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

RAY WEBBER, Assistant Construction Engineer, and
W. C. HILL, Soils Engineer, Oregon State Highway Department

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

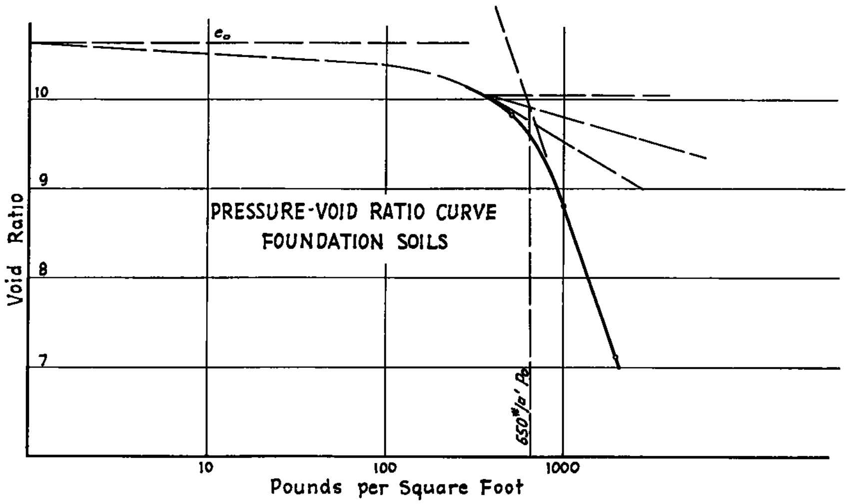

Figure 1.

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 ½ ($p_1 + p_2$) = 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

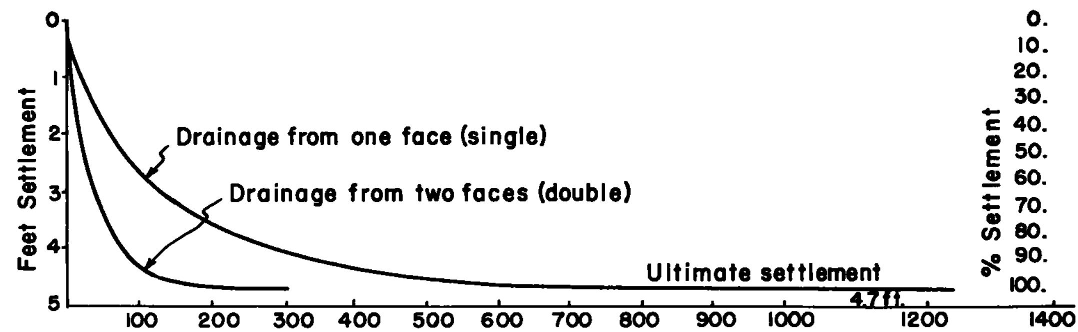
Rate of Settlement in days = "t" = $\frac{H_d^2 T}{C_v}$

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

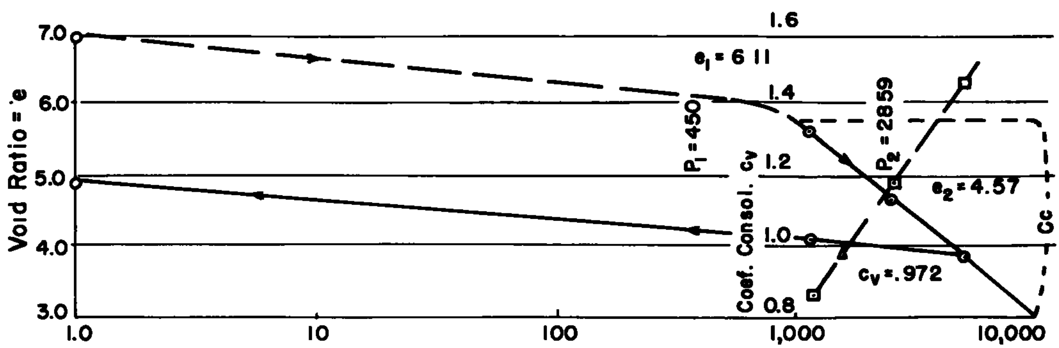
% Consol.	Settl't. Feet	T	Rate of Settlement in Days Single Drn.	Double Drn.
10	0.5	.008	3	1
20	0.9	.032	13	3
30	1.4	.069	28	7
40	1.9	.126	52	13
50	2.3	.198	82	21
60	2.8	.288	119	29
70	3.3	.405	167	42
80	3.8	.566	233	58
90	4.2	.846	349	87
95	4.5	1.134	468	116
100	4.7	$\pm$3	$\pm$ 1236	$\pm$ 308

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

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



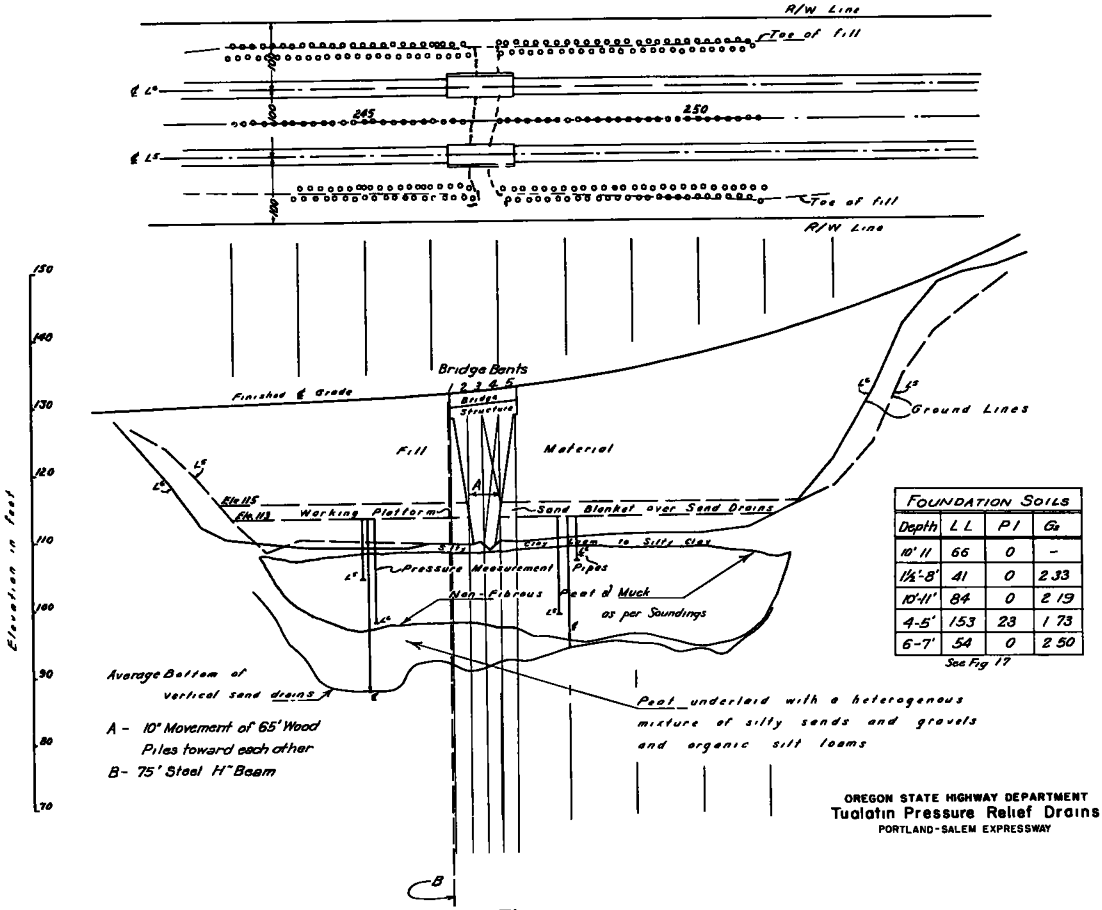
Settlement - Time Curve



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

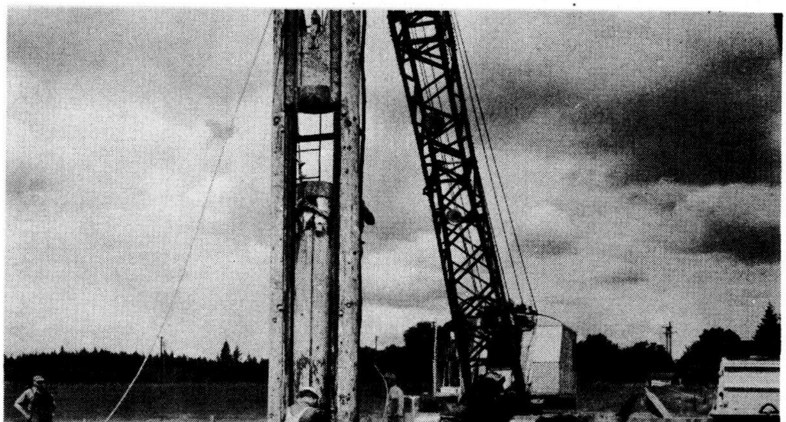


Figure 4. Centering wood spud over pressure relief drain layout. Permeable drain material buckets on right.

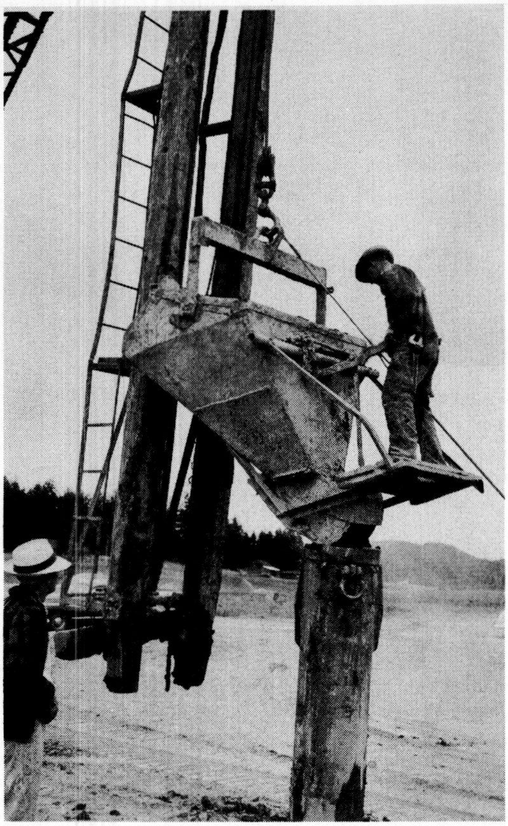


Figure 5. Filling hollow steel mandrel with ¼ in. -0 drain material. Note the control platform welded to the bucket.

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.

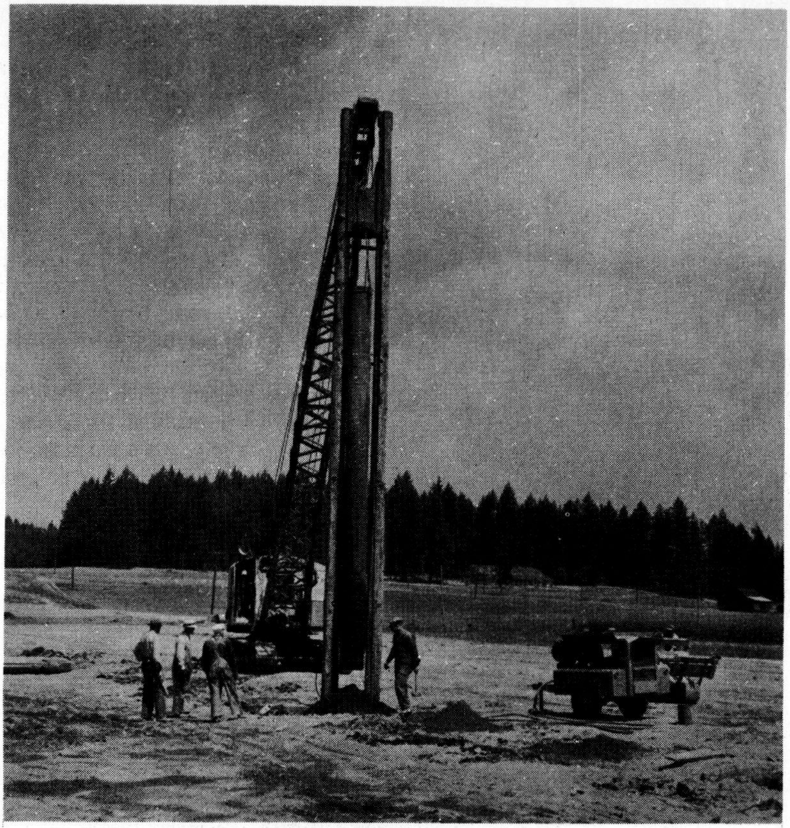


Figure 7. Mandrel pulled showing surplus ¼ in. -0 drain material. Note type of air compressor used.

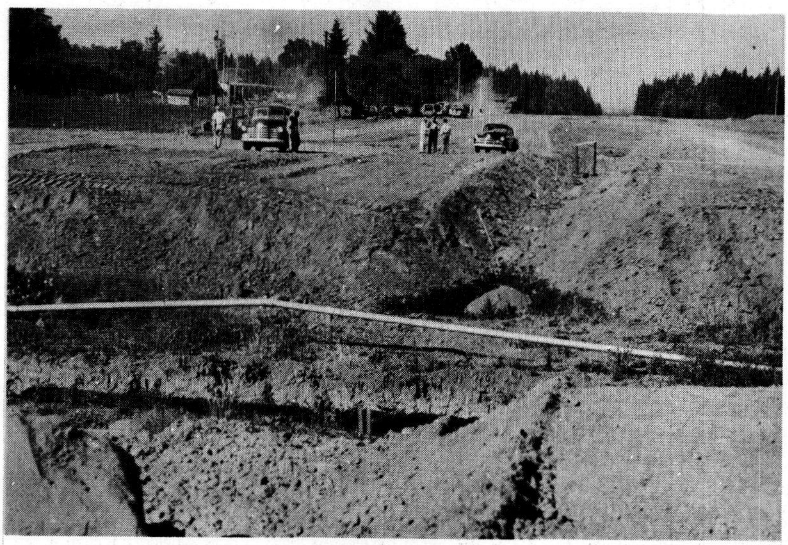


Figure 8. First lift of embankment L 245. Note activity of pressure relief drain, right center, settlement pipe center background and gauge pipe, right.

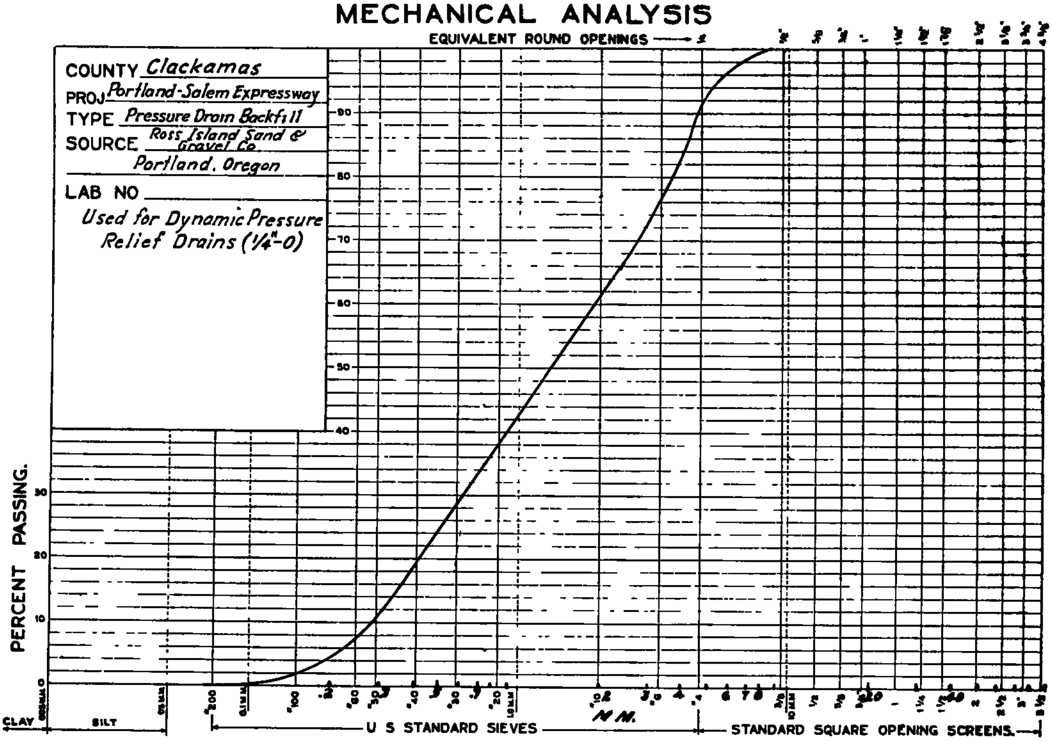


Figure 9.

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

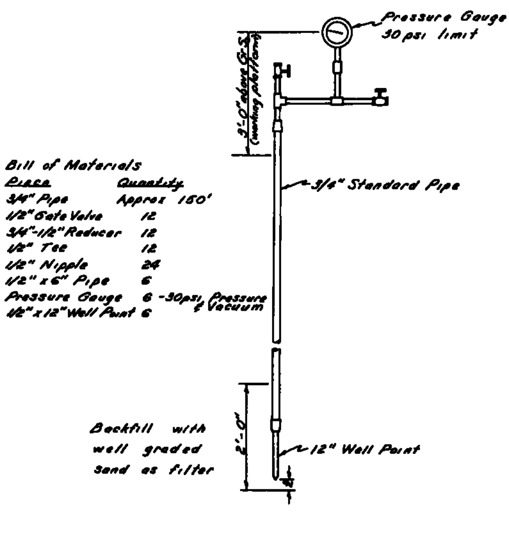


Figure 10. Sketch diagram, pore pressure measuring device.

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

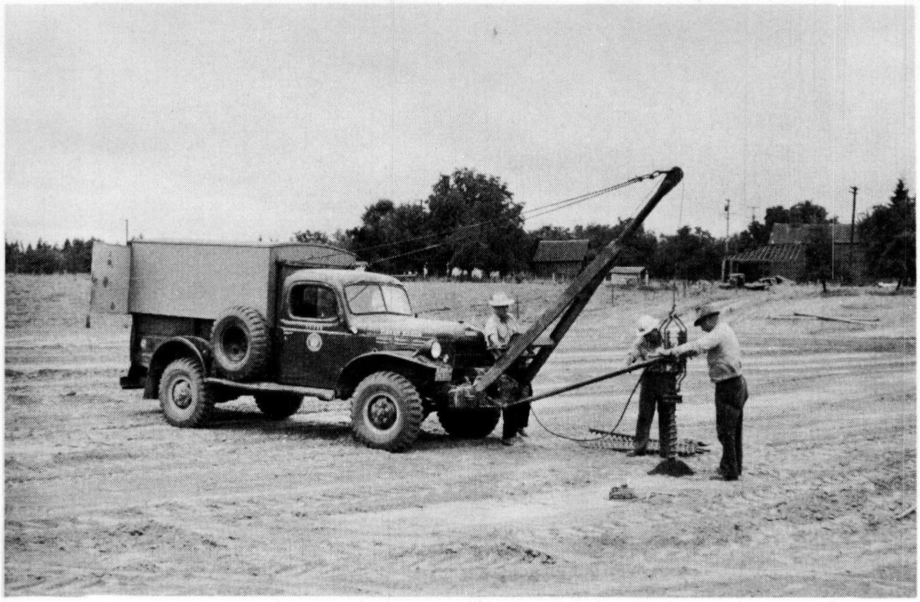


Figure 11. Boring holes for pressure piezometric gauges.

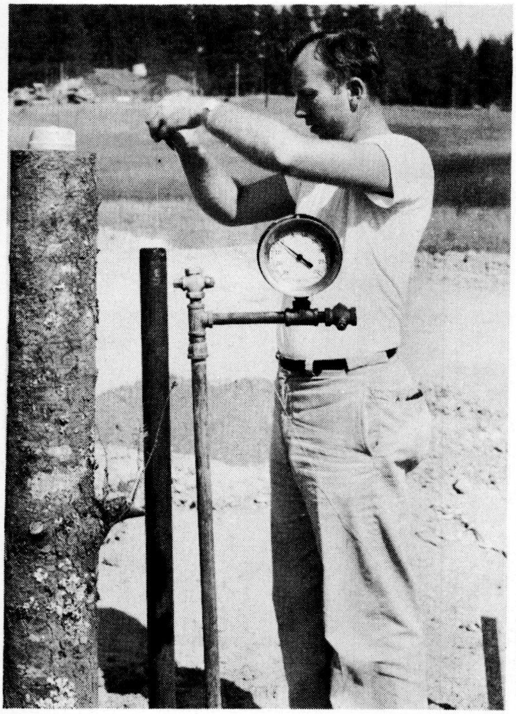


Figure 12. Checking water rise against pressure piezometric gauge. Vacuum of ± 1½ inches initially applied. Gauge reads in pounds per square inch plus and inches of mercury minus.

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.

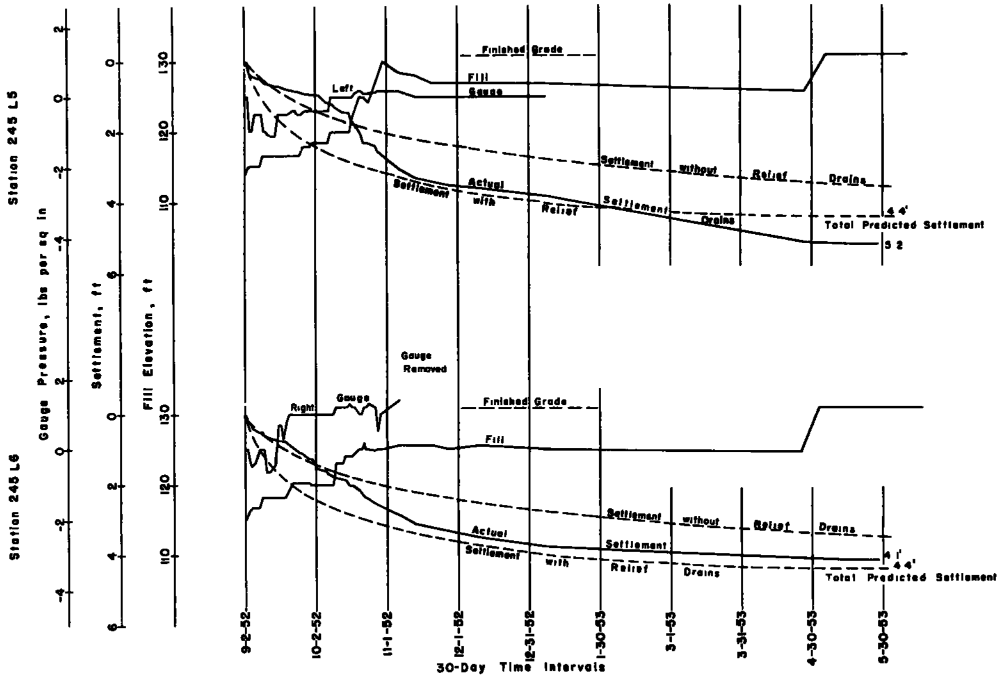


Figure 13.

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-

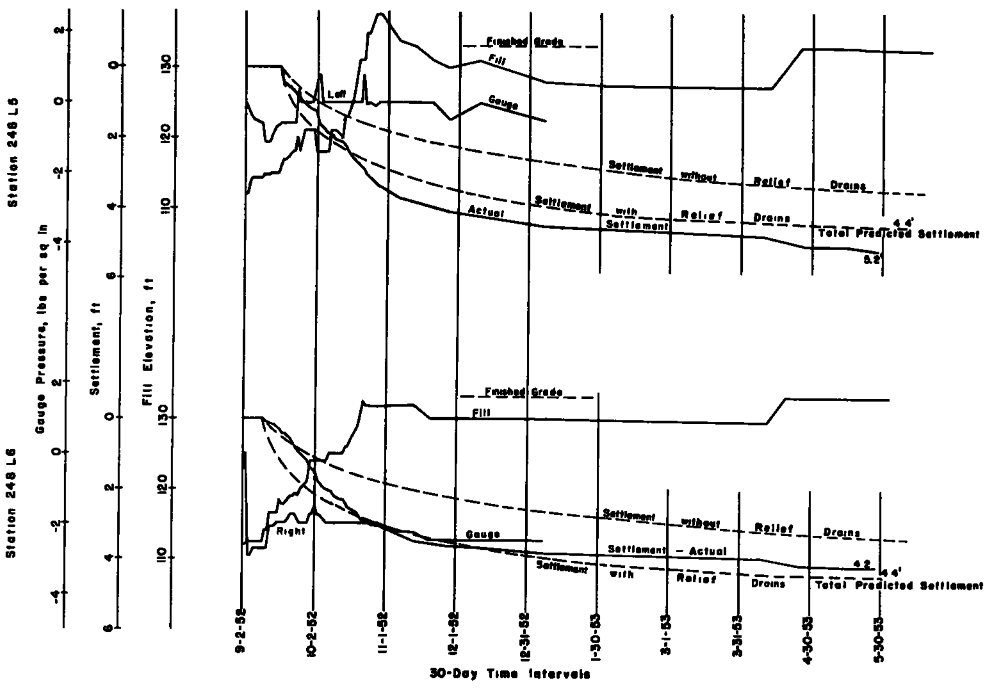


Figure 14.

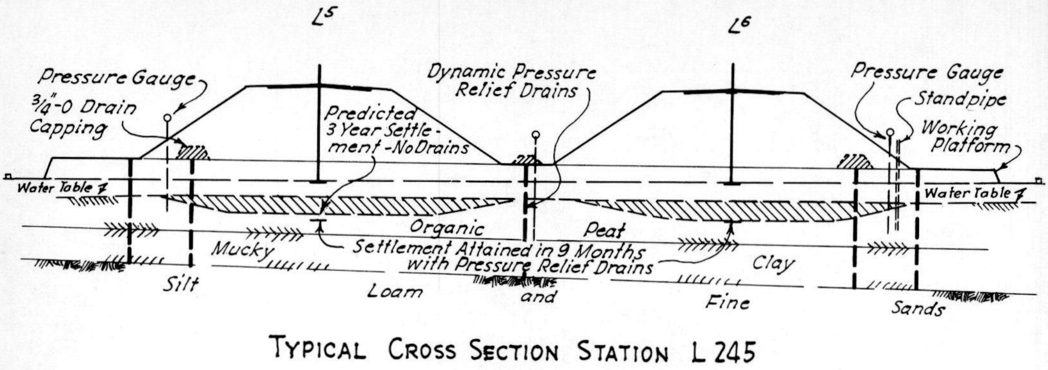


Figure 15.

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

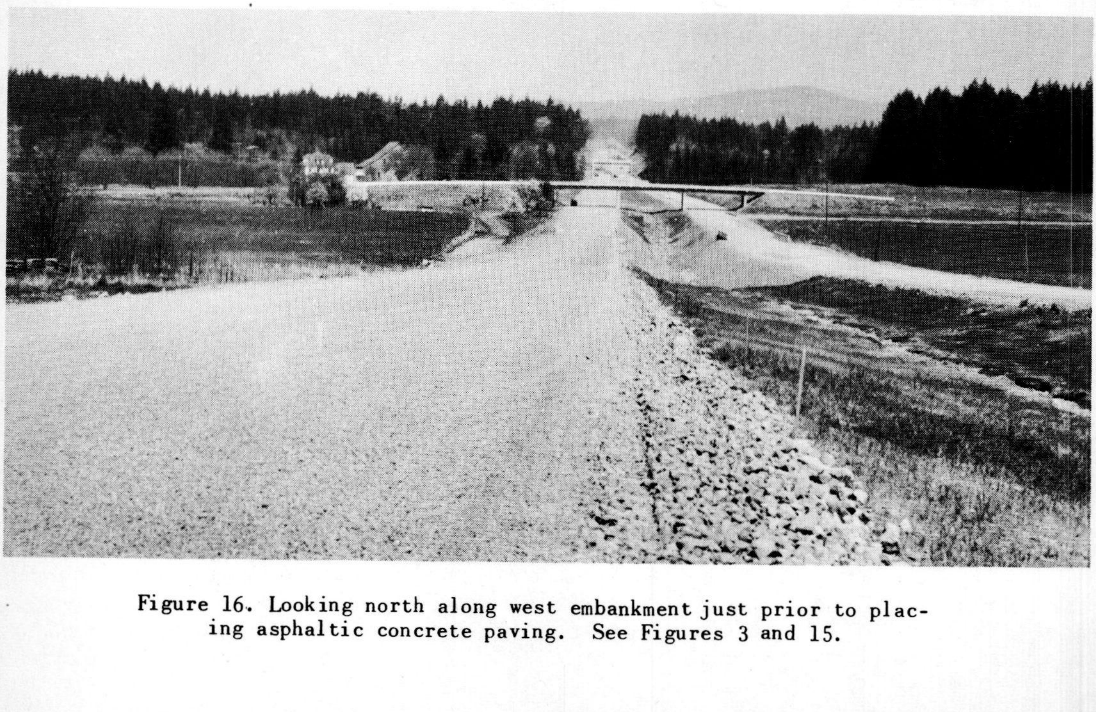


Figure 16. Looking north along west embankment just prior to placing asphaltic concrete paving. See Figures 3 and 15.

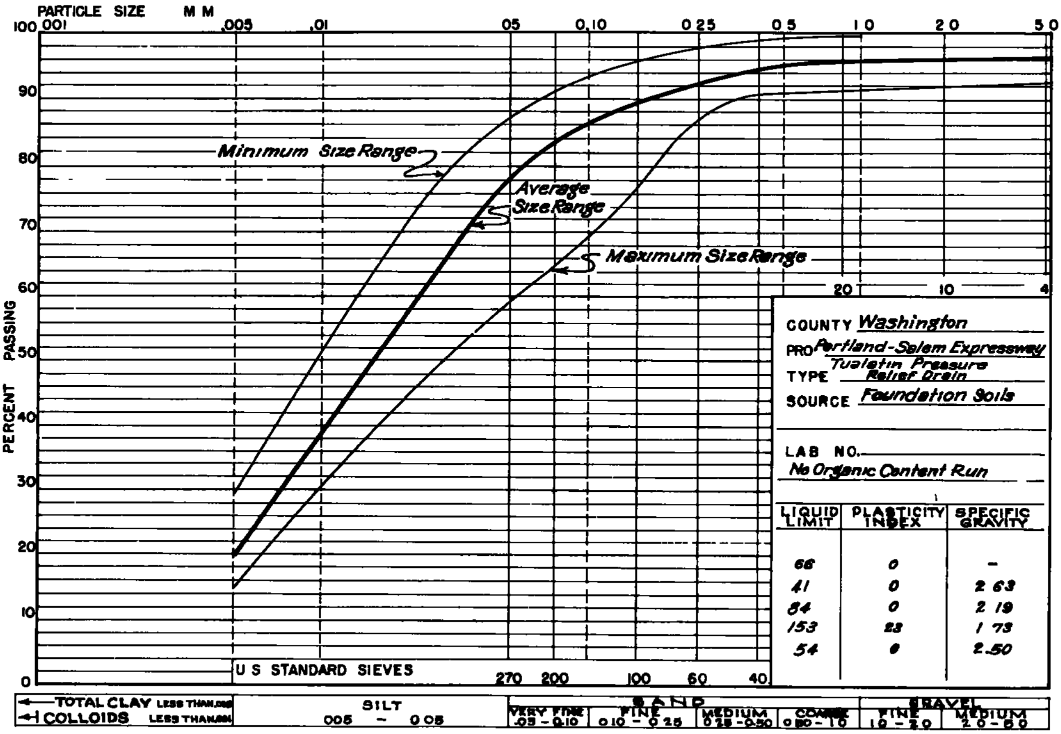


Figure 17.

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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