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 sta-

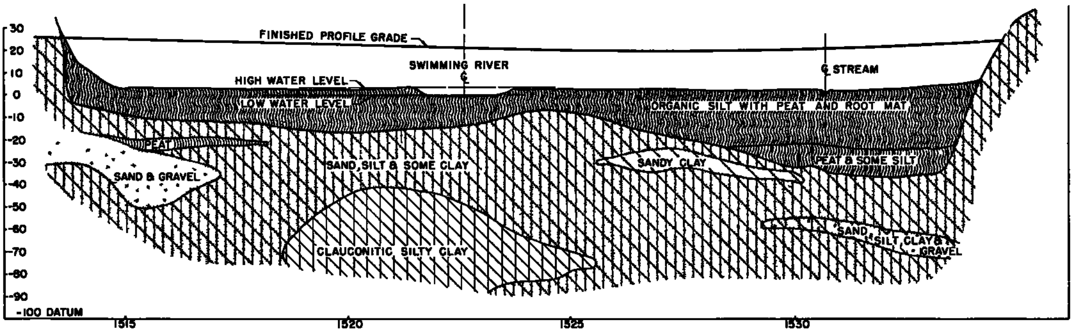


Figure 1. Soil profile, southbound roadways.

bilize 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.

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.

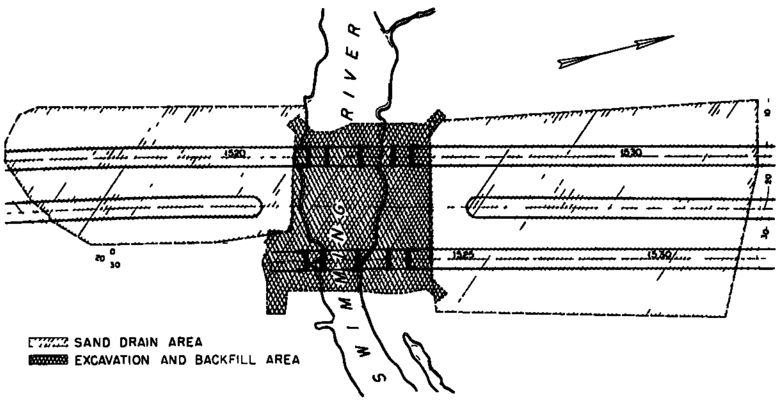


Figure 2. General plan.

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

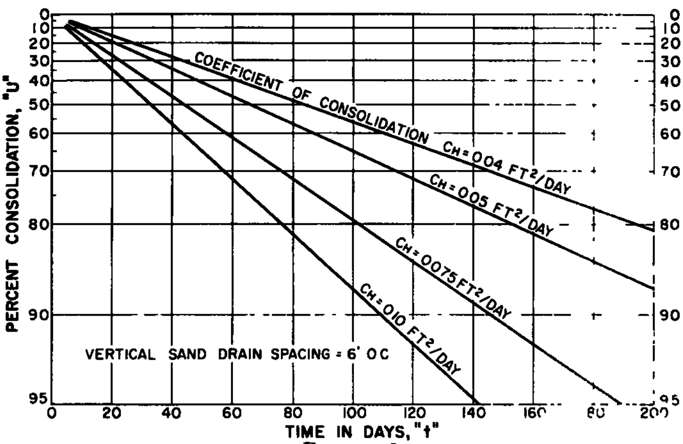


Figure 3.

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,

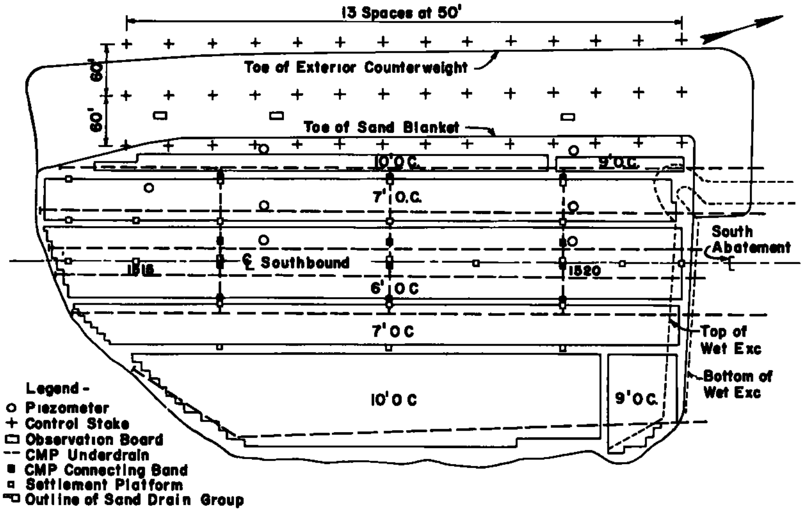


Figure 4. Typical design plan.

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.

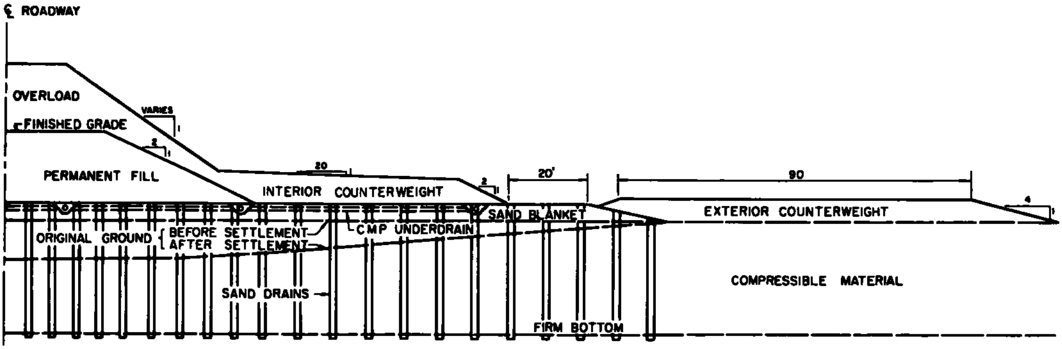


Figure 5. Typical half section.

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

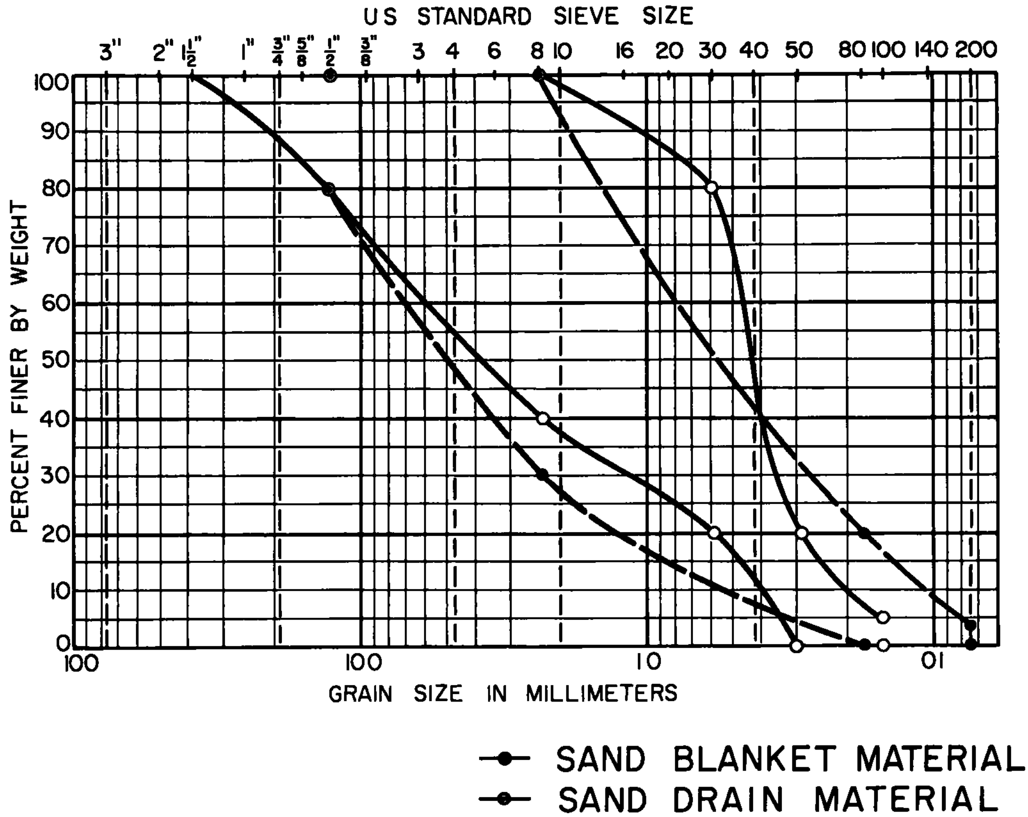


Figure 6. Gradation limits.

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.

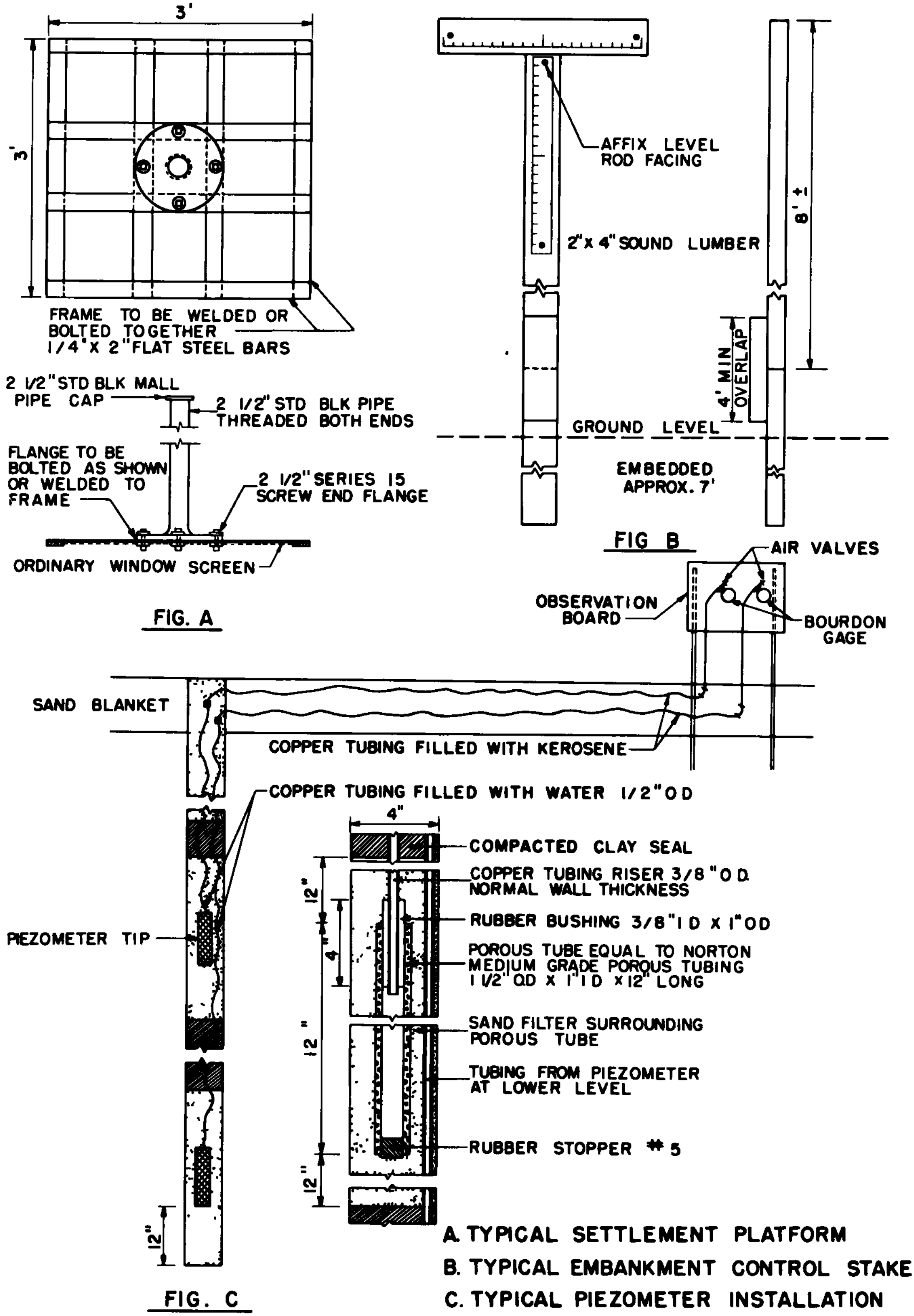


Figure 7. Detail of control devices.

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-

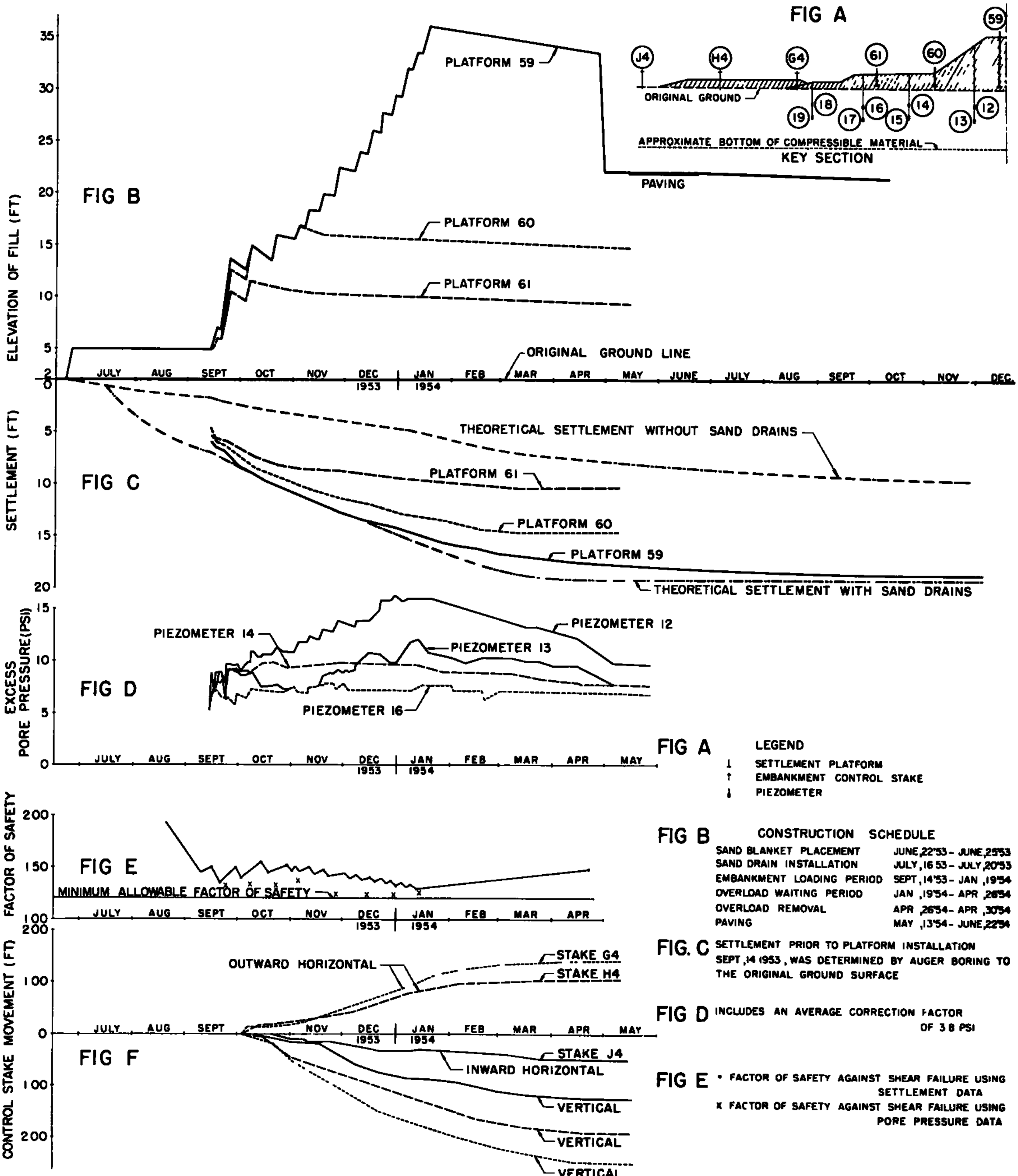
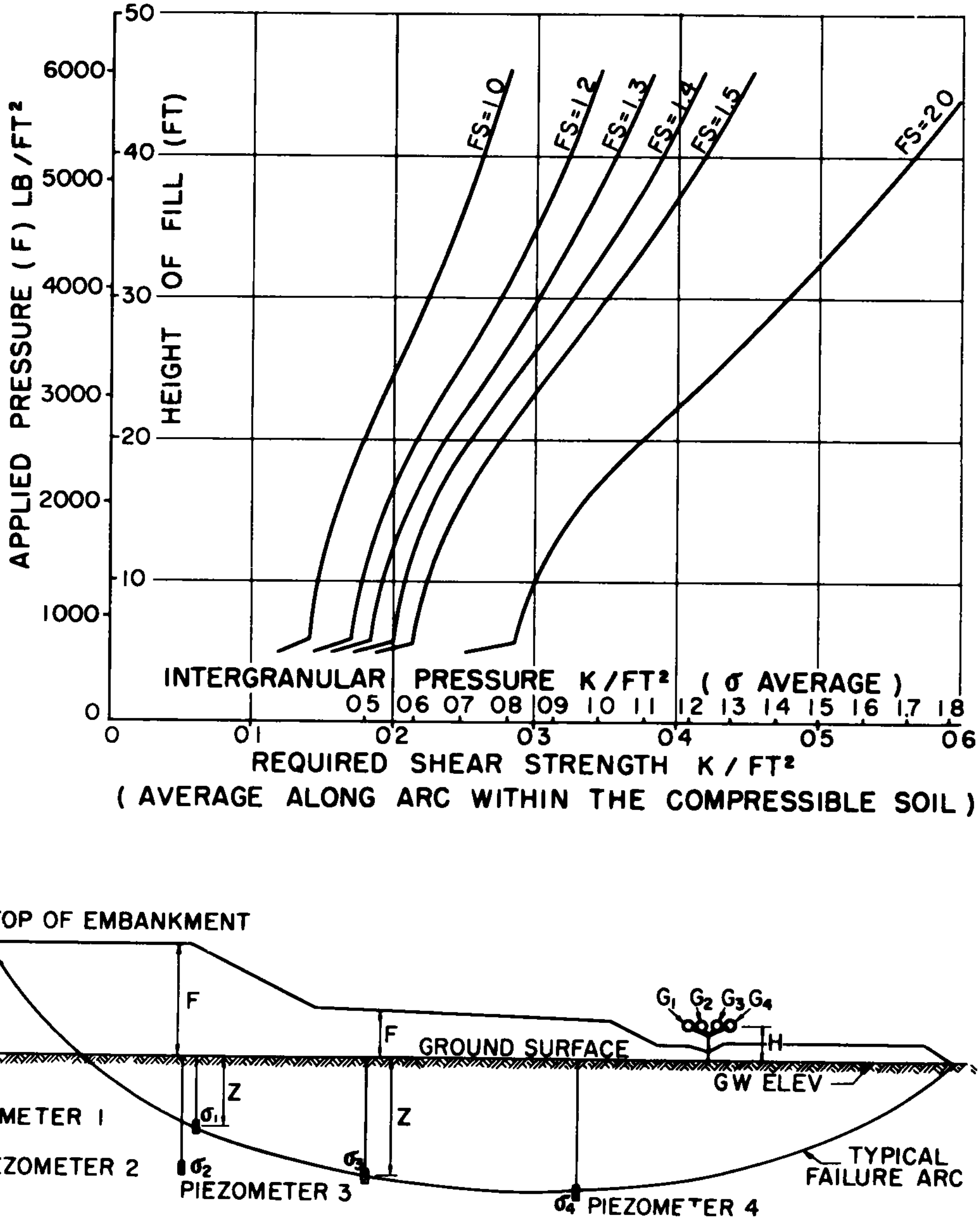


Figure 8. Field observation data at Station 1532 + 00, southbound roadway.

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-

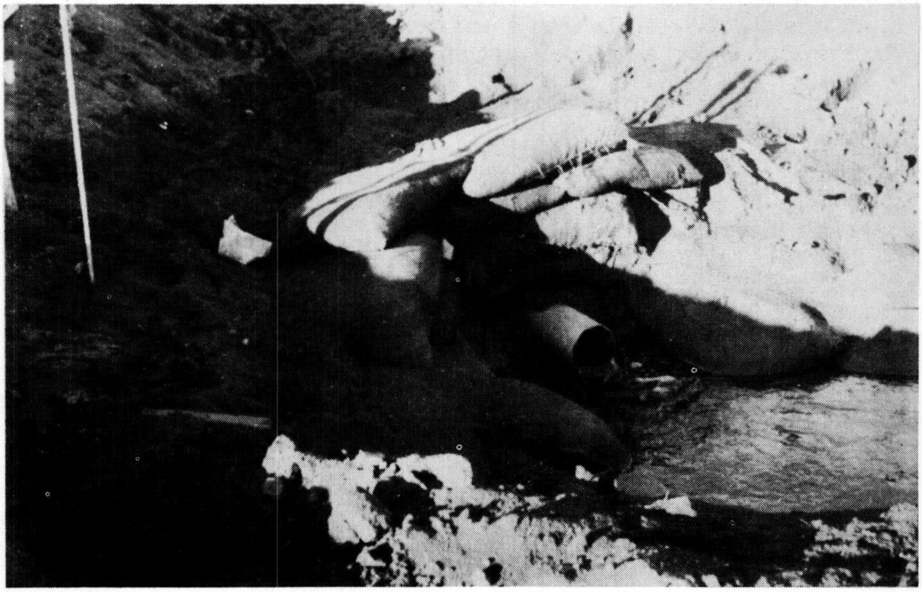


Figure 10. Corrugated-metal underdrain discharging.

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,

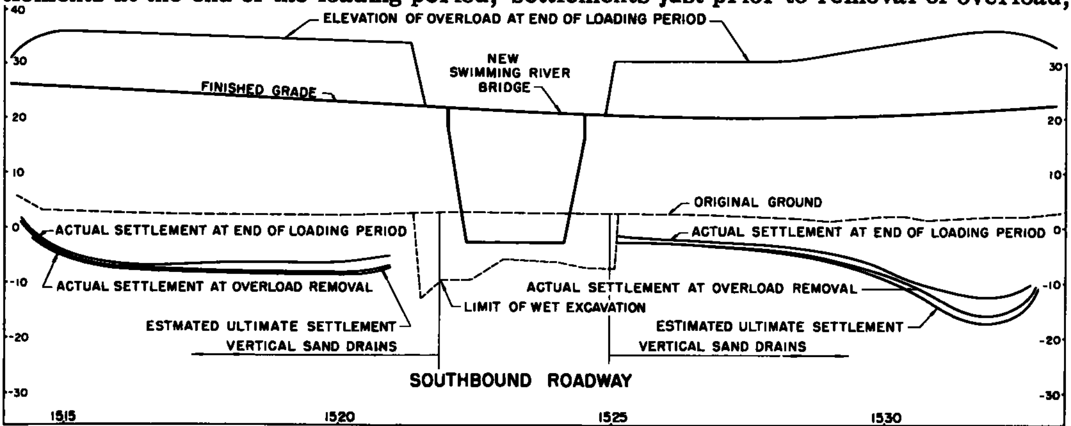


Figure 11. Typical profile showing the final overload, estimated and actual settlements.

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.

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