisture contents at Napa River decreased with depth. The moisture content was about 110 percent of the dry weight at elevation -10. The moisture decreased lineally with depth to a moisture of about 75 percent at elevation -50. The moisture contents before construction are shown in Figures 17 and 18.

The moisture content of the soft silty clay in the non-sand drain areas showed a minor decrease 4 years after construction (Fig. 17). Two years after completion of the fill the moisture decrease was about 3 percent and after 4 years was about 10 percent.

The moisture content of the soft foundation soil as determined from the samples from borings in the sand drain areas did not show a decrease during construction (Fig. 18). Two years after construction the moisture content had decreased 15 percent and after 4 years had decreased 20 to 25 percent.

Assuming that all of the measured settlement is due to drainage of water from the soil, the decreases in moisture content are in reasonable agreement with the measured settlements. This is true in both the sand drain and non-sand drain areas.

The nuclear gages measure the moisture content of the soil in pounds of water per cubic foot. The moisture content of the soil is therefore reported in these units in this report. These units of pounds of water per cubic foot are a volume measure of moisture content. The nuclear access tubes failed by corrosion after 1962 so that no data are available after that date.

The moisture content as determined by the nuclear gages in the non-sand drain area is shown in Figure 19. The moisture content in the non-sand drain area did not show a significant decrease in this 2-yr period. This is in agreement with the moistures determined from the samples from the borings.

The moisture content as measured by the nuclear gages in the sand drain area is shown in Figure 19. No significant change in moisture content is indicated during construction, which is in agreement with the moistures determined from the samples from the borings. During the first 2 years after construction there was a decrease in moisture content of 1½-2 lb of water per cubic foot indicated in the sand drain area. This is in reasonable agreement with the moisture change noted by the sample borings.

MOVEMENT MEASUREMENTS

In order to better understand the way in which the failure occurred, two methods of measuring movements were used. Heave stakes were used to measure the movement of the surface of the fills. Both horizontal and vertical movements were measured. The movement of the foundation soil was measured by means of a slope indicator.

The heave stake lines were installed on the berms. The failure to the right side produced a heave of the existing roadway (Fig. 3). The rate of movement increased at the time of failure, and gave some notice that movement was occurring.

The reloading of the fill was resumed on September 8, 1960, and the rate of movement on the left side continued at the same rate as prior to reloading. From September 21 to 27 a slight increase in the rate of movement of the heave stakes occurred (Fig. 4). However, this movement was not considered alarming. The readings on September 30, after the failure had occurred, indicated a large horizontal movement had occurred.

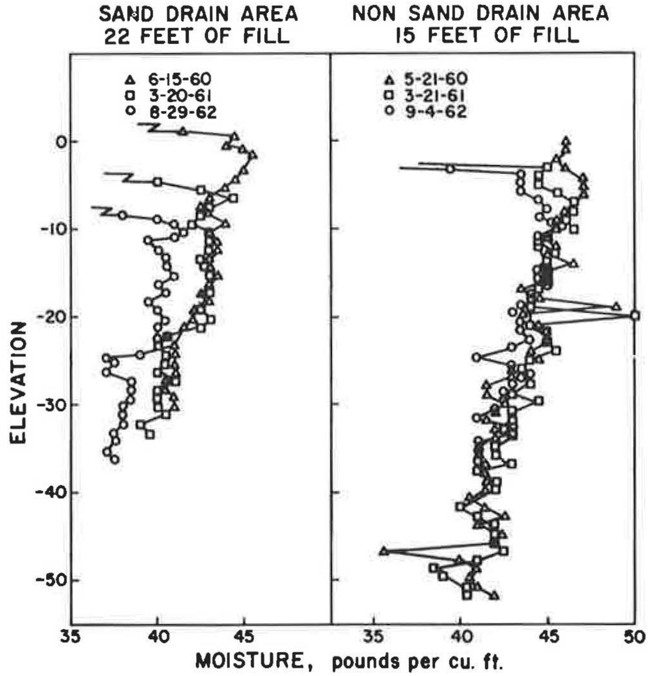

Figure 19. Moisture variation with depth as determined by subsurface nuclear gages.

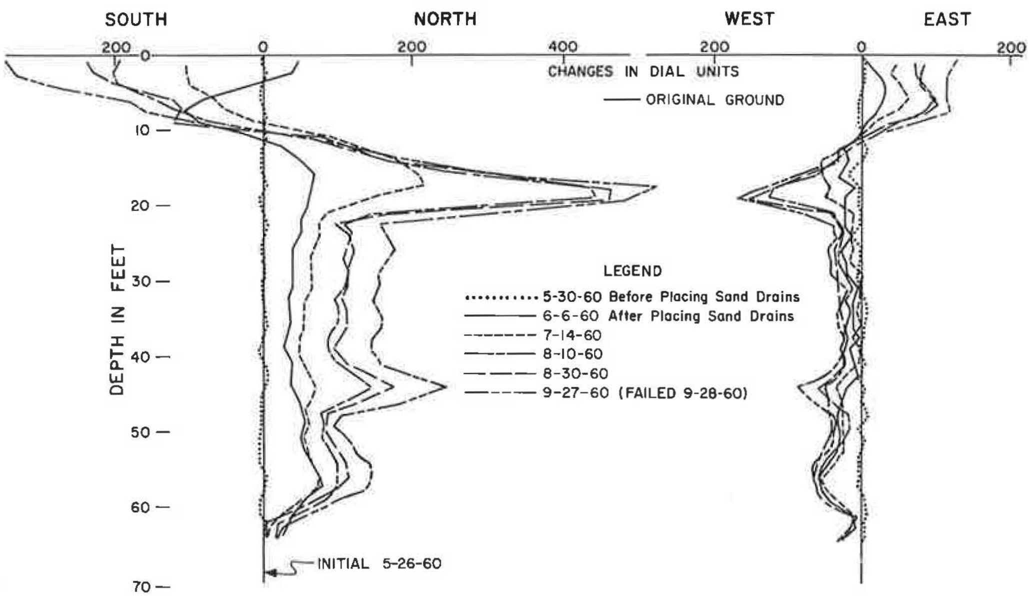

Figure 20. Movement measured by slope indicator.

The main deficiency in the use of heave stakes is the time required to perform the necessary surveying, reduction and analysis of the data. A time lag of one day was normal. Also large manpower would be required to make the necessary measurements once or twice daily. For this reason heave stakes are limited in the routine application to stability control during normal construction.

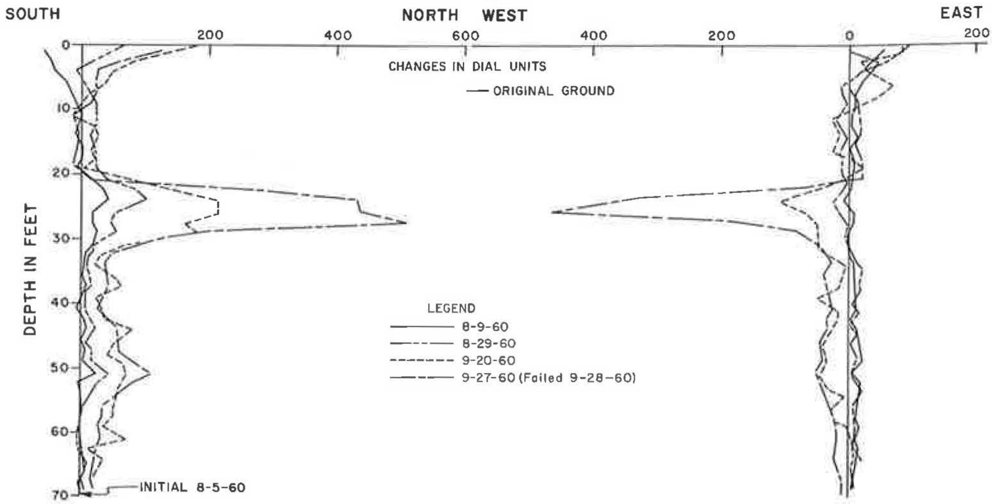

Figure 21. Movement measured by slope indicator.

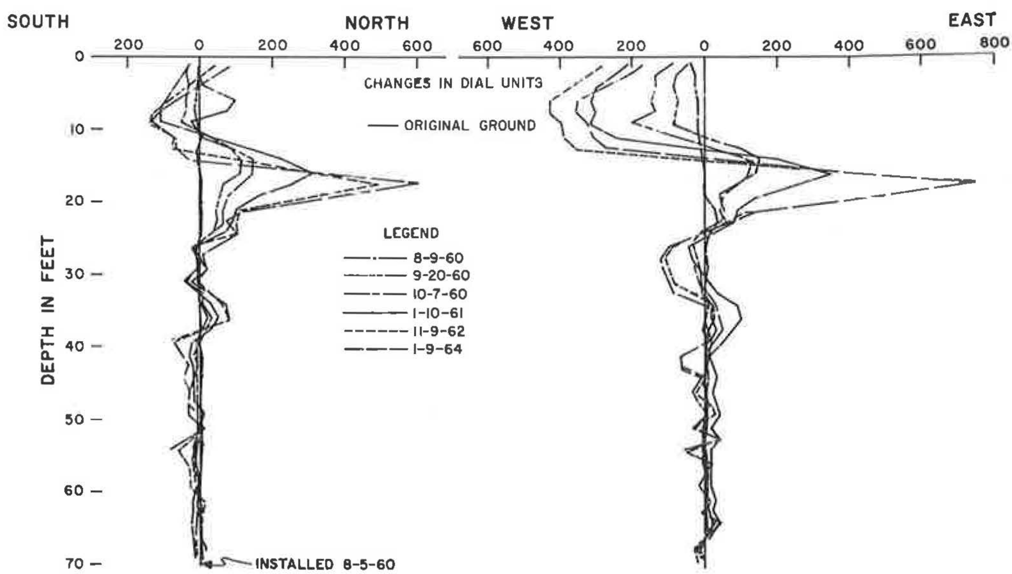

Figure 22. Movement measured by slope indicator.

The slope indicators were installed when the working table was completed. Two slope indicators were installed 10 to 15 ft from the edge of the future sand drain area. The lateral movement of the foundation soil caused by the driving of the sand drains is shown in Figure 20. The soft foundation soil was moved laterally during the contractor's operations of placing the sand drains. This may be due to the use of a closed mandrel.

The data obtained from a slope indicator to the left of the fill in the area that failed are shown in Figure 21. This installation was destroyed during construction and reinstalled after the berms were widened on the left. As loading of the main fill was resumed, after the failure on the right, a minor movement was noted at a depth of 25 ft

below the top of the tubing. On the morning of September 27 this installation was read. When the data were reduced and plotted on the morning of the 28th, after the failure, it was found that the slope indicator had indicated a major movement occurred at a depth of 25 ft. This indicates how the time delay is critical in stability problems. On Septemper 28, 1960, it was found that the tubing had failed at a depth of about 20 ft.

A series of long-term readings are shown in Figure 22 from an installation to the right side. During the loading of the main fill prior to September 27, 1960, a minor movement occurred at a depth of 15 ft. This movement has continued to 1965 at a fairly uniform rate and is now at the limit of the slope indicator scale. This indicates a movement of the foundation soil is occurring which has not been recorded by the heave stakes. This is evidently a plastic type flow slowly occurring. This condition has been noted on several other slope indicators that have long-term readings. However, no cracking or mud waves have been observed.

The heave stakes and slope indicators are both useful installations. However, they both have one deficiency for use as a stability control device during routine construction. This deficiency is the time required to obtain the necessary results.

CONCLUSIONS

This experimental sand drain project produced several items of information of benefit to the design and construction of future sand drain projects.

1. The consolidation of the soft foundation soil in sand drain areas with conditions similar to Napa River will occur at a rate 2 to $2\frac{1}{2}$ times slower than estimated from theoretical studies.

2. Sufficient time must be available to construct fills on soft soils at a slow rate. It may be necessary to construct fills on soft soils under separate contracts and utilize stage construction.

3. Where sand drains are installed in soft foundation soils similar to the conditions at Napa River, it should be accomplished by a method that will not disturb the soft foundation soil.

4. With the slow rate of consolidation of the soft foundation soils, sand drains may be required under struts as well as under the main fill.

ACKNOWLEDGMENTS

The research described in this paper was conducted by the California Division of Highways, Materials and Research Department. The Materials and Research Department was under the supervision of F. N. Hveem (retired) and J. L. Beaton. The work was performed by the Foundation Section under the direction of A. W. Root (retired) and Travis W. Smith.

The construction of the fill was under the supervision of the California Division of Highways District 10, under J. G. Meyer, District Engineer. The Construction Engineer was W. F. Fleharty and the Resident Engineer, L. E. Daniel.

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Summary of Treatments for Highway Embankments on Soft Foundations

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This paper summarizes the procedures that have been developed in the New York State Department of Public Works for the evaluation and solutions of foundation problems involving highway embankments.

Early in the line selection stage of design, the critical soils areas are located and sufficient data obtained to determine the soil properties. By establishing close coordination between the location engineers and soils engineers, it is often possible to avoid critical soils problems having expensive solutions by minor shifts in alignment. The most economical and satisfactory solution to an embankment foundation problem is determined not only by the soil properties, but also by consideration of construction time, right-of-way, location of project, cost and availability of construction materials, and highway geometrics. Experience with various methods of treatment is described.

•THE BASIC function of a highway engineer is to design, construct and maintain a satisfactory and adequate highway system utilizing the most economical methods available. In the New York State Department of Public Works organization the Bureau of Soil Mechanics is responsible for providing the location, design, and construction engineers with complete information on the location of critical foundation areas, the type of treatment required to provide a stable embankment foundation, and the necessary treatment details to be incorporated into the contract documents.

New York State has a great variety of soil deposits that present embankment foundation problems. Tidal marsh deposits containing organic silts and clays are found in the lower Hudson Valley and in western Long Island. Many of the major river valleys in upstate New York were at one time large postglacial lakes. Varved silts and clays up to 200 ft in thickness now fill these former lake locations. In central New York, near Syracuse, these silt and clay deposits are often found under a surface cover of peat and soft marl. In western New York, the former shore lines of Lake Ontario and Lake Erie reached inland beyond their present boundaries leaving extensive lacustrine deposits of silts and clays. The St. Lawrence Valley and Champlain Valley on the northern boundary of the state contain deposits of sensitive marine clays. All over the state, including the upland areas, small postglacial lakes or ponds existed and have been subsequently filled with highly compressible peat and muck.

A combination of a large highway construction program and extensive areas of soft soil deposits presents a great number of embankment foundation problems. Each year the Bureau of Soil Mechanics investigates 30 to 50 major foundation problems. This paper outlines the procedures that have been developed over the past 20 years to provide, in the highway design, the most economical and satisfactory solution to these problems. The proper time to solve all the details of foundation problems is during the design phase. If this work is left until construction, the cost of treatment will be higher and the results most likely will not be as successful.

Paper sponsored by Committee on Embankments and Earth Slopes and presented at the 45th Annual Meeting.

The fundamental requirements of any treatment are (a) to provide a method of constructing an embankment that will be stable against lateral movement or shear failures of the foundation soil during and after construction, and (b) to eliminate all post-construction differential settlement of the foundation soil that would be detrimental to pavement performance and that would make the riding characteristics of the roadway dangerous.

Every embankment foundation problem usually has more than one solution. However, the most economical and satisfactory solution depends upon a careful evaluation of the following five factors:

1. Characteristics of the foundation soil—Sufficient subsurface investigations and laboratory testing must be conducted to determine the extent, depth, and the strength and consolidation properties of the critical layers.

2. Highway alignment and grade requirements—Often expensive foundation treatments may be avoided by a slight line shift to avoid a limited area of poor foundation conditions. Similarly, a decrease in grade line, when possible, may diminish the stability and settlement problems and the cost of any necessary foundation treatment.

3. Available construction time—The most economical method of treatment often is the use of time to allow the foundation soils to consolidate and gain shear strength. Combinations of controlled rate of construction, surcharge treatment, and waiting periods are used for this method of treatment. The usual construction contract runs for about two years. Stabilization periods of from six to nine months may be used on portions of a project with little inconvenience or delay to the contractor.

For projects where embankment foundation problems exist over a major part of the area, consideration may be given to stage construction by letting a separate grading contract followed by a paving and structures contract after sufficient stabilization time has elapsed.

4. Construction materials—The selected method of treatment will often depend upon the availability and cost of construction materials. For example, a project involving unsuitable material excavation of swamp deposits requires a suitable granular material to provide a stable underwater backfill. When a highway location is in mountainous country there usually will be excess rock excavation available for underwater backfill at a reasonable cost. However, in an urban area the cost of granular underwater backfill may be considerably more expensive. On urban highway projects, excess excavation often is not available, since the required fills usually exceed the material available from cuts. Also, many urban areas are adopting regulations restricting development of gravel pits and excavations making it necessary to import material from distant sources for the backfill material at a considerable cost.

One solution for embankment foundation stability problems is to construct the fills with a lightweight material, such as slag, cinders, or lightweight aggregate obtained from expanded shale. The use of this material depends upon its availability and cost, including transportation.

5. Location of project—The cost of right-of-way often influences the type of treatment selected. The use of stabilizing berms may be impractical due to the high cost of right-of-way in an urban or industrial area. The effect of the highway construction upon adjacent structures, utilities, highways and railroads must be considered. Embankment settlements may also cause detrimental settlement to adjacent facilities unless protective measures are incorporated into the highway design.

SOILS INVESTIGATION PROGRAM

The soils investigation program is planned so that the major foundation problem areas may be investigated and evaluated early in design in order to assess the magnitude of the problems involved. This policy not only applies to highway embankment foundation problems, but also to structure foundations and highway cuts. For critical foundation problems, the cost of treatment may become so large that the designer may desire to compromise on other design requirements in order to reduce or eliminate the foundation treatment costs.

For example, the preliminary alignment considered for a typical Interstate highway crosses a swamp area approximately 1000 ft long. An exploration program is made to determine the subsurface soil profile and to evaluate the soil properties. The results show that the treatment will require excavation of the unsuitable material and replacement with granular borrow to an average depth of 20 ft. Preliminary estimates indicate that the cost of providing a stable foundation to original ground surface will be in the order of $350,000. With this information, the designer determines if the proposed line can be moved to areas of more favorable soil conditions. Concurrently, additional explorations are conducted on possible alternative alignments to determine the foundation conditions and the economics of treatment. Frequently hundreds of thousands of highway construction dollars can be saved by using this exploration procedure and by working closely with the designer in the preliminary stages of a project. This coordination also eliminates the costly and frustrating situation of scrapping completed designs because of critical foundation conditions which are "discovered" after the design is nearly complete.

In order to provide the designer with the necessary information at the proper time, the following procedures have been adopted for major highway projects:

1. Terrain reconnaissance survey—During the initial line selection study, the boundaries of the major soil deposits are determined by field inspection and by a study of aerial photographs and agricultural soil maps. A summary of highway design considerations is made based upon a general knowledge of the engineering properties of each particular soil type and the past experiences in highway construction and pavement performance on each soil type. Reports on the detailed procedures of terrain reconnaissance surveys used by the New York State Department of Public Works have been published previously (1, 2, 3).

2. Preliminary soils investigations and design—This phase of the exploration program is usually confined to the soil deposits where major embankment foundation problems are expected. Sufficient subsurface explorations and testing are required to develop the soil profile, and the strength and compressibility characteristics of each soil stratum. This allows the soils engineer to determine the general type of treatment required and to prepare a preliminary cost analysis. A conference is arranged with the designer to determine the most practical solution. The selected solution may be influenced by non-soil factors such as highway alignment, grade requirements, anticipated time for construction, right-of-way costs and limitations, land usage, available construction materials, and design class of highway. Often, the result of comparing all the factors involved is a shift of the line to an area with better soil conditions or a decrease in the grade line to improve embankment foundation stability and settlement problems. This type of approach insures the most economical solution to a foundation problem while assuring that all other requirements of the highway design are satisfied.

3. Final foundation design—Additional explorations, testing and analyses are made to determine the details and limits of the foundation treatment. Sufficient drawings, special notes and specifications are required for the contract documents, so that the desired end result of the selected treatment may be achieved. A description of the foundation conditions and the general treatment has been found to be effective in giving the contractor and supervising engineer an understanding of the purpose, desired results, and methods of payment for the treatment.

METHODS OF TREATMENT

This section discusses the various methods of treatment that may be used to provide a stable foundation for the highway embankment. The selected method, or combination of methods, not only depends on the soil properties, but upon an evaluation of all the other design factors discussed previously.

Removal of Soil by Excavation

This treatment is used for swamp deposits that are predominantly organic. A typical peat or muck may contain, by volume, nine parts water and one part organic and inert

soil. This material has a very low shear strength and is highly compressible under embankment loads. It is very difficult to stabilize this soil in place satisfactorily so as to obtain a smooth pavement for high-speed traffic. Explorations through old country and town roads constructed across peat deposits indicate that extensive maintenance has been required in order to keep the roadways above high water. As much as 10 ft of asphaltic concrete has been found on several old roads crossing swamps. This type of settlement and maintenance cannot be tolerated on modern highways.

Sufficient explorations should be taken to indicate the depth of excavation on the contract cross sections and to predetermine quantities. A contour plan of the depth of unsuitable material is made available to the project engineer for large projects where the swamp bottom is irregular.

The width of excavation should be such that the embankment slopes will not be unstable resulting in settlements or lateral movement of the roadway shoulder. Typical sections for various cases of excavation are shown in Figure 1. Some designers become concerned that the stability of the backfill against the adjacent peat or muck may be critical and that the backfill will spread and crack. A circular arc stability analysis

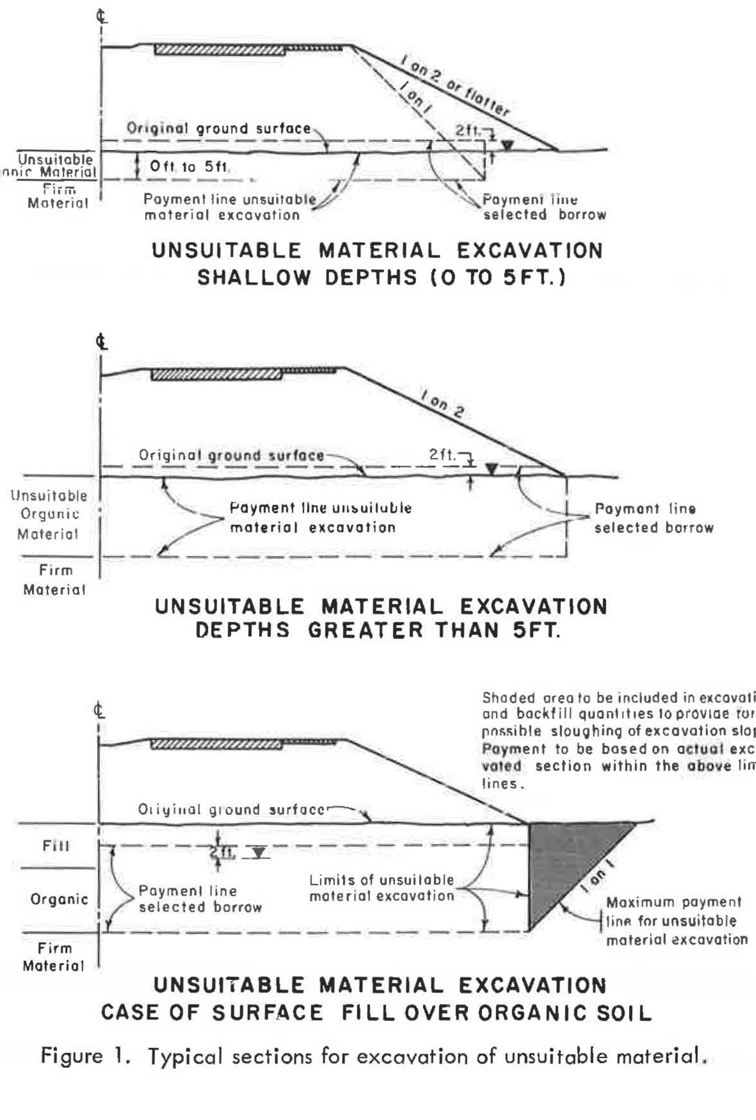


Figure 1. Typical sections for excavation of unsuitable material.

or sliding wedge analysis often shows that this condition should be critical. However, experience on many projects has indicated backfill with vertical side slopes underground will be stable against this type of movement. A possible explanation is that the organic soils supply more passive resistance against lateral movement than the available methods of analyses indicate. Also, explorations on several completed projects show that the backfill below ground surface remained in a vertical plane and practically no bulging of the backfill occurred.

On one project where the depth of underwater backfill was large as compared with the fill height above ground, cracks appeared in the embankment. The area over the crack was surcharged in order to force displacement. However, no displacement occurred and the surcharge apparently served to increase the intergranular strength of the backfill and increase the stability of the area since no post-construction distress appeared.

On large excavation projects, disposing of unsuitable material may be a problem. The specifications should indicate that the material is to be placed in spoil banks outside the right-of-way or used in construction to flatten slopes. Soils with high organic contents will shrink over 50 percent on drying.

During construction, the vertical excavation slope in peat or muck will be stable for several days before backfilling, provided that the excavation is kept full of water. When the water has been pumped out of excavations, the sides have caved in, resulting in needless additional excavation. This is a basic slope stability problem. When the organic material is under water, it has a submerged unit weight of 5 to 10 pcf. When the excavation is drained, the saturated unit weight of the adjacent soil increases by approximately 62.4 pcf from the submerged weight. The increase in effective weight greatly decreases the stable height of excavation. This fact is often overlooked by engineers in the field. Frequently, engineers require the contractor to lower the water level so that the excavation may be inspected. When this is done, sloughing problems will develop on the sides of the excavation.

For fills with widths of the order of 200 ft, it is not practical to carry the entire width of backfill across the swamp at one time. The usual procedure is to progress the backfill in strips 50 to 70 ft wide. Advancing fill fronts are skewed away from the completed portions to help eliminate the possibility of unsuitable material being entrapped between the fills. This method has been used successfully on several projects with no adverse post-construction settlements at the boundary between adjoining backfill strips.

Frequently, the unsuitable organic soils are underlain by very loose silt or soft clayey silt that appear to be unsuitable material for an embankment foundation when uncovered by excavation equipment. However, when backfill is placed on this material, it will consolidate rapidly and provide an adequate foundation. A very difficult problem to control in the field is the problem of over-excavation when the underlying soil is loose or soft. In order to control this practice, which has resulted in very costly overruns of excavation and backfill quantities, the following item is included in the New York State Department of Public Works Construction Specifications for excavation. "The Contractor shall remove all muck, peat and other organic swamp deposits to the payment limits as established by the Engineer prior to such excavation. No reimbursement will be made for any excavation in excess of such payment limits." The purpose of this clause is to determine the properties of the questionable material before large quantity overruns develop. A fast method is to obtain samples from test pits or auger holes ahead of the excavation and determine the moisture content of the questionable soils. The moisture content and soil description can be correlated with strength and consolidation properties from testing on similar swamp soils.

Figure 2 shows a moisture content correlation chart based on the laboratory test results from many types of swamp soils. This chart is often used to obtain the necessary information for a field decision on excavation depths that have not been determined by previous explorations. An important limitation of this chart is that it only applies to swamp soils where the precompression loads are low and the soils are soft or loose. For soils outside of swamp areas, the strength and consolidation characteristics may have been modified by precompression or soil structure and the correlation will not apply.

CHART FOR ESTIMATING SUITABILITY OF SOILS IN SWAMP DEPOSITS FOR HIGHWAY EMBANKMENT FOUNDATIONS BASED ON SOIL IDENTIFICATION AND MOISTURE CONTENT

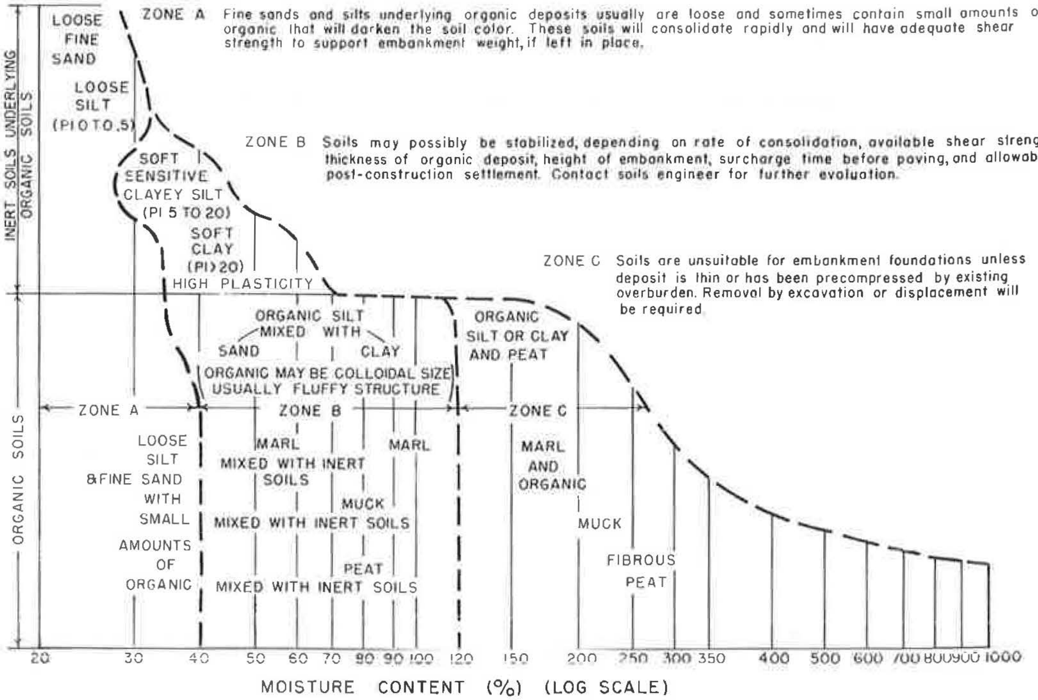
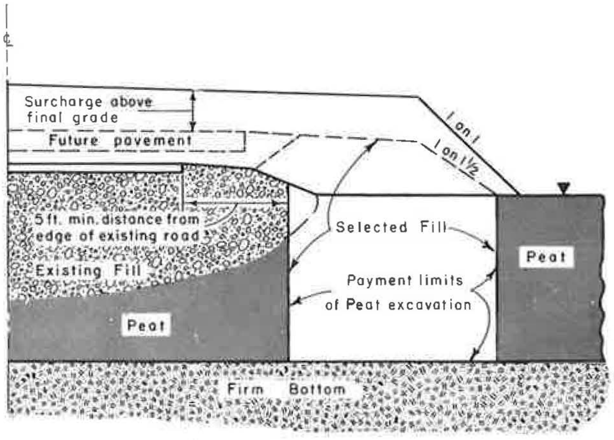


Figure 2. Moisture content correlation chart.

The following case history illustrates the economics of overruns on unsuitable material excavation and replacement. The north approach roadways to the Throgs Neck Bridge in the Borough of Bronx, New York City, were located in a shallow tidal swamp about 1½ mi long. Soil conditions consisted of 5 ft of peat mixed with organic clay underlain by 4 ft of soft clayey silt over firm sand. Excavation and replacement of the surface organic deposit was required. However, in the initial excavation work, the underlying clayey silt was also removed because it appeared to be very wet, soft, and unsuitable material for a foundation. Previous laboratory testing, and analyses, had indicated that this material would consolidate rapidly and provide a satisfactory foundation. It was decided not to excavate the soft clayey silt. If this over-excavation had been allowed to continue, the overrun in cost for excavation and backfill for the project would have been over $800,000.

Another application of the problem of unsuitable material removal involves the stripping of sod and topsoil under embankments outside of swamp areas. This material, which usually contains a small amount of organic, will compress rapidly during construction under the embankment loads and will provide an adequate foundation for the roadway. New York State Specifications require stripping of sod and topsoil only when the embankment height is less than 6 ft or when topsoil is required as an item on the project. The savings in construction costs with the elimination of sod and topsoil stripping is significant. The cost for 6 in. of stripping and 6 in. of backfill under a 15-ft high embankment on the Interstate System is of the order of $30,000 per mile.

A special problem of embankment stabilization by excavation is encountered for widening projects on secondary roads crossing swamps. These old roads are usually floating on fills that have penetrated by displacement and settlement to a considerable depth below the swamp surface. The existing roadway is reasonably stable, since the underlying soil is well compressed after 30 years or more of loading. Any new embankment material placed on the ad-



TYPICAL SECTION FOR WIDENING HIGHWAY

Figure 3. Typical section for widening highway on swamp soils. (Note: Length of open excavation must be carefully controlled to preserve stability of existing embankment.)

jacent swamp surface will undergo settlement of perhaps several feet, thus creating a differential settlement problem across the highway section. Figure 3 shows the typical section for the successful method of treatment used in New York State. The organic material under the widened section is excavated and backfilled with suitable granular soil. This method has been used successfully to depths of 20 ft. Failure of the existing roadway into the open excavation is prevented by keeping a minimum length of open excavation parallel to roadway centerline. Specifications require that the distance from the toe of excavation to the toe of backfill shall be less than 15 ft at all times during each day's operation and less than 3 ft at the conclusion of a day's operations. Also, no traffic should be allowed within 10 ft of the open excavation. After completion of excavation and backfill, a surcharge may be used, if necessary, to decrease differential settlements across the roadway. The excavated material may be used to flatten slopes.

Removal of Soil by Displacement

The displacement procedure is accomplished by placing sufficient embankment material on the foundation soil to cause the underlying soil to displace by shear failure in the direction of least resistance. The essential design features for a successful displacement operation are to have sufficient embankment weight to force out the underlying soil and to have a sufficient depth of mudwave excavation before the advancing fill front so that the direction of displacement can be controlled and the fill will continue to sink to the desired depth. Also, the advancing fill front should have a steep front face. The displacement method is used for peat and muck deposits greater than 30 ft in depth where complete excavation may become difficult, and to remove very soft clays or organic silts that would not stand on a steep excavation slope under water.

In a swamp with an irregular to firm bottom difficulty is encountered obtaining complete displacement when the displaced soil is directed against a rising firm bottom surface. The front of the fill should be skewed as necessary to direct the displacement toward the deeper portion of the swamp. Frequently, a drawing is included in the plans indicating the direction of successive fill fronts as the embankment is constructed across a swamp area.

In deep swamps, culverts should be located where the organic material is shallow, usually near the edge of the swamp. The possibility of culvert settlement due to consolidation of deep backfill and possible pockets of entrapped organic material is elimiated.

Occasionally the organic material is underlain by very soft clays that increase in strength with depth. For this case a detailed laboratory testing and stability analysis is required to predict accurately the depth of displacement for design.

The method of payment for removal of unsuitable material by displacement methods presents a problem, since it is not practical to measure quantities in the field. The method used in New York State is to predetermine the quantities of removal by detailed explorations and by laboratory testing and analyses in order to predict as accurately as possible the volume of displaced material. A special payment item is used in contracts for a predetermined quantity of excavation between designated station limits. The above method of measurement and payment has been successful and satisfactory on numerous projects. Use of this payment method is not recommended if there is any uncertainty on the behavior of the soil material under displacement action since the predicted quantities could be erroneous. A typical section for this treatment is shown in Figure 4. A waiting period before paving is desirable to allow the backfill and any entrapped organic soil to consolidate. Sometimes a surcharge is used to eliminate any expected post-construction settlement.

The rate of backfill placement should not exceed the rate of removal of a mudwave material. If this rule is not enforced, the backfill can trap pockets of unsuitable material resulting in undesirable settlements after paving.

The surface root mat is removed at least 30 to 40 ft ahead of the backfill to promote displacement. All mudwave material is excavated that rises above a designated elevation for a 30 to 40-ft distance in front of the backfill in order to insure continuing displacement to the desired depth. The critical elevation for excavation may be determined from stability analyses. However, the water elevation in the excavation at the time of construction is very important and often cannot be predicted. Any mudwave material rising above water will have a much greater counterweight effect than submerged material and should be removed. All excavated material should be cast behind the advancing fill front or removed from the site.

Controlled Rate of Construction

The two previous methods involved removal of the foundation soil. The following methods involve treating the soil in place to provide a stable foundation. Controlled rate of construction is the most economical method when it can be used, since it involves no additional construction materials. The only requirement is adequate time.

Controlled rate of construction is used where the foundation soil would undergo shear failures if the embankment is built rapidly. However, as a soil consolidates, an accompanying increase in shear strength occurs. The consolidation-strength relationships may be determined by laboratory testing. Stability analyses are conducted to determine the factors of safety for various heights of embankment and various degrees of consolidation throughout the subsurface soil profile. The maximum safe height of embankment

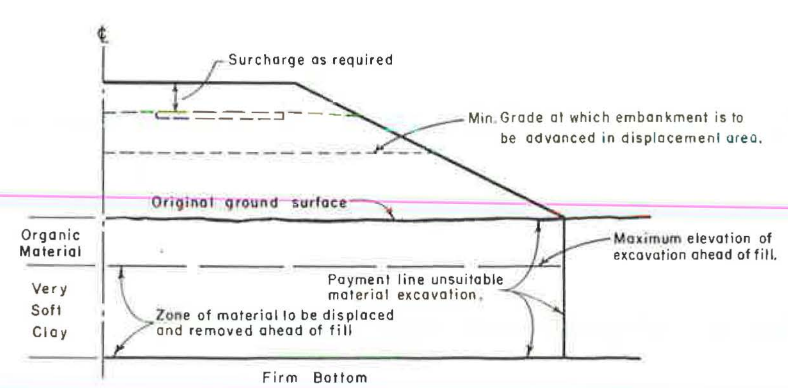


Figure 4. Typical section for displacement of unsuitable material.

that can be placed with no rate restrictions is first determined in order that the maximum consolidation may be obtained during the waiting period. The time interval between fill increments and the height of increments is next established by analysis. As each increment is placed, the factor of safety should not decrease below an established minimum value.

The minimum factor of safety for design is influenced by the completeness of the explorations, the quality of the undisturbed samples, the consistency of the test results, and the sensitivity of the soil structure. Where the information is dependable, a minimum factor of safety of 1.25 is used for embankments. Where structures such as bridge abutments are involved, the minimum factor of safety is set at 1.50 at the time structure work is scheduled to begin.

Occasionally, situations arise that justify a "calculated risk" and lower factors of safety are used. This is usually in areas where an embankment foundation failure would not damage any structures or drainage pipes, the shear failure would not cause future differential pavement settlement, and there is sufficient right-of-way available to place a counterweight berm if required. The consequences of a failure in terms of economics and future highway performance should be thoroughly considered before using this philosophy of design.

Field instrumentation such as settlement platforms and piezometers should be installed to check the field settlement with the design estimates. The waiting periods between fill increments may be decreased if the consolidation occurs faster than predicted. Also, collection of field performance data is valuable for use in designing future projects.

Stabilizing Berms

When the weight of an embankment causes shear stresses greater than the shearing strength available in the foundation soil, a possibility exists that the embankment will cause the underlying soil to displace laterally. The zone along which this movement takes place approximates a circular arc with the center of arc over the side slope of the embankment.

The factor of safety against failure is the ratio of the resisting moment to the driving moment along the arc. The driving moment is determined from the weight of the embankment material and the resisting moment determined from the available soil shear strength along the failure arc. The purpose of a berm placed against the outer embankment slope is to provide a counterweight that will reduce the overturning moment on the failure arc and increase the factor of safety.

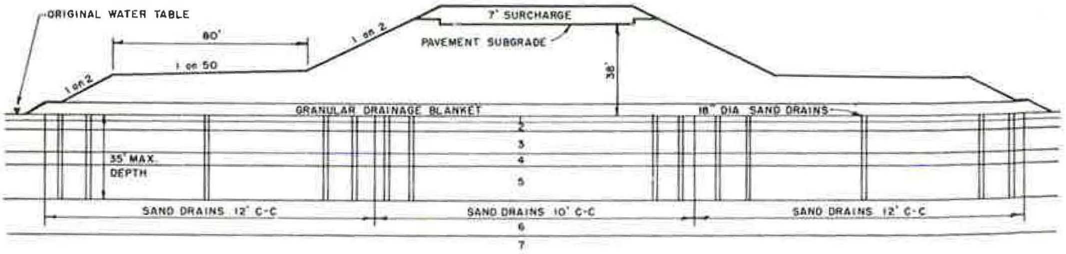
An exception to this analysis occurs if the weak foundation soil is relatively thin and the circular arc type failure does not occur. The failure condition consists of a lateral squeeze movement of the soft material resulting in sinking and sometimes spreading of the embankment.

The stability of various combinations of berm widths and heights are found to determine the most economical and satisfactory solution. The toe of the berm should extend beyond the arc of the most critical circle. Also, the berm should be checked for stability to ascertain that the berm itself will not fail. A series of step berms have been used to achieve stability of the berm embankment.

The stability analysis by graphical methods to determine the most critical factor of safety can be very time consuming. New York State has developed a stability analysis program for the electronic computer (4) to reduce the time required for this analysis. The program has been designed with sufficient flexibility to determine the factors of safety at any stage of construction for various degrees of consolidation of the foundation soil. Also, the program can determine the most economical berm requirements and compute the stability of embankments placed on sloping ground surfaces.

Sand Drains

Another method used to consolidate and stabilize foundation soils is the use of sand drains. Sand drains are columns of pervious sand installed in the soft foundation soils on a grid pattern varying from 6 to 20 ft on centers. A blanket of pervious sand is



TYPICAL SECTION
SCALE 1" = 40'

LAYER	THICKNESS	SOIL	AVE. MOISTURE CONTENT %	LIMITS			AVE. SHEAR STRENGTH		AVE. CONSOLIDATION CHARACTERISTICS		
				LL	PL	PI	CONSOLIDATED UNDRAINED C-LBS/FT²	UNCONFINED STRENGTH LBS/FT²	COMPRESSION INDEX	COEFFICIENT OF CONSOLIDATION C_v - FT²/DAY	INITIAL VOID RATIO
1	3'	SOLVAY SLUDGE	280	150	128	22	$\phi = 0^\circ$ C = 350	300	4.00	3.0	7.50
2	2'	BLACK PEAT	200	-	-	-	$\phi = 0^\circ$ C = 350	300	1.50	0.1	4.00
3	10'	GRAY MARL	70	48	35	13	$\phi = 20^\circ$ C = 200	180	0.54	1.0	1.85
4	5'	SILT, SAND AND MARL	30	26	20	6	$\phi = 15^\circ$ C = 200	250	0.1 - 0.2	0.5-1.0	0.75-1.00
5	15'	SOFT CLAY	38	32	18	14	$\phi = 10^\circ$ C = 200	180	0.35	HORZ 0.12 VERT. 0.07	1.05
6	15'	FIRM SILT	25	NPL			$\phi = 25^\circ$ C = 300	400	0.02	2.0-3.0	0.69
7	-	FIRM SAND	20	NPL			-	-	-	-	0.58

Figure 5. Typical cross section and avenue soil properties, Oswego Blvd., Syracuse, N.Y.

placed on the top of the drains to allow the water moving out of the top of the drains to flow laterally from under the embankment. The superimposed embankment weight causes a pressure in the soil pore water in the underlying soil. The presence of the sand drains decreases the time for the consolidation to occur. As the soil consolidates, the shear strength increases. Therefore, the principal benefit of sand drains is to increase the rate of consolidation of the subsoil and the stability of the embankment.

Over the past 15 years, the New York State Department of Public Works has installed sand drains on eight highway projects. In general, the results indicated that the design procedures based on Terzaghi's theory of consolidation for compressible soils and developed by R. A. Barron (5) for application to sand drain design are reasonable, sufficiently accurate, and yield satisfactory results. Post-construction subsurface investigations and testing indicated that the change in moisture content due to consolidation and the increase in shear strength was close to the values predicted in design.

A typical sand drain treatment project was used for the construction of Oswego Boulevard on the outskirts of Syracuse, N.Y. Embankments up to 45 ft in height were constructed on surface organic materials underlain by soft clays extending to a depth of 35 ft. Figure 5 shows a typical cross section of the highway embankment, which also required stabilizing berms in addition to the sand drain treatment.

Post-construction explorations were made 21 months after construction to determine the characteristics of the foundation soils. Figure 6 shows a comparison of foundation settlements as determined by four different methods. Figure 7 shows the theoretical time-settlement plot compared with the field records. Figure 8 shows the theoretical pore pressure behavior compared against the actual records.

The one project with unsatisfactory results occurred in a tidal marsh organic clay deposit on Long Island. Long-term secondary settlements were detrimental to the performance of the roadway. It is our opinion that the cause of this settlement was due to the disturbance of the soil structure by the displacement method used to install the sand drains. A mandrel with a flap gate on the end was employed. Six years later, on another project several miles away, sand drains were installed in a very similar organic clay deposit, but this time using a non-displacement method. This procedure used the hollow shaft flight auger. The secondary settlement was much less and the pavement

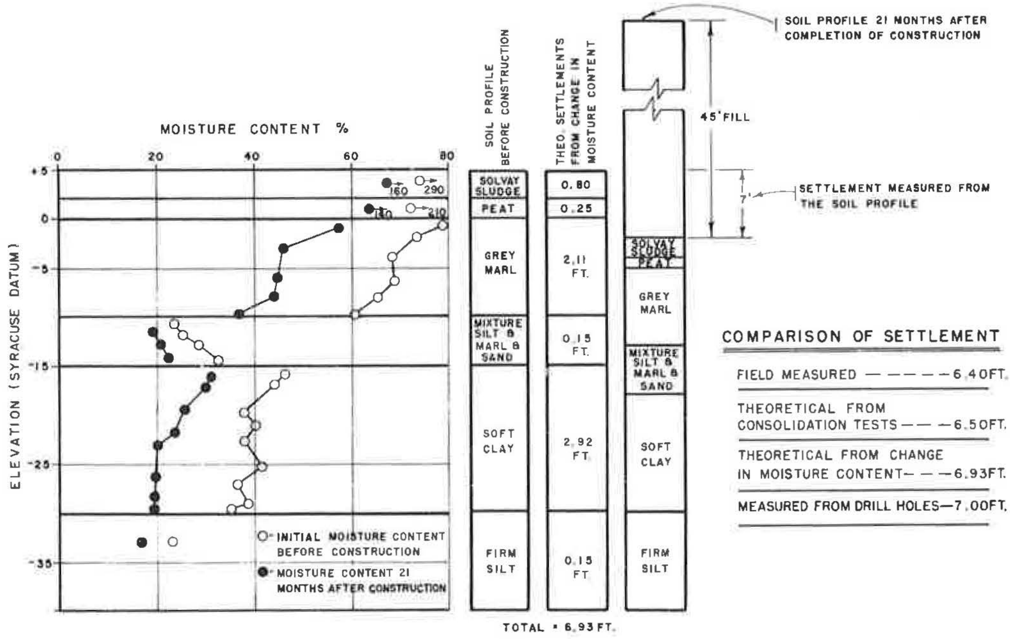


Figure 6. Moisture content—depth and settlement comparisons, Oswego Blvd., Syracuse, N.Y.

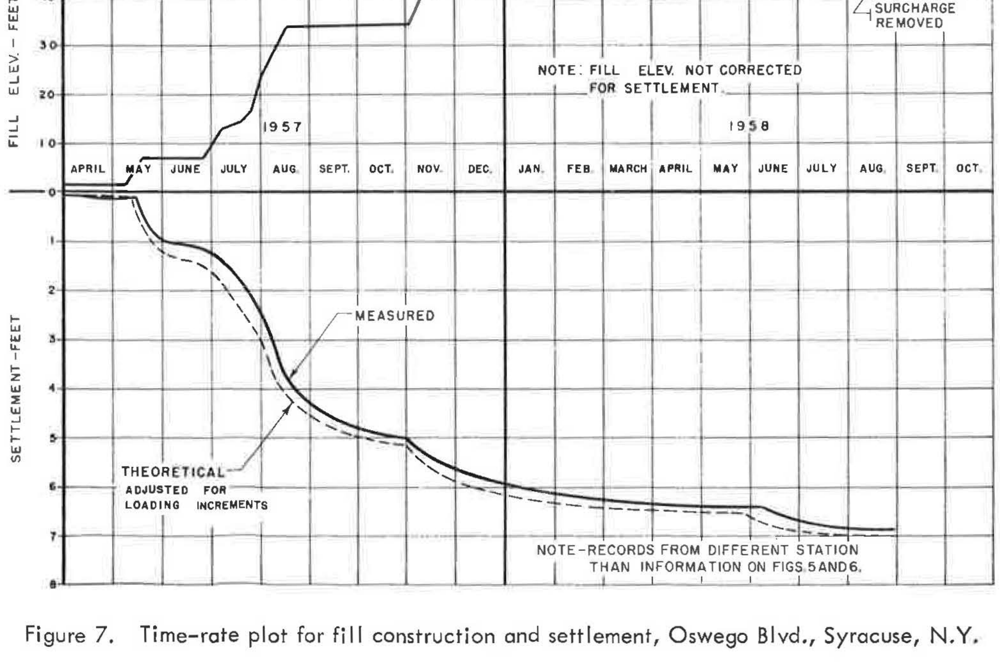


Figure 7. Time-rate plot for fill construction and settlement, Oswego Blvd., Syracuse, N.Y.

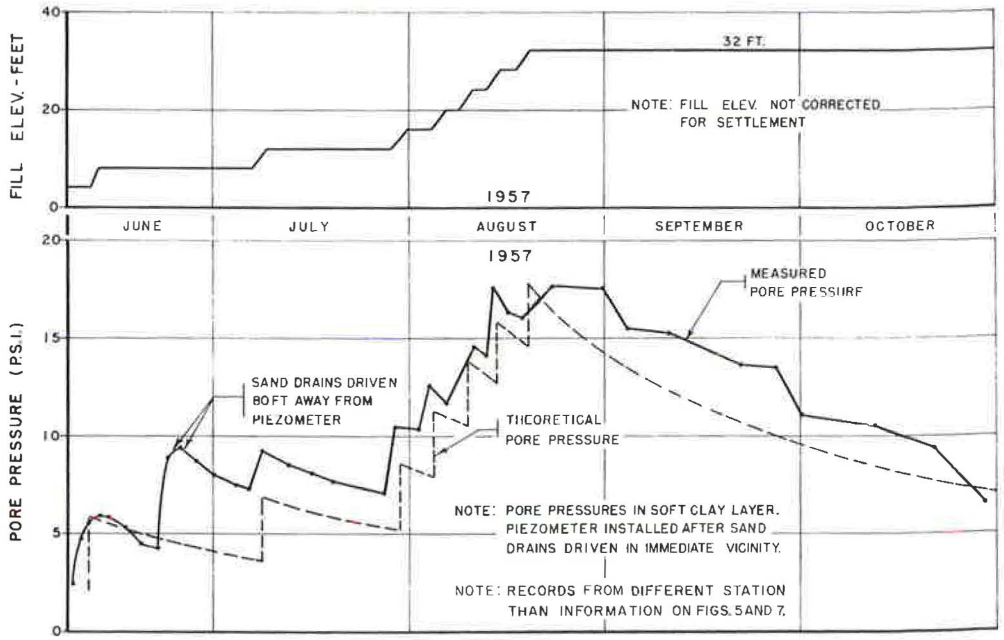


Figure 8. Time-rate plot for fill construction and pore pressures, Oswego Blvd., Syracuse, N.Y.

performance has been satisfactory to date except in one area where there was considerable peat mixed with the organic clay. Sand drains are not recommended for stabilization of deep deposits of high organic soils such as peat which are subject to long-term secondary consolidation.

The selection of sand drains for a solution to a foundation problem is also influenced by economics. The linear feet of drains required and the volume of free-draining sand can be large and this treatment can be quite expensive. For example, on six projects, the cost of sand drains per lineal foot varied from $0.72 on a large project to $1.65 on a smaller project. Because these projects were constructed over five years ago, a factor for increased labor costs would have to be added for present-day estimates.

Lightweight Fill

Another solution of stability problems is to decrease the weight of the embankment by using a material of a lighter density than ordinary earth embankment material. In steel-producing areas, quantities of water-cooled slag are often available. Water-cooled slag has a compacted wet density on the order of 70 to 80 pcf. Ordinary earth embankment density will vary from 125 to 135 pcf. The use of the lighter material will reduce the embankment weight approximately 40 percent, thus decreasing the stresses on the underlying soil. These lighter materials can be used if economically available.

Other materials that can be used are cinders and lightweight aggregate obtained from expanded shale. Cinders are becoming increasingly scarce and are difficult to obtain. In the northeastern U.S., there are 31 lightweight aggregate plants, but the quantity that can be obtained is often limited in terms of the volumes needed for highway embankments. The in-place weight of lightweight aggregate is 55 to 70 pcf, and in New York State the cost per cubic yard in-place is approximately $4.00 to $5.00 plus transportation charges.

Because of the limited quantities available, the principal use of lightweight fill has been in the vicinity of structure abutments where an alternate end berm treatment would involve lengthening the structure.

Surcharge

Embankments are often built to heights above future pavement grade to decrease the post-construction settlement. The effectiveness of the surcharge is dependent upon several factors which should be analyzed, such as the time-settlement characteristics of the foundation soil, and the ratio between surcharge height and final fill height. As the surcharge height-fill height ratio decreases, the effectiveness of the surcharge also decreases. The loading intensity of the surcharge increment on the compressible layers should be checked by usual pressure distribution methods. If the fill is high and the compressible layer is deep, then the surcharge will be relatively ineffective.

Also, the effect of a surcharge loading on the stability of the embankment for a foundation shear failure should be checked. Often a surcharge would have been desirable from the standpoint of settlement problems, but would have made the stability critical.

Surcharges have been used effectively in the areas of bridge abutments located on soils such as loose silts, fine sand, and clayey silts that consolidate rapidly. By preloading the abutment area, the structure settlement may be reduced to a magnitude that allows the use of a spread footing foundation instead of piles. The economics of a surcharge treatment should also be studied, since, in New York, payment is required for placing the material and also for removal to subgrade elevation.

CONCLUSION

The most satisfactory and economical solution to highway embankment foundation problems may often be obtained in the early design stages when alignment or grade changes may be made to eliminate or reduce the cost of foundation treatment. This requires a preliminary exploration program to provide the information for an early evaluation of the foundation problem.

The final selected treatment and detailed design is based upon a more extensive investigation program with emphasis placed upon careful undisturbed sampling and testing to obtain the most reliable design information possible. The selected solution to each problem is often influenced by factors other than the soil properties such as available construction time, availability and cost of construction materials, right-of-way and location of project.

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Discussion

PHILIP KEENE and ROBERT J. ISABELLE, Respectively, Engineer of Soils and Foundations and Assistant Highway Engineer (Foundations), Connecticut Highway Department—The author has given a clear presentation of the various methods of treatment which are used by his Department. His paper should be of considerable value to the practicing highway engineer and to the soils specialist.

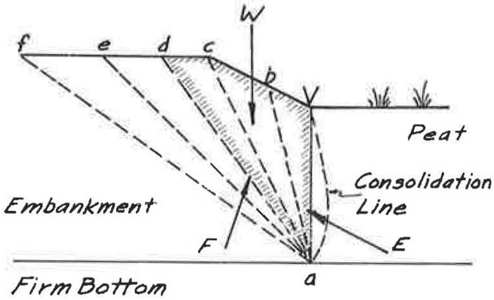

Figure 9.

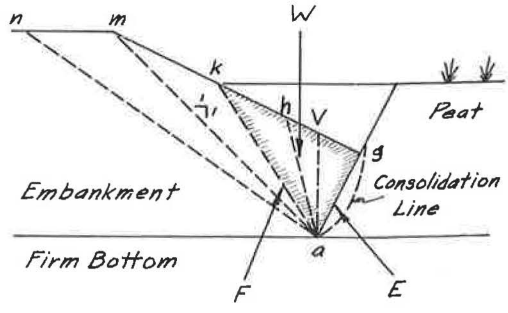

Figure 10.

The writers wish to discuss the desirability of the lateral limit of muck excavation for depths greater than 5 ft, described under "Removal of Soil by Excavation" and shown in Figure 1 of the paper. It is true that after excavating the muck and backfilling with select material, there will be no failure by shear through the muck which would result in large displacements and a "mud wave." However, there will be continuous small lateral and downward movement of the muck, due to consolidation, since the pressure from the backfill greatly exceeds the pressure which existed before construction. This movement, at first, would be a type of primary consolidation; in later months or years it would be secondary consolidation. The latter might be a few inches per year in magnitude and is similar to long-term settlements mentioned by the author. This movement is shown by the dotted line in Figures 9 and 10.

When the depth of muck is much greater than the height of embankment above swamp elevation, and the excavation limit is vertically below toe of slope, as line av in Figure 9, a crack in the shoulder area at d will result. This is because the wedge avd would gradually move to the right and cause a rupture along ad. If the muck experiences appreciable lateral consolidation after the shoulder is paved, lateral movement of the wedge avd would occur, the crack at d would open up and the shoulder would require resurfacing.

A safer practice, used by the Connecticut Highway Department, is to excavate out to point a in Figure 10, where a is on a 1 to 1 "directional" line through the top of slope, point m. Using this method, cracking due to lateral movement of a wedge of embankment would occur at point k, where it would cause no trouble. Field observations have verified this.

It can be added that the critical wedges, mentioned above, are determined by analysis of various trial wedges, such as avb, avc, avd, ave, etc., in Figure 9. For each trial wedge, the forces W (weight of wedge), F (frictional resistance in the embankment) and E (resistance of muck) are plotted. The wedge resulting in the maximum value of E is the critical one. The direction of F is assumed as 35 deg from the normal to the rupture plane of the wedge. Different assumptions can be made for the direction of F and E and the slope of the excavation limit, but the same conclusion would be arrived at, so long as point a is unchanged.

LYNDON H. MOORE, Closure—The author welcomes the comments presented by Messrs. Keene and Isabelle. The suggested wedge analysis method for analyzing the stability of the backfill against lateral movement is probably the best available at the present time. The author mentions in his paper that this type of analysis does indicate a critical stability condition, but experience with many swamp crossings has indicated that this does not prove to be a serious problem in the field. Although there have been a few instances where cracks appeared near the sides of the backfill during construction, there never have been any cases observed in New York where post-construction

distress of the shoulder or pavement has occurred due to the lateral movement of the backfill. Also, drill holes have indicated that the lateral boundaries of the backfill against the muck material have remained nearly vertical.

The author is not prepared to present a theoretical analysis of why the spreading does not occur. There is no doubt that on deep swamps there must be a pressure transmitted to the peat soil.

The additional width of excavation and backfill proposed by the discussors to satisfy the wedge analysis could amount to considerable additional cost on a long swamp crossing. It may prove to be more economical if a surcharge were substituted prior to paving to preload the sliding wedge and achieve a certain degree of equilibrium before paving.

Also, when the muck is excavated on a slope flatter than vertical, then the weight of the backfill will have a vertical component acting upon the compressible organic material. The backfill weight will cause considerable consolidation of the peat and certainly would result in cracking along the proposed failure plane outside of the shoulder limits. If this consolidation and movement of failure wedges is considerable, then a series of successive failure wedges could progress up the side slope of the shoulder of the embankment.

In conclusion, the writer agrees with the discussors that theoretically this is a problem that should be considered in design. However, from experience in New York State, it has been found that additional widening of the excavation beyond the toe of the embankment is not necessary to construct a stable backfill and embankment for a swamp crossing.

Observational Approach and Instrumentation for Construction on Compressible Soils

YVES LACROIX, Woodward-Clyde-Sherard and Associates

This paper describes the observational approach for construction on compressible soil. This approach consists of using observations and measurements to evaluate the performance of structures, both existing and under construction, for the purpose of deciding on corrective measures or improving design and construction of future structures. The paper contains a description of simple, practical instrumentation which provides the necessary quantitative observations. Observations and measurements, methods of recording, and typical interpretations are illustrated by case histories.

•THE OBSERVATIONAL approach or learn-as-you-go procedure, as it was often referred to by Terzaghi, consists of observing and measuring in the field the effects of successive construction steps on the surrounding soils and structures. The uncertainties usually involved in construction on compressible soils are often overcome by conservative design, but the added expense of being conservative does not always guarantee the success of the project. The observational approach often provides a more satisfactory answer.

The observational approach to engineering problems is possible because, in most instances, unsatisfactory performance or failure does not suddenly occur but is preceded by signs that can be recognized. By careful observation using adequate instrumentation and by competent interpretation of the results obtained, it is possible to predict the behavior of the structure being constructed, and the effect of construction on the adjacent structures. If necessary, changes may be made in the construction procedures or in the design of the structure, as construction proceeds, depending on the results of the observations.

Field measurements of full-scale structures also provide data which can be used directly in the design of other structures with similar soil conditions. The observational approach is, therefore, of great benefit when a series of structures is built in stages at the same location, making possible improvement in the design based on prior experience.

TYPE OF OBSERVATIONS

The most frequent type of observation is the measurement of horizontal and vertical movements. The movements occur in structures, at the ground surface, or at various depths in the subsoil. The observations may consist of measuring the displacement of reference points or of recording the formation and width of cracks. The stresses or forces in the soil mass such as those occurring beneath a footing or at the back of a retaining wall can also be measured, but such measurements are less frequent because this type of measurement is somewhat difficult.

The elevation of the water table and the porewater pressure in the soil are often important observations because variations in the position of the water table and variations of porewater pressure with time greatly affect the stability and settlement of structures on compressible soils.

Paper sponsored by Committee on Embankments and Earth Slopes and presented at the 45th Annual Meeting.

The field observations have one characteristic in common. To be usable in the observational approach, the successive values of the variable must be measured both as a function of time and as a function of the factors which may influence the variable. In this manner, the trend of the observations can be established and extrapolation becomes possible.

Observations can also consist of evaluating or measuring the engineering properties of subsurface materials during construction, at which time many measurements are possible at a reasonable cost. Although this type of observation is a part of the observational approach, the methods employed for determining the engineering properties of soils in situ are not discussed in this paper.

INSTRUMENTATION AND OBSERVATION PROGRAM

An instrumentation program consists of three phases: (a) installation of instrumentation, (b) observations and readings, and (c) interpretation. Because all the phases are equally important, every effort should be made when planning an instrumentation program to allot sufficient time and money for the satisfactory completion of the latter phases.

The instrumentation and observation program should be designed by an engineer having considerable experience in this field. The engineer should have sufficient authority so that he can carry out the field program with the cooperation of all the parties concerned. It is essential that the main objective of the instrumentation program be well defined and understood in advance.

The proportion of field instruments which become inoperative is usually high and it is advisable to use a greater number of instruments than would otherwise be necessary. Although initial readings are taken on all the instruments, regular readings may be limited to a selected number of instruments. The instrumentation should be installed at locations determined by a compromise between adequacy of data obtained, protection from construction activities, and ease of reading.

The recording of field readings should be facilitated by the use of adequate surveying equipment, convenient measuring scales, and clear and complete data sheets. The use of a mixture of units such as feet, inches, centimeters, and non-decimal fractions should not be permitted. The numbering system of the instruments should not be changed during the progress of the work.

Every effort should be made to obtain reliable zero readings, and there should be no doubt about the validity of calibration charts to which all subsequent readings are referred. If the instruments can be retrieved at the end of the program, the zero readings and calibrations should be checked. A field reading generally cannot be checked when an error is discovered at the time the results are interpreted. This lack of control should be compensated by taking the same measurement several times.

The results of the field observations should be assembled, as soon as they become available, in easily readable tables which exclude intermediate calculations. The data should be plotted to a carefully chosen scale to permit distinction between random variations and trends in measurements. It is often useful to compare a plot of the main variable vs time, with a temperature chart, river level chart, or other secondary variables plotted vs time.

INSTRUMENTATION

The field instrumentation should be as simple as possible because complicated instruments often become inoperative. Simplicity should be preferred to accuracy because in foundation engineering it is generally only necessary to obtain answers with an accuracy of ±15 percent. The effects of climate and weather should be taken into account in choosing instrumentation, as perfect waterproofing and protection from freezing are very difficult to achieve. The instrumentation should be designed to provide readings which are not affected by temperature, because the fluctuation of outdoor temperatures is commonly rapid and erratic. Instruments should be designed for a well-defined purpose. Multipurpose instruments should not be used when the main purpose of the instrument is compromised.

The following paragraphs describe a list of instruments which fulfill the demands of most instrumentation programs. More detailed descriptions of the instruments are available in the technical literature.

Bench Marks and Reference Points

Permanent bench marks of known reliability are an important requirement when measuring settlements or heaves. In many instances special installations are required. At least two permanent bench marks should be provided as there is always the possibility that one may be destroyed. If the permanent bench marks are some distance from the site, temporary bench marks should be installed at more readily accessible locations. The relationship between the temporary bench marks and the permanent bench marks should be determined, from time to time, to assure the continued reliability of the temporary bench marks. Many types of bench marks are described in the references.

Reference points, ranging from bronze screws set in masonry to simple scratch marks, may be placed on structures. The type of reference point used depends on the type of measurement and the duration over which the measurements are to be carried out, a more elaborate and better protected reference point generally being required for measurements extending over a period of years.

Surface Settlement and Lateral Movement Rod

Many types of reference points may be installed to permit the measurement of settlement, heave or horizontal movement using ordinary surveying methods. The surface settlement and horizontal movement rod shown in Figure 1 has been found convenient for installation at the surface of embankments. The measurements can be made to an accuracy of 0.01 ft.

Foundation or Embankment Settlement Plate

The foundation or embankment settlement plate shown in Figure 2a provides a means of measuring the settlement at a point in a foundation or embankment as fill material is placed. A base plate is placed on the foundation or embankment at a specific elevation and, as the fill is constructed, steel pipe sections are progressively extended vertically. Elevations of the pipe sections are measured before and after each additional section is placed. Because it is necessary to add successive lengths of pipe and because of the necessity of successive surveys when adding the pipe sections, the accuracy of the settlement value is rarely greater than 0.1 ft.

If the lower end of the pipe is perforated it provides a means of measuring water levels in the embankment. If reference pipes of two different diameters are provided, so that one pipe can telescope over the other, the settlement of plates at two different levels in the embankment may be measured by one installation, as shown in Figure 2b.

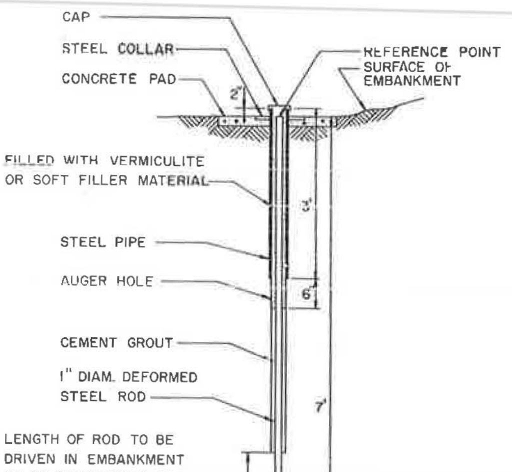

Figure 1. Surface settlement and lateral movement rod.

Water Level Settlement Gage

The foundation or embankment settlement plate described in the preceding section requires that pipes be extended vertically through the fill at the location of the plate, interfering with construction of the

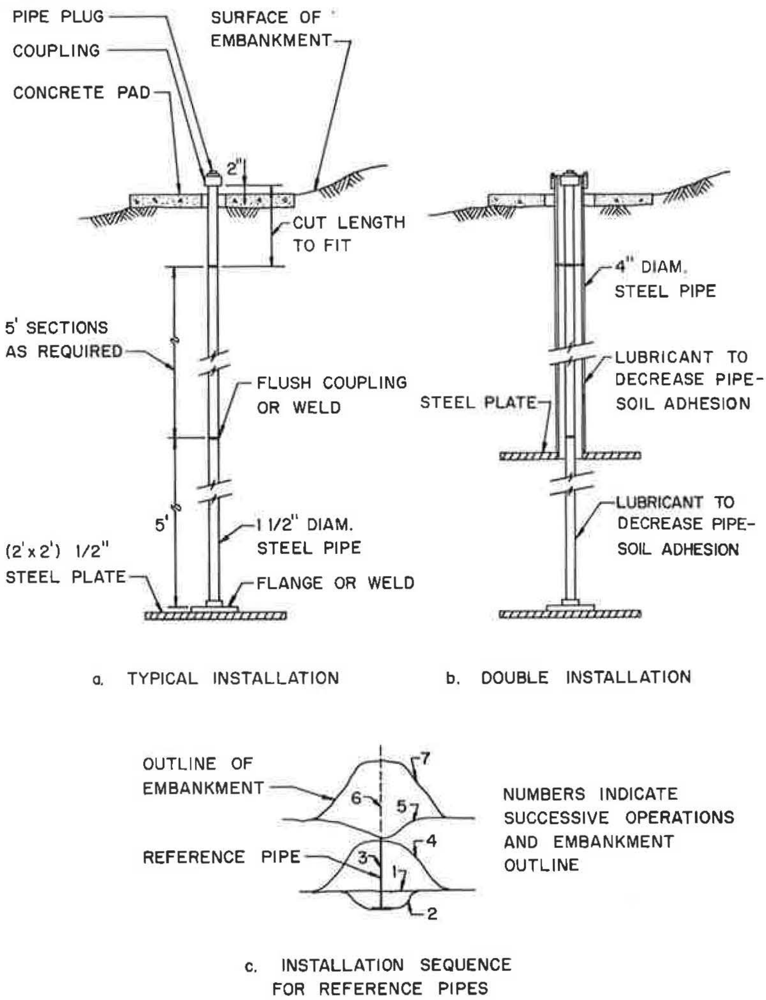

Figure 2. Foundation or embankment settlement plate.

fill. The water level gage shown in Figure 3 can be used to measure the settlement within an embankment with respect to a point located at the same elevation outside of the embankment. The device can also be used to measure the settlement of the back of a retaining wall as shown in the figure. The accuracy of the measurement is rarely greater than 0.05 ft.

Device for Measuring Heave of Bottom of Excavation

The heave of the bottom of an excavation can be measured by installing the heave point shown in Figure 4a within the soil mass prior to excavation. The heave point is attached to rods and pushed into the soil until it has reached the elevation at which heave is to be measured. The rods are then disconnected from the heave point and withdrawn. Alternatively, if the upper soil strata are too hard, the heave point is lowered to the bottom of a boring which penetrates the hard strata, and the heave point is pushed through the softer soils until it reaches the desired elevation as shown in Figure 4b. The elevation of the top of the point is recorded at the time of installation and again at the time the point is retrieved on completion of the excavation. The accuracy of the measurements is rarely greater than 0.05 ft.

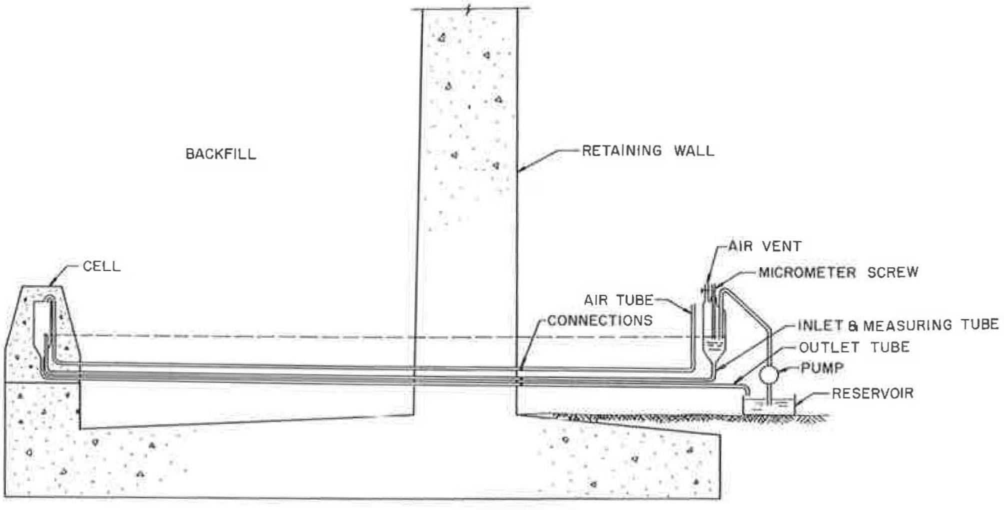

Figure 3. Water level settlement gage.

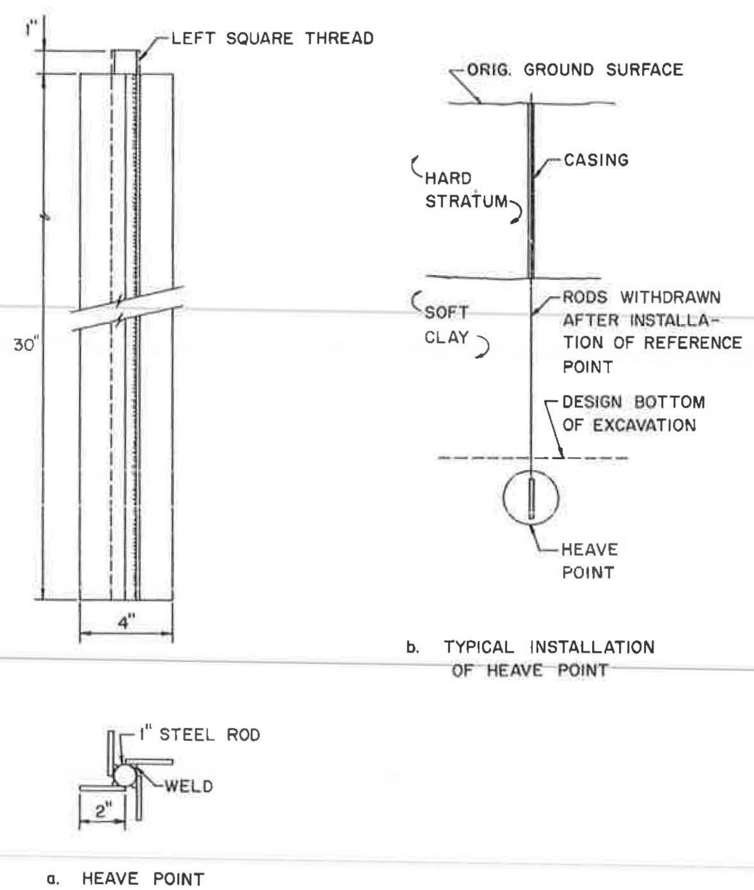

Figure 4. Device for measuring heave.

Wilson Slope Indicator

The Wilson slope indicator is an instrument for measuring deflections at depth. The instrument consists of a tiltmeter, a control box, an electric cable, and a 3-in. grooved aluminum casing which is installed in a 5-in. boring. The tiltmeter consists of a pendulum which, when activated by an electrical current, moves a conductor against a resistance and subdivides it into two resistances forming one-half of a Wheatstone bridge. The other half of the bridge is located in the control box. The pendulum is enclosed in a cylinder about 2. 5 in. OD and about 15 in. long. The tiltmeter cylinder is lowered into the aluminum casing by means of the electric cable and is kept in the constant azimuth of one set of grooves. Because the casing contains four grooves at right angles, the angle between the vertical and the casing can be measured in two rectangular directions. By integration of the readings, which are made at several depths, at different times, the successive deflections of the casing can be calculated. If the casing is fixed to a structural member, the bending moments in the member can be calculated from the deflection of the member, taking into account the structural properties of the member. The maximum angular deviation that the tiltmeter can measure is about 8 deg from the vertical. The accuracy of the readings is about 0. 001 radians which corresponds to about 1 in. in 100 ft. However, under favorable conditions displacements as small as 0. 1 in. may be measured.

Piezometers

In relatively pervious material with a coefficient of permeability $k>10^{-3}$ cm/sec, a wellpoint piezometer of the type shown in Figure 5a may be employed. A typical installation of such a piezometer is shown in Figure 5c.

In fine-grained soil with $k<10^{-3}$ cm/sec, a porous cylinder is used in place of the wellpoint. This cylinder may consist of plastic which is or is not surrounded by a sand filter. The porous cylinder has a lead consisting of plastic tubing extending to the ground surface. A typical plastic porous cylinder piezometer is shown in Figure 5b.

The water level in piezometers may be determined by means of an electrical probe. This probe basically consists of a thin double-conductor electric cable. The conductors

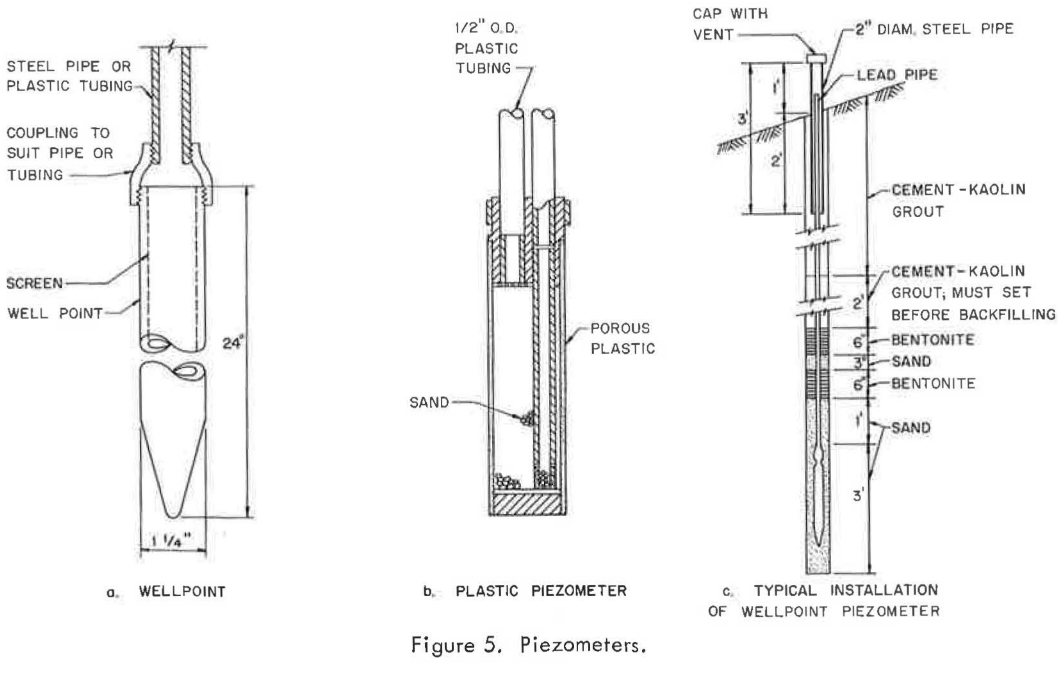

Figure 5. Piezometers.

at one end of the cable are connected to an ohmmeter and battery; the other end of the cable is introduced into the piezometer tube. When the conductor at the lower end of the cable comes in contact with the water, a sharp variation in resistance is indicated on the ohmmeter. If the piezometric level is higher than the top of the lead tube it may be measured by a Bourdon gage.

Because it is not possible to push an electric cable into the lead tube of a piezometer over great horizontal distances, another type of reading device may be used. The outlet of a small diameter air tube is connected to the lead tube at a predetermined elevation which must be below the lowest expected piezometric level. The vertical distance between the outlet of the air tube and the piezometric level is calculated from the air pressure which must be applied to the inlet of the air tube in order to produce a small air flow. The inlet of the air tube can be located in any convenient place outside the construction area. A piezometer equipped with an air tube is referred to as an air-activated or bubbler piezometer, which is described in the references.

Strain Gages

Several types of strain gages can be used to measure the strain occurring in a structural member under load. Some of the instruments available include the Whittemore mechanical gage, the Baldwin SR-4 electrical gage, and the vibrating wire gage.

The Whittemore gage consists of a frame formed of two parallel bars connected near their ends by spring fulcrum plates which prevent motion of one bar relative to the other except in a longitudinal direction. A steel point is attached to one side of each bar. The steel points are inserted in small holes drilled in the member. The gage is 10 in. long and the relative motion of the bars is measured by a dial micrometer graduated to read 10^{-4} in.; thus, unit strains as low as 10^{-5} can be measured with the gage. Although the Whittemore gage is accurate enough for foundation engineering, a very meticulous operator is required and readings are time consuming.

Baldwin SR-4 strain gages operate on the principle that the electrical resistance of wires attached to a structural member is related to the change in length of the wires as a result of strain in the member. These gages must be waterproofed and extreme precautions must be taken to keep the resistance of the electrical connections constant. Baldwin SR-4 strain gages can be incorporated in a load cell as shown in Figure 6. This type of load cell has been successfully employed for a period of nearly a year as outlined in the case history given later.

Vibrating wire strain gages typically consist of a steel tube with circular steel plates attached to each end of the tube. An axial pretensioned wire is stretched between the center of the plates, and an electromagnet within the instrument plucks the wires and picks up its vibrations. Strains, due to the load to be measured, are transmitted to the tube and to the wire altering the frequency of the wire. The gage is read by tuning a similar wire in a receiver to the same frequency as the wire in the instrument by observing traces on a cathode ray tube. The vibrating wire gage has the advantage that changes in the properties of the electrical circuit do not alter the frequency of vibration of the wire. However, the gage is not yet commonly employed.

Earth Pressure Cells

The earth pressure at the interface of a structure and soil, and the earth pressure within a soil mass, may be measured with earth pressure cells. However, such measurements are difficult and are usually limited to special projects.

The Carlson stress meter, which is described in the references, is solid and sufficiently sensitive and reliable provided it is properly installed. However, in common with electrical gages, difficulties often arise in measurement due to changes in the resistance of electrical connections. Vibrating wire pressure cells which eliminate this problem, are to be preferred but are expensive.

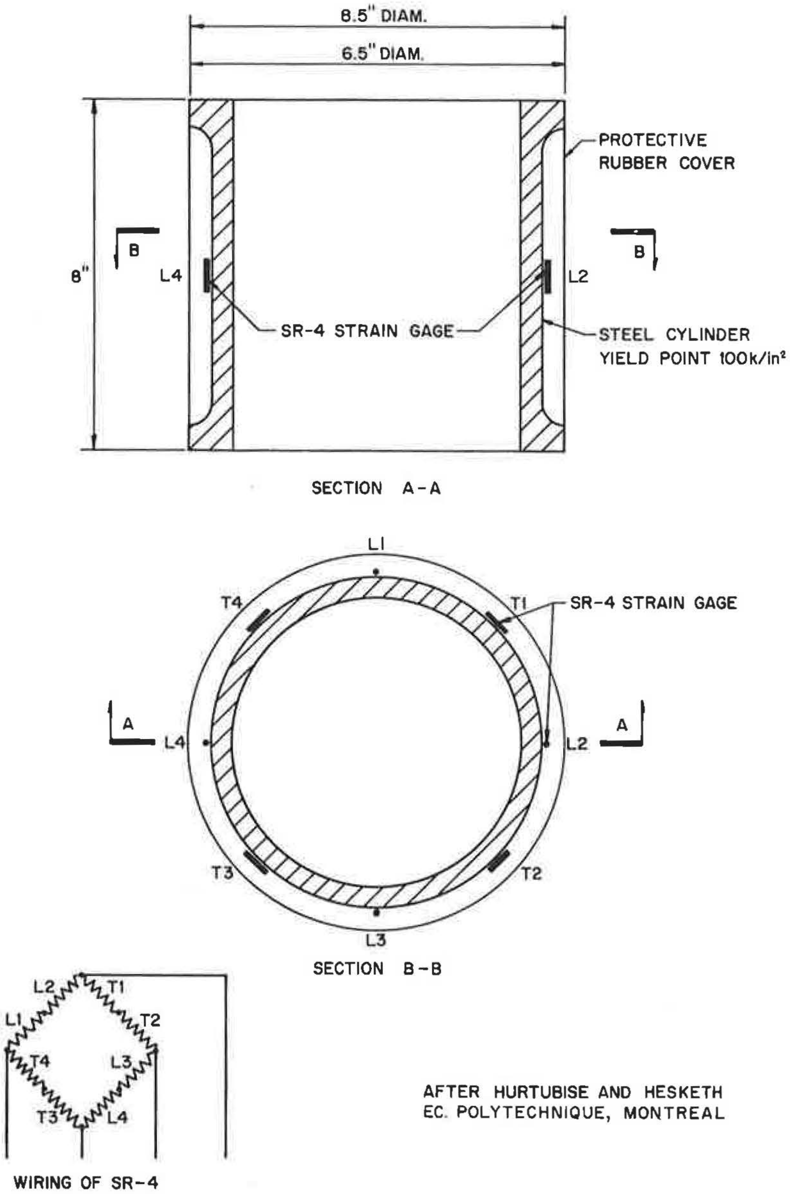

Figure 6. Load cell.

CASE HISTORIES

Observation of an Anchored Wall in Montreal

This case history describes the observational techniques and instrumentation employed in the construction of the side walls of a deep excavation for the Berri-DeMontigny subway in Montreal.

The excavation through strata of sand, silt, and very dense till was extended to a depth of about 60 ft into shale (Fig. 7). The excavation was supported by a wall

consisting of 30-in. diameter reinforced concrete drilled piers at 10-ft centers with a gunite membrane between the piers. The piers were tied back with steel cables anchored into the shale outside the excavated area. The excavation was close to streets, utilities, and multistory buildings founded on shallow spread footings.

The instrumentation consisted of reference points on the surrounding buildings and on the top of the piers (Detail A, Fig. 7) and of load cells in selected tie-backs. The load cells, which incorporated Baldwin SR-4 strain gages (Fig. 6), were designed for a maximum load of 400 kips. Calibration readings taken before and after use indicated that the drift in the zero readings was no greater than five kips.

Because of the large size of the excavation and the number of reference points, it was impractical to survey all of them at frequent intervals. However, all the reference points were surveyed two or three times after they were installed to obtain a good set of initial readings. Then, regular observations were limited to selected areas, the frequency of the readings at any one location being dependent on the trend indicated by

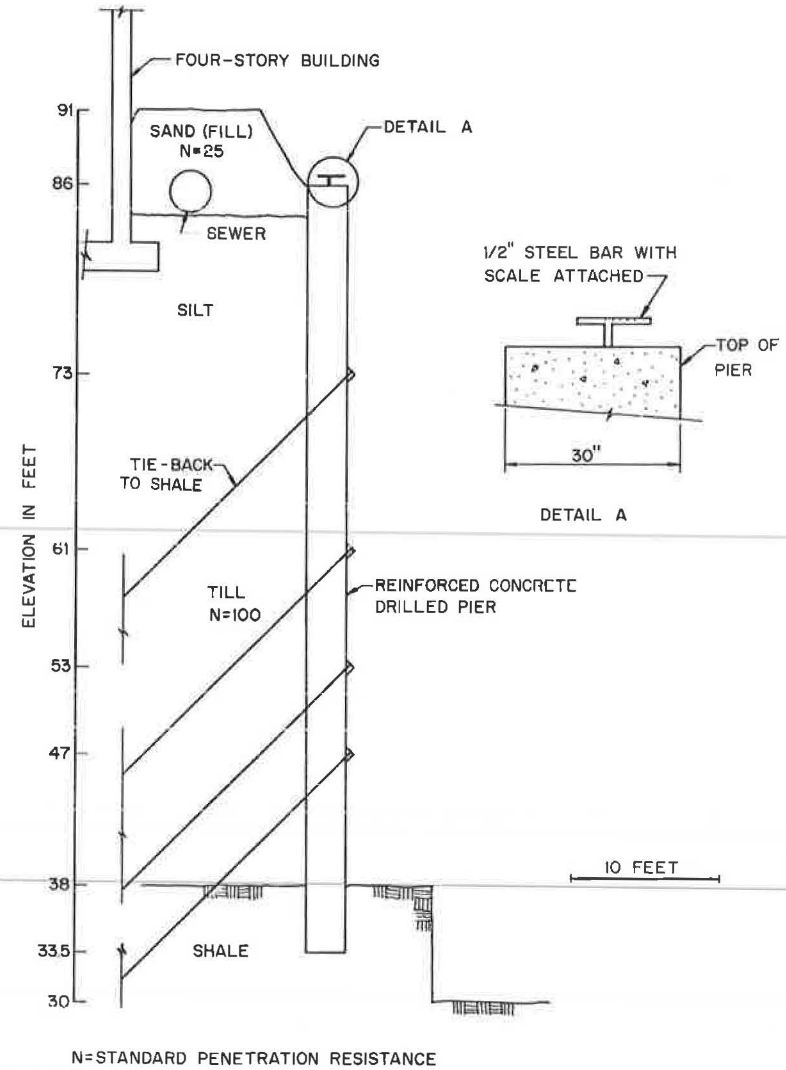

Figure 7. Berri-DeMontigny subway station, Montreal: typical section of anchored wall.

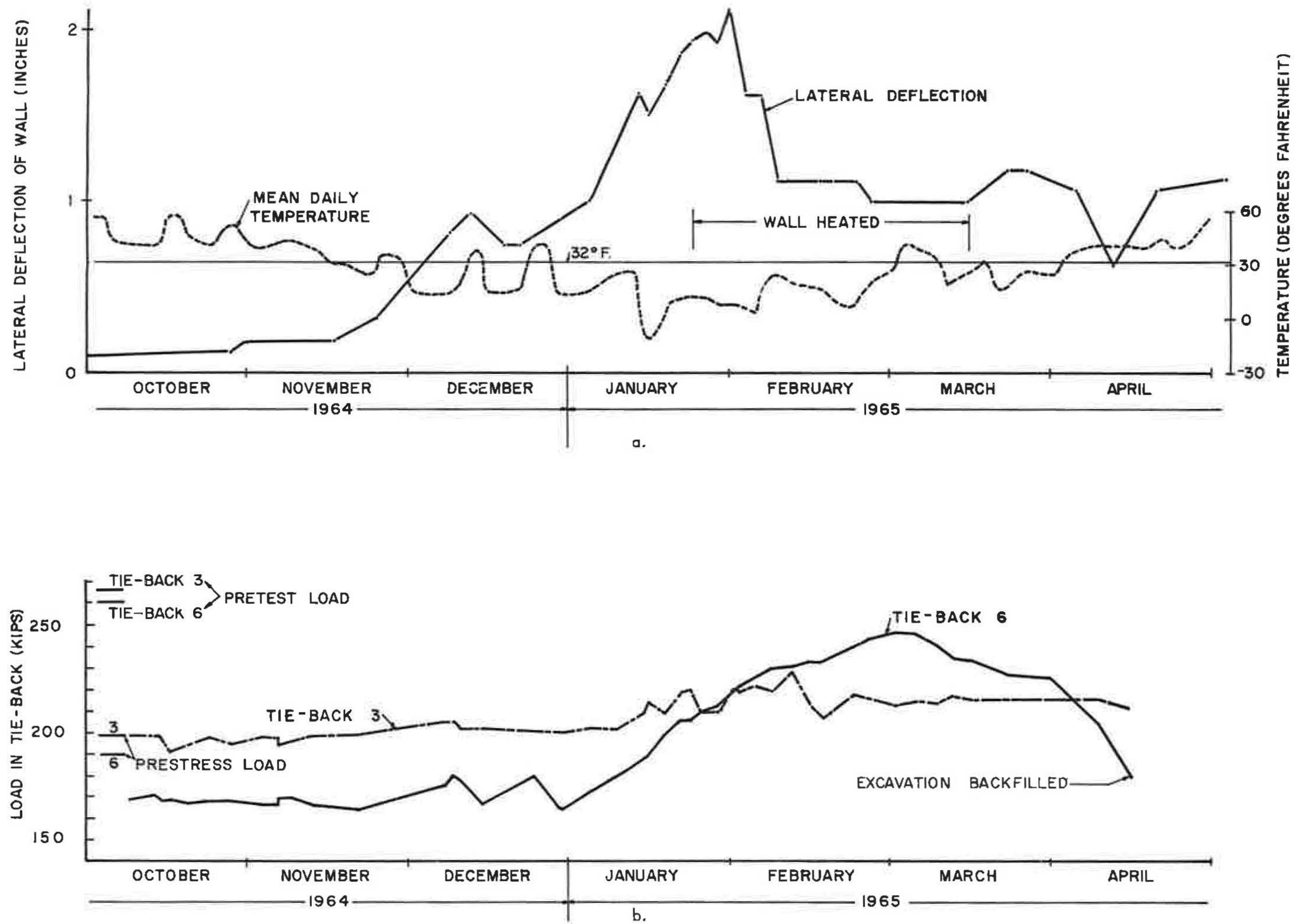

Figure 8. Berri-DeMontigny subway station, Montreal: observations on anchored wall.

previous measurements at the location. If abnormal movement had occurred in areas not surveyed regularly, the availability of the reference points and the initial readings would have permitted a measurement of the total movement.

When the tie-backs were installed they were pretested to a load at least equal to the design load. The load was then released to a prestress load equal to about 75 percent of the design load. As the excavation was continued the load in the tie-backs increased but not to the extent anticipated in the original design, and it was possible to reduce the number of tie-backs at some locations by as much as 30 percent.

The results shown in Figure 8 are typical of the observations which were made on the wall (Fig. 7) during the winter of 1964-1965, when the depth of the excavation did not change. The deflection of the wall in the direction of the excavation was not significant until the middle of November 1964 when freezing temperatures first began to occur (Fig. 8a). In early December, overflow and leakage from a sewer saturated the silt behind the wall. The load in the tie-backs and the deflection of the wall increased sharply, probably due to freezing of the material behind the wall. Although the wall moved back slightly during a period of mild temperatures in December, the deflection continually increased during prolonged freezing temperatures. By the middle of January the total deflection was of the order of 2 in. Further movement of the wall was considered undesirable because settlement of the adjacent building was likely to occur. The lateral movement was attributed to freezing of the material behind the wall, and heating of the wall was adopted as a remedial measure.

An enclosure consisting of a timber framework and a plastic covering was installed around the wall adjacent to the building. The temperature within the enclosure was maintained at about 36 F. The wall moved back soon after heating began and the total deflection remained at about 1 in.

The loads in two typical tie-backs during this period are shown in Figure 8b. The load in tie-back 3, located within the heated portion of the wall, did not increase any further once heating was begun. The load measured in tie-back 6, in an adjacent unheated wall, increased during the entire cold period and did not decrease until the weather became mild in the spring.

At this site, the observations indicated that (a) the design assumptions as to the magnitude of the earth pressure were conservative—several tie-backs could be safely omitted; and (b) prolonged freezing temperatures caused an increase of the load in the tie-backs and of the deflection of the wall. When the maximum permissible deflection of the wall had been reached, heating of the wall was effective in preventing further deflection.

Observation of a Braced Excavation in Chicago

This case history describes the observational techniques and instrumentation employed by the Chicago Park District in the construction of a braced excavation for an underground garage.

A sheet pile wall was driven around the periphery of the site through sand fill and soft clay to a layer of stiff clay (Fig. 10b). The foundation of the garage consisted of a slab constructed on the soft clay; such a solution proved feasible as the weight of the garage was less than the weight of the excavated material.

Because of the proximity of major buildings, including the Chicago Orchestra Hall and the Chicago Art Institute, the construction of the garage was to be undertaken in such a manner that the movements of the surrounding area would be minimal. Provisions were made to obtain readings of deflection of the sheet pile wall using the Wilson slope indicator during the construction period. The instrumentation also included reference points on adjacent buildings and on the sheet pile wall. Heave measuring devices were installed at the foundation level of the proposed garage.

The construction of the garage was done in stages as shown in Figure 9. The successive deflections of the sheet pile wall at one typical location, measured using the Wilson slope indicator, are shown in Figure 10a. These deflections have been plotted assuming that the bottom of the wall did not move. From October 24, 1963, when the slope indicator was installed, to March 26, 1964, when excavation was started in this

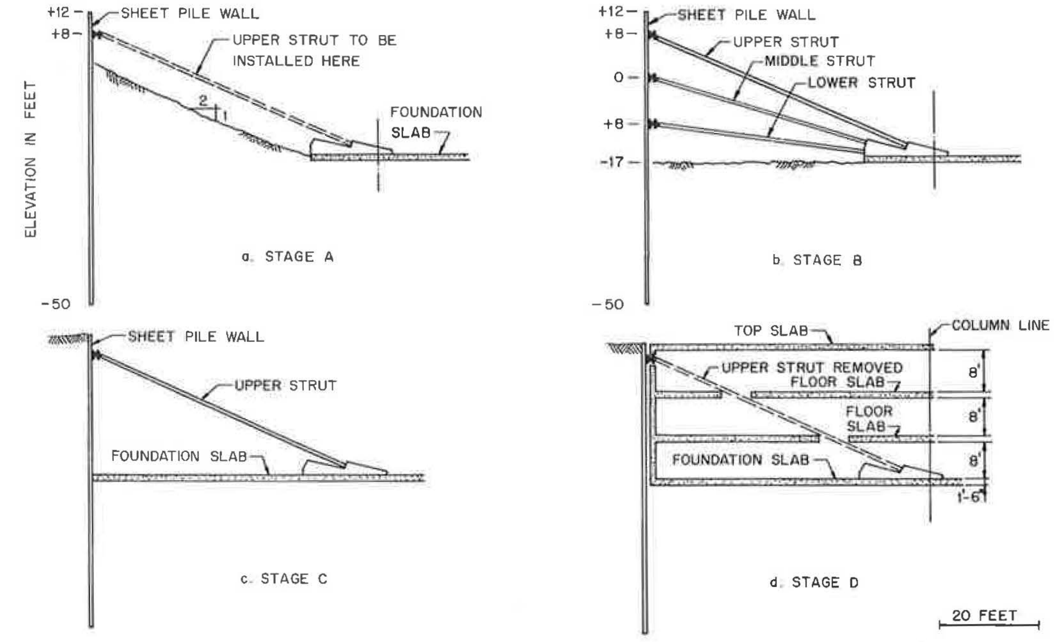

Figure 9. Underground garage, Chicago: construction stages.

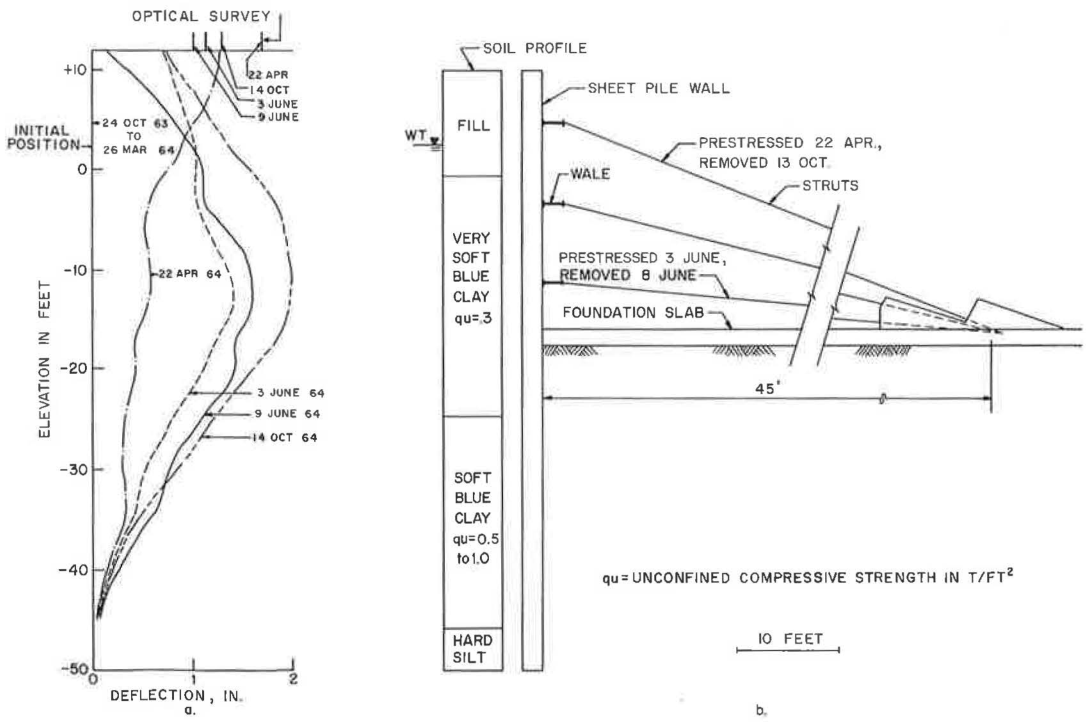

Figure 10. Underground garage, Chicago: soil conditions and observations on braced sheet pile wall.

area, the sheet pile wall remained vertical, as shown by the slope indicator readings. Initially, only the material in the center of the garage was excavated and a wedge of soil was allowed to remain against the sheet pile wall. The foundation slab in the center of the garage was constructed and the upper strut installed and prestressed (Stage A in Fig. 9a). Just before the upper strut was prestressed the wall had deflected as shown by the curve for April 22, 1964, in Figure 10a. The soil against the sheet pile wall was excavated in sections and the middle and lower struts were installed and prestressed (Stage B, Fig. 9b). At this stage, the deflections of the wall were as shown by the curve for June 3, 1964, (Fig. 10a). The foundation slab was extended to and poured in contact with the sheet pile wall. As soon as the strength of the foundation slab was sufficient to provide an effective support for the wall, the lower and middle struts were removed, which greatly facilitated construction (Stage C, Fig. 9c). After removal of the lower strut and just prior to the removal of the middle strut, the deflections were as shown by the curve June 9, 1964, in Figure 10a. The floors of the garage were constructed leaving openings for the upper struts and these struts were removed after the top slab was installed (Stage D, Fig. 9d). The deflections of the wall at this stage were as shown by the curve for October 14, 1964, in Figure 10a. Subsequent observations of the wall showed that the deflection remained constant.

The result of the optical survey shown at the top of Figure 10a confirms the trend of movement of the sheet pile wall during the various construction phases. However, the optical survey data show greater movement of the top of the wall than the slope indicator readings. Better agreement can be obtained between the optical survey and the slope indicator readings by assuming that the bottom of the sheet pile wall moved about ½ in. between March 26 and April 22, 1964.

At this site, the observations confirmed the predicted behavior of the wall and established that the deflections were minor and would not affect the adjacent buildings. The slope indicator readings showed that deflection of the sheet pile wall extended below the bottom of the excavation at all construction stages. Although such deflections are unavoidable, they are at a minimum when the struts are placed at a close vertical spacing. The removal of the intermediate struts after pouring the foundation slab caused only small deflections which indicate arching of the soil between the upper strut and the foundation slab. The heave of the foundation was small, on the order of ½ in. Therefore, recompression of the clay due to loading from the garage was small.

Observation of Groundwater Table at an Industrial Site

This case history illustrates the use of instrumentation in determining the groundwater table at an industrial site.

The site was located on a river terrace and the soils consisted of alluvial deposits. The soil stratification was irregular, the materials consisting of sand and gravel with variable amounts of silt. Most of the foundations consisted of spread footings constructed at a wide range of depths below the original ground surface, depending on grading requirements in various areas of the site. It was also desirable to establish the footings above the water table to avoid disturbance of the granular materials and to facilitate construction. Instrumentation was therefore installed to determine the groundwater table.

The instrumentation consisted of piezometers installed in selected exploratory borings. The tips of the piezometers were placed at varying depths and were surrounded by a sand filter. The upper portion of each boring was filled with cement grout to prevent ingress of runoff water. The water table as shown by the water level in the piezometers was observed at frequent intervals.

The position of the water table as measured in the piezometers was confirmed when excavations were made for the footings, with the exception of one particular area where the water table was higher than anticipated. The soil stratification in this area consisted of loose silty sand from the ground surface to a depth of 10 ft, medium-dense gravelly silty sand from 10 to 18 ft, and very dense sand and gravel below 18 ft, (Fig. 11). The tip of the piezometer was located in the dense sand and gravel at a depth of 30 ft and the water level measured in the piezometers was at a depth of 15 ft. However,

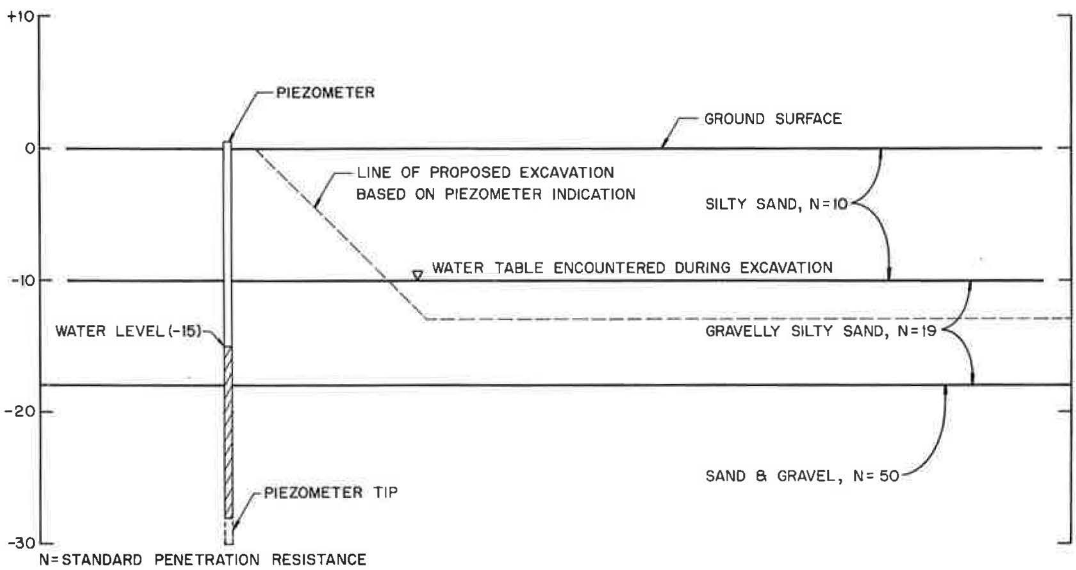

Figure 11. Industrial site: soil conditions and water table observations.

on excavation the water table was encountered 5 ft higher than anticipated on the basis of the piezometer readings. The relevance of the piezometer readings was therefore re-examined.

The relatively impervious nature of the gravelly silty sand stratum had not been recognized, in that this stratum was impervious enough to permit the development of a perched water table at a depth of 10 ft. This perched water table was not disclosed by the piezometer because the tip of the piezometer was located in the underlying pervious sand and gravel. The water table as implied by measurements of water levels in piezometers must, therefore, be critically interpreted in conjunction with the soil profile.

CONCLUSIONS

Because it is frequently impossible to predict the behavior of compressible soils in advance, engineers and contractors must often resort to radical and expensive solutions or accept an unknown risk. The observational approach reduces the number of cases in which such alternatives are necessary because it is possible to learn as you go on the basis of field observations. The observational approach also provides useful data for the design of new structures in similar soil conditions.

The observational approach requires adequate instrumentation and competent recording and interpretation of the results. Good organization of the field program is a prerequisite and cooperation among the owner, engineer, and contractor is essential.

The observational approach is widely accepted on projects such as large earth dams. There is no reason why it should not be applied to smaller projects.

ACKNOWLEDGMENTS

The writer wishes to express his gratitude to J. V. Sheehan of Woodward-Clyde-Sherard and Associates, who provided considerable help during the preparation of this paper, and to S. Revay, general manager of Dufresne Engineering Company Ltd., and R. A. Black, chief engineer of the Chicago Park District, for permission to publish the data presented in the case histories. The writer was employed by Dr. R. B. Peck, consulting engineer to the Chicago Park District, during the design and construction of the underground garage, and the writer's firm was retained by Dufresne Engineering Company Ltd. during the construction of the Berri-DeMontigny subway station.

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Method of Installation as a Factor in Sand Drain Stabilization Design

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•THE EFFECTS of the method of installation of sand drains on the in situ characteristics of the subsoil being treated are critically important in evaluating the design assumptions and cost of construction in establishing the feasibility of employing sand drain stabilization for any given project. It is the purpose of this paper to review the factors entering into the design of sand drain installations for use in embankment foundation stabilization, and to develop the concept of "grid efficiency" as a basis for comparing the performance of drains installed by various methods. This concept is employed to explain the greater effectiveness of drains installed by the newly developed flight auger method as compared to results obtained with drains installed by the mandrel method in connection with highway projects in the Flushing Meadows area of New York City, where both methods of installation have been employed in similar soils.

General

In saturated soils, the rate of consolidation of the soil under load is directly related to its permeability, and the volume change, as represented by its settlement, is essentially equal to its volumetric reduction in moisture content. Although soil consolidation, or settlement, is normally measured in the vertical plane, the flow of water from the soil may occur in any direction. The time required to achieve any degree of consolidation is proportional to the mathematical square of the maximum distance that water in the compressible soil must travel to leave the consolidating portion of the stratum involved (1, p. 265ff). Sand drains represent artificially installed zones through which such water may leave the soil, and by arbitrarily establishing a spaced grid of drains, the maximum distance that water must move (drain horizontally) to leave the system can be fixed to meet the requirements of any stabilization schedule (1, p. 290ff). A schematic representation of the direction and maximum distance of water flow for the sand drain and non-sand drain conditions are shown in Figure 1 for single and double drainage.

Economic Considerations

Sand drain stabilization is not employed automatically when roadways are to be constructed over compressible subsoils, but rather when there is a time limit imposed by a construction schedule. Even then it is used only when it is more economical to do so compared to alternative construction procedures, such as excavation of unsuitable material, overloading, use of viaduct, as well as combinations of these and other possible alternatives. Depending on factors controlling the construction, such as right-of-way limitations, availability of materials, etc., the use of sand drain stabilization may effect substantial savings over alternative procedures in the construction of highways over compressible soils. Therefore, sand drains represent an engineering tool that can be rationally employed to minimize total construction costs where unsuitable or unstable subsoil conditions exist.

Paper sponsored by Committee on Embankments and Earth Slopes and presented at the 45th Annual Meeting.

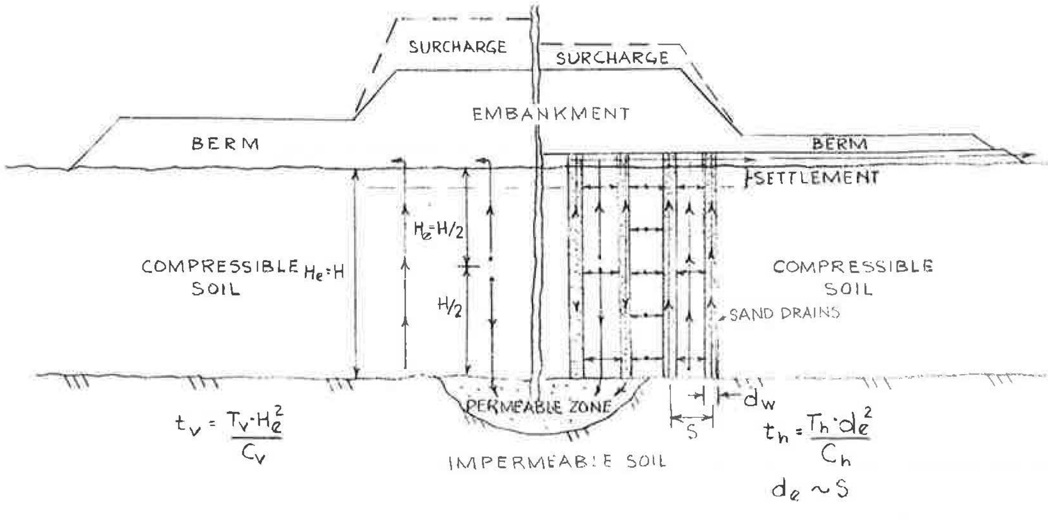


Figure 1.

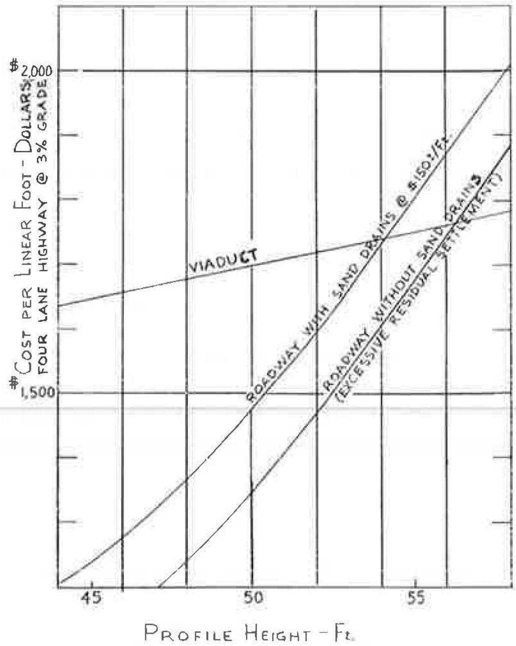


Figure 2.

The results of an economic evaluation of the feasibility of a sand drain installation are shown in Figure 2, based on an actual cost study prepared for a proposed 4-lane highway approach to a high-level crossing of a major waterway, utilizing a maximum 3 percent roadway grade. The figure relates the incremental (per foot) cost of construction for each of three methods of construction with the profile height above existing ground. The three methods of construction used are a viaduct, a surcharged embankment stabilized with a single berm, and a surcharged embankment with a single berm and sand drains. Because the available right-of-way limited the roadway construction to the use of a single berm, the surcharge height for the main embankment was limited by the maximum height of berm that could be constructed safely. As a result, it was not possible to adequately surcharge the embankment to preconsolidate the soil sufficiently to meet design requirements within the 18-month period allowed for the work. Although other alternatives were considered, including partially excavating the compressible soil, it was established that sand drains were the only economically feasible means to achieve the desired result. It was determined that the use of 18-in. diameter drains spaced 10 ft on centers in a rectangular pattern would adequately expedite settlement to meet the 18-month schedule. The non-sand drained system would require 10 years for stabilization to achieve the same degree of consolidation. Figure 2 shows that the break-even point between the sand drain section and the viaduct occurs at a profile height of 54 ft. Thus, the most economical construction would utilize a

sand drained embankment where the profile height is less than 54 ft and a viaduct where the height is 54 ft or more. Inasmuch as the time required for sand drain stabilization varies approximately with the square of the grid spacing used, if the work schedule were extended beyond 18 months, a wider spacing could be employed which would reduce the incremental cost of construction. This in turn would raise the profile height at which the break-even point occurs. The reverse would be true if the construction schedule were reduced. In this manner, the proper selection of sand drain spacing can be tied to any schedule established for the stabilization period, and thus minimize the total cost of construction for the project.

Project Experience

The Whitestone Expressway is part of the Federal Interstate highway system, and passes through the Flushing Meadows area of New York City. A portion of this road, section 61-1, was designed by Praeger-Kavanaugh and uses sand drain stabilized embankment in areas where the underlying organic deposits exceeded 20 ft in depth. Although this was not the first project in the area to utilize sand drain stabilization, it was the first project which required the use of nondisplacement sand drain techniques—thereby prohibiting the use of the heretofore commercially accepted mandrel method of installation (see Appendix). This change in specification requirements was based on experience obtained in connection with an earlier use of sand drains for a section of roadway through the same area at the time of the construction of the Long Island (formerly Horace Harding) Expressway. In the latter case, the mandrel method of installation was employed. Although primary consolidation of the subsoil occurred as had been estimated in design, it was found that the secondary post-construction settlements were many times the value expected or predictable from the results of laboratory tests. As a result of the extensive post-construction maintenance required, the road was shut down only one year after its completion to permit regrading an area which had settled 2 ft in that time. The organic soil involved was originally 35 ft thick (liquid limit of 80 percent and plastic limit of 40 percent) in which sand drains were installed 6 ft on centers in a square pattern. Similar settlements occurred in the remainder of the sand drain area where the grid spacing was 10 ft on centers.

As a result of investigations by Babylon District 10 of the New York State Department of Public Works and the Bureau of Soil Mechanics, it was theorized that the excessive secondary settlements noted were related to the disturbance effects of the mandrel method of sand drain installation. Because the subsoil conditions at the location of the Whitestone Expressway section showed characteristics that were essentially the same as found in connection with the Long Island Expressway work (liquid limit of 80 percent and plastic limit of 40 percent, and moisture contents in situ averaging 90 percent or more), it was decided to avoid the use of the usual mandrel method, and to require the application of a nondisplacement method. The correctness of this decision is clearly evident in Figure 3, which compares the percent of primary consolidation vs time for the Whitestone and Long Island Expressway projects. For the Whitestone project, where the minimum spacing employed was 8 ft on centers in a square grid, the time to attain 100 percent (estimated) of primary consolidation was found to be $1/30$ that required for the Long Island Expressway area where the grid spacing was 6 ft on centers. Correcting for the difference in grid spacing, consolidation in the sand drain section for the Whitestone project occurred more than 40 times more rapidly than the best results achieved for the Long Island Expressway construction. Furthermore, the secondary settlements recorded for the Whitestone sand drain sections were found to be predictable from laboratory test results. These and other findings will be discussed in greater detail after a brief review of sand drain theory and practice, which will form the basis for understanding the substantial difference in results obtained in the foregoing projects.

THEORETICAL ASPECTS

The deformation of a saturated compressible soil matrix under load is related to the value of the load applied, and its volume change is related to the change in its moisture

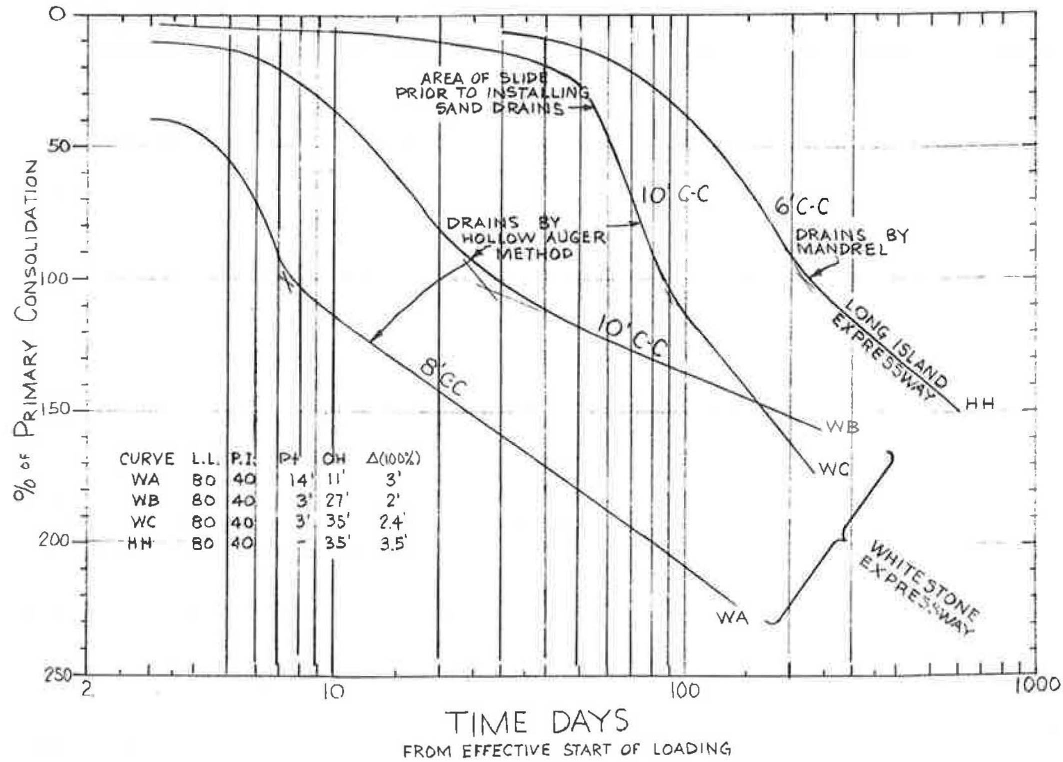


Figure 3.

content. The proportion of the load supported by the soil matrix is related to the degree of settlement observed at any given time. Thus, when a load is initially applied to a soil, and no settlement has had time to occur, the full value of the load is supported by the water within the matrix which reflects as an increase in its normal hydrostatic pressure. This pressure increase is termed pore pressure. As consolidation progresses, and the load on the soil is transmitted from the water to the matrix, the pore pressure diminishes proportionately until 100 percent of primary consolidation is attained, at which point the pore pressure theoretically is zero. From a practical point of view, 90 percent of primary consolidation is the maximum that is normally attempted under a given load (as the 100 percent condition theoretically requires an infinite time to achieve), and the remaining 10 percent is eliminated when necessary by means of surcharge or overload. In nature, settlement is found to occur beyond the point of 100 percent of primary, and such continued deformation is termed secondary settlement. Although the magnitude and rate of secondary consolidation is normally considerably less than that of the primary, there are instances where its occurrence may be detrimental to the proposed construction.

Terzaghi's theory of consolidation (1) demonstrates that the time required for any degree of consolidation (primary) to occur is related to the square of the maximum distance that water must travel to reach a zone of zero pore pressure (Fig. 1). The higher the permeability of the soil, the faster the point of 100 percent consolidation and zero pore pressure can be approached. In the case of sand, if pore pressures were to be developed they would dissipate almost instantaneously if the material were coarse, whereas in the case of fine sand some small time lag may be involved. By using essentially coarse sand in the formation of a sand drain, and spacing these columns in the soil to be consolidated, the maximum distance that water must travel to reach these zones of zero pore pressure can be arbitrarily established to meet any time requirement

for the completion of any degree of consolidation under a given applied load. The fact that the water in the soil flows horizontally to the drain is generally beneficial, as the permeability of many compressible soils is considerably greater in the horizontal direction than in the vertical direction. This factor is of even greater significance in soils which are varved, or interspersed with layers of coarse materials which can be intercepted by the sand drains to form better drainage paths for the water leaving the finer-grained soils. Even such relatively homogeneous soils as organic clays may have a horizontal permeability which is 10 times its value in the vertical direction. Thus, not only do sand drains provide a convenient means to arbitrarily establish the time required for a specific degree of consolidation, but they also permit the utilization of the best soil drainage characteristics. Inasmuch as the normal vertical soil drainage occurs at the same time as the sand drains function, consolidation due to sand drains supplements the vertical consolidation effects (Fig. 2).

The most comprehensive theoretical analysis concerning sand drain installations was presented by Barron (2), whose work forms the basis of most modern sand drain practice. Aside from the soil characteristics themselves, the variables involve the diameter of the drain, d_w, and the effective diameter of the zone of drain influence, d_e. The latter factor is in turn related to the grid pattern employed and the drain spacing, S. The larger the drain diameter, the more effective the sand drain grid, so it is possible to investigate the economics of the variation in the total cost of an installation by varying the sand drain spacing as opposed to varying the size of the drain to achieve the same end result.

Aspects Related to Design

The fact that soils increase in strength with consolidation is generally taken for granted; however, this fact is often neglected in the design of sand drain installations (3, p. 185). By taking the increase in soil strength into account, it may be possible to minimize or even eliminate potential stability problems which may ordinarily require the incorporation of counterbalancing berms in the embankment section. Thus, a complete evaluation of the improvement in soil strength with consolidation may be of significant economic importance in the development of a design to reflect the lowest possible cost for construction.

Sand drain designs can be developed with or without the use of an overload to achieve the desired degree of primary consolidation. Overloads are used in conjunction with sand drain stabilization where they are economically justified to (a) eliminate residual primary consolidation as a consideration in post-construction maintenance, (b) minimize the effect of secondary consolidation, and (c) supplement the effect of sand drains in attaining the required degree of consolidation.

Where embankment stability is a major consideration and the extent of counterbalancing berms is to be minimized due to right-of-way cost or availability, and depending on soil conditions, the use of overloading may be eliminated in favor of accepting some increase in roadway maintenance (4).

Sand Drain Diameter

The size of sand drains used in the United States and elsewhere generally ranges between 6 in. and 30 in. in diameter. The most common drain size used is on the order of 18 in. in diameter. Because the cost of the sand backfill for the drain is normally a small part of the total cost, and labor and equipment are essentially independent of the size of the drain installed, the use of 18-in. diameter drains is recommended as a standard. Drains of this size have been found to function in installations where settlements have represented 25 percent of the initial length of the drains. In areas where sand suitable for drain backfill is a major part of the installation cost, an investigation of the economics of using smaller size drains is warranted.

SAND DRAIN INSTALLATION METHODS

Although sand piles have been in use for centuries for the support of construction in marginal terrain, it was not until 1926 that the concept was formally established for

using such columns as drains for the stabilization of a water-laden compressible earth mass (5). The California Division of Highways undertook the first field installations of sand drains, and largely as a result of the work of Porter (6), the original concept was translated into a practical reality as the mandrel method of installation. This method involves the use of a pipe or casing, termed a mandrel, which is driven or otherwise inserted into the soil to support the sand drain cavity. A hinged cap is located at the driving end of the mandrel. During the driving operation, the cap prevents the intrusion of soil which would contaminate the interior of the mandrel. Upon withdrawal of the mandrel, the sand backfill is inserted into the mandrel for passage into the sand drain cavity with the hinged cap in the open position (Fig. 4B). The sand backfill replaces the mandrel as support for the formed cavity, and at the same time provides a highly permeable conduit for the passage of water from the compressible soil. The driving operation in the mandrel method of installation is equivalent to the procedure for driving foundation piling, and the effects on the soil characteristics are similar.

The failure of many of the field installations of sand drains placed by the mandrel method to meet design requirements resulted in the avoidance of its use by many of our more prominent soils engineers. However, because the economic feasibility of projects could depend on the validity of the theory of sand drains, the concept could not be easily rejected. It was reasoned that the failure of mandrel installations was not related to deficiencies in the theory, but rather to the disturbing effects of the method of installa-

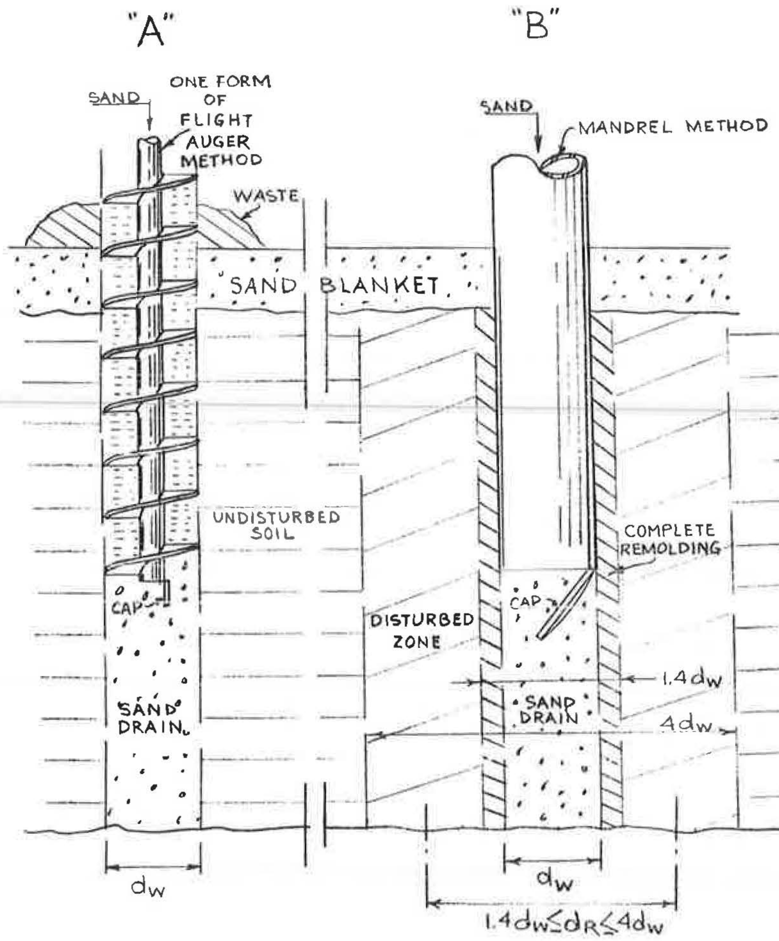


Figure 4.

tion, and the difficulties involved in fully evaluating such effects (3, p. 178 ff). Thus the flight auger method of installation was developed (7). In this method, the flight auger is penetrated into the soil by helically cutting into the earth (Fig. 4A). When the required depth of penetration is reached, the auger is rotated while held in its vertical position, which shears a core of soil from the earth mass. The core is removed, forming the sand drain cavity, which can then be filled with sand to form the required drain. The outer flight diameter forms the dimension of the finished drain. By using a hollow auger shaft or stem, the sand backfill can be placed into the cavity simultaneously with the removal of the core of earth. A movable cap at the base of the auger shaft is used to prevent the contamination of the shaft interior during the insertion of the auger into the soil, and it is moved out of position during the backfill process.

Volumetric displacement in forming the cavity can be minimized or entirely eliminated by controlling the rate of insertion of the auger, thus avoiding disturbance of the sidewalls of the sand drain cavity. The simultaneous backfill procedure through the hollow shaft insures the dimensional continuity of the drain, and avoids the possibility of disturbance that may be associated with the reduction of soil stresses (8). Compared with the mandrel method, the flight auger method can be considered as a non-displacement method which minimizes any disturbance of in situ soil characteristics.

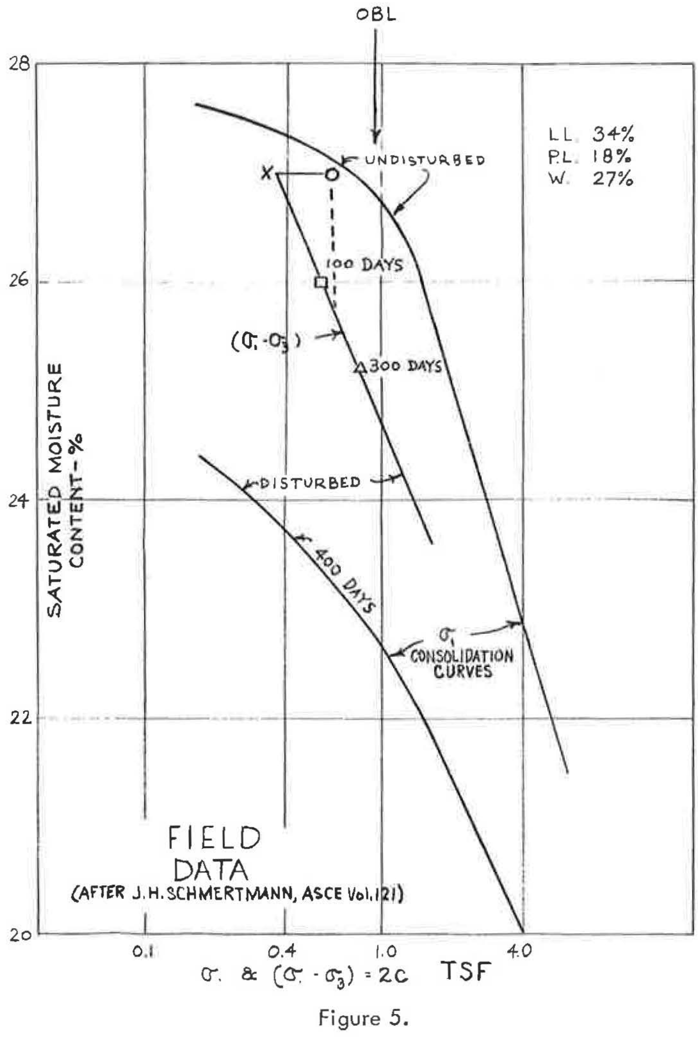


Figure 5.

In addition, due to its rotary cutting procedure, should the subsoil be varved or otherwise contain seams of coarse-grained materials, the flight auger method maximizes the utilization of this favorable soil drainage condition, compared with a vertical cutting process which would more readily distort the stratification.

Effects of the Mandrel on Soil Characteristics

The effects of disturbance associated with the use of a driven mandrel are equivalent to those associated with driving piles into compressible soils. These effects include (a) reduction in soil strength, (b) increase in soil settlement, and (c) decrease in soil permeability. Although all of these effects may not occur in nonsensitive soils, some of these effects are expected to occur in all compressible soils, and the greater its sensitivity the more marked the disturbance effects expected.

The effect of pile driving on soil strength is adequately substantiated in the literature (9, 10, 11). These reported observations include data indicating a simultaneous reduction in soil volume due to a reduction in moisture content, resulting in unanticipated settlements. In a number of instances, the resultant increase in soil compressibility resulted in excessive settlements which resulted in the abandonment of the foundation or extensive reconstruction (11, 12). A typical example of this is shown in Figure 5, which is based on the work of Schmertmann (9) in connection with a glacial clay having a sensitivity of 2.5, with liquid and plastic limits of 34 and 18 percent, respectively. The effects of pile driving resulted in the reduction in shear strength (cohesion) from 0.29 tsf to 0.19 tsf at a constant moisture content of 27 percent. The minimum value of strength for the soil at its initial moisture content was established as 0.3 tsf from the result of laboratory tests on remolded samples. It took 3 months for the clay to recover its initial strength by draining without further loading, which occurred at a moisture content of about 23.5 percent. Thus, the soil consolidated slightly more than 5 percent (sp. g. assumed as 2.7), which is greater than any expected heave of soil due to the volume of piling driven, or compensation due to any reduction in the compression index due to remolding. For a 50-ft thick clay stratum, assuming a net settlement due to disturbance of only 2.5 percent (allowing for heave and reduction in C_c), the increase in total settlement due to pile (or mandrel) installation would be on the order of 15 in. This could be a substantial increment in settlement for a normal embankment, which would have the effect of increasing the cost of construction for a sand drain stabilized area. Coupled with the effects on design of the loss of strength, which may result in a reduction in the rate of filling or the need for counterbalancing berms, the remolding of the subsoil and related disturbance that can be expected to result from the use of sand drains in soils of even moderate sensitivity may substantially affect the cost and progress of construction.

Some of the effects of driving a mandrel in compressible soil are shown in Figure 4B. It can be shown mathematically that the volume of soil displaced by a mandrel having a diameter d_w is equivalent to the volume of soil exterior to the formed cavity having an outer diameter of 1.4 d_w. This volume, having been completely displaced in the formation of the sand drain cavity, represents the zone of complete remolding, and its characteristics can be readily established by laboratory test. However, volumetric displacement does not end abruptly at 1.4 d_w, and its resultant disturbance effects extend to at least 2 d_w and possibly as far as 4 d_w for even moderately sensitive soils (13). Furthermore, pore pressure increases are not limited to the remolded zone around the cavity formed by a mandrel (15), and may occur as much as 50 ft away from the point of driving.

Laboratory consolidation and permeability data published in connection with undisturbed and remolded soil samples (3, p. 94ff) indicate that disturbance induces an increase in total settlement and considerably reduces the rate at which it will occur. Figure 6 compares the pressure void ratio characteristics for the undisturbed and remolded cases of a soil with liquid and plastic limits of 49.5 and 30.5 percent, respectively, and a natural moisture content of about 52 percent. The pressure-void ratio curves demonstrate and confirm the field findings that soil disturbance will result in soil consolidation in excess of what would be obtained for the undisturbed case at any

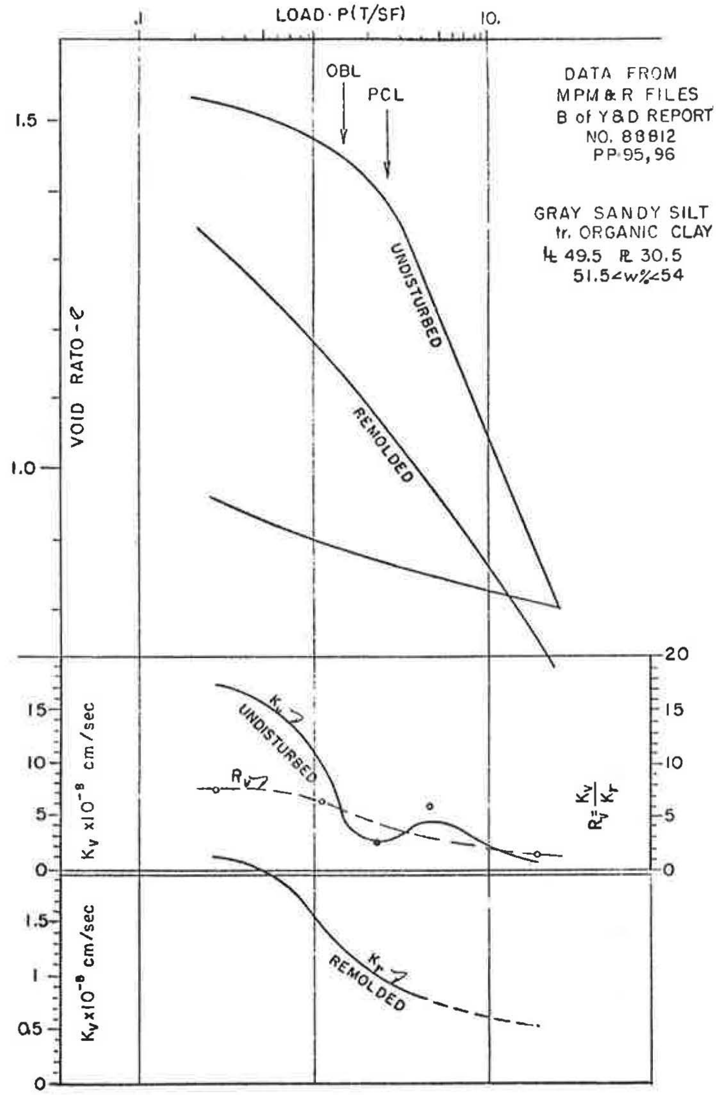


Figure 6.

given load application. The figure also indicates a considerable reduction in the coefficient of permeability, k, as a result of disturbance even at loads above the preconsolidation load. It may therefore be inferred that although the reduction in soil permeability due to disturbance is large for preconsolidated soils, due to the effects of disturbance the reduction for normally consolidated soils may also be substantial. The permeability ratio, R (ratio of horizontal or vertical permeability to the remolded value, whichever is specified), for the vertical case is on the order of 5.0 for the in situ condition of the soil in Figure 6. Because the value of horizontal permeability of a clay soil is normally greater than the vertical value, even in homogeneous clays, the value of R for the horizontal case may be considerably larger than 5.0.

Recognizing that soil characteristics vary, there may be instances where the disturbance effects due to the driving of a mandrel may not result in excessively reducing the value of soil permeability. However, the results of laboratory tests alone may not be completely indicative of related effects of disturbance due to difficulties involved in

obtaining and handling laboratory test specimens from which "undisturbed" characteristics are established (14). The normal disturbance to soil samples inherent in sampling and testing techniques may erroneously minimize the spread between true in situ characteristics, particularly with regard to determining the ratio of horizontal permeability to its remolded value. Nevertheless, the disturbance effects due to the mandrel method must be evaluated to establish any increase in construction cost due to this method of sand drain installation compared to the flight auger or other methods which eliminate or minimize disturbance.

Laboratory Comparison of Auger and Mandrel Installations

Field observations by the writer indicate that cavities formed by means of a controlled flight auger will not distort varves that may exist in the soil, and that the vertical movement of clay or silt contained within the auger flights does not materially smear the more granular soil strata by virtue of such movement. Although the reason for this may not be immediately obvious, it was nevertheless found to be the case in a number of instances during the field installation of sand drains when such observations were possible.

A laboratory model was developed to compare the results of forming a cavity in varved soil by means of a controlled flight auger as compared to the use of a mandrel. The results are shown in Figure 7 (gray silt, some clay with varves of gray silt and clay; liquid limit 60%, plastic limit 26%, moisture 60%).

Test No. 1 was performed using a $\frac{5}{8}$-in. auger, and Test No. 2 was performed using a $\frac{1}{2}$-in. rod, representing a mandrel. The original height of each specimen was the same. In Test No. 1, the auger advance was controlled at less than 1 pitch (distance between flights along the shaft) length per revolution. The development of a clean cavity with essentially no distortion of the varves is evident. The size of the cavity formed was essentially $\frac{5}{8}$-in. in diameter, conforming to the size of auger used. The upper portion of the sample was easily separated from the lower portion at a granular parting, and it was clearly evident that the horizontal drainage path was in no way obstructed. No remolding of soil forming the cavity was visible either at the sidewalls or at the level of maximum auger penetration. In contrast to this, Test No. 2, which was performed with the equivalent of a mandrel, shows considerable disturbance extending for a distance equivalent to 3 d_W. Complete remolding of the soil adjacent to the cavity was noted, and it was not possible to separate the sample at any granular parting. Disturbance of the soil at the level of maximum penetration is clearly shown in the figure. An equally important observation is the fact that the final cavity dimension was $\frac{17}{32}$ in. at the top, $\frac{13}{32}$ in. at the middle, and $\frac{14}{32}$ in. at the bottom. This represents an average reduction in drain diameter of about 8 percent. This occurrence

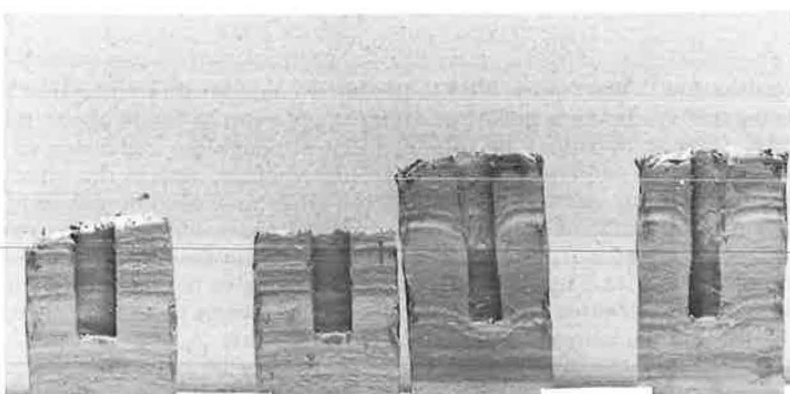


Figure 7.

in the field would theoretically reduce the effectiveness of the installation by at least a 15 percent increase in time, which does not include the other effects of disturbance due to using a mandrel (15).

WHITESTONE EXPRESSWAY OBSERVATIONS

The general soils profile for the Whitestone Expressway project, Section 61-1, is shown in Figure 8, on which are superimposed the locations of settlement platforms and the areas where sand drains were installed by the hollow shaft flight auger method. Typical profiles of subsoil characteristics before the start and after the completion of construction are shown in Figures 9 and 14. Typical settlement observations are shown in Figures 10 through 13, with Figure 13 representing an area where a slide failure had occurred prior to the installation of sand drains while the contractor was excavating to install a sewer parallel to the roadway.

The areas with sand drains installed 10 ft on centers show a characteristic primary consolidation break (1) within 20 to 30 days after the effective start of loading, after which the settlement curve exhibits a linear relationship with the logarithm of time (Fig. 10). The settlement curves for the areas where an 8-ft square grid was used show a similar break within 9 to 12 days after the effective start of loading. These observations indicate a value of 2 to 3 ft^2/day for the coefficient of horizontal consolidation for the organic clay. If the ratio of consolidation coefficient is taken as equal to the ratio of permeability coefficients, then the horizontal permeability ratio, R_h, is on the order of 40. Laboratory testing could establish a value of 10.0 for R_h (boring 4-24, test T-14), which is less than the field results indicate.

This difference is undoubtedly due to the sensitivity of the soil, which is about 7.0, and the disturbance inherent in sampling and testing procedures. An order of magnitude of 40 for R_h can be seen as a possibility by reviewing the permeability data in Figure 6. Here, the vertical value, R_v, is about 5.0; however, organic clays may have a ratio of horizontal to vertical permeability of as much as 10, which would then indicate a factor of as much as 50.0 for R_h. On this basis, it is possible to conclude that the hollow shaft flight auger method of installation of sand drains has little or no soil disturbance effect on the sand drain cavity formed or on the soil beyond the cavity, when used in soils similar to that in the Flushing Meadows area where the organic material has a sensitivity of about 7. By the same token, there is every reason to expect that the hollow shaft flight auger method of installation can be applied to all other clay and silt soils with equally good results.

For the partially disturbed area (Fig. 13), the permeability ratio R_h is found to be on the order of 5.0, based on a comparison of the Whitestone field data with that obtained for the mandrel installation for the Long Island Expressway work.

The settlement platforms for the Whitestone project were removed when the overload was excavated, and new points added. Where possible these new readings, taken along the curb of the finished roadway, were added to the last available (or extrapolated) platform readings to establish a continuous curve. The average coefficients of secondary compression, expressed per unit of soil thickness per logarithmic cycle of time, are (after overload removal):

Grid Spacing (ft)	Peat (% of total)	Coeff. of Sec. Comp.
8	50	0.031
10	10	0.008
10	10	0.029[a]

[a]Area of slide failure prior to sand drain installation.

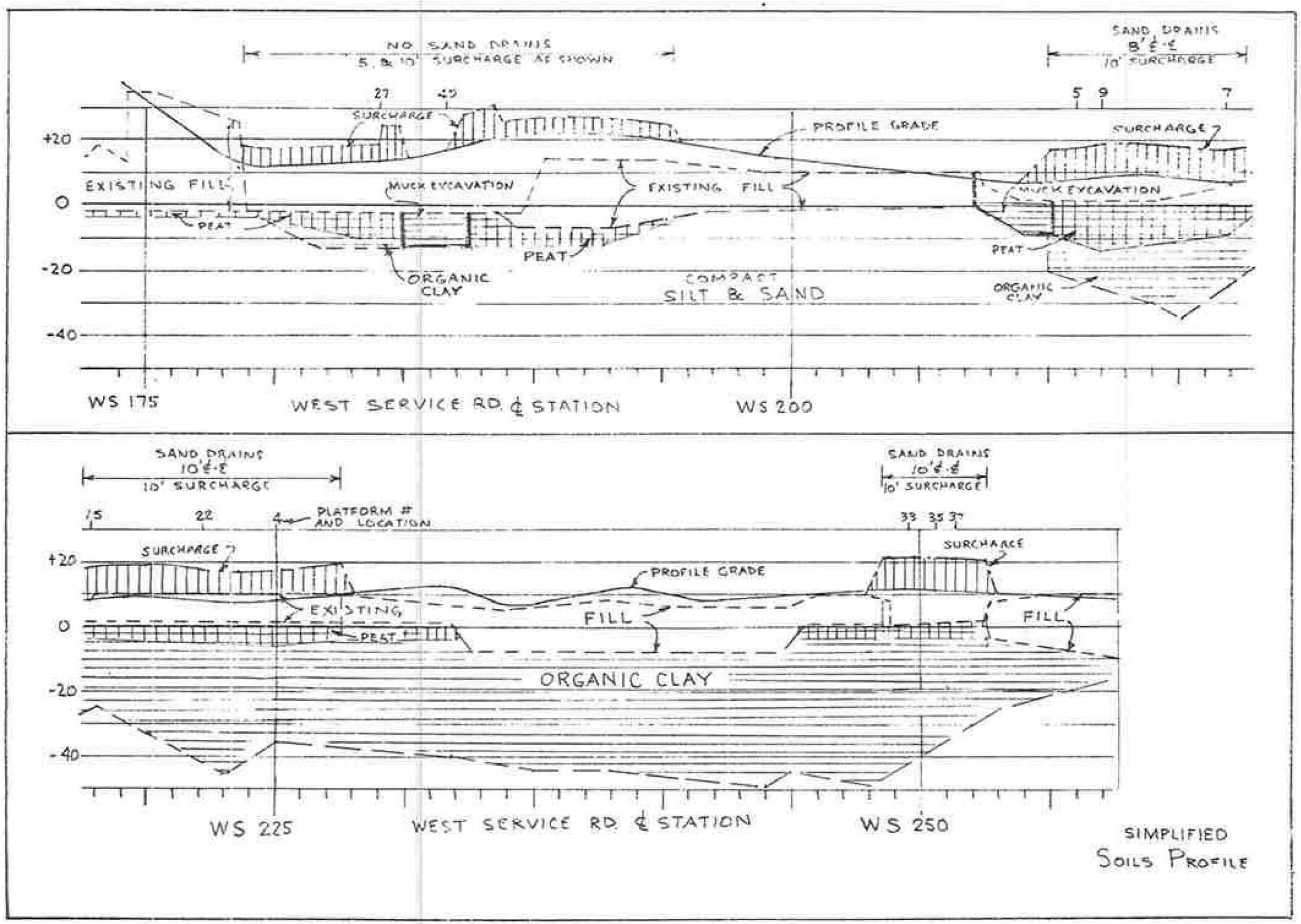
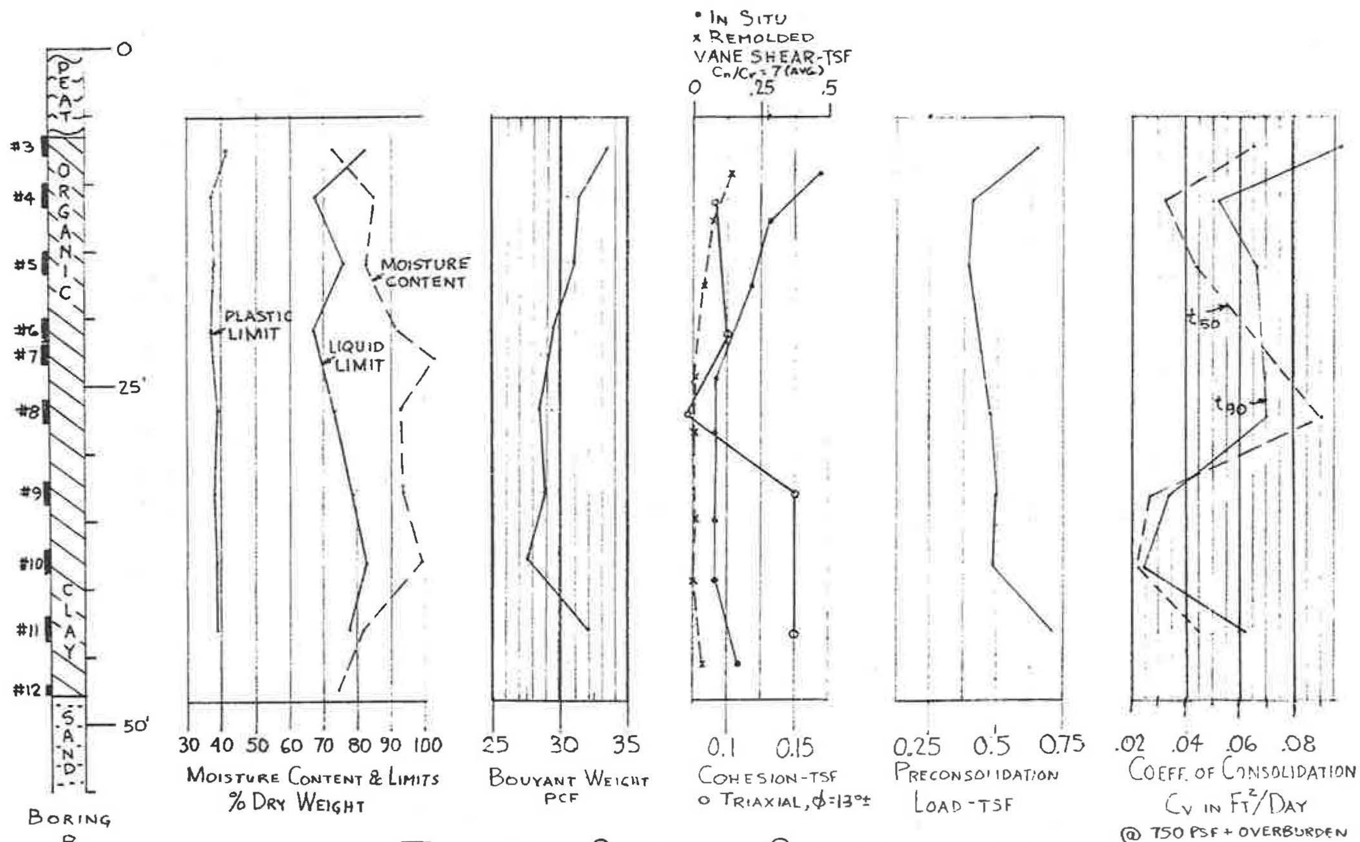

NO SAND DRAINS
5 & 10' SURCHARGE AS SHOWN
SAND DRAINS
8' ¢-¢
10' SURCHARGE
27
45
5
9
7
SURCHARGE
PROFILE GRADE
SURCHARGE
+20
EXISTING FILL
MUCK EXCAVATION
EXISTING FILL
MUCK EXCAVATION
0
PEAT
PEAT
PEAT
-20
ORGANIC CLAY
COMPACT SILT & SAND
ORGANIC CLAY
-40
WS 175
WEST SERVICE RD ¢ STATION
WS 200
SAND DRAINS
10' ¢-¢
10' SURCHARGE
SAND DRAINS
10' ¢-¢
10' SURCHARGE
15
22
4
PLATFORM # AND LOCATION
33
35
37
SURCHARGE
PROFILE GRADE
SURCHARGE
+20
EXISTING
FILL
FILL
0
PEAT
ORGANIC CLAY
-20
-40
WS 225
WEST SERVICE RD ¢ STATION
WS 250
SIMPLIFIED
SOILS PROFILE

Figure 8.



Typical Subsoil Characteristics

OBTAINED PRIOR TO CONSTRUCTION
STA. WS 222+65±, 30±Lt.

Figure 9.

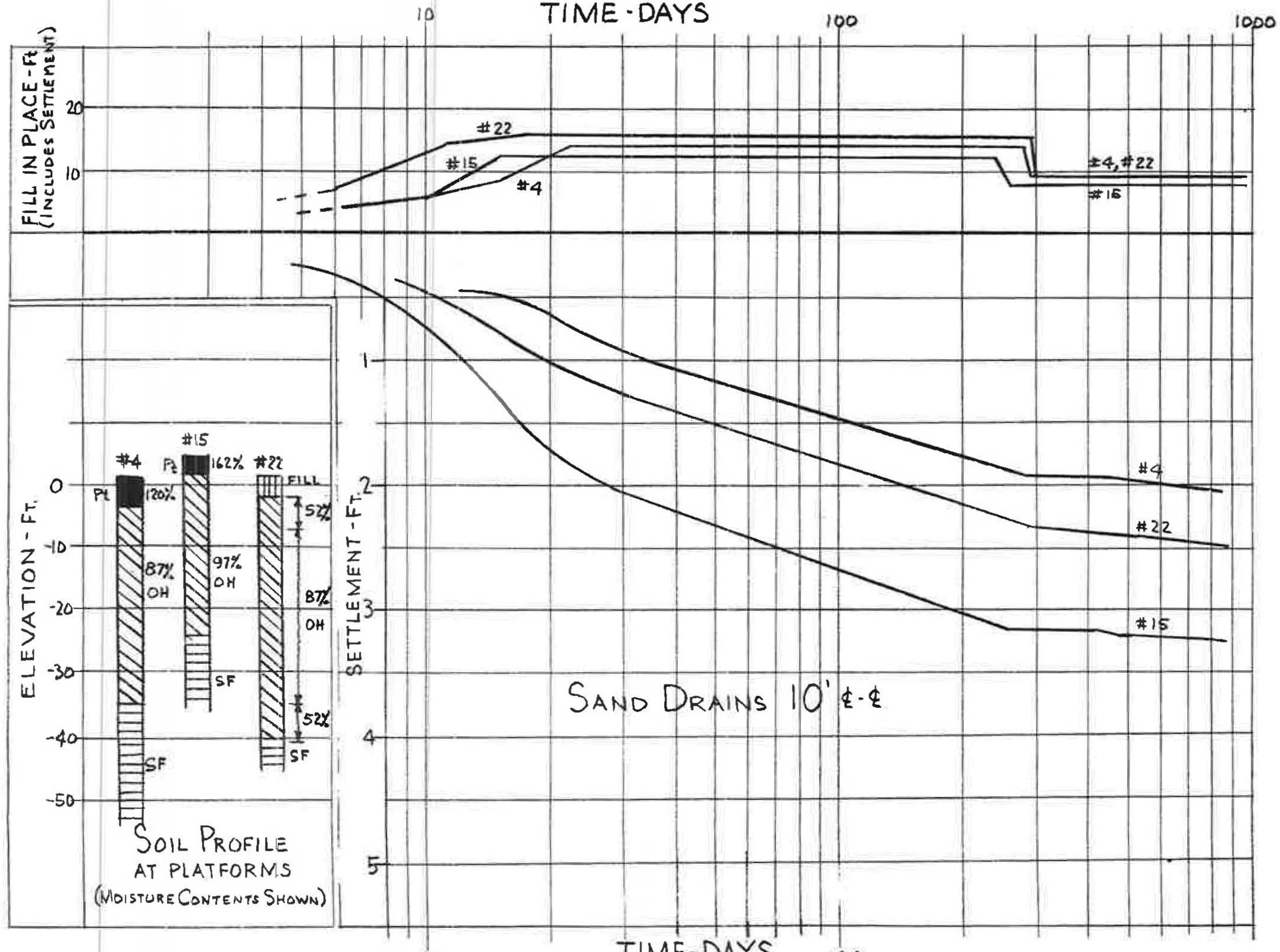
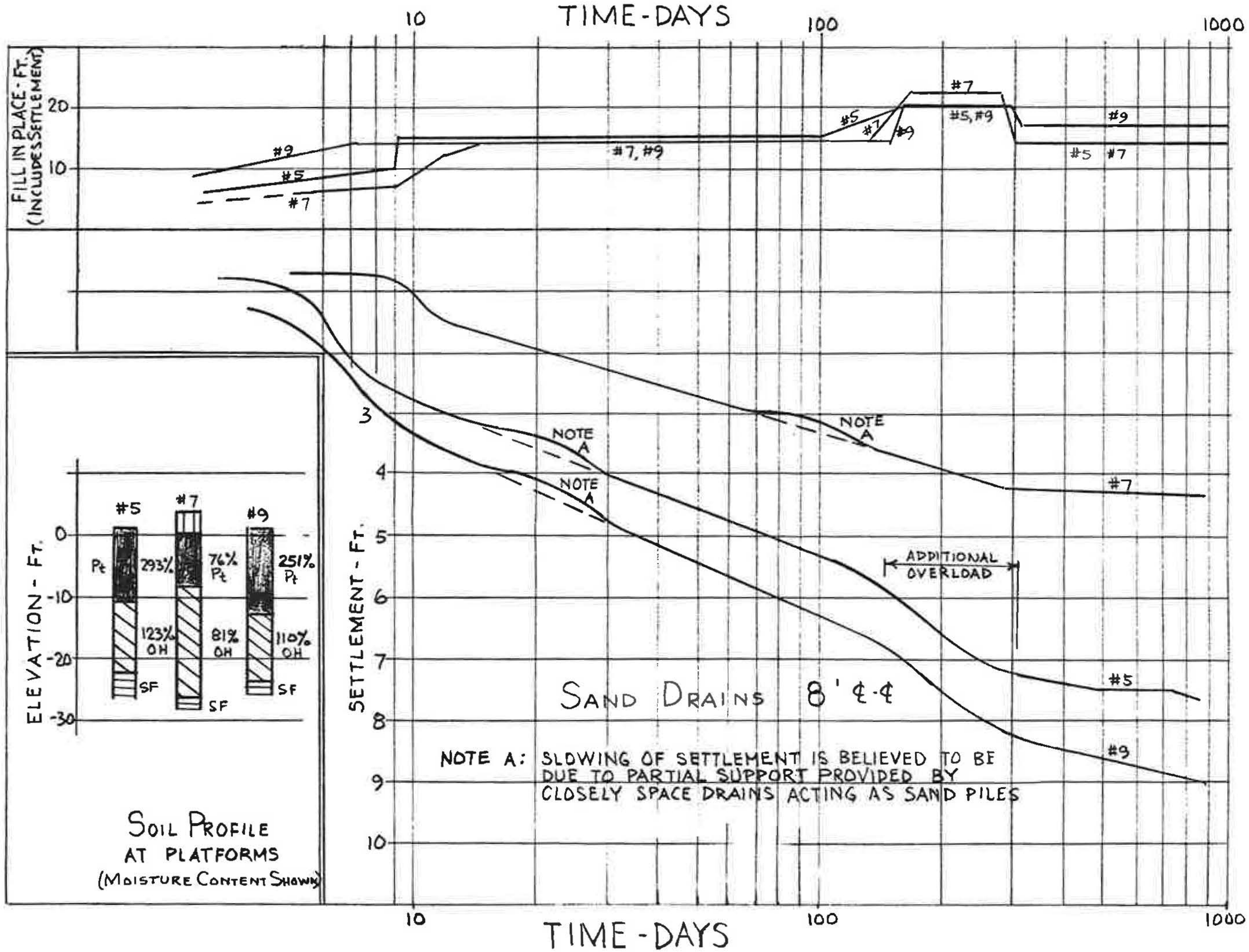

TIME - DAYS
10
100
1000
FILL IN PLACE - Ft
(INCLUDES SETTLEMENT)
20
10
#22
#15
#4
#4, #22
#15
SETTLEMENT - FT.
1
2
3
4
5
#4
#22
#15
SAND DRAINS 10' ¢-¢
ELEVATION - FT.
0
-10
-20
-30
-40
-50
#4
#15
#22
Pt
120%
162%
FILL
52%
87%
OH
97%
OH
87%
OH
SF
SF
52%
SF
SOIL PROFILE
AT PLATFORMS
(MOISTURE CONTENTS SHOWN)
TIME-DAYS
10
100

Figure 10.



TIME-DAYS
10
100
1000
FILL IN PLACE - FT.
(INCLUDES SETTLEMENT)
20
10
#9
#5
#7
#7, #9
#5, #9
#5 #7
SETTLEMENT - FT.
3
4
5
6
7
8
9
10
NOTE A
ADDITIONAL OVERLOAD
SAND DRAINS 8' ¢-¢
NOTE A: SLOWING OF SETTLEMENT IS BELIEVED TO BE DUE TO PARTIAL SUPPORT PROVIDED BY CLOSELY SPACE DRAINS ACTING AS SAND PILES
ELEVATION - FT.
0
-10
-20
-30
Pt
293%
76% Pt
251% Pt
123% OH
81% OH
110% OH
SF
SOIL PROFILE AT PLATFORMS
(MOISTURE CONTENT SHOWN)

Figure 11.

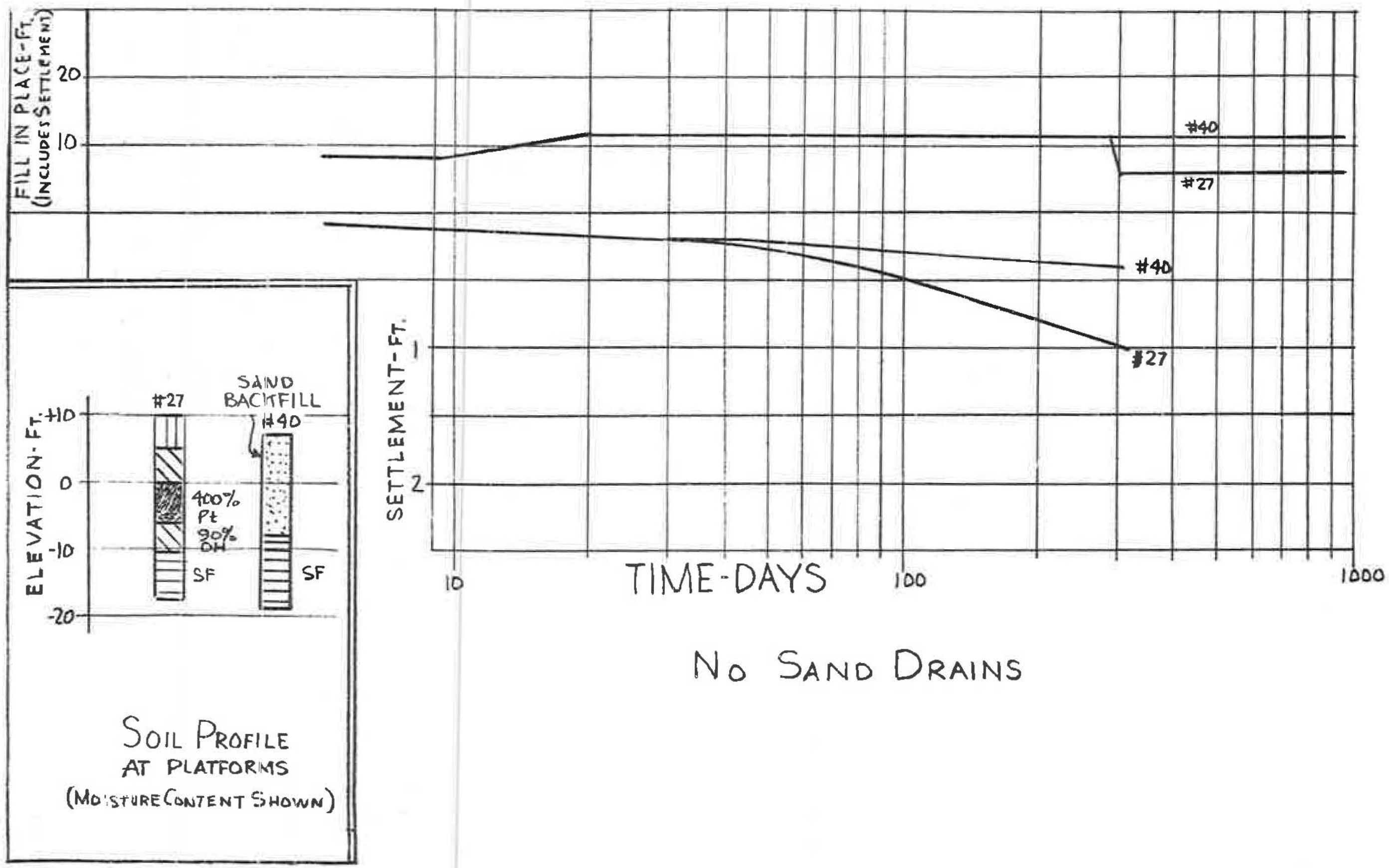

FILL IN PLACE-FT.
(INCLUDES SETTLEMENT)
20
10
#40
#27
#40
#27
SETTLEMENT-FT.
1
2
10
TIME-DAYS
100
1000
NO SAND DRAINS
ELEVATION-FT.
+10
0
-10
-20
#27
SAND
BACKFILL
#40
400%
Pt
90%
OH
SF
SF
SOIL PROFILE
AT PLATFORMS
(MOISTURE CONTENT SHOWN)

Figure 12.

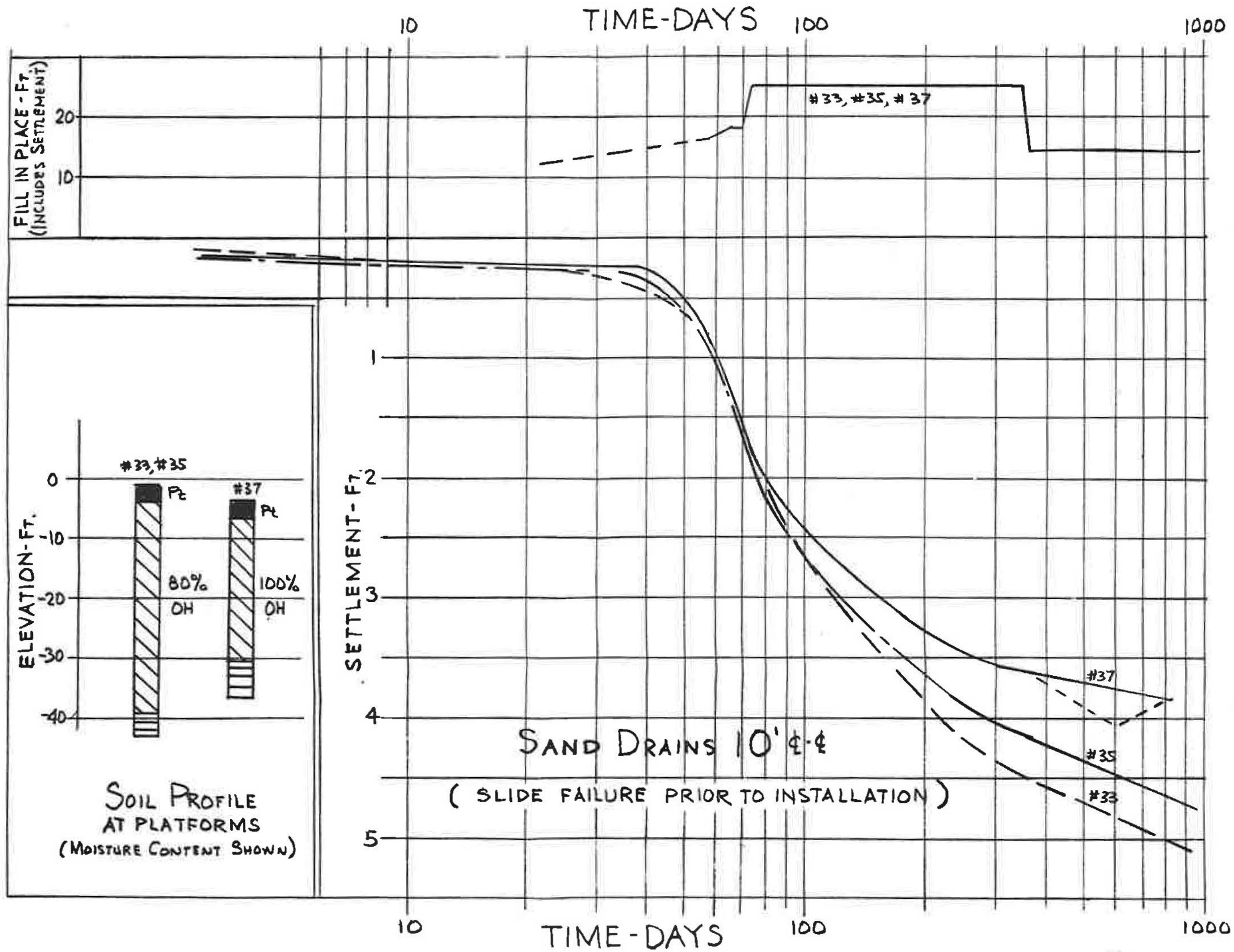
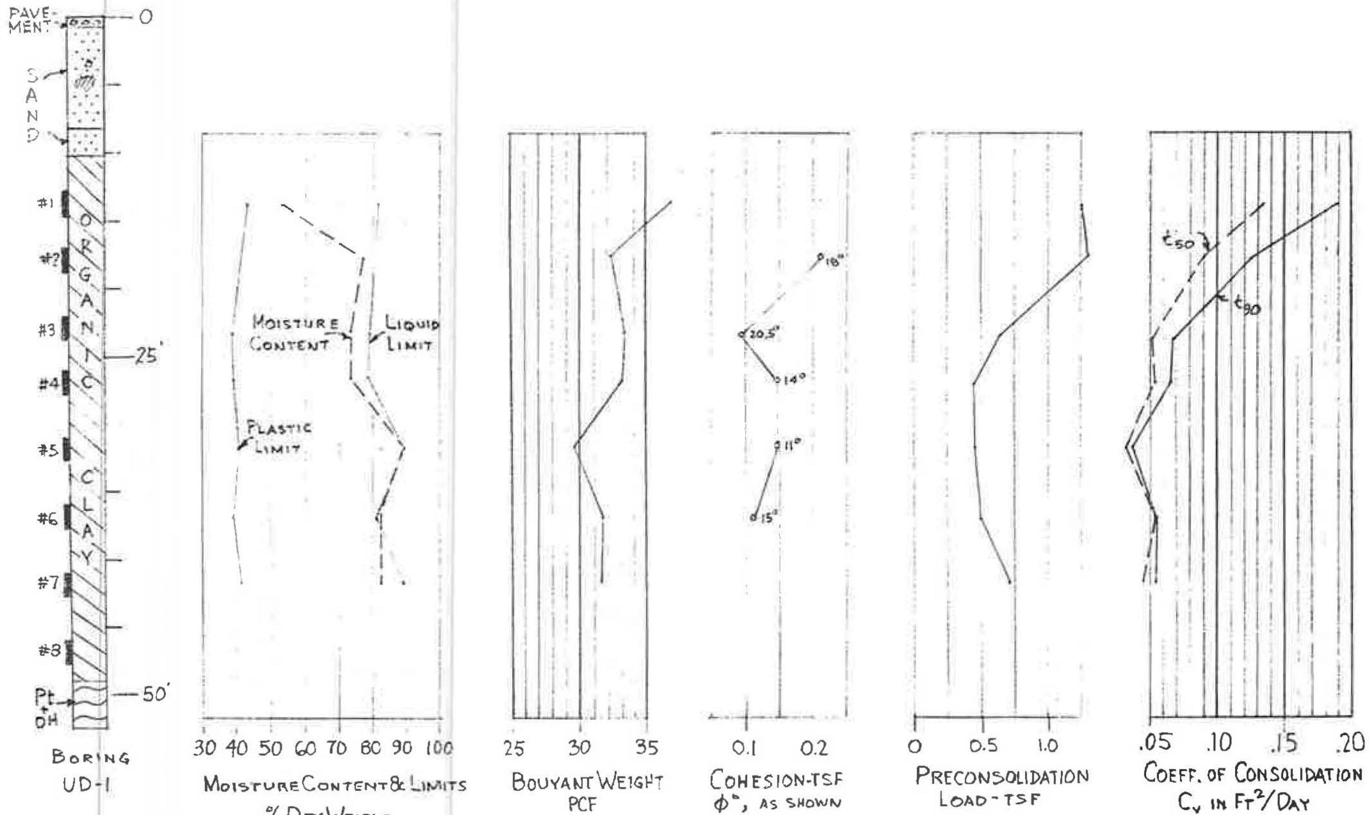

TIME-DAYS
10
100
1000
FILL IN PLACE - FT.
(INCLUDES SETTLEMENT)
20
10
#33, #35, #37
SETTLEMENT - FT.
1
2
3
4
5
#37
#35
#33
SAND DRAINS 10' ¢-¢
(SLIDE FAILURE PRIOR TO INSTALLATION)
ELEVATION - FT.
0
-10
-20
-30
-40
#33, #35
#37
Pt
80%
OH
100%
OH
SOIL PROFILE
AT PLATFORMS
(MOISTURE CONTENT SHOWN)

Figure 13.



TYPICAL SUBSOIL CHARACTERISTICS
AFTER CONSTRUCTION
STA. WS 222+09 ℄

Figure 14.

The latest post-construction settlement readings are given in Table 1. Despite the occurrence of a slide failure which disturbed the subsoil in one area prior to the installation of sand drains, its coefficient of secondary compression is less than half the value reported for the mandrel installation in similar material for the Long Island Expressway project. The coefficient for the undisturbed organic clay soil, 0.008, is consistent with laboratory results, and on the order of 10 to 15 percent of values reported for the mandrel installations. Where peat represents about 50 percent of the soil, the

TABLE 1

SUMMARY OF POST CONSTRUCTION SETTLEMENT

STATION*	AVG. THICKNESS, Ft. Pt	OH	SETTLEMENT, FT. 7/-/63 TO 3/22/65 600± DAYS	PLATFORM	AVG. SETTLEMENT, FT. 7/-/63 TO 3/22/65 600± DAYS	REMARKS
WS 211			.43	#5		
212			.80	#9		
213			.69			SAND DRAINS
214	12'	17'	.69		.5'	8' ℄-℄
215			.39			
216			.26			
217			.16	#7		
WS 218			.13	#15		
219			.09			
220			.06			
221			.25			
222	5'	25'	.16	#22	.2'	SAND DRAINS
223			.39			10' ℄-℄
224			.33			
225			.19	#4		
226			.14			
227			.11			
WS 228			.16			
229			.21			
230	4'	30'	.23		.3'	NO SAND DRAINS
231			.28			
232			.44			
WS 245			.30			
246	4'	40'	.44		.5'	NO SAND DRAINS
247			.46			
248			.84			
WS 249			1.06			AREA DISTURBED
250	5'	19'	.68	#33,35	.6'	PRIOR TO INSTALLING
251			.21			SAND DRAINS 10' ℄-℄
252			.33	#37		
253			—			

* MARKERS AT CURB LINE

secondary coefficient was expected to be higher than the results reported for the organic clay, due to the soil type involved, and a secondary coefficient of 0.031 cannot be considered unusual. This value is nevertheless much smaller than that reported for the mandrel installations in organic clay, where little or no peat was involved. Thus, it may be inferred that the use of the auger method of sand drain installation will minimize the coefficient of secondary compression, and minimize post-construction settlements.

Grid Efficiency

The effectiveness of any sand drain installation may be established by comparing the number of sand drains required to stabilize a given area in a given amount of time using a method which avoids soil disturbance with the number of drains of equal diameter meeting the design requirements taking into account the disturbance effects of the second method considered. This comparison, expressed as a percentage, is termed "grid efficiency," E, and its reciprocal value is termed "grid factor," F.

If R_h for a soil is 15, and a 15-ft square grid spacing is established for a proposed auger installation, then the efficiency and grid spacing can be obtained utilizing Fig-

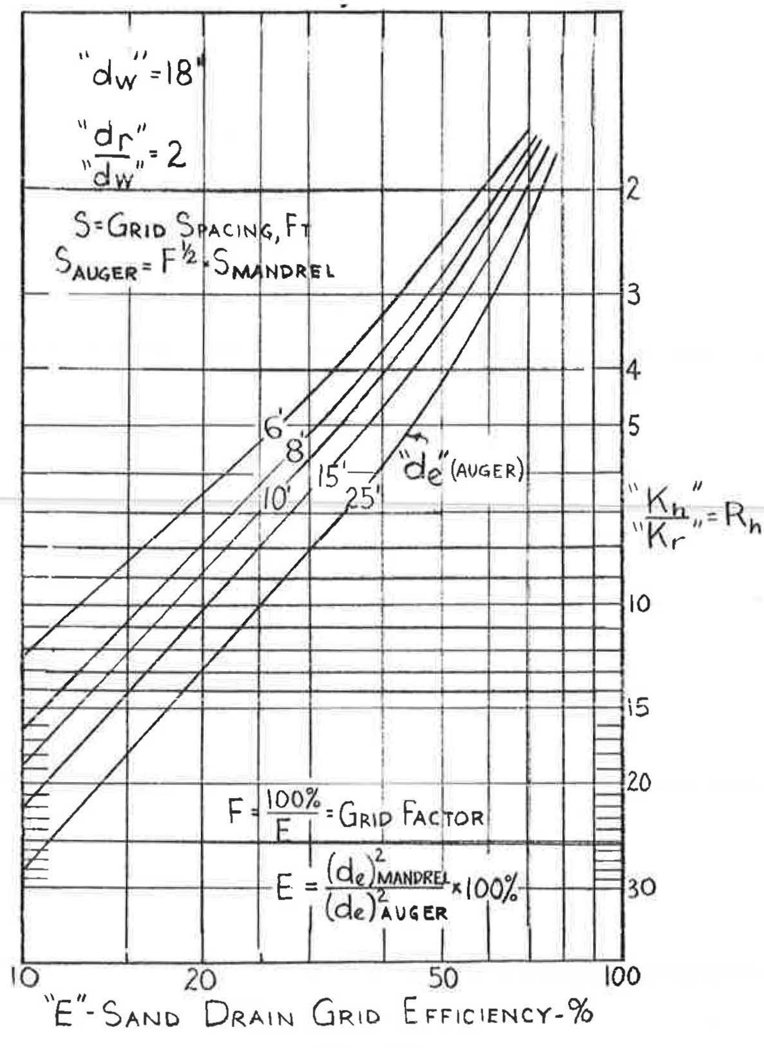


Figure 15.

ure 15, relating R_h with grid efficiency, using d_e as a parameter, for 18-in. drains. The curves are based on sand drain theory (2) for the case where the effective diameter of the mandrel remolded zone does not exceed 2 d_w. The value of d_e (for a square grid 15 ft on centers) is 17 ft. By entering Figure 15 at the R_h value of 15, and proceeding to the interpolated curve for d_e equal to 17, E is established as 15 percent and F is 6.6. Therefore a mandrel installation with drains spaced at about 6 ft on centers would be required to be equivalent to an auger installation with drains 15 ft on centers, for the above conditions. In this case, for every dollar spent on the mandrel installation, six dollars could be afforded to attain an equivalent design by means of the hollow shaft flight auger method. Additional savings would accrue to the auger installation in minimizing primary and secondary settlements, as well as maximizing soil strength by virtue of soil consolidation without soil disturbance.

Based on the results of the Whitestone Expressway project compared to the results obtained in connection with the mandrel-installed sand drain areas for roadwork related to the Long Island Expressway construction, the hollow shaft flight auger method of sand drain installation appears to be as efficient a means for stabilizing soils, without inducing soil disturbance, as can be ideally obtained. The results of the auger installation, being 40 times more efficient than the mandrel installation for the work in the Flushing Meadows soils, performed approximately 4 times better than the best available laboratory test data would indicate. The latter undoubtedly results from the reduction in permeability characteristics attributable to disturbance effects resulting from present-day sampling and testing techniques.

CONCLUSIONS

The theoretical basis for sand drain design (2) is completely adequate to permit the evaluation of various methods of sand drain installations. By developing comparative cost estimates for equally effective sand drain stabilization grids installed by the mandrel and hollow shaft auger methods, the most economical means of soil treatment can be established. Cost factors that need to be included as part of the cost of the mandrel grid installation are:

1. Settlement increase due to soil disturbance;
2. The increase in the total number of sand drains required to achieve the desired result in the time allowed for stabilization;
3. The increase in dimension of berms, overload (a decrease in permissible embankment height would otherwise result), or other stabilization aids due to the disturbance effects of the mandrel; and
4. Increase in post-construction maintenance due to the increase in secondary compression factors as a result of soil disturbance.

There is every indication from the field results of the sand drain areas of the Whitestone Expressway project that the hollow shaft flight auger method of sand drain installation minimizes soil disturbance so as to produce a highly efficient means for stabilizing soils in accordance with the application of sand drain design theory without the necessity of compensating for any disturbance. In this fashion, sand drain stabilization can be used as an effective engineering tool.

ACKNOWLEDGMENTS

The author wishes to acknowledge the assistance and encouragement received from the New York State Department of Public Works, its Babylon District as well as the Bureau of Soil Mechanics, without which this paper could not have been developed. The author also wishes to thank the firms of Preager-Kavanaugh and Tully and DiNapoli, Inc., the engineer and contractor, respectively, for the Whitestone Expressway Section 61-1, for their confidence and assistance in connection with the implementation of the auger method of stabilization.

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Appendix

ABSTRACTED FROM NEW YORK STATE DEPARTMENT OF PUBLIC WORKS SPECIFICATIONS FOR FIWE
(Whitestone Expressway) Section 61-1

Continuous Hollow Shaft Auger Method of Installation

(a) The rate of advance of the auger shall not be greater than one pitch length per revolution and shall be adjusted so that the volume removed by the helix at any depth is equal to the volume displaced by the hollow shaft at that depth. Suitable means shall be provided such that soil does not enter the bottom of the shaft during the advancement of the auger.

(b) At the end of the auger advance for each sand drain, the auger shall be held stationary and rotated at least one revolution. The sand shall then be placed in the hollow shaft, and the auger withdrawn in such a manner that a continuous column of sand, having a minimum diameter of 18 inches, extends from the bottom of the drain to the surface of the ground.

(c) The rate of auger withdrawal shall be regulated so that the sides of the hole are supported at all times, either by the sand backfill or the soil in the auger helix. The auger shall not be rotated upon extraction without permission of the Engineer.

(d) The outside diameter of the hollow shaft shall not be greater than sixty percent of the outside diameter of the helix. The auger shall be of constant diameter and shall be straight to within one inch of axial deviation when suspended in a vertical position.

Notes: The remainder of the specification can be written to suit the needs of the specific project and local working conditions (9).