Foundation Settlements in the Ft. Randall Dam Embankment

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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

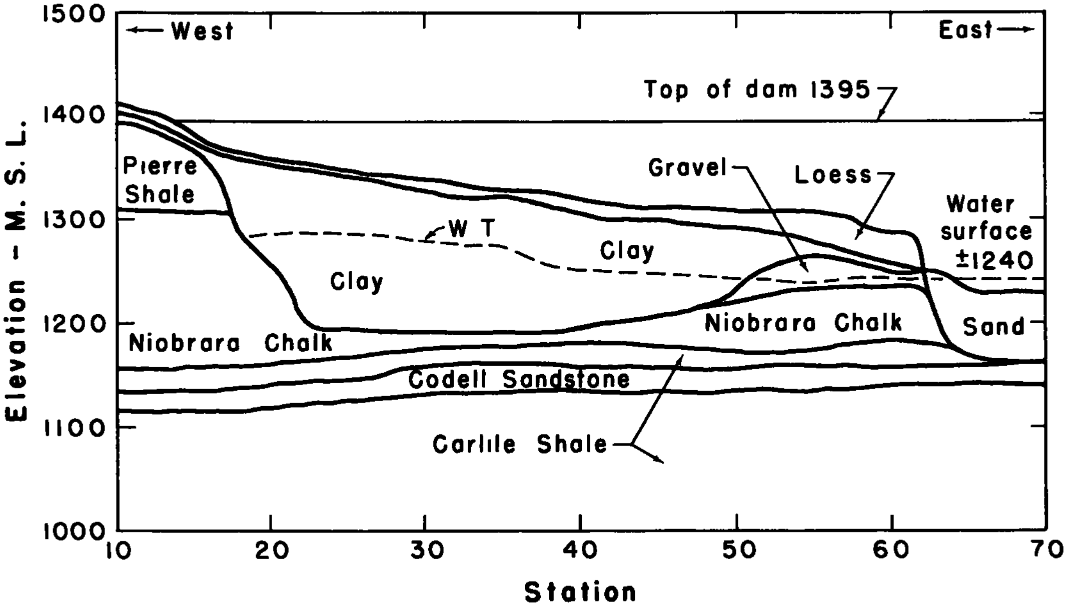


Figure 1.

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

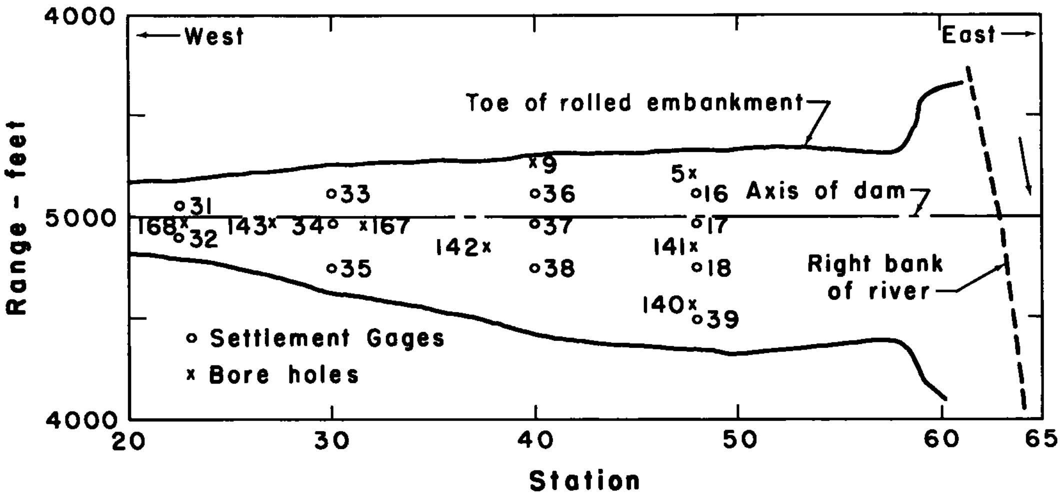
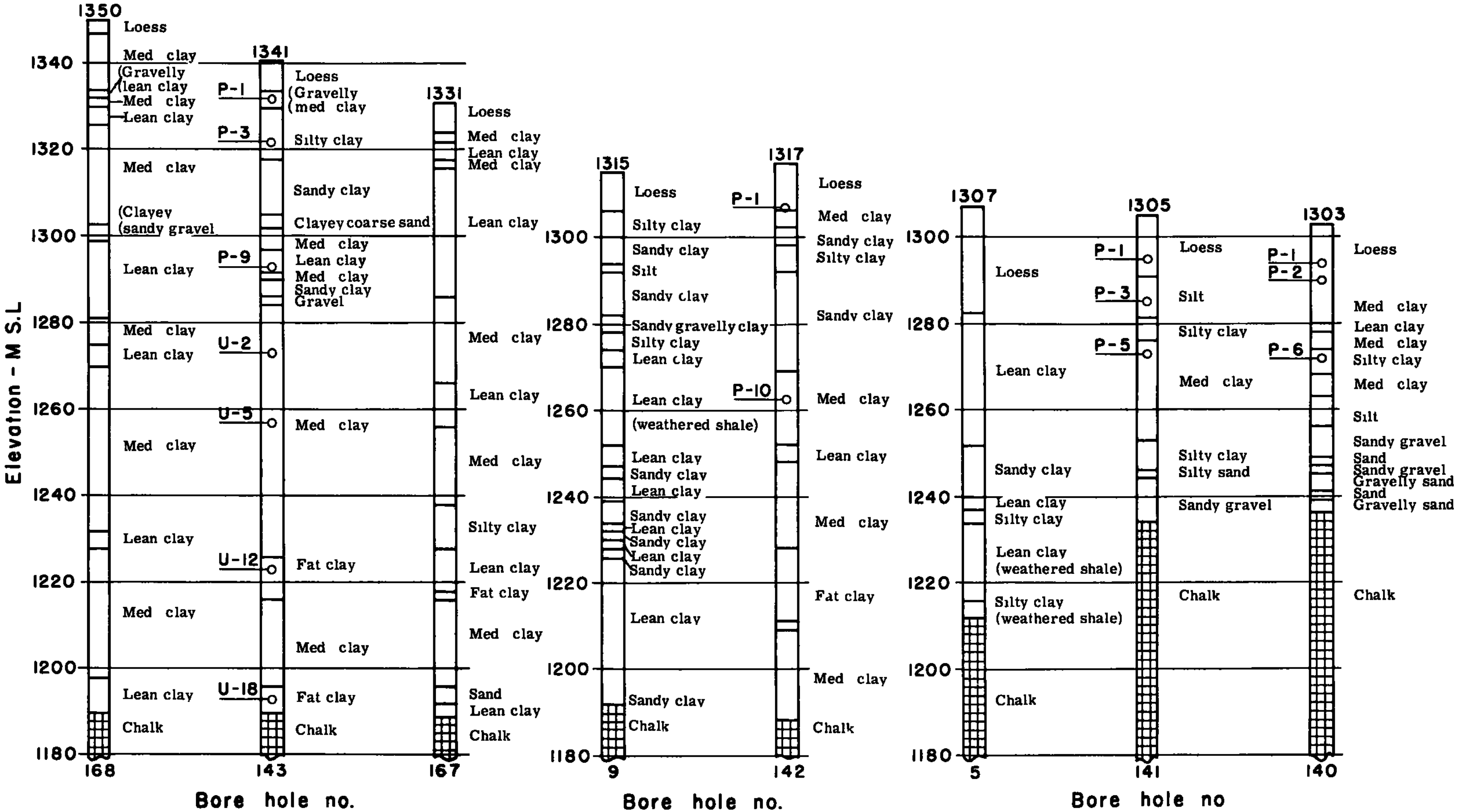


Figure 2.



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
168
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
143
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
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
9
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
142
Bore hole no.
1307
Loess
Lean clay
Sandy clay
Lean clay
Silty clay
Lean clay
(weathered shale)
Silty clay
(weathered shale)
Chalk
5
1305
P-1
P-3
P-5
Loess
Silt
Silty clay
Med clay
Silty clay
Silty sand
Sandy gravel
Chalk
141
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
140
Bore hole no

Figure 3.

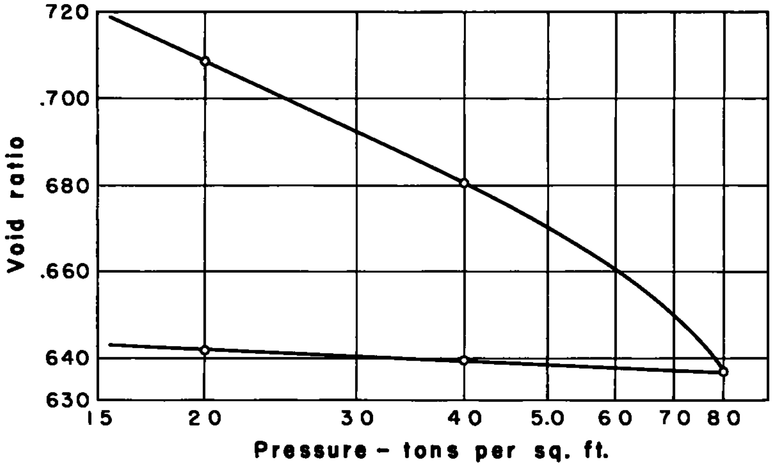


Figure 4. Void ratio - pressure diagram, sample 143, P-9.

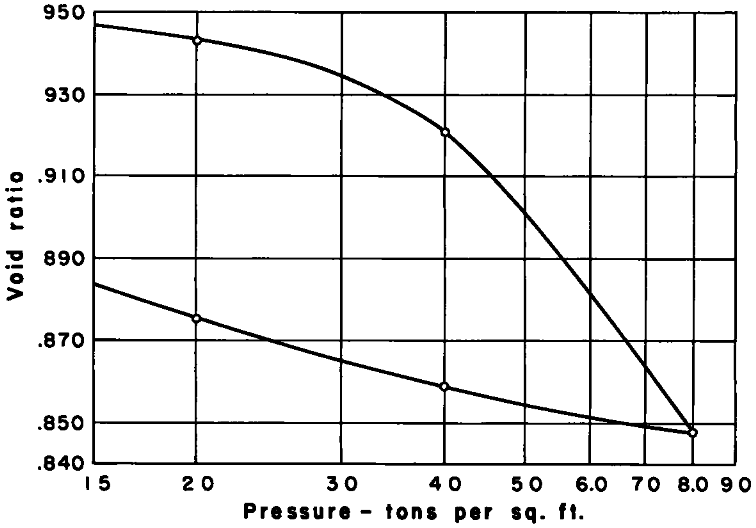


Figure 5. Void ratio - pressure diagram, sample 143, U-5.

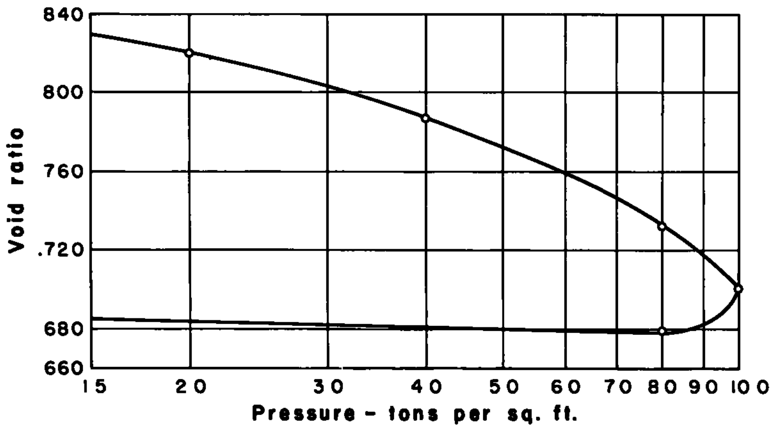


Figure 6. Void ratio - pressure diagram, sample 142, P-10.

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

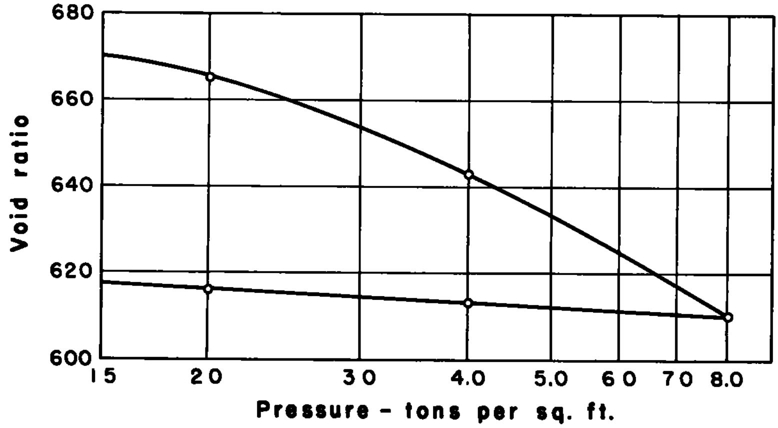


Figure 7. Void ratio - pressure diagram, sample 141, P-3.

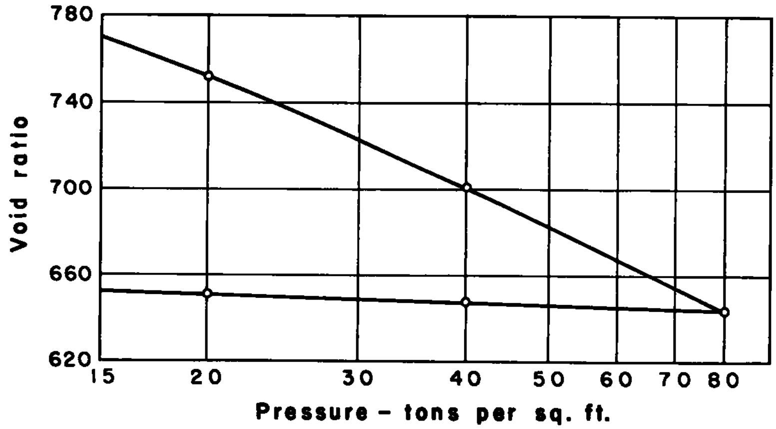


Figure 8. Void ratio - pressure diagram, sample 141, P-5.

TABLE 4

CONSOLIDATING PRESSURES AT MID-PLANE AT THE VARIOUS SETTLEMENT GAGES

Gage No.	Sta.	Height of Embankment, h. ft.	q_o 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								2.250	0.262	0.314	-110	3.38
34	30 + 00	67	4.02	30	540	295	1.83	0.698	0.681	1.430	35	3.67
35								0.157	0.061	2.268	250	1.13
36								2.372	0.227	0.323	-110	4.66
37	40 + 00	88	5.28	30	725	400	1.81	0.681	0.698	1.524	35	4.95
38								0.157	0.065	2.598	250	2.44
16								2.629	0.183	0.227	-110	5.04
17	48 + 00	92.5	5.55	30	850	570	1.49	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:

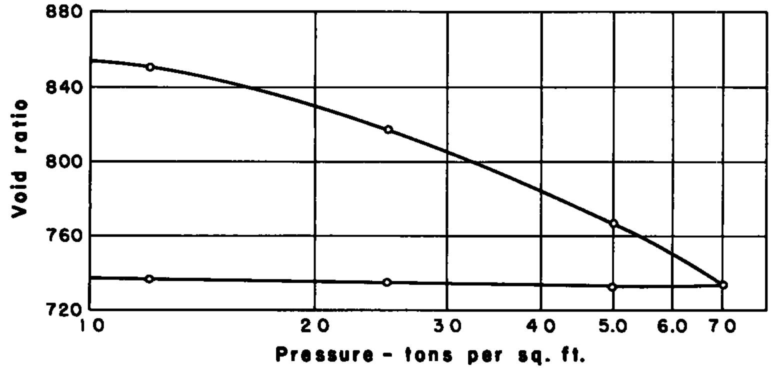


Figure 9. Void ratio - pressure diagram, sample 140, P-6.

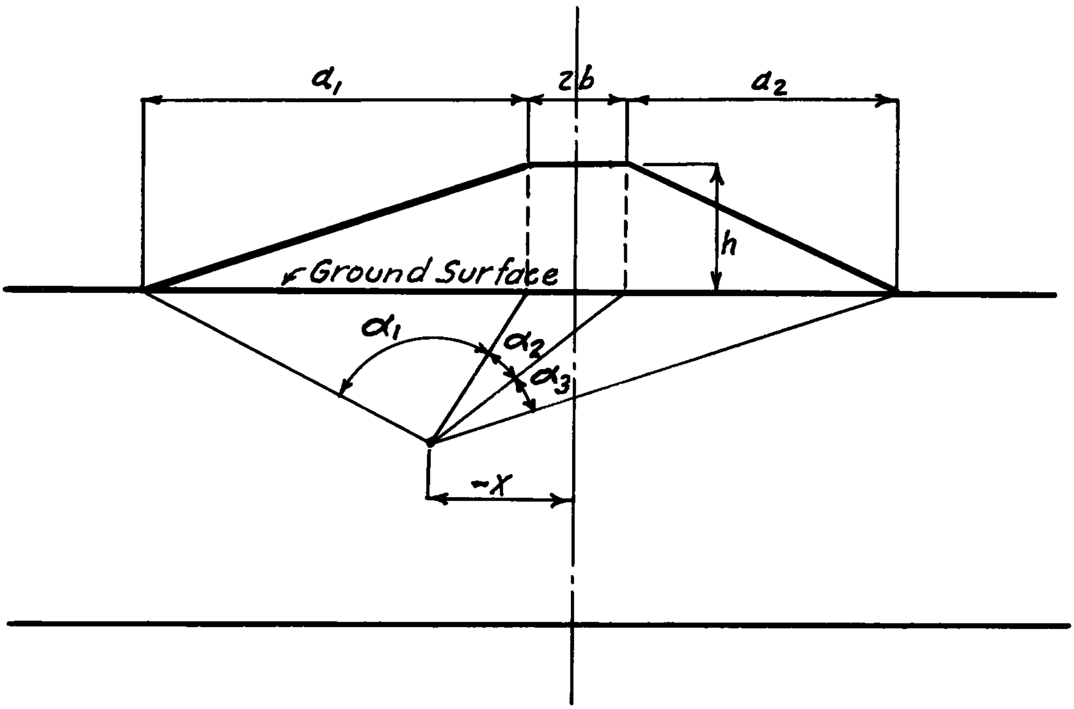


Figure 10.

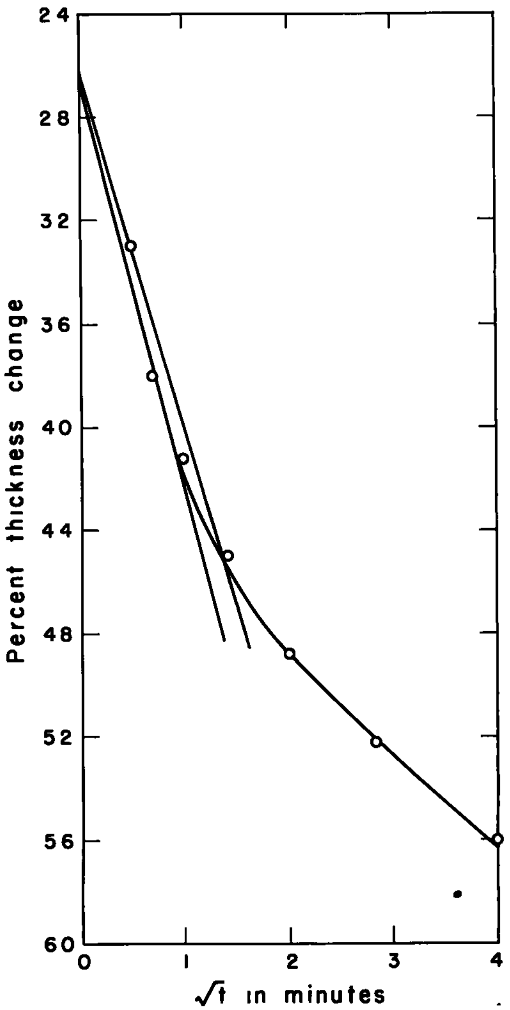


Figure 11. Laboratory time curve, sample 141, P-3. Load IT/sq ft.

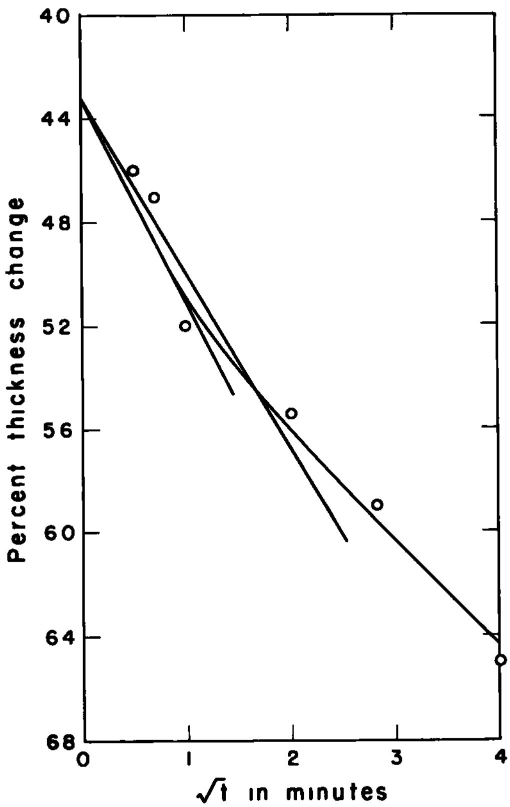


Figure 12. Laboratory time curve, sample 141, P-3. Load 2T/sq ft.

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

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

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_0 = wh = unit weight of soil times height of embankment

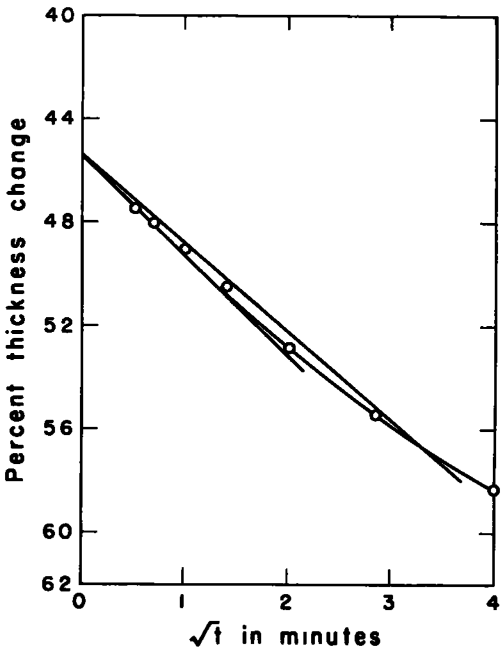


Figure 13. Laboratory time curve, sample 141, P-3. Load 4T/sq ft.

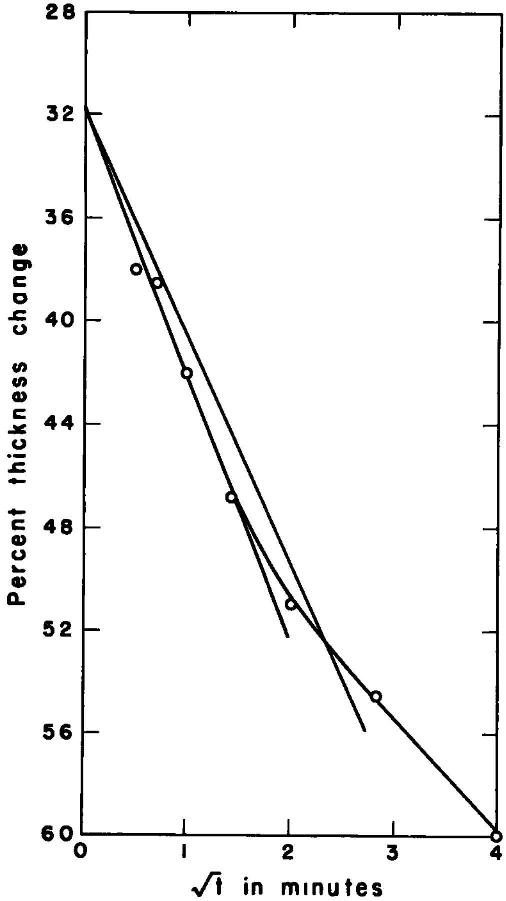


Figure 14. Laboratory time curve, sample 141, P-3. Load 8T/sq ft.

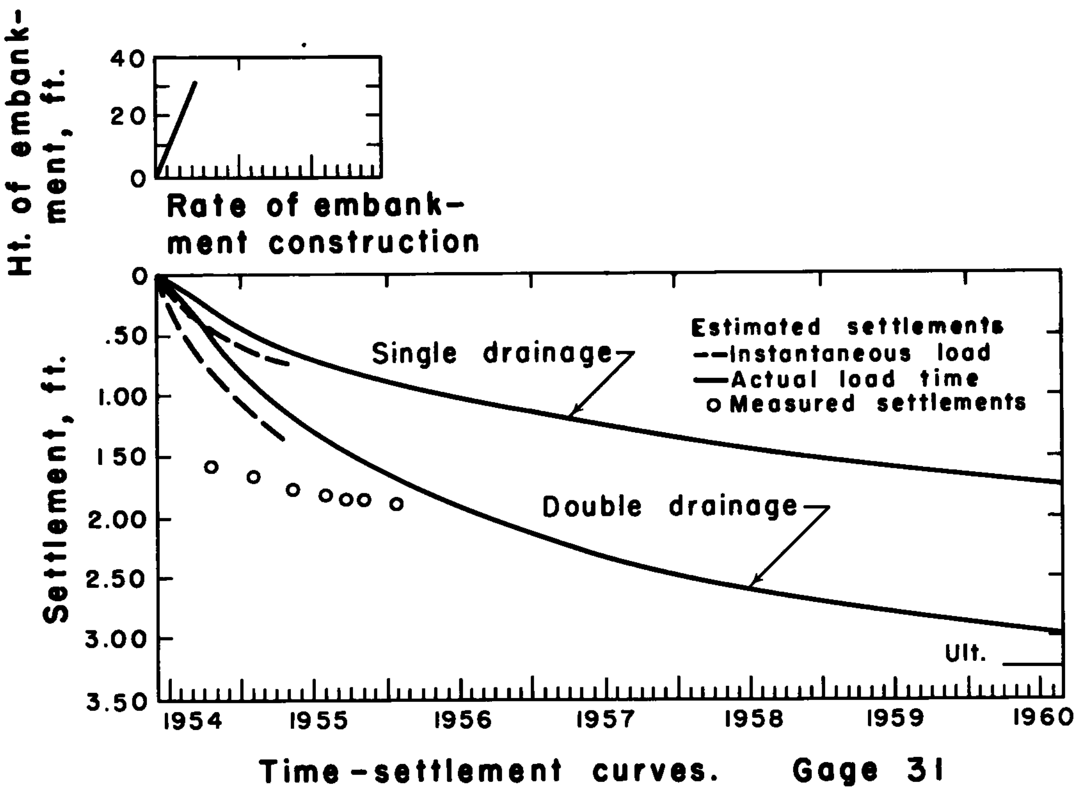


Figure 15.

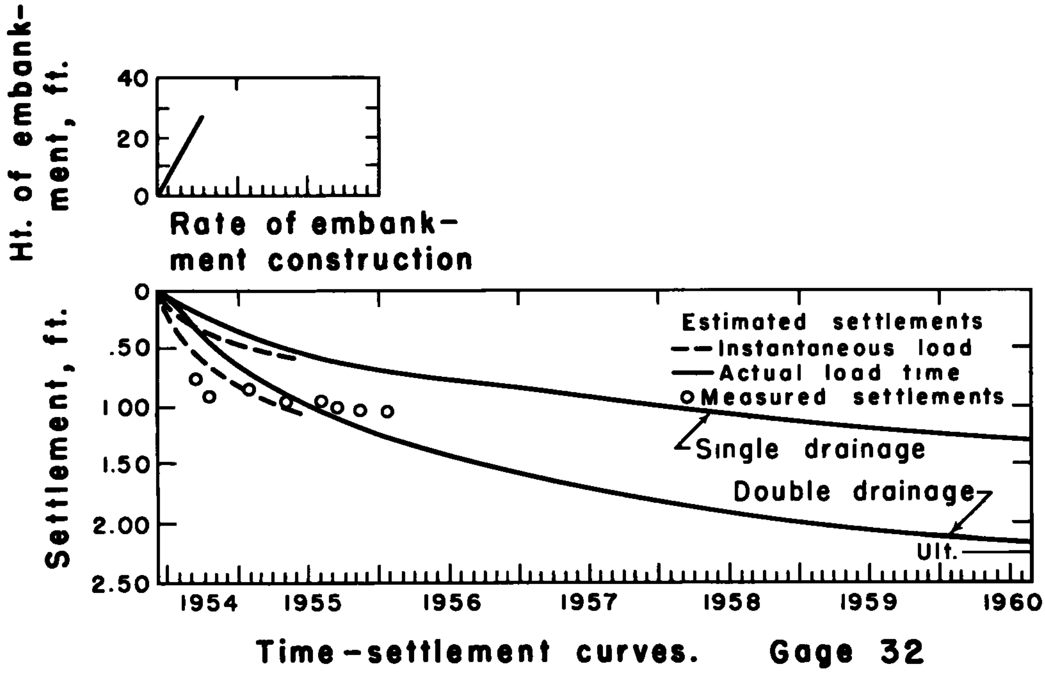


Figure 16.

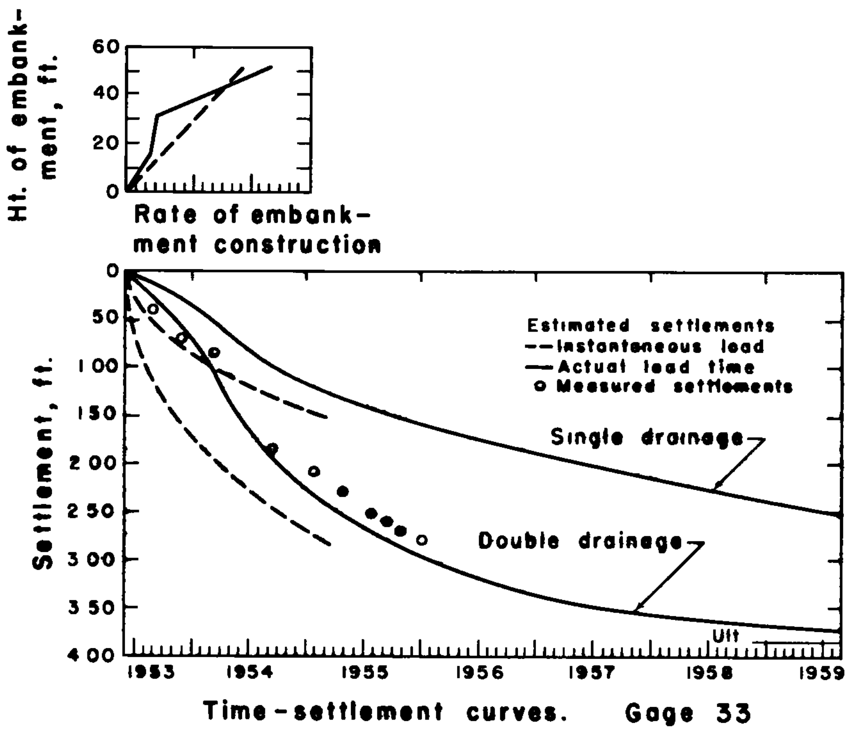


Figure 17.

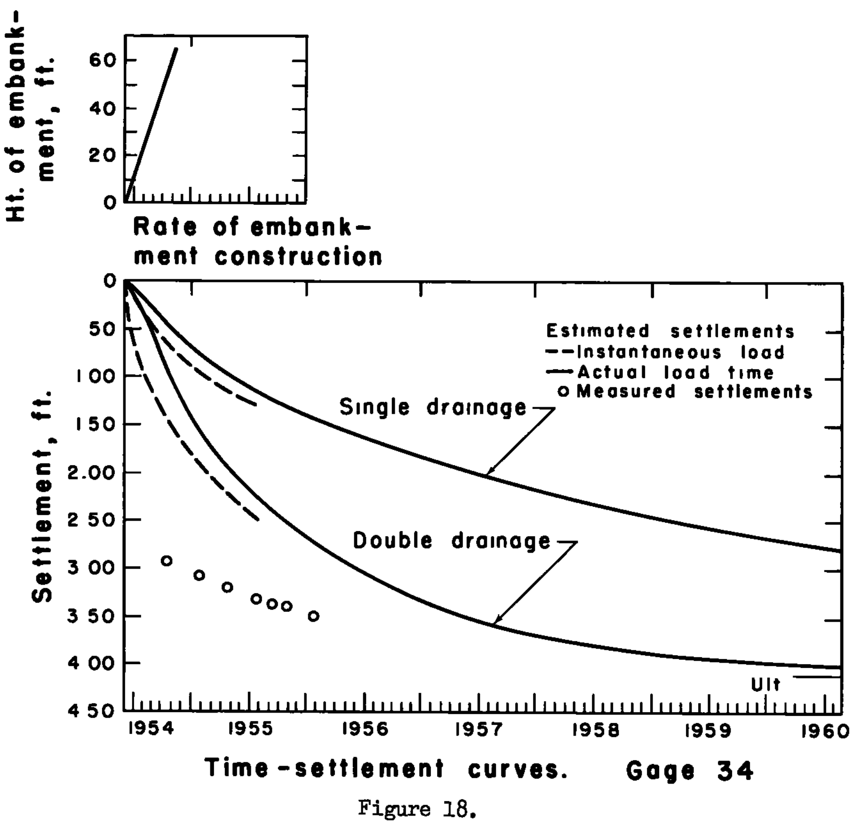


Figure 18.

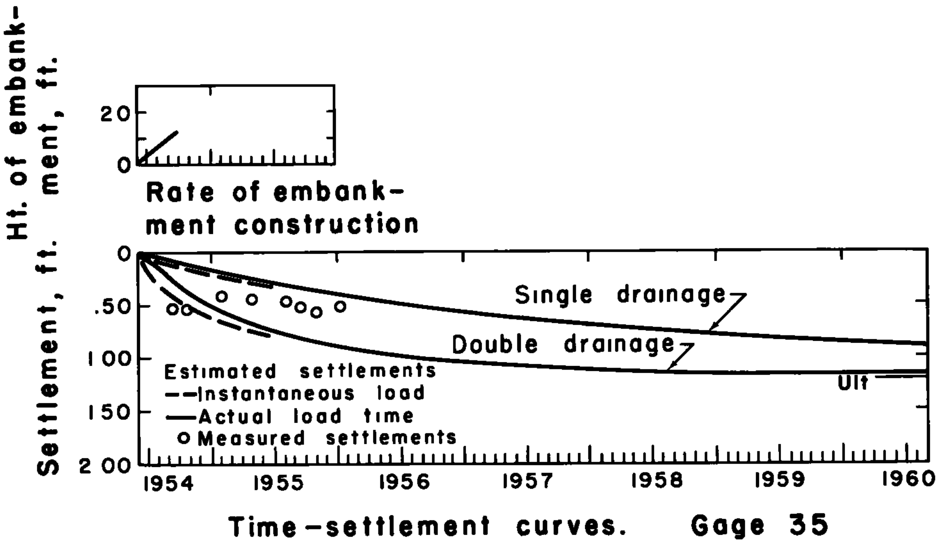


Figure 19.

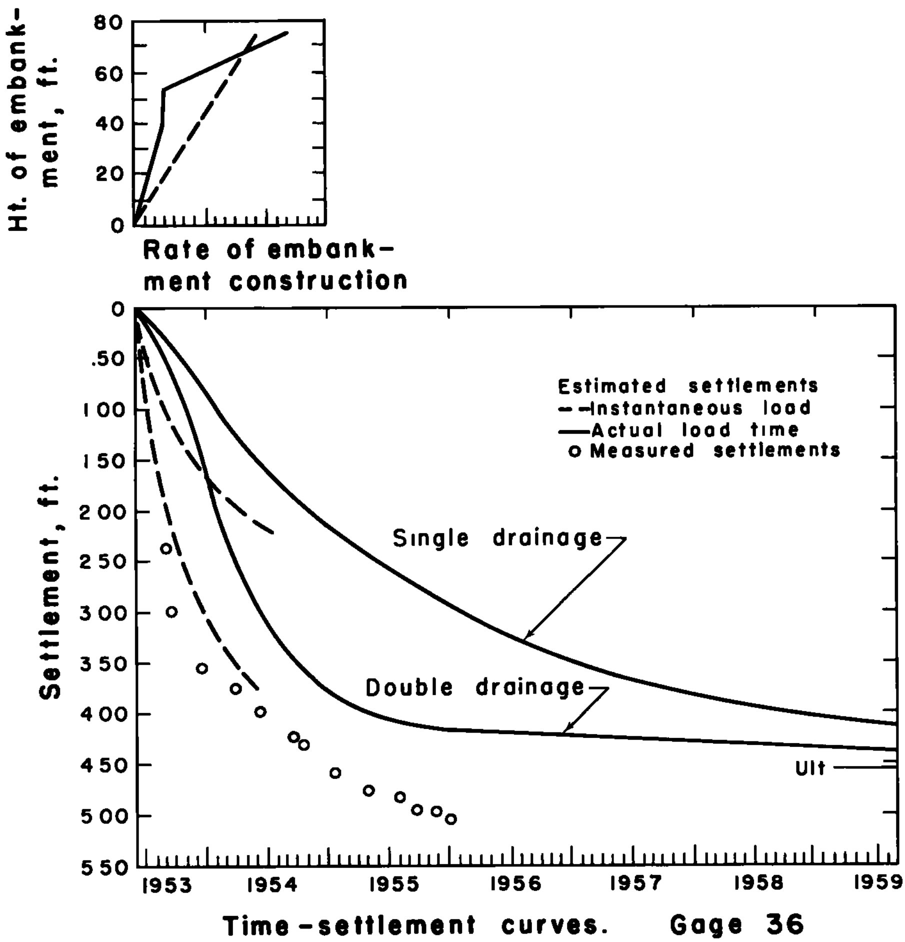


Figure 20.

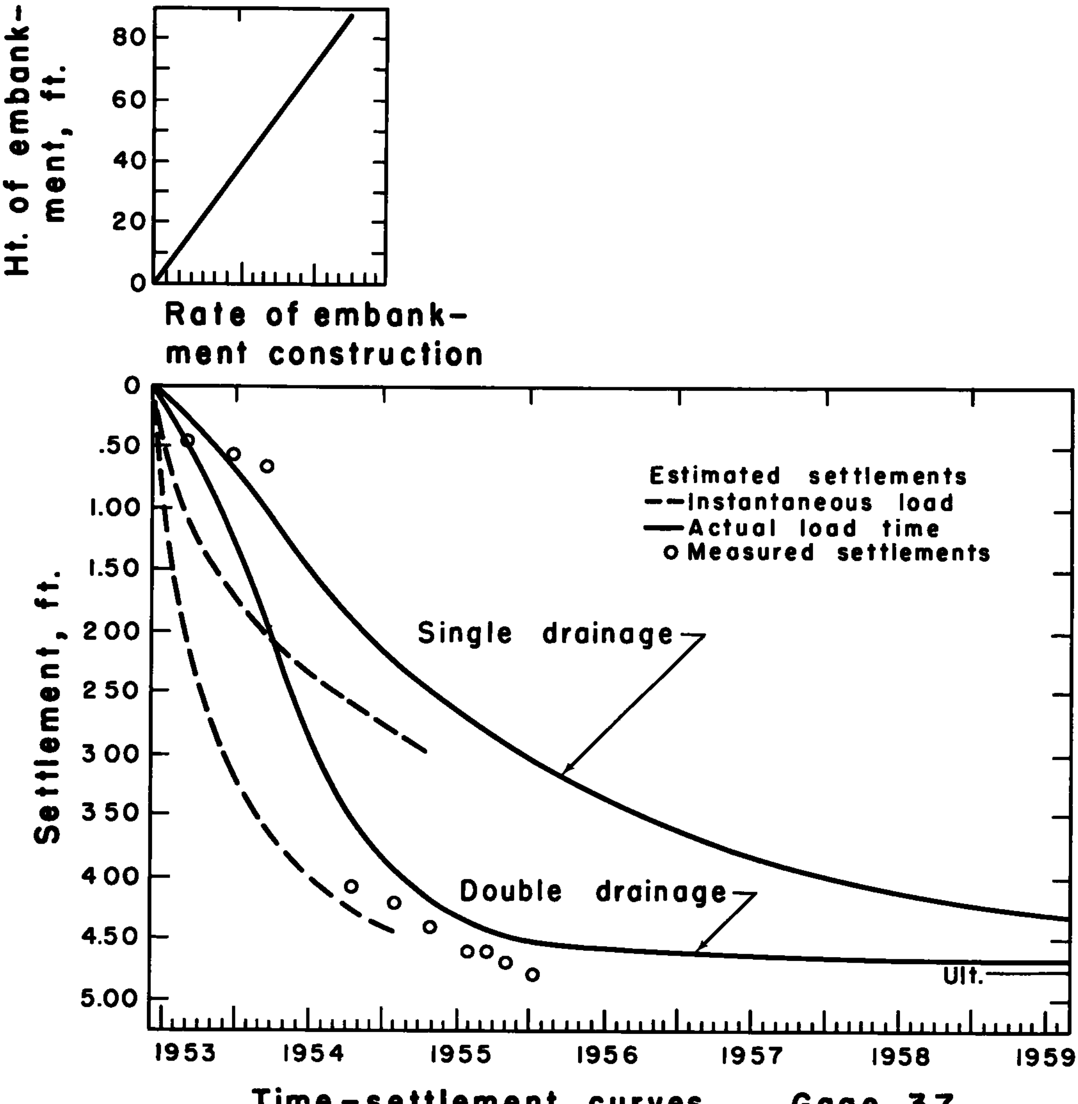


Figure 21.

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

$$p_2 = p_1 + p_c \qquad (3)$$

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.
			143, P-9		107	
22 + 50	31, 32	4.0 to 6.5	143, U-5	4, 8	49	78
			143, P-9		157	
30 + 00	33, 34	3.3 to 7.0	143, U-5	2,4,8	55	106
			143, P-9		205	
30 + 00	35	3.3 to 4.4	143, U-5	2, 4	61	133
40 + 00	36, 38	3.4 to 8.4	142, P-10	2,4,8	211	211
			141, P-3		85	
48 + 00	16, 17	1.7 to 7.1	141, P-5	2,4,8	125	105
			141, P-3			
48 + 00	18	1.3 to 4.7	141, P-5	1,2,4	134	168
					203	
48 + 00	39	1.2 to 2.1	140, P-6	1.2,2.5	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	Single	Settlement, ft Gage 31	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

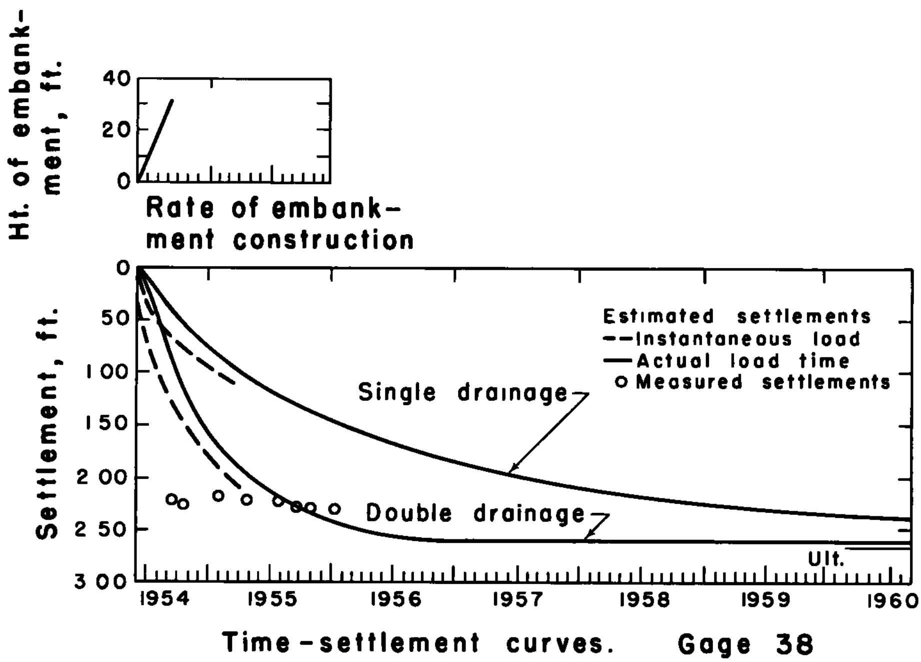


Figure 22.

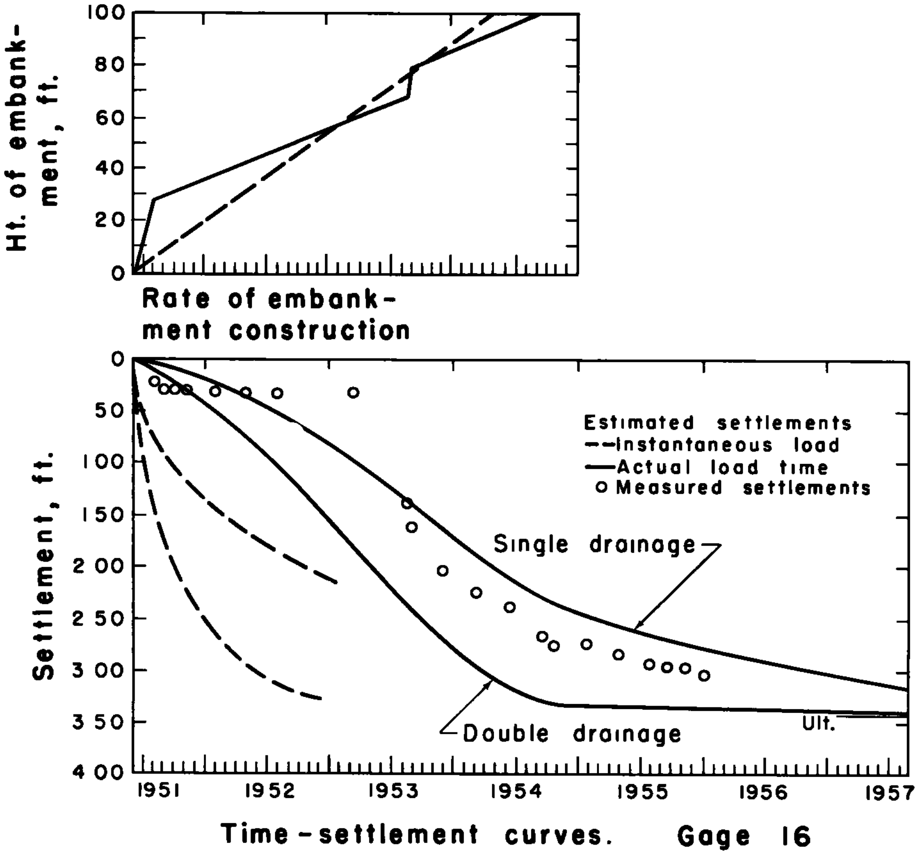


Figure 23.

The final void ratio, e_3, 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

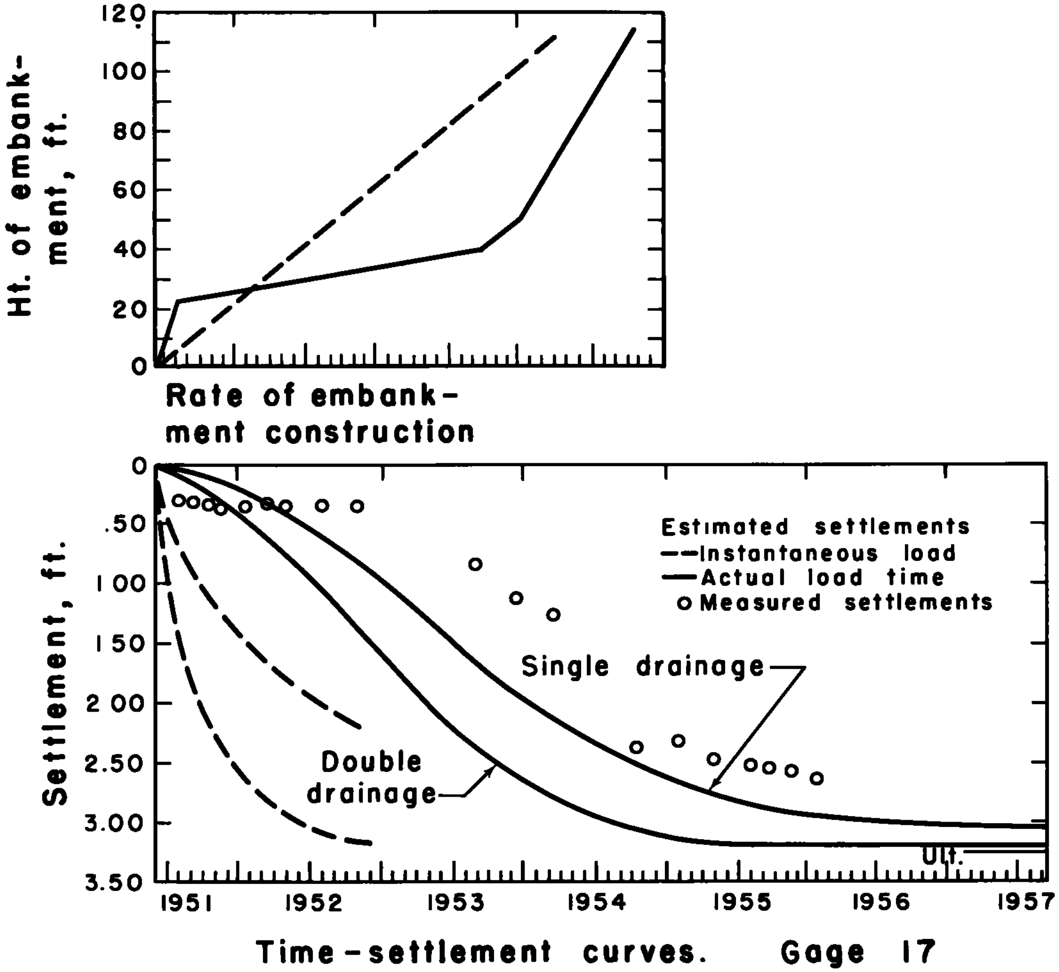


Figure 24.

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

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

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

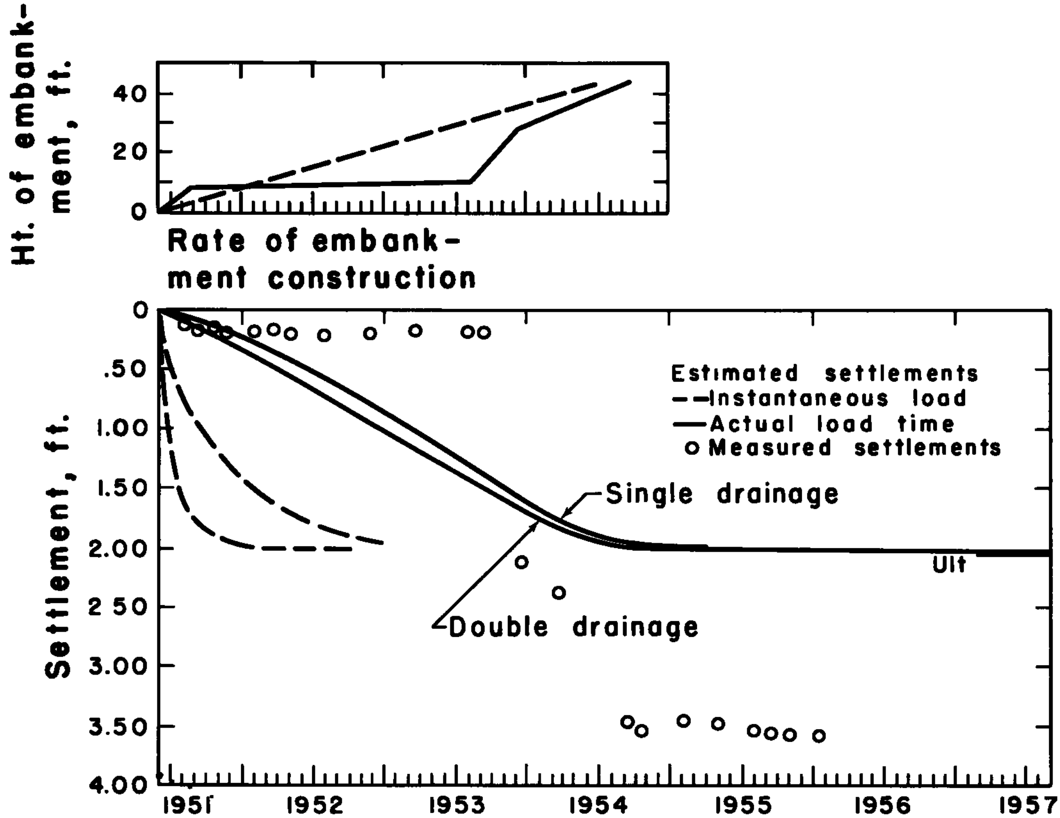
For single drainage

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

in which

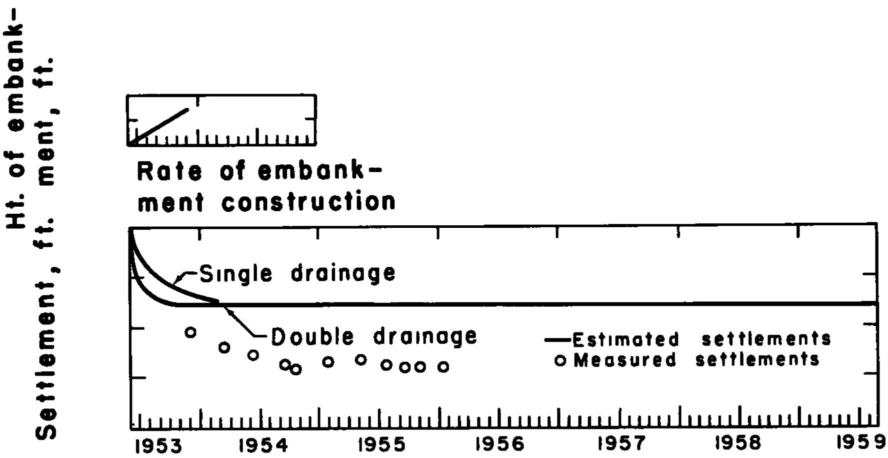
t = elapsed time in which a specified percentage of ultimate settlement develops
d = thickness of compressible layer
c = coefficient of consolidation
T = the "Time Factor" in the Terzaghi equation

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.



Time-settlement curves. Gage 39

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.