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Vertical Sand Drains for Stabilization of Embankments

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Modification of Sand-Drain Principle for Pressure Relief in Stabilizing Embankment Foundation

RAY WEBBER, Assistant Construction Engineer, and
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A 20-foot double embankment on the Portland-Salem Expressway crosses a peaty, poorly drained area. The application of the sand-drain principle to the perimeter of the embankment area provides outlets for the free water under pressure of the embankment loads and permits accelerated consolidation of the foundation soils under these loads without lateral flow beyond the toes of the embankment.

Two bridge approach areas in this fill, not protected by pressure relief drains, showed signs of lateral flow through slight horizontal movements of bridge piling.

●A STUDY of embankment failures on Oregon highways, where they are underlaid with unstable ground, has revealed an unusual rise of water through the embankment material. This hydrodynamic action within the embankment has been apparent at times 10 ft. above the original ground surface even though the static water table was several feet below that level prior to the construction of the fill.

During the construction of such an embankment on the Oregon Coast Highway several years ago movement of the foundation soil occurred at the toe of the slope. In order to permit continued construction and avoid property damage or extra right-of-way costs through an ownership adverse to the improvement, an attempt was made to increase the stability of the foundation soils by placing vertical drains along the embankment toes. Holes were drilled on 10-ft. centers with an 8-in. power auger to a depth of about 20 ft. without the use of casing by highway department drill crews. The depth to hard bottom ranged up to 40 ft. Upon withdrawing the auger steel, the holes partially closed, permitting only incomplete filling of the holes with dune sand.

Immediate results were observed as the dynamic embankment pressures forced the foundation pore water through the drilled zones to the surface of the native soil blanket or working platform, which was about four ft. above the original ground surface. These pressures were sufficient on some of the holes to force out the dune sand soon after the first filling. These relief drains continued to function as long as the moving embankment created dynamic pressures forcing the flow of water out of the foundation zone to the outer area pierced by the auger holes. Complete success was attained in stabilizing the toe area of this project without affecting the area outside the right-of-way limits.

Experience on this and other Oregon highway projects where modifications of this idea have been used has indicated that relief of excessive pressures can be obtained by the use of vertical drains put down from an impervious working platform about four ft. above the surface of the unstable ground along the perimeter of an embankment. This will provide the release of pressures in excess of about 2 psi., which appears to be well below the danger point for lateral flow of the foundation material under the outer slope area of our embankments. Later application of this hydrodynamic pressure relief by vertical sand drains along the toes of slopes has indicated an apparent control of foundation consolidation even though the lateral or transverse distance between the lines of drains exceeded 100 ft.

Three important conditions probably account for the effectiveness of these drains: (1) horizontal movement of foundation pore water is greater than the vertical flow; (2) the danger zone of lateral flow of foundation material due to hydrodynamic pressure lies between the shoulder line and toe of the slope; and (3) the submerged portion of the settling embankment presents a convex surface to hydrodynamic pressures, deflecting them to the toe area where the pressure relief drains provide an outlet for the pore water under excessive pressure.

This report describes the use of the above principles on a recent Oregon State Highway Department project where physical measurements closely agreed with movements predicted.

Since this use of vertical sand drain holes in relieving internal pressures below an embankment does not follow, in its entirety, the orthodox application of sand drains, any reference to such a system has been avoided. We prefer to consider this method of foundation stabilization as a hydrodynamic pressure relief.

TULATIN BEAVER DAM AREA, PORTLAND-SALEM EXPRESSWAY

A weak foundation area across a shallow valley located about one-half mile south of the Tulatin River in the southwest corner of Washington County, Oregon, was to carry a double, 20-ft. embankment for north- and southbound traffic. The foundation soils consisted of about 26 ft. of organic, non-fibrous, peaty soil. Below this lay the organic

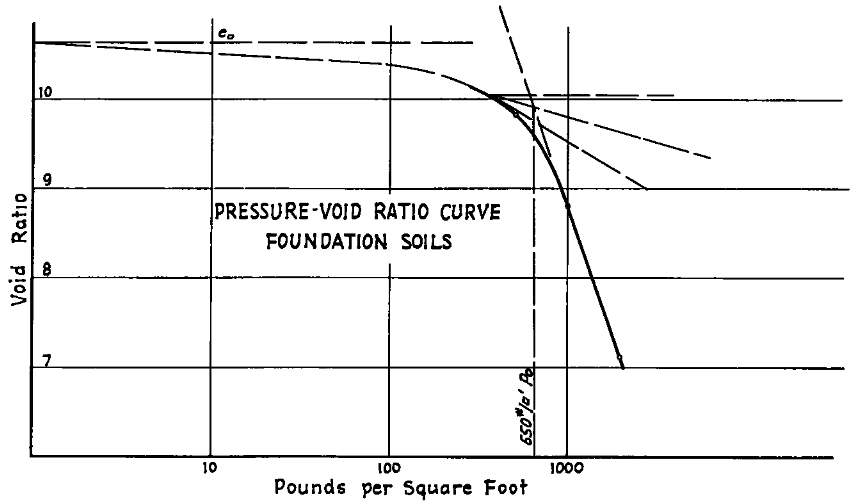

Figure 1.

silt loams and mixed silty sands and fine gravels of the Wapato soils of the locality. The embankment was to be built during the fall and winter of 1952. The next fall two bridges, one for each lane, were to be built on pile foundations at approximately the mid-point of each fill as shown in Figure 2.

In order that the foundation material might be identified and the strength characteristics determined, foundation explorations were made by a portable, vertical power auger. Samples of the organic soil were taken in an as undisturbed condition as possible with a special tube sampler. The consolidation analysis of this material revealed that a total embankment settlement of about 5.1 ft. for the proposed 20-ft. embankment might be expected over a period of about 15 years if the organic soils acted as a compressible clay. Since the construction program called for paving this arterial highway within two years, it was thought that some means of accelerating this subsidence should be sought, and that the work should be let by contract.

FOUNDATION STUDIES

The foundation soils were known to be highly-organic, non-fibrous peat with probably a much higher coefficient of permeability than clays and, therefore, a higher but unknown rate of consolidation. Undisturbed samples of these organic foundation soils were taken with a 2½-inch-diameter tube sampler forced into the soils by hand or by driving with a fence post driver of the closed-pipe type. Routine tests on five samples taken between 2 and 11 ft. below the ground surface showed a gradation indicated in Figure 17. The liquid limit percentages ranged between 41 and 153 and the plasticity indices ranged between 0 and 23. There were sufficient inorganic soil particles to indicate a range in specific gravity of from 1.73 to 2.65. No ignition test was made to determine the percentage of organic material, as it is felt that such a test run at high enough temperature

TABLE 1

TULATIN PRESSURE RELIEF DRAIN FOUNDATION CONSOLIDATION

Embankment Ht. 20 ft. Roadbed Width 38 ft. Embankment Slope 1½ : 1

Foundation Soils:

Depth to Water 3 ft. Depth of Foundation Soils 23 ft.

Unit Weight, assumed 37 pcf., submerged = W_{found}

Specific Gravity 2.67 = G_s Cohesion, "c" = 101 psf. ϕ = 26½ deg.

Embankment Soils: Clay, Wt. 125 pcf. = $W_{emb.}$

COMPUTATIONS

$$W_{found} = \frac{(G-1)(62.4)}{1 + e_{bottom}} = \frac{(2.67-1)\ 62.4}{1 + 5.76} = 15 \text{ pcf. (1st approx.)}$$

$$= \frac{(2.67-1)(62.4)}{1 + 6.04} = 14.8 \text{ pcf.}$$

Preconsolidation pressure, probably dessication = ± 500 pcf.

Soil Pressures:

Depth ft.	Material	Layer Pressure psf.	Accum. Pressure psf.
0 - 3	Clay	300	300
3 - 13	Peat	10 (15) = 150	450
13 - 23	Peat	10 (15) = 150	600

Fill Pressures: Emb. Soils

Depth	m	n	f (m,n) (Newmark)	Pressures (psf.)
0	o	o	o	20 (125) = 2500
3 ft.	$^{34}/_{3}$ = 11	$^{46}/_{3}$ = 15	.25	4 (.25) (2500) = 2500
13 ft.	$^{34}/_{13}$ = 3	$^{46}/_{13}$ = 3½	.243	4 (.243) (2500) = 2430
23 ft.	$^{34}/_{23}$ = 1½	$^{46}/_{23}$ = 2	.223	4 (.223) (2500) = 2230

Effective Pressures:

Depth	0	3 ft.	13 ft.	23 ft.	e	Average (Simpson)
Nat. Soils (p_1)	0	300	450	600	6.11	450 psf.
Fill prs (Δp)	2500	2500	2430	2230	- - -	- - -
Total $p_2 = p_1 + \Delta p$	2500	2800	2880	2830	4.57	2859 psf.

Average Total Pressure $\frac{1}{2}(p_1 + p_2)$ = 1654 psf.

Settlement Estimate:

I. Void Ratio Method $= \frac{2H}{1 + e_1}(e_1 - e_2)$

$$\frac{20}{1 + 6.11}(6.11 - 4.57) = 4.3 \text{ ft.}$$

II. Compression Index Method $= \frac{2H}{1 + c_1}\left(\frac{p_2 - p_1}{\frac{1}{2}(p_1 + p_2)}\right)(.435)(C_c)$

$$\left(\frac{20}{1 + 6.11}\right)\left(\frac{2859 - 450}{1654}\right)(.435)\ (2.79) = 5.0 \text{ ft.}$$

Average Computed Settlement Used = 4.7 ft.

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to insure complete combustion of the organic material also destroys certain inorganic soils and, therefore, can be misleading.

Foundation stability computations based on Terzaghi's approach (1, 2, 3) showed that the foundation soils would fail by shear and that lateral flow of the organic soil would take place outside the right-of-way boundaries unless wide counterbalances were constructed on either side of the embankment. These counterbalances, in addition to requiring considerably more right-of-way than normal through expensive truck garden land, would not prevent the undesirably slow settlement due to foundation consolidation.

The most-representative-appearing, 1-inch-thick sample of the "undisturbed" founda-

TABLE 2

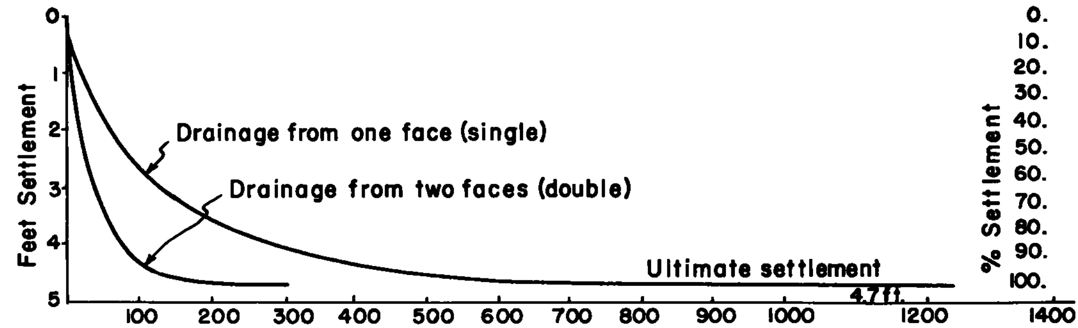
Rate of Settlement in days = "t" = $\frac{H_d^2\,T}{C_V}$

H_d = Depth of Drainage Face ,
T = Time Factor dependent on pressures and drainage
C_V = Coefficient of Consolidation for Average Pressure

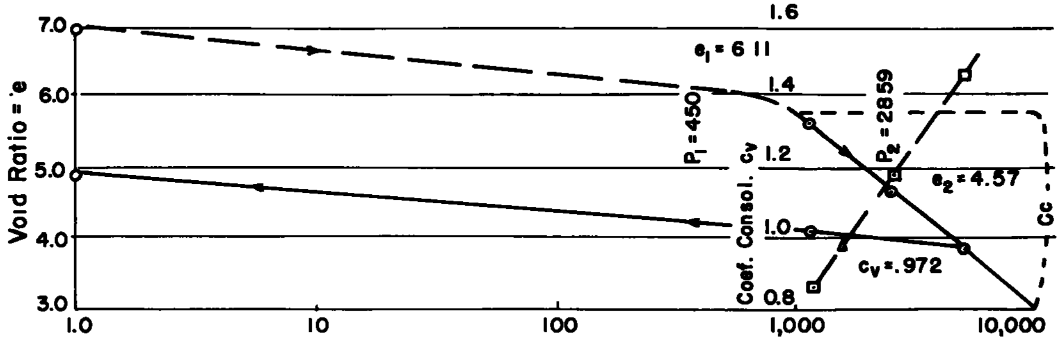
% Consol.	Settl't. Feet	T	Rate of Settlement in Days Single Drn.	Double Drn.
10	0.5	.008	3	1
20	0.9	.032	13	3
30	1.4	.069	28	7
40	1.9	.126	52	13
50	2.3	.198	82	21
60	2.8	.288	119	29
70	3.3	.405	167	42
80	3.8	.566	233	58
90	4.2	.846	349	87
95	4.5	1.134	468	116
100	4.7	$\pm$3	$\pm$ 1236	$\pm$ 308

Single Drainage
"t" = $\frac{20^2}{.972}$ T = 412 T

Double Drainage
"t" = $\frac{10^2}{.972}$ T = 102 T



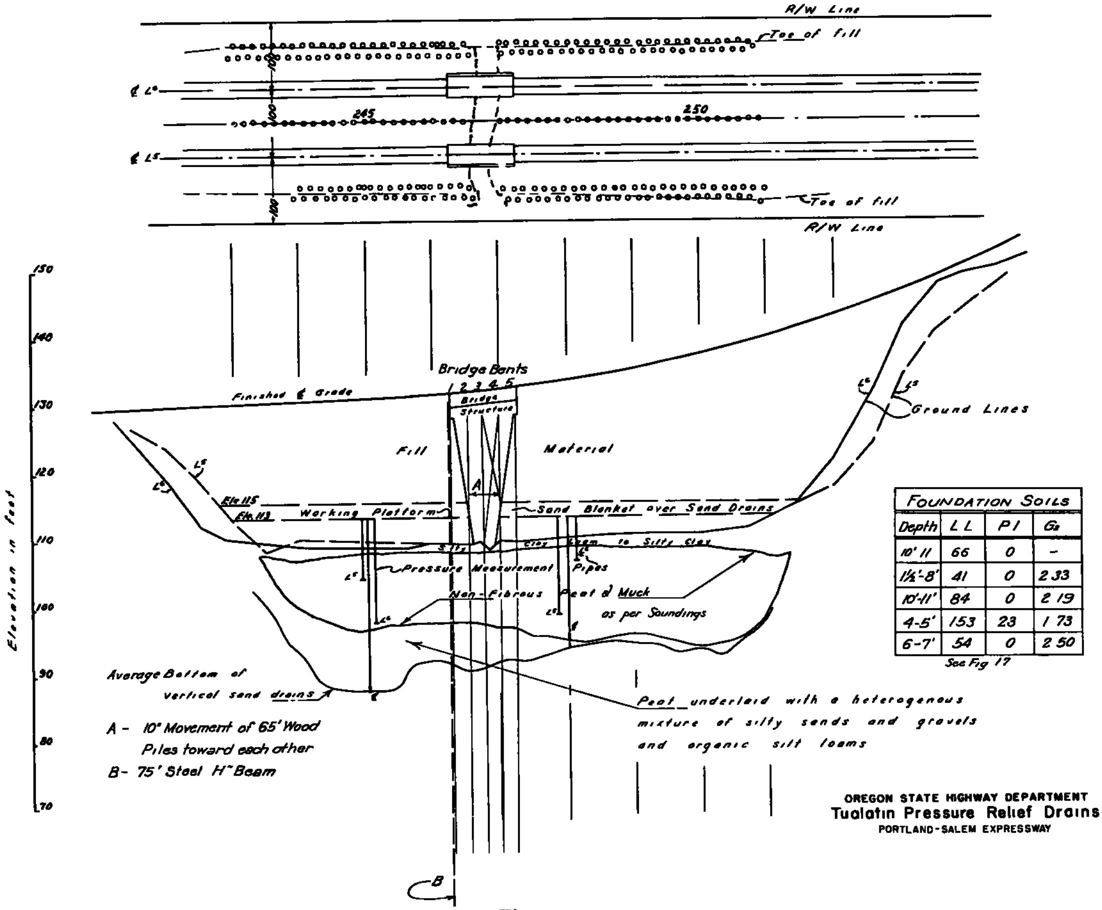
Settlement - Time Curve



Void Ratio - Logarithm Pressure Curve

tion soil was subjected to the usual consolidation tests, using loadings up to 5,000 psf., which approximated the loads of the embankment and the foundation soils in place. The typical results are shown in Figure 1 and Table 1 and 2.

Some method of increasing the stability of the foundation soils and accelerating the normal consolidation was found to be necessary. The most logical approach to increasing the rate of flow of the interstitial water normally held in the organic foundation soils from the pressure areas to the outside seemed to be the installation of vertical pressure relief drains along the perimeter of the embankments. Laminated structure of the organic soil was suspected from inspection of the samples in the field. This, plus the evidence that the preconsolidation load had been light, led to the placing of a double row of staggered, vertical drain holes along the outer toes of the embankments and a single row between the inner toes, as shown in Figure 2 and Figure 3. Double drainage would thus be established through the foundation soils instead of single-faced drainage which an impervious working platform laid upon the natural ground would normally suggest. The outer rows would pick up the pore water forced outward by the greater pressures below the more massive portions of the embankment. There would then be developed an effect similar to a retaining wall between the two rows of the outer drains. Hydrodynamic pressure establishes radial flows to the vertical drains and thus increases the shear



FOUNDATION SOILS

Depth	L L	P I	Gs
10' 11	66	0	–
1½'-8'	41	0	2 33
10'-11'	84	0	2 19
4-5'	153	23	1 73
6-7'	54	0	2 50

Figure 2.

strength of the surrounding soils.

PRESSURE RELIEF SYSTEM

Several types of pressure relief drains were considered. Permeable concrete tile in vertical auger holes was abandoned because of economic reasons, as was the use of concrete sands. An available supply of ¼ in. - 0 crushed rock rejects led to the use of that material in the drain holes and drainage cover. The gradation of the material is shown in Figure 9.

After discussions with contractors, it was decided to install 18-in. diameter holes as spuds and pipe of that size were available. These holes were spaced 15 ft. apart longitudinally in staggered rows 15 ft. apart laterally along the outer toes of the em-

Figure 3. Working platform looking north, summer, 1952. Companion figure to Nos. 15 and 16. Pressure relief drain layout shown by excess ¼ in. -0 drain material.

bankments and 15 ft. apart along the inner toes. The transverse distance between rows of drains was about 110 ft., as shown in Figure 3 and Figure 15. The working platform upon which the pile driving equipment was to operate was a departure from the usual permeable sand blanket, being a dense soil blanket about 3 ft. thick laid over the natural ground surface. This would allow for an initial flow of foundation pore water under

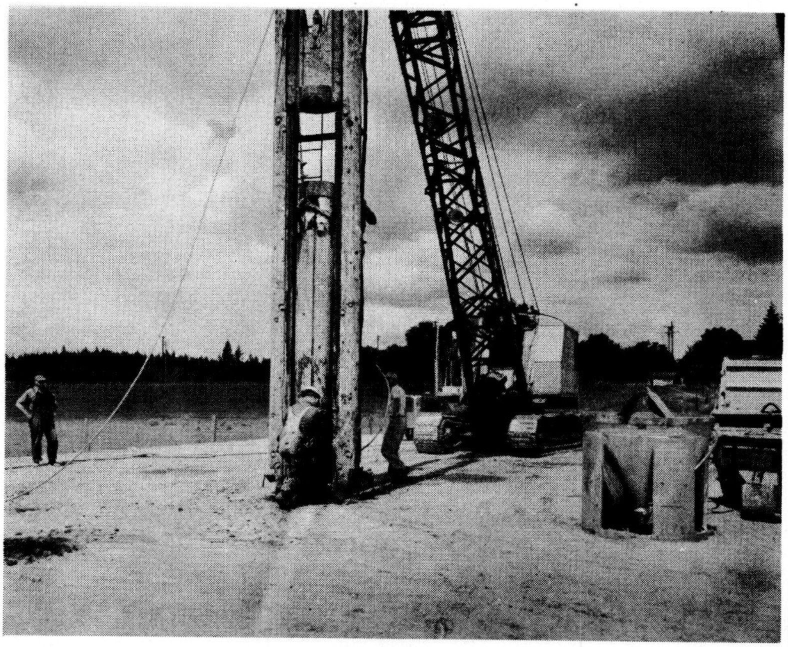


Figure 4. Centering wood spud over pressure relief drain layout. Permeable drain material buckets on right.

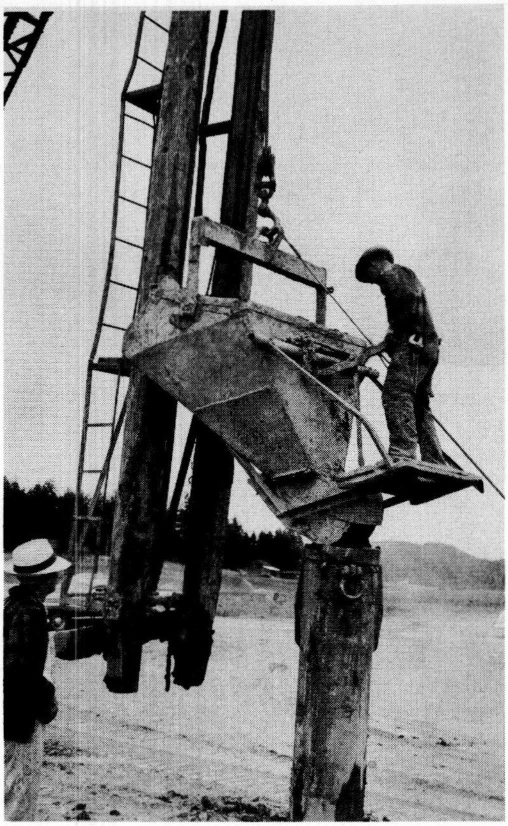


Figure 5. Filling hollow steel mandrel with ¼ in.-0 drain material. Note the control platform welded to the bucket.

dynamic pressures above 1.5 psi., well under the total pressures that were expected in the area covered by the drains.

CONSTRUCTION

When the platform had been brought to grade the drain locations were laid out and the holes punched through the platform and the soil above the organic material by a wood spud spotted and held in place by a swinging lead, as shown in Figure 4. The spud was withdrawn and a hollow steel 18-in. diameter mandrel with closed bottom inserted and driven to firm bottom. With the mandrel on firm bottom, the driving head was removed, the tube filled with ¼ in.-0 material, as shown in Figure 5, and a pressure cap placed, clamped and sealed. About 80 psi. of air was then applied (see Figure 6). As the mandrel was withdrawn from the hole, as shown in Figure 7, the bottom of the tube opened and a continuous column of pervious material remained behind in the soft, compressible, saturated soil.

Upon completion of the pressure relief drain installation, this same ¼ in.-0 material was placed on the working platform over the inner rows of the vertical drains by dump trucks and covered with the silty clay embankment soils. This windrow provided the necessary outlet for the foundation pore water to the creek crossing the mid-point of the embankment. As shown in Figure 15, the platform was extended beyond the toes of the embankment slopes. This appeared to have the affect of forcing more water through the vertical drains than if they had been installed at the edge of the platform.

During construction of the embankments care was taken to limit the height of each lift to six ft. in each 8-hour shift so as not to build up excessive foundation pressures. Upon completion of the fill to grade, there

Figure 6. Clamping the air pressure cap on steel mandrel filled with ¼ in.-0 drain material. Note the driving cap on the ground lower right.

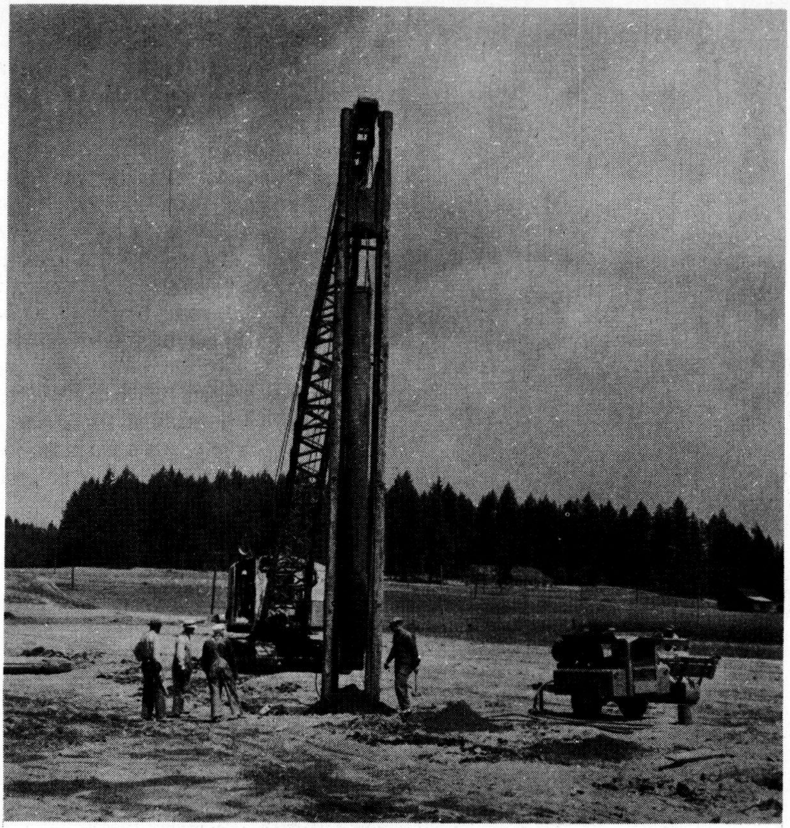


Figure 7. Mandrel pulled showing surplus ¼ in. -0 drain material. Note type of air compressor used.

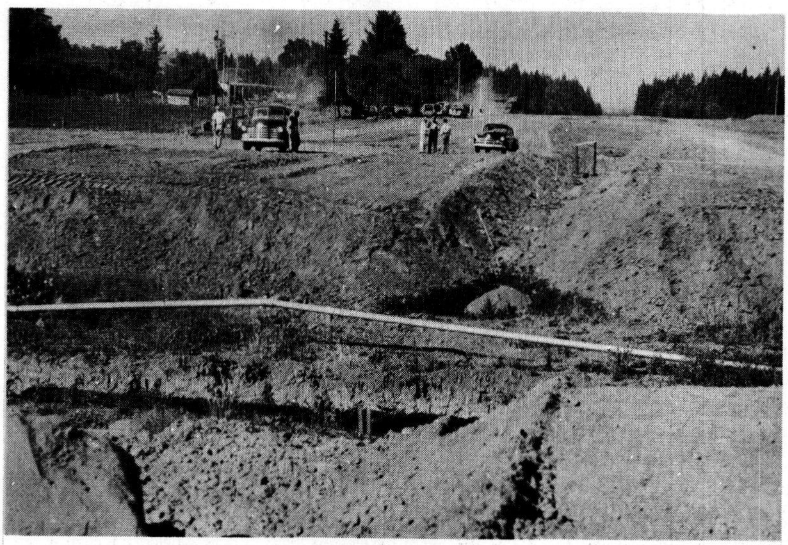


Figure 8. First lift of embankment L 245. Note activity of pressure relief drain, right center, settlement pipe center background and gauge pipe, right.

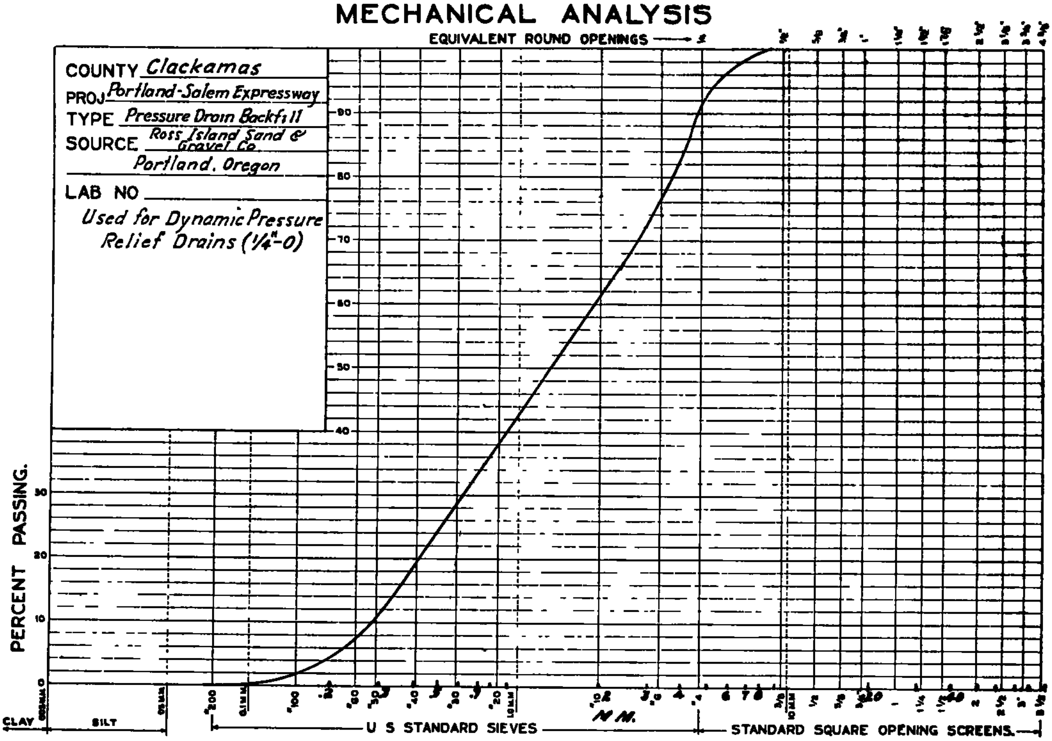


Figure 9.

was a resting period of six months for observation of the settlement. At the end of that period the major portion of the consolidation had apparently taken place, and the embankment was again restored to grade and the gravel base and asphaltic concrete paving laid. Only a minor settlement at the bridge ends has occurred during the past few months.

RECORDS

Continuous records of the behavior of the embankment and its foundation during the construction period were obtained by means of settlement plates, pressure piezometers (Figure 10) and an open standpipe. One pressure gauge was checked against the open standpipe during the early stages of the settlement, as shown in Figure 12. The settlement plates were of the usual design–2 ft. sq. by ¼-in. steel plates on which were welded a 2-in. pipe coupling. As the fill was raised 4-ft. pipe sections were screwed on and the elevations read on their tops. One of the pipes is shown in the background of Figure 8. The distances from the pipe tops to the base plates were accurately measured at least once a week to determine if settlement had pulled the plates from the pipes.

Only minor vertical and horizontal movements of the adjacent meadows were observed during construction of the embankments. Tacked hubs set every 50 ft. on a line 10 ft. out from the toes of the slopes were checked weekly with a transit and level. The greatest movement observed in either direction

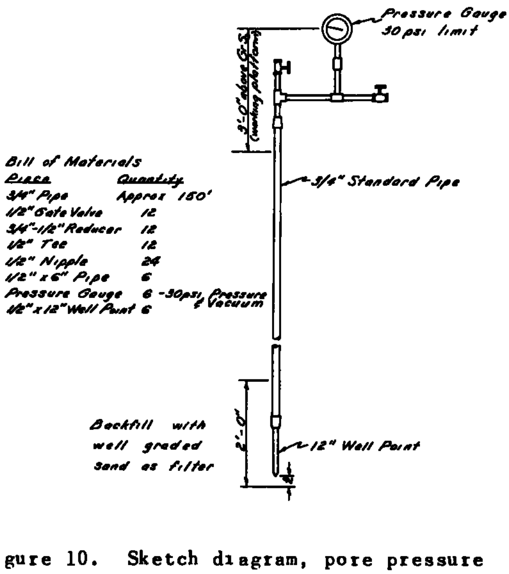


Figure 10. Sketch diagram, pore pressure measuring device.

was upward and amounted to about 0.20 ft. During the period of January to April 1953 this movement disappeared as the meadow surface readjusted itself to within 0.1 ft. of

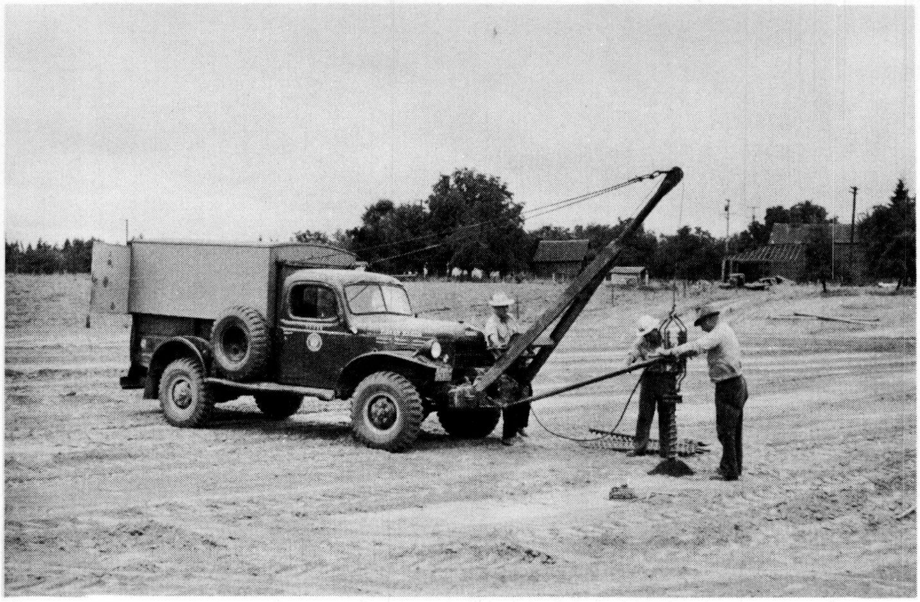


Figure 11. Boring holes for pressure piezometric gauges.

its original elevation.

Settlement and pressure readings were maintained on those plates and gauges functioning from the beginning of the job in September 1952 to May 1953, a period of nine months. By that time the rate of settlement had decreased appreciably and the pressures had leveled out or disappeared, as can be seen in Figure 13 and Figure 14. Sometime during the consolidation of the embankments the center gauges which had been set rather deep in the foundation soils ceased to register. We believe that these piezometers had been forced into a clay layer and that the well points were plugged and unable to transmit pore water pressures. One of the gauges (right, Station 245-L[6]) was removed before the others, as it was felt that the open standpipe could serve to register any fluctuation in the pore water pressures at that point. The pressure change here, however, was so minor that no changes in water elevation could be discerned. By the middle of December 1952, the pressure fluctuations were so small that it was difficult to obtain accurate gauge readings, and they were removed to another project being started. The settlement readings were continued, however, until the grade was finally brought to line and it was necessary to remove the pipes to allow for the construction of the base course.

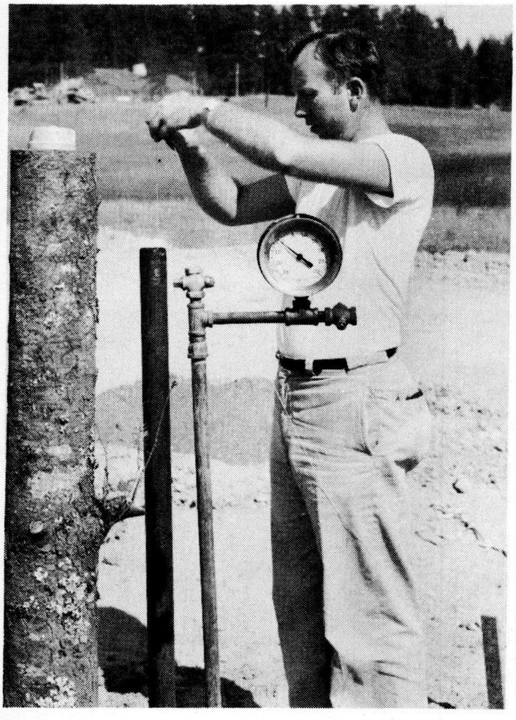


Figure 12. Checking water rise against pressure piezometric gauge. Vacuum of ± 1½ inches initially applied. Gauge reads in pounds per square inch plus and inches of mercury minus.

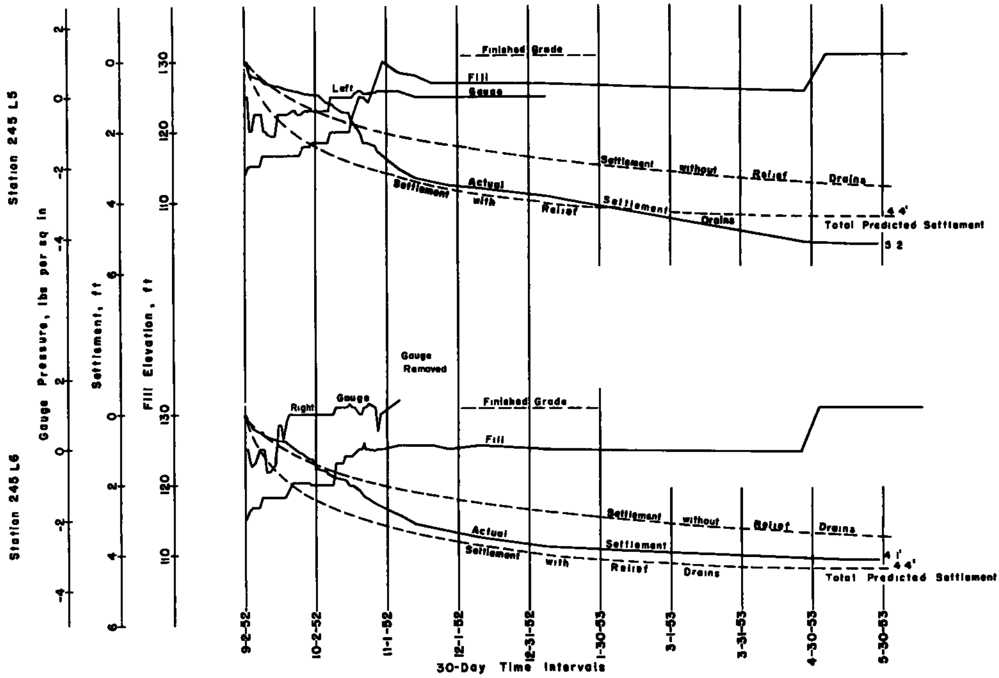


Figure 13.

SETTLEMENT ANALYSIS

The Terzaghi theory of consolidation is usually applied to settlements of structures founded on clay soils. It was not certain how close this theory would forecast the em-

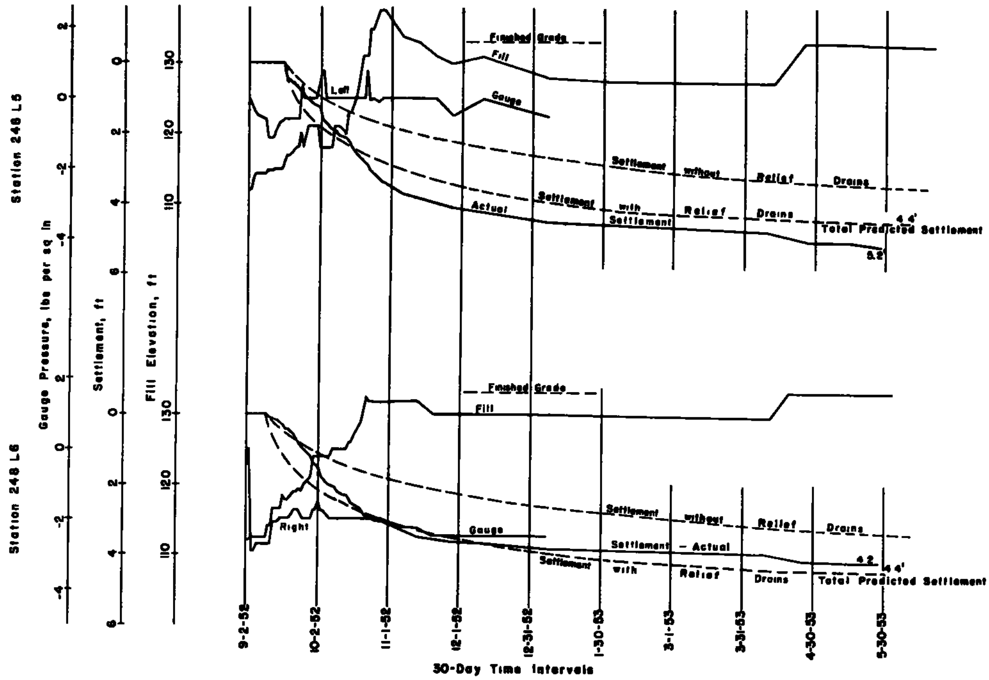


Figure 14.

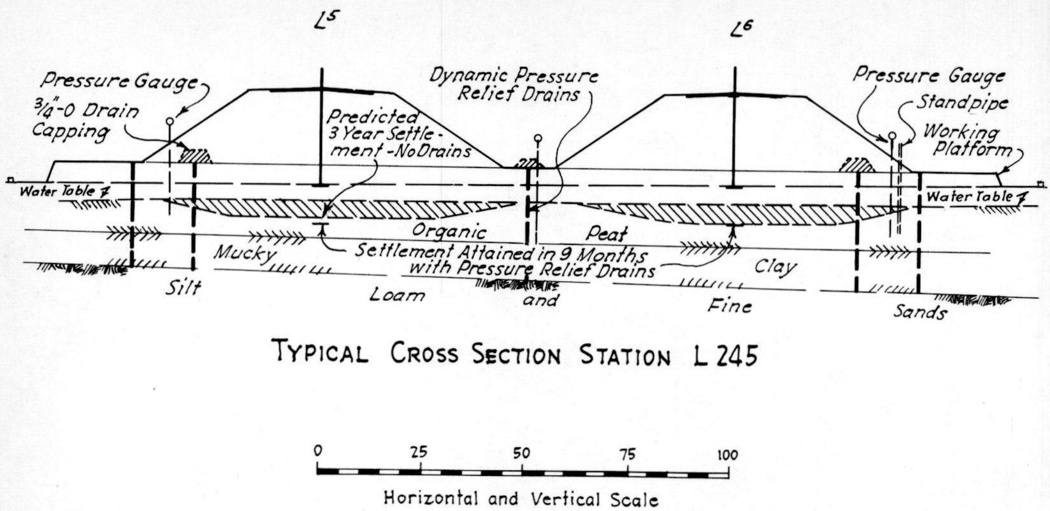


Figure 15.

bankment settlement over the organic soils found on this project. The computations follow principles cited in several texts and reports (4, 5, 6) and follow, in general, a method used successfully on other highway embankment problems (7).

The time rate of consolidation factor "T," reflecting a double-faced or "open" type of drainage, appears to represent the settlement that has taken place, Figure 13 and Figure 14. The selection of this factor was influenced by principles suggested by Terzaghi (8, 9) and Spangler (6). It may be, however, that other factors not uncovered by this investigation and the tests made on the foundation soils, such as the relatively shallow depth of the firm bottom and the existence of horizontal drainage channels within the organic foundation material, assisted in the rapid settlement of the embankments.

FOUNDATION MOVEMENT IN UNDRAINED AREA

The embankment ends at the bridge structure shown in Figure 2 were built without the

Figure 16. Looking north along west embankment just prior to placing asphaltic concrete paving. See Figures 3 and 15.

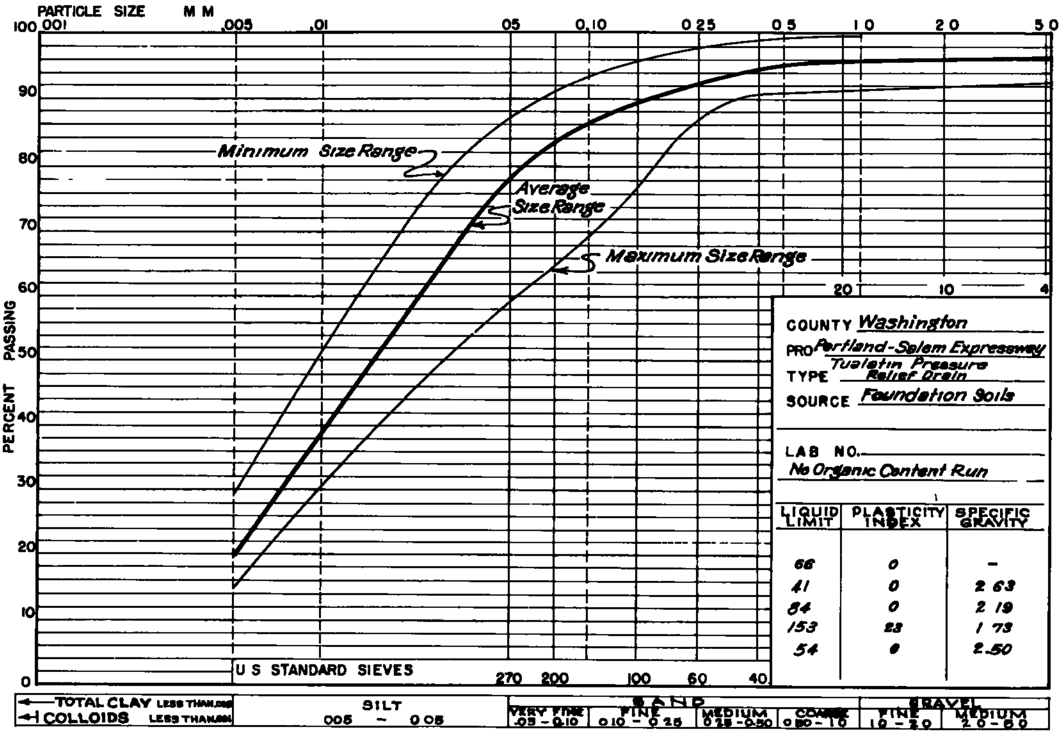


Figure 17.

protection of pressure relief drains. During the driving of the 65-ft. piling supporting the structure some difficulty was encountered in obtaining the required 21-ton bearing capacity, particularly in Bent number 1. However, driving tests made on one of the piles in this group the day following the setting of the piles failed to move it. The piles "froze" in the underlying soils. All of the bridge piling was driven to full length with an average cut-off length of about 3 ft.

Within 7 months after the piles had been driven horizontal movement of the piles in Bents 2 and 4 was observed. The tower bracing between Bents 3 and 4 warped out of line, showing that the piles were being bowed toward the center of the opening. A further investigation, made about 10 months after the bridge had been built, showed that lateral pressure was developing under the precast concrete beams of Bents 1 and 5. One of the pilings in Bent 1 which had shown signs of being broken during driving had settled away from the precast concrete beams about 3 in. and had been pushed about 2 in. toward the center of the bridge. A 75-ft. steel H beam was driven adjacent to the end of the bridge, a bracket welded to it and steel and lead wedges forced between the bracket and the bridge beam to take the place of the wood piling.

This slow, plastic movement of the foundation soils has continued up to date, resulting in a total movement of the piles in Bents 2 and 4 of about 10 in. toward each other. There has been no noticeable settlement of the structure, nor is there any anticipated.

This behavior of the piling emphasizes the stability of the balance of the project where the pressure relief drains have been installed. We believe that if the pressure relief drains had been carried around the ends of the embankments under the structures, little or no distortion of the foundation soils would have taken place.

CONCLUSIONS

A view of the completed project is shown in Figure 17. The knowledge of soil mechanics relating to the consolidation of organic soils, such as those found in this shallow valley crossing, has not been advanced far enough to allow more than an intelligent guess as to what the rate and amount of settlement would have been on this project. We believe that the close correlation between what was predicted and what actually happened is of interest to the highway engineering profession.

Several conclusions relevant to this type of problem appear obvious from the behavior of the foundation soils on this project:

1. Hydrodynamic relief drains will accelerate settlement of an embankment built upon unstable foundation material.
2. Perimeter treatment of the embankment with drains will provide sufficient increase in shear strength in the foundation material so that lateral flow can be largely prevented.
3. Low level working platforms provide early release of the hydrodynamic pressures through pressure relief drains and that this early release of pressure will provide a stable foundation through the first stages of construction.
4. Where embankment toes are not protected by drains the slow plastic flow of the foundation soils can affect the stability of the embankment and any structure placed upon it.
5. Low level impervious platforms serve as a water cut off and deflect water to drain system. They also provide some structural stability not present in sand blankets.

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Sand Drains for Embankment on Marl Foundation

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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.

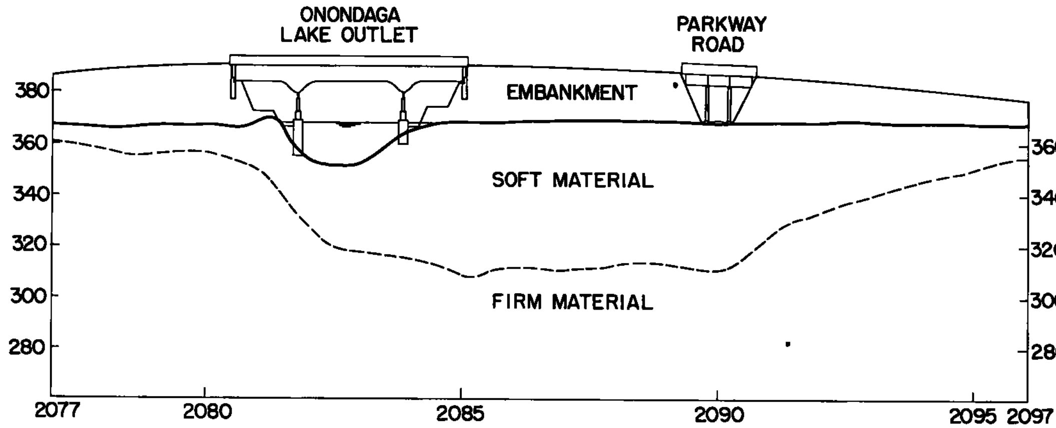

Figure 1. Problem requirements.

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.

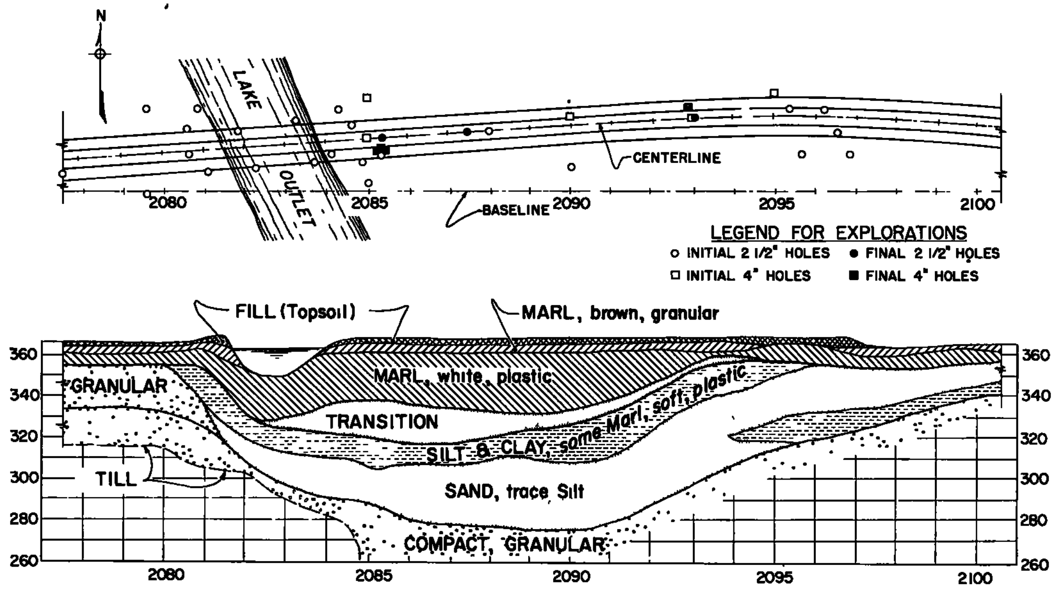

Figure 2. Plan and profile of line.

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.

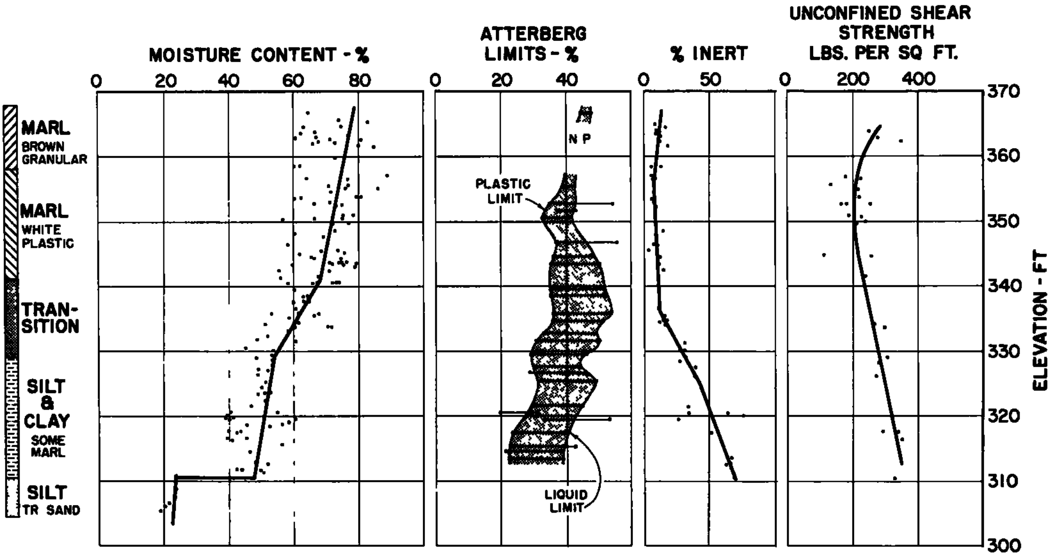

Figure 3. Soil characteristics.

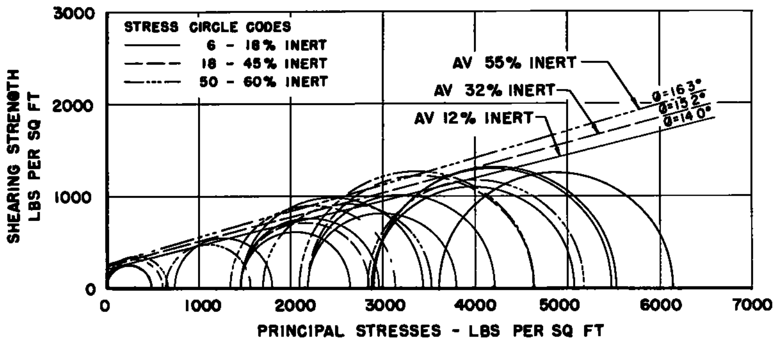

Figure 4. Typical triaxial-shear strengths.

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

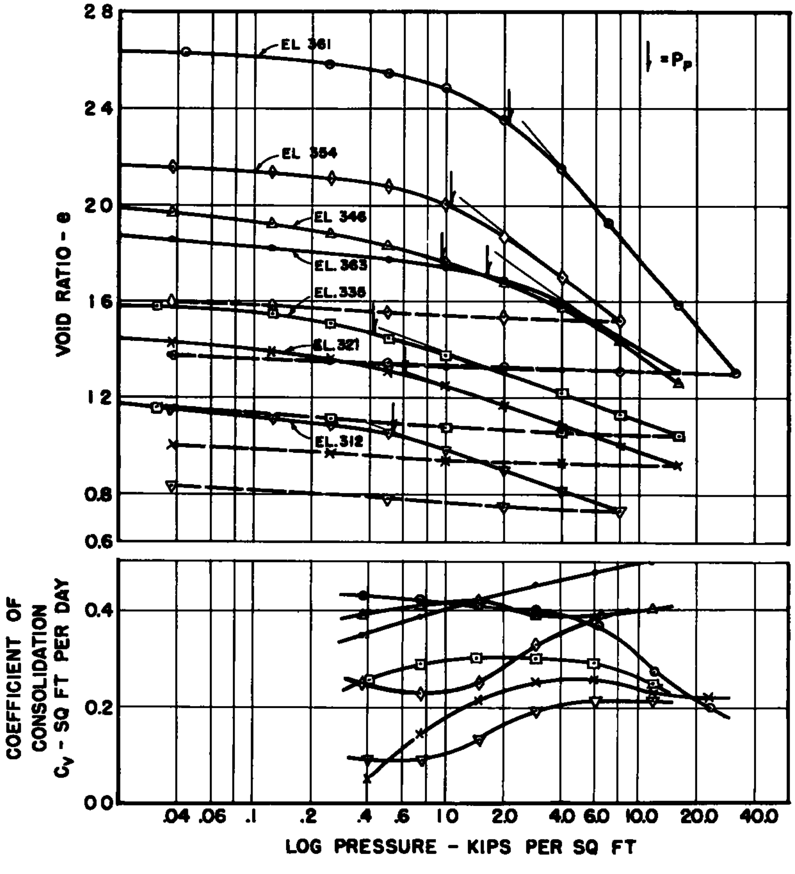

Figure 5. Consolidation test data.

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

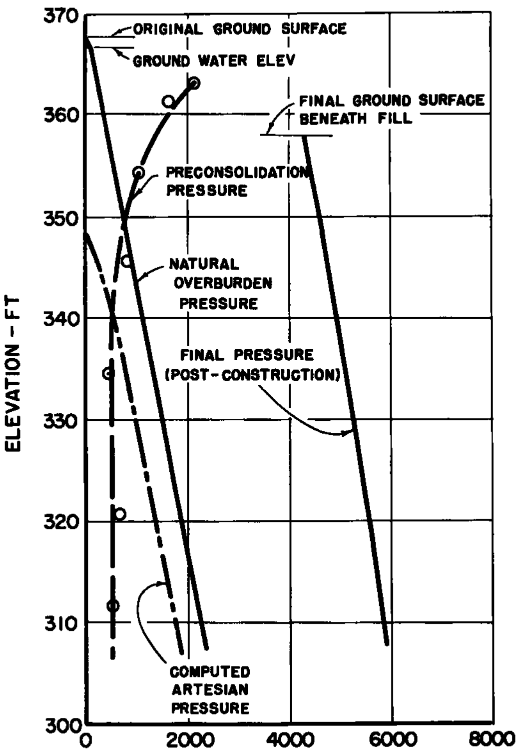

Figure 6. Depth versus pressure distribution.

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-

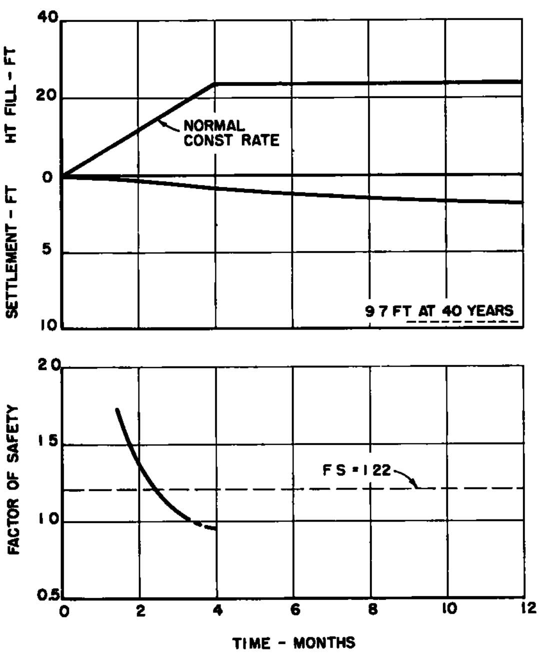

Figure 7. Normal construction rate, no foundation treatment.

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.

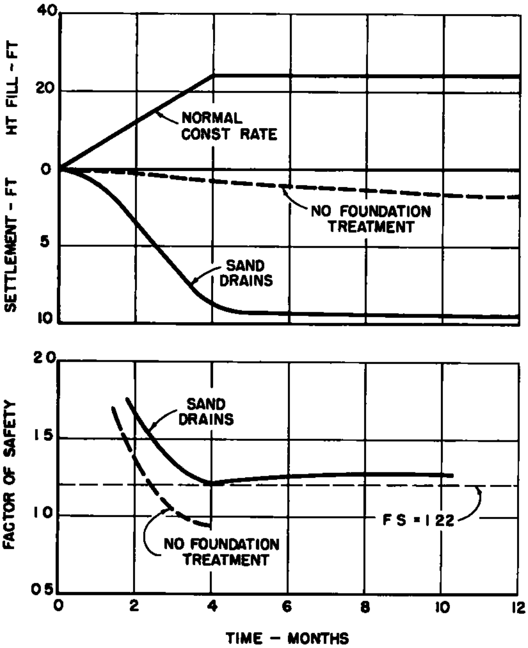

Figure 8. Normal construction rate, sand drains.

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.

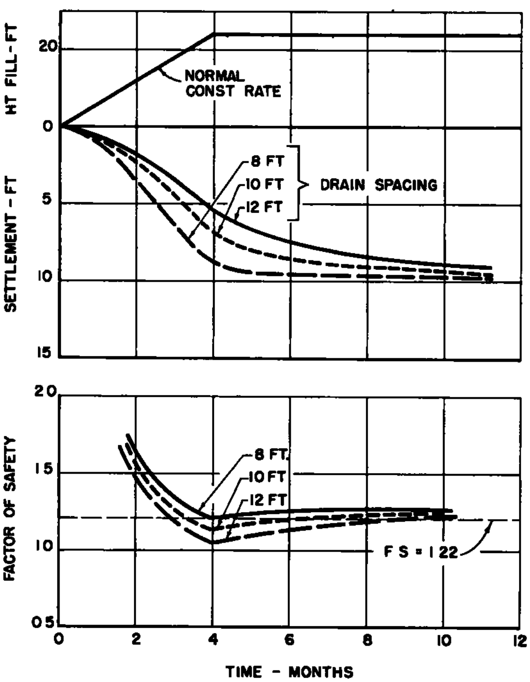

Figure 9. Normal construction rate, sand drains, various spacing.

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-

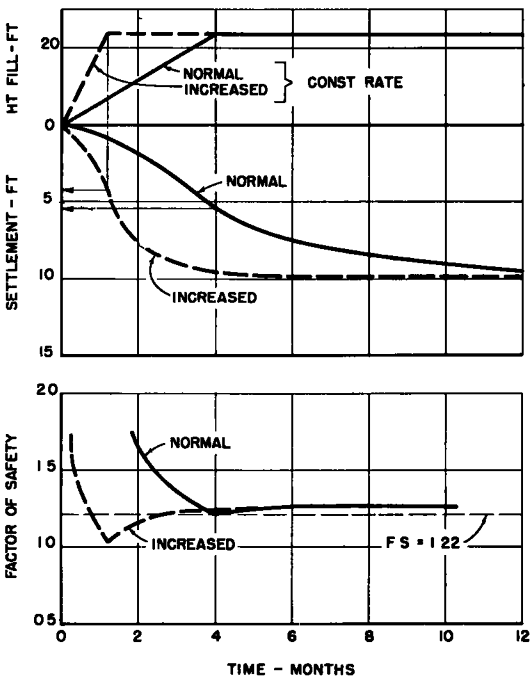

Figure 10 Sand drains, increased construction rate.

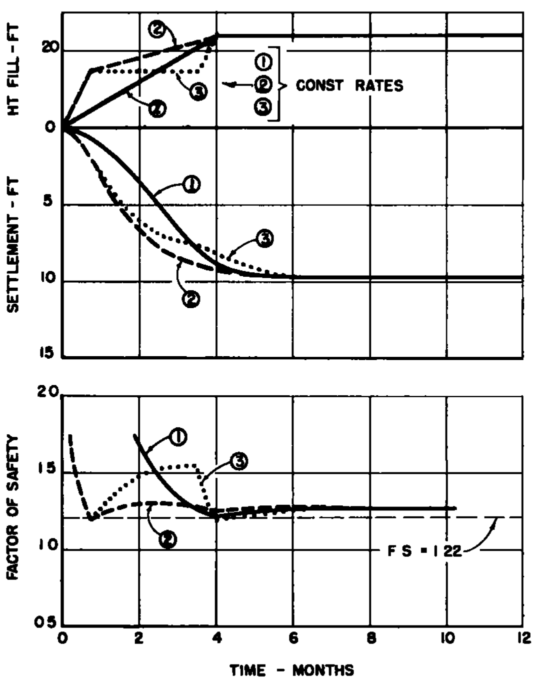

Figure 11. Sand drains, various construction rates.

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.

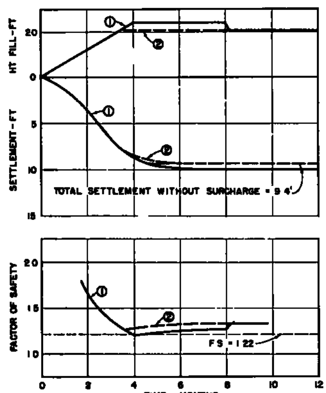

Figure 12. Surcharged embankment.

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 $2\frac{1}{4}$ 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.

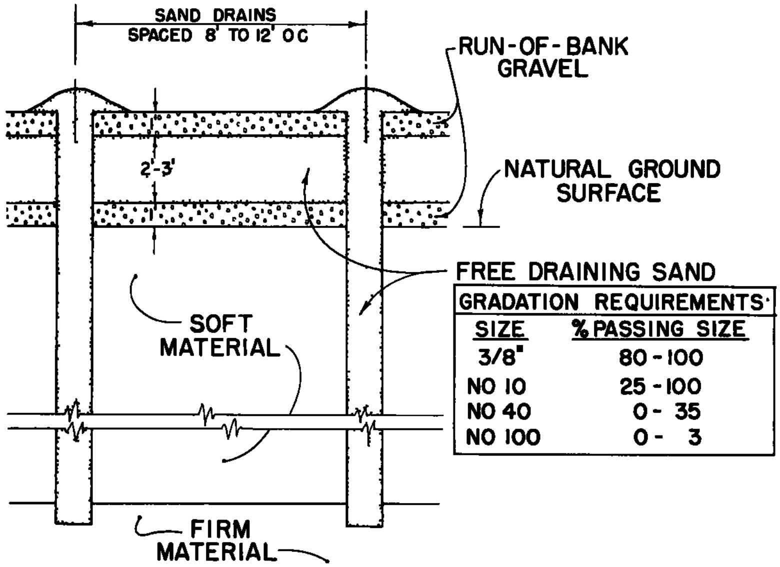

Figure 13. Drainage blanket details.

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

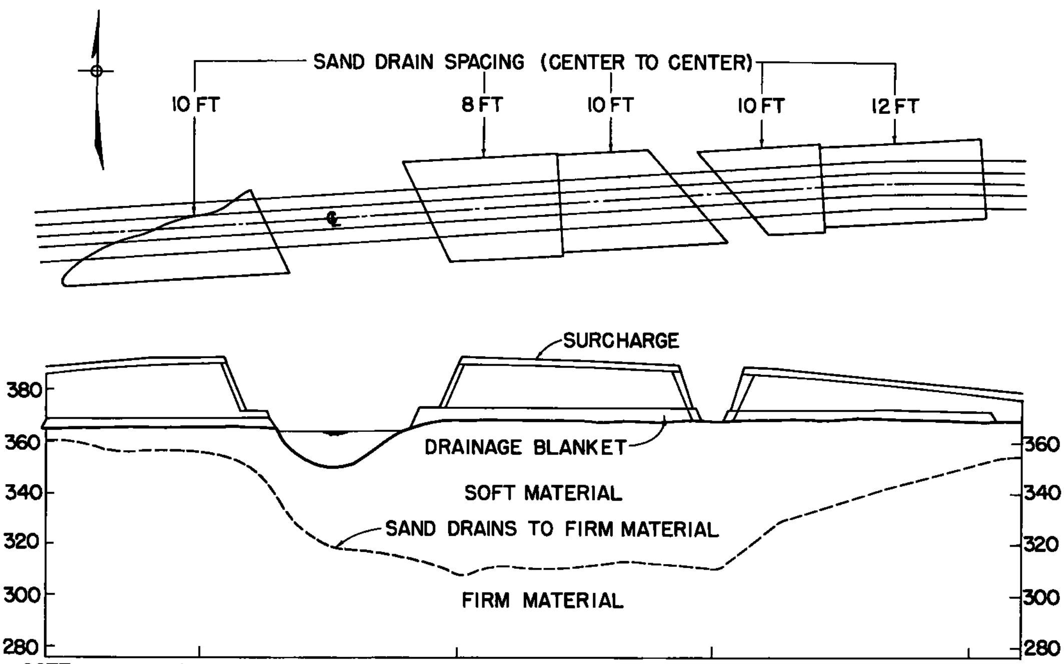

Figure 14. Final design, plan and profile.

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.

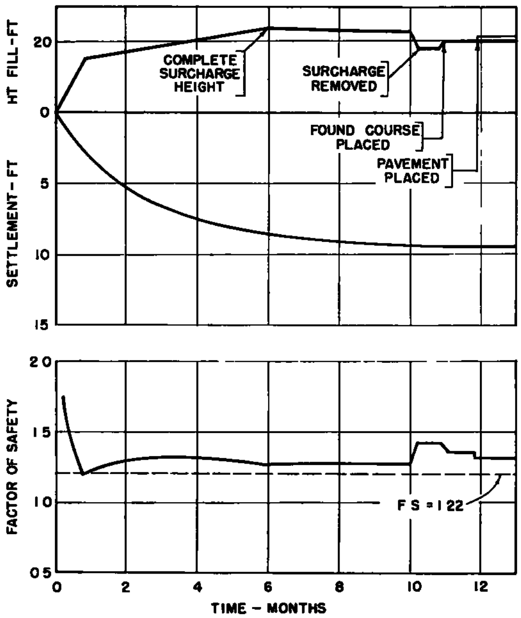

Figure 15. Design conditions sand drains, surcharged restricted construction rate.

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

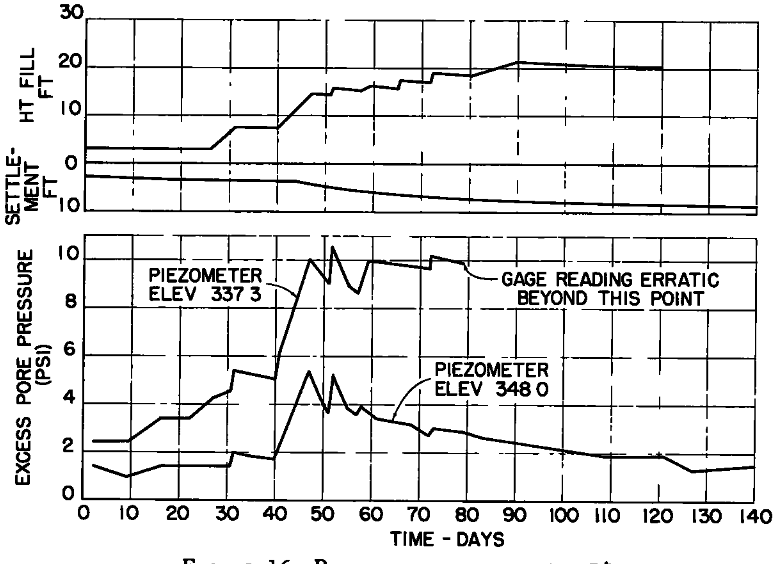

Figure 16. Pore-pressure measurements.

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.

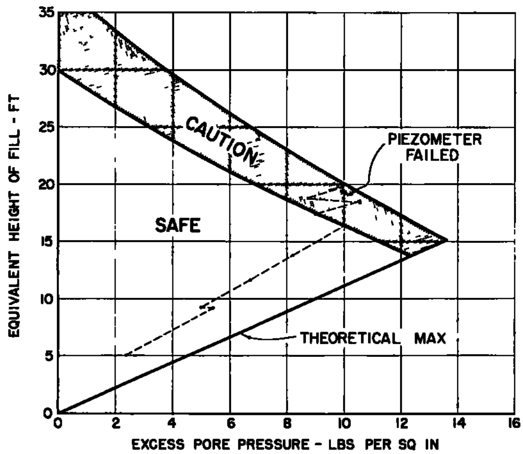

Figure 17. Excess pore-pressure control chart.

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.

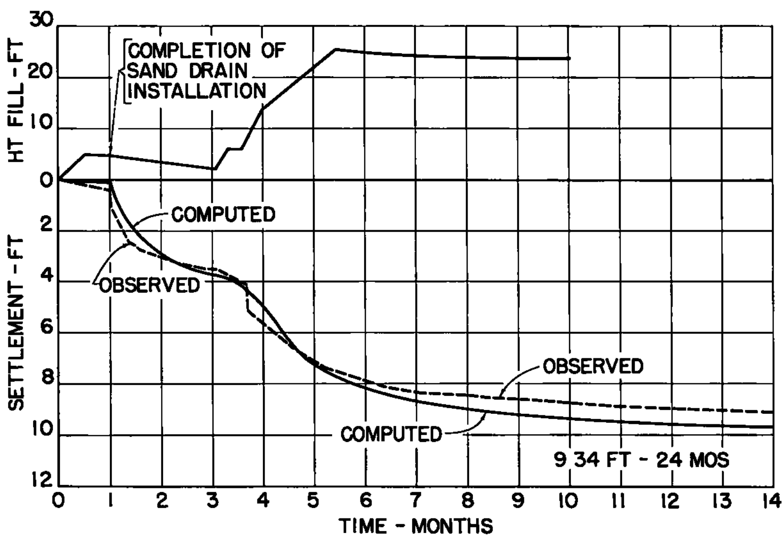

Figure 18. Settlement, computed versus observed (Station 2085 + 70).

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

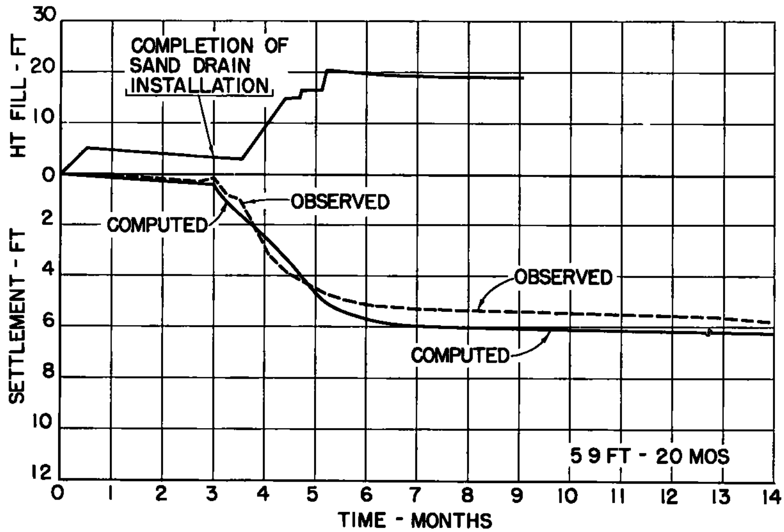

Figure 19. Settlement, computed versus observed (Station 2091 + 55).

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-

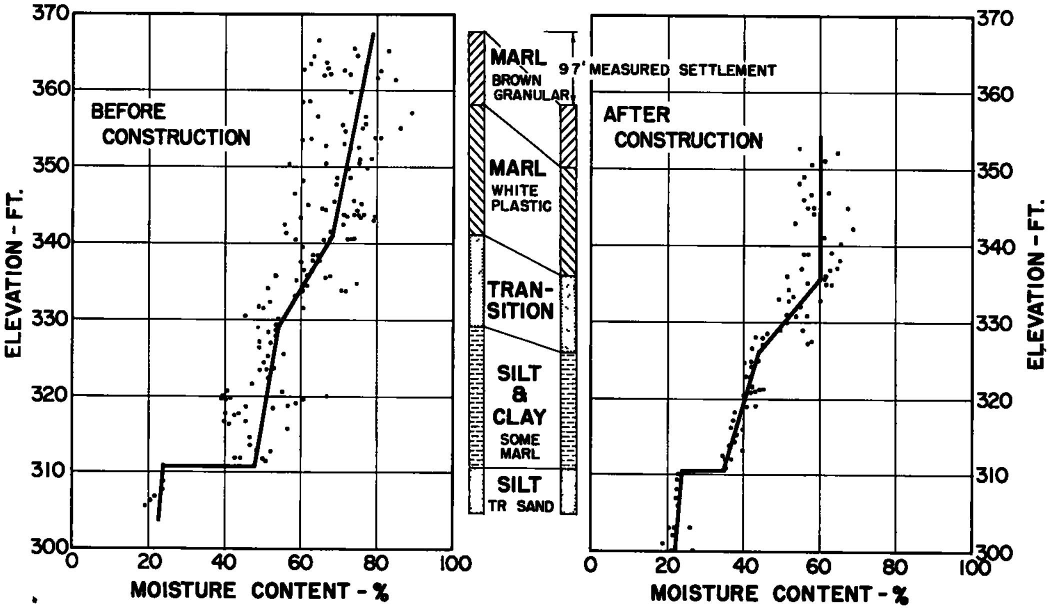

Figure 20. Moisture content before and after construction.

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

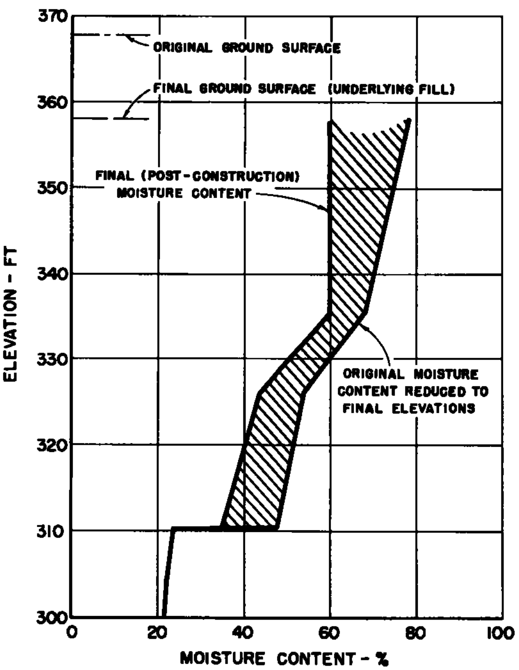

Figure 21. Moisture content before and after construction, corrected for settlement.

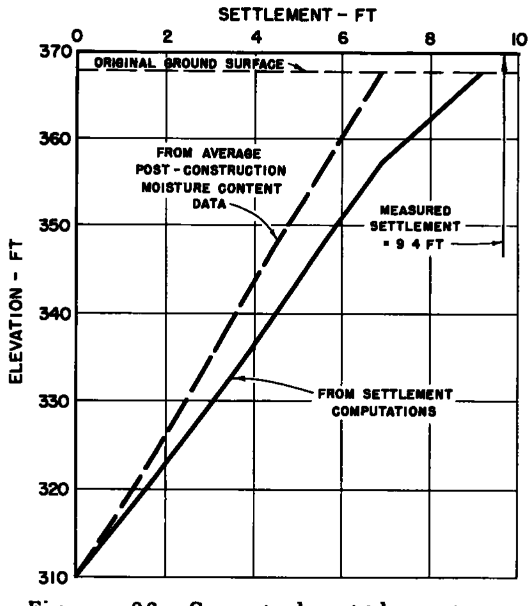

Figure 22. Computed settlements.

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.

Stabilization of Marsh Deposit

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This paper presents the method of stabilization of marsh deposits at Swimming River, Red Bank, N.J. with emphasis on the use of vertical sand drains. The aim of the design was to accomplish the predicted settlement within a specific construction time and to insure stability of the embankment against shear failure at all times.

This paper describes construction materials and order of procedure used in the stabilization work and such control devices as piezometers, embankment control stakes, and settlement platforms. During construction, a simple and quick method of determining the approximate percent consolidation accomplished within the compressible soil mass in each stage of loading was used.

This report indicates the workability of the sand drain method of stabilization at Swimming River where highly compressible soil is located. Settlement of great magnitude was accomplished in a relatively short time with no shear failure. The correlation between the predicted and actual settlement in most areas indicated that this method of stabilization with close control of field work can be depended upon to produce satisfactory and predictable results.

Drawings show a typical soil profile, limits of stabilization, a typical design plan including the layout of the control devices, a typical half-section of the embankment, details of the control devices, a typical field observation record, field control chart, and a typical construction profile including actual and predicted settlements, etc.

● SECTION 7 of the Garden State Parkway starts at the Raritan River and runs southeast and then south for approximately 30 miles. Two miles southwest of Red Bank, New Jersey, in the central portion of the section, the parkway crosses the Swimming River, a tributary of the Navesink River.

SOIL CONDITION

Borings taken at the Swimming River site disclosed an upper layer of recent marsh deposit, from 10 to 45 feet deep, composed of dark brown organic silt intermingled with root mat and peat. The spoon sampler used in the borings was pushed down by hand through this upper layer, indicating extreme softness and high compressibility. Some pockets of sand and a little gravel were found at various depths in this layer. The marshy deposit is underlain by a mixture of sand, silt, and clay of variable relative density. Underlying this layer is the Navesink formation, a dark green glauconitic marl with a shell base.

Some borings indicate that the Wenonah and Mount Laurel sands, fine and micaceous, underlie the marl. Geologic maps indicate that the deep bed rock consists of sands and glauconitic clays of Upper Cretaceous Age. They are a conformable series striking approximately N 55 deg. E and dipping southeastward at a rate of about 33 feet to the mile. Beyond the area of stabilization, the soil consists mainly of Red Bank Sand, a coarse rusty sand up to 100 feet thick.

A typical soil profile along the centerline of the southbound roadway is shown in Figure 1.

PHYSICAL CHARACTERISTICS OF SOIL

Eight undisturbed samples of the compressible materials in this area were classified and tested. The test results are listed below:

Natural water content, percent of dry weight	191 to 570
In-situ void ratio	5.2 to 6.8, one sample 12.3
Specific gravity	2.08 to 2.59
Dry unit weight	9.9 to 26.0 pcf.

Horizontal permeability at 20 C.	4×10^{-8} cm. per sec.
Ratio of horizontal to vertical permeability	1.2 to 1.7
Unconfined compressive strength at the natural water content	0.1 ton per sq. ft.

METHOD OF STABILIZATION

To construct the fills safely on top of this compressible material, as high as 23 feet in places, and to minimize the future maintenance costs, means had to be taken to stabilize the soil during the construction period. Several methods of stabilization were considered, including deep well pumping, stabilizing berms as counterweights, displacement possibly aided by surcharge and blasting, well-points, excavation and backfill, and vertical sand drains.

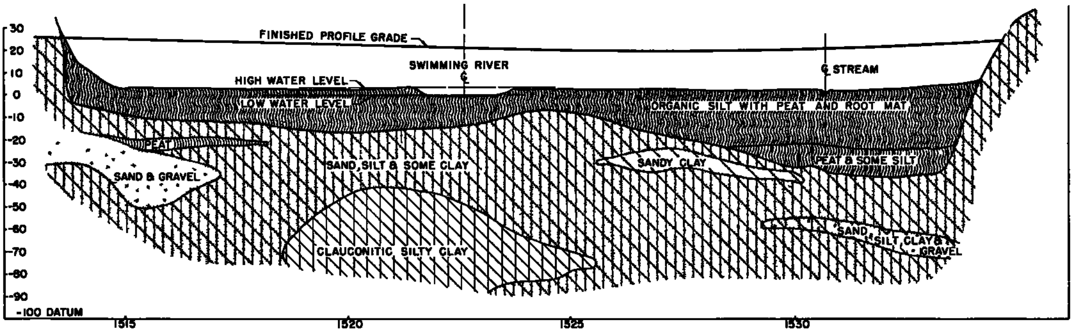


Figure 1. Soil profile, southbound roadways.

Because of the impervious nature and depth of the compressible material, and great width of the fill, all but the latter two methods were eliminated as impractical and too costly. Alternate designs were prepared, Alternate A calling for the use of vertical sand drains, with excavation in the vicinity of the new bridge, and Alternate B, excavation of all the compressible material. Excavation in the bridge area was included in Alternate A so that construction of the piers and abutments would not be delayed.

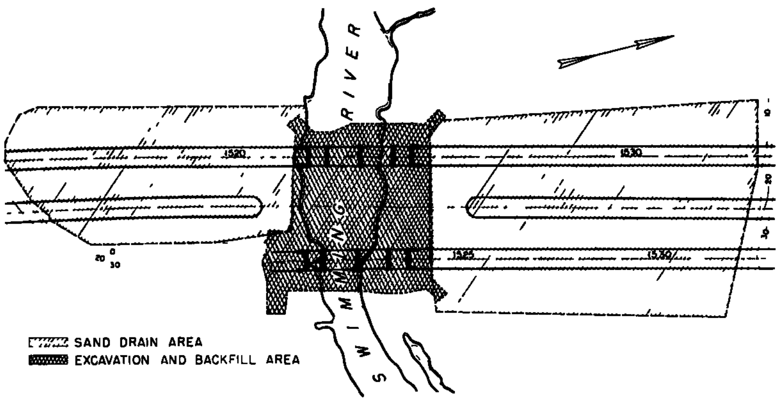


Figure 2. General plan.

Contractors were invited to bid on Contract 19, which includes the alternate designs of the stabilization work and it was found that the contract job could be done with a saving of approximately $370,000 in favor of the bid with Alternate A.

A general plan showing the limits of the excavation and sand drains is given in Figure 2. In this paper only the part of stabilization by means of vertical sand drains is presented in detail.

DESIGN

The aim of the design was to determine a proper spacing and size of vertical sand drains, a minimum height of overload, extent of counterweights and a feasible construction schedule in such a combination that the embankment would be safe against shear failure (commonly called "mud waves") at all times and within a specified construction time be completed, leaving a permissible residual settlement in the future. To fulfill these two requirements, settlement computation and slope stability analyses were made at each stage of loading under various possible combinations of design. The theory of sand drain design is presented in detail in various texts and papers. Therefore, it will not be discussed here.

The following basic data and assumptions were used in the design:

1. From a series of consolidated-quick triaxial shear tests (Qc tests) on the compressible soil samples: (a) the average in-situ shear strength=180 psf. and (b) the design apparent friction angle=17 deg. To reduce the hazardous outward "creep" or progressive deformation of the compressible material which may induce shearing of the drains under the embankment load, and also to allow the possible reduction of strength due to remolding effect of sand drain driving or other disturbance during construction, approximately 60 percent of the peak shear strength as found in the Qc tests under each consolidation

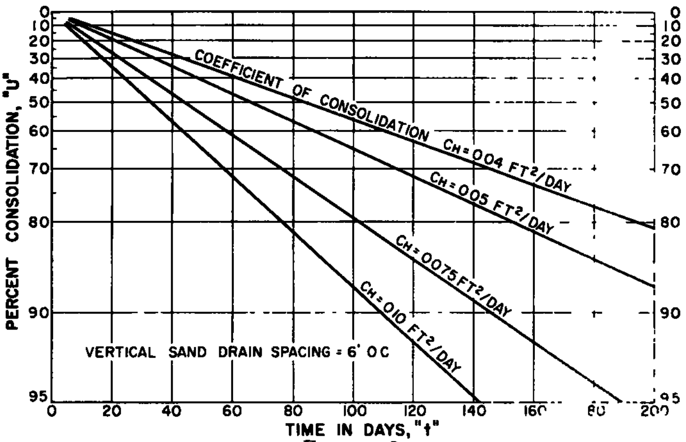


Figure 3.

pressure was used. This design strength corresponds approximately to 2 percent axial strain. The minimum allowable factor of safety against shear failure used in this design was 1.2.

2. The preconsolidation load=500 psf. The design compression index C_c=3.0.

3. Due to erratic variations in stratification of the deposits in that area and the remolding condition of the soil adjacent to the drain periphery, the horizontal coefficient of consolidation, C_H, was assumed to be equal to the vertical coefficient of consolidation, C_v, although the horizontal permeability was found to be slightly higher than the vertical permeability. The design value was 0.10 feet^2 per day at and less than the preconsolidation load, decreasing to 0.025 feet^2 per day at two tons psf. A typical design curve for time versus horizontal percent consolidation for various horizontal coefficients of consolidation, C_H, and for six feet on centers vertical sand drain spacing, is shown in Figure 3. In this design, the consolidation due to combined horizontal and vertical flow was computed.

4. In the design the effect on the consolidation process of the following factors was not considered: (a) Change of permeability of the soil adjacent to the drain periphery due to remolding or smear effect of sand drain driving. (b) Resistance to flow of the expelled water up the drain, and any ineffective vertical drainage as a result of disturbance and distortion of the drain. (c) Soil displacement and possibly heave by the volume of the drain, if closed end mandrel is used. (d) Shearing strain developed in the soil mass by differential settlements.

By recognizing the slab action of the compacted embankment fill, the change of its uniform load distribution over the zone of influence for each drain due to any differential settlement and unequal strain within the soil mass was neglected. Theoretically, with free strains permitted, the soil immediately adjacent to the drain should consolidate faster than the soil farther away from the drain, thus resulting in a greater settlement near the drain. However, this greater settlement is somewhat restrained by the combined effect of the relatively stiff sand column, although in a loose state and the slab action of the compacted embankment fill.

A system of 20-inch-diameter vertical sand drains was designed with a square pattern, 6 feet on centers under the main fill, 7 feet under the slopes of the main fill and 9 and 10 feet beyond the slopes and under the interior counterweight. To insure the stability of the embankment against shear failure, under the required rate of loading schedule, an exterior counterweight was used for a distance of approximately 90 feet at a thickness of 6 feet. To provide horizontal drainage of pore water from the sand drains, a blanket of free draining sand, three feet thick, was placed over the marsh.

A network of 6-inch and 8-inch corrugated-metal-pipe underdrains, some perforated,

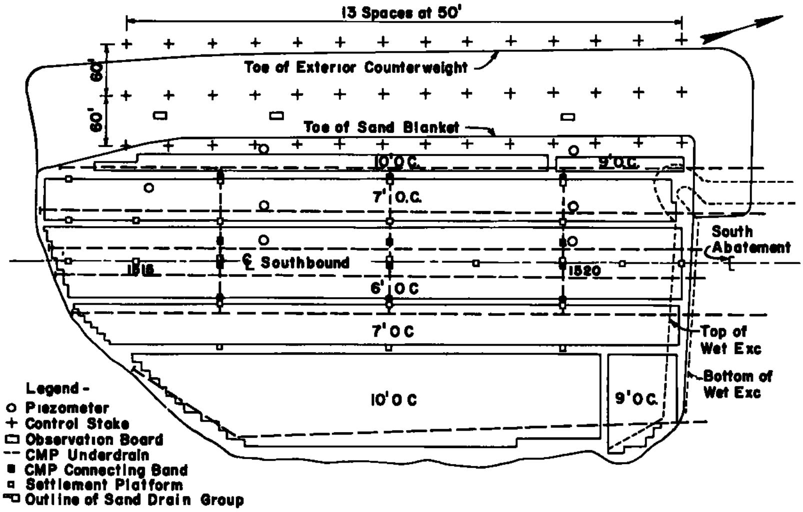


Figure 4. Typical design plan.

were placed in stone trenches which were installed within the sand blanket. With this design the embankment and overload, a maximum total of 50 feet of fill, could be placed in a period of four months without shear failure and in the meantime the desired amount of the ultimate settlement could be reached.

To determine the actual progress of consolidation within the soil mass during construction, the design had 94 settlement platforms placed over the stabilization area spaced longitudinally 50 feet to 100 feet apart. At approximately every 100 to 150 feet, five platforms were placed transversely to determine settlement cross section. The settlement platforms, each three feet square with a 2½ inch marked vertical pipe, also served to measure the amount of fill actually placed. Thirty-two piezometers were set at various depths in the compressible material, placed to indicate the pore pressure conditions over the entire area, particularly in the anticipated critical sections.

To detect any lateral displacement of the embankment during construction, three rows of embankment control stakes, 60 feet apart, were placed parallel to the main fill. The stakes were made of timber and consisted of a vertical post, 2 in. x 4 in., and a horizontal cross piece, driven several feet into the marsh surface. The stakes were fitted with facings so that they could be checked for vertical and horizontal movements.

Vertical movements were determined from elevation readings. Horizontal movements were determined by aligning a transit between two fixed points and sighting on the graduated cross piece affixed to the stake.

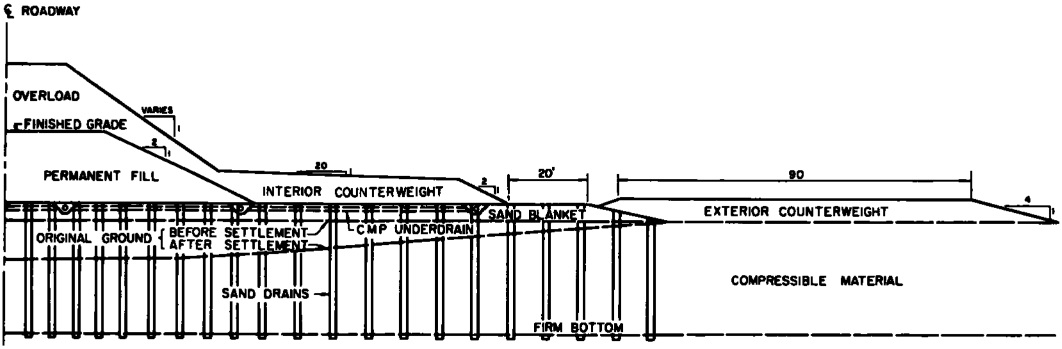


Figure 5. Typical half section.

A typical design plan showing the control devices is given in Figure 4 and a typical half-section is shown in Figure 5.

MATERIALS

The specifications for the sand drain and sand blanket materials call for free draining materials containing a minimum of fine particles. The gradation of the materials should be such that it will offer least resistance to the water expelled from the surrounding compressed soil and yet will allow the adjacent soil to be washed into the interstices of the material, thus obstructing the flow. Well-graded sand and gravel is usually not desirable due to its relatively low porosity.

Figure 6 shows the specified gradation limits of the sand drain and the sand blanket materials.

All the embankment borrow materials which will eventually settle below the ground water table were sand and gravel having less than ten percent fines passing the No. 200-mesh sieve (Class III borrow).

Embankment materials, including overload, on top of the Class III borrow were sand and gravel containing fines less than 35 percent passing the No. 200-mesh sieve (Class II borrow). The determination of the percentage fines passing the No. 200-mesh sieve was made by wet sieving in accordance with the AASHO standard testing procedure. Class III borrow was also used for interior counterweight and the working table on top of the sand blanket. Unclassified materials were specified for the exterior counterweight.

CONSTRUCTION

In general, the order of procedure used in the stabilization work at Swimming River was as follows: (1) clearing and grubbing; (2) temporarily diverting the existing river for excavating the unsuitable material at the new bridge location; (3) excavating and backfilling in the new bridge area; (4) returning the river to its original channel; (5) placing the sand blanket and the working table; (6) installing the vertical sand drains; (7) placing the exterior counterweight; (8) installing the pipe underdrains in the sand blanket, piezometers, settlement platforms and embankment control stakes; (9) placing the embankment fill (Class III and II borrow) and the interior counterweight by dry fill method; (10) placing overload (Class II borrow); and (11) removing overload using the material to build the future reversible lanes between the northbound and southbound roadways and to flatten the side slope of the embankment.

The first order of work under the contract was clearing and grubbing of tree stumps. To support the sand blanket and construction equipment, the existing root mat was not disturbed.

Two dikes were then built across the existing river bed and a 40 foot diversion channel carried the river north of the bridge area. Placement of the sand blanket and installation of sand drains was started in the section south of the river during the excavation operation. To avoid displacement of the underlying compressible soil, the sand blanket

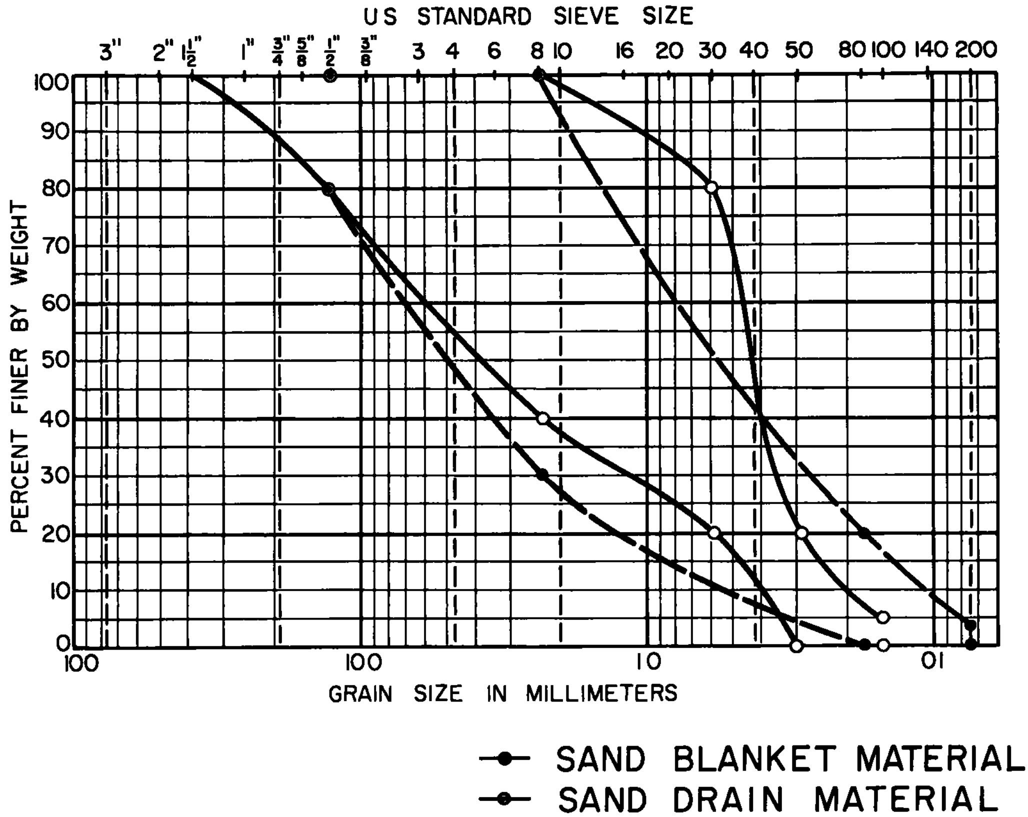


Figure 6. Gradation limits.

material was spread and pushed over the marsh by light bulldozers. After the excavated area was backfilled, the river was returned to its original channel and the placing of sand blanket and installation sand drains continued.

During the placement of the sand blanket, it was found that high initial consolidation brought the level of water above the blanket. To provide a working table above the water surface for the rigs to drive sand drains, as much as 6 feet of gravelly material on top of the blanket was required. A total of 434,000 linear feet of vertical sand drains averaging 32 feet in depth were installed by driving a closed-end, 20-inch mandrel with a removable bottom. A steam operated, 7-ton hammer was used to drive the mandrel until 10 blows per foot were required. The mandrel was filled with selected sand through a "skip" and then gradually withdrawn while steam pressure applied on top of the sand held the sand in place. To overcome arching of sand in the mandrel and to insure proper density of the sand in the hole, an initial pressure of 100 psi. was applied.

After the drains were installed, the corrugated-metal-pipe underdrains were placed in stone trenches in the sand blanket to provide a free path for the excess pore water to be discharged directly into the river. The exterior counterweight was placed during this stage of construction so that its full weight would be utilized to insure the safety of the embankment against shear failure during the loading period. Several small mud waves developed during the placement of the sand blanket and exterior counterweight, but they were of a local nature and after being enclosed, did not cause further difficulties.

Piezometers, embankment control stakes, and settlement platforms were next installed just prior to the start of embankment fill. The details of these control devices are shown in Figure 7. To avoid interference with sand drain installation, the settlement platforms were set on the working table after completion of sand drains. Auger borings were taken next to each platform to determine the settlement that had taken place prior to their installation.

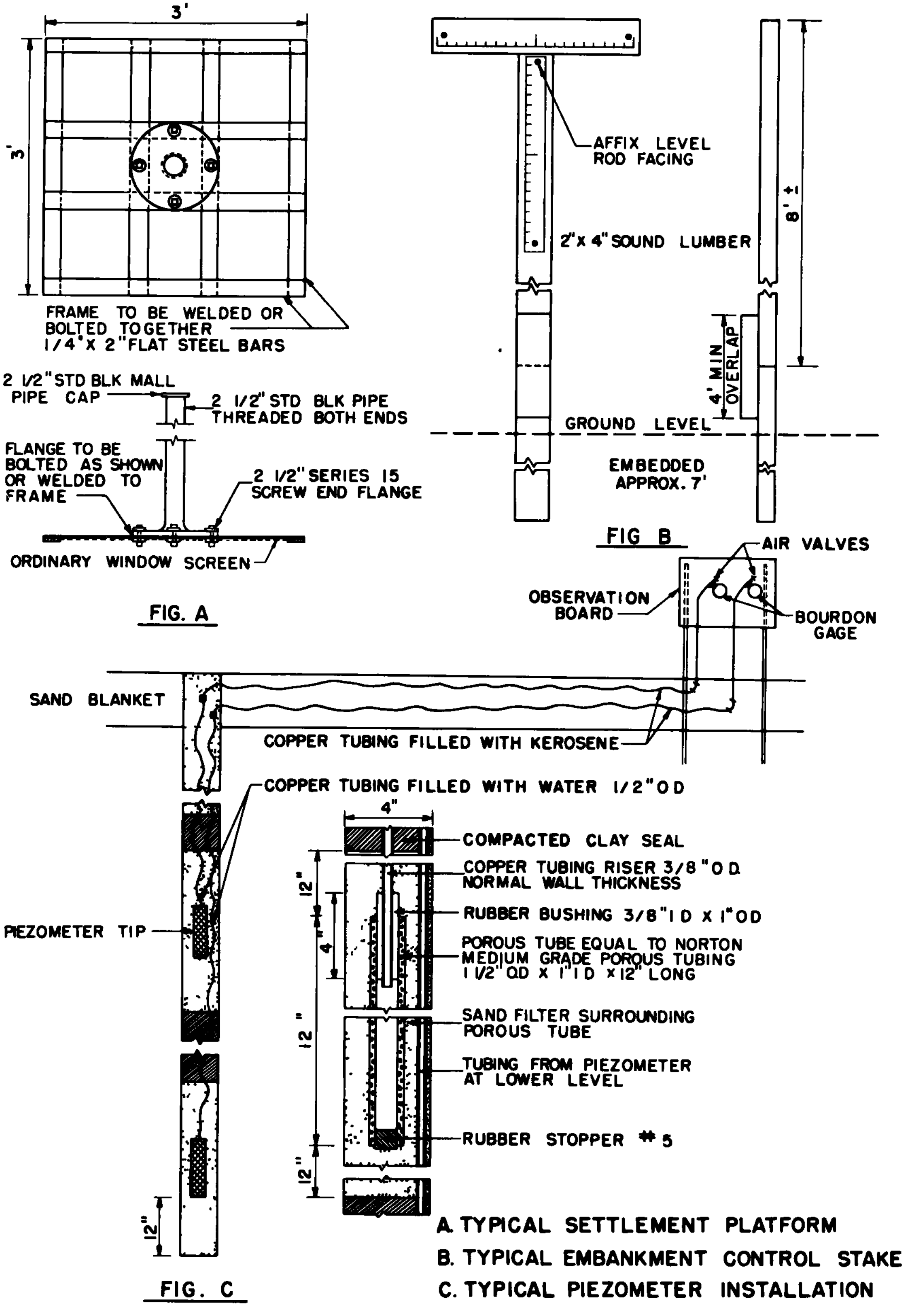


Figure 7. Detail of control devices.

PIEZOMETERS

Among all the above mentioned control devices, only the piezometers warrant further discussion. To get a sensitive and satisfactory measurement of pore pressure in an impervious soil, the following four requirements must be fulfilled: (1) Simple, sensitive and inexpensive; (2) Minimum amount of gas in the system (apportion of which may be due to galvanic action of the metals of which usual piezometers are made); (3) Sufficient intake area at the piezometer tip; and (4) Small cross sectional area of standpipe or tubing to insure rapid response to changing pore pressure, i. e., to decrease the time-lag. However, the diameter of tubing should not be smaller than ⅜ inch, since bubbles of air or gases may be retained in the tubing instead of rising to the surface.

Based on the above design criteria, the piezometer as shown was developed. It is essentially the same as that originated by A. Casagrande. The 1½ inch O. D. porous tube piezometer tip, made of an abrasive refractory material, was obtained from the Norton Company, Worcester, Mass. The ½ inch O. D. saran tubing was obtained from Brown Wales, Boston, Mass. However, it was later found that the saran tubing became very brittle and was liable to fracture in cold weather. Therefore, ⅜ inch O. D. copper tubing was substituted. The sand filter surrounding the porous tube was the same material used in sand drains. Clay for slurry was Bentonite (Volclay KWK &33) manufactured by American Colloid Company, Chicago, Illinois. Bourdon gages of outdoor type had graduations from 15 psi. of vacuum to 30 psi. pressure and were accurate to within ½ percent. A four inch diameter casing was found satisfactory for installing two or more piezometer tips in the same location but at different elevations and a three inch casing for single installation in each location.

The general procedure for installing a piezometer is briefly as follows:

1. Drive casing to one foot below the required bottom of piezometer tip.
2. Thoroughly clean out the entire length of casing by washing.
3. Obtain dry samples of the material from bottom of casing to a depth of a foot.
4. Pull the casing up a foot, preferably by jacking and backfill with saturated sand.
5. Attach piezometer tip to the copper tubing by rubber bushing and lower the system down the casing until the tip is rested on the sand in the bottom of the hole. Seal all joints of the system with a nonhardening joint sealing compound insoluble in kerosene or water.
6. Pull the casing up an additional foot corresponding to the desired elevation of the top of the tip. Then saturated sand is poured in to fill the space around the tip. Continue to pull the casing up to a foot above the top of the tip and backfill the open hole with more saturated sand.
7. Pour and then tamp a Bentonite clay slurry in the form of balls into casing to form a seal above the sand backfill.
8. Withdraw casing.
9. Backfill with lightly tamped sand.
10. Connect the horizontal copper tubing to the vertical tubing from the piezometer tip. Fill the vertical tubing with water up to the level of the tee fitting. Lay the horizontal tubing in a zigzag shape on top of the sand blanket to allow for future stretch due to settlement. Fasten the tubing to the gage and then fill with kerosene.

During the observation period, bleeding off gases formed in the tubing through a relief valve was frequently required to get true gage readings. The gage stem was tilted downward to prevent gases from collecting inside the gage. In this design a single-line horizontal tubing was used. However, it was later found that double horizontal lines, i. e., by bleeding kerosene through the lower intake line and forcing gas out of the upper return line with kerosene, could give more satisfactory results. With this system gases can be bled off more efficiently and the devices can be checked at any time to see if they operate properly. If any line was broken, no kerosene would come through the return line. It is also comparatively easy to rebleed the system just before winter to remove any water that may migrate in the lines and, by freezing, make the devices inoperative.

EMBANKMENT FILL

During the placement of embankment fill, constant readings were taken on the piez-

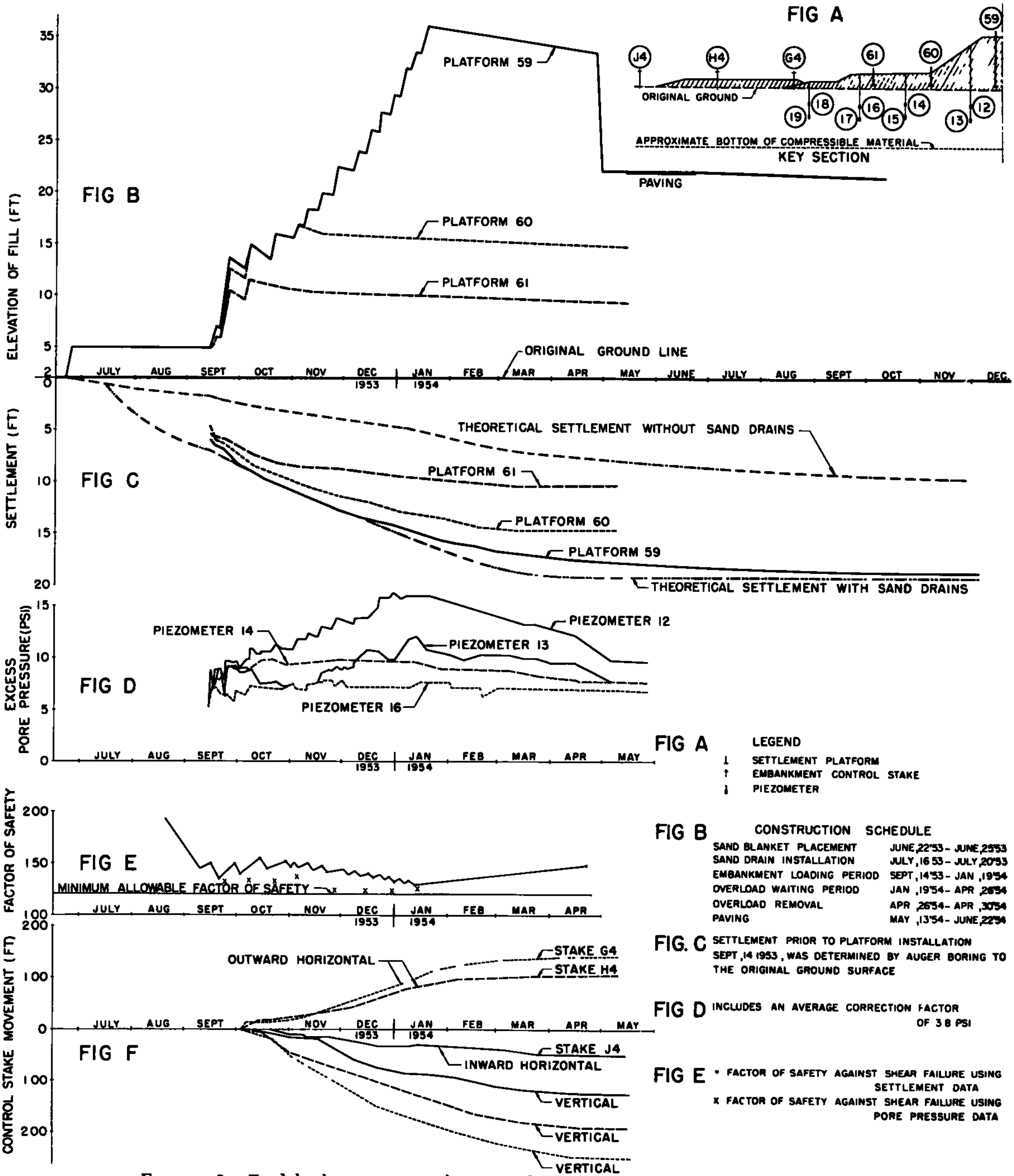
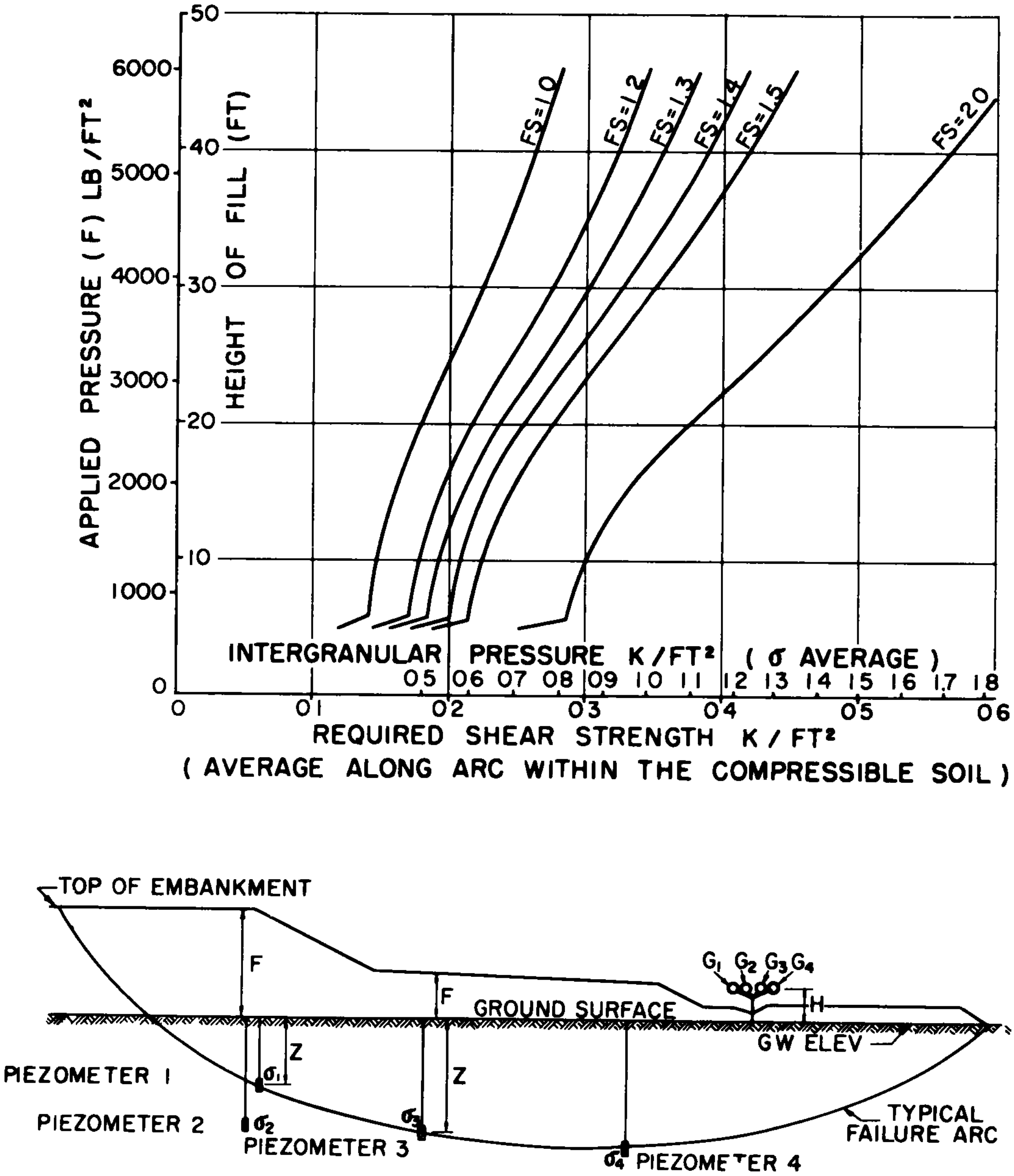


Figure 8. Field observation data at Station 1532 + 00, southbound roadway.

ometers, settlement platforms and control stakes, and the elevations of fill, pore-water pressures, settlement, and control-stake movement were plotted and studied. These data served to control the rate of loading. The records at a critical section, Station 1532 + 00, southbound roadway, are shown in Figure 8. In this figure the theoretical time-settlement curves under the main fill with and without sand drains are indicated.

The above field data provided two quick methods for determining the percent consolidation at any particular time, one by using the excess pore pressure data and the other by comparing the settlement record with the estimated total for the fill in place. In many



LEGEND

G = GAGE READING (PSF)

H = ELEVATION HEAD (PSF)

F = APPLIED PRESSURE (PSF)

Z = OVERBURDEN INTERGRANULAR PRESSURE (PSF)

σ = INTERGRANULAR PRESSURE (PSF)

$\sigma = F + Z - H - G$

$\sigma_{AVER.}$ = WEIGHTED AVERAGE OF $\sigma_1, \sigma_2, \sigma_3, \sigma_4$

Figure 9. Field control for checking embankment stability.

cases, the percent consolidation obtained by these two methods did not check, indicating probably that some piezometers did not register true readings. Based on the percent consolidation obtained, a check on the factor of safety against shear failure for each successive load increment was made. Figure 9 describes the method of checking on the embankment stability by using pore-pressure data. The curves in this figure also serve to check same by using average intergranular pressure computed from settlement data.

The embankment was placed and compacted in 6-inch layers. The specifications call for a minimum of 90 percent Modified AASHO density above a plane located 3 feet above the existing water level to a plane 5 feet below the finished pavement surface. The layers of all embankment including the lower portion of the overload, above a plane 5 feet below the finished pavement surface and within the limits of subgrade treatment at transitions from cut to fill were compacted to a minimum of 95 percent Modified AASHO density. However, sand with a plasticity index of less than three was compacted to a min-

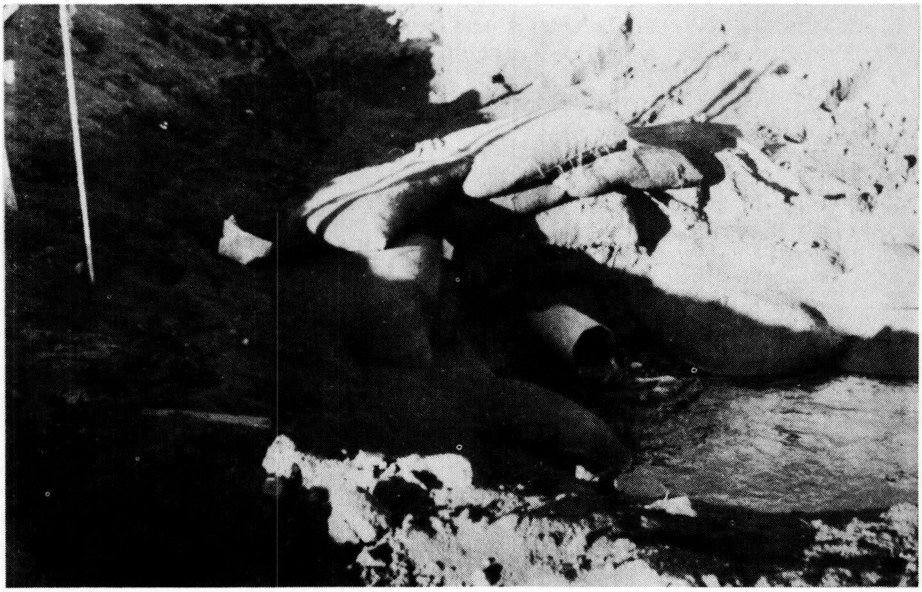


Figure 10. Corrugated-metal underdrain discharging.

imum of 98 percent Modified AASHO density. The upper portion of the overload, which was later removed, was not compacted any more than was necessary to permit the movement of construction equipment. All these elevations were estimated based on the anticipated settlement in each area.

OBSERVATION WELLS

At one time during the placement of the embankment fill, excess pore pressures as high as 15 psi. were obtained on two of the piezometers. A possible reason for the high piezometer readings was trapped air or gas in the lines. Although the lines were bled constantly, if the entrapped gas was deep in the line it could not be removed by this method. For this reason it is believed that double-line piezometers should give better results.

To determine if there was any built-up pressure in the sand blanket due to the upward flow of water from the sand drains and to check the piezometer readings, three observation wells, made of 4-inch standpipes, were installed in the critical areas. Two were driven into the sand blanket and one into the compressible material and the water surface in the standpipes was observed. Both pipes in the sand blanket showed no excess pore pressure and the one in the compressible material eventually indicated excess pore pressure of approximately 50 percent of the amount as registered by the piezometer. These results indicated that the sand blanket was operating effectively and the embankment construction continued. In addition, auger holes driven into the sand blanket at the toes of the interior counterweight did not disclose any excess pore pressure.

PIPE UNDERDRAINS

The outlets of the pipe underdrains set in the sand blanket were checked constantly and most were found to be discharging. After some settlement the outlets were under water when the tide was high and silt tended to collect, thus requiring that the outlets be cleaned regularly. The steady flow out of these pipes indicated that the sand drains were working effectively (see Figure 10).

CRACKS

As expected, many cracks appeared over the area during the placement of fill and during the waiting period. These were caused by differential settlement, with the largest at the extremities of the stabilized area, following the contours of the existing stable material. Others appeared at the toes of the northbound and southbound embankments. Numerous shrinkage cracks were also noted throughout the area.

MOVEMENT OF THE CONTROL STAKES

Throughout the fill operation and during the waiting period, the control stakes showed a constant outward movement. The steady rate of movement did not indicate failure but was caused by shearing strain mobilized by the greater shear strength required as the embankment was increased. A maximum horizontal movement of 1½ feet at a maximum rate of 0.01 foot per day was observed at the most critical area. All the control stakes indicated downward movement, with a maximum of 2½ feet.

The only sign of failure during the loading period was a small mud wave that developed at the outer edge of the exterior counterweight. No movement was noted by the nearby control stakes, indicating that the mud wave was caused by a local slope failure. Fill was placed around the mud wave to prevent further movement and no other disturbances occurred.

OVERLOAD

The overload grade, drawn according to estimated settlement, had to be revised during the loading period because of a slower actual rate of settlement observed at the area of deepest compressible material, i. e., the northerly 350 feet of the stabilization area. This may be due to the use of slightly high coefficients of consolidation in design for a range of pressure and to distortion of the sand drains in areas of excessive settlement, retarding the upward flow. The overload grade was raised in this area in an attempt to speed consolidation.

Figure 11 shows the final overload profile along the southbound roadway and the settlements at the end of the loading period, settlements just prior to removal of overload,

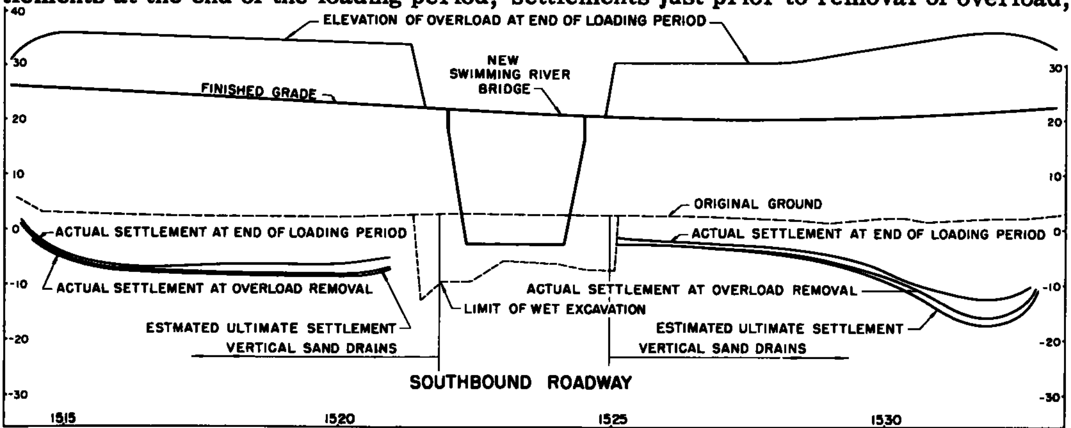


Figure 11. Typical profile showing the final overload, estimated and actual settlements.

and estimated ultimate settlements under the main fill.

After approximately a 3-month waiting period, the overload was removed and the embankment brought to grade. The excess material was used to form a future reversible lane between the north and the southbound roadways and to flatten the embankments to 1-on-4 side slopes. Few check loadings on the settlement platforms during the month subsequent to overload removal showed that no rebound or settlement took place south of the bridge. The area adjacent to the bridge, on the north side, showed a maximum rebound of 2 inches. In these areas the actual total settlement at the time of overload removal corresponded closely with the estimated ultimate values shown in Figure 11. After the overload was removed, no attempt was made to check the water content and shear strength of the stabilized soil.

In the vicinity of the deepest compressible material, the northerly 350 feet, the time-settlement curve did not reach the expected settlement at overload removal (Figure 8). The maximum total settlement of 18 feet recorded under the mainfill in this area at the time of paving, as compared to the estimated ultimate settlement of at least 19 feet, at the same location, indicated that additional settlement was expected. The completion date set for this section of the parkway did not allow additional time for the dissipation of this additional settlement. To compensate for it in part, and yet not seriously affect the riding qualities of the pavement after it is open to traffic, the grade elevation was raised a maximum of 0.4 foot at the most critical area.

PAVEMENT

The flexible pavement was constructed in June 1954. Since then, level readings on top of the pavement and along the centerlines of the roadway have been taken continuously. As was anticipated, the northern-350-foot portion of both the roadways have been settling at a decreasing rate aggregating to a maximum grand total of approximately 18.5 feet, or a net settlement of the pavement of 0.5 foot, four months after pavements were constructed. As a result of this trend, maintenance in these critical areas will be required during the next few years.

CONCLUSION

The consolidation of a highly compressible soil, with low shear strength as found at Swimming River indicates the workability of the sand-drain method. Settlements of great magnitude were accomplished in a relatively short time with no shear failures. The correlation between the predicted and actual settlement indicated that this method of stabilization, with close control of field work, can be depended upon to produce satisfactory and predictable results.

ACKNOWLEDGMENTS

The design and supervision of construction for this section of the parkway was undertaken for the New Jersey Highway Authority by Edwards, Kelcey, and Beck, consulting engineers, with H.J. Leonard as project engineer. The original design and studies of the method of stabilization were made by G.R. Halton, soil consultant for this firm.

The author wishes to thank the authority for using the observation data incorporated in this paper, and E.A. Henderson, soil engineer for the authority, for his recommendation and encouragement in its presentation. Appreciation is extended to the firm of Edwards, Kelcey, and Beck for encouragement and time devoted to the preparation of this paper. Grateful acknowledgment is also expressed to C.J. DeVito, soil engineer for Edwards, Kelcey, and Beck, for his help in preparing the figures and charts included in this report and in reviewing the manuscript.

Economic Aspects of Vertical Sand Drains

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The highway engineer has a number of methods to consider as a solution to the problem of crossing marshland with road construction. His choice of method will depend on engineering aspects and economic considerations. It is assumed that the cost of bridge structures will generally exceed various types of fill construction. Hence, discussion is restricted to economic consideration of mud-excavation, surcharged-mud, and sand-drain methods.

Relationships are developed which indicate the construction cost differentials involved in considering mud excavation and the sand-drain method. The proper consideration of the surcharged-mud method as a solution to the problem is discussed. Sources of information related to these problems are numerous and the author is conscious that economic studies are given careful attention by all highway departments. However, work to date has shown that the economic considerations are a function of many parameters, the exact interrelationships of which the engineer can only estimate. Formulation, however, permits some compounding which makes the other parameters stand out prominently. Total cost differential involving costs related to time and maintenance are discussed.

● SELECTION of the vertical-sand-drain method for the stabilization of muck is dependent upon the exercise of keen engineering judgment and a careful study of economic aspects. A phase of the problem, but by no means the only requirement, dictates a comparison of sand drains to other methods of treatment.

Methods to be compared may be briefly summarized as follows: (1) complete removal of soft soil and replacement with acceptable embankment material; (2) partial removal of soft soil and replacement with acceptable embankment material; (3) drainage methods, including vertical sand drains; (4) floating embankment on soft soil; and (5) bridge structures.

In general, the use of bridge structures cannot compare favorably to the other methods on a construction cost basis. Accordingly, they will not be considered at this time.

The floating embankment is a specialized solution, applicable under rather stringent conditions and, therefore, is dismissed from this discussion,

The economic problem to be discussed reduces itself to a comparison of Methods 1 and 2 to Method 3. It is in this aspect of muck stabilization that interesting and conflicting ideas have been advanced. It is by no means accepted that complete removal of soft soil with the replacement of acceptable embankment material is restricted economically to depths of muck less than 10 feet. This statement, however, is frequently made. It appears that local circumstances may alter this contention considerably. This particular problem will be discussed in some detail.

GENERAL ECONOMICS OF STABILIZATION

The problem of economic comparisons of the various methods of muck stabilization may be stated in a general way, but data are not yet available to give conclusive results nor to permit a complete analysis.

In general, the total cost of a method includes right-of-way requirement costs, construction cost with its related problems, a charge applicable to the method due to time of construction, and annual maintenance costs as a result of the adoption of a given method.

The method using the vertical sand drain is so new, comparatively, that complete service records related to annual maintenance costs are meager and inconclusive. Prospective charges applicable to a method due to time of construction must be predicated on laboratory tests and keen engineering judgment. These estimates may be subject to appreciable change during actual field operations.

As a result of the factors briefly cited above, it appears that a comparison of construction costs represents the only firm index susceptible to analytical treatment at this time. Other phases of the economic problem must be reserved for actual situations and will serve to alter the conclusions arrived at by construction cost comparisons.

CONSTRUCTION-COST COMPARISONS

Complete generalizations of the methods of stabilizing muck cannot be made. One soon becomes impressed by the many variables involved and the interrelationships that exist adding more complication to the problem. One is conscious of the problems faced by the state highway departments and by the realization that a method which meets with success in one area may be inadvisable in another part of the country. Accordingly, the writer wishes to emphasize that demonstrations in this paper are not intended as generalizations. They represent a comparison of methods where engineering judgment indicates a reasonable measure of success.

A simplified geometric sketch of the sand drain method of construction is given in Figure 1. This formed the basis of costs applicable to this method and also lists the nomenclature in units suitable for calculation and comparable to field terminology.

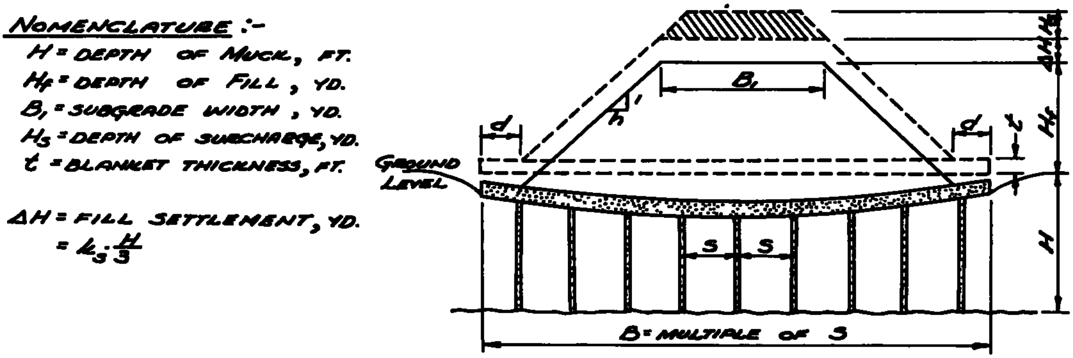


Figure 1. Cross-section for sand-drain method.

In general, the total fill material required is defined as the summation of the volume due to settlement, volume due to required embankment cross-section reflecting widening due to settlement, drainage blanket volume, and volume due to surcharge load. Hence, values for B, D_B, D_S and blanket volume follow directly from the geometry of the sketch. Settlement is expressed as a uniform percentage of muck depth, H, across the entire width, B, this percentage being expressed as K_S, in decimal equivalent form. It should also be pointed out that the depth of drains was taken as (H + t), i.e., the depth of muck including meadow mat and drainage blanket thickness with no overrun into lower support stratum considered. A geometric arrangement of tributary sand drain area is considered to apply, having a well ordered relationship to total blanket width, B.

In accordance with the geometry of Figure 1, the following relationships may be developed:

$$\text{Width} = B = B_1 + 2h\left(H_f + K_S\frac{H}{3} - \frac{t}{3}\right) + 2d$$

Fill borrow, cu. yd. per yd. $= D_B = \left[B_1 + h\left(H_f + K_S\frac{H}{3} - \frac{t}{3}\right)\right] \cdot \left[\left(H_f + K_S\frac{H}{3} - \frac{t}{3}\right)\right]$

Surcharge borrow, cu. yd. per yd. $= D_S = \left[B_1 - h\,H_S\right]H_S$

Blanket volume, cu. yd. per yd. $= B \times \frac{t}{3}$

These expressions of volume per lineal yard of fill may be combined in the following expression of [Cost per sq. yd.] $_{SD}$, this nomenclature being understood to mean cost per sq. yd. tributary to a sand drain:

$$\left[\text{Cost per sq. yd.}\right]_{SD} = \left[\frac{H+t}{A_S}\right]C_D + C_B\left[\frac{D_B}{B} + \frac{t}{3}r_B + \frac{D_S}{B}r_S\right]$$

in which:

C_D = Sand drain driving cost per ft.

C_B = Borrow cost per cu. yd.

A_S = tributary area in sq. yd. per sand drain

$$\text{Ratio } r_B = \frac{\text{Blanket cost per cu. yd.}}{\text{Borrow cost per cu. yd.}}$$

$$\text{Ratio } r_S = \frac{\text{Surcharge cost per cu. yd.}}{\text{Borrow cost per cu. yd.}}$$

This expression for cost is considered to be the unit construction cost directly attributed to the sand drain method. The ratios, r_B and r_S, will depend upon local conditions. The ratio, r_B, is considered to vary from 1.0 to 4.0 but has a relationship to C_B, which in turn is considered to vary from \$0.30 to \$1.60 per cubic yard. For high values of C_B, r_B will be relatively low; for low values of C_B, it will probably be quite high. The ratio, r_S, may vary from 1 to 2. If a separate item for overload removal and disposal is included in the bid proposals, this ratio will probably vary from 1.25 to 2.00.

A simplified geometric sketch of a total excavation and replacement method following the standard practice of the Michigan State Highway Department is given in Figure 2.

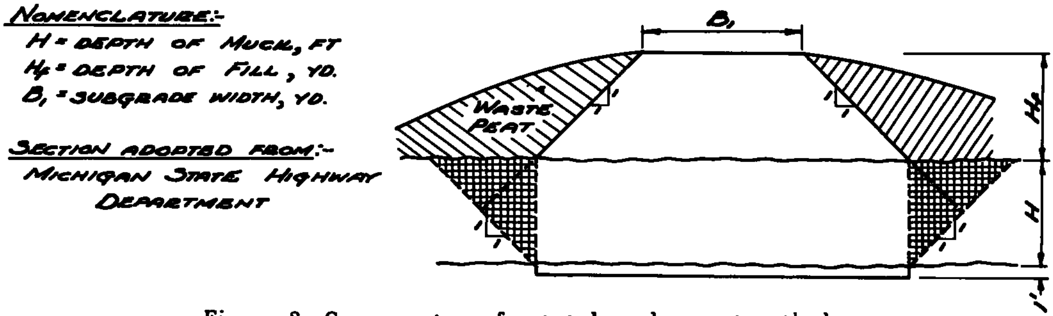


Figure 2. Cross-section for total-replacement method.

This arrangement is reported as applicable for depths of muck, H, up to 10 feet, but it is the author's opinion that some increase in depth is possible. The unit construction cost directly attributed to this total replacement method is given as:

$$\Big[\text{Cost per sq. yd.}\Big]_{TR} = C'_B\left[\frac{R_B}{B} + \frac{R_E}{B}\, r_e\right]$$

in which:

$$\text{Total excavation, cu. yd. per yd.} = R_E = \Big[B_1 + 2H_f\Big]\cdot\left[\frac{1+H}{3}\right] + \left[\frac{H}{3}\right]^2$$

$$\text{Total borrow, cu. yd. per yd.} = R_B = \Big[B_1 + H_f\Big]\cdot\Big[H_f\Big] + R_E$$

C'_B = Borrow cost per cu. yd. for total replacement method

B = Total width, sand drain section

$$\text{Ratio } r_e = \frac{\text{Excavation cost per cu. yd.}}{\text{Borrow cost per cu. yd.}}$$

The insertion of width B, of the sand-drain method, reduces data to an evaluation comparable to sand drain construction. The ratio, r_e, will be a function of local conditions and may assume wide variation. The author recognizes that it bears a functional relationship to depth h, but no reliable data could be obtained to estimate this variation. Data to permit such an evaluation would be most welcomed. The borrow cost per cubic yard has been expressed as C'_B, with the recognition that conditions of the bid proposal may result in a value differing from C_B, the borrow cost per cubic yard of the sand-drain method.

With determinable values for the unit construction cost for comparable methods, a unit cost increment can be expressed as:

$$\Delta C = \Big[\text{Cost per sq. yd.}\Big]_{SD} - \Big[\text{Cost per sq. yd.}\Big]_{TR} = \frac{H+t}{A_S} C_D + C_B\left[\frac{D_B + r_B\, t/3\, B + D_S r_S}{B}\right] - C'_B\left[\frac{R_B + R_E r_e}{B}\right]$$

Hence, for positive values of ΔC, the total-replacement method is economically favorable; for negative values, the sand-drain method controls. This equation may be further modified by introducing the relationship:

$$C_D \approx K_d\left[\frac{7}{H+t} + 1\right]$$

where K_d is a constant varying from 0.60 to 1.20. The value of C_D, so determined, includes both the driving cost and porous sand fill. It reflects the functional relationship of driving cost with respect to length of drain including the unit driving cost and a chargeable cost per foot for equipment moving and setup. Hence, it may be noted that as the drains get very short, the unit price rises to extreme values.

By introducing the approximate relationship for C_D and rearranging terms, we obtain

$$\frac{\Delta C}{C_B} = \left[\frac{K_d}{C_B A_S}\right]\cdot [7+H+t] + \left[\frac{D_B + r_B t/3\, B + D_S r_S}{B}\right] - \frac{C'_B}{C_B}\left[\frac{R_B + R_E r_E}{B}\right]$$

The primary objective of this study is to present a basis for the determination of the critical muck depth, H_{CRIT}, i.e., the depth for which the cost of the sand-drain method balances the replacement method. Hence a reduction of the labor of calculation is realized by expressing the equation in a grouped constant form. If the condition $C'_B = C_B$, is considered to exist, we obtain the result:

$$\frac{\Delta C}{B} = \left[\frac{K_d}{C_B A_S}\right] \cdot [7+h+t] + \left[\frac{D_B + r_B t/3\, B + D_S r_S - R_B - R_E r_E}{B}\right]$$

This is the form that has been utilized for studies in this paper.

A simplified geometric sketch of a partial excavation and displacement method of muck stabilization following the standard procedure of the Michigan State Highway Department is shown in Figure 3. For details of the method, the reader is referred to published standards and data of this organization. For a comparison of the sand drain method and this partial replacement method we get the following relationship:

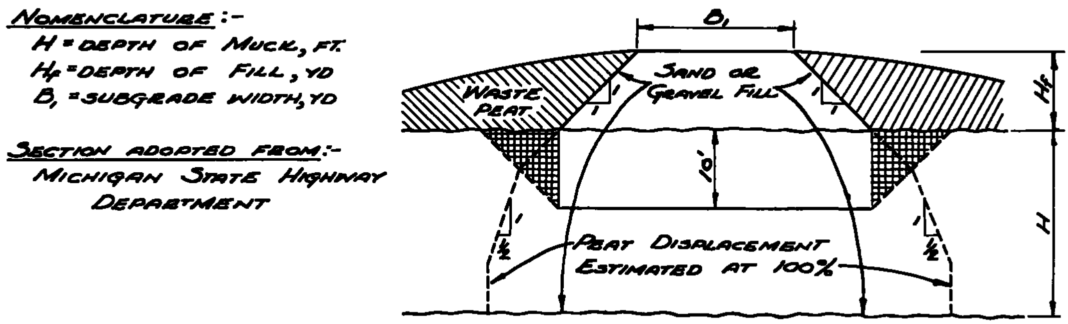


Figure 3. Cross-section for partial-replacement method.

$$\frac{\Delta C}{B} = \left[\frac{K_d}{C_B A_S}\right] \cdot [7+H+t] + \left[\frac{D_B + r_B t/3\, B + D_S r_S - P_B - P_E r_E}{B}\right].$$

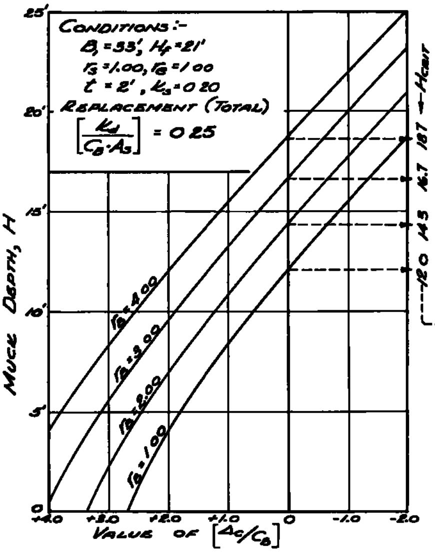
in which the nomenclature is identical to that given previously, except for:

Total excavation, cu. yd. per yd. $= P_E = \left[B_1+2H_f+\frac{10}{3}\right]\cdot\left[\frac{10+H}{6}\right] - \left[\frac{10H}{36}\right] + \left[\frac{H}{6}\right]^2$

Total borrow, cu. yd. per yd. $= P_B = \left[B_1+2H_f+\frac{10}{3}\right]\cdot\left[\frac{H}{3}\right] + \left[B_1+H_f\right] H_f + \frac{H}{12}\left[\frac{3H-20}{6}\right]$

An example of the determination of the critical muck depth, H_{CRIT}, for a defined set of conditions is shown in Figure 4 together with a tabulation of all the data developed for one set of curves.

A graphical presentation of the variation of critical muck depth, H_{CRIT}, versus variation of the economic ratio $(K_d/C_B A_S)$ for both the total-replacement method and partial-replacement method compared to the sand-drain method is depicted in Figure 5. It must be emphasized that generalizations should be drawn with caution from this graph. The



$\frac{k_d}{C_B A_s}$	Value of H_{crit} when r_e = 1.00	2.00	3.00	4.00
0.01	1.2	3.0	4.4	5.9
0.05	2.6	4.1	5.7	7.3
0.10	4.2	5.9	7.7	9.3
0.15	6.1	8.1	10.0	12.0
0.20	8.7	11.0	13.1	15.0
0.25	12.0	14.3	16.7	18.7
0.30	16.3	19.2	21.8	24.4
0.35	22.7	25.2	28.0	31.0

Table of Values - H_{crit}. Depth of muck for which total replacement cost balances sand drain cost

Figure 4. Example of determination of critical muck depth, H_{crit}.

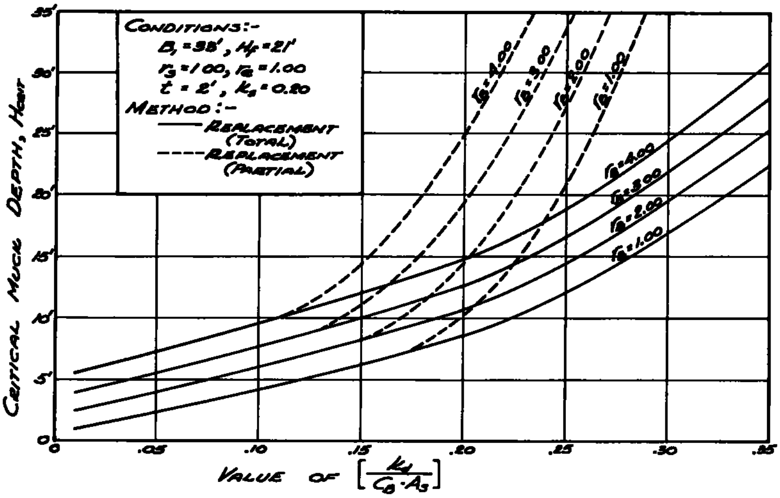


Figure 5. Graph of variation of critical muck depth with economic ratio with method as a variable.

conditions depicted, may in their extreme, bear little resemblance to reality. The use of value, r_e = 1.00, is unrealistic for values above 15 feet, since it reflects inordinately on the value of C_B, and additionally, this depth is probably the extreme condition of stability for material of the type represented. Because of the construction procedure, these remarks are not so directed to the partial-replacement method.

The graph of Figure 5 shows quite emphatically why the partial-replacement method is utilized for muck depths above 10 feet and also shows how the blanket cost ratio influences economical depth. For conditions of low borrow cost, together with close spacing of drainage wells, the replacement method may compare favorably with sand drains in the range from 0 to 30 feet.

The variation of critical muck depth due to fill width is depicted in Figure 6. It should be noted that fill width exerts a minor influence.

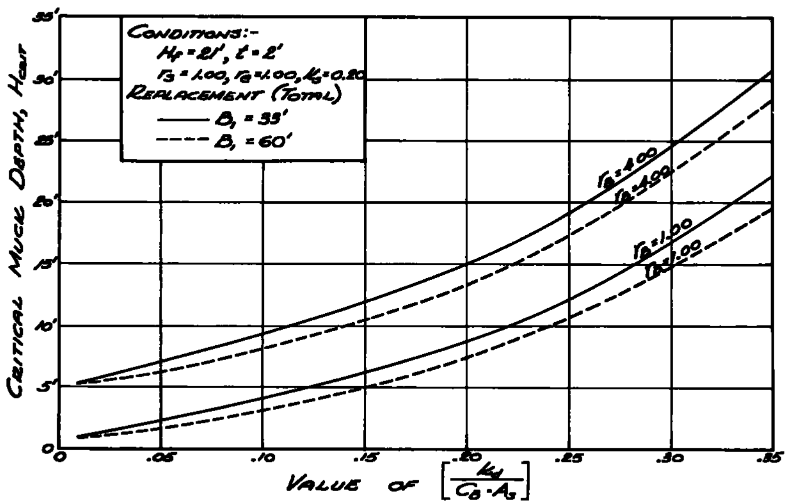


Figure 6. Graph of variation of critical muck depth with economic ratio with fill width as a variable.

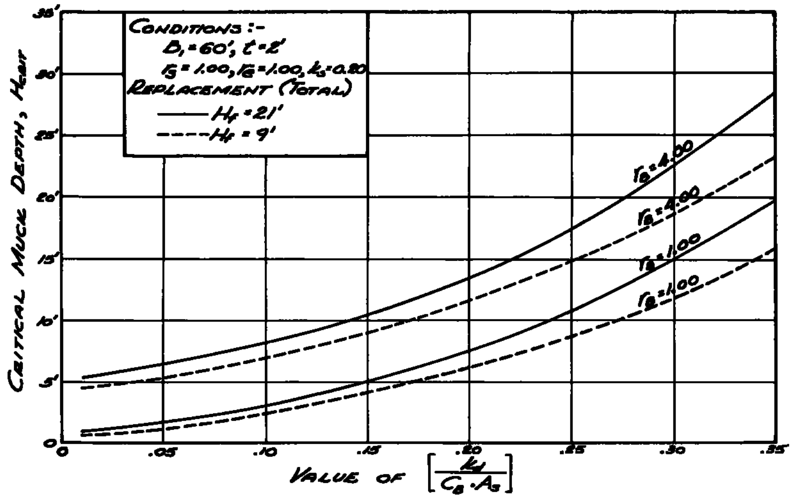


Figure 7. Graph of variation of critical muck depth with economic ratio with fill depth as a variable.

The graph of Figure 7 indicates that the replacement method becomes less favorable as the depth of fill decreases.

CONCLUSIONS

This presentation has been intended to focus attention on a number of interesting problems related to economic comparisons of methods of muck stabilization. It is felt that a mode of study permitting rapid comparisons of the important parameters may result.

A number of interesting cases rate investigation: (1) section geometry variation; (2) critical muck depth versus variation of r_e; (3) critical muck depth versus variation of

K_S & t; (4) critical muck depth versus variation of r_S; (5) the effect of including an approximate expression for the variation of r_e with depth, H; (6) effect of economic value of construction time for comparable methods; and (7) effect of relative maintenance costs on economic depth determinations.

ACKNOWLEDGMENT

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The contractual arrangements were made before the recent resignation of William F. Childs, Jr., as chief engineer of the commission, and the subsequent appointment of Norman Pritchett, who succeeds him. J. Trueman Thompson is chairman of the department and research contract director. The contract is administered by a Coordinating Research Committee composed of Walter C. Hopkins, deputy chief engineer of the commission; C.A. Goldeisen, A. L. Grubb, and Allan Lee, representing the commission; and J. Trueman Thompson, Robert S. Ayre, Walter C. Boyer, and Wen-Hsiung Li, representing the department.

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Control of Slide by Vertical Sand Drains

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OF the many adverse agents of nature, to the engineer and geologist, a slide is not the least. Many preventative and remedial measures have been tried, but no one method is applicable to all cases, and the remedy must be custom-tailored to suit the situation. The paper does not discuss a new method, but is a case history of a slide which was brought under control by the use of vertical sand drains.

In 1947, the Virginia Department of Highways began the relocation and widening of US 220 between Clifton Forge and Iron Gate in Alleghany County, Virginia. It was realized from the beginning that the probability of slides was imminent due to the following factors:

The location of this road is in the Valley and Ridge Province which embraces the major portion of the Appalachian Valley. The terrain is extremely rough and mountainous, with the mountains being cut by numerous streams forming a number of water gaps.

The immediate site of this road is near the famed Iron Gate Gorge where the Jackson River cuts through Rich Patch Mountain, revealing one of the best known of all anticlines in the Appalachian Valley. This anticline rises 700 feet above the grade of the new road and is composed of interbedded sandstones, limestones, and shales of Silurian and Devonian ages. Joint planes set up by the forces of folding have developed in the sandstone at right angles to the bedding, giving rise to large rectangular blocks of rock, which came in handy in building the retaining walls which will be mentioned later.

The new location runs parallel to the axis of the anticline and is sandwiched between the Jackson River and the mountain side. Excavation showed that through this entire section the road passed through a thick mantle of talus, mostly sandy soil, and the heretofore-mentioned boulders. An analysis of this soil gave the following results: a liquid limit of 21 percent; nonplastic; shrinkage limit of 16 percent; optimum moisture, 10.5 percent; and a density of 123 pcf.; 49.3 percent of the material passed a No. 10 sieve. Of this, 20 percent was coarse sand, 52 percent fine sand, 15 percent silt, and 13 percent clay. This soil was classified as A-2 sand silt. It was in this talus material that the slides first showed evidence of developing.

As in 99 percent of all cases of slides, water was the chief offender. It could percolate through the beds of sandy soil and badly fractured rock and break out in the slope above the new grade.

To further complicate matters, the old road (which was still carrying traffic) was perched on a narrow shelf-like indentation. It was under this that the slide first showed signs of starting.

The engineering staff believed that a massive retaining wall would have a tendency to check the slide. For once nature was on our side, for the large sandstone blocks which normally would have been waste material were used to construct the wall. This wall was approximately 4 feet through and 3 feet high. No mortar was used.

Owing to the character of the material, the excavation for the footing of the wall was only carried forward about 5 feet ahead of the actual construction of the wall. This wall was quite effective in checking any large earth movements at the base of the slide, but further creep and seepage caused the bank to slough off and break back into the pavement of the then-existing road.

A detailed study disclosed the presence of a horizontal bed of plastic varved impervious clay about 35 feet above the grade. The water draining through the loose sand would reach this bed of clay, then follow it and break out on the slope.

The geological staff of the highway, in consultation with the Engineering Geology Branch of the United States Geological Survey, considered several possibilities as remedies, among them being benching, grouting, and a flatter slope. However, it was finally decided to try vertical sand drains, even though there was some question as to whether they would be effective. It was known that these drains had performed with excellent results in the marsh lands of New Jersey and California, but as far as was then known this was a pioneering attempt to use them to drain a slide.

It must be borne in the mind that this method is different from the normal sand drains, wherein a layer of sand is placed upon a marshy section and the water forced upward through the sand drain. In our case the sand drains were installed in hopes that the water would be carried down and discharged in the porous material at the bottom of the hole.

A well-drilling company contracted to do the work. An ordinary churn drill was used. An 8-inch hole was drilled and cased with 6-inch casing. Ten of these holes, each 80 feet deep, were drilled in the ditch line of the old road. This placed the bottom of the hole 10 feet below the grade of the new one. It was anticipated that the water would drain through the sand and gravel in the old flood plain of the river and hence find its way into the river itself. When the first hole was finished, it was found that contrary to expectations, the water would not drain through the tightly compacted gravel and clay. Five sticks of dynamite were exploded at the bottom of each hole, which allowed the water to drain freely out to the river. No other drainage was necessary. The hole was then filled with well-graded concrete sand. The casing was pulled and sealed with a bituminous cap.

It had been estimated that this work would cost in the neighborhood of \$8,000, whereas the actual cost was a little over \$500 per hole or approximately \$6.25 per foot. The total cost was a little over \$5,000.

This area has been carefully watched for the past 7 years, and with the exception of the normal sloughing of any new cut, no slides have occurred. It is therefore concluded that these drains are operating efficiently and that their use may have a more-widespread application to the control of various drainage problems than those confined to marshes and tidal sections.

HRB: M-326

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government designated by the President of the United States, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY-COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.