# Sand Drains for Embankment on Marl Foundation

GEORGE W. McALPIN, Jr., Director, and M.N. SINACORI, Associate Soils Engineer
Bureau of Soil Mechanics, New York State Department of Public Works

This paper describes the engineering details involved in the establishment of an embankment on a soft foundation consisting of 60 ft. of marl and silt.

After establishing the soil profile by field explorations, undisturbed soil samples were tested to determine the strength and consolidation characteristics of the foundation materials. Analyses were then made of various design treatments, consideration being given to the availability of materials, the rights-of-way requirements, the possible construction sequence and required time of completion, and the overall costs. The use of sand drains was found to be most advantageous.

A description is given of the preconstruction consideration involving the design layout of the drains (spacing and depth); the diameter of drains; the necessary construction control devices, such as pore-pressure devices, settlement plates and alignment stakes; rate of construction and surcharge. Comparison is made between the anticipated and the actual behavior of the embankment and its foundation during construction.

The data obtained from field measurements are presented and are compared to those obtained from preconstruction analyses. Comments are made concerning the effectiveness of sand drains at this installation and a note made of the general application of this procedure of treatment.

● THE project described by this paper involved the placement of an embankment on a soft marl foundation, as required to carry the New York State Thruway over an intersecting road and the outlet of Onondaga Lake. The paper develops two main topics: (1) the explorations, laboratory testing and design considerations required to evaluate the major variables involved and (2) the observations and measurements made during and after construction with a comparison of these data to the actions predicted from design computations.

## PROJECT DESCRIPTION

The site of this project is a city park in the vicinity of Syracuse, New York, just north of Onondaga Lake. This lake is the remnant of a more extensive shallow glacial lake. The terrain is flat and approximately 3½ ft. above adjacent water levels.

The geometry of the proposed construction required a minimum clearance of 19 ft. over the lake outlet, and 14½ ft. over the parkway road. These requirements dictated a grade line shown in Figure 1, with a maximum height of embankment above original ground of 22 ft. The cross section provided for two lanes of pavement in each direction, with a dividing mall. The embankment had a top width of 117 ft., and an approximate bottom width of 205 ft. The soil profile consisted of a maximum of 70 ft. of soft material underlain by firm material.

## FIELD EXPLORATIONS

Field explorations were made throughout the entire foundation area to define the soil profile, using regular 2½-in. cased holes and taking standard spoon samples at definite intervals within each hole. A total of 24 such holes were taken. Undisturbed samples, using 3½-in. Shelby tubes, were obtained of the soft materials in preparation for laboratory testing.

In addition to the regular holes required for developing the profile and preparing the design, several undisturbed sample drill holes were placed five months after the completion of construction, and three additional 2½-in. cased holes were placed some 20 months after the completion of construction to aid in rechecking the characteristics of the foundation materials.

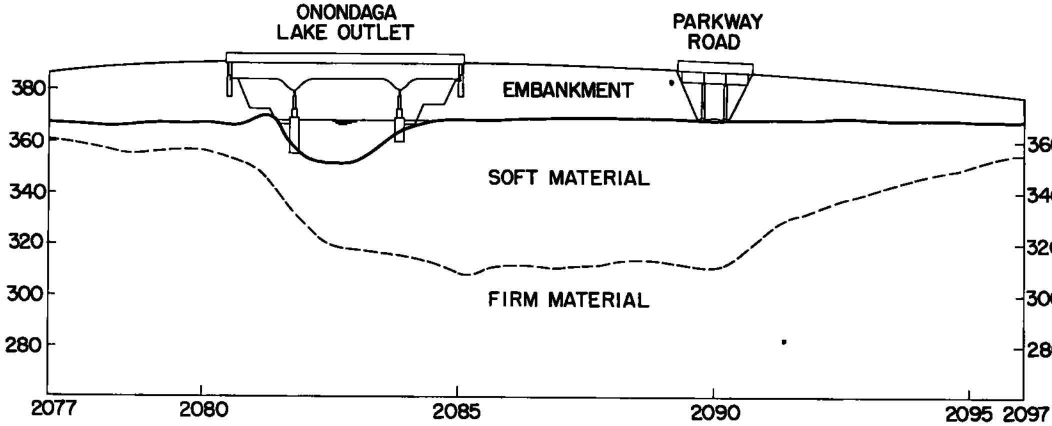


Figure 1. Problem requirements.

## SOIL PROFILE

The soil profile developed shows an upper layer of marl which, with depth, gradually becomes mixed with silt and clay. Below the marl, and silt and clay, are layers of mixtures of sand, silt and gravel, all overlying glacial till. East of the outlet, the soft compressible material extends to a maximum depth of 70 ft. This consists of an upper layer 3 to 6 ft. thick of light-brown marl which contains many coarse shells and appears granular; 15 to 25 ft. of white soft and plastic marl; a transition zone 10 to 15 ft. thick of marl, which increases in silt and clay content; and a lower layer, approximately 15 to 20 ft. thick, of soft silt and clay with minor mixtures of marl.

Within the area west of the lake outlet, the marl depth is rather shallow and eventually runs out. East of Station 2100, the marl layer again becomes quite shallow. Figure 2 shows the location of all drill holes, the average profile, and the approximate depth of soft material.

## FOUNDATION CHARACTERISTICS

Marl is composed of calcium carbonate mixed with a variable proportion of impurities.

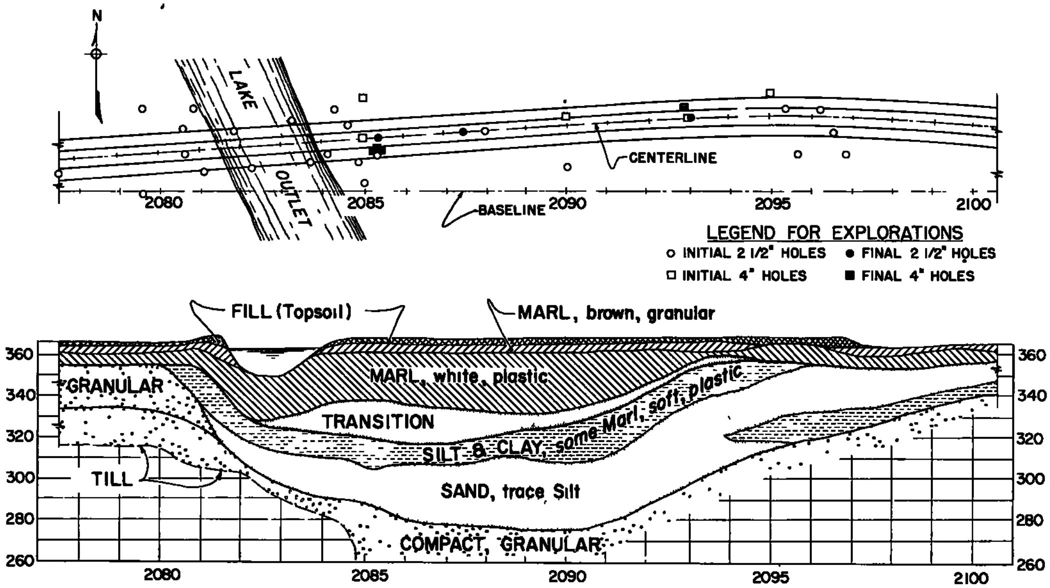


Figure 2. Plan and profile of line.

To determine the proportion of impurities, dilute hydrochloric acid was used to dissolve the calcium carbonate, with the residue referred to as the percent inert.

Plots of average natural moisture content, Atterberg Limits, percent of inert material, and unconfined shearing strength are shown in Figure 3. The natural moisture content for the upper marl layers averaged from 68 to 78 percent, and was approximately 20 percent greater than the liquid limit. The natural moisture content for the silt and clay layers was approximately 50 percent and was only slightly higher than the liquid limit. The percent of inert material varied from 5 to 15 percent for the marl layers; from about 15 to 35 percent for the transition zone, and from 35 to 75 percent for the lower silt and clay layer with some marl.

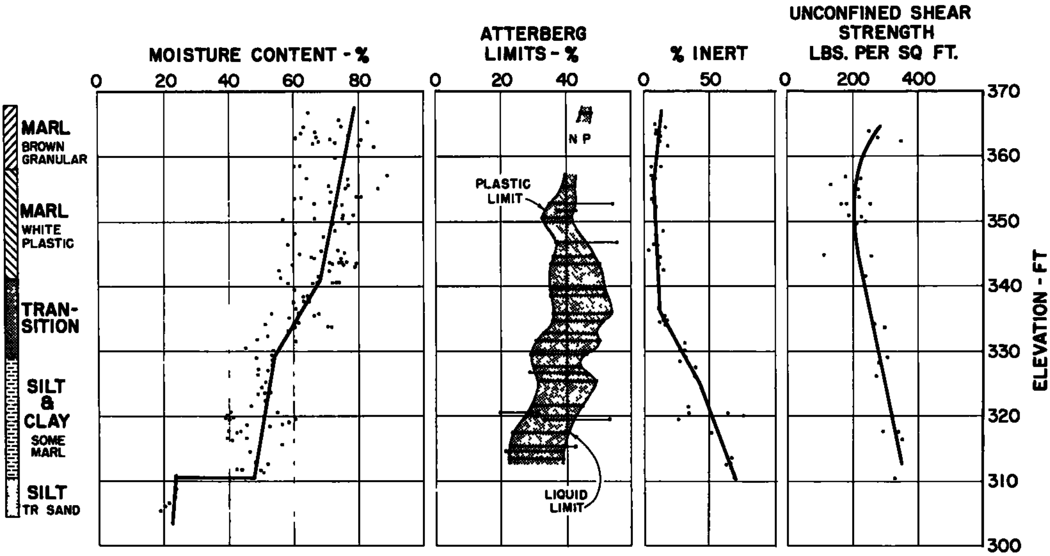


Figure 3. Soil characteristics.

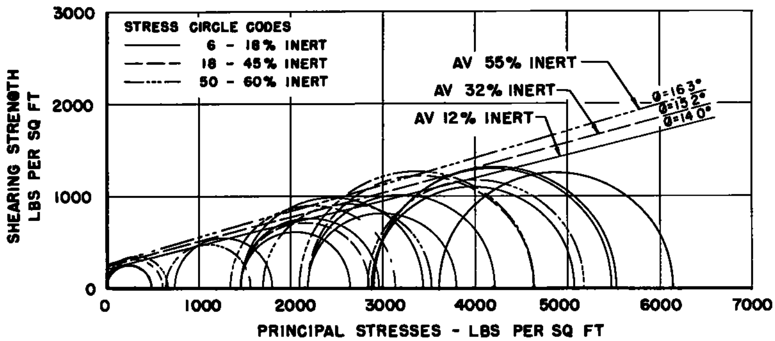


Figure 4. Typical triaxial-shear strengths.

The average unconfined shearing strength for the soft material varied from an average of 200 psf. for the upper marl layers to an average of 300 psf. for the lower silt and clay layers. Under triaxial-compression tests, the shearing strengths varied slightly with the percentage of inert material within the sample. The cohesion ranged from 200 to 250 psf., and the angle of internal friction from 12 to 16 deg. A typical triaxial shear strength relationship is shown in Figure 4.

Results of typical consolidation test data are given in Figure 5, along with computed values of the coefficient of consolidation. Figure 6 shows the pressure-distribution relationship against elevation. By comparing the preconsolidation pressures obtained from the consolidation test data with the natural overburden pressure, it may be seen that the upper 15 to 20 ft. of the foundation were precompressed to a value higher than the overburden pressure. Below this level the preconsolidation pressure was lower than the overburden. An explanation for this condition was obtained during the drilling operations when it was noted that artesian pressure existed in the lower granular layers. The design for this project gave consideration to the effects of each of these pressure features.

## ENGINEERING CONSIDERATIONS

Preliminary analyses indicated that the low strength and high compressibility of the foundation material presented a special design problem. For the areas between Station 2085 and Station 2091, stability analyses showed that the foundation would not support a fill built to the required grade by normal construction procedures. Figure 7 shows the

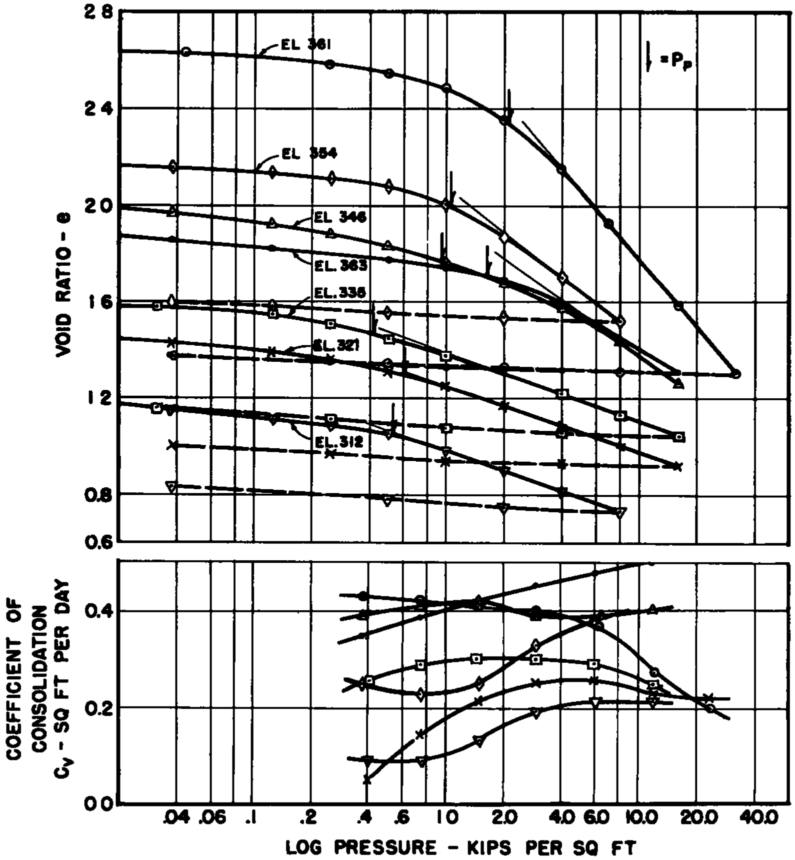


Figure 5. Consolidation test data.

reduction in factor of safety against failure for a normal construction rate of four months, with the fill built to a maximum height of 23 ft.

It should be noted that a factor of safety of 1.0 would be reached in approximately 3½ months, corresponding to an embankment height of 20 ft., and that foundation failure would be expected at this height of embankment or above. In computing the factor of safety, allowances have been made for the changing load conditions and for the increase in the strength of the foundation soil as a result of consolidation. Foundation shear failure could have been prevented by the use of counterweight side berms. However, such a design would have resulted in a maximum settlement of 9.7 ft., which would have been gradual and continuous over a period of about 40 years.

Due to the presence of bridge structures within the areas of questionable foundation and the riding quality requirements of high speed traffic, it was essential that the final

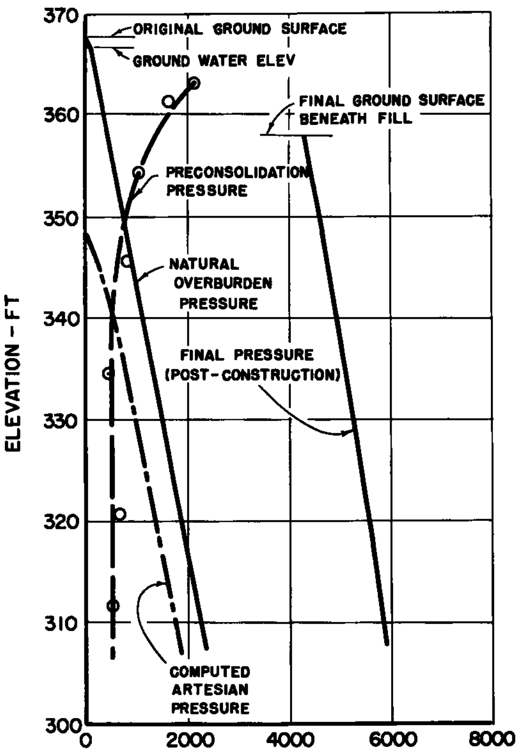


Figure 6. Depth versus pressure distribution.

design not only provide for stability, but also eliminate all settlement in the fill and foundation prior to the final paving operations. In addition, the urgency of completing the project and permitting its opening to traffic at an early date necessitated a minimum time for construction.

## METHODS OF DESIGN CONSIDERED

Many possible methods of treatment were considered and were judged by the economic considerations involved and the results to be obtained, consistent with the engineering requirements to be satisfied. Some of these methods considered for this project are as follows:

Change in alignment. Not possible due to restricted right-of-way.

Lowering of grade line. Not possible due to the minimum clearance requirements for the structures within the area.

Use of light-weight materials to lessen embankment weight. Such materials could not be obtained within an economic haul distance.

Removal of soft foundation material. Uneconomical and impractical due to the great depth of soft material and large quantities involved.

Partial excavation. Partial excavation to economic depths would have offered stability, but would not have solved the long duration settlement problems.

Displacement of the soft material. Uneconomical due to large quantities involved and the uncertainty of the quality of results obtained.

Use of side berms for stability. Procedure satisfactory for obtaining stability, but would not affect settlement which would be of high magnitudes and of long duration.

Use of sand drains. For this project, the use of sand drains to expedite settlement and aid in the rapid increase of strength in the foundation during the construction period was considered to be the most feasible for that portion where the compressible materials were of considerable depth. By the use of sand drains and a normal construction rate of approximately 4 months, most of the estimated settlement would occur within a period of 5 months from the beginning of construction. During the period of placement, the factor of safety would decrease to a minimum value immediately at the end of the construc-

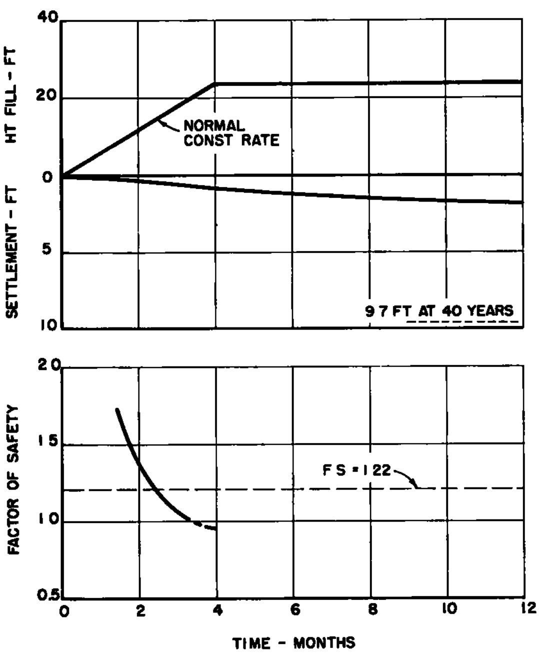


Figure 7. Normal construction rate, no foundation treatment.

tion period, and then would increase as further consolidation would tend to increase the strength of the foundation soil.

Figure 8 shows the effect of sand-drain installation on settlement and stability. This figure also shows the comparison of settlement and stability with the normal construction period without the use of sand drains, as also indicated in Figure 7. For this project, the minimum economical factor of safety was considered satisfactory at 1.22.

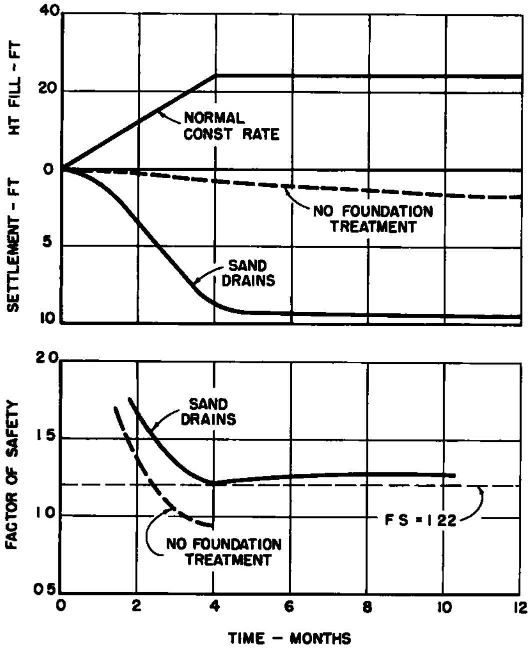


Figure 8. Normal construction rate, sand drains.

## STABILITY AND SETTLEMENT

Any special treatment of a foundation needs to satisfy the two main engineering requirements of stability and settlement.

### Stability

The embankment planned must be stable against any lateral displacement. In the design, it was essential to provide a construction procedure that would permit the completion of the embankment without a displacement of the foundation soil.

Embankment failures generally follow one of two common types; the squeezing or sinking type of failure, and the rotational type of failure. The first type consists of the embankment sinking as a unit into the soft foundation, producing mud waves on both sides. The latter consists of the embankment breaking and one section rotating downward and outward, producing a mud wave on the side of movement. Rotational failures may occur simultaneously on both sides of an embankment. All critical embankments are generally checked for both types of failures.

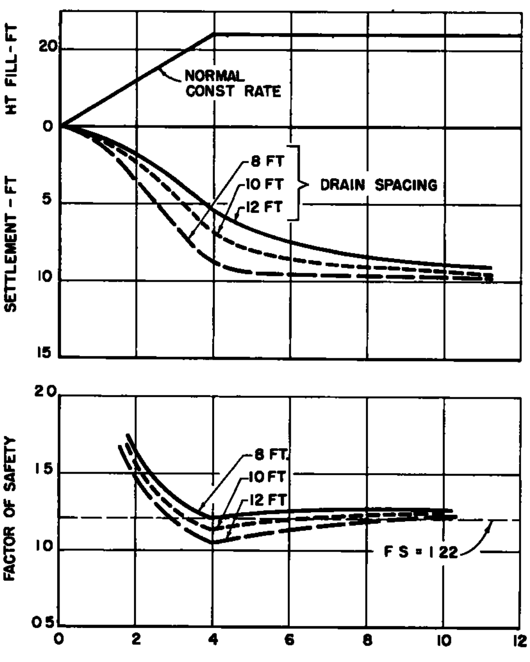


Figure 9. Normal construction rate, sand drains, various spacing.

For this project, the rotational type of failure was the more critical and was investigated in detail. The procedure used was the graphical circular arc method. In developing the detailed stability analyses, the strength increase in the foundation due to consolidation under the superimposed weight of embankment was considered. This was done by first determining from laboratory triaxial-shear tests on undisturbed samples, the initial strength of the foundation soil as found prior to construction, then determining the strength of the soil completely consolidated under the added load of the proposed embankment. For simplicity, the strength of the foundation soil was assumed to change from its initial to its final consolidated value in direct proportion to the degree of consolidation taking place under the weight of the embankment.

In the analysis, the location of the center of the most-critical circle is obtained by trial and error and varies with each assumed section and with each degree of consolidation. The method permits an accurate accounting of such variables as sloping water tables, variable densities and strengths, artesian pressure, degree of consolidation, and irregular ground surface.

The measure of stability has been expressed by a numerical value represented by the factor of safety which reflects the ratio of the resisting forces to the overturning forces obtained from the graphical analysis method. By this procedure, the factor of safety can be computed for the varying stages of construction, reflecting the changing height of fill and changing degree of consolidation, and can be plotted as shown in Figures 7 to 12.

## Settlement

A highway embankment may be safe against failures but may be entirely inadequate because of possible future settlement. Settlement is the gradual compression of a soil layer which involves volume reduction without lateral displacement of the soil mass. For this project, it was necessary that the final design provide for the elimination of all settlement prior to the final paving operations.

A total of 24 consolidation tests were run on segments of samples located at regular depth intervals from each of five separate drill holes. Samples were set up to apply loads and run permeability tests in both horizontal and vertical directions, with respect to their natural positions.

Computations to obtain the magnitude of settlement at the various locations both transversely and longitudinally within the embankment areas were made, using normal procedures based on the ratios of load to void, the preconsolidation pressure, the overburden pressure, and final total pressure. Settlements were computed by the methods of depth increments to reflect variations in the material and pressure distributions.

The time-settlement rate was computed from the coefficient of consolidation values corresponding to the load-increment ratio from overburden and preconsolidation to final load. For sand-drain computations, the overall time-settlement rate is a combination of two component parts: (1) that due to vertical drainage as a result of normal consolidation and (2) that due to radial drainage into the sand drains. The two effects are com-

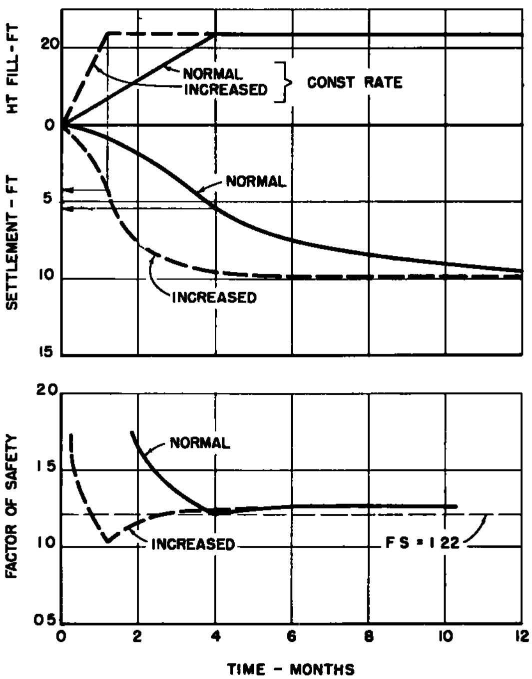


Figure 10 Sand drains, increased construction rate.

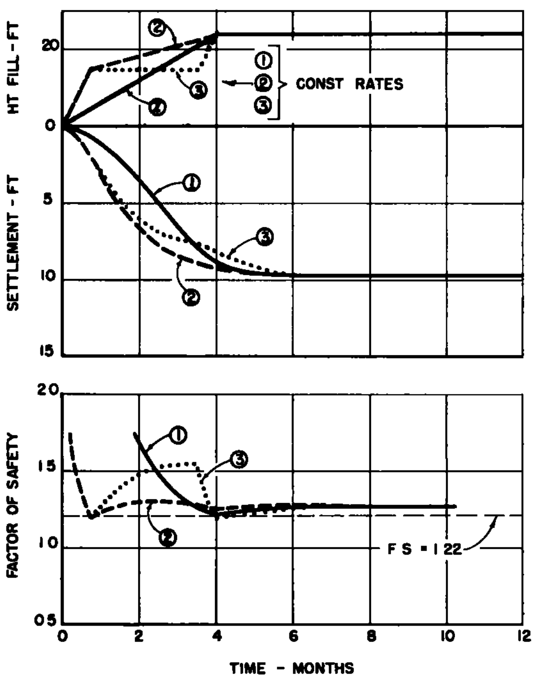


Figure 11. Sand drains, various construction rates.

puted separately and then added to reflect the proper time ratios and the effects of the rate of load application.

The ratios of the coefficient of permeability in a horizontal and vertical direction are also evaluated to develop the proper relationships of the two component parts of drainage. In this instance, the horizontal coefficient of permeability for the materials was only slightly higher than the vertical coefficient of permeability. To compensate for some of the other effects indicated below, the computations were actually based on equal coefficients of permeability in both directions.

For a proper evaluation of sand-drain effects, consideration was also given to other factors which affect the assumed behavior of sand drains: smear of the various layers due to the driving and pulling of the casing during the sand drain installation, possible remolding and its effects on the strength of the foundation soil around each sand pile, and the unequal strain or differential settlement that may result from radial flow. Although these effects cannot be determined with any degree of accuracy, an approximation of their possible effects can be made and can be reflected in the values of the minimum factor of safety considered satisfactory for the overall project.

## SAND-DRAIN-DESIGN RELATIONS

The proper design of a sand-drain project reflects a balance among the various factors which affect stability and settlement. The relationships between applied load and soil characteristics which are of most importance in the solution of such problems are: (1) the amount of settlement in the foundation increases with the increase in the applied load; (2) the rate of settlement increases with a decrease in the distance which which water must travel to escape from the soil mass during consolidation; and (3) the shearing strength of the soil increases with consolidation.

These relationships are affected by the diameter of sand drains installed, the spacing of sand drains, the rate and period of construction, the height of embankment surcharge, and the time such surcharge is retained.

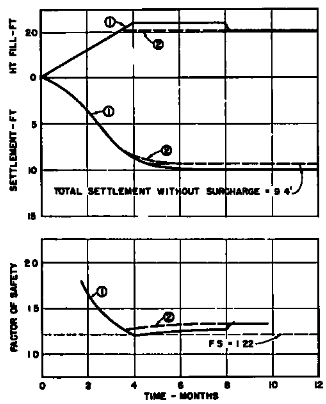


Figure 12. Surcharged embankment.

Sand drains 18 to 22 inches in diameter have been accepted as standard for the type of construction equipment currently available and the depths of penetration anticipated for this project. Consequently, an assumed diameter of 18 in. was used for all design considerations, and the effects of diameter variations were not studied in detail. A description of the effects of some of the other factors follows:

### Spacing

The spacing of sand drains determines the distance that the water in the soil voids must travel to escape during the consolidation process. The greater the spacing, the slower will be the rate of consolidation and, consequently, the slower will be the rate of increase in the strength of the foundation.

Figure 9 shows the relationship of the effects of sand-drain spacing on the rate of settlement and on the change in the foundation factor of safety for a normal embankment construction rate of four months. As indicated in the figure, after an elapsed time of approximately 8 months, practically all the settlement has occurred for the 8-ft. spacing, whereas an additional settlement of some 9 in. would be expected for the 10-ft. spacing and 16 in. additional for the 12-ft. spacing.

Due to the reduced rate of consolidation and reduced rate of foundation strength increase, the factors of safety at the end of the construction period would also be reduced from 1.22 for the 8-ft. spacing, to 1.14 for the 10-ft. spacing, and to 1.05 for the 12-ft.

spacing. It should also be noted that the 8-ft. spacing would require one drain for each 64 sq. ft. of foundation area; the 10-ft. spacing one drain for each 100 sq. ft., an increase of 63 percent in the number of drains; and the 12-ft. spacing one drain for each 144 sq. ft., an increase of 125 percent in the number of drains compared to the 8-ft. spacing.

## Construction Rate and Period

The rate and period of construction are other variables affecting both settlement and foundation stability. Figure 10 shows the effects of two uniform rates of construction, the normal rate of four months duration, and an increased rate in which the total embankment load is placed in approximately one month.

As would be expected, an increase in the rate of load application results in an increase in the rate of settlement. Consequently, the increased rate of construction shows greater settlement at the end of the same 4-month construction period. However, the increase in the rate of settlement is slower than the increase in the rate of load application, and less total settlement takes place at the end of the 1-month construction period than at the end of the normal 4-month period. This reduced total settlement at the end of the rapid construction period reflects a proportionate reduction in the strength relations and the computed factor of safety.

Varying the construction rate will also affect the settlement and the stability relations. Figure 11 shows the effects of three separate construction rates within an overall 4-month construction period.

Case 1 reflects the same uniform 4-month construction period covered in Figure 10, in which the minimum factor of safety occurs at the end of construction.

In Case 2, approximately $\frac{9}{10}$ of the total embankment height are shown reached in 20 days, and then the fill continued to full height at a reduced uniform rate for an overall 4-month period. The factor of safety drops rapidly as the fill height is increased, reaching a minimum at the end of the 20-day period. It then gradually rises, showing minor variations which are the result of changes of strength increase affected by the rates of applied load and settlement.

If desired, the rate of fill increase could be adjusted to result in an overall constant factor of safety to the end of the construction period after having reached the minimum value during the first stage of rapid construction. In Case 2, the settlement is more rapid due to the increased rate of load application during the early stages.

In Case 3, the rapid rate is shown for the first 20 days as in Case 2, but it is then followed by a waiting period of approximately 2¼ months before it is brought to full height at the initial rapid rate. Here the factor of safety drops to 1.2 at the end of the initial construction period, increases during the waiting period to over 1.5, and then reduces to the initial value during the later stages of construction as the additional weight of fill is placed on the foundation.

These computed values of factor of safety and settlement could vary over a wide range, depending on the time rates of construction assumed or anticipated for the project. Normally the most-critical time rate of construction that the successful bidder may undertake is assumed for design. In many instances, however, where an unrestricted time rate of construction becomes critical, the final computations reflect a definite maximum construction rate, which rate is then made part of the proposal and contract plans, and strictly enforced during construction.

## Surcharge

In addition to the design considerations of sand-drain spacing and construction rate, the effects of surcharging the embankment must be analyzed. A surcharge is an additional height temporarily added to the fill beyond final pavement grade to expedite settlement and prestress the foundation. Generally, the greater the ratio of height of surcharge to height of normal fill, the more effective becomes the surcharge. The actual amount of surcharge required depends on the time-settlement relations of the compressible layer, the normal embankment height, the strength of the foundation, and the overall time required for completion of the project.

As the amount of settlement in a given period of time increases with an increase in applied load, a surcharge could be removed after the settlement expected under the normal embankment height has been reached. A surcharge maintained for the full time required to eliminate all settlement under even the surcharge height will tend to prestress the foundation and eventually increase the factor of safety after its removal.

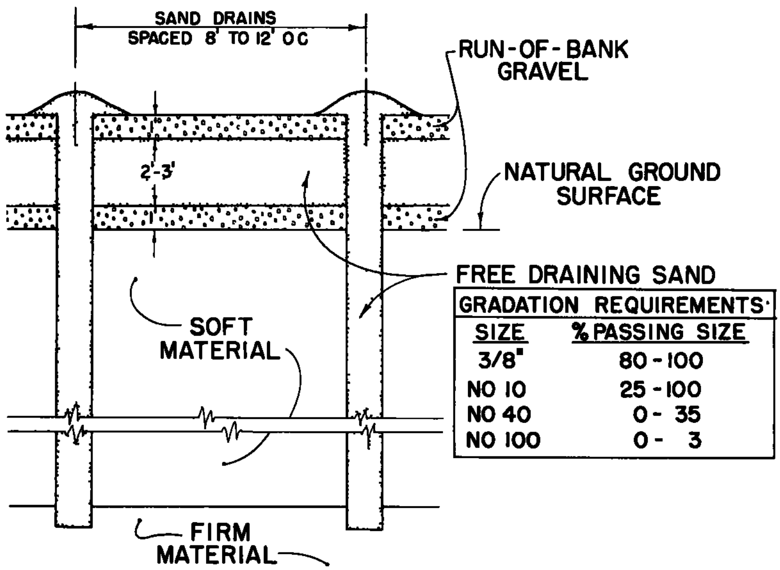


Figure 13. Drainage blanket details.

The curves in Figure 12 show the effects of surcharging on settlement and stability. In this case, a surcharge 3 ft. high would reduce the time for settlement under the normal embankment height by approximately 2 months. A higher surcharge would give a further time reduction in settlement but would cause such a reduction in the factor of safety to be considered dangerous. The same 3-ft. surcharge left in place to eliminate all settlement under the surcharge will prestress the foundation and permit approximately 0.6 ft. of additional settlement.

## Sand Gradation

In concluding the discussions on design, mention must be made of the consideration given to the gradation of the sand used for backfilling the drains and the granular drainage blanket, the anticipated schedule of operations, and the seasons of the year and climatic conditions that might be expected during construction. These features affect the scheduling of the different operations and the final completion date.

The sand placed in the sand drains should be sufficiently permeable to permit the passage of water squeezed from the foundation without adding measurably to the flow frictional resistance and of such gradation to act as a filter to prevent movement of the fines into the drains themselves.

The drainage blanket serves two general purposes: (1) to provide a working platform for the contractor's equipment during the installation of the drains and (2) to permit free passage of water flowing from the drains to the outlets provided. The material planned for the sand drains and the permeable sand drainage blanket was specified to meet the gradation requirements listed in Figure 13.

For this project, the available material meeting this requirement was processed material and was tested to show a minimum value of coefficient of permeability greater than 500 ft. per day, placed in a dense state, which value was used in the design. Based on this value of permeability, the drainage-blanket thickness required to remove the water squeezed from the foundation during settlement was set at 3 ft. for the deep foundation areas, where large settlements were expected, and 2 ft. of all other areas.

As these thicknesses of 3 ft. and 2 ft. were not sufficient to provide a strong platform to hold the contractor's equipment for the installation of the sand drains, the sand blanket was reinforced with less costly and more-readily available run-of-bank sand and gravel, placed a foot directly on the original ground below the clean free draining sand and a foot above the free draining sand layer, as shown in Figure 13. In addition to strengthening the overall blanket, the placement of the sand and gravel layer below and above the free-draining sand minimized possible contaminations of the clean sand and permitted effective use of its full thickness.

## FINAL DESIGN

The final design prepared for this project reflected what was considered an economical combination of sand drain spacing, rate of construction, height of surcharge, and time for retaining surcharge, all consistent with the overall time anticipated for the project and the quality of results desired.

The spacing of sand drains was set as shown in Figure 14, which included 8-ft. spacings within the deep foundation area and adjacent to the lake outlet bridge east abutment, 10-ft. spacings in the remainder of the moderately deep foundation areas and on both sides of the lake park bridge, and 12-ft. spacings at the easterly end of the project where the soft foundation was shallower.

Considering the settlement-stability relations, the embankment could have been planned for completion in a minimum period of 4 months. However, due to possible unanticipated delays in installing the sand drains, the plans and proposal were prepared to require a minimum of 6 months for the overall construction period, with the following wording:

"It is anticipated that the embankment can be built uniformly throughout the entire area to a height of 10 ft. above the granular drainage blanket without any rate of restriction due to foundation requirements but that the remaining height may be restricted to such a rate that the overall time for embankment construction will be approximately six months."

Due to the critical stability relations under the normal embankment height, the amount

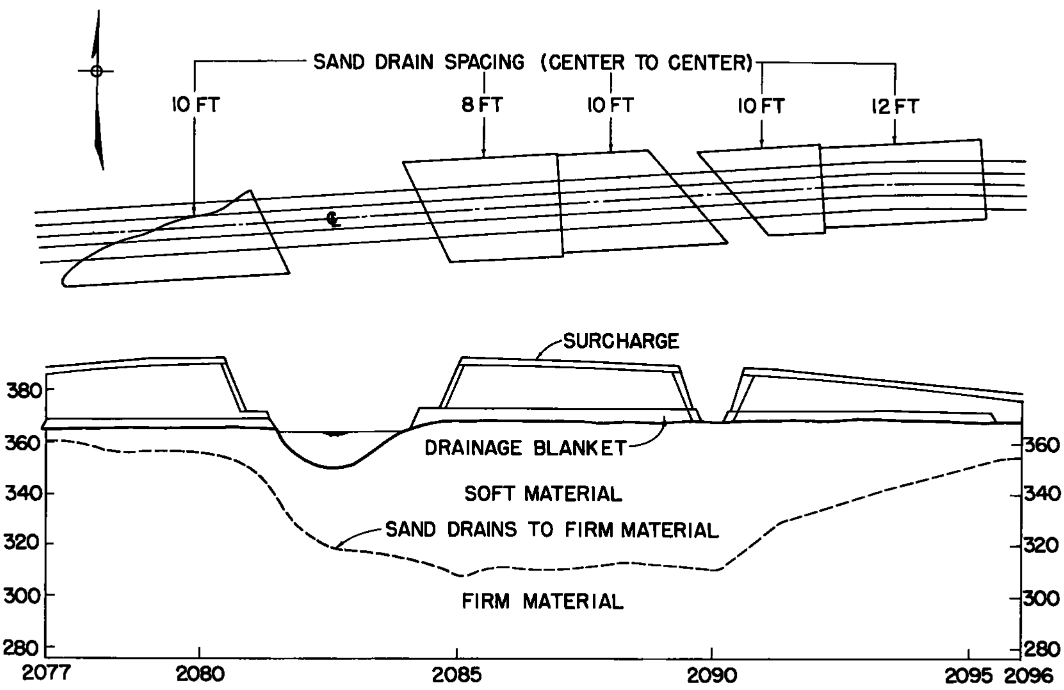


Figure 14. Final design, plan and profile.

of surcharge permitted in the final design was limited to 3 ft. above final grade. Consequently, it was also required that the surcharge be retained for a minimum period of 6 months before removal to permit full foundation settlement and adjustment under this small increment.

As discussed earlier, the drainage blanket consisted of a foot of run-of-bank sand and gravel placed on the original ground surface; a layer of free-draining sand, either 2 or 3 feet thick, having a minimum coefficient of permeability of 500 feet per day; and a top layer of run-of-bank sand and gravel a foot thick to strengthen the working platform and to minimize possible contamination of this clean free draining sand.

Figure 15 shows the relationship of rates of settlement and factors of safety for the designed rate of construction, surcharge waiting period, removal of surcharge, placement of the foundation course gravel, and final placement of the pavement. According to this figure, the minimum factor of safety was 1.22, and full settlement was anticipated before the removal of surcharge. Considering the many assumptions made and the fact that actual time for construction might be somewhat longer than proposed, the designed minimum factor of safety of 1.22 was assumed to be satisfactory for this project.

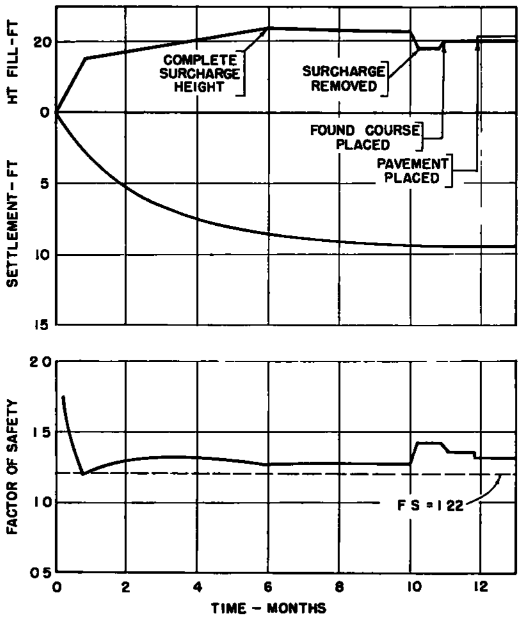


Figure 15. Design conditions sand drains, surcharged restricted construction rate.

Portions of the project on either end of the sand drain limits contained soft founda-

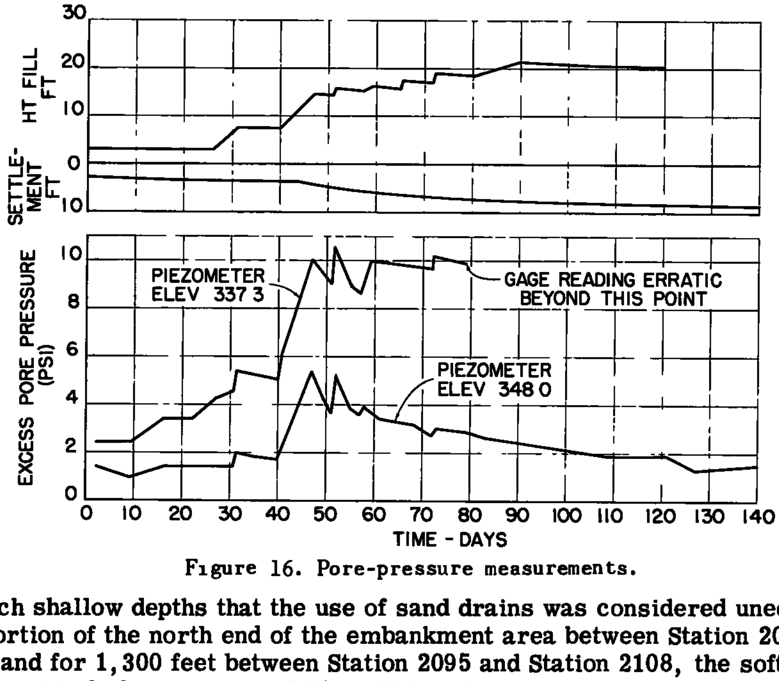


Figure 16. Pore-pressure measurements.

tions of such shallow depths that the use of sand drains was considered uneconomical. Within a portion of the north end of the embankment area between Station 2077 and Station 2081, and for 1,300 feet between Station 2095 and Station 2108, the soft foundation material consisted of an average of 10 to 12 ft. of marl. These areas were stabilized by adding a surcharge to a minimum height of 3 ft. above final grade and maintaining this surcharge for 6 months. The soft foundation material was sufficiently shallow to develop full consolidation within this 6-month period under the surcharge provided.

## FIELD CONTROL

As the design was based on obtaining definite strength increase in the foundation material resulting from consolidation due to weight of superimposed embankment, both settlement and pore pressures were measured during construction, and the relationships between relative strength increase and rates of embankment construction were evaluated. Pore pressure measuring devices, settlement platforms, and alignment stakes were installed and observations made during and after the construction period.

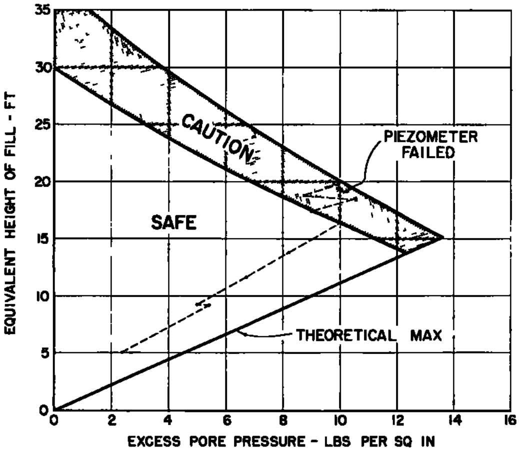


Figure 17. Excess pore-pressure control chart.

The pore-pressure-measuring devices originally planned were porous Norton tubes. As these tubes were unavailable at the time of installation, regular well points were substituted. These devices were set at the proper elevations and connected to a Bourdon gauge beyond the limits of the fill with ½-inch o. d. plastic Saran tubing.

The settlement platforms used were 3-by-3-feet-square steel plates set on the original ground and firmly connected to 2½-inch-diameter standard pipe, which was extended in length as the fill height increased.

Readings obtained from the settlement platforms proved to be consistent and satisfactory. The pore pressure readings obtained from piezometers were quite erratic.

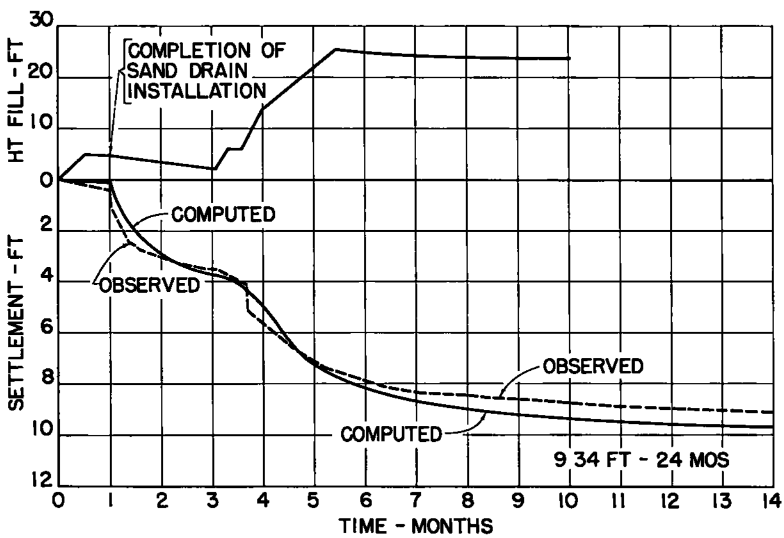


Figure 18. Settlement, computed versus observed (Station 2085 + 70).

Figure 16 shows typical readings obtained from both settlement and the pore pressure devices.

For purposes of control, a chart was prepared which permitted the plotting of the foundation excess pore pressure against the equivalent embankment height, corrected for settlement, location of water table, and unit density of the component parts. On this chart were superimposed the corresponding factors of safety obtained from the design computations.

It was the intention that a mere plotting of the corrected pore pressure values obtained from field measurements and the corresponding fill height would give the field inspectors a direct guide of the relations developed. As the pore pressure readings were somewhat erratic, the use of this chart was effective only in a general way. The settlement readings obtained from the settlement platforms were much more reliable, and the measured rates and amounts of settlement converted directly to the degrees of consolidation were mainly used for construction control.

## COMPUTED AND OBSERVED SETTLEMENTS

A high point in any paper about sand drains is a comparison of the designed and actual rates of construction and the computed and measured settlements. Figure 18 gives such

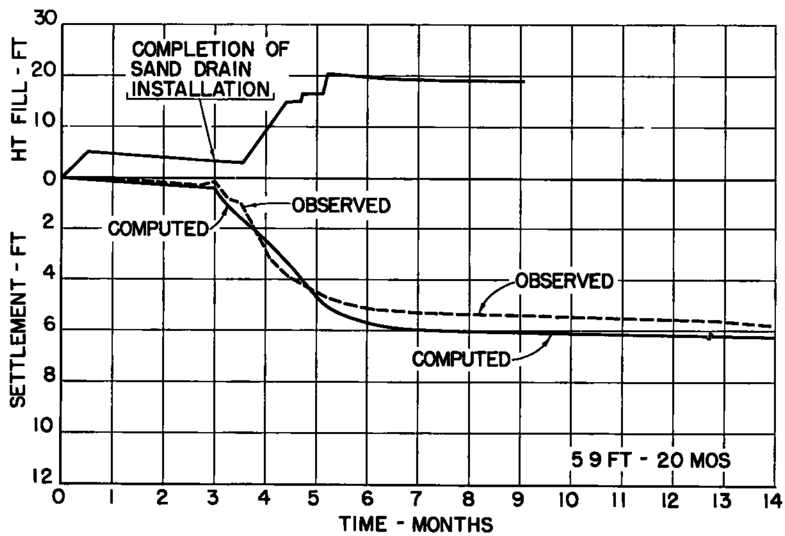


Figure 19. Settlement, computed versus observed (Station 2091 + 55).

a comparison for Station 2085 + 70. The actual construction rate shows a minor delay following the placement of the sand blanket, during which time sand drains were installed. The computed settlement was readjusted to fit the actual construction period, and is shown plotted along with the observed settlement. These show close agreement with each other.

The agreement is of particular interest since the computations considered numerous unusual properties of the foundation materials, including the relief of artesian pressure. The large settlements computed and actually observed immediately following the sand-drain installation were attributed to the relief of artesian pressure and the subsequent increase in the unit weight of the foundation soil.

Figure 19 gives similar relationships for the condition at Station 2091 + 55. Here again, both the rate and amount of computed settlement are in close agreement with those observed.

## MOISTURE CONTENT CHANGE

It is generally assumed that true settlement should be accompanied by a reduction in moisture content, with the volume of water squeezed out being equivalent to the settlement observed. To verify this relationship, moisture contents of the foundation soil were obtained by direct sampling both before and after construction.

Figure 20 shows the plots of moisture content versus elevation before and again after construction. Each plot includes values obtained from two holes, located within an over-

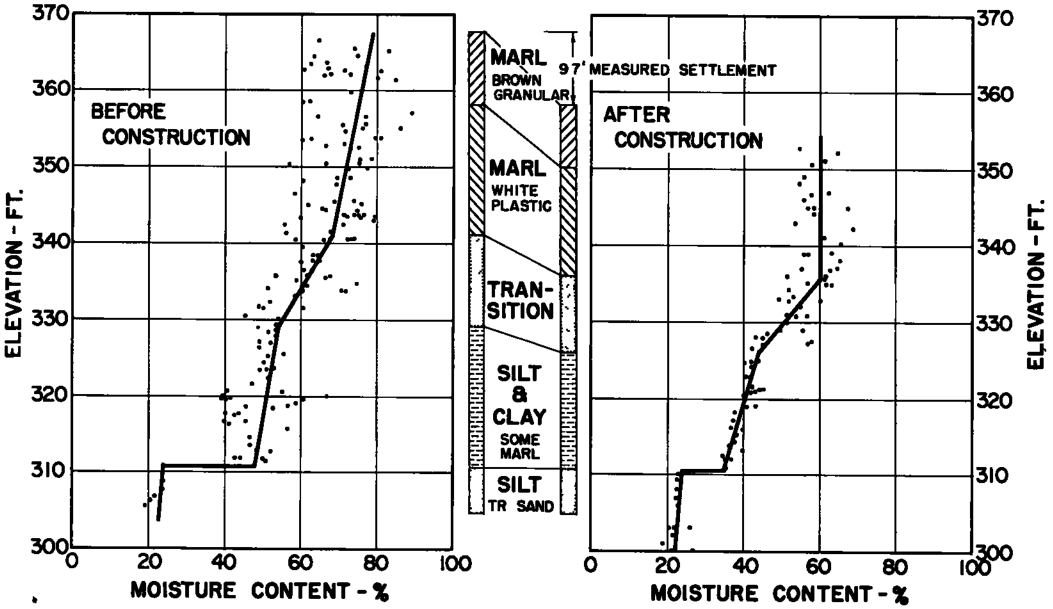


Figure 20. Moisture content before and after construction.

all distance of 200 ft. in the deep soft foundation area between Station 2085 and 2087. The line drawn through these points represents weighted average moisture content values to permit evaluating the before and after conditions. The shortening of the profile in the after-construction plot is the result of settlement, adjusted to reflect the proportionate settlement in each layer.

In Figure 21, the elevations of the before-construction moisture contents have been modified to allow for settlement at each level and permit a direct comparison with the moisture contents measured after construction. The distance between the curves at any elevation represents the loss in

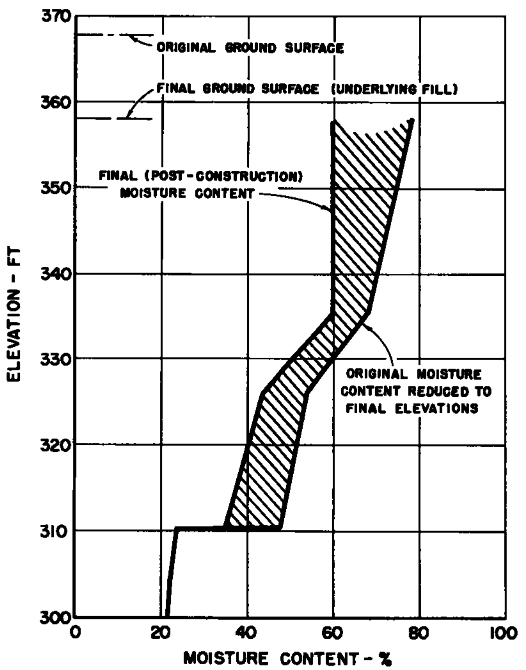


Figure 21. Moisture content before and after construction, corrected for settlement.

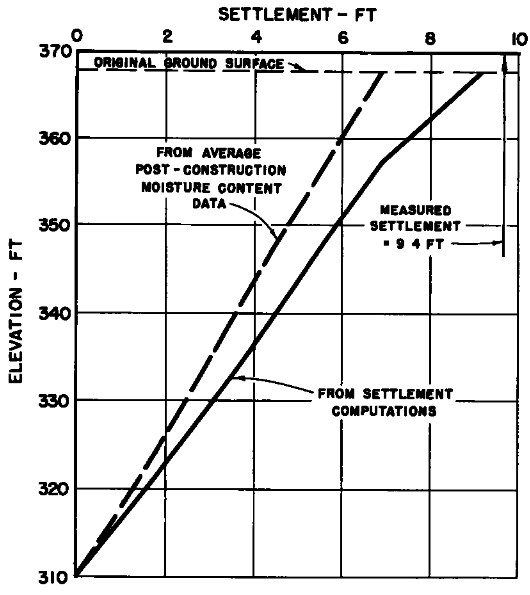


Figure 22. Computed settlements.

moisture content as the result of construction, and the shaded area reflects the total moisture loss, which is a measure of the settlement in the foundation.

An attempt has been made to evaluate the settlement directly attributable to the reduction in moisture content by converting the loss in moisture content to direct volume changed and to settlement. This has been plotted, shown in Figure 22, as a summation starting from the lower firm layer and reflecting the total sum at the original ground surface. The computed settlement for the same conditions has also been plotted for comparison.

In Figure 18, it was shown that the measured settlement agreed very closely with that predicted by preconstruction computations. This measured settlement is also indicated in Figure 22 on the settlement scale. As there is close agreement among these relations, it follows that the settlements occurring on this project can be accounted by a change in the moisture content of the foundation material.

## CONCLUSION

The procedures employed on this project for field explorations, laboratory evaluations and design analyses, together with the observations and measurements made before, during, and after construction, permit the following statements:

1. The use of sand drains represents only one of several procedures that can be used for the treatment of soft foundation areas. The choice of type of treatment depends primarily on the economics and the desired rate of project completion.
2. Where sand drain treatment is to be used, a careful analysis must be made of all factors to assure a satisfactory job and one that is engineeringly economical.
3. The relative close agreement of settlement data for this project—those obtained by direct measurement, those computed from soil characteristics developed in the laboratory, and those computed from moisture changes in the foundation soil—indicates that the action of sand drains is fairly well defined by existing theories of soil mechanics.