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DEPARTMENT OF TRANSPORTATION

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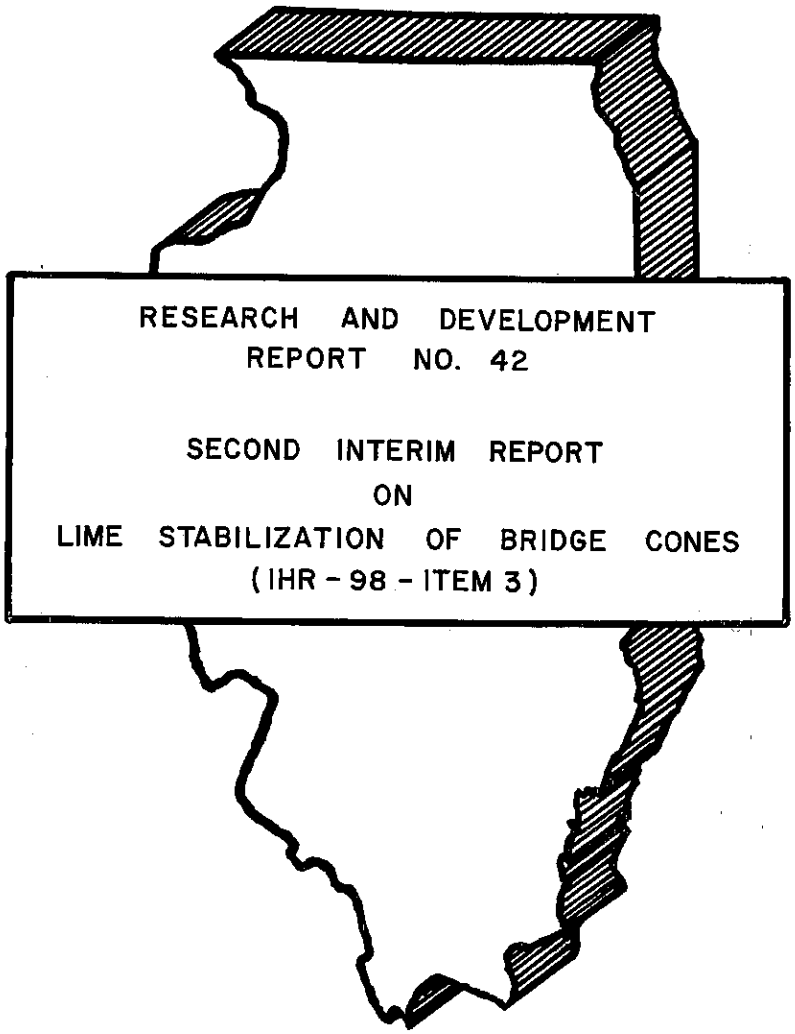

RESEARCH AND DEVELOPMENT REPORT NO. 42

SECOND INTERIM REPORT ON LIME STABILIZATION OF BRIDGE CONES

(IHR - 98 - ITEM 3)

RESEARCH & DEVELOPMENT ILLINOIS DIVISION OF HIGHWAYS

SPRINGFIELD, ILLINOIS 62764 — MARCH 1972

State of Illinois
DEPARTMENT OF TRANSPORTATION
Bureau of Research and Development

SECOND INTERIM REPORT
ON
LIME STABILIZATION OF BRIDGE CONES
IHR-98 - Item 3

by

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A Research Project Conducted by
Bureau of Research and Development

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LIME STABILIZATION OF BRIDGE CONES

A B S T R A C T

This is the second interim report on a study of the use of lime stabilization to reduce embankment settlement at bridge approaches. The study was initiated to determine the value of lime stabilization of fill material as a means for minimizing embankment settlement by adding strength to resist internal movement and to more uniformly distribute the weight of the embankment to the underlying subsoils.

The first report described the construction of the first installation and presented the preliminary settlement data compiled during the construction period. This report covers additional settlement data compiled at the first test site after the approach slabs were placed, describes the construction and instrumentation of the second trial installation, and presents preliminary settlement data taken at the second site.

Because of the relatively small settlement taking place beneath and within the stabilized and unstabilized embankments at both test sites, no conclusions can be drawn regarding the effectiveness of the lime treatment in reducing the settlement of the approach pavements. There is some indication that the settlement within the stabilized embankment at the second trial installation was less than the settlement of the unstabilized embankments; however, more data are needed to confirm this apparent trend.

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LIME STABILIZATION OF BRIDGE CONES

INTRODUCTION

This is the second interim report on a study of the use of lime stabilization for reducing the settlement of pavements at bridge approaches. The bump resulting from this settlement often creates an unpleasant and sometimes unsafe driving condition which has drawn wide criticism from the traveling public. It is also a problem which requires extensive maintenance to correct before the situation becomes hazardous. This study was undertaken to determine the value of stabilizing embankments with lime to alleviate the settlement problem occurring at bridge approaches.

Because bridge foundations are designed and constructed as essentially unyielding supports, the irregular pavement surface at bridge approaches almost always can be attributed to the settlement of the earth embankments immediately adjacent to the bridge structure. Although some settlement may take place even within a properly compacted embankment, most is believed to occur within the natural soils underlying the fill. It is possible that poorer embankment soils induce nonuniform loading conditions on the underlying soils to create uneven and locally excessive settlements.

This study was conceived on the premise that the use of lime treated fill material could add strength to the embankment structure to minimize internal movement and to more uniformly distribute the load to the underlying subsoils. A limited field investigation is being conducted at two trial installations to determine the effectiveness of the lime treatment in reducing settlements of approach pavements.

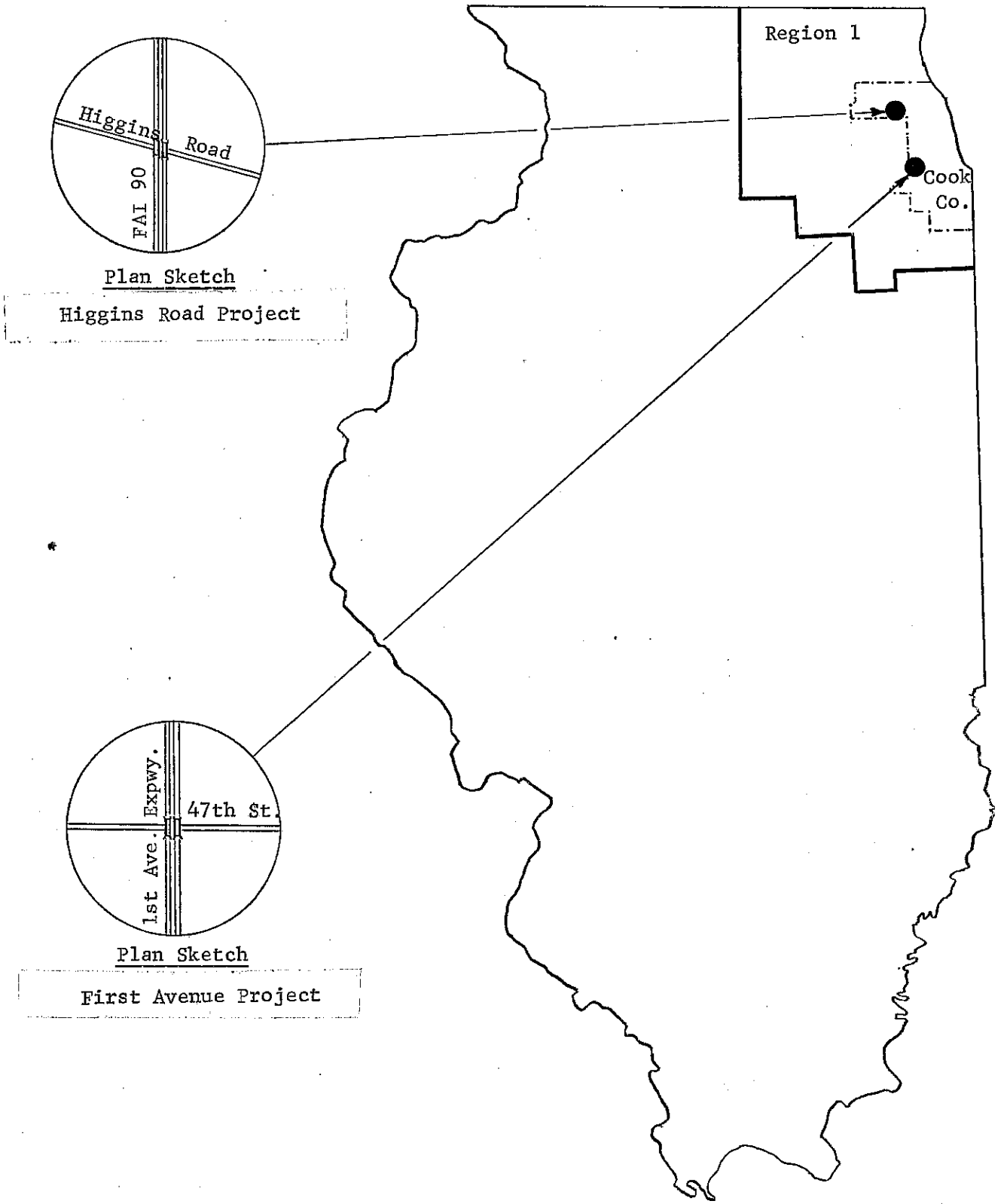


Figure 1. Location of experimental lime stabilized embankments

beneath the embankments. The small changes in measured elevations at both test sites appear to verify the theoretical analysis. At some locations the vertical movements of the approach slabs were upward rather than downward; however, all vertical displacements were relatively small in magnitude. Because the magnitude of the measured vertical movements is small, an effective evaluation of the use of the lime treatment for reducing embankment settlement is difficult. The selection of study locations having greater potential for serious settlement probably would have enhanced the experiment. Within the very limited range of the small settlements that have occurred, it appears that the lime treatment has not had much effect.

Research Objectives

The primary objective of this investigation is to evaluate the effect of lime stabilizing bridge embankments for reducing the settlement of bridge approach pavements. Settlement occurring beneath and within the embankments during and after construction are being measured and analyzed. Periodic measurements of the settlement of the approach slabs also are being made to determine the total effect of the lime treatment for improving the riding quality of the approaches.

The secondary objective of the study is to investigate the feasibility of using longer approach slabs to reduce the effect of differential settlement by increasing the settlement transitioning length.

FIRST AVENUE PROJECT

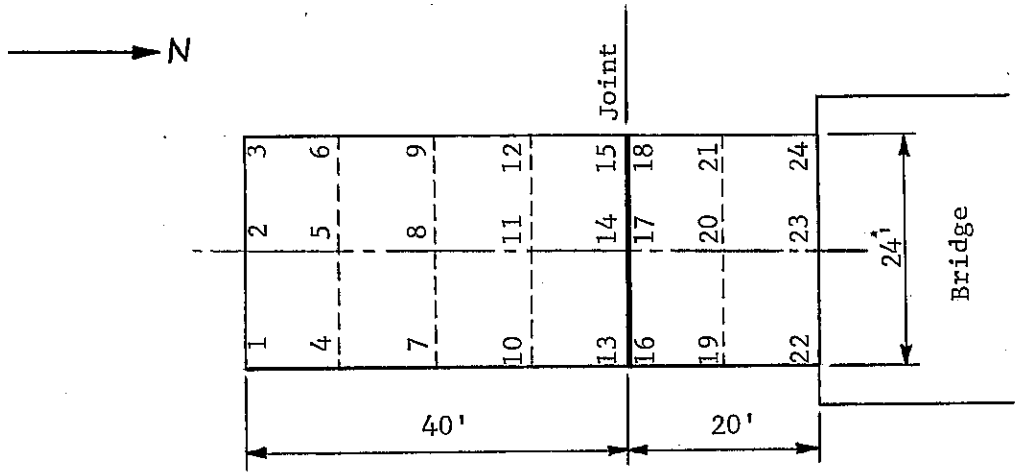
At the First Avenue site, the construction of the stabilized cone began on June 19, 1968 and was completed by September 5, 1968. In the first interim report, settlement measurements compiled during the construction period were compared with theoretical settlements correlated to the rate of embankment construction

for both embankments. This comparison indicated no significant difference in the rates of settlement for the stabilized and unstabilized embankments.

Since the differential settlement causing the rough bridge approaches is due to subsidence of the embankments after the approach slabs are in place, the collection of settlement data after completion of the embankments was deferred until the approach slabs were constructed. By October of 1969 all approach slabs were completed and the collection of settlement data was resumed.

Elevation measurements of the approach slabs were taken quarterly for two years from October 1969 to October 1971. Using the October 1969 reading as a base elevation, the cumulative variations from this reference are presented in tabular form in Tables 1 thru 4.

Table 1 indicates that the approach slabs at the stabilized embankment moved upward slightly over the two-year period with the majority of upward movement at most points occurring between October 28, 1969 and December 3, 1969. Shortly after this time period, observations indicated that the shoulders at this embankment had heaved with respect to the approach pavements. The observed heave of the shoulders and the recorded upward displacement of the approach slabs are not believed to have been caused by the lime added to the embankment. During installation of the small settlement rods on top of the embankment beneath the shoulders, excavation through the gravel subbase already in place was necessary. At this time the thickness of the subbase was observed to be nonuniform, with the bottom of the subbase varying from 10 to 20 inches below the top of the stabilized shoulder. Pockets formed by the nonuniform subbase may have prevented free drainage of water from the subbase, and the subsequent freezing of the trapped water may have caused the observed heave in the shoulders and approach slabs.



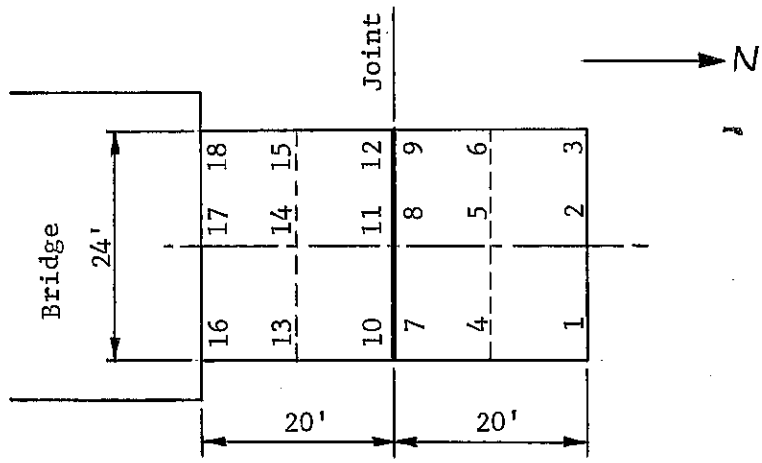
Location of elevation reference points.

TABLE 1.

VERTICAL MOVEMENTS OF SOUTHEAST APPROACH SLAB AT FIRST AVENUE
(Stabilized Embankment)

Date	10-28-69	12-3-69	3-24-70	6-29-70	9-23-70	3-24-71	6-10-71	10-19-71
Point	Elevation	Variations in Elevations						
1	633.12	+.01	+.03	+.02	+.02	+.03	+.01	+.02
2	632.95	+.02	+.04	+.03	+.03	+.04	+.02	+.03
3	632.67	+.04	+.04	+.05	+.06	+.03	+.07	+.07
4	633.21	0	+.02	+.01	+.01	+.02	0	+.01
5	632.99	+.02	+.03	+.01	+.02	+.03	+.02	+.03
6	632.65	+.03	+.04	+.06	+.05	+.01	+.05	+.05
7	633.24	0	+.02	+.02	+.02	+.01	+.02	+.02
8	632.98	+.03	+.02	+.01	+.02	+.02	+.01	+.02
9	632.64	+.04	+.05	+.06	+.06	+.01	+.05	+.05
10	633.30	+.01	+.03	+.02	+.02	+.03	+.01	+.03
11	633.01	+.01	+.02	0	+.01	+.01	+.05	+.01
12	632.67	+.02	+.03	+.04	+.04	-.01	+.03	+.03
13	633.39	+.01	+.02	+.01	+.01	+.02	+.01	+.02
14	633.04	0	+.04	+.01	+.02	+.03	+.01	+.02
15	632.68	+.03	+.03	+.04	+.06	+.04	+.02	+.04
16	633.38	+.02	+.04	+.02	+.02	+.04	+.02	+.03
17	633.05	0	+.05	+.01	+.01	+.02	+.01	+.01
18	632.68	+.03	+.03	+.05	+.05	+.06	+.02	+.03
19	633.42	+.01	+.02	+.01	+.01	+.07	0	+.02
20	633.08	+.01	+.06	+.04	+.01	+.04	+.03	+.02
21	632.72	+.02	+.03	+.03	+.03	+.03	+.01	+.02
22	633.43	-.01	+.01	+.02	+.02	+.08	+.02	+.02
23	633.05	+.03	+.05	+.04	+.05	+.04	+.04	+.03
24	632.71	+.01	+.01	+.04	+.04	+.04	+.03	+.03

NOTE: - denotes downward movement
+ denotes upward movement



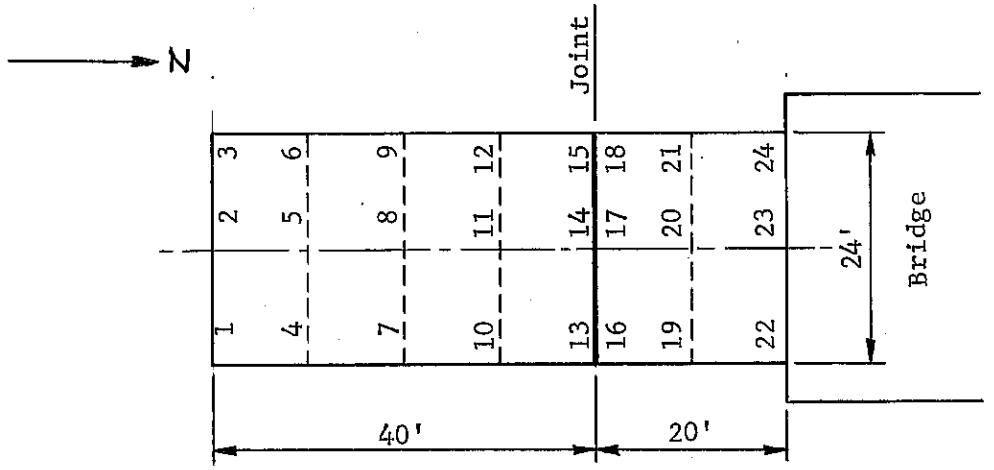
Location of elevation reference points.

TABLE 2.

VERTICAL MOVEMENTS OF NORTHEAST APPROACH SLAB AT FIRST AVENUE
(Unstabilized Embankment)

Date	10-29-69	3-24-70	6-29-70	9-23-70	3-24-71	6-10-71	10-19-71
Point	Elevation	Variations in Elevations					
1	633.23	-.01	0	0	0	0	-.01
2	632.99	+.01	+.01	+.02	+.03	-.02	+.02
3	632.72	+.02	+.03	+.03	+.03	+.01	+.05
4	633.29	0	-.01	-.01	-.02	-.06	-.01
5	633.04	+.01	+.01	0	0	-.06	+.01
6	632.74	0	+.01	+.01	0	0	+.01
7	633.37	-.01	-.02	-.02	-.01	-.05	-.02
8	633.10	+.01	-.02	+.01	+.01	-.01	0
9	632.76	+.01	+.01	+.01	+.02	+.01	0
10	633.37	0	-.01	-.02	-.01	-.02	-.01
11	633.09	+.01	+.02	0	0	0	+.01
12	632.76	+.01	+.01	+.01	+.01	+.01	+.01
13	633.42	+.01	-.01	0	-.01	-.02	-.02
14	633.12	+.02	+.01	+.02	+.02	+.01	0
15	632.78	+.01	+.01	0	0	0	-.01
16	633.46	0	0	0	+.01	0	-.01
17	633.16	+.01	+.01	0	-.02	+.01	0
18	632.79	+.01	+.02	+.02	+.02	+.02	+.02

NOTE: - denotes downward movement
+ denotes upward movement



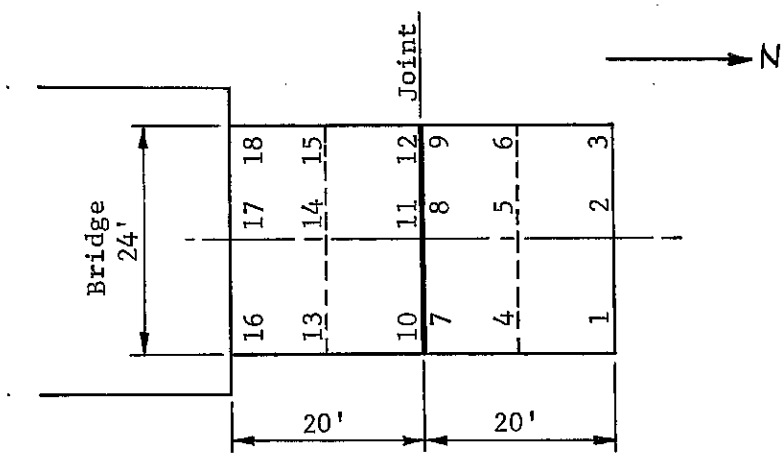
Location of elevation reference points.

TABLE 3.

VERTICAL MOVEMENTS OF SOUTHWEST APPROACH SLAB AT FIRST AVENUE
(Unstabilized Embankment)

Date	10-28-69	12-3-69	3-24-70	6-29-70	9-23-70	3-24-71	6-10-71	10-19-71
Point	Elevation	Variations in Elevations						
1	633.05	0	0	-.01	0	-.02	-.01	-.02
2	632.86	0	+.02	-.01	0	-.01	+.04	-.01
3	632.65	+.02	+.03	+.03	+.04	-.01	+.08	+.01
4	633.07	+.03	+.02	+.01	+.01	0	-.04	0
5	632.88	-.01	+.01	0	-.01	-.02	+.02	-.01
6	632.69	+.02	+.03	+.02	+.02	-.01	+.04	0
7	633.13	0	+.01	+.01	+.01	-.01	-.01	0
8	632.97	-.01	+.02	0	0	-.02	0	-.01
9	632.71	+.02	+.03	+.04	+.05	0	+.04	+.01
10	633.23	0	+.02	+.01	+.01	0	0	+.01
11	632.99	+.02	+.03	+.02	0	+.01	+.01	+.02
12	632.74	+.01	+.02	+.02	+.02	-.01	+.02	0
13	633.33	0	0	-.01	-.01	-.01	-.01	0
14	633.03	-.01	+.02	0	+.01	.0	-.01	-.01
15	632.77	+.03	0	+.02	+.02	0	+.03	0
16	633.32	+.01	+.02	0	0	0	0	0
17	633.04	-.01	+.01	0	0	-.01	-.04	-.01
18	632.77	+.03	+.01	+.01	+.02	0	+.03	+.02
19	633.38	+.02	0	+.01	-.02	-.03	-.01	-.01
20	633.06	0	+.04	+.02	+.01	0	-.03	+.01
21	632.75	+.01	+.02	+.01	+.02	0	+.02	.0
22	633.45	+.02	+.02	+.01	0	-.01	-.01	-
23	633.14	+.01	+.02	-.02	-.01	-.01	+.01	+.01
24	632.77	+.04	+.02	+.02	+.03	0	+.01	+.01

NOTE: - denotes downward movement
+ denotes upward movement



Location of elevation reference points.

TABLE 4.

VERTICAL MOVEMENT OF NORTHWEST APPROACH SLAB AT FIRST AVENUE
(Unstabilized Embankment)

Date	10-29-69	3-24-70	6-29-70	9-23-70	3-24-71	10-19-71
Point	Elevation	Variations in Elevations				
1	633.12	-.01	-.03	-.02	-.01	-.03
2	632.92	-.03	-.02	-.04	-.04	-.05
3	632.65	+.01	0	0	0	-.03
4	633.20	-.02	-.02	-.02	-.02	-.02
5	632.95	-.01	-.01	-.02	-.01	-.03
6	632.69	-.02	-.01	-.01	-.01	-.02
7	633.30	-.01	-.03	-.02	0	-.01
8	633.01	0	-.01	-.01	-.01	-.02
9	632.73	0	0	+.02	+.02	0
10	633.31	-.01	-.03	-.03	-.03	-.02
11	633.02	+.01	-.01	-.01	0	-.02
12	632.73	0	0	+.01	+.01	-.01
13	633.39	-.01	-.03	-.04	-.05	-.04
14	633.09	-.02	-.02	-.02	-.02	-.04
15	632.79	0	-.01	-.01	-.01	-.02
16	633.49	0	-.01	-.02	-.02	-.01
17	633.16	+.01	+.01	0	0	0
18	632.81	+.01	+.01	+.01	+.01	0

NOTE: - denotes downward movement
+ denotes upward movement

Table 2 shows very minor changes in elevation occurred at the approach slabs resting on the unstabilized northeast embankment. The readings indicate that the west edge of the approach slab rose slightly while the east edge settled a small amount. These slight variations in the elevation of the slabs have little effect on the riding quality of the approaches and can be considered negligible.

As shown in Table 3, the vertical movement of the approach slabs resting on the unstabilized southwest embankment was slight and can also be considered negligible.

The approach slabs at the unstabilized northwest cone settled more than the other approach slabs at this site. As shown in Table 4 the maximum settlement of about 5/8 inch occurred near the center of the end approach slab farthest away from the abutment. The settlement at this location is also of minor significance regarding the riding quality of the approach pavement.

In comparing the standard 20-foot approach slabs with the 40-foot slabs constructed at the south embankment, no noticeable difference in behavior has been observed which can be attributed to the difference in length of the slabs. Because of the relatively small settlements occurring at this location, an effective evaluation cannot be made of the use of longer approach slabs for reducing the effect of differential settlement by increasing the settlement transition length.

At this site most of the subsoil settlement was expected to occur within the five-foot clay layer directly beneath the embankment since the soil below this level consists of hard impervious clay. Based on the Terzaghi theory of one-dimensional consolidation, the total settlement of the five-foot layer was

predicted to be about 1-1/2 inches. Time consolidation relationships between the laboratory sample and the field layer indicate that approximately 90 percent of the total consolidation of the field layer would occur in one year.

Field settlement measurements indicate that an average of approximately 1-3/4 inches of settlement occurred within the subsoils underlying both the stabilized and unstabilized embankments during the first year after the embankments were completed. An additional settlement of 1/8 inch was recorded during the following two years. The fact that the measured settlements are slightly larger than the predicted ultimate settlement may be due to a small amount of settlement occurring in the soil beneath the five-foot clay layer underlying the embankments.

Since nearly one year elapsed between completion of the embankments and the placement of the approach slabs, the major portion of the settlement should have occurred before the approach slabs were in place. This explains why little settlement has been observed since the construction of the approach slabs.

In Table 5 the vertical movements beneath and within the embankments are tabulated for the time period after the approach slabs were in place. The movements beneath the embankments are given by Platforms 1 thru 4, while the movements within the embankments are given by Plates 1 thru 4. Platforms 1 thru 4 are located 20, 30, 40, and 60 feet from the back of the abutments, and beneath the east shoulder of the northbound lanes on a longitudinal line about three feet from the edge of the roadway. Plates 1 thru 4 are located 20, 30, 40, and 60 feet from the back of the abutments and in the shoulders about two feet from the platforms with the same number designation.

The tabular values of the vertical movement of Platforms 1 thru 4 at the stabilized south embankment indicate that very little settlement has occurred in the subsoils beneath the embankment since the approach slabs were placed. The

TABLE 5.

VERTICAL MOVEMENTS OF EMBANKMENTS AT FIRST AVENUE

South Embankment (Stabilized)									
Date	10-28-69	12-3-69	3-10-70	3-24-70	6-29-70	9-23-70	3-24-71	6-10-71	10-19-71
Location	Elevation	Variations in Elevations							
Platform 1	633.57	+.03	0	0	0	0	0	0	-.01
Plate 1	633.73	-.01	+.03	+.01	-.01	0	-.02	0	-.01
Platform 2	633.35	+.05	+.08	+.05	+.06	+.06	+.05	+.05	+.04
Plate 2	633.68	-.01	-	+.02	+.03	+.02	-	-	-
Platform 3	633.30	0	+.02	+.01	0	0	0	0	0
Plate 3	633.51	-.01	+.03	+.02	+.01	+.01	+.02	+.03	+.03
Platform 4	633.20	+.01	+.03	+.02	-	+.01	+.01	+.01	-.02
Plate 4	633.38	0	-	+.03	+.02	+.02	+.02	+.03	+.03

North Embankment (Unstabilized)									
Location	Elevation	Variations in Elevations							
Platform 1	633.58	-.02	-.01	-.01	-.01	-.01	0	-.01	-.03
Plate 1	633.62	-.01	-.03	-.03	-.05	-.06	-.06	-.09	-.11
Platform 2	633.29	0	0	0	0	0	0	0	-.01
Plate 2	-	-	-	-	-	-	-	-	-
Platform 3	633.28	+.01	+.01	0	0	0	0	0	-.01
Plate 3	633.22	0	-.03	-.01	-.02	-.03	-.03	-.03	-.04
Platform 4	633.15	0	-.02	0	0	-.01	-.01	-.01	-.01
Plate 4	633.03	0	+.01	+.02	+.02	+.02	0	+.02	+.02

NOTE: - denotes downward movement
+ denotes upward movement

initial elevation at Platform 2, which is buried five feet below natural ground, appears to be in error since the next reading indicates a sharp rise in elevation. An increase in elevation at this location is considered unlikely because of the physical nature of the subsoil underlying the platform.

Plates 1 thru 4 indicate that the top of the stabilized embankment rose approximately 3/8 to 1/2 inch except at Plate 1 where a slight settlement was recorded. This apparent rise is consistent with elevations taken on top of the approach slabs at this embankment. Since the settlement plates were placed just beneath the subbase, the rise at the top of the embankment cannot be explained by the nonuniform subbase. Additional data will be collected for further study of this phenomenon.

The settlement of the subsoil beneath the unstabilized embankment was slight as evidenced by the recorded settlements of Platforms 1 thru 4 at the north embankment. A comparison of the platform settlements at both embankments indicates that the lime treatment had little effect in reducing settlement of the subsoil at this location.

Of the four plates located on top of the north unstabilized embankment, Plate 2 was damaged prior to October 1969 and elevations of this plate could not be made thereafter. The magnitude of the changes in elevation of the remaining plates was small except at Plate 1 located 20 feet from the back of the abutment. Measurements taken at Plate 1 and Platform 1 indicate a settlement of approximately one inch within the embankment proper, which is apparently due to inadequate compaction of the unstabilized material within the vicinity of the reference plates. However, an equivalent amount of settlement was not observed for the approach pavement near the same location since the slab is supported by a pile bent located 20 feet from the back of the abutment.

HIGGINS ROAD PROJECT

The second trial installation was incorporated in a construction project consisting of twin structures carrying the northbound and southbound lanes of Interstate Route 90 over Higgins Road. The two south bridge cones were selected as the experimental embankments incorporating the lime stabilization. The two untreated north cones were constructed by conventional methods.

Instrumentation

The stabilized south cone and the unstabilized north cone of the northbound lanes were instrumented for obtaining comparative settlement measurements. Prior to constructing the embankments, three settlement platforms were seated on top of the natural ground at distances of 20, 40, and 60 feet from a line corresponding to the back of the abutments. A fourth platform was buried five feet below natural ground at a distance of 35 feet from the back of the abutments. All four platforms at each embankment were located on a line parallel to and three feet from the edge of the pavement in the west shoulder of the northbound lanes. Two additional platforms were placed under the south stabilized embankment with one beneath the east shoulder of the southbound lanes, and the other beneath the east shoulder of the northbound lanes (Figures 2 and 3).

All the settlement platforms consist of 3- by 3-ft by 1/4-in. steel plates. Threaded steel fasteners were welded to the center of the base platforms. Reference rods consisting of 3/4-in. high-strength steel pipe were then screwed into the threaded fasteners, and extended in 5-foot increments as the embankments were constructed. Steel pipe was used for the reference rods on this project, because of the frequent breakage of the 3/4-in. black pipe used during the construction of the embankments at the First Avenue project. Although breakage occurred at the second test site, the use of the stronger, more ductile, structural pipe reduced the susceptibility of the reference pipes to damage during construction.

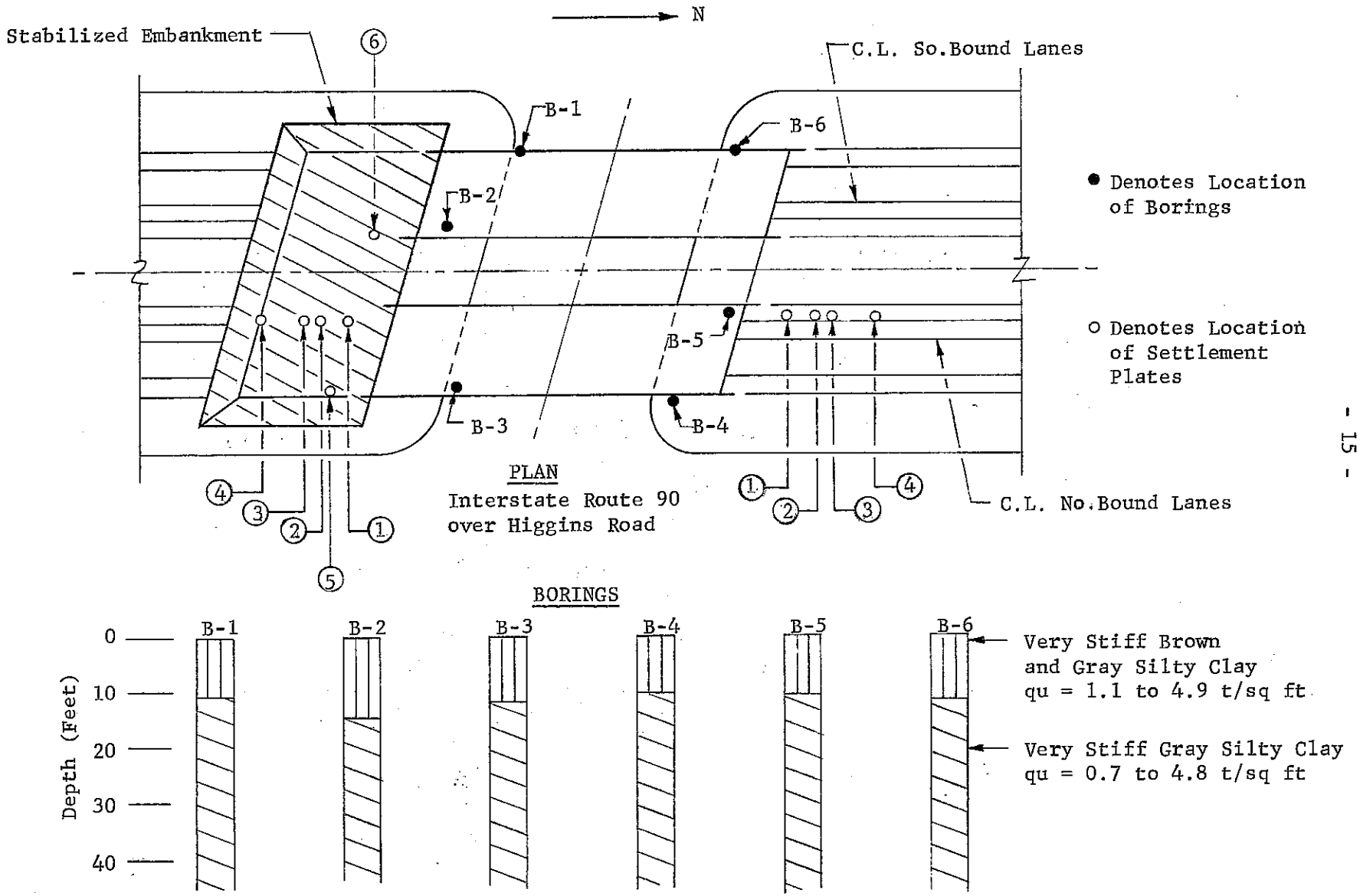


Figure 2. Borings and boring locations at I-90 over Higgins Road

Figure 2. Borings, boring locations, and settlement platform locations at I-90 over Higgins Road

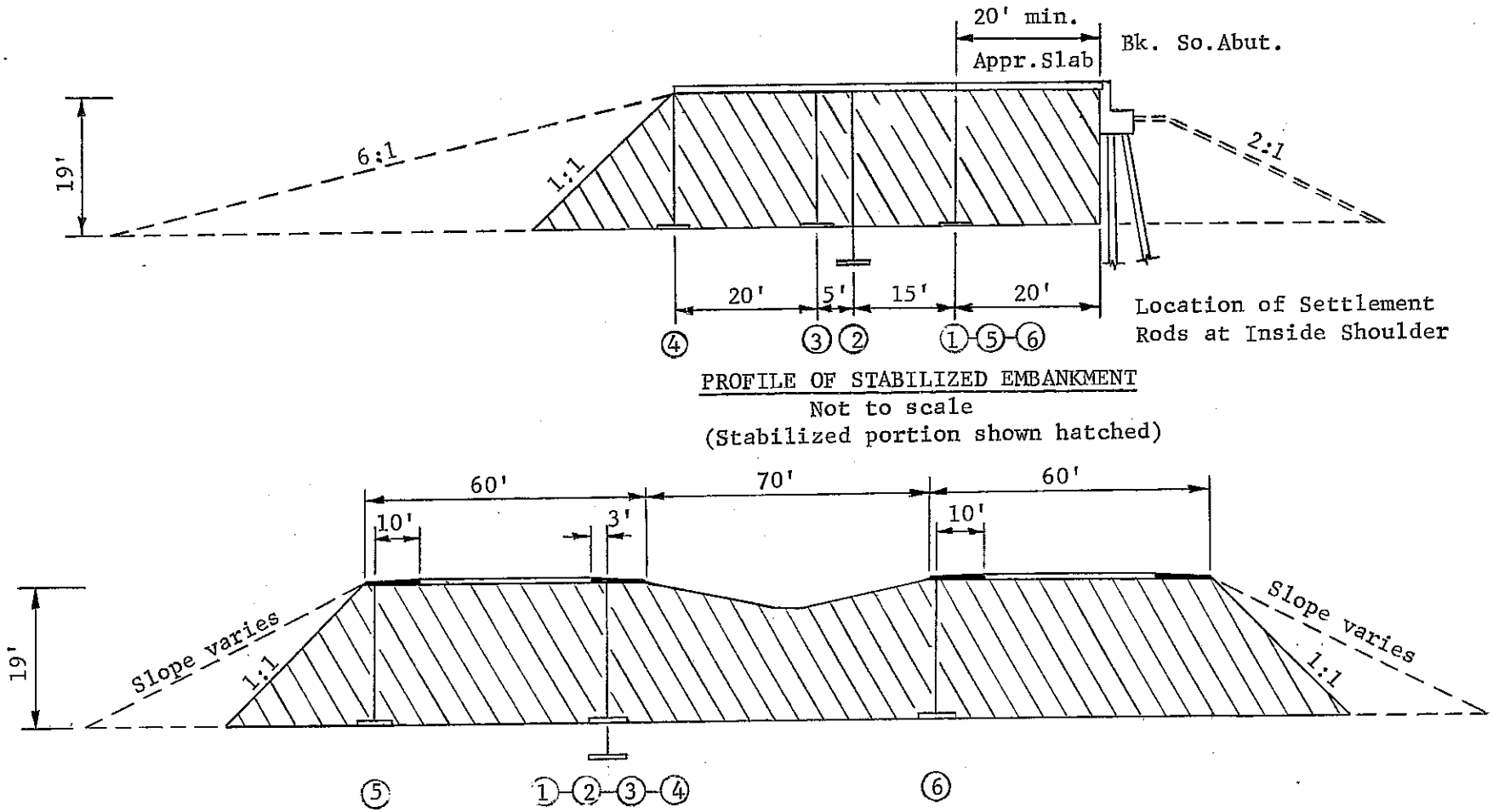


Figure 3. Location of settlement platforms and reference rods within stabilized embankment.

Smaller settlement plates, 4- by 4- by 1/8-in., were placed on top of the embankments directly beneath the paved shoulders in order to measure the settlements within the embankment proper. These small plates were placed within two feet of the pipes extending upward from the larger settlement platforms. Steel rods of 1/4-in. diameter and about 1-ft 4-in. long were welded to the center of the steel plates. The steel rods were then encased by sleeves of 3/4-in. pipe which were cut flush with the top of the shoulders.

Stabilized Embankment Materials

The borrow soil used in the lime stabilized embankment at Higgins Road is classified as A-7-6(16) clay with a moisture-density relationship of 105 lb per cu ft at optimum moisture of 20.0 percent. The natural soil consisted of 52 percent clay, 13 percent sand, and 35 percent silt, and was used in both the stabilized and unstabilized embankments.

At the First Avenue site, the borrow material was classified as A-7-6(15) clay with a moisture density relationship of 107 lb per cu ft at optimum moisture of 19.6 percent. This soil contained 62 percent clay, 8 percent sand, and 30 percent silt.

The borrow material at both trial installations contained a sufficient quantity of clay to be well above the minimum 15 percent clay content specified to insure reaction with the lime additive.

Laboratory tests were conducted using the natural soil from the Higgins Road site to establish moisture-density relations and strength gain of the lime soil mixture. The moisture-density relations were determined for a specimen containing 5 percent lime using a 5.5-lb rammer and a 12-in. drop. From this test, the optimum moisture content of the lime soil mixture was established as 21.4 percent at a dry density of 102.7 lb per cu ft.

Unconfined compression tests were then performed on duplicate specimens containing 3, 4, and 5 percent lime. The cylindrical samples measuring 2 inches in diameter by 4 inches in length were compacted in 3 equal layers by 20 blows of a 4-lb compacting hammer falling 12 inches. The specimens were cured at a temperature of 120°F for 48 hours, and after cooling to room temperature were tested in compression at a constant rate of deformation of 0.05 inches per minute. The results of these tests are shown in Figure 4.

The results of the tests indicate no significant strength gain above a lime content of 4 percent. In order to allow for construction losses and uneven distribution of the lime, the field percentage of lime was established as 5 percent.

In comparing the strength gains of the soil at the two trial installations, it is noted that while a strength gain of approximately 400 percent was achieved at the First Avenue project at a lime content of 5 percent, only a 72 percent gain in strength was obtained at the Higgins Road project with the same percentage of lime added. Considering the similarity of the two soils used as borrow material, the reason for the large difference in strength gain is not apparent at this time. Based on the lower gain in strength achieved at the Higgins Road project, it appears that the lime stabilization would be less effective at this site than at the First Avenue installation.

Construction of Stabilized Embankments

The procedures established for constructing and field testing the stabilized embankment at the Higgins Road project are essentially the same as those procedures used at the First Avenue site. They are presented below as a matter of interest.

(1) Test for moisture content of borrow soil

(2) Initial pulverization of the borrow soil

(3) Addition of lime and initial mixing of the lime-soil composition

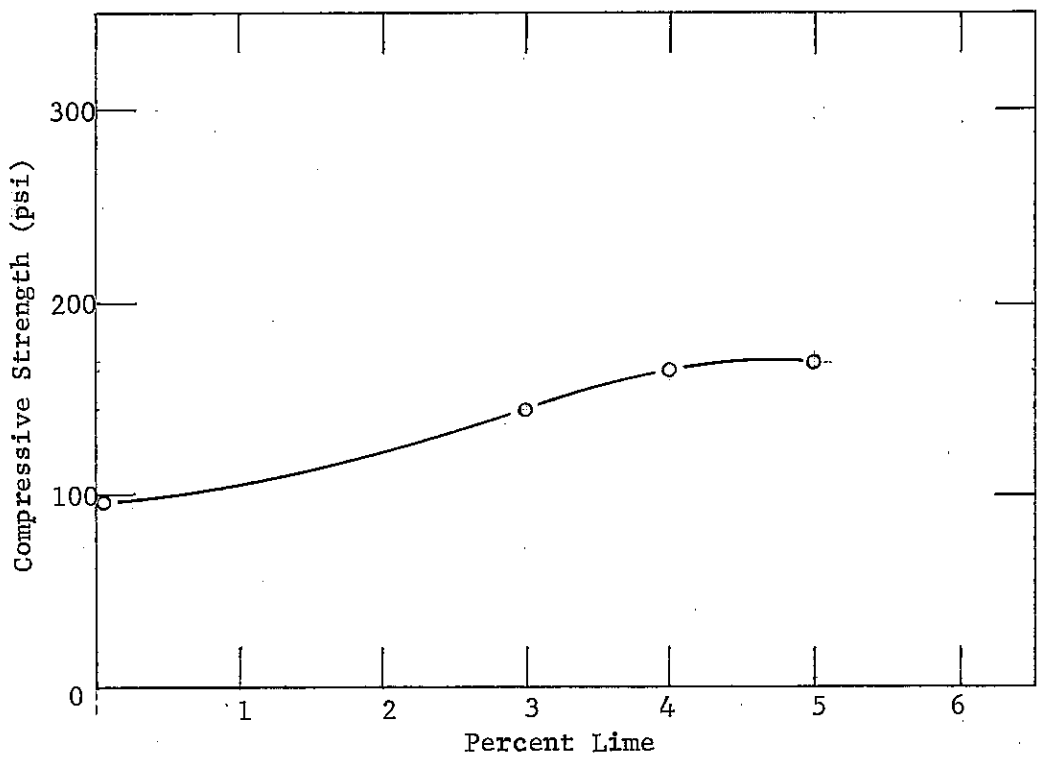


Figure 4. Compressive strength of lime-soil mixtures

(4) Test for moisture content of the lime-soil mixture

(5) Addition of water and final mixing of lime-soil composition

(6) Test for moisture content after final mixing

(7) Test for gradation

(8) Compaction of lime-soil composition

(9) Test for compaction

Table 6 gives a summary of the contractor's daily progress on the stabilized embankment including the quantity of materials used for each lift and the test results of samples taken from each lift.

The procedures used for the construction of the Higgins Avenue project are similar to those used for the First Avenue project, except for certain notable exceptions. Detailed descriptions in this report are limited to the differing procedures.

The original plan for both test sites was to treat both of the south embankments of the dual structures with lime, and construct both of the north cones by conventional methods to serve as controls. The stabilized portion of each south embankment at both sites was to be bounded by a vertical line at the back of the abutment and by a 1:1 slope extending down from each shoulder line and a line 60 feet from and parallel to the back of the abutments.

This plan was altered at the First Avenue project because the original estimate of water for the lime stabilized embankment had been low and the unit bid price for water was unrealistically high. For this reason only the southeast cone was stabilized with lime.

On the Higgins Road project both south cones were treated with lime; however, the original limits for the stabilized portion of the embankments were changed. Early in the project the contractor requested and received permission to extend

TABLE 6.

QUANTITIES AND TEST RESULTS

Date 1970	Lift	Moisture Content of Borrow %	Lime Added lbs.	Gradation Passing No.4 Sieve %	Compaction %	Moisture Content After Compaction %
5/19	0	--	44,400	--	--	--
5/20	1	17.5	44,900	41.0	97.5	17.0
5/21	2	19.0	44,900	50.0	104.0	16.0
5/21	3	18.7	46,100	52.0	109.2	21.0
5/22	4	16.9	47,900	51.4	105.9	17.8
5/23	5	21.0	49,200	50.5	102.0	19.7
5/26	6	18.6	37,600	55.0	106.0	18.6
5/28	7	17.9	42,500	52.0	106.6	16.3
5/28	8	22.7	42,000	58.2	112.2	15.0
5/29	9	24.2	41,900	51.0	98.5	22.8
6/4	10	25.0	36,000	50.3	106.9	20.8
8/15	11	21.0	36,000	54.5	101.8	16.5
8/17	12	19.7	36,000	52.7	105.3	18.5
8/17	13	19.3	40,000	52.7	99.3	18.6
8/20	14	19.1	45,000	56.8	99.6	16.9
8/21	15	25.0	36,000	61.0	99.6	19.5
8/24	16	22.7	40,000	60.0	97.0	16.3
8/24	17	21.2	30,400	60.0	97.9	17.9

the lime treatment to the entire embankment area between the roadway rather than stabilize each roadway embankment independently from each shoulder to natural ground at 1:1 slope.

The original gradation specification for lime treated embankment at the First Avenue Project required at least 60 percent passing a No. 4 sieve before compaction. Without a waiting period for initial curing of the lime soil mixture, difficulty was experienced in meeting this specification. In lime treated embankment construction a curing period of 24 to 48 hours is beneficial before final pulverization. This aging period for the lime soil mixture is uneconomical for continuous multilayer construction confined to the limited area of bridge embankments. To be economically feasible, the rate of construction of the stabilized embankments at this site was estimated to be three lifts per day. In order to attain this rate of construction, the gradation requirement minimum was reduced to 50 percent passing a No. 4 sieve (4.76 mm).

At the Higgins Road project, initial difficulty was experienced in meeting the reduced gradation specification. In attempting to meet this specification one disker and one pulverizer were used by the contractor. Because of the larger area of stabilized embankment at this project, the construction rate of one lift per day was nearly equivalent to the average rate of 2.6 lifts per day finally achieved at the First Avenue project. Therefore, it was believed that the economic attainment of the 50 percent gradation requirement could be made on this project. With the addition of a second pulverizer, the 50 percent gradation requirement was successfully achieved.

At the First Avenue project the embankments were constructed to their final elevation, and excavation of the completed cone was required before constructing

the abutments. This procedure was not used at the Higgins Road project since the contractor elected to build the embankment up to the level of the bottom of the abutments, and then construct the abutments before completing the embankments. The lime stabilized soil was then pulverized and compacted by manual effort at the back of the abutment, as it was near the settlement reference rods.

The stabilized embankment at the First Avenue site was constructed in lifts approximately six inches thick while at Higgins Road the thickness of each lift was about one foot. Phenolphthalein tests and compaction tests conducted in the field at Higgins Road indicate that adequate distribution of the lime was achieved, and that specified compaction requirements were met in the construction of the one-foot lifts.

At the First Avenue project, timber approach piles were used at both the stabilized and unstabilized embankments. The rate of pile precoring through the stabilized material was substantially slower than through the untreated embankment. At the Higgins Road site, approach piles were used at the untreated embankment but not at the stabilized embankment since the fill height and the subsoil properties indicated that no approach piles were necessary.

The First Avenue installation included an evaluation of approach slabs 40 feet in length in comparison with the conventional 20-foot approach slabs. The use of the longer slabs was based on the premise that the additional length would provide a longer settlement transition length, and reduce the effect of the settlement on the riding quality of the bridge approach. At the Higgins Road project, only the conventional 20-foot approach slabs were constructed. They were constructed on a 16° skew, however, so that the length of the short side was 20 feet and the length of the long side was 30 feet. At the north embankment, two approach slabs were

used with their adjoining ends resting on an approach pile bent. At the south stabilized embankments, only one approach slab was incorporated without the use of a pile bent.

Laboratory Analysis

To predict the settlement that theoretically would occur in the subsoils beneath the embankments, a number of consolidation tests were conducted on Shelby tube samples taken at the test site. Two Shelby tube borings were made near the location of both the stabilized and unstabilized embankments. Near the stabilized embankment, one boring was made to a depth of 12 feet and the other made to a depth of 10 feet. At the unstabilized embankment both borings were made to a depth of 12 feet. Settlement which would occur beyond a depth of 12 feet was judged to be negligible.

The samples were contained in Shelby tubes cut in increments approximately two feet long. Specimens for consolidation testing were cut from the center of each two-foot length of Shelby tube to reduce the probability of the specimen drying out before the tests were conducted. A total of 14 consolidation tests were conducted on specimens taken at the unstabilized embankments and a total of 11 consolidation tests were conducted on samples taken near the stabilized embankments. From this number of specimens, the variation in soil properties with depth was determined over a relatively large area near the embankments. The consolidation tests were conducted by the Bureau of Materials in Springfield. The results of the consolidation tests were used to plot void ratio-log pressure curves and time-consolidation curves.

A typical void ratio-log pressure curve plotted from the consolidation test is shown in Figure 5. Point b in Figure 5 is located at the intersection of a line extended from the lower straight line portion of the e-log p curve with the

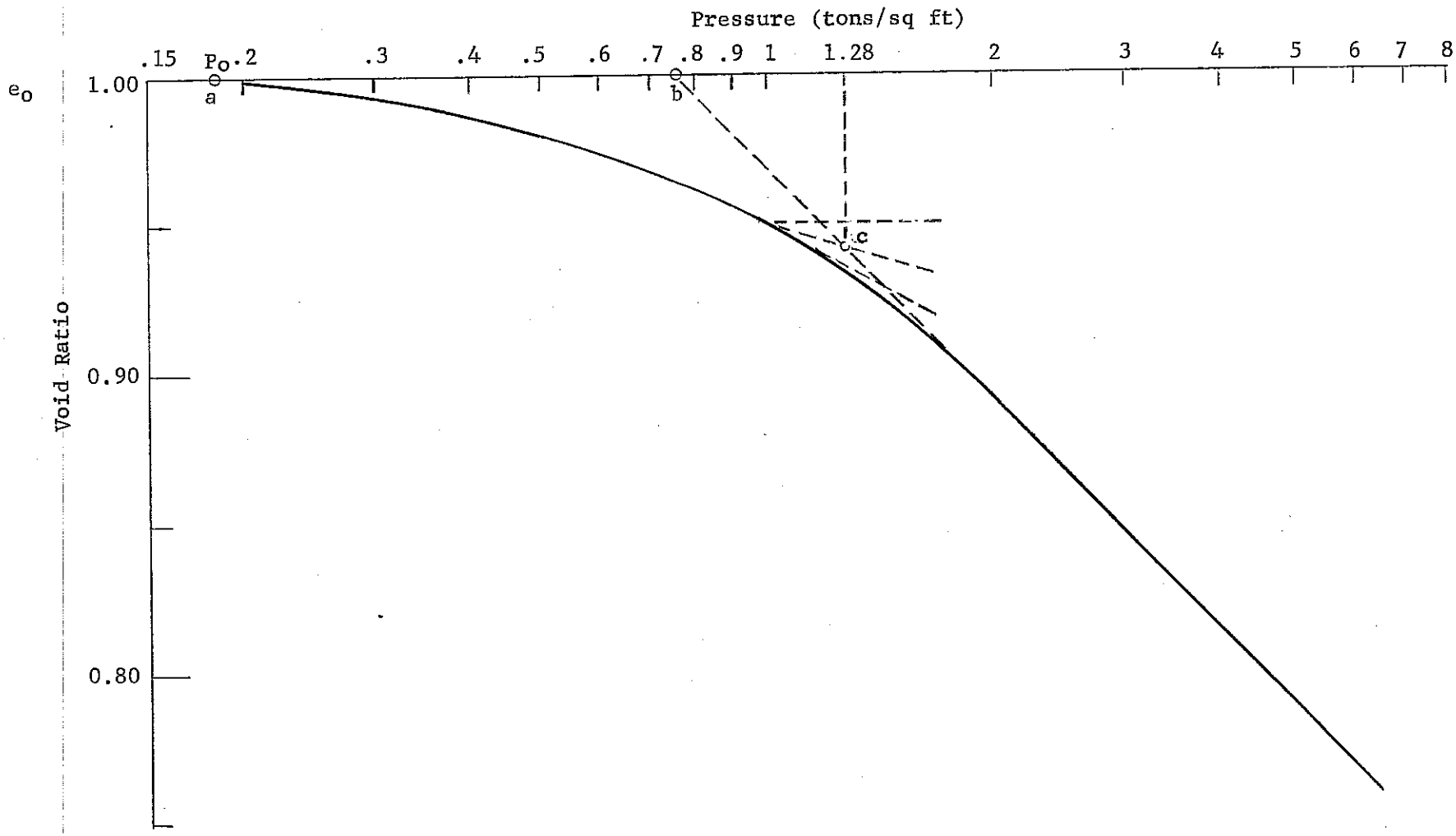


Figure 5. Typical void ratio-pressure curve showing preloaded condition of subsoil.

horizontal axis drawn through e_o which is the void ratio under existing field conditions. The fact that point b falls to the right of point a, which represents the pressure (P_o) under the existing field conditions, indicates a preloaded condition. Evidence of preloading is not unusual in areas where soil deposits once present have been eroded away, or where the soil was previously loaded by a glacier. The preloading effect also occurs where desiccation has taken place. During the process of desiccation, moisture in the soil is gradually lost by evaporation. As the water evaporates, the solid soil particles are drawn into closer contact because of the increased surface tension of the water. Desiccation increases the strength of the soil, and has the same effect as if the soil had been preloaded by a former overburden. Past desiccation of the soil is suspected to be the primary cause of the preloaded condition at this site. Terzaghi (3) states that, in the Chicago area, a thick layer of soft normally loaded clay is covered by a layer of stiff clay which has been precompressed by desiccation.

Procedures developed by Casagrande were used in conjunction with the e-log p curves to estimate the magnitude of the preloading pressure. The manner in which this method is applied is shown in Figure 5. From the point of maximum curvature of the e-log p curve, a horizontal line and a tangent line are drawn as shown dashed in the figure. A line bisecting the angle formed by the horizontal and tangent lines is then extended to intersect the line extending from the lower straight line portion of the e-log p curve. According to Casagrande, this point of intersection lies at a pressure corresponding to the magnitude of the preload pressure.

On this sample curve the magnitude of the preload was equal to 1.28 tons per square foot, which is approximately equal to the pressure exerted by the 19-foot bridge embankments. Analyses of the e-log p curves from all of the samples tested

indicate that the amount of preloading pressure in the area of the embankments ranged from 1.0 to 2.0 tons per square foot. Since the subsoils were preloaded by an amount equal to or greater than the pressure caused by the embankments, little settlement should theoretically occur at this location.

Accurate prediction of the ultimate settlement which would occur in a layer of preloaded clay is extremely difficult. According to Terzaghi, if the settlement of a preloaded clay is estimated with the assumption that no preloading has taken place, the estimated settlements should be in the order of 4 to 10 times greater than the actual settlements.

Using the average value of the compression index as determined from the void ratio-log pressure curves, the ultimate settlement that would occur in the 12-foot clay layer beneath the embankments if no preloading had taken place was found to be about 6 inches. Since the clay had been preloaded the theoretical amount of settlement would be about 1/4 to 1/10 times this amount or from about 0.6 to 1.5 inches.

The time required for the field clay layer to reach a given degree of consolidation is dependent upon whether the clay layer is free to drain at the top and bottom surfaces or at one surface only. Based on the borings taken at the test site, it appears that the 12-foot clay layer would be free to drain at both the top and bottom surfaces. Therefore, double drainage of the clay layer was assumed when computing the theoretical settlements.

Data from the consolidation tests were also used to plot time-consolidation curves for the test specimens. From the time-consolidation curves the coefficient of consolidation of the clay was determined. Using the coefficient, the theoretical time settlement curves were plotted for both the stabilized and unstabilized embankments (Figures 6 and 7).

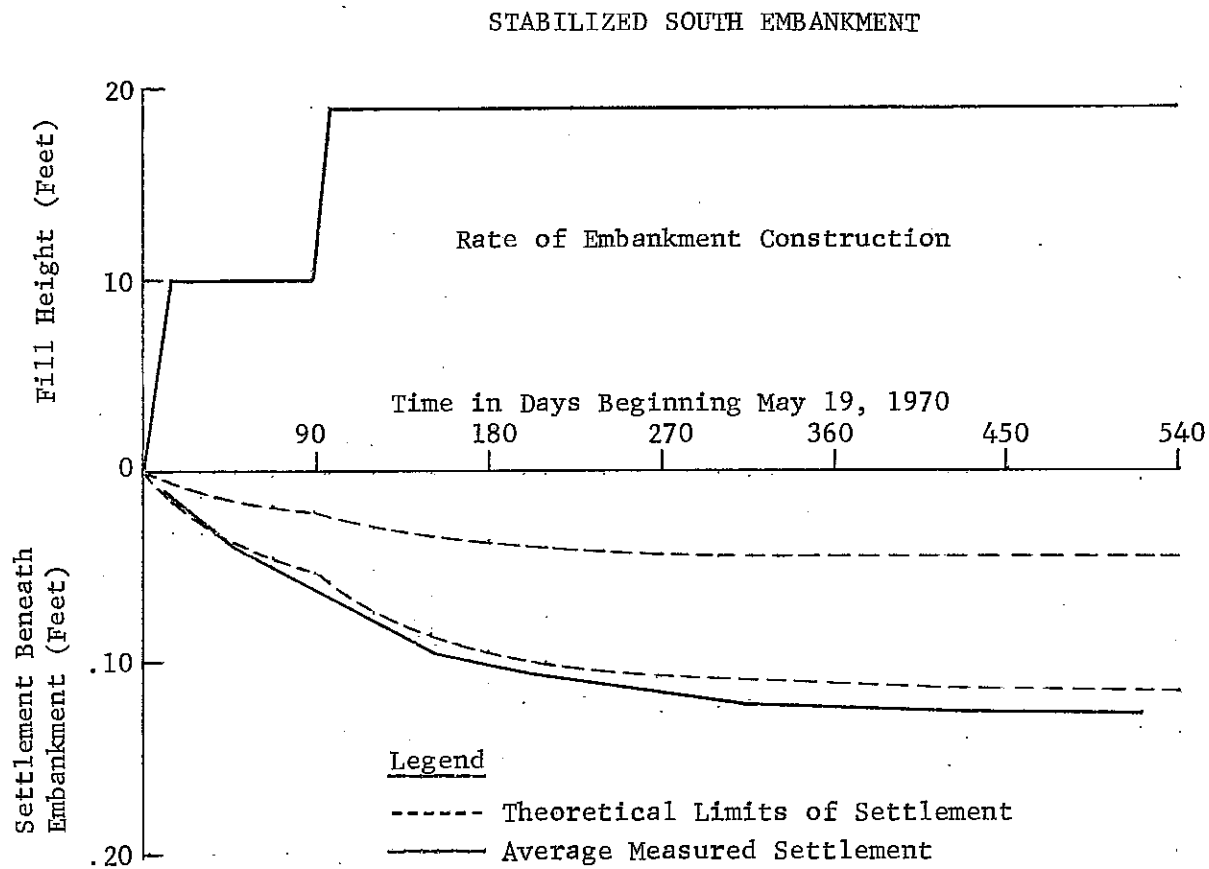


Figure 6. Theoretical and measured settlement of stabilized embankment correlated to rate of embankment construction (Higgins Road Project)

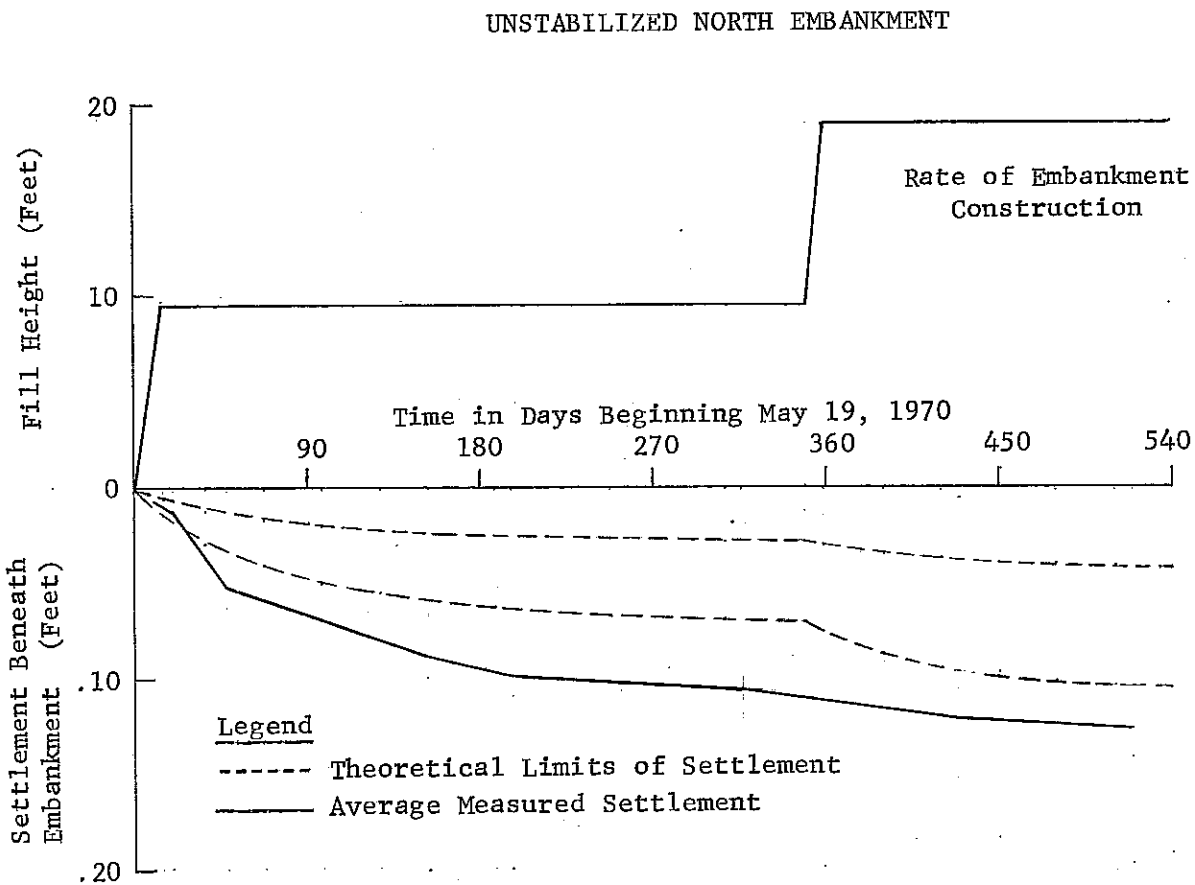


Figure 7. Theoretical and measured settlement of unstabilized embankment correlated to rate of embankment construction (Higgins Road Project)

The theoretical curves for the two embankments differ somewhat because of the difference in the rate of construction of the embankments. Initially, the embankments were constructed almost simultaneously up to the level of the bottom of the abutments. Work on the embankments was then halted until the abutments were constructed. The construction of the stabilized embankments was resumed in less than 90 days, but nearly a year elapsed before the unstabilized embankments were completed. This difference in rate of construction results in a variation in the theoretical settlement curves for the two embankments. However, in both cases most of the settlement would theoretically occur within 540 days after the beginning of construction. The theoretical settlements for this time interval are plotted as dashed lines in Figures 6 and 7 with the upper dash line representing 1/10 and the lower dashed line representing 1/4 of the settlements which would occur had no preloading taken place.

Field Settlement Measurements

As the embankments were constructed, reference pipes were extended from the settlement plates in five-foot increments. Elevations were taken at the top of the pipes as the work progressed. Of the six settlement pipes at the stabilized embankment and the four pipes at the unstabilized cone, two pipes (one at each embankment) were broken and lost during construction operations and, despite efforts to recover them, were never found. The pipes lost were pipe 4 at the stabilized south embankment and pipe 2 (buried plate) at the unstabilized north embankment (Figure 3). The remaining settlement pipes were considered to be sufficient for evaluating the amount of settlements occurring beneath the embankments. Settlement measurements were made at frequent intervals during initial construction work and were made quarterly thereafter.

After the embankments were completed, the settlement pipes were cut and capped in July 1971, by contractor forces without recording the length removed from the top of each pipe. Therefore, the settlement measurements taken after construction was completed could not be correlated precisely with the measurements taken during construction. At the time the settlement pipes were cut and capped, over a year had elapsed since the start of the embankment construction. Therefore, a definite trend of settlement had been established, and most of the predicted settlement had already occurred. The resulting gap in the continuity of the data was filled by estimating the amount of settlement that would occur based on the previous rate of settlement.

Since the settlements varied somewhat from pipe to pipe, the average value of the settlements occurring at all the pipes was used in establishing the measured settlements as plotted by the solid lines in Figures 6 and 7. At the stabilized embankment (Figure 6) the settlements measured for 540 days closely approach the larger limit of predicted settlement. The plotted measured settlements nearly coincide with the lower dashed line representing 1/4 of the settlement that would have occurred had the soil not been preloaded.

The measured settlements at the unstabilized embankment fall somewhat further below the dashed line representing the larger limit of predicted settlement. The magnitude of the maximum difference between the larger theoretical limit and the measured settlement was equal to about 3/8 inch. This is not an unreasonable variation since the methods used to establish the theoretical limits for the settlement of pre-loaded clay give only approximate results.

The trend of the settlement curves suggests that most of the settlement that could be anticipated took place the first year. This probability was also suggested by the theoretical analysis.

Although more settlement occurred at the unstabilized bridge cone than occurred at the stabilized cone relative to the theoretical limits, the slight variation does not indicate a significant difference in the behavior of the two embankments. Subsoils more susceptible to settlement probably would have enhanced the value of the experiment.

SUMMARY COMMENTS

At the present time, no conclusive evaluation of the effectiveness of lime as a stabilizing agent to control settlement of bridge embankments can be made as a result of the experimentation. No settlements of consequence have occurred either where the stabilizing process has been used or where the embankments have been constructed by normal procedures without stabilization.

At the first installation, the impervious nature of the soil underlying the five-foot clay layer directly beneath the embankments indicated that the settlement would be limited to the five-foot layer. From the results of consolidation tests conducted on soil samples taken from the five-foot clay layer, an ultimate settlement of 1.5 inches was predicted. The tests also indicate that approximately 90 percent of this settlement would theoretically occur during the first year after the embankments were completed.

Field settlement measurements indicate that an average of 1 3/4 inches of settlement occurred within the subsoils underlying both the stabilized and unstabilized embankments during the first year after the embankments were completed, and shortly before the approach slabs were constructed. During the two years following the construction of the approach slabs, the average settlement beneath both embankments was equal to about 1/8 inch. The total settlement of the subsoils three years after completion of the embankments was approximately 1 7/8 inches.

The predicted value of ultimate settlement was based on information derived from borings which indicated that most of the subsoil settlement would be limited to the upper five feet of clay beneath the embankments. The fact that the measured settlements are slightly larger than the predicted ultimate settlement may be due to a small amount of settlement occurring in the soil beneath the five-foot layer.

Analysis of the soil underlying the embankments at the second trial installation indicated that the soil had been preloaded some time in the past by a load equal to or greater than the embankment overburden. The approximate ultimate settlement of the preloaded clay at this site was predicted to range from about 5/8 inch to 1 3/8 inches, 90 percent of which would occur within one year after completion of the embankments.

Field settlement measurements taken for 540 days after the beginning of construction indicate that approximately 1 1/2 inches of settlement occurred beneath both the stabilized and unstabilized embankments. This value is reasonably close to the upper limit of predicted settlement of 1 3/8 inches. Very little additional settlement is expected to occur within the subsoils at this site.

The small settlements occurring beneath the embankments do not preclude an evaluation of the lime as a stabilizing agent within the embankments proper. Theoretically, the addition of lime to the borrow material would add strength to the compacted embankment and reduce the settlements occurring within the embankments themselves. At the first installation, settlement data collected quarterly for two years indicate that little settlement has occurred within the stabilized or unstabilized cones. This indicates that the addition of lime does not materially reduce the subsidence of an embankment constructed by proper compaction procedures. The limited amount of data available on the settlement within the embankments at

the second test site does not allow a proper evaluation of subsidence within the embankments proper to be made at this time.

At First Avenue, elevations taken on top of the approach slabs for two years after the slabs were in place indicate that the vertical movements of the slabs have been small. A surprising development at the first installation was the apparent heave of the approach pavements and shoulders at the stabilized embankment as indicated by the recorded elevations of the approach slabs and the visual observations of the shoulders. The heave is not believed to be caused by the lime additive but is possibly due to the freezing of water trapped within pockets in the coarse aggregate subbase which varied in thickness by as much as 10 inches. Although drainage of the subbase was provided, the pockets formed in the subbase were too low to be adequately drained. Swelling of the lime stabilized embankment is another possible explanation for the upward movement of the approach slabs, although the possibility is considered unlikely.

At certain locations the upward movement of the approach slabs was equal to 3/4 inch. The shoulder heaved even more, as evidenced by upward movement relative to the approach slabs, exceeding 1 inch at some locations. Further evidence that the heaving was not caused by the lime treatment is the fact that some heaving of the shoulders took place at the southwest approach which was not lime stabilized.

At the second test site, not enough information is presently available to establish a trend of settlement at the approach slabs, although based on limited data it appears that the approach slabs at the unstabilized cones have settled slightly more than their counterparts at the stabilized embankments. More information on the behavior of the approach slabs at the second test site will be presented in the next report.

At the First Avenue site, a comparison of the 20-foot approach slabs with the 40-foot slabs constructed at the south embankment indicated no noticeable difference in behavior which can be attributed to the difference in length of the slabs. Because of the relatively small vertical movements occurring at this location, an effective evaluation cannot be made of the use of longer approach slabs for reducing the effect of differential settlement.

IMPLEMENTATION

The findings of this research to date have not provided sufficient evidence to indicate that the lime stabilization of bridge embankments materially reduces the settlement of bridge approaches. Therefore no recommendation is made to implement the results of the research at this time.

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