Report No. CDOH-DTD-R-89-8

MONITORING OF NONDURABLE SHALE FILLS IN SEMI-ARID CLIMATES

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Final Report
December, 1989

Prepared in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
CDOH-DTD-R-89-8		

4. Title and Subtitle	5. Report Date
Monitoring Of Nondurable Shale Fills In Semi-Arid Climate	December, 1989
	6. Performing Organization Code 1525A 71.30
7. Author(s) Majid Derakhshandeh	8. Performing Organization Report No. CDOH-DTD-R-89-8
9. Performing Organization Name and Address Colorado Department of Highways Division of Transportation Development 4201 E. Arkansas Avenue Denver, CO 80222	10. Work Unit No. (TRAIS) 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Colorado Department of Highways 4201 E. Arkansas Avenue Denver, CO 80222	13. Type of Report and Period Covered Final 8 Years 14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the U. S. Department of Transportation Federal Highway Administration

16. Abstract

In this research study the performances of three embankments along I-70 in western Colorado were monitored and studied. The embankments were built using the borrow shale material from the adjacent cuts near the roadway. Standard construction procedures were followed with no special shale testing in the field.

During this study, the shale samples from all three embankments were tested in the laboratory, and it was found that all three embankments consisted of nondurable shale material.

The long-term performance of the above embankments indicates that nondurable shales can successfully be used as embankment fill material in semi-arid climates such as Colorado.

Implementation Based on the findings of this study, both the jar-slake and the slake-durability tests are highly recommended to identify the shale quality in the field. Based upon the findings of the tests, recommendations could be made to treat the shale as soil-like in thin (8-inch) lifts or as rock-like in thick (2-3 feet) lifts during the embankment construction.

17. Key Words	18. Distribution Statement
Durable shale, nondurable shale, slake-durability index, jar-slake index, semi-arid climate, embankment	No restrictions. This report is available from the Colorado Department of Highways Technical Reference Center, 4201 E. Arkansas Avenue, Denver, CO 80222

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	52	

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MONITORING NONDURABLE SHALE FILL
IN
SEMI-ARID CLIMATE

I. INTRODUCTION

Shale is often among the most troublesome materials for construction of highway embankments by virtue of its weakness in comparison with other rocks and the possible further deterioration of its low strength over the service life of the embankment. It follows that treatment of shale as rock in embankments can result in costly failures. Conversely, not all shales have such adverse characteristics. Some, instead, may hold up quite satisfactorily over the long term and can be treated as rock. Consequently, fixed conservative design and construction procedures might be unnecessary and costly.

The need for comprehensive guidance on the use of shales in highway embankments and procedures for evaluation and treatment of existing shale embankments led to a 4-year study by the Federal Highway Administration (FHWA) titled "Design and Construction of Compacted Shale Embankments". The results of this study were published in five volumes (references 1 through 5) and they covered areas such as literature review, remedial treatment of shale embankments, design, field and laboratory investigation, and a final report to include the results of the previous volumes.

The above reports are extensive, but contain minimum information on the behavior of nondurable shale fills in semi-arid climates such as Colorado. As a result, this research study was initiated to examine the long-term performance of nondurable shale fills in a semi-arid climate and add to the available inventory of data on this topic.

II. Literature Review

Construction of large and high embankments to complete the modern highway system in much of the United States has required using economically available shales from adjacent cuts and borrow areas. Settlements of 1 to 3 feet in many shale embankments have required frequent overlaying to maintain the original grade. In some instances, raising of bridge abutments founded on approach embankments of shale has also been required. In some shale embankments, continuing settlements are followed by slope failure and slides; while in others, the settlement stops and no further distress occurs. The most severe settlements and slope failures have occurred in the east central states, where the climate is humid. Repair of failures is expensive, amounting to nearly two million dollars, in one case, for three slides where reconstruction was required over period of 18 months.

The primary reason for excessive settlement and slope failures in highway shale embankments appears to be deterioration or softening of certain shales with time construction. Some shales are termed nondurable and are rock-like when excavated, but when placed as rockfill, deteriorate or soften into weak clay soil. On the other hand, some shales, often interbedded with limestone or sandstone, are durable and keep their integrity a long time after completion of construction.

In arid areas, embankments constructed of durable or nondurable shales generally perform well if the embankment material is adequately compacted. But, this may not be the case in the more humid areas where nondurable shale is used as construction material, and no adequate drainage is provided to prevent water from mixing with shale material.

This translates into the fact that the successful use of excavated materials from cuts in shale formations for highway embankments requires adequate compaction of all fill materials and sufficient drainage to prevent harmful saturation of the completed embankment. These two main requirements are often difficult to achieve because of variable stratification of shale formations.

The main difficulty is determining which shales (durable) can be placed in thick lifts (2 to 3 ft.) and which shales (nondurable) must be placed as soil and compacted in thin lifts (8 to 12 in.). Shale formation features in cuts and other borrow areas should be considered early in the preliminary design to assess the need for specifying and the feasibility of controlling selective excavation and separate placement of a durable shale in rockfill lifts (at the base of the embankment and/or outer shells of the embankment) and (b) nondurable shale and soil in thin lifts (or inner sections of embankments). As an alternative, the cost of breaking down all materials during excavation and placement for compaction in thin lifts should be compared with selective excavation and placement to arrive at the best solution. Figure 1 illustrates some of the difficult stratigraphic and shale conditions that require special construction procedures to achieve adequate compaction and drainage.

Gradation requirements for nondurable shales placed as soil should limit large rock sizes and provide adequate fines, while for durable shales placed as rockfill, excessive fines must be limited. For example, if a 10 ft. thick section of thin shale layers in a cut contained about 50 percent nondurable shale, the entire section should be considered soil-like and compacted in thin lifts (8 to 12 in.). In this case, an excessive amount of large shale or hard rock sizes would prevent adequate compaction, as illustrated in Figure 2a. The excessive amount of large rock (upper drawing, Figure 2a) produces a loose and pervious structure. The shale pieces, cracked by stresses at contacts would soften and break down further as water infiltrates down into the completed

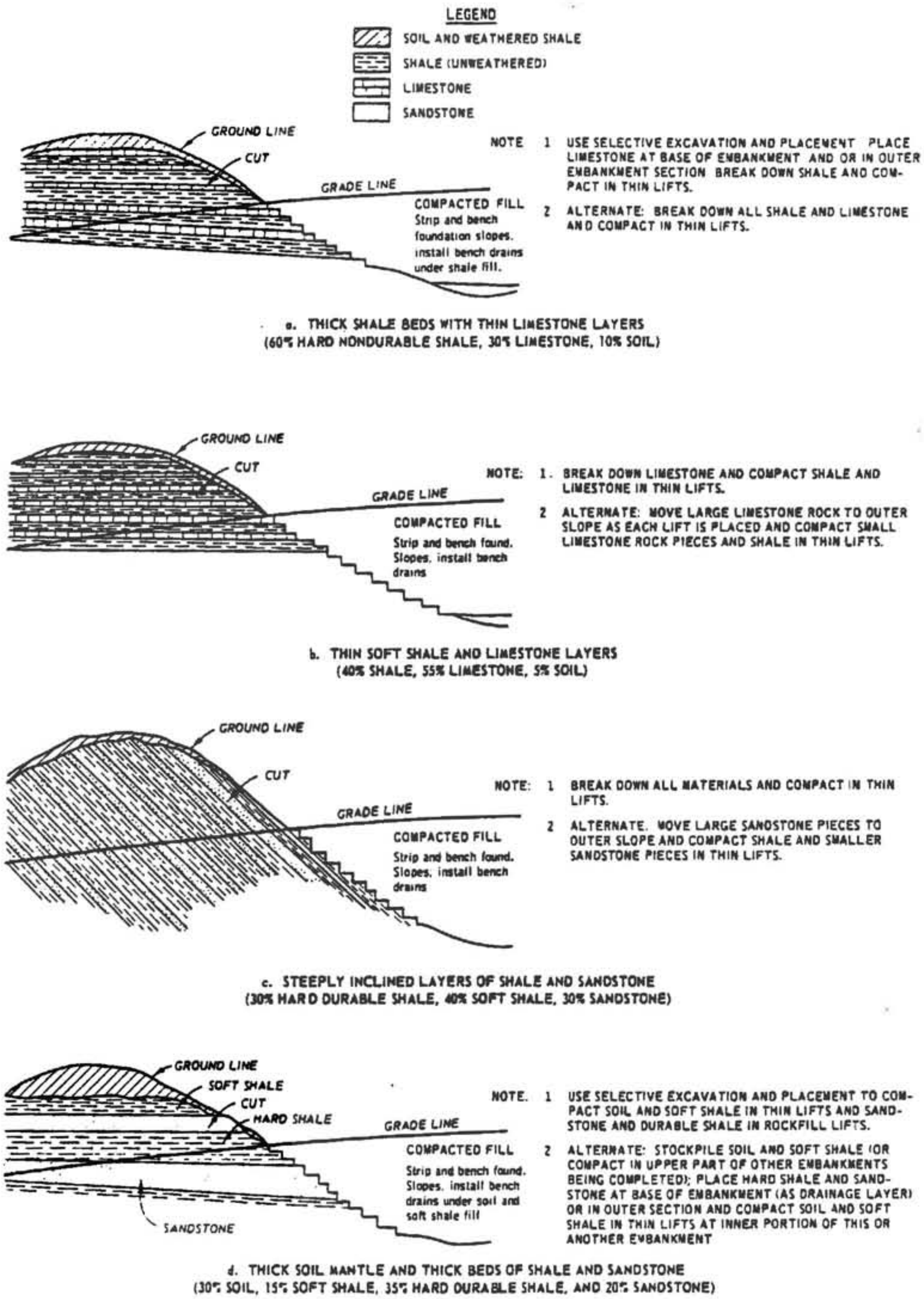


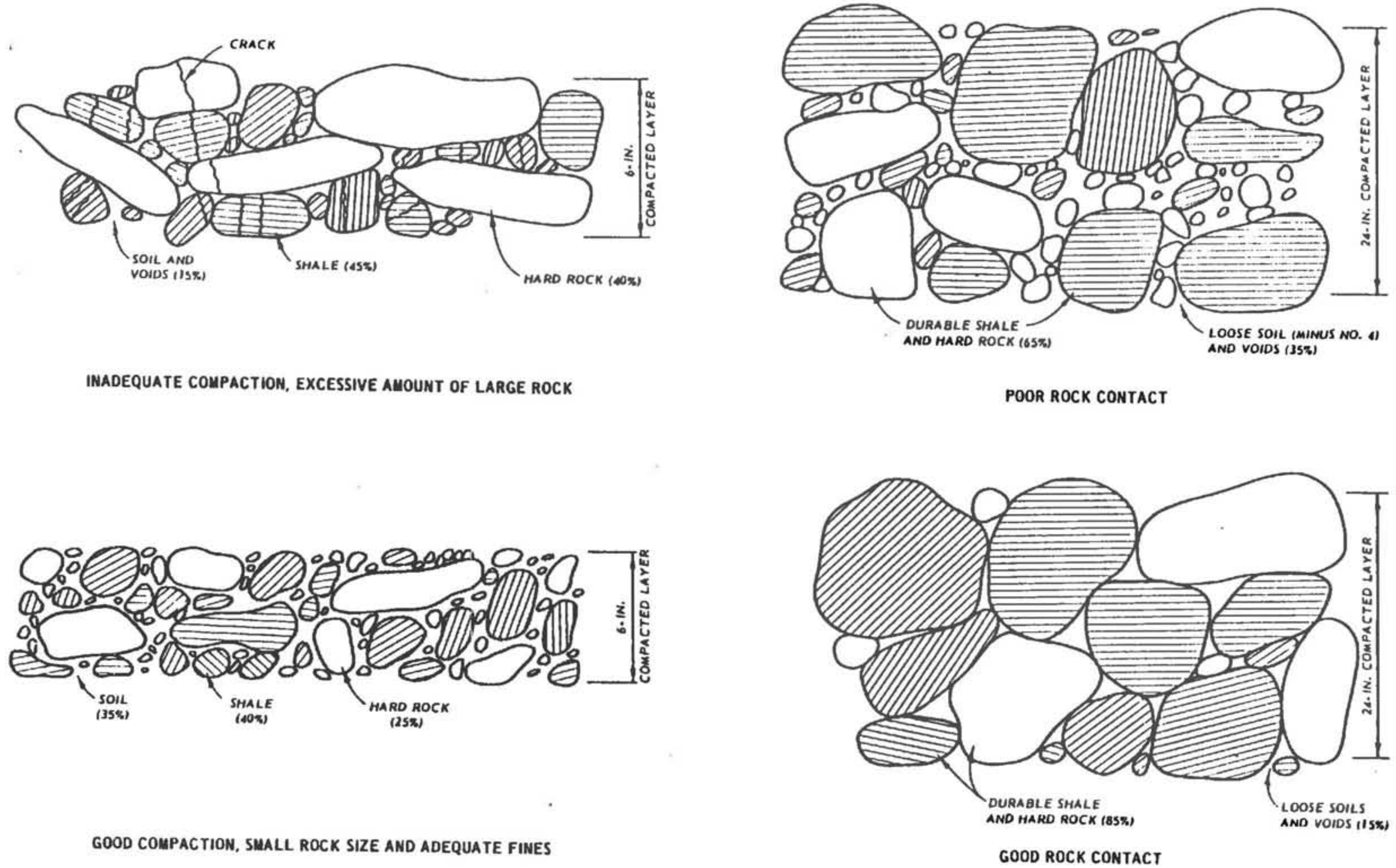
Figure 1. Examples of difficult shale and stratigraphic features and possible solutions

embankment. On the other hand, if the 10 ft. thick section contained 60 percent or more of durable shale, then the material could be used as rockfill. But, in this case, an excessive amount of fine-ground material could prevent adequate compaction between durable rocks, as illustrated in Figure 2b. In the upper drawing of Figure 2b, the loose soil between rocks would soften and deform under infiltrating water, resulting in large settlements. Therefore, the following specifications, as outlined in Reference No. 5, should be considered to prevent post-construction problems in shale embankments.

A. Nondurable Shales As Soilfill

Compaction studies of minus 3-in. earth rock mixtures using an 18-in. diameter mold by Donaghe and Townsend (1976)[6] showed that maximum dry density decreased significantly when the gravel content exceeded 60 percent. As shown in Figure 3, the highest maximum dry density was 138 PCF for 40 percent gravel and 25 percent fines (minus 200 sieve), compared to 135 PCf for 60 percent gravel. When the amount of fines was reduced to 15 percent, the maximum dry density increased to 142 PCF.

Using the gradation curves from the earth rock mixture, and assuming a maximum rock size of 12 in. for an 8-in. lift, a proportional gradation curve is constructed (using the offset distance A, based on the



a. Nondurable shale in soilfill

b. Durable shale in rockfill

Figure 2. Effects of rock size on nondurable shale and of soil (minus No. 4) on durable shale

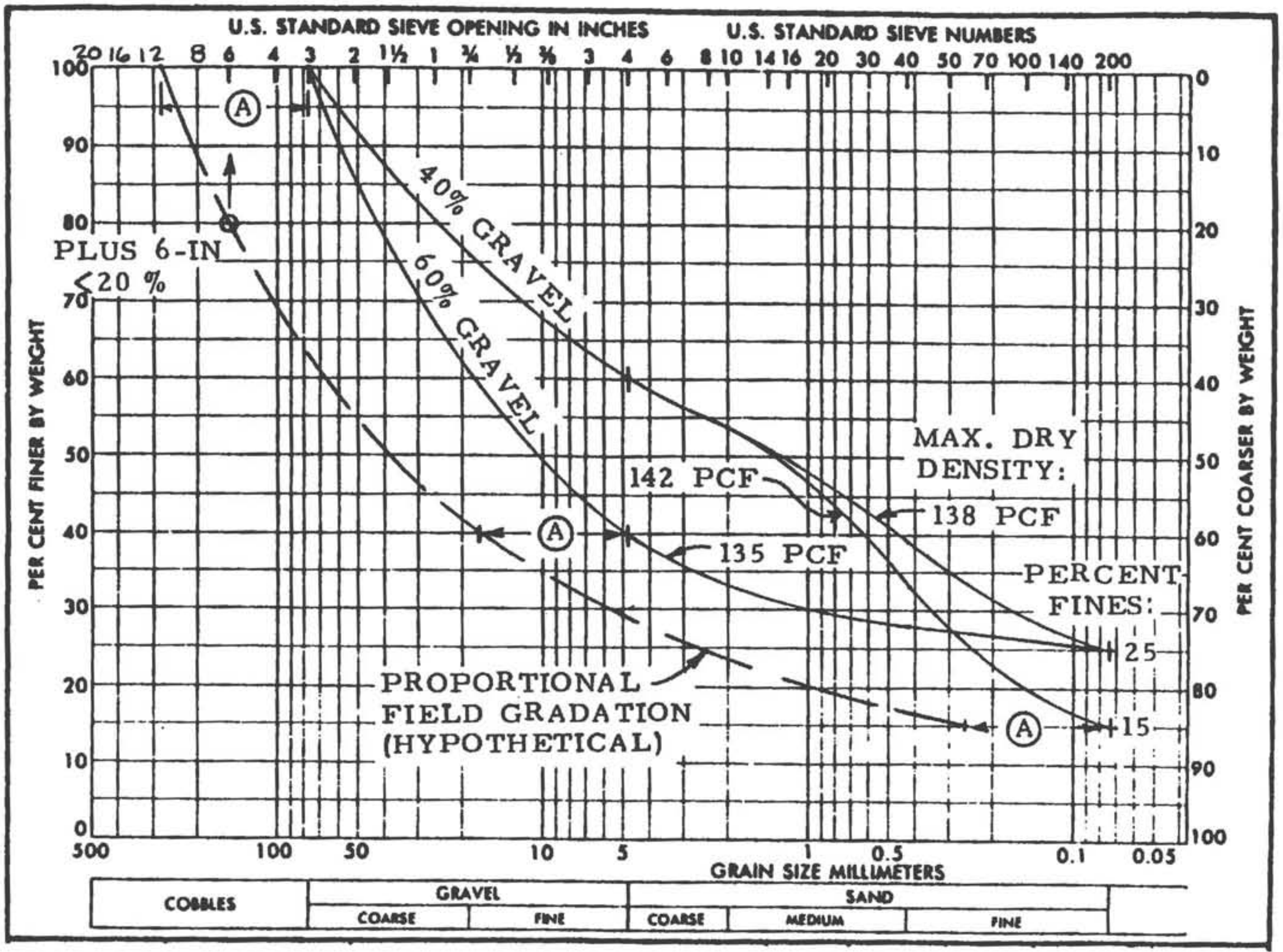


Figure 3, Basis of gradation requirements for nondurable shales used as soilfill difference between the maximum size of 12 in. to 3 in. as shown in Figure 3, for the dashed curve). This proportional curve indicates a well graded material with 20 percent of plus 6-in. size rock. Since a 6-in. size is easily recognized in the field and rocks larger than 12 in. should be prohibited or limited to a smaller percentage, criteria limiting plus 6-in. rock size to less than 20 percent should be used as a minimum. Hard nondurable shales that do not contain sufficient fines may require an additional limitation of about 60 percent on the plus 1-in. size (or a requirements for about 40 percent, minus 1-in. material).

The use of heavy compaction equipment on a rocky mixture of nondurable shale and hard rock shown in upper drawing of Figure 2a would not produce adequate density because hard rock such as limestone would not break down. Conversely, small rock sizes and soil can be well compacted in thin lifts using conventional compaction equipment.

B. Durable Shale As Rockfill

Durable shales and rock used as rockfill require good contact to achieve a stable mass that will not deform or settle. As illustrated in the upper drawing in Figure 2b, the large shale and rock are floating in loose soil. It would be practically impossible to obtain good compaction of the soil even with very heavy equipment. Thus, the loose soil structure would compress and deform with time under infiltrating water and result in large settlements. A much more stable mass of rock and shale is obtained when large pieces are pushed together to form a large number of contacts, as shown in lower picture in Figure 2b. To achieve the desired clean rock, the amount of soil (minus No. 4) should be limited to not more than 20 percent for lifts as thick as 24 inches.

III. Project Location and Geology

Three embankments along I-70 in Western Colorado were selected to be instrumented and monitored during this study. Location of these three sites are marked and they are illustrated in Figures 4 and 5.

Site No. 1 was an embankment 50 foot high with a 2 to 1 slope about 1/4 mile east of the Rulison Interchange as shown in Figure 5. The exposed shale on this site is mostly nondurable and it could easily be detected visually. Figure 6 shows the shale formation that was used during the 1980 construction of the "Rulison cut". Note the small monads, especially the one in the lower left quadrant of the photo. These began as hard, dense silty shale rocks (shaley siltstone), but due to weathering and moisture absorption during many years, they have disintegrated into much finer and softer material.

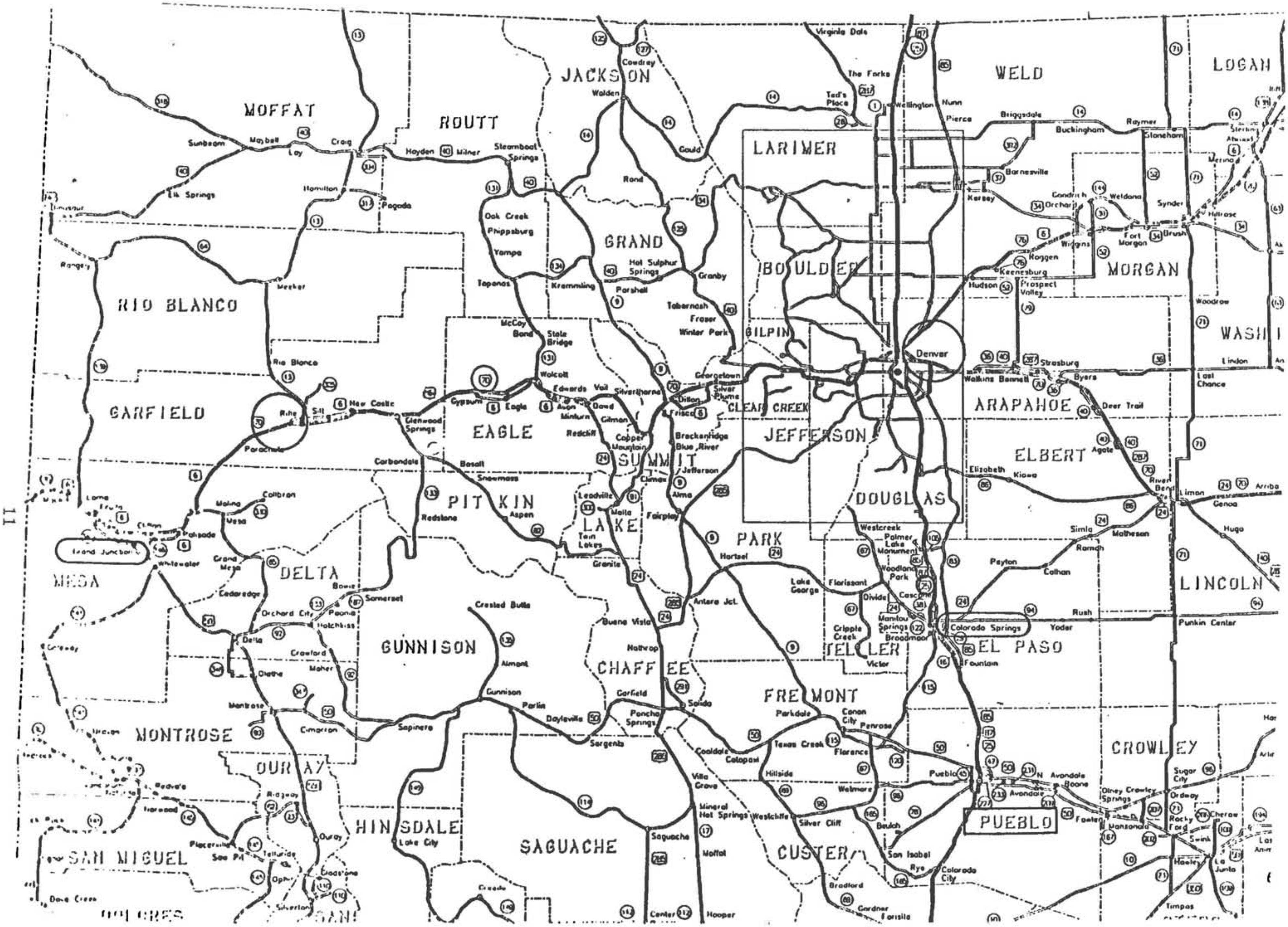


FIGURE 4. GENERAL MAP OF THE STATE OF COLORADO

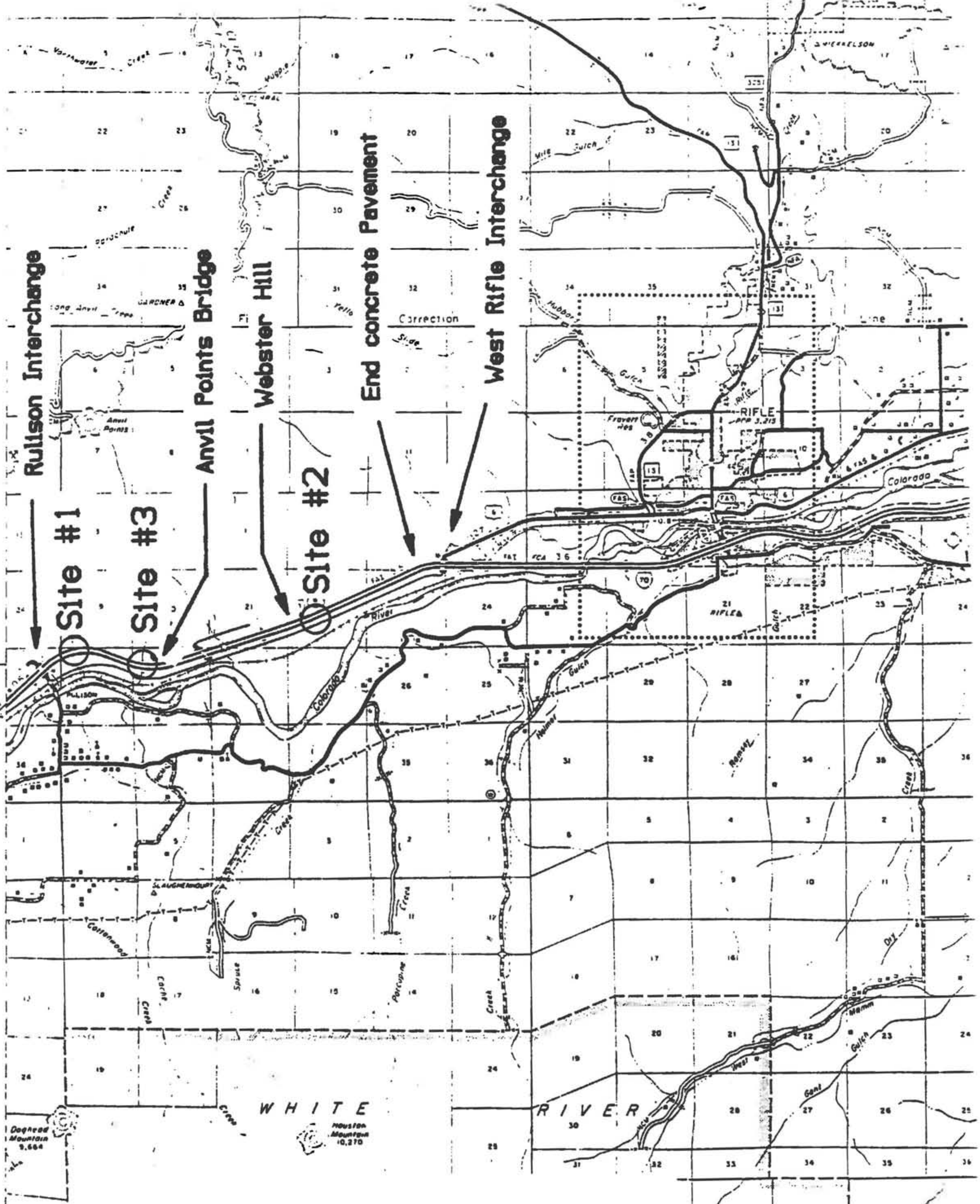


Figure 5. locations of test sites on a county map

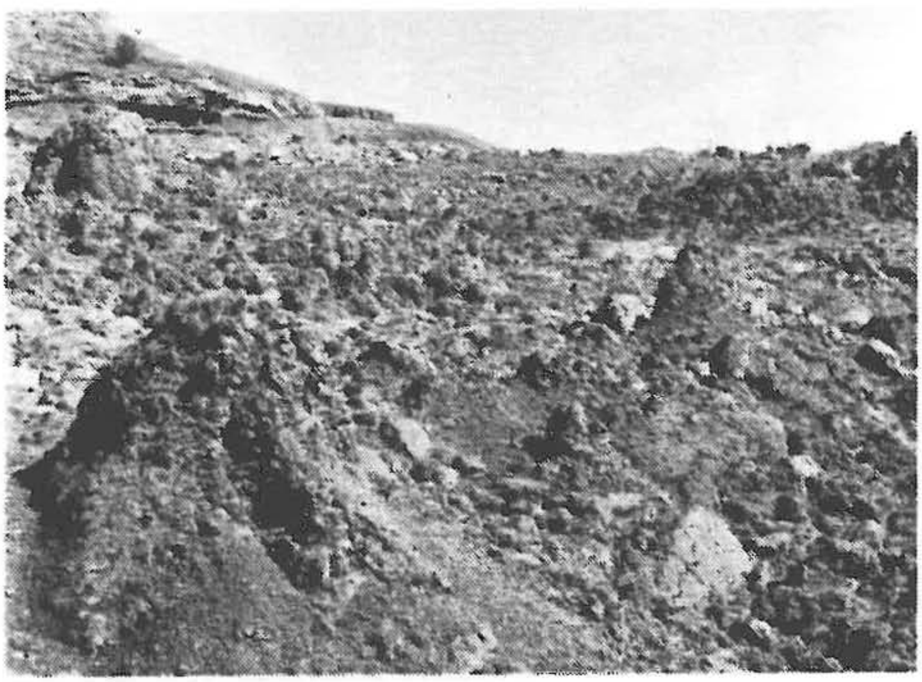


Figure 6. Shale formations near site no.1

Site No. 2 was selected east of the Webster Hill cut, as shown in Figure 7. The embankment on the east side of this hill stretches for more than 2 miles, and its height descends from 100 to 10 feet from the top to the bottom of the hill, as shown in Figure 8. The shales on this site were weathered and had rocklike appearance as shown in Figure 9.

Site No. 3 was selected westbound just west of the Anvil Point Bridges, See Figure 10. The embankment on this location was less than a mile in length and its height varied from 30 to 5 feet from the top to the bottom of the slope. The shales on this location were very similar in appearance and durability to the shales on Site No. 2.

Construction of I-70, west of Rifle to Rulison on the western slope in Colorado was completed in 1980 and early 1981, and nearly three million yards of excavated shale was incorporated into the embankments. The shales were taken from cuts and adjacent borrow areas where they were in an undisturbed natural state.

On each site, at least one hole was drilled to install the inclinometer/Sondex system. Boring logs were obtained to visually categorize the embankment material, and they are illustrated in Appendix A. All three embankments consisted of a mixture of shale and sandstone from the adjacent cut areas.

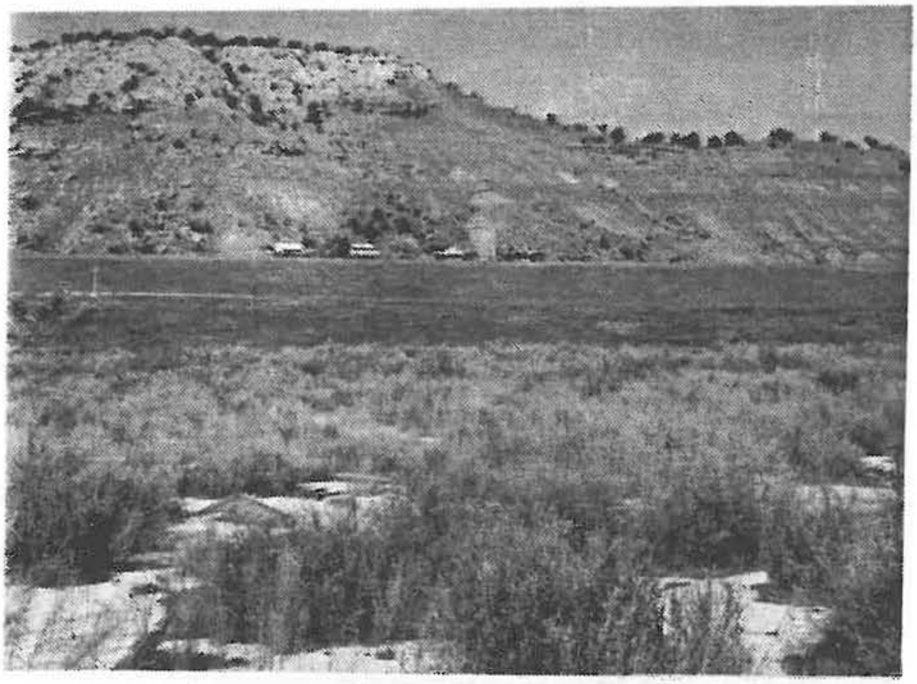


Figure 7.
View of the embankment
located on site no. 2

Figure 8.
Embankment just east
of site no. 2

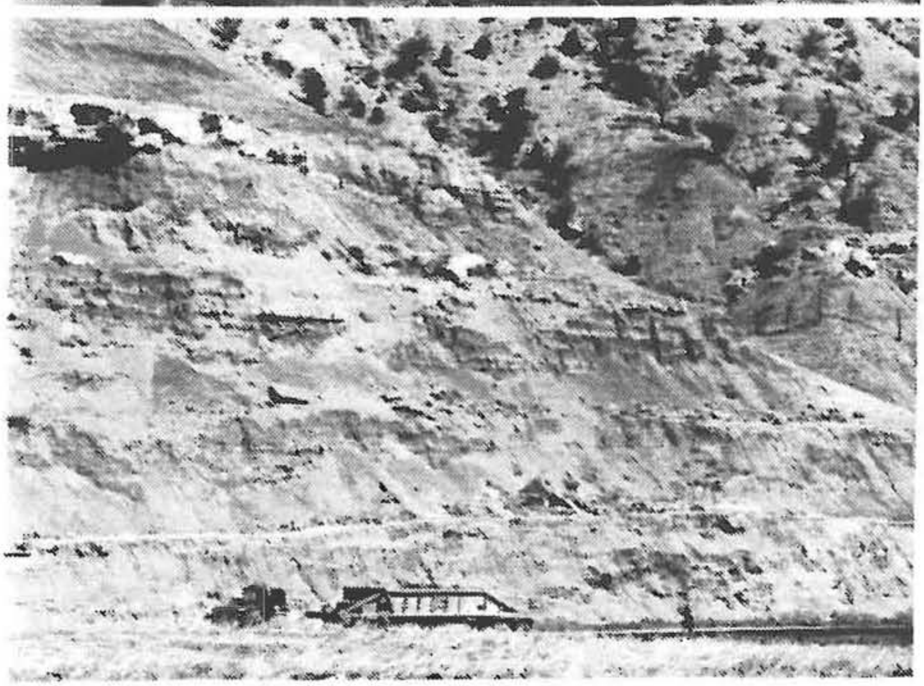


Figure 9.
Shale formations near
site no. 2

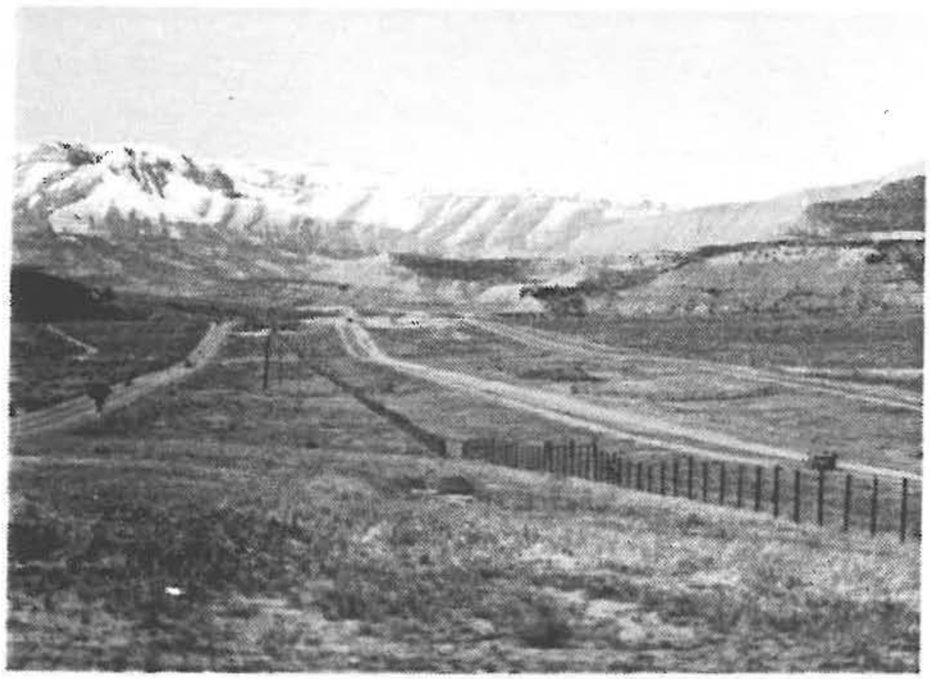


Figure 10. View of site no. 3

The shale formations in this part of the country are categorized as Wasatch shales and their slaking characteristics vary from durable through disintegration overnight. To demonstrate this fact, three rocks were excavated from the construction site in March 1980. They were placed outside in small containers and their behaviors were observed for about 2 months. Figure 11 shows these rocks two days after excavation. By day 19, one sample had completely disintegrated, one sample was severely broken, and one remained intact, as shown in Figure 12. Monitoring was continued into April, 1980 and deterioration into clay soil was observed in portions of the two slaking samples.

IV. Construction Procedures

Standard construction procedures were adopted and used to build the embankments along I-70 in Western Colorado. The shale or mixture of shale and sandstone were cut from the adjacent slopes, and they were simply hauled in and compacted to 95 percent maximum dry density in 2 to 3 foot thick lifts. No special field test such as jar-slake test was performed to identify the approximate nature of the shales, and as a consequence, no specifications were provided to control the lift thickness for nondurable or durable shales during the actual construction.

Figure 11. Selected shales from the construction site two days after excavation

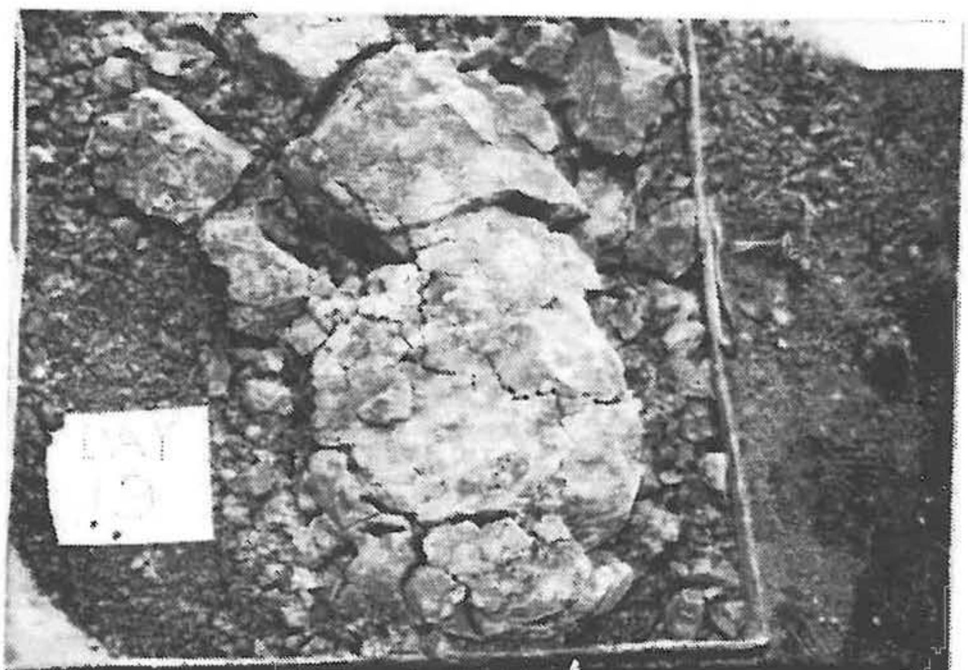


Figure 12. Disintegration of the above selected shales nineteen days after excavation

V. Instrumentation

A total of 3 slope indicators (inclinometers), one on each site, were used to measure the lateral movements of the selected highway embankments. In addition, each inclinometer was encased in a collapsible tubing (Sondex) with stainless steel rings set at an initial two-and-a-half foot spacing to measure the settlement and/or expansion of the fill material. Figure 13 shows sketches illustrating the mechanics of the inclinometer and Sondex systems. These two instruments are capable of measuring movements as small as a quarter of an inch in either the horizontal or vertical directions. In addition, standard surveying instruments were used to measure the change in surface profile in each site. Table 1 shows the distribution of instruments on each site.

	No. of Vertical Inclinometer	No. of Survey Sondex Casing	No. of Survey Points
Hole 1	1	1	32
Hole #2	1	1	36
Hole #3	1	1	34

TABLE 1. Distribution of instruments on each site

VI. Laboratory Tests

Numerous index tests have been proposed to assess shale durability, (Chapman, 1975)[7]. Based on Chapman's comparative studies, the following index tests are highly recommended:

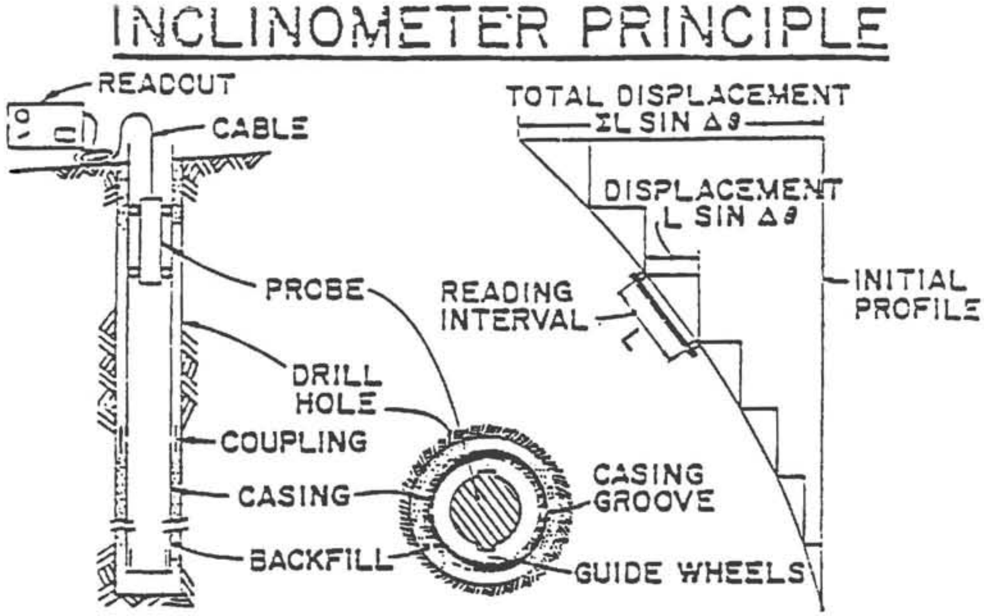


Figure 13a. COMPONENTS OF INCLINOMETER EQUIPMENT AND A SKETCH OF ITS MEASURING PRINCIPAL

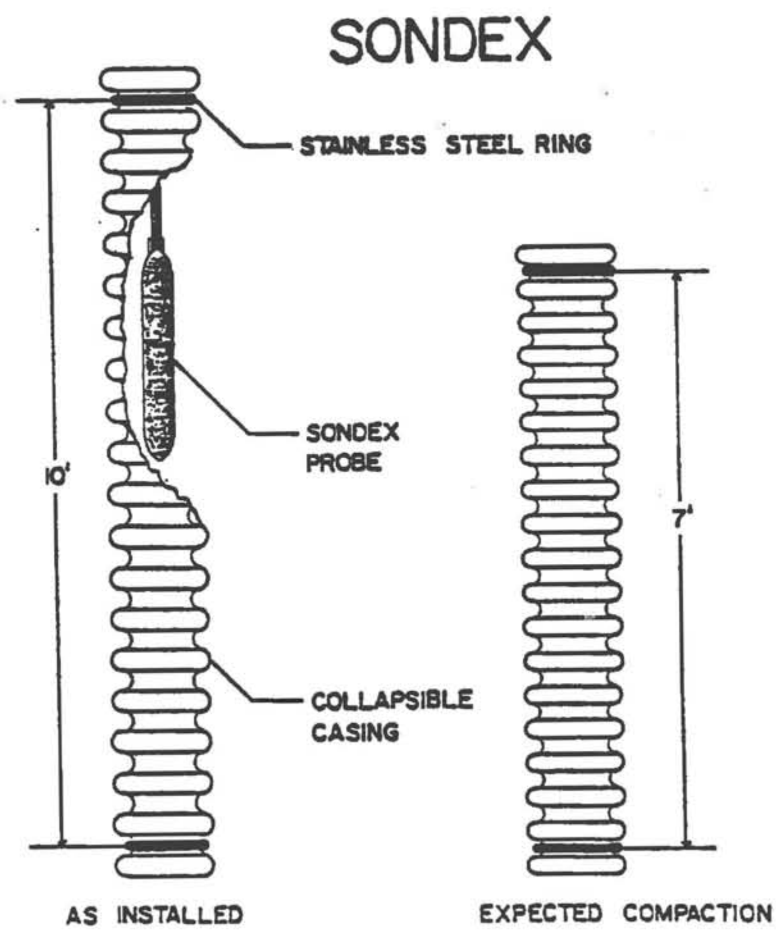


Figure 13b. SKETCH OF VARIOUS COMPONENTS OF A SONDEX SYSTEM

a: Jar-slake test

b: Slake-durability test

A. The jar-slake test is qualitative with six descriptive degrees of slaking determined from visual observation of oven-dried samples soaked in tap water for 24 hours. The six values of the jar-slake index, I_J, are listed in Table 2.

I_J	DESCRIPTIVE BEHAVIOR
1	Degrades into a Pile of flakes or mud
2	Breaks rapidly and/or forms many chips
3	Breaks rapidly and/or forms few chips
4	Breaks slowly and/or forms several fractures
5	Breaks slowly and/or forms few fractures
6	No change

TABLE 2. Description of the six values given to jar-slake index,

Reaction to the jar-slake test usually occurs within the first 10 to 30 minutes, and a standard of 24 hours is recommended as a convenient maximum time for initial testing of a large number of samples. As experience is gained with shales in a particular formation, the maximum time can be reduced to 2 hours or less. During our jar-slake tests, we adopted the maximum 24 hours for all the tests.

B. Slake-Durability Test

The slake-durability test is performed on 10 pieces

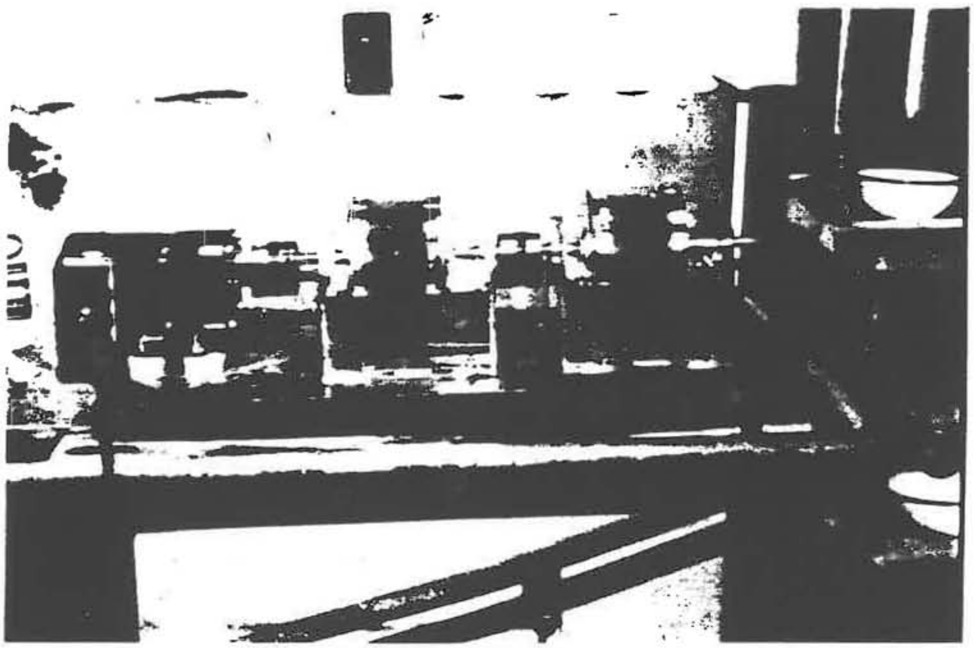


Figure 14. Slake-Durability test device

of oven-dried shale (40 to 60 grams each) submerged and rotated in a wire drum cage (No. 10 screen) at 20 rpm for 10 minutes. The procedure is repeated on the material retained in the drum after oven-drying. The two cycle slake-durability index, I_D, is the percent of oven-dried material retained after the test.

$$I_D = \frac{\text{Dry Weight After Two Cycles}}{\text{Dry Weight Before Testing}} \times 100$$

The testing apparatus is shown in Figure 14.

C. Results of Jar-Slake Tests and the Slake Durability Tests

A total of 6 shale samples were obtained to study their durability behavior. These samples were collected by digging into the shale formations adjacent to the selected test embankment sites. The shales varied in color, with most variation in the cut slopes adjacent to Site No. 2. The natural water content of the collected samples were determined in the laboratory and they are presented in Table 3.

Bag#	Site No.	Wet Weight of Sample (gm)	Dry Weight of Sample (WS) g/m	Weight of Water WW gm	Water Content (WW/WS) X 100
1	1	1253.8	1250.4	3.4	.27
2	1	491.0	489.6	1.4	.29
3	2	777.4	775.9	1.5	.19
4	2	852.6	850.0	1.1	.13
5	3	369.8	368.1	1.7	.46
6	3	386.3	381.1	5.2	1.36

TABLE 3. Water Content of the Collected Samples

Jar tests and slake-durability tests were performed on each test sample and the results are presented in Tables 4 and 5.

Sample Bag #	Site No.	Color	Total Weight Sample Submerged In Water (gm)	IJ
1	1	Reddish Gray	76.10	1
2	1	Gray	53.40	2
3	2	Gray	123.80	3
4	2	Gray	59.10	2
5	3	Yellowish Gray	80.80	1
6	3	Gray	32.40	2

TABLE 4. Results of the Jar-Slake

Bag #	Site No. (gm)	Dry Wt. of Sample (gm)	Dry Wt. After 2 cycles (gm)	I_D
1	1	486	239	49.2
2	1	589	553	93.9
3	2	436	282	64.7
4	2	480	287	59.8
5	3	280	44	15.7
6	3	355	231	65.1

TABLE 5. Results of the slake-durability tests

Additional Laboratory Tests

In addition to the slake tests, the Atterberg limit tests were also performed on the recovered samples, and the results are illustrated in Table 6.

Sack#	Site #	Liquid Limit (L.L.)	Plastic Limit (P.L.)	Plasticity Index (P.I.)
1	1	23	16	7
2	1	24	16	8
3	2	22	15	7
4	3	33	19	14
5	3	26	18	8

TABLE 6. Atterberg Limit Test Results

The tested specimens generally appeared to be silty with little clay content except for the specimen obtained from Sack No. 5. This is evident from the Atterberg limit test results. The plasticity index (P.I.) of 14 for the sample obtained from Bag No. 5 is approximately twice that of other samples. This translates into the fact that the embankments composed of material with higher plasticity indices have higher clay contents, and therefore, they are susceptible to larger settlement or expansion when the moisture conditions are appropriate.

VII. Field Observation

The field observation consisted of the results obtained from the inclinometers, Sondex systems, and the plane surveying of the pavement on top of each test embankment. All three embankments were monitored for 56 months and the following are the results of our field observations.

The inclinometer data was plotted, and they are shown in Figures 15 through 17. The test embankments No. 1 and 2 showed 0.386 and 0.348 ft. of lateral movements, respectively. The test embankment No. 3 showed 0.484 ft. of lateral movement which is higher than the lateral movements of the other two test embankments.

The Sondex information was also plotted, and they are displayed in Figures 18 through 20. The maximum settlement of the top ring which is an indicator of the total overall settlements are .44, .32, and 0.99 ft. of settlement, respectively, on sites No. 1, 2, and 3. Site No. 3 shows the largest settlement corresponds well with the inclinometer data.

The last set of information was simply obtained by directly marking the pavement on top of each test embankment and using surveying instruments to monitor the elevation of each marked point from time to time. The results are shown in Tables 7 through 9. The last set of readings were taken on

INCLINOMETER DATA FOR OBSERVATION WELL NO. 1

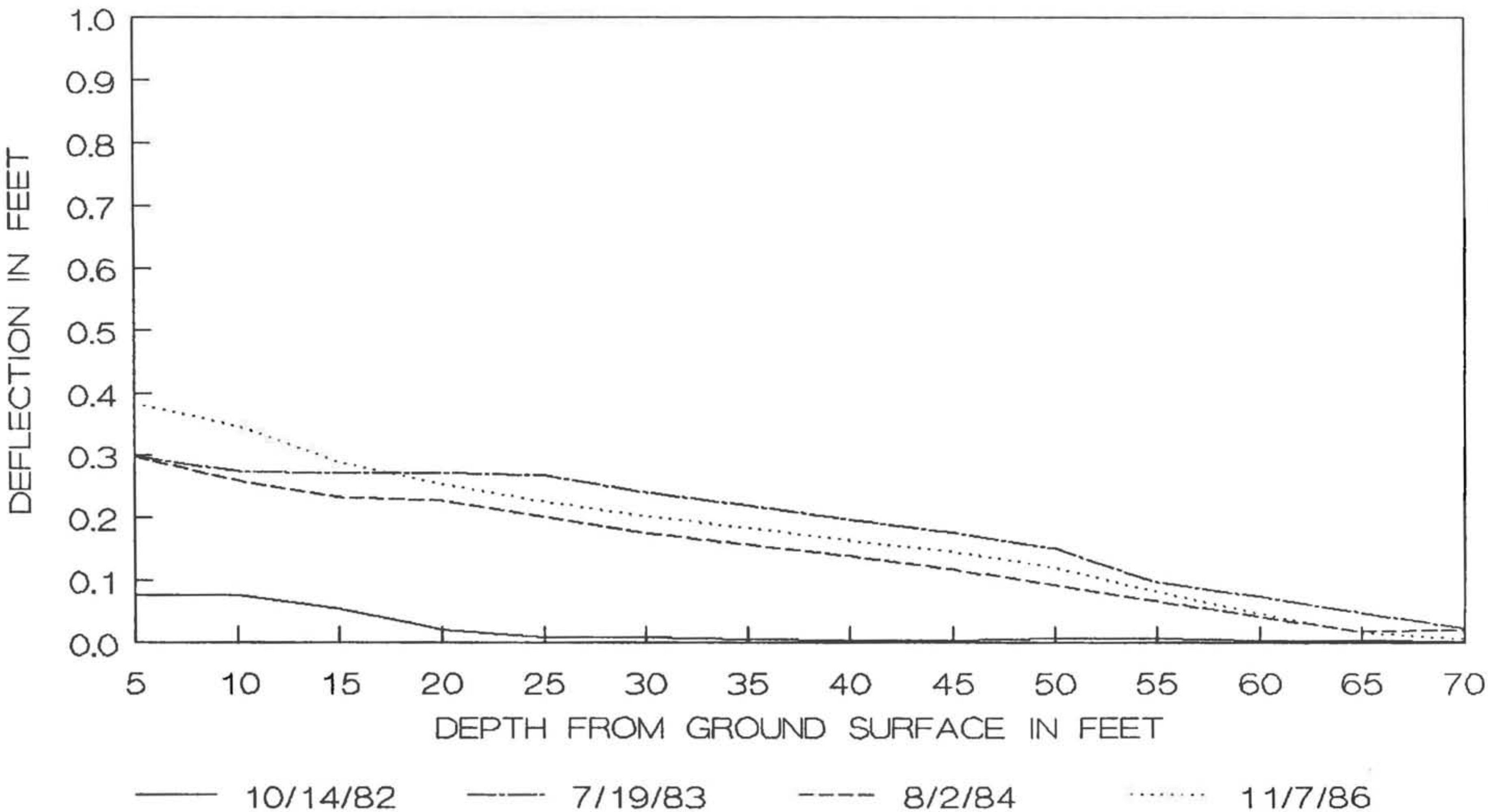


Figure 15

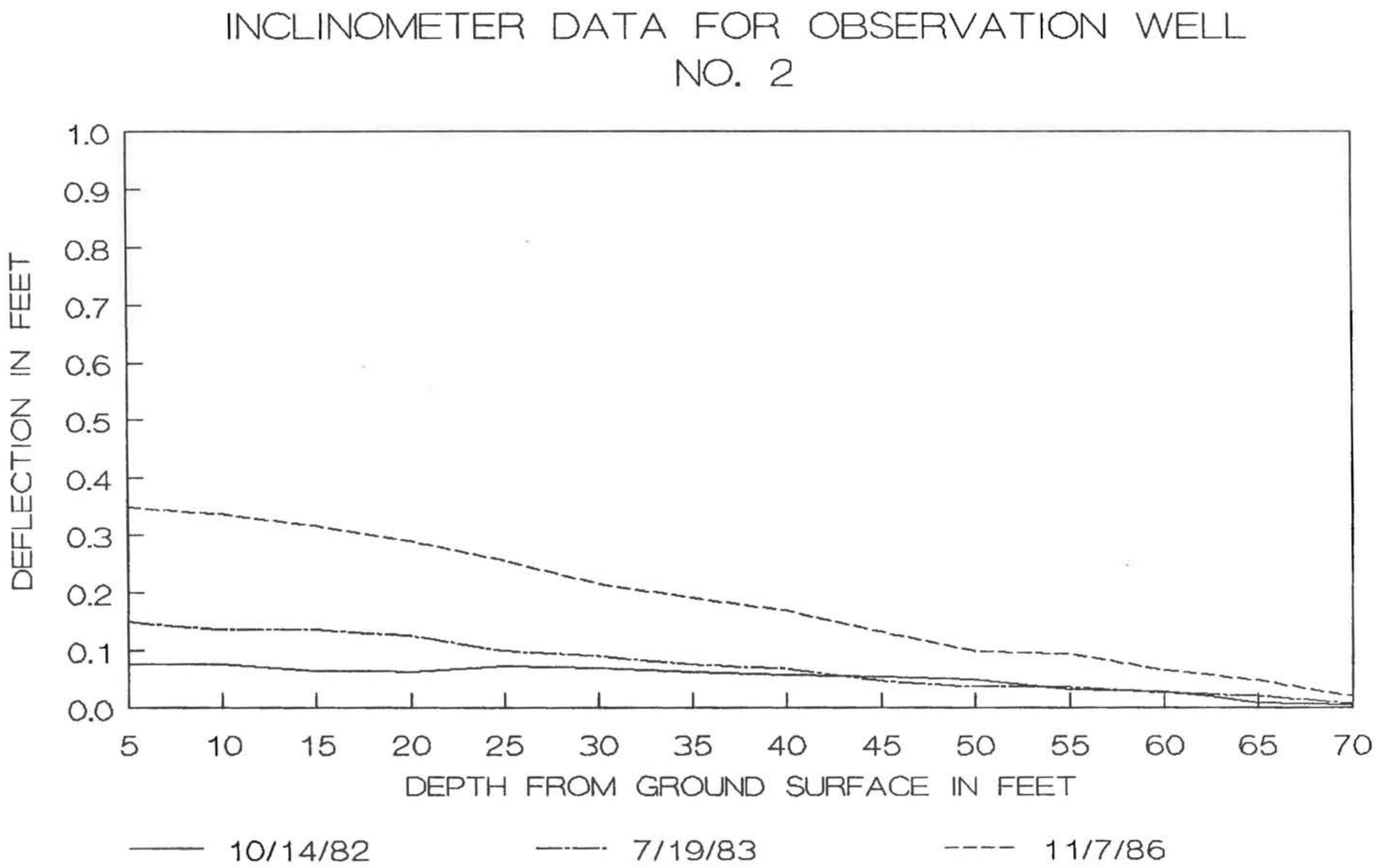
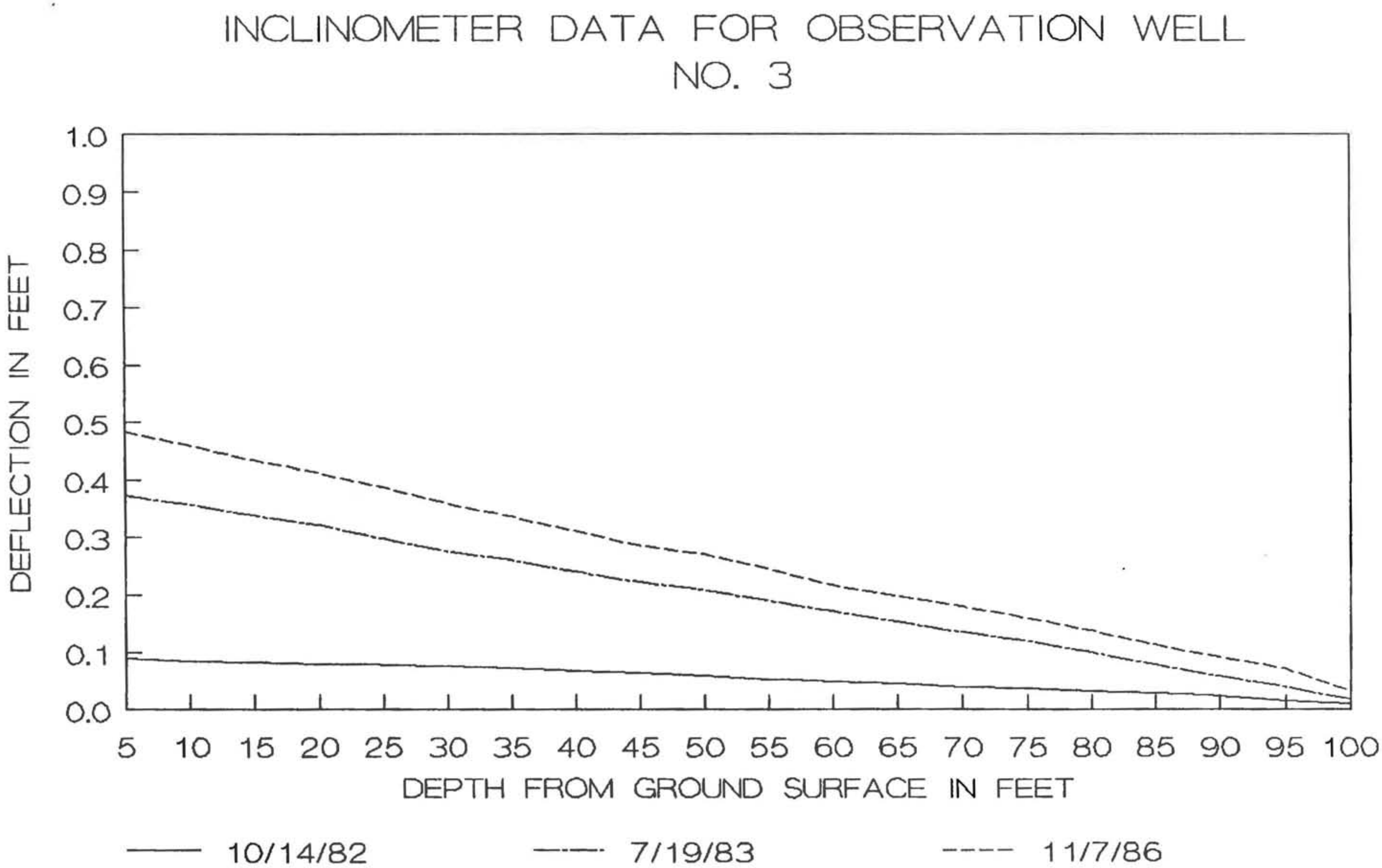

INCLINOMETER DATA FOR OBSERVATION WELL
NO. 2
DEFLECTION IN FEET
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
DEPTH FROM GROUND SURFACE IN FEET
10/14/82
7/19/83
11/7/86

Figure 16



INCLINOMETER DATA FOR OBSERVATION WELL NO. 3
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
DEFLECTION IN FEET
5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100
DEPTH FROM GROUND SURFACE IN FEET
10/14/82
7/19/83
11/7/86

Figure 17

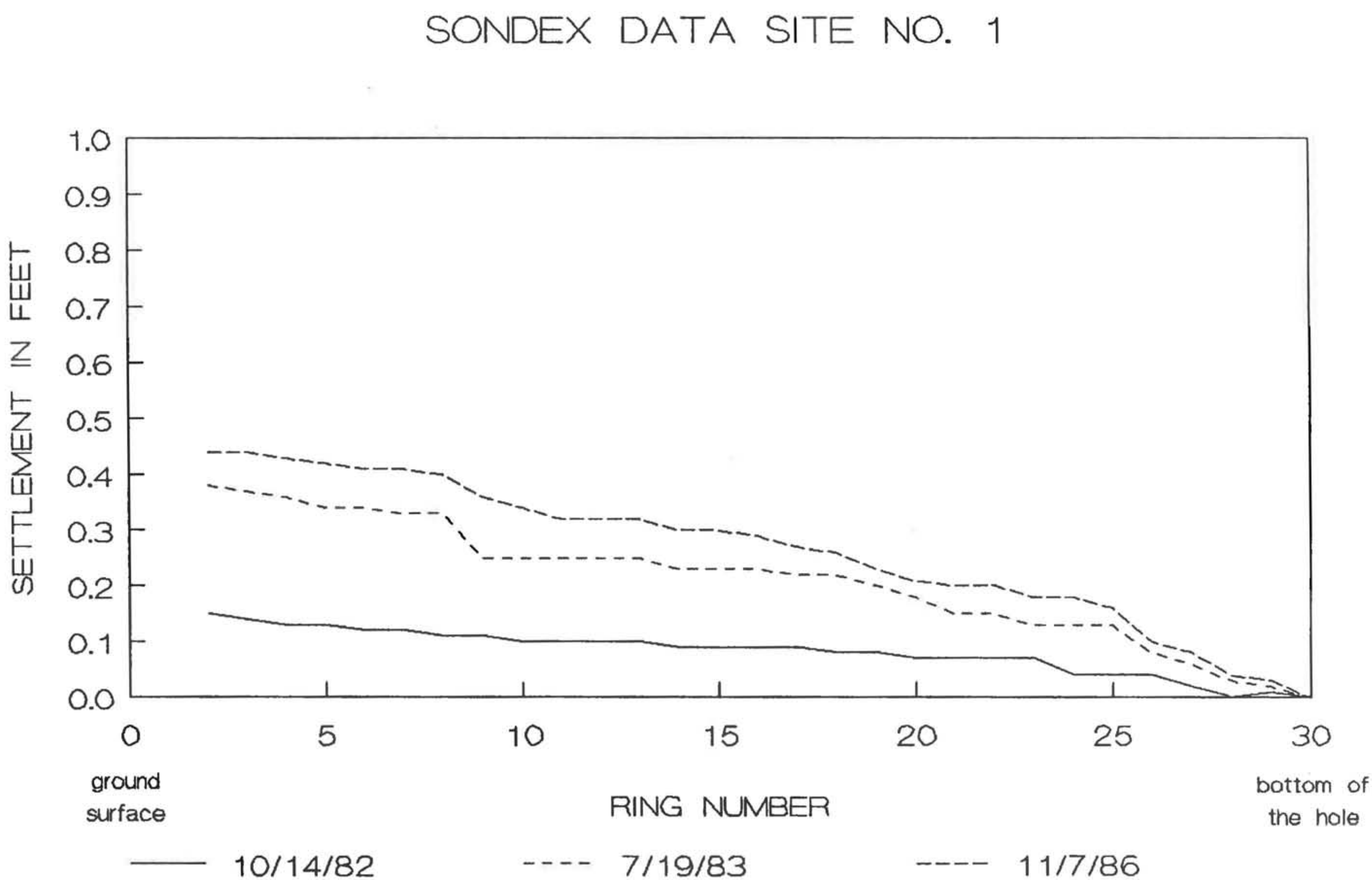

SONDEX DATA SITE NO. 1
SETTLEMENT IN FEET
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0
5
10
15
20
25
30
ground surface
RING NUMBER
bottom of the hole
10/14/82
7/19/83
11/7/86

Figure 18

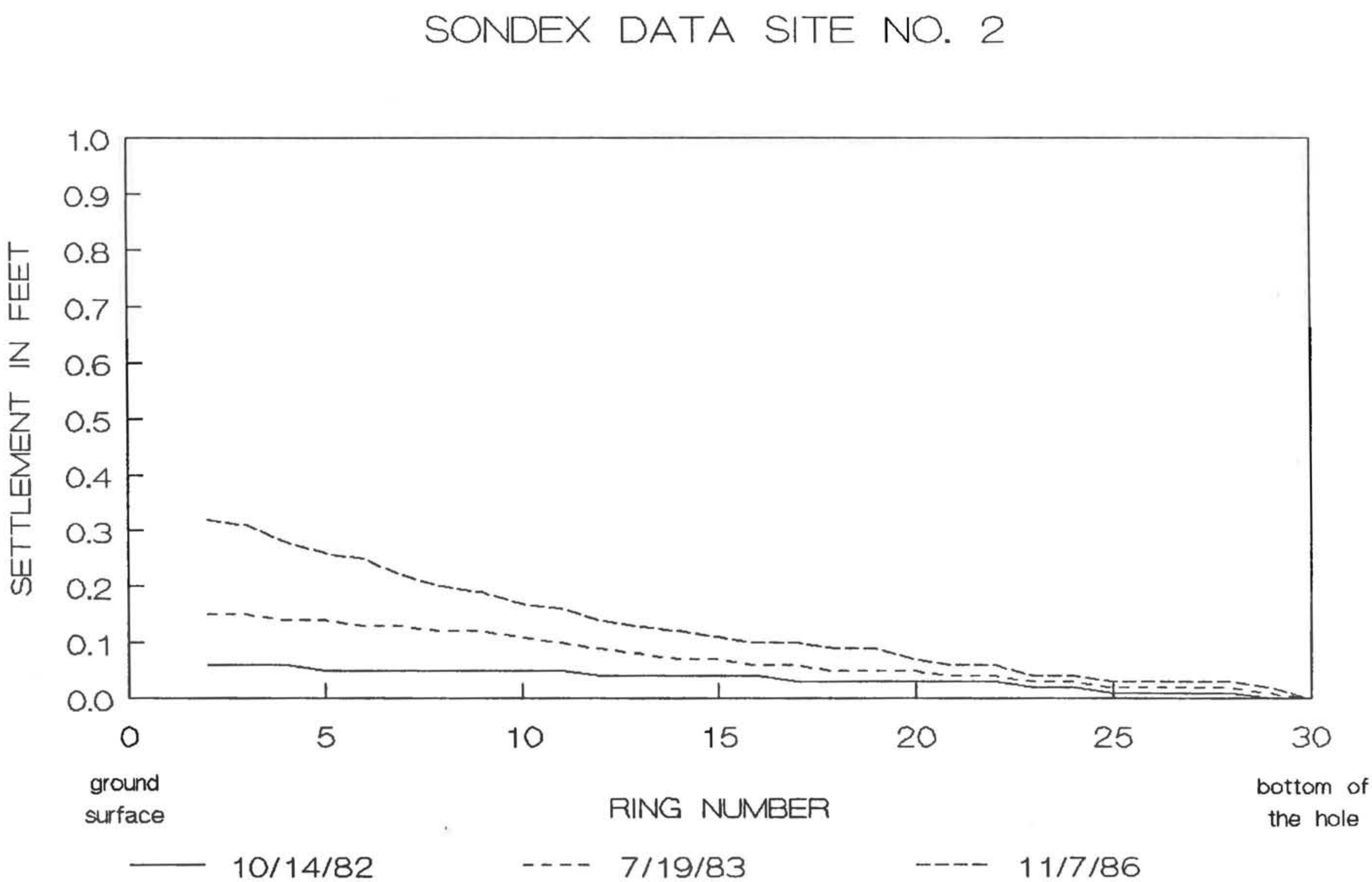

SONDEX DATA SITE NO. 2
SETTLEMENT IN FEET
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0
5
10
15
20
25
30
ground surface
RING NUMBER
bottom of the hole
10/14/82
7/19/83
11/7/86

Figure 19

SONDEX DATA SITE NO. 3

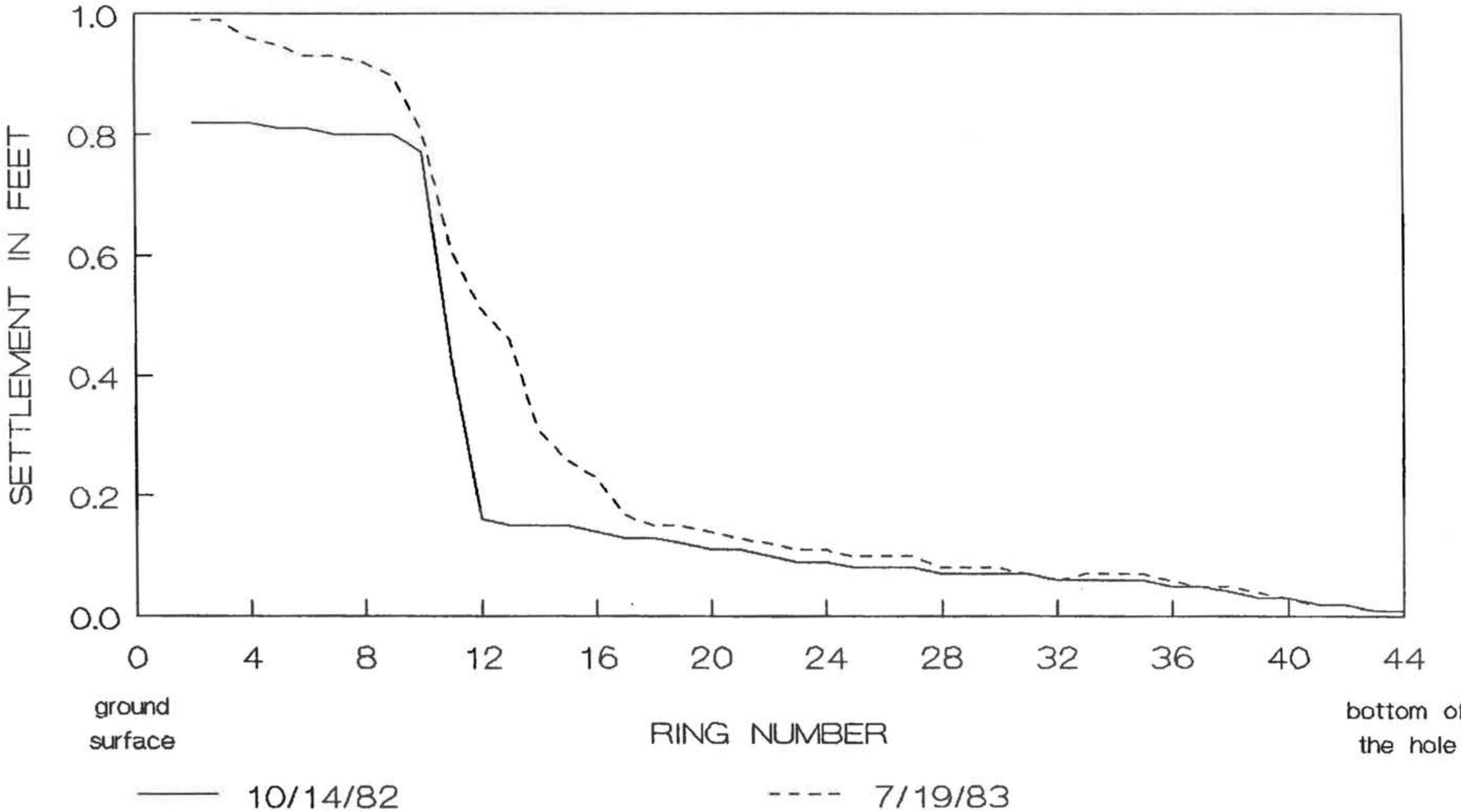


Figure 20

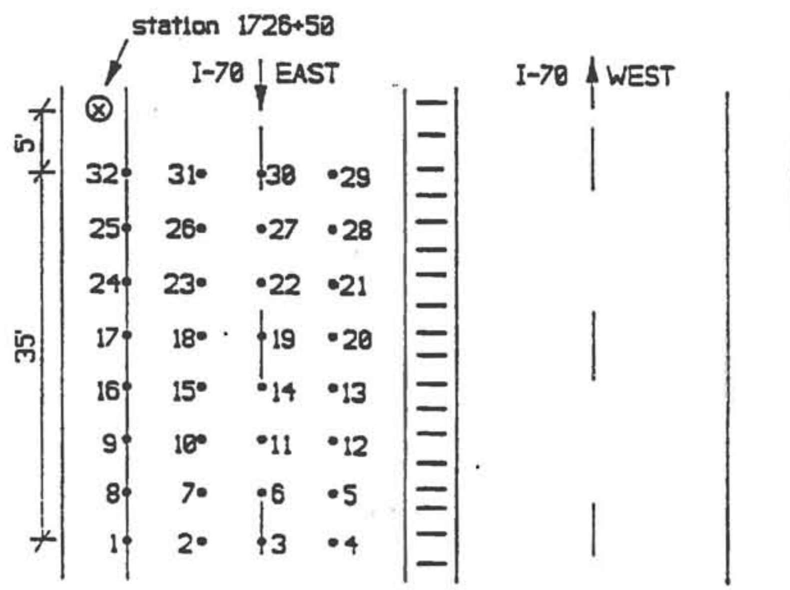
HOLE NO.1

	DATE / LOCATION	6/23/81 INITAIL READING	11/7/86 ELEVATIONS	11/7/86 SETTLEMENT FEET
1	S.E.	5212.49	5212.18	0.31
2	R.T.	5213.50	5213.20	0.36
3	C.L.	5214.22	5213.85	0.37
4	L.T.	5.06	4.70	0.36
5	L.T.	5.10	4.75	0.35
6	C.L.	4.26	3.91	0.35
7	R.T.	3.55	3.20	0.35
8	S.E.	2.59	2.24	0.35
9	S.E.	2.74	2.32	0.42
10	R.T.	3.59	3.25	0.34
11	C.L.	4.28	3.96	0.32
12	L.T.	5.13	4.81	0.32
13	L.T.	5.13	4.84	0.29
14	C.L.	4.29	3.99	0.30
15	R.T.	3.62	3.30	0.32
16	S.E.	2.86	2.49	0.37
17	S.E.	2.88	2.62	0.26
18	R.T.	3.60	3.34	0.26
19	C.L.	4.26	4.02	0.24
20	L.T.	5.08	4.84	0.24
21	L.T.	4.99	4.83	0.16
22	C.L.	4.18	4.01	0.17
23	R.T.	3.53	3.32	0.21
24	S.E.	2.83	2.65	0.18
25	S.E.	2.90	3.61	0.29
26	R.T.	3.48	3.28	0.20
27	C.L.	4.09	3.94	0.15
28	L.T.	4.85	4.75	0.10
29	L.T.	4.73	4.62	0.11
30	C.L.	4.00	3.85	0.15
31	R.T.	3.43	3.24	0.19
32	S.E.	2.90	2.70	0.20

S.E.	SHOULDER EDGE
R.T.	RIGHT LANE CENTER
C.L.	CENTER LINE
L.T.	LEFT LANE CENTER
X	LOCATION OF THE INCLINOMETER/ SONDEX SYSTEM
T.P.	TOP OF PIPE

Table 7 - Survey results on site no. 1

Table 7.
Survey data on site no. 1



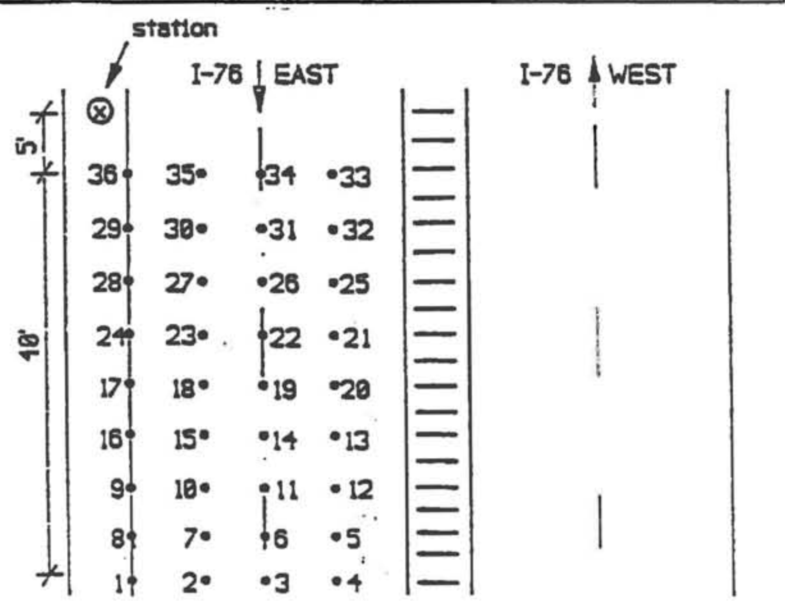
			HOLE NO.2	
	DATE	6/23/81	11/7/86	11/7/86
		INITAIL		SETTLEMENT
	LOCATION	READING	ELEVATIONS	FEET
1	S.E.	5287.66	5287.56	0.10
2	R.T.	5287.73	5287.65	0.08
3	C.L.	7.84	7.78	0.06
4	L.T.	7.95	7.90	0.05
5	L.T.	7.55	7.50	0.05
6	C.L.	7.43	7.35	0.08
7	R.T.	7.28	7.19	0.09
8	S.E.	7.21	7.12	0.09
9	S.E.	6.80	6.71	0.09
10	R.T.	6.86	6.76	0.10
11	C.L.	7.00	6.91	0.09
12	L.T.	7.13	7.06	0.07
13	L.T.	6.76	6.68	0.08
14	C.L.	6.63	6.52	0.11
15	R.T.	6.51	6.37	0.14
16	S.E.	6.44	6.31	0.13
17	S.E.	6.15	5.89	0.26
18	R.T.	6.16	5.95	0.21
19	C.L.	6.28	6.15	0.13
20	L.T.	6.44	6.35	0.09
21	L.T.	6.08	5.99	0.09
22	C.L.	5.95	5.79	0.16
23	R.T.	5.84	5.57	0.27
24	S.E.	5.78	5.48	0.30
25	S.E.	5.44	5.23	0.21
26	R.T.	5.50	5.33	0.17
27	C.L.	5.63	5.50	0.13
28	L.T.	5.74	5.60	0.14
29	L.T.	5.43	5.38	0.05
30	C.L.	5.29	5.19	0.10
31	R.T.	5.16	5.05	0.11
32	S.E.	5.09	5.00	0.09
33	S.E.	4.75	4.69	0.06
34	R.T.	4.83	4.74	0.09
35	C.L.	4.94	4.87	0.07
36	L.T.	5.11	5.05	0.06

LEGEND

S.E.	SHOULDER EDGE
R.T.	RIGHT LANE CENTER
C.L.	CENTER LINE
L.T.	LEFT LANE CENTER
X.	LOCATION OF THE INCLINOMETER/ SONDEX SYSTEM
T.P.	TOP OF PIPE

Table 8 - Survey results on site no. 2

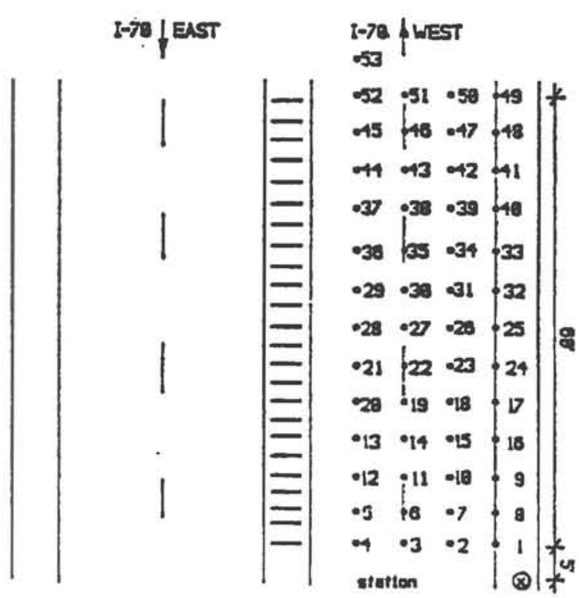
Table 8.
Survey data on site no. 2



	DATE LOCATION	4/1/81 INITAIL READING	6/15/82 ELEVATIONS	6/15/82 SETTLEMENT FEET	10/13/83 ELEVATIONS	10/13/83 SETTLEMENT FEET
1	S.E.	5277.36	5277.30	-0.06	5277.30	-0.06
2	R.T.	5277.42	5277.44	0.02	5277.44	0.02
3	C.L.	5277.55	5277.67	0.12	5277.66	0.11
4	L.T.	5277.81	5277.84	0.03	5277.84	0.03
5	L.T.	5278.30	5278.35	0.05	5278.36	0.06
6	C.L.	5278.05	5278.23	0.18	5278.22	0.17
7	R.T.	5277.84	5278.00	0.16	5278.00	0.16
8	S.E.	5277.70	5277.87	0.17	5277.88	0.18
9	S.E.	5278.02	5278.32	0.30	5278.33	0.31
10	R.T.	5278.22	5278.48	0.26	5278.48	0.26
11	C.L.	5278.46	5278.67	0.21	5278.67	0.21
12	L.T.	5278.70	5278.83	0.13	5278.83	0.13
13	L.T.	5279.00	5279.18	0.18	5279.17	0.17
14	C.L.	5278.85	5279.07	0.22	5279.09	0.24
15	R.T.	5278.63	5278.85	0.22	5278.87	0.24
16	S.E.	5278.42	5278.70	0.28	5278.71	0.29
17	S.E.	5278.75	5279.01	0.26	5279.02	0.27
18	R.T.	5278.98	5279.20	0.22	5279.22	0.24
19	C.L.	5279.20	5279.42	0.22	5279.44	0.24
20	L.T.	5279.32	5279.50	0.18	5279.50	0.18
21	T.P.	5279.23	5279.20	-0.03	5279.15	-0.08
22	L.T.	5279.69	5279.85	0.16	5279.85	0.16
23	C.L.	5279.53	5279.73	0.20	5279.74	0.21
24	R.T.	5279.32	5279.53	0.21	5279.54	0.22
25	S.E.	5279.10	5279.34	0.24	5279.37	0.27
26	S.E.	5279.49	5279.72	0.23	5279.73	0.24
27	R.T.	5279.64	5279.84	0.20	5279.85	0.21
28	C.L.	5279.81	5280.02	0.21	5280.03	0.22
29	L.T.	5280.03	5280.14	0.11	5280.16	0.13
30	L.T.	5280.23	5280.27	0.04	5280.39	0.16
31	C.L.	5280.08	5290.25	0.17	5280.28	0.20
32	R.T.	5279.90	5280.07	0.17	5280.08	0.18
33	S.E.	5279.75	5279.92	0.17	5279.93	0.18
34	S.E.	5280.09	5280.27	0.18	5280.37	0.28
35	R.T.	5280.19	5280.34	0.15	5280.36	0.17
36	C.L.	5280.34	5280.51	0.17	5280.53	0.19
37	L.T.	5280.56	5280.66	0.10	5280.67	0.11
38	L.T.	5280.82	5280.90	0.08	5280.93	0.11
39	C.L.	5280.62	5280.77	0.15	5280.80	0.18
40	R.T.	5280.45	5280.58	0.13	5280.61	0.16
41	S.E.	5280.34	5280.50	0.16	5280.54	0.20
42	S.E.	5280.50	5280.69	0.19	5280.71	0.21
43	R.T.	5280.63	5280.79	0.16	5280.81	0.18
44	C.L.	5280.84	5280.98	0.14	5280.97	0.13
45	L.T.	5281.04	5281.17	0.13	5281.18	0.14
46	L.T.	5281.23	5281.33	0.10	5281.32	0.09
47	C.L.	5281.01	5281.13	0.12	5281.13	0.12
48	R.T.	5280.84	5281.01	0.17	5280.99	0.15
49	S.E.	5280.75	5280.92	0.17	5280.91	0.16
50	S.E.	5281.03	5281.09	0.06	5281.09	0.06
51	R.T.	5281.11	5281.18	0.07	5281.17	0.06
52	C.L.	5281.26	5281.33	0.07	5281.32	0.06
53	L.T.	5281.43	5281.51	0.08	5281.48	0.05

Table 9 - Survey results on site no.3

Table 9.
Survey data on site no. 3



11/7/86, and the maximum differential settlements of 0.32 and 0.25 feet were measured on Sites No. 1 and 2. On Site No. 3, it was observed that a new layer of asphalt pavement was poured in prior to 11/7/86, and all the marked points were covered. Therefore, the last set of readings taken on 6/13/83 were used to measure the differential settlements on this site. The maximum differential settlement on this site after just 7 months is about 0.30 feet in a stretch of 35 feet of roadway. According to the inclinometer and Sondex data, it is reasonable to assume that the rate of relative movements had substantially increased between 10/13/83 and mid-1985, and because of that the pavement was overlaid in mid-1985.

VIII. Analysis of the Results

The primary objective of this study was to monitor the overall performance of the three selected embankments along I-70 in the Western Slopes of Colorado. All three embankments were constructed using the shale material which was provided from the adjacent cuts close by the roadway.

To define the durability characteristics of the shales on each site, samples were obtained and both jar-slake and slake-durability tests were performed on each sample. The results are summarized in Table 10. According to the results of the jar-slake tests, the jar-slake Index, I_J, varies between 1 and 3 for all the collected shale samples. This, according to the descriptions given on Table 2, classifies all the tested

SAMPLE NO.	SITE NO.	IJ	ID (o/o)	P.I.
1	1	1	49.2	7
2	1	2	93.9	8
3	2	3	64.7	7
4	2	2	59.8	14
5	3	1	15.70	8
6	3	2	65.1	-

TABLE 10. Summary of the results obtained from jar, slake-durability and Atterberg limit tests.

shales as being weak and nondurable when exposed to water. This was also visually observed during the jar-slake tests in the laboratory as shown in Figure 21. Shale samples were placed in 6 different jars and water was added. All samples except Sample No. 3 reacted rapidly to the addition of water and tests were almost completed within the first 15 to 20 minutes. Sample No. 3 reacted slower than the other samples, and it took about one hour before its rate of disintegration was completed. All samples were left over night and inspected 24 hours later for jar-slake index evaluation.

All six samples were then tested using slake-durability test apparatus as shown in Figure 14. After two dry-wet cycles, the slake-durability index, I_D, was calculated for each sample and the results are presented in Table 10. Figures 22 and 23 show the selected samples before and after the slake-durability tests. To identify the durability of these samples, Table 11, by (CHAPMAN, 1975)[7] was used as a guideline. According to this table, shale samples with I_D less than 90 percent are considered nondurable and they must be treated as soil-like. Table 10 illustrates that all the collected shale samples contained an I_D less than 90 percent except for Sample No. 2. But this sample will also be considered nondurable since its I_J was determined to be 2, which means that it deteriorates and breaks rapidly in water and it could form many chips.

Figure 21. Jar-Slake durability test in progress

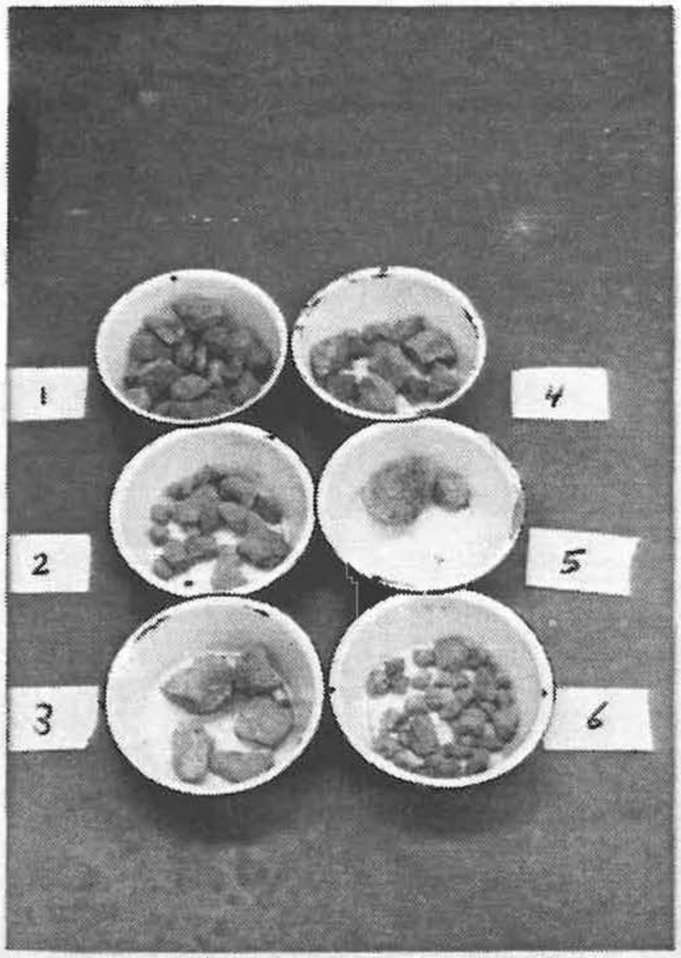


Figure 22. Samples prior to slake-durability test

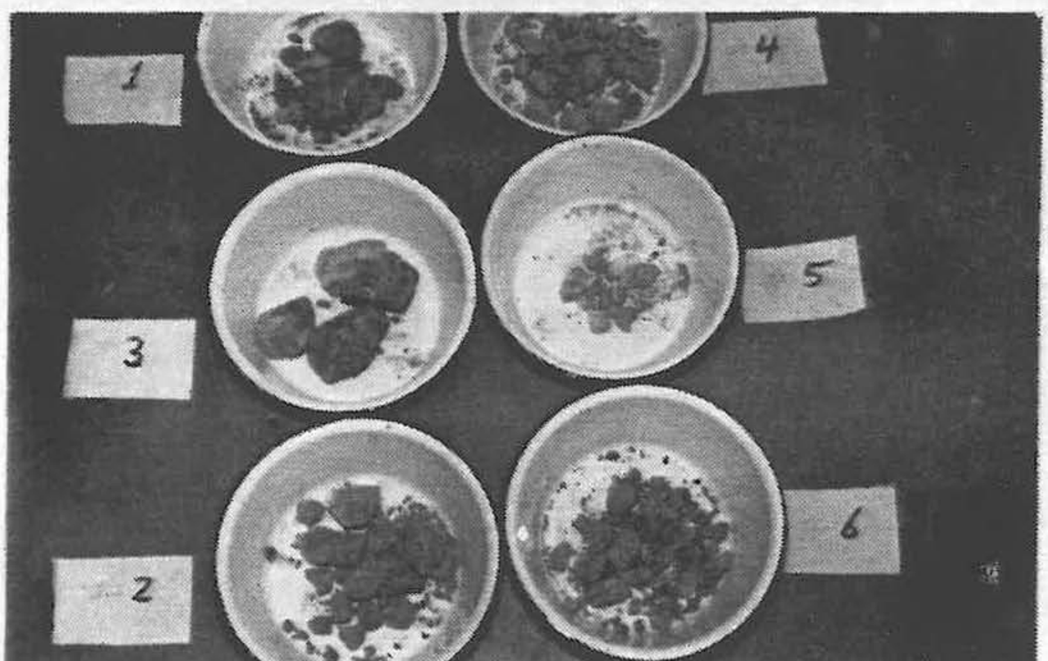
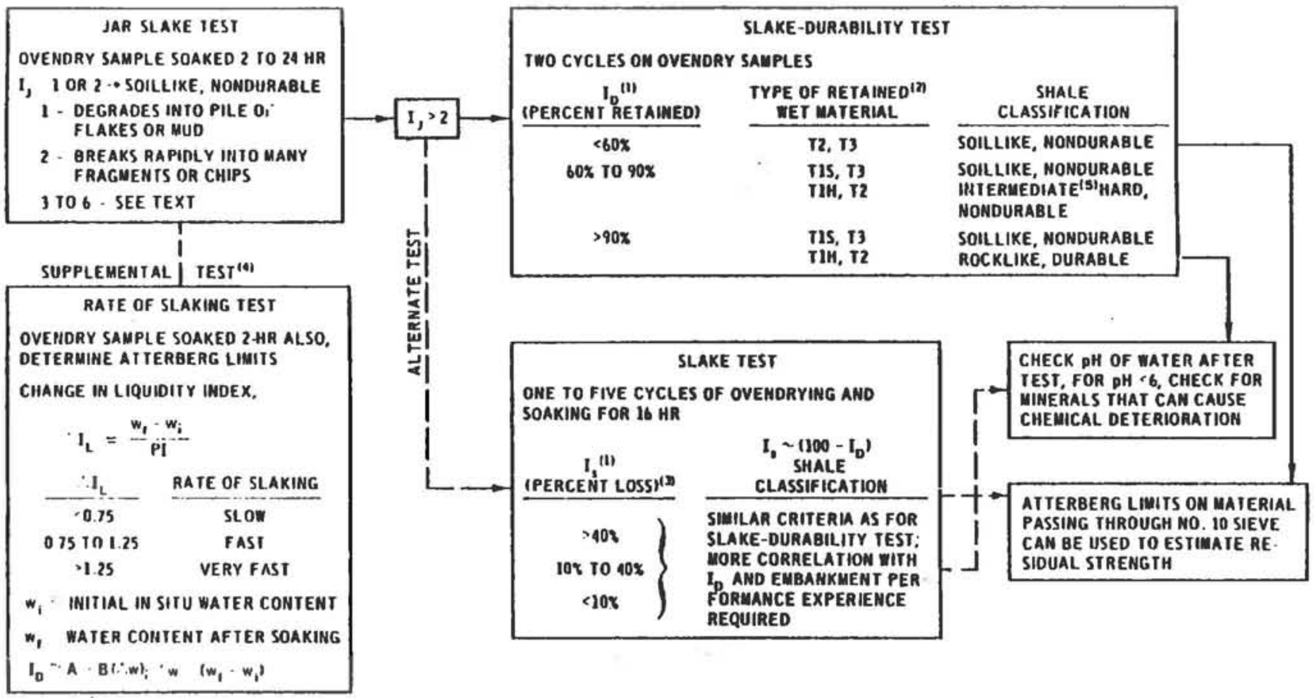


Figure 23. Remains of the samples after the slake-durability test



NOTE (1) DIFFERENT LIMITING VALUES MAY BE JUSTIFIED ON BASIS OF LOCAL EMBANKMENT PERFORMANCE EXPERIENCE.

(2) TYPE T1 - NO SIGNIFICANT BREAKDOWN OF ORIGINAL PIECES.
TYPE T1S - SOFT, CAN BE BROKEN APART OR REMOLDED WITH FINGERS.
TYPE T1H - HARD, CANNOT BE BROKEN APART.
TYPE T2 - RETAINED PARTICLES CONSIST OF LARGE AND SMALL HARD PIECES.
TYPE T3 - RETAINED PARTICLES ARE ALL SMALL FRAGMENTS.

(3) USING NO. 10 SIEVE.

(4) CAN BE PERFORMED ON JAR-SLAKE TEST SAMPLES IF IN SITU NATURAL WATER CONTENT IS KNOWN. PI SENSITIVE TO DEGREE OF PULVERIZATION.

(5) REQUIRES SPECIAL PROCEDURES TO ASSURE GOOD DRAINAGE AND ADEQUATE COMPACTION (95% T-99) FOR LOOSE LIFT THICKNESS UP TO 24-IN. MAXIMUM.

Table 11. Recommended durability index tests and suggested classification criteria for shales used in highway embankments

One interesting observation was made on Sample No. 5, with the lowest I_g and I_D values equal to 1 and 15.7 percent, respectively. Based on this evaluation, it is concluded that the shale Sample No. 5, which was collected from Site No. 3 had the least amount of durability against water and should be categorized as a weak nondurable shale. This was also verified from the Atterberg limit tests which produced the highest plasticity index (P.I.) value for Sample No. 5, as illustrated in Table 10.

In addition to laboratory tests, field observations was also continued for about 5 years, and it is interesting to compare the correlation between the laboratory test results and the field observations. According to inclinometer and Sondex data, Embankment No. 3 showed the largest lateral and vertical movements (settlements) between 1981 and 1986. The largest measured lateral and vertical movements were 0.48 and 0.99 ft., respectively, on Site No. 3, as illustrated in Figures 17 and 20. This correlates well with the laboratory data obtained on shale Sample No. 5, which was collected from Site No. 3.

Finally, due to large differential settlements on Site No. 3, the pavement was overlaid in mid-1985, and it appears that the rate of settlement within the embankment has slowed down. The pavements on top of the other two embankments have had some minor differential settlements, but they have not been overlaid.

IX. Conclusion

It is the conclusion of this study that nondurable shales can be used as embankment material in semi-arid climates such as lower elevation areas on the Western Slope in Colorado. Their long-term performance can be generally satisfactory if they are treated as soil-like and adequate drainage is provided to keep out the surface water.

X. Implementation

At the present time, it is apparent that standard construction procedures are being used during embankment construction using shale material. No testing is currently specified to determine the shale durability, and therefore, no special construction control is adopted. During this study, we experimented with the simple jar-slake and slake-durability tests as recommended in Report No. FHWA-RD-78-141. It was found that these two tests can be performed at very low cost, and the results can help the construction project engineers to decide on how to treat the shale as embankment material. If the results of slake-durability and jar-tests specified a shale to be nondurable, then an 8 inch lift thickness should be used. On the other hand, if the test results specified a durable shale, then the material can be placed as rockfill in thick lifts of 2 to 3 ft.

REFERENCES

1. "Design and Construction of Completed Shale Embankments, Vol. 1, Survey of Problem Areas and Current Practices," Interim Report, August 1975, Report No. FHWA-RD-75-61.

2. "Design and Construction of Compacted Shale Embankments, Vol. 2, Evaluation and Remedial Treatment of Shale Embankments," Interim Report, August 1975, Report No. FHWA-RD-75-62.

3. "Design and Construction of Compacted Shale Embankments, Vol. 3, Slaking Indexes for Design," Interim Report, February 1977, Report No. FHWA-RD-77-1.

4. "Design and Construction of Compacted Shale Embankments, Vol. 4, Field and Laboratory Investigations, Phase III," Interim Report, October 1978, Report No. FHWA-RD-78-140.

5. "Design and Construction of Compacted Shale Embankments, Vol. 5, Technical Guidelines," Final Report, December 1978, Report No. FHWA-RD-78-141.

6. Donaghe, R. T., and Townsend, F. C., "Scaling and Replacement Effects on the Compaction Characteristics of Earth-Rock Mixtures," Soil Specimen Preparation for Laboratory Testing, STP 599, American Society for Testing Materials, Philadelphis, PA, June 1976.

7. Chapman, D. R., "Shale Classification tests and Systems: A Comparative Study," Joint Highway Research Project, JHRP-75-11, Purdue University, June 1975.

APPENDIX A

Foundation Boring Logs

STATE OF COLORADO
DEPARTMENT OF HIGHWAYS
DIVISION OF HIGHWAYS
DOH Form No 267
Revised. September. 1978

Project 1525
Location E. of Rulison Interchange
Structure
Route I-70-1(74) County Garfield
Date Drilled 2/31/81

FOUNDATION BORING LOG

Top Hole Elev. ________ Geologist H. Swanson Station 1726 & 50 EB Boring No. 1
4" from edge of pavement, EB lanes.

Elev.	Depth	Description of Material	BPF*	Remarks
1.5"Below final grade	0.0-2.0	River gravel A-1		
	2.0-2.5	Shale & SS rock		Slow & hard
	2.5-4.0	Basalt 2 hours to drill		Very hard
	4.0-33.0	Fill - SH. SS. Clay silt		Sample 1D
	33.0-70.0	Clay		Sample 1E
	70.0-75.0	Gravel		
		Installed 70' of slope indicator tube.		
		Finished 11:30, 2/4/81		

* Standard Penetration Test (AASHTO T 206-74)

BORING LOG No. 1

Water level upon completion ________ Elev.________ Date________ Time________
Water level (24 hrs.)________ Elev.________ Date________ Time________

STATE OF COLORADO
DEPARTMENT OF HIGHWAYS
DIVISION OF HIGHWAYS
DOH Form No. 267
Revised, September, 1978

Project 1525
Location Webster Hill, EB
Structure
Route I-70 County Garfield
Date Drilled 2 & 4/1981

FOUNDATION BORING LOG

Finished Highway Grade
Top Hole Elev. / Geologist H. Swanson Station Rt. Shoulder Boring No. 2
1' from edge of asphalt mat

Elev.	Depth	Description of Material	BPF*	Remarks
	0.0-2.5	Gravel A-1		
	2.5-52.0	Fill-silt, clay, shale, sandstone		Sample 2B
	52.0-60.0	Clay soil		
	60.0-69.0	Clay - wet		
	69.0-75.0	Gravel		
		Installed 68 ft. slope ind tubing w/sondex		
		Completed 2/5/1981		

* Standard Penetration Test (AASHTO T 206-74)

BORING Log No. 2

Water level upon completion ______ Elev.______ Date______ Time______
Water level (24 hrs.)______ Elev.______ Date______ Time______

STATE OF COLORADO
DEPARTMENT OF HIGHWAYS
DIVISION OF HIGHWAYS
DOH Form No. 267
Revised. September. 1978

Project 1525
Location Anvil Pt. Bridge (WB)
Structure
Route I-70 County
Date Drilled 2/2/81

FOUNDATION BORING LOG

Top Hole Elev. Hwy. Grade Geologist Station Left shoulder 10' from west abutment, 5' from guard rail Boring No. 3

Elev.	Depth	Description of Material	BPF*	Remarks
	0-.35	HBP		
	.35'-20'	Gravel		
	2.0'-35'	Shale, sand stone, silt		
	35-45.0	Clay soil		
	45-60	Wet clay soil		
	60-100	Gravely clay		
		Installed 100' of inclinometer tube.		

* Standard Penetration Test (AASHTO T 206-74)

BORING LOG No. 3

Water level upon completion ________ Elev.________ Date________ Time________
Water level (24 hrs.)________ Elev.________ Date________ Time________