HIGHWAY RESEARCH BOARD

Bulletin 173

Analyses of Soil Foundation Studies

National Academy of Sciences—
National Research Council
publication 533

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PRESENTED AT THE
Thirty-Sixth Annual Meeting
January 7-11, 1957

1958

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Contents

Foundation Settlements in the Ft. Randall Dam Embankment

M. G. SPANGLER, Research Professor of Civil Engineering, Iowa State College, and
ALFRED L. GRIEBLING, Captain, Corps of Engineers, United States Army

An estimate of the magnitude and time rate of settlement due to compression strains in the foundation soil at 12 points beneath the embankment section of the Ft. Randall dam has been made. These estimates have been compared with actual measured settlements which had occurred up to January 1956, approximately a year and a half after completion of the embankment. The compression strains were estimated by means of the Terzaghi theory of consolidation, using geological profiles and soil test data furnished by the Omaha District, Corps of Engineers, U.S. Army. The measured settlements were also obtained from the Omaha District.

In general, the order of magnitude of the estimated ultimate settlements compares favorably with the measured settlements up to the present time. The estimated rates of settlement do not compare as favorably with the measured rates as do the ultimate settlements, due primarily, it is believed, to the difficulty of estimating the drainage conditions within and below the alluvial sediments which comprise the foundation soil.

The paper contains a description of the assumptions and methods employed in making the settlement estimates, together with the geological and test data and the measured settlements. Also presented is a complete typical calculation of stress distribution and settlement at one point.

● THE RELIABILITY and usefulness of methods of estimating the settlements of engineering structures, caused by compression strains in the underlying foundation soil, can only be checked by making comparisons between estimated settlements and actual observed settlements after a structure is completed. During construction and subsequent to completion of the earth embankment section of the Ft. Randall Dam, the Omaha District of the Corps of Engineers made a number of measurements of settlements of the natural ground surface at several points beneath the embankment. This agency also had made an extensive geological investigation of the dam site prior to construction, and consolidation tests of a large number of undisturbed samples from the foundation soil strata had been made. These data and other pertinent information were made available to the authors by the Omaha District and have been used to make prediction estimates of probable settlements at twelve selected points under the embankment. The predicted settlements have been compared with actual measured settlements up to January 1956. The purpose of this paper is to present the assumptions and methods employed in making the prediction estimates and to show the comparison between estimated and actual settlements.

The Ft. Randall Dam is located on the Missouri River about 6 miles south of Lake Andes, Charles Mix County, in southeastern South Dakota. It is a multiple purpose dam and is a major unit in the comprehensive Pick-Sloan Plan for development of the Missouri River Basin. The rolled earth structure has a maximum height of 160 ft and is 10,000 ft long. The width of the embankment section is 60 ft at the top, and, including an impervious upstream blanket and both upstream and downstream chalk berms, the maximum width at the base is 4,500 ft. The volume of earth fill is 28,000,000 cu. yd and of the dumped chalk in the berms is 22,000,000 cu yd.

A generalized geological section along the longitudinal axis of the right or west bank portion of the dam is shown in Figure 1. This section reveals an extensive alluvial terrace deposit of clays and silts extending from about Sta. 17 to Sta. 50 and ranging in thickness up to about 160 ft. The Omaha District of the Corps of Engineers made

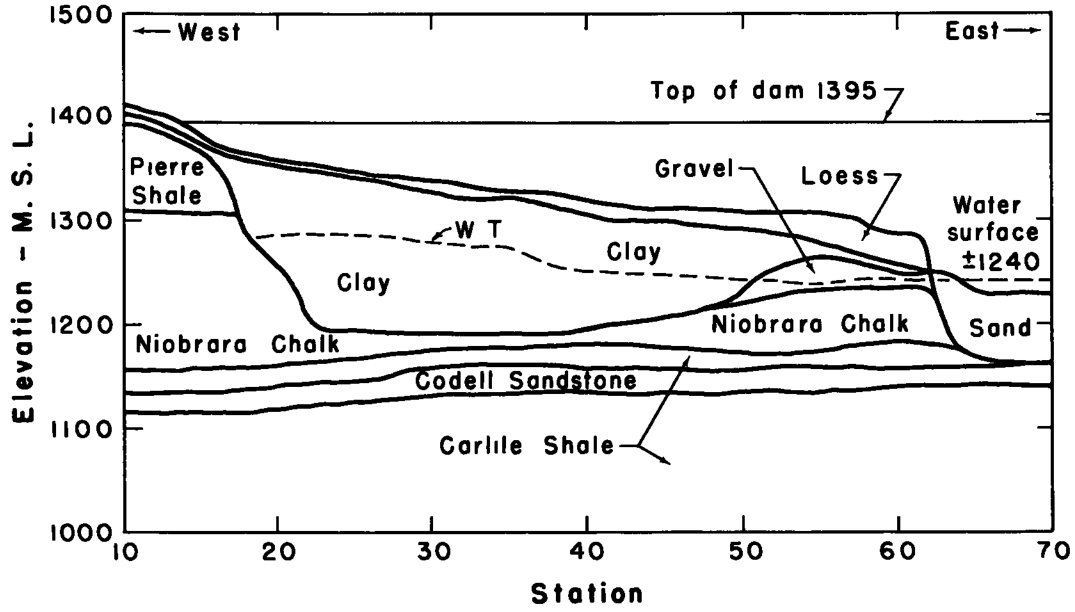


Figure 1.

extensive borings at the site. They also installed a number of combination piezometers and settlement plates with riser pipes through which the settlement of the natural ground surface could be measured both during construction of the embankment and after its completion. Twelve of these points at which settlements were measured have been selected for this comparative study. Their location in plan is shown in Figure 2.

Eight bore holes, chosen because of their proximity to the twelve settlement gage points studied, were selected as representing the character of the foundation soil under the embankment. The location of these bore holes is also shown in Figure 2. The boring logs for the holes and the positions from which undisturbed samples were taken for consolidation tests are shown in Figure 3.

Tests conducted by the Missouri River Division Laboratory of the Corps of Engineers in Omaha established the engineering properties of the foundation soils encountered. These are summarized in Table 1. The unit weights of embankment materials are also

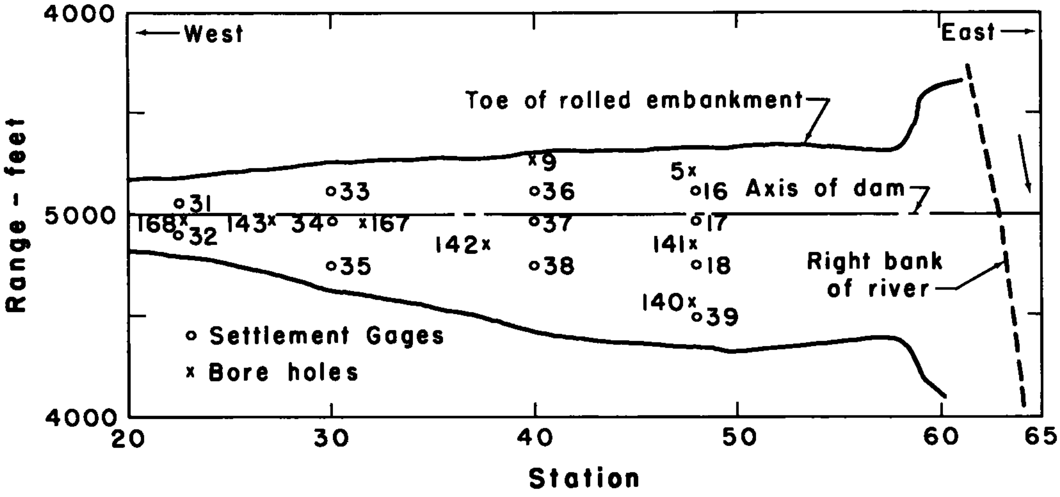


Figure 2.

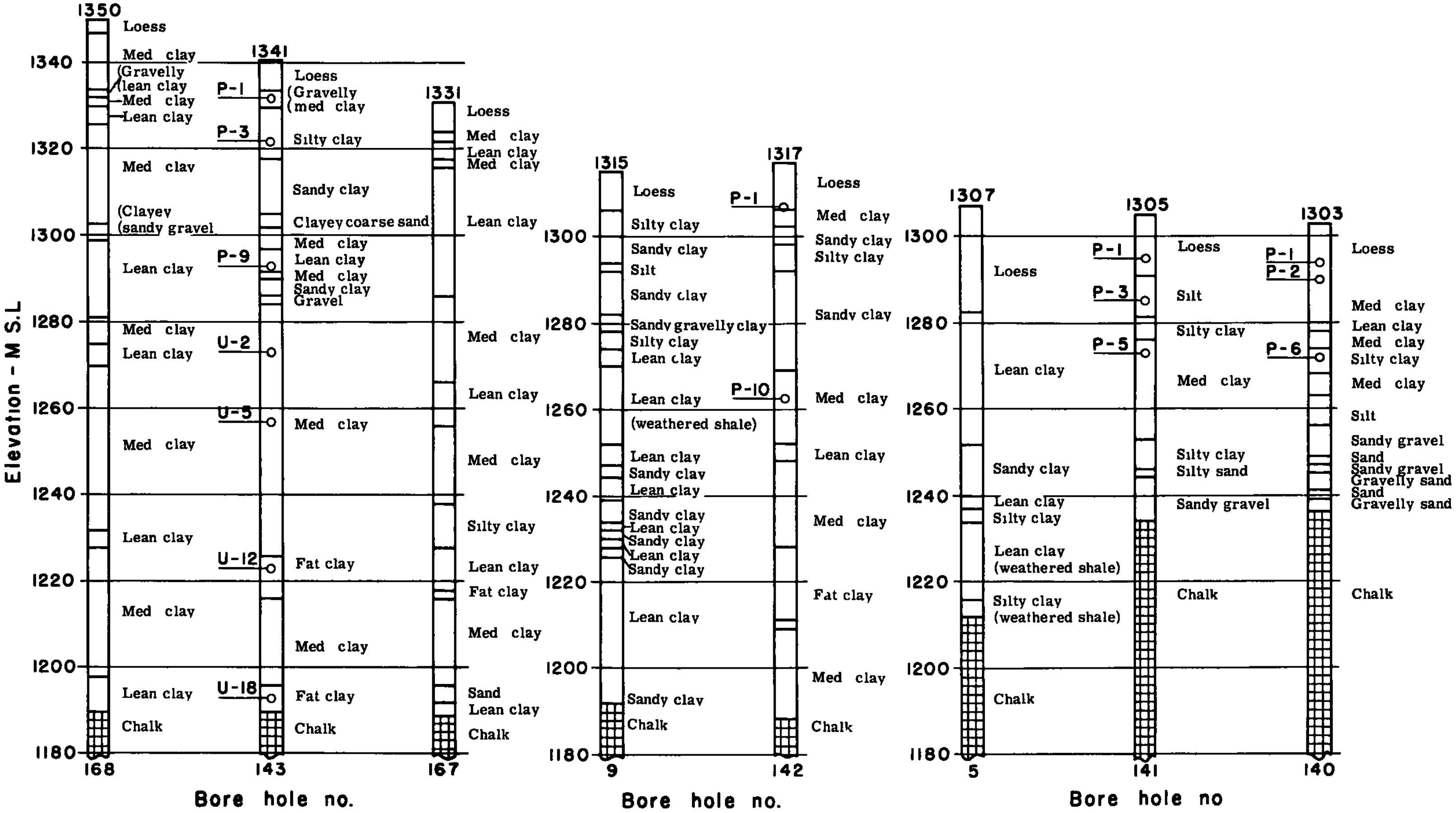

Elevation - M S.L
1340
1320
1300
1280
1260
1240
1220
1200
1180
1350
Loess
Med clay
(Gravelly
(lean clay
Med clay
Lean clay
Med clay
(Clayey
(sandy gravel
Lean clay
Med clay
Lean clay
Med clay
Lean clay
Med clay
Lean clay
Chalk
1341
P-1
P-3
P-9
U-2
U-5
U-12
U-18
Loess
(Gravelly
(med clay
Silty clay
Sandy clay
Clayey coarse sand
Med clay
Lean clay
Med clay
Sandy clay
Gravel
Med clay
Fat clay
Med clay
Fat clay
Chalk
1331
Loess
Med clay
Lean clay
Med clay
Lean clay
Med clay
Lean clay
Med clay
Silty clay
Lean clay
Fat clay
Med clay
Sand
Lean clay
Chalk
168
143
167
Bore hole no.
1315
Loess
Silty clay
Sandy clay
Silt
Sandy clay
Sandy gravelly clay
Silty clay
Lean clay
Lean clay
(weathered shale)
Lean clay
Sandy clay
Lean clay
Sandy clay
Lean clay
Sandy clay
Lean clay
Sandy clay
Lean clay
Sandy clay
Chalk
1317
P-1
P-10
Loess
Med clay
Sandy clay
Silty clay
Sandy clay
Med clay
Lean clay
Med clay
Fat clay
Med clay
Chalk
9
142
Bore hole no.
1307
Loess
Lean clay
Sandy clay
Lean clay
Silty clay
Lean clay
(weathered shale)
Silty clay
(weathered shale)
Chalk
1305
P-1
P-3
P-5
Loess
Silt
Silty clay
Med clay
Silty clay
Silty sand
Sandy gravel
Chalk
1303
P-1
P-2
P-6
Loess
Med clay
Lean clay
Med clay
Silty clay
Med clay
Silt
Sandy gravel
Sand
Sandy gravel
Gravelly sand
Sand
Gravelly sand
Chalk
5
141
140
Bore hole no

Figure 3.

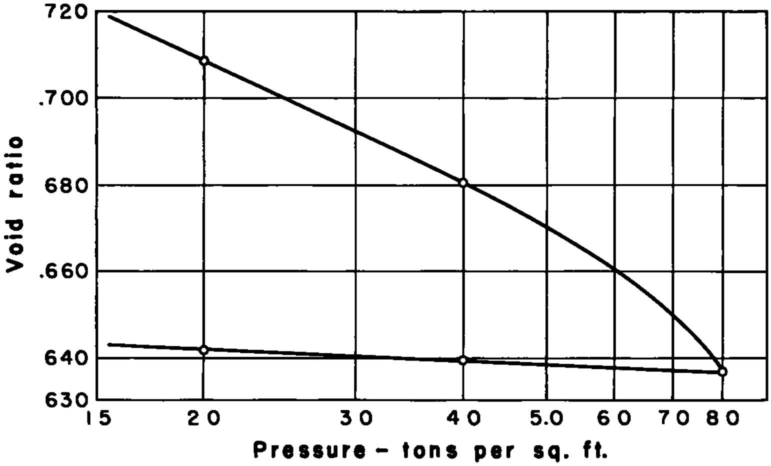


Figure 4. Void ratio - pressure diagram, sample 143, P-9.

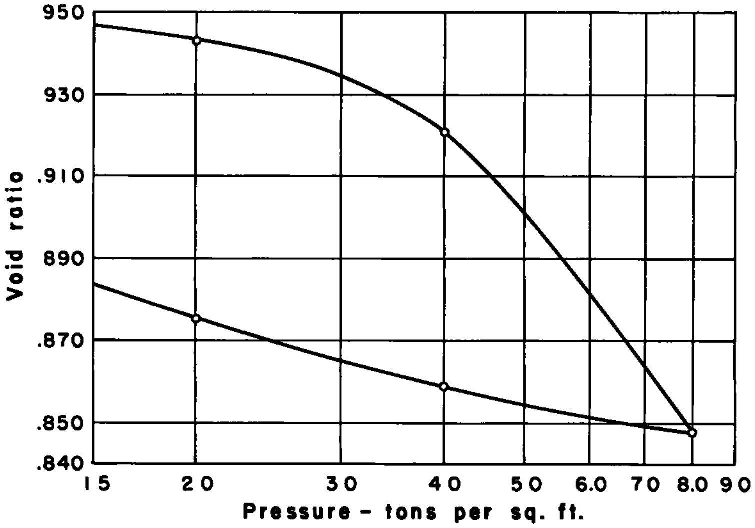


Figure 5. Void ratio - pressure diagram, sample 143, U-5.

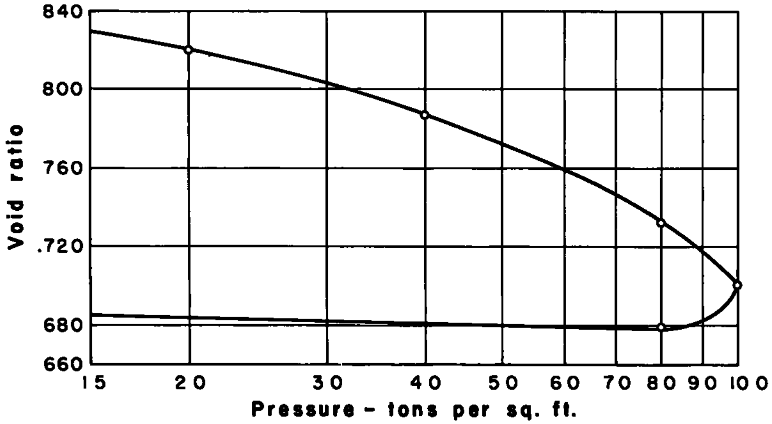


Figure 6. Void ratio - pressure diagram, sample 142, P-10.

TABLE 1

ENGINEERING PROPERTIES OF FOUNDATION AND EMBANKMENT SOILS FT. RANDALL DAM

Type of Material	Dry Density pcf.	Moisture Content, %	Liquid Limit	Plasticity Index	Unit Weight pcf.
Loess	74-82	11-16	33	10	89
Clay	90	20-35	45-90	23-58	115
Clay (submerged)	-	-	-	-	53
Rolled earth	-	-	-	-	120
Dumped chalk	-	-	-	-	112

TABLE 2

SELECTED CONSOLIDATION SAMPLES

Station	Settlement Gage No.	Bore Hole No.	Sample No.
22 + 50	31, 32	143	P-9, U-5
30 + 00	33, 34, 35	143	P-9, U-5
40 + 00	36, 37, 38	142	P-10
48 + 00	16, 17, 18	141	P-3, P-5
48 + 00	39	140	P-6

TABLE 3

INITIAL MID-PLANE PRESSURES AND VOID RATIOS AT THE VARIOUS SETTLEMENT GAGES

Gage No.	Sta.	Initial Pressure, P_1, Tsf.	Sample No.	Initial Void Ratio e_1
31	22 + 50	4.07	143, P-9	0.680
32			143, U-5	0.920
33			143, P-9	0.689
34	30 + 00	3.25	143, U-5	0.933
35				
36				
37	40 + 00	3.43	142, P-10	0.797
38				
16	48 + 00	2.04	141, P-3	0.665
			141, P-5	0.750
17	48 + 00	1.67	141, P-3	0.668
			141, P-5	0.763
18	48 + 00	1.32	141, P-3	0.671
			141, P-5	0.775
39	48 + 00	1.24	140, P-6	0.858

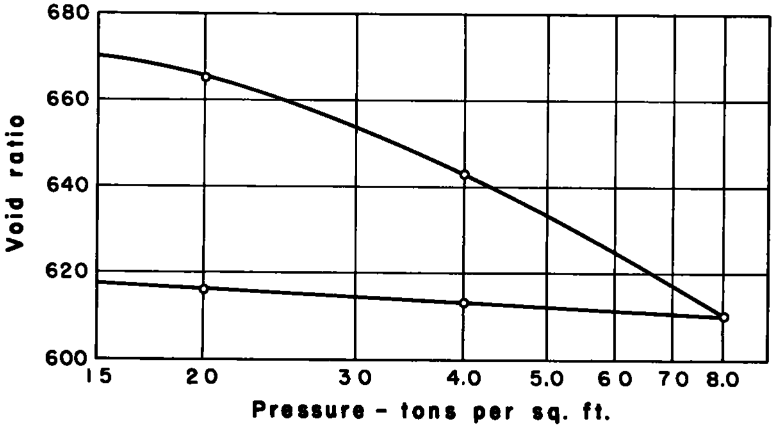


Figure 7. Void ratio - pressure diagram, sample 141, P-3.

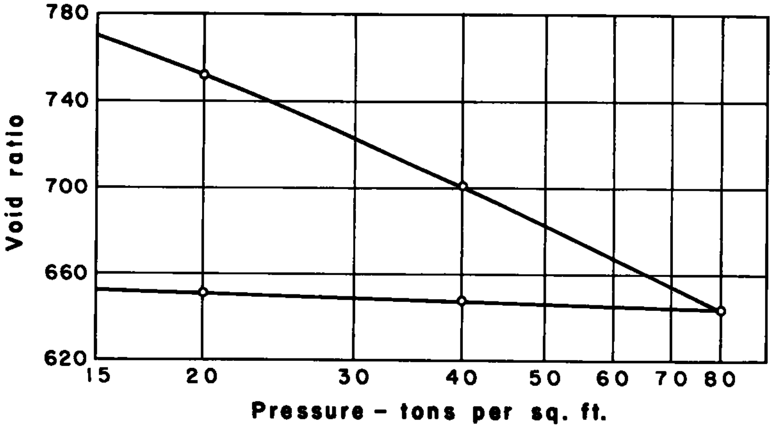


Figure 8. Void ratio - pressure diagram, sample 141, P-5.

TABLE 4

CONSOLIDATING PRESSURES AT MID-PLANE AT THE VARIOUS SETTLEMENT GAGES

Gage No.	Sta.	Height of Embankment, h. ft.	q_0 Tsf	b ft.	a_1 ft.	a_2 ft.	k	rad.	rad.	rad.	x ft.	Consol. Pressure Tsf.
31	22 + 50	48.5	2.91	30	300	200	1.50	1.527	0.550	0.506	-50	2.48
32								0.366	0.305	1.736	100	1.82
33	30 + 00	67	4.02	30	540	295	1.83	2.250	0.262	0.314	-110	3.38
34								0.698	0.681	1.430	35	3.67
35								0.157	0.061	2.268	250	1.13
36	40 + 00	88	5.28	30	725	400	1.81	2.372	0.227	0.323	-110	4.66
37								0.681	0.698	1.524	35	4.95
38								0.157	0.065	2.598	250	2.44
16	48 + 00	92.5	5.55	30	850	570	1.49	2.629	0.183	0.227	-110	5.04
17								0.467	0.878	1.710	35	5.44
18								0.079	0.035	2.925	250	3.43
39								0.026	0.009	2.818	510	0.88

TABLE 5

ESTIMATED ULTIMATE SETTLEMENTS AT THE VARIOUS SETTLEMENT GAGES

Gage No.	Sta.	Thickness of Clay Layer, d, ft.	Sample No.	Initial Pressure P_1, Tsf	Initial Void Ratio e_1	Consolidating Pressure P_C, Tsf	Final Pressure P_2, Tsf	Final Void Ratio e_2	Estimated Ultimate Settlement, ft.	Average Ultimate Settlement, ft.
31			143, P-9	4.07	0.680	2.48	6.55	0.655	2.38	3.24
			143, U-5		0.920			0.871	4.09	
32	22 + 50	160	143, P-9		0.680	1.82	5.89	0.662	1.71	
			143, U-5		0.920			0.884	3.00	2.36
33			143, P-9	3.25	0.689	3.38	6.63	0.654	2.94	
			143, U-5		0.933			0.870	4.73	3.84
34	30 + 00	142	143, P-9		0.689	3.67	6.92	0.651	3.19	
			143, U-5		0.933			0.865	5.02	4.11
35			143, P-9		0.689	1.13	4.38	0.677	1.01	
			143, U-5		0.933			0.914	1.40	1.21
36			142, P-10	3.43	0.797	4.66	8.09	0.732	4.52	4.52
37	40 + 00	125	142, P-10		0.797	4.95	8.38	0.729	4.74	4.74
38			142, P-10		0.797	2.44	5.87	0.759	2.65	2.65
16		82	141, P-3	2.05	0.665	5.04	7.09	0.617	2.36	3.45
			141, P-5		0.750			0.653	4.54	
17		69	141, P-3	1.67	0.668	5.44	7.11	0.617	2.11	3.23
	48 + 00		141, P-5		0.763			0.652	4.35	
18		54	141, P-3	1.32	0.671	3.43	4.75	0.636	1.45	
			141, P-5		0.775			0.687	2.68	2.07
39		49	140, P-6	1.24	0.858	0.88	2.12	0.829	0.77	0.77

included in this table. As indicated, the dumped chalk weighed about 8 lb per cu ft less than the rolled earth, but since the chalk occupied only the outside fringes of the embankment, a uniform unit weight of 120 pcf was assumed for all embankment materials for the purpose of this estimate.

Six of the consolidation tests were chosen to represent the compression characteristics of the foundation soil. These were selected because of their proximity to the location of the settlement gages and because they were deemed to be representative of the foundation material. The numbers of the consolidation test samples used to estimate the settlements at the several gage locations are shown in Table 2. The void ratio-pressure diagrams for the six test samples are shown in Figures 4 to 9.

Estimates of the ultimate compression strain in the foundation material at each of the gage points were computed from the formula:

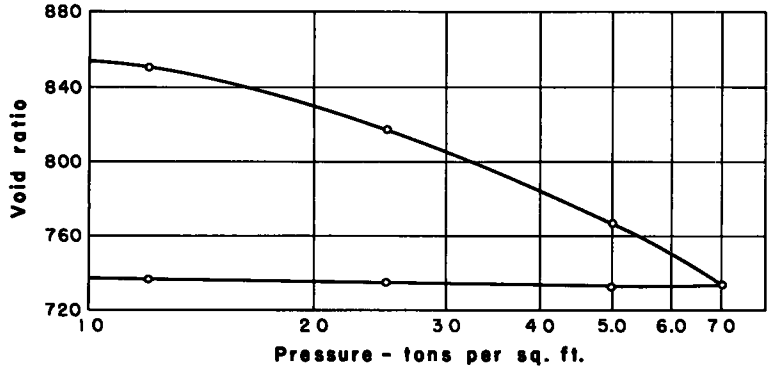


Figure 9. Void ratio - pressure diagram, sample 140, P-6.

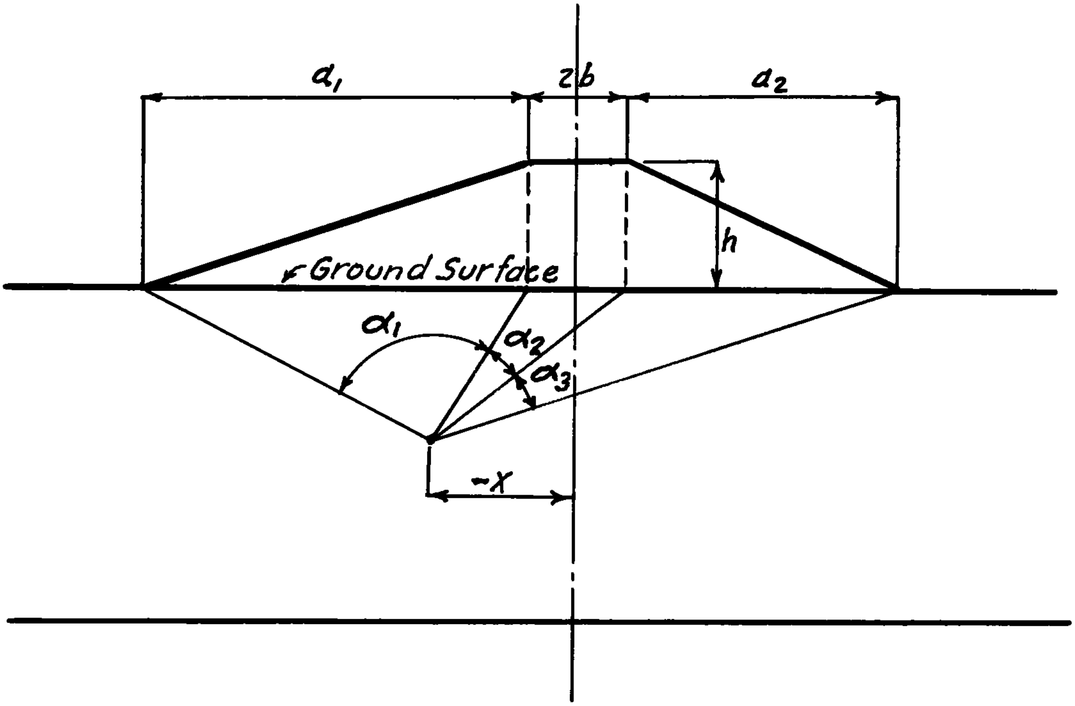


Figure 10.

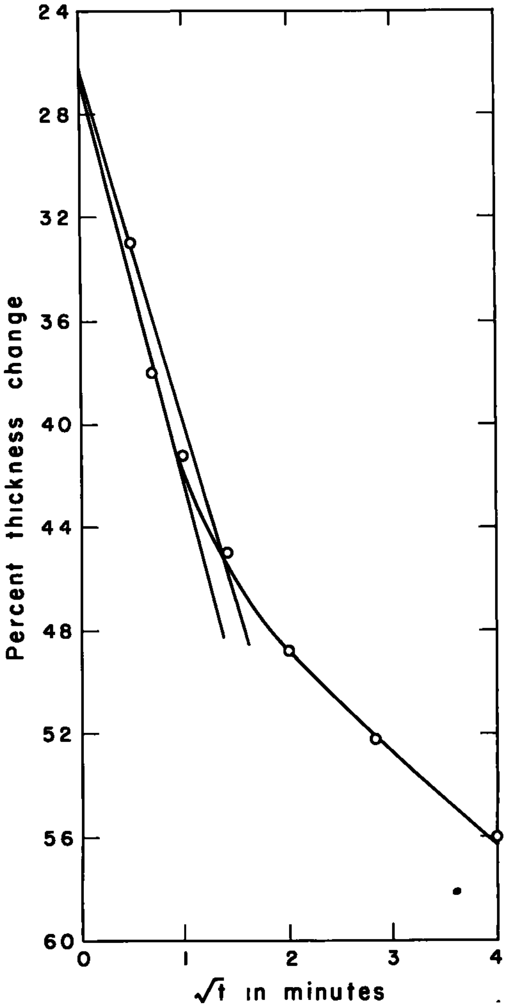


Figure 11. Laboratory time curve, sample 141, P-3. Load IT/sq ft.

TABLE 6

VALUES OF THE TIME FACTOR

Percent of Ultimate Settlement, q.	Time Factor
0.1	0.008
0.2	0.031
0.3	0.071
0.4	0.126
0.5	0.197
0.6	0.287
0.7	0.403
0.8	0.567
0.9	0.848

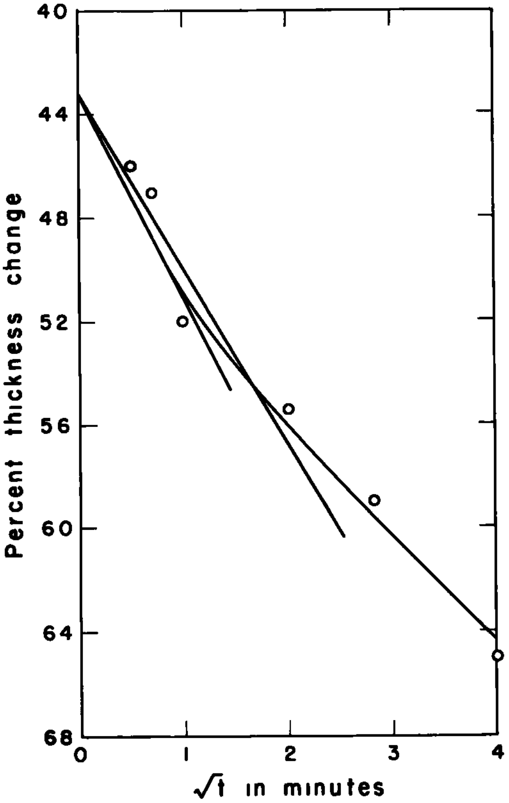


Figure 12. Laboratory time curve, sample 141, P-3. Load 2T/sq ft.

$$s = d\,\frac{(e_1 - e_2)}{(1 + e_1)} \quad \ldots\ldots(1)$$

in which

s = the ultimate primary compression strain

d = thickness of the compressible stratum

e_1= void ratio at mid-plane of compressible stratum at initial pressure, p_1, prior to construction of embankment.

e_2= void ratio at mid-plane of compressible stratum at final pressure, p_2, after completion of embankment.

The initial pressure p_1 was determined by calculating the weight of all material lying above the mid-plane using the unit weights of foundation material shown in Table 1. Then the corresponding values of initial void ratio were taken from the void ratio-pressure diagrams for the appropriate consolidation test samples. A typical computation of this kind is given for gage numbers 31 and 32 at Sta. 22 + 50. The elevation of the ground surface at this station was 1,350; the top of the clay stratum, 1,347; the water table, 1,286; the bottom of the clay stratum, 1,190; and the mid-plane of the clay, 1,270. Thus we have:

Loess	3 x 89 =	267 psf
Clay above W. T.	61 x 115 =	7,015
Clay below W. T.	16 x 53 =	848
		8,130 psf

Thus p_1 = 4.065 Tsf and e_1 from the consolidation curves is 0.680 for sample 143, P-9 and 0.920 for sample 143, U-5. The initial pressures and void ratios at mid-depth for all gage points are shown in Table 3.

The consolidating pressures at the mid-plane of the compressible stratum were calculated by the method developed by D. L. Holl (1) for stress in an elastic medium caused by a trapezoidal load applied at the surface. This method involves the determination of the angles at the point in the foundation where the stress is desired, which are subtended by the projection on the ground surface of the distances between the shoulders and toes of the embankment and the distance between the shoulders, as illustrated in Figure 10. With these angles expressed in radians, the dimensions of the embankment and the unit weight of the embankment soil known, the vertical stress at any point, due to the embankment load, is

$$\sigma y = -\frac{q_0}{\pi}\left[(\alpha_1 + \alpha_2 + \alpha_3) + \frac{b}{a_1}(\alpha_1 + k\alpha_3) + \frac{x}{a_1}(\alpha_1 - k\alpha_3)\right] \quad . \; . \; . \; . \; . \quad (2)$$

in which

σy = vertical stress at a point in the foundation

q_o = wh = unit weight of soil times height of embankment

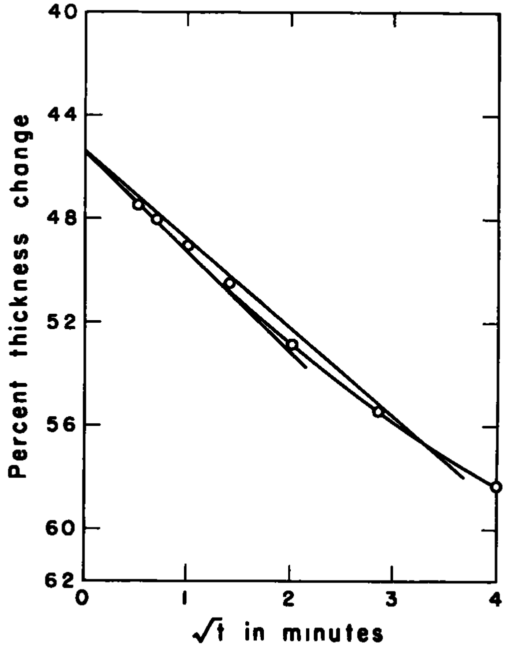


Figure 13. Laboratory time curve, sample 141, P-3. Load 4T/sq ft.

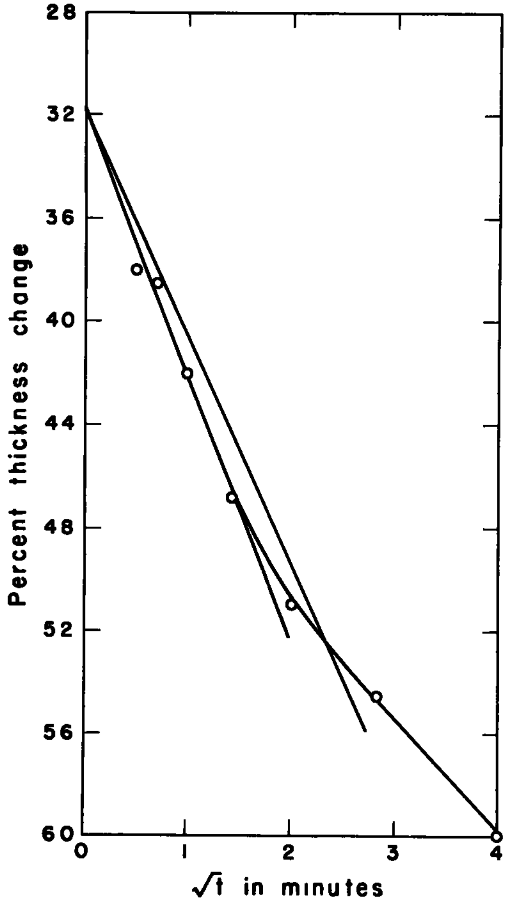
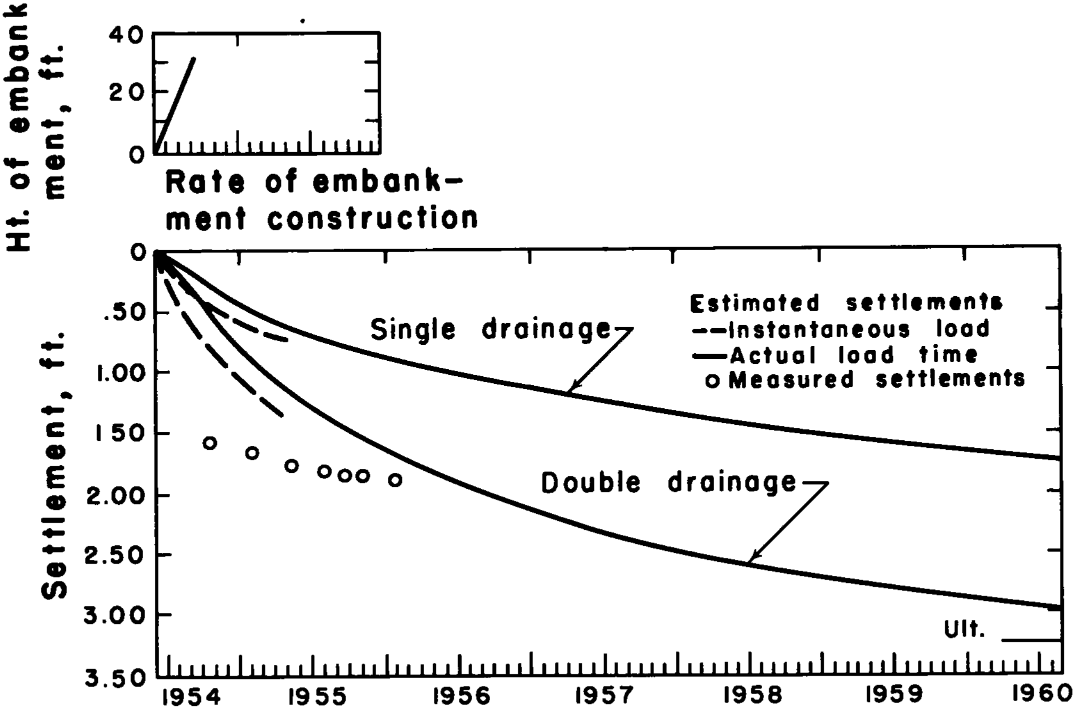
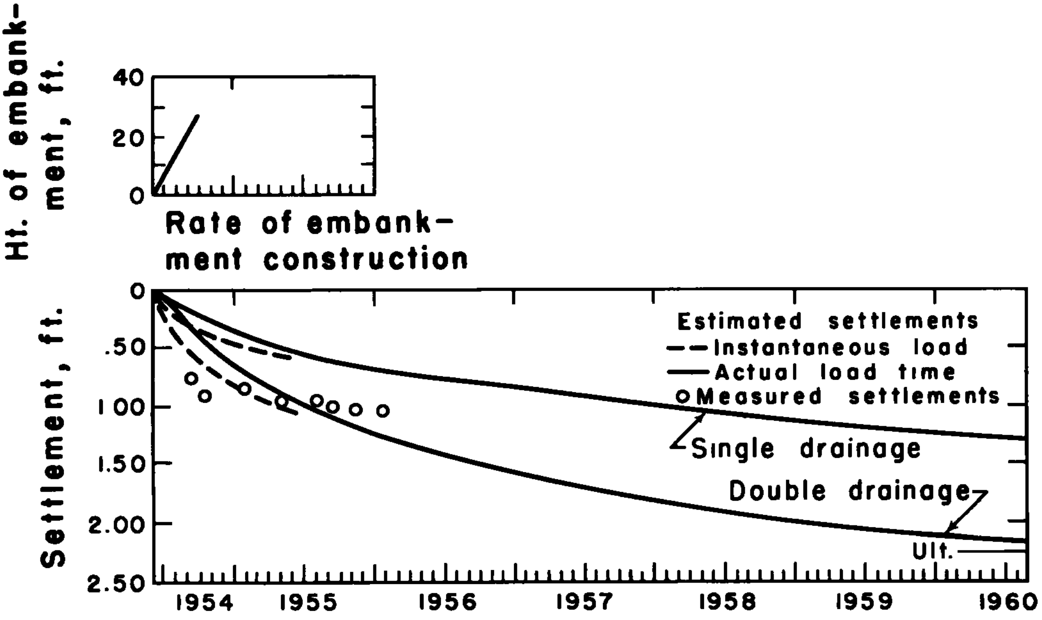


Figure 14. Laboratory time curve, sample 141, P-3. Load 8T/sq ft.



Time-settlement curves. Gage 31

Figure 15.



Time-settlement curves. Gage 32

Figure 16.

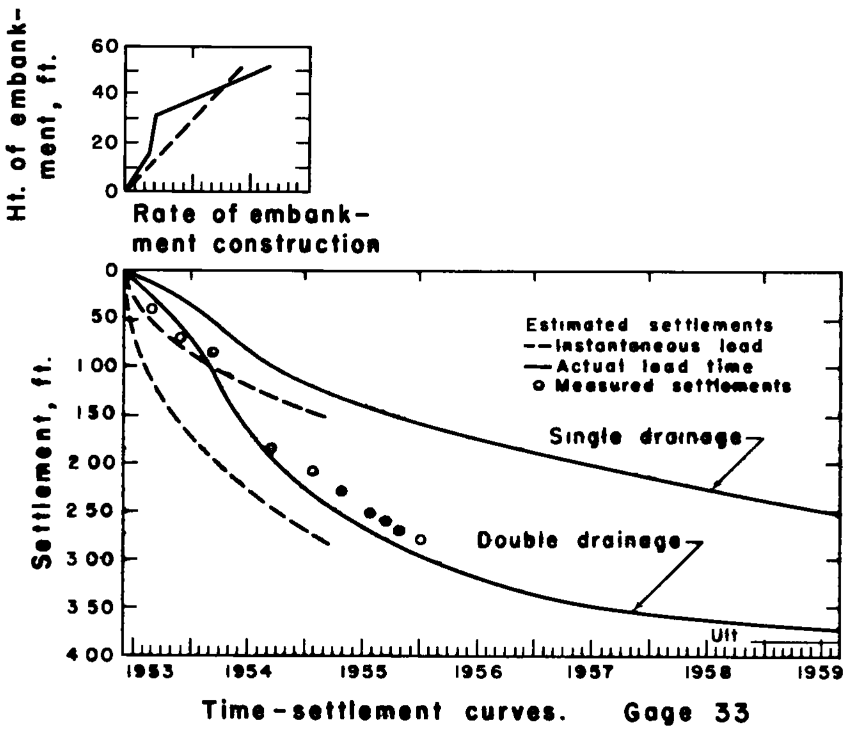


Figure 17.

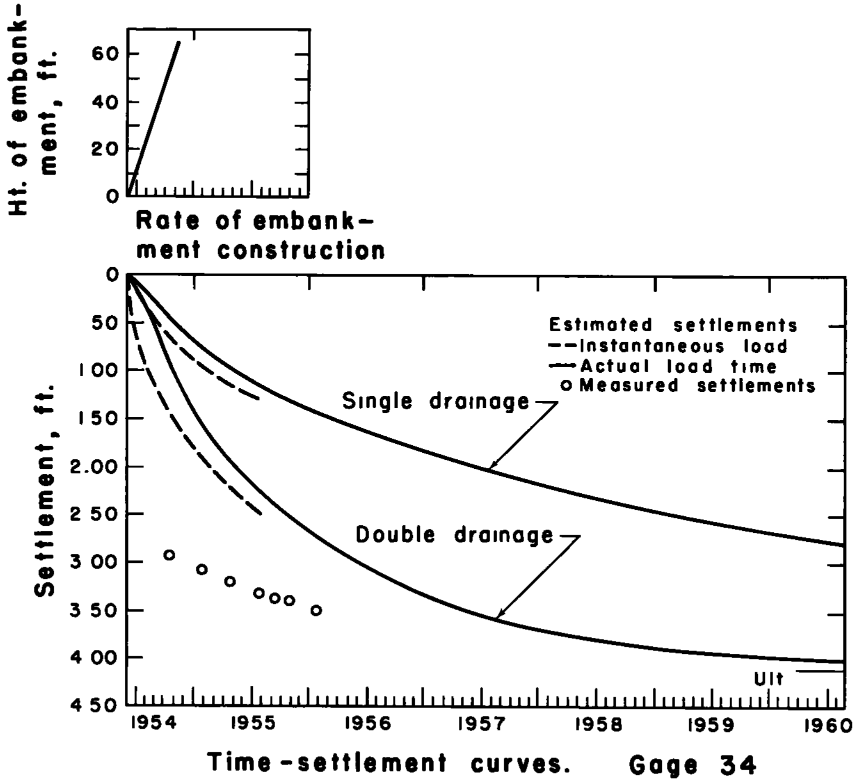


Figure 18.

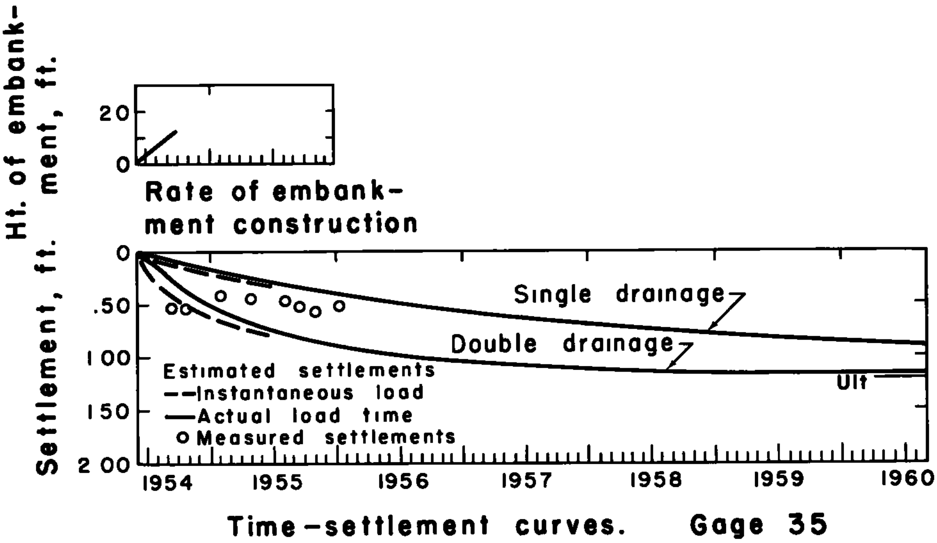


Figure 19.

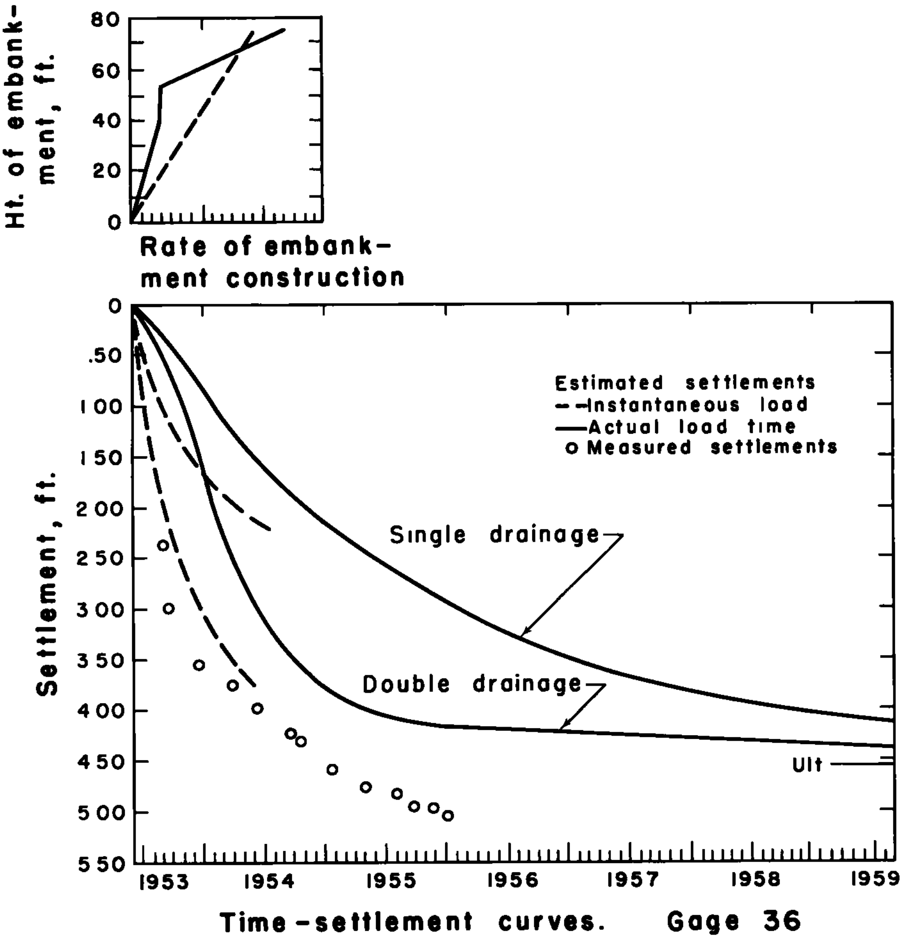


Figure 20.

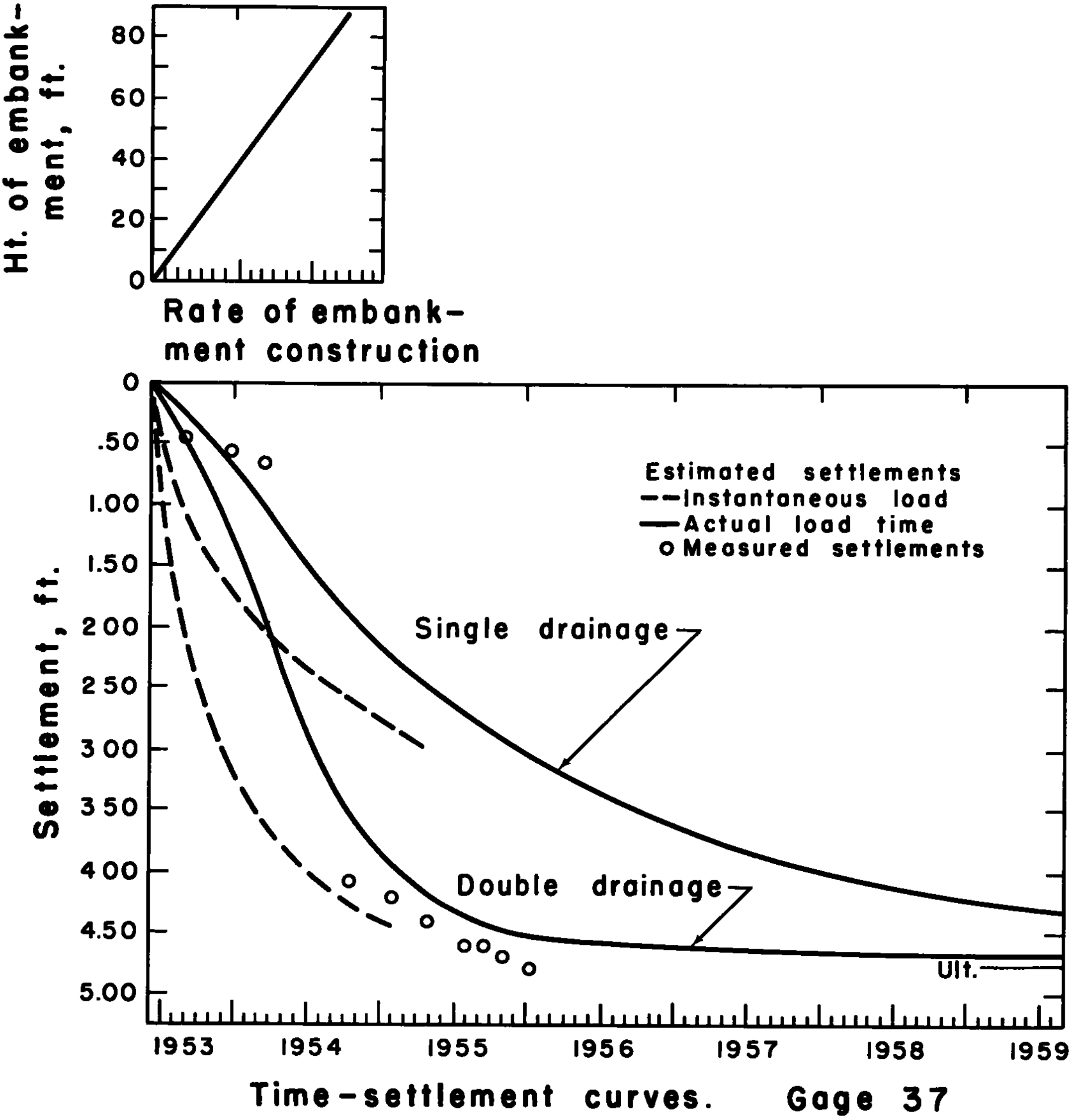


Figure 21.

α_1 = angle subtended by horizontal distance from shoulder to upstream toe
α_2 = angle subtended by horizontal projection of crest of embankment on its base
α_3 = angle subtended by horizontal distance from shoulder to downstream toe
b = ½ crest width
x = distance from axis of dam to point
a_1 = horizontal distance from shoulder to upstream toe
a_2 = horizontal distance from shoulder to downstream toe
k = $\frac{a_1}{a_2}$

The Ft. Randall embankment was not a simple trapezoid in cross-section, as indicated in Figure 10, because of variable slopes of the rolled earth section and because of the chalk berms, which had a much flatter slope. However, it was possible to estimate an equivalent trapezoidal cross-section and to calculate stresses in the under soil without, it is believed, appreciable error. The consolidating stresses thus calculated at the midplane of the clay stratum under the various gage points are shown in Table 4.

The final stress, p_2, is equal to the initial stress, p_1, plus the consolidating stress, p_c. That is

$$p_2 = p_1 + p_c \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

TABLE 7

COMBINED AVERAGE COEFFICIENTS OF CONSOLIDATION

Station	Gage No.	Pressure Range Tsf.	Sample No.	Test Loads Tsf.	Average c ft.^{2}/mo.	Combined Av. c ft.^{2}/mo.
22 + 50	31, 32	4.0 to 6.5	143, P-9 143, U-5	4, 8	107 49	78
30 + 00	33, 34	3.3 to 7.0	143, P-9 143, U-5	2,4,8	157 55	106
30 + 00	35	3.3 to 4.4	143, P-9 143, U-5	2, 4	205 61	133
40 + 00	36, 38	3.4 to 8.4	142, P-10	2,4,8	211	211
48 + 00	16, 17	1.7 to 7.1	141, P-3 141, P-5	2,4,8	85 125	105
48 + 00	18	1.3 to 4.7	141, P-3 141, P-5	1,2,4	134	168
48 + 00	39	1.2 to 2.1	140, P-6	1.2,2.5	203 274	274

TABLE 8

TIME - SETTLEMENT COMPUTATIONS AT GAGES 31 AND 32

$d = 160$ ft $\quad \frac{d}{2} = 80$ ft $\quad c = 78$ sq ft per month (samples 143, P-9 and 143, U-5)

Double Drainage

$$t = T \frac{(\frac{d}{2})^2}{c}$$

t mo. = 82.2 T

Single Drainage

$$t = T \frac{d^2}{c}$$

t mo. = 328.8 T

% Ult. Settlement	Elapsed Time, mo. Double	Elapsed Time, mo. Single	Settlement, ft Gage 31	Settlement, ft Gage 32
10	0.7	2.6	0.32	0.24
20	2.5	10.0	0.65	0.47
30	5.8	23.2	0.97	0.71
40	10.3	41.2	1.30	0.94
50	16.2	64.8	1.62	1.18
60	23.6	94.5	1.95	1.42
70	33.0	132.0	2.27	1.65
80	46.5	186.0	2.60	1.89
90	70.0	280.0	2.92	2.12
100	-	-	3.24	2.36

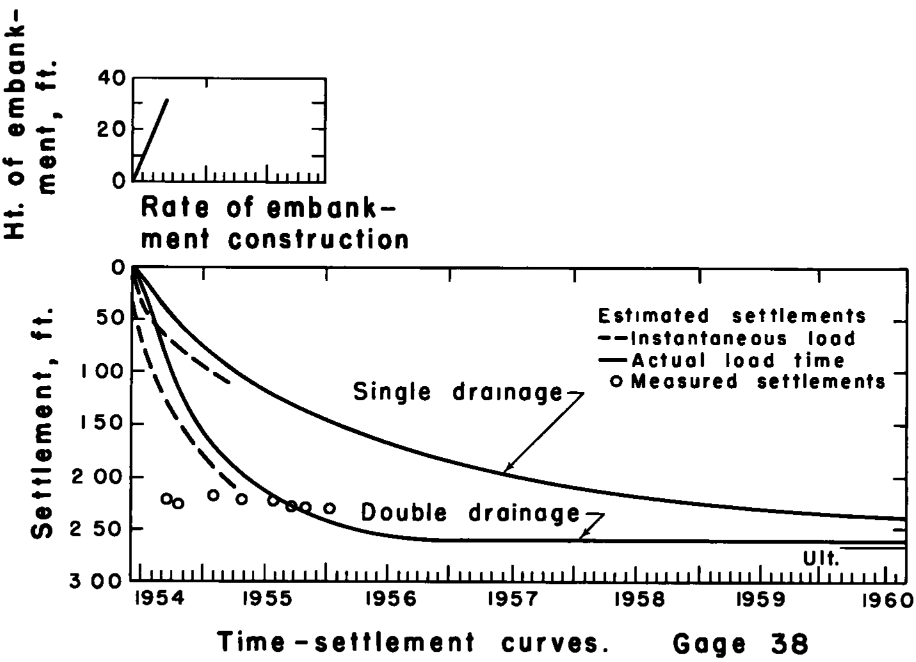


Figure 22.

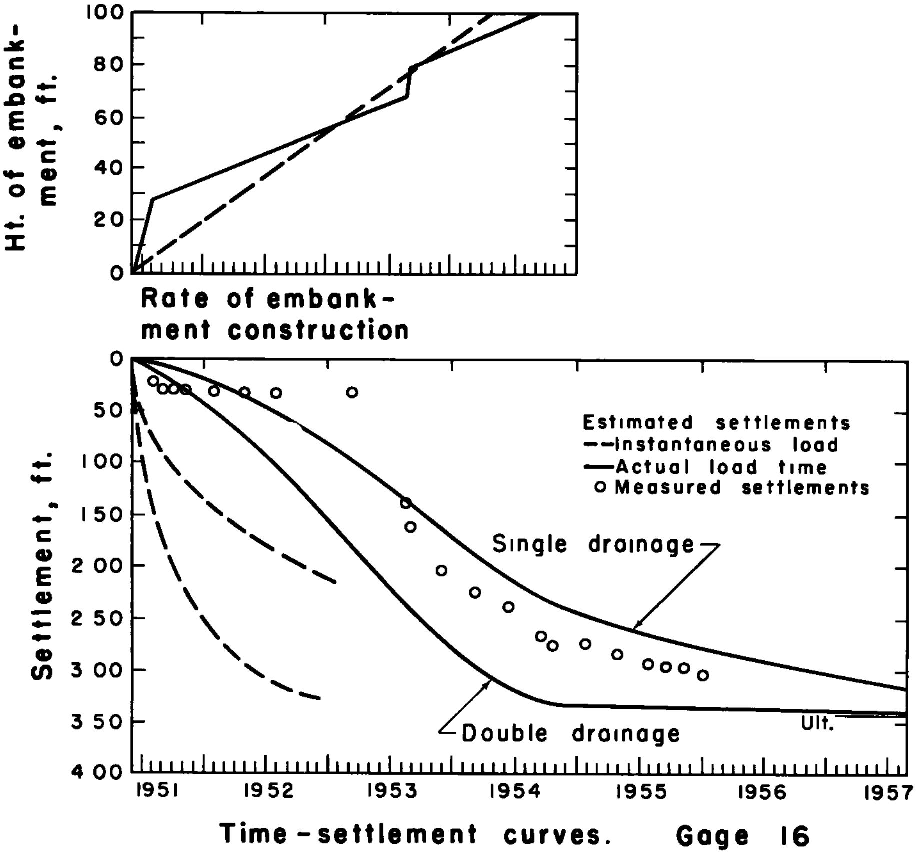
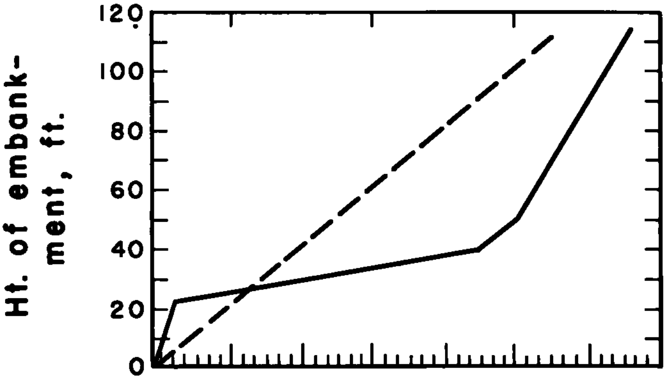


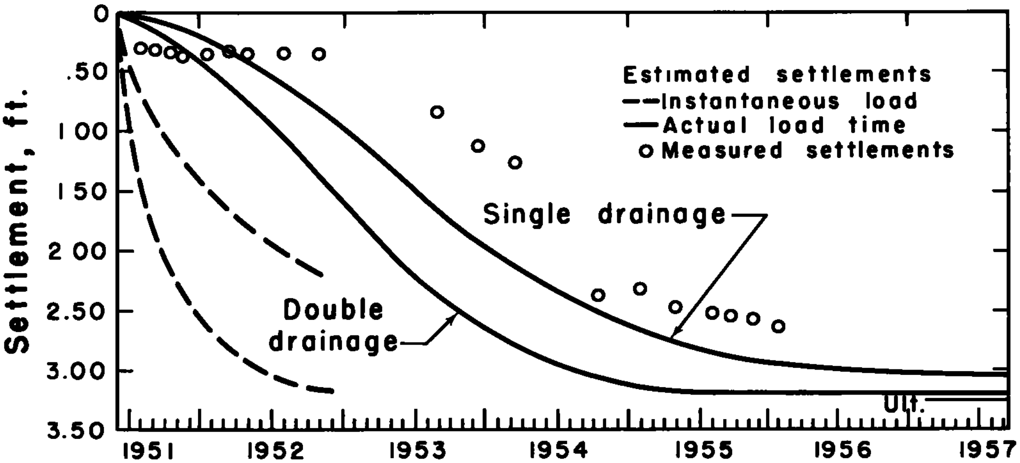
Figure 23.

The final void ratio, e_2, may then be determined from the appropriate void ratio-pressure diagrams, and the ultimate compression strain or settlement of the base of the embankment calculated from Eq. 1. The values of ultimate settlement thus estimated are shown in Table 5.

Estimates of the time-settlement relationship at the several gage points were made in accordance with the Terzaghi theory of one-dimensional consolidation. In the application of this theory it is necessary to know whether the compressible stratum is free to drain in both the upward and downward directions (double drainage) or whether it will drain in one direction only (single drainage). A study was made of the boring logs of a large number of bore holes in the foundation material. This study failed to reveal definitely whether double drainage or single drainage conditions prevailed, since there did not appear to be a continuous stratum of free-draining material below the compressible clay, and the underlying Niobrara chalk was reported to be relatively impervious. Some of the bore holes penetrated relatively thin layers of sand and gravel at varying elevations, but these same layers were not always encountered in adjacent holes. Therefore it was decided to estimate the time-settlement relationships for both the single and double drainage conditions, although it was suspected that the latter case might be the more appropriate, since alluvial deposits of this kind frequently contain partings and inclusions of sandy material which may provide more or less continuous and tortuous



Rate of embankment construction



Time-settlement curves. Gage 17

Figure 24.

paths for the escape of water but which may not be revealed by subsurface exploration.

The time required for various percentages of the estimated ultimate settlements to develop were calculated by the following formulas;

For double drainage

$$t = T\frac{\left(\frac{d}{2}\right)^2}{c} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

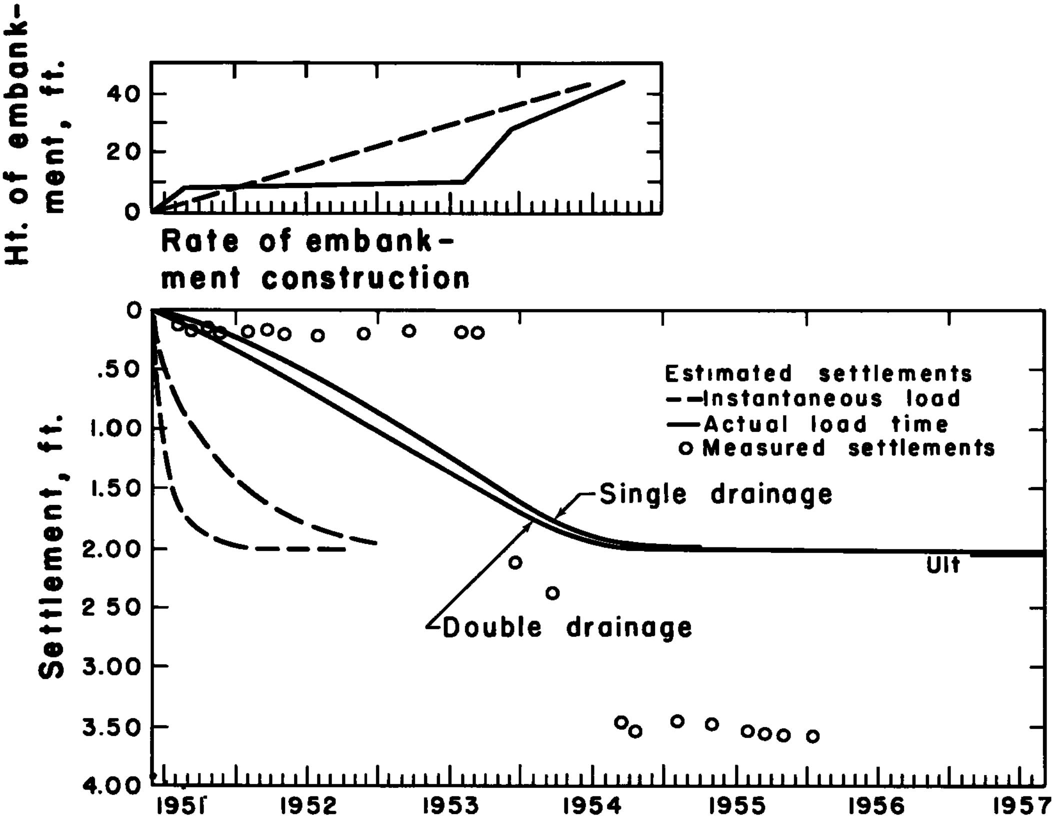
For single drainage

$$t = T\frac{d^2}{c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

in which

t = elapsed time in which a specified percentage of ultimate settlement develops
d = thickness of compressible layer
c = coefficient of consolidation
T = the "Time Factor" in the Terzaghi equation

Values of the coefficient of consolidation were determined from the consolidation test data by means of Taylor's square-root-of-time fitting method. From two to four values of the coefficient were obtained for each test sample and then an average value representing the range of pressure at each gage point was determined for use in the rate of settlement calculation. Four typical time versus compression curves, representing sample 141, P-3 under test loads of 1, 2, 4 and 8 tons per sq. ft. are shown in Figures 11 to 14. The combined average coefficients of consolidation for all test samples and



Time-settlement curves. Gage 18

Figure 25.

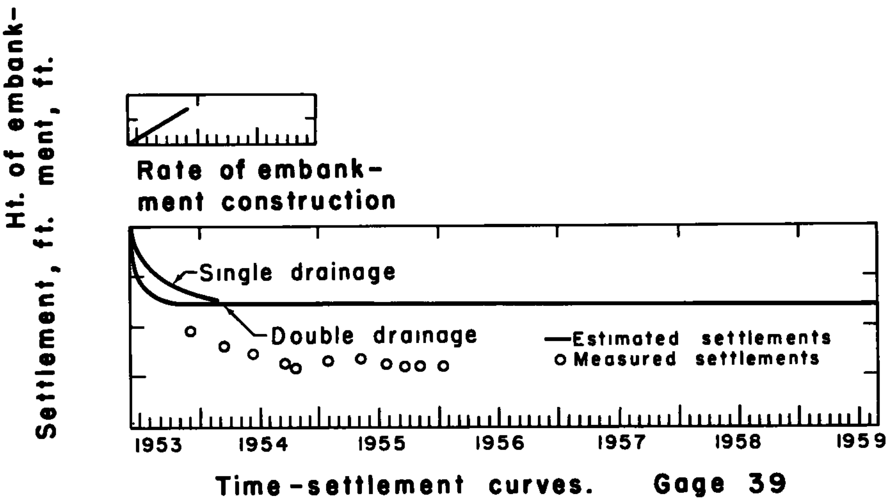


Figure 26.

gage points are given in Table 7.

Typical computations of time versus settlement - those for gages 31 and 32 - are shown in Table 8, for both single and double drainage and for instantaneous application of the load. Obviously the embankment loads were not applied instantaneously or even in a very short time. The time of construction of the embankment varied from about 4 months to 40 months. Therefore it was necessary to modify the time-settlement curves for instantaneous loading to account for the effect of actual loading time. This was done by the approximate method suggested by Terzaghi and Gilboy and outlined by Taylor (3).

The estimated settlements at all gage points, modified to take into account the time of construction of the embankment, are shown graphically in Figures 15 to 26. The actual measured settlements at the gage points up to January 1956 are also shown on these graphs.

The authors consider that the general agreement between the estimated settlements and the actual measured settlements is very good. Certainly it can be said that the order of magnitude of both the estimated and actual values is the same. Specifically, the authors rate the comparison as good at gages 31, 32, 33, 36, 37, 38, 16 and 17; as fair at gages 34 and 35; and as poor at gages 18 and 39. The results tend to strengthen confidence in the Theory of Consolidation and the various methods of approach employed in this study.

Also, the measured rates of settlement lead to the conclusion that the assumption of double drainage of the compressible layer was the most appropriate in this case. Although the available boring logs did not reveal a definite, continuous drainage element at the bottom of the clay, it is believed that the alluvial character of the soil was such that downward drainage actually did develop to a considerable extent and that the rate of settlement was hastened thereby.

REFERENCES

1. Holl, D. L., Plane-Strain Distribution of Stress in Elastic Media, Bulletin 148, Iowa Engineering Experiment Station, 1941.
2. Spangler, Merlin G., Soil Engineering, International Textbook Co., Scranton, Pennsylvania, 1951.
3. Taylor, Donald W., Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York, 1948.
4. Griebling, Alfred L., Foundation Settlements - Ft. Randall Dam Embankment, unpublished thesis, Library, Iowa State College, 1956.

Measurement of Forces Produced in Piles By Settlement of Adjacent Soil

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● SETTLEMENT of a soil layer through which a pile penetrates in passing to a firm stratum will tend to transfer load to the pile by negative skin friction. That is, the action of the pile is to support the settling soil, and the direction of the stresses acting on the surface of the pile is downward. The drag of the soil on the pile can produce forces of large magnitude; therefore, this type of loading may be an important consideration in the design of a pile foundation.

The most common loading condition which can lead to drag forces of considerable magnitude is the case in which a fill is placed over a compressible layer, and piles are driven through this layer to a firm stratum prior to complete consolidation of the soil. A similar condition will exist if the fill is placed after the piles are driven. Chellis (2) lists several examples of pile failure attributed to negative friction and Moore (7) describes a failure of a pile under an estimated drag load of nearly 200 tons. Florentin and L'Heriteau (5) also report a case of pile failure produced by negative friction and indicate that the magnitude of the drag forces may be large.

Design of pile foundations for drag forces is sometimes based on a single pile analysis, such as the method suggested by Moore (7), or the design may be based on a cluster or group pile analysis as outlined by Terzaghi and Peck (13). In either event there may be considerable uncertainty concerning some of the design factors such as lateral soil pressure, friction between the piles and soil, bond between cohesive soils and piles, influence of pile driving on shear strength of soil, and the extent to which shear strength of the soil is mobilized.

At the site of a proposed abutment for a bridge on the Connecticut Turnpike a fill approximately 50 ft high was placed on top of a layer of marine mud. It was anticipated that consolidation of the soft mud layer would not be complete at the time the supporting piles for the abutment were driven to a firmer stratum underlying the mud. Therefore, a particularly severe loading condition for drag forces might be imposed on the piles. In view of the many uncertainties involved in the design of the pile foundation for drag forces, an experimental investigation of the nature and magnitude of these forces was made. This paper describes the investigation which was carried out as a joint research project of the Connecticut State Highway Department and the Civil Engineering Department of the University of Connecticut. In July 1955 three 12-in. Monotube fluted piles equipped with electric strain gages on the inside surfaces were driven at the site of the bridge abutment. Readings of the strain gages were taken over a period of several months, and the corresponding drag loads in the piles were computed. Several borings were made at the site of the tests, and soil tests were made to determine certain changes in the soil characteristics resulting from consolidation of the mud layer. In addition, several control measurements, including pore water readings, were taken.

THE TEST SITE AND SOIL CONDITIONS

The pile tests were conducted at the site of the east abutment of the Connecticut Turnpike crossing of the main line of the New Haven Railroad in West Haven, Connecticut. The crossing consisted of a 6-span steel beam and slab bridge with the five piers and two abutments supported on piles. The particular site selected for the pile drag tests was at the north end of the east abutment (Fig. 1) where three piles equipped with electric strain gages (shown circled by the dotted line) and eight additional piles were driven in July 1955. The original ground elevation at the test site was approximately 6-ft above mean sea level, with a deposit of marine mud underlying the area

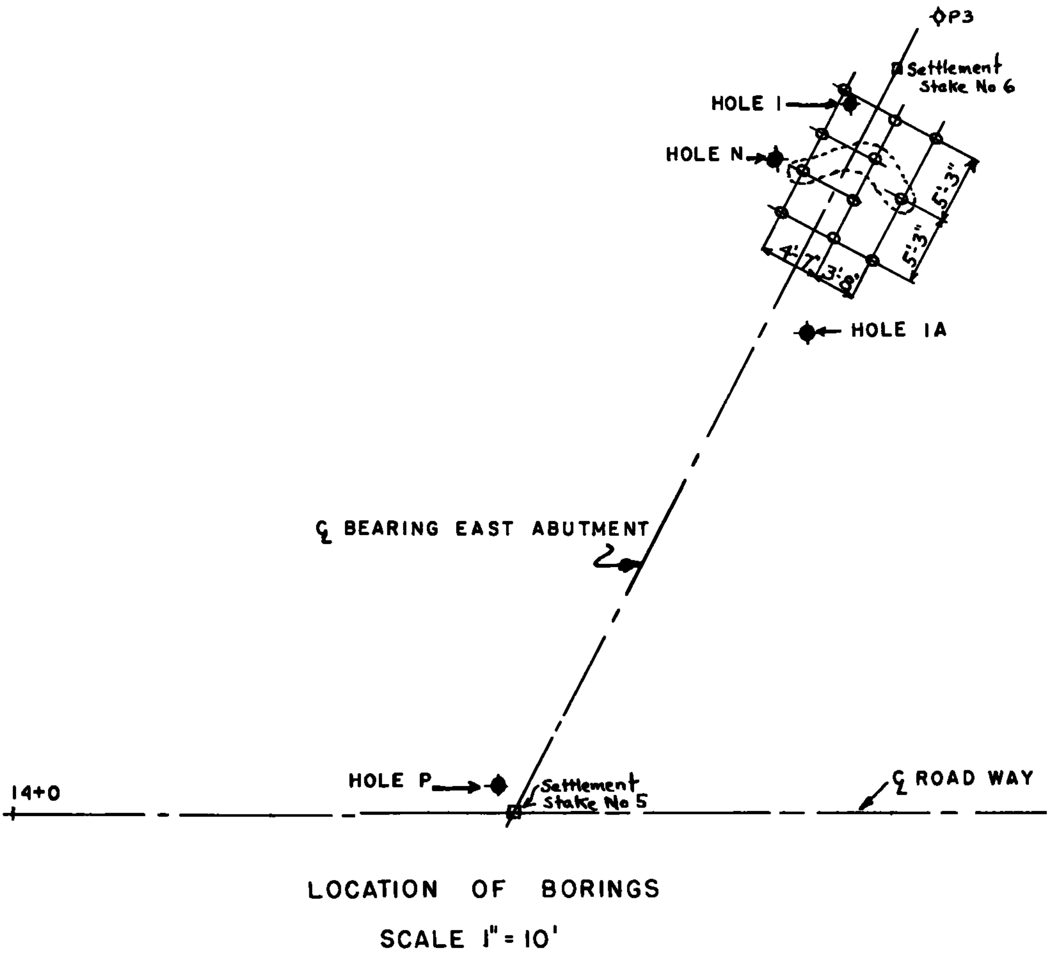


Figure 1. Location plan at east abutment for: test piles; borings for holes N, P, 1, 1A; settlement stakes 5, 6; piezometer P3.

to a depth of about 25 ft below mean sea level. From 1951 to late 1954 fill was placed for the approach to the bridge, finally reaching an elevation of +50 ft including a 5-ft overload. The fill up to elevation +25 ft was hydraulic fill, consisting of medium to fine sand. From elevation +25 ft to final grade the fill was a trucked-in bony glacial till. To expedite settlement of the mud layer, sand drains were installed late in 1951.

The effect of the placement of the embankment on the underlying mud can be seen from Figure 2. By late 1955 over 11 ft of settlement had taken place. The settlements (Fig. 3) at the east abutment (as shown for settlement stakes No. 5 and No. 6, Fig. 1) are larger than at pier No. 5 where the thickness of the mud layer was less and the fill was brought up only to elevation +15 ft. Another important effect of the high fill at the east abutment is that an appreciable settlement took place after the test piles were driven in July 1955.

Starting in 1953 several borings were made near the location of the test pile cluster (Holes N, 1, 1A, Fig. 1). No undisturbed samples were taken in Hole N in 1953; however, undisturbed samples were taken in 1953 in Hole P, near the centerline of the roadway at the east abutment (Fig. 1). Undisturbed samples were taken in Holes 1 and 1A in 1954 and 1955, respectively.

The boring logs (Fig. 4) for the holes at the east abutment show that beneath the fill there is a thin layer of peat and then a layer of clayey silt extending to an elevation of approximately -25 ft. From elevation -25 ft to -38 ft there is a medium sand and then below elevation -38 ft there is a very thick deposit of reddish brown fine sand and silt.

SOIL TESTS

Laboratory tests were performed on samples taken during the borings made in 1953, 1954, and 1955. The tests on the 1953 samples were performed by the Connecticut State Highway Department Laboratory. Tests on samples taken in 1954 and 1955 were performed in the Soil Mechanics Laboratory at the University of Connecticut (12). When the results of the tests on the 1953 and 1954 samples are compared with the results of the 1955 soil tests (Fig. 5) the moisture content at the middle of the clayey silt changed from about 88 percent in 1953 to 70 percent in 1954 and 60 percent in 1955. There was also a considerable change in shear strength as measured by unconfined compression tests. Thus, the cohesion for undisturbed samples changed from about 325 lb per sq ft in 1953 to about 1,200 lb per sq ft in 1955 (the solid line curve, Fig. 5). Cohesion values for remolded samples for 1955 are shown by the dotted line curve. Comparison of the results for undisturbed samples with results for remolded samples shows that the average sensitivity of the clayey silt is 5. 33 in 1955. The consolidation tests carried out on the 1955 samples indicate that the consolidation of the clayey silt was still not complete although most of the total settlement had occurred.

In connection with the tests on the samples taken in December 1955 it should be pointed out that the embankment of the east abutment underwent certain changes during 1955. The embankment was benched out in two stages to elevation +35 ft (Fig. 6). Then, in order to drive the piles, the till was excavated to elevation +13 ft, and the area was backfilled with sand in June and July 1955. In November 1955 the fill was excavated to elevation +13 along the entire length of the abutment, except around the

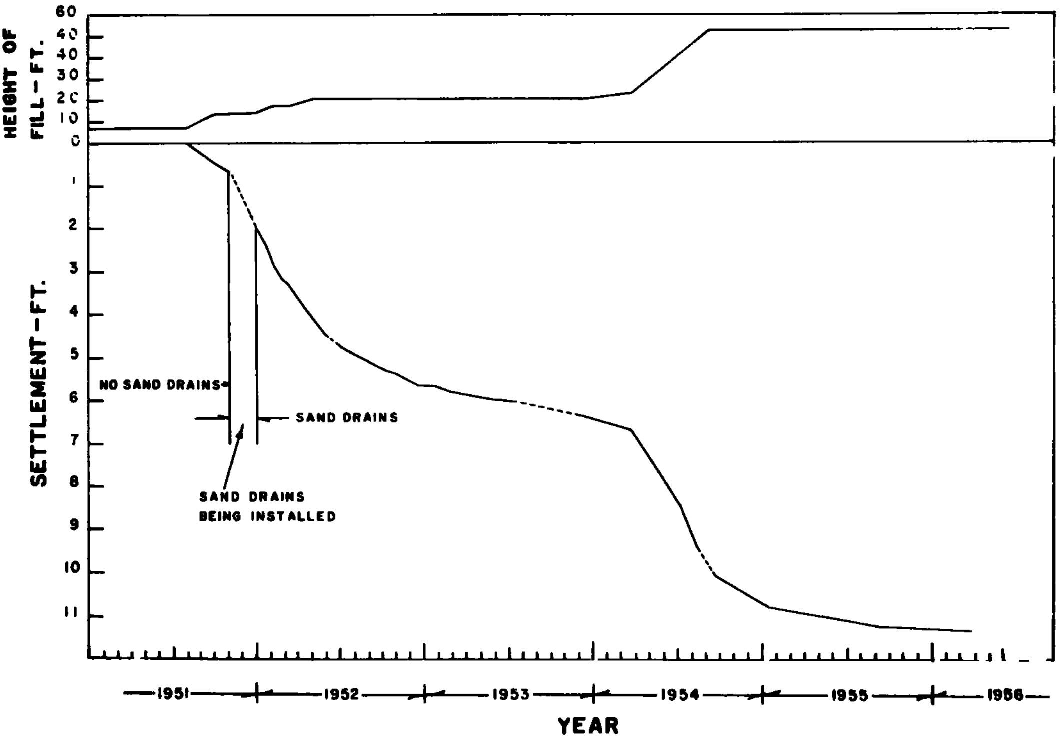


Figure 2. Settlements at the site of the east abutment.

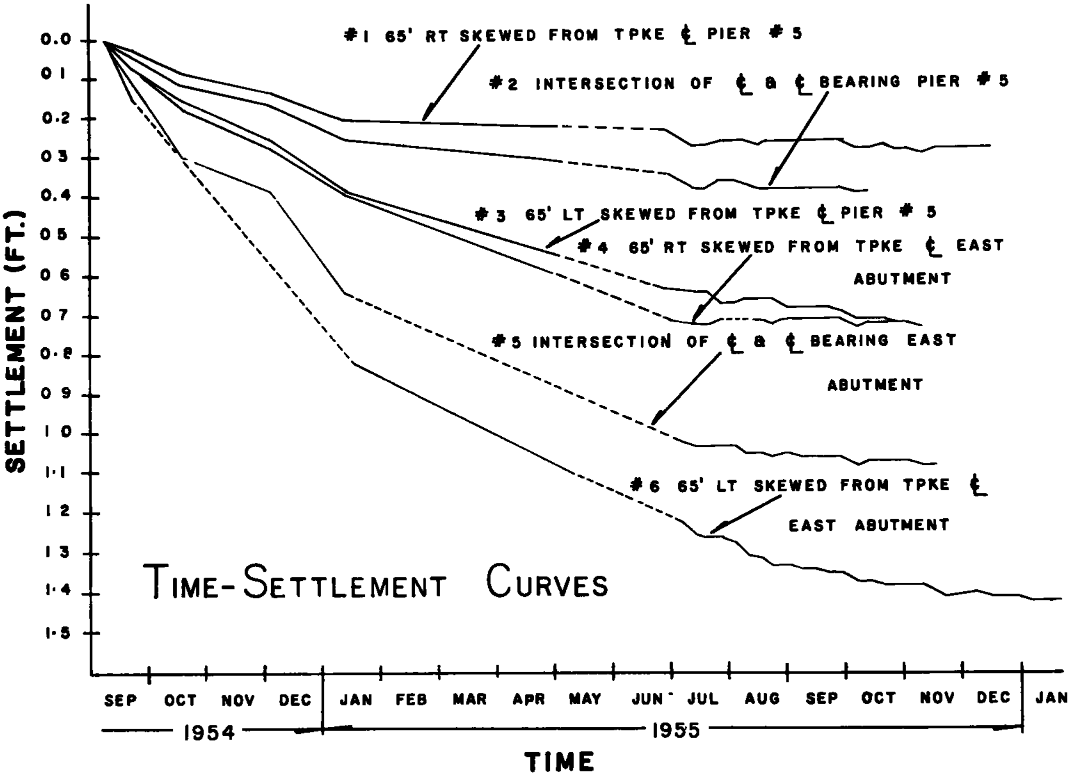
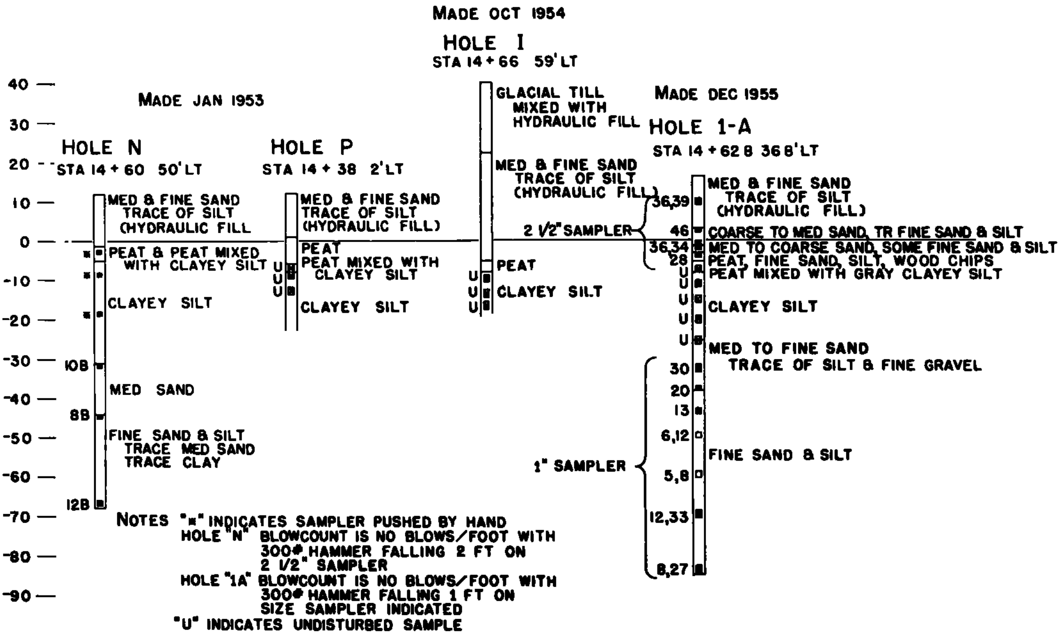


Figure 3. Settlement data for the sites of the east abutment and Pier 5.



NOTES "*" INDICATES SAMPLER PUSHED BY HAND
HOLE "N" BLOWCOUNT IS NO BLOWS/FOOT WITH 300# HAMMER FALLING 2 FT ON 2 1/2" SAMPLER
HOLE "1A" BLOWCOUNT IS NO BLOWS/FOOT WITH 300# HAMMER FALLING 1 FT ON SIZE SAMPLER INDICATED
"U" INDICATES UNDISTURBED SAMPLE

Figure 4. Boring logs for holes N, P, 1 and 1A.

11 test piles, to drive the piles for the abutment.

EXPERIMENTAL PROCEDURE

The initial foundation design for the east abutment was based on three lines of piles, one line of batter and two lines of vertical, to be driven from elevation +35 ft through the fill and mud layers and into the underlying sand. Preliminary estimates indicated that the combined load from drag and the structure might require a penetration below the mud of about 100 ft or a total length of nearly 170 ft. Over half of the estimated penetration was accounted for by anticipated drag forces. In view of the importance of drag forces at the West Haven site and the uncertainties in design, it was decided to investigate the nature and magnitude of these forces.

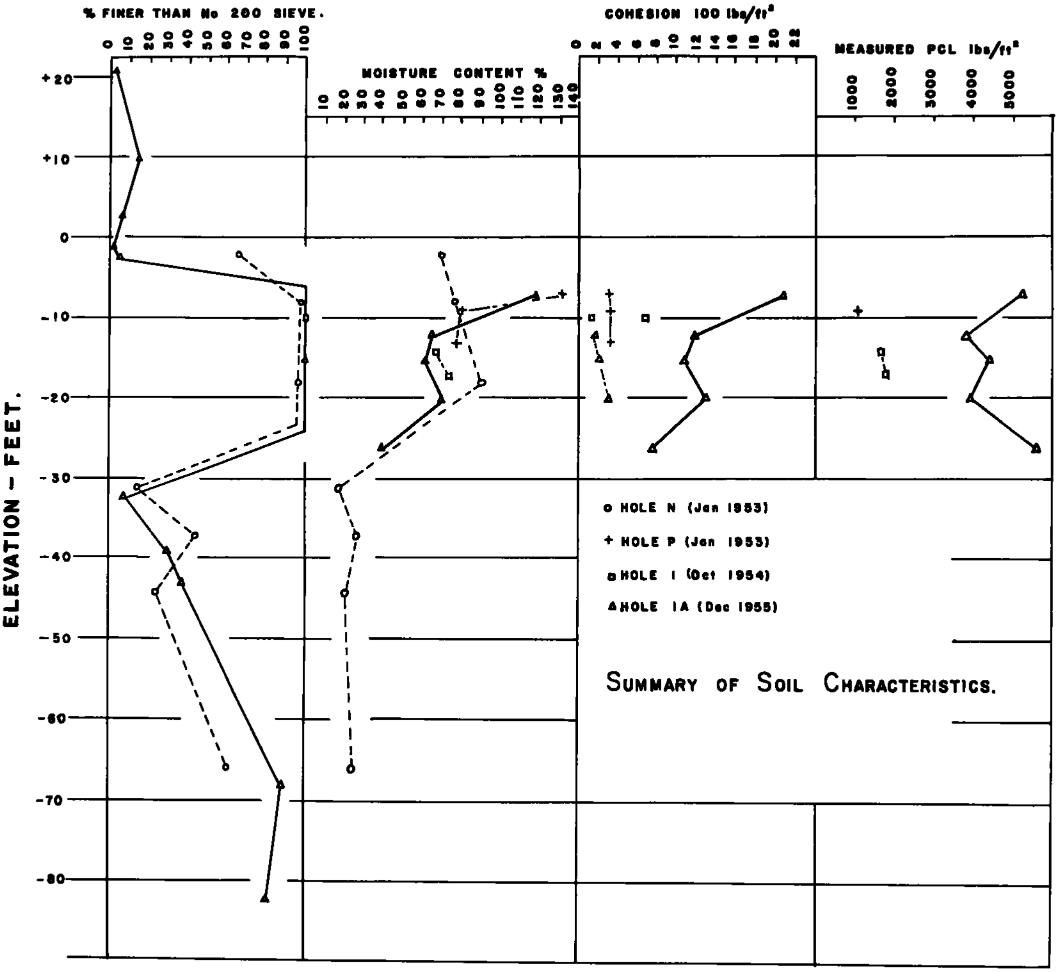


Figure 5.

Three experimental piles, two vertical piles and one batter pile, were to be equipped with SR-4 electric strain gages on the inside surfaces and would be part of a cluster of eleven piles driven for group effect. The piles selected were Monotube fluted piles No. 3 gage Type F (tapered) for the nose sections and No. 3 gage Type N 12, 12-in. diameter, for the extensions. The tip diameter was 8 in. Initially it was planned to use for each pile a 30-ft nose section, two 40-ft extensions, and two 30-ft extensions making a total length of 170 ft. However, later one of the 40-ft extensions was omitted.

GAGE INSTALLATION

Several preliminary studies were made to determine the most efficient method of gage installation (10). One method was the use of hand holes cut in the pile at 5-ft spacings. This method had proved satisfactory in the studies by Schlitt (11) and others (9). However, it was abandoned in favor of full length splitting of the pile shells. Although this method required more cutting and welding, it resulted in greater ease of installation and provided for closer inspection of the work. Reese and Seed (8) also used this method in their studies on 6-in. diameter steel pipe piles. The procedure finally adopted was as follows:

1. Each pile section was ripped full length along the center of diametrically opposite flutes so chosen as to avoid the factory weld (Fig. 7).
2. A steel messenger cable was placed under tension in each half of the pile section to support the gage lead cables.
3. Type A-5 SR-4 electric strain gages were installed with Duco cement at 5-ft intervals along the pile sections for zones expected to produce drag and at 10-ft intervals in other zones. Two active gages were provided at each station with two additional active gages at alternate stations (Fig. 8). A temperature gage was provided at each gage station.
4. After the gages were glued to the shell, they were waterproofed and given mechanical protection. An asphalt paving cement was spread over a gage and fused to the steel by curing with heat lamps, then covered with Armstrong A-2 adhesive and given another coating of asphalt cement.
5. The two halves of the shell were then clamped and adjusted to the original dimensions. Gages were protected by cooling and the shell was welded.

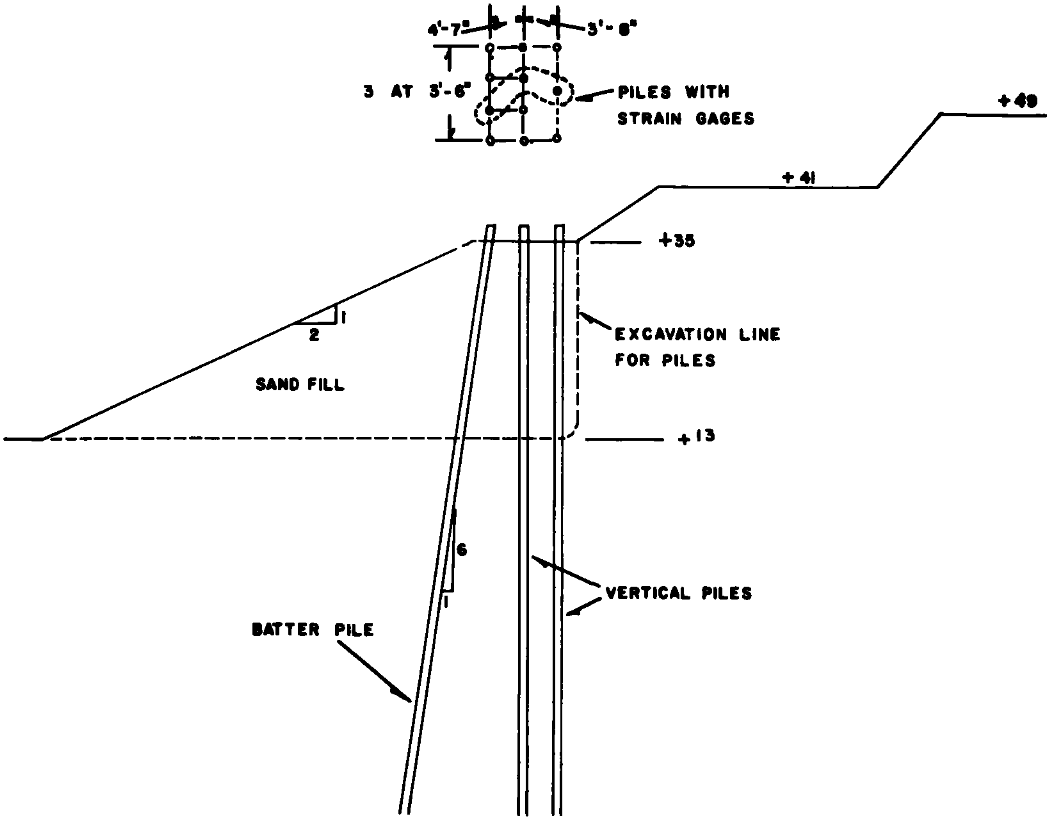


Figure 6. Elevation of the embankment at the east abutment.

Studies showed that the residual stresses produced by the splitting and rewelding were of small magnitude and would have only a negligible effect on the resistance of the pile to further load. It was also found by test in the laboratory that gages installed in this manner would correctly indicate loads at the gage stations.

Figure 7. Pile section cut in halves lengthwise for gage installation.

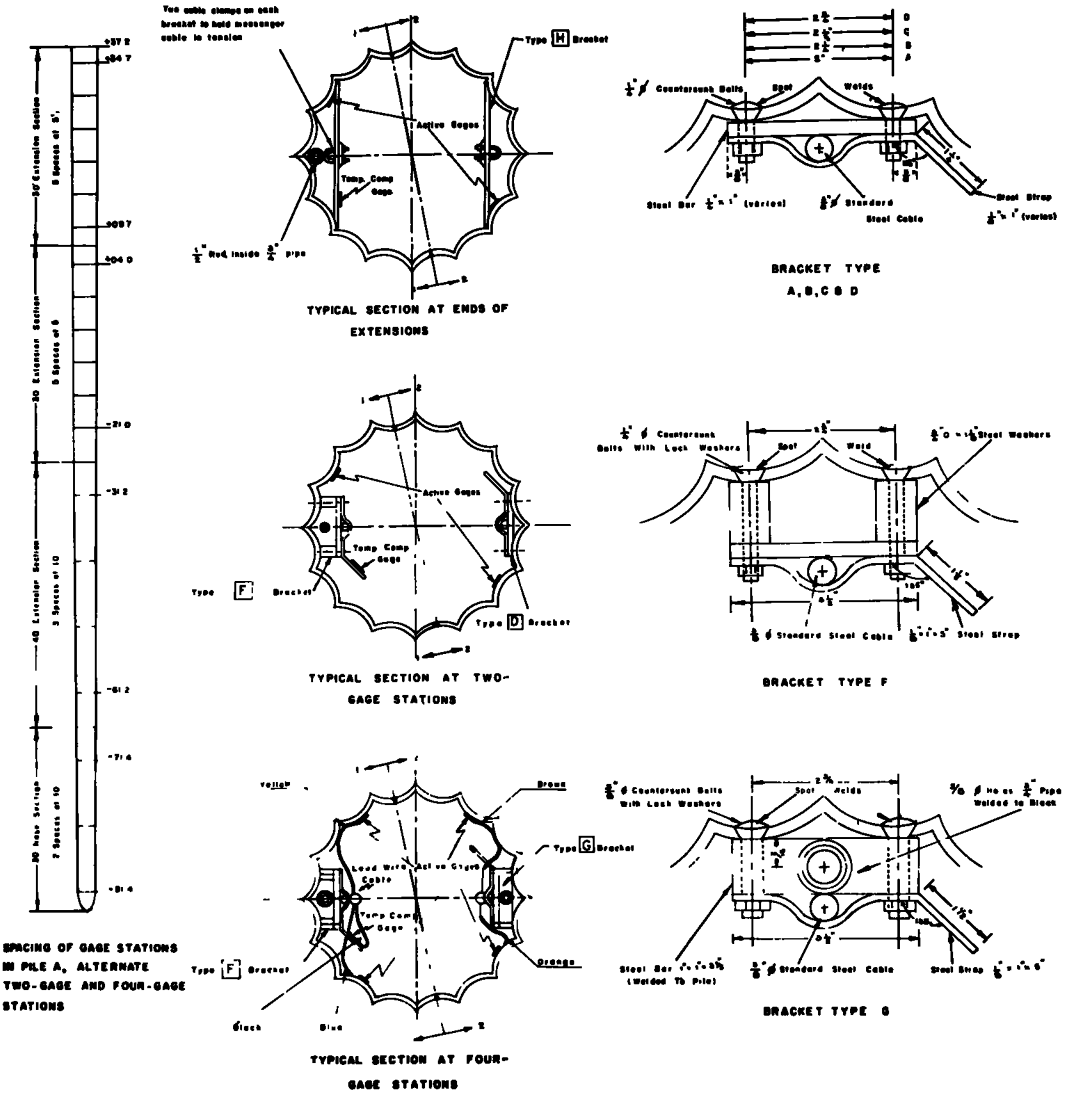


Figure 8. Location of gages along piles as driven and typical sections.

MECHANICAL SYSTEM

As a check on the SR-4 gages, a mechanical system of measuring the strain over 15-ft gage lengths was also installed. This system consisted of rods and pipes starting at different cable brackets and extending to the top of the pile. As each member was fixed to the pile shell at its lower end only, any relative motion between the upper ends of the various rods and pipes would be caused by strain occurring in the pile shell between the levels at which the members were anchored. The upper portion of each rod was located inside a pipe fastened at a higher level and restrained laterally by the higher cable brackets. For ease in field splicing, the rods and pipes installed in any extension section were threaded to connect with the rods in the next section.

Unfortunately, the vibrations produced in the pile sections by hard driving tore the mechanical system loose, and damaged a large number of the SR-4 strain gages.

CONSTRUCTION PHASE

The installation of the gages was carried out at the University of Connecticut, and

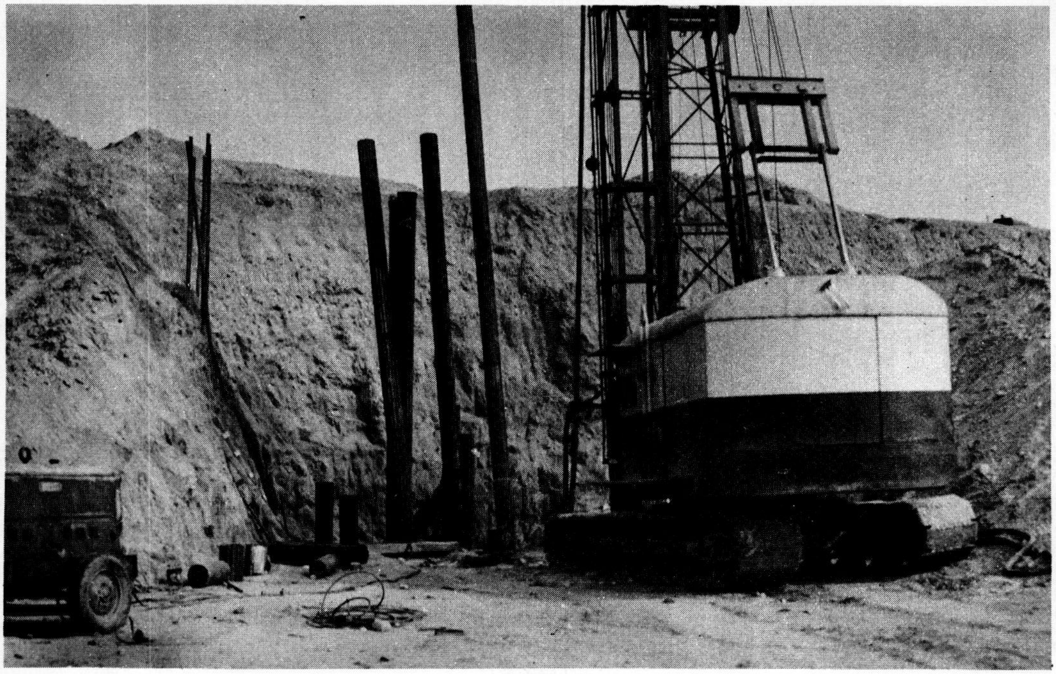
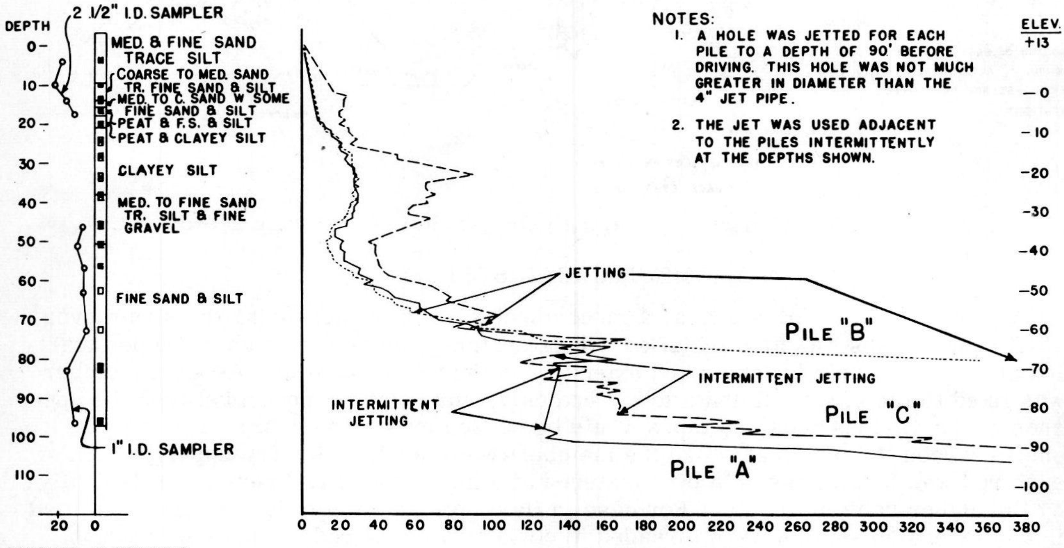


Figure 9. Pile test site during pile driving operations after excavation of embankment to elevation +13 ft.

the sections were trucked to West Haven.

At the job site partial assembly of the pile sections was completed. The cable connectors for the nose and the adjacent extension section for each pile were joined and



NOTES:

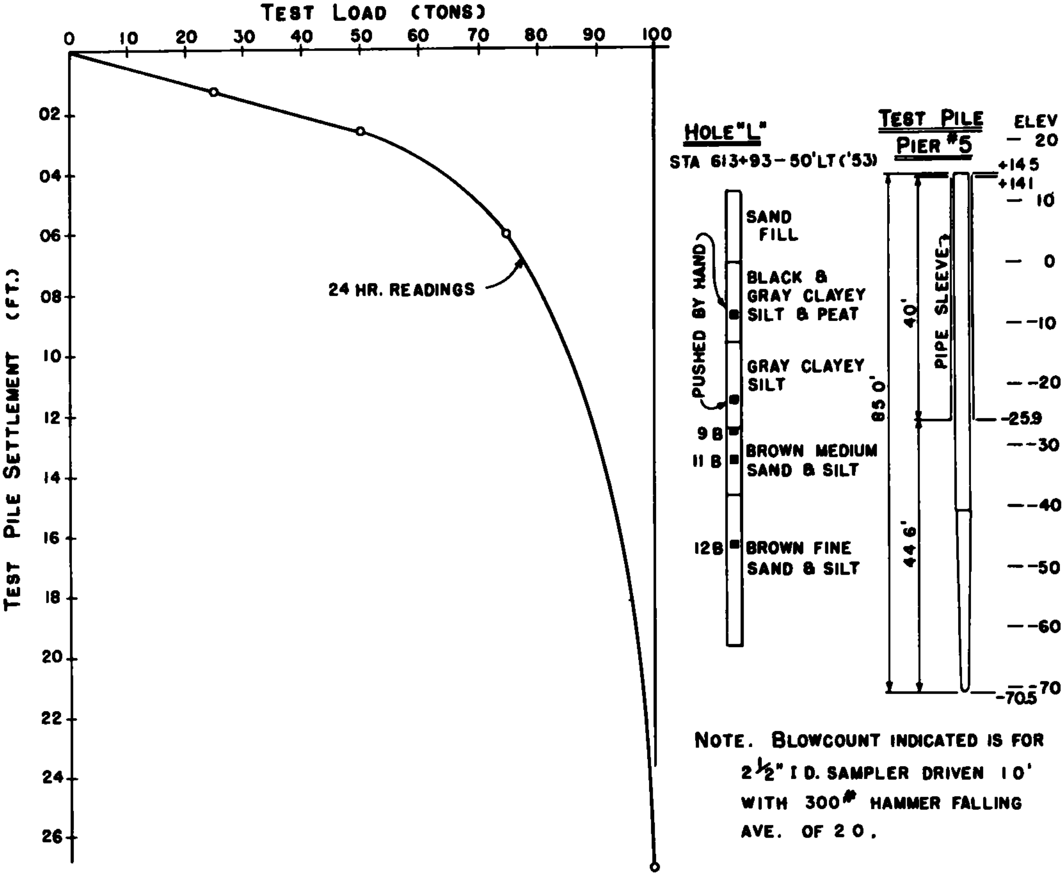
1. A HOLE WAS JETTED FOR EACH PILE TO A DEPTH OF 90' BEFORE DRIVING. THIS HOLE WAS NOT MUCH GREATER IN DIAMETER THAN THE 4" JET PIPE.
2. THE JET WAS USED ADJACENT TO THE PILES INTERMITTENTLY AT THE DEPTHS SHOWN.

Figure 10. Driving records for the three experimental piles A, B, C. Piles A and C are vertical piles, interior and exterior, respectively. Pile B is an exterior batter pile.

waterproofed, and the welded pile splice was made. The connections between the other two sections of the piles were made in a similar manner. Thus, for driving purposes, each pile consisted of a 70-ft section, which included the nose section, and a 60-ft extension which carried the mechanical strain system. It was necessary to make only one splice in each pile after the driving had begun.

One of the piles in the test pile group which had no strain gages was to be driven initially to determine the best method of driving and any special precautions necessary to keep the damage to the strain gage system to a minimum.

However, the first attempts to drive this pile met with failure, because the upper 13 ft of the fill consisted of very dense trucked-in coarse glacial till. After the contractor had exhausted all his resources in attempting to penetrate the fill, even resorting to jetting and the use of a spud, it was decided to excavate a portion of the fill down to elevation +13 ft (Fig. 6 and 9). The compactness and density of the upper portion of the fill is illustrated by the fact that it took two days and a total pull of 140 tons to extract the spud driven to 38 ft below the proposed bottom of the footing. After the excavation was completed, the remainder of the pile was driven. Jetting was necessary to start this pile moving after the delay between the initial attempts to drive it and the time when the driving was renewed. The remainder of the pile group including the experimental piles were then driven with a 10B-3 hammer (Fig. 9).



NOTE. BLOWCOUNT INDICATED IS FOR 2½" I D. SAMPLER DRIVEN 1 0' WITH 300# HAMMER FALLING AVE. OF 2 0.

Figure 11. Test pile results at Pier 5.

The piles were not driven to any specified formula value, but were driven to a predetermined length. Since the experience with the first pile indicated that some difficulty would be encountered in achieving this desired penetration, it was decided to jet a hole

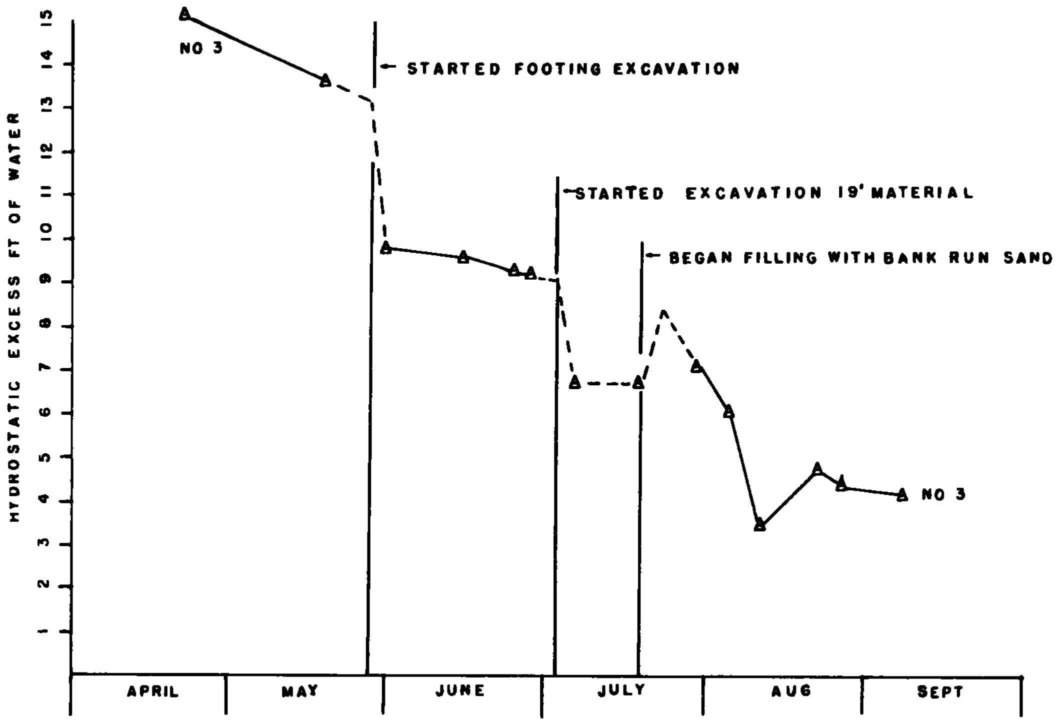


Figure 12. Pore water pressure measurements at the north end of the east abutment, 1955.

90 ft deep for each pile before it was driven. It was later found necessary to resort to jetting adjacent to each pile to minimize the energy absorption and possible damage to the strain gage system. The driving records for the three experimental piles with strain gages are shown in Figure 10.

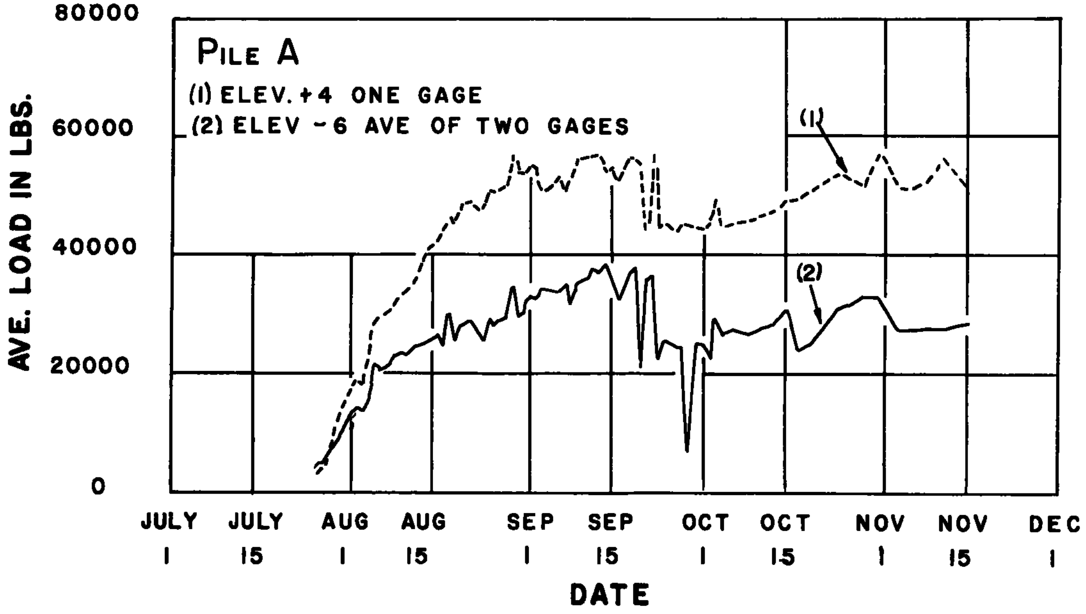


Figure 13. Indicated soil drag loads in Pile A at elevations +4 ft and -6 ft.

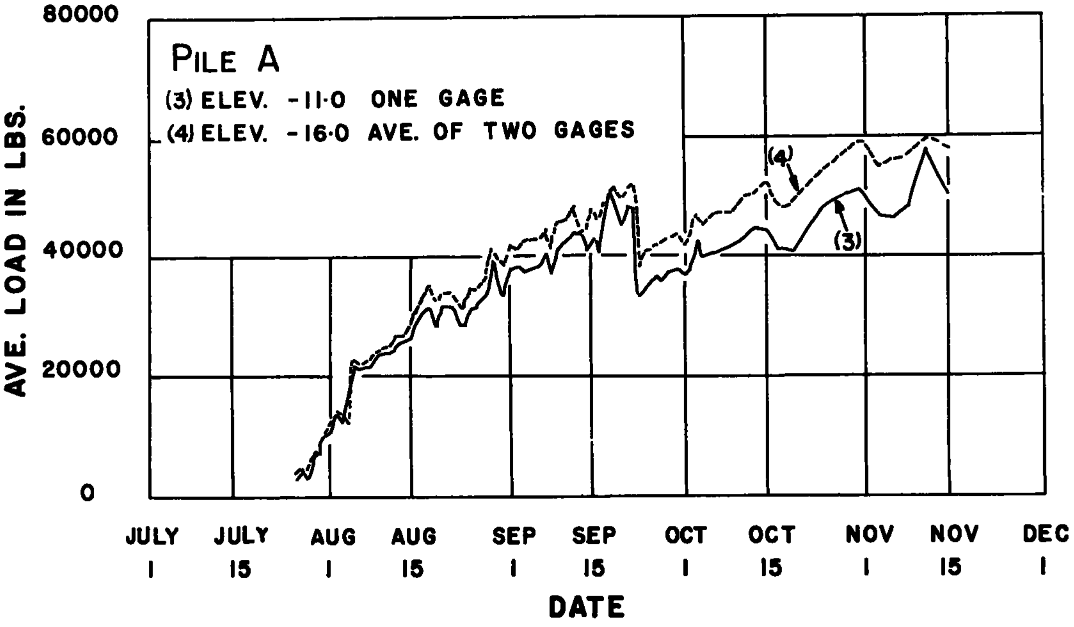


Figure 14. Indicated soil drag loads in Pile A at elevations -11 ft and -16 ft.

GAGE READINGS

The three experimental piles were driven between July 12 and July 14, 1955. The area around the piles from elevation +13 ft to +35 ft was then backfilled with bank run sand and compacted (Fig. 6). The resistances of the electric strain gages to ground were checked and connections were made to an instrument shed which housed the balancing and switching units. On July 25 the gages were zeroed in and readings were begun on July 26, 1955. In order to minimize drift a DC bridge was used (Anderson Model 301 Strainmeter).

Readings of all active gages were made daily for a period of several weeks and then two or three times per week. Around the middle of November 1955 excavation was started at the east abutment in preparation for the driving of the supporting piles. The material around the upper part of the experimental piles slumped as a result of the construction operations. Since the loading conditions had been changed by the partial removal of the fill, regular reading of the gages was discontinued at this point.

TEST PILES

Two special pile load tests were performed at piers near the east abutment to obtain definite values of the frictional resistance offered by the soil underlying the mud. One was near the south end of Pier No. 4, and the other was near the north end of Pier No. 5. These piles were load tested in four load increments to 82 tons and 100 tons, respectively. To eliminate possible effects of the fill and clayey silt on the test results, the piles were driven through 18-in. pipe sleeves which had previously been driven to the bottom of the clayey silt and cleaned out. The test pile at Pier No. 4 was driven to a final blow count of 64 blows per foot for the last foot and the one at Pier No. 5 was driven to a final blow count of 95 blows per foot, both with a 10B3 hammer. The load test results for the pile driven at Pier No. 5 are shown in Figure 11.

CONTROLS

At the same time that the planning stages of the project were initiated, controls were installed at the project site. These included settlement stakes at each end and the center of Pier No. 5 and the east abutment (Fig. 2 and 3) and a cluster of six piezometers adjacent to the north end of the east abutment. Three of these piezometers were installed at a radius of 1.5 ft from the theoretical location of a 20-in. diameter sand drainage well and at the approximate quarter points in the clayey silt layer. Three more were installed at an approximate radius of 3 ft from the sand well and were spaced vertically in the same manner as the other three. Both the settlement records shown in Figures 2 and 3 and the pore water pressure measurements illustrated in Figure 12 indicate that considerable consolidation of the mud layer took place after the test piles were driven.

Readings were made on the settlement stakes and piezometers until they were either destroyed or made inaccessible by construction operations.

TEST RESULTS

Readings of individual strain gages were converted to stress and then the average stress at each elevation was multiplied by the corresponding steel area to obtain the indicated drag load. Initially, gages were distributed throughout the length of the three experimental piles (Fig. 8). However, a large number of gages were damaged when the mechanical system failed during driving operations. In addition, some gages ceased to operate satisfactorily during the course of the investigation although, in general, the methods of gage installation and waterproofing proved to be satisfactory for long term tests.

The indicated drag loads for pile A, the interior vertical in Figure 6, are plotted in Figures 13, 14, 15, 16 as a function of time. Figure 17 shows the distribution of the drag loads for three different dates approximately six weeks apart. In general, the distribution of load for the exterior vertical and batter piles followed a pattern similar to that shown for pile A. During the period November 15-29, 1955, fill was removed at the east abutment to permit driving of the entire pile foundation of about 100 piles. The fill was excavated to elevation +13 ft up to the face of the cluster of 11 test piles and as a result the sand fill between the test piles slumped to an average elevation of

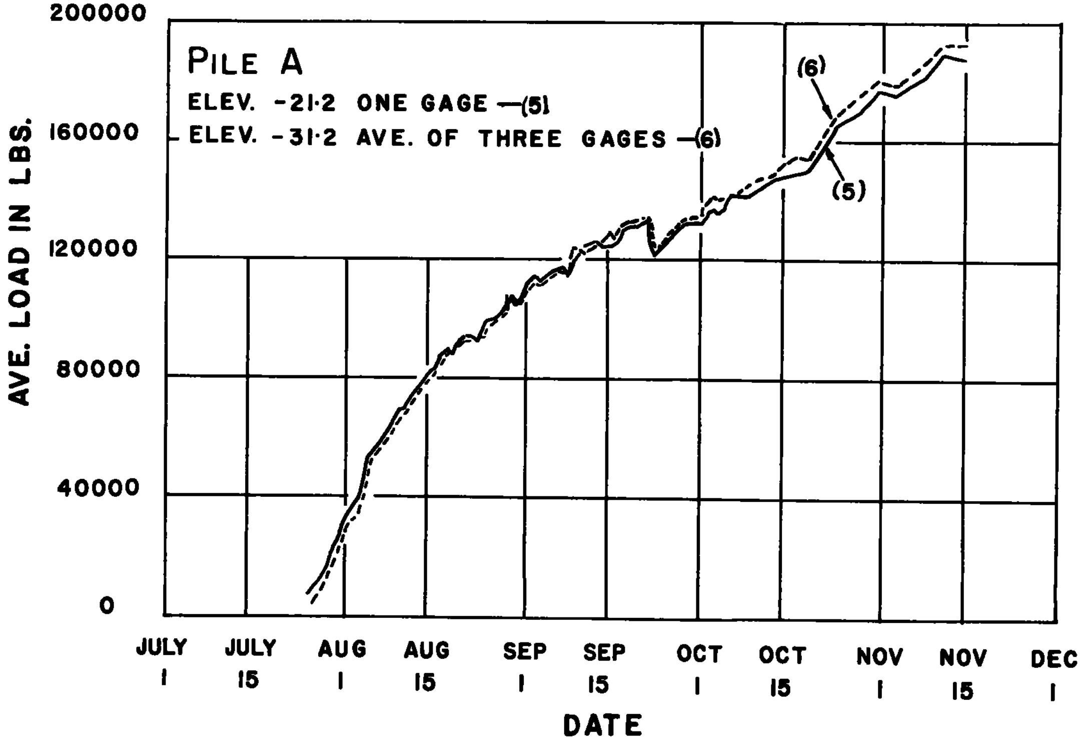


Figure 15. Indicated soil drag loads in Pile A at elevations -21.2 ft and -31.2 ft.

TABLE 1

RESPONSE OF STRAIN GAGES TO PARTIAL UNLOADING - PILE A

Elevation (ft)	Number of Gages	Indicated Load (kips)		Change in Load Nov. 15-29 (kips)
		Nov. 15	Nov. 29	
+4	2	50.5	35.6	14.9
-1	1	44.6	24.9	19.7
-6	2	28.1	8.4	19.7
-11	1	50.1	22.7	27.4
-16	2	57.8	31.7	26.1
-21	1	187.2	169.7	17.5
-31.2	3	192.3	175.9	16.4
-51.2	2	141.4	128.5	12.9
-61.2	1	141.6	121.8	19.8

about +20 ft. The response of the strain gages in pile A to the unloading is illustrated in Table 1. The start of large scale construction operations at the east abutment, with altered loading conditions for the piles, marked the end of the formal test period. However, periodic strain readings indicated that, at least for several weeks after the removal of load, the drag load remained substantially constant.

The progressive consolidation of the mud layer resulted in increased drag loads. However, the effect of the continuing consolidation was different for different layers. As shown in Table 2 and Figure 17, most of the drag caused by the soil above the clayey silt was accumulated in the first three weeks. On the other hand, the transfer of load by the clayey silt took place at a much slower rate and the total load in the pile on August 15 amounted to only about one-half of the November 15 load. Further, comparison of the load gains shows (Table 2) that the increases above elevation -31.2 ft in

TABLE 2

CHANGE IN LOAD - PILE A

Elevation (ft)	Indicated Load Change (kips)		Percent of Nov. 15 Loads		
	Aug. 15 - Oct. 1	Oct. 1 - Nov. 15	on Aug. 15	Change Aug. 15 - Oct. 1	Change Oct. 1 - Nov. 15
+4	3.2	6.0	82	6	12
-1	0.6	0.2	98	0.1	0.1
-6	-1.1	3.6	91	-4	13
-11	9.7	13.6	54	19	27
-16	12.0	16.7	50	21	29
-21	51.4	55.2	43	27	30
-31.2	55.5	58.1	41	29	30
-51.2	43.0	73.0	18	30	52
-61.2	36.0	83.5	16	25	59

the two arbitrary six-week periods (August 15 to October 1, and October 1 to November 15) were roughly equal although somewhat greater gains were measured in the second period. The increased loads measured below the clayey silt in the positive friction zone point to a redistribution of the positive frictional stresses with a probable growth in resisting stresses at lower elevations with time.

The maximum indicated drag loads do not appear unusually large when account is

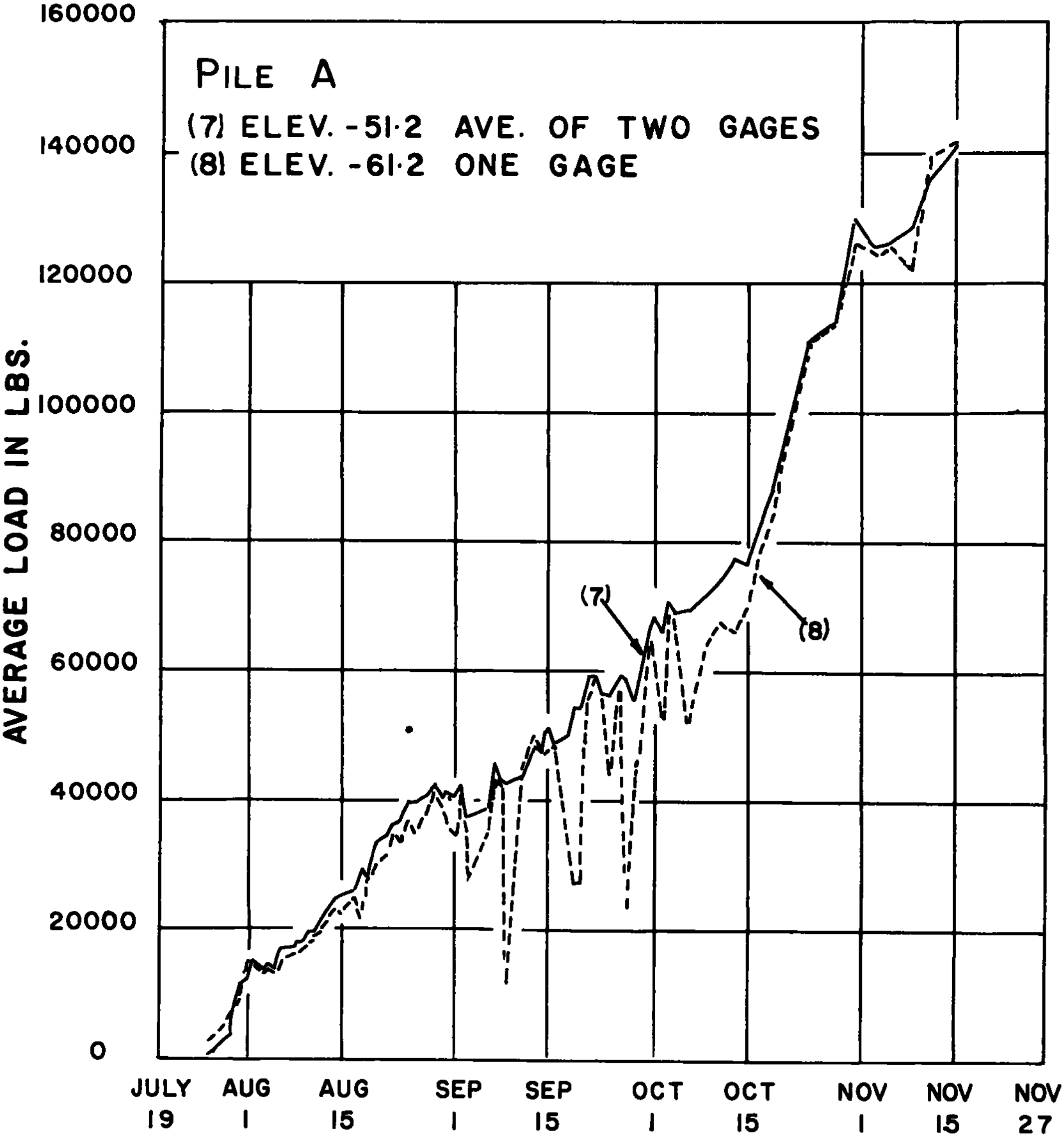


Figure 16. Indicated soil drag loads in Pile A at elevations -51.2 ft and -61.2 ft.

taken of the combination of a high fill on a soft stratum and the fact that the mud layer was incompletely consolidated when piles were driven. However, explanation of the distribution of load obtained in the test (Fig. 17) requires further study of the mechanism of stress transfer from soil to a pile. The maximum indicated bond stresses for the load distribution exceed to a considerable extent the cohesion of the silt as measured by unconfined compression tests on undisturbed samples. This could mean that the bond between the silt and pile is not approximately equal to the shear strength of the soil, but instead is much greater. Or, the shear strength of the soil near the pile may increase with time to a value much higher than that for soil unaffected by the pile driving operation. Gray (6) has suggested an increase in shear strength near the pile and other investigators have also pointed to the fact that often soil will adhere to a pile when it is pulled from the ground. From Figure 17, the maximum indicated bond stress for a load transfer of 130,000 lb in 5 ft is about 8,000 lb per sq ft. However, at a distance of one and a half diameters from the face of the pile the required shear stress is reduced to about 2,000 lb per sq ft. From Figure 5, a shear strength of the order of 2,000 lb per sq ft may conceivably be attained in the silt when sufficiently consolidated.

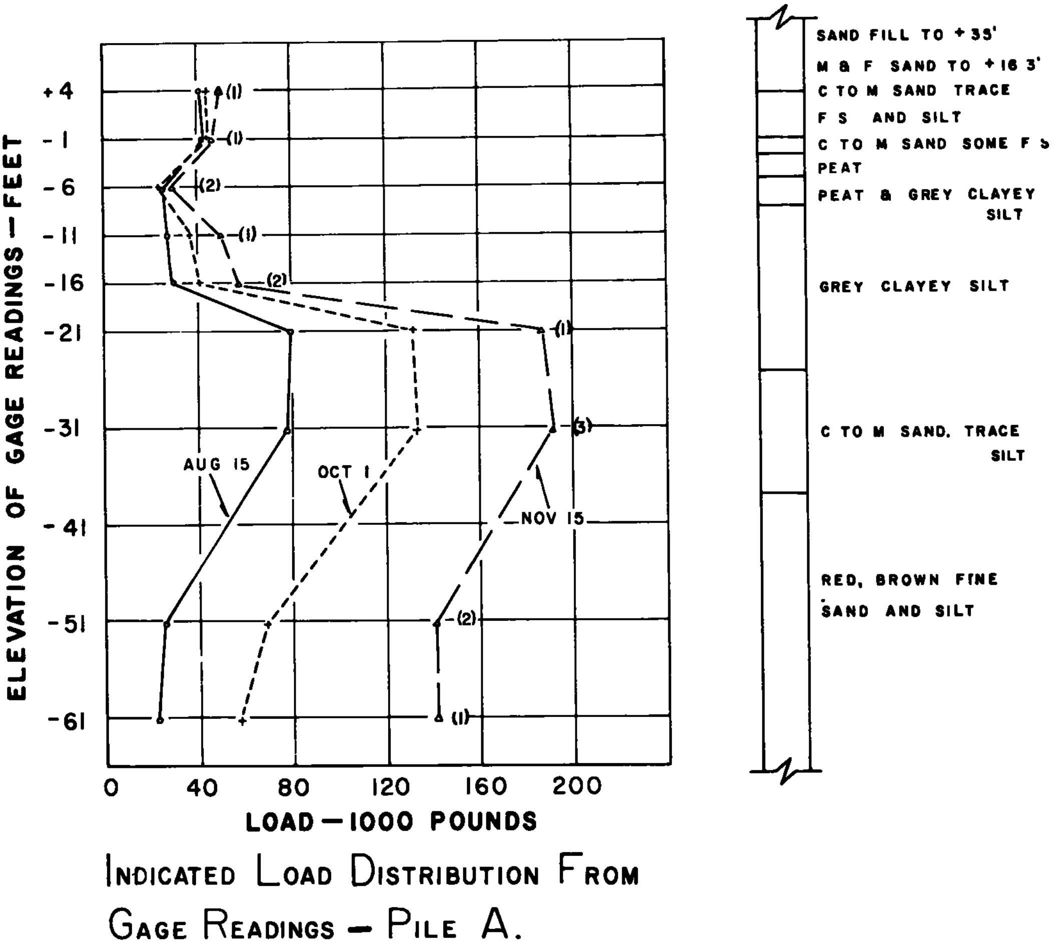


Figure 17. Indicated distribution of soil drag loads in Pile A for three different dates.

COMPUTED DRAG LOAD

For the test pile site, the results of extensive laboratory testing and field observations indicate that the mud layer was incompletely consolidated under the loads applied, especially toward the middle of the layer, at the time piles were driven. Tests also show that the shear strength of the clayey silt, as an indicator of possible bond stresses, was greatly increased by consolidation (Fig. 5). The manner in which the fill was replaced around the upper portion of the piles suggests that the lateral pressure of this soil on the pile corresponded to the active Rankine state. The jetting operation through fill material down to elevation -5 ft also points toward the active Rankine state. The following computations illustrate two procedures for estimating the maximum drag loads produced by the severe loading conditions at the test pile site.

Single Pile Analysis

α = 110 lb per cu ft Elev +35 ft to +6 ft

α = 48 lb per cu ft Elev +6 ft to -5 ft (submerged)

α = 38 lb per cu ft Elev -5 ft to -25 ft (submerged)

Coefficient of lateral soil pressure = ⅓ (Elev +35 ft to -5 ft). Angle of friction between pile and soil of 30 deg (Elev +35 ft to -5 ft).

Based on these values, the drag load in the pile is 29,500 lb at Elev +6 ft and 53,500 lb at Elev -5 ft. From Elev -5 ft to -25 ft the drag of the silt can be estimated on the basis that the cohesion of the soil is fully mobilized at a distance from the pile surface equal to the radius of the pile. For an average cohesion of 1,200 lb per sq ft (Fig. 5) the drag of the silt is 150,000 lb. This gives a total drag load of 203,500 lb at elevation -25 ft compared with approximately 200,000 lb as obtained from the test. In this procedure it is assumed that the deformation in the silt is sufficient to develop the shearing strength of the soil and that the bond stresses at the surface of the pile are large in comparison to the shear strength.

Cluster Analysis

The influence of the batter piles (Fig. 6) decreases with depth. Therefore, the pile cluster analysis can be made on the basis of the group of seven vertical piles. The perimeter of the cluster at the face of the piles is 32.3 ft and the total area within the cluster is 4.67 ft by 11.5 ft or 53.7 sq ft. Full arching of the soil between the piles is assumed.

The cluster analysis, based on the same soil properties as used in the single pile analysis, gives a total drag load of 63,400 lb in the pile at elevation +6 ft and 100,300 lb at elevation -5 ft. From elevation -5 ft to -25 ft the drag produced by arching of the soil within the cluster and a cohesion of 1,200 lb per sq ft on the perimeter amounts to 116,300 lb. Therefore, the total drag load at elevation -25 ft is 216,000 lb as compared with approximately 200,000 lb from the test results. In this procedure, the stress transfer to the piles above elevation -5 ft requires larger bond stresses than were indicated for the single pile analysis.

The two methods of analyses indicate about the same total drag load at the bottom of the silt. However, it is clear that they are very different in terms of distribution of load and it cannot be assumed that the methods will always give results as close as obtained here. It must also be emphasized that for any given loading condition involving negative skin friction the magnitude of the maximum drag load in the piles is a function of the consolidation that takes place after the piles are driven.

ACKNOWLEDGMENT

The investigation described in this paper was carried out as a joint research project of the Connecticut State Highway Department and the Civil Engineering Department of the University of Connecticut. Particular acknowledgment is made of the cooperation and assistance of Philip Keene, Engineer of Foundations, who initially suggested the project and made available considerable data obtained by the Connecticut State Highway Department.

A. J. Silva and R. E. Machol, graduate assistants, and B. K. Ramiah, research assistant, all of the University of Connecticut, assisted in the experimental program.

The assistance of many members of the Connecticut State Highway Department is also greatly appreciated.

REFERENCES

1. Casagrande, A., "The Structure of Clay and Its Importance in Foundation Engineering," Journal, Boston Society of Civil Engineers, April 1932.
2. Chellis, R.D., "Pile Foundations," New York, McGraw-Hill Book Co., Inc., 1951, Chapter 16.
3. Crandall, L.L., "Electrical Resistance Strain Gauges for Determining the Transfer of Load from Driven Piling to Soil," Proceedings, Second Int. Conf. on Soil Mech. and Foundation Engr., Vol. 4, p. 122 (1948).
4. Cummings, A.E., Kerkhoff, G.O., and Peck, R.B., "Effect of Driving Piles Into Soft Clay," Transactions, American Society of Civil Engineers, Vol. 115, pp. 275-350 (1950).
5. Florentin, J. and L'Heriteau, G., "About an Observed Case of Negative Friction on Piles," Proceedings, Second Int. Conf. on Soil Mech. and Foundation Engr., Vol. 5, p. 155 (1948).
6. Gray, Hamilton, "Discussion of Experiences with Predetermining Pile Lengths by W.W. Moore," Transactions, American Society of Civil Engineers, Vol. 114, p. 371 (1949).
7. Moore, W.W., "Experiences with Predetermining Pile Lengths," Transactions, American Society of Civil Engineers, Vol. 114, p. 357 (1949).
8. Reese, L.C. and Seed, H.B., "Pressure Distribution Along Friction Piles," Proceedings, American Society for Testing Materials, Vol. 55, p. 1156 (1955).
9. Report of Committee, "Steel and Timber Pile Tests — West Atchafalaya Floodway," Proceedings, American Railway Engineering Association, Vol. 52, pp. 149-202 (1951).
10. Report on "A Study of Methods of Installing SR-4 Electric Strain Gages in Monotube Pile Sections," Progress Report, University of Connecticut, March 1955.
11. Schlitt, H.G., "Group Pile Loads in Plastic Soils," Proceedings, Highway Research Board, Vol. 31 (1952).
12. Silva, Armand, J., "A Study of Soil Characteristics in Relation to Forces Produced in Friction Piles," Master of Science Thesis, University of Connecticut, June 1956.
13. Terzaghi, Karl and Peck, Ralph B., "Soil Mechanics in Engineering Practice," New York, John Wiley and Sons, Inc., 1948, pp. 473-474.

Discussion

GREGORY P. TSCHEBOTARIOFF, Professor of Civil Engineering, Princeton University; Associate, King and Gavaris, Consulting Engineers — The paper deals with a topic of considerable practical importance. The detrimental effects of drag on piles produced by the settlement of adjacent soil have been widely recognized in a qualitative manner for some time. Precise quantitative information has, however, been lacking and any attempts to obtain such information are most welcome.

It is unfortunate that circumstances beyond the control of the authors, namely the presence of fill of such density that 22 ft of it had to be excavated to permit driving of experimental piles — after which the fill had to be replaced again, created unusual and most unfavorable conditions for this study. The special nature of these conditions has not been given full consideration by the analysis in this paper of some rather peculiar results of measurements which it reports.

Specifically, the drag value remained within what will be shown to be reasonable limits down to elevation -16 ft, that is, over a total distance of 51 feet: 29 ft through granular fill above water level, plus 11 ft through granular fill below water level, plus 11 ft through peat and clayey silt. Then, over a distance of only 5 ft through the same clayey silt, as reported on Figure 17, the drag value suddenly was more than trebled, being increased by some 325 percent to a total value of 186 kips or 93 tons. This is

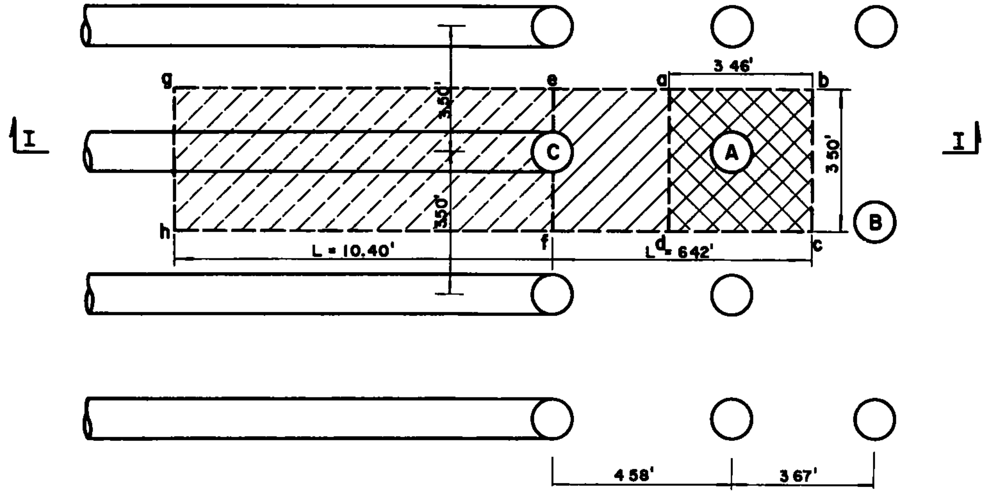


Figure 18. Plan showing column abcd of soil carried by Pile A (see Fig. 19 for Section I-I).

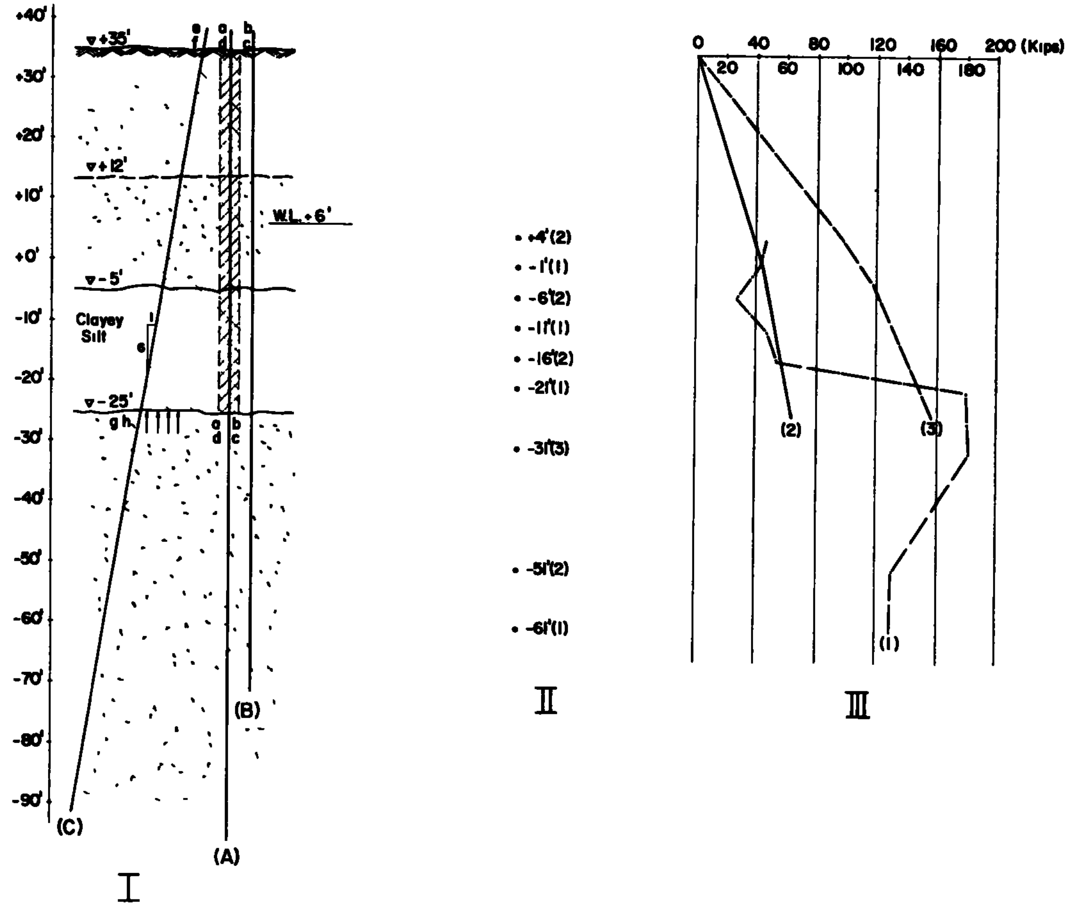


Figure 19. (I) Section I-I on Figure 18; (II) Elevations of SR4 strain gages (number of gages shown in brackets); (III) Computed pile loads: (1) from SR4 readings (by the authors); (2) weight of soil column abcd; (3) weight of soil prism bcefgh.

more than the weight of the soil around that pile.

The authors state that this load of 93 tons does not appear unusually large. This statement and the explanations presented by the authors in its support cannot be accepted by the writer for reasons which are as follows:

(1) The authors leave out of consideration one of the essential criteria which limits maximum possible drag loads, the maximum possible weight of the soil which surrounds the pile. A study of this additional criterion refutes the contention of the authors which attributes the high strain gage readings below a 54 ft depth to vertical drag loads only. The following points are of importance:

(a) The response of the strain gages to partial unloading as reported by Table 1 was somewhat erratic, but the average recorded by the 15 gages at all elevations was 18.7 kips. This value corresponds to the weight of a column abcd of soil 15 ft high shown double hatched in plan in Figure 18 (the excavation is stated to have resulted in a drop of the soil surface between the piles from +35 ft to + 20 ft).

(b) It will be noted from Figure 18 that the area abcd approximately corresponds to (i.e., it equals 83 percent of) the maximum area of soil which could be assigned to the pile A had all the piles around it been vertical.

(c) It will be further noted from curve 2 (Fig. 19) that the weight of the column of soil abcd closely agrees with the drag values computed from readings on all 8 strain gages at the 5 elevations down to and inclusive of elevation -16 ft, i.e. through all of the fill and through two-thirds of the depth of the compressible clayey silt layer.

(d) An extreme but most improbable assumption is that due to the presence of batter piles on one side of it, the pile A would carry a larger prism bcefgh which would include all soil under the batter piles as shown on Figure 18 and 19. The weight of this prism would exceed by some 250 percent values given by the 8 strain gages at the 5 elevations down to and inclusive of elevation -16 ft, as shown by curve 3 on Figure 19, but even then the weight of that prism would fall short of the drag computed from readings on the 4 strain gages at the next two elevations, that is, 4 ft above and 6 ft below elev -25 ft of the surface of the deep non-cohesive granular stratum in which the piles are embedded.

(2) References by the authors to allegedly similar observations made elsewhere are not relevant to a numerical evaluation of this case:

(a) The drag load of nearly 200 tons estimated by Moore (7) caused a pile to fail under conditions which were substantially different from the present ones. It can be seen from Figures 17 and 18 (pp. 366-367) that Moore's piles were spaced much more widely apart than the piles of the present case (Figures 1 and 18) so that a much greater weight of soil could come to hang on them.

(b) It is true that soil will often adhere to a pile when it is pulled from the ground. For instance the writer has observed a shell of stiffer clay one or two inches thick around the face of excavated concrete piles. This is natural since such piles facilitate consolidation of the clay along their faces by permitting drainage through them. In the case of extracted H-piles clay will frequently remain stuck between their flanges since the surface of adhesion of clay to metal is then almost twice as great as the clay shearing surface related to this phenomenon. It is also conceivable that thin patches of clay might adhere even to cylindrical surfaces of steel due to rusting or other chemical action on the clay. But the writer knows of no evidence (and the authors have not produced any) which would justify their suggestion that the zone of increased strength of the clay might in the present case extend up to a distance of one and a half diameters from the face of pile A.

(3) Some of the assumptions on which the authors base their "single pile" and their "cluster" analysis are in contradiction with the observations which they report. Thus, they allegedly obtain agreement at the bottom of the silt layer between the drag loads as computed by them from the strain gage readings and as estimated by them on the basis of soil strength data. This alleged agreement is obtained by assuming a load transfer due to an average potential cohesion of 1.2 ksf (Fig. 5), 6 in. from the face of the pile along the entire 20 ft depth of the silt layer, from -5 ft to -25 ft. This assumption is in direct contradiction to Figure 17 of the authors which shows variations of the total drag load with depth. Figure 20 (II) shows the variation of the shearing strength

needed at face of pile if the curve in Figure 17 for November 15 is to be explained by drag loads as is attempted by the authors. The shearing strength needed to explain the computed drag loads at the face of the pile of 12-in. diameter equaled 1.2 ksf for the first 5 ft only (from -6 ft to -11 ft); then it dropped to 0.51 ksf over the next 5 ft, but increased to 8.05 ksf over the subsequent 5 ft. At a distance of 6 in. from the face of the pile these values would be reduced to one-half of the figures given.

(4) The authors offer no explanation which could account for the enormous differences shown on Figure 20 (II) in the effective shearing strength for different parts of the silty clay layer needed to explain their hypothesis that only drag forces act on the piles. The following shows that such differences cannot be assumed actually to exist:

(a) The trend of the actual variation of the potential shearing strength with depth (Figures 5 and 20, III) for the December 1955 boring indicates a greater shearing strength of some 2.1 ksf in the upper part of the clayey silt, even though it is stated to be mixed there with some peat. In the lower part the actual potential shearing strength is only approximately half as large, 1.2 ksf. This actual trend is the reverse of the one shown on Figure 20 (II) as corresponding to the hypothesis of purely axial drag loads.

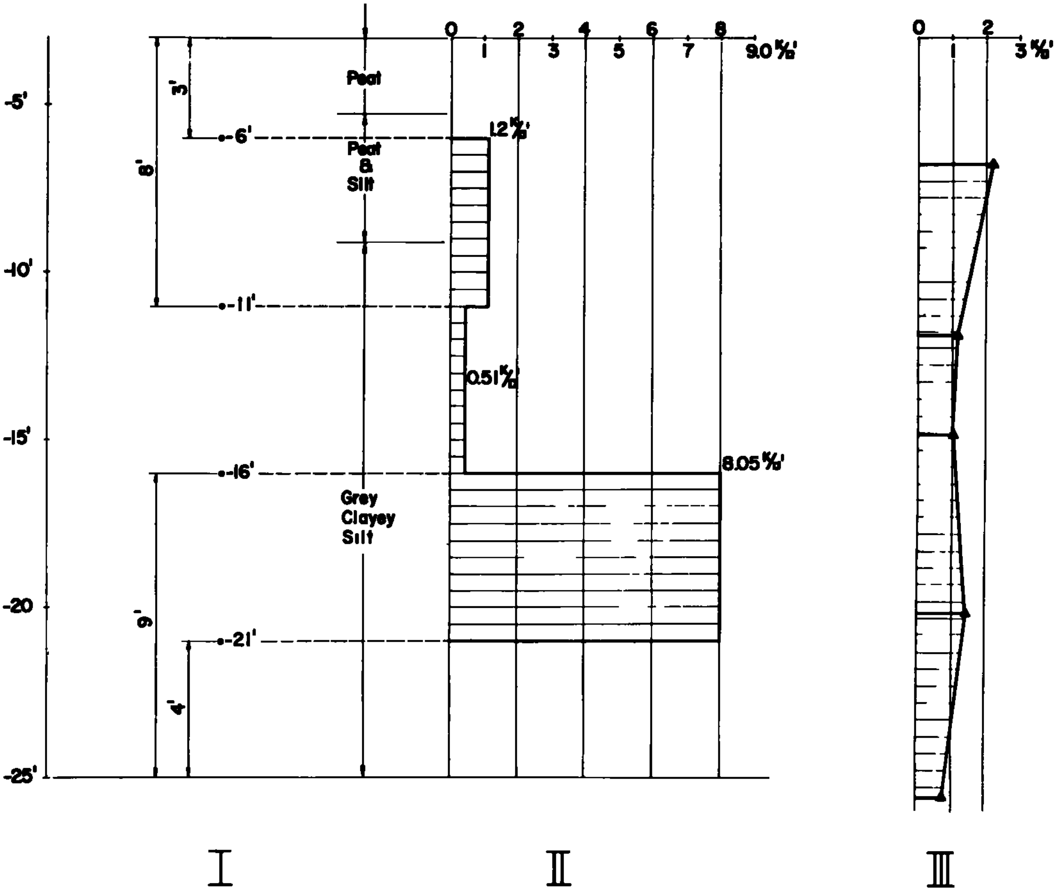


Figure 20. (I) Distance of points of measurement from pervious boundaries; (II) Shearing strength needed at face of pile if curve (I) of Figure 19 (also Figure 17) is to be explained by axial drag loads only; (III) Actual shearing strength of cohesive layer on December 1955 as per Figure 5 .

(b) Although the low rate of transfer of load (0.51 ksf) at the center of the clayey silt layer may be explained by a smaller degree of consolidation, this explanation cannot hold for the differences between the top (1.2 ksf) and the bottom (8.05 ksf)

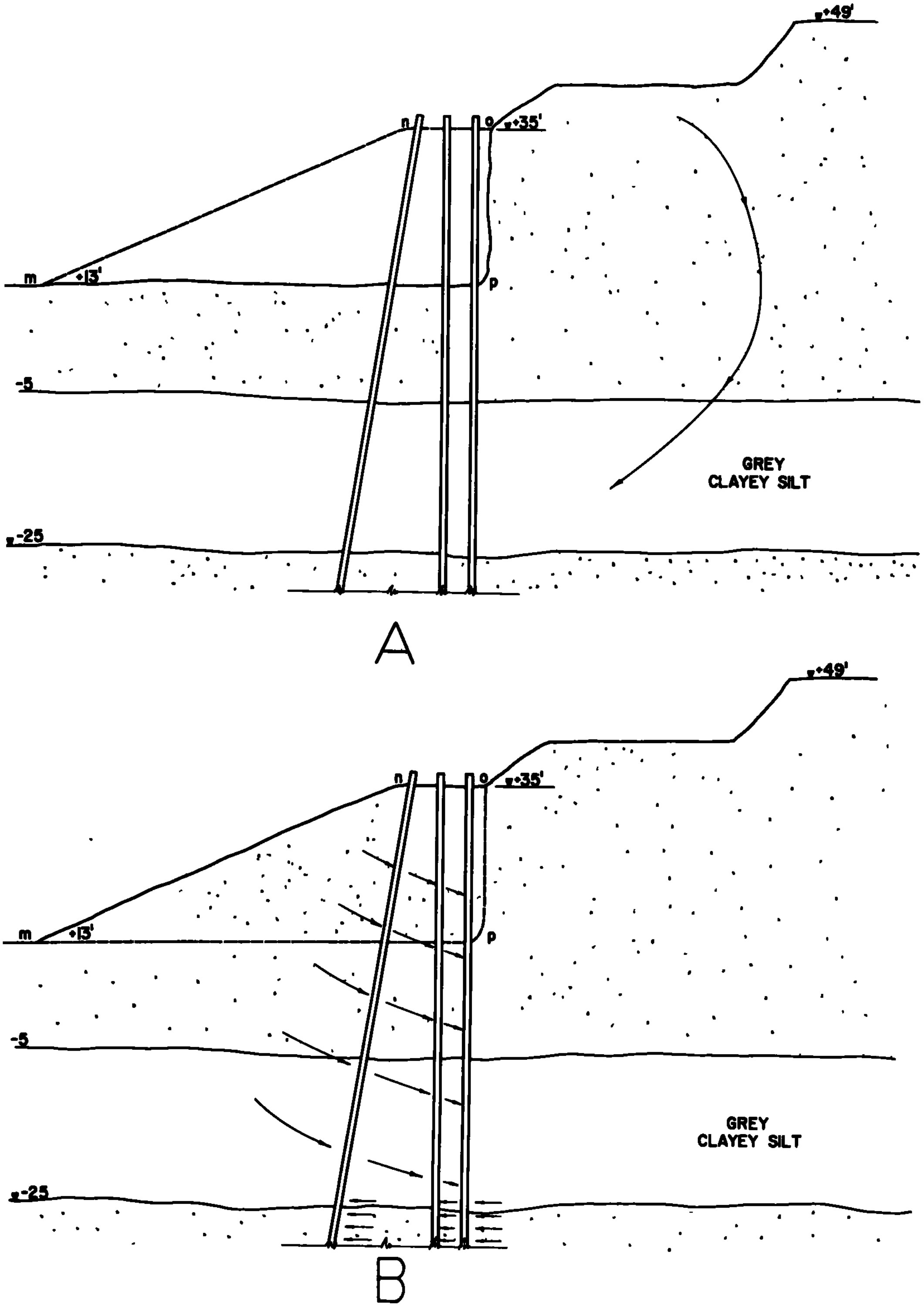


Figure 21. Soil deformations induced: (A) First by excavation of embankment wedge mnop and then (B) By its replacement after pile driving.

parts of the layer since the distances to pervious boundaries (Fig. 20, I) are approximately the same in both cases.

(c) The average ratio of the cohesion to the preconsolidation loads (PCL values) reported in Figure 5 is approximately 0.30. This is a reasonable value for this type of material. It therefore follows that a preconsolidation load of 26.8 ksf would have to be applied in order to obtain the cohesion of 8.05 ksf (Fig. 20, II). This value of the preconsolidation load is 26.80/4.34 = 6.2 times greater than the unrelieved weight of the overburden at elev -21 ft. This ratio would have to be much higher if the actual vertical pressures at that elevation are considered, since a large part of the weight of the overburden has already been transferred by drag to the piles. Thus, pressures of very high passive intensity would have to be present between elev -16 ft and -21 ft in order to explain strain gage readings there by drag only. Therefore, since drag essentially is a phenomenon of active or neutral pressures, it is not possible to explain these readings by drag only. Furthermore, it is inconceivable that pressures several times higher than the weight of the overburden could be localized for unknown reasons in the lower part of the clayey silt layer without affecting the overlying parts of that layer.

(5) It is most unfortunate that no reliable settlement readings of the experimental piles themselves appear to have been taken. This data could have shown in conjunction with the load test pile data (Fig. 11) whether 93 tons drag load actually had been applied to pile A.

(6) The statement by the authors that strain gages on the experimental piles B and C recorded in general a distribution of load similar to that of the pile A which is discussed by their paper contains basic contradictions with their hypothesis of drag loads as well as a clue to the solution of the problem:

(a) Vertical soil loads due to gravity forces cannot produce axial loads only in a batter pile; such forces must inevitably produce in a batter pile bending as well.

(b) It therefore follows that if vertical piles reacted to changes in load in a manner identical to that of a batter pile, then these changes in load must have produced bending not only in the batter pile, but in the vertical piles also.

(7) All the inconsistencies in the explanation by "drag" of these peculiar measurement results are resolved if the causes and the nature of possible bending of pile A are studied.

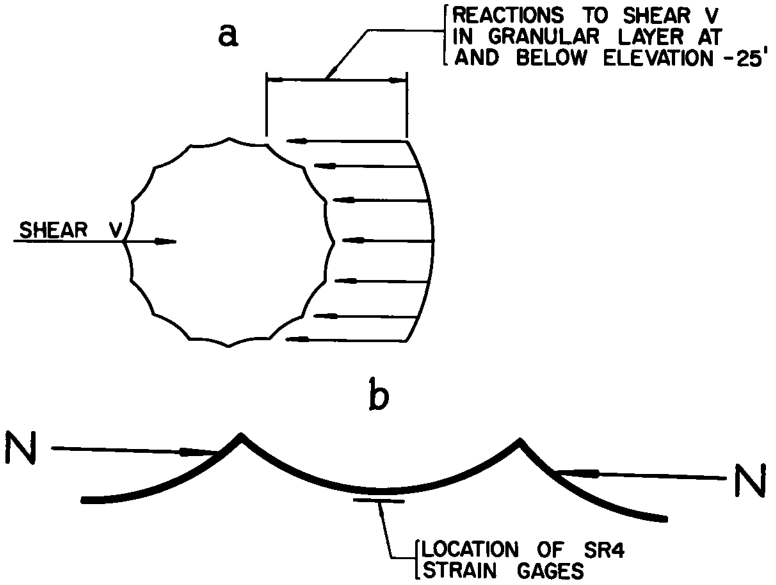


Figure 22. Sketch illustrating probable effect of unsymmetrical loading on SR4 strain gages mounted on empty shells of fluted Monotube piles.

(a) The removal of the prism mnop of fill shown (Fig. 21A) must have induced tendencies for slight soil movements (indicated by arrows on that diagram) prior to and during the pile driving.

(b) The replacement of the prism of fill mnop after the pile driving must have induced a reversed trend indicated by the arrows in Figure 21(B). As a result, compared to their zero readings on the strain gages, the pile shells must have then acted as slender dowels fixed in the non-cohesive granular soil stratum at elev -25 ft. The effects would have been felt just above and below elev -25 ft, exactly where a sudden and abnormal increase in strain gage readings was noted at elev -21 ft and below that level (Fig. 19).

(c) The fixation at elev -25 ft must have induced bending (primary) of the entire shell of the pile, plus the transmission of a shear V and resulting one-sided loading at that elevation.

(d) This type of one-sided loading (Fig. 22a) is bound to produce secondary bending in a horizontal plane through the fluted portions of the shell (Fig. 22b), that is, at right angles to the direction of the SR4 gages. Stresses exceeding the yield point can then easily be induced in a horizontal plane at the location of the SR4 strain gages and, due to the Poisson ratio effect can be recorded by them as high axial strains in a vertical direction. The primary and secondary bending combine to produce a complicated stress picture which it is now unfortunately impossible to analyze further since only 39 percent of the originally small number of strain gages actually operated.

CONCLUSIONS

1. The actual drag load on pile A does not exceed the values recorded down to elev -16 ft, some 60 kips or 30 tons.
2. The sudden increase of strain gage readings at and below elev -21 ft does not mean that the drag was increased there to some 180 kips or 90 tons as suggested by the authors. These readings merely reflect the bending and dowel action of the thin pile shells due to one-sided lateral pressures caused by the replacement after pile driving of some 22 ft of excavated fill.
3. The end slope of a high embankment is not the best place to study axial drag loads on vertical piles.
4. Thin fluted shells are not the best kind of piles for drag measurements by means of strain gage readings if any axially unsymmetrical loading is possible.
5. The number of SR4 gages installed was much too small in the first place. The usual practice of providing under laboratory conditions 50 percent extra gages in excess of the minimum needed should be changed under field conditions to provide 200 to 300 percent extra gages because of the hazards of field installation and operation.
6. A positive result of the whole investigation is the need which it demonstrates for consideration of bending stresses in all abutment piles driven through soft cohesive soil.

EDWARD V. GANT, JACK E. STEPHENS, and LYLE K. MOULTON, Closure—The discussion presented by Professor Tschebotarioff represents an interesting and thoughtful review of certain aspects of the investigation described in the paper. Professor Tschebotarioff's comments on the experimental procedure are especially helpful. However, his main thesis is that the loads measured for elevations below about -16 ft were produced by bending rather than by negative friction. He also comments on the lack of agreement between the values for drag load below elevation -16 ft, as determined in the investigation, and the magnitude and distribution of the loads which he computes on the basis of certain assumed boundary conditions.

Throughout the investigation particular attention was paid to the possibility of bending. Initially strain gages were installed (Fig. 8) in such a way as to eliminate bending effects. Careful analysis of all the gages which remained active failed to show anything more than rather minor bending in the vertical pile A. This was also true for the other vertical pile studied. The batter pile did seem to subject to bending, evidently caused by the eccentricity of the drag loads. In answer to a question raised by Professor Tschebotarioff on the distribution of load in the batter pile, it should be noted that the writers stated that the distribution in the exterior vertical and the batter pile followed a pattern similar to that shown for pile A (Fig. 17). It was not stated that the distributions were identical. It was unfortunate that most of the gages in the exterior vertical and batter piles were damaged. However, for the few gages which continued to operate in those piles it appeared that there were increases in axial loads in the cohesive layer at about the same elevation as for pile A.

There are other reasons also why the bending action described by Professor Tschebotarioff must be questioned. Thus, there is no explanation offered as to how the bending action would increase with time as the measured forces did, Figures 13-16. On the other hand, there is a large amount of experimental evidence available to indicate that for most compressible soils the shear resistance, and probably the bond between the pile and the soil, does increase with a decrease in excess hydrostatic pressure and added consolidation (Fig. 5 and references (4) and (8)). For the compressible soil adjacent to the experimental piles the buildup in shear strength, and therefore in drag forces, was produced by the re-consolidation of the soil disturbed by pile driving, and the continuing consolidation produced by the recent fill. Another factor which has some bearing on the question of bending is the effect of the removal of the large volume of fill during the period November 15-29, 1955. The bending action described by Tschebotarioff is based on the removal and replacement of fill over the relatively short length of abutment occupied by the experimental piles. In the period November 15-29, fill was removed to elevation +13 ft over the whole length of the abutment, except within the pile group where the fill slumped to elevation +20 ft. While the fill was not replaced immediately, it does seem that if bending of the type described actually occurred then the gages should have indicated a large change in the pile loads. Instead, the changes were fairly small (Table 1). In fact, Tschebotarioff draws a comparison between these changes and the weight of soil removed from within the pile group, and no mention is made of bending effects caused by the removal of the soil.

Tschebotarioff states that one of the essential criteria which limit maximum possible drag loads is the maximum possible weight of the soil which surrounds the pile. This leads to considerable ambiguity since the criterion does not define the role of the soil outside the pile group. Actually, according to his calculations, Tschebotarioff assumes (Fig. 18 and 19) that the soil outside the pile group has no effect on the interior pile A so far as drag is concerned. However, this overlooks the fact that the soil outside the group tends to settle relative to the piles and the soil within the pile group and, as a result, shear stresses are mobilized on vertical planes at the boundary of the pile group. If the deformation is great enough then the maximum shear resistance of the soil and the bond stresses on the pile surface are mobilized.

The mechanics of stress transfer from the soil outside the pile group to the soil within the group and individual piles is obscure and ideas on the distribution of the load along the pile cannot be fixed until further test results are available. Some of the contributing factors, however, can be identified. Thus it is known that the reconsolidation of a compressible soil after remolding by action of pile driving may result in values of unconfined compressible strength greater than for the undisturbed soil, (4), (8). Reese and Seed (8) found that the rate of increase in shear strength for soil adjacent to a pile agrees with the rate of decrease in excess hydrostatic pressure. Cummings, Kerkhoff, and Peck (4) have suggested the possibility of horizontal drainage as a factor in the growth of shear strength in the soil near a pile. For the test site in West Haven, there was the possibility of horizontal drainage at all levels of the compressible silt since there were vertical sand drains in the area. In addition the effect of the overlying fill was to produce a continuing consolidation through a combination of vertical and horizontal drainage. The conclusion must be, then, that the sum of the effects was to produce a large shearing resistance in the soil near the piles. The value could conceivably vary along the length of pile, depending on the extent of remolding at different depths and the rate of decay of excess hydrostatic pressure. There is little known concerning the variation in shear strength with distance from a pile. Casagrande (1) suggests that extensive remolding takes place near a pile with some disturbance up to one and a half diameters from the pile. If this is the case then the value of one and a half diameters might be considered the maximum distance within which the shear strength is increased over the value for undisturbed soil. Possibly the value varies over the length of pile and, as the writers pointed out, this may account for the variable rate of transfer of load to the pile A. It is, of course, impossible to reconcile the measured distribution with the calculations presented by Tschebotarioff, since he neglects the deformations that are produced in the soil outside the pile group and the resulting load transfer.

The purpose of the writers in presenting sample calculations of pile drag loads was simply to compare the results obtained from conventional design methods with the results obtained in the investigation. An additional result was the indication of the assumptions used in the comparison. It is certainly not to be expected that computations of the type used will result in a distribution of pile drag loads compatible with measured values. The best that can be hoped for in this type of approximation is that the maximum computed values to be used in design are somewhere near the actual values. For the case considered, and using the assumptions as shown, the design values did approximate the measured values. All cases, however, cannot be considered alike. In each case the consolidation that is likely to take place after piles are driven, and the effect of such consolidation on soil properties, should be carefully studied before a decision is reached on the possible build-up of drag loads.

Tschebotarioff has made an interesting comparison between the average decrease in load along the pile in the period November 15-29, 1955 and the volume of soil within the pile group that slumped when the soil outside the pile group was excavated to elev +13 ft. The principal criticism of the comparison is that the assumption is made that the unloading results in equal decreases in loads at all lower levels. The fact of the matter is that the decreases measured were not all the same (Table 1). The factors which contributed to the decreases were the decrease in the weight of soil between the piles and the rebound of the compressible layer. The decrease in the weight of soil within the pile group took place when the fill outside the pile group was excavated to elev +13 ft and the soil within the pile group slumped to an average elevation of +20 ft. However, pile A was an interior pile and the fill was somewhat higher around the pile than elevation +20 ft. The rebound of the compressible layer was caused by the decrease in total stress within the layer. Further, the decrease in total stress was, of course, caused by the excavation of fill to elev +13 ft, which was about 7 ft below the average height of fill within the pile group. Therefore, it is reasonable to expect that the rebound would result in differences in the decreases in load at the several levels. The use of an average decrease and the comparisons made therefrom are therefore not considered to be meaningful.

Tschebotarioff evidently did not understand the purpose that the writers had in mind in giving some reference to the literature on drag loads. The references to Moore (7), Chellis (21), Florentin and L'Heriteau (5) were made merely to give the reader some knowledge of what had been written on the subject and, at least, the opinion of others.

California Experience in Construction Of Highways Across Marsh Deposits

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California Division of Highways

Embankments have been constructed across marsh deposits in California by several different methods, including controlled rate of placement and use of berms, removal of soft compressible material by stripping or dredging, displacement of the weak soil by the embankment, and installation of vertical sand drains. Examples of each type of design are cited, with plots of observed settlement.

The importance of adequate exploration, testing and analysis is emphasized. The uncertainties and limitations of the application of theoretical soil mechanics principles are pointed out. Use of field permeabilities is proposed for calculating rates of settlement, and a method of measuring in-place permeability is described. Conventional methods of predicting settlement, derived from the theory of consolidation, are not always reliable when applied to fibrous peat. Examples are presented of embankment construction across peat beds, with comparisons of theoretical and observed settlement.

No one method of construction across marsh deposits is suitable or economical for all conditions. After thorough exploration and testing, the stability and settlement can be estimated for different designs by applying the principles of soil mechanics. The choice of design usually will be based on cost comparisons, taking into account the adequacy of service to the highway user.

● CONSTRUCTION of highways across marsh deposits always presents an important and difficult engineering problem, usually expensive to solve. As a result of improper design, excessive differential settlement may create a hazard to the highway user and necessitate costly reconstruction or repairs; or the embankments may be so unstable that failures or slipouts occur during construction and perhaps throughout the life of the facility. Placing the highway across marsh areas on pile trestles or structures is costly, and is not always a satisfactory solution, as the integrity of the structures may be jeopardized by the effects of settlement unless the piles are designed to resist the loading incurred by negative skin friction resulting from settlement of the marsh deposits.

In designing embankments two basic questions must be answered: (a) will the foundation soil support the required embankment load without displacement or shear failures; and (b) will excessive settlement occur due to consolidation of the marsh soil. Fortunately, by the application of soil mechanics, reasonably accurate solutions of these problems are possible in most cases. Methods for analyzing settlement by use of the theory of consolidation have been set forth by Terzaghi (1). The reliability of the answers depends largely on the thoroughness of the exploration, the quality of undisturbed samples, the use of suitable test procedures, and most important, the proper analysis and interpretation of the test data.

Based on the results of such soil tests and analysis, the soil engineer can estimate the permissible loading and the magnitude and rate of settlement for various loading conditions and methods of construction; the design then becomes largely a matter of engineering economics. If the marsh deposits comprise thick layers of weak highly compressible mud or peat, construction is likely to be costly, and the marsh area should be avoided if possible. But too often avoidance of the unstable areas will result in circuitous routing or substandard alignment, leaving no satisfactory alternative but to traverse the marsh area.

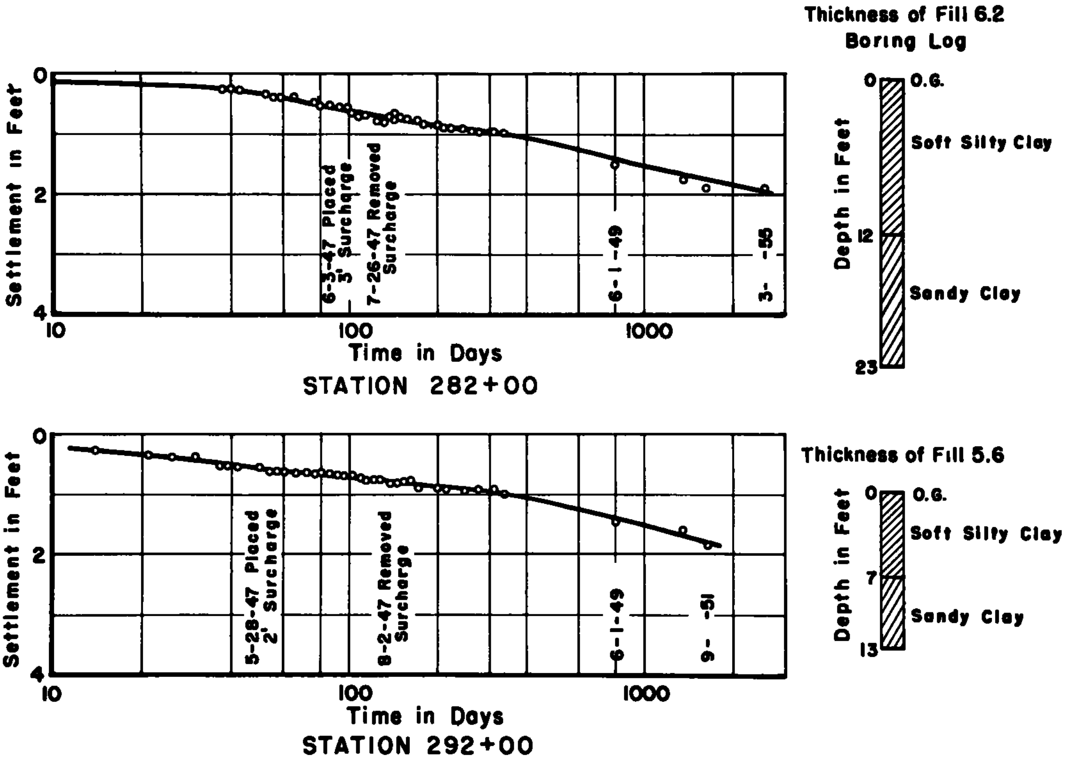


Figure 1. Observed settlement: Bayshore Freeway.

Several methods of construction have been used successfully in California, including: (a) controlled rate of placement and use of berms, (b) partial or complete dredging or stripping of the weak compressible soil, (c) displacing the soft material by the weight of the embankment without stripping, (d) use of vertical sand drains.

All of the special treatments are relatively costly, and frequently funds are not available to construct a project if stripping or sand drains are required. On less important routes carrying light traffic the cost of extensive foundation treatment may not

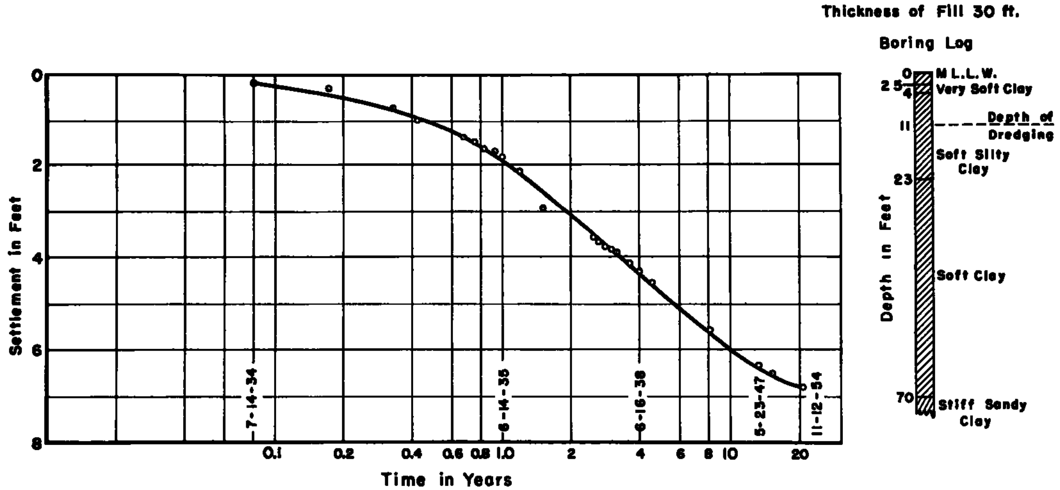


Figure 2. Observed settlement: San Francisco Oakland Bay Bridge Approach, Sta. 317 + 50 (L 21-S).

be warranted. If stability analyses show that the marsh soil will support the required height of embankment without danger of shear failures, the embankments may be constructed by conventional methods. If the analyses indicate that the factor of safety is dangerously low the embankment should be designed with berms and the rate of placement of the fill material should be carefully controlled. Embankments constructed by this method over thick layers of highly compressible soil will continue to settle over a long period of time. The post-construction settlement can usually be reduced by placing an overload or surcharge, provided the construction schedule is such that the surcharge can be left in place for a sufficient length of time prior to paving. Surcharges must be used with discretion, however, as the weak foundation soil often will not safely support the additional load. Furthermore, the surchrage is worse than useless unless left in place long enough to effect appreciable consolidation, which often requires a long waiting period.

It is important that reliable estimates be made of the magnitude and rate of settlement in order to determine whether the settlement can be tolerated, and for evaluating the probable cost of maintenance, including periodic reconstruction when required. Where interchange structures or bridges might be damaged by settlement of the approach fills, special foundation treatment, such as stripping or sand drains, is almost mandatory in the areas adjacent to the structures.

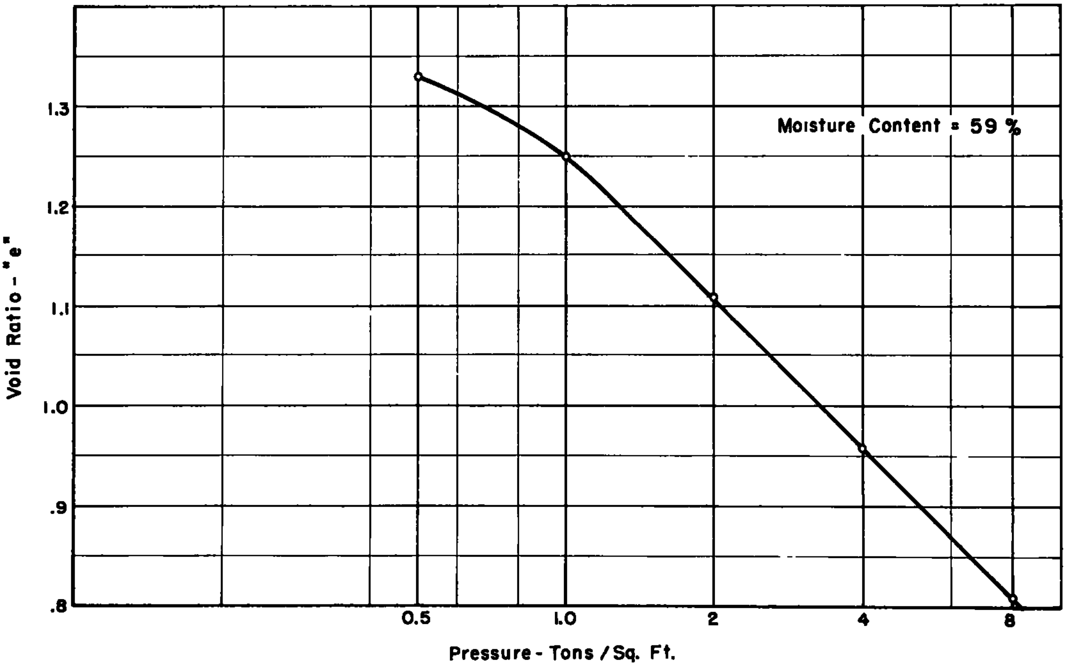


Figure 3. Typical pressure-void ratio curve, soft silty clay: San Francisco - Oakland Bay Bridge - East Approach.

CONSTRUCTION WITHOUT TREATING THE MARSH SOIL

Low embankments have been constructed across marsh areas without removal or treatment of the weak soil on several highway projects in California. Figure 1 shows plots of observed settlement at two points on a low embankment constructed over a marsh area where the soil consisted of 7 to 12 feet of soft silty clay (described locally as "bay mud"), underlain by stiff sandy clay. On this project sand drains were installed in the embankment areas adjacent to structures, but no special foundation treatment was applied on other portions of the project, where the height of fill did not exceed six or eight feet. Differential settlement after paving has resulted in noticeable

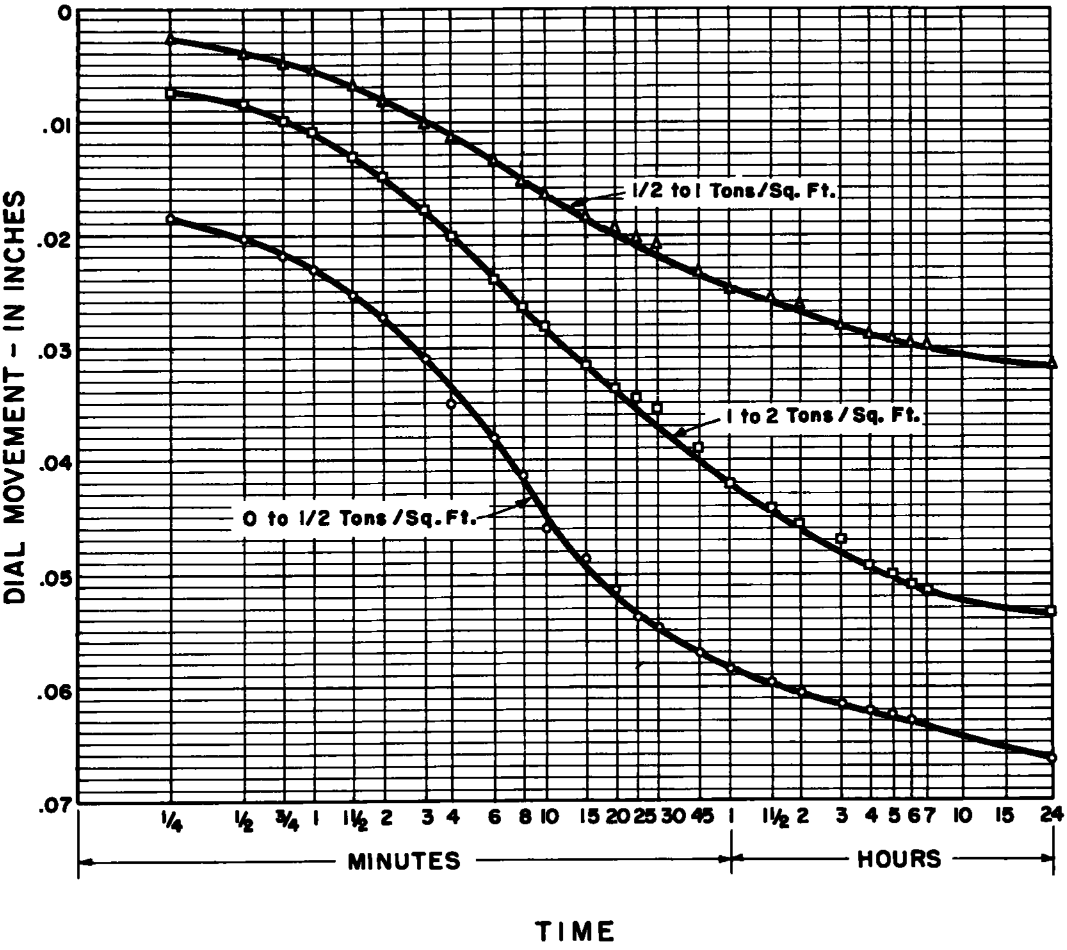


Figure 4. Typical laboratory time-consolidation curves, soft silty clay: San Francisco - Oakland Bay Bridge, East Approach.

distortion of the roadway profile and cross-section.

The easterly approach of the San Francisco-Oakland Bay Bridge is one of the earlier California projects involving embankment construction across thick deposits of very weak, highly compressible soil; here an embankment was constructed across a long section of tidal land where the depth of soft, silty clay (bay mud) was as much as eighty feet, and the height of fill was generally fifteen to twenty-five feet above the mud-line. About twelve feet of the very soft mud was removed by dredging, after which the sand fill was placed hydraulically, in stages. Settlement of the embankment has been recorded during the twenty years which have now elapsed since the fill was placed. Figure 2 is the settlement curve for a reference point at Station 317+50 where the depth of soft material prior to dredging was about seventy feet.

Note that the settlement to date at this point has been about seven feet. During the period from one to ten years after construction, the settlement was proportional to the logarithm of time, and occurred at a diminishing rate thereafter; the slope of the curve indicates that the primary consolidation may now be nearing completion. Figures 3, 4 and 5 are somewhat typical of the laboratory load-consolidation and time-consolidation curves, respectively, for the soft silty clay in this vicinity.

It is of interest to note that, although the settlement of this fill subsequent to paving ranged from 1 ft to 8 ft in a length of about 8,000 ft, the differential settlement has not seriously affected the riding qualities of the road and has not impaired the service to traffic.

STRIPPING OF THE UNSTABLE SOIL

If the thickness of the weak compressible soil layer is relatively small, the most economical and reliable treatment may be to strip and waste the unstable soil. This method was used on a project near Petaluma, California, where a high fill was to be constructed across a marshy area where the soil consisted of a ten-foot layer of very soft compressible soil, underlain by a stiffer and less compressible soil. The soft material was stripped to a depth of eight to ten feet and replaced by granular material from roadway excavation. Figure 6 is a plot of observed settlement of a point set at the bottom of the fill in the stripped area, where the thickness of embankment over the settlement platform was 69 ft. The observed settlement was due primarily to consolidation of a sandy clay layer eleven feet in thickness underlying the soft material which was stripped. The lapse of time between grading and pavement was such that the settlement was virtually completed before the pavement was placed. On other projects, where all compressible soil was stripped, practically no movement was recorded.

On the above project stripping was planned because the soil had such low strength that it would not support the required height of embankment. On other projects, however, compressible soil has been stripped under approach fills at structures in order to prevent excessive settlement which would damage the structure, even though the

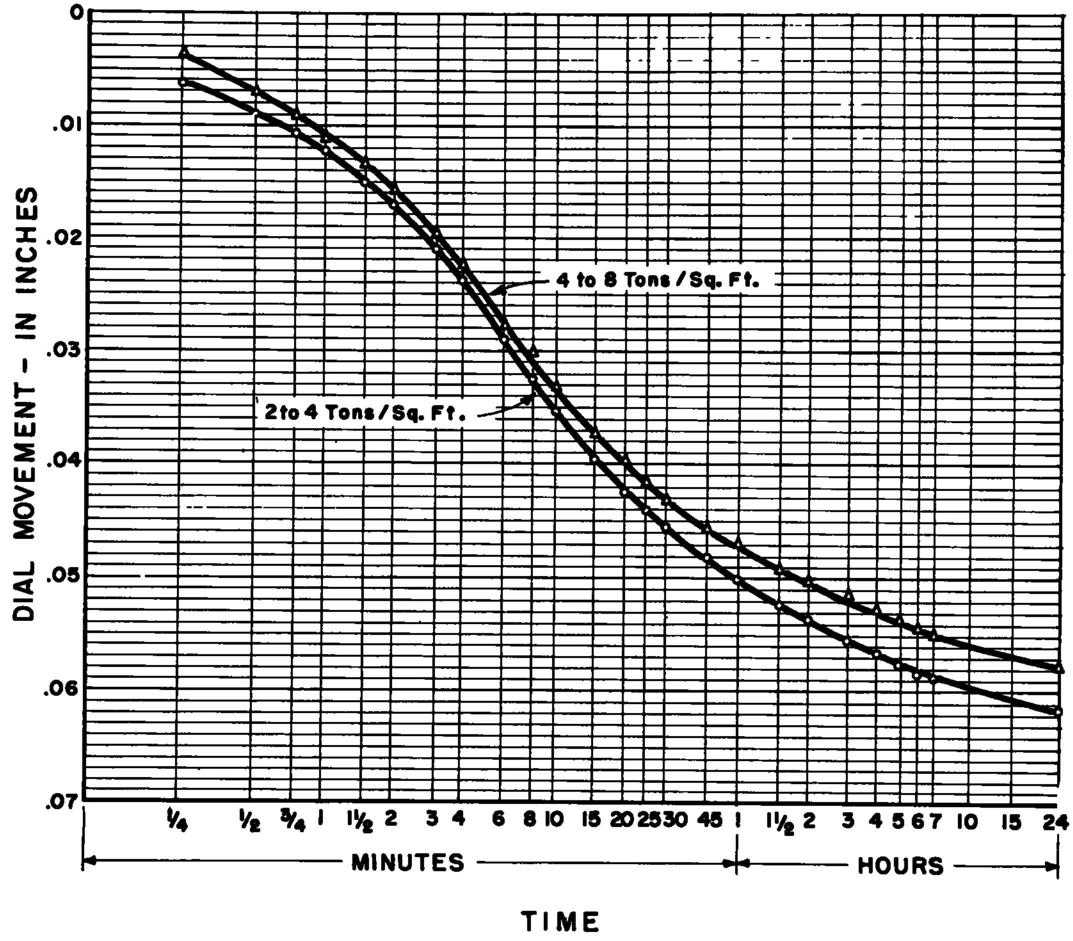


Figure 5. Typical laboratory time-consolidation curves, soft silty clay: San Francisco - Oakland Bay Bridge, East Approach.

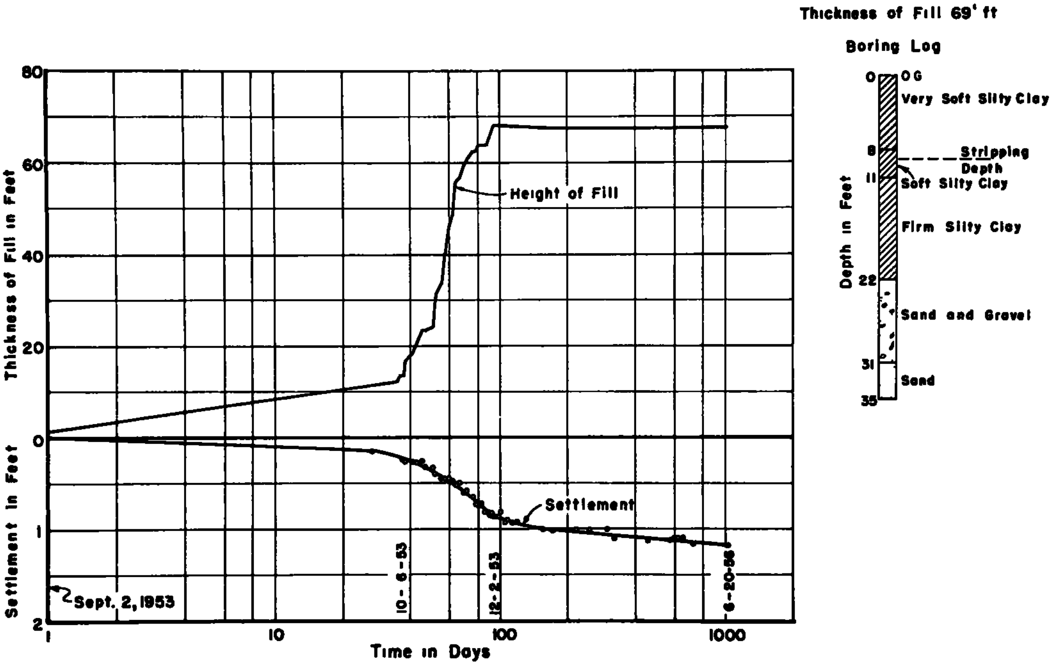


Figure 6. Observed settlement near Petaluma: IV-Son-1-F, Sta. 857 + 60.

shear strength of the soil might be sufficient to permit construction of the proposed fill without failures or slipouts of the fill.

If the depth of weak compressible soil is great, it is usually impractical to strip down to firm material. In such cases it may be possible to construct the required embank-

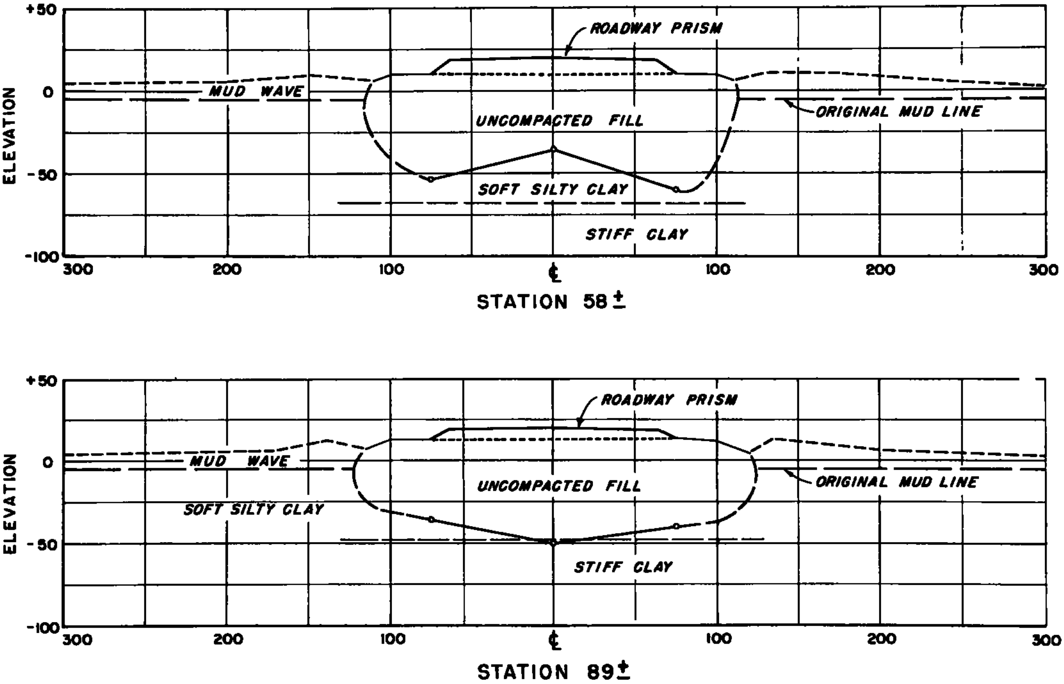


Figure 7. Cross-sections of fill at Candlestick Cove.

ment by removing only a portion of the weak material, by stripping to a depth which will permit the necessary loading without shear failures. Careful analysis must be made to determine the required depth of stripping, and even though no shear failures occur settlement of the embankment may continue for a long period of time. Partial stripping is seldom recommended if other more positive types of treatment are feasible

DISPLACEMENT OF MARSH SOIL BY WEIGHT OF THE EMBANKMENT

Displacement of peat by blasting has been practiced extensively in other regions, but very little work of this type has been done in California. The railroads have had considerable experience with displacement of soft mud by end-dumping rock into swamp areas until a more or less stable fill resulted. In most cases no concerted effort was made to displace all of the soft material, and often the completed embankment was subject to periodic shear failures; or, even if actual failures did not occur, the differential settlements over a long period of time were so great that maintenance costs were excessive, and normal service was difficult or impossible to maintain.

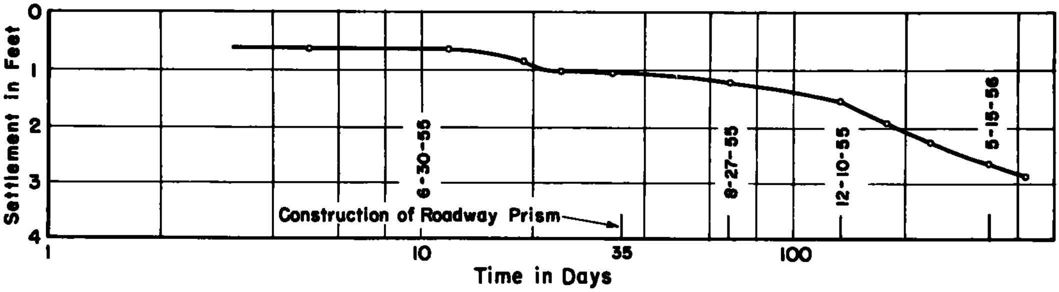
Construction of a freeway south of San Francisco, between Sierra Point and Candlestick Point, necessitated the crossing of an arm of the bay where the depth of water was about five feet; the soil in the bay at this location consisted of from forty to eighty feet of soft silty clay or "bay mud" underlain by relatively firm sandy clay or sand. After exploring the area and studying various methods of construction, it was decided to construct a short experimental section of embankment to determine whether sufficien mud could be displaced without dredging to permit construction of a stable embankment. It was found that an embankment of the required height could not be "floated" on the soft mud, but that with proper control of the embankment construction most of the soft material could be displaced during the filling operation (2). The embankment has now been completed across the two mile length of open water, and paving of the roadbed is in progress. It was, of course, impossible to displace all of the mud, and considerable settlement is anticipated. Figure 7 shows two cross-sections of the fill, and Figure 8 shows the recorded settlement at the same locations. At Station 58 the elevation of the bottom of the soft mud is about -67, and the thickness of mud remaining under the fill ranged from 7 ft to 37 ft at various locations on the section; corresponding values for Station 89 are: bottom of mud elev -49, and thickness of remaining mud 0 to 25 ft. Embankment was constructed to about elevation +19 at both stations.

VERTICAL SAND DRAINS

Numerous papers have been published describing the use of vertical sand drains in construction of embankments over marsh deposits. The first installation of sand drains by the California Division of Highways in 1934 has been described by O. J. Porter (3). Since that time vertical sand drains have been installed on numerous projects.

As a comprehensive survey and evaluation of sand drain installations throughout the United States is now in progress by others, this paper will not present any detailed analysis of sand drain projects in California. It is the author's opinion, based on study of sand drain projects in California, that when properly used, sand drains are effective in increasing the shear strength of marsh soils during the loading period and reducing settlement subsequent to construction. It is emphasized, however, that vertical sand drains are not a panacea for all foundation troubles, and should not be used indiscriminately.

Success of vertical sand drain treatment depends largely on two factors: (a) the rate of increase in shear strength of the soil must be sufficient to prevent the occurrence of shear failures, and (b) the rate of consolidation of the foundation soil must be such that the major portion of the settlement will occur during construction, and subsequent settlement will be within tolerable limits. It should be obvious that sand drains cannot be used successfully if the shear strength of the soil during the loading period will be so low that shear failures are inevitable, or if the soil consolidates at such a slow rate that primary settlement is not completed until years after the embankment is constructe Yet there are records of projects where sand drains were installed under such conditions, either because of neglecting to make the necessary stability calculations, or because of inaccuracies in computing the time rate of settlement.



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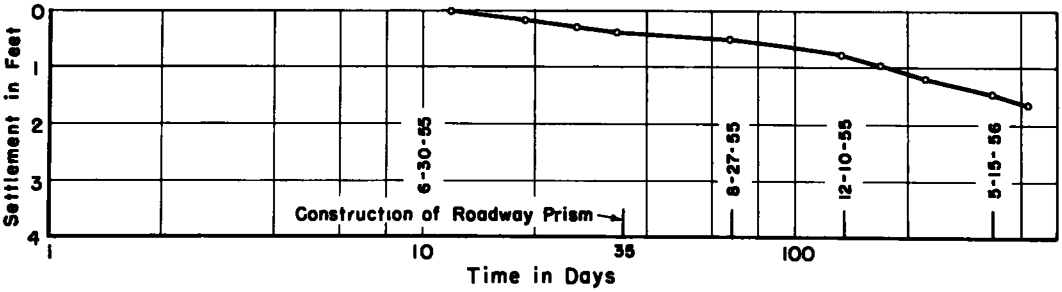

STATION 89+00

Figure 8. Observed settlement at Candlestick Cove.

There are many uncertainties and indeterminates involved in the analyses of stability and settlement in connection with sand drains, as will be discussed later. These known difficulties, however, strongly emphasize the importance of thorough, meticulous sampling, testing, and analysis before adopting sand drains as a treatment of marsh deposits.

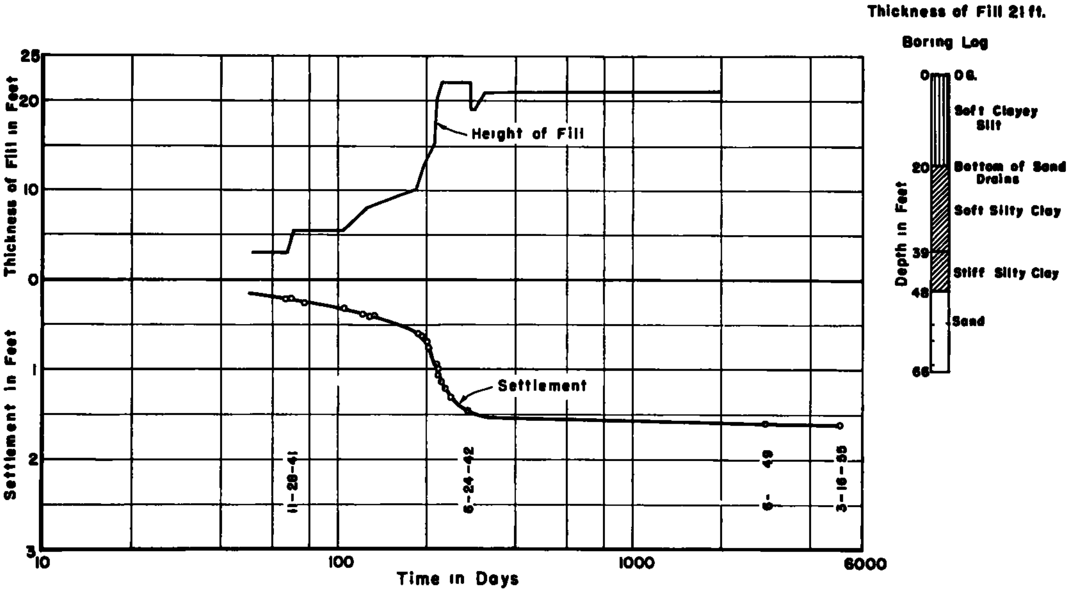

Figure 9. Observed settlement at Eureka slough-sand drain area.

Sand drains were installed at the approaches to the Eureka Slough Bridge in northern California, where embankments up to 25 ft in height were to be constructed over a layer of weak compressible soil. Figure 9 is a plot of the observed settlement at one of the settlement devices, where the embankment was 21 ft in height and the thickness of mud was about 20 ft. A total settlement of 1.5 ft has been recorded, but only about 0.2 ft has taken place after paving. No fill failures occurred in the sand drain area.

Recent construction north of the Antioch Bridge in the Delta area of California necessitated construction of embankments over deposits of fibrous peat. Sand drains were installed in two areas, one adjacent to the bridge end, and the other where the new road bed was located along the toe of an existing levee. At the latter location, where the height of embankment was to be 6 to 8 ft above original ground surface, there was 28 ft of soft fibrous peat and 22 ft of peaty clay underlain by firm silty sand. A typical cross-section of the completed embankment is shown in Figure 10. The contact between the fill and the peat was determined by borings. There was no evidence of shear failures or displacement. Settlement of about 12 ft occurred at Station 80 + 10, for which the time-settlement curve is shown in Figure 15. Maximum settlement of over 18 ft was recorded in this area.

Figures 20, 21 and 22 show settlement records on portions of three projects on which vertical sand drains were installed. These plots show the observed settlement within sand drain areas, and for comparison, similar records for adjacent areas where no sand drains were used. The settlement during construction, prior to paving, is plotted, as measured by settlement platforms. Also shown on the plots is the settlement which occurred after the pavement was constructed. These post-construction settlements were measured either by pavement profiles or elevations on surface monuments.

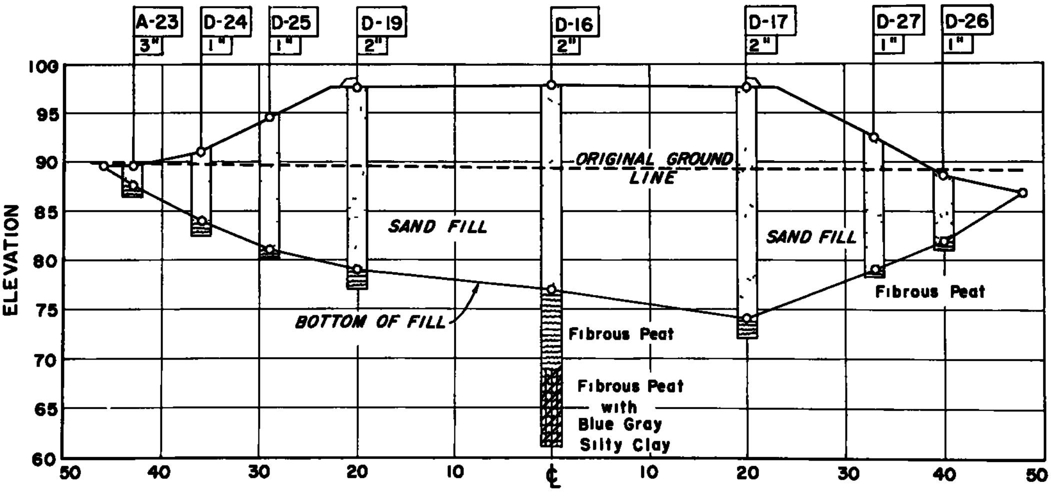

Figure 10. Cross-section of fill in sand drain area: North Approach Antioch Bridge, X - Sac. - 11 - C, Station 18 + 75.

It is emphasized that the height of fill, depth of compressible soil, and soil types are not usually identical in the sand drain areas and the adjacent areas without sand drains. Even though the settlement data do not provide a direct comparison, it is felt that the records of observed settlement may be useful in evaluating the effect of the vertical sand drain treatment on these particular projects.

TIME RATE OF CONSOLIDATION

The strength and load-consolidation characteristics of saturated inorganic clays can be evaluated with considerable confidence. There is, however, some uncertainty in

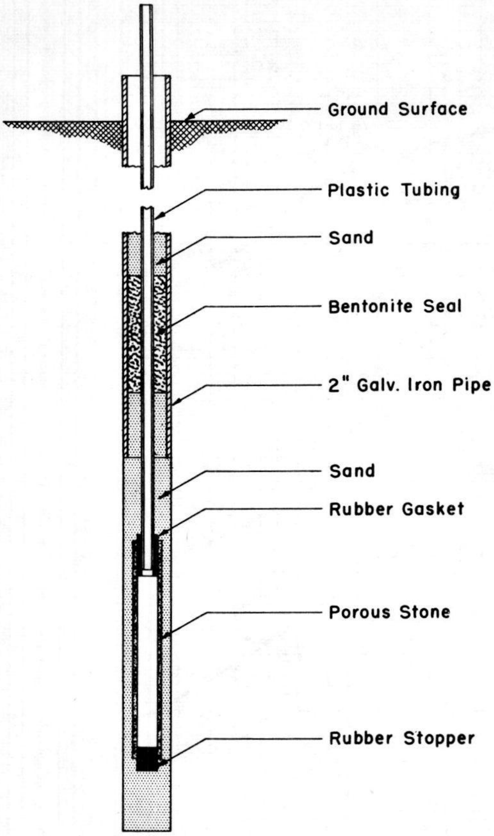

Figure 11. Schematic sketch of piezometer for field permeability measurement.

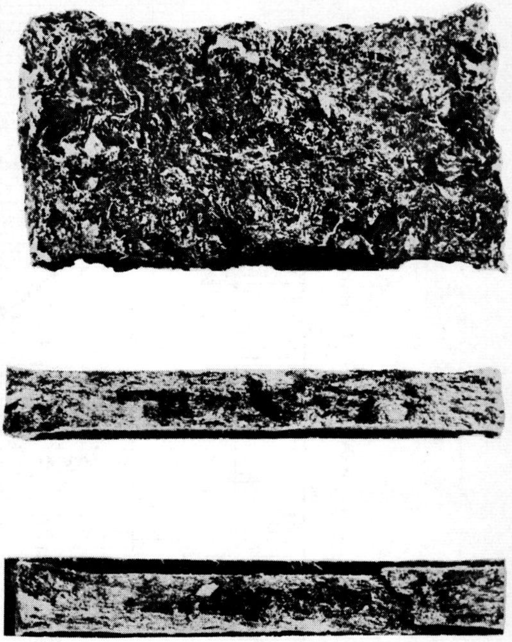

Figure 12. Oven-dried peat, original water content of 600%. Upper photo shows vertical section of unconsolidated material. Lower photo shows similar material after 75% consolidation under 8 T.S.F. load.

estimating time-consolidation relationships, except in the rare instances where the soil is relatively homogeneous and isotropic. The effect of sand drains on the time rate of settlement is computed by the method proposed by Barron (4). This calculation requires determination of the coefficient of consolidation in both the vertical and horizontal directions. Although horizontal permeability is probably the most important factor in estimating the effect of sand drains, no satisfactory reliable laboratory test method has been devised for measuring permeability as it affects radial flow. Determination of field permeabilities may provide a more reliable method of estimating rates of consolidation.

The field permeability test is made by installing, in the soil layer being studied, a porous stone about 1½ in. in diameter and one to five ft in length, with a plastic tube leading from the porous stone to the ground surface. The porous stone is commonly installed in a 2½ in. diameter hole, with sand backfill. A schematic drawing of the permeameter is shown on Figure 11. The piezometer is left in place for several days before taking readings, to allow the system to reach pressure equilibrium. After careful checking to assure that the piezometer is functioning properly, the piezometer head in the tubing is lowered by pumping out water. The piezometer level is measured and recorded at measured time intervals, and the ratio of measured head to original head is computed for each time interval; the log of this ratio is plotted against time. The basic time lag and coefficient of permeability are then computed, using the method described in Waterways Experiment Station Bulletin No. 36 (5).

Instead of lowering the piezometer level and performing the test with a rising level, as outlined above, the piezometer level may be raised by adding water to the system, and the test performed with a falling head. The question of which method may be more advantageous depends on conditions. Work done by this department, based on the procedure described by Waterways Experiment Station, indicates better correlation be-

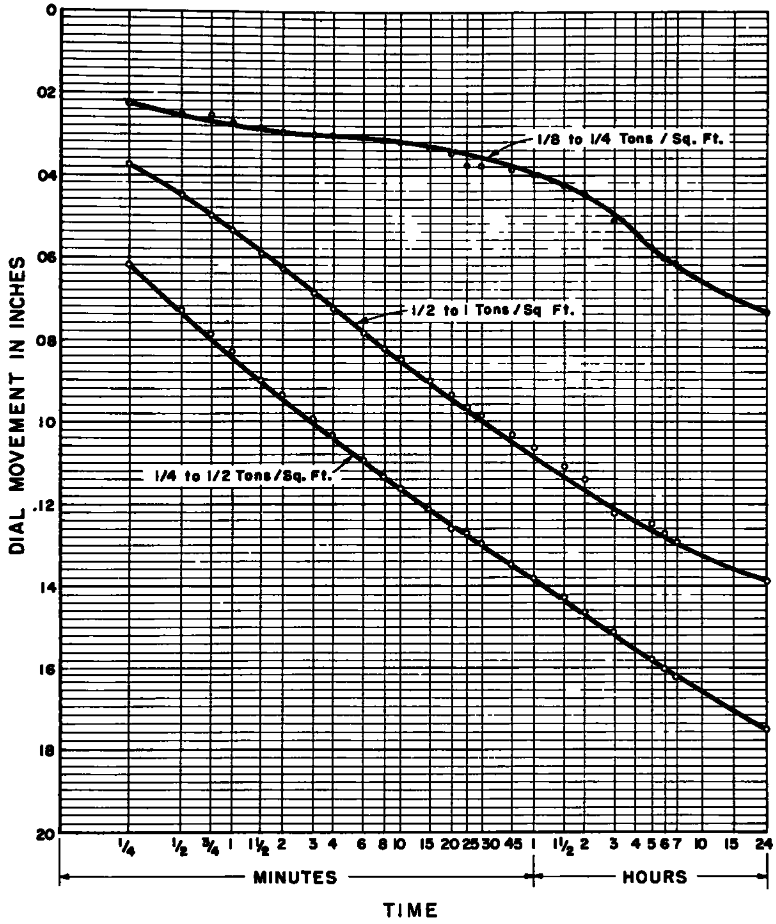

Figure 13. Typical laboratory time-consolidation curves: Antioch Peat.

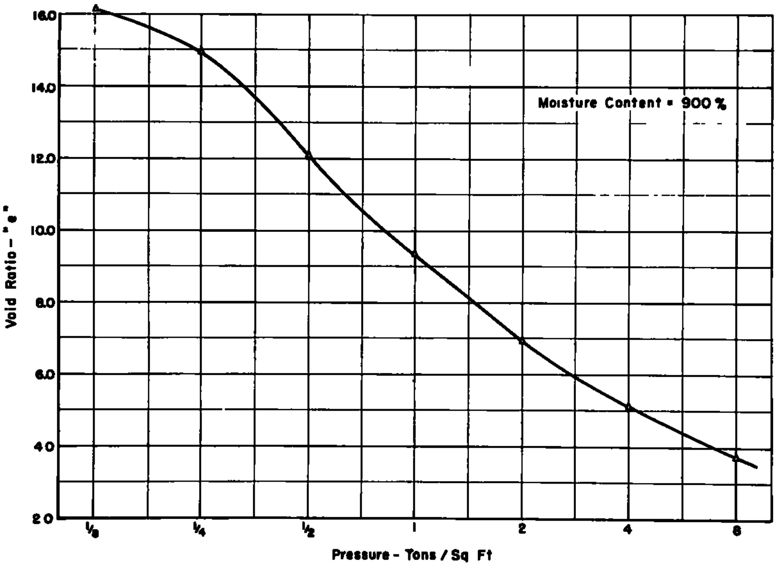

Figure 14. Typical pressure-void ratio curve, peat: Mokelumne River to Potatoe Slough.

tween field permeabilities and observed rates of settlement than by the use of coefficient of consolidation determined in conventional laboratory consolidation tests. However, much additional investigation will be necessary before any conclusions of general validity can be formulated.

CONSOLIDATION OF PEAT

It is the author's opinion that consolidation properties of fibrous peats cannot be evaluated accurately by application of the usual consolidation testing procedures and the generally accepted theories of consolidation. For inorganic clays there is reasonably good agreement between estimated and observed amount of settlement, and at least a semblance of correlation between computed and actual rates of settlement when based on consolidation tests of good quality undisturbed samples. This does not appear to be true in the case of fibrous peats, where numerous difficulties are encountered in the interpretation and application of consolidation test data, and the accuracy of settlement predictions for peats is likely to be of a low order.

There is need for a rational system of classifying organic soils and peats. Such soil may consist of pure peat with virtually no disintegration of the plant forms, various gradations of mineral grains and partially decomposed plant remains, or soil composed primarily of silt and clay but with high organic content. Figure 12 is an enlarged photograph of a sample of peat from the Delta area north of the Antioch Bridge. In its natural state the moisture content of the samples was from 550 percent to 650 percent. In order to show more clearly the texture of the peat, a cross-section of an oven dry sample 1 in. high by 2 in. diameter was photographed and enlarged three diameters. Also shown is an enlarged photograph of a similar sample after consolidating under a load of 8 tons per sq ft, and oven drying. Note the dense texture and apparent absence of fibers in the consolidated specimen.

The shapes of time-consolidation curves for peats vary greatly, and it is difficult to select a typical curve, especially at loads of ½ ton per sq ft or less. Figure 13 illustrates the more common types of time-consolidation curves, and Figure 14 a load consolidation curve for peats in the Delta area of California with moisture contents of over 1,000 percent. Work done by this department confirms the conclusions reached by Thompson and Palmer (6) and others, that there is little similarity between time-consolidation curves for peat and those obtained for clays. The concept of primary and secondary consolidation, generally accepted for clay soils, is not considered applicable in the case of peats, where rate of consolidation does not appear to be a function of drainage. This is evidenced by the fact that the rate of strain in the consolidation tests of peat is independent of the height of specimen.

In general, the estimates of settlement derived from consolidation tests of peat with the usual twenty-four hour loading period are considerably lower than actual settlements recorded during and after construction. It has been found by W.A. Lewis (7) that after a load increment in the consolidation test is left in place for fifty days the rate of consolidation has become exceedingly small, although slight compression may still be occurring. This long loading period was not proposed as a routine test procedure, and would not be practicable for such use. Some modified method of testing and interpreting the test data is needed which will permit more realistic estimates of settlement of fills constructed over peat beds.

If it is accepted that the rate of consolidation of peat is not a function of drainage, it is logical to question the use of vertical sand drains in peat deposits. There is substantial, although not conclusive evidence, that sand drains effect an increase in shear strength of the peat during construction. Unfortunately, no records are available on embankments of critical height constructed by California Division of Highways over identical peat deposits with and without sand drains. Only by such evidence could it be proved conclusively that the use of sand drains permitted construction of a stable embankment which, without sand drains, would have resulted in shear failures. However, based on California experience with construction over peat beds, it is believed that vertical sand drains accelerate the increase in strength of peat deposits during the loading period. On the Antioch project referred to above, for example, a total thickness of embankment as great as 25 ft was constructed with sand drains, with no measurable displacement or shearing, even though settlements of as much as 18 ft have occurred. On previous fill construction over similar peat beds in the same region shear failures or displacement of the peat occurred under much lighter loading. At this sand drain location the moisture content of the peat before construction averaged about 600 percent, and unconfined compressive strength ranged from 0.2 to 0.3 tons per sq ft; samples taken from the same location after completion of the embankment had moisture contents of about 300 percent and unconfined compressive strengths of 0.4 to 0.8 tons per sq ft.

It has been suspected for several years that settlement analyses of peat by the conventional methods developed for clays are not always reliable. Because no better meth has been developed, the magnitude and rate of settlement have been estimated by the same procedures as for clay soils. Previous experience in construction over the peat deposits of the Delta area of the California central valley has been helpful in estimating settlements on proposed construction in the same region, but peat soils in other areas may not have the same consolidation or strength characteristics.

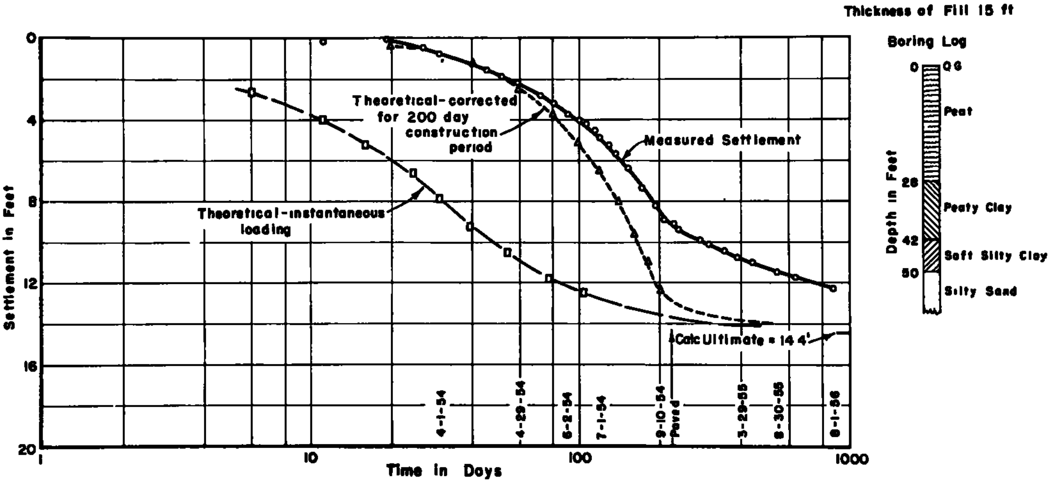

Figure 15. Comparison of measured and theoretical settlement: North Approach to Antioch Bridge - sand drain area, Sta. 80 + 10.

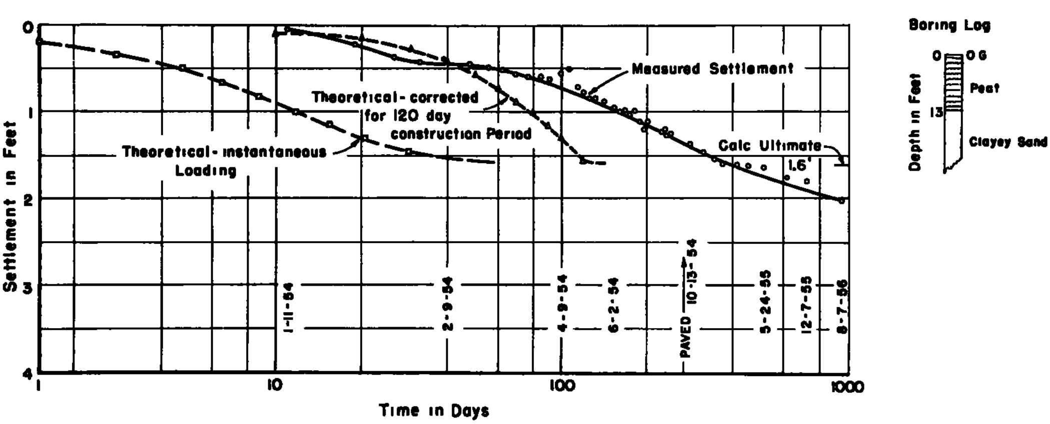

Figure 16. Comparison of measured and theoretical settlement: North Approach to Antioch Bridge, Sta. 113 + 00.

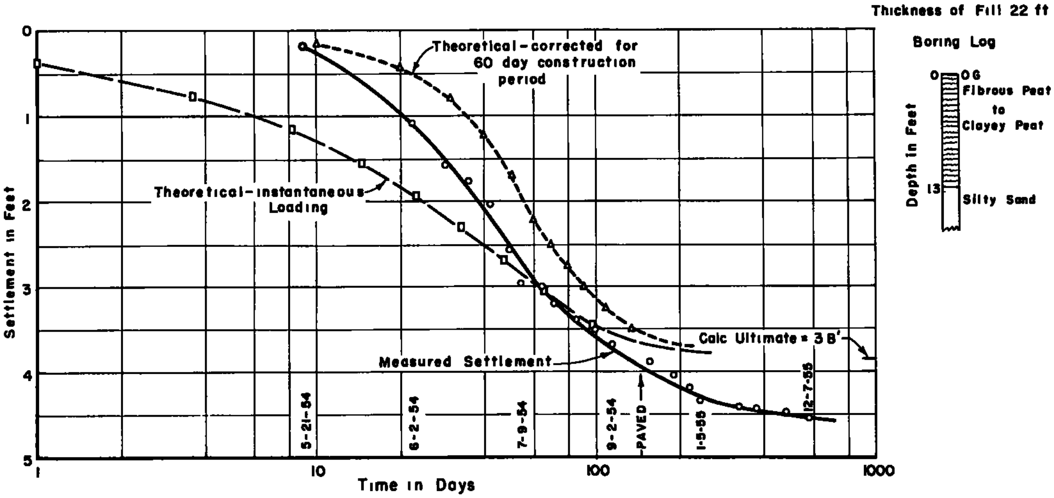

Figure 17. Comparison of measured and theoretical settlement: North Approach to Antioch Bridge, Sta. 214±.

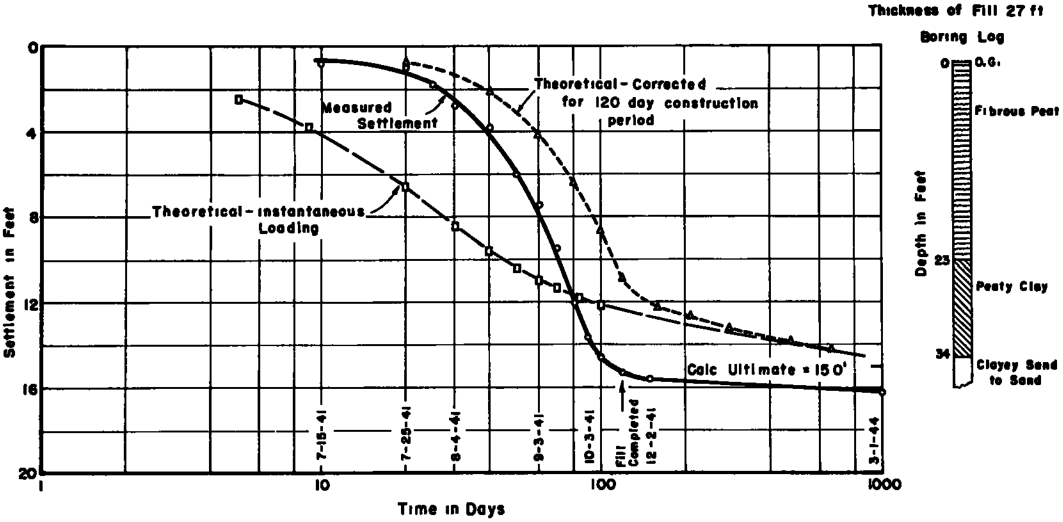

Figure 18. Comparison of measured and theoretical settlement: Mokelumne River to Potato Slough - sand drain area, Sta. 11±.

In general, the settlement occurring during construction approximates the ultimate settlement for the peat as computed from the 24-hr consolidation tests by the same procedure used for clays, but there are numerous unexplained discrepancies. The rate of settlement greatly diminishes a short time after loading of the fill is discontinued; thereafter, the settlement follows a straight line on a semilog plot, but the slope of the line varies markedly even between points where the fill loads are comparable and the peat appears similar as to thickness of bed and moisture content. For light fills over these peat deposits the post-construction rate of settlement commonly ranges between 0.5 and 1.5 ft per log cycle of time, and the rate does not appear to be proportional to the thickness of the peat layer.

Comparisons of computed and observed settlements of embankments over peat deposits are shown by Figures 15 to 19. Two of the comparisons, Figure 15 and Figure 18, are for areas in which sand drains were installed. The settlement calculations for all of the theoretical curves were based on conventional consolidation tests with a

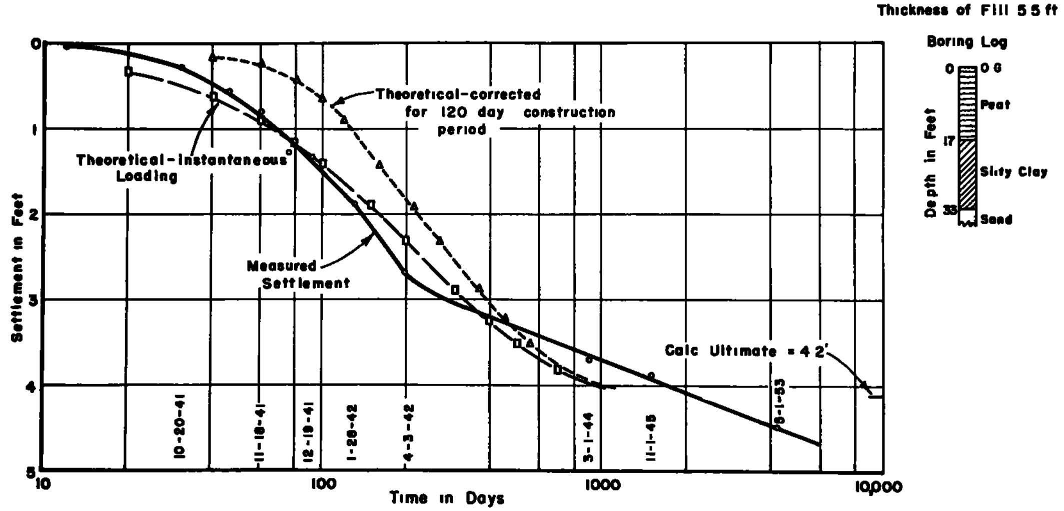

Figure 19. Comparison of measured and theoretical settlement: Mokelumne River to Potato Slough, Sta. 121±.

24-hr loading period for each load increment. The time-settlement relationships for the peats, however, were derived by use of field permeabilities.

Note that in the sand drain area at Station 11, Figure 18, the total settlement to date has been 16.2 ft, but only about 0.6 ft occurred after construction. The calculated ultimate settlement was 15.0 ft, or 93 percent of the measured settlement. In this instance there was good agreement between estimated and actual settlements.

At the other sand drain area at Station 80 on the Antioch project, Figure 15, the

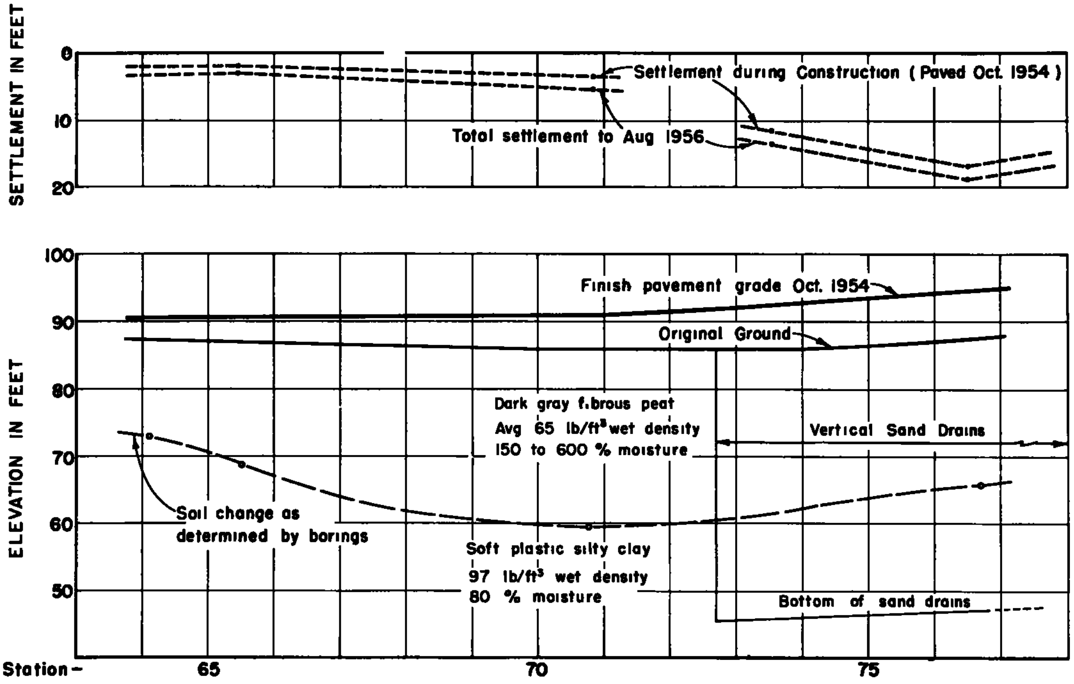

Figure 20. Observed settlement in areas with and without sand drains: North Approach to Antioch Bridge, X-Sac-11-C.

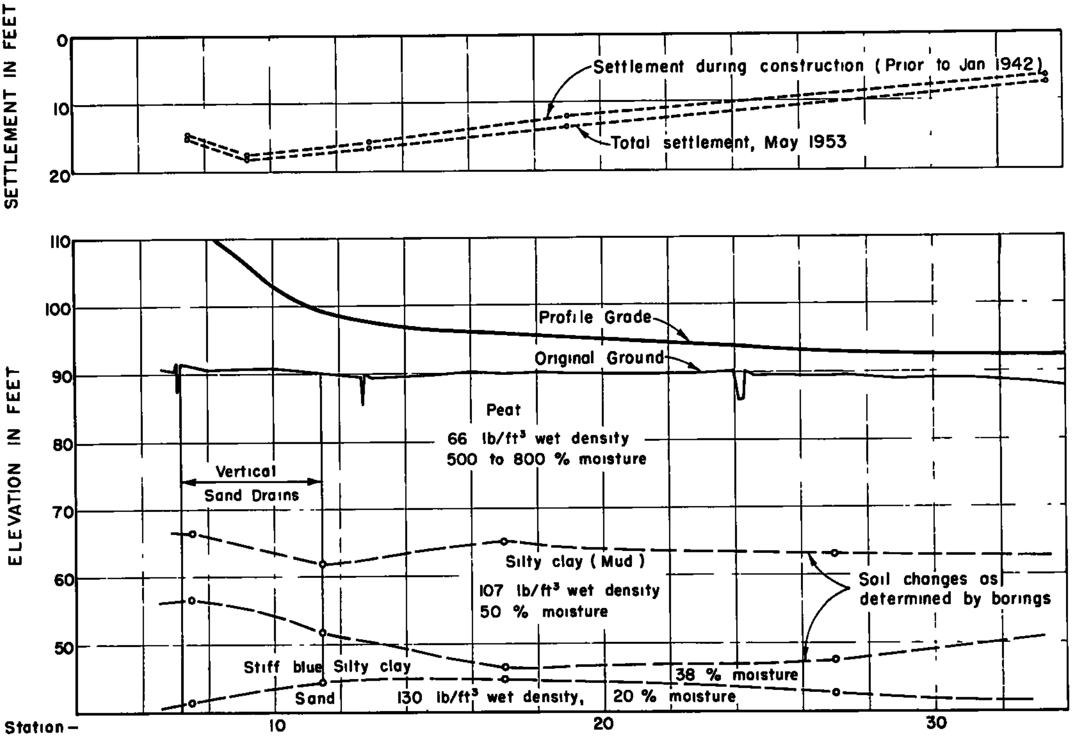

Figure 21. Observed settlement in areas with and without sand drains: Mokelumne River to Potato Slough, X-SJ-53-C.

measured settlement to date has been 12.3 ft, which is 86 percent of the calculated ultimate, and the settlement is not completed. In this case, however, almost four ft of settlement has taken place subsequent to paving, and the fill is still settling.

At the other three locations, where no sand drains were installed, the settlements were as follows:

Figure	Station	Calculated Ultimate	Observed To Date	After Paving
16	113	1.6 ft	2.0 ft	1.0 ft
17	214	3.8 ft	4.6 ft	0.6 ft
19	121	4.2 ft	4.5 ft	2.5 ft

It is evident that there is a fair agreement between calculated and observed total settlements, but the magnitude of the post-construction settlement varies widely.

It is questioned whether primary consolidation can be identified in the consolidation tests of peat; if there is primary consolidation it probably takes place during the very early stage of the test, and is complete in one to ten minutes. The long-time consolidation, which is evident in both the laboratory test and under field loading, is proportional to the logarithm of time. This phenomenon is not clearly understood, but the long-term compression may be due to plastic deformation, slow structural failure of the peat fibers, and slow disintegration of the plant forms in the past.

It is possible, also, that this is not entirely secondary consolidation, at least during the intermediate stages of the total consolidation period. The initial permeability of the peat before loading is relatively high, but diminishes rapidly as the peat is compressed; accordingly, at some time subsequent to the completion of the initial "primary consolidation" the permeability of the peat may decrease to the point where the rate of consolidation is influenced or controlled by the dissipation of pore pressure. For example, on the Antioch project mentioned above, field permeabilities of the peat

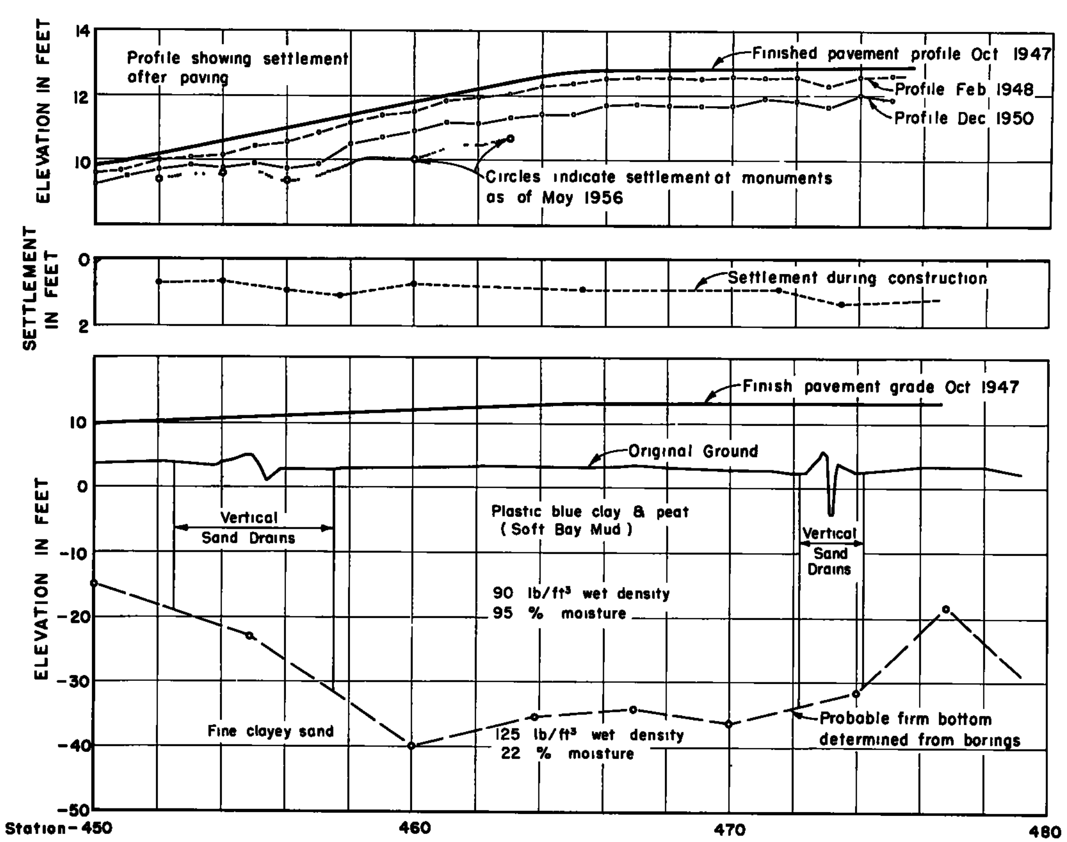

Figure 22. Observed settlement in areas with and without sand drains: Bayshore Freeway, IV-SM-68-F.

layer were determined before and after loading at three locations. The field permeabilities were as follows:

Permeability in ft/hr

Station	Initial	During Loading	Settlement At Time Of Second Measurement
70+75	1.2 x 10^{-3}	1.2 x 10^{-4}	4.0 ft
214+00	1.4 x 10^{-2}	2.6 x 10^{-5}	3.6 ft
215+00	1.7 x 10^{-2}	3.3 x 10^{-5}	4.4 ft

On other projects in the Delta area the permeabilities of the peat, as measured by the field method, ranged from 1 x 10^{-2} ft/hr for no loading to 1 x 10^{-5} ft/hr under fills of ten to fifteen ft. The supporting data are admittedly meager, and these results are cited as being perhaps indicative, rather than conclusive.

Fairly adequate test data and long-time settlement records are available on several California highways projects involving embankment construction over peat beds. It was hoped that by analysis of these data a reliable empirical or rational method could be formulated for estimating rate and magnitude of settlement. However, because of numerous unexplained anomalies and discrepancies, no satisfactory procedure has yet been developed for predicting settlement of embankments constructed over thick beds of peat.

SUMMARY AND CONCLUSIONS

Various methods of embankment construction over marsh deposits have been employed on California highways, and each has been successful when used where conditions were appropriate. By proper utilization of the principles of soil mechanics the performance of the structure can be predicted for the various alternate construction methods. The economic phases of the design cannot be ignored, and a rational decision on type of treatment can be reached only by thorough evaluation of engineering economics, with consideration of cost of construction, probable costs of maintenance and reconstruction, and the adequacy of service to traffic.

It is seldom economical to design embankments across marsh deposits without some type of special treatment unless preliminary investigation indicates that the foundation soil will support the proposed loading with some reasonable factor of safety. The time element is often the controlling factor — stable embankments and levees have been successfully constructed over marsh deposits by slow stages during a period of several years, whereas, attempts to build similar embankments in one or two years have resulted in complete failure.

Where less important roads carrying light traffic must traverse extensive marsh deposits it may be possible to support light embankments without stripping of the marsh soil or other costly treatment. Even though settlement of the road may occur for many years, and periodic restoration to grade may be required, this may be less costly than extensive foundation treatment; or, even if the over-all ultimate costs of the two were comparable, funds would not be available for the large initial investment involved in the stripping or sand drain treatment.

The most positive treatment of marsh deposits is, of course, removal of all soft compressible material and replacement with granular fill material. If the depth of the unstable soil is only a few feet, stripping is generally the most economical solution, but is not feasible where the peat or mud is of great depth. Vertical sand drains have been used successfully in cases where the rate of consolidation of the marsh soil was sufficiently rapid that the weak material acquired early shear strength to prevent shear failures, and where most of the primary settlement occurred during the construction period. The sand drains are more likely to be effective if the fill load can be applied slowly over a long period of time, and a surcharge canbe left in place for several months before the road is paved.

No one method of construction over marsh deposits is suitable for all conditions. A rational design can be achieved only after thorough investigation and analysis by judicious application of the principles of soil mechanics, together with an engineering economics study.

ACKNOWLEDGMENT

All of the foregoing data were selected from job records in the Materials and Research Department of the California Division of Highways, which is under the supervision of F.N. Hveem, Materials and Research Engineer. The author wishes to acknowledge, with sincere appreciation, the assistance given by all those who helped compile the data. Special credit is due Travis W. Smith for his valuable suggestions and assistance, and to William G. Weber and William F. Kleiman who helped to compile and analyze the data.

REFERENCES

1. Terzaghi, K., "Theoretical Soil Mechanics," J. Wiley and Sons, 1943.
2. Smith, Vincent, "Open Water Fill," California Highways and Public Works, Nov.-Dec., 1955.
3. Porter, O.J., "Studies of Fill Construction Over Mud Flats," including a description of experimental construction using vertical sand drains to hasten stabilization. HRB Proceedings, Dec. 1938.
4. Barron, R.A., "Consolidation of Fine Grained Soils by Drain Wells," Trans. ASCE Vol. 113, 1948, p. 718.

5. "Time Lag and Soil Permeability in Ground Water Observations," Bulletin No. 36, Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.

6. Thompson, James B. and Palmer, L.A., "Report of Consolidation Tests with Peat," Special Technical Publication No. 126, American Society for Testing Materials, 1951.

7. Lewis, W. A., "The Settlement of the Approach Embankments to a New Road Bridge at Lackford, West Suffolk," Geotechnique, Vol. VI, No. 3, Sept., 1956.

Review of Uses of Vertical Sand Drains

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Vertical sand drains have been for the stabilization of soft and compressible soils when the foundation soil is either too weak to support a proposed fill or structure or is so compressible that large and long continued settlements would occur following completion of construction. Credit for the idea of using sand drains for stabilizing soils apparently belongs to Daniel E. Moran, who submitted a patent application for sand drains in 1925 (filed August 5, 1925) which was granted in 1926 (patented August 31, 1926, Patent No. 1,598,300). This patent has been used solely for the protection of the engineering profession. Moran apparently clearly understood the mechanics involved for his patent claimed " . . . the method of strengthening a body of earth which consists in forming drains at numerous points in the area of the mass and compacting the material laterally also at numerous points to force the water out of it by way of such drains."

The first application of Moran's invention to highway fill foundations was proposed by him as a means of stabilizing the mud foundation beneath the easterly roadway approach to the San Francisco - Oakland Bay Bridge. This proposal led to laboratory and field experiments with sand drains by the California Department of Highways in 1933 and 1934, which were described by O.J. Porter in a paper published in the First International Conference on Soil Mechanics and Foundation Engineering in 1936 (1). The success of the laboratory experiments and field test sections led to further use and development of the method, first by the California State Highway Department, and then, in the eastern United States by the Corps of Engineers in 1940 - 1942, and by many state highway departments since that time. The number of sand drain installations which have been made now total about 100 and installations have been made in 16 different states and in several foreign countries. Sand drains have been used for the stabilization of weak and compressible foundation soils beneath earth fills, primarily highways and airfield fills; beneath warehouse floors; within cellular cofferdams; and in earth dams.

The use of sand drains covers a period during which theoretical design procedures were not initially available and in which the design procedures changed from largely an empirical approach to one based upon theoretical concepts as well as past experiences. An equally substantial advancement occurred simultaneously in field installation techniques and equipment. While a large number of records have been published in which sand drains have been successful some installations have not been satisfactory, the number of the latter has not been known and causes of failure have not been analyzed. In order to determine the usefulness and possible limitations of vertical sand drains for the stabilization of soils, the Bureau of Yards and Docks, Department of the Navy, decided to undertake a review to determine what is known and what is not known about the use of sand drains. The results of this review, which has been conducted by the firm of Moran, Proctor, Mueser and Rutledge, will be discussed briefly to the extent that it is complete. The review has included a thorough examination of available theoretical design methods and of experiences in the use of vertical sand drains.

DESIGN METHODS

Design of sand drains refers primarily to determination of rate of consolidation of soft and compressible soils in which sand drains have been installed and which subsequently are loaded by the weight of fill. Secondarily, the shear strength of the soft soil, the rate of gain of shear strength with consolidation and the over-all stability of the fill during its stages of placement are a vital part of design. In the primary design, factors to be determined for a soil profile determined by borings include: diameter and spacing of sand drains, thickness of drainage blanket, rate of placement of fill, amount and duration of surcharge fill loading, amounts of settlement to be anticipated during and subsequent to construction period, and values of settlements or pore water pressures to be used for control of construction operations. In the secondary but equally important part of the design, stability analyses for all stages of the construction operations are required because construction controls should, in most cases, be based on maintenance of stability.

The theory for the primary design of sand drains is based upon an extension of Terzaghi's basic work on the consolidation of clay soils and was largely developed by R.A. Barron during 1940-42. Prior to his work, Rendulic, under Terzaghi's direction formulated and solved the differential equation for consolidation by radial flow to a well in 1935. Carrillo worked on the same problem about the same time as Barron and published his results in March 1942. Barron's work, which was the most extensive, was done independently of the work of Rendulic and Carrillo and was presented in complete form in the 1948 Transactions of the ASCE (2). This paper constitutes a basic reference on the theory of vertical sand drains. In order to determine if the theoretical design procedures are sound, F.E. Richart of the University of Florida made a detailed review for our office of the mathematical theory of consolidation of soils for both vertical and radial drainage conditions. This review found that the mathematical solutions are correct and that the reliability of sand drain design analyses is limited primarily by the closeness with which the assumptions made as to soil behavior agree with the actual properties of the soil. These assumptions and uncertainties include those of Terzaghi's consolidation theory and others such as:

1. The effect of disturbance of the soil caused by installation of the drains on the coefficient of consolidation. This disturbance includes a smearing action of the soil at the surface of the drain plus a disturbance of the soil to some unknown distance from the drain, both of which affect the coefficient of consolidation.

2. The effect of more rapid consolidation, and hence settlement, of soil near the sand drain in causing arching of the overlying foundation soil and fill.

Barron evaluated the effect of a smeared zone of finite thickness having a reduced permeability adjacent to the drain. During the course of our review, Richart developed charts which simplify the use of Barron's analysis in computing the effect of a smeared zone on the rate of consolidation and which show that the effect of smear is to reduce the effective diameter of the sand drain. These charts will be included in a final report to be published by the Bureau of Yards and Docks, Department of the Navy. The assumptions for the thickness of the smeared layer and the coefficient of permeability in the affected zone are still matters of judgment, for our review did not disclose that any satisfactory field data have been developed for evaluating the effects of smear caused by driving of sand drains. Pile driving observations show that serious remolding occurs for a distance of one-half to one diameter outside of a pile. It is, therefore, necessary for each installation to be considered individually, on the basis of judgment and possibly with the benefit of consolidation tests performed on undisturbed and remolded samples, to estimate the effects of smear and disturbance.

Barron also obtained theoretical solutions for two limiting cases: one of no arching, a "free-strain" case; and a second case where arching occurs and redistributes the fill and foundation loads to result in an equal settlement, an "equal-strain" case. Fortunately, the solutions to these two limiting cases do not differ substantially when used for sand drain design purposes with the exception of evaluating piezometer observations.

One of the assumptions made in deriving the theory of consolidation and in the theoretical work by Barron is that the voids of the soil are completely filled with water. Practically, however, many soils in which sand drains are installed contain or evolve gas. It has been observed that open-ended piezometers installed in organic soils have frequently discharged gas which could be ignited and would burn for sometime. Our investigation has developed analyses to indicate the effect of gas on the consolidation process. If the load on a sample of soil containing gas is suddenly increased the gas will compress practically instantly and the sample will, in effect, be partially consolidated under the increment of load even though there has been no drainage of water. As time proceeds, water drains from the soil with the result that part of the load carried by the water is transferred to the soil grains and the hydrostatic pressure decreases. The volume of gas simultaneously increases with the result that the observed settlement is less than the settlement corresponding to the volume of water which is drained during any interval of time. The over-all effect of gas in the soil is to increase the coefficient of consolidation during the period when the loads are being increased and to decrease it after all loads have been applied. The results of analyses of the effect of gas on the rate of consolidation are too involved to include in detail but demonstrate that the effect of gas can be substantial and should be given consideration in evaluating the results of laboratory tests and in analyzing piezometer observations. The effect of gas in the soil has another effect also, which is to reduce the heaving and disturbance caused by a close spacing of sand drains.

SOIL PROPERTIES AFFECTING SAND DRAIN DESIGN METHODS

Coefficient of Consolidation

The most significant soil properties entering into the design of sand drains are the compressibility in the vertical direction and the permeability in vertical and horizontal directions. The coefficient of consolidation for drainage in the horizontal direction, defined as

$$c_v - r = \frac{k_r \; (1 + e)}{a_v \qquad w}$$

governs the consolidation process and is the most important single soil property in design, but unfortunately it is not readily determinable. Laboratory consolidation tests of the usual type determine the coefficient of consolidation for vertical instead of horizontal drainage. If a consolidation test is performed on a sample which is rotated 90 deg., the direction of drainage corresponds to prototype conditions but the compressibility is determined for differently oriented soil particles and the results may not be comparable to the field behavior of the soil. Normally performed consolidation test results can be corrected to furnish a value for horizontal drainage by: (a) performing permeability tests in the laboratory or in the field for both horizontal and vertical drainage directions; or (b) assuming a ratio between the horizontal and the vertical permeability on the basis of experience and inspection of the soil samples. Field methods of determining the coefficient of permeability in vertical and radial directions are receiving considerable attention and may prove to be reliable if done carefully. Neither method by itself is particularly satisfactory. In many cases the ratio of the horizontal to the vertical permeability has been assumed to be higher than it really was. This ratio is greatly affected by the presence of even thin layers of silt or sand which can be found only if continuous sample borings are made, however, there is a possibility that smear caused by installation of the sand drains can nullify the effect of thin pervious layers. Promising work is in progress at Northwestern University on a new type of consolidation test apparatus which provides directly the coefficient of consolidation for horizontal drainage with vertical settlement and it is hoped that it will soon be practicable to use it in consolidation testing for design of sand drains.

Disturbance

The effect of disturbance of the soil on its consolidation properties is to lower the coefficient of consolidation. The effect decreases with increasing water contents and increasing loads for organic silts and clays with water contents between 37 percent and 98 percent. For these materials and loads at the preconsolidation stress the coefficient of consolidation in the undisturbed state was from 2 to 24 times as great as in a remolded state. The ratios decreased to between 2 and 7 for loads equal to one ton per sq ft above the preconsolidation stress. The variability of the coefficient of consolidation with load, and with disturbance, makes it necessary to use conservative values in sand drain designs. The assumption made in the theoretical analyses that the coefficient of consolidation is constant is satisfactory but approximate. This soil property decreases greatly for loads in the vicinity of the preconsolidation stress but is reasonably constant for greater loads.

Secondary Compression

Many, but not all, soils in which sand drains are installed exhibit large secondary compressions which are not directly related to excess pore water pressures and hence are not accounted for in the Terzaghi theory of consolidation nor in its adaptation by Barron to the design of sand drains. These compressions or settlements occur simultaneously with primary consolidation but continue after primary consolidation is complete. The relative importance of secondary compression is greater on sand drain installations than on most applications of the theory of consolidation. This is partly due to the higher amounts of secondary compressions usually exhibited by soils in which sand drains are installed. An important reason, however, is that the time of primary consolidation is greatly reduced, with the result that the amount and relative importance of secondary compression following completion of primary consolidation is greatly increased. The general concept of secondary settlements is that they are the result of a plastic time lag or plastic resistance to compression but relatively little is known about this phenomenon. Tests have shown that the rate of secondary compression is proportional to the logarithm of time and the amount is directly proportional to the thickness of the compressible layer. The amount of secondary compression is often as high as, or higher than, 0.03 ft per ft thickness of layer per cycle of time. Thus, for a ten year period immediately following a normal construction program, the secondary compression for a fifty foot compressible stratum would be 1.5 ft if surcharge fills were not used or were not maintained long enough. Of this amount, 0.45 ft would occur during the first year after construction and 1.0 ft during the first five years. The second ten year period would, however, show a secondary compression of only 0.45 ft and the third ten year period a settlement of 0.27 ft. These figures illustrate that secondary compressions can be of practical importance for some soils and that repaving may be required fairly soon after construction if surcharge fills are not used.

It is believed on the basis of both field, laboratory and theoretical considerations, that secondary compressions can be largely and possibly entirely eliminated by preloading fills provided that the surcharge load is maintained for a long enough time to reach consolidation equivalent to ultimate under a load which is greater than the final load remaining after the surcharge has been removed. The degree of preconsolidation required to eliminate future secondary compressions can only be estimated approximately at this time.

Stratification

The details of stratification are difficult to determine but have a profound effect on the rate of consolidation. Continuously sampled borings in representative locations are necessary to define stratification. Soils may be stratified in many ways but the effect is similar though the degree of influence may differ greatly. If a soil contains layers of more pervious material, the effect will be to accelerate the component of the rate of settlement which is due to vertical drainage. The effect of more pervious strata depends upon their permeability, spacing, continuity and thickness. If a sand layer is thin, installation of the drains may smear the sand, separating it from the drain and thereby greatly reducing the efficiency of the sand layer in accelerating horizontal consolidation. If a soil is highly stratified, sand drains may be unnecessary, or, if drains are used, may make an accurate estimate of their effect difficult or impossible.

SUMMARY RE DESIGN

The dependence of sand drain design methods upon the Terzaghi theory of consolidation makes it necessary to inquire into the validity and applicability of the theory, especially because of doubts which have been expressed concerning the physical concept that loading causes hydrostatic excess pressures which result in drainage and settlement with time. Our review found that: (a) the mathematical solutions are correct for the basic assumptions made, (b) the physical concepts involved have been recognized and understood by some practical engineers before the mathematical theory of consolidation was formulated, and (c) the applicability of the theory of consolidation to practical work has been verified. The conclusion is reached, therefore, that the theory of consolidation, and sand drain design procedures, are applicable but their accuracy is limited by the degree of agreement between assumed and actual soil properties.

The influence of actual soil properties leads to two important limitations in the use of vertical sand drains and of theoretical methods of design. The first is that sand drains are effective only in accelerating settlement due to primary consolidation and that current design methods apply only to such primary consolidation. Settlements due to secondary compressions are essentially independent of whether or not sand drains are used. The second limitation is that some soils have such an extremely low coefficient of consolidation that sand drains at customary or economical spacings cannot effect a significant amount of consolidation in the short time generally available for construction. Additional limitations involving the sensitivity of a soil to disturbance may exist but cannot be formulated at this time.

The practical significance of the first limitation is that in soils exhibiting large secondary compressions, sand drains will probably not be effective in eliminating future settlements because they are effective only in accelerating primary consolidation. However, if the soil profile also includes very soft soil with a large primary consolidation, sand drains may be effective in increasing the rate of gain of shear strengths and providing stability under loads otherwise not possible. This limitation in the use of sand drains can be made evident from the results of laboratory consolidation tests and design analyses. If soils exhibit mainly secondary compressions with rapid primary consolidation, sand drains should be considered only for increase in shear strengths. Surcharge loading fills may be applicable, if maintained long enough to compensate for secondary compressions under the weight of the final fill. No other specific statement of the applicability of sand drains to particular conditions is needed because any advantages of using sand drains become evident if tests and analyses are made.

The practical significance of the second limitation in the use of sand drains is that: (a) a complete laboratory testing program is necessary to recognize the highly impervious and slow consolidating soils, and (b) whether or not this limitation applies is dependent upon the particular case involved and especially the time available for surcharge loading. In many highly plastic and impervious soils a three or six month loading period is not long enough to accomplish significant consolidation with any practical spacing of drains and magnitude of surcharge loading.

EXPERIENCE WITH SAND DRAINS

Installations Made

In studying experiences with sand drains the records of over 100 sand drain installations have been reviewed and the field performance data from 25 installations have been analyzed in detail to determine past experiences and especially to uncover cases where troubles have been encountered. The greatest single application of sand drains has been to stabilize foundations beneath highway fills. The drain diameters and spacings most generally used are summarized below:

Item	Range	Most Frequent Values
Diameter of sand drains	6 in. to 30 in.	75 percent between 18 in. and 20 in.
Drain spacing:		
(a) "n" values[a]	4 to 42	35 percent between 4 and 6 75 percent are less than 9
(b) feet	6 ft to 20 ft	30 percent between 6 ft and 8 ft 75 percent between 6 ft and 10 ft

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[a] $n = \frac{d_e}{d_w}$

The types of difficulties experienced on sand drain installations can generally be grouped as follows:

1. Shear slides during construction.
2. Slow rate of consolidation.
3. Excessive settlements following construction.

The above types of troubles have been experienced in almost every area where sand drains have been installed, but to widely varying degrees. Over a dozen installations have experienced major troubles of one type or another. In some cases the troubles were so serious that the sand drains did not perform any useful function. In other cases the troubles were corrected during construction with satisfactory results. Of particular interest and importance are the experiences at some installations where shear failures took place and where the sand drain system was redesigned, new drains installed, and the fill completed without difficulty.

Shear Slides

The most serious single type of trouble on sand drain installations has been with slides during construction. These slides, which extend through the weak soil in which the drains are installed, have occurred at practically every stage of construction. Slides have occurred when the sand blanket was being placed, during placement of a working mat, as the fill was being placed, and when the fill was almost completed. In some cases the slides were arrested by berms and by halting construction before any apparent damage to the sand drains had taken place. In other cases the slides sheared the drains which became practically ineffective. In several cases very large mud waves developed with heights in the range of half the fill height to one extreme which was even higher than the fill. The damage to the drains by slides was sufficient in several cases to require installation of new drains in the area affected.

The histories of sand drain installations reveal one especially significant fact. Serious slides have occurred only on projects where no stability analyses were made prior to construction. In all cases where adequate stability analyses had been made no slides occurred or they were local and easily corrected or their possibility had been anticipated and control measures established. Minor slides at the beginning of work on several jobs were used as a basis for correcting the assumed shear strength used in stability analyses and the remainder of the work was completed without slides.

There has unfortunately developed an apparent belief among some engineers that sand drains automatically increase the strength of a weak soil at a fast enough rate so that shear failures of the foundation cannot occur. It cannot be sufficiently stressed that the design of a sand drain installation where the stability of the foundation is critical, the usual case, requires not only careful design of the sand drains themselves but also complete stability analyses of the fill and foundation and studies of how the contractor can and should place the sand blanket, install the drains, and place the fill. One result of our review is the conclusion that, while many shear slides have occurred on sand drain installations, the stability analysis methods developed by soil mechanics are satisfactory in preventing this type of trouble.

Slow Consolidation

Several installations consolidated so slowly that the rate of fill placement had to be decreased and the surcharge fill could not be maintained for the length of time anticipated. In reviewing these records we usually found that the spacing of the sand drains and the probable rate of consolidation had been determined from either very crude approximations or the results of previous installations with no use made of the thoretical design procedures developed by Barron. In some cases the design charts in Terzaghi's "Theoretical Soil Mechanics" were used and the engineer was unaware that they had been superseded by Terzaghi's revised charts in an article in Civil Engineerin in October 1945. In other cases the results of using sand drains were predicted before the results of laboratory tests were available and the predictions were not revised. Review of the data available from one unsuccessful installation, designed under severe time restrictions, showed that shear slides should have been anticipated, that consolidation would proceed very slowly and that the scheduled surcharge loading period was so short that little consolidation could result. We have concluded from our review that existing laboratory consolidation tests, despite their shortcomings when applied to the design of sand drains, are still capable of predicting the general rate of consolidation under field conditions. Failure to make carefully performed tests on good undisturbed samples and to use the theoretical design procedures available are the main causes for disappointments in the rate of consolidation.

Excessive Post-Construction Settlements

The settlements following construction have been sufficient at a number of installations to require repaving of highways once or twice within the first ten years of use. The settlements have been so rapid on some jobs that repaving was necessary within two or three years after construction was complete. In reviewing the records and soil information available, it became apparent that the reasons were either that the primary settlement of the foundation was not complete at the end of construction or that high secondary compressions were occurring. When the primary settlement was found to be incomplete at the end of construction, it was apparent from the properties of the soils that this result could have been expected for the time permitted for consolidation. Cases where the field rate of primary settlement was sharply lower than would be expected from laboratory consolidation tests were not found.

Settlements due to secondary compressions are dependent upon whether or not the soil was preconsolidated by surcharge fills to loads in excess of the final fill loads. It was found that settlements due to secondary compressions were high when no surcharge was used and that they decreased as the ratio of the surcharge to the final fill loads increased. At one sand drain installation where no surcharge fill was used, and where the soil conditions consisted of 5 ft of fibrous organic matter and organic silt overlying 20 to 25 ft of soft dark gray clayey silt, primary settlements were complete by the end of construction but large secondary settlements were experienced. Within two years after construction the roadway had to be repaved. This was repeated four years later by which time the maximum secondary settlements had reached nearly one foot.

On another installation a five to seven foot surcharge fill was left in place for over a year and practically complete primary consolidation under it was obtained. When the surcharge was removed, a rebound of 0.1 ft to 0.2 ft occurred but, nevertheless, later settlements occurred following paving which are attributed to secondary compressions. The amount of settlement in a three year period was as high as 0.25 ft and decreased with increasing amount of load removed. This case illustrates the secondary settlements that may occur even with an apparently generous surcharge loading period. While moderate uniform settlements following construction can generally be tolerated, surcharge fills appear necessary to keep secondary settlements to a minimum near bridge abutments and at transitions to hard ground. In a few cases where comparisons of field and laboratory secondary compressions were possible, it was found that the agreement was relatively good. On this basis it is believed that the individual loads in the laboratory consolidation tests should be maintained long enough to define the slope of the secondary compressions and that these data should be used as a guide in estimating probable field secondary compressions. Estimation of the probable amount of secondary compression should be regarded as part of the design procedure although this has not generally been done. Considerable field and laboratory research is needed, however, on the phenomenon of secondary compressions.

Construction Control

The review of cases where troubles have been experienced showed that both design and practical considerations are important and that neither can be slighted. It is especially important that the specifications provide adequate controls for such items as:

1. The normal maximum allowable rate of fill placement.
2. Varying the rate or temporarily stopping fill placement to permit dissipation of

temporarily high hydrostatic pressures.

3. The permissible lift thickness of fill and maximum end and side slopes during placement.
4. Disposition of surcharge fill.
5. Placement methods for sand blanket and working mat, need for mats or casting of material into position for sand blanket and working platform.
6. Control measures such as piezometers, settlement plates and side stakes.
7. Gradation of sand drain fill, sand blanket and drainage windrows, if required.

In several cases where troubles were experienced the specifications were found to have been violated or adequate controls for construction operations were not provided. Proper inspection during construction is probably more essential on sand drain work than on almost any similar construction activity.

Technical Control

Settlement plates, piezometers and side stakes to determine lateral and vertical movements have been used with success in controlling field operations and in checking the field behavior against results of design analyses. This finding is, however, only partially true with respect to the behavior of piezometers. The piezometers commonly used are of either the closed system type with a Bourdon gage or are of the open Casagrande type utilizing a small diameter ($^{3}/_{8}$ in.) standpipe. The closed system type of which there are several variations, has been the most common.

In a large number of cases the piezometers failed to drop as fast as they should have for the period following the application of all loads. In many cases the piezometer readings remained stationary or actually increased. The causes are not known but it is believed likely that accumulation of gas in and around the piezometer point and riser pipes is responsible. We have not found any cases where the piezometers have been tested to determine their basic time lag and response in the manner recommended by Hvorslev (5). These simple tests reveal the presence of gas in the piezometer system and are recommended for every piezometer installation. In connection with the use of piezometers for determining excess hydrostatic pressures, it should be remembered that the placement of a fill occupying a fairly large area generally raises the normal or static ground water level beneath the fill.

Emergency Corrective Measures

Sand drains are often used under exceptionally difficult soil conditions and despite careful and complete investigations and designs, troubles may result because of variations in the soil profile or soil properties not revealed by the investigational program or because of an effort on the part of the engineer to use the minimum possible factor of safety. In addition, the need for the completed work may require an accelerated construction schedule, or the contractor may have fallen behind in his work. If the above conditions develop, emergency measures of one type or another may be required, and our review has shown that many have been used.

If slides develop during construction, placement of fill should be immediately stopped and stability analyses made to determine the shear strength of the soil at failure. Borings to determine unexpected changes in the subsoil profile may also be required. With this information and revised stability analyses, a decision can be made as to the need for local or general corrective measures. These measures may consist of:

1. A decreased rate of loading.
2. Berms
3. Additional sand drains.
4. Lowering the ground water level in some drains by installing wellpoints.
5. Lowering the hydrostatic pressure in an underlying more pervious stratum, if one exists.
6. Use of vacuum type wellpoints in the sand drains.
7. Accelerating drainage by electro-osmosis.

If a need develops during construction to accelerate the rate of consolidation, this may be done by adding sand drains or by increasing the rate of fill placement and the amount of surcharge to the limit permitted by stability considerations. The permissible rate of filling and amount of surcharge can be increased by adding berms. A surcharge can also be effectively added by pumping from the drains as in methods 4 to 7. Thus, methods exist for improving the behavior of a sand drain installation after it is in operation, if the need arises. The sooner these methods are used the more satisfactory will be the results.

SUMMARY

The review of installations with unsatisfactory performance records showed that the reasons were, in general:

1. Failure to make complete stability analyses.
2. Improper design of the sand drains including determination of anticipated rates of consolidation.
3. Lack of control requirements in the specifications on the contractor's operations.
4. Violation of the drawings and specifications by the contractor, or lax or inadequate inspection.

Of the above causes, a failure to make stability analyses was the most common reason for failures. In a few cases reviewed the limited data available did not permit the conclusion to be drawn that sound design or construction procedures had been violated nor that sand drains did not behave according to the theory of consolidation. Thus, while it cannot be stated that the drainage of water from soil can always be facilitated by using sand drains, it can be stated that for the cases which were reviewed, there is not one instance where sand drains were properly and completely designed, installed and inspected and still failed to effect an increased rate of consolidation.

The results of the review, both of the theoretical aspects and of actual sand drain installations, forcibly demonstrates the need for a thorough initial design and that layout of sand drains on a purely empirical basis is not satisfactory. The initial design should be based on an adequate number of continuously sampled borings, good undisturbed samples, and carefully performed strength and consolidation tests on representative undisturbed samples.

The use of sand drains involves many minor uncertainties which need to be resolved on the basis of full-scale field investigations. These include such items as:

1. Smear
2. Disturbance
3. Permissible sizes and spacings of drains installed by driven closed end mandrels
4. Secondary compression

Item 4 above actually represents a major research study requiring intensive theoretical, laboratory and full-scale field investigations. In the meantime collection and analyses of data from field installations will provide guides for practical design and invaluable basic data for the intensive research.

ACKNOWLEDGMENT

The project summarized in this paper has been carried out by the firm of Moran, Proctor, Mueser and Rutledge, Consulting Engineers, New York, under contract with the Bureau of Yards and Docks, Department of the Navy. The officer in charge of this contract has been Captain A.S. Klay (CEC) USN and the supervising engineer has been L.A. Palmer of the Bureau of Yards and Docks. For the engineers Philip C. Rutledge has been supervising partner and Stanley J. Johnson has been the engineer in charge of the investigation. Theoretical analyses have been made by F.E. Richart, Jr., Department of Civil Engineering, University of Florida, as a part time employee of the engineering firm and as a consultant.

Special acknowledgment is made of the excellent cooperation received from all state highway departments contacted and from several consulting engineers and toll road authorities. The information so received has made possible the review of actual experiences in the use of sand drains.

REFERENCES

1. Porter, O.J., "Studies of Fill Construction Over Mud Flats Including a Description of Experimental Construction Using Vertical Sand Drains to Hasten Stabilization," Proc., First International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 229-235, Harvard University, Cambridge, Mass., 1936.
2. Barron, R.A., "Consolidation of Fine-Grained Soils by Drain Wells," Transactions, ASCE, Vol. 113, pp. 718, 1948.
3. Housel, W.S., "Checking Up on Vertical Sand Drains," Highway Research Board Bulletin 90, 1945.
4. Gould, J.P., "Analysis of Pore Pressure and Settlement Observations at Logan International Airport," Harvard Soil Mechanics Series No.34, Harvard University, Dec.1949.
5. Hvorslev, M. Juul, "Time Lag and Soil Permeability in Ground-Water Observations," Bulletin 36, Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.
6. Taylor, D.W., "Research on Consolidation of Clays," Publication No. 82, Department of Civil and Sanitary Engineering, M.I.T., August, 1942.
7. Thompson, J.B. and Palmer, L.A., "Report of Consolidation Tests with Peat," A.S.T.M. Spec. Publication No. 126, Symposium on Consolidation Testing of Soils.

Discussion

W.S. HOUSEL, Professor of Civil Engineering, University of Michigan, and Research Consultant, Michigan State Highway Department—This paper has been awaited with a great deal of interest ever since it became known that such a comprehensive study was being made for the Bureau of Yards and Docks of the Department of the Navy. The writer desires to comment on several phases of the subject and compliment the authors on their unusually thorough and objective review of a somewhat controversial subject. It should be noted that their final report will be published by the Navy and will include the detailed data which have necessarily been summarized in the present paper.

The authors' frank recognition of the importance of mass stability is particularly welcome and the following statements under the heading of "Shear Slides" cannot be overemphasized:

> The most serious single type of trouble on sand drain installations has been with slides during construction. These slides which extend through the weak soil in which drains are installed have occurred at practically every stage of construction. . . when the sand blanket was being placed, during placement of the working mat, as the fill was being placed, and when the fill was almost completed.
>
> It cannot be sufficiently stressed that the design of sand drain installations where the stability of the foundation is critical, the usual case, requires not only careful design of the sand drains themselves but also complete stability analyses of its fill and foundation and studies of how the contractor can and should place the sand blanket, install the drains, and place the fill. One result of our review is the conclusion that, while many shear slides have occurred on sand drain installations, the stability analysis methods developed by soil mechanics are satisfactory in preventing this trouble.

While heartily endorsing this positive statement of an important basic problem, one cannot help but be somewhat amused by the authors' reluctance to call the underlying phenomenon by its real name, shearing displacement or plastic flow, instead of "secondary compression." For many years the writer has been objecting to the studied efforts of the Terzaghi school of soil mechanics to relegate the basic phenomenon of plastic flow to the role of secondary compression and to pretend that this was one of the unsolved mysteries of soil mechanics. It seems that this paper may present the opportunity to bring the difference in basic concepts to some sort of a climax that may clarify the issues involved.

As the writer sees the situation, the difference originates in the basic idea of the theory of consolidation, which pictures the soil-water system as a two-phase system in which water and soil solids act as separate entities. In this concept Terzaghi first defined and has never changed the view that cohesion was the product of internal pressure created by the surface tension of adsorbed water, with shearing resistance a function of internal friction (1) (2). In discussing this question, it may be well to outline several past discussions that are significant in illustrating the conflict of ideas.

This concept of the function of surface tension led Casagrande in 1932 to the view that a saturated clay, submerged as it generally is below the surface water table, would be reduced to a suspension of solid particles in water without any static resistance to displacement. He voiced this conclusion as follows (3):

> I have tried to illustrate that the whole problem of building foundations on clay boils down to these two simple principles: first, do not disturb the natural structure of the clay: if you do, no human being is able to restore its original strength; second, decide on a certain rate of settlement which you do not wish to exceed and determine that pressure which will cause that rate of settlement; the difference between the building load and the above pressure is the weight of soil which must be removed before erecting the building.
>
> A definite bearing value of clay does not exist . . . . The engineer must learn that the kind of questions he asks an expert regarding the properties of a clay underground should not be, "How much load may I put on this soil?" Or, in an apparently more scientific manner, "What is the bearing capacity or bearing value of this clay?" His question should be, "How must I design my foundation so that the rate of settlement under a given building load will not exceed certain limits?"

The writer took issue with this denial of a definite bearing capacity which eliminates static shearing resistance due to cohesion and declared that a definite bearing power of clay does exist (4). In support of this declaration the results of several series of plate loading tests were presented, with subsequent settlement measurements on full-size structures which were successfully designed for a limited settlement on the basis of the loading tests.

The last time the writer heard the statement that plastic clay had no shearing resistance was in another discussion by Casagrande at the Purdue Soil Mechanics Conference in 1937. This discussion was never published, although it would have been useful to document changing views on soil mechanics. Complete recognition of the shearing resistance of cohesive clays has long since ceased to be a matter of debate. All soil mechanics laboratories conduct shearing resistance tests of one kind or another and use the results to compute bearing capacity by some one of a dozen or more available formulas. It seems rather odd, however, that shearing displacement, the Siamese twin of shearing resistance, must still be called "secondary compression." Furthermore, it should be noted that although the view that soil has no static shearing resistance has gone into oblivion, the basic concept of soil as a two-phase system is still with us accompanied by other complexities, some old and some new.

The next episode in the story took place at the International Conference on Soil Mechanics and Foundation Engineering at Harvard University in 1936. It was at that conference that Terzaghi began laying the groundwork for an explanation of a continuing rate of settlement at a substantially uniform rate as a secondary time effect or secondary consolidation. The writer's views were expressed at that time in a discussion, from which the following abstracts have been taken:

> Next, I wish to comment on the subject of the settlement of structures and certain aspects of continued settlement. According to my observations of time-settlement relations, there appear to be two basic phenomena which may be represented by time-settlement curves.

> In the first place there is consolidation of the soil due to volume changes which represent the compression of void spaces in the soil structure. In porous soils this part of the settlement may be relatively large, but in well-consolidated materials it may be relatively small. This consolidation will take place over a period of time, which may be four or five years in a large mat foundation, two or three months in the case of a pier footing, or considerably less than an hour for a smaller test area. In clay soils with water filled voids and a relatively high degree of impermeability, I have yet to encounter conclusive evidence that the migration of water through the soil due to applied pressures within yield value of the soil under plastic flow is of more than negligible importance.
>
> After the period of consolidation one of two situations may arise. For a certain intensity of pressure one may say that the consolidation has been complete, the pressure being less than the yield value there is no continued or progressive settlement and the settlement curve approaches a horizontal asymptote. For a higher intensity of pressure the consolidation is also complete but the load is greater than the yield value of the soil and settlement continues. It appears, as mentioned by Dr. Terzaghi earlier in the discussion, that such settlement continues at a uniform rate and the settlement curve approaches a sloping asymptote.
>
> I cannot see, however, anything about this situation new or awaiting explanation by investigators of soil mechanics. This is entirely in accord with the conceptions of plastic substance, outlined, I believe, by James Clerk Maxwell approximately in the middle of the last century. It is not at all surprising that plastic clays follow the laws of plastic flow which are quite well known, in fact it would be surprising if they didn't.
>
> According to these principles, Bingham, Nadai, and others, define a plastic material as a substance which will sustain a certain shearing stress without movement but at a higher stress will be deformed gradually without rupture, the rate of deformation being directly proportional to the stress in excess of the field value.
>
> The determination of yield value in my opinion is the most important factor which practical foundation engineering has to consider. Incidentally this point bears on a question put to the Conference which, so far as I am aware, has not been definitely answered. The yield value according to definition as applied to cohesive soils is the shearing resistance at zero normal pressure assuming, of course, that no dynamic effects are introduced due to rapid load application.
>
> * * *
>
> These examples are not all, but many investigators have uncovered similar evidence. Thus, in addition to consolidation we have with us plastic flow of plastic soils if that be strange.

In the past twenty years there are many times when the subject of plastic flow versus consolidation has been discussed without any significant change in opposing viewpoints. There have also been many occasions within the writer's recollection when failure to look this problem full in the face and recognize shear failures for what they were, has led practicing engineers astray.

It is the overemphasis on consolidation to the exclusion of shearing displacement that has led to a number of notable failures of the consolidation theory to provide reliable control as in the case of the Norfolk Naval Air Station (5). While it may be over-simplification, the writer has attempted to summarize his objection to the theory by the statement that the water cannot be squeezed out of clay without shearing displacement when that moisture is held by the same molecular forces that are the source of shearing resistance due to cohesion.

Although failure to recognize shearing displacement has resulted in many failures it may be that renewed interest in the subject, via the current investigation of sand drains, may succeed in clarifying the relation between consolidation and shearing displacement where other efforts have failed.

One new idea that seems to have appeared in the present paper has to do with the effect of gas on sand drain installations. As stated by the authors the theory of consolidation has always been limited to saturated soils, where it is assumed that the voids are filled with water. In reading the comments on this point, one wonders if the introduction of gas into the discussion is a first step in the attempt to extend the theory of consolidation to unsaturated soils, with further complications in theoretical soil mechanics.

REFERENCES

1. Terzaghi, K., "Compressive Strength of Clay," Engineering News-Record, Vol. 95, p. 796, Nov. 12, 1925.
2. Housel, W.S., "Shearing Resistance of Soil — Its Measurement or Practical Significance," Proceedings, ASTM, Vol. 39, 1939.
3. Casagrande, A., "The Structure of Clay in Foundation Engineering," Jour. Boston Soc. Civil Eng., Vol. 19, No. 4, April 1932.
4. Housel, W.S., "Bearing Power of Clay is Determinable," Eng. News-Rec., Feb. 23, 1933.
5. Housel, W.S., "Checking up on Vertical Sand Drains," Highway Research Board Bulletin 90, 1945.

P. C. RUTLEDGE and S. J. JOHNSON, Closure—Professor Housel's discussion is welcomed because it calls attention to basic differences in the understanding of soil mechanics terms.

Professor Housel is either indulging in a play on words or taking advantage of the differences in understanding of terms when he implies that the authors, and what he calls "the Terzaghi school of soil mechanics", ignore shear strengths and plastic deformations in clays. Life for soil mechanics engineers would indeed be simpler if secondary compression or consolidation could be dismissed simply as a plastic deformation which occurs only at stresses above some determinable yield value as indicated by Housel.

Perhaps the basic difference in viewpoints is in separation of phenomena. The authors, and many of their colleagues in soil mechanics, prefer to separate the phenomena of volume change in soil, which is consolidation, from those of shear deformation and shear strength which do not necessarily involve volume change and which, in clays, do take place without volume change. To clarify these two groups of phenomena, and some soil mechanics terminology, the following definitions are offered:

Primary consolidation is decrease in volume of a soil through decrease in volume of its pore spaces, accompanied by a compression or squeezing out of pore fluid, whether gas or liquid or both. Primary consolidation is independent of shear stresses and occurs under conditions of equal stress in all direction when shear stresses and deformation are zero, although the more common case of one-dimensional consolidation in laboratory tests and in nature does involve some shear stresses and deformations.

Secondary compression is a volume change phenomenon which continues after completion of primary consolidation and is characterized by a straight-line relation between volume change and logarithm of time. The proportionate magnitude of secondary consolidation in comparison with the primary, by which it is invariably preceded, varies with soil type, state of stress and temperature, being a maximum for highly organic soils. However, secondary compression continues after primary consolidation at all magnitudes of stress, with the proportion between the two being approximately independent of magnitude of stress. Secondary compression has been attributed by some research investigation to a plastic readjustment of stress between soil grains and theories have been developed on this basis. However, no theory put forward to date explains adequately all of the physical phenomena, such as effects of temperature, observed in secondary compression.

Shear deformation is the phenomenon of change in shape under the action of stress and invariably requires unequal stresses in coordinate directions; in other words, the presence of shear stresses. This type of deformation can occur without volume change, and in clay soils does. Since shear deformations do not require volume change, which in saturated clays can only proceed at a slow rate, they frequently precede consolidation. In other words, stability or freedom from large shear deformations is a primary requisite in successful foundations and earthworks, including sand drain installations, but it does not preclude subsequent settlements due to consolidation. An example is an earth fill of uniform thickness but of large areal extent completely covering a deposit of soft or compressible soils. Once such a fill is in place no significant shear stresses are created in the soft soil, but settlements of large magnitude can take place due to consolidation. This is the basic concept in the Casagrande paper of 1932, with which Housel takes issue by selective quotation out of context.

Plastic deformation is a restricted part of shear deformation defined in two different ways. In classical mechanics, plastic deformation is that part of change in shape which is not completely recovered upon release of stress. In more sophisticated mechanics it is shear deformation, which occurs gradually under constant stress and hence is a time phenomenon. By the latter definition some materials exhibit plastic deformation under all magnitudes of stress, whereas others deform plastically only when some stress, called the yield value, has been exceeded. It must be emphasized that plastic deformation requires shear stress, but does not require volume change. In fact, in most metals plastic deformation takes place at constant volume.

These definitions make it self-evident that secondary compression, shear deformation, and plastic deformation are not different names for the same physical phenomenon, as suggested by Housel, but are separate and distinct behavior characteristics which must be treated separately if a clear understanding of the behavior of soils is to result.

Professor Housel's quotation and discussion of Casagrande's 1932 paper on "The Structure of Clay in Foundation Engineering" appears to miss completely the point of the quoted material. Two important conclusions that are exemplified by Casagrande's paper are that (a) excessive primary consolidation can occur without exceeding the bearing capacity of the soil, and (b) the bearing capacity can be exceeded before the amount of consolidation is of practical significance. Thus, Casagrande's statements do not deny the existence of a definite or limiting bearing capacity of a soil; his statements only emphasize that, before the safe bearing capacity has been reached, the amount of consolidation that will ultimately be experienced can be and generally is the factor that determines how much load should be applied to a foundation. Thus, " the Siamese twin of shearing resistance, shearing displacement" cannot and has not been called "secondary compression". Apparently Housel does not observe the important difference between the definitions previously given. Unless these differences are observed there is no common meeting ground for factual discussion of the various phenomena involved. This does not imply that secondary compression, shear deformation and plastic deformation are not all related to the existence of shear stresses in the soil. What it does say, however, is that these three behavior characteristics are not one and the same to which three different names have been applied.

Professor Housel flatly rejects the idea that consolidation of a clay can occur without shearing displacement, although he recognizes that this may be an over-simplification. This opinion is erroneous and misleading. The consolidation process should be evaluated on its basic assumptions as a simple matter of water flowing through clay due to hydraulic gradients created by applied loads, ignoring theoretical concepts of molecular forces. The applicability of the theory of consolidation to normal field conditions is dependent upon whether or not decreases in water content are actually found under field loading conditions. They have been found at so many sites that the basic applicability of consolidation theory to field loading conditions has been adequately demonstrated.

Although Housel appears to be trying to discredit the theory of consolidation, in reality he is discussing details about which there is not real disagreement, despite his implications. For example, his statement that "I have yet to encounter conclusive evidence that the migration of water through the soil due to applied pressure within yield value of the soil under plastic flow is of more than negligible importance", merely says, in effect, that for low loads, probably less than the preconsolidation load, the expected settlement and water content decreases are so small that they may be difficult to detect in the field.

Those who use the theory of consolidation do not deny—rather they endorse the concept that settlement, the preconsolidation stress, the applied load, shear deformation, secondary compression, and plastic deformation are related in over-all sense. A relationship between these items is obvious, in that maximum previous loading affects the preconsolidation load, and hence settlements, as well as the existing shear strength of the soil, which affects shear and plastic deformations. These factors are not grouped, but are considered separately by the school of soil mechanics criticized by Housel.

The authors and Professor Housel are in complete agreement on the necessity of considering both shearing displacement and consolidation theory in the design and use of vertical sand drains and for providing field construction control. The authors view this need as imperative and believe that any failure to do so, or to have done so in the past, merely demonstrates an incomplete analysis of a problem and a temporary stage in the application and correct use of soil mechanics to practical problems. As ideas and experiences are exchanged and discussed by engineers the required design procedures to meet the needs of the situation will be formulated and become more frequently employed.

Description of a Method of Predicting Fill Settlement Using Voids Ratio

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This paper contains a simplified explanation of the theory and related computations for estimating fill settlement, using the voids ratio. Included is a problem involving two layers of compressible soil. The problem is solved in a step-by-step method, with comments for each step. Necessary charts and graphs are included for predicting the settlement of a fill.

● MANY highway engineers do not fully understand the principles involved in analyzing the settlement of highway embankments. The primary factor in estimating settlement is based on the amount of water and air which is squeezed or pressed out of the compressible foundation soil by the fill load. The rate of settlement is controlled by the character of the compressible foundation soil and the distance the water and air must travel to escape. Obviously a clay soil will drain water slower than a silty or peaty type soil. Any layers of granular material in the foundation soil will result in accelerating the time of settlement, since the excess water will travel horizontally as well as vertically.

ACCURACY OF ESTIMATE

It must be stressed that computations are correct for each soil test. However, since a soil test may represent many thousands of cubic yards of non-homogenous soil, there results at best an average representation of the soil mass. For highway embankment foundations the condition of average representation by sampling is more pronounced than in the case of structure foundations where the effective mass of foundation soil is much less.

STANDARD FORM OF COMPUTATION

The following data has been selected from the publications of soils authorities. It has been my experience that a continuity of study on this subject involves many references which in turn make it difficult to follow the subject. This probably is the principle reason that many highway engineers are not eager to pursue the subject. A standard procedure generally offers a better understanding of a complex problem.

ESTIMATE OF SETTLEMENT

The computation of the amount of settlement involves the average void ratios of the natural ground before loading with the fill (e_1) and after loading with the fill (e_2) and the thickness of the layer, or:

$$\text{Settlement} = \text{thickness of layer times } \left(\frac{e_1 - e_2}{1 + e_1}\right)$$

ESTIMATE OR THE RATE OF SETTLEMENT

The rate of settlement is determined by the ability of the soil to drain away the excess water and reduce air voids in the soil mass under the pressure caused by the fill load. The basic formula used to estimate the time of settlement (t), is:

$$t = \frac{T\,h^2}{c_V}$$

t = time to settle

T = Terzaghi time factor (depends on pressure and direction of drainage relationships). Select values from Charts C or D.

h^2 = longest vertical drainage path

c_V = average coefficient of consolidation

SETTLEMENT COMPARED TO SHEAR FAILURE

A fill settlement should not be confused with the slide or shear type failure. It might be stated that foundation consolidation is contained within the fill section, and the shear type results in action outside of the fill section. Of course, both types of action can occur in the same embankment.

The above section represents settlement on a foundation soil of sufficient strength to support the fill load. In general terms the action is vertical and the amount of settlement depends on the quantity of water and air pressed out of the foundation soil by the weight of the fill.

The above section represents an embankment placed on a foundation soil which is too weak to support the load of the fill. The pressure of the fill acting vertically causes the weak foundation soil to rise or shear outside of the fill section. If the foundation soil is very unstable a slide can develop in the fill section; this condition is indicated on the right side of the above sketch. The condition on the left side prevails when the pressure of the fill exceeds the strength of the foundation soil, but to a lesser degree than that indicated on the right side of the sketch.

SAMPLING

The importance of sampling cannot be over emphasized. There are several factors involved in sampling which can seriously affect the mathematical approach to an estimated settlement of an embankment foundation; chief of which are:

1. The non homogeniety of soils will be a cause of concern. If a layer of clay within a mass of silty type material is selected for testing, the results would be of questionable value; since such material has a definite effect of slow consolidation. Hence the laboratory technician can become a salient factor in the end result of the computations. Also, since the undisturbed sample is very small (generally about 2 in. in diameter x about 1 in. thick) when compared to the volume of foundation soil, the non homogeniety of the soil demands careful selection of the samples.
2. The method of sampling, where it is generally impractical to remove a sample by means of a pit or excavation (most questionable highway embankment foundations being submerged) are subject to serious consideration. At best a so-called undisturbed sample is disturbed, but to a lesser degree if obtained with best design equipment. Usually best samples are obtained by means of samplers using a removable series of rings, inserted into an outer split shell or containing body. Samples are selected by sawing the soil between adjacent rings and inserting the ring and its contained soil directly into the consolidation apparatus.
3. The technique of taking a sample for consolidation testing can produce varied results. In this respect I believe it best that one man be assigned to supervise or take such complex samples. All strata of the foundation soil must be observed and recorded and zones of typical consolidating soils sampled. Layers of granular material within the soil mass must be recorded and the type of material on which the compressible material rests must be identified. Such observations have a direct bearing on the time element computations for estimated settlement, as will be shown later. With a specialist taking all samples (a soils engineer is desired but not required) most con-

sistent results are obtained and more complete data will be obtained.

4. Improper shipping and handling methods can result in altering or even destroying a properly taken sample, again a sampling specialist can take personal care in transporting samples to the laboratory or take proper action in preparing the undisturbed sample for shipment.

Other factors could be described but it is the intention here to bring attention to some of the problems which fundamentally affect the results of computing estimated settlements. Consistent sampling procedures will reduce the judgment and procedural errors or at least bring them to the attention of the soils engineer responsible for making the estimate. It might be said that if highway engineers were aware of this condition a better understanding of the problem would result and an estimate would be more acceptable and understandable to engineers involved.

TESTING AND REPORTING

The laboratory generally reports the void-ratio and the average coefficient of consolidation (c_V) of the sample at increment loads. The applied test loads must exceed the proposed embankment load (wet weight) by at least 25 percent. The height of the intended embankment must be given to the laboratory, this being for purposes of determining the range of pressure to be applied to the sample. Test loads should range from less than 800 lb per sq ft to at least 25 percent greater than the unit weight of the embankment. The specific gravity and natural moisture should be reported. Other tests are helpful such as screen and hydrometer analysis, liquid limit, plasticity index, ignition loss, etc.

EXAMPLE OF COMPUTATIONS (See Fig. 1, Appendix)

With the data from Figure 2, prepare a graph of the logarithm of the applied pressures against the void ratios (P-e curve). This is prepared on semi-log paper; also on the same sheet, plat a graph of the coefficient of consolidation (c_V) against the computed averages of the applied pressures. Figure 3 shows the P-e and c_V curves from the laboratory data of Figure 2.

Line 4

Compute the unit weight of the submerged foundations soils. Figure 2 gives the specific gravity and natural moisture of the samples. With this data using Chart A, the wet weight of soil is determined, by subtracting 62.4 (unit weight of water) the submerged weight of the soil is obtained. Note: in most problems the water level will rise to the ground level upon application of the fill load; this being the case of most fill foundations which cover much more area than bridge foundations, etc.

Line 5

The wet weight of the fill soil per cubic foot is required to compute the total load of the fill. Often this value is approximately 125 pcf, but it can be computed from the dry weight as:

$$\text{wet weight soil} = \text{dry weight} \times \left(\frac{\%\ \text{moisture}}{100} + 1\right)$$

or the wet fill soil may be compacted in the standard mold and its unit weight determined per cubic foot.

Line 6

Highway fill foundations are considered for problem purposes to have the dimensions of a rectangle one side of which is equal to the average width of the fill and the other side is equal to 4 x the depth of the compressible soils. For study purposes this large rectangle is analyzed as a ¼ section, it being the theory that the maximum load is on one corner of a ¼ rectangle. The values A and B represent the dimensions of ¼ of the larger rectangle.

Line 7

This is the average width of the fill or:

$$\frac{\text{top width} + \text{bottom width}}{2}$$

Line 9

The computation for Δp at this point determines the total load per sq ft of the fill at the ground surface.

Lines 10 and 11

Determine the influence coefficient at the mid-point of the layer, or:

$$m = \frac{A}{\text{Depth from ground surface}}$$

$$n = \frac{B}{\text{Depth from ground surface}}$$

With m and n known the influence coefficient is determined from Chart B. This value is used in computing the decrease pressure in the lower regions of the soil mass due to the fill load. See line 13 for use.

Line 12

The average initial static ground pressure within 1st layer is determined, or:

P_1 = Unit weight foundation soil x ½ thickness of layer

This is the theoretical pressure of the natural ground at the mid-point of the layer without the addition of the fill weight. From Figure 3 determine the void ratio for this pressure from the P-e curve of the layer in question and enter in appropriate column as e_1.

Line 13

The effect of the fill pressure at the mid point of the layer is determined. This computation involves the use of the m and n influence coefficient and expansion to cover the 4 corners of the pressure center or:

Δp = 4 x influence coefficient x fill load, (Δp from line 9)

$p_2 = p_1 + \Delta p$ This pressure is the total average pressure acting in the layer and determines the value of e_2 as taken from Figure 3 at the pressure p_2 for the layer in question. At this point it is well to complete $\frac{p_1 + p_2}{2}$ for the average pressure at the mid point of the layer to determine the average coefficient of consolidation (c_v) from Figure 3 for the layer No. 1 of compressible soil; enter value in the appropriate column.

Lines 14 and 15

Determine the m and n values and the corresponding influence coefficient for the bottom of the layer; same procedure as for lines 10 and 11.

Line 16

This computation is the same as for line 13 but is for the bottom of the layer, or:

Δp = 4 x influence coefficient x fill load.

The void ratio is not required at this point; only the average values of e are required to estimate the subsidence.

Lines 17 and 18

Determine values for m and n at the average depth of the layer below ground surface. Same procedure as lines 10 and 11.

Line 19

Compute the natural ground pressure at the middle of the 2nd layer. In this instance it is necessary to compute the total weight per sq ft of the top layer and add to this the average pressure of the 2nd layer. Generally two unit weights of foundation soil are involved necessitating individual computations as:

p_1 = (unit weight of top layer x thickness) + (unit weight of bottom layer x $\frac{1}{2}$ thickness of layer)

For this value of p_1 select the corresponding e_1.

Line 20

Compute the effect of the fill load at the mid point of the 2nd layer. Same procedure as for line 13. Select e_2 for the pressure p_2, ($p_2 = p_1 + \Delta p$); also select c_v for $\frac{1}{2}(p_1 + p_2)$ from Figure 3, layer No. 2.

Lines 21 and 22

Determine the m and n values for the bottom of the layer, same procedure as lines 10 and 11.

Line 23

Determine the effect of the fill load at the bottom of the layer, same procedure as line 13. The void ratio is not required at this point.

Line 25

Compute the settlement of each individual layer, from:

$$\text{Settlement} = \text{thickness of layer times } \left(\frac{e_1 - e_2}{1 + e_1}\right)$$

Values of e_1 and e_2 are taken for each layer from the work sheet Figure 1, having been determined from Figure 3 for the pressures p_1 and p_2 at the middle of the layer. The computed settlement of each layer is entered on line 38 under the columns headed by "Settlement" No. 1 layer and No. 2 layer. Further computations involve entering the percent of settlement to complete these columns.

Lines 26 to 29 on right side of work sheet, Figure 1

Charts C and D indicate the effect of time of settlement. It will be noted that the magnitude of the top and bottom pressures affect the T factor. In Cases IV and V the value of u or ratio of top pressure (Δp) to the bottom pressure (Δp) must be determined in order to select the proper T value. After determining the proper case (and value of u if required) enter the corresponding T factor from Charts C or D for the percent consolidation on the work sheet for the individual layers.

ESTIMATE OF TIME OF SETTLEMENT

$$t = \frac{T h^2}{c_v}$$

This formula estimates the time of settlement for the computed percent of settlement, using the applicable T factor; h is the longest vertical path the water must travel to escape from the layer. In the case of a granular bottom under a compressible soil layer, the water travel can be both up and down or the longest vertical path will be $\frac{1}{2}$ the thickness of the layer in question. It will be noted that foundation soils which are free to drain at the top and bottom are identified as Case I of Chart C. If c_v or coefficient of consolidation is given in days the resulting computation for t will be in days.

Complete the work form by computing the t value for all percent indicated in the form and enter the data under the proper column for the individual layers.

PREPARE GRAPH OF SETTLEMENT VS TIME

Following the computations of the amount of settlement and the corresponding time to attain such settlement, prepare a graph on 10 x 10 cross-section paper for each layer using the amount of settlement against the time to attain this settlement (see Fig. 4).

By adding the settlement values of each layer (as graphed) at selected time intervals prepare a graph representing the total settlement of both layers; see Figure 4. This graph can be studied by the engineers who are concerned and be an aid in establishing required quantities to maintain the fill height. It is also very helpful in considering the subsidence during the construction period. Also it will reveal the probability of maintaining a grade line following the construction period.

Once this form and example (Fig. 1) are understood, it will be much easier to refer to the tests prepared by the authorities of the subject. The above example was selected for the purpose of explaining the procedure. If a field problem involves more layers additional sheets may be used for the additional layers.

Some engineers believe that the estimated time of settlement is quicker than the computed value, particularly when there is evidence of varving or layering in the foundation soils. Under such conditions the error, if any, could be attributed to sampling, drill logging the data, etc. If the drill data indicates layers or formations through which the excess water and air can migrate horizontally, a value of ½ the thickness of the layer (longest vertical path for the water to escape) can be used. Using the sample form many layers can be computed by adding more sheets. Of course, intensive sampling might be considered impractical; with this in mind some engineers are dividing the thickness of the compressible layer by 1.5 for anticipated drainage at one surface and by 3.0 if two faces are possible.

ACKNOWLEDGMENT

I wish to acknowledge valuable assistance by the following persons who helped me to prepare this paper: Jaime de La Fuente, Professor of Soils, "Mayor de San Andres" University, La Paz, Bolivia, and William C. Hill, Soils Engineer, Oregon State Highway Department, Salem.

REFERENCES

1. Lambe, T. William, "Soil Testing for Engineers," pp. 74-87, 1954.
2. Sloane, R.L., "Settlement Analysis," Roads and Streets, pp. 53-58, Feb. 1952.
3. Spangler, Merlin Grant, "Soil Engineering," pp. 251-294, 1951.
4. Sowers and Sowers, "Introductory Soil Mechanics and Foundations," pp. 29-38 and 115-126, 1951.
5. Highway Research Board Bulletin No. 115, "Vertical Sand Drains for Stabilizing Embankments," 1956.
6. Taylor, Donald W., "Fundamentals of Soil Mechanics," pp. 268-310, 1950.
7. Tschebotarioff, Gregory P., "Soil Mechanics, Foundations and Earth Structures," pp. 95-119, 1951.
8. American Society of Civil Engineers, "Subsurface Exploration and Sampling of Soils for Engineering Purposes," 1949.
9. Highway Research Board Bibliography No. 15, "Survey and Treatment of Marsh Deposits," 1954.
10. Highway Research Board, Bulletin No. 90, "Sand Drains," 1954.

Appendix

NOMENCLATURE AND NOTES FOR FIGURES 1-4, CHARTS A AND B AND EXAMPLES I-V

p_1 = Natural ground pressure at mid point of layer.
Δp = Consolidating pressure of fill load.
$p_2 = p_1 + \Delta p$.
$p_1 + p_2 \div 2$ = Average pressure for determining average value of c_V.
e = Void ratio = ratio of the volume of soil solids to the volume of voids.
c_V = Coefficient of consolidation.
m = A ÷ depth considered.
n = B ÷ depth considered.
Infl. Coeff. = Factor for computing effect of fill load at various depths.
u = Ratio used in Cases IV and V for determining T factor = Δp top ÷ Δp bottom.
T Factor = Time factor for determining settlement rate (from Chart C or D).
t = Time of settlement.

Plat P-e curve on semi log paper, using applied pressures and corresponding void ratios, also plat c_V curve using average applied pressures and average coefficient of consolidations.

LINE 4, Using Chart A, determine submerged unit weight of foundation soils.

LINE 5, This is the wet weight of the fill material per cubic foot.

LINE 6, A and B represent the dimensions of ¼ of a theoretical pressure area.

LINE 7, This is the average width of the fill section.

LINE 9, Compute fill pressure at ground surface or; Δp = unit wght. of fill x fill height

LINE 10, m = A ÷ depth to mid point of layer (from ground surface).

LINE 11, n = B ÷ depth from ground surface to mid point of layer. With m and n known, select the influence coefficient from Chart B.

LINE 12, p_1 = unit weight of foundation soil x depth to mid point of layer.

LINE 13, Δp = pressure of fill at mid point of layer = 4 x fill load x infl. coeff.

LINES 14, 15, 16, same as Lines 10, 11, 13, except compute for bottom of layer.

LINES 17, 18, 19, 20, same as Lines 10, 11, 12, 13, except for mid point (total dist. from ground surface) of 2nd layer. Note, two unit weights of submerged foundation soil are generally involved.

LINES 21, 22, 23, same as Lines 10, 11, 13, except for bottom of layer.

LINE 25, Settlement = thickness of layer x $e_1 - e_2 \div 1 + e_1$, e_1 and e_2 are taken from the P-e curve for pressures p_1 and p_2 respectively.

LINES 26 and 28, if Case IV or V is involved, compute u = Δp at top ÷ Δp at bottom of layer. Enter appropriate T factor from Charts C or D in column as indicated. Determine, $t = T \times h^2 \div c_V$, in which h = the longest vertical drainage path. If the bottom layer is granular, h = ½ thickness of layer because water is free to travel in both vertical directions. c_V is selected from the coefficient of consolidation curve at the computed pressure for each layer or $p_1 + p_2 \div 2$.

LINES 29, 38, compute percent settlement of each layer, 100 percent is the values from Line 25. Compute time to settle and enter in appropriate column. If c_V is given in sq ft/day t will be in days, if c_V is given in sq ft/year t will be in years. Prepare a graph of percent settlements against corresponding t for each layer, from this graph accumulate the settlement of both layers at selected periods of t for a graph of total subsidence.

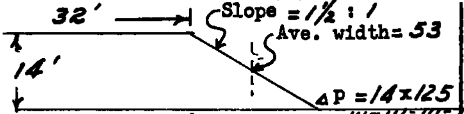
NOTE:

For estimating purposes, it can be considered that foundation materials will be entirely submerged. Generally the effect to the fill pressures will cause water to rise to the original ground line or top of the compressible soil.

OREGON STATE HIGHWAY DEPARTMENT
CONSTRUCTION DIVISION
SOILS SECTION
ESTIMATE OF EMBANKMENT FOUNDATION SETTLEMENT FOR HIGHWAY USE

1 Section______ Highway ______ County ______
2 Station 43+00, ft. lt. 10, ft. rt. ___, ℄___. Sampled by Adamson
3 Lab. no.______, depth 7 to 8 ft. Lab. no.______, depth 10 to 11 ft.
4 Unit wght of submerged soils; layer #1= 35 pcf; layer #2= 44 pcf.
5 " " " embankment soil (wet wght.) = 125 pcf.
6 A=½ ave. width of emb.= 26.5 ft. B=½ (4xdepth of compressible soils)= 120.0 ft.

7
8
9



	Infl. Coeff.	Depth	p_1	Δp	p_2	$\frac{p_1+p_2}{2}$	e	c_v		
9		0.0		1750						
10 m at 4.5 ft.= $\frac{26.5}{4.5} = 5.9$	0.249								Layer #1	Compressible soils
11 n at 4.5 ft.= $\frac{120}{4.5} = 26.7$										
12 p_1 at 4.5 ft.= 35 x 4.5		4.5	158				e_1=1.62			
13 Δp at 4.5 ft.= 4x 1750 x 0.249		4.5		1743			e_2=1.42			
14 m at 9 ft.= $\frac{26.5}{9} = 2.94$					1901	1030		0.95		
15 n at 9 ft.= $\frac{120}{9} = 13.33$	0.247									
16 Δp at 9 ft.= 4x1750 x 0.247		9.0		1729						
17 m at 34.5 ft.= $\frac{26.5}{34.5} = 0.77$	0.18								Layer #2	
18 n at 34.5 ft.= $\frac{120}{34.5} = 3.48$										
19 p_1 at 34.5 ft.=(35 x 9)+(25.5 x 44)		34.5	1437				e_1=1.09			
20 Δp at 34.5 ft.= 4x1750x 0.18		34.5		1260			e_2=1.04			
21 m at 60.0 ft.= $\frac{26.5}{60.0} = 0.44$	0.12				2697	2067		1.16		
22 n at 60.0 ft.= $\frac{120}{60} = 2.0$										
23 Δp at 60.0 ft. = 4x1750 x 0.12		60.0		840						

24 ← Type of soil; describe, Clay - Sand

25 SETTLEMENT; layer #1= 9.0 x $\frac{1.62-1.42}{1+1.62}$ = 0.68 ft; layer #2 = 51.0 x $\frac{1.09-1.04}{1+1.09}$ = 1.22 ft.

26	Settlement		T Factor		t Days	
27 %	#1	#2	#1	#2	#1	#2
28						
29 10	0.07	0.09	0.01	0.01	1	22
30 20	0.14	0.24	0.03	0.02	3	45
31 30	0.20	0.37	0.07	0.04	6	90
32 40	0.27	0.49	0.13	0.09	11	202
33 50	0.34	0.61	0.19	0.15	16	336
34 60	0.41	0.73	0.29	0.23	25	516
35 70	0.48	0.85	0.40	0.39	34	874
36 80	0.54	0.98	0.57	0.51	49	1143
37 90	0.61	1.10	0.85	0.79	72	1771
38 100	0.68	1.22	3.0 ±	3.0 ±	255	6726

Layer #1; u= $\frac{1750}{1729}$ = 1.01 Use Case V

Layer #2; u = $\frac{1729}{840}$ = 2.05 Use Case V

Layer #1; t = $\frac{T \times 9^2}{0.95}$ = T x85

Layer #2; t = $\frac{T \times 51^2}{1.16}$ = T x2242

Ray Webber 5/31/56 6-15-56
Computed by, date, Check by, date

Figure 1.

LABORATORY DATA

LAYER #1
Depth of sample 7 ft. to 8 ft.
Specific gravity 2.58
Natural moisture 69.1
Ignition loss at 1000° C = 7.9%
L L = 31, % Silt = 22
P I = 0, Pass 200 screen =39%

LAYER #2
Depth of sample 10 ft. to 11 ft.
Specific gravity 2.62
Natural moisture 48.5
Ignition loss at 1000° C = 10.6%
L L = 40, % Silt = 30
P I = 0, Pass 200 screen = 45%

Applied Pressure	Voids Ratio	Coeff. of Consol. Sq. Ft. / Day	Applied Pressure	Voids Ratio	Coeff. of Consol. Sq. Ft. / Day
0	1.785		0	1.235	
		0.865			1.853
500	1.584		750	1.284	
		0.797			1.284
1000	1.511		1500	1.079	
		1.034			1.116
2000	1.415		3000	1.027	
		0.872			1.102
4000	1.308		6000	0.968	
500	1.340		750	0.985	
0	1.390		0	1.013	

LOG OF DRILL HOLE

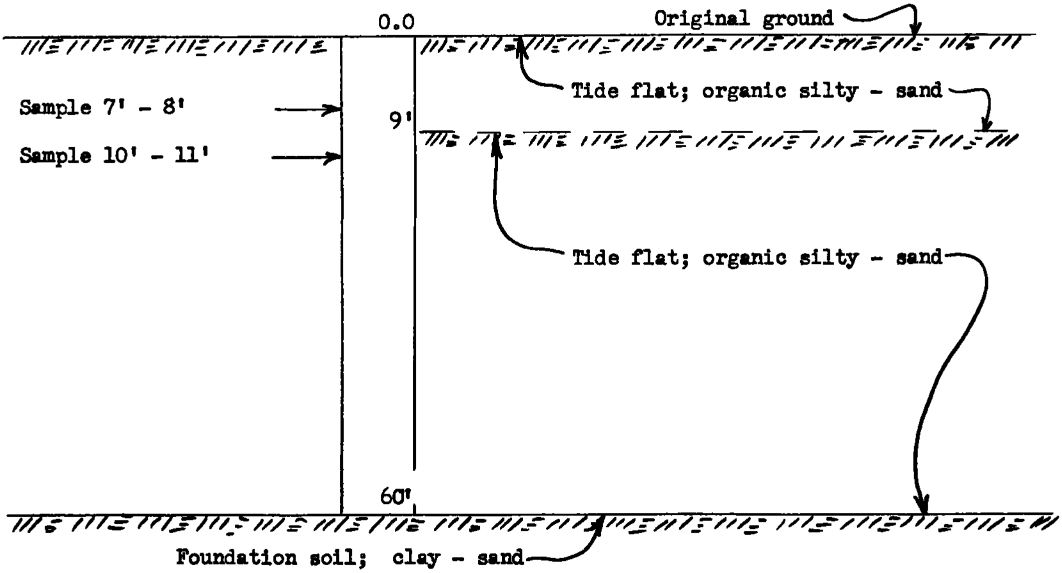


Figure 2.

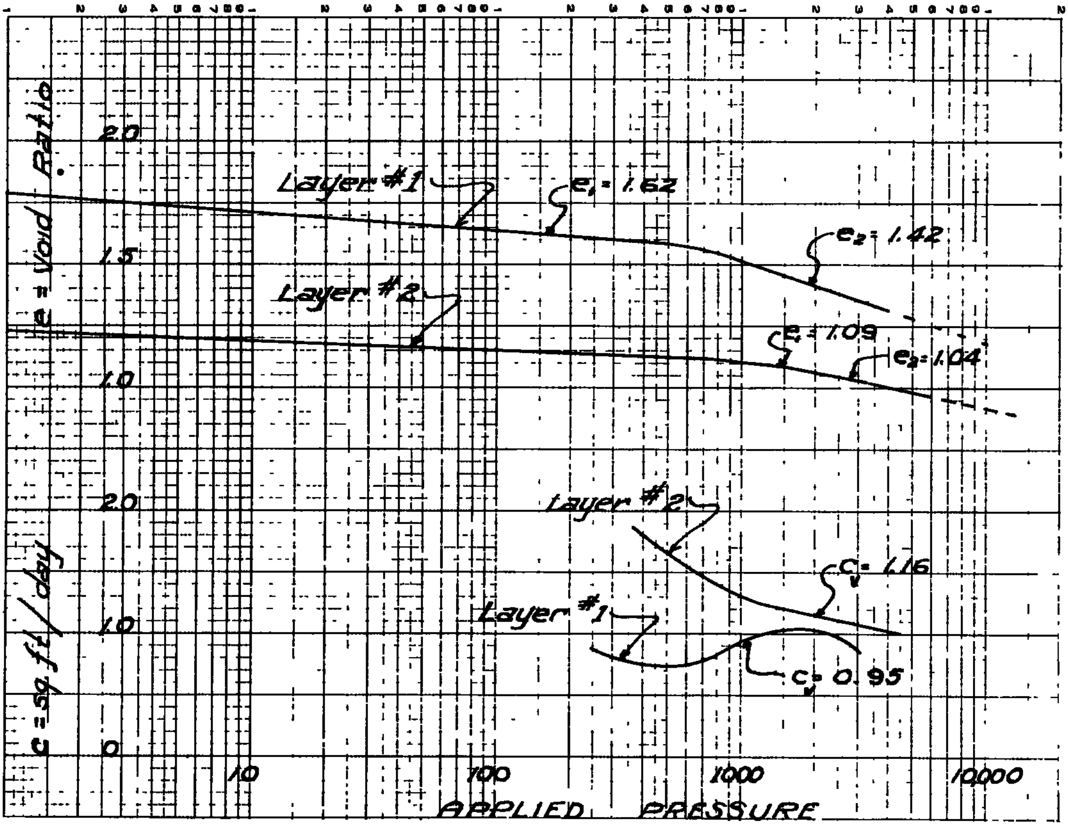


Figure 3.

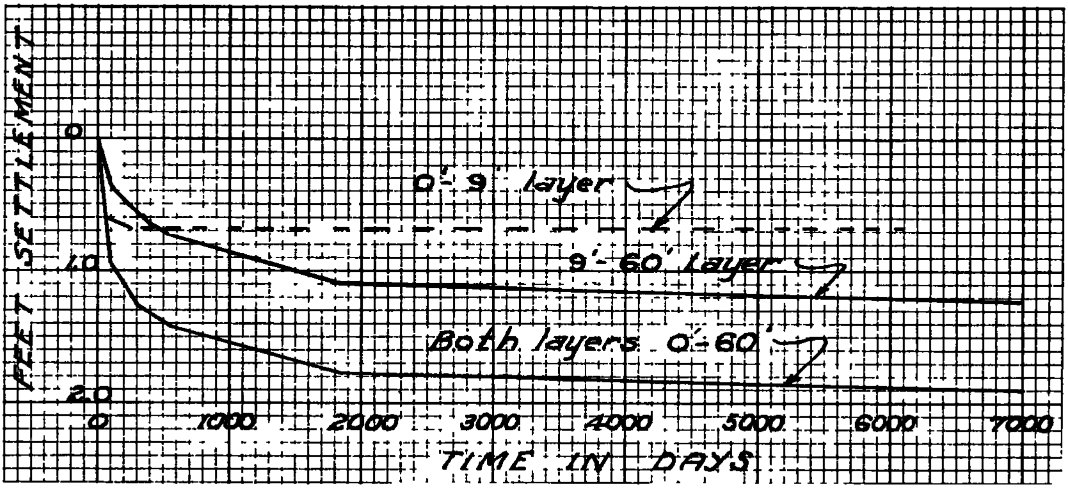


Figure 4.

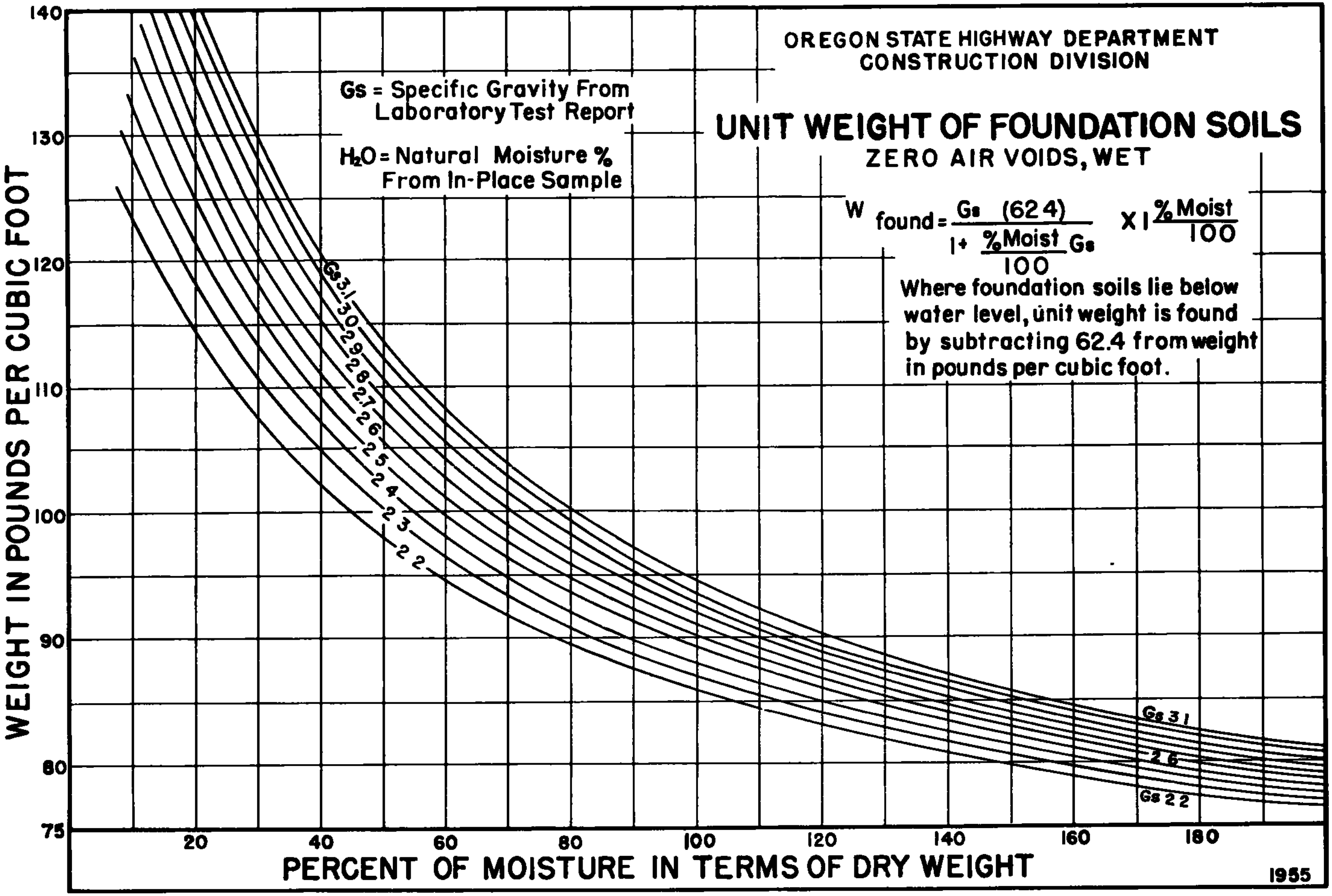

OREGON STATE HIGHWAY DEPARTMENT
CONSTRUCTION DIVISION
UNIT WEIGHT OF FOUNDATION SOILS
ZERO AIR VOIDS, WET
$W_{found} = \frac{G_s\,(62.4)}{1 + \frac{\%\,Moist}{100}\,G_s} \times 1\frac{\%\,Moist}{100}$
Where foundation soils lie below water level, unit weight is found by subtracting 62.4 from weight in pounds per cubic foot.
Gs = Specific Gravity From Laboratory Test Report
H₂O = Natural Moisture % From In-Place Sample
Gs 3.1
3.0
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
Gs 3.1
2.6
Gs 2.2
WEIGHT IN POUNDS PER CUBIC FOOT
140
130
120
110
100
90
80
75
PERCENT OF MOISTURE IN TERMS OF DRY WEIGHT
20
40
60
80
100
120
140
160
180
1955

Chart A.

NEWMARK CHART FOR VERTICAL STRESSES DUE TO EMBANKMENT LOADS

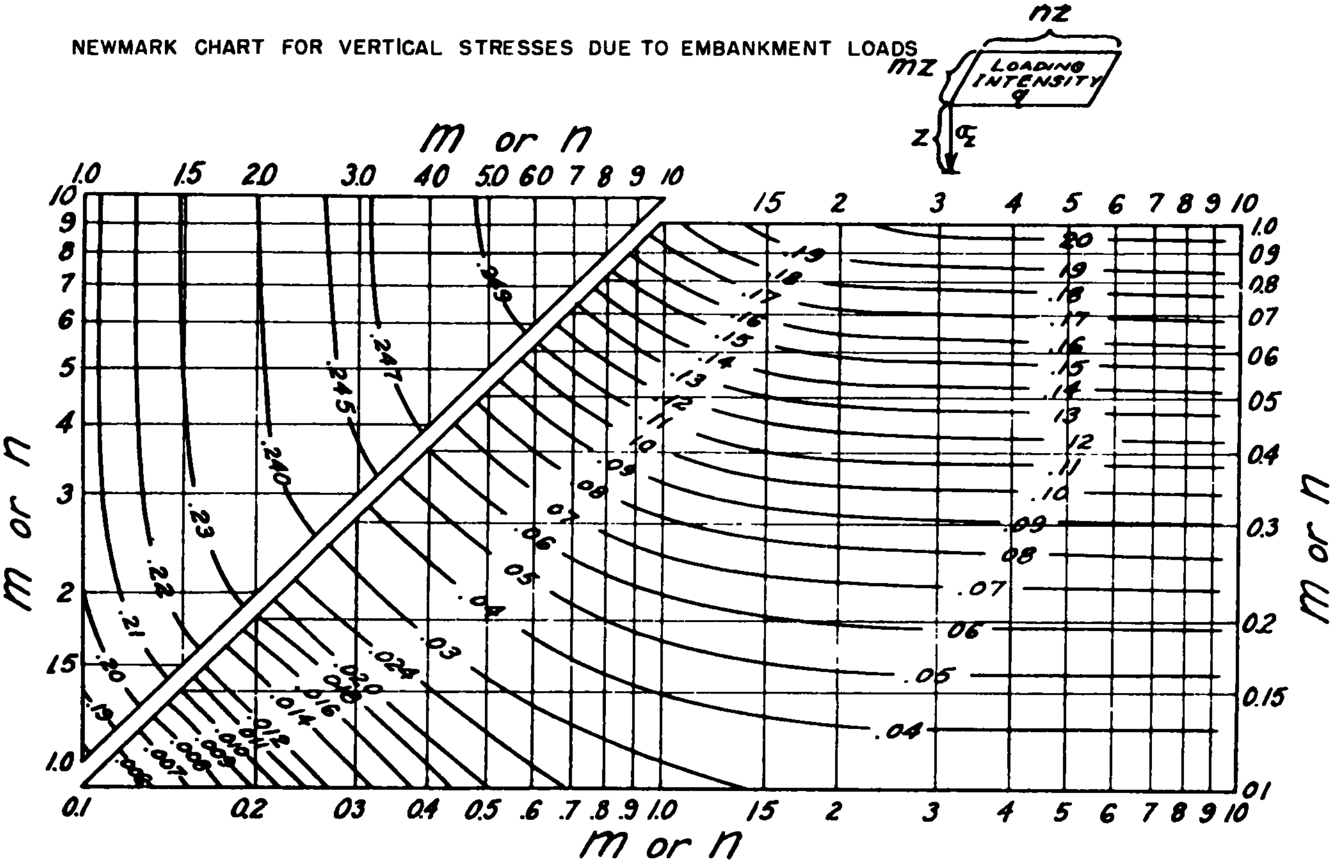
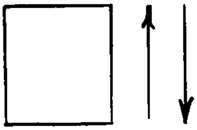


Chart B.

OREGON STATE HIGHWAY DEPARTMENT
CONSTRUCTION DIVISION
SOILS SECTION
FOUNDATION CONSOLIDATION INVESTIGATION

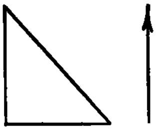
CASE I



Pressure distribution equal with drainage in both directions.

% Cons. =	10	20	30	40	50	60	70	80	90	100
T factor =	.008	.032	.069	.126	.198	.288	.405	.566	.846	3 ±

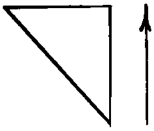
CASE II



Pressure distribution diagram with flow from maximum to zero pressure.

% Cons.	10	20	30	40	50	60	70	80	90	100
T factor	.049	.101	.158	.223	.296	.385	.502	.656	.951	3 ±

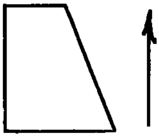
CASE III



Pressure distribution diagram with flow from zero to maximum pressure.

% Cons.	10	20	30	40	50	60	70	80	90	100
T factor	.002	.008	.024	.049	.097	.194	.279	.437	.717	3 ±

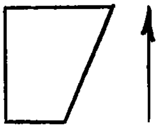
Chart C. Internal pressure and drainage relationships for Terzaghi time Factor "T".



CASE IV

Pressure distribution trapezoid with lesser pressure in direction of flow.

		RATIO OF TOP						BOTTOM PRESSURES				
	u=	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Percent Consolidation	10	0.05	0.04	0.04	0.03	0.03	0.02	0.02	0.02	0.01	0.01	0.01
	20	0.10	0.09	0.08	0.07	0.06	0.06	0.05	0.04	0.04	0.04	0.03
	30	0.16	0.15	0.13	0.12	0.11	0.10	0.09	0.09	0.08	0.07	0.07
	40	0.22	0.21	0.19	0.18	0.17	0.16	0.15	0.15	0.14	0.13	0.13
	50	0.30	0.28	0.27	0.26	0.24	0.23	0.23	0.22	0.21	0.20	0.20
	60	0.38	0.37	0.36	0.34	0.33	0.32	0.32	0.31	0.30	0.29	0.29
	70	0.50	0.48	0.47	0.46	0.45	0.44	0.43	0.43	0.42	0.41	0.40
	80	0.66	0.64	0.64	0.62	0.62	0.60	0.60	0.59	0.58	0.57	0.57
	90	0.96	0.94	0.92	0.91	0.89	0.88	0.87	0.87	0.86	0.85	0.85
	100	3.±	3.±	3.±	3.±	3.±	3.±	3.±	3.±	3.±	3.±	3.±



CASE V

Pressure distribution trapezoid with greater pressure in direction of flow.

		RATIO OF TOP						BOTTOM PRESSURES			
	u =	1	1.5	2	3	4	5	7	10.0	20.0	100
Percent Consolidation	10	0.008	0.008	.008	.008	.008	.008	.004	.004	.004	.004
	20	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	.008
	30	0.07	0.05	0.04	0.04	0.04	0.04	0.04	0.04	0.03	0.02
	40	0.13	0.10	0.09	0.09	0.08	0.08	0.07	0.06	0.06	0.05
	50	0.19	0.17	0.15	0.15	0.14	0.14	0.13	0.12	0.11	0.10
	60	0.29	0.25	0.23	0.22	0.21	0.21	0.21	0.20	0.19	0.17
	70	0.40	0.37	0.39	0.34	0.33	0.33	0.32	0.31	0.30	0.28
	80	0.57	0.53	0.51	0.49	0.49	0.49	0.48	0.47	0.45	0.44
	90	0.85	0.81	0.79	0.78	0.77	0.77	0.76	0.74	0.73	0.72
	100	3.±	3.±	3.±	3.±	3.±	3.±	3.±	3.±	3.±	3.±

Chart D. Pressure distribution correction for T (time factor).

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, 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.