PROCEEDINGS OF THE

TWENTY-FIFTH ANNUAL

Highway Geology Symposium

MAY 23 & 24, 1974

SPONSORED BY

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N. C. STATE UNIVERSITY, RALEIGH

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PROCEEDINGS OF THE 25th ANNUAL
HIGHWAY GEOLOGY SYMPOSIUM

SHERATON-CRABTREE INN
RALEIGH, N. C.
MAY 22 - 24, 1974

Sponsored By
NORTH CAROLINA DEPARTMENT OF TRANSPORTATION
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PREFACE

The 25th Annual Highway Geology Symposium was held at the Sheraton-Crabtree Inn in Raleigh, North Carolina on May 23 and 24, 1974. A get acquainted party, hosted by suppliers of drilling equipment, geophysical equipment, and others began the social activities in the evening of May 22, 1974. These companies were most generous in their support of symposium activities and include the following:

Vulcan Materials Company	Brainard-Kilman Drill Company
Acker Drilling Company	Slope Indicator Company
Soil Test, Inc.	Mobile Drilling, Inc.
Law Engineering and Testing Co.	Sprague and Henwood, Inc.
Troxler Electronics Company	Ararat Rock Products
Long-Year Drill Company	Central Mine Equipment Company

A field trip through 3 local geologic areas, including a stop at a nuclear generating site under construction was held on May 23, and also included stops in Coastal Plain, Triassic, and Slate Belt formations. Although heavy rains somewhat curtailed field trip activities, the participants were able to view a very interesting and unusual geologic area. Also during the day, visiting ladies were treated to a tour of historical Raleigh and a luncheon at the Faculty Club of N. C. State University.

The Annual Banquet was held on the evening of May 23, with 150 registrants and guests. The Banquet speaker was Mr. Billy Rose, State Highway Administrator, who spoke on the status of highway transportation and future options pertaining to all types of transportation.

The technical session, including a varied format of papers on engineering geology subjects was opened by Mr. John H. Davis, Chief Engineer of the N. C. Division of Highways, who welcomed all participants to the program.

On behalf of the Symposium Steering Committee, I would like to thank the co-sponsors, Mr. Steven G. Conrad, State Geologist, and Dr. C. J. Leith, Head, Department of Geosciences, N. C. State University for their work and cooperation. Also much appreciation is due to Messrs. W. D. Bingham, H. F. Koch, and J. S. Britt for developing and directing the field trip.

We also wish to thank the more than 250 people who attended the technical session and helped make this one of our most successful meetings.

A. C. Dodson, Chairman
Steering Committee

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AN EVALUATION OF SAND DRAIN INSTALLATION METHODS IN RECENT ALLUVIUM

INTRODUCTION

Ladies and gentlemen, I appreciate the opportunity to be here today. I would like to take this occasion to discuss the work on the proposed project site some of you may have seen on the field trip five years ago when this symposium was in Illinois. As of today the project has been designed, constructed, and tentatively evaluated. So for the next several minutes I want to tell you about the design, construction, and evaluation of vertical sand drain installation methods on a test project on the alluvial floodplain of the Mississippi River near St. Louis.

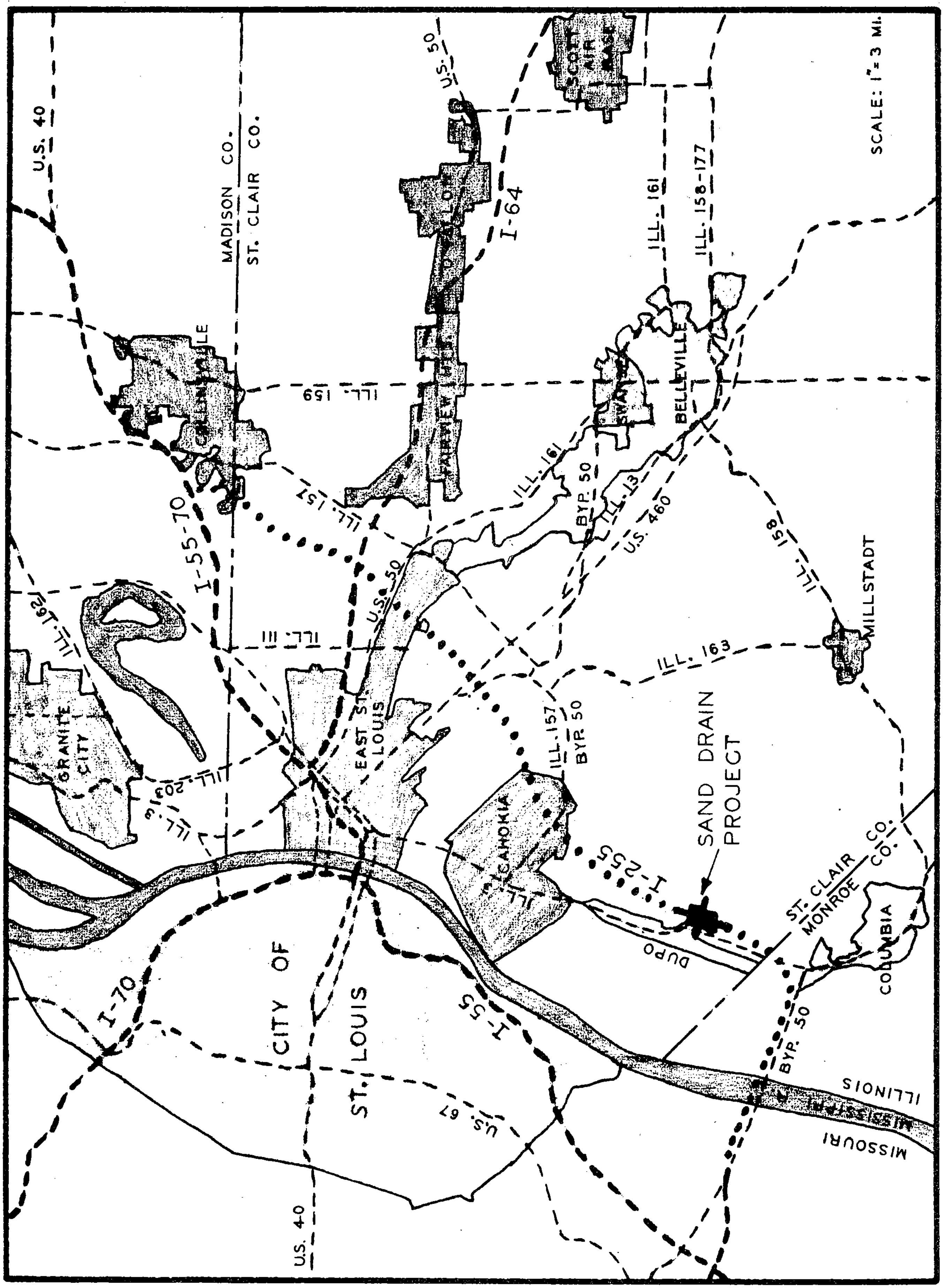
Geological and Physiographical Features

Physiographically, the project is located at the extreme northeasterly edge of the Salem Plateau Section of the Ozark Plateaus Province. Geographically, the project is located at the easterly edge of the alluvial valley of the Mississippi River locally known as the American Bottoms. The loess covered limestone bluffs which mark the easterly limit of the American Bottoms lie just a few hundred yards east of the project.

Figure 1 shows the geographical situation of the project in relation to the region. This project is adjacent to Dupo, Illinois, in southwestern St. Clair County. The city of St. Louis, Missouri, is 8 mi northwesterly of this location.

The topography within the limits of the project is nearly level with a depressional swampy area near the interchange which

FIG. 1. GEOGRAPHICAL LOCATION OF PROJECT



stands water a major portion of the year. The major portion of the project alignment traverses an area which is not under cultivation and displays old abandoned oil wells. Surface drainage generally is poor as is the subsurface drainage.

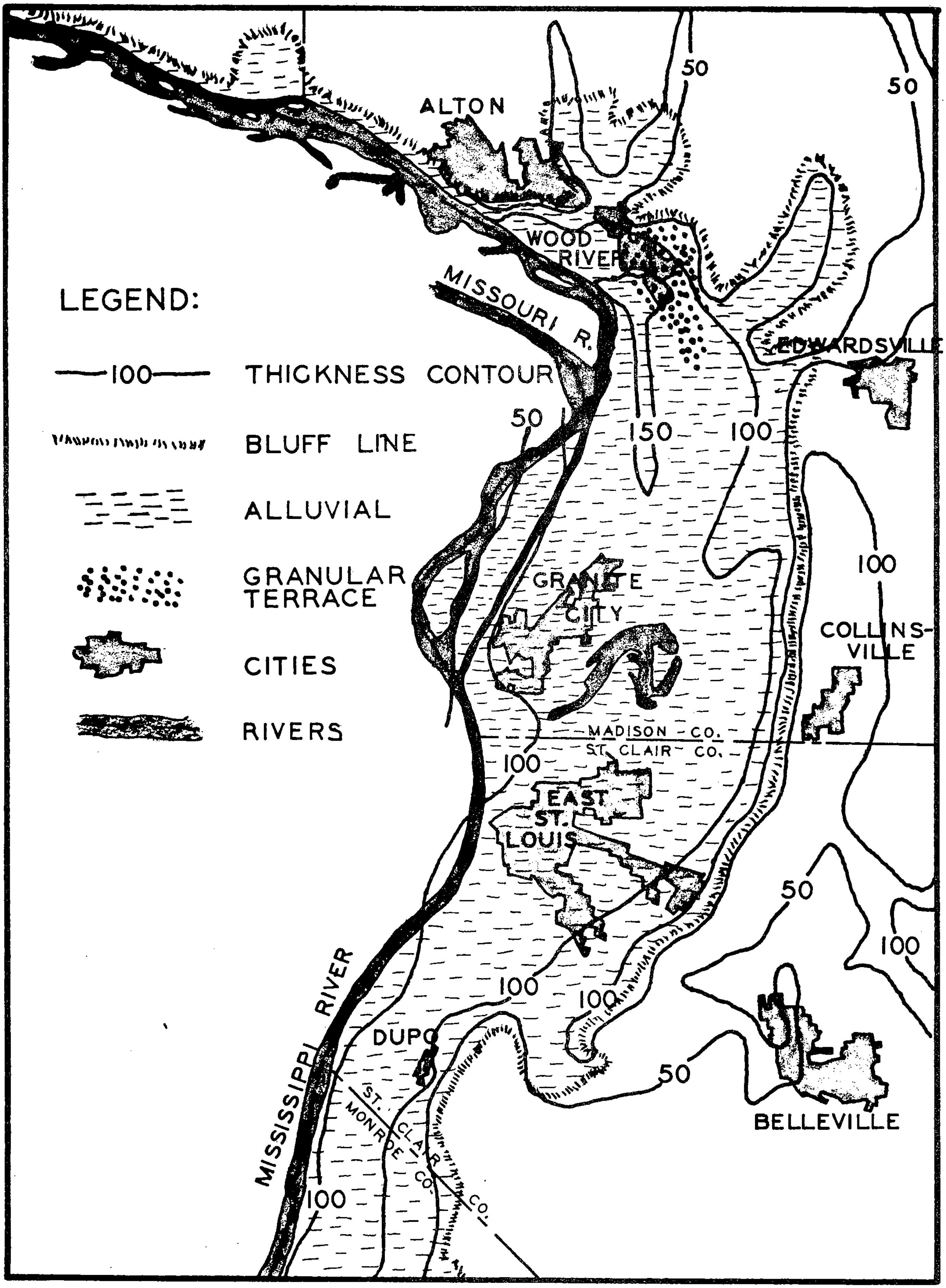
Figure 2 depicts a sketch of the surficial geology showing the thickness of the unconsolidated deposits of the general area. Convergence of the Missouri and Mississippi Rivers occurs about 5 mi downstream from Alton. Together these rivers have produced the bedrock valley and deposited the glacial outwash and the more recent alluvial sediments that partially fill the valley. The valley fill averages about 100 ft thick consisting of an upper unit composed primarily of clay, silt, and sandy silt with a lower unit of sand and gravels.

History of Structures in the Project Area

A limestone quarry is situated near the project site at the edge of the nearby bluffs. The crushed stone stockpiles which the company has built are underlain by the swampy subsoils. The entire working platform which supports the stockpiles has settled into the subsoil several feet with a foundation failure occurring beneath the ag lime pile. Officials of the company estimate that the entrance road to the quarry from nearby Illinois Route 3 has settled in excess of 10 ft within the last 15 years.

Residents of Dupo, located adjacent to the project, relate that the embankment for Route 3 has settled differentially to an appreciable degree since it was constructed 40 yrs ago. Even though the embankment for this roadway is 5 ft or less in height, it is evident from inspection that differential settlement has

FIG. 2. SURFICIAL GEOLOGY



occurred. The amount of settlement, however, is of unknown magnitude since the available records do not show the thicknesses of several previous patches and overlays.

Roadway Features

The location on which the subject project was built is part of an interchange area for proposed Interstate Route 255 and relocated Illinois Route 3. The mainline facility at this location consists of a controlled access, divided highway meeting the minimum design standards for interstate highway service as a portion of Interstate Route 255. Figure 3 shows the geometrics of the facility in the immediate area of the subject project.

At this location there is a relocation of Illinois Route 3 for a distance of 6500 ft. It is on this relocation that the subject test project is shown in the figure. At the site of the project, Route 3 consists of two lanes at 16 ft. each separated by a barrier median.

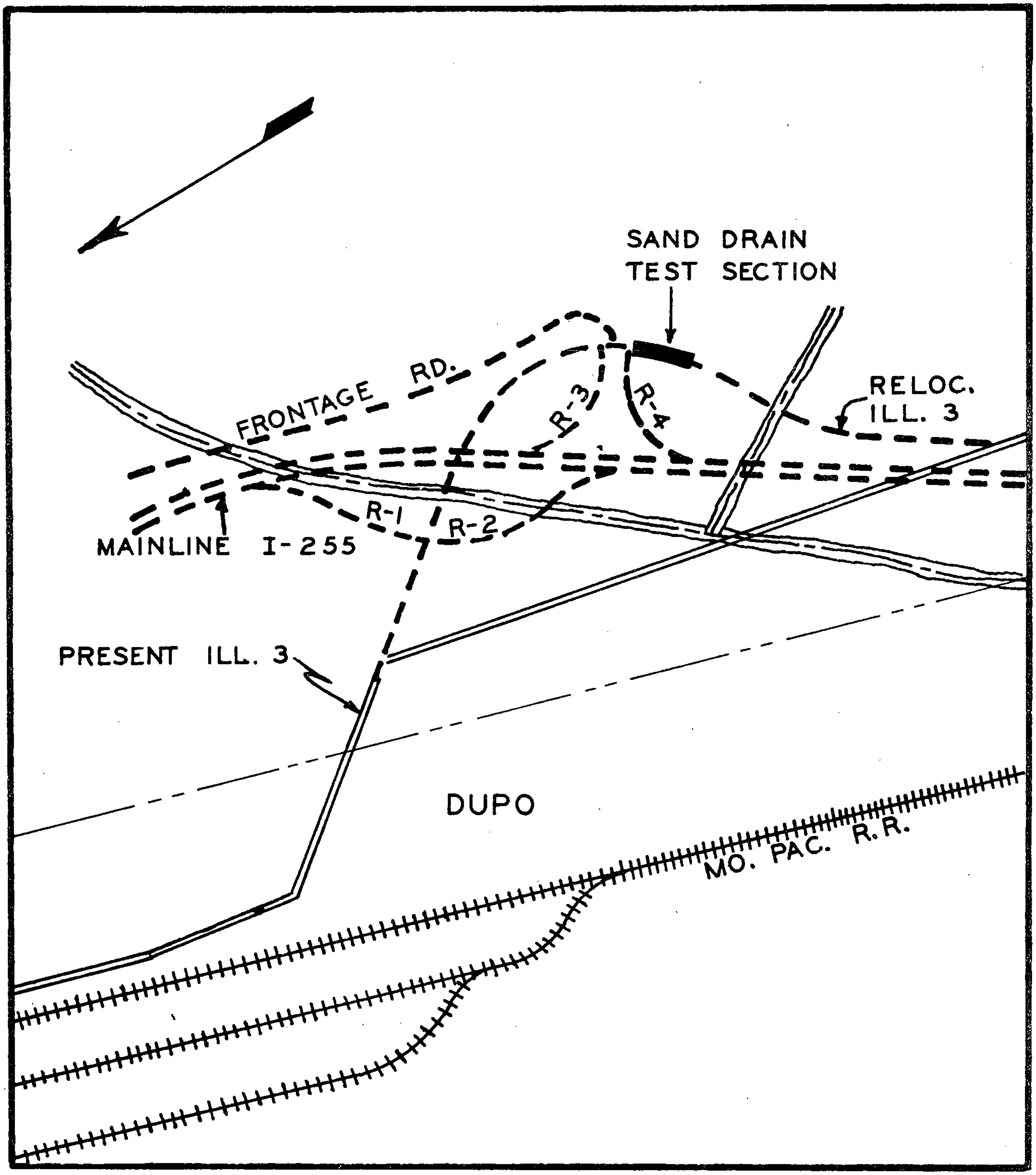
DESIGN AND PRELIMINARY ENGINEERING

The field investigations, laboratory testing, design considerations for the foundation problems, and analyses for the overall alignment took place over a period of several years. The comments to be made for this aspect apply not only to the test section but also for that portion of the mainline traversing the adjacent area.

Soils Investigations

Several types of soils investigations were made during the

FIG. 3. LAYOUT SKETCH OF IMPROVEMENT



preliminary engineering phase. The field investigations included hand auger borings, swamp probes, split spoon borings, shelby tube borings, and vane shear borings. From the samples obtained selections were made for over a dozen different lab tests during the investigation period.

Field Investigations. During the normal soil survey in Illinois, subsurface investigations for roadways consist of hand auger borings every 300 ft in fill sections and 100 ft in cut sections. This type of boring permits visual field identification plus provides samples for routine classification tests. In addition, it can be determined in the field, from the hand auger borings, if additional subsurface exploration is desirable. In this case the auger borings revealed the need for such additional information.

Auger borings and swamp probes were made extensively to delineate the areas of severe foundation problems. The 2-in. auger borings were primarily utilized to delineate the subsoil profile on the basis of the HRB classification system. Disturbed samples were obtained for the purpose of visual matching and obtaining samples for determining the Atterberg limits. Swamp probes were made with a "Michigan type" sampler which retrieves a relatively undisturbed sample approximately 3/4 in. in diameter. Visual identification of the swamp probe samples in conjunction with moisture content, loss on ignition, and liquid limit determinations assisted in the location and scheduling of split spoon, vane shear, and shelby tube borings.

Shelby tube borings were made for the purpose of obtaining "undisturbed" samples for unit weight determinations, unconfined compressive strength tests, triaxial tests, and consolidation tests. 2-in. shelby tubes were utilized with continuous sampling for obtaining samples for the previously mentioned tests. At several locations intermittent 3-in. shelby tube samples were obtained to provide better specimens for consolidation and triaxial tests. The shelby tube borings were located in such a manner as to yield good coverage of the area relative to consolidation characteristics of the subsoil.

Split spoon borings were made along the roadway primarily for additional soil classification information and to assist in the determination of sand elevations. In addition, some of the split spoon borings were carried deeper into the sand to probe for buried clay pockets. Sampling was at 18-in. intervals with standard penetration tests performed at each sample location. In addition, field unconfined compression tests were made on cohesive samples.

Vane shear borings were made with a Swedish vane shear tester to determine in-situ shear strengths of the clays. A comparison was made to compare in-situ shear strengths with the unconfined compressive strengths. The sensitivity of the soil was checked by comparing undisturbed shear strengths with remolded shear strengths. The great majority of the sensitivity ratios ranged from 1.5 to 3.5 with an average value about 2.7 which would classify this clay deposit as insensitive.

Subsurface conditions. The subsurface conditions encountered on this project are basically uniform in that the soil profile consists generally of a soft clay (15 to 35 ft in depth) underlain by deep sand which grades from fine and very fine to coarse at greater depths. In the depressional areas the thickness of the soft clay layer averages from 18 to 24 ft with the entire layer possessing an unconfined compressive strength ranging from 0.1 to 0.4 tsf. On the other hand, in the areas where there is some degree of drainage gradient, the clay layer ranges from 18 to 35 ft in depth with surficial soils possessing higher unconfined compressive strengths than the lower strength subsoils.

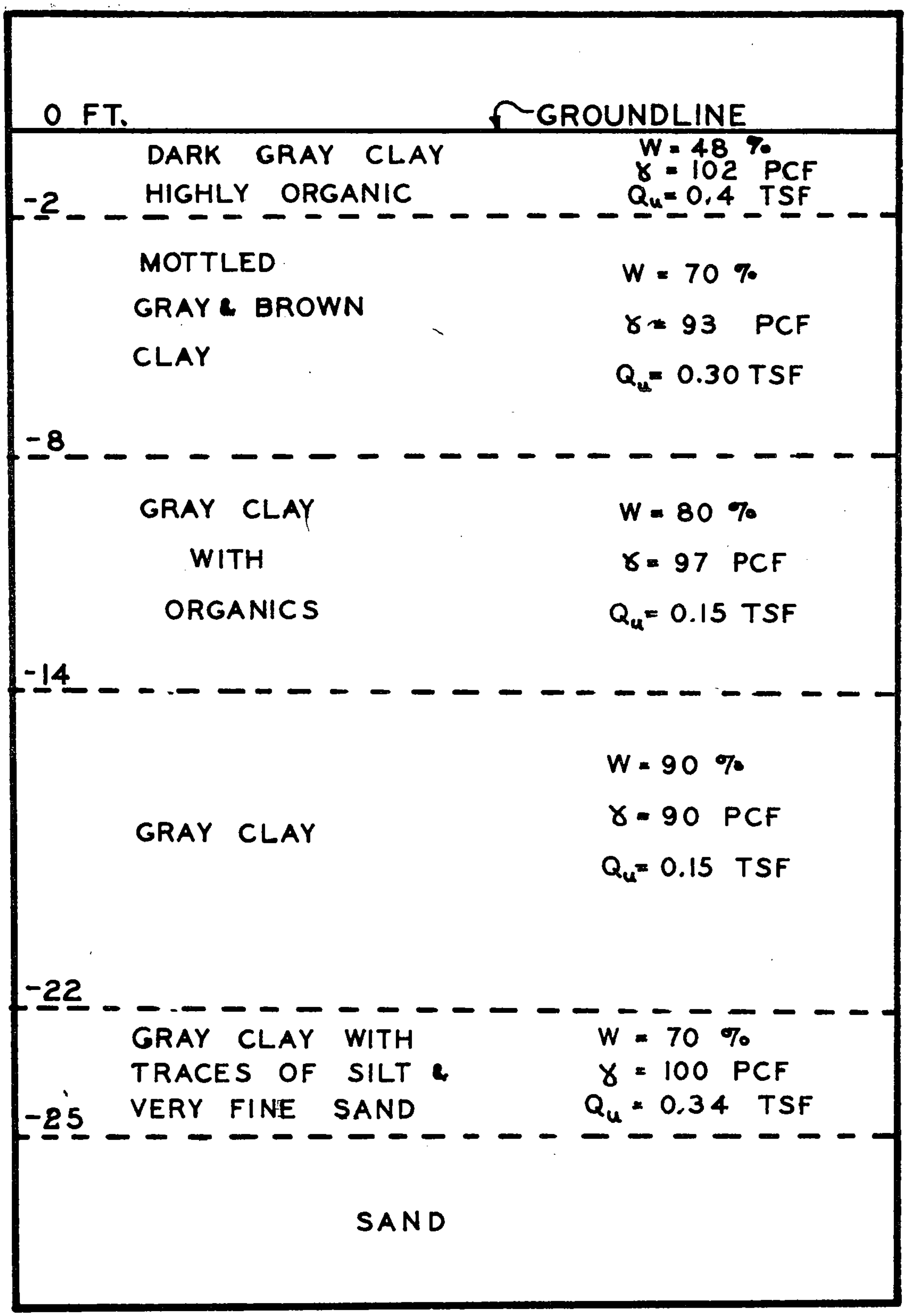
Moisture contents of the subsoils ranged from 30% to over 200% based on the oven dry weight of the sample. There were several samples obtained which were highly organic, some of which were more like peat than soil, that had moisture contents in excess of 100%. The major portion of the soft inorganic clays, however, possessed a natural moisture content in the range of 60 to 90%.

Figure 4 portrays the typical soil profile of the area. It was noticed in the shelby tube samples that some of the samples had numerous slickensides. In others there appeared to occur a "blocky" structure in the sample. The clay layer encountered is basically a continuous layer with very little or no interbedding with non-cohesive layers.

To aid in the estimation of the depth of the clay layer at any given location, a contour map of the top of sand elevation was prepared. This map, which is shown in Figure 5, was prepared from a grid system constructed by using the data from the various borings. This contour map was later used in construction to estimate the depth of the sand drains. Note the variability of the top of sand elevation in this figure.

<u>Laboratory Testing</u>. Once the soils profile was generally

FIG. 4. TYPICAL SOILS PROFILE .



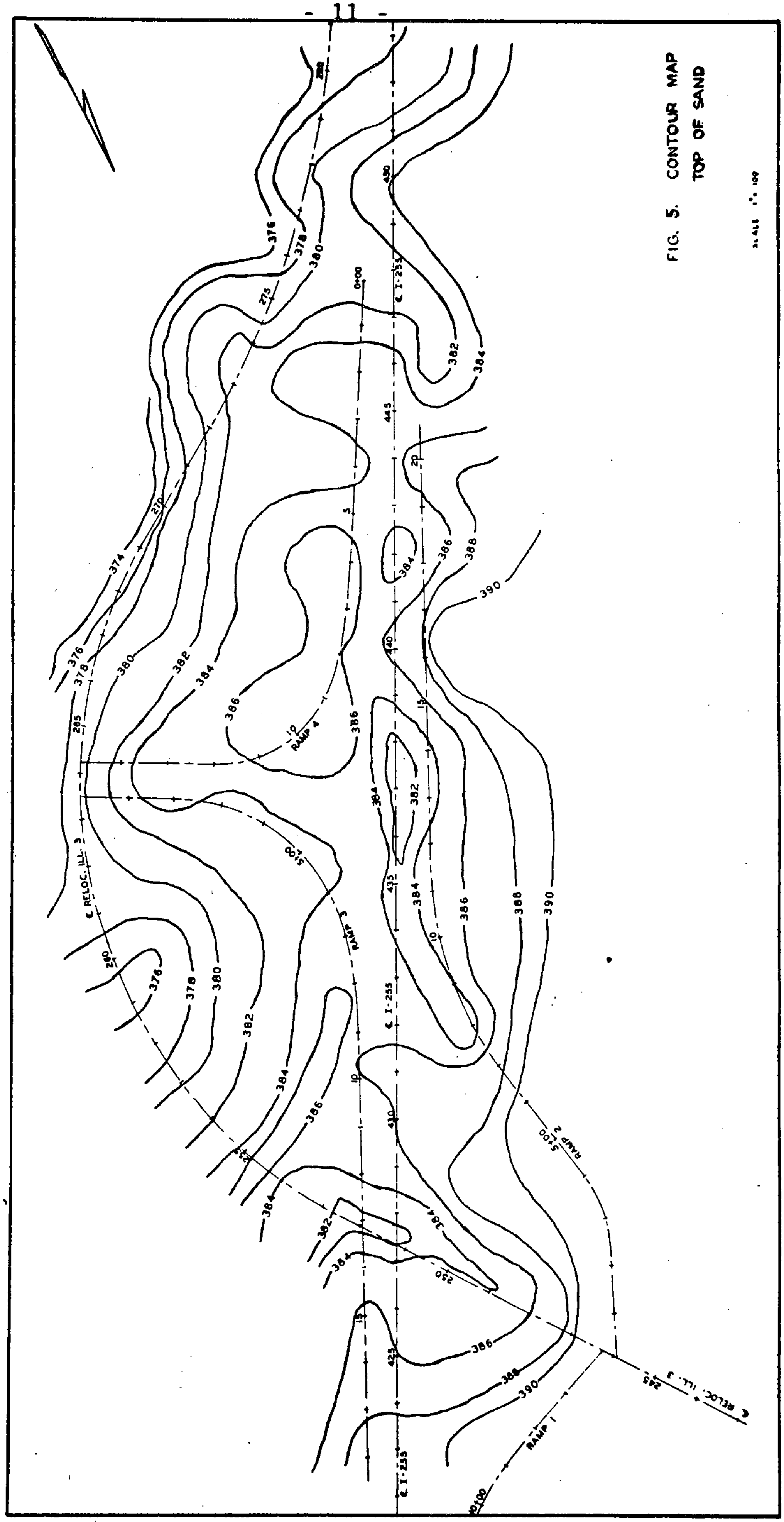


FIG. 5. CONTOUR MAP
TOP OF SAND

established by the hand auger borings and swamp probes, the basic classification tests of moisture contents, liquid limits, plastic limits, and specific gravities were performed. After the completion of these tests it became evident the most important characteristics of the subsoils on the project were the shear strength and consolidation parameters.

Numerous shelby tube samples were taken for the purpose of determining those parameters along with the natural moisture contents and unit weights of the soil. Table 1 shows a summary of the number and types of tests performed for this project.

TYPE OF TEST	NO. PERFORMED
Consolidation	80
Unconfined Compressive Strength	746
Triaxial	18
Liquid Limit	197
Plastic Limit	110
Natural Water Content	1867
Unit Weights	897
Specific Gravity	46
Loss on Ignition	32
Grain Size Analysis	30

Table 1. Summary of Laboratory Tests Performed.

<u>Analysis of Data</u>

Based upon the laboratory testing it was quite evident that both differential settlement and foundation stability posed severe problems on this project. Faced with this situation, analyses were made to determine just what the extent of the settlement and stability problems were.

Settlement Analysis. Consolidation tests were performed on samples from 52 boring locations. In determining sites from which consolidation samples were taken, three factors were considered --- height of fill, depth of the clay layer, and moisture content.

Test results of representative consolidation tests are shown in Table 2 with each test identified by a boring number, station, and reference to the roadway centerline. In this table columns six and seven show the amount of primary settlement and the time required for 90% of that settlement to occur at the given fill height, respectively. Column eight shows the amount of secondary settlement which was predicted from the slope of the secondary compression tangent. Column nine shows the total amount of primary settlement which would be produced by a load equal to the fill height plus a 5-ft surcharge. Column 10 shows the amount of time required to produce 90% of the settlement shown in column six by a load equal to the fill height plus 5 ft of surcharge.

As typical information, Figure 6 shows the time-settlement curves for a consolidation test on boring 40 ST. The log fitting method was used to plot all the test curves with the loading increment doubled for each subsequent curve after the initial increment.

All settlement times were computed by the utilization of the coefficient of consolidation (c_v) curves shown with the void ratio-log pressure curves in Figure 7. Values of c_v used to make the plot for these curves were obtained by solving

(1) Boring No.	(2) Station	(3) Ref. to R'dway C.L.	(4) Fill Height (ft.)	(5) Thickness Clay Layer (ft.)	(6) Primary Settlement (in.)	(7) Primary t_{90} (yrs.)	(8) Secondary Settlement (in.)	5-ft. Surcharge	
								(9) Settlement (in.)	(10) Time t_{90} (mo.)
7ST	422+00	C.L.	6.8	14	6.1	15.2	0.1	11.7	47.0
11ST	430+00	C.L.	6.1	14	7.2	1.7	1.0	12.5	7.7
13ST	436+00	15' Rt.	7.5	19	13.4	29.1	2.0	20.8	110.0
16ST	440+00	50' Lt.	8.8	17	7.9	2.1	1.1	13.0	12.0
21ST	447+00	45' Rt.	7.5	18	21.7	13.4	1.6	34.8	41.0
23ST	448+00	C.L.	7.3	18.5	27.4	13.7	5.7	42.6	42.0
25ST	449+00	6' Rt.	5.8	17	7.3	0.08	1.1	15.2	4.9
38ST	258+40	C.L.	10.8	22.0	21.2	10.8	3.6	27.2	67.0
39ST	267+00	7' Rt.	10.2	27.5	36.1	17.0	6.4	49.9	108.0
40ST	267+00	12' Rt.	10.2	25.5	41.9	42.3	9.6	56.7	247.0
41ST	268+65	C.L.	9.0	23	21.6	20.0	4.5	30.8	98.0
42ST	273+50	8' Lt.	8.0	17	7.5	3.2	1.8	12.2	13.0
43ST	274+60	C.L.	7.3	20.5	16.3	12.2	0.7	24.6	49.0
44ST	275+49	2' Lt.	6.8	22	8.9	1.8	1.9	16.8	8.7
45ST	277+67	C.L.	6.5	21	6.6	2.4	1.3	11.3	8.8

Table 2. Summary of Settlement Time and Magnitudes of Settlement from Representative Consolidation Tests.

FIG. 6. TIME-SETTLEMENT CURVE: BORING 40 ST

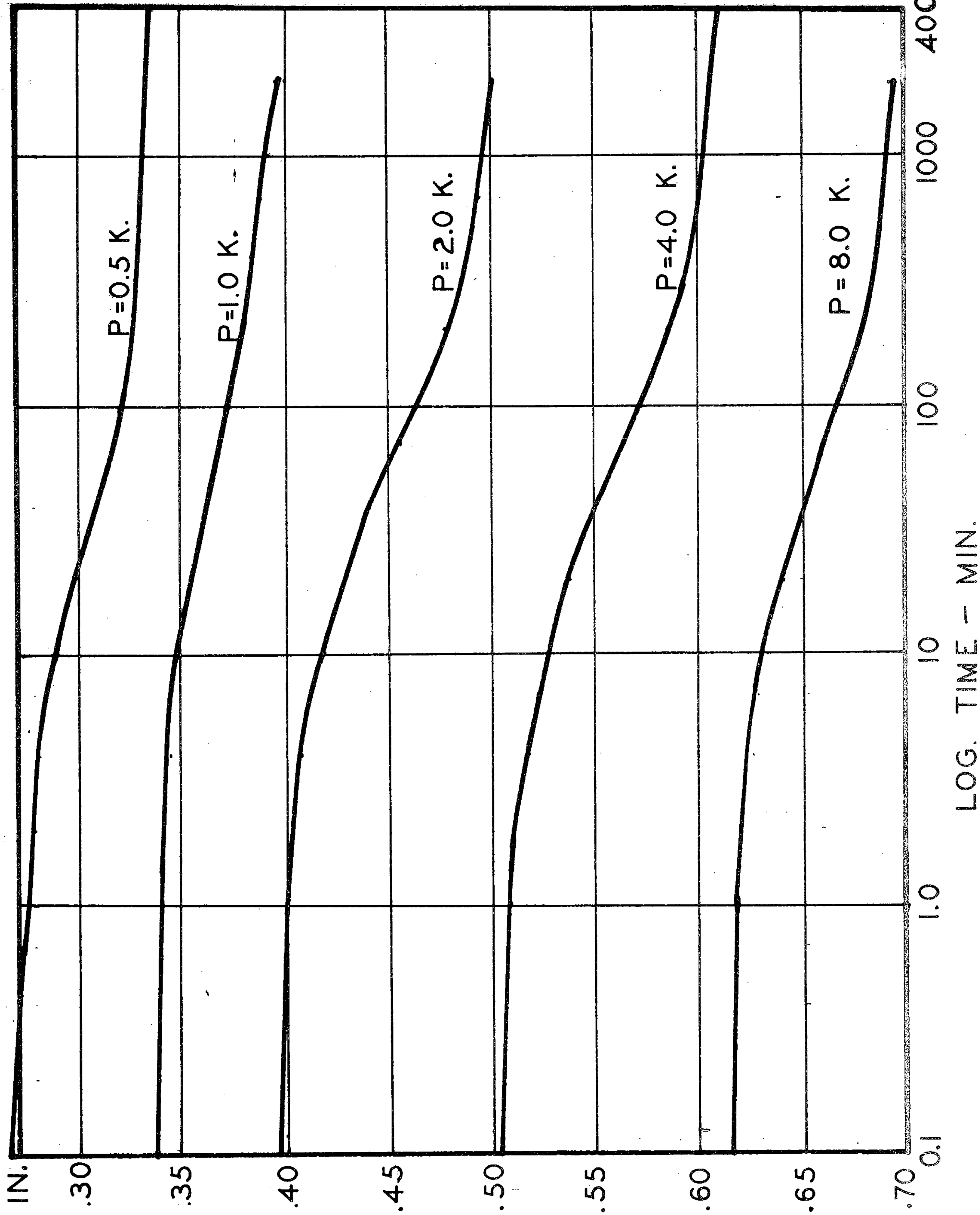
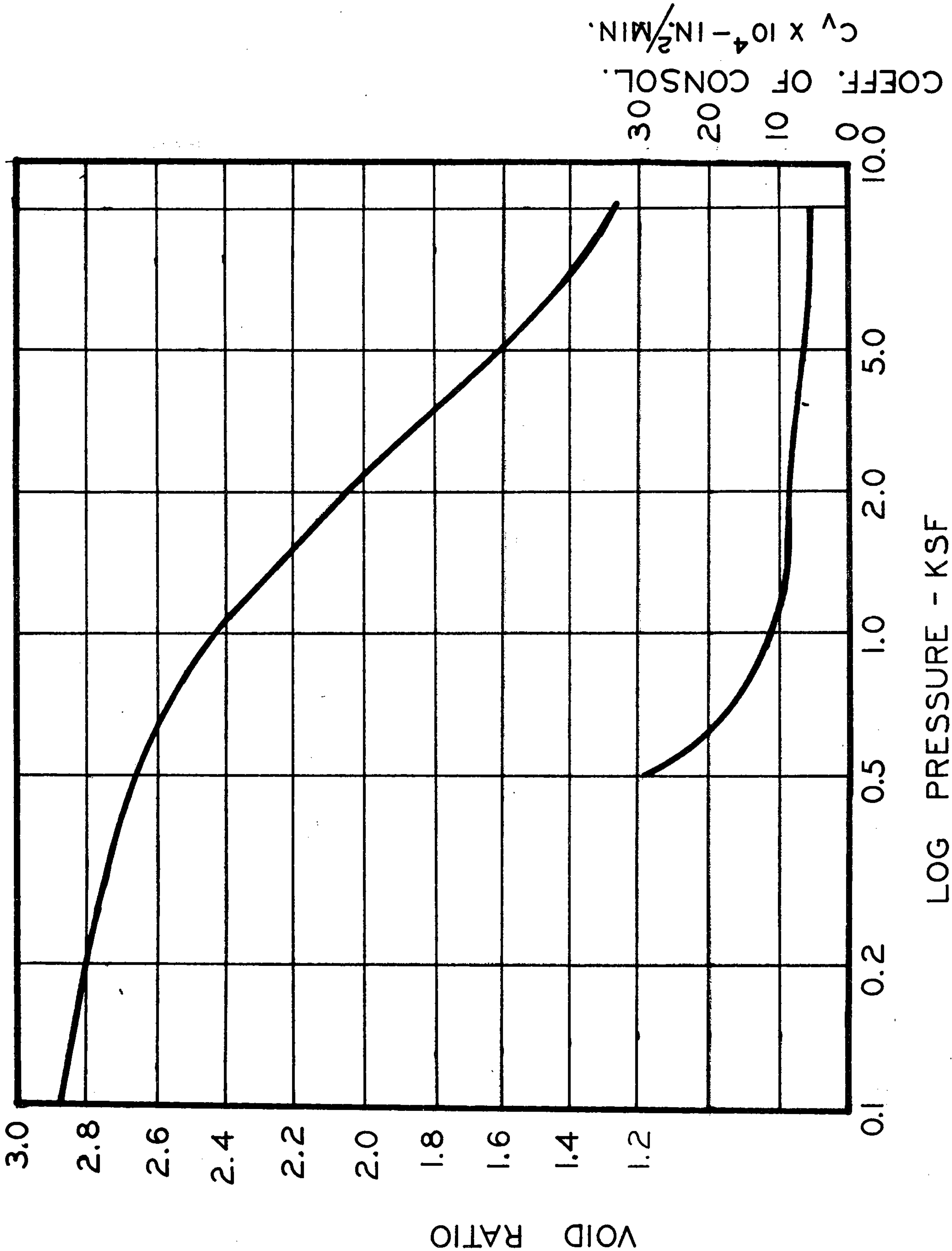


FIG. 7. VOID RATIO CURVE: BORING 40 ST



the formula $c_v = \frac{H^2 T_v}{t}$ where

T_v = time factor

H = half thickness of the consolidating layer

t = the laboratory time for 50% consolidation

The coefficient of consolidation in all cases was calculated as an average value corresponding to that existing at 50% consolidation and plotted at the pressure for that load increment. The predicted field times were computed from the above formula by using the appropriate c_v value from the curves and solving $t = \frac{H^2 T_v}{c_v}$ where H in this case is the half thickness (double drainage) of the consolidating layer in the field.

Table 3 shows a summary of a comparison between the horizontal and vertical coefficients of consolidation for eight locations.

BORING NO.	STATION	COEFFICIENTS OF CONSOLIDATION		C_h/C_v
		VERTICAL	HORIZONTAL	
8ST	442+00	0.0020 0.0052	0.0036 0.0047	1.8 0.9
9ST	422+00	0.0038	0.0071	1.9
11ST	430+00	0.0036	0.0099	2.8
14ST	437+05	0.0048 0.0020	0.0059 0.0025	1.2 1.3
21ST	447+00	0.0014	0.0014	1.0
24ST	448+00	0.0036	0.0051	1.4
44ST	275+49	0.0057 0.0028	0.0050 0.0038	0.9 1.4
53ST	12+00	0.0018	0.0024	1.3

Table 3. Comparison of Vertical and Horizontal Coefficients of Consolidation.

The vertical coefficients were determined in the manner just described for normal consolidation tests. Horizontal coefficients were calculated similarly from consolidation tests performed on shelby tube samples which were rotated 90° from their natural position. Basically, the ratio of the horizontal coefficient to the vertical coefficient averages 1.5 which tends to be rather conservative. The reason for this conservatism obviously lies in the fact that there isn't any difference in the test results if the sample doesn't contain a stratification lense.

Evaluation of the consolidation test results basically becomes a matter of comparing individual tests for magnitude and time. Consideration was given not only to the differential in magnitude of settlement but also to the differential in consolidation time. Examples of this is evident for the first two tests shown in Table 2. While the magnitudes are similar (6.1 in. vs. 7.2 in.) there is appreciable difference in the periods of time to achieve 90% primary consolidation (15.2 yrs. vs. 1.7 yrs.). If such tests as these are indeed indicative of field results, the riding performance of the roadway would be significantly influenced during most of the 20-yr. design life of the pavement surface.

For the purpose of visual presentation of the problem of differential settlement (magnitude only), Figures 8A and 8B show the proposed pavement profile as compared to the profile as it would exist after completion of primary consolidation. Figure 8A shows the profile for mainline I-255 while Figure 8B shows the profile for relocated Illinois 3. Based upon an

FIG. 8A. DIFFERENTIAL SETTLEMENT: I-255

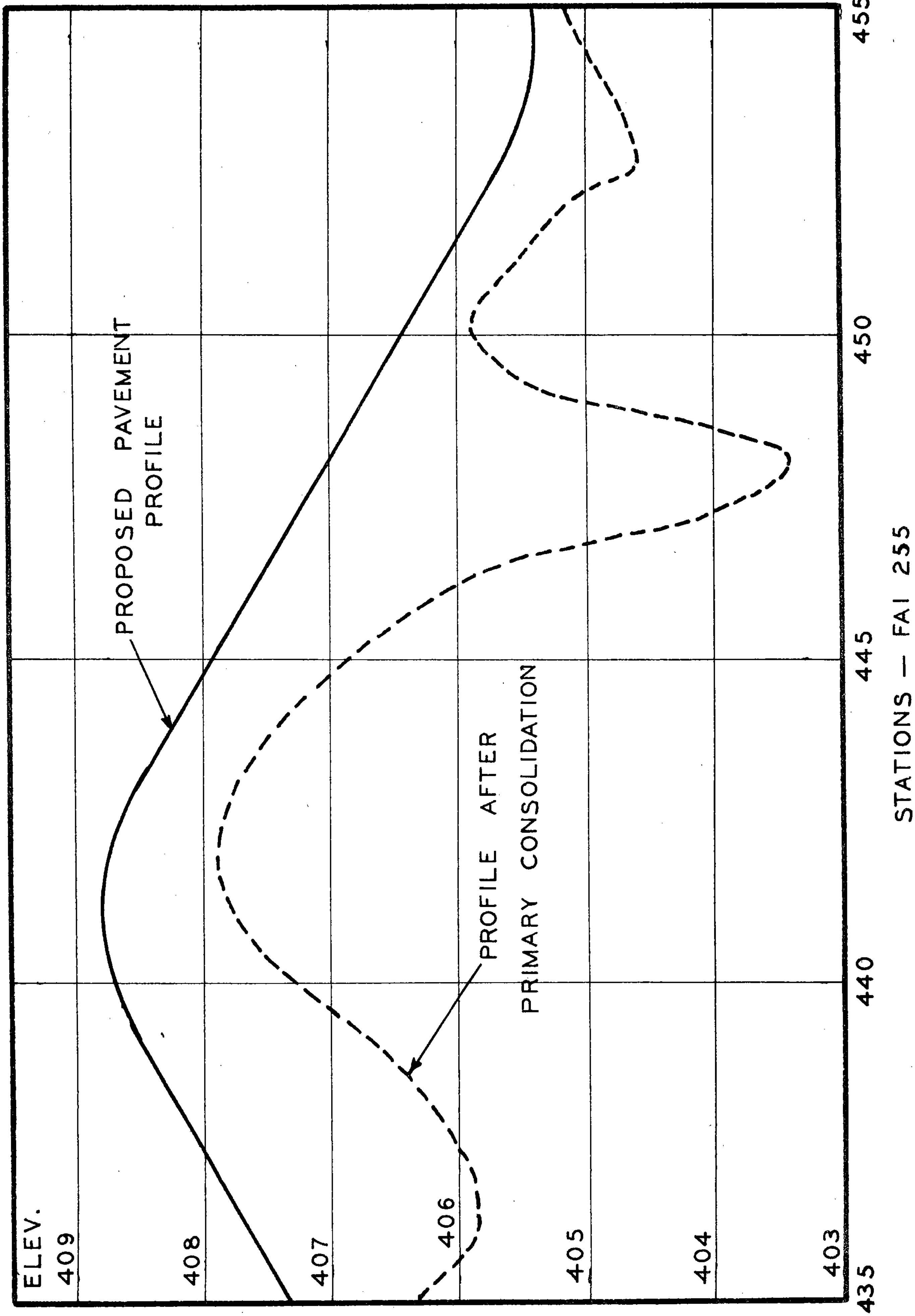
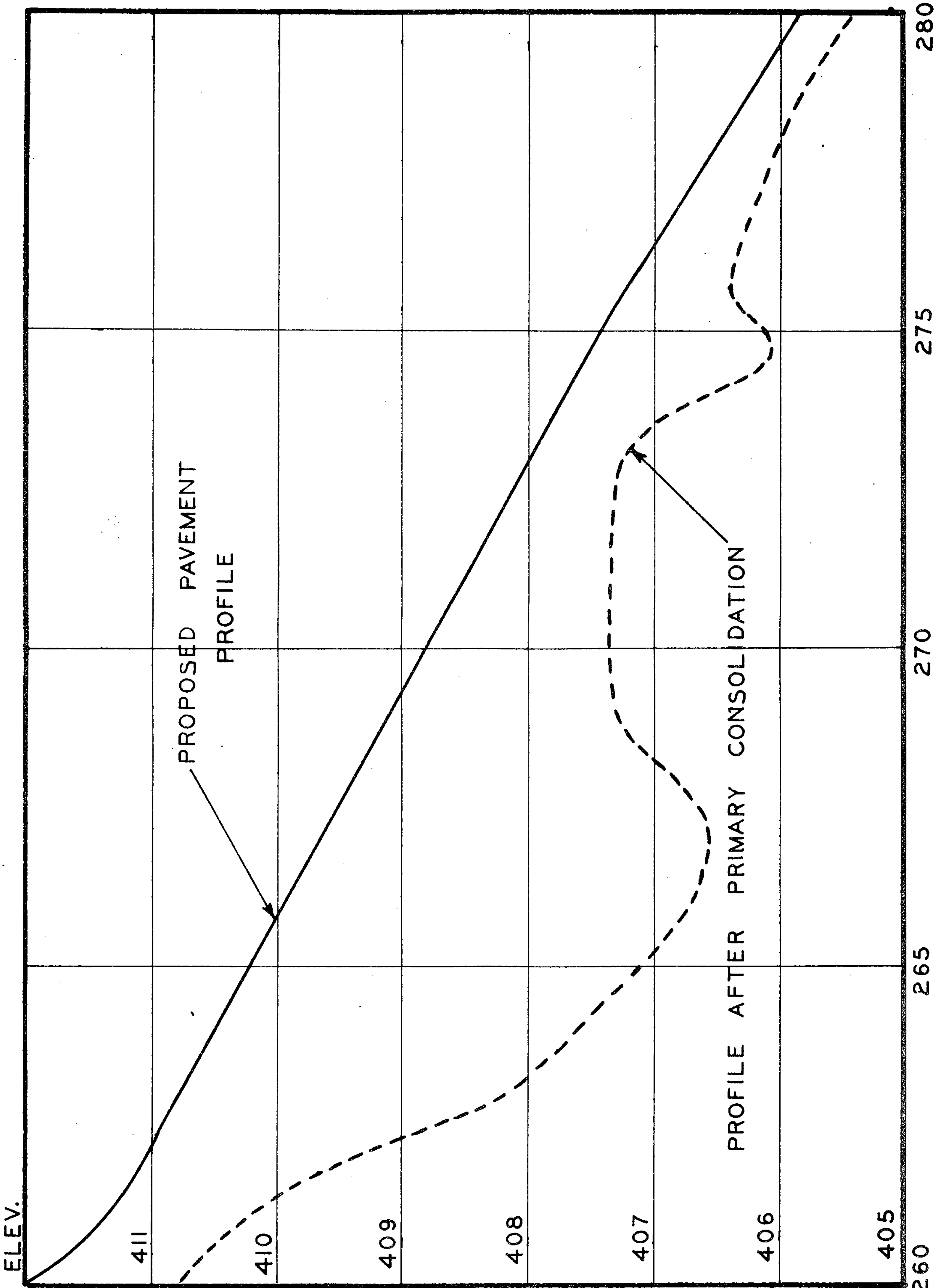


FIG. 8B. DIFFERENTIAL SETTLEMENT : ILL. 3



examination of all the consolidation test results it was determined that differential settlement was a serious problem over much of the proposed alignment. From this decision we proceeded to investigate stability problems to determine if possible treatments for settlement would be limited by the stability aspect.

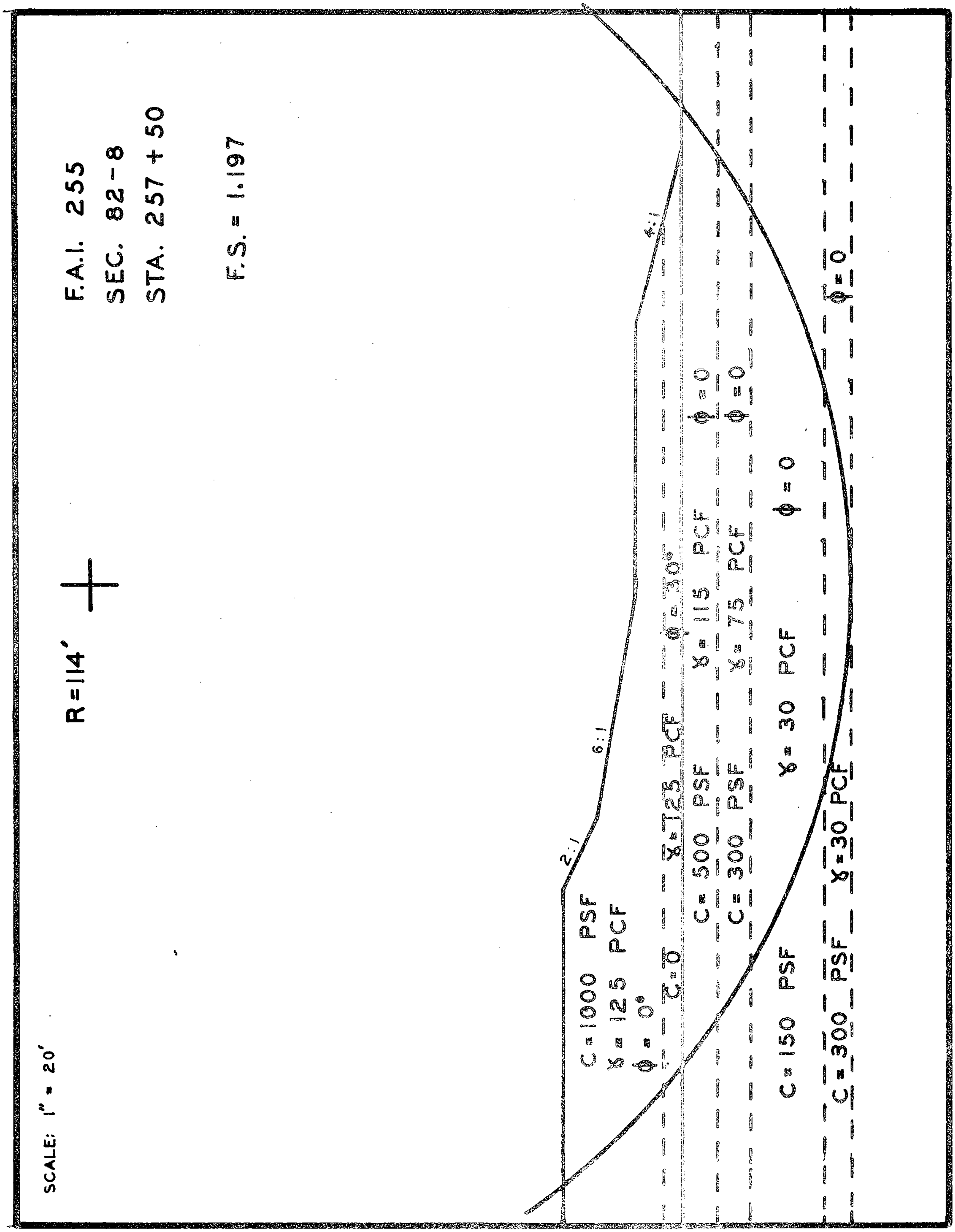
Stability Analyses. The various foundation boring logs were examined to determine, in conjunction with proposed fill heights, which areas were potentially critical in so far as the ability of the subsoil to support the loading was concerned. Several slip circle and sliding wedge analyses were made.

Slip circle analyses were performed at 24 locations in the area of the proposed project. The centers of the critical circles were searched for on a 5-ft grid system by the computer. Figure 9 shows the graphical analysis of one of the slip circles. This circle shows the embankment with a 5-ft surcharge and a 3-ft thick sand blanket as part of the cross section.

Normally, the State of Illinois considers 1.30 as a minimum acceptable factor of safety. In this particular instance it was decided that it would be more practical to use 1.20 as the minimum factor of safety and that added precautions should be taken during construction. The computer program utilized, is set up to search for the minimum width of berm in 5-ft increments to provide an adequate factor of safety if the initial run utilizing the standard cross section is inadequate. In this example a 40-ft berm was required for F.S. = 1.20.

Sliding wedge analyses were performed at several locations since we have found that often when the thickness of the weak

FIG. 9. SLIP CIRCLE GRAPHICAL ANALYSIS



subsoil layer is relatively thin, i.e., 10 to 30 ft thick, the danger of a spread type failure is imminent. The method used to make these computations was taken from the "NAVFAC Manual DM-7" which is described therein as the analysis of transitional failures.

This method of analysis considers the active and passive forces involved in the translation of a wedge of soft foundation soil due to the embankment load on a relatively firm base underlying the softer material. In this case the top of the sand layer is the firm base. Figure 10 shows a typical wedge analysis for Station 256+00.

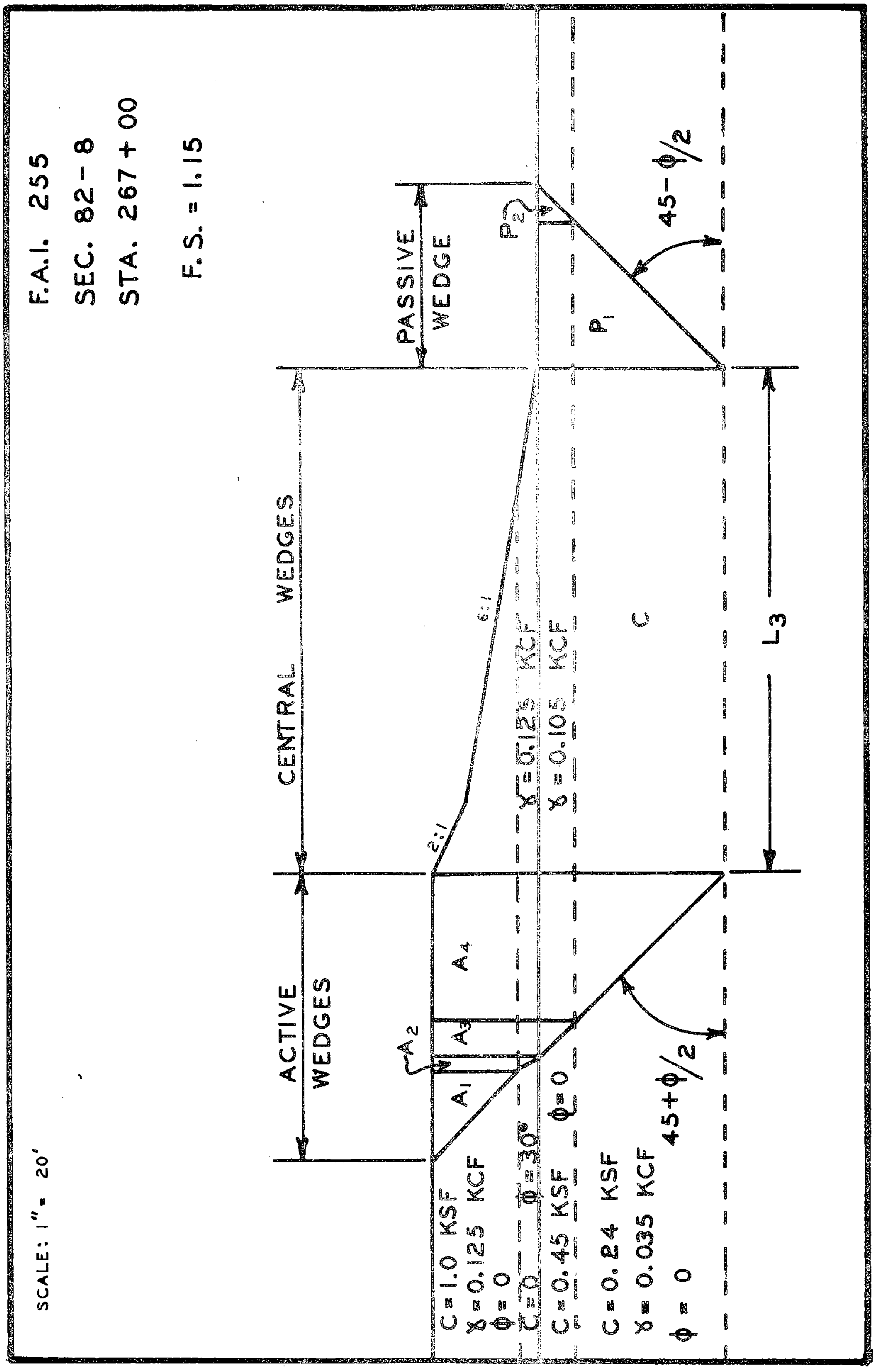
Table 4 shows a tabulation of the factors of safety resulting from both the sliding wedge and slip circle analyses. In the necessary instances, the required berm width to produce a factor of safety of 1.20 is shown.

Alternative Treatments

Upon deciding that the predicted differential settlements were too great to tolerate, various methods of subsoil treatment were investigated. The various possible treatments considered were:

1. Removal and replacement of the unsuitable subsoil.
2. Partial removal and replacement of the subsoil with the employment of structures.
3. Use of lightweight granulated slag for the embankment.
4. Installation of vertical sand drains and a surcharge.

FIG. 10. SLIDING WEDGE ANALYSIS



STATION	ROADWAY	SLIP CIRCLE F.S.	SLIP CIRCLE BERM REQ'D.	SLIDING WEDGE F.S.	SLIDING WEDGE BERM REQ'D.
416+00	I-255	---		2.77	
420+10	"	1.915			
430+00	"	1.388			
431+00	"			1.15	10
437+00	"	1.158	10	1.58	
440+00	"	1.519		1.69	
446+00	"	1.544		1.80	
448+00	"	1.395			
450+00	"	1.466			
452+20	"	1.558			
468+00	"	1.499			
471+00	"		140		
473+00	"		80		
248+00	Rte. 3	1.560			
249+00	"	1.068	35		
250+18	"	1.082	15		
252+33	"	0.819			
255+00	"	1.073	20	1.12	25
255+50	"	0.984	65	0.94	165
257+50	"	1.197		1.00	120
258+40	"	1.088	20	1.15	30
259+00	"	1.097	20	1.17	20
267+00	"	1.180	20	1.10	35
273+50	"	1.362		1.42	
274+60	"	1.239		1.22	

Table 4. Tabulation of Factors of Safety for Stability Analyses.

Granulated Slag. Computations for the magnitude of settlement and the time required were made substituting granulated slag at 90 pcf for regular embankment at 125 pcf. These computations were made for the conditions of:

1. All granulated slag embankment.
2. Slag embankment with 5 ft removal of the subsoil.
3. Slag embankment with 10 ft removal of the subsoil.

It was found that under all these conditions, differential settlement was still a significant factor. With the best condition of 10 ft of removal of the subsoil, the magnitude was reduced approximately 50% but the time to achieve 90% consolidation was still on the order of 5 to 10 yrs. As a result this alternative was not given any further consideration.

Removal and Replacement. One rather positive method of foundation treatment is the total removal and replacement of the unsuitable subsoil. Figure 11 represents a sketch of the typical embankment section used for the cost computation of the treatment.

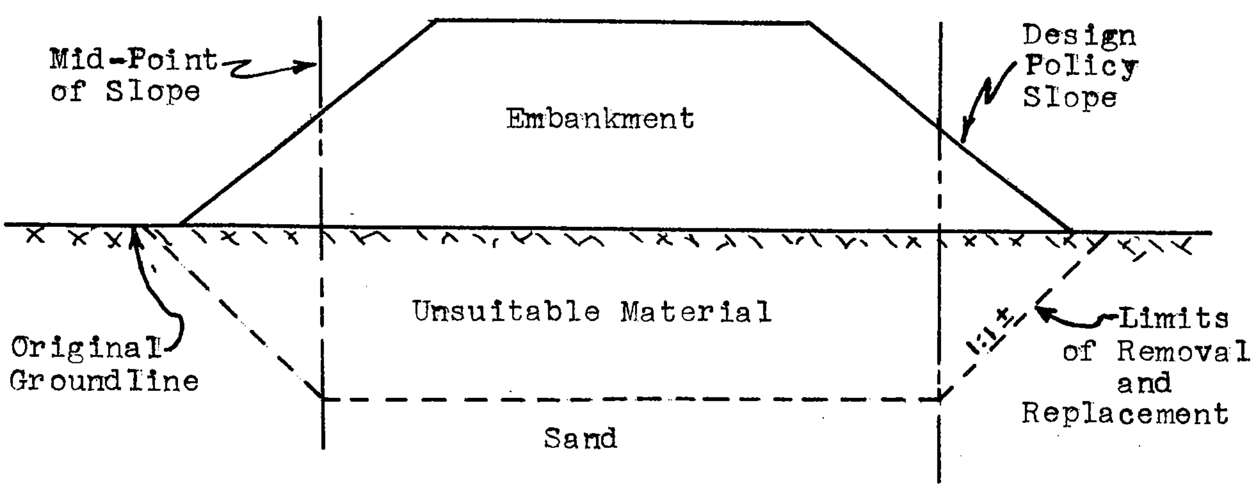


Figure 11. Removal and Replacement Cross Section.

At this particular location a large quarry is adjacent to the project. It was anticipated that waste caprock would be utilized for backfilling the unsuitable excavation with soil overburden utilized as embankment material. For the total project this treatment was estimated to cost $8,355,000.

Combination of Removal and Replacement and Structures. The third alternative investigated consisted of employing removal and replacement at the intersections of the interchange ramps with Route 3 and at locations where the depth of unsuitable material did not exceed 10 ft. The remainder of the roadways over unsuitable material would be placed on continuous structure. The total cost for this alternative was estimated to be $11,541,000.

Vertical Sand Drains. The final method of foundation treatment investigated was the use of vertical sand drains with the use of a surcharge. For the purpose of cost estimates for the entire project the features in Table 5 were assumed relative to the sand drain length and spacing. In addition, it was assumed that a

Roadway	Station Limits	Drain Length	Drain Spacing	Settlement Time
I-255	413 to 420	12 ft	7 ft	9 mo
"	420 to 425	12 ft	12 ft	8 mo
"	425 to 438	17 ft	7 ft	10 mo
"	438 to 473	28 ft	12 ft	7 mo
Rte. 3	247 to 294	20 ft	12 ft	7 mo
Ramps	2, 3, & 4	20 ft	12 ft	8 mo

Table 5. Physical Features of Sand Drains for Cost Evaluation.

5-ft surcharge and a 3-ft sand blanket would be utilized with 18-in. diameter closed-mandrel sand drains. This alternative was estimated to cost $4,164,000.

Sand Drain Design

From the results of the cost comparison of the various alternatives just discussed, it was evident that the use of vertical sand drains was the most feasible method of overcoming the problem of differential settlement. In addition, at unstable locations, berms were selected as a feasible method of overcoming stability problems. The preliminary investigations and the study of alternate treatments were all presented in a preliminary soils report. From this point a refinement of the sand drain design and installation was pursued. In the next few moments the purpose, design procedures, and engineering considerations involved in the selection of the sand drain system will be discussed.

Theoretical Considerations. The two principal elements in the design of a vertical sand drain system are, 1) the determination of the magnitude and time rate of volume change of the compressible soil which is to be loaded, and 2) the determination of the stability of the foundation soil with the superposed fills during construction. Both of these elements were thoroughly explored on a theoretical basis through consolidation testing and stability analyses.

The magnitude and rate of volume change depend upon the following construction factors:

1) The diameter and spacing of the sand drains.

2) The thickness of the drainage blanket.

3) The rate of fill placement.

4) The amount and duration of surcharge loading.

Sand drain installations effect vertical consolidation through radial flow to the drains. The rate of this radial flow is affected by such factors as smear, drain resistance, horizontal coefficient of consolidation, and drain diameter and spacing.

Smear occurs at the periphery of the drain due to installation procedures with the effect of forming a less permeable zone at the periphery. Moran, Procter, Mueser, and Rutledge have found this smear effect to be insignificant if the ratio of horizontal permeability in the undisturbed region to the horizontal permeability in the smeared zone is equal to or less than 4.0.

The effect of drain resistance has been shown to be insignificant for properly graded sand as long as the ratio of the influence zone of one drain (6 to 12 ft for this project) to the diameter of the drain is equal to or less than 15, and the ratio of the influence of one drain to the height of the clay layer is equal to or less than 1.0.

Drain Spacing Computations. Spacings of the sand drains were determined by trial and error to meet pre-conceived conditions using the coefficients of consolidation obtained from the laboratory consolidation tests. The following relationships were utilized in making the computations:

1) Double drainage would exist in the field---upward into the sand blanket and downward into the underlying sand.
2) The length of the drainage path without sand drains is equal to one-half the thickness of the consolidating layer.
3) The length of the reduced drainage path due to the sand drains is equal to one-half the drain spacing (center to center).
4) The fractional drainage distance is one-half the consolidating layer depth divided into one-half the drain spacing.
5) The fractional time created by the presence of the sand drains is equal to the square of the fractional distance.
6) The total time required to achieve 90% of the primary consolidation with the sand drains and a 5-ft surcharge is equal to t_{90} for the settlement due to the fill height mulitplied by the fractional time.

Table 6 shows a tabulation of the sand drain spacing computations for those locations of representative tests that were shown earlier in Table 2. Those results noted with an asterisk are for the use of sand drains only with no surcharge. In making the computations the governing factors for arriving at the sand drain spacings were:

1) Achieve 90% consolidation within 12 mo.
2) Use 12 ft center to center spacing as the probable maximum.
3) Use 6 ft. center to center spacing as the practical minimum.

It may be seen in this tabulation that only in two instances is the desired maximum time of 12 mo. not met. This wasn't considered to be significant in relation to the total project. In many cases the primary settlement may be expected to be achieved in considerably less than 12 mo.

Station	Thick. Clay Layer (ft)	Drain Spacing (ft)	t_{90} with Surch. (mo)	Lgh Reduced Drainage Path (ft)	Fract. Dist.	Fract. Time	Req'd Total Time (mo)
422+00	14	6	46.8	3	.429	.184	8.6
430+00	14	12	7.7	6	.855	.73	5.6
436+00	19	6	110.4	3	.316	.10	11.0
440+00	17	10	13.2	5	.588	.346	4.56
447+00	18	8	40.8	4	.444	.197	8.05
448+00	18.5	8	42.0	4	.432	.187	7.85
449+00	17	12	4.9	6	.706	.498	2.44
258+40	22	6	67.2	3	.272	.074	5.0
267+00	25.5	6	247.0	3	.235	.055	12.9
268+65	23	6	985.0	3	.261	.068	67.0
273+50	17	10	1.1	5	.588	.346	0.4
274+60	20.5	10	49.0	5	.488	.238	11.7
275+49	22	10	8.7	5	.455	.207	1.8
		10	21.6	5	"	"	4.47*
277+67	21	10	8.8	5	.476	.227	2.0
		10	28.8	5	"	"	6.55*

Table 6. Tabulation of Representative Sand Drain Spacing Computations.

In analyzing the results of the sand drain installation, one must keep in mind the overall benefits to be derived from the installation. If the primary settlement can be achieved within a 12 mo period for a major portion of the alignment, then it would not be practical to delay the completion of the project for one or two isolated locations.

<u>Physical Features of the Sand Drains</u>. Previous work by various people involved in studies of sand drain installations has shown that there is no appreciable increase in the efficiency of a sand drain for diameters larger than 12 in. A build-up of smear, however, tends to reduce the effective diameter of the sand drains because of the soil around the periphery having a lower permeability than the undisturbed soil farther away from the drain. It was decided for this project that the maximum true smear effect would not extend more than 3 in. beyond the sand drain perimeter. This is not to say that some remolding doesn't occur farther away than 3 in. from the perimeter of the drain, but this is not true smear.

Based upon these reasons it was decided to install 18-in. diameter drains for those installation methods which cause smear thereby giving an effective drain diameter of 12 in. For those installation methods not causing smear, such as jetting, it was decided that 12-in. diameter drains were satisfactory.

The length of the sand drains were to be variable depending upon the depth to the underlying sand. Design quantities for estimating lineal feet of drains were calculated through the utilization of the sand contour map shown previously. It was anticipated that each drain should penetrate the sand layer a depth of at least 2 ft. Exact drain length was intended to be determined in the field at the time of installation. This information was to be ascertained by observation of a change in the drilling rate or resistance, or a change in driving resistance once the sand was penetrated.

SAND DRAIN TEST SECTION

Completion of the soils investigations and analyses for the entire 16 mi of the proposed I-255 alignment revealed several areas where the most feasible method of foundation treatment would be the utilization of vertical sand drains. Knowing this, several publications and papers were reviewed for analysis of the past performances of various types of sand drain installations. It became readily evident that performance predictability of similar type installations is quite difficult. The major reasons for this inconsistency are related to wide variations in design procedures and construction procedures and controls among projects. Most of the projects reviewed, on which the sand drain installations were less than successful, appeared to be attributable to inadequate design investigations and development.

Test Section Concept

In view of these variable performances of past installations over the country, and considering the probable large quantity of work to be done on I-255, it was deemed feasible to construct a test section.

Purpose. The test section was developed for the purpose of evaluating:

1) The effectiveness of different methods of sand drain installation.
2) The reliability of time-settlement predictions from lab data as compared to actual field performance.
3) The performance of closed-type and open-type piezometers.
4) The effectiveness of construction control devices.

Conditions. The historical review of previous sand drain installations indicated that the more commonly used methods that merited consideration were:

1) The driven or jetted closed-end mandrel.
2) The hollow stem auger.
3) The unsupported, externally jetted.

While driven mandrel drains had been installed in Illinois before, the jetted and augered drains had not. Therefore in order to gather first-hand knowledge, projects were visited in New Hampshire and Connecticut to observe installation of those types of sand drains. In this manner it was possible to determine potential problems in advance as well as incorporate desirable specifications into the test section.

The area selected for the test section consisted of a portion of the final alignment for relocated Illinois Route 3. This area appeared to be challenging in that the surface condition made it difficult to place the sand blanket and control installations, the predicted settlement was substantial in relation to the fill heights, and there was marginal stability over a portion of the alignment.

It was felt that it might be desirable to compare each installation method at different spacings. Furthermore, it seemed reasonable to us that a minimum length for each area was 200 ft. Therefore, we arrived at the decision to have six test areas, with two areas for each of the methods and with two spacings for each method.

Design Features

Prior to the development of the test section, spacing computations had already been performed as shown on previous slides. For the area of the test section we were able to utilize a 1200 ft section in such a manner as to have 600 ft each of two spacings. Other pertinent data previously compiled such as the sand contour map and stability analyses were utilized in the project development. This phase of design development consisted mainly of selecting and specifying what types of controls and specifications would be needed for an adequate installation. Figure 12 shows a cross section of a typical sand drain installation that was used as a guideline to develop the test section.

Monitoring Devices. It was felt that construction and post-construction monitoring devices should be adequate to yield the necessary field data yet be uncomplicated. In this respect, we felt the only data really needed was a record of the time-settlement data for the subsoil and some method of warning of impending foundation failure due to the marginal stability of the subsoil to support the loading.

Settlement platforms to be placed on the surface of the ground were designed to measure the time-settlement relationships of the subsoil. Figure 13 shows the settlement platform developed for this project. It simply consists of a steel plate

FIG. 12. TYPICAL SAND DRAIN INSTALLATION

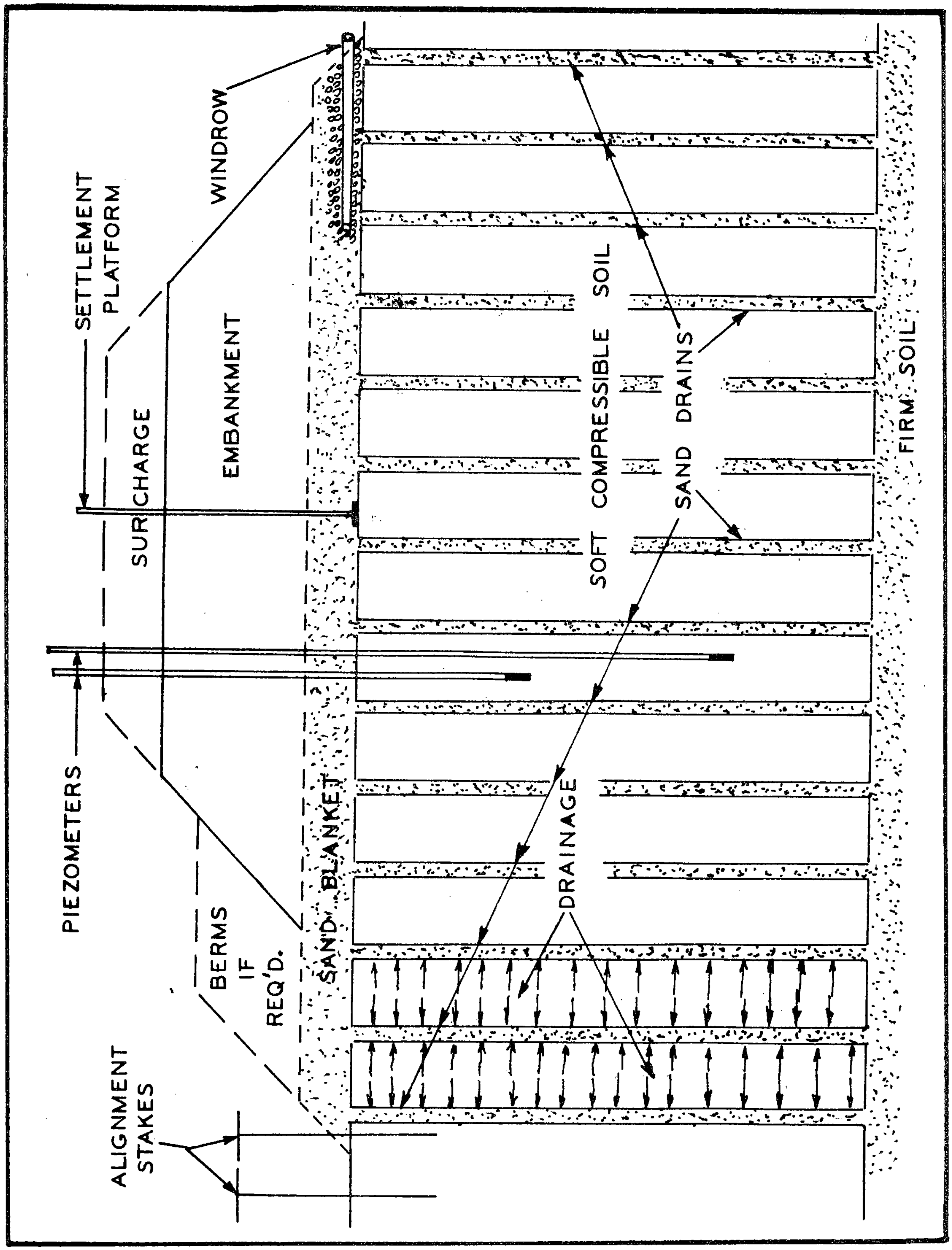
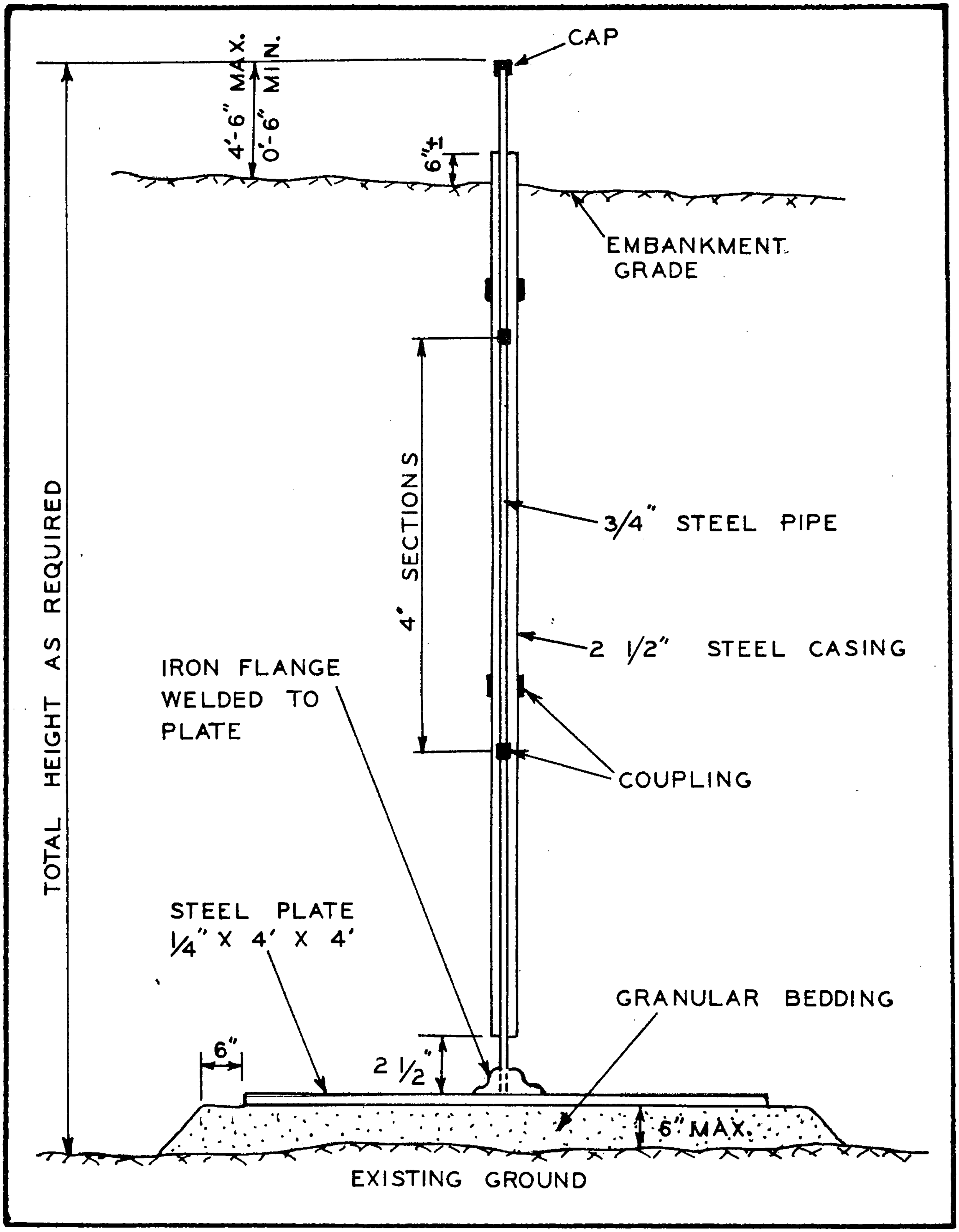


FIG. 13. SETTLEMENT PLATFORM DESIGN



with a pipe attached to it to extend through the embankment on which to take periodic elevation readings. With the settlement anticipated and the desire for as accurate information as possible, a 2-in. casing was placed around the elevation pipe through the embankment to negate any effect of negative skin friction.

Piezometers were deemed necessary to provide data on excess pore pressures caused by the drain installations and the loading of the embankment and surcharge. This information was intended to provide input for controlling embankment placement particularly in those areas of marginal stability. Pneumatic-type and open-type piezometers were selected not only to monitor the excess pore pressures but for the purpose of comparing one type to the other.

Alignment stakes were selected as a type of device to primarily indicate lateral movements beyond the toe of the embankment. These devices were designed as T's with a 2 ft horizontal arm and a 10 ft vertical leg to be embedded 6 ft into natural ground. They would serve to indicate movements caused by the bulge of rotational movement, horizontal translation, or mud waves. Both the vertical and horizontal arms were to have graduated scales to 0.01 ft for detection of vertical or horizontal movement.

Since these stakes were to be placed in two rows, parallel to the toe of slope of the roadway which was on a horizontal curve, we elected to reference them to pile supported monuments through triangulation. These monuments also would provide good bench mark references in an otherwise unstable area. The alignment stakes were specified to be of either cedar or red wood

2 x 4's so as to be weather resistant.

Site Preparation. Prior to any installations it was required that the site be prepared in one of two manners. In areas where the surface soil was capable of providing good support it was required that all vegetation be disked and trees and brush be removed. In marshy areas it is desirable to leave the natural mat of vegetation in place to help improve the working platform. In these areas, trees and stumps were to be removed with the remainder of the marsh growth to be flattened by a suitable method such as back-dragging a dozer blade.

In addition, an old access road about 350 ft long existed near the centerline. It was required that this road be removed prior to covering that area with the sand blanket.

Sand Blanket. The sand blanket was designed 4 ft thick at the centerline and tapering to 2 ft thick at the edges to serve as a horizontal drainage layer between the embankment and natural ground. This cross section was selected to facilitate clean-up after sand drain installation and to minimize the head that would occur due to the settlement which would lower the top of the sand blanket 2 to 4 ft at the centerline.

In as much as settlement within the sand blanket is of no consequence, it was specified that the blanket could be placed by end-dumping in full depth (4-ft) layers. The gradation of the sand blanket material was specified as clean sand, relatively free of deleterious material, meeting the following gradation:

Sieve Size	3/8 in.	#4	#16	#50	#100
% Passing	100	85-100	50-85	0-30	0-10

It was specified as the contractor's option to use the sand blanket as a working platform for drain installation or to place up to 12 in. of embankment material to use as a working platform.

Sand Drains. In the design of the sand drains it is necessary to determine the spacings, type of drain, diameter of drain, length of drain, and gradation of the sand. The drain spacings for the test section were included in the computations for the entire project which was shown earlier in Table 6. For this particular area those spacings were 6 ft center to center from Station 265+25 to Station 271+00 and 10 ft center to center from Station 271+00 to Station 277+00. Estimated lengths of the drains were determined from the sand contour map as previously mentioned. In addition, I previously mentioned how the type of drain and diameters were selected from studies of other projects installed in this country. In the case of the test section the closed-end mandrel and auger drains were chosen with 18 in. diameters and the jetted drains were chosen with a 12 in. diameter for reasons explained earlier.

The gradation for the sand to fill the drains was selected after considering local availability, permeability of the sand, and filter effect against the action of the surrounding soil. Sand was required that was relatively free from deleterious material within the following gradation limits:

Sieve Size	3/8 in.	#4	#16	#50	#100
% Passing	100	85-100	50-85	0-25	0-4

It can be seen that the sand for the drains is coarser on the fine end than for the blanket. This was done under the assumption that in the field some intrusion of fines into the drains would occur. Even with this occurring we estimated that the above initial gradation would provide an ultimate drain sand as clean as the blanket material which is free-draining.

Auger drains were specified to be installed with an auger having a minimum helix outside diameter of 18 in. with a maximum hollow stem outside diameter of 11 in. The rate of advance of the auger was required not to be greater than one pitch per revolution and then held stationary and rotated at least one revolution once the tip elevation was reached. Backfilling was to take place under air pressure of 100 psi to prevent arching of the sand with the pressure decreased as the withdrawing tip neared the surface to prevent "blow-out."

Mandrel drains were required to be installed by driving or jetting a closed-end mandrel with a minimum outside diameter of 18 in. Backfilling was required under air pressure of 100 psi to help assure a continuous column of sand with reduction of pressure required as withdrawal neared the surface to prevent "blow-out."

Jetted drains were required to be installed by any nonrotary jetting method accomplishing a minimum of a 12-in. diameter hole with pressure and volume regulated in such a manner as to provide satisfactory progress without backflushing adjacent completed sand drains. Diameters of the jetted holes were required to be checked a minimum of four times daily. The uncased holes were required to be filled with water prior to and during placement of the sand backfill.

At the start of the project it was mandatory to conduct in-place experimental determinations of the wash period once the tip elevation of the drain was reached. This was to be accomplished by sampling the water from each of five test holes installed for wash periods of 0, 1, 2, 3, 4, and 5 minutes. The washing period to be selected was the minimum time required to maintain the amount of minus #200 soil particles within the wash water below 10% by weight.

It was further specified that the contractor had to provide a collection system for removing the sediment from the wash water prior to discharging the wash water into any local streams.

Drainage Windrows. Transverse windrows of granular material were required to be placed every 100 ft across the entire width of the sand blanket for the purpose of preventing possible build-up of excess pore pressures within the sand blanket. The windrows were to be constructed approximately 18 in. wide by 12 in. deep with 6 in. minimum of sand cover top and bottom from gravel, crushed stone, or slag meeting the following gradation:

Sieve Size	1 1/2 in.	1 in.	1/2 in.	#4
% Passing	100	90-100	25-65	0-10

A 6-in. diameter perforated pipe, 10 ft in length, was to be placed at the discharge ends of each windrow to prevent sloughing of the sand blanket edges. A perforated cap was to be installed over the inlet end of the pipes.

Embankment. The embankment was to be constructed of borrow material classified as either A-4, A-6, or A-7-6 soils with a group index of 17 or less. The reason for this restriction of borrow was due to areas of marginal stability. In these areas it was necessary to have embankment material capable of developing a shear strength equivalent to an unconfined compressive strength of 1.0 tsf. In addition, to assure the A-4 soils would develop this strength it was necessary to specify that they not be compacted at moisture contents exceeding 110% of the optimum content for the maximum Standard Laboratory Dry Density.

It was specified that the contractor's rate of embankment placement might be affected by excessive pore pressure measurements. If it became necessary to stop embankment placement, the rate of dissipation would govern the resumption of work.

Construction Operations

Aside from the determination of the sand drain spacings, perhaps the most important aspect of a sand drain project is the installation and operation of construction controls. Regardless of the amount of time and diligence spent in the design stages, no sand drain project can be expected to be successful without the proper installation procedures and monitoring. It is absolutely necessary, to assure success, that the contractor and the resident engineering staff be competent and diligent. For the next few minutes I would like to show you some slides taken during the progress of construction operations.

First, I would like to show you the plan layout and typical embankment cross sections developed in the design stage. Figure 14 shows the embankment section from Station 266+20 to Station 271+00 where the drain spacing is 6 ft and the subsoil stability was marginal. Figure 15 shows the embankment section from Station 271+50 to Station 276+52 where stability was not

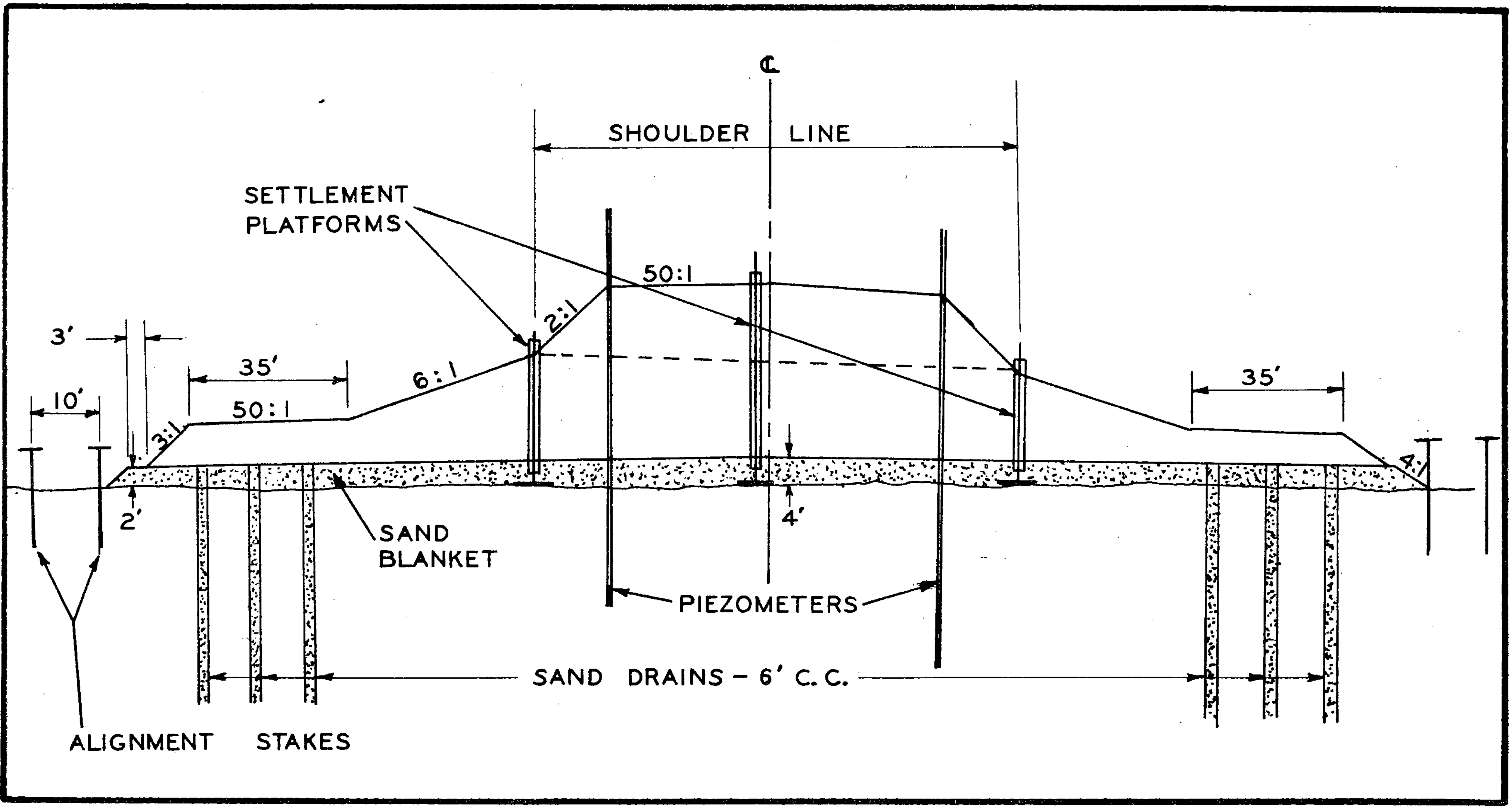


FIG. 14. EMBANKMENT X-SECTION STA. 266+20 TO 271+00

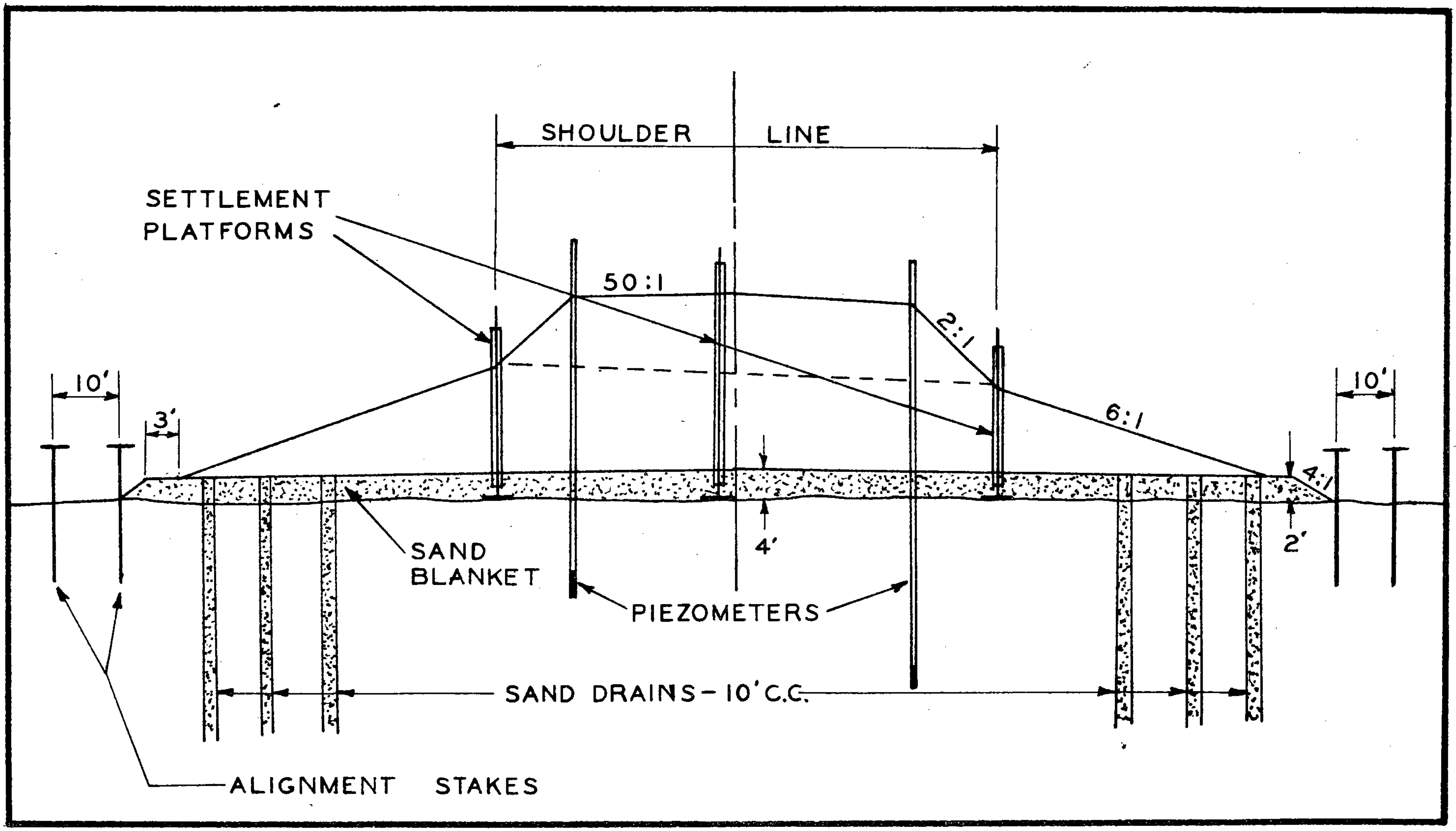


FIG. 15. EMBANKMENT X-SECTION STA. 271+50 TO 276+52

a problem and the drain spacing was 10 ft. Figure 16 shows the plan layout of the various test areas and the locations of the monitoring devices.

Site Preparation. Site preparation consisted primarily of leveling cattails and reeds with the removal of a few trees. The reeds and cattails were leveled by back-dragging a dozer blade. Most of the trees had such a relatively shallow root system that when a dozer pushed them over roots and all were removed.

Monitoring Devices. The next operation involved the placement of all the settlement platforms and some of the piezometers. The piezometers were placed at this point in time to monitor pore pressure build-up due to sand blanket placement and drain installation. The alignment stakes were placed after the sand blanket was in place. These were referenced to four monuments placed on pile-supported footings at the corners of the project.

Sand Blanket Placement. The sand blanket was placed by end-dumping from trucks and spreading with a dozer to the lines and grades shown on the plans.

Jetted Sand Drains. The jetted sand drains were placed in adjacent areas (3 and 4) for the 6 ft and 10 ft spacings for the sake of convenience of water supply and settling basin. Water from the City of Dupo's line was pumped into a storage reservoir from which water was pumped for jetting. As the cuttings from the jetting were washed to the surface they were raked away with hoes from the periphery of the drain. The wash period determined for this project was 1 min.

NOTE: Figures 17 through 59 were colored slides of construction operations and have been omitted from the text.

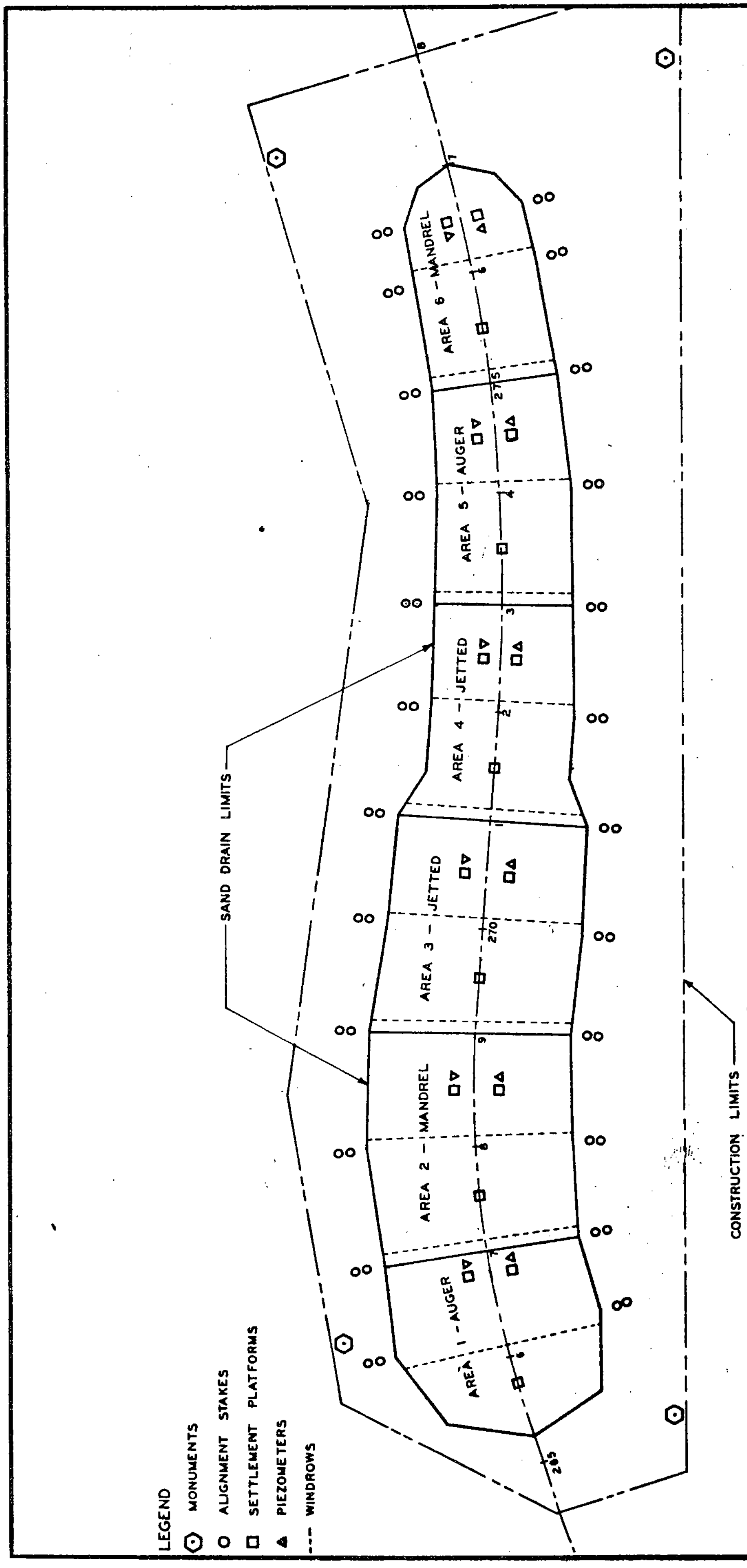


FIG. 16. TEST SECTION LAYOUT PLAN

SCALE: 1 IN. = 60 FT.

Backfilling was accomplished by placing a hopper over the completed drain and dumping sand into the hopper. The sand was kept in a fluid state by running water into the hopper with a hose. Some of the jetted holes were pumped out for inspection and revealed a perfectly uniform hole.

Augered Sand Drains. Due to the controlled rate of augering, the installation of the augered drains was considerably slower than either of the other two methods. Backfilling the drains was accomplished by dumping sand into a skip which in turn was raised up the leads to dump into the hollow stem auger. Air pressure was then applied to the closed system, the trap door was released, and the auger withdrawn.

Mandrel Sand Drains. In the area with 6-ft spacing, the installation of the mandrel drains caused the settlement platforms to heave. One advantage the mandrel method has over the other two is the absence of clean-up after installation. Backfilling of the drains was accomplished by dumping sand into a skip which in turn was raised up the leads and dumped into the mandrel. Air pressure was then applied to the system and the mandrel withdrawn.

Drainage Windrows. Once the drains were installed and the top of the sand blanket cleaned up, the excavations for the drainage windrows were made. Crushed stone was placed in the windrow and then covered with sand up to the top of the sand blanket. The perforated pipes in the ends of the windrows were installed simultaneously with the crushed stone.

Embankment. Embankment placement was somewhat hampered by the relatively small area in which to work and by the presence of the piezometers and settlement platforms. It was necessary to halt embankment construction in the 6-ft mandrel area due to excessive pore pressure increases on two or three occasions.

Post-Construction Monitoring and Evaluations

At the risk of sounding trite, I would like to say that the merits of this project lies in the end results. At this time I want to show you the post-construction results which represent the fruits of our labors in the design and construction stages. These results can probably be best summed up by looking at the data for the time-settlement curves and the pore pressure readings. For the purpose of this presentation I have simplified the field data onto 8 1/2 x 11 in. sheets which were taken from graphs that were originally plotted on sheets 3 to 5 ft long.

Pore Pressures. In general, none of the methods of installation caused excessive pore pressures of significant magnitude in those areas where the drain spacing was 10 ft. In those areas where the drain spacing was 6 ft, however, excessive pore pressures developed to a small degree with the jetted method and to a considerable degree with the mandrel method. The installation of the drains alone in the 6-ft mandrel area caused quite excessive pore pressures.

Figure 60 shows the relationship of the embankment construction to the subsoil pore pressures for the auger method. As the embankment and surcharge peaked out at an elevation of 415 ft, some pore pressure did build up in the area of 6-ft spacing, but they dissipated rather rapidly. The pore pressure build-up for the area of 10-ft spacing was rather insignificant.

FIG. 60. PORE PRESSURE: AUGER METHOD

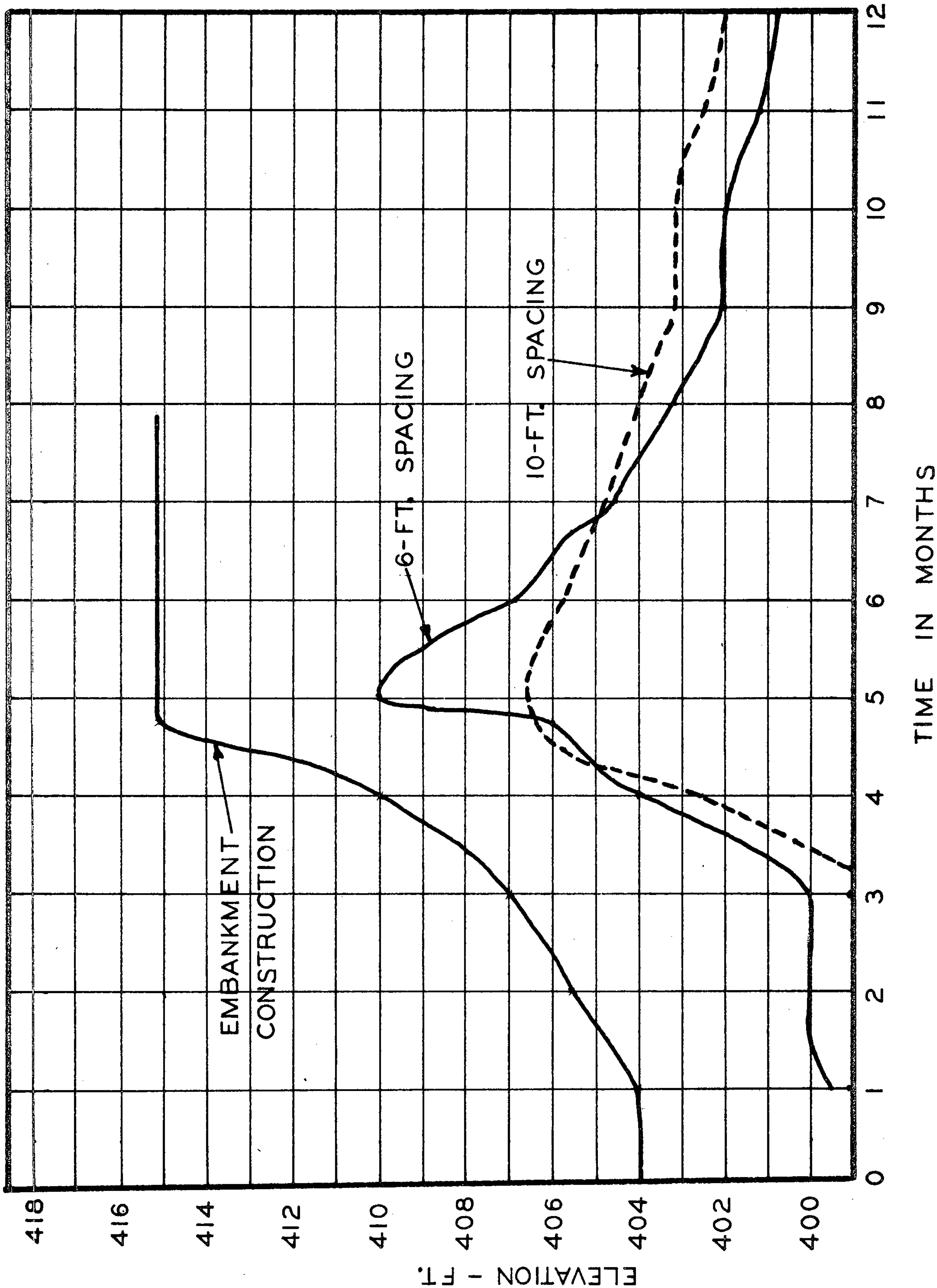


Figure 61 shows the embankment construction-pore pressure relationship for the jetted method of installation. There was some excessive pore pressure build-up for the area with the 6-ft drain spacing. This build-up was not great enough to interfere with the contractor's embankment placement operation. The pore pressure increase in the area of 10 ft spacing was much less significant. In both instances the dissipation occurred rather rapidly.

Figure 62 shows the pore pressure history for those areas in which the sand drains were installed by the mandrel method. In this instance the pore pressure build-up was a significant factor for the area of 6-ft drain spacings. The increase was great enough that embankment construction had to be halted on occasion. While the pore pressure increase for the 10-ft spacing was much less, it is notable how slowly dissipation occurs in both instances. Even after a year the pore pressures for the 6-ft spacing is still significant.

As a point of interest, I might mention that we used a level of pore pressure increase of 1.5 times the height of fill increase over a given time period as the critical indicator. In applying this rule of thumb on the job, other factors such as rate of placement, previous readings, etc., were also taken into consideration.

Settlement. The time-settlement curves I'm about to show you represent readings taken from settlement platforms in the 6-ft drain spacing areas for each method of installation. In each case a theoretical curve is shown as developed from lab tests on the boring nearest the respective settlement platform.

FIG. 6.1. PORE PRESSURE: JETTED METHOD

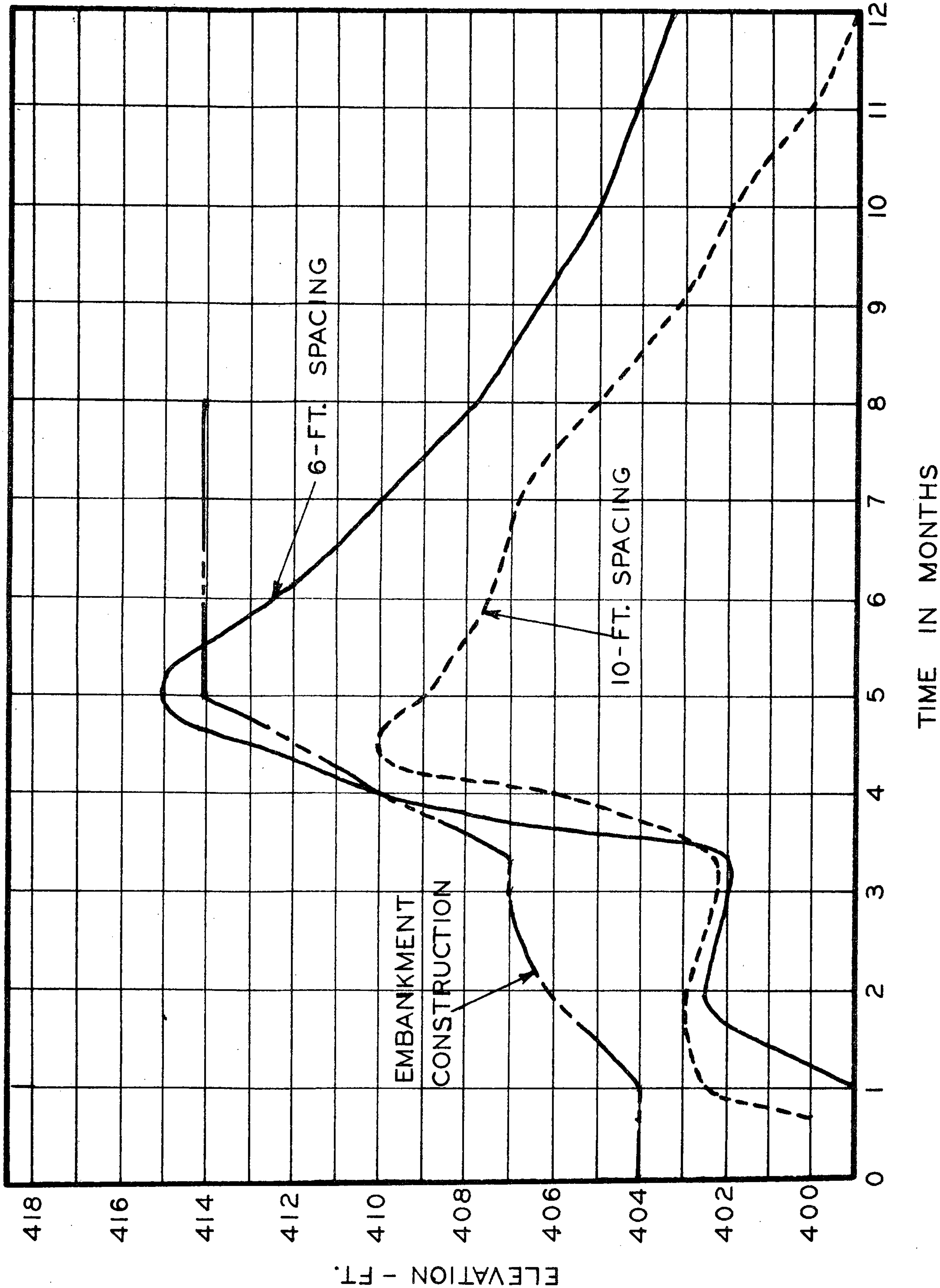
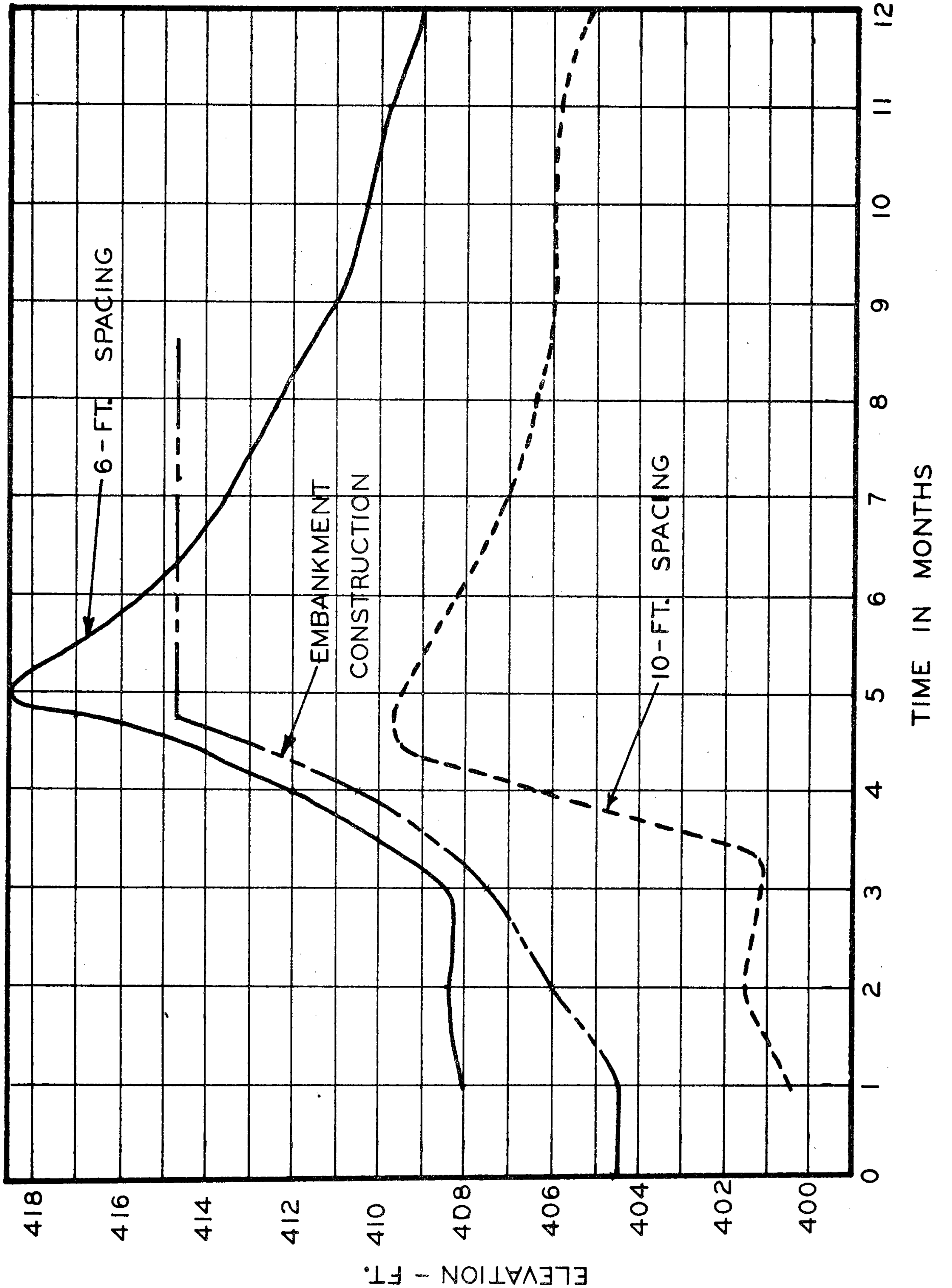


FIG. 62. PORE PRESSURE: MANDREL METHOD



On the time scale of these charts, a time of 20 weeks corresponds to approximately 0 on the previous pore pressure plots. The sharp descents at 20 weeks signify the completion of the installation of the sand drains.

Figure 63 represents the time-settlement relationship for the auger method. In this case the field curve very closely approaches the lab curve at 50 weeks. This curve, in conjunction with the total dissipation of excess pore pressures, is conclusive as to the effectiveness of this installation.

Figure 64 shows the time-settlement curves for the jetted method. While the field curve is somewhat displaced from the lab curve, the slope of the field curve in conjunction with complete dissipation of pore pressures indicated primary consolidation is complete.

Figure 65 displays the time-settlement curves for the mandrel method. Note the upward movement of the field curve just prior to 20 weeks. The settlement platforms were actually heaved due to the installation of the drains. Not only is the field curve significantly above the lab curve, but Figure 63 also showed considerable pore pressure that has not dissipated. This leads us to believe that primary consolidation is not complete.

Piezometer Comparison. We have not found a significant difference in the information yielded by the open-type and pneumatic-type piezometers. Figure 66 shows a portion of the curves for the two types of piezometers installed adjacent to one another in the field. About the only conclusions drawn from this portion of the test were:

FIG. 63. TIME-SETTLEMENT CURVES: AUGER METHOD

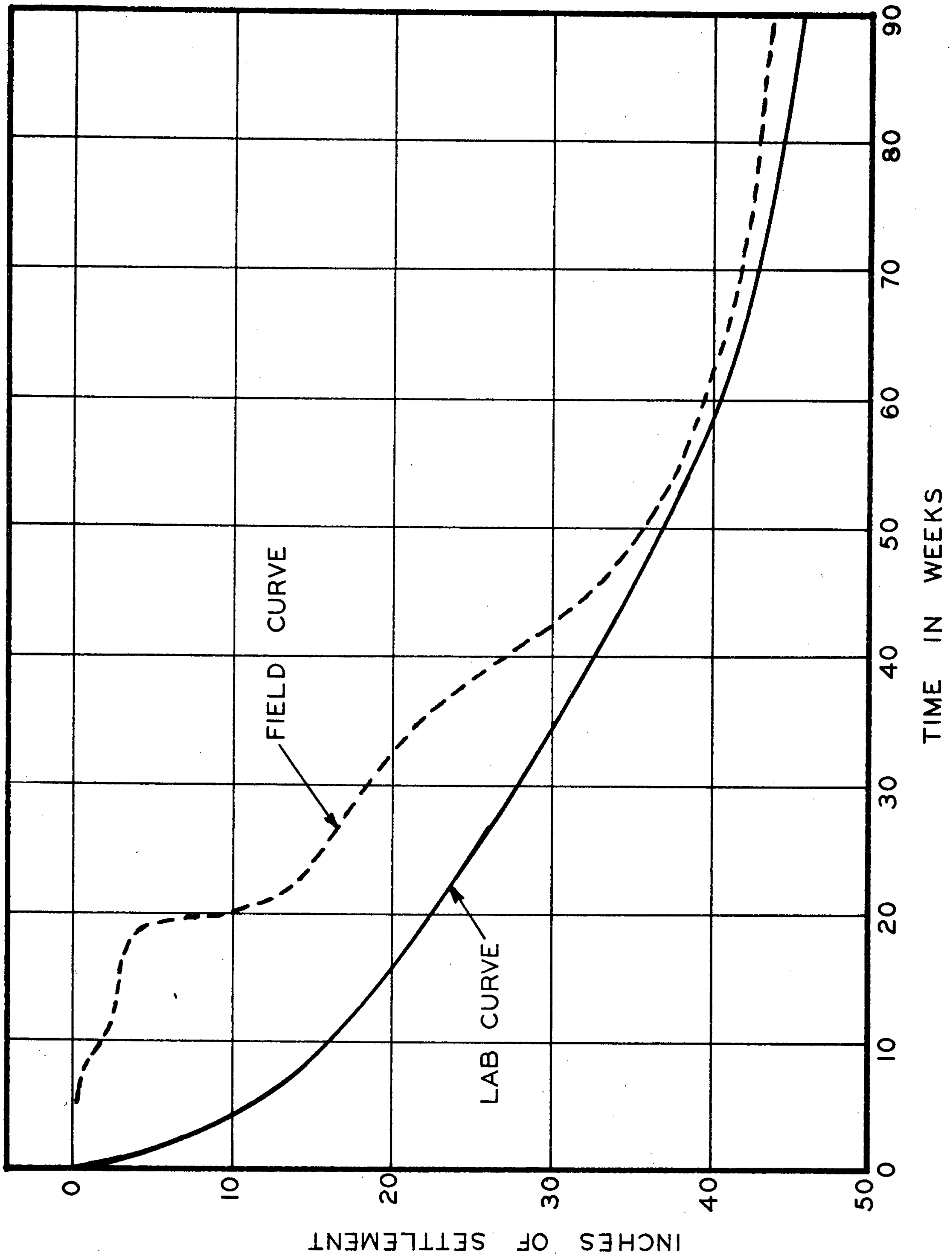


FIG. 64. TIME-SETTLEMENT CURVES: JETTED METHOD

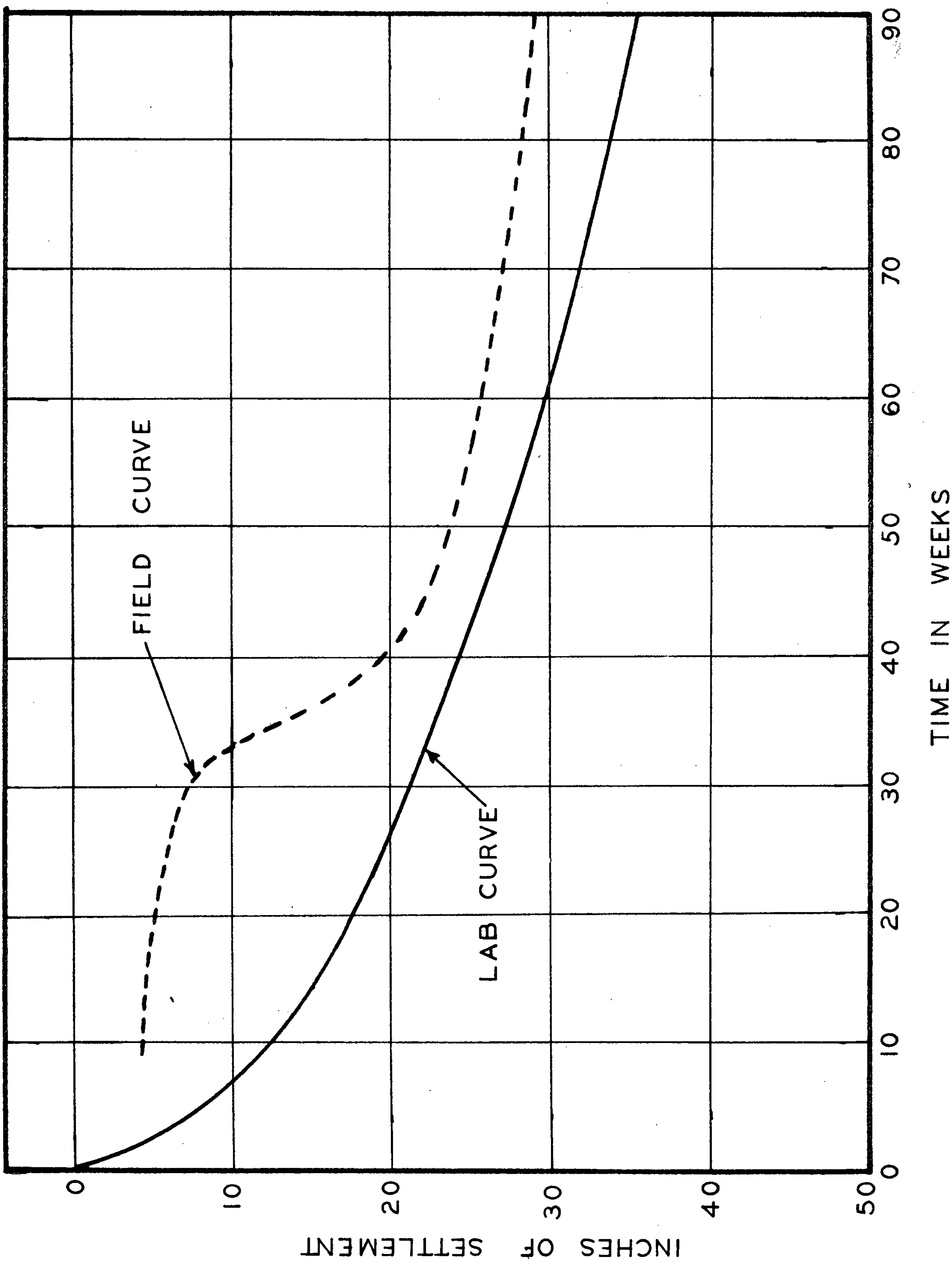


FIG. 65. TIME-SETTLEMENT CURVES: MANDREL METHOD

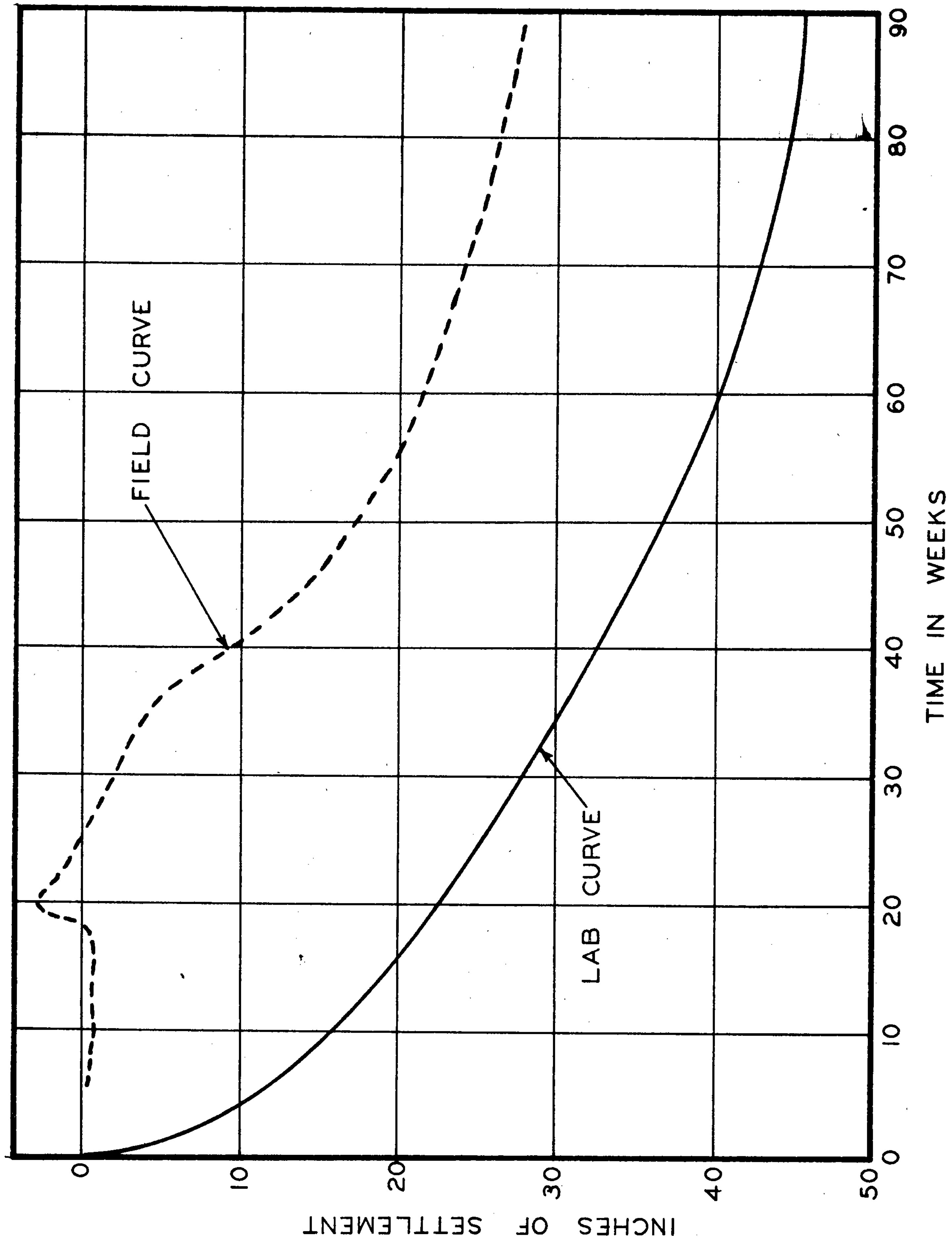
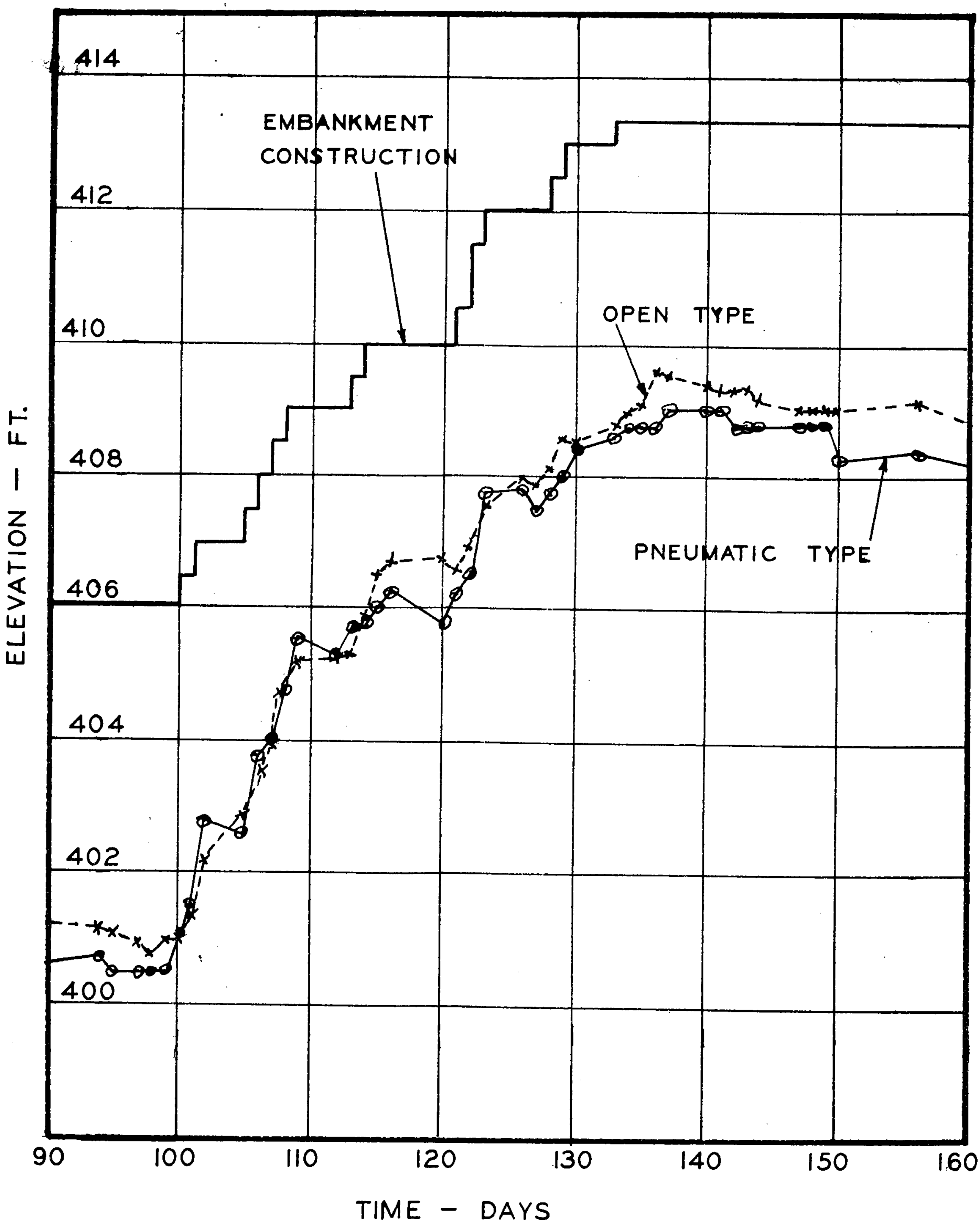


FIG. 66. COMPARISON OF PIEZOMETERS



1) The pneumatic-type piezometer responds to changes with less time lag than the open-type.
2) The pneumatic-type displays more variable readings for no readily apparent reason.
3) The pneumatic-type is easier to read.

SUMMARY

Based upon the results of the test section I feel the following observations are valid conclusions:

1) There is very little difference in the final result of sand drain installations utilizing either the auger or jetted method with either one giving satisfactory results.
2) The mandrel method of installation is not nearly as effective as the other two methods probably due to the fact it is a displacement-type installation.
3) The major portion of the primary consolidation occurs more rapidly in the field than in the lab.
4) There is apparent validity to increasing drain diameter to compensate for smear effect.
5) There is not a significant difference in the data yielded by either the pneumatic-type or open-type piezometer.
6) The design procedures utilized in developing this project provided rather reliable and adequate information.

In general, it is felt that we have gained adequate knowledge and confidence from the test section to develop an effective sand drain system for the major portion of the future work on I-255.

INTRODUCTION TO THE GEOLOGY AND MINERAL RESOURCES OF NORTH CAROLINA *

BY

STEPHEN G. CONRAD

*The contents of this paper were excerpted by the author from previous publications for presentation at the 25th Annual Highway Geology Symposium, Raleigh, North Carolina, May 24, 1974.

CONTENTS

INTRODUCTION

North Carolina extends across three physiographic provinces that are generally present in the eastern part of the United States: the Coastal Plain, the Piedmont and the Blue Ridge. The principal geologic belts correspond very closely to these physiographic provinces.

The Coastal Plain is underlain by sedimentary rocks that range from Cretaceous to Recent in age and consist largely of loosely to unconsolidated sediments that include clays, sands, gravels, limestones and marls. The Piedmont is composed of much older sedimentary and volcanic rocks that have been variably altered by metamorphic processes and intruded by igneous rocks that range from gabbro to granite in composition. The sedimentary and volcanic rocks are generally considered to be late Precambrian to early Paleozoic in age and the igneous intrusive rocks from early to middle Paleozoic in age. An exception are the downfaulted Triassic basins which are composed of unmetamorphosed sedimentary rocks that have been intruded by late Triassic or Jurassic dikes and sills of gabbroic composition.

The Blue Ridge is composed of a billion year old Precambrian basement complex which consists mostly of plutonic gneisses. The basement complex is unconformably overlain by younger Precambrian sedimentary and volcanic rocks that have undergone several episodes of metamorphism and are now very complex gneisses and schists. The regional geologic pattern is interrupted in several areas by complex structural "windows" in which younger Precambrian and Paleozoic sedimentary rocks are exposed.

Coastal Plain Geology

The Coastal Plain of North Carolina is part of a much larger accumulation of Mesozoic and Cenozoic sediments that are present along the east coast of North America from southern Florida to the Grand Banks of Newfoundland. These sediments overlie pre-Mesozoic basement rocks that are similar to those found in the Piedmont. The post-Triassic sediments form a wedge-shaped mass that thickens from a feather edge along the Fall Line to 10,000 feet at Cape Hatteras.

In North Carolina the Cretaceous and Tertiary rocks crop out in broad belts that roughly parallel the present Atlantic coastline. The older formations of Cretaceous age crop out along the western edge of the Coastal Plain and become buried progressively deeper toward the coast. The younger formations of Tertiary and Quaternary age crop out southeast of the Cretaceous formations and are buried less deeply near the coast.

Cretaceous System

The outcropping Cretaceous formations are from oldest to youngest, the Tuscaloosa, Black Creek and Peedee formations. The Tuscaloosa Formation crops out mainly in the southeastern part of the State south of the Neuse River in Harnett, Moore, Cumberland, Hoke, Rockingham and Richmond Counties. It is composed chiefly of light-colored sand and clay, with some gravel near the base of the formation. The Black Creek Formation overlies the Tuscaloosa Formation and crops out, or occurs near the surface in Green, Wayne, Sampson, Bladen and Robeson Counties. It is composed chiefly of beds and laminations of dark carbonaceous clays and fine to medium sands.

The Peedee Formation overlies the Black Creek and crops out east of the Black Creek Formation in Pitt, Lenoir, Duplin, Pender and Columbus Counties. It is composed of dark-gray to green glauconitic sands and clays, and locally thin impure limestone beds.

Rocks of Lower Cretaceous age are present in the subsurface throughout much of the Coastal Plain. They occur at or near the surface in Halifax County and are no doubt present in other areas as well.

Tertiary System

The only Tertiary Formations that crop out in North Carolina are the Castle Hayne Limestone of Eocene age and the Yorktown Formation of Upper Miocene age. Paleocene, Lower to Middle Miocene, and Oligocene sediments are present in the subsurface.

The Castle Hayne Limestone is at or near the surface in a relatively narrow belt that extends through Craven, Jones, Onslow, Duplin, Pender and New Hanover Counties. It consists of marl, sand and a distinctive indurated shell-rock or shell limestone. The hard shell limestone is the only source of crushed stone in the eastern Coastal Plain region. The Castle Hayne Limestone is an important source of ground water.

The Yorktown Formation underlies a large portion of the Coastal Plain north of the Neuse River. It varies in composition from blue massive clay to lighter colored shell beds to sands and sandy clay.

Quaternary System

The Coastal Plain of North Carolina is covered by a veneer of sands and clays that almost everywhere covers the older formations.

These surficial deposits vary from a few feet to over 40 feet thick and occur as belts 10 to 15 miles wide that lie at different elevations above sea level. The deposits are commonly considered to be marine terraces of Pleistocene age. Each terrace is bounded on the west by a scarp that marks the presence of the shoreline at the time the terrace was formed.

Piedmont Geology

The Piedmont in North Carolina is part of the Appalachian Piedmont that lies between the Blue Ridge and the Coastal Plain from New Jersey to Alabama. At its northern extremity, the Piedmont is only 10 miles wide, but it widens to the south and reaches its maximum width of nearly 150 miles in North Carolina. The Piedmont is underlain mostly by deformed and metamorphosed crystalline rocks of late Precambrian to early Paleozoic age and intrusive rocks that range from gabbro to granite are common. Downfaulted basins filled with unaltered Triassic sedimentary rocks are also present. Except along the major stream valleys, the rocks of the Piedmont are everywhere covered by a mantle of thoroughly weathered residual material, called saprolite, that in many places exceeds 50 feet in thickness.

The complex geology of the crystalline rocks of the Piedmont of North Carolina is at best only generally understood at the present time. However, because of similar rock types, structures and areal distribution, the Piedmont is devisible into parallel geologic belts that have a regional strike from southwest to northeast. Across strike from southeast to northwest, these geologic belts are known as the Carolina Slate Belt, the Charlotte Belt, the Kings Mountain Belt and the Inner Piedmont Belt. The Brevard Belt, a prominent regional structural feature, forms the western boundary of the Inner Piedmont Belt and separates it from the Blue Ridge.

Triassic Rocks

Within the crystalline rocks of the Piedmont from Nova Scotia through the Carolinas, are a series of downfaulted basins containing unaltered sedimentary rocks, in large part redbeds, that are of Triassic age. The major part of two of these basins, and a small subsidiary basin, occur in North Carolina.

Deep River basin: - This is the largest of the southern basins and extends, from just over the South Carolina line through Anson County, northeastward to near Oxford in Granville County, a distance of about 150 miles. The basin varies from 5 to about 18 miles in width. Different parts of the basin have been termed the Wadesboro, the Sanford, the Deep River and the Durham basins, but these names are actually subdivisions of a single basin.

The Deep River basin is bordered on the east and west sides by normal faults with the area between depressed in the form of a graben. However, greater downward movement apparently took place along the eastern border fault. The beds in the Deep River basin dip southeast toward the eastern border fault (Jonesboro Fault) and consist of red, brown, purple or gray claystone, shale, siltstone and sandstone. Conglomerate and fanglomerate predominate adjacent to the border fault. Where the Deep River Basin has been studied in detail, the rocks have been divided into three formations. A middle dark gray shale and coal bearing unit (Cumnock Formation) separates the lower Pekin Formation and the upper Sanford Formation. Reptile and plant fossils are locally abundant in the Pekin and Cumnock Formations.

Dan River basin: - The Dan River basin begins near Germanton, Stokes County and strikes northeast along the Dan River across Stokes and Rockingham Counties, and continues for some distance into Virginia. The portion in North Carolina is about 40 miles long and averages 5 to 7 miles in width. The rocks in the Dan River basin are much like those in the Deep River basin in composition and stratigraphic position. However, in the Dan River basin the rocks dip to the northwest toward the western border fault.

About 25 miles southwest of the Dan River basin in the northwest corner of Davie County, there is a small, isolated area of Triassic rocks that dip to the northwest toward a border fault.

Carolina Slate Belt

The Carolina slate belt is the easternmost geologic belt in the Piedmont province. It occupies much of the eastern Piedmont and crops out in two regions. The larger western region enters North Carolina in Union County on the southwest and extends northeastward through the central part of the state to Person, Granville and Vance Counties where it continues into Virginia. A smaller eastern region occurs in Halifax, Nash, Wilson, Johnston, Harnett, Moore, Richmond and Anson Counties. The eastern and western regions are separated to the north by an area of medium-grade metamorphic rocks and a large granite body, and to the south by the Deep River Triassic basin. On the west, the Carolina slate belt rocks are in contact with igneous and medium-grade metamorphic rocks of the Charlotte belt. The Gold Hill fault marks this boundary between Union and Davidson Counties. On the east, the slate belt rocks continue beneath the Cretaceous and Tertiary deposits of the Coastal Plain.

The rocks of the Carolina slate belt are of volcanic and sedimentary origin, all of which have been subjected to low-grade regional metamorphism. The dominant sedimentary rock is a well bedded, finely laminated rock that has been termed slate, volcanic slate, shale, mudstone, argillite and siltstone. Coarser grained sedimentary rocks that are also abundant include graywacke, siltstone, sandy siltstone and fine sandstone. Less abundant sedimentary rocks include quartzite, conglomerate, and very rarely limestone. Interbedded with the sedimentary rocks are extensive subagueous and subaerially deposited volcanic rocks that range from rhyolite (acid) to basalt (mafic) in composition. Both flow and pyroclastic deposits are present.

Based on a few radiometric age dates and very meager fossil evidence, the Carolina slate belt is generally considered to be of early Paleozoic age.

Igneous intrusive bodies are common in the northern half of the slate belt. These intrusive bodies range in size from narrow dikes and sills to plutons several miles across. They are granitic to gabbroic in composition and some are metamorphosed and others are not. They range from early Paleozoic to Triassic in age.

Charlotte Belt

The Charlotte belt lies northwest of the Carolina slate belt and is about 40 miles wide at its maximum width. It is composed mostly of plutonic, or igneous, rocks that form a complex mixture of granites, diorites and gabbros. Toward the northeast end of the belt in Guilford County, the plutonic rocks begin to finger out in gneisses and schists of the Inner Piedmont belt.

Although plutonic rocks are much more abundant in the Charlotte belt than in the other Piedmont geologic belts, much of the area is composed of granitoid gneiss that apparently originated as sedimentary and volcanic rocks. These medium-grade metamorphic rocks may be in part equivalent in age to the sedimentary and volcanic units present in the adjacent Carolina slate and Kings Mountain belts.

One of the most interesting features of the Charlotte belt is the so-called Concord ring-dike that occurs about 12 miles northeast of Charlotte in western Cabarrus County. This is an almost circular belt of coarse grained, massive syenite that is a mile wide and six miles in diameter. The syenite weathers to large boulders and the ring-dike forms a distinctive topographic high. The ring-dike encircles a gabbro-diorite intrusive body.

Radiometric age dates indicate the plutonic rocks of the Charlotte belt are from 415 to 385 million years in age. Some of the plutons are metamorphosed and others are not. The granitic rocks of the Charlotte belt are the principal source of commercial stone, both crushed and dimension, in the Piedmont section of the State.

Kings Mountain Belt

Beginning at the South Carolina line and continuing in a northeasterly direction for about 40 miles through Cleveland, Gaston, Lincoln and Catawba Counties, is a narrow belt of low-grade metamorphic rocks of sedimentary and volcanic origin that comprise the Kings Mountain belt. Through this area the Kings Mountain belt separates the Charlotte belt and the Inner Piedmont belt.

Rocks in the Kings Mountain belt have been altered by low-grade regional metamorphism, but their sedimentary and volcanic origin are clearly evident. Rock types include slate, phyllite, quartzite, conglomerate and marble. Especially prevalent are sericite, or muscovite, schist and hornblende schist, parts of which are clearly volcanic in origin. Quartzite beds, some of which contain kyanite, are especially evident because they form the prominent ridges in the area such as the Pinnacle and Crowder's Mountain.

The Kings Mountain belt has been tightly compressed by folding and all of the beds dip at nearly vertical angles. The age of the rocks in the belt is not clear, but recent workers in the area suggest they may be correlative with the Carolina slate belt.

Inner Piedmont Belt

The largest of the geologic belts in the Piedmont of North Carolina is the Inner Piedmont belt. It occupies a broad area 50 to 60 miles in width across the state. The western border is marked by the Brevard belt and on the east it is in contact with the Kings Mountain belt to the south and the Charlotte belt to the north. The contact with these two belts on the east appears to be more of a change in metamorphic grade than one of rocks of different age.

The belt occupies the zone of highest regional metamorphic grade in the Piedmont of North Carolina and consists mostly of a great variety of complexly mixed mica gneisses and schists with lesser amounts of hornblende gneiss and schist. The belt is characterized by generally low and irregular dips of the foliation. Except along its margins, the dip of the gneisses and schists is rarely steep and dips of less than 45 degrees are more common than dips of more than 45 degrees. In Stokes County, a domed-shaped structure composed of interlayered quartzite and schist rises above the surrounding gneisses to form the Sauratown Mountains. Because of their obvious sedimentary origin the rocks in this area have been correlated with the Kings Mountain belt, but the age and structural relationship with the surrounding gneisses remains uncertain.

Granitic rocks are abundant in the Inner Piedmont belt, but are subordinate to the gneisses and schists. Where mapped in detail, the granitic bodies consists of widely spaced sheets and lenses that crosscut the intruded gneiss and schist and represent two distinct ages of igneous activity. Commercial muscovite pegmatite dikes, some of which contain spodumene (lithium ore), are associated with the granitic rocks, particularly in the Shelby-Kings Mountain area. Ultramafic rocks, occurring as small, isolated bodies are scattered throughout the belt.

The high regional metamorphic grade makes the origin, stratigraphic sequence and geologic history of the Inner Piedmont belt extremely difficult to decipher. However, it is generally believed that the gneisses and schists represent upper Precambrian and lower Paleozoic sedimentary and volcanic rocks that have undergone several episodes of regional metamorphism and igneous intrusive activity.

Brevard Belt

The Brevard belt, often referred to as the Brevard fault zone, is a narrow belt of low-grade metamorphic rocks that separates the Inner Piedmont belt from the Blue Ridge belt. It is present from northern North Carolina to Alabama, a distance of more than 325 miles. It enters North Carolina in southwestern Transylvania County and strikes northeastward through Henderson, Buncombe, McDowell, Burke, Caldwell, Wilkes and Surry Counties. The segment of the belt in North Carolina varies from less than one mile to over five miles in width. Although the Brevard belt marks the southeast edge of the Blue Ridge geologic belt, it does not exactly correspond to the southeast edge of the Blue Ridge physiographic province. It lies northwest of the Blue Ridge scarp from Transylvania County to McDowell County where it descends to the Piedmont province and remains for the rest of its length in North Carolina.

Most of the rocks in the Brevard belt have been intensely sheared and show evidence of having been derived by retrogressive metamorphism of the bordering rocks. Others are progressively metamorphosed sedimentary rocks that locally show primary textures and structures. Crystalline limestone, or marble, lenses are common in the southwestern portion of the belt in North Carolina. Throughout most of its length, rocks in the Brevard belt are steeply dipping or vertical.

The Brevard belt is bordered on the northwest and southeast sides by faults. Where detailed studies have been conducted in areas that span the belt, significant contrasts in rocks that lie to the southeast in the Inner Piedmont belt and those to the northwest in the Blue Ridge belt have been demonstrated. Differences in metamorphic grade, structural pattern, relative proportions of schist, gneiss and amphibolite and abundance and character of rocks are obvious.

Because of its unique character and importance of any interpretation of the geologic history of the area, the Brevard belt has been of interest to geologists since it was first recognized early in this century. No less than nineteen theories have been advanced to explain the Brevard belt including most forms of faulting and combinations of faulting and folding. Whatever its origin, the Brevard belt is one of the most complex structural features in the southern Appalachians and has been involved in much of the tectonic development of the region.

Blue Ridge Geology

The Blue Ridge in North Carolina is part of the Blue Ridge belt that extends for more than 700 miles from southern Pennsylvania to northwestern Alabama. It ranges in width from less than 5 miles in Maryland and Pennsylvania to as much as 70 miles in North Carolina and Tennessee. The belt is composed of variously metamorphosed Precambrian rocks, and other than the Canadian shield, is one of the largest exposed areas of Precambrian rocks in North America. Because of a pronounced change in structural pattern, the Blue Ridge belt is divided into two sections. That part lying north of Roanoke, Virginia, is called the northern Blue Ridge, and that part to the southwest where it disappears beneath the Coastal Plain in central Alabama is called the Southern Blue Ridge.

Blue Ridge Belt

In North Carolina, the Blue Ridge belt is bordered on the southeast by the Brevard fault zone and on the northwest by Paleozoic rocks of the Valley and Ridge and Unaka belts in Tennessee and southwest Virginia. In very generalized terms the Blue Ridge belt is composed of an anticlinal core of older Precambrian basement rocks unconformably overlain by a vast thickness of younger Precambrian metasedimentary and metavolcanic rocks. Intrusive into these Precambrian rocks are diverse types and ages of plutonic rocks. Large scale, low angle thrust faults have transported part, if not all, of the southern Blue Ridge belt northwestward as much as 35 miles over the unmetamorphosed Paleozoic rocks of the Valley and Ridge belt. Subsequent uplift and erosion has exposed the younger overridden rocks in "windows" at several places.

The basement core is composed of massive, granitic-textured rocks and layered and non-layered granitic gneisses mixed with various proportions of biotite and hornblende schist, amphibolite and other nongranitic rocks. The massive granitic-textured rocks have been mapped as the Max Patch Granite and Cranberry Gneiss and occur mostly on the northwest side of the Blue Ridge belt in Swain, Haywood, Madison, Yancey, Watauga and Ashe Counties. Recent radiometric age determinations on samples of Cranberry Gneiss from Ashe County have given ages of 1,207 million years to 1,297 million years and these are the oldest rocks thus far found in the Southern Blue Ridge. The Blowing Rock Gneiss and Wilson Creek Gneiss are granitic gneisses of the Precambrian basement complex that occur within the Grandfather Mountain Window in Avery, Caldwell and Watauga Counties. These rocks are also in excess of 1,000 million years in age. Other rocks of the Precambrian basement core occupy a large area in parts of Macon, Jackson, Madison, Haywood and Buncombe Counties and consist of a complex mixture of granitic gneiss and layered and non-layered gneisses composed mostly of biotite, hornblende, garnet and muscovite.

Unconformably overlying the older Precambrian basement core is a vast thickness of metasedimentary and metavolcanic rocks of younger Precambrian age. Along the west side of the Blue Ridge belt through parts of Cherokee, Graham, Swain, and Madison Counties, these younger Precambrian rocks comprise the Ocoee Series. The rocks of the Ocoee Series are only slightly to moderately metamorphosed and consist mostly of clastic sediments that range from shale to conglomerate. The total thickness of the Ocoee may be as much as 25,000 feet. It is broken by a number of major thrust faults and is complexly involved by folds and faults with the underlying basement rocks.

Much of the Blue Ridge belt in North Carolina is composed of mica gneiss and schist and hornblende gneiss and schist, apparently derived from metamorphism of sedimentary and volcanic rocks. These rocks underlie a large part of Macon, Jackson, Haywood, Transylvania, Henderson, Buncombe, Yancey, Mitchell, Watauga, Ashe and Alleghany Counties. Form many years, these rocks were considered to be early or middle Precambrian in age and were referred to as the Carolina Gneiss and Roan Gneiss. However, more recent work has shown these rocks to non-conformably overlie the basement core and to consist mostly of metamorphosed sedimentary and volcanic rocks of late Precambrian age. These rocks are now considered to be, at least in part, more highly metamorphosed equivalents of the Ocoee Series. Other thick sequences of metasedimentary and metavolcanic rocks that lie on the older Precambrian basement in the Blue Ridge belt of North Carolina include the Mount Rogers Formation in the northwest corner of Ashe County and the Grandfather Mountain Formation exposed within the Gradfather Mountain Window.

In several restricted areas in the North Carolina Blue Ridge Belt, sedimentary rocks of apparent lower Paleozoic age are found. The most extensive of these is the Murphy belt which begins in Graham County and strikes southwestward through parts of Clay and Cherokee Counties where it continues for some distance into Georgia. Other areas in which Paleozoic rocks are exposed include the Hot Springs Window in Madison County and the Grandfather Mountain Window in Avery, Caldwell and Watauga Counties. The presence of limestone, dolomite and marble are distinguishing features of the areas of Paleozoic rocks.

Intrusive into the older Precambrian basement core and the overlying young Precambrian rocks are numerous igneous bodies that range from Precambrian to Paleozoic in age and from mafic (basic) to felsic (acid) in composition. These include the Beech, Brown Mountain and Whiteside Granites, the Bakersville Gabbro and numerous pegmatites and "alaskite" bodies in the Spruce Pine mining district. Small bodies of metamorphosed ultramafic rocks (dunites) many of which have been mined for asbestos, olivine, vermiculite, and corundum, occur throughout the Blue Ridge belt from Clay County on the southwest to Alleghany County on the northeast.

The older Precambrian basement rocks were metamorphosed at least once prior to the deposition of the overlying younger Precambrian rocks. The Upper Precambrian rocks and the underlying basement rocks were subsequently subjected to one and probably several episodes of regional metamorphism during the Paleozoic era. These metamorphic effects, plus the folding and faulting that has taken place make the Blue Ridge belt one of the most geologically complex areas in the country.

MINERAL RESOURCES

Because of its complex geological history and wide variety of rocks that were formed from Precambrian time to the present that include the products of practically every known geological and mineralogical process, North Carolina is endowed with an abundance of rocks and minerals that are of economic importance to our industrialized society. Of the over three hundred varieties of rocks and minerals known to occur in the State, over seventy have uses in commercial or industrial processes, and between forty and fifty of these have been produced at one time or another in commercial quantities.

Metallic Minerals

Metallic minerals are associated principally with the igneous and metamorphic rocks of the Piedmont and Blue Ridge geologic belts. Chromite, copper, gold and silver, iron, lead and zinc, manganese, molybdenum, nickel, tin, titanium and tungsten all occur in the state to some extent.

Although there are no active metal mines in the state today, noteworthy amounts of gold, copper, iron and tungsten have been produced in the past. North Carolina was the leading producer of gold prior to the discoveries in California in 1849. During the time it was operated in the 1950's, the Hamme Mine in Vance County was the largest single tungsten mine in the United States. Considerable reserves of tungsten ore remain in this deposit and as economic conditions change it will undoubtedly be reopened in the future. Because of its geologic similarity to important metal mining districts in Canada, the Carolina slate belt has been the target of extensive exploration during the past few years, and is considered to have good potential for the discovery of massive sulfide (copper, lead, zinc) deposits.

Non-Metallic Minerals

Non-metallic minerals include a wide variety of rocks, minerals and other naturally occurring substances that are mined for their economic value, and it is this group of rocks and minerals that have dominated the North Carolina mineral industry since the turn of the century.

High quality primary, or residual, kaolin deposits occur in the Franklin-Sylva district and in the Spruce Pine District. These deposits were known as early as 1767, but systematic mining was not begun until 1888 near Webster in Macon County. Mica mining began on a large scale in western North Carolina about 1868 and it has been a principal producer of mica in the United States since that time. Sheet mica was the principal product for many years, but transistors have to a large extent eliminated many of the uses for sheet mica. No significant amounts of sheet mica have been produced in North Carolina since about 1962, but scrap and ground mica are still produced in large quantities.

Feldspar is another non-metallic mineral that has been mined in North Carolina for many years. Mining began in the Spruce Pine district in 1911 and is the principal feldspar producing district in North America.

Pyrophyllite, which is a high alumina mineral that occurs exclusively within rocks of the Carolina slate belt, was first mined commercially in North Carolina in about 1855 and it has been mined almost continuously since that time. Talc deposits, associated with crystalline limestones of the Murphy marble belt in Cherokee and Swain Counties have been mined since 1859.

Although first recognized as early as 1906, the economic significance of the spodumene bearing pegmatites of the Kings Mountain district was not realized until 1942. Today, this relatively small area in Cleveland and Gaston Counties, contains more than 80 percent of the known lithium ore reserves in the United States.

Olivine, a magnesium silicate mineral that has been used principally as a refractory material but is becoming increasingly more important as a molding sand in the foundry industry, occurs extensively throughout the Blue Ridge area. Vermiculite and anthophyllite asbestos also occur in the same rocks (dunites) as the olivine.

Clays of different kinds are found in varying amounts throughout the Coastal Plain, Piedmont and Mountain regions of the state. These clays provide the raw materials from which over one billion bricks are produced annually in North Carolina.

Crushed stone and sand and gravel annually account for over one half of the total mineral value produced in North Carolina. The Sandhills region in Anson, Moore, Lee, Harnett and Richmond Counties is the principal producing area of sand and gravel, but sand is mined from practically every county in the state. Granite and related crystalline rocks in the Piedmont and Blue Ridge geologic belts provide the principal sources of crushed stone in North Carolina. However, numerous quarries are located in the dense fine-grained rocks of the Carolina slate belt. Crystalline limestone and marble are quarried in limited amounts in the Piedmont and Mountain regions and shell limestone from the Castle Hayne Limestone is the principal source of crushed stone in the Coastal Plain. This formation also provides the high calcium raw material for the only cement plant in the State.

The most recent addition to the mineral industry in North Carolina is phosphate. A deposit of this material was found to underlie a large portion of Beaufort County in the early 1950's and subsequent exploration delineated a minable ore body that contains on the order of two billion tons of phosphate ore. Mining operations were begun by one company in 1965, and to date over 175 million dollars have been committed to this mining and chemical fertilizer products facility.

Other non-metallic minerals that have been mined in the past or considered to have potential for future mining, include corundum, garnet, barite, graphite, kyanite, sillimanite, beryl, columbite-tantalite, allanite-gadolinite, monazite, illmenite, zircon and uranium minerals.

Mineral Fuels

Mineral fuels provide the primary sources of energy currently consumed in the world and include coal, petroleum, natural gas and uranium. North Carolina is difficient in the mineral fuels and must import these resources from outside sources in order to meet its energy requirements.

Coal: - The only area in North Carolina that contains coal of potentially commercial value is the Deep River coal field which lies along Deep River in Chatham, Moore and Lee Counties. The coal is associated with sandstones and shales of Triassic age and occurs in beds from only a few inches to a maximum of 48 inches in thickness. The Cumnock seam, which averages about 40 inches in thickness, is the only minable coal present.

Coal has been mined intermittently from a number of underground mines developed in the Cumnock seam. However, the coal seam is deeply buried, badly broken by a number of faults, and there have been several major mine disasters. The most recent effort to mine coal from the Deep River field was between 1947 and 1953.

A detailed report on the Deep River coal field was published by the U. S. Geological Survey in 1955, and it estimated that the Deep River coal field contained 110,337,000 short tons of coal to a depth of less than 3,000 feet of which 55,170,000 tons were recoverable under conditions existing at that time.

Petroleum and Natural Gas: - The only area of North Carolina that is considered to have any potential for oil and gas production is the eastern one-third of the Coastal Plain and its continuation under the continental shelf. This area is underlain by a wedge-shaped block of sedimentary rocks that are largely of marine origin.

From 1925 until the present 112 wildcat exploration wells have been drilled in the onshore area of the State in the search for oil and gas. Although traces of gas have been detected in a few of these wells there have been no commercial quantities of oil or gas found to date.

Uranium Minerals: - A number of the uranium bearing minerals are known to occur as minor constituents in the pegmatites of North Carolina, principally in the Spruce Pine district and adjacent areas. Radioactive minerals have been found in schists and underlying granitic rocks in northern Burke, Mitchell, Avery and adjacent counties. However, none of these occurrences are large enough to be of commercial value.

Associated with the phosphate deposits in Beaufort County are trace amounts of uniformly distributed uranium. If this uranium could be recovered as a byproduct of the chemical fertilizer industry, North Carolina would have a significant uranium resource.

The Triassic basins and the Upper Cretaceous continental sediments may be favorable environments for the occurrence of uranium.

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CONSTRUCTION OF A REINFORCED EARTH FILL ALONG I-40 IN TENNESSEE

By

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Introduction

In October 1973, approximately two months after the initial row of concrete panels was set, the first reinforced earth structure to be constructed along a highway project in the southeast was completed. The structure, which is 831 feet long and 39 feet high at its highest point, spans one of the many landslide problem areas along Interstate-40 near Rockwood.

The slide, which occurred in January 1973, involved a section of sidehill fill between stations 2034+00 and 2042+00 along the east-bound-lane that had been in place for approximately four years (Figures 1 and 2). Fortunately, this particular section of roadway had not been paved at the time of failure. Sliding was attributed to pore-pressure buildup in a zone that extended from the base of the soil fill into the foundation material, which consisted of colluvium overlying highly weathered shale (Figure 3). The weight of the fill over the years no doubt increased the consolidation along the colluvium-residuum interface. This resulted in a constriction of the drainage channels in this zone which impeded percolation; thereby creating the slow buildup of pore-pressure that ultimately produced the failure.

Slope inclinometers installed at about the center of the slide (station 2038+00) revealed the failure plane to be at a depth of 38 feet near the roadway shoulder, 22 feet at the toe of the fill, and 24 feet approximately 100 feet below the toe of the fill. As in virtually all the other slides investigated along this alignment, failure occurred along the colluvium-residuum contact (Royster, 1973).

Similar failures had occurred at three other locations along the project. These were corrected by removing the failed mass to below the shear zone, replacing the lower portion with a rock

FIGURE 1. GROUND VIEW OF SLIDE CENTERED AT STATION 2038+00 OF THE EAST-BOUND LANE.

FIGURE 2. AERIAL VIEW OF THE 2038+00 SLIDE AS IT APPEARED IN JANUARY 1973.

pad, and then carrying the fill to grade with soil (Figure 4). The rock pad, which is made up of buttress-type rock, acts as the foundation for the fill while at the same time serving as an outlet for subsurface drainage and as a buttress against further upslope movement. This method, though quite effective, proved to be rather expensive, mainly because of the tremendous excavation quantities. For example, a slide involving 600 feet of roadway, just to the west of this failure in which this method was used, cost almost $600,000.00 to repair. Excavation alone amounted to slightly less than $250,000.00. Calculations using this method of repair indicated that the costs of the 2034+00 - 2042+00 slide would be in the area of $950,000.00. This prompted the Department to look at other alternatives. In addition to Reinforced Earth, the alternates considered were relocation and a gabion wall. Relocation was immediately ruled out because it would have involved realigning at least a mile of the roadway, and because it would also have meant excavating the median to just below the west-bound-lane; thus, jeopardizing sections of that portion of the roadway which had already been opened to traffic. The gabion wall was decided against because the costs would have been comparable to the first alternative-- that of removal and replacement--and because the design did not readily lend itself to the peculiarities of this particular problem.

Reinforced Earth--What it is

Reinforced Earth, the relatively recent creation of Henri Vidal, a French engineer, is a patented process wherein metal strips are strategically placed within a mass of essentially non-cohesive granular soil to form a gravity section capable of withstanding very high external loadings (Gedney, 1973). According to a company brochure on the subject, the principle of Reinforced Earth is simple: "independent individual granular elements of a Reinforced Earth body are linked together by the reinforcing strips due to the friction forces developed between the earth and the reinforcing material" (Technical Memorandum, 1971). Facing elements--either light metal shaped in a semi-elliptical cross section or interlocking concrete plates to which the metal strips are attached--retain the earth at the free end of the structure.

Application to the Rockwood Problem

As a result of a preliminary design and cost estimate prepared by the Reinforced Earth Company, the Department decided that Reinforced Earth would be a feasible approach to the

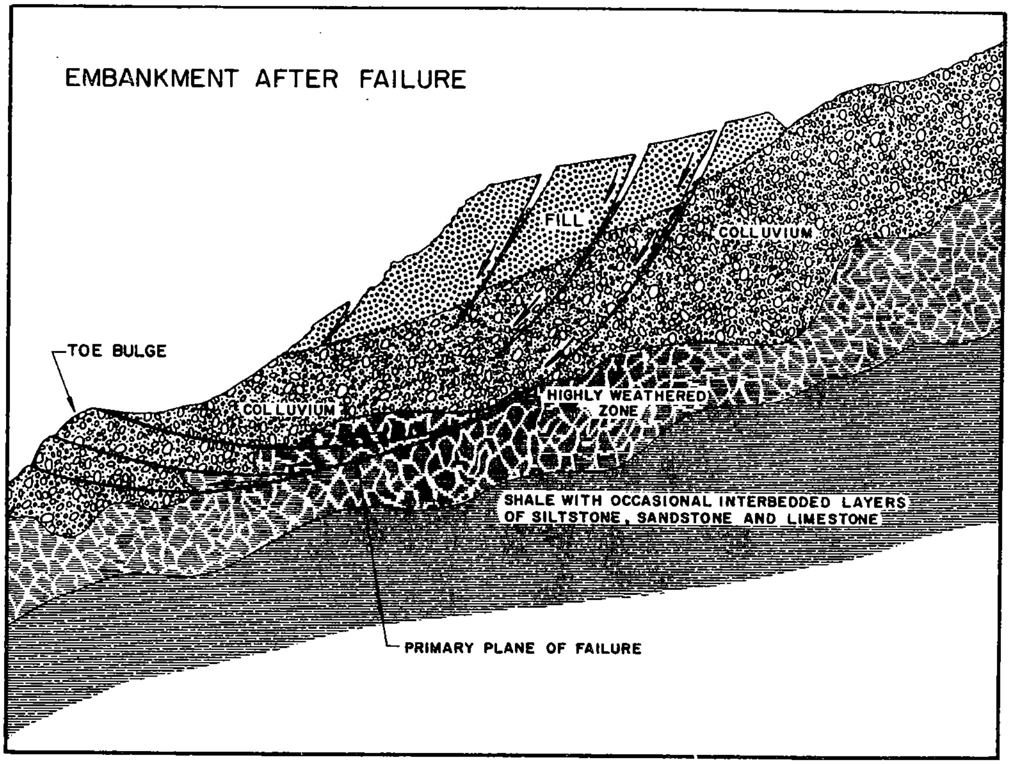

FIGURE 3. SCHEMATIC OF THE MATERIALS AND CONDITIONS OF THE 2038+00 SLIDE AT THE TIME OF FAILURE.

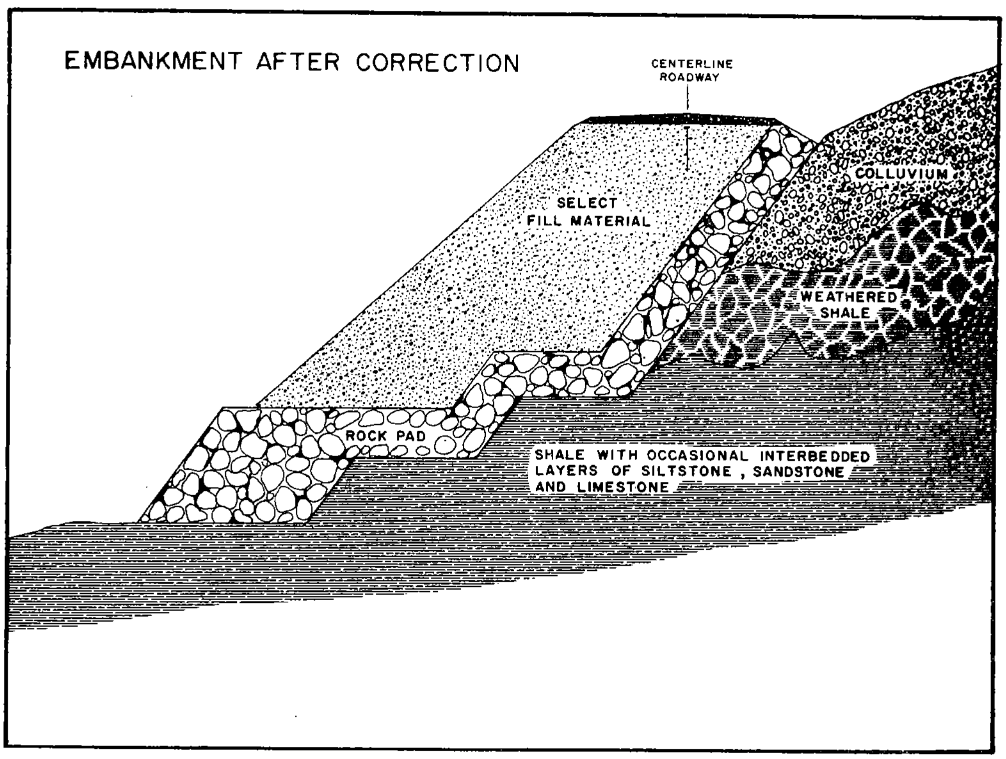

FIGURE 4. A SIMILAR FAILURE WAS REPAIRED USING THE METHOD SHOWN ABOVE.

problem if a nearby source of granular material could be located. As it turned out, the Division of Soils and Geological Engineering was able to locate a very excellent supply of material just off the right-of-way within approximately one mile of the project.

The plan was to excavate a trench longitudinally along the roadway through the full length of the failed interval. Excavation was to be accomplished on a 1:1 slope on both sides down through the shear zone into a foundation of relatively unweathered shale or siltstone. The trench was to be wide enough to accommodate the full roadway template, plus the width of a rock pad which would extend 5 to 10 feet beyond each side of the reinforced portion of the fill (Figure 5). The rock pad was incorporated into the design to serve as the foundation for the reinforced earth structure and to provide a more positive drainage outlet. The shale formation that underlies the area tends to slake and weather quite rapidly when exposed. The rock, in addition to providing a uniform foundation surface, was expected to offset some of the support lost through weathering by insuring a stronger keying and interlocking effect along the shale contact.

Materials and the Construction Sequence

As indicated earlier, the first order of business was to locate a source of granular material to serve as the reinforced volume. The material was obtained from a geologic formation known as the Sewanee Conglomerate which crops out approximately one mile west of the site. The Sewanee is a fine-to coarse-grained conglomeratic sandstone containing numerous quartz pebbles. At most localities it is fairly well cemented but at this particular location, due to its close proximity to a major fault zone and its steep angle of dip (60°- 85°), it contained several friable zones which permitted easy quarrying. The narrow hogback ridge along which the formation outcropped was drilled and cross sectioned and was found to contain approximately 30,000 cubic yards of usable material (the embankment required approximately 23,000 cu/yds.). The gradation recommended by the Reinforced Earth Company calls for 100% passing the 10-inch sieve, 75%-100% passing the 4-inch sieve, and not more than 15% passing the 200 mesh sieve. Two random samples from the outcrop had the following gradations:

	Total Percent Passing	
Sieve Size	Sample #1	Sample #2
1"	100%	100%
3/4"	100%	99%

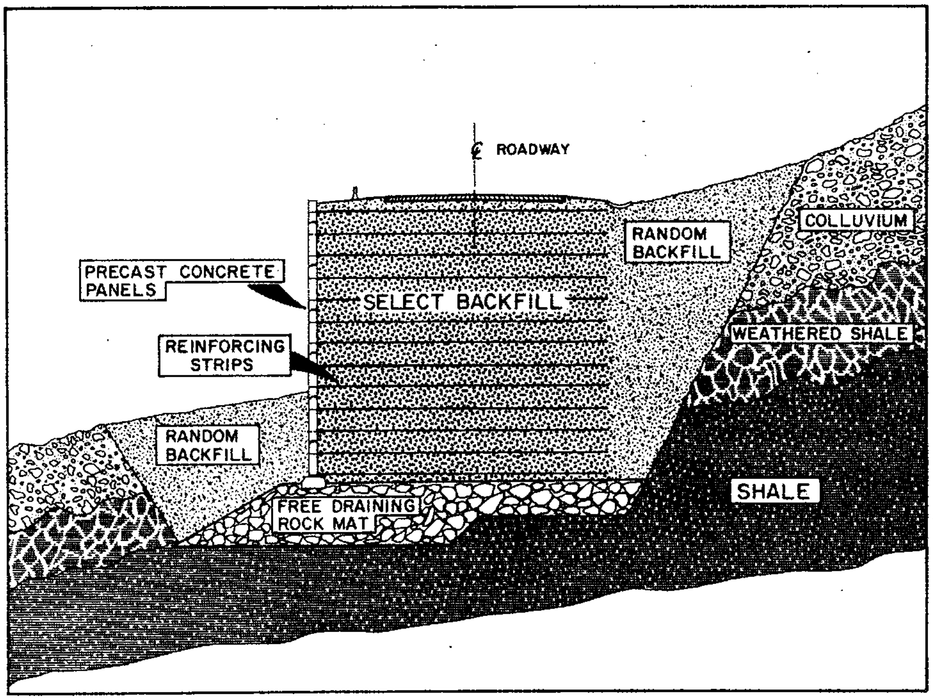

FIGURE 5. SCHEMATIC OF THE PLAN USED TO CORRECT THE 2038+00 FAILURE.

FIGURE 6. AERIAL VIEW OF THE BEGINNING OF THE TRENCH EXCAVATION AS IT APPEARED IN JUNE 1973.

Sieve Size	Total Percent Passing Sample #1	Sample #2
3/8"	92%	94%
4M	74%	86%
10M	65%	68%
40M	29%	20%
100M	12%	7%
200M	7%	5%

There were exceptions, of course, especially in the larger fractions, but probably 75% of the material by the time it was placed came very close to fitting the above gradations. No crushing or screening of the material was necessary. Chunks which did not break down to 10 inches or less in the shooting, loading, unloading, and spreading process were removed by hand prior to being compacted.

The excavation for the trench went along fairly smoothly. Some minor sloughing and slumping of the slopes occurred but due to the relatively dry periods in which excavation was carried out (June and July), no major problems were encountered (Figure 6). The rock pad was constructed of non-degradable limestone obtained from a nearby quarry. The specification called for 50% of the material to be greater than one cubic foot, with no more than 10% passing the No. 2 mesh sieve. It also had to be free of coal, shale and soil materials (Figure 7). The upper one to two feet of the pad was constructed of a finer gradation of stone to provide a uniform base on which to place the 1' X 0.5' concrete footing that supported the first row of panels. The footing, which serves merely to facilitate alignment of the panels both vertically and horizontally, was not required to be reinforced (Figure 8).

The design called for four different types of concrete panels, two types of which had different thicknesses. The following table lists the number of each of the four types used:

	Panel Schedule			
Thickness	"A"	"B"	"C"	"D"
7-1/8"	577	77	7	76
8-5/8"	281	-	70	-

The full-sized panels are about 5 feet in each dimension. The precise location of each type of panel was designated on the plans. The "C" panels, for example, were half sections, used

FIGURE 7. THE IN-PLACE ROCK PAD AS VIEWED IN AUGUST 1973.

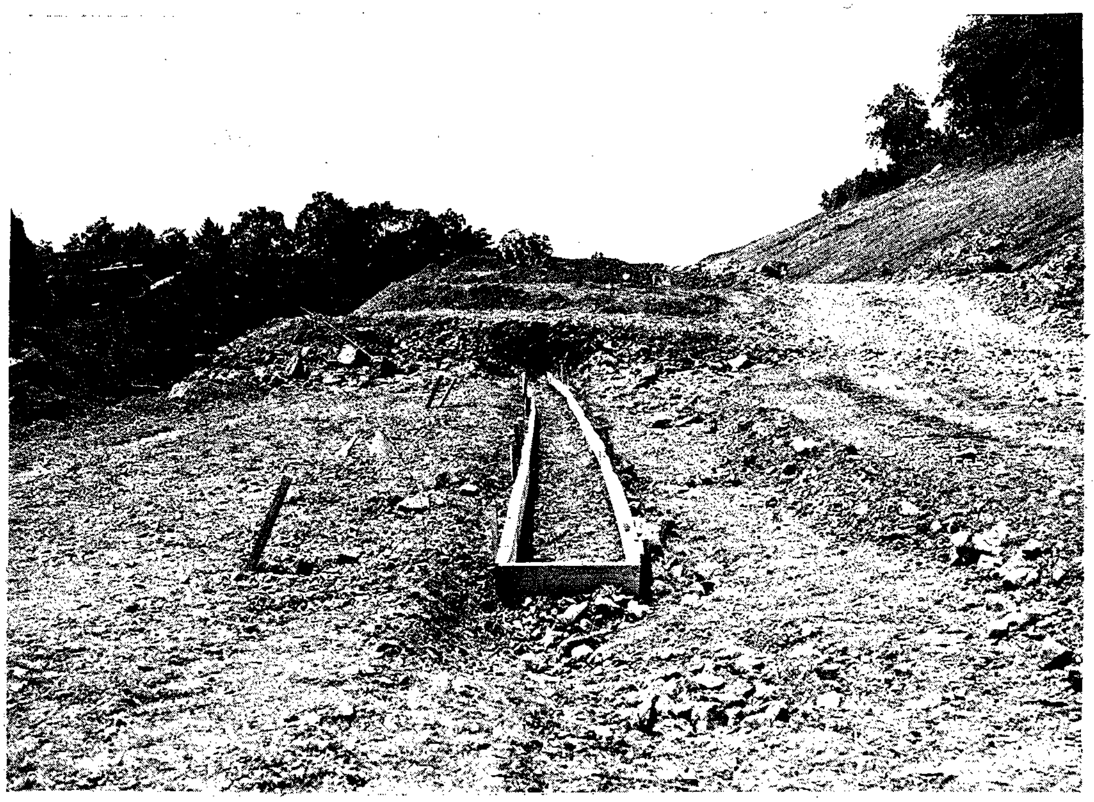

FIGURE 8. FORM FOR THE 1' X 6" LEVELING FOOTING ON WHICH THE FIRST ROW OF PANELS WILL BE SET.

at the base to fill in between the full-sized "A" panels (Figure 9). The "D" panels were the half sizes used between the type "B" panels at the top of the wall. The main differences between the various types, aside from thickness, are the grooving and interlocking features and in the number of tie points for the reinforcing strips (Figure 10). The thicker panels (8-5/8") were used along the lower portion of the wall.

Fourteen different lengths of galvanized reinforcing strips, ranging all the way from 9 feet to 32 feet, were used. The strips, which are approximately 3mm (1/8") thick, were of two different widths: 80mm (3.14") and 60mm (2.37"). The Reinforced Earth Company recommends that the strip lengths in the structure be at least 80% of the total structure height; hence, the 32 foot lengths in the nearly 40-foot high wall.

The reinforcing strips were attached to the tie points on the panels with two 1" X 1/2" bolts (Figure 11) and then stretched to their full lengths across the fill (Figure 12). As with the panels, a schedule and detailed plan for the placement of the reinforcing strips was provided (four thousand seven hundred and ninety-seven strips, totaling 117,986 feet were used on the project). Each row of strips was then covered with approximately 30 inches of sand placed in 8-10 inch compacted lifts (Figure 13). Polyurethane foam strips (2" X 2") were used to seal the vertical joints between the panels, and strips of cork four inches wide by one inch thick were used along the horizontal joints. Each succeeding row of panels were kept in a plumb position by clamps and wedges until the reinforcing strips could be attached and covered with the select backfill material (Figure 14).

Compaction was accomplished with a heavy vibratory roller along the main body of the fill (Figure 15) and a power hand tamper near the panels (Figure 16). Extreme care had to be exercised to prevent bending the tie-points on the panels during the tamping operation. Maximum laboratory density of the sand (AASHO T-99) averaged 115 pcf at 16% moisture. The required field density was 95% of maximum laboratory density, and was determined with a nuclear gauge. As is usually the case with sand, the application of large quantities of water was necessary to obtain the required density.

To prevent erosion and slumping of the sand into the void between the end of the reinforced portion of the fill and the inside trench slope, it was necessary to carry the random backfill material up to grade along with the select backfill

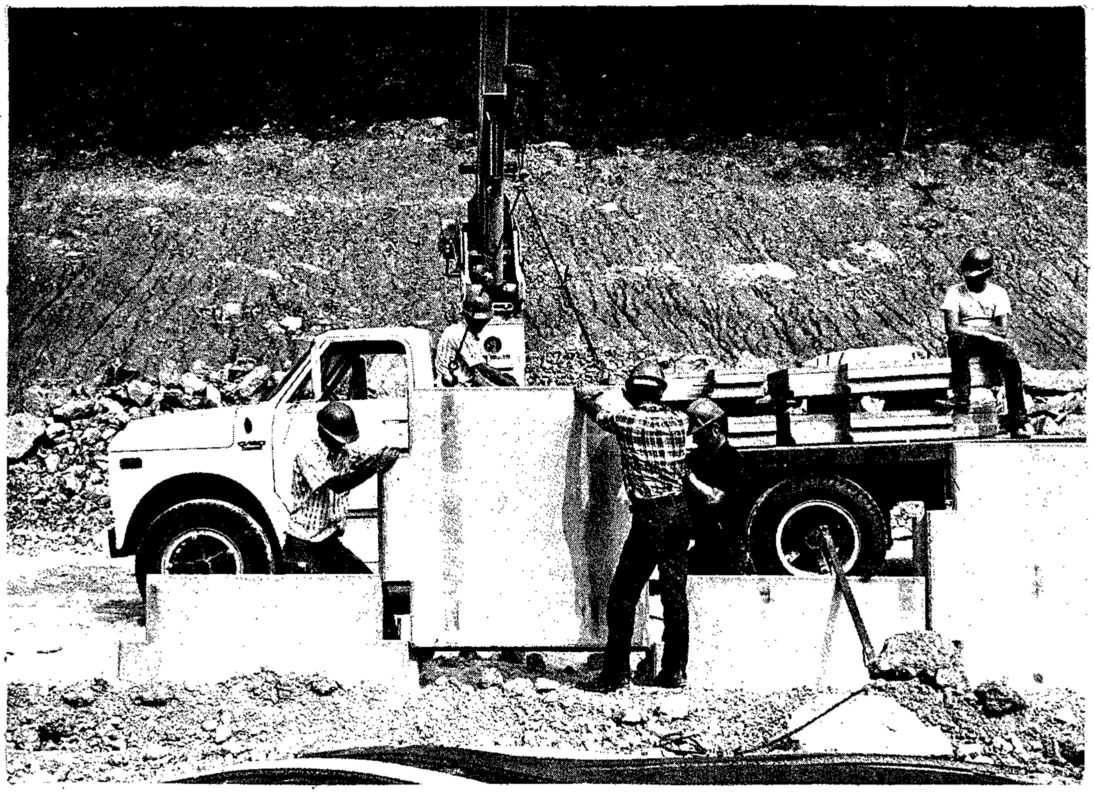

FIGURE 9. THE FIRST ROW OF PANELS BEING SET ON AUGUST 29, 1973.

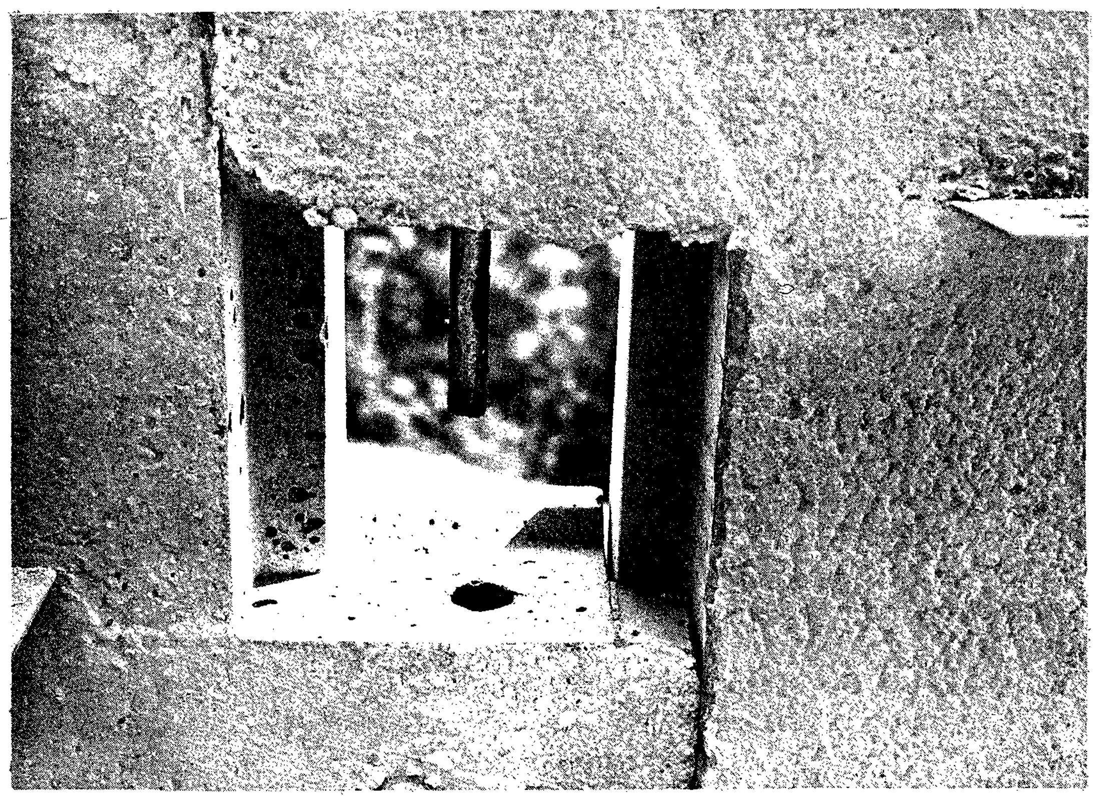

FIGURE 10. CLOSE-UP OF THE INTERLOCKING FEATURES OF THE PANELS.

FIGURE 11. THE REINFORCING STRIPS ARE ATTACHED TO THE PANELS WITH TWO 1/2" X 1" BOLTS.

FIGURE 12. THE REINFORCING STRIPS ARE 1' TO 3' APART HORIZONTALLY AND 1 1/2' TO 3' VERTICALLY.

FIGURE 13. THE SAND IS SPREAD AND COMPACTED IN 8-INCH TO 10-INCH LAYERS.

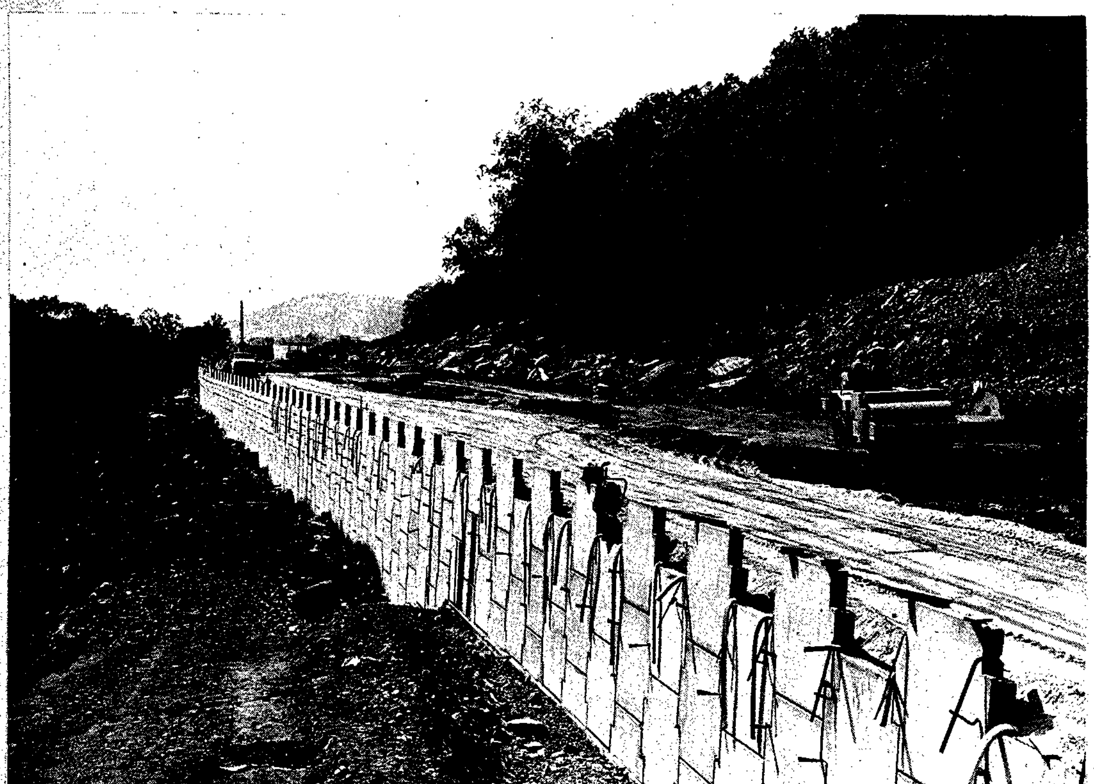

FIGURE 14. THE UPPER PANELS ARE KEPT IN A PLUMB POSITION BY WEDGES AND CLAMPS UNTIL THE REINFORCING STRIPS ARE ATTACHED.

FIGURE 15. COMPACTION WAS ACCOMPLISHED WITH A VIBRATORY ROLLER TO AT LEAST 95% OF THE MAXIMUM LABORATORY DENSITY.

FIGURE 16. COMPACTION NEAR THE WALL WAS ACHIEVED WITH A GASOLINE POWERED HAND TAMPER.

(Figure 17). Once the elevation of the wall approximated the crest of the outside trench slope, the trench was filled in and graded against the lower portion of the wall to a 5:1 slope (Figure 18).

Drainage in the area of the structure was accomplished with two drains; one, a 24-inch perforated metal pipe below the wall at station 2039+27 to handle the subsurface flow from a wet weather spring, and the other, a 30-inch pipe through the upper part of the wall, to carry the surface runoff (Figure 19). It was deemed preferable and more economical to carry the surface water in this manner rather than to construct a deep catch basin for collection and removal under the wall.

Conclusion

As stated in the introduction, the construction of the reinforced earth structure itself required approximately eight weeks. This included almost two weeks of "slow time" while waiting for delivery of additional panels and cork. Weather was not a factor, because once the wall was started it could be worked in spite of some of the light rains that fell during the construction period. Although the contractor had no previous experience in Reinforced Earth construction, no significant problems were encountered. A technical representative from the Reinforced Earth Company was on hand for the first several days to train the construction personnel and to assist in getting the project started. He returned to the site on at least two other occasions to review the progress.

The final cost estimate for the project, excluding paving, etc., is as follows:

Excavation	43,808 cu/yds at $1.97	=	$ 86,301.76
Rock Pad	4,740 cu/yds at $6.00	=	$ 28,440.00
Random Backfill	16,257 cu/yds at $1.97	=	$ 32,026.29
Select Backfill	22,811 cu/yds at $4.50	=	$102,649.50
R/E Package*	24,248 sq/ft at $8.55	=	$207,320.40
	Total	=	$456,737.95

*Includes panels, reinforcing strips, and erection

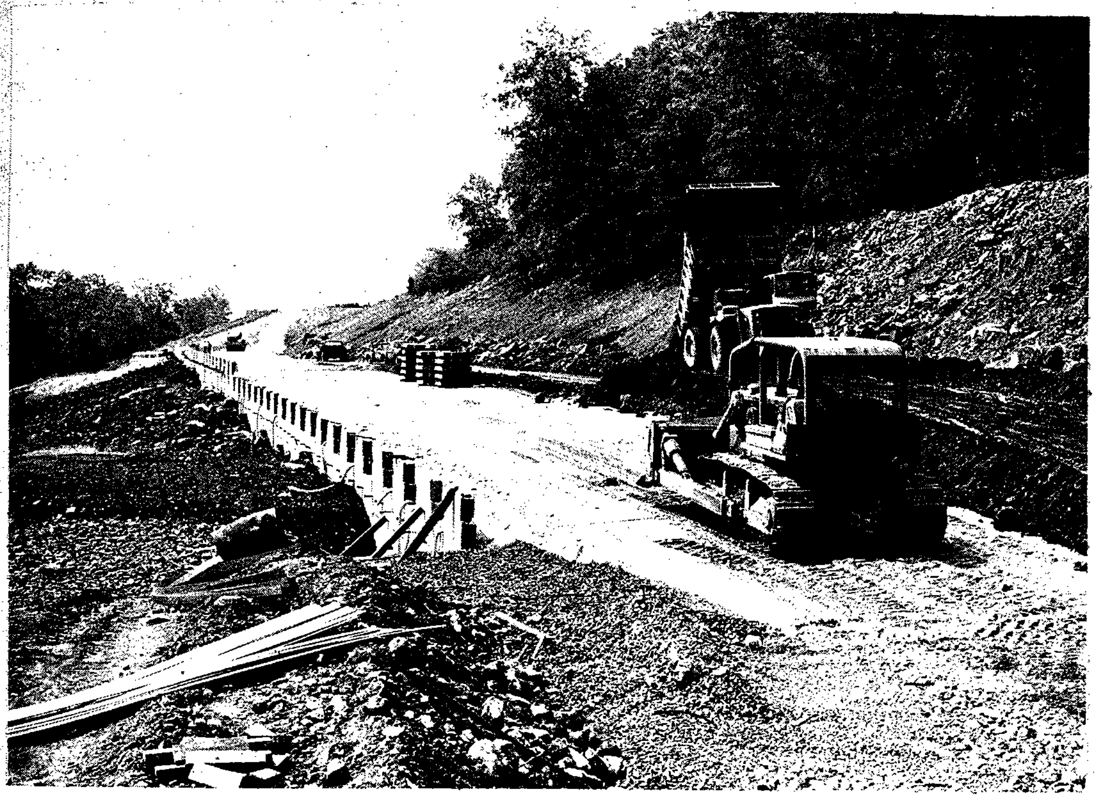

FIGURE 17. THE RANDOM BACKFILL WAS CARRIED TO GRADE ALONG WITH THE REINFORCED VOLUME.

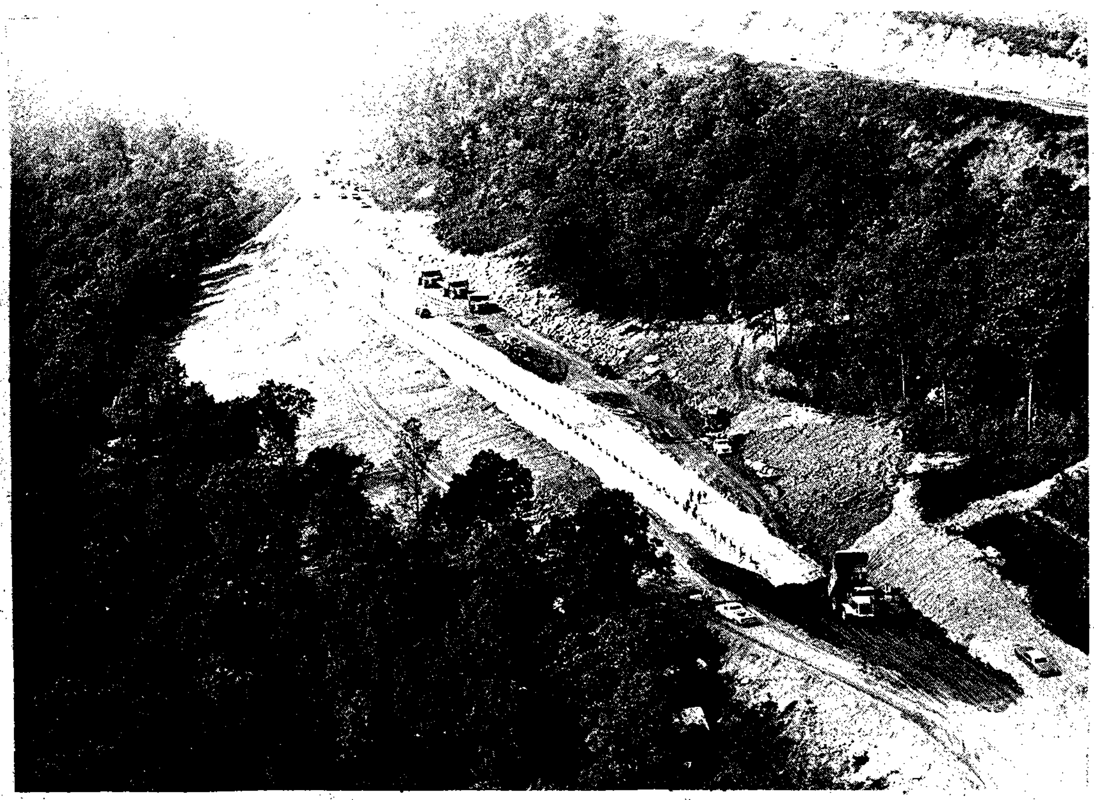

FIGURE 18. THE OUTER TRENCH SLOPE WAS FILLED IN AGAINST THE BASE OF THE WALL ON A 5:1 SLOPE RATIO.

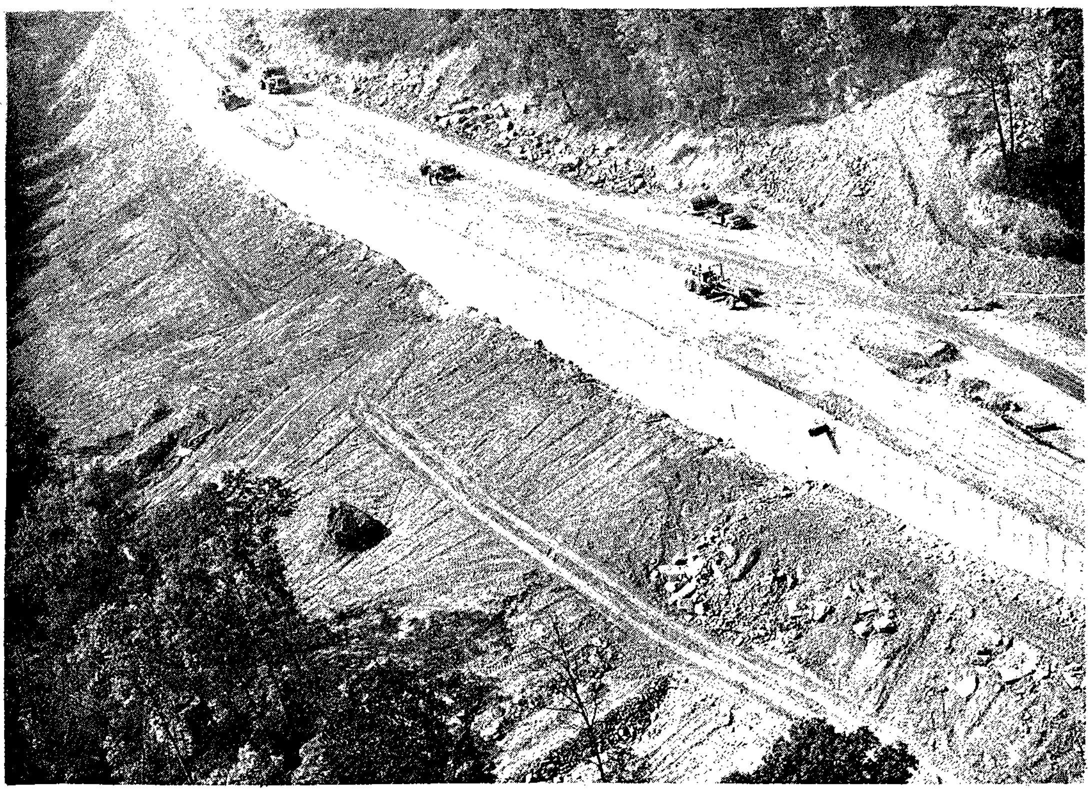

FIGURE 19. TO AVOID CONSTRUCTING A DEEP CATCH BASIN ON THE UPPER SIDE OF THE STRUCTURE, DRAINAGE WAS CARRIED THROUGH THE WALL IN A 30" PIPE.

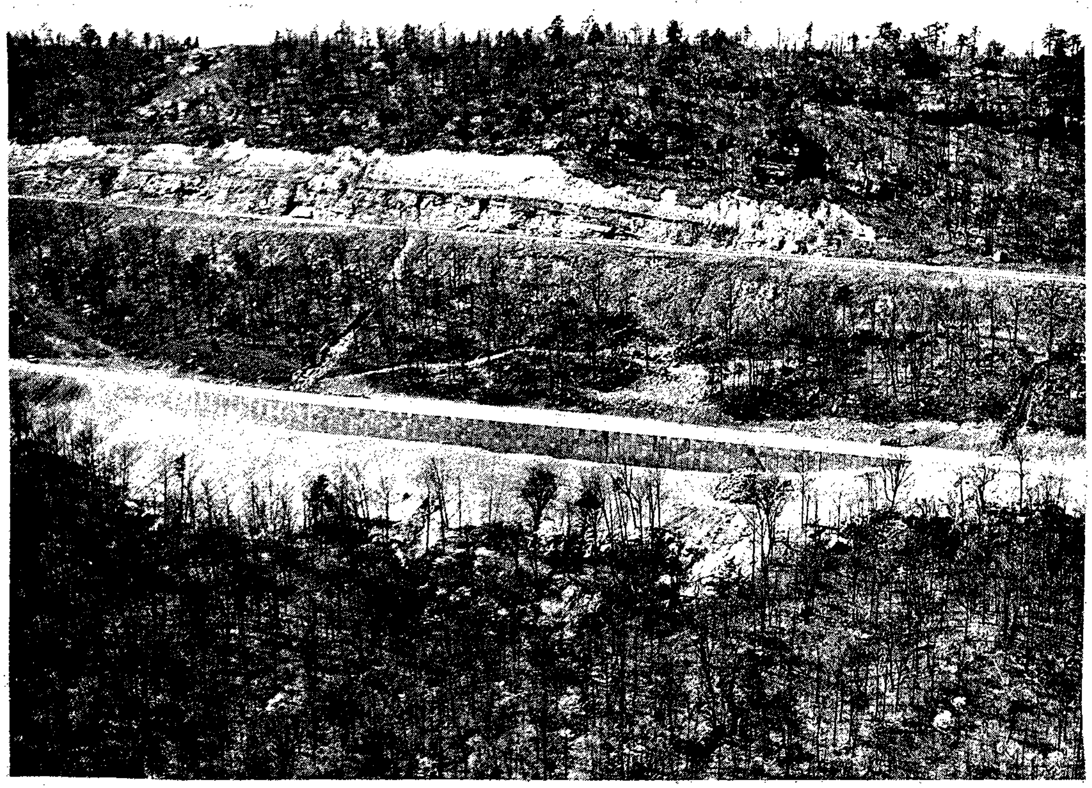

FIGURE 20. LONG RANGE VIEW OF THE COMPLETED STRUCTURE.

Since the roadway has not been opened, it is not possible to provide a report on the performance of the structure under traffic. However, as of the writing of this paper (May 1974), no appreciable deflections or settlements have been noted. If there are weaknesses, they almost certainly have to be in the transition areas at the ends of the structure. This is not to say that the structure is inherently weak at these points, but rather that the embankments themselves may lack the desired stability. Our experience has been that no fill along this alignment is without suspect that has been constructed over the colluvium-residuum sequence described previously.

From what we have observed to date, Reinforced Earth is a highly acceptable alternative in the correction of landslides of the type and with the conditions that have been described. Like any other new method, however, only time can provide the real test.

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ENVIRONMENTAL ATLAS OF THE TEXAS COASTAL ZONE AND ITS ROLE IN LAND USE PLANNING

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INTRODUCTION

The Texas Coastal Zone is marked by diversity in geography, resources, climate, and industry. It is richly endowed with extensive petroleum reserves, sulfur and salt, deepwater ports, intracoastal waterways, mild climate, good water supplies, abundant wildlife, commercial fishing resources, unusual recreational potential, and large tracts of uncrowded land. The Coastal Zone is a vast area of about 20,000 square miles, including approximately 2,100 square miles of bays and estuaries, 375 miles of Gulf coastline, and 1,425 miles of bay, estuary, and lagoon shoreline. About one-quarter of the State's population and one-third of its economic resources are concentrated in the Coastal Zone, an area including about six percent of the total area of the State.

The Texas shoreline is characterized by interconnecting natural waterways, restricted bays, lagoons, and estuaries, low to moderate fresh-water inflow, long and narrow barrier islands, and extremely low astronomical tidal range. Combined with these natural coastal environments are bayside and intrabay oil fields, bayside refineries and petrochemical plants, dredged intracoastal canals and channels, and satellite industries. The attributes that make the Texas Coastal Zone attractive for industrialization and development also make it particularly susceptible to a variety of environmental problems. The Texas Coastal Zone is thus balanced between maintenance by natural physical, chemical, and biological processes, and effects of industry, urban concentration, and coastal land development.

Parts of the Coastal Zone are among the fastest developing industrial, urban, and recreational regions in Texas; the Zone is at best a precariously balanced natural complex of dynamic environments with a history of almost yearly hurricane impact. Adequate plans to meet the potential problems of pollution, land and water use, and conservation are critically needed to insure proper development of this vital Texas region. A regional analysis and inventory of the total coastal resources of Texas is vitally important and must be based on accurate maps of physical and biological environments, landforms, areas of significant processes, genetic sedimentary or substrate units, and man-made features. The Environmental Geologic Atlas of the Texas Coastal Zone (fig. 1) is designed to present information on the nature of the Coastal Zone, what is happening to it, and at what rate changes are taking place. Such information is needed for long-range resource planning and management. Mapping is the fundamental base necessary to provide answers to these critical questions.

Role of Environmental Geology in the Coastal Zone

Development of guidelines for proper and prudent management of the Texas Coastal Zone depends upon adequate knowledge of the nature and distribution of natural environments, land and water capability, and man's impact on the Coastal Zone. Processes and environments are a fundamental part of the geological character of this dynamic region. Many areas of the Coastal Zone are changing under man's accelerating impact. Because the Zone is balanced in terms of erosion and deposition, hurricane impact damage, salinity variations within bays and estuaries, plant stabilization of sediments, and a myriad of other critical features, man's impact can significantly affect the natural environmental balance. At the same time, the necessity of resource use in man's modern industrial society is obvious. Development, exploitation, and industrialization practices, however, should be compatible with the natural limitations imposed on the region by its physical, chemical, and biologic setting.

Regional climatic, sedimentary, biologic, and physical process variations along the Texas Coast clearly preclude a rigid coastwide system of resource management. Any fair system of management must be based upon the concept of natural local and regional variation of environments; correspondingly, flexible guidelines should be firmly based upon these variations in properties, composition, and behavior under various land uses. Environmental geologic maps provide the fundamental data needed to create such a system of resource management.

One principal goal of the Environmental Geologic Atlas of the Texas Coastal Zone is to obtain an understanding of the natural systems *before* human impact irreversibly changes the character of the Zone. Only by the understanding of the natural coastal system can proper and

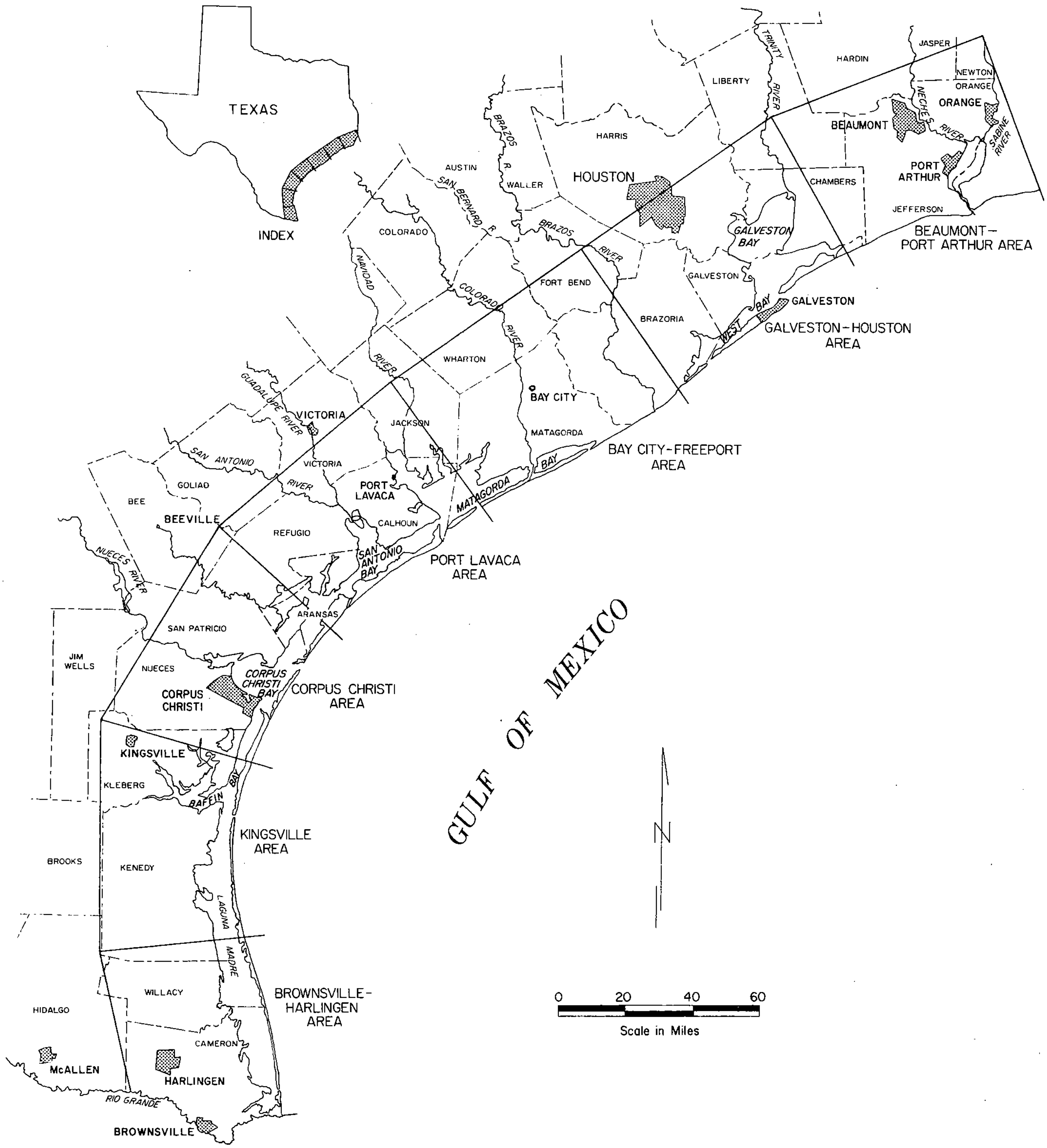

Figure 1. Index of the Environmental Geologic Atlas of the Texas Coastal Zone. After Fisher and others, 1972.

compatible use of the region be determined. Maps of environmental units along the 400-mile-long Coastal Zone provide a benchmark with which to evaluate future changes and to diagnose appropriate use of the coastal regime.

Wise conservation should include the proper use of Coastal Zone resources within prudent guidelines that will insure minimum modification of the environmental quality of the region. For this reason, each kind of land use should be evaluated in terms of its potential effects on the geological and biological units of the Zone. Proper use will result when each of man's coastal activities are properly located in such a manner as to insure necessary minimum environmental damage.

The key to proper land and water use is the basic inventory of the coastal environments, sediment types, processes, and biological conditions. The Environmental Atlas provides this fundamental information that can serve as the basis for evaluating coastal legal problems, socioeconomic problems, industrial development, pollution, recreational needs, problems of public and private ownership, and other factors involving the natural framework of the Coastal Zone.

Several aspects of the Texas Coastal Zone make a long-term resource management program imperative; in turn, this requires a thorough knowledge of the environmental geology of the Coastal Zone. Since the Coastal Zone is the center of rapid geological and physical changes coupled with a rapidly expanding population, an Environmental Atlas provides a current record of the status of dynamic coastal environments and processes, as well as a permanent record of exploitation, erosion, and human modification. Dynamic environments can be monitored by periodic mapping that indicates the significant direction and approximate rate of physical, biological, and chemical changes. The environmental map is the common denominator for communication among coastal scientists through which technical input can be integrated and applied. Just as important, economists, planners, utilities specialists, power suppliers, sanitary engineers, lawyers, legislative councils, regional councils of governments, and many other groups can better plan, plot, refer, and digest environmental data using the Atlas maps.

The Coastal Environmental Atlas Project

The Environmental Geologic Atlas project was initiated in 1969 when the need for a thorough regional analysis of natural processes, environments, lands, water bodies, and other coastal factors became urgently apparent. Without an adequate environmental inventory, further specialized scientific studies, as well as regional planning for improved use of coastal resources, could proceed neither efficiently nor effectively. Because of impending environmental problems in the region, staff members of the Bureau of Economic Geology assigned the project a high priority and proceeded with the mapping in the summer of 1969. Approximately 25 man-years of geologic and cartographic effort were expended in the four-year period of preparation.

The Coastal Zone, defined from the inner continental shelf to about 40 miles inland, includes all estuaries and tidally influenced streams and bounding wetlands. For purposes of presentation, the Zone was divided into seven areas (fig. 1) from the Texas-Louisiana boundary southwestward to the Rio Grande: (1) Beaumont-Port Arthur, (2) Galveston-Houston, (3) Bay City-Freeport, (4) Port Lavaca, (5) Corpus Christi, (6) Kingsville, and (7) Brownsville-Harlingen. Each of these seven coastal areas is covered by a separate Environmental Geologic Atlas (fig. 2) containing a descriptive text, statistical tables, an environmental geology map (scale 1:125,000), and eight special-use environmental maps (scale 1:250,000). The seven Coastal Atlases cover approximately 20,000 square miles.

GEOLOGY AND GEOLOGIC HISTORY

The Texas Coastal Zone is composed of several active, natural systems of environments—fluvial and deltaic systems, marine barrier-strandplain-chenier systems, and bay-estuary-lagoon systems, as well as an eolian (wind) system in South Texas and marsh-swamp systems in the more humid middle and upper coastal regions. Geologists are also aware that the Coastal Zone is entirely underlain by sedimentary deposits that originated in ancient but similar coastal systems. These ancient sediments were deposited by the same natural processes that are active in shaping the present coastline; for example, longshore drift, beach swash, wind deflation and deposition, tidal currents, wind-generated waves and currents, delta out-building, and river point bar and flood deposition.

Active and relict coastal systems (fig. 3) may be divided into three principal groups based on their relative ages: (1) natural systems that originated more than about 30,000 years B.P. (before

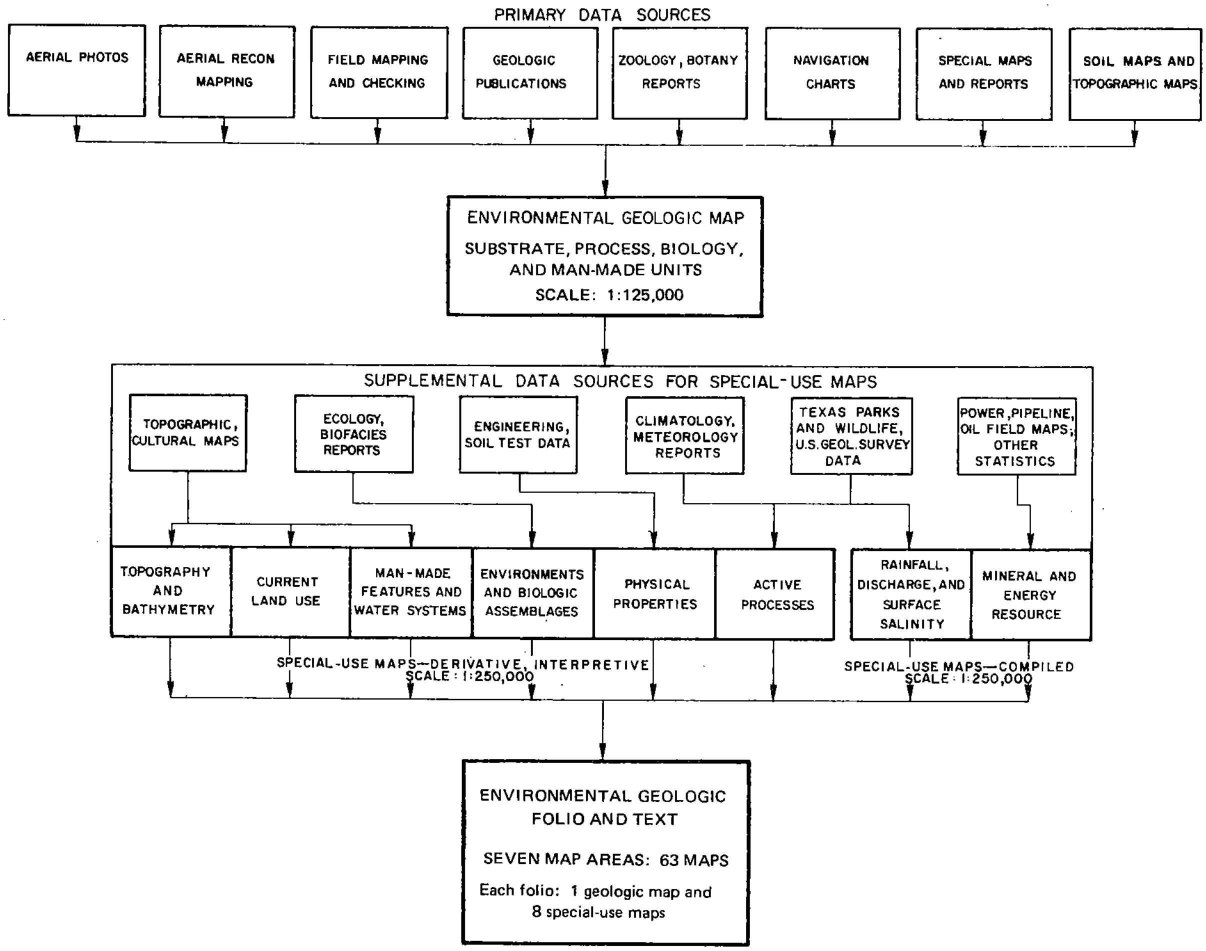

Figure 2. Sources and flow of data for the Environmental Geologic Atlas of the Texas Coastal Zone. After Fisher and others, 1972.

present) during interglacial periods of the *Pleistocene* ice age; (2) natural systems termed *Holocene* that originated between approximately 18,000 and 4,500 years B.P.; and (3) natural systems herein termed *Modern* that have been developing since about 4,500 years B.P. and are currently active.

Modern coastal systems are characterized by a distinctive suite of natural environments in which certain geologic processes result in deposition of unique sedimentary deposits. These deposits are similar in every respect to older sedimentary deposits of Pleistocene or Holocene age and, therefore, can be interpreted to have originated within genetically similar but ancient environments. For example, Modern river or fluvial systems are composed of levee, point bar, and flood-basin environments, in which certain types of sediment are deposited by specific geologic processes. Similarly, point bar, levee, and flood-basin deposits of Pleistocene or Holocene age can be interpreted as having been deposited in similar environments within an ancient river system.

A knowledge of processes that are active within Modern environments is critical if the environmental impact of various types of human activity is to be evaluated. Stated simply, natural environments must be properly understood if they are to be managed and protected. Just as important environmentally, but perhaps less obvious to most citizens, is an understanding of the ancient sedimentary substrates underlying the Coastal Zone. These relict deposits of ancient systems of coastal environments determine to a great extent the suitability of coastal lands for various uses and human activities. Similarly, the sedimentary

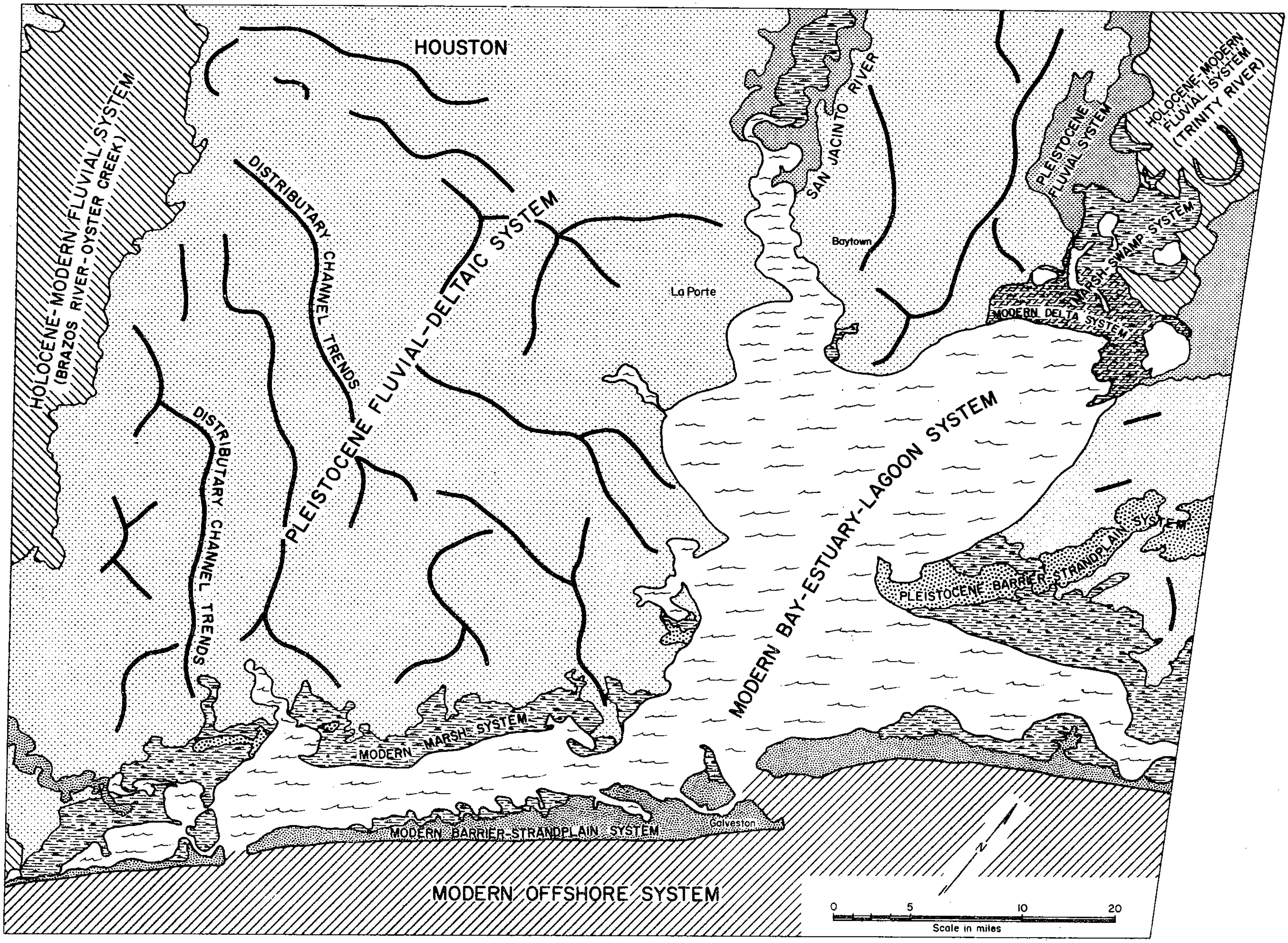

Figure 3. Natural systems defined by environmental mapping in the Galveston-Houston area. These systems are composed of genetically related environments, sedimentary substrates, biologic assemblages, areas of significant physical processes, and man-made features. Simplified from the Environmental Geology Map of the Atlas. After Fisher and others, 1972.

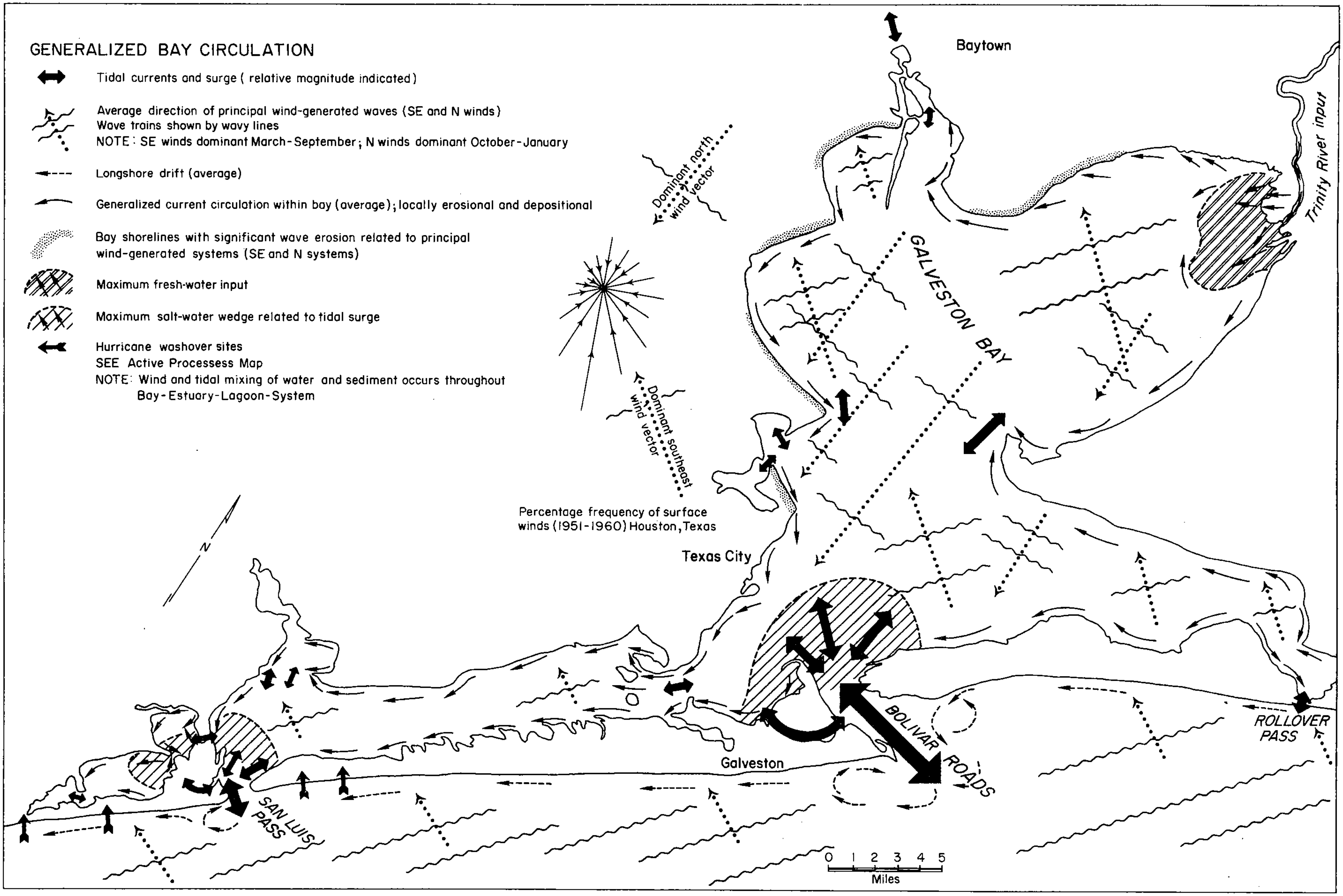

Figure 4. Circulation, waves, sediment transport, and other physical processes, bay-estuary-lagoon system, Galveston-Houston area. After Fisher and others, 1972.

deposits of these older Pleistocene and Holocene systems dictate the nature of soils, wildlife, vegetation, ground water, natural resources, and all manner of aspects that are important to the environmental quality of the region. For these reasons, it is critical that the nature of the environments, processes, and sediment substrates for all *active* coastal systems and all relict sedimentary substrates for all *ancient* coastal systems be determined and mapped so that a scientific basis for environmental management can be developed.

A principal goal of the Environmental Geologic Atlas of the Texas Coastal Zone is to present the nature of active environments and relict sedimentary deposits. An appreciation of the geologic history of this dynamic region will enable the reader to envision the sequence of geologic events that has created and shaped the present Texas Coastal Zone. The geography of the region has evolved slowly through time as climate, sea level, and other environmental factors have changed. The present Coastal Zone is, therefore, but one frame in a kaleidoscope of changing rivers, shifting beaches, and subsiding plains. Past geologic events and current geologic processes join in characterizing the nature of the *total* coastal environment, as well as to point inevitably to future changes that man must learn to understand, predict, and manage. In short, the Coastal Zone is characterized by natural change; man's activities may significantly affect the rate and direction of these changes.

CLIMATE AND DYNAMIC COASTAL PROCESSES

The climate of the Texas Coastal Zone strongly dictates the relative importance of many significant geological processes. A principal factor is the direction and intensity of persistent winds that control the orientation and size of wave trains approaching the shoreline (fig. 4). In turn, the angle at which waves strike the coast affects the nature of longshore drift. The direction of wind-driven currents and waves in relationship to the orientation of tidal passes may increase or diminish the magnitude of astronomical tides that coincide with the wind activity. The amount of open-bay fetch and the direction of wind-driven tides within a bay also control the effectiveness of wind-tidal activity; for example, broad fetch and persistent wind aligned with the axis of a narrow, funnel-shaped bay result in high wind tides. The angle at which hurricanes strike the coast, likewise, affects the magnitude of floodtides, especially in narrow upper bay areas. The duration and intensity of winds control the nature and direction of bay currents that erode, transport, and deposit sand and mud. Bay shorelines strongly reflect the depositional or erosional character of currents, just as longshore drift smooths the seaward side of strandplains and barrier islands.

Just as important as wind in controlling coastal processes is the combined and interrelated effect of rainfall, evaporation, and temperature. Effective precipitation controls the nature and density of coastal plants, which are critical in a climatic regime where wind is a primary factor. Plants stabilize coastal sands that, if unvegetated, will be deflated and transported as eolian or wind dunes. The density of vegetation is especially critical in stabilizing and shielding coastal barriers and shorelines against hurricane impact. Effective rainfall and associated plant cover also stabilize inland soils.

ENVIRONMENTAL GEOLOGY MAP

Environmental geology units for the entire Coastal Zone (fig. 1) were interpreted from and plotted on three hundred and twenty 7.5-minute Edgar Tobin Aerial Surveys photomosaics and corresponding U. S. Geological Survey topographic maps, both at a scale of 1:24,000, or approximately 2.5 inches per mile. All environmental maps are printed on a regional base map of the Coastal Zone constructed especially for the Atlas by the Bureau of Economic Geology. The base map was compiled from 7.5-minute U. S. Geological Survey quadrangle maps; 5-foot topographic contours, available bathymetric contours, updated culture, and all paved roads are included.

Mapping involved extensive aerial photographic interpretation, field work, aerial reconnaissance, and utilization of available published data for the region. General sources and flow of data used in mapping are shown in figure 2; specific sources of data are noted in the text. Interpretation and mapping of environmental geologic units were based on a genetic grouping of the major natural and man-made features of the Coastal Zone. Units mapped were interpreted to be of first-order importance to the environmental character of the Zone. First-order environmental units include the following: (1) a wide variety of sedimentary substrates (sand, mud, shell) and associated soil units displaying distinct properties and composition; (2) units displaying a variety of natural processes, including storm channels, tidal passes,

Table 1. Environmental geology map units, Texas Coastal Zone.

HOLOCENE-MODERN SYSTEMS

Fluvial-Deltaic Systems

Meanderbelt sand, tree-covered, inactive, entrenched stream
Meanderbelt sand, prominent grain, inactive, non-entrenched stream
Fluvial sand and flood-basin mud, undifferentiated, inactive, non-entrenched stream
Meanderbelt sand and silt, sparsely grass- and shrub-covered, inactive within an entrenched stream
Interdistributary silt and mud, locally includes bay, lacustrine and crevasse splay
Flood basin, overbank mud and abandoned channel, mud-filled, inactive, within entrenched stream
Levee deposits, fresh-water, marsh-covered
Levee and crevasse splay deposits, silt, sand and mud, grass-covered
Levee and crevasse splay deposits, silt, sand and mud, tree-covered
Delta-plain mud and sand, sparsely grass-covered
Delta-plain mud and sand, grass-covered
*Delta front and channel-mouth bar sand shoal (active and abandoned)
*Prodelta mud and silt
Point bar, sand, tree-covered, along active streams
Point bar, sand, bare or sparsely vegetative, along active streams
*Abandoned channel and course, mud-filled
*Abandoned channel and course, swamp-covered, mud-filled
*Abandoned channel and course, fresh-water, marsh-covered, mud-filled
*Marsh, salt-water, mud and locally sand substrate
*Marsh, fresh- to brackish-water, mud and locally sand substrate
*Marsh, fresh-water, mud and locally sand substrate
*Swamp, mud and locally sand substrate
*Tidal creek, fresh- to brackish-water, marsh-covered, mud-filled
*Tidal creek, mud-filled
*Wind-tidal flat, sand and mud, transitional between bay and stream
Small active, headward-eroding stream, tree-covered to barren
*Berm and beach ridge, abandoned, sand and shell
*Fan and fan delta, sand, subaerial

Barrier-Strandplain-Chenier and Offshore Systems

Shelf mud and sand, mottled
Shoreface, mud, burrowed
Shoreface, sand and muddy sand, burrowed
Coastal mudflat, subject to tidal inundation
Beach, sand and shell
Barrier flat, sand and shell, very sparse grass
Beach ridge and barrier or strandplain flat, grass-covered, sand and shell
Sandflats and/or coppice sand-dune fields, active
Fore-island blowout dunes and back-island dunes, sand, active
Fore-island dune ridge, sand
Chenier beach ridge, sand, grass-covered
Chenier flat, sand and shell, grass-covered
Strandplain-barrier oak motte
Stabilized blowout dune complex, sand, grass-covered, hummocky
Washover channel, sand-filled, inactive
Washover channel, sand, active
Washover fan, sand, subaerial, vegetated
Washover fan, distal, sand, subaerial, barren, active
Ebb-tidal delta, mud and sand, subaqueous
Ebb-tidal delta, sand, subaqueous, proximal to channel
Ebb-tidal delta, mud and sand, subaqueous, distal to channel
Flood-tidal delta, mud, subaqueous
Flood-tidal delta, mud and sand, subaqueous, distal to channel
Flood-tidal delta, sand, subaqueous, proximal to channel
*Tidal channel, mud and some sand, active
Tidal channel, sand, active
Tidal channel, mud- and sand-filled, inactive
Inlet-related shoal, sand, accretionary in passes
Wind-tidal flat, subaerial, burrowed
Tidal flat, sand
*Swale between beach ridges, mud-filled
*Swale between beach ridges, fresh- to brackish-water, marsh-covered, mud-filled
*Marsh, salt-water, mud and locally sand substrate
Back-island sandflats with small migrating dunes, unvegetated
Wind-deflation trough and storm runnel on barrier flat, sand

Marsh-Swamp System

*Marsh, salt-water, mud and locally sand substrate
*Marsh, brackish-water, closed system, mud and locally sand substrate
*Marsh, fresh- to brackish-water, mud and locally sand substrate
*Coastal lake or pond, partially or completely mud-filled
*Marsh, fresh-water, and poorly drained depressions, mud and locally sand substrate
*Swamp, mud and locally sand substrate

Bay-Estuary-Lagoon System

*Fan and fan delta, sand, subaerial, along bay margin
*Berm or beach ridge, abandoned, sand and shell (P/M)
Bay- or lagoon-margin sand, locally with mud and shell, subaqueous
Bay- or lagoon-margin sand and mud; shell berms, beaches and spits, active, subaerial
Bay-margin oolites and quartz sand
Bay-margin quartz sand and calcite-coated grains
Sand shoal with some oolites
Bay and bay-margin sandy mud, mottled, some shell
Grassflat, muddy sand with shell
Bay sand with mixed shell
Bay and lagoon mud, mottled, some mixed shell
Bay sand with oyster shell
Bay mud with oyster shell
Bay sand
Bay mud and silt
Bay and lagoon sand, muddy
Bay mud, laminated, rare shell
Bay sand and muddy sand, locally with oyster shell
*Prodelta mud and silt
*Delta front and channel-mouth bar sand shoal (abandoned and active)
Oyster reef
Oyster reef flank, sand or mud, abundant shell
Interreef mud with oyster shell
Serpulid reefs and related shell-rich sand and beach rock
Wind-tidal flat, sand, loose, rarely flooded
*Wind-tidal flat, sand and mud, extensive algal mats, alternately emergent-submergent
Wind-tidal flat, mud and sand, algal-bound mud, gypsiferous, firm
Wind-tidal flat, mud and sand, algal mats, depressed relief, wet and soft

Wind-tidal flat, sand and mud, barren to sparsely vegetated, subaerial
Marginal residual sand apron on windward side of rincons and potreros, wind-deflation lag deposit
Eolian accretionary bars and ridges, sand and clay, on wind-tidal flats
Transitional zone, wind-tidal flat to eolian sand sheet, wind deflation

Eolian System

Active dune complex, sand, commonly banner dunes with small barchans
Active dune blowout areas, sand, local depressed relief, eolian grain prominent
Sand sheet with strong relict grain of base-leveled dunes
Sand and loess (silt) sheet with no relict grain
Moderately stabilized dunes, sand and loess (silt) sheet, brush-covered
Well-stabilized dune sands, live oak-covered
Sand and loess (silt) sheet deflation area, active
*Clay-sand dunes, accretionary, active
*Clay-sand dune complexes, inactive (H/M)
*Loess sheet, thin, silty, overlies calichified Pleistocene fluvial sand (H/M)
*Loess sheet, thin, discontinuous, silty, overlies Pleistocene deltaic mud and calichified sand (H/M)

Other Map Units

Point bar (fluvial) accretion grain
Beach ridge (barrier-strandplain-chenier) accretion grain
Spoil heap or mound, subaerial
Reworked spoil, subaerial
Spoil, subaqueous
Made land
Barchan dune orientation in banner dune complex
Longitudinal dune orientation in back-island dune field
Beach ridges, accretionary, relict (barrier-strandplain)
Wind accretion ridges, rincons and potreros
Serpulid reefs

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* Unit may occur within more than one system
M Modern
P/M Pleistocene to Modern
H/M Holocene and Modern

PLEISTOCENE SYSTEMS

Fluvial-Deltaic Systems

Meanderbelt sand
Flood-plain mud
Flood-plain mud veneer over meanderbelt sand
Distributary sand and silt
Interdistributary mud
Interdistributary mud with sand veneer
Delta-front mud and sand, veneered by thin mud
Delta-front mud and sand
*Abandoned channel and course, mud-filled (P/M)
*Abandoned channel and course, swamp-covered (M)
*Abandoned channel and course, fresh-water, marsh-covered (P/M)
*Tidal creek, fresh- to brackish-water, marsh-covered (M)
*Tidal creek, unvegetated, mud-filled (M)
*Tidal creek, fresh-water, marsh-covered (M)
*Tidal creek, grass-covered, mud-filled (P/M)
*Marsh, fresh-water, and poorly drained depressions, mud and sand
*Upland oak motte
*Loess sheet, thin, overlies Pleistocene fluvial sand (M)
*Loess sheet, thin, discontinuous, overlies Pleistocene mud and sand (M)
Circular to irregular depressions on distributary/fluvial sand (P/M)
*Clay-sand dunes, active (M)
*Clay-sand dunes, inactive (H/M)
Lakes and ponds, coastal, mud and sandy mud-filled (P/M)
*Beach ridge and berm, margin of lakes, shell (P/M)
*Swale between beach ridges, margin of inland lakes, mud-filled (P/M)

Barrier-Strandplain Systems

Barrier-strandplain sand, tree-covered
Barrier-strandplain sand, grass-covered
Live oak-covered beach ridge, relict
Swale between beach ridges, unvegetated, mud-filled (P/M)
Swale between beach ridges, grass-covered, mud-filled (P/M)
Sheet sand, locally mud-veneered, back side of Pleistocene strandplain
Well-stabilized dune sand, dense live oak mottes (M)
*Marsh, fresh-water and poorly drained swales, mud- and sand-filled (M)

tidal flats, fluvial channels, wind erosion, and other dynamic properties of significance in maintaining and modifying the coastal environments; (3) biologic features such as reefs, marshes and swamps, subaqueous grassflats, and plant-stabilized sediment where biologic activity is of principal importance; and (4) man-made features such as spoil heaps, spoil wash, dredged channels, and made land where man's activities have resulted in significant environmental modification. Approximately 125 specific environmental geologic units are recognized and mapped in the Texas Coastal Zone.

Environmental geology map units are grouped into higher order natural systems (fig. 3). Systems such as fluvial-deltaic, barrier island, and bay-estuary-lagoon, for example, include a variety of natural substrate, biologic, or process units and environments that are genetically interrelated as to origin and distribution within the Coastal Zone. Man-made features are separately grouped to differentiate clearly natural and artificial features. The origin of the various natural units in the Coastal Zone determines their main features, composition, and character; hence, their origin is basic to consideration of resource evaluation and use. Natural systems delineated (table 1) include (1) fluvial-deltaic, a series of relict Pleistocene substrates (fig. 5) and Modern environments and substrates formed both by older and present-day

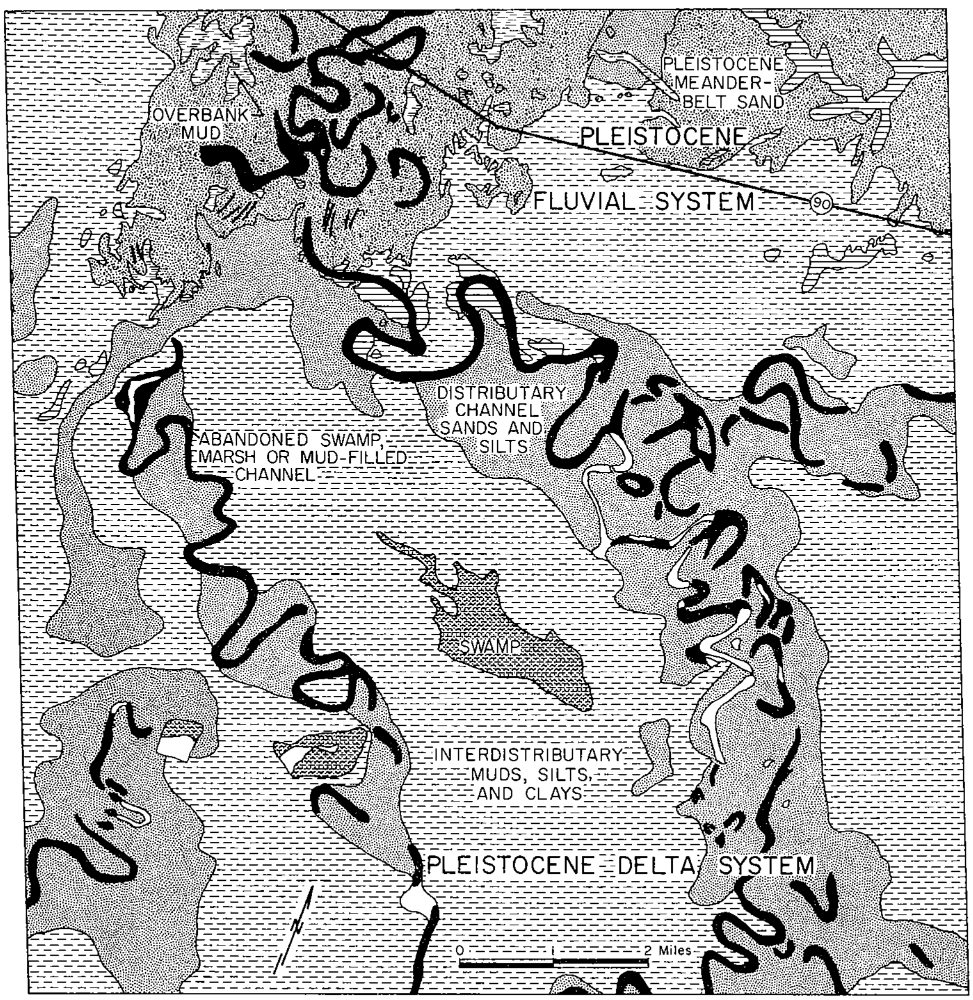

Figure 5. Pleistocene fluvial-deltaic facies, coastal uplands in vicinity of Devers, Beaumont-Port Arthur area. Meanderbelt sands appear to grade coastward into elongate belts of sand and silt that were deposited principally during delta building. After Fisher and others, 1973.

rivers and deltas (fig. 6); (2) barrier-strandplain-chenier, a suite of relict Pleistocene substrates (fig. 7) and Modern environments and substrates formed at the interface of the land and Gulf (figs. 8, 9); (3) marsh-swamp, including a variety of Modern, permanently wet, grassed and wooded lands of the low-lying coastal areas (figs. 6-10); (4) offshore, embracing various units of the Modern barrier island shoreface and inner continental shelf developed seaward of Gulf beaches (figs. 8, 9); (5) bay-estuary-lagoon, consisting of Modern subaqueous or submerged environments formed between the mainland and the barrier islands (fig. 11); (6) eolian, consisting of a variety of wind-dominated facies which occur primarily in the coastal bend region south of Corpus Christi (fig. 12); and (7) man-made features such as spoil and made land (fig. 10).

Environmental geology maps are presented at a scale of 1:125,000, or 2 miles per inch. Compilation work maps (1:24,000) are maintained on open file at the Bureau of Economic Geology. The currency of aerial photographs, topographic maps, and navigational charts used in the project can be determined by reference to index maps, which provide specific information on the date of photography and map or chart revision. Edgar Tobin Aerial Surveys photomosaics provided uniform coverage of the entire Coastal Zone.

Remapping in future decades with updated aerial photography and other multispectral remote sensing devices carried by aircraft and satellites will provide a valuable historical reference to rates and degree of both natural and man-made changes in the Coastal Zone. The Atlas is, therefore, an open-ended document which can be updated to maintain a current record of the change and modification of the region. It is also anticipated that the Atlas will serve to stimulate interest in and provide the environmental base line for many more specialized and localized studies addressed to specific pollution, land use, ecology, economic, and resource problems.

SPECIAL-USE ENVIRONMENTAL MAPS

Following preparation of the Environmental Geology Map for each of the seven areas of the Coastal Zone, a series of Special-Use Environmental Maps was prepared to present more specific information for a variety of potential users. The series of eight maps is designed for direct and specific use in the evaluation and proper utilization of the natural resources and environments of the area. They were constructed through (1) interpretation and derivation of units mapped for the Environmental Geology Map, (2) compilation of data from diverse sources projected onto the environmental base map, and (3) a combination of both derived and compiled data (fig. 2). Selection of the kinds of special-use environmental maps was based on a survey of the greatest need and potential use by both professional and lay people concerned with proper resource use and environmental management (table 2).

The series is composed of the following maps: (1) Physical Properties; (2) Environments and Biologic Assemblages; (3) Current Land Use; (4) Mineral and Energy Resources; (5) Active Processes; (6) Man-Made Features and Water Systems; (7) Rainfall, Stream Discharge, and Surface Salinity; and (8) Topography and Bathymetry. They comprise only a basic series of maps; a variety of other specific-use maps may be prepared by overlaying or combining any of the environmental map units (table 1). For example, the pipeline network of an area can be compared directly with the distribution of active and potentially active surface faults to identify those areas where faulting might result in damage to or rupture of the pipeline. Likewise, current land use can be compared to areas of hurricane flooding to determine kinds and amounts of land use affected. Map units can be grouped according to physical properties (Physical Properties Map) and then evaluated in terms of various land uses (table 3). Statistical analyses of all units and features included on the Environmental Geology Map and the various Special-Use Environmental Maps are summarized in tables.

Physical Properties Map

The special-use map delineating physical properties is designed to provide regional data for a variety of physical uses applicable both to land surface and to approximately 60 feet below the surface. Specific types of uses and activities that can be evaluated from these data are primarily those involving construction, excavation, drilling, channelization, and waste disposal. Table 3 includes an evaluation of the degree of suitability of each physical properties group for potential engineering uses.

The many geologic, biologic, active process, and man-made map units of the basic Environmental Geology Map are graded and organized into major

Table 2. Special-use environmental map units, Texas Coastal Zone.

PHYSICAL PROPERTIES MAP

Group I. Dominantly clay and mud
Group II. Dominantly sand
Group III. Dominantly clayey sand and silt
Group IV. Coastal and estuarine marshes, fresh to brackish
Group V. Inland swamp and fresh-water marsh
Group VI. Tidal flat and salt marsh
Group VII. Made land and spoil
Group VIII. Transitional wind-tidal flat and eolian sand sheet
Group IX. Clay-sand dunes and dune complexes
Group X. Eolian sand sheet
Group XI. Active sand dunes and dune complexes
Group XII. Loess sheet, silt and fine sand
Pit or quarry
Sludge pit or miscellaneous waste-disposal pit
Sewage-disposal site
Solid waste-disposal site
Salt dome, shallow piercement
Active or potentially active fault

CURRENT LAND USE MAP

Agriculture, cultivated land and orchards
Range-pasture
Woodland-timber
Swamp-timber
Live oak mottes
Fresh-water marsh
Saline- and brackish-water marsh
Residential-urban
Industrial, refineries, rail yards
Park and recreational facility, formally defined public facility
General recreational land, public beaches
Wildlife refuge, formally defined, government operated
Government land, federal and state, excluding recreational
Transitional area between wind-tidal flat and eolian sand sheet
Undifferentiated urban land, greenbelts
Made land
Spoil
Barren land
Oil and gas field
Education site
Pit or quarry
Solid waste-disposal site
Pipeline
Offshore petroleum production platform
Airfield
Salt dome
Sulfur production site
Brine production site
Liquid petroleum gas storage
Sludge pit
Sewage-disposal site
Artificial reservoir

ACTIVE PROCESSES MAP

Lower shoreface and shelf
Normal surf and breaker zone
Site of active or potentially active hurricane washover
Shoreline, erosional
Shoreline, depositional
Shoreline in erosional-depositional equilibrium
Shoreline, artificially stabilized
Area of slow to moderate bay deposition
Area of rapid deposition within bay
Area of active reworking and redistribution of spoil
Area of shoaling, moderate to high wave energy
Area of moderate erosion to slight deposition, tidal channel
Area of wind-tidal flooding
Area of intensive wind deflation
Eolian sand dunes, active
Oyster reef deposition
Inland lake, area of wave erosion and beach-ridge accretion
Area inundated by marine water, Hurricane Carla
Area inundated by marine water, Hurricane Beulah
Area inundated by river flooding and rainfall runoff, Hurricane Beulah
Hurricane Carla recording tide or river gage
Hurricane Carla recording site, still high water mark
Hurricane Carla storm surge and river flooding, debris or driftline
Hurricane Beulah recording tide or river gage
Hurricane Beulah storm surge and river flooding, debris or driftline
Hurricane Beulah recording site, still high water mark

MINERAL AND ENERGY RESOURCES MAP

Sand, includes all subaerial sandy deposits
Mud, includes all subaerial muddy deposits
Oyster reef
Oyster shell, dredged from bottom of bay
Serpulid worm reefs
Pit or quarry
Oil or gas field
Salt dome
Sulfur production site
Liquid petroleum gas storage site
Brine production site
Salt dome oil field
Cement plant
Lime plant
Aluminum plant
Petrochemical plant
Power generation plant
Utility line or cable
Pipeline
Offshore petroleum production platform

ENVIRONMENTS AND BIOLOGIC ASSEMBLAGES MAP

Subaqueous Environments and Assemblages

Shelf, open marine
Lower shoreface
Upper shoreface, surf zone
Shoreface adjacent to tidal delta
Inlet and tidal delta
Bay margin, shoal water
Bay and lagoon margin, seasonally hypersaline
Grassflats, shallow bay margin, dense grass
Grassflats, hypersaline, sparse to moderate grass
Subaqueous sandflats
Open bay, lower end with tidal influence
Open bay with reefs
Enclosed bay with reef
Enclosed bay away from tidal or river influence
Enclosed hypersaline bay or lagoon center
Restricted hypersaline bay and lagoon margin
Restricted bay center, hypersaline
River influenced bay, low salinity
Sand and oolite shoal
Reef, dense oysters
Reef flank and margin, oysters
Serpulid reef (relict) and interreef shoals
Subaqueous spoil
Fresh- to saline-water bodies

Subaerial Environments and Assemblages

Beach, swash zone
Vegetated barrier flat, foredune ridge, blowouts
Unvegetated coastal mudflats
Vegetated strandplain flat, foredune ridge, and beach ridge
Washover channel, fan, and wind-deflation trough and storm runnels
Active dunes, coppice dunes, blowouts
Sandflats, wind-tidal
Eolian ridges and active clay-sand dunes
Salt-water marsh
Brackish-water marsh
Brackish- to fresh-water marsh
Swamp
Inland fresh-water marsh
Frequently flooded fluvial areas
Berms along bay-lagoon margin
Intense wind-deflation and wind-tidal activity
Prairie grasslands
Poorly drained depressions, mud substrate
Poorly drained depressions, sand substrate
Loose sand and loess prairies
Live oak mottes and groves
Brushland
Made land
Grass and locally scrub oak-covered ridges
Mixed pine and hardwood forest
Small prairies in forested uplands
Fluvial grassland
Barren land

RAINFALL, STREAM DISCHARGE AND SURFACE SALINITY MAP

Calculated average surface salinity
Extreme low surface salinity
Extreme high surface salinity
Salinity measurement station, data for graphs
Salinity measurement station, data for contouring
Rainfall recording station
Discharge measurement station

MAN-MADE FEATURES AND WATER SYSTEMS

Man-Made Features

Urban and residential areas
Industrial areas
Made land
Subaerial spoil
Subaqueous spoil
Jetty or pier
Pipeline
Offshore production platform
Airfield
Solid waste-disposal site
Sewage-disposal site
Seawall
Sludge pit
Undifferentiated urban land

Water Systems

Open ocean
Tidal inlet and pass
Lagoon, bay and estuary
Transportation canal and channel
Tidal-affected stream
River or stream
Slough or abandoned course and cutoff
Lake or pond (perennial)
Lake, pond, or playa (ephemeral)
Drainage or irrigation ditch and canal
Artificial reservoir
Wind-tidal flats

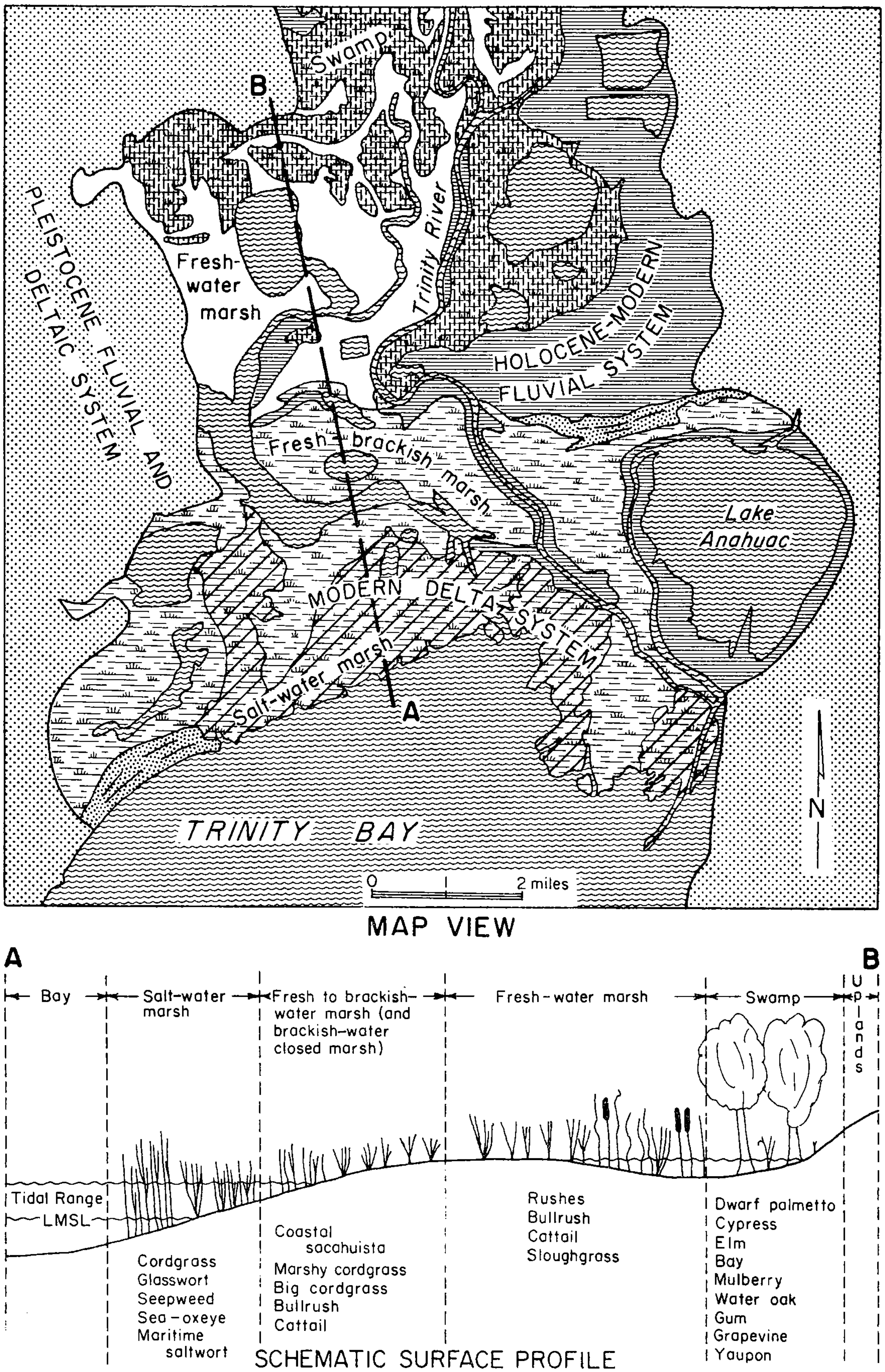

Figure 6. Modern marsh system overlying Holocene and Modern fluvial and deltaic facies in the lower Trinity River valley, Galveston-Houston area. Schematic profile from Trinity Bay northward across various marsh and swamp environments of the Wallisville area. After Fisher and others, 1972.

groups (table 2); each group is composed of units having common physical features and properties. Principal physical properties groups and land areas outlined on the Physical Properties Map include (1) areas of dominantly clay and mud soils and substrates, (2) areas of dominantly sand soils and substrates, (3) areas with soils and substrates consisting primarily of clayey sands and silts, (4) fresh- to brackish-water coastal marshes, (5) inland fresh-water marshes and wooded swamps, (6) tidal flats and salt marshes with frequent tidal inundation, and (7) made land and spoil. Additional physical properties groups occur in the eolian-dominated areas of South Texas (Kingsville, Brownsville-Harlingen, and Corpus Christi areas).

All these units have been derived from basic map units on the Environmental Geology Map by applying quantitative test data to the areally defined and mapped environmental geologic units (fig. 2). Land units are characterized in a qualitative manner only; available test data are too limited and too local in distribution to ascribe precise quantitative parameters to the various units throughout the entire area. Data presented on the Physical Properties Map of the Atlas should not be substituted for specific site testing and evaluation, but they can be used to grade large tracts of land for a particular suitability.

In addition to the major physical land types shown, principal zones of active or potentially active surface faults are defined. Current waste-disposal sites, pits and quarries, and sludge pits are also plotted.

Environments and Biologic Assemblages Map

The Environments and Biologic Assemblages Map depicts the distribution of major biologic

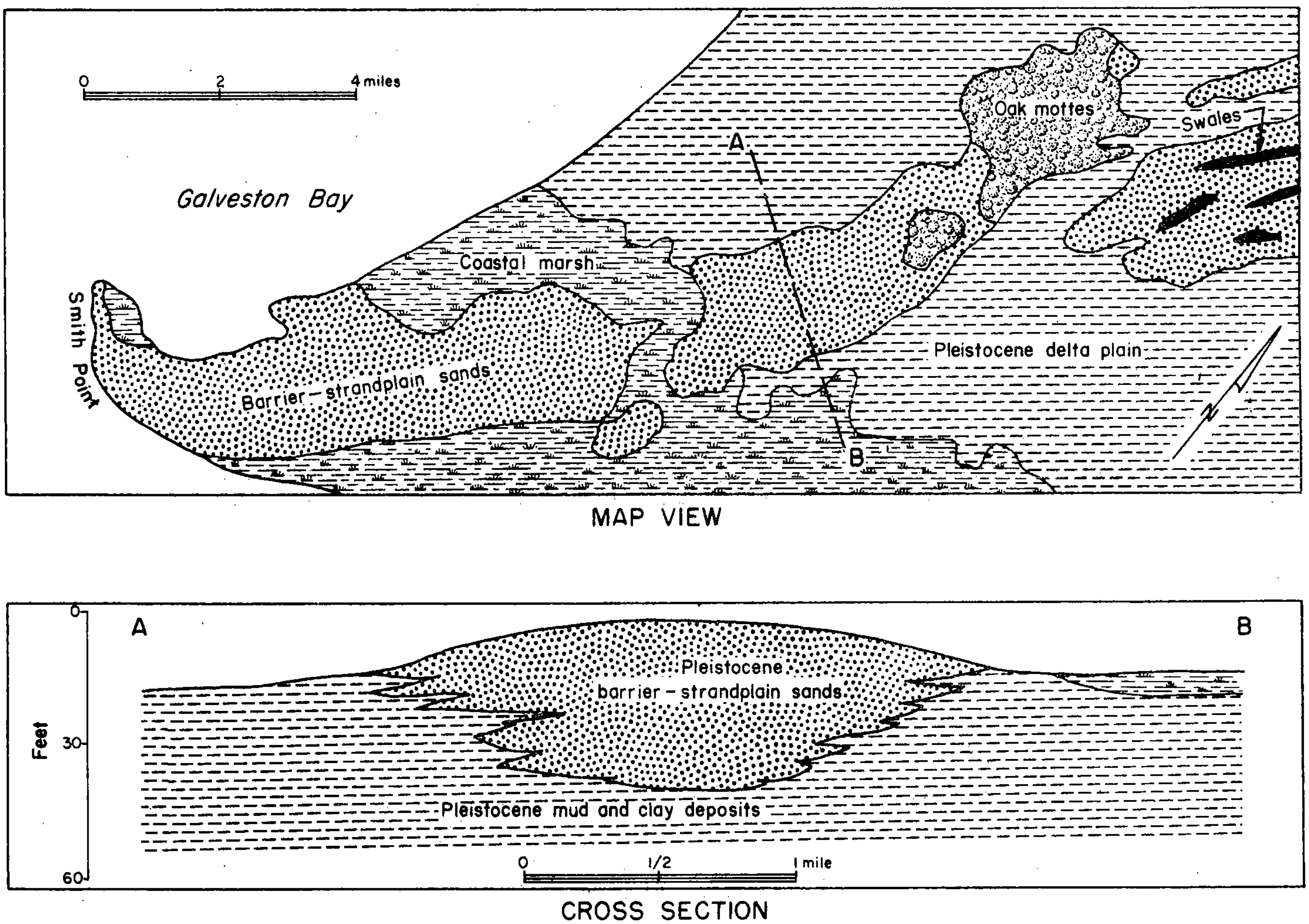

Figure 7. Pleistocene barrier-strandplain sands, Smith Point area, Chambers County, Texas. After Fisher and others, 1973.

communities and the environments they inhabit. These include (1) subaqueous environments and assemblages of the bays, estuaries, tidal passes, shoreface, and open shelf, defined primarily by assemblages of fixed or mobile benthonic (bottom-dwelling) organisms, which are chiefly faunal; and (2) subaerial environments and assemblages, defined primarily by land vegetation. A number of the biologic assemblages are of first-order environmental significance and, accordingly, appear as specific map units on the basic Environmental Geology Map. These include such units as reefs, the various wetland environments, and most of the Modern grass-covered barrier-island and associated units. Other natural environments have been derived from the basic Environmental Geology Map by utilizing previously known and compiled information on animal and plant distribution in the Texas Coastal Zone (fig. 2). Several environmental geologic units are occupied by single biologic assemblages. For example, the Pleistocene delta distributary channel sands and interdistributary muds originally supported extensive coastal prairie grasslands, but much of this assemblage and natural

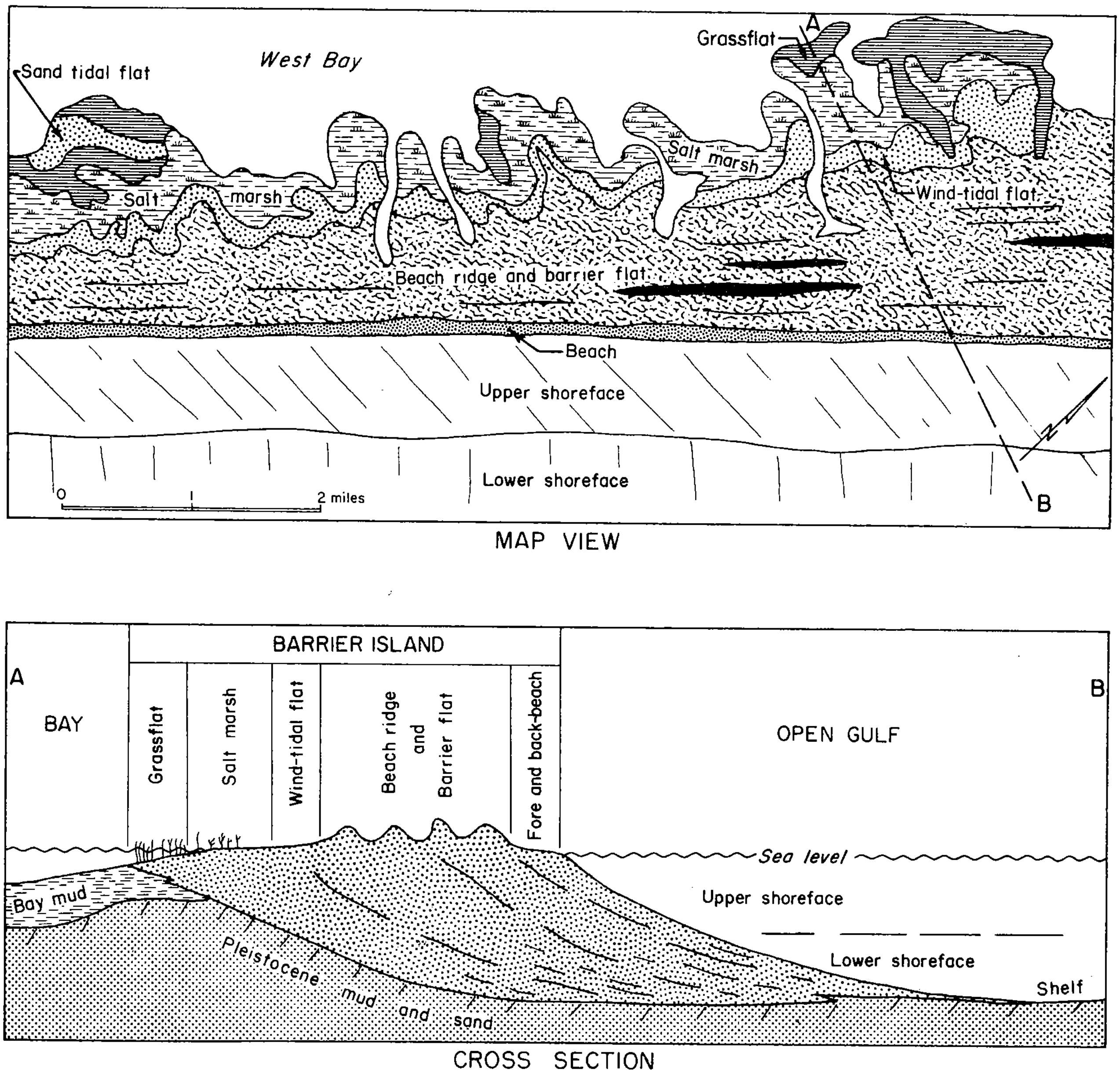

Figure 8. Modern barrier-bar environments and facies, Galveston Island. Cross section after Bernard and others, 1970. After Fisher and other, 1972.

Table 3. Evaluation of the natural suitability of physical properties groups for various coastal activities and land uses, Galveston-Houston area, Texas. After Fisher and others, 1972.

Suitability is evaluated on the basis of natural properties and may be improved by special engineering and construction methods. Significant properties considered as positive criteria for evaluating land-use suitability (+ = satisfactory; − = unsatisfactory; o = possible problems):

(1) Road construction: earthen structures and fill material—low shrink-swell potential, low compressibility, and low plasticity.
(2) Road construction: base material—low compressibility, low shrink-swell potential, and high shear strength.
(3) Road construction: grade material—low compressibility, low shrink-swell potential, and high shear strength.
(4) Fill material: topsoil—loam or sandy/silty clay composition.
(5) Fill material: general, below topsoil—silty/sandy clay composition with low to moderate shrink-swell potential.
(6) Foundation: heavy—high load-bearing strength, low shrink-swell potential, and good drainage.
(7) Foundation: light—low shrink-swell potential.
(8) Underground installations—low shrink-swell potential, high load-bearing strength, and good drainage.
(9) Buried cables and pipes—low shrink-swell potential and low corrosivity.
(10) Excavatability—ease of digging with conventional machinery.
(11) Waste disposal: septic systems—moderate permeability, low to moderate shrink-swell potential, and good subsurface drainage.
(12) Waste disposal: solid waste—low permeability and good surface drainage.
(13) Waste disposal: unlined liquid waste retention ponds—low permeability.
(14) Earthen dams and dikes: low permeability, moderate shear strength, and moderate compressibility.
(15) Water storage: unlined reservoirs or ponds above ground-water level—low permeability.
(16) Water storage: reservoirs or ponds below ground-water level—high permeability.

GENERAL PHYSICAL PROPERTIES	PRINCIPAL ENVIRONMENTAL GEOLOGIC MAP UNIT	LAND USE: ROAD CONSTRUCTION (1) Earthen structures and fill material	ROAD CONSTRUCTION (2) Base material	ROAD CONSTRUCTION (3) Grade material	FILL MATERIAL (4) Topsoil	FILL MATERIAL (5) General—below topsoil	FOUNDATIONS (6) Heavy	FOUNDATIONS (7) Light	(8) Underground installations	(9) Buried cables and pipes	(10) Excavatability	WASTE DISPOSAL (11) Septic systems	WASTE DISPOSAL (12) Solid waste	WASTE DISPOSAL (13) Unlined liquid waste retention ponds	WATER STORAGE (14) Earthen dams and dikes	WATER STORAGE (15) Unlined reservoirs or ponds above ground-water level	WATER STORAGE (16) Reservoirs or ponds supplied by ground water
Group I Dominantly clay and mud, low permeability, high water-holding capacity, high compressibility, high to very high shrink-swell potential, poor drainage, level to depressed relief, low shear strength, high plasticity, high to very high acidity, high corrosivity	Interdistributary muds, barrier-strandplain-chenier swales, abandoned channel-fill muds, overbank fluvial muds, mud-filled coastal lakes and tidal creeks	−	−	−	o	−	−	o	−	−	+	−	+	+	o	+	−
Group II Dominantly sand, high to very high permeability, low water-holding capacity, low compressibility, low shrink-swell potential, good drainage, low ridge and depressed relief, high shear strength, low plasticity	Beach, foredunes, barrier-strandplain-chenier vegetated flats, Pleistocene barrier and strandplain sands	+	+	+	o	+	+	+	+	+	+	o	−	−	−	−	+
Group III Dominantly clayey sand and silt, low permeability, moderate drainage, moderate water-holding capacity, low to moderate compressibility and shrink-swell potential, level relief with local mounds and ridges, high shear strength	Meanderbelt sands, alluvium, levee, crevasse splay, distributary sands, bay-margin sand and mud, Pleistocene fluvial, distributary, delta-front sands	+	+	+	+	+	o	+	o	+	+	+	o	o	+	o	o
Group IV Coastal marsh, fresh to brackish, very low permeability, high water-holding capacity, very poor drainage, depressed relief, low shear strength, high plasticity, high organic content, subject to salt-water flooding, high to very high corrosivity	Fresh to brackish and closed brackish marsh, marsh-filled abandoned coastal lakes and tidal creeks	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−	+
Group V Inland swamp and marsh, permanently high water table, very low permeability, high water-holding capacity, very poor drainage, very poor load-bearing strength, high organic content, subject to frequent flooding, very high acidity	Swamp, inland marsh, marsh-filled channels	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−	+
Group VI Tidal flat and salt marsh, permanently high water table, very low permeability, high water-holding capacity, very poor drainage, very poor load-bearing strength, very high corrosivity, subject to frequent tidal inundations	Tidal flat and salt marsh	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−
Group VII Made land and spoil, properties highly variable, mixed mud, silt, and sand, reworked spoil commonly sandy and moderately sorted with properties similar to those of Group III	Subaerial spoil heaps or mounds, subaerial reworked spoil, subaqueous spoil, made land	HIGHLY VARIABLE: USE WITH CAUTION															

environment has been modified and converted into agricultural lands (compare with Current Land Use Map).

The Environments and Biologic Assemblages Map is not meant to be a biologic assay of the area but rather to show areal distribution of the type and number of major environments that are defined by dominant biologic assemblages. In short, it outlines the natural condition of the Coastal Zone. Comparison with current land use readily shows the extent of man's modification of the natural biologic environment.

Current Land Use Map

A number of factors in the Texas Coastal Zone contribute to diversified and extensive land and water use. The Zone is endowed with extensive mineral resources, notably oil, gas, and chemical raw materials (sulfur, salt, and lime), which

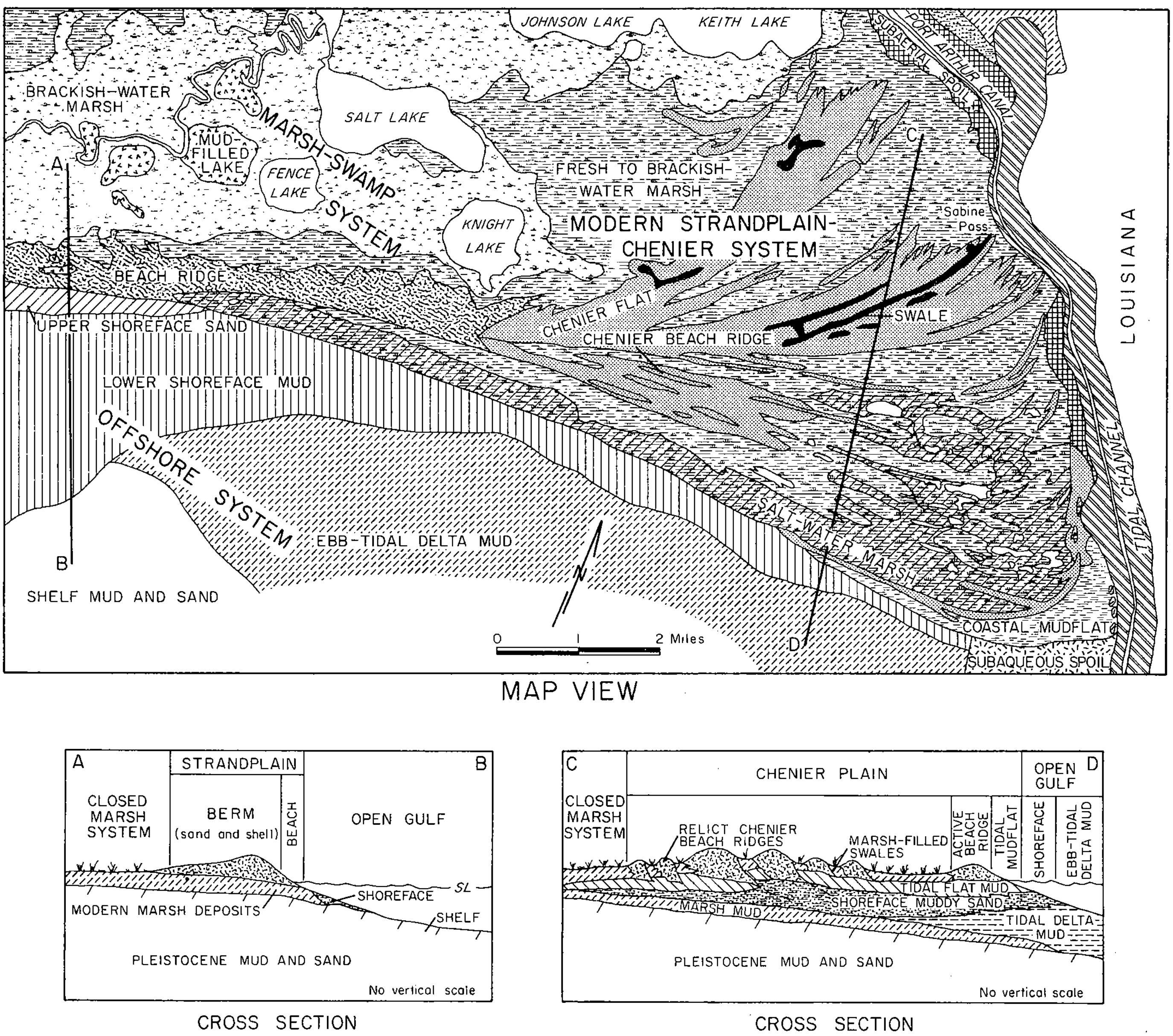

Figure 9. Modern strandplain-chenier and offshore systems along Gulf of Mexico, Beaumont-Port Arthur area. Chenier beach ridges were deposited along muddy coastline near Sabine Pass; sandy strandplain deposits occur southwestward of cheniers where muds are less abundant. Generalized cross sections contrast strandplains and chenier plains. After Fisher and others, 1973.

support one of the major petroleum-refining and petrochemical centers of the world. It is an area with fertile and productive lands that support extensive agriculture. And, finally, it embraces major port facilities with extensive intracoastal waterways and ship channels that have led to a high-volume flow of imports and exports.

Many of the factors that have led to diverse land and water use in the Coastal Zone have also led to current and potential limitations and conflicts. Many of the resources of the area have varied uses, both present and potential. For example, water bodies are used simultaneously for transportation, commercial and sport fishing, recreation, oil and gas well locations, pipeline routes, as a source of fill for real estate developments, and as a part of a waste-disposal system. Certain of these uses are obviously in conflict. The natural area is one of rapid and dramatic physical change involving active shoreline processes, hurricane flooding and damage, subsidence, and surface faulting; these dynamic changes interface with a variety of land and water uses. Further, the area embraces a fundamental legal boundary with the shore zone largely privately owned and the bay, estuary, and offshore areas publicly owned. Because the legal boundary is also a high-energy geological boundary, actions taken by one proprietor have an immediate and significant effect on others.

Current land use is classed in major use categories on the Current Land Use Map of the Atlas. Most of the information utilized in compiling the map was taken or derived from 7.5-minute U. S. Geological Survey topographic maps and similar Edgar Tobin controlled photomosaics (fig. 2); supplementary data were obtained

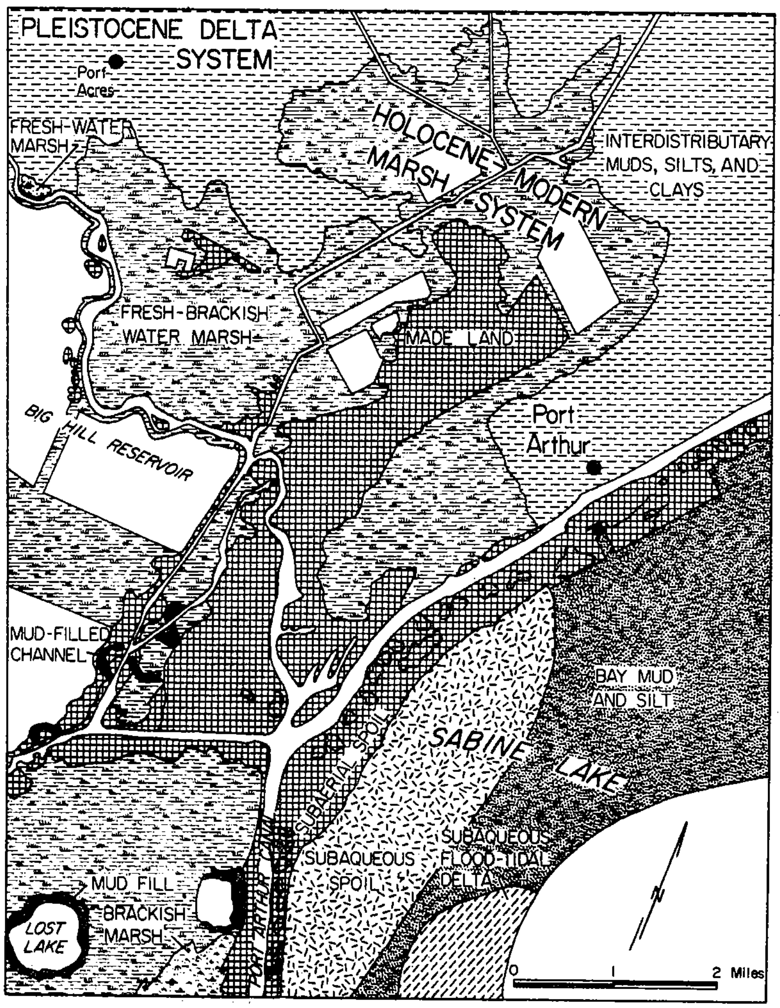

Figure 10. Extensive areas of made land cover former marshlands in the Port Arthur area. Subaerial and subaqueous dredge spoil has been dumped into the western end of Sabine Lake. After Fisher and others, 1973.

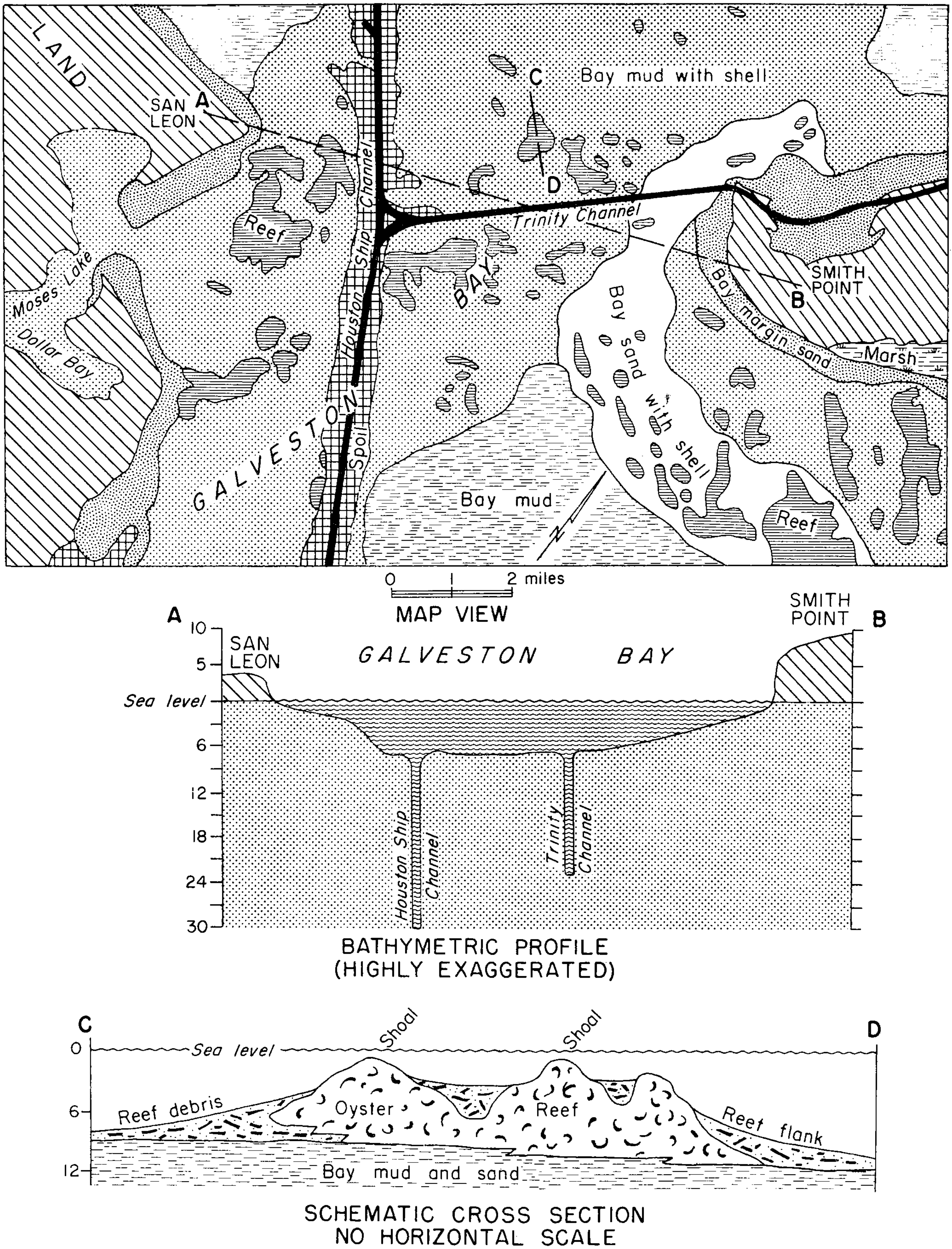

Figure 11. Modern bay-estuary facies within Galveston Bay. Bathymetric profile illustrates man's impact on the bay floor; the nature of living oyster reefs is shown by schematic cross section. After Fisher and others, 1972.

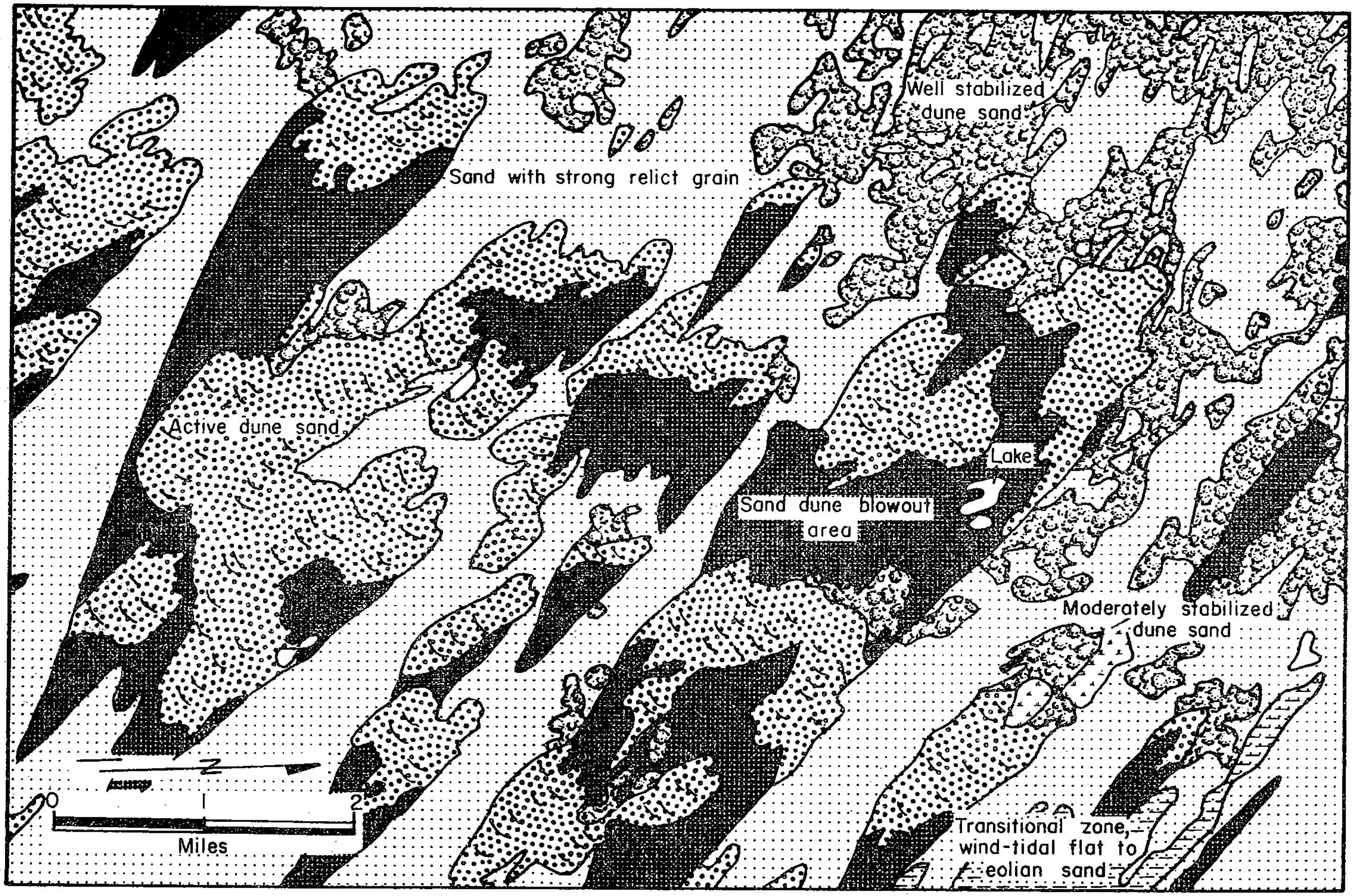

Figure 12. Banner dune complex, eolian system, Kingsville area. Banner dunes are composed of small barchan dunes.

by field observation and by derivation from the Environmental Geology Map. Base materials available for the entire area are generally on the order of a decade old. Where more recent base materials on a detailed scale existed, they were used to bring land use as up-to-date as possible; updating should be completed at least every decade or whenever new coastwide aerial photography becomes available.

Major classes of current or, in some cases, potential land use include (1) agricultural lands, (2) timber or wooded lands, (3) marshes or grassed wetlands, (4) urban lands, (5) government lands (State and Federal), (6) formally designated wildlife refuges, (7) general recreational lands, (8) made and reclaimed lands, (9) dredged spoil lands, and (10) artificial surface reservoirs. The major classes—agricultural, timber, marsh, and urban lands—are divided into smaller land use units. The Current Land Use Map shows location and distribution of oil and gas fields, educational sites, shallow piercement salt domes, sulfur mines, salt or brine plants, LPG storage sites, pits and quarries, sludge pits, sewage-treatment and disposal sites, solid waste - disposal sites, offshore petroleum production platforms, and airfields. Major pipeline, transportation-navigation, and irrigation-drainage networks are indicated.

Mineral and Energy Resources Map

Most of the Texas Coastal Zone is richly endowed with mineral and energy resources. Chief among these resources are oil and natural gas, which serve not only for fuel but also provide raw materials for many petrochemical processes. In addition, the Zone contains important resources of chemical raw materials—sulfur and salt, and shell for lime. The abundance of these chemical and petroleum raw materials and their occurrence in a zone with ocean access make this area one of the major petrochemical and petroleum-refining centers of the world.

The Mineral and Energy Resources Map of the Atlas shows the occurrence and distribution of all known mineral deposits, including oil and gas fields, salt domes, sulfur deposits, clay deposits, and general fill and aggregate materials. Also shown are existing pits and quarries, cement plants, brine production sites, and sulfur production sites. All shallow-piercement salt domes and moderate to deep-seated domes that have been proved by drilling, as well as those used currently for the underground storage of LPG, are indicated. The energy-distribution network is outlined by all major pipeline transmission facilities, major power or utility transmission lines, and power generation stations.

Active Processes Map

The Active Processes Map of the Atlas outlines the major physical and biological processes of the Coastal Zone that are critical for a variety of land and water uses. The main features of the map are a delineation of areas inundated by hurricane-surge floods and characterization of the bay and Gulf shorelines in their present state—erosional, depositional, or stabilized. In addition, such features as depositional rates within the bays, subaqueous areas of high energy, areas of extensive reef development, and areas of spoil reworking are indicated.

Man-Made Features and Water Systems Map

The Man-Made Features and Water Systems Map of the Atlas combines on one sheet the various features of the Coastal Zone that are the products of man's construction activities and the various kinds of surface water systems, including both natural and artificial water bodies. Presentation on a single map is for cartographic convenience.

Rainfall, Stream Discharge, and Surface Salinity Map

The Rainfall, Stream Discharge, and Surface Salinity Map of the Atlas summarizes certain salient climatic features. Data were selected for the three-year period, 1965-1967, for which detailed and continuous coverage exists.

Topography and Bathymetry Map

The Topography and Bathymetry Map included in the Atlas is a basic tool in the evaluation of land and water use and capability. Topography is indicated on the map with a distinct but graduated color pattern for each 5-foot interval of ground elevation. Elevations range from zero or sea level to approximately 100 feet in the most inland portions of the Coastal Zone. Topographic control used for this map (scale 1:250,000) and on the Environmental Geology Map, (scale 1:125,000) was compiled from U. S. Geological Survey detailed 7.5-minute topographic maps at the scale of 1:24,000.

Table 4. Coastal Zone land and water resource units—use and capability. Evaluations are based on natural capability which can be improved by engineering. Blank areas represent either no environmental problems or the use is not applicable on the particular resource capability unit. After Brown and others, 1972.

RESOURCE CAPABILITY UNITS / ACTIVITIES			Liquid Waste Disposal			Solid Waste Disposal	Construction of Offshore and Bay Platforms	Coastal Construction			Inland Construction				Dredging of Canals and Channels, and Spoil Disposal ★	Excavation (includes extraction of natural materials)	Filling for Development	Draining of Wetlands	Well Development	Devegetation	Transversing with Vehicles (marsh buggies, air boats, dune buggies, motorcycles)	Use of Herbicides, Pesticides, Insecticides
			Surface Disposal of Untreated Liquid Wastes	Disposal of Untreated Liquid Wastes, Subsurface, Shallow	Maintenance of Feed Lots	Disposal of Solid Waste Materials		Construction of Jetties, Groins, Piers	Construction of Storm Barriers and/or Seawalls	Placement of Pipelines and/or Subsurface Cables ▲	Light Construction	Construction of Highways	Heavy Construction	Flooding (through dam construction)								
WATER CAPABILITY UNITS	Bays, Estuaries, and Lagoons	River Influenced Bay Areas Including Prodelta and Delta Front	X	X		X	O	O	X	O					X		X		O			
		Enclosed Bay Areas	X	X		X	O		X	O					O		O		O			
		Living Oyster Reefs and Related Areas	X	X		X	X	X	X	X					X		X		X			
		Dead Oyster and Serpulid Reefs and Related Areas	X	X		X	O	O	X	O					X		X		O			
		Grassflats	X	X		X	X	X	X	X					X		X		X			
		Mobile Bay-Margin Sand Areas	X	X		X	X	X	X	X					X		X		O			
		Tidally Influenced Open Bay Areas	X	X		X	O	O	X	O					X		X		O			
		Subaqueous Spoil Areas	X	X		X	O	O		O							O		O			
		Inlet and Tidal Delta Areas	X	X		X	X	X	X	X					X		X		O			
		Tidal Flats	X	X		X			X	X	X	X	X		O	X	X		O			
LAND CAPABILITY UNITS	Coastal Wetlands	Salt-Water Marsh	X			X	X		X	X	X	X	X	X	X	X	X	X	O	X	X	X
		Fresh-Water Marsh	X			X			X	O	X	X	X	X	X	X	X	X	O	X	X	X
		Swamps	X			X				O	X	X	X	X	O	O	X	X	O	X	O	X
	Coastal Barriers	Beach and Shoreface	X	X		X		X	O		X	X	X		X	X			X		O	
		Fore-Island Dunes and Vegetated Barrier Flats	X	X	X	X			X		+	+	+		X	X			O	X	X	X
		Washover Areas	X	X	X	X		X	X	X	X	X	X		X		X		X			
		Blowouts and Back-Island Dune Fields	X	X		X				O	X	X	X		X				X			
		Wind Tidal Flats	X	X		X					X	X	X		O		X		O			
		Swales	X	X		X					X	X	X		X	X	X	X		X		X
	Man-Made	Made Land and Spoil	X	X	X	X							O							X		
	Coastal Plains	Highly Permeable Sands	X	X	X	X									O	X			O	X		X
		Moderately Permeable Sands	X	X	X	X									O	X			O	X		X
		Impermeable Muds	O							O	O	O	O						O			O
		Broad Shallow Depressions •	O								X	O	X						O			O
		Highly Forested Upland Areas •												X					O	X		X
		Steep Lands, Locally High Relief	X			X					O		O			X				X		O
		Stabilized Dunes	X	X	X	X						O			X	X			O	X	X	X
		Unstabilized, Unvegetated Dunes	X	X		X				O	X	X	X		X				X			
		Fresh-Water Lakes, Ponds, Sloughs, Playas	X			X									X		X	X	O			X
		Mainland Beaches	X	X		X		X	X		X	X	X		X	X	X		X	O		
		Areas of Active Faulting and Subsidence	O	O		O				X	O	O	X						O			
	Major Floodplain Systems	Point-Bar Sands	X	X	X	X					O				X				O	X		X
		Overbank Muds and Silts	X	X	X	X				O	O	O	O	O								O
		Water	X			X								O	X							X

X Undesirable (will require special planning and engineering)
O Possible problem(s)
\+ Barrier Flat only (no construction on dunes)
• Substrate variable
▲ Also occurs in Offshore Construction
★ Also occurs in Offshore Canals and Dredging

Bathymetric contours are shown at intervals of six feet, or one fathom, and are also represented by distinct gradational color patterns for ready determination of bottom relief and configuration. These contours are shown also on the Environmental Geology Map and were compiled from 7.5-minute topographic sheets and U. S. Coast and Geodetic Survey nautical charts. Depths range from zero or mean sea level to more than 30 feet. Deepest areas are within the active tidal channels, dredged channels, and the inner shelf. Depth of the navigation channels varies according to project depths and other specifications.

RESOURCE CAPABILITY: UTILITY IN LAND AND WATER MANAGEMENT

A basic goal of the Environmental Geologic Atlas of the Texas Coastal Zone is an adequate inventory of the natural resources of the Zone. Flexible management of the Texas Coastal Zone should be based on the natural capability of resource and environment units. Such units, termed *natural resource capability units* (Brown and others, 1971), have been derived from the maps included in the Atlas (table 4).

Particularly important to the maintenance of environmental quality are those properties and characteristics of natural resource units that limit their use for specific purposes or activities. Examples are: (1) potential for flooding by hurricane surges or by overbanking rivers; (2) unfavorable shrink-swell conditions; (3) tendency for corrosion of pipes and conduits placed in certain substrates; (4) degree of permeability, which determines the extent of transmission of pollutants into ground-water aquifers and nearby surface water bodies; (5) steep slopes, which are susceptible to gravity failure and extreme erosion from runoff; (6) extremely flat lands that are poorly drained and that pond water following heavy or prolonged rainfall; (7) impermeability, which exaggerates ponding and drainage problems; (8) persistent winds in arid areas, which cause wind erosion and migration of sediments in the form of dunes; (9) tidal flooding of broad, low-lying coastal flats by wind-driven water from bays, estuaries, and lagoons; (10) density of stabilizing vegetation on sand substrates, which maintains stability of sediments in high-energy wind and water environments; (11) wave energy dissipated along shorelines with resulting erosion and redistribution of sediments; (12) zones of active or potentially active faulting; (13) tendency for subsidence; and (14) erosional susceptibility of various sediments and soils to wind and water.

Evaluation of natural resource capability units depends upon the types of human activities that result in use of the units—currently or potentially. A wide variety of land and water use activities occur within the Coastal Zone (table 4); other potential activities will develop as population and urban-industrial expansion continues in the Zone.

Natural resource capability units display different capabilities and tolerances under the impact of human activities. For example, a highly permeable sand is a very poor host for a solid waste - disposal site simply because of its capability for transmitting wastes into aquifer systems, but the same permeable sand provides an excellent foundation for coastal structures. In turn, a relatively impermeable clay unit provides a secure host for solid waste disposal without aquifer pollution, but it is a very unsatisfactory foundation material. A brackish-water marsh not only can tolerate but is in fact defined on its capacity to accommodate changes in salinity; salt-water marshes, by contrast, can tolerate little fresh-water influx. A washover channel on a barrier island is a natural outlet for hurricane surges; it is an exceedingly poor site for construction. Many land and water resource units and their capabilities for particular uses are obvious; others are more subtle. A resource capability unit, therefore, must be evaluated in terms of each coastal activity; that is, environmentally significant physical properties may indicate that the capability unit will be severely affected by one activity, while another activity may prove to be entirely compatible with these properties.

By these examples it can be seen that in order to evaluate the impact of a specific coastal activity on a natural resource unit, it is necessary to evaluate the unit in terms of its limiting environmental capability properties. In this manner an activity can be evaluated in terms of the environmental stress it exerts on the resource unit; if the limiting environmental capability properties are compatible with the activities, no environmental impact will occur. On the other hand, if the activity adversely affects the resource unit because of the incompatibility of the activity and the limiting environmental capability properties, problems can be predicted and avoided or properly engineered.

Derivation of land and water capability maps from environmental geology maps provides an inventory of natural units that charts the distribu-

COASTAL WATER BODY AND LAND CLASSIFICATION

I. Bays, lagoons, and estuaries

A. River-influenced bay
B. Enclosed bay
C. Reef and reef-related areas (absent)
D. Grassflats (absent)
E. Mobile bay-margin sands (absent)
F. Tidally influenced open bay
G. Subaqueous spoil
H. Tidal inlet and tidal delta
I. Wind-tidal flats (absent)

II. Major river systems

A. Point-bar sands
B. Overbank muds and silts (not illustrated)
C. Water (including related lakes and sloughs)

III. Coastal wetlands

Salt marsh, fresh-water marsh, swamps

IV. Coastal plains

A. Highly permeable sands
B. Moderately permeable sands
C. Impermeable muds
D. Broad, shallow depressions (not illustrated)
E. Highly forested upland areas
F. Steep lands
G. Stabilized (vegetated) dunes and sand flats (absent)
H. Unstabilized (unvegetated) dunes (absent)
I. Fresh-water lakes, ponds, sloughs, playas
J. Mainland beaches (absent)
K. Areas of active faulting and subsidence (not illustrated)
L. Headward-eroding streams

V. Made land and spoil

VI. Coastal strandplain

A. Beach and shoreface
B. Vegetated strandplain and chenier flat
C. Washover areas
D. Active dunes (absent)
E. Tidal flats (absent)
F. Swales

Figure 13. Schematic map of land and water resource capability units, Beaumont-Port Arthur area.

tion of kinds and grades of natural resources. A schematic map of the Beaumont-Port Arthur area (fig. 13) illustrates the nature and distribution of land and water capability units; detailed, cartographically accurate maps can be constructed (derived from the Environmental Geology Map) to chart these vital environmental units. In any area these basic resource units can be evaluated in terms of current and projected human activities; the limits of their capabilities for various uses allow for the development of guidelines that will permit maximum use consistent with minimum environmental degradation.

A suite of special maps can be constructed from a basic natural resource capability map by evaluating all the units of a region in terms of all possible uses or activities; each natural resource unit on the map, therefore, can be graded as to capability for each specific use, providing a basis for evaluating the potential impact of an activity. In this manner, potential environmental stresses can be predicted far in advance in order to provide a firm, logical, and just basis for environmental management and decision-making with the full realization of the economic, political, and social alternatives.

USING THE DATA

Statistical Measurements

The areal extent of each map unit, the length of linear features, and the number of specific environmental units within each map area are listed in tables. For example, the area covered by fresh-water marsh or the area being used as rangeland was calculated and tabulated by counties. In addition, the percentage of each unit within an individual map area was determined. The total length of features, such as pipelines, erosional shorelines, or bay shoreline, was measured along with the number of specific sites on the maps, such as power generation plants, waste-disposal pits, and airports. The areal extent of units is listed in square miles; linear features are in miles. Measurement of areal data is based on point-count methods, cross-checked by planimeter techniques. Average values proved to exhibit greater than 90 percent accuracy. Linear features were measured by map-measuring wheels, and average values display greater than 95 percent accuracy. Accuracy of quantitative data is principally limited by the scale of the maps and the nature of the polyconic map projection.

Environmental Resource Subject Guide

An extensive, alphabetized index of information concerning the Coastal Zone has been compiled to afford easy access to desired information. The index provides a subject guide for locating information of a general nature, as well as information not specifically included in the map legends; both map and text sources are indexed. Following is an example of how this material may be used: One may wish to determine areas with very low permeability that would serve as satisfactory solid waste - disposal landfill sites. By referring to *permeability* in the index, the reader is directed to the Physical Properties Map, to specific pages in the text, and to a table evaluating land use suitability in the particular map area. In this manner, the areas of low permeability can be located on the Physical Properties Map. Reference to the text and to a table listing land suitability provides additional description and evaluation of landfill suitability. In addition, if the user wants to know the percentage of current solid waste - disposal sites within a map area that are improperly located, he can evaluate each site based on the properties at each location (Physical Properties Map) and then determine the percentage that are poorly located. Interpretation of data in this general manner will naturally depend upon the experience of the user in the specific subject of interest.

Generating Additional Data

For cartographic convenience and feasibility, basic data are presented on a series of nine maps. Combining information from two or more maps may provide additional insight into the nature of an area or provide a specific solution to an environmental problem. Many other special maps can be prepared by the user to present any combination of properties or characteristics necessary. For example, to evaluate an area in terms of potential for recreational parks, characteristics desirable for this particular land use must be defined. If the desired recreational land should be well drained, above hurricane tidal effects, vegetated with mixed pines and hardwoods, remote from oil fields, sulfur mines, pipelines, power lines, and industrial or residential developments, the coincidence of these several factors, obtained by overlapping the special-use environmental maps depicting these required properties, outlines areas suitable for this type of recreational development. All of the recreation requisites can be obtained from various maps of the Environmental Geologic Atlas of the Texas Coastal Zone; a map that shows the location and grades potential recreation sites can, thereby, be prepared by the user.

If an industrial site is desired within a region, the area can be analyzed using the Atlas. For example, reference to the Physical Properties Map outlines areas with suitable foundations and other related properties; the Current Land Use Map indicates the current use and approximate value of the land, as well as location of airfields or residential areas for employees; the Mineral and Energy Resources Map indicates availability of construction materials, pipeline facilities, railroads and highways, and principal power lines; the Topography and Bathymetry Map shows the slopes and land configuration which might bear on the site selection; the Rainfall, Stream Discharge, and Surface Salinity Map illustrates climatic data that might be critical; the Man-Made Features and Water Systems Map shows the nature of drainage systems, reservoirs, made land, and other related factors within the area; and the Environments and Biologic Assemblages Map provides information on the nature of vegetation at potential sites. In this manner, an environmental analysis may be made to evaluate a site or area for a specific, potential land use; or a broad area may be analyzed in order to locate or outline favorable sites for specific uses.

Other maps that may be made from the Atlas could outline areas of positive or negative suitability for a specific use; or the entire area could be grouped into various capability or use grades from excellent to poor on the basis of the number of desirable land factors which coincide. The varieties of special-use environmental maps that can be prepared from the basic Environmental Geology Map and units on the eight Special-Use Environmental Maps are essentially unlimited. By combining maps of the Atlas with other sources of economic, planning, industrial, transportation, or sociological data, a broad spectrum of environmental problems and management goals can be solved, or at least outlined and properly defined.

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GEOLOGIC AND SEISMIC ENGINEERING FOR NUCLEAR POWER PLANTS IN THE SOUTHEAST

by

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1. INTRODUCTION

1.1 Requirements of a Nuclear Power Plant

A nuclear power plant like a transportation system is one of the most complex and expensive of all engineering endeavors. Like a conventional fossil fuel plant it consists of a number of individual components, each having its own special requirements. In addition, each of these is a part of a system whose parts must function together with a common objective: producing electric power cheaply, efficiently and safely. Although various types of plants are in use, most include the following:

1. The nuclear reactor and the containment structure which surrounds it. Within the containment are cooling systems, control systems and monitoring systems.

2. A heat exchanger that converts the heat produced by the nuclear reaction to steam that generates electricity, similar to the steam boilers in a conventional coal fuel power plant. Some plants directly use steam that is produced within the nuclear reactor; other plants include heat exchanger that confines the reactor steam within the containment structure and produces new, independent steam that is then fed to the turbo generators.

3. Electrical turbines and generators which convert the heat energy of the steam power to electrical energy for transmission to the power lines.

4. Electrical switching station and transformers such as those normally associated with electric power generation.

5. A cooling system that provides water to cool the various systems, particularly the condensers of the steam turbines during power production.

6. An emergency cooling system which cools the nuclear reactor in case of a sudden shutdown of the plant during which the nuclear heat energy cannot be utilized in generating electricity.

7. A system for handling the radioactive fuel and the spent fuel.

8. A system for generating emergency electric power to enable the nuclear power plant to shut down safely in case some accident should occur that would disrupt normal electric power generation.

Some of these components are common to all thermal electric generating stations. Others, such as the reactor containment vessel and the emergency cooling water system for the reactor, are peculiar to nuclear power plants. However, there are major differences between nuclear and the fossil fuel steam electric generating station. Because of the lower steam temperature and pressures, the thermal efficiency of the nuclear plants is somewhat less than that of modern fossil fuel steam electric stations. Larger amounts of cooling water are required and greater amounts of heat are dissipated for each unit of power produced. Moreover, the turbine generators for a nuclear power plant are approximately four times the physical size of the equivalent machines in a modern fossil plant. Therefore, from the engineering point of view, the nuclear power plants present an unprecedented scale of power plant engineering and construction.

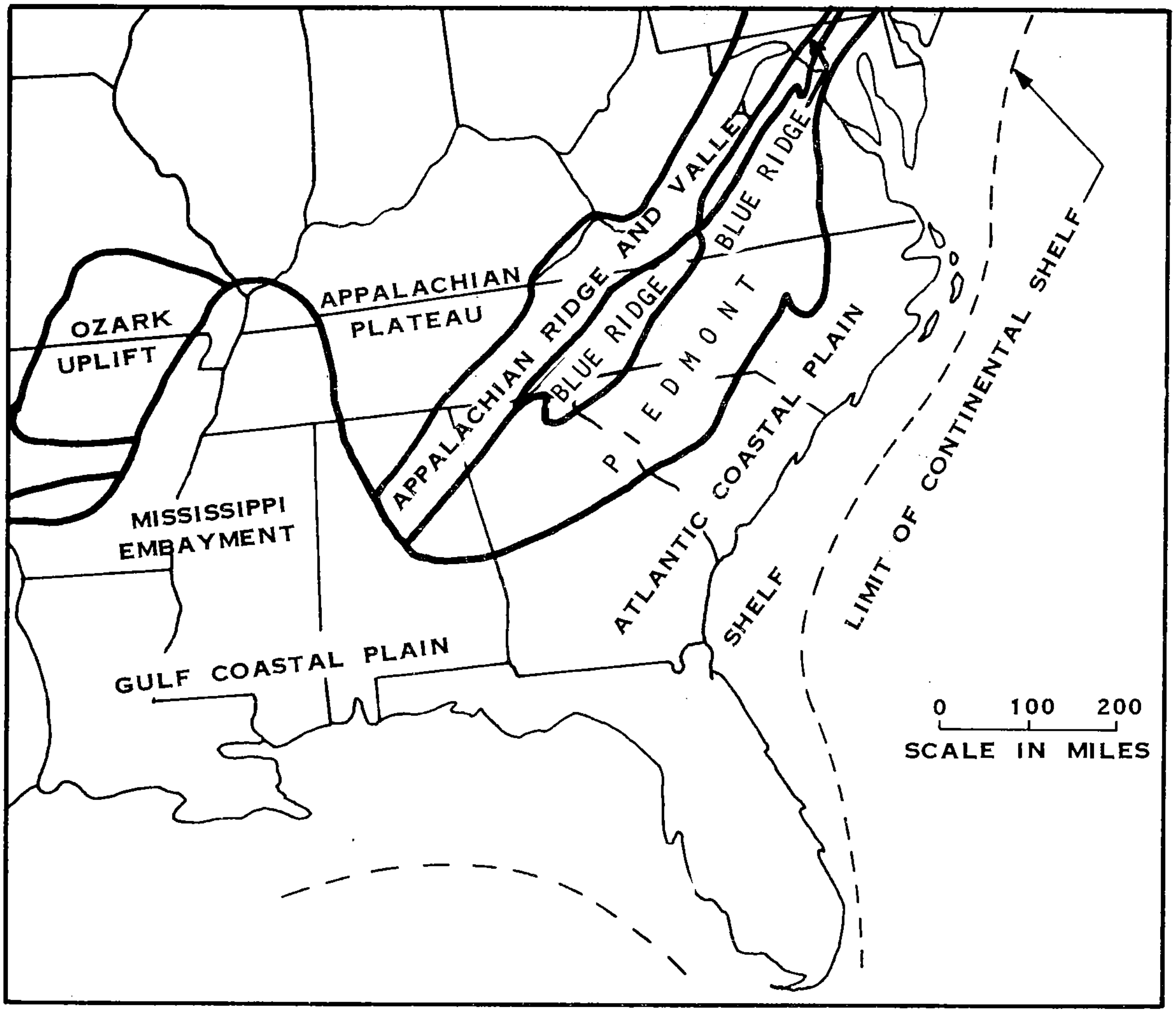
The geologic and seismic requirements for a nuclear power plant are really no different from any other large engineering structure whose failure could endanger man or his environment.

1. The foundations must provide support for all of the structures and particularly for those whose failure could endanger life or the surroundings. The foundation design must include a reasonable margin of safety against the effects of unusual natural hazards: earthquakes, tornadoes, hurricanes, and tectonic movements, such as faulting.

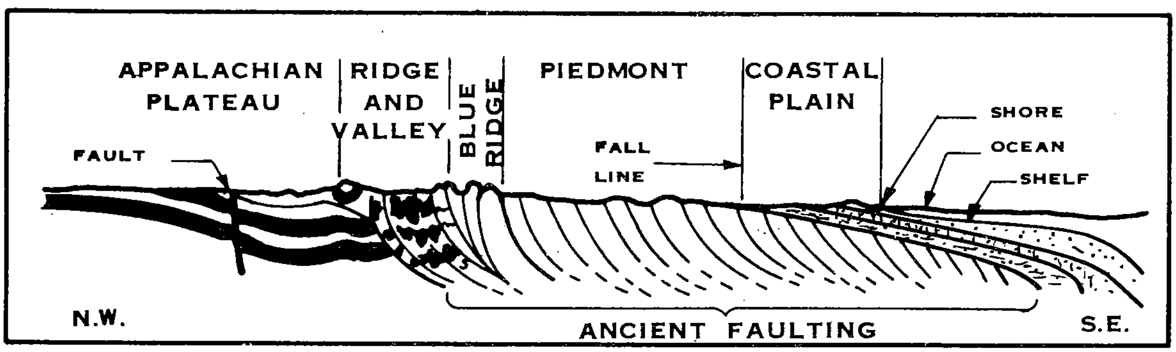
2. The site must be safe from the effects of flooding from nearby streams, lakes or other bodies of water.

3. Those waste products which could endanger the surrounding public or environment must not be allowed to escape.

Again, these requirements are valid for both the conventional steam electric generating plants and for nuclear power plants. The difference is in degree. Because of size and complexity, the nuclear power plant requires more extensive and exacting geotechnical design. Moreover, the fuel and the waste products may be potentially a more serious risk to the public than are the wastes from the conventional thermal power plant (although that point is still a matter of controversy).



MAP OF GEOLOGIC PROVINCES IN SOUTHEAST



CROSS SECTION N.W. TO S.E. ACROSS KENTUCKY, TENNESSEE, NORTH CAROLINA AND SOUTH CAROLINA: VERTICAL SCALE EXAGGERATED 500,000 : 1

FIGURE 1 GENERALIZED GEOLOGY OF SOUTHEASTERN U.S.

1.2 Critical Factors in the Geotechnical Requirements of Nuclear Plants

The typical loads involved in nuclear power plants are not unduly heavy compared to the large conventional steam-electric generating stations or other big structures being built today. The weight of a nuclear power plant is only a fraction of that of a major concrete or earth dam and the concentrations of loads are much less than those of a city skyscraper. The area required for the plant is no larger than that for a medium-sized industrial site: typically one-quarter to one-half mile square for all of the major components (although for economic, topographic or political reasons it may be necessary to encompass larger areas).

The engineering design related to site conditions, however, involves a number of systems whose tolerances are much smaller than those of other structures. First, certain portions of the structure (particularly the mechanical piping) are more critical than in conventional process industries. This is because of the fluids whose loss could contaminate the atmosphere or the ground. Second, there are psychological constraints generated by those elements of the public who associate nuclear power with the atomic bomb. (Some of those who once carried banners stating "BAN THE BOMB" are now carrying similar placards that equate nuclear power plants to war.) Third, a few irresponsible members of the scientific and engineering professions obtain headlines by half-truths and even false statements regarding the limits of our scientific and engineering knowledge. They are being used by those who, for fear, prejudice or selfish gain, would discourage the sound development of nuclear energy. Thus, the geotechnical engineering for nuclear power plant design is far more demanding than the design of comparable structures, even when some of those may be far more hazardous to human life and to the preservation of the environment than the nuclear power plant.

2. GEOLOGIC STRUCTURE

The Southeastern United States is characterized by a wide range of geologic structures and ages. These are shown in a very generalized form in Fig. 1.

2.1 Piedmont

The Piedmont is possibly the oldest and most unusual. Its western boundary is the Appalachian Mountains. It extends in a northeast-southwest belt from Alabama into New England. It is as wide as 120 miles in Georgia and South Carolina, but narrows to the north to about 10 miles across eastern Pennsylvania. The rocks are old. Some may be Precambrian, older than 660 million years; with few noted exceptions, none are younger than the Paleozoic Era which ended about 230 million years ago. They consist of ancient sediments that were metamerphosed by pressure and heat when they were buried deep below the present ground surface. Their sedimentary origin is barely evident and often disputed because they have been converted into crystalline rocks, predominately gneiss and schist, with some zones of slate and meta volcanics.

The pressures producing the alterations were probably generated by the tectonic forces of drifting global crustal plates. The predominate force was compression, acting in a northwesterly direction, which produced the Appalachian uplift and which caused innumerable wrinkles and fractures with a northeasterly trend, or strike. The previous high stress and temperature levels within the presently exposed rocks are demonstrated by the plastic flow of the minerals and their rearrangement in parallel bands and sheets that resemble close stratification and which gives the rocks a candy-stripe appearance.

Superimposed on the predominate structure are numerous small wrinkles as well as cross-folds and fractures. Some of the fractures are simple tension cracks that accompanied the warping and folding. Others are strike-slip faults that exhibit horizontal displacement of aportion of rock mass against the other. Faulting is diagnosed by offsetting of the rock bands, by the polished or striated sliding surfaces or "slickensides" or by the ground rock "gouge" along the cracks. The great age of these fractures is demonstrated by the complete rehealing of many, by subsequent mineral deposition or by plastic readjustment of the rock so that the wavey slickensided surfaces now fit together perfectly.

During the late Paleozoic Era and the ensuing Triassic Era (180 to 230 million years ago), localized upswelling of hot plastic magma produced massive intrusions of granite and similar rocks into the existing bodies of country rock. Stone Mountain, Georgia is a prominent example of a broad granite intrusion. Numerous thin wall-like intrusions or "dikes" also occurred, when the plastic or molten rock squeezed up into the previous cracks.

The uplift forces waned long ago and the heaving gradually diminished, allowing erosion of the mountainous region to a plain whose surface is now only 400 to 1000 feet above sea level.

2.2 Blue Ridge

The western edge of the Piedmont apparently received a greater degree of upthrust, forming the Blue Ridge Mountains. These consist of similar rocks, but with more intense ancient wrinkling, folding and fracturing, striking northeasterly.

2.3 Erosion

The crystalline rocks still retain their identity from the Blue Ridge to the edge of the Continental Shelf as far as 100 miles off the present Atlantic Coast. They dip downward to the Southeast retaining the effects of the ancient thrust that was upward to the Northwest.

Soils and soluble weathering products from the weathering of the uplifted Piedmont and Blue Ridge were transported eastward to the ancient ocean and westward into an inland shallow sea that occupied what is now the land between the Rockies and the Appalachians during the Paleozoic Era.

2.4 Triassic Basins

There are only isolated remnants of the oldest sediments toward the East. Localized areas of the crystalline basement dropped downward sharply during the Triassic Era, 180 to 230 million years ago. The eroded sediments formed claystones, siltstones, fine sandstones and even some low-grade coal. The deposits were probably continuous over much of the old sea bottom, but all other than those trapped in the Triassic basins were subsequently eroded and redeposited further east in what is now the Coastal Plain, described later. A few of the Triassic basins are exposed in the present Piedmont and others have been found by deep boring beneath the younger Coastal Plain sediments.

2.5 Coastal Plain

These sediments comprise the present Coastal Plain, the third major geologic province of the Southeast. Geographically, the Coastal Plain consists of three parts: the Mississippi Embayment is its center (a triangle with its vertex pointing up the River) and including much of Louisiana, all of Mississippi and southern Alabama. It merges in a broad crescent into the Atlantic Coastal Plain that includes the Florida peninsula, the southern 3/5 of Georgia and South Carolina and narrows in an elongated triangle with a vertex near New York. Westward it forms a second arc, the Gulf Coastal Plain of eastern Texas.

The boundary between the exposed crystalline basement of the Piedmont and the Coastal Plain sediments is known as the "Fall Line". It is marked by changes in topography from rolling hills to more gentle undulations and by rapids in the streams which formed the inland limit for riverboat navigation a century ago. Thus, the Fall Line was the locus for major settlements for growing towns during the early settlement of the Southeast. Washington, Richmond, Columbia, Augusta, Columbus, and Montgomery are on or within a few miles of the Fall Line.

The sedimentary formations lying southward and eastward of the Fall Line are much younger (Cretaceous to Recent) than those of the crystalline basement. Because of the continuing downwarping of the basement toward the East and South and the changes in sea level, these sediments outcrop in more or less parallel bands that are successively younger toward the sea. The formations consist of sands, silts, clays and calcareous deposits which have become indurated by the weight of overburden and by geochemical changes since their deposition. The induration has not been uniform and so the formations that underlie this area range from sands and silts and clays, which have been physically hardened by desiccation and consolidation, to siltstones, claystones and limestones which locally are as hard and strong as the rocks found in older, more indurated regions.

2.6 Paleozoic Sediments

The eroded materials, which were washed westward into the inland sea, formed alternating layers of sandstone and shale. Thick deposits of limestone accumulated by the deposition of shells, algaes and chemical precipitation. The strata were more or less continuous; many can still be traced from Alabama into Canada. The deposits that can be found today are predominantly of the Paleozoic Era which means that they accumulated 230 to 600 million years ago.

2.7 Appalachian Uplift

The last episode of major northwesterly thrust and uplift probably occurred at the close of the Paleozoic Era or during the early Triassic Period. There is no evidence of major tectonic activity during approximately the last 200 million years. The Blue Ridge and part of the Southern Piedmont were thrust over the Paleozoic sediments to the West producing a more or less continuous fault along the boundary that is marked today by an abrupt mountainous wall, including the Great Smoky Mountains.

2.8 Ridge and Valley Province

A belt of Paleozoic sediments about 70 miles wide adjoining the Blue Ridge was badly distorted into a series of parallel corrugations whose axes trend north-east-south-west. This is the Ridge and Valley Province and it extends from Alabama on the south into Pennsylvania. Within the area are numerous ridges that parallel the folds. The compression of the belt produced numerous minor wrinkles, cross-folds, joint cracks, and faults.

2.9 Appalachian Plateau

West of the Ridge and Valley the Paleozoic sediments lie nearly level, largely undistorted by the thrust. There is gentle warping producing a very broad structural dome with gentle rock dips near Nashville, Tenn. and a flat arching of the sediments near Cincinnati that ends with another broad dome at Lexington, Ky. In northern Arkansas and southern Missouri the formations are thrust upward to form the Ozarks. South of the Ozark uplift

Fig. 2a Baton Rouge Fault (between arrows) looking WNW, showing stream contorted by fault (Infra red by author).

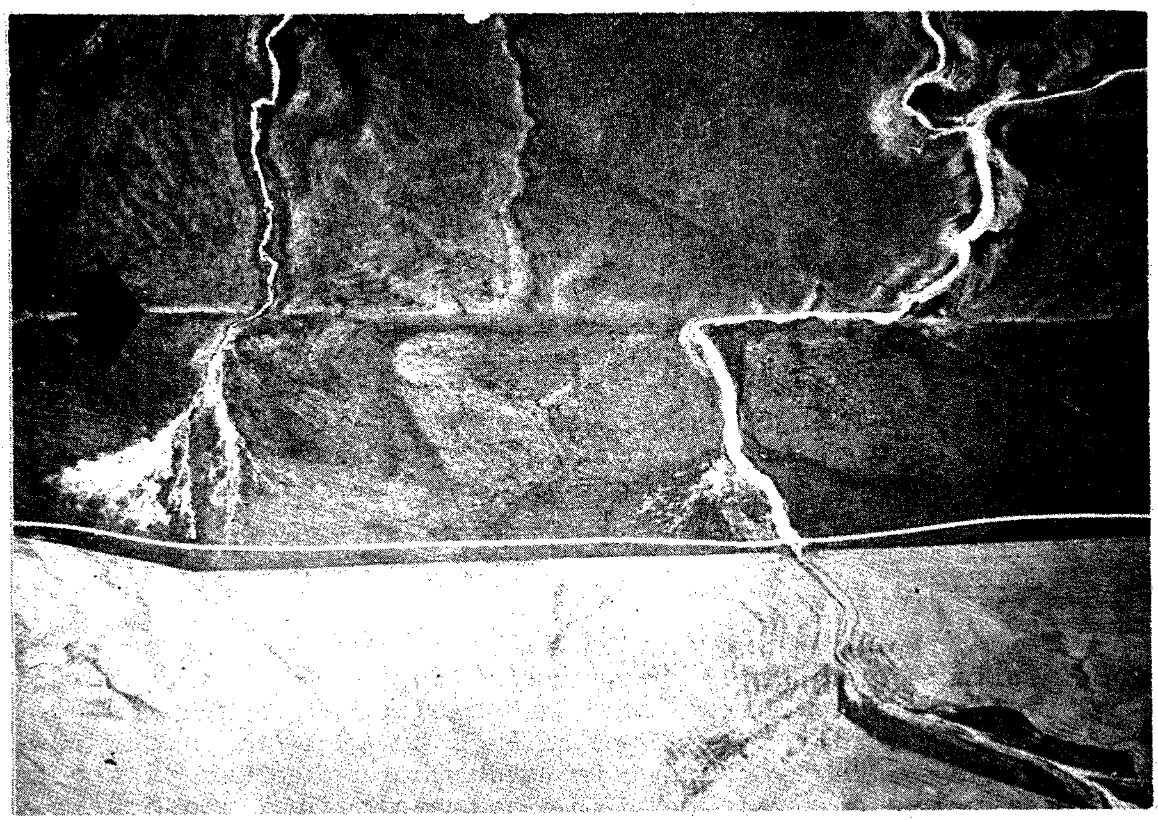


Fig. 2b San Andreas Fault expressed by shifts in streams crossing the fault, and by a narrow trough.

there is a small region of E-W uplifting that extends beneath the sediments of the Mississippi Embayment and which may be a continuation of the Appalachian uplift as expressed by the Blue Ridge and Piedmont.

3. FAULTING - MAJOR STRUCTURE

With the exception of the Baton Rouge Fault, there has been no fault movement observed in the southeastern United States during historic time. Even the Baton Rouge Fault is subject to some argument. It appears to be the northern hinge of wedge settlement from the accumulating sediments of the Mississippi in the Mississippi Embayment. Two faults are shown in Fig. 2.

During historic times minor cracks have been observed in the earth's surface. Most of these are arcuate and appear to be the result of slumping or landsliding. Other cracks have been the result of stress relief expansion induced by erosion during the two million years since the most recent major tectonic uplift.

3.1 Identifying Geologic Structures, Faults and Discontinuities

Structural features are abundant in the Piedmont, Blue Ridge and Ridge and Valley Provinces. Every deep excavation reveals fractured rocks, evidence of ancient faulting (such as slickensides) and offsets in the bedding and layering of the rock. An inexperienced geologist may become overexcited at his first view of those real but old faults. Some (with excessive imaginations) have even conjured up visions of the San Francisco Earthquake occurring anywhere in the southeastern United States.

Most of the geologic structures and particularly the offsets and slickensides that accompany offsets in the southeastern United States are hidden from view. In the Coastal Plain, they are covered with more recent sediment, varying in thickness from a few feet near the Fall Line to thousands of feet near the Atlantic and Gulf Coasts. In the Piedmont and Blue Ridge, where the crystalline rocks are exposed, the rocks are blanketed with thick accumulations of residual soil (highly weathered rock). While some of this weathered rock, terned "saprolite", retains the relict structure of the original rock, in some places the weathering has been so severe and so deep that all the former structure is obscured.

3.2 Soil Profile

Ultimately, the process of soil profile formation near the ground surface completely obliterates the ancient structure. It is replaced by a new structure that reflects the final stages of weathering. In this zone, typically 5 to 10 feet thick, the local environmental effects completely mask the character of the parent materials below. Thus, inexperienced geologists and others who limit their study to the ground surfaces are unlikely to see the details of rock structure which underlie the region. They are surprised when they see that the geologic structure is far more complicated than a ground surface examination (including occasional rock outcrops or shallow rock cuts) might indicate. Because of the blanketing by the Coastal Plain's sediments and the residual soils that cover the crystalline areas of the Piedmont and Blue Ridge and also the sediment to the west, it is necessary to make unusually thorough studies of geologic structures in evaluating the effects of those structures on the performance of foundations.

3.3 Boring and Sampling

Detailed soil sampling and rock core boring are an essential part of any study of site conditions. These reveal the details of the geologic structure at the site. They also provide the samples that are eventually tested to determine the engineering properties needed for design. Unfortunately, the usual small diameter soil sample or rock core only reveals a statistically small part of the site conditions. A vertical boring can entirely miss a vertical crack in the soil or rock a foot or two away from that boring. Thus, the conventional soil and rock boring are supplemented by sloping borings and large samples particularly where there are indications of steeply dipping rock structure.

Rock core borings reveal cracks and structural defects, but ordinarily they do not show the direction of the structure because the core of rock rotates in the sampling tube. Therefore, special provisions are required to determine the orientation of fractures. Complex rock core drilling, using swivel-mounted interior tubes, which allow the core to remain stationary and which score the core in a predetermined direction, are one approach to identifying the direction and dip of any rock fractures. Similarly, sophisticated soil sampling techniques can obtain samples with a known orientation.

3.4 Bore Hole Imagery

Small diameter television cameras and small diameter photographic cameras which can be lowered in the bore hole view the walls of the core hole. The records can be interpreted to obtain the orientation of fractures and other structure that are encountered. All of these techniques are slow, expensive and the results are not always capable of conclusive interpretation. For example, it is difficult to obtain high quality television or photographic data below the ground-water level. However, a program of core boring and soil sampling sedigned specifically to obtain data on structures can be evaluated by knowledgeable engineers and geologists.

3.5 Bore Hole Logging

Various methods of bore-hole logging provide useful data for correlating the strata throughout a site. Each soil and rock strata exhibits certain subtle peculiarities that vary with depth in such a way that they exhibit a pattern of "signature". The data obtained from a sensor that is lowered in the bore hole and which measures the soil or rock properties at close intervals and which plots a continuous record of the properties as a function of depth. Some of the properties that are measured include electric self potential, electric resistivity, nuclear radiation and absorption and scatter of nuclear radiation from a source within the measuring device. The patterns or signatures from numerous holes are compared to determine the dip of the formations or the location and amount of possible offsets.

3.6 Test Trenches and Pits

Deep trenches excavated across the site provide a larger look that displays the interrelationship of the structural details that are difficult to see in core borings. Obviously, such trenches are extremely expensive and often are difficult to construct. At one site, three test trenches required a total excavation of 25,000 cu yd of soil and rock. A program of test trenches should not be undertaken until data from core borings and geologic mapping have been evaluated so that the test trenches are located where they will provide the greatest amount of information for the least amount of money. However, if properly located, they are probably the best single tool for correlating all of the data obtained in core borings to obtain an integrated picture of the details of rock and soil structure.

Where the rock is so deep that it is impractical to reach it by trenches, caissons (holes 3 to 4 ft in diameter provided with steel liners which extend to the rock and which even can be core drilled into the rock) provide large windows for direct examination by engineers and geologists. Large samples of the materials can be obtained for detailed laboratory testing to determine the age of the minerals and the physical properties of the soils and rocks encountered.

Highway cuts in the vicinity of the site and railroad cuts provide ready-made test trenches. Although these have not been located with the purpose of providing geologic or engineering data, they can be extremely useful supplements to the data obtained directly on the site. Moreover, they provide an enlarged view of the near-site and regional conditions. Unless the cut has been made relatively recently (within the last few months), rainwash and the disintegration of the materials upon exposure can be misleading. Therefore, it is necessary to dig into the surface of these cuts to obtain a fresh exposure for examination. This is particularly important when viewing the weathered rock so frequently encountered in the Piedmont and Blue Ridge regions.

3.7 Age of Geologic Structures

It is particularly important to be able to determine the geological age of any structural defects ~~that are~~ encountered. The predominance of geological opinion is that major tectonic activity ceased in the southeastern United States during the Triassic Age (about 180 million years ago). Although there is some evidence of small continuing movements in the Southeast, there is no indication that these movements are either significant or tectonically related.

One method of aging is to evaluate the character of the minerals in the vicinity of cracks and fissures. For example, a fault surface that is wavey and exhibits no locally fractured material must have developed under conditions in which the rocks were in a semiplastic state; otherwise, the movement would have been accompanied by fracture and the formation of broken rock. Therefore, the faulting must have taken place in an entirely different geologic environment, millions of years ago when the site was buried deep below the earth and subjected to high stresses and high temperatures.

Intrusions of materials, such as quartz, which cross the fault or fracture without offsets, indicate that the structure is older than the quartz intrusion. Since quartz is deposited in a far different geologic environment than that presently near the ground surface, its presence indicates that the fracturing took place in a different geologic age. An example can be seen in Figure 3. Deposits of other minerals may occur under present geologic conditions; because their rate of deposition is slow, the amount of the mineral is a clue to its age.

A third method is to examine deposits on the surface of the fracture itself. Leaching of organic compounds from the ground surface may contain enough carbon to obtain a carbon isotope dating. Although the maximum age that can be determined this way is approximately 35,000 years, such dating can provide a minimum age for the fracture. Deposits of potassium-bearing minerals may permit potassium-argon dating. For example, some of the faults which have been in nuclear site test pits in the North Carolina Piedmont have been aged by this means. Their ages range from 180 to 230 million years, approximately. Although the accuracy of such isotope aging is seldom better than + or - 20 percent, there is really little engineering significance in such a range of error. From the engineering point of view, if the fault has not moved

Fig. 3 Quartz dike exposed in caission with intruded brotite that permitted potassium-argon dating. Dike crossed a fault without an offset (Photo by author in 60 ft. deep caisson 48 in. in diameter, in N. Carolina. Age of dike about 180 million years).

Fig. 4 Lineaments of stream offsets in South Georgia

within 35,000 years, the likelihood of its moving within the projected life of a nuclear power plant (perhaps 25 to 50 years) is extremely remote. Thus, it is reasonable to conclude that if there has been no movement for millions of years (regardless of how many millions), movement is extremely unlikely during the life of the plant.

As was previously stated, the soil-forming processes close to the ground surface completely obliterate the structure of the rock. These soil-forming processes, which develop definite horizons within 6 to 10 ft of the ground surface, proceed slowly. Thousands of years are required for the downward leaching of clay minerals from the topsoil of A-horizon at the ground surface to develop a clay and iron-rich B-horizon below. If the geologic structure does not extend into the B-horizon, this established that the age of that structure is greater than the period of time required for the development of the soil horizons. This ranges from thousands to millions of years.

By combination of isotope aging, mineral indentification, the character of the zone of movement and various intrusions, it is possible to determine the age of most of the structural features with reasonable accuracy. With minor (and questionable) exceptions, it can be shown that there has been no tectonically induced structural movement in the southeastern United States for many million years.

3.8 Inferred Structure

The evaluation of deep geologic structure on a regional basis in the eastern United States is difficult because of the cover of vegetation, sediments and residual soils. However, it is possible to infer structure because it is frequently reflected in the surface topography. For example, an examination of the currently active San Andreas Fault in California shows that it is marked by definite topographic features: strings of elongated ponds, vertical cliffs in which the newly exposed material erodes and the abrupt horizontal displacement of stremas along the line of the fault (as shown in Fig. 2). Thus, it similar features are observed in other areas, it is possible that they reflect faulting.

Detailed studies of aerial photographs, ERTS imagery (television-like photographs obtained from the ERTS satellite), high-altitude aerial photography, photographs and images obtained in various colors of light and infra-red bands, radar imagery and low-level examination of detailed topographic features, are the tools of this "remote sensing" or indirect evaluation of ground structure. The data obtained must be considered to be inferences. They point out areas of geologic interest or structural suspicion. Those indications which are sufficiently well-defined should be studied in more detail by direct methods. However, the absence of such indications is good evidence of the absence of critical geologic structure.

A good example of the use of aerial photographs to determine the locus of prominant geologic structure is the study of stream patterns in the Georgia Coastal Plain where the basement crystalline rock structure is completely covered by sediments thousands of feet deep. As can be seen in Fig. 3, the rivers follow the regional slope toward the Atlantic, exhibiting a strong southeastward parallelism. However, a number of these streams exhibit abrupt shifts in direction. These shifts occur in similar amounts in adjacent streams and they lie more or less within narrow straight belt. Furthermore, the belts occur in sets of two's and three's as can be seen in Fig. 4. Geophysical evidence suggests that these shifts are reflect gentle flexure or folds in the underlying rock. The stream shifts occur in sweeping curves and not the abrupt shifts that accompany faulting. The location of the Hatch Nuclear Power Plant of the Georgia Power Company was selected so that it would not fall on the extension of any of these lineaments of stream shifts. If it had, it would have been necessary to study their structure by intensive geophysical exploration and by deep drilling to insure that they do not reflect active faulting beneath the nuclear plant.

In the Piedmont region the stream patterns frequently exhibit parallelism in the short reaches of second and third order (the large creeks and the small rivers into which these creeks empty). The stream parallelism occurs in sets, forming parallelogram-like patterns. This is termed "angulate drainage" which is a distortion of the typical random pattern found in igneous rock, such as granite, or the trellis pattern that is prominant in folded sedimentary rocks, such as shale-sandstone. Detailed studies of this parallelism in numerous areas in the vicinity of nuclear power plants in the Piedmont show that these patterns are reflect rock jointing. While jointing is related to rock flexure, it develops without structural offsets. These stream patterns provide wide area information on the expected directions of fractures in the rock. They give the structure designer evidence of the excavation problems he is likely to encounter. Such shifts occur in Fig. 5.

From the foundation engineering point of view, joints do not significantly reduce the rock bearing capacity. However, they are related to ease of excavating the rock, possible damage to the rock by overblasting and, particularly, the stability of rock slopes. These must be taken into account in designing deep excavations and permanent rock and soil slopes.

The patterns of stream drainage, vegetation, land use and other topographic details, also help pinpoint areas in the region which should be investigated further to determine if there is any profound structure which might affect the site. For example, the Piedmont Region is underlain by numerous intrusive bodies, some of which are in the form of narrow, wall-like masses termed "dikes".

Fig. 5 Angulate abrupt changes in river alignment in South Carolina Piedmont, parallel to jointing of gneiss bed rock. (Oblique low level air photo by author.)

Fig. 6 Circular land use pattern produced by ring dikes above a massive intrusion near Charlotte, N.C. (USDA photos)

While most of the dikes lie in parallel lines following both major and minor structural weaknesses, a few isolated systems provoke unusual patterns. One is near the town of Concord, N.C. A large mass of igneous rock intruded the older metamorphic materials from below forcing its way upward and causing the rocks above to both melt and bulge upward. Circular tension cracks developed in the rocks above this intruding mass. Igneous dikes filled these circular cracks. Although this intrusion took place long ago, probably during the Triassic Period (approximately 300 milllion years ago), the difference in the minerology of the intrusive rocks, compared to the surrounding materials, produces differences in land fertility and land use. This is reflected in both ordinary aerial photographs and in false-color infrared photographs: the patterns of the fields and particularly the patterns of the crops and their growth vigor are reflected in a series of concentric circles whose outer diameter is several miles.

A second example is shown in Fig. 7a, a map of a portion of eastern Florida centered around the St. Lucie Nuclear Power Station of the Florida Power & Light Co. near Ft. Pierce. From time to time in the past, various geologists have suggested faults parallel to the Atlantic Coast. The evidence of this faulting has been weak.

The swampy swale parallel to the present offshore bar and tidal lagoon is mildly suggestive of the trench that accompanies such well-known faults as the San Andreas. However, the trough in Florida is a broad, shallow swale rather than the concentrated ditch-like notch characteristic of the San Andreas. This difference, however, has been suggested to be caused by the difference in the soils and the age of the possible fault. A second bit of evidence has been obtained from the cuttings obtained during water well drilling. This evidence is fragmentary at the best because the water well cuttings were secured during churn or rotary drilling and only represent fragments which were washed to the surface. Therefore, the level from which these fragments came cannot be determined accurately. Furthermore, the drilling didn't include continuous sampling and, therefore, the elevations of the cuttings are known only within tens of feet. However, based on a few widely-spaced deep wells, there is a suggestion of an offset of the deeper strata.

The "faults" inferred from the offsets are shown in Fig. 5 as solid lines. As can be seen, these lines coincide with the swales. However, the offset evidence from the wells does not suggest such a definite location; it has only been hypothesized that any faults inferred from the borings should coincide with the swales.

These coastal swales can be seen quite clearly in the high-level ERTS imagery in the band 7 (infrared) range, Fig. 7c. The

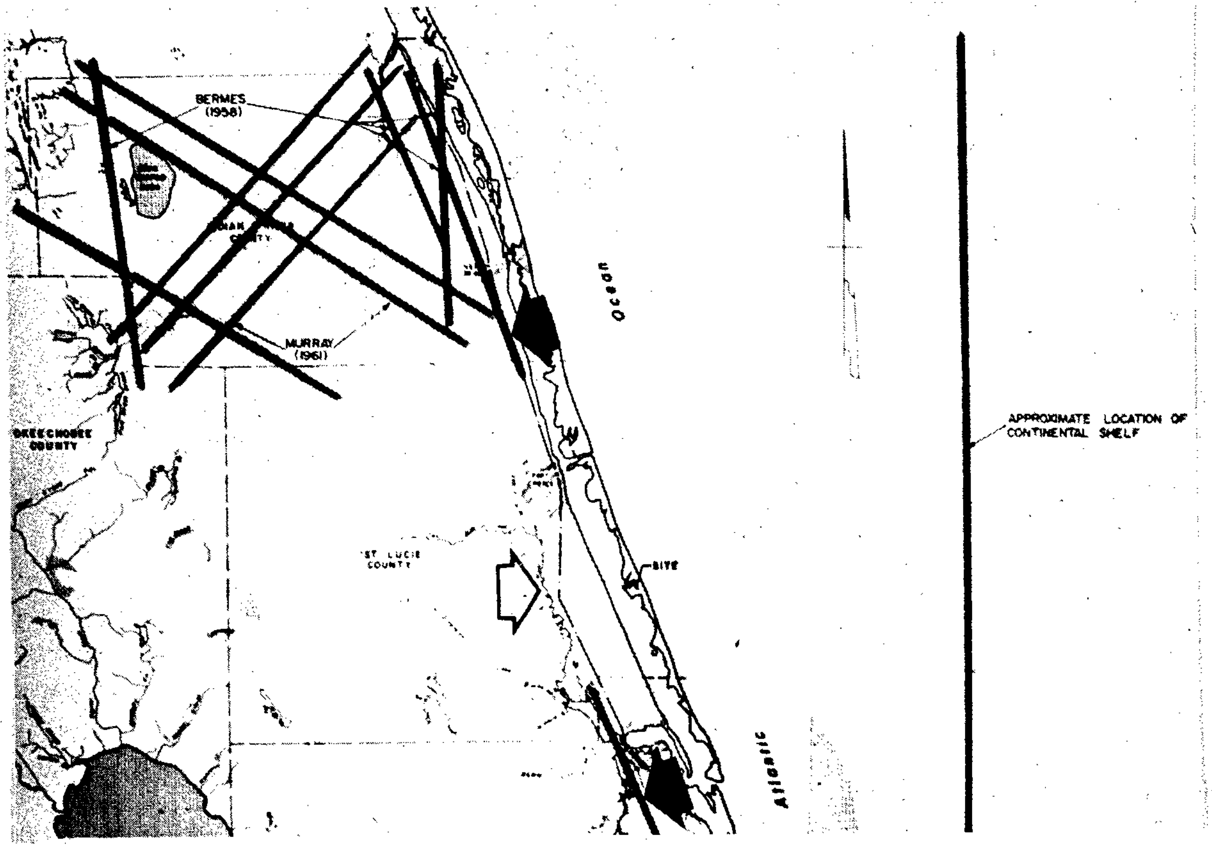


Fig. 7a Map of E coast of Florida, centered on Fort Pierce, showing "faults" inferred from swale, tidal river alignment and deep well cuttings that suggest "offsets".

Fig. 7b Oblique infra red photo shore of Fig. 7a (looking E at point of open arrow) with old marsh behind ancient shore that has been mistaken by some interpreters as a fault. (By author)

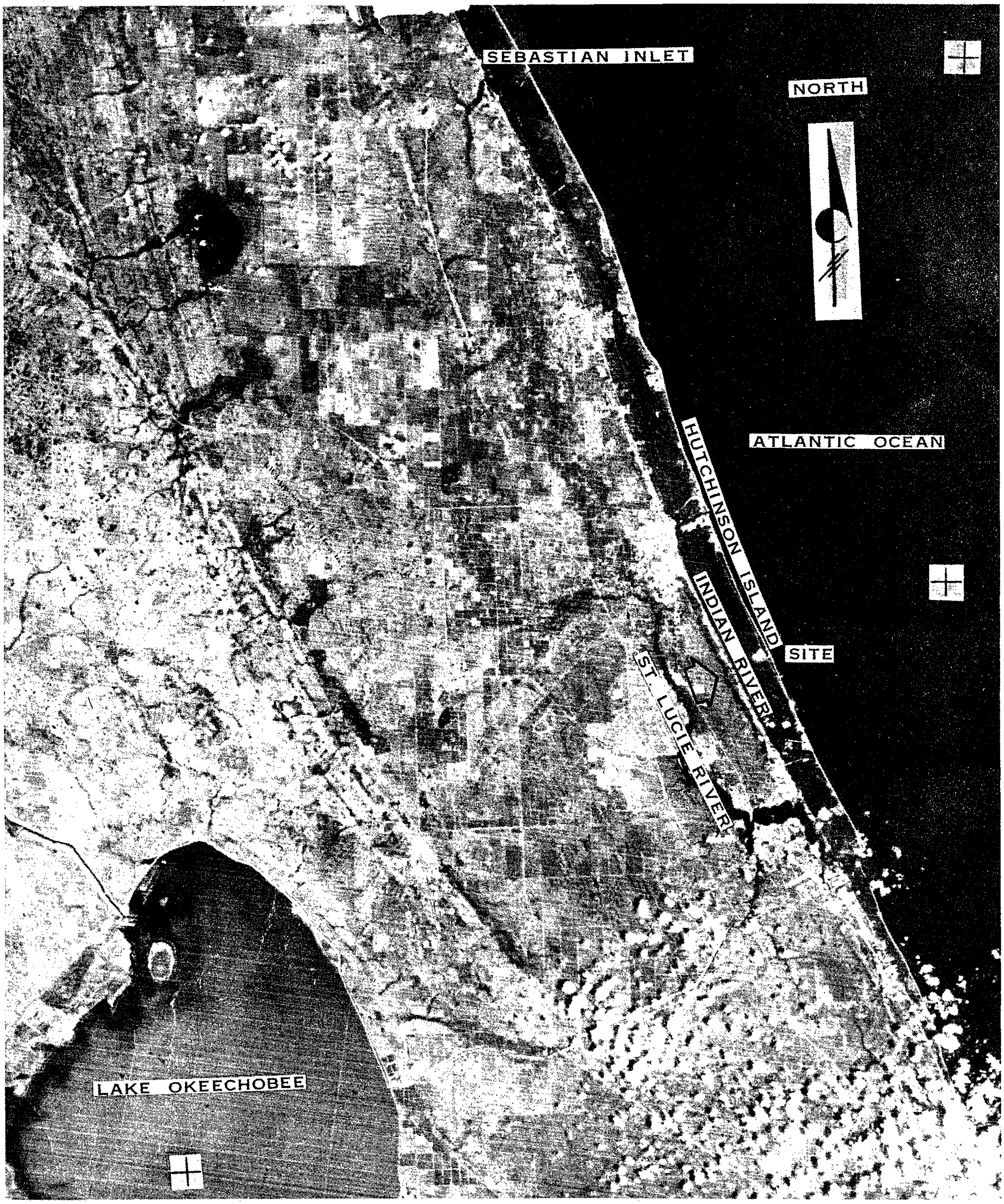


Fig. 7c ERTS imagery of area of Fig. 7 a,b taken by satellite scanner at altitude of about 600 miles. White tuffs in lower right are clouds. (Open arrow at location of Fig. 7 b, points to old marshy swale).

ERTS imagery was obtained by a camera, similar to a TV camera. The camera scans a very narrow band of the earth's surface from side to side at right angle to the satellite path to an elongated television picture. The imagery is obtained in several spectral ranges representing different colors and extending into the new infrared range which is invisible to the naked eye and even invisible to extended photographic films. Band 7 is the longest wave presently being obtained. It is particularly useful in that it shows low wet areas (which have a low infrared reflectance and radiation) as dark tones, whereas living vegetation (and clouds) are displayed as light tones. The tidal lagoon, known as the Indian River, is black; the continuous offshore sand bar is nearly white. The swale which parallels the "Indian River" and which has been inferred to be the trace of a fault, can also be seen on this imagery, although not as clearly. It is shown by the arrow on Fig. 6. An oblique infrared photograph made by the author from an aircraft flying at 7000 ft is shown in Fig. 7b, The construction work for the St. Lucie Plant can be seen in the far background on the offshore barrier beach and island. The coastal lagoon is denoted by C. The swale is shown very clearly by the dark band parallel to the tidal lagoon (the dark tone of this area in the infrared photography indicates a broad swampy swale which is unlike known fault traces). The hypothesized "faults" of the invarious investigators, both north and south of the St. Lucie/Ft. Pièrce area, end miles from the site. However, the continuation of the swale has, to some persons, suggested a continuation of a "fault" (if it indeed were present).

To check the hypothesis, deep borings were made on both sides of the swale opposite the St. Lucie Plant site. The boring locations were based on the trace of the swale as shown on the air photographs and the ERTS imagery. These borings extended into the Miocene formations which underlie the younger coastal sediments. These borings included detailed sampling of the materials encountered. They demonstrated that there is no offsetting or faulting of the Miocene formations (13 to 25 million years old) opposite the St. Lucie Nuclear Plant site. These borings also raise the suspicion that the inference of faulting, both south and north of the St. Lucie Plant, is not well founded because the swale is not a fault-related feature. A close examination of the aerial photographs of the region shows many similar swales parallel to the shore. All of these have the shape of successive offshore bars and coastal lagoons, similar to the present barrier beach and Indian River Lagoon. Each successive swale and the separating higher sand ridge occur at higher elevations above sea level. These correspond to hypothesized fluctuations of the sea during the interglacial periods of the Pleistocene Epoch. Thus, a study of the ERTS imagery, infrared photographs and ordinary aerial photographs help to identify possible structural features which should be investigated in detail and has confirmed their true geomorphological nature. Deep geophysical reflection seismic studies show that the well offsets represent only continuous small undulations in the gently dipping strata.

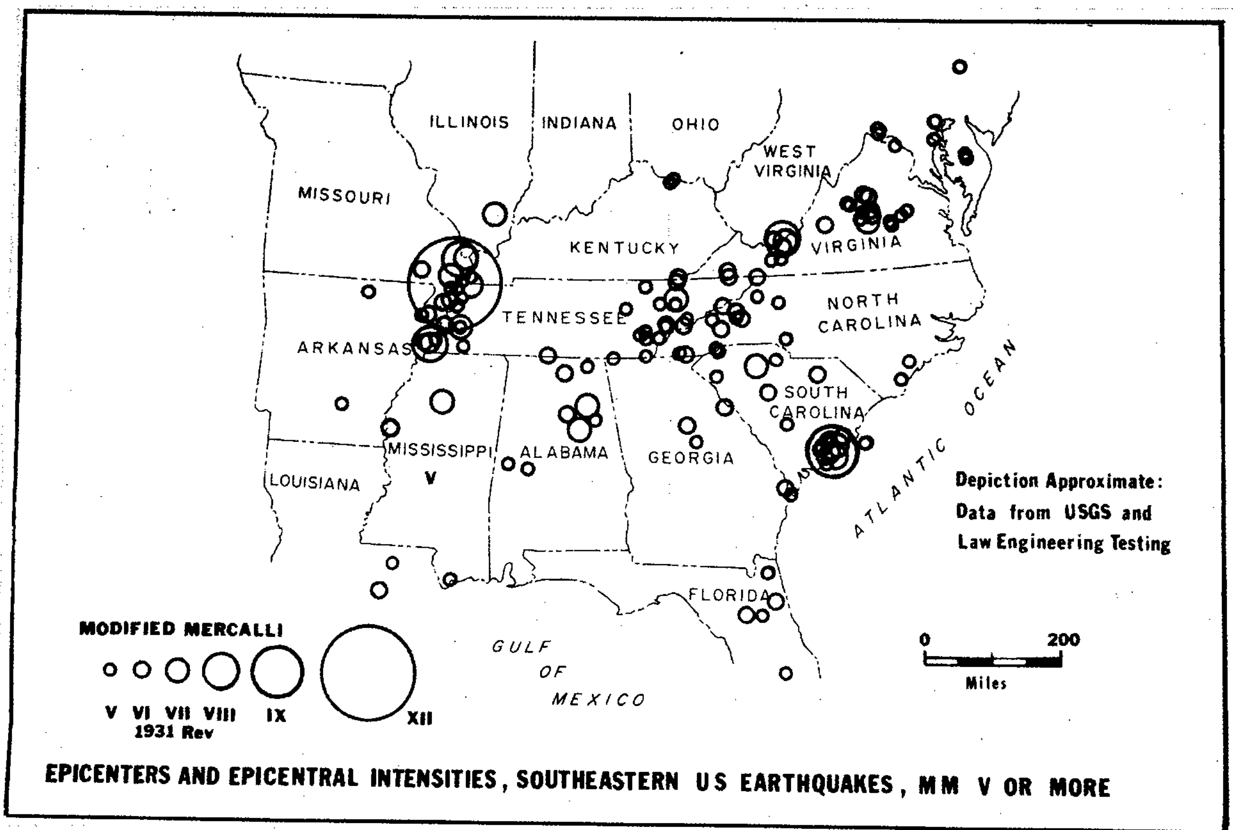


Fig. 8 Earthquake epicenter in the Southeastern United States (Law Engineering).

Fig. 9 Damaged columns in New Olive View Hospital caused by San Fernando Earthquake of 1971 (Photo by author).

4. EARTHQUAKES AND THEIR EFFECTS

4.1 Tectonic Patterns

A glance at a map showing the geographic locations of the centers of earthquakes (earthquake epicenters) shows that there is a definite pattern in several major belts of earthquake activity can be visually dientified. One of these is the perimeter of the Pacific Ocean. It includes Alaska, the Pacific Coast of the United States, Central America, the Pacific Coast of South America, Japan and the Philippines. These epicenters describe a crude oval on the surface of the globe.

A line drawn perpendicular to that oval following the Himalayan Mountains and crossing Iran, Turkey, Greece, Italy through Portugal and northwest Africa, describes the second zone. A third zone can be found in the Caribbean, including Central America, the northern coast of South America, the outer islands of the West Indies, the Dominican Republic and Cuba, forming an ellipse whose western end merges with the Circum-Pacific belt.

Recent research indicates that these earthquake belts coincide with the movements of large plates of the earth's crust. These plates drift on the plastic molten rock below, probably impelled by convection currents in the earth's hot interior. These impinge on one another generating strains in the earth's crust which finally produce fracturing. The shock, vibration and noise of the fracture is an "earthquake".

The central portions of these plates are generally devoid of earthquake activity. Occasional, small scattered earthquakes occur within them. Within some of the plates, however, there may be a localize strain buildup followed by release in the form of isolated larger earthquakes. These localized areas exhibit continuing low level activity; the larger events occur sporadically but within the same localized zone.

The southeastern United States, fortunately, is one of those areas which appears to be centered in a large tectonic plate and far from the edges which are associated with major continuing earthquake activity. Within the area the occasional earthquakes are of modest intensity and occur in rather random locations. There appears to be little pattern to the earthquake activity and no demonstrated relationship between the earthquake and specific major tectonic structures as can be seen in Fig. 8.

There are two exceptions: (1) The vicinity of Charleston, South Carolina; (2) an elliptical zone more or less centered on the Mississippi Valley, extending from St. Louis to Memphis.

Fig. 10a Earthquake-triggered land slide blocking road following San Fernando Earthquake of 1971 (By author).

Fig. 10 b Pavement cracked by San Fernando Quake of 1971 (by author)

Fig. 10c Landslide scarns in Alaska Quake of 1964 (USGS Photo)

4.2 Effects of Earthquakes

Earthquakes, in the past, have demonstrated they can profoundly affect engineering structures that are not designed to resist them. For example, the San Fernando Earthquake of January, 1972, caused severe damage in relatively new buildings including the new Olive View Hospital which was just being put into operation. The building was wracked from side to side. The columns in the lowest floors were torn apart, converting the concrete to rubble that was confined only by the large reinforcing bars, Fig. 9. It toppled stairwells and collapsed a one-story pavilion by shaking the roof free from its column supports.

Although it has been claimed by some alarmists that this hospital is an example of the inadequacy of earthquake engineering, it should be pointed out that the building was structurally designed for an earthquake less than half as strong as that which occurred. The reasons for the discrepancy appear to have been more political than technical; the building code did not reflect modern concepts. Similarly, bridges were severely damaged, abutments were broken and some high-level bridge decks were flipped completely off their T-shaped supports to land on other highways beneath in some of the more complex freeway interchanges. They also were not designed to withstand earthquakes. Only the occurrence of that earthquake during the early morning prevented severe loss of life.

Earthquakes also have a profound effect on the earth itself. The San Fernando Earthquake generated a number of small landslides which blocked highways and obstructed small streams. The famous Good Friday Earthquake, which occurred in Alaska in 1964, probably produced its greatest damage by the landslides which were triggered by the earth motion. Large areas of residential Anchorage slide 1/4 to 1/2 mile toward the sea, although the ground surface was relatively flat (except for a cliff above the shore), Fig. 10.

The earth itself may be changed: soil fissures reflect the movement of the underlying rocks. There may be vertical and horizontal displacements of the ground which shift fence posts, break roadways and buckle railroad rails. Loose sands may momentarily become liquefied allowing the buildings to settle and hollow buried tanks to float to the ground surface.

Some of the most vivid legends of ancient man are those of the damage and terror generated by earthquake activity where the "solid ground" suddenly became mobile and unsupporting. Thus, the real danger of structural damage due to earthquakes is augmented by the psychological fear generated by loss of the groud's stability. Thus, while on the average, very few people are killed every year by earthquakes in the United States (compared to the number of people killed by automobiles), the public becomes far more excited with the risk of earthquake damage than it is with the risk of loss of life due to auto accidents. This fear in the public must be reflected in the conservatism in the geotechnical evaluation of sites for the potential ground motion during earthquakes.

4.3 Ground Motion

The motion of the ground during an earthquake is quite complex. Under pressure the earth's crust warps until it cracks like a green stick breaks upon bending. The cracking may be concentrated at a point or it may travel along a surface of weakness that already exists in the ground, such as a previous earthquake fault. The earth may move upward or downward laterally. The sudden release of the accumulated warping strain produces shaking within the rocks. These vibrations take the form of energy waves: pressure waves, like sound waves, that radiate out from the source of strain release and shear waves in which the earth moves sideways in a shaking motion.

Of these two forms of major energy, the shear waves are the greatest engineering problem. The ground motion radiates outward from the earthquake epicenter through the hard earth's crust. The shear waves are confined to the rigid crust because the plastic inner core of the earth will not withstand shear stresses. As the energy propagates outward, the earth shakes horizontally. This is shown in plan in Fig. 11. The ground surface moves at right angles to the earthquake wave travel and then reverses direction and moves in the opposite direction. This reversal occurs many times as the earthquake wave passes a point. A structure on the surface of the ground is first wracked in one direction and then the opposite direction as can be seen in Fig. 11. The movement is seldom as regular as depicted in the idealized diagram. Viewed in cross-section (Fig. 11-lower), the successive movements of a building during the passage of an earthquake at a point are shown perpendicular to the direction of the wave propagation. At one instant the ground moves in one direction away from its original position. The ground surface stretches forming tension cracks. Because of inertia, the lower portion of the building moves before the upper stories commence motion. Therefore, the building is wracked vertically in one direction. However, as the wave passes, the ground motion reverses. The soil tension is replaced by compression. The building is wracked in the opposite direction causing a change in curvature in the vertical direction and reversed distortion. Because of the compressive stresses in the soil, there may be a slight rise in the ground surface contrasting with the previous slight subsidence in the tension zones as the wave passes. Finally, the top of the building reverses its horizontal movement shortly after the bottom of the building shortly after the bottom of the building has started to move in the opposite direction. The form of motion developed vertically

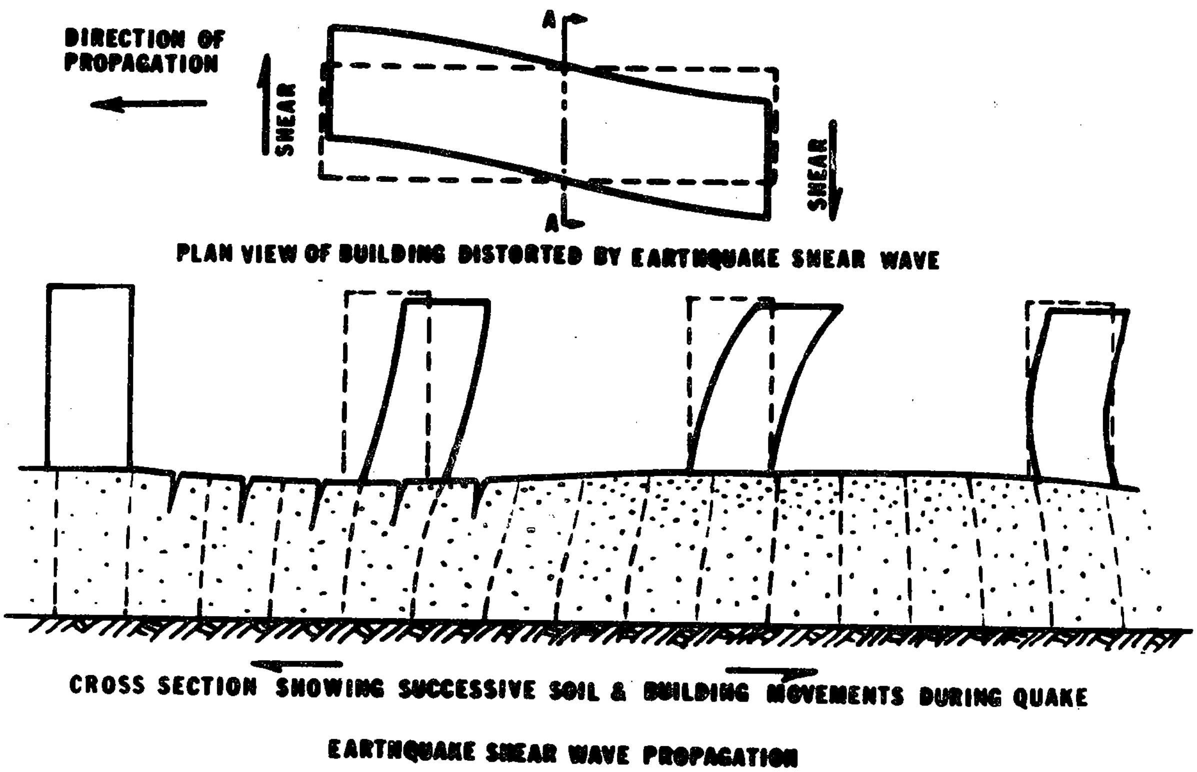


Fig. 11 Ground Motion during an earthquake

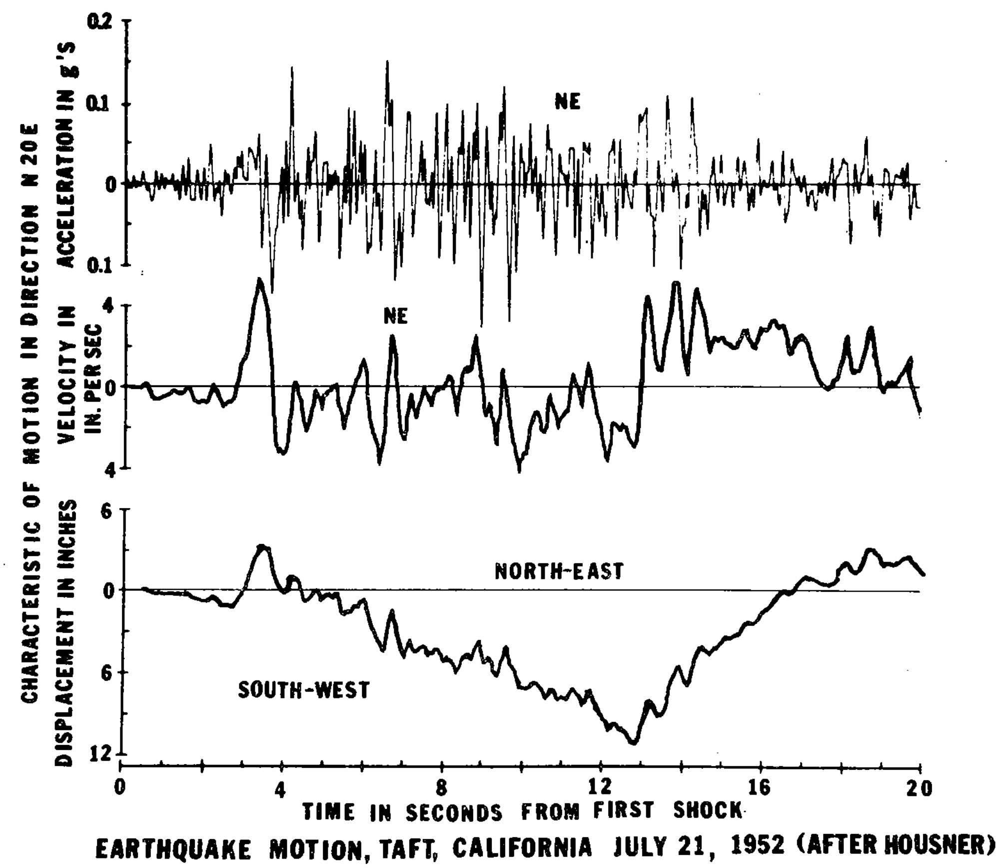


Fig. 12 Time history of ground motion during earthquake.

in the building is similar to the whipping of a slender fishing pole held vertically in the hand and quickly moved back and forward in the horizontal direction. Thus, the shear waves of the earthquake impose complex severe strains and stresses on structures that are supported on the ground surface.

The motion of the upper stories of the building may be even larger than the motion of the ground surface. Similarly, the motion of the soil surface can be larger than the motion of the underlying bedrock. Thus, depending on the frequency of the earthquake motion compared to the natural frequency of the soil and the building on top of it, there will be either an increase (amplification) of the earthquake motion or a decrease (attenuation) of the earthquake motion within the soil and any structure. This explains way one structure may suffer severe damage whereas a neighboring building remains totally undamaged, although both are subject to identical ground motion. It also explains why structures founded on certain geological deposits are more susceptible to damage than similar structures founded on different deposits even though both are equally distant from the earthquake epicenter.

4.4 Time-History of Motion

The ground motion that develops during an earthquake can be recorded by a strong motion seismograph. This is a special instrument which is triggered by a small predetermined level of ground motion. When that motion level is reached, the device automatically records the acceleration on a chart that moves rapidly enough that the details of the motion can be studied. Such a record from an earthquake which occurred July 21, 1952 near Taft, California is shown in Fig. 12. This record was obtained through the California Institute of Technology and had been previously published by G.W. Housner (1). The record shows that the earthquake lasted approximately 20 seconds. The components of motion were recorded in three directions at right angles to one-another; only the northeast record is depicted; the others are similar. The figure shows a period of small accelerations lasting about four seconds followed by about ten seconds of strong accelerations and then about six seconds of moderate accelerations. In this particular earthquake, accelerations approached 0.15g several times in both the northeast-southwest directions during the strong motion interval of the earthquake.

The ground velocities were obtained by integration of the acceleration record. Velocities of about 4 in. per second were reached several times in the northeast direction. The displacement or total movement of the ground surface was obtained by integrating the velocity diagram. Most of the movement was toward the southwest and eventually approached 12 in. toward the end of the period of strongest ground motion. At the end of the earthquake, the ground had returned to approximately its initial position: a net displacement of 0.

As can be seen from the record, the movements are quite irregular; yet a definite pattern of repeating frequencies can be seen in the acceleration record showing that certain frequencies predominated during the earthquake.

Although each earthquake has its own distinct pattern of movement, nearly all of the strong motion records obtained throughout the world exhibit similar patterns. Most of the strong motion records have come from California and Japan, with a scattering from other parts of the world in which large earthquakes occur frequently. There are presently no strong motion records for earthquakes in those parts of the United States in which only infrequent earthquakes of moderate intensity are observed. However, it is reasonable to presume that these earthquakes do not exhibit more severe patterns of motion than those in California because the patterns of damage have been no worse.

4.5 Southeastern Earthquakes

The general pattern of Southeastern earthquakes is shown in Fig. 8. This is based on the records of the U.S. Geological Survey supplemented by interviews and contemporary accounts gathered by engineers and geologists of the Law Engineering Testing Company. The locations of the epicenters of earthquakes are determined most accurately by high amplification seismographs. These are entirely different than the strong motion instruments previously mentioned. Instead, they operate continuously with sufficiently high amplifications that they even record quarry blasts, nuclear explosions in Asia and, small earthquakes at great distances. They record the time that the earthquake wave first reaches the seismograph station. If the time at which the earthquake occurred is known, the distance of the earthquake from the seismograph can be calculated from the time of travel of the waves through the earth. When an earthquake is recorded by several stations, both the time of its occurrence and its epicentral location can be calculated with reasonable accuracy. If the seismographs are sufficiently close to the epicenter, it is even possible to calculate the depth in the ground at which the motion or fracture first occurred.

Because of the infrequent occurrence of earthquakes in the eastern United States, there had been little interest in high amplification seismographsstations until after World War II. Those stations which had been established previously were in widely scattered universities and did not operate continuously. As late as 1973 there were only a few stations in the Southeast that were operating continuously enough that accurate locations of

Table 1

EARTHQUAKE INTENSITY - MODIFIED MERCALLI (1931)

Intensity	Characteristics	Acceleration
I	Not felt; animals uneasy	0.00.-0.05g
II	Barely felt by people at rest	
III	Felt indoors, like passing truck; some hanging objects swing	0.003-0.01
IV	Like passing heavy trucks; hanging objects swing, autos rock, windows, dishes, etc. rattle, walls creak	.007g-.02
V	Felt outdoors, sleepers awakened, small objects upset, doors swing open or close	0.015-0.04
VI	Felt by all. People frightened and run outdoors. Dishes, glassware broken. Furniture moved, overturned. Weak masonry cracked, small bells ring.	0.03-0.1
VII	Difficult to stand; noticed in moving autos, furniture, masonry broken, chimneys break at roof line, ponds become turbid.	0.07-0.2
VIII	Auto steering affected; masonry damage, some collapse, frame houses move on foundations; twisting and fall of large chimneys, branches broken from trees	0.17-.5
IX	General panic. Masonry damaged or destroyed; frame buildings wracked, underground pipes broken; sand boils, soil cracks	0.35-1g
X	Most masonry buildings destroyed; wood frame buildings destroyed, landslides, railroad rails bent	0.8-2
XI	Railroad rails badly bent; underground piping destroyed	
XII	Total damage; objects thrown into air	

moderate-size earthquakes could be obtained. Even today the network of seismographs is not adequate to determine the locations of very small earthquakes (except those that by chance are close to the seismograph stations).

Interest in a more extensive network of seismograph stations has been spurred by two developments. First, the need for detecting foreign nuclear explosions encouraged the U.S. Government to establish a world-wide network of seismograph stations capable of detecting and pinpointing these explosions. Second, the interest in earthquakes generated by the critical demands of modern large structures including nuclear power plants has refocused attention on earthquakes in the southeastern United States. Third, the interest of the general public has been awakened by the great damage produced in the Good Friday Earthquake in Alaska in 1964 and the San Fernando Valley Earthquake of January 1972. Hopefully, within the next decade, there will be enough high amplification seismographs in the southeastern United States that even the smaller earthquakes can be located accurately and any patterns of earthquake activity can be established.

4.6 Southeastern Epicenters

Most of the epicenters in the southeastern United States shown on Fig. 8 have been estimated from the effects of the earthquake felt by people and damage to structures. The effect of the earthquake is described by its "intensity" on a scale of I to XII (or more). The scale is not linear. Each increasing unit of intensity represents approximately a doubling of the earthquake's acceleration. This scale, in abbreviated form, is given in Table 1.

The pattern of earthquakes in the southeastern United States (Fig. 8) is seen to be scattered with some areas appearing to have somewhat greater susceptibility than others. Most of these clusters appear in the Appalachian Mountain Region from Virginia through the Tennessee/North Carolina boundary and extending into Alabama. With two exceptions, the intensities at the epicenters are relatively small (less than VIII).

There are two areas of continuing activity. One is near Charleston, S.C. In 1888 a large earthquake shook the city and surrounding towns doing catastrophic damage. It was felt over a much wider area than its epicentral intensity (between IX and X) would suggest. For example, the church bells rang in St. Augustine, Florida. This is a significant effect over a distance of several hundred miles. There has been almost continuous earthquake activity in that area since then, although most of the earthquakes have had relatively small epicentral intensities. For some undisclosed geological reason, this localized area appears to be unusually susceptible to continuing activity.

An even larger area can be seen along the Mississippi River with its southern end at Memphis and its northern end near St. Louis. One of the largest earthquakes ever to occur in the United States took place in 1811 near the small town of New Madrid. The area was sparsely populated and, therefore, there are few reliable accounts of the effects near the epicentral location. However, this earthquake was felt throughout most of the southeastern United States. In fact, considering the epicentral effect, the earthquake was felt over a much larger area than in most other earthquakes in recorded history.

By way of contrast, most of the earthquakes which have occurred in other parts of the world are more concentrated considering the intensity of damage at the earthquake epicenter. This suggests either that the Eastern earthquakes occur at a very great depth below the ground surface or that the nature of the rock slippage or gracture is somewhat different (less concentrated) than earthquakes which occur elsewhere. Particularly, there are two bits of conflicting evidence. The depth of the earthquake slippage deduced from a limited number of after shocks about 10-15Km, however, the large areas over which earthquakes of modest epicentral intensity are felt would suggest that the depth of the focus might be many tens of kilometers. This contradiction might be explained by a horizontal slippage between plates making up the shallower portions of the earth's crust, such as the slippage between leaves of a book when the book is bent. Such faults, parallel to the ground surface, termed "sole faults" are believed to exist on the basis of geophysical evidence. It is possible that the Eastern earthquakes are related to adjustments of strains along these faults and that the earthquakes, therefore, would not be accompanied by any surface expression of the fault movement. Certainly this hypothesis fits the observed facts.

In the western United States and other seismic belts, the significant earthquakes have been found to be related to faults that can be seen at the ground surface. In other words, the continuous warping within the earth's crust is relieved by cracking and slipping that extends to the ground surface. No such relationship has ever been observed in the eastern United States. There is no known fault movement that has accompanied any Eastern earthquake, although the eastern United States contains innumerable ancient faults. This tends to confirm the hypothesis of a mechanism of slippage far below the surface of the earth. Moreover, no definite relationship has been demonstrated between earthquakes in the eastern United States and any particular geologic structure expressed at the ground surface. Of course, it might be argued that the greater number of earthquakes in the Appalachian Mountain Region is a reflection of structure. This is probably true. However, there has been no correlation between the earthquakes and specific fault structures that has been observed. Many such correlations have been suggested based on various topographic features and on the drawing of straight lines or arcs connecting various earthquake epicenters and relating those lines to geologic structure. So far, all are hypothetical and have not been verified by observed ground fracturing.

Because of the increased importance of earthquake engineering, the U.S. Geological Survey has established more closely-spaced networks of strong motion seismographs in the vicinity of Charleston and New Madrid. The purpose of these networks is to determine if the seismic activity in these areas is widespread and occurs along some line or in a pattern suggesting an unknown structure. Although the final evaluation of these studies must await accumulation of more data, preliminary studies indicate that both the Charleston area and the Mississippi Valley/New Madrid areas are isolated zones of elevated activity which are not related to the remainder of the southeastern United States.

4.7 Evaluating Earthquakes in the Southeast

Because of the lack of detailed instrumentation, most of the earthquakes in the southeastern United States have been located and their intensities evaluated based on the observed effects of the earthquakes on man and structures. The basis for this evaluation is the Modified Mercalli Scale of 1931. (This scale, in an abbreviated form, is shown in Table 1.) Although this may appear to be crude and subjective, it may have more engineering significance than some of the instrument-determined ratings in areas where high amplification seismographs are located, because ultimately, the engineering rating of an earthquake must be based on its effect on man and his works. For example, earthquake intensities of less than IV are just noticeable by man. Intensities of VII produce minor structural damage, such as cracks in walls, but fortunately not damage of great engineering significance. Intensities of VIII produce structural damage which is costly to repair. Intensities of IX to X are hazardous to life. Ultimately, the combined effect of such ratings gives a realistic appraisal of the true risk from such an earthquake.

The effect observed by any one person is subject to prejudice and thus may be in error depending on the bias, ignorance or sensitivity of that person or the person who converts this to an "Intensity Rating". The exposure of persons to earthquakes may influence their sensitivity to such a subjective rating. For example, persons living east of Los Angeles (40 miles from the epicenter) felt the San Fernando Earthquake as one of many similar moderate earthquakes which occur with great regularity in southern California. One man taking a shower felt the shock and merely assumed that his wife had fallen out of bed. It was not until after he finished his shower and questioned his wife did he realize he had felt an earthquake. Yet, he was within the region in which the intensity was IV to V. By way of contrast, when an earthquake

Fig. 13a High stacks of baby food on store shelves near epicenter of Wallha quake of 1972, top jars fell. Same effect could be produced by jumping on floor.

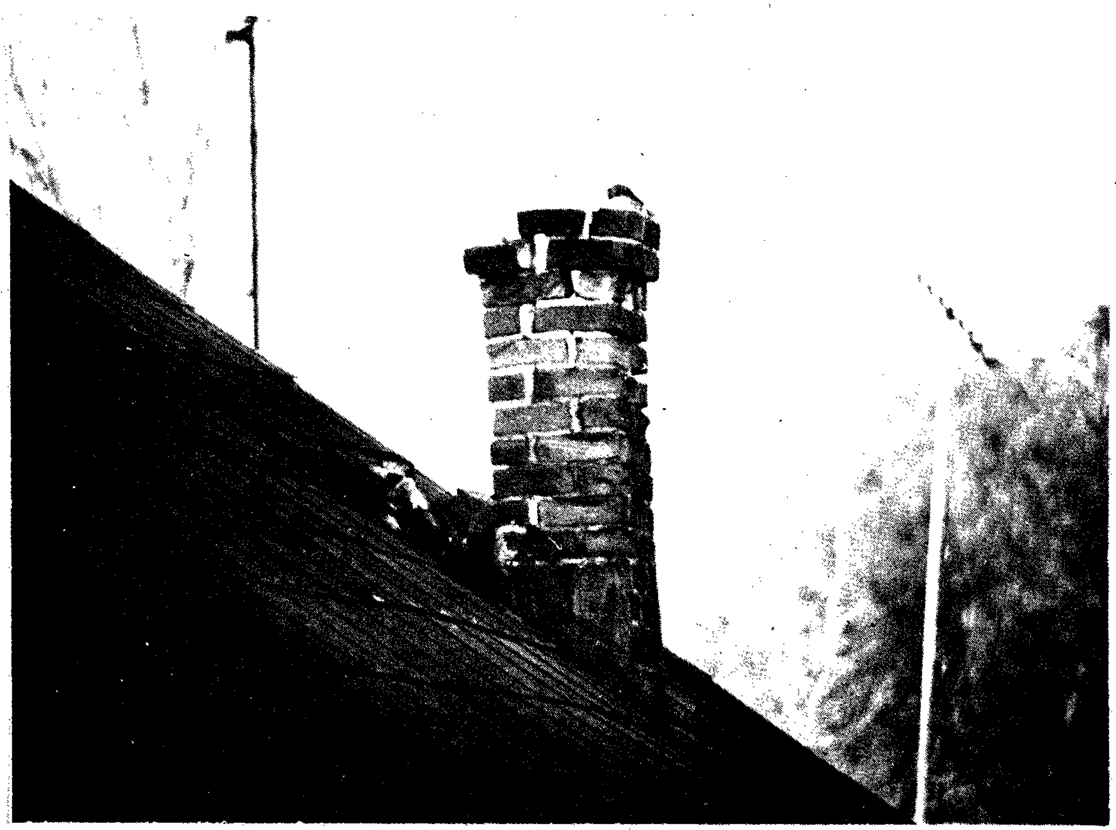


Fig. 13b Old chimney in Newry S. C. (7 m E.of epicenter) that broke from disrepair months before the earthquake and remained unchanged during the earthquake.

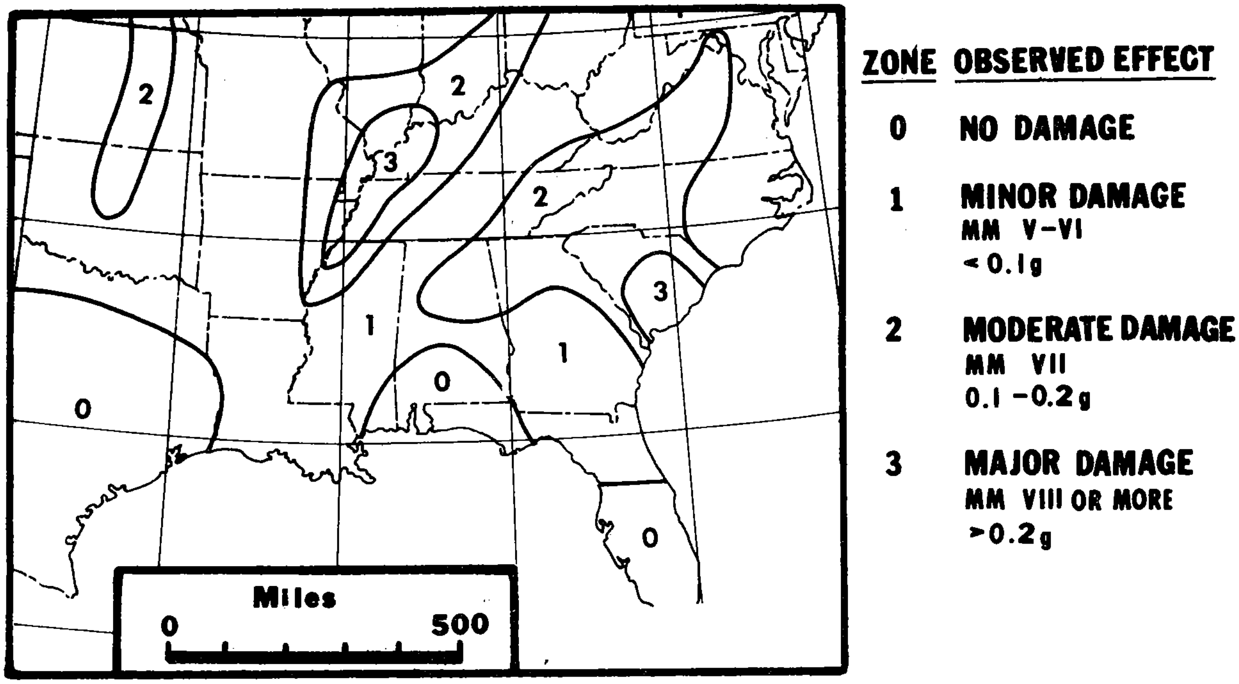
of similar intensity is felt in the southeastern United States, it is the subject of great apprehension and much discussion. In other words, in a subjective rating, that which is commonplace tends to be downgraded; that which is unusual tends to be overrated.

The example of earthquake rating by the subjective process is the small quake which occurred between the towns of Seneca and Walhala, S.C. in April 1972. The earthquake was felt over a radius of about 40 miles. Although it took place early in the morning, it was in a rural area where people arise early- many were seated at breakfast or were doing farm chores when the earthquake was felt. Since it was so unusual, people searched for definite indications of earthquake damage. A team of observers from the Duke Power Co. and Law Engineering Testing Co. interviewed numerous persons in the area, particularly those in locations were people congregate, such as country stores, country gas stations and similar public spots. Data were obtained from public officials who are accustomed to dealing with damage and disasters, such as police and fire departments. Some of the evidence collected can be seen in Fig. 13.

It is evident that an earthquake rating based on subjective application of the Modified Mercalli Scale (or any other scale) requires in-depth personal interviews of those who felt the earthquake and a realistic evaluation of the ground motion in terms of the sensitivity of persons involved and the quality of the structures which may have been damaged.

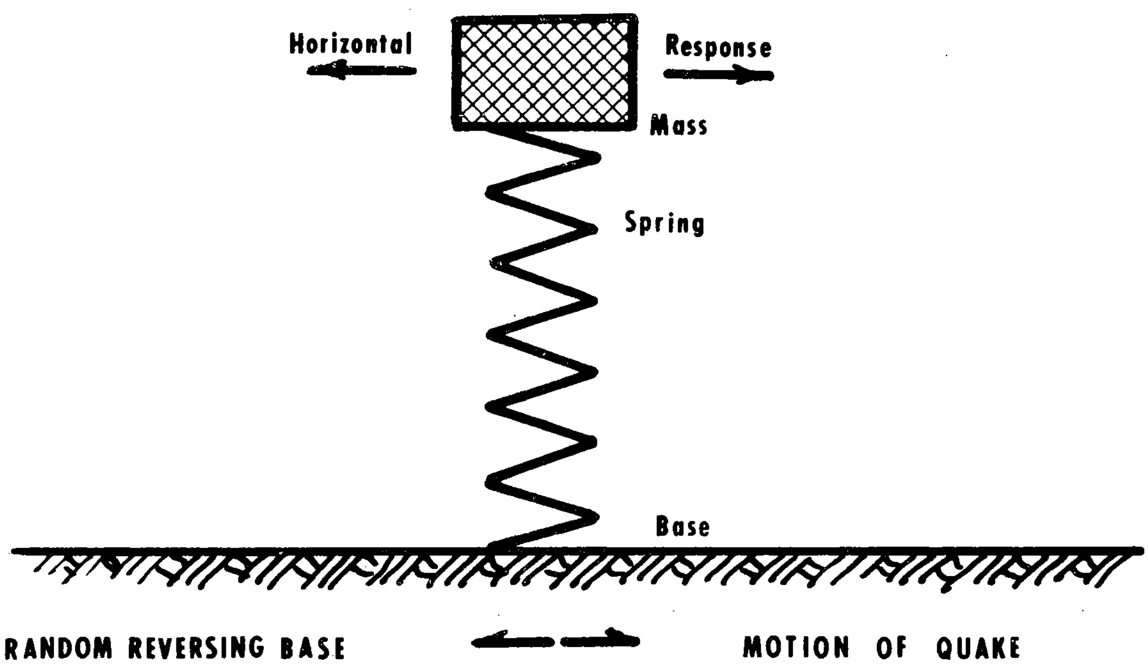
4.8 Earthquake Regionalization

Because it has not been possible to relate Southeastern earthquakes to specific geological structure, the earthquakes have been regionalized on the basis of geologic or tectonic regions in which the structure of the underlying rock is generally similar and which the patterns of seismic activity during historic times have been similar. The most recent version of such regionalization published by the U.S. Geologic Survey is shown in Fig. 14. This is a portion of the U.S. Regional Map by Algermisson. Although the map was originally entitled "Earthquake Risk Map", it reflects the intensity of the greatest earthquakes. In areas depicted zero, there has been either no activity within historic times or the quake was so small that there has been no damage. In areas denoted I, there has been some noticeable effect on humans, but only minor damage during historic times. This is equivalent to epicentral intensities of Modified Mercalli V to VI, with accelerations less than 0.1g. In areas denoted II, there has been moderate, but not critical, damage caused by earthquakes of epicentral intensity of Modified Mercalli VII. This corresponds to maximum accelerations of between 0.1 and about 0.15g. In areas denoted III, there has been significant structural damage corresponding to earthquakes of



EARTHQUAKE EXPERIENCE ZONES, SOUTHEASTERN USA
(ADAPTED FROM ALGERMISSON AND USGS, 1969)

Fig. 14



Single Degree of Freedom Response System

Fig. 15

epicentral intensities of Modified Mercalli VII or more. The accelerations may exceed 0.2g. The zone numbers are not proportional: a rating of II does not represent twice as strong an earthquake as I; a rating of III is not 1.5 times a rating of II. Instead, it suggests that in zone II moderate damage could occur and that it is necessary to take precautions for earthquake design of large structures. In areas denoted III, where major damage has occurred, detailed dynamic analyses are necessary for the safety of most structures. However, because of the complexity of nuclear power plants, it is necessary to consider the dynamic effect of earthquakes even in areas zoned 0 and 1.

In summary, the earthquakes in the southeastern United States, with the exception of two isolated areas, are of moderate intensity and are scattered with relation to geologic structure. Because of this lack of relation to structure, seismic design is based on seismo-tectonic regions (areas in which geology is consistent with observed earthquake activity during historic times). Although most of the earthquake intensities and epicentral locations in the East have been determined by the effect of the earthquake on man and man-made structures, this approach is fundamentally sound from the engineering point of view. The essence of engineering design is to incorporate measures for preventing these effects; therefore, the best way of evaluating the effects is to observe the behavior of structures during earthquakes.

5. ENGINEERING PROPERTIES

The engineering properties of the soils and rocks beneath the site must be determined for the proper design of the plant foundation, the walss which support any structures that are deeply embedded in the ground and to determine the dynamic response of the ground during an earthquake. It is particularly critical when the bedrock at the site is overlain by either residual soils or a deep blanket of sediments which differ in their elastic properties from the rock below.

5.1 Static Tests

The ordinary static tests are utilized for the design of the foundations. These tests include triaxial and direct shear of the rock and consolidation tests of any compressible strata. Since large structures are involved which may cross many of the joint fractures, it is important that both intact portions of the residual soil and rock as well as the fractured surfaces are tested. Undisturbed samples are obtained from test trenches and from large tube samples which cut across the crack surfaces. The shear characteristics of these surfaces are then tested in the laboratory in such a way that the shear movement occurs across the pre-existing plain of weakness.

Unfortunately, the laboratory tests do not integrate the effects of the irregularities that are commonly present on joint cracks or even ancient slickensides, nor do they integrate the effects of fractures on the total mass. Therefore, large-scale, in-place testing of the soil and rock may be necessary to determine the relationship between the laboratory tests and the field behavior. While such tests are difficult and expensive, they provide a more realistic picture of the way in which the laboratory tests must be interpreted to determine the shear strength of the mass.

5.2 Site Permeability and Ground Water Flow

In-place permeability tests are necessary to determine the seepage characteristics of the ground because small-scale laboratory tests cannot possibly integrate the varying effects of pervious lenses, fractures and other discontinuities in the soil and rock mass. The tests are commonly conducted in bore holes. The holes are drilled through the particular strata in which permeability determinations are required. A crude test merely consists of filling the bore hole with water and determining the rate at which the water falls. From this an average effective permeability can be determined. The stratum in which the test is made can be isolated by the use of casing above and packers or plugs in the hole below the level desired. More sophisticated tests are made by a similar procedure in which a constant rate of water inflow is maintained at a constant pressure. The effect on the groundwater table is monitored by small diameter observation wells placed in patterns around the inflow well. This procedure, while much more expensive and time-consuming than the simple falling water level test, provides information about a much larger portion of the soil and rock mass. It permits testing of the formation at elevations higher than the present ground-water level. If the present ground-water level is very high, the test can be reversed by pumping water out of the central well and monitoring the depression of the groundwater level in the surrounding observation wells. This procedure is limited because some of the critical formations may be unwatered by the test itself. In any of these flow tests, it is important that the water pressure gradients involved are not so high that the formation is disrupted by the water pressure. If this should happen, the results will not be indicative of underground seepage.

The present flow of the ground water at the site is important in that any loss of liquid nuclear materials could contaminate ground water or eventually reach streams and thus contaminate rivers. Generally, a large-scale pattern of observation wells is established at the site and within a half mile or a mile of the prospective plant location. From these observation wells the ground water flow patterns can be established. If there are alternating pervious and impervious layers, there may be several aquifers each of which has a different water pressure level. This requires a careful identification of the pervious and impervious strata from the core boring records.

There are two major purposes in making these seepage studies. The first is to determine the impact of ground water lowering during plant construction on the adjacent aquifers. For example, in many rural areas, the water supplies of small farms are dependent on the high level surficial aquifers. Unwatering for a structure that extends 100 ft below the ground surface could dry up wells and create severe hardship and losses for nearby farms and homes. The second and more critical problem is the possible loss of liquid nuclear materials into the ground and their contamination of the ground water. The problem is alleviated by the fact that some of the nuclear ions are absorbed and retained by the soil. This occurs by ion adsorption and ion exchange. Most of these studies have shown that percolation through soil for distances comparable to the area of the typical nuclear plant site are sufficient to remove the radioactive wastes and prevent ground water contamination.

A major exception to this would be where there are large underground openings, such as limestone caves or the porous zones in vesicular basalt in which there would be little or no ion exchange or ion adsorption.

5.3 Dynamic Properties

The vibration of the site during an earthquake is governed by the elastic properties of the soil. Furthermore, the response of soil structures, such as embankments, fills and small dams is determined by their elastic properties. Because of the difference between their elastic properties and those of the rock beneath, the earthquake ground motion transmitted through the bedrock can be either amplified or attenuated in the soil and soil structures above.

In order to determine the soil behavior during earthquakes, studies of the dynamic properties of the soil and rock are necessary. Refraction seismic exploration is a useful tool in nuclear plant investigation in two ways. First, it provides information on the boundaries between successively harder strata which can supplement data of the direct boring procedures. For example, in one nuclear site in the Piedmont, seismic refraction lines were extended a half mile in each of four directions from the plant site to determine if there were unusual variations in the rock-soil boundary which should be investigated in more detail by direct borings. The seismic exploration over these long distances was completed in a much shorter time and cheaper than borings. Second, the refraction seismic data determines the compression wave velocity of the soil and rock formations. If the density of the soil is determined by direct sampling, the elasticity incorporates the effect of the defects or geologic discontinuities within the mass whereas the laboratory tests for the elastic modulus only involve the intact material subjected to the testing. There is a serious drawback to the determination of the elastic modulus from the seismic refraction. The amount of ground movement strain in the refraction testing is very small compared to the strain level in an earthquake. Since the elasticity is dependent on the amount of motion, an elastic constant determined by the seismic refraction test is higher than that effective during earthquakes. Thus, this data should be considered an upper limit for the elastic properties.

Laboratory tests do not include the total effects of the soil and rock stratification. Therefore, their results constitute a lower limit for the dynamic properties of the soil and rock materials. By using the laboratory tests to establish a lower limit at large strain levels and the geophysical tests to determine the upper limit at small strain levels, it is possible to determine the range of dynamic properties which will be involved in the soil and rock during an earthquake of any level.

6. DESIGN CRITERIA

The design criteria for a nuclear power plant includes the basic requirements of safety against foundation failure, freedom from excessive settlement and freedom from disruption from environmental changes, including the effects of ground water fluctuations and seismo-tectonic activity. They are really no different from those for any other major engineering structure whose damage or destruction would produce severe changes in the environment or endanger life and property. However, because of some of the uncertainties involved and the limited experience with nuclear power plants as well as the general fear of the public for anything encompassing the work "nuclear", criteria applied to these basic foundation design requirements are somewhat more stringent than those applied to engineering structures of equal risk to the public, such as large dams.

6.1 Distance from Capable Faults

Movement along an active fault, such as the San Andreas Fault, could cause damage to any structure, including a nuclear power plant. Although there are a number of buildings directly astride the San Andreas Fault or its branches, including a large winery (and a portion of the University of California at Berkeley Campus), it certainly is imprudent with today's technical knowledge to found a structure on a fault that is likely to move. Such a fault has been defined by the AEC as a "capable fault".

This is a fault which has shown evidence of a movement during the past 35,000 years or evidence of two or more movements during the past 500,000 years. The distance from a critical structure from such a fault depends on whether the fault is a well-defined narrow plane of fracture or whether it is a fracture zone. This must be determined by indirect evidence, such as stream shifts and ground offsets and by direct observations from changes in level of marker beds in borings. Generally, the critical structures should be more than 3/4 mile from a zone that is capable of undergoing movement during the foreseeable life of the plant - the real significance of the term "capable fault".

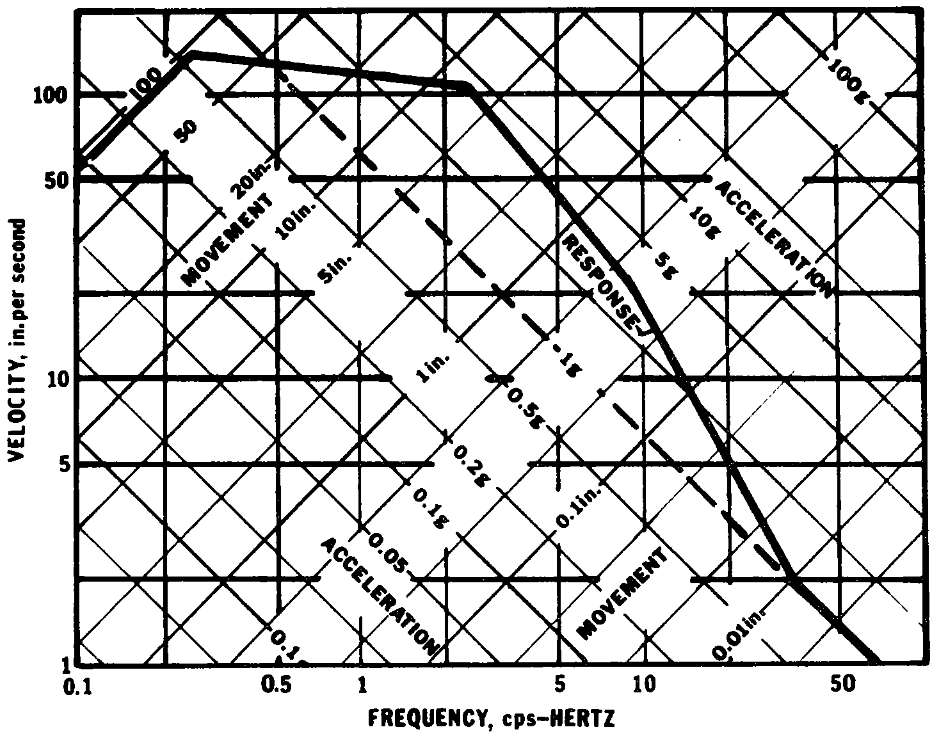
6.2 Design Acceleration

A nuclear power plant must be designed so that it can be shut down safely during the largest earthquake which can reasonably be expected to occur at the site. A number of different approaches have been taken to determine the size of this safe shutdown earthquake for the dynamic design of all of the critical structural elements within the plant. In the western half of the United States, earthquakes are associated with faults of known activity. For design it is assumed that the largest earthquake of record whould occur at any point along the fault which it has previously taken place. It is further assumed that this maximum earthquake will occur at the point on the fault that is closest to the plant site. Various curves relating earthquake motion as a function of distance from the epicenter on its causative fault have been established for the western United States. These are used to compute the earthquake's effect at the site.

In the eastern United States the problem is less determinate, somewhat simplier and more conservative. As has been mentioned previously, the eastern United States can be divided into zones in which the tectonic activity during historic times has been reasonably consistent. It is assumed that the largest earthquake that has occurred within such a seismo-tectonic zone could be expected to occur anywhere within it. Thus, in effect, the largest earthquake in the plant's seismo-tectionic region is presumed to occur at the plant site. As a further measure of conservatism, it has been considered prudent that the largest earthquake which can reasonably be expected to occur in the future has an acceleration 1.2 to 1.5 times greater than the largest earthquake which has been observed.

6.3 Duration of Earthquake

There are no strong motion earthquake records for the eastern United States. The duration of the earthquakes, therefore, must be based on the subjective observations of people during such



RESPONSE SPECTRA OF SINGLE DEGREE OF FREEDOM STRUCTURAL SYSTEM AT 2% DAMPING TO EARTHQUAKE OF 1g HORIZONTAL GROUND ACCELERATION

Fig. 16 Typical response spectra of a design earthquake earthquakes. Typically they are similar to duration to those of comparable effect on the West Coast - 15 to 20 seconds in total length. The earthquakes may be accompanied by irregular foreshocks which warn of the impending earthquake and by after-shocks occurring at irregular intervals for several days afterwards. Both the foreshocks and aftershocks are of much smaller intensity than the main event. In the absence of instrumentally determined data, it is assumed that the earthquakes in the eastern United States are of similar duration (15 to 20 seconds).

6.4 Earthquake Motion

The movements experienced during an earthquake are quite complex as can be seen in Fig. 11. Such strong motion records have been obtained in the western United States, in Japan, in India and other areas subject to frequent, large earthquakes. These records display a random motion in which there is an initial interval of a few seconds increase in acceleration. This is immediately followed by an interval of intense motion in which the accelerations vary considerably including several episodes which are greater than half of the maximum. After 4 or 5 seconds of intense motion, there is a somewhat larger interval of declining motion.

Strong motion records have been collected by such investigators as Newmark, Blume, Housner and others from various parts of the world. These records have been combined to produce an envelope that encompasses the worst of all of the earthquakes which have been instrumented anywhere within the world.

The earthquake motion can be expressed in a number of ways. One is a time history of acceleration or ground motion or ground velocity or ground displacement, such as shown in Fig. 12. However, a more significant expression is the effect of the motion on an engineering structure. The simpliest form of structure consists of a weight supported by a spring in which a single degree of motion in the direction of the earthquake motion develops. Although the earthquake motion occurs at varying irregular frequencies, there are certain impulses which occur at reocurring intervals defining the "predominant frequency". If this predominant frequency should be equal to the natural frequency of the spring-mass system, resonance results. Depending on the damping, amplitude of motion of the mass will exceed the ground motion: a condition termed "amplification". Thus, from the engineering point of view, the most meaningful way of expressing the motion of an earthquake is in terms of the response of such a single degree of freedom system (as shown in Fig. 15) with varying degrees of damping. It is presumed that the natural frequency of the spring-mass system matches the most critical or predominant frequency of the earthquake. The plot of earthquake ground motion applied to such a variable spring-mass system is shown in Fig. 16.

This is a tri-part logarithmic plot which shows the displacement of the mass, the maximum velocity of the mass and the maximum acceleration for different natural frequencies of the spring-mass system. It permits evaluating any earthquake in terms of its effects on a simple structure. If the structure has a damping of 100 percent of the critical, the maximum acceleration is equal to the maximum acceleration of the earthquake and there is no amplification; if the damping is less (depending on the soil-structural system represented by the single degree of spring-mass system), the maximum acceleration, velocities and displacements may be amplified. The effects of different earthquakes are often compared in terms of the response of a single degree of freedom system with damping of 2 percent (representing many engineering structures) or 10 percent (representing soil-rock systems).

In areas in which there are a number of strong motion records, the design of the engineering structures may be based on the time-history records of several large nearby earthquakes, expanded or reduced (normalized) to the maximum acceleration to be expected at the site in question. In the eastern United States, where very few strong motion records have been observed, synthetic earthquake records involving random ground motion resembling observed earthquakes are utilized. These synthetic earthquakes produce a response in a single degree of freedom system that is at least as great as the combined envelope of the observed largest earthquake records obtained throughout the world, normalized to the peak accelerations of the design earthquake for the particular site.

6.5 Site Amplification and Attenuation

Because of the variable effects of soils on earthquake motion, the most consistent, reliable data on ground motion have been obtained from instruments supported on rock or other firm ground that is not underlain by weak or resilient materials. Thus, the design earthquakes (time histories from compiled observations or from theoretical random motion that equals or exceeds the combined envelope of strong motion previously mentioned) refer to the motion of such a firm base. From the engineering design point of view, it is desirable to place the foundations of heavy structures (such as the reactor containment) on such a firm base. There are situations where this is impractical or impossible.

The base motion of the firm gravel or rock is altered as it propagates through the soil above. The change in the motion depends on the stiffness of the soil compared to that of the base, the thickness of the soil and the damping of the system. These properties determine the "fundamental period" of the soil column. If the period is close to the "predominant period" of the earthquake, it is possible that the earthquake will produce motion at the soil surface than at the firm base (or on a nearby rock outcrop).

A soil column, typically 30 to 60 ft thick, of moderate density and regidity, may have a fundamental period that falls within the range of the predominant frequencies of a typical earthquake. In such cases, the earthquake motion is amplified (resonance). During a small earthquake the ground motion is small and the corresponding damping is slight. Therefore, amplifications at resonance will be large. By way of contrast, in a large earthquake the motion will be great enough to produce large strains, plastic movement of the soil and much more hysteresis damping. The amplification of the large earthquake will be less than that of a small one because of the greater damping.

If the soil is very thick, the fundamental period of the soil column can be much longer than the predominant earthquake frequencies. Depending on the damping of the soil, the motion at the ground surface will generally be less than that at the firm base. The motion is attenuated by the soil, which is analogous to the cushioning of a small object placed in a box of sawdust. Thus, it is not possible to determine in advance whether the soil mass will amplify or attenuate the earthquake.

The problem may be compounded by the distance from the epicenter of the earthquake. The greater the distance, the smaller the base motion at the site. In addition, the higher frequencies of the earthquake are more severly attenuated than the lower frequencies. These lower frequencies, in some instances, define longer predominant periods at great distances. These longer periods may fall in the range of the fundamental frequencies of thicker soil deposits. Thus, it is theoretically possible that a distant earthquake whose motion in bedrock at the site is quite small may be amplified so as to produce a greater effect at the ground surface than a closer earthquake whose bedrock motion at the same location is of greater intensity, but of a shorter predominant frequency. While this possibility has been suspected in some earthquakes, there is no evidence that this has actually occurred. Regardless, all possibilities of amplification and attenuation must be considered for each critical structure before a final design can be established.

6.6 Liquefaction

The effect of the earthquake motion on the strength of the soil itself is a factor at some sites. Rocks, stiff clays and dense sands do not appear to be adversely affected by earthquake motion. Loose saturated fine sand and possibly very soft clays can be weakened by the continuing reversing motion imparted by an earthquake. In some instances, loose saturated fine sands have lost their strength completely to become viscous liquids with densities equivalent to the saturated soil (100 to 110 lb per cu ft). Hill sides have suddenly flowed outward and buildings have sunk into the ground. Paradoxically, buried hollow structures float upward because of their bouyancy. If the site investigations find loose sands and soft saturated clays, their potential for liquefaction must be studied by dynamic shear testing. The soil is subjected to repeating, alternating stresses that simulate the stress reversals in the shear waves in the earthquake. The level of stress and the number of stress reversals required for a failure are compared with the level of stress and the number of cycles of strong motion of the design earthquake and a safety factor determined. If the safety factor is too low, either the offending soils must be removed or stabilized to provide a safe foundation despite the effect of the design earthquake. At one site, the St. Lucie Nuclear Power Plant (near Ft. Pierce, Florida), this entailed excavating loose sands to approximately 50 ft below sea level and replacing them with the same sands densified sufficiently that they can not liquefy.

7. CONCLUSIONS

The geotechnical design requirements for nuclear power plants are no different from those of any other critical engineering structures that involves public health and safety. However, because of the lack of long-time experience with nuclear plants, and the public fear of anything atomic, the geotechnical investigations must be unusually thorough. It is likely that the advances in technology generated by the studies made for these nuclear power plants will aid in the design of other non nuclear structures.

A number of conclusions can be drawn from the studies for nuclear power plants in the southeastern United States. First, the geologic structures are old. There is no evidence of contemporary movement of any except perhaps the Baton Rouge Fault. All the present evidence indicates that the geologic structures, including faults and joint cracks and folds, can be adequately factored into the design of the nuclear power plants.

The earthquakes in the region have been of moderate intensity (with the exception of the area between St. Louis and Memphis along the Mississippi River and the vicinity of Charleston, S.C.). These earthquakes have occurred at random locations and have only been of moderate intensities with accelerations not exceeding about 0.12g during historic times. None of these earthquakes have been found to be related to any of the known geologic structures in the region. It is possible that they are associated with different forms of tectonic movement than the earthquakes in the western United States.

Regardless, the design earthquakes are based on the more critical intense, strong motions which have been recorded in parts of the world where earthquakes occur frequently and with larger intensities. Until a significant number of strong motion records become available in the eastern United States, this conservative approach is prudent. As larger networks of high sensitivity seismographs become available to help pinpoint the epicentral location and focal depth of the earthquakes and as strong motion recorders become available to develop the time histories of the eastern earthquakes, a more realistic design approach may evolve. Nevertheless, the present technology permits a conservative, safe design for nuclear power plants to withstand the geotechnical forces they may be reasonably expected to feel during their projected lifetime.

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ERTS and Multispectral Photography

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Abstract

ERTS-1 imagery can be used to study a wide variety of phenomena. For the geologist its most apparent use is in structural studies. Lineaments are conveniently mapped in many cases, and a regionwide, synoptic view of the structural geology may be obtained. Other uses include land use studies, highway route planning, and studies related to water quality in large bodies of water.

Multispectral photography has been used only limitedly in solving geologic problems, but it has been used successfully in conjunction with vegetative studies, water quality studies, and with studies of marshes.

Introduction

Remote sensing of the earth involves a wide range of techniques and methods. Aerial photography is perhaps the best known method. This paper discusses two methods of remote sensing and some of the associated techniques. One method is satellite observations of the earth and the other is photographic.

ERTS-1

The existence of ERTS-1 in its sun synchronous polar orbit is generally well known to geologists. The multispectral scanner aboard the satellite records reflectances from the earth below in four bands of the electromagnetic spectrum:

0.5 to 0.6 microns (Band 4)

0.6 to 0.7 microns (Band 5)

0.7 to 0.8 microns (Band 6)

0.8 to 1.1 microns (Band 7).

Experience has shown that band 5, 6, and 7 are the most useful. Band 4 seems most affected by haze and seems to exhibit the least contrast. In a general way band 5 is best for distinguishing different types of land use and studying suspended matter in water bodies. Bands 6 and 7 are most useful in separating water bodies from their surroundings and for marking the limits of wet areas. Combinations of bands 5, 6, and 7 aid in recognition of various vegetative types. In some cases band 6 and 7 emphasize geologic structures better than band 5; in other cases band 5 brings out the structural information better.

Perhaps the biggest advantage of the ERTS imagery is that it is a map of the way things are, and as such it has multidisciplinary and interdisciplinary uses. An individual ERTS-1 image covers about 10,000 square miles, providing a synoptic view of an area that at this time is unobtainable otherwise.

The data are received from the satellite and processed into digital tapes and various photographic products. Several techniques exist for converting the black and white photographic products to clear images. One of these techniques is color additive viewing. By manipulating the light intensities and filters in a color additive viewer, one can emphasize one set of features and suppress others. The materials can be studied in the black and white format also.

In general, resolution on the ERTS-1 imagery is about 300 feet, although under favorable circumstances objects with dimensions smaller than 300 feet may be recognized. Long, narrow, linear features often stand out well.

Multispectral Photography

The existence of a wide range of films and filters makes possible the recording on film of selected parts of the electromagnetic spectrum. In normal aerial black and white photography, essentially all of the visible spectrum is recorded on the film. If only certain portions of the spectrum are desired, appropriate filter-film combinations may be used to restrict the energy recorded. Multispectral photography is an application of this idea. The light reflected from the earth may be broken down into bands by filters. Broad band filters may be used, and essentially all of the light may be recorded in one or another of the bands. Narrow band filters allow only very selected portions of the spectrum to reach the film; all but those selected portions of the spectrum desired may be blocked from the film. Thus, for example, by the proper choice of filters one may investigate only the green and blue parts of the spectrum, or only the blue, orange, and infrared portions.

Multispectral photography implies the simultaneous recording of a scene by several camera lenses. The lenses may be mounted on individual cameras designed to image simultaneously, or the lenses may be mounted on one camera so that the camera shutter exposes the film behind each lens essentially simultaneously. North Carolina State University's multispectral camera is of the latter type, and four simultaneous images of an individual scene are made in the blue, green, red, and near-infrared portions of the spectrum.

Applications

The question naturally arises: "How can these tools be applied to geologic problems, and specifically, how can they be applied to problems associated with highway construction and maintenance?" Perhaps the best way to answer the question is to describe briefly several ways that ERTS imagery and multispectral photography have been used or information gained from the techniques. Other possible applications may then come to mind.

ERTS-1 Imagery

If rock types can be distinguished in normal aerial photography on the basis of color and tone, they can probably be distinguished on ERTS imagery. If rock types cannot be distinguished in normal photography, they probably cannot be clearly distinguished on the ERTS imagery visually, although some photographic enhancement techniques may be useful. On the other hand, application of computer technology to the data may make some differentiations possible. Eventually, however, one must make field checks. As with normal aerial photography of humid regions, vegetative cover limits the ability to distinguish rock types. On the other hand, comparison of images from different seasons can aid in interpreting boundaries between different rock types. There can be subtle differences in reflection from the soils and rock exposures as well as differences in the vegetation. Much of the emphasis in the geologic interpretation of ERTS-1 imagery has been placed upon recognition of photo-linears and from them recognition of geologically significant lineaments. Whether a given lineament is significant or not depends upon the geologic context within which it is set. Of course, not all "lineaments" seen on the imagery can be identified in the field, and some are man-made. Since fracture patterns are believed to be good guides for mineral exploration and because many lineaments are believed to record fractures, there has been a considerable amount of work done in interpreting these features.

Figure 1 is a reproduction of part of ERTS-1 image 1568-15272-6 (February 11, 1974) showing the High Rock Lake - Badin Lake area of North Carolina. The geology of this area has been described in part by Connley (1962) and Stromquist, et al (1971). The major structural trends are readily seen in this small scale reproduction of an ERTS-1 image.

Geologic mapping of the quadrangles to the west of the Denton and Albemarle quadrangles is not yet published. However, ERTS-1 imagery gives us the opportunity to see the structural trends on a regionwide basis and to do reconnaissance mapping of the area much more rapidly than would be otherwise possible. Use of the imagery in geologic mapping at a scale of 1:125,000 seems possible.

Figure 2 (Image No. 1569-15331-6, February 12, 1974) shows the northern part of the Brevard Zone in North Carolina and structural features adjacent to it. The intersection of the Grandfather Mountain structure and the inner piedmont occurs just west of Lake James. The apparent structural control of the stream drainage pattern along the inner piedmont is also shown, especially in the vicinity of Lake James. Of interest also is the east-west topographic alignments shown in the Hickory Nut mountains northeast of Lake Lure.

One of the most successful uses of ERTS imagery has been the monitoring of the shallow sounds of North Carolina. Suspended matter is well recorded in bands 4 and 5, and current patterns and water mass distribution can be studied. Figure 3 (Image No. 1133-15150-5, December 3, 1972) and Figure 4 (Image No. 1205-15150-5) show Albemarle Sound in the northeastern part of the state at

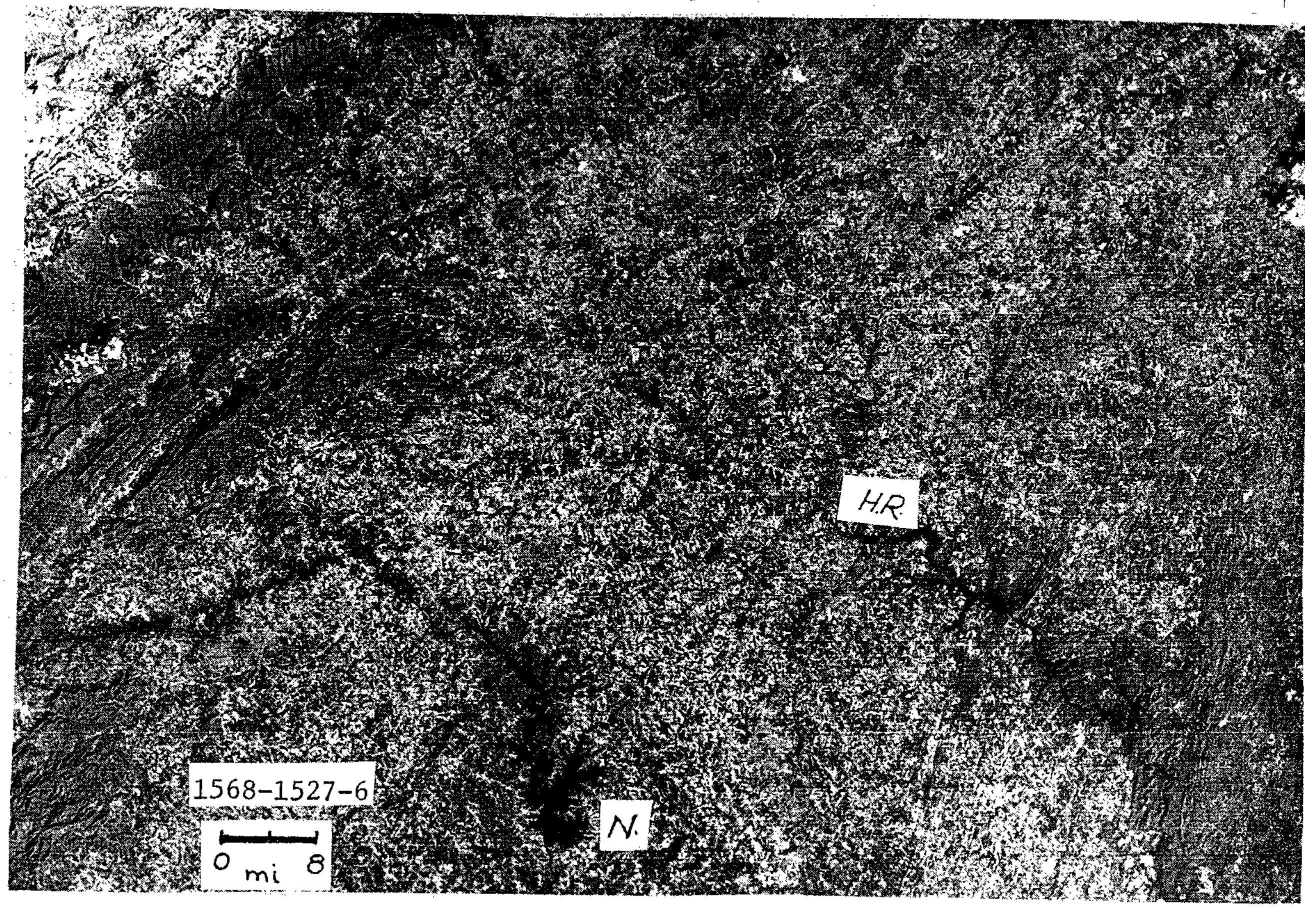

Fig. 1. ERTS Image 1568-15272-6 showing Piedmont structures. High Rock Lake (HR), Lake Norman (N).

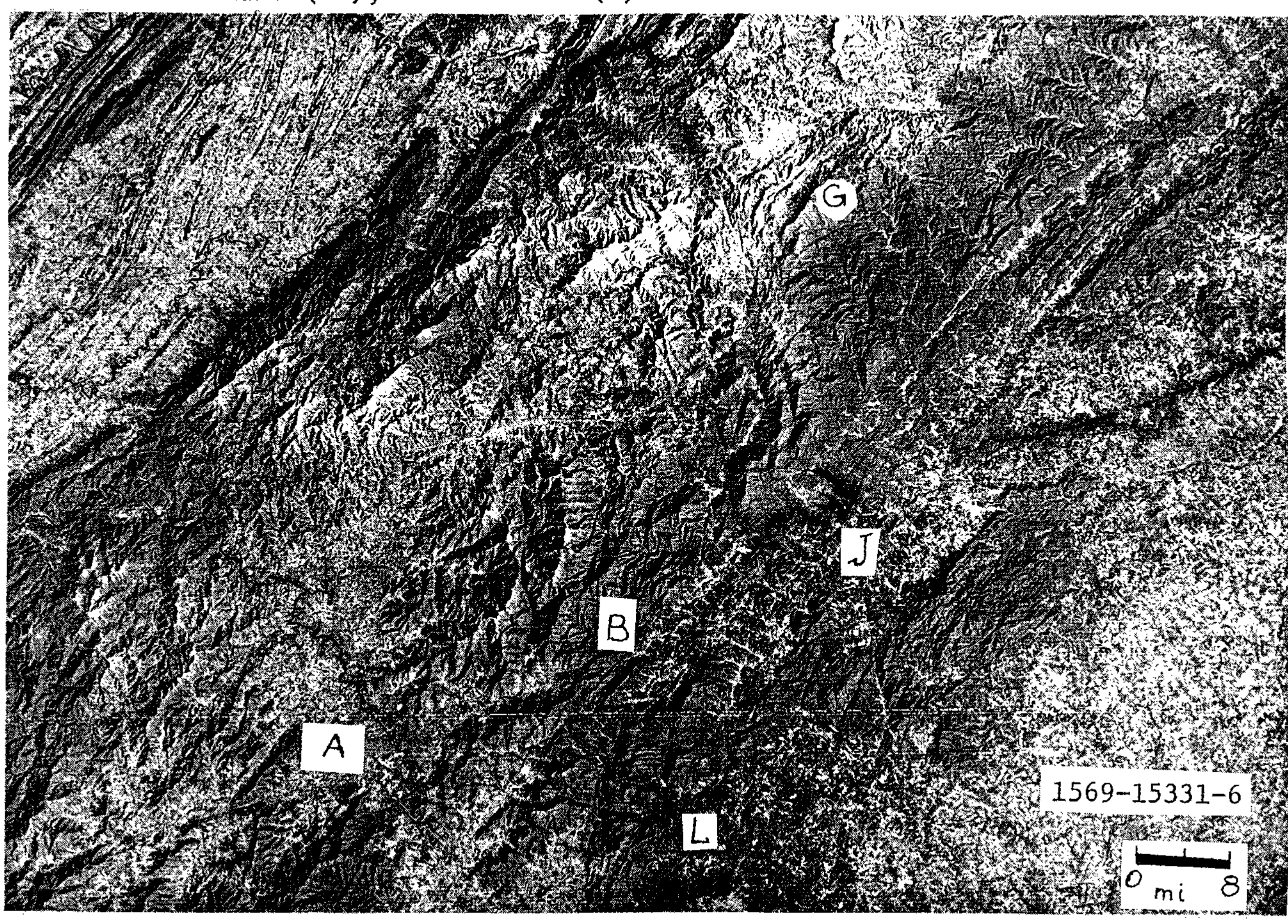

Fig. 2. ERTS-1 Image 1569-15331-6 showing northern end of Brevard Zone (B), Lake James (J), Grandfather Mtn. (G), Asheville (A), Lake Lure (L).

Fig. 3. ERTS-1 Image 1133-15150-5, December 3, 1972. Albemarle Sound, wind from southwest.

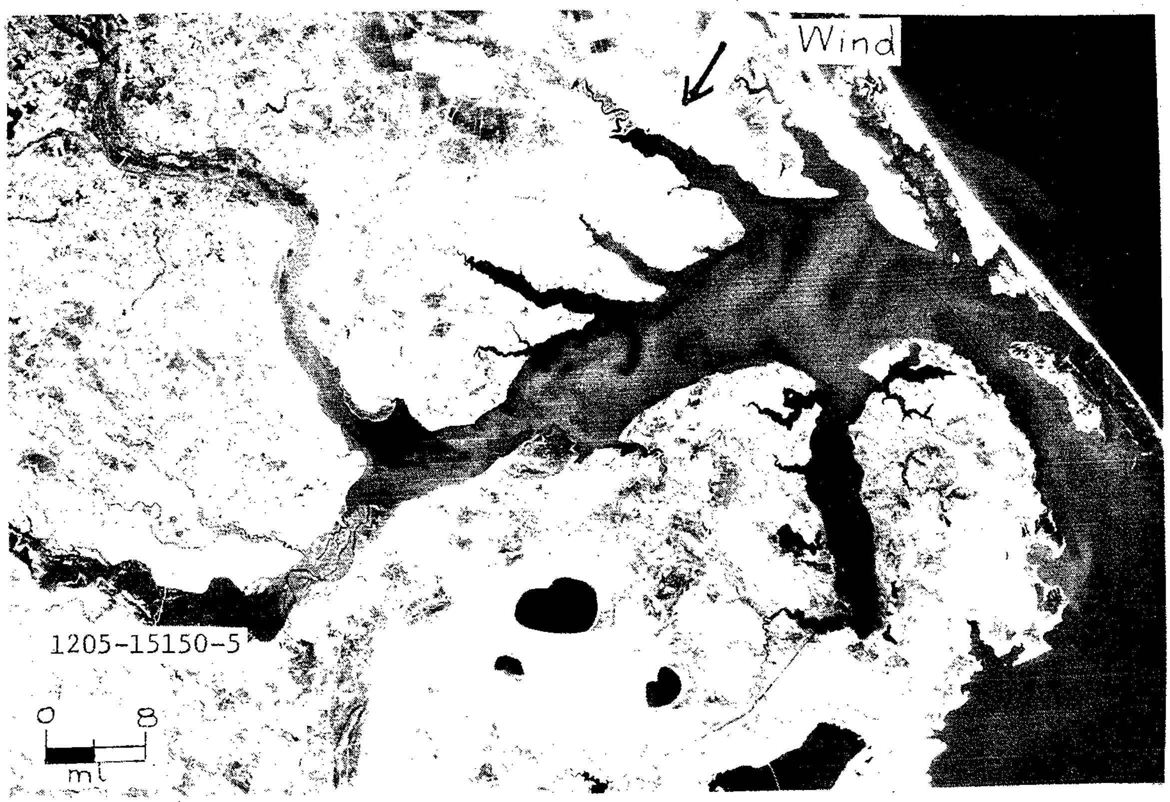

Fig. 4. ERTS-1 Image 1205-15150-5, February 13, 1973. Albemarle Sound, wind from northeast.

two different times. In Figure 3 the suspended sediment is shoved northward in response to a southwesterly wind. With the wind blowing from the northeast the suspended material is concentrated against the south shore and tongues of less turbid water extend from the Pasquotank and other rivers into the sound (Figure 4). By utilizing the facts that the several parts of the visible and near-visible parts of the electromagnetic spectrum penetrate to different depths and that the ERTS imagery is split into the four bands, it is possible to make some estimates about the vertical distribution of the suspended materials even without groundtruth. Attempts to make these determinations for the Albemarle Sound area are part of an on-going investigation into the water quality problems of the sound. The sediment load entering the larger lakes of North Carolina have been monitored by ERTS, and ideas about the major areas of erosion developed.

Details of the channels in Oregon Inlet can be seen in the enlargement made from ERTS-1 Image No. 1492-15063-5 (Figure 5). This image could probably be used to understand better the dynamics of the inlet and to guide a channel maintenance program, and the image probably represents the best navigational chart available for the area at the time it was made. Careful scrutiny shows the pattern of the flood tide delta.

Highways are concerned with the movement of people. The urbanizing piedmont cresent of North Carolina is growing along its highways, and an ERTS-1 image made in October 1973, (Image No. 1459-15235-5) illustrates this fact (Figure 6). Enlargements made from this and similar images emphasize the growth patterns of the urban areas. The coastal plain-piedmont contact and the differences in land use patterns are also shown.

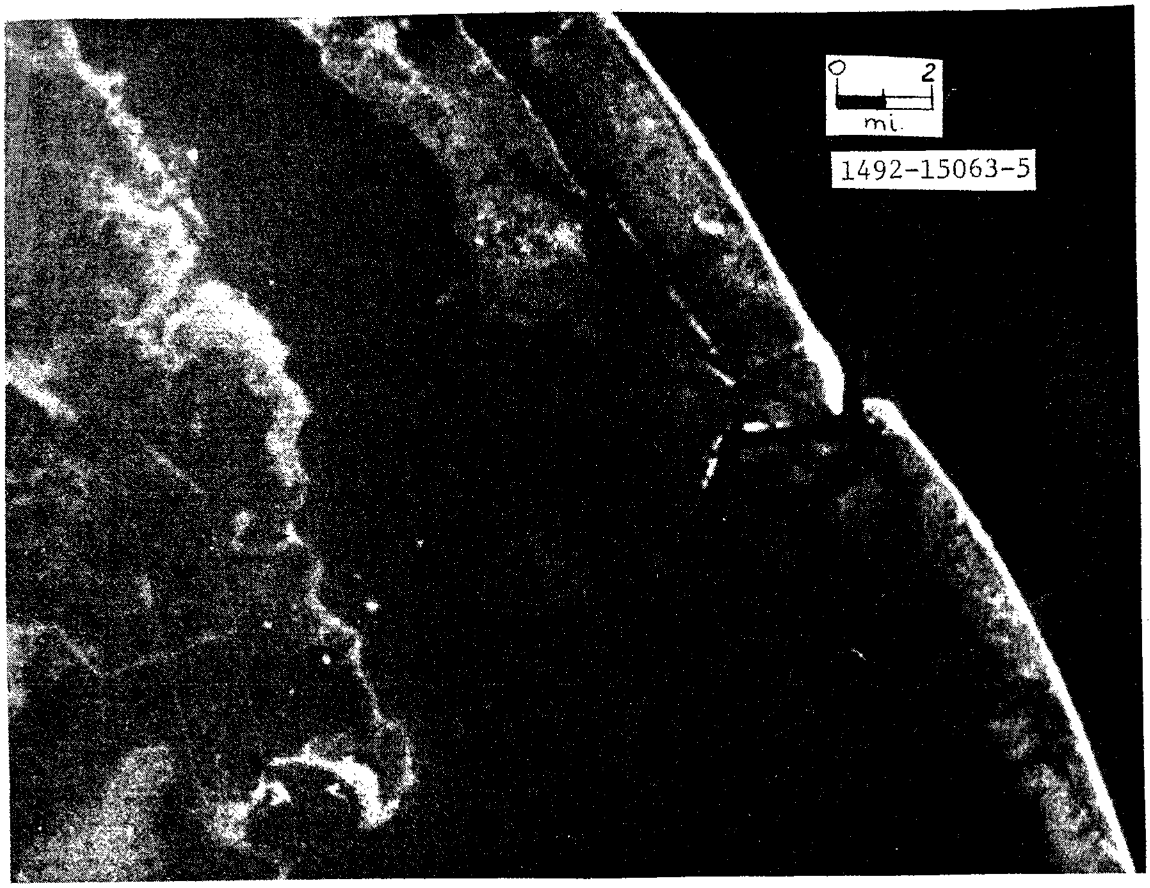

Fig. 5. ERTS-1 Image 1492-15063-5, November 27, 1973. Oregon Inlet, approximately one hour after high tide.

Fig. 6. ERTS-1 Image 1459-15235-5, October 25, 1973. Raleigh (R), Durham (D), Greensboro (G), Coastal Plain (C).

Quarries may be recognized, and their relationship to the growth of a city may be noted, a point worth emphasizing when discussing planning and zoning controls. Geologically important and environmentally sensitive areas can be monitored and evaluated in terms of the processes transpiring.

Presently there are two ERTS-1 related studies being made within the area shown. One supported by the North Carolina Office of Earth Resources is aimed at the evaluation of ERTS-1 imagery as a tool in geologic mapping in the piedmont and as a tool in other regionwide studies. The second study is aimed at evaluation of the forest resources in a portion of the piedmont. This study will provide information related to decisions to be made about development of a new state park between Greensboro and Winston-Salem. In the Raleigh area ERTS imagery is being used to study urban forests.

When one examines ERTS imagery on which is displayed a good representation of the local geology, he is forced to ask himself, "Can this information help solve a problem?" One of the problems of considerable concern to the North Carolina State Department of Transportation for some time has been the landslide potential and the actual slide along Interstate 26 in the vicinity of Tryon, Polk County, North Carolina. Can ERTS imagery or similar imagery tell us something about the possibilities of landsliding in this and similar circumstances?

In order to answer these questions, ERTS imagery of this area taken in October, 1972, May, 1973, and July, 1973 (Images 1083-15375, 1299-15383, 1371-15373) has been examined. The imagery was studied both in black and white at scales of 1:1,000,000 and 1:500,000 and in the color additive viewer at a projected scale of approximately 1:125,000.

The ERTS imagery shows the regional structural trends that are shown on the 1:250,000 scale geologic map of the area (Hadley and Nelson, 1971). One of the facts derived from the study is that bands 6 and 7 (infrared) record in all three images a belt of relatively high reflectivity extending from Mill Spring on the north to just south of the slide zone (Figure 7). In the other two bands the belt does not seem to be recognizable. If one examines a topographic map of the area, he finds that the western boundary of the belt of high infrared reflectivity coincides approximately with the mountain ridge which includes Tryon Peak and White Oak Mountain and that this boundary marks the approximate western boundary of the slide area. The eastern boundary of the belt of high reflectivity is near the base of the ridge containing Tryon Peak, and it marks the approximate eastern boundary of the slide area. The general impression gained from study of the negative prints of the area is that the slide zone is laced with northeastward-trending lineations which give a northeastward-trending grain to the topography. No readily identifiable transverse lineations have been recognized.

How could ERTS-type imagery have helped in the design of the highway? Probably the best use to which the imagery could have been put would have been to emphasize the need for very careful study of the area prior to beginning of the highway construction. The existence of the marked difference in reflectivity in the infrared band might have caused extensive questioning of the geologic data available and perhaps led to a more intensive geologic investigation of the possibilities of landslides along this particular stretch of the route than was made. It is obvious that the scale of the imagery would not have permitted a prediction of possible individual slide areas, but the space imagery could have emphasized the slide potential and caused studies beyond those undertaken to be made. Such studies might have identified a high probability for sliding.

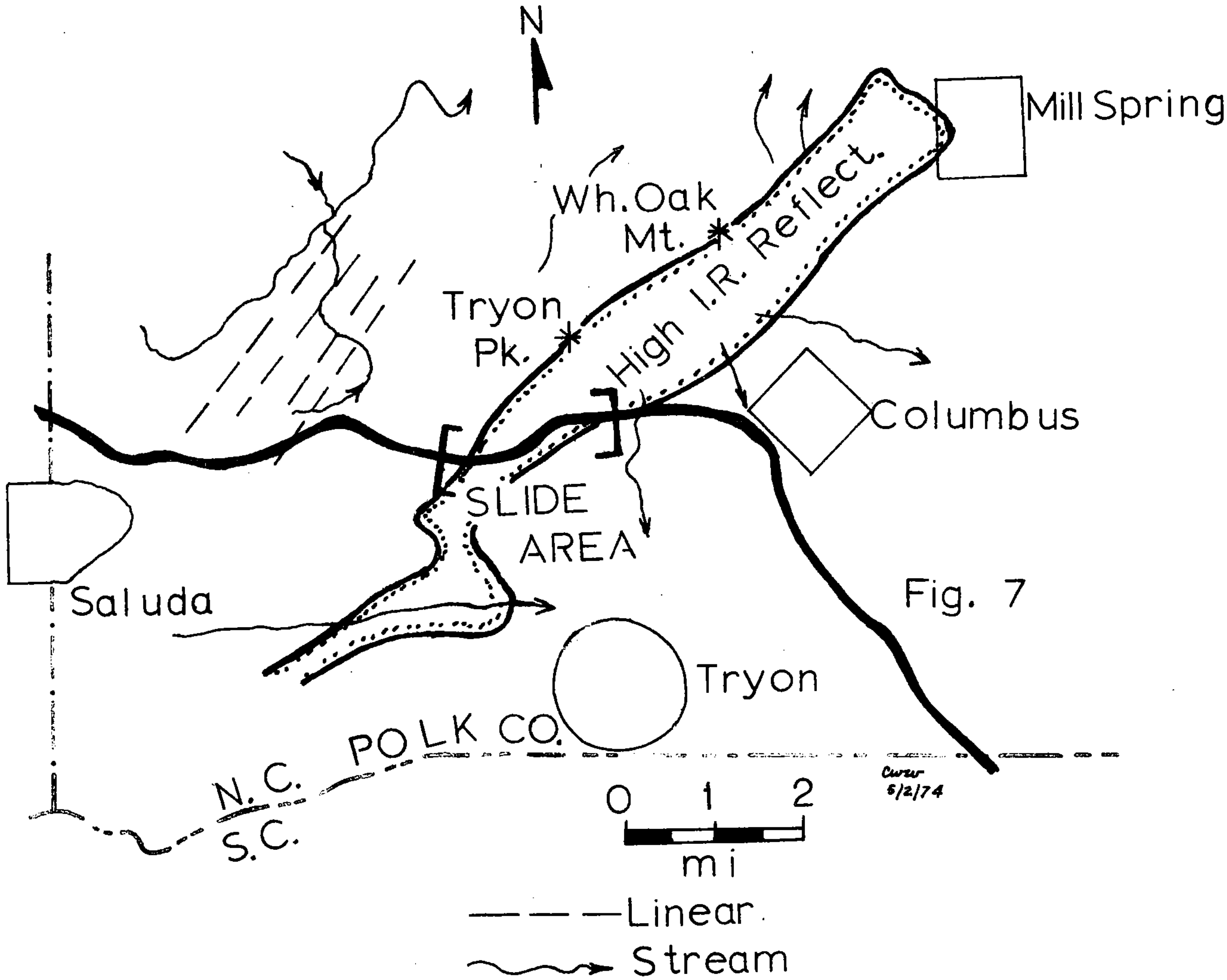

Fig. 7. Tryon slide area on I-26, Polk County, North Carolina.

Multispectral Photography

What is the advantage of multispectral photography over conventional photography? In many instances there is no advantage, and it is probably not the appropriate tool. However, where reflectances from one object mask the reflectances from another, or reflectances overlap, multispectral photography can be used to separate the two phenomena. Vertical distribution of suspended materials in water can be studied, and pollutants with weak reflectances in one part of the spectrum but stronger in another part can be isolated.

Multispectral photography is a useful tool in differentiating between and among different types of vegetation, and vegetation under stress can sometimes be recognized before the stress becomes apparent visually. Soil moisture differences can be mapped, and it is not implausible to believe that indirect evidence of soil water movement and even shallow groundwater movement might be monitored with successive overflights of a critical area. Certainly sinkholes and insipient sinkholes could conceivably be more easily recognized with this type of imaging than with conventional photography.

In any of these studies the advantage of the multispectral photography is that the black and white images can be reconstituted into a conventional color format, into a false color infrared format, or some format which emphasizes a particular feature but is unavailable in any film type. Additionally, the scene can be studied in the black and white format.

Figures 8, 9, and 10 illustrate some possible uses of multispectral photography. Figure 8 shows the subtle differences between two water masses found near Edenton, North Carolina, in August, 1973. Appropriate photographic treatment of the scene would enhance the differences, and if groundtruth were available, it might be possible to measure the vertical distribution of the

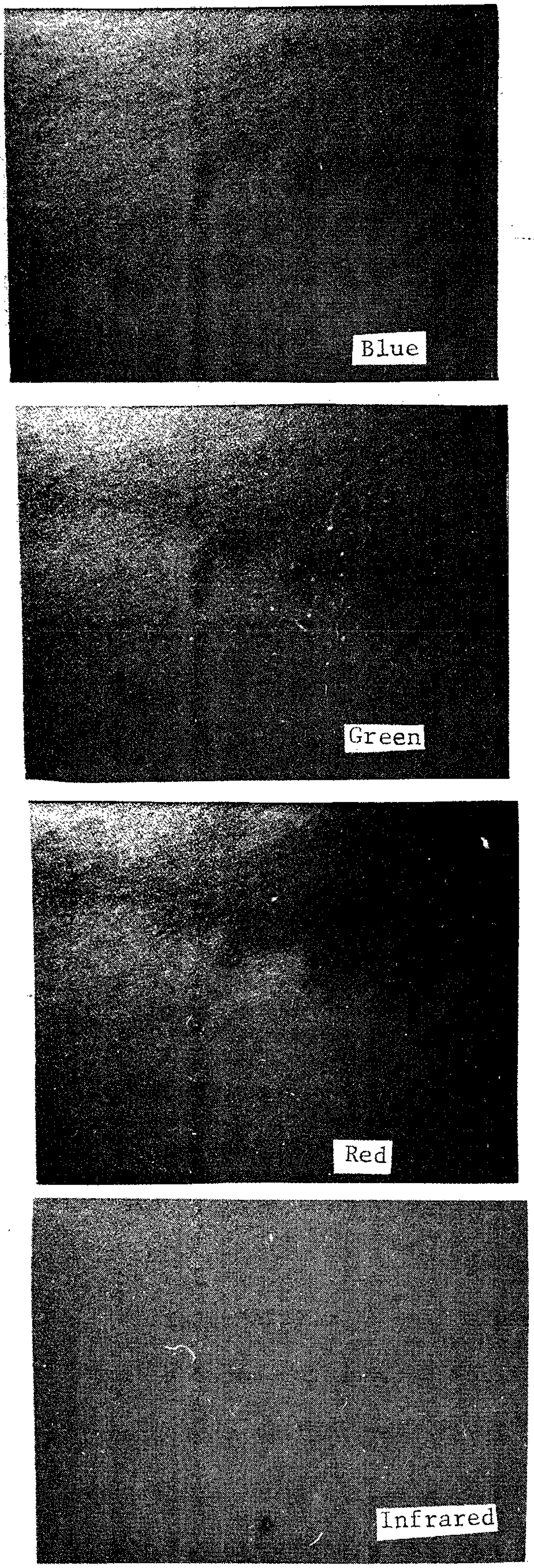

Fig. 8. Edenton Area, Aug. 9, 1973.

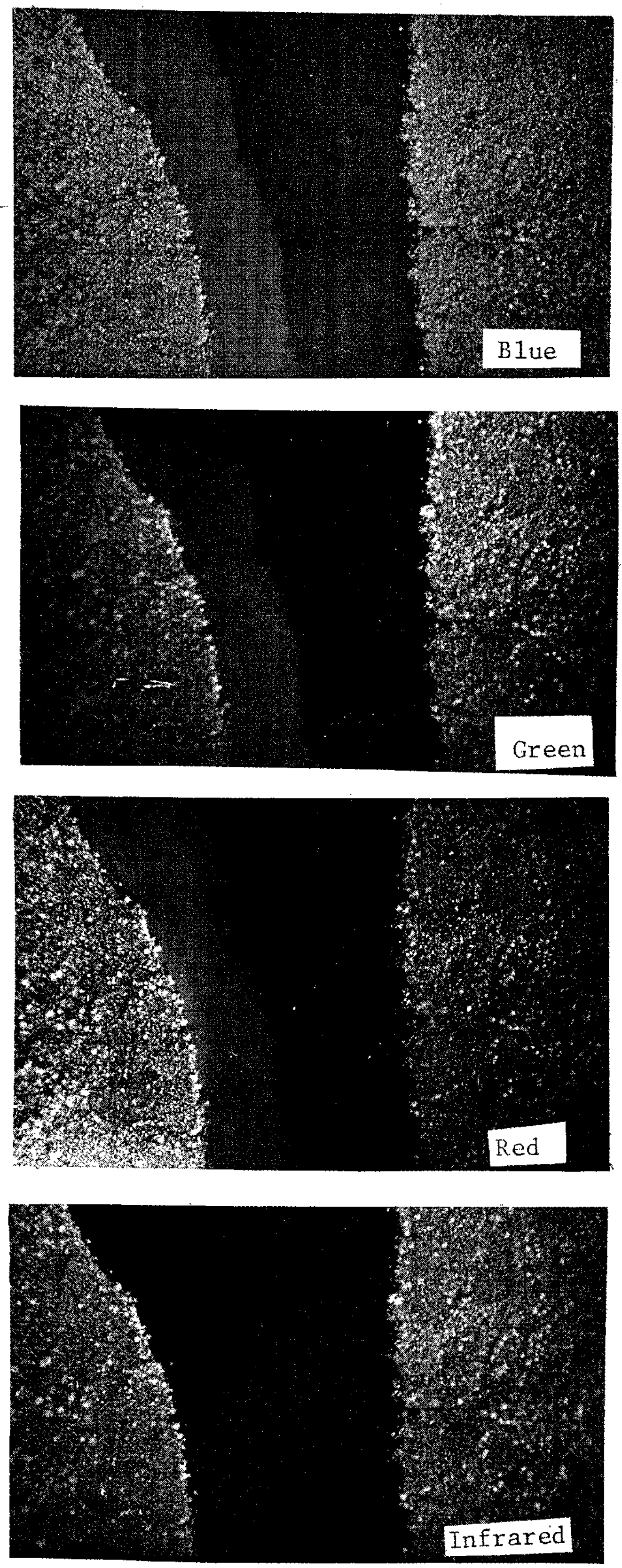

Fig. 9. Chowan River, N. C., near mouth of Meherrin Creek. Feb. 21, 1974.

suspended matter in the darker of the two water masses. Water quality studies being made in this area have been sampling sparsely and apparently not at critical points or along critical transects.

A more obvious meeting of two water masses is shown in Figure 9 where the lighter colored water from the Meherrin River enters the Chowan River containing the dark, organic-rich effluent from an industrial plant in Virginia. Although the phenomenon can be observed with the naked eye, the multispectral photography allows us to have a measure of the vertical concentrations within the two water masses if appropriate groundtruth has been collected. Also the dynamics of the system can be studied better, for conventional photography does not provide the capability for making studies of vertical movements in the water.

Figure 10 is a scene from the settling pond of the Texas Gulf Inc., phosphate mine near Aurora, North Carolina. Vegetative differences are shown by the different tones and shades of gray. When this scene is placed in a color additive viewer, the subtle differences in vegetative and sediment distribution stand out along with the major vegetative differences. The same techniques can be used in identifying marsh vegetation and separating the several types of vegetation.

A multispectral photograph of the new Raleigh Sewage Plant site taken last February is shown in Figure 11. The different reflectances from the undisturbed areas and from the disturbed areas should be noted. The relatively high red reflectance from the reddish saprolite is seen in the red band. The reflectance from the Neuse River is chiefly in the red band, and the blue band records more energy return than the green band. No groundtruth was obtained during the flyover, but if a portion of the area had been noticeably wetter, the infrared band would probably have shown this by a lower reflectance. The

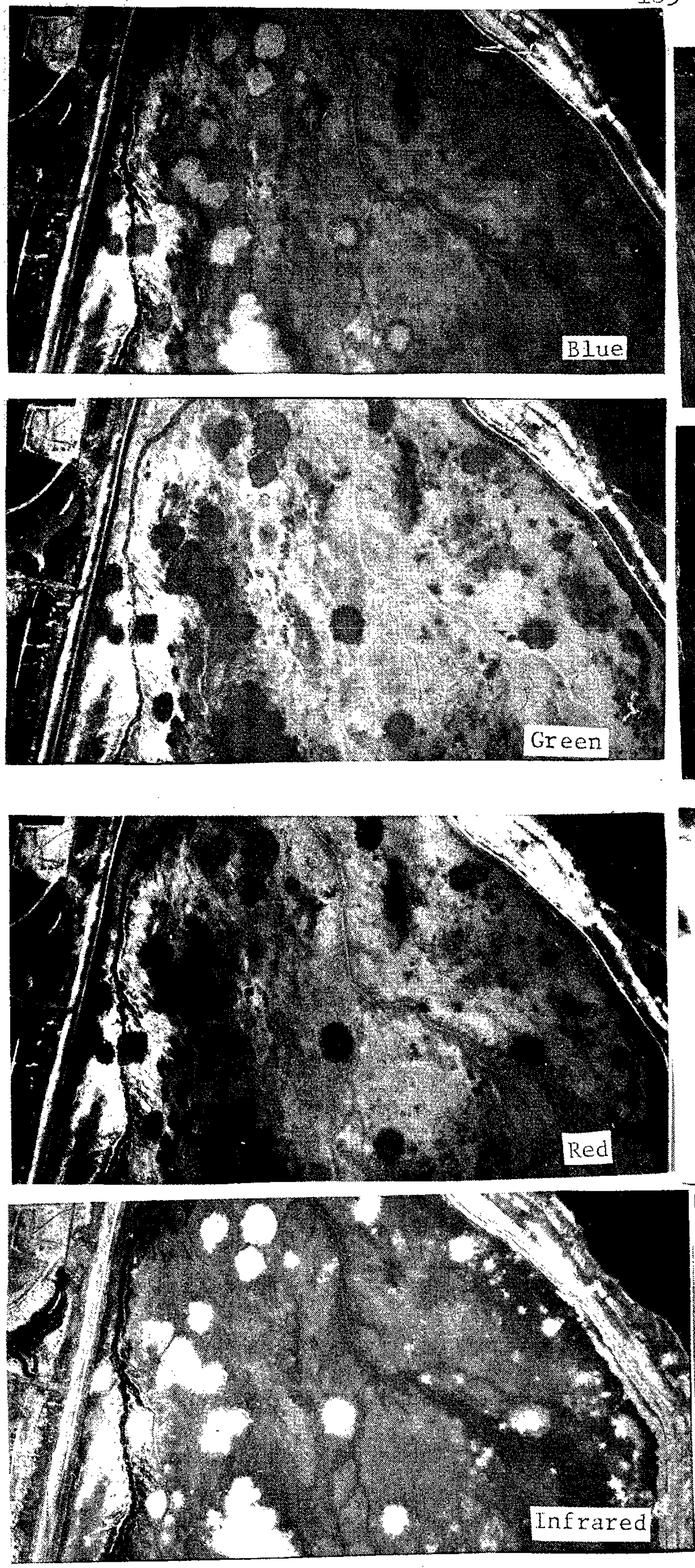

Fig. 10. TGI Holding Pond, May 18, 1973.

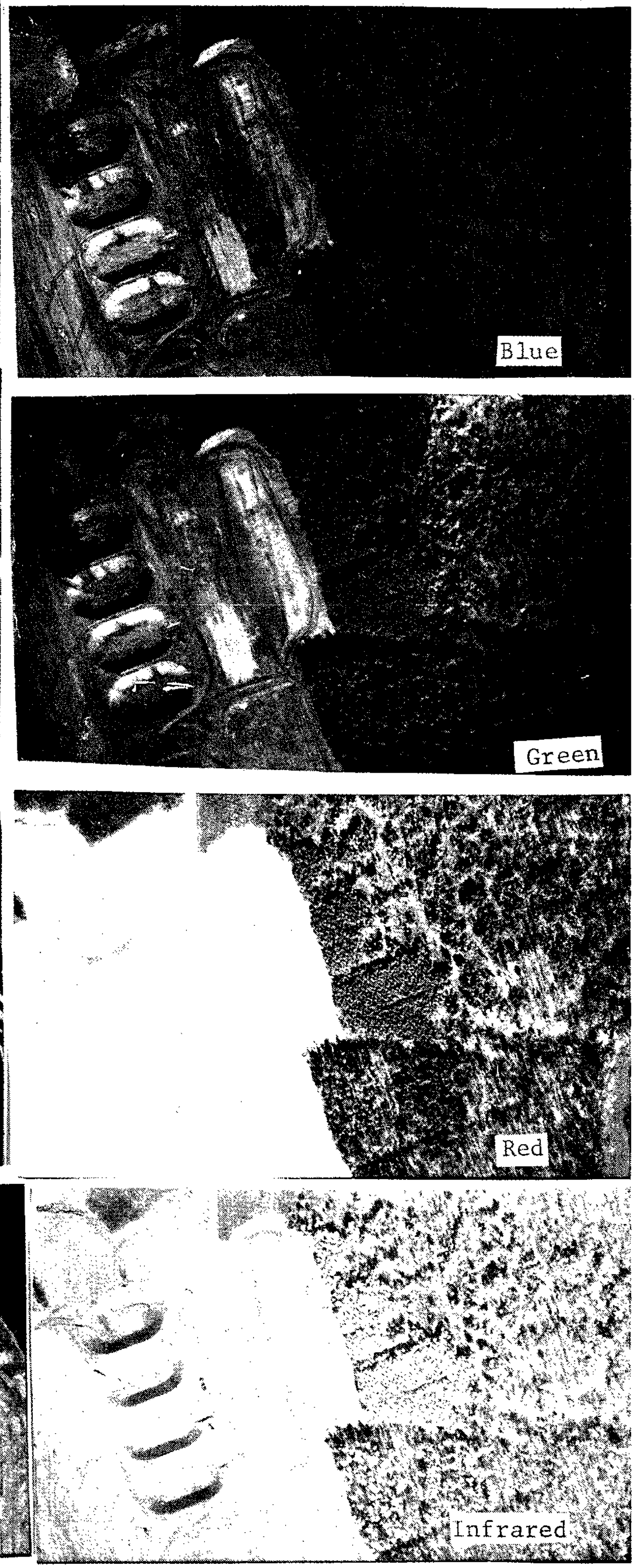

Fig. 11. Raleigh Sewage Treatment Plant site, Feb. 21, 1974.

images made during this study will provide baseline data about the site, both geological and botanical. When the plant goes into operation, sewage effluent will be placed on the ground, and the changes in vegetation will be monitored. A groundwater monitoring system is being developed, and it is expected that some experiements will be made relating changes in groundwater quality to behavior of the vegetation.

Summary

ERTS-1 imagery is a useful tool for reconnaissance studies related to a number of fields. In geologic applications regional trends not suspected or fully appreciated may be recognized. The repetitive coverage of the satellite allows examination of an area at different seasons, under different soil moisture and vegetative conditions, and at different sun angles.

Phenomena not seen under one set of conditions may possibly be recognized under another set. Additionally, the splitting of the electromagnetic spectrum into the four bands helps separate features which might be obscure if all parts of the spectrum were recorded on the single image.

Multispectral photography has apparently had little or no systematic practical applications to gathering of geologic data. Probably one of its most useful applications might be in recognition of soil moisture patterns in potential slide areas. It probably can be used to delineate subtle features in soils and saprolite which might provide clues to potential slide areas or to other potential problem areas. Another possible use in regions where winter salting of highways is the practice in the monitoring of possible groundwater contamination through monitoring the vegetation for signs of stress. It has a considerable potential for studying sediment movement in shallow water areas of the coast and large lakes.

Acknowledgments

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SUBSURFACE EXPLORATION

STATE OF THE ART

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SUBSURFACE EXPLORATION STATE OF THE ART

The field of geotechnical science has made significant advances during the past decade. This is true not only of the technical area but also of the demand worldwide for geotechnical services by other disciplines as well. This increased demand is directly attributable to failures, a fact that may easily be verified by a review of court dockets or contractural claims being processed by governmental agencies. Most engineers, architects, and administrators now realize that the low cost of geotechnical investigation and design--less than one percent of construction cost--offers a substantial return on funds invested.

Unfortunately, the field of highway engineering has lagged behind other types of construction in utilizing geotechnical knowledge in the design-construction process. Many States still do not possess in-house geotechnical abilities and engage consultants only in the event of failures. This approach, of course, results in overconservative design, numerous contractural claims, and excessive maintenance expenditures.

A recent study by the Federal Highway Administration has shown that more than $100 million are spent annually for correction of landslides on the Federal-aid system alone. This represents over two percent of annual Federal-aid construction expenditures. In addition, it is estimated that over $3 million each year are wasted on ultraconservative design. Thus, the value of the geotechnical field to the highway industry can easily be seen.

"Don't begin until you count the cost. For who would begin construction of a building without first getting estimates and then checking to see if he has enough money to pay all the bills. Otherwise, he might complete only the foundation before running out of funds, and then how everyone would laugh. 'See that fellow there?' they would mock. 'He started that building and ran out of money before it was finished'." This advice of Jesus, found in the 14th Chapter of Luke, The Living Bible, is recognized not only for its spriitual significance but also as sound councel to our profession today. It is entirely possible that the man of whom Jesus spoke exhausted his funds during construction of the foundation while attempting to cope with unforeseen soil conditions.

Thus, the prime objective of any soils investigation is to "count the cost"; to gather information which will allow the interaction of the proposed structure and the supporting soil to be accessed with a reasonable degree of accuracy. In order to accomplish this objective, the field investigation must identify all soil and rock strata which have an influence on design and construction, and must allow the definition of the pertinent physical properties and the strength and deformation characteristics of each strata. The importance of thorough planning for each subsurface exploration cannot be overemphasized. Exploration techniques should be tailored to the specific requirements of each project.

Exploration Methods

One of the oldest methods of subsurface exploration known to man is the use of test pits. Today, the use of test pits is not considered to be a routine investigative technique, however, their merits should not be overlooked. Because of expense, test pits are generally restricted to relatively shallow depths (5 to 10 feet). The value of a test pit is derived from the detailed, visual examination of stratification. Undisturbed block samples may also be obtained for subsequent laboratory analysis, and in situ, direct shear tests may also be conducted. Test pits are especially useful when weathered conditions exist at soil-rock interfaces or where extensive solution of rock has occurred.

Mechanized augers mounted on a truck or a jeep are widely used to develop generalized soil profiles which are invaluable aids in general roadway design. It must be noted however, that auger borings are not adequate for foundation design, slope stability or embankment settlement since problems in these geotechnical areas require undisturbed sampling capability. Thus, auger borings should be considered to provide preliminary information only which assists in pinpointing areas requiring a more sophisticated exploration technique.

The most popular technique of subsurface exploration today is the rotary wash boring. The popularity stems from the fact that it is the most rapid technique for advancing borings in virtually all types of soil and rock. This versatility, coupled with the fact that this type of boring results in minimal disturbance to the soil at the bottom of the borehole, has caused the rotary wash boring to be widely utilized for subsurface explorations.

The boring is advanced by a rotating bit with the cuttings being removed from the hole by circulating water on drilling fluid. In soils such as soft clays or sands, where the hydrostatic pressure from the wash water is not sufficient to hold the boring open, powdered bentonite clay can be added to the water to form a "heavy fluid" known as drilling mud. The increase in pressure against the walls of the boring is usually sufficient to prevent collapse. Drilling mud is also useful in preventing the loss of water circulation when small cavities or fissures are encountered. Occasionally, conditions such as large cavities, exceptionally soft soil, or gravels are encountered which necessitate the use of casing to support the boring sidewalls and allow circulation of the drilling fluid.

Rotary wash borings are extremely difficult in thick deposits of coarse gravel. The use of casing in situations such as this is tedious, time consuming and expensive. This led to the advent of the hollow stem auger. As the name implies, the stem on which the continuous helical spirals are mounted is hollow. The lower auger flight contains a device which prevents the intrusion of soil into the hollow stem. This allows sampling devices to be lowered through the hollow section to obtain samples at any depth desired. In effect, the hollow stem auger acts as a means of advancing the boring and casing the boring simultaneously. The use of this versatile equipment has grown steadily in popularity in the past few years.

As stated previously, the scope of each subsurface exploration is dependent upon the character of soil deposits encountered and the type of information required for competent design and construction. Thus, exploration programs must remain flexible to allow necessary modifications when unforeseen or potentially dangerous soil conditions are encountered. Therefore, the following guidelines are presented to reflect the current acceptable level of effort for the majority of highway projects.

The development of a highway profile requires borings which are spaced approximately 1,000 feet and are carried to a minimum depth of 6 feet below subgrade. In traversing terrain, whose geological characteristics are complex or variable, borings should be made at closer intervals. On projects where uniform stratigraphy and reliable historic data exists, the spacing of the borings may be increased.

For projects involving deep cuts, high fills, or slope stability, borings along the centerline of the roadway should be located at the quarterpoints of the section being investigated. Often it is necessary to develop soil cross sections with at least three borings as well. These borings should be carried to a depth which adequately defines the stratification which could influence the design of the proposed construction.

A minimum of three borings should be made on each structure. In order to define traverse bedding planes, the borings should not be along a straight line. When erratic subsurface conditions exist, the number of borings should be increased to at least one at each abutment and pier location. There are several empirical methods available for determining the depth to which foundation borings should be taken. The most widely accepted method is that of the Geotechnical Institute of Belgium. This guideline states that the borings should be extended to a depth where the additional stress created by the structure and the adjacent fills is less than 10 percent of the existing overburden pressure. In addition, structural exploration must include sampling capabilities in order that pertinent stress-strain characteristics of bearing strata may be subsequently determined by laboratory testing.

Often, proposed construction sites are viewed as being inaccessible to subsurface exploration equipment during the preliminary phase of project development. Unfortunately, the rugged terrain in which these sites are located invariably contain critical soil or rock conditions which have a major influence on design and construction. When the mobility of the various types of commercially available vehicles is considered, it is easily seen that inaccessibility is no longer a valid reason for eliminating subsurface exploration.

Sampling Devices

Of all the decisions to be made concerning subsurface investigations, the most important is the determination of sampling methods. An exploratory program which meets all of the requirements concerning location and depth of borings cannot possibly meet the objectives of the investigation unless the samples obtained allow the definition of pertinent physical soil properties. Without the aid of prior information, the decision concerning sample types must be made in the field at the onset of the drilling operations. This necessitates the presence of a well-trained individual to evaluate the results of the initial borings in light of the proposed construction and to subsequently instruct the drill crew as to the sampling procedure to be used. In order to avoid inefficient operations it is obvious that a drill crew must be well equipped with proper sampling devices before embarking on the exploration.

Samples can be generally divided into two categories; disturbed and undisturbed. Disturbed samples are those which are relatively complete but the sample has undergone severe structural disturbance. As the name implies, undisturbed samples are those in which the structural disturbance is kept to an absolute minimum. While it is physically impossible to obtain a perfect undisturbed sample, there are many devices and techniques which may be employed to minimize disturbance.

For undisturbed sampling of cohesive soils, the most widly used device is the Thin Wall Open Drive Sampler, more commonly known as the Shelby Tube. The minimum requirements for Shelby Tubes are set forth in AASHO T207-70 and ASTM 1587-67.

Often, when very soft clays are encountered, the Shelby Tube is ineffective in obtaining samples since the adhesion of the clay to the inner wall of the tube is insuffecent to support the weight of the sample until it can be withdrawn from the borehole. To overcome this problem, the piston sampler can be used. The piston sampler is basically a Shelby Tube with an inner piston which can be released or withdrawn during the sampling procedure. Consequently, any downward movement of the sampler will create a partial vacuum over the sample which assists in sample retention within the tube.

The relationship between sample type and the degree of disturbance for a sensitive clay is presented in Figure 1. While the effect of sampling would be minimized for clays having less sensitivity, this figure does point out the importance of using the proper sampler for a particular soil.

Currently, the most widely used device in the sampling of cohesionless soils is the split barrel sampler. A complete description of the device and the test procedure can be found in AASHO T-206-70 or ASTM B 1586-67. Samples obtained with a split barrel sampler are classified as disturbed. For sands, however, the standard penetration value allows the relative density of a material to be established with a resonable degree of accuracy. This relative density can then be reproduced in the laboratory for subsequent triaxial testing to determine the angle of internal friction.

The use of standard penetration values to determine the shear strength of cohesive soils is, at best, a crude approximation. In effect, the standard penetration test is an in-place dynamic shear test and since dynamic testing cannot access the time dependent stress-strain relationships of clays, the safety factors of resulting designs can range from less than one to ultraconservative values. For this reason, the use of standard penetration values in cohesive soils is discouraged.

Another common practice involving the testing of cohesive materials is also discouraged. Often, cohesive split barrel samples are subsequently used in conducting unconfined compression and triaxial tests. It must be pointed that in the majority of soils, the shear strength thus derived does not even give a good approximation of the remolded strength of the clay due to the change in structural characteristics and density. Thus, designs resulting from this procedure are often ultraconservative and erroneous.

When material is encountered during the course of the exploration which is too hard to penetrate using the techniques previously discussed, core drilling is employed to obtain samples. There are many different types of core drills available today; each designed to accomplish a specific objective. For most exploration programs involving rock, the single type core barrel will provide adequate samples for examination and testing. This barrel consists of hardened steel tubing with a rock bit attached to the end. Depending upon the hardness of the rock, the bits may be diamond or tungsten carbided tipped.

In the coring operation, the barrel and the bit rotate while the drilling fluid is circulated through the borehole to return the powdered rock cuttings to the surface. As the bit advances, the rock core extends upward into the core barrel. The ratio of the length of core recovered to the distance drilled is known as core recovery, which is indicative of the soundness of the rock. In sound, homogenous rock a recovery of over 90 percent may be expected; in rock containing seams, a 50 percent recovery is typical; while in rock which is decomposed, the recovery may be little, if any. The latter two cases lead to the development of the double tube core barrel which employs a thin inner barrel which remains stationary during drilling to protect the core against vibration and erosion by the drilling fluid, thus increasing core recovery.

For construction which calls for deep cuts in rock masses, the direction, inclination and frequency of bedding planes, faults, and cracks become essential data which must be gathered to determine the stability of rock mass. Normal rock cores can provide only a portion of this information. The use of core orienting barrels greatly assists in overcoming exploration difficulties in this area. A small knife located in the inner barrel cuts a scratch on the core thus allowing the core to be reoriented on the surface and measured to determine the direction and slope of a core fissure. Another method which is gaining in popularity is the use of the borehole camera to examine the walls of the borehole thus allowing the extent of cracks and joints to be accessed.

The Denision core barrel employs rock coring techniques to obtain high quality samples of soils which cannot be effectively sampled with split barrel samples or Shelby Tubes. It is especially useful in hard cemented soils or in dense sands or gravels. A saw toothed bit is used to cut the core which is subsequently protected by a nonrotating inner barrel. Sample loss upon removal of the barrel from the borehole is prevented by a catcher located at the bottom of the tube.

No discussion of exploration or sampling methods would be complete without an accessment of the role of the crew chief and the driller. Often, the quality of their work governs the degree of objective accomplishment of the entire soils exploration program. For reasons of economy and efficiency, these individuals must function independently in the field on most investigations. Their decisions concerning location and depth of borings, and the type of samples are of prime importance in gathering accurate data for subsequent design.

In many organizations, the plan of attack for exploration is derived in the office and allows little if any deviation on the part of the crew chief-driller team. This inflexibility stymies growth of these key individuals and encourages an exploration objective of "drilling holes." Those organizations which require independent action on the part of this team, in the event of unusual subsurface conditions, have found that the additional expense of training has been money well spent. When the crew chief and driller are included in the exploration-design team, their job enrichment results in a comprehensive exploration effort. Decentralization within a soils organization must be the key word.

The crew chief must continually supervise the drilling operation to ascertain that good sampling and testing procedures are followed. He must be aware of the importance of sample handling and preparation to insure a minimum amount of disturbance. The importance of accurate logs cannot be overemphasized, and for this to be accomplished, the crew chief must work closely with the driller. In addition, the preparation of soil profiles in the field by the crew chief greatly assists in evaluating subsurface conditions to determine if sufficient information is being obtained. All too often, however, the development of the soil profile is not made until the drill crew completes the exploration and has moved to another project. In this case, the time delays and extra costs of obtaining the necessary data could often be avoided by training the crew chief to make proper use of the field soil profile.

Of all the personnel involved in a subsurface exploration program, the experienced driller has probably more "feel" for the conditions which exist than any other single individual. By evaluating the operation of the drilling equipment he can detect changes in materials, bedding characteristics, etc. This important information must be communicated to the crew chief to allow proper logging, sampling, and in some cases, modification of the scope of the exploratory program. In short, no subsurface investigation can be any better than the exploration work done by the crew chief and the driller.

In light of this, it is evident that the crew chief and the driller must be well trained personnel. An exposure to the work of the laboratory for these individuals has been shown to be an important area in their training since their understanding of sample preparation and testing better equips them to operate decisively in the field.

Since the work of a drilling crew is usually accomplished under adverse working conditions and, from the surface, appears to consist primarily of manual labor, the pay scales are often relatively low when compared to other technical workers. This administrative assessment has severely hampered the development of a strong geotechnical unit within many highway departments and will continue to do so until steps are taken to establish more equitable pay scales.

In Situ Strength Tests

As research continued to demonstrate the effect of disturbance on the strength properties of soils, more effort was invested in developing techniques and equipment which could be used in the field with minimal soil disturbance to determine the in situ strength and strain parameters of soils.

The vane shear test was developed by Cadling in 1948 to measure the in situ undrained shear strength of clays. Within a few years, its popularity had spread to several countries. Today, a soils exploration involving soft clays is not considered complete if it does not include a number of vane tests.

Over the period of time since the inception of the vane, it was noticed that the results were often higher than those produced in laboratory tests. Initially this discrepancy was attributed to the rate of loading. In 1972, however, extensive work done by Bjerrum of the Norwegian Geotechnical revealed that the plasticity index of the clay must be considered when using vane strengths in foundation design or embankment stability analyses. Figure 2 presents the recommended reduction factor to be applied for vane tests in clays of varying plasticity indicies.

The Iowa Bore Hole Shear Device is another instrument used to measure in situ strengths. In this test, an expandable shear head is positioned in a smooth 3-inch boring. The jaws are then expanded pneumatically with a constant pressure against the side of the borehole. A mechanical shearing force is then executed at a rate of 0.002 inches per minute until failure occurs. By repeating the procedure at the same location, Colombs failure envelope may be developed. There are several uncontrolled variables in the testing procedure, however. These variables include the effect of disturbance and the amount of drainage which occurs. Since disturbance and pore pressure are major factors in determining strength parameters, it is recommended that the Iowa Borehole Shear Test be used in conjunction with standard laboratory shear tests.

The Menard Pressuremeter is a device which has been used extensively in Europe for the past 15 years. It was introduced in the United States approximately 7 years ago. The method basically consists of seating an expandable cylinderical probe in the borehole and, using fluid pressure, applying incremental loads against the borehole wall. Under each increment, the changes in the borehole volume is measured using a glass manometer at the ground surface. The results are plotted as a stress-strain curve and provide information equivalent, at least in part, to a consolidation test and to a triaxial shear test.

Settlement calculations for consolidation of clays using pressuremeter data correlate quite well with those resulting from standard laboratory consolidation tests. Settlements of footings on sands have also been more accurately predicted using pressuremeter data rather than the standard penetration method. The prediction of shear strengths with the device, however, has not been as successful. This is due, in part, to the difficulty in defining the ultimate or failure stress. Pressuremeter results are often erratic and in excess of vane shear results. This is an excellent example of why the designer must have a thorough understanding of each test method employed during a subsurface exploration. The assumptions of each test, the limitation of each test, and the correlation between test methods must be evaluated if a sound design is to be produced.

Another device which is becoming more popular in the United States is the Dutch cone penetrometer which is basically a stainless steel probe which is forced into the ground with hydraulic pressure. The probe most widely used today consists of a 60-degree cone at the tip of the probe and a skin friction jacket which travels behind the cone. Readings are obtained at the ground surface which define the force required to advance first the point, and then the point and the friction jacket together. These results can be used to estimate the point bearing and skin friction for pile foundations. The use of the Dutch cone to determine strength parameters is fairly difficult and requires a knowledge of the type of soil being encountered by the probe. Thus, this device should be used in conjunction with standard borings.

One definite advantage of the Dutch cone is speed with which probes can be made. Since soundings can be made in soft soils to a depth of 40 to 50 feet in approximately 20 minutes, the cone penetrometer has become an extremely economical method for subsurface exploration. By using the penetrometer in conjunction with borings, rapid detailed surveys of erratic deposits of soft clays, silts, and peats can be made, thus reducing the exploration costs considerably since fewer borings are required. Small diameter undisturbed samples can also be obtained with this device.

In summary, while significant advances have been made in the state-of-the-art of in situ testing, no device or technique has been shown to be completely definitive. Therefore, in situ testing should always be used as a supplement to borings and laboratory testing in the design process.

Supplementary Techniques

Through the efforts of research, remote sensing techniques have made tremendous advances in the past two decades in the field of subsurface exploration.

The most valuable remote sensing technique today is aerial photography. Air photo interpretation is based on the principle that subsurface conditions are reflected in surface features. Drainage patterns and vegetation give indications of general soil conditions. Old landslide scarps and terraces, sinkholes, and swamps or marshy areas can be easily identified and avoided early in project development. In addition, air photos provide an excellent method of preliminary reconnaissance, since site access and equipment type may be determined. Air photos also allow definition of regional structure including faults, folds, joint patterns for tectonic activity, and formation changes. All corridor studies should include a review of aerial photos by qualified individuals to determine the probable soil and rock conditions along the proposed route.

The use of direct air reconnaissance in helicopters or low speed-high speed wing aircraft has the same advantages of air photos with additional benefits as well. This reconnaissance method allows a rapid review of the route from various angles and altitudes. Excellent hand photos can be made for later reference. Unfortunately, this inexpensive method is seldom used because it is considered by many who are uninformed to be an extravagant waste of time and money.

Infrared sensing, microwave radar, and sonar have also been adapted to exploration work. These devices, particulary infrared sensing, have assisted in further pinpointing the location of muck pockets, sinkholes, springs, and shear zones which are difficult to detect with aerial photography. Further advancement in remote sensing will be forthcoming in the next decade, as a substantial amount of research effort is currently being devoted to this field.

Although there are several methods of geophysical exploration, only two have found wide acceptance in highway engineering, the electrical resistivity method and the seismic velocity method. The electrical resistivity method is based upon electrical conductance (the reciprocal of resistivity) of a soil. A saturated clay is a good conductor and hard rock is a poor conductor. It is easily seen that unlimited possibilities exist between these two extremes. Consequently, the value of this method is in direct proportion to the user's ability to relate the resistivity data to the physical features of the soils encountered.

The resistivity method most commonly used in the United States is the Wenner procedure. The assumptions in this method are that each soil layer is homogenous and isotropic and has a definite resistivity value; each layer is parallel to the ground surface; and that the layers are separated by distinct interfaces. These conditions are rarely found in nature. In addition, the presence of metal pipes, ground water, etc., influence the data considerably.

The seismic method of subsurface exploration is based on the principle that the rate of propagation of elastic waves is a function of the elastic constants of the media through which the waves travel. Currently, this method is widely used throughout the United States, however, certain drawbacks do exist. One definite limitation is that successive soil layers must have the property of increased seismic velocity. The layers must also be fairly thick, i.e. greater than one-fourth of the depth to the top of the layer. The position of the water table can have a large influence on the results. Finally, complex or sloping stratigraphy is virtually impossible to analyze.

Thus, the need of routine borings in connection with geophysical techniques can readily be seen.

Conclusion

In some quarters, one can still hear the remark that 'geotechnical engineering is not an exact science.' That statement is true, but is must be kept in mind that neither is the art of concrete, asphalt or even steel design and construction. The degree of success in each disciplinary area is directly dependent upon the ability of those responsible to collect, interpret, design, and supervise the construction of desired facilities.

Perhaps, the reason that the geotechnical discipline is subject to remarks such as this, can be found in the fact that errors in judgment, unforeseen conditions and failures usually result in expensive, time-consuming delays. All too often, these delays could have been avoided with a thorough subsurface exploration program. The methodology for precise geotechnical exploration is available; it is the responsibility of the profession to see that it is properly applied. A chain is only as strong as its weakest link. There can be no adequate compensation in the design and construction phases to neutralize the effects of shortcuts, nearsightedness, or pseudoeconomy in the geotechnical exploration phase of the project.

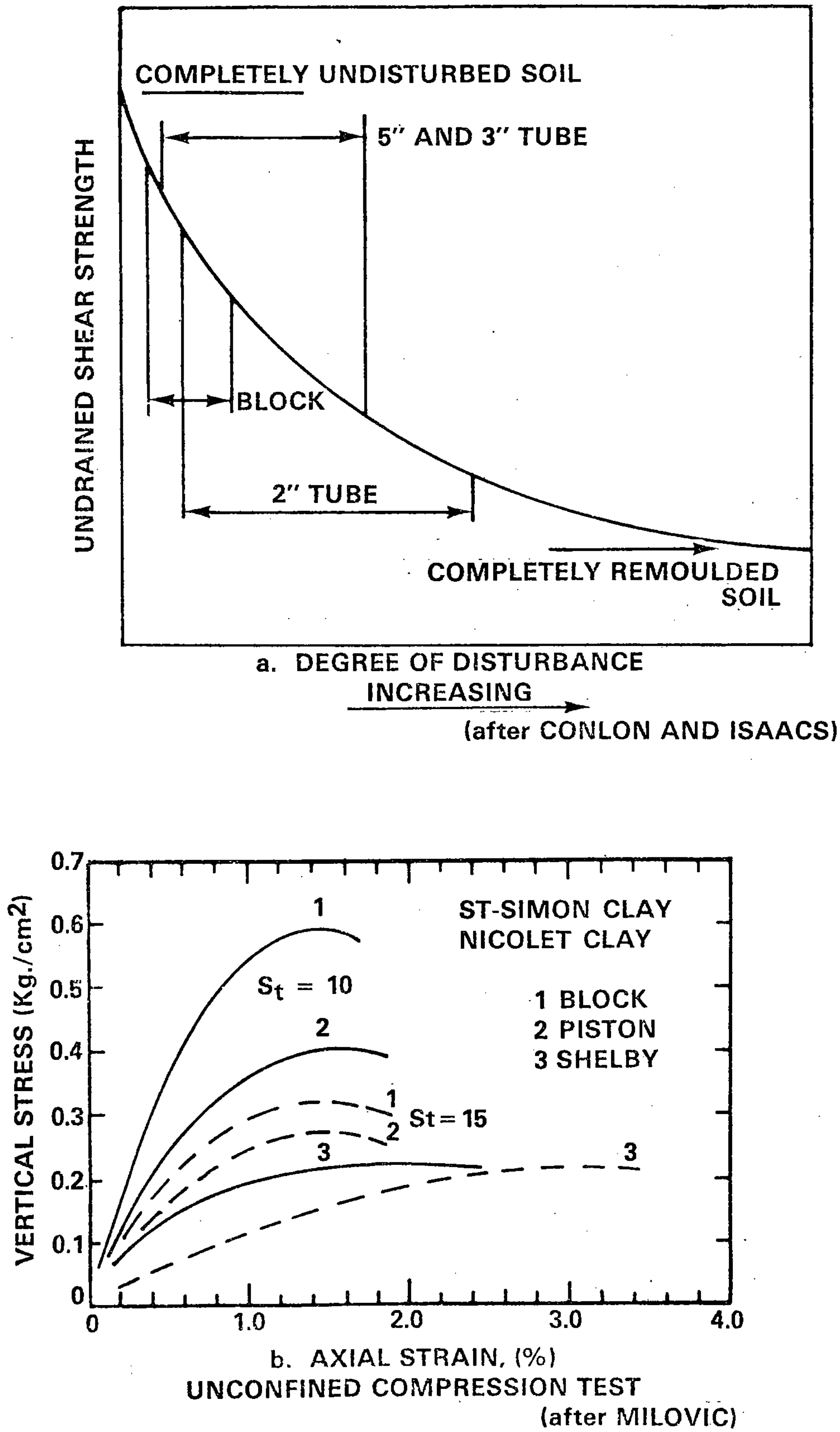
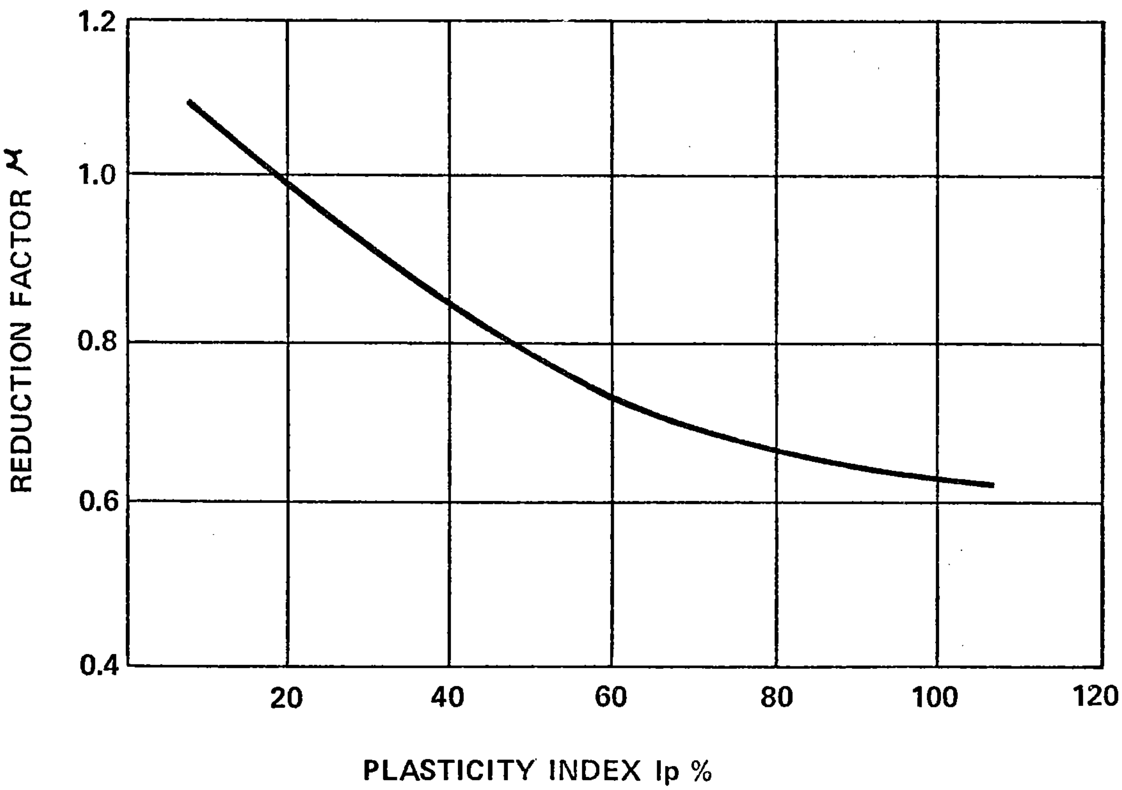


FIG.- 1. INFLUENCE OF SAMPLE TYPE AND SIZE ON DISTURBANCE

FIG. -2:

REDUCTION OF FIELD VANE SHEAR STRENGTHS BY BJERRUM



ENGINEERING GEOLOGY OF I-26 LANDSLIDES POLK COUNTY, NORTH CAROLINA

BY
CLAY E. SAMS AND CHARLES H. GARDNER
LAW ENGINEERING TESTING COMPANY

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ENGINEERING GEOLOGY OF I-26 LANDSLIDES POLK COUNTY, NORTH CAROLINA

BY

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I. INTRODUCTION

Interstate Highway 26 extends northward from the Piedmont of South Carolina, ascends the southern slope of Miller Mountain a short distance inside the North Carolina line in Polk County, and continues into Asheville, North Carolina. Miller Mountain is part of a high northeast to southwest trending ridge which includes Tryon Peak and White Oak Mountain. When construction on the Miller Mountain section of highway began in late 1968, several very large cut slope slides were released in the saprolite and colluvium of the mountain slope. The Miller Mountain slide area is the only section of I-26 where construction is incomplete; no grading work has been done on this section since 1969. These slides were investigated on a preliminary basis by the North Carolina Highway Commission (currently named the Department of Transportation, Division of Highways), and by another consultant. A more extensive geotechnical investigation was subsequently undertaken by Law Engineering Testing Company, beginning in mid-1971 and substantially completed in mid-1972.

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The location and geotechnical engineering aspects of the slide site and vicinity are described herein, accompanied by a brief description of the regional and area geology setting. Also presented are shear strength parameters determined from direct shear tests on hand-cut block samples of preconstruction and construction shear surfaces. A brief description of the safety factors and influence of remedial actions and the corrective scheme being utilized is presented.

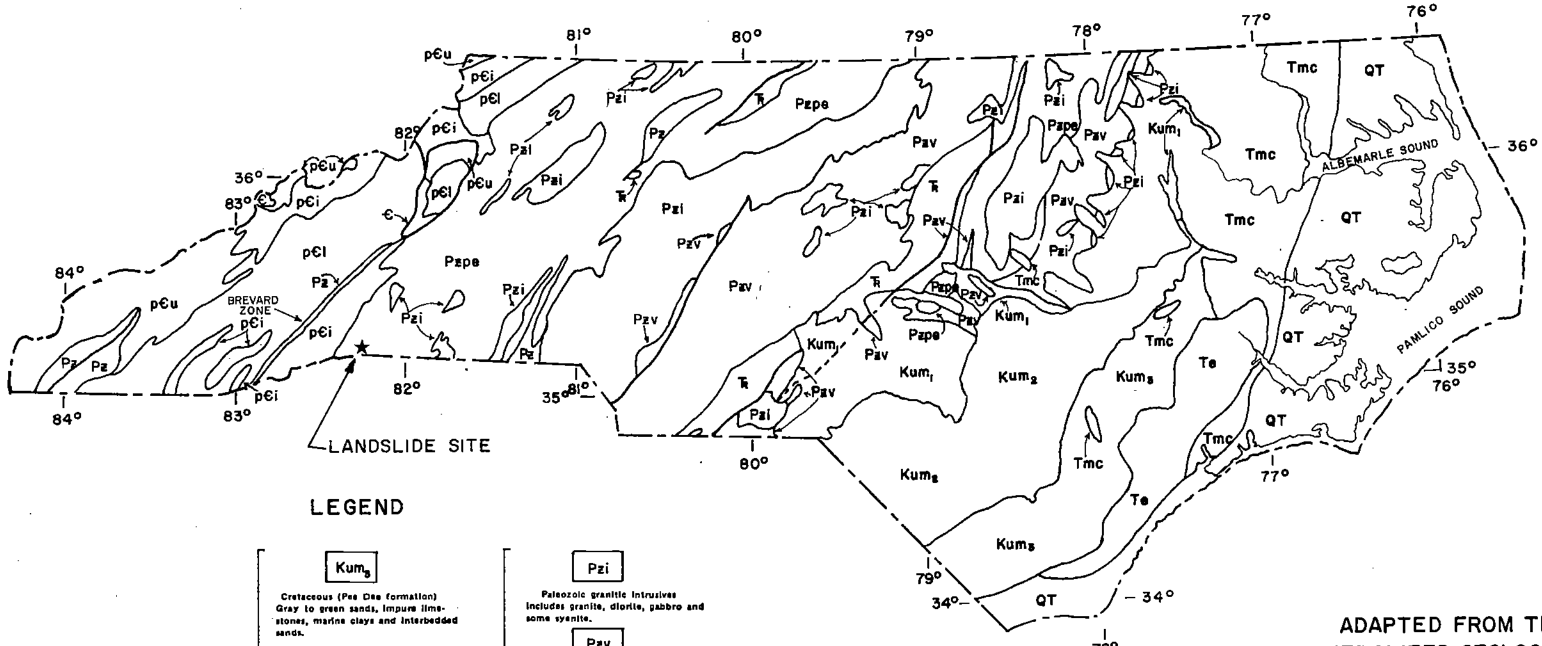
II REGIONAL GEOLOGY

A. Regional Physiographic and Tectonic Location

Figure 1 shows the site location on the Generalized Geologic Map of North Carolina. The site lies on the boundary between the Blue Ridge and Piedmont geologic provinces. Physiographically, the site lies on an outlier of the east front range of the Blue Ridge Province. With respect to lithologic and tectonic classification, the site is southeast of the Brevard Zone and lies within the Inner Piedmont Belt.

The Inner Piedmont Belt is characterized by highly deformed pre-Cambrian and early Paleozoic metamorphic and igneous rocks. Most of the rocks in the region consist of gneisses and schists resulting from high grade metamorphism of sedimentary, intrusive, and possibly volcanic materials. Some igneous intrusions in the belt have been dated as late Paleozoic. Most rock forming and deformational events were completed by the end of the Paleozoic. Some faulting and basic dike intrusion occurred during Triassic time. No significant rock forming or deformational events are known to have occurred since the Triassic period.

Rocks existing near the present day surface in the region were deeply buried when deformed, and underwent semi-elastic folding as well as plastic deformation. Faults and shear zones in the region were typically healed by recrystallization. Joints of microscopic width usually represent the only open fractures.



LEGEND

CENOZOIC

QT — Quaternary. Surficial sands and clays.

Tmc — Miocene (Yorktown formation). Massive marine clay and interbedded shell beds and marls.

Te — Eocene (Castle Hayne limestone). White sandy shell limestone to dense silicified gray limestone.

MESOZOIC

Kum_3 — Cretaceous (Pee Dee formation). Gray to green sands, impure limestones, marine clays and interbedded sands.

Kum_2 — Cretaceous (Black Creek formation). Black to gray, interbedded sands, clays and marls.

Kum_1 — Cretaceous (Tuscaloosa formation). Tan, red and gray sands and interbedded clays.

₸ — Triassic. Red, brown purple and gray sandstone, siltstone, shale and conglomerate, and some coal.

PALEOZOIC

Pzi — Paleozoic granitic intrusives. Includes granite, diorite, gabbro and some syenite.

Pzv — Paleozoic volcanics. Metavolcanic and metasedimentary rocks of Carolina Slate belt.

Pz — Paleozoic metasedimentary. Quartzite, schist and marble of probable Paleozoic age.

Є — Cambrian (?) Chilhowee. Quartzite, shale and

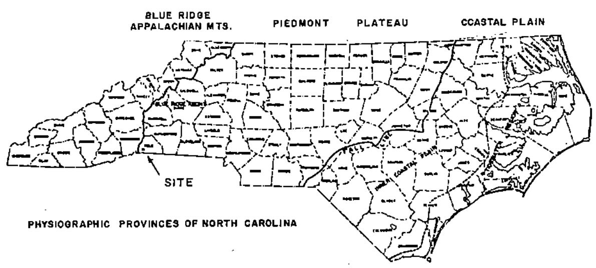
Pzpe — Paleozoic and Precambrian. Gneiss and schist.

PRECAMBRIAN

pЄu — Upper Precambrian. Ocoee and related metasedimentary and metavolcanic rocks.

pЄl — Lower Precambrian. Predominently mica gneiss, hornblende gneiss and mica schist.

pЄi — Precambrian granite and granite gneiss.



PHYSIOGRAPHIC PROVINCES OF NORTH CAROLINA

ADAPTED FROM THE
GENERALIZED GEOLOGIC MAP
OF
NORTH CAROLINA

DEPARTMENT OF CONSERVATION AND DEVELOPMENT
DAN E. STEWART, DIRECTOR

DIVISION OF MINERAL RESOURCES
STEPHEN G. CONRAD, STATE GEOLOGIST

1966

SCALE IN MILES
40 0 40 80

LAW ENGINEERING TESTING CO. CHARLOTTE, NORTH CAROLINA	SITE LOCATION MAP		
I-26 LANDSLIDE POLK COUNTY, NORTH CAROLINA	DWN. BY	CDH	SCALE:
	CKD. BY		DRAWING NO.
	APPR'D.		FIGURE 1

DUNCAN-PARNELL, INC., CHARLOTTE

The age and origin of the Blue Ridge topographic front overlooking the Piedmont Plateau has not been adequately explained.

B. Regional Weathering Profile Characteristics

The weathering profile below is not unique to the southern Piedmont, but is present in residual soils derived from in-place weathering of igneous and metamorphic rocks found on almost every continent. The generalized weathering profile generally contains the following elements (Sowers, 1954):

1. The Upper Zone - A crust of red sandy clays;

2. The Intermediate Zone - A thick zone of micaceous sandy silts and silty sands (saprolites);

3. The Partially Weathered Zone - A transitional material between the Intermediate Zone and the unweathered rock, consisting of gravelly micaceous silty sands and sandy silts with lenses of relatively sound rock.

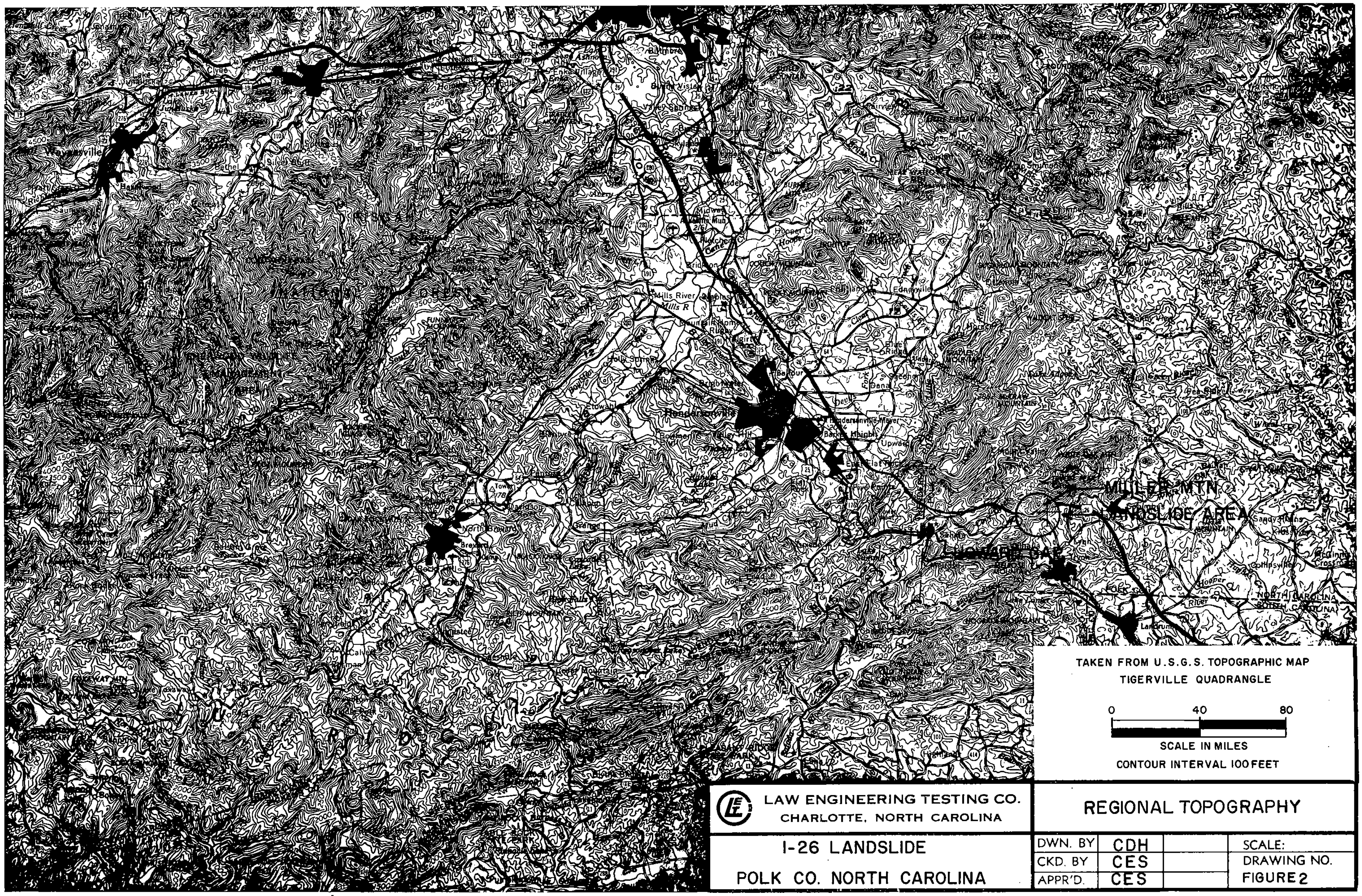
Soils of the Upper zone were not identified at the site except as colluvial material but, in an undisturbed state, are usually homogeneous, with little or no evidence of the structure of the rock from which they were derived. The Intermediate Zone, as implied, is formed by the incomplete weathering of the rocks. The soils still retain many of the original rock's characteristics - joints, faults, banding, etc. (Soils of the Intermediate and Partially Weathered zones may properly be termed saprolites). An important characteristic of the saprolites is their content of various types of weathered and unweathered mica, from the original rock, including muscovite, biotite and vermiculite (Sowers, 1954). The original rock in most areas is foliated. Some of the mineral bands may be almost all mica, oriented with planes of cleavage parallel to foliation. The strike and dip of the saprolite foliation reflects the local structure, which is usually complex.

The most important factors affecting the depth of weathering are the composition of the rock and the abundance of joints and faults. Some of the bands of rock minerals may be more resistant to weathering than others. The fissures allow surface water to percolate deeply into the rock mass, thus developing weathered zones. These factors combine to cause the depth of weathering to be irregular and erratic. The boundary between the Intermediate or Partially Weathered zone and the rock is, thus, not a sharp one. The degree of weathering decreases with increasing depth until the rocks become sound with only occasional zones of partial weathering, and at greater depths are completely unweathered.

III. LOCATION AND TOPOGRAPHY

Interstate 26 will extend from Charleston, South Carolina in the Coastal Plain, to Greenville/Spartanburg in the Piedmont of South Carolina, and then on to Asheville in the North Carolina mountains. Just inside North Carolina, as shown on Figure 2, the highway leaves the Piedmont at elevation 1500±and begins climbing to a higher plateau, elevation 2000 - 2500. Most of this elevation change takes place where the road ascends the southern slope of Miller Mountain. The highway climbs from elevation 1100 to about elevation 1850 at Howard Gap over a distance of about 12,000 feet.

Miller Mountain/Tryon Peak/White Oak Mountain and the higher mountains to the southwest rise from a plateau which has elevations similar to those of the Blue Ridge drainage areas (including the Asheville Plateau) to the northwest, and then descend to lower elevations to the southeast (the Piedmont). However, the plateau in this area and the associated mountains have been cut off from the true Blue Ridge drainage divide (to the Gulf of Mexico) by the Green River, which drains to the Atlantic. The site area, then, consists of a topographic outlier of the Blue Ridge front, and may at one time have been in the drainage divide between the Gulf and Atlantic regions.



TAKEN FROM U.S.G.S. TOPOGRAPHIC MAP
TIGERVILLE QUADRANGLE

0 40 80

SCALE IN MILES

CONTOUR INTERVAL 100 FEET

LAW ENGINEERING TESTING CO.
CHARLOTTE, NORTH CAROLINA

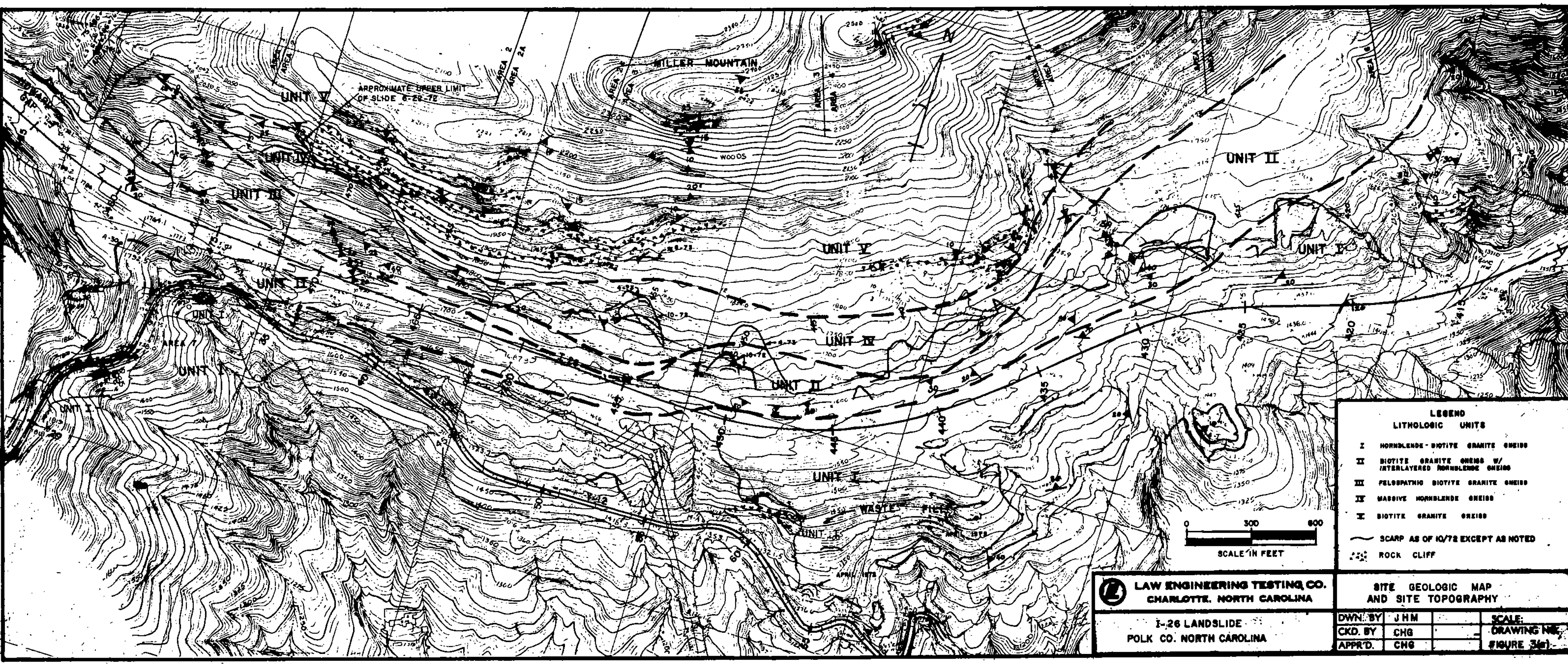
REGIONAL TOPOGRAPHY

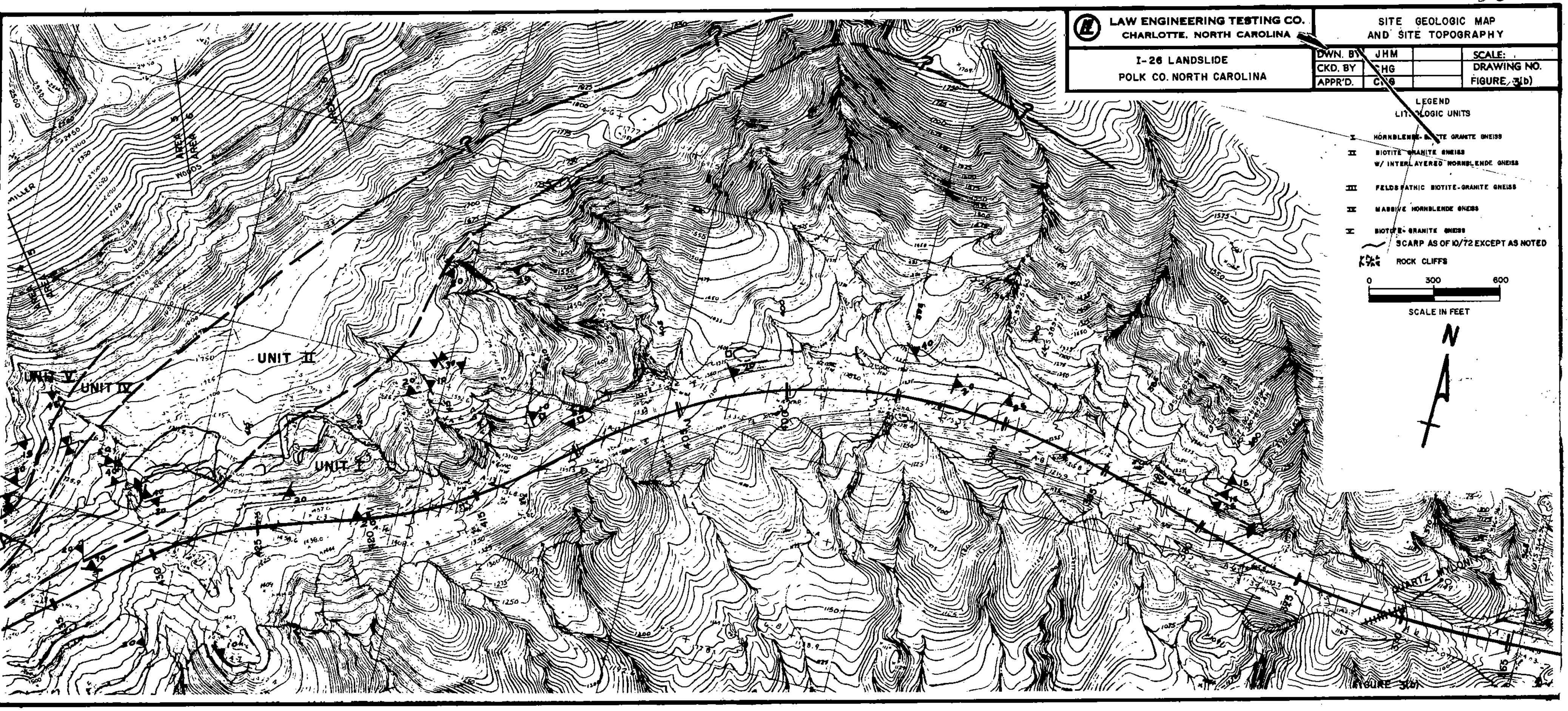
I-26 LANDSLIDE	DWN. BY	CDH		SCALE:
	CKD. BY	CES		DRAWING NO.
POLK CO. NORTH CAROLINA	APPR'D.	CES		FIGURE 2

The south slope of Miller Mountain is characterized by a general dissection into ridges and hollows sloping southward and by several levels of sloping terraces and steeper scarps. The development of surface drainage is notably absent on the terraces. The topography of the site is shown on Figures 3 (a) and 3 (b). As studies subsequently described herein showed, the slope can be classified as a colluvial slope - or one which has had a geologic history of mass wasting. Some areas of the mountainside have hummocky ground and bent tree trunks, evidences of natural instability.

The topography of the south slope of Miller Mountain has features that are both puzzling and tantalizing. Engineers and geologists would like to be able to better understand the relationship between geologic processes and hillside forms. However, such understanding is hampered by the brief span of geologic time that has been available for the actual observation of slope evolution.

There is evidence in the literature that some geomorphologists believe slopes may form in a discontinuous process. A "threshold of stability" (implying factors of safety close to unity) is exceeded at intervals (factor of safety less than unity). A period of stability then ensues. However, hillslope evolution may be affected by non-geomorphic factors, including climatic change (and human activity). Characteristic slope angles (those that frequently occur) of an area are thus related to local conditions and morphological history, and are not intrinsic features of slope development.

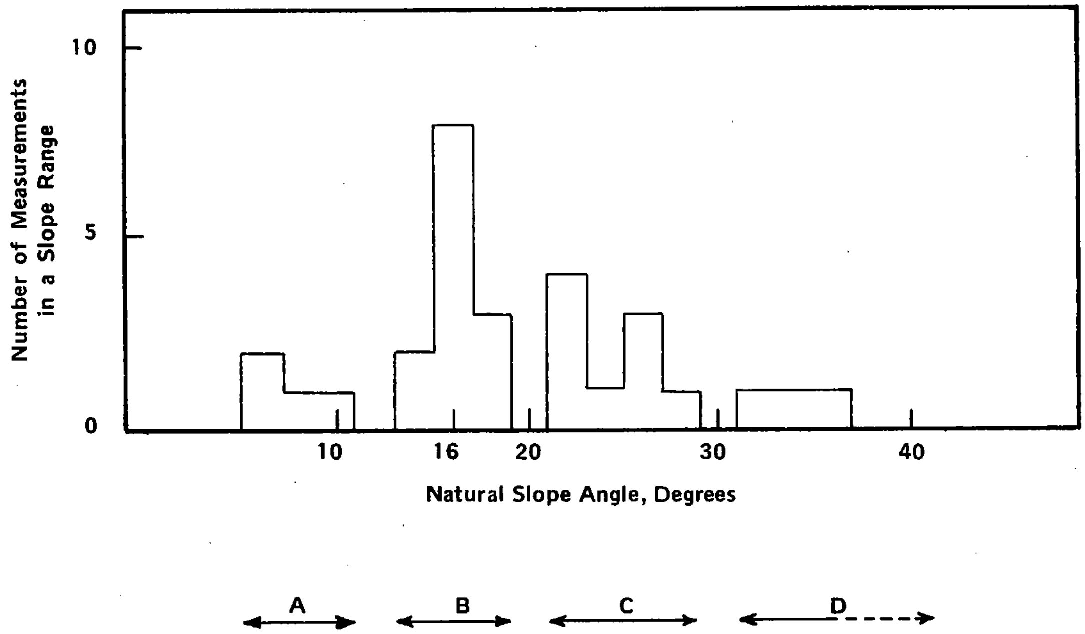

LEGEND
LITHOLOGIC UNITS
I HORNBLENDE - BIOTITE GRANITE GNEISS
II BIOTITE GRANITE GNEISS W/ INTERLAYERED HORNBLENDE GNEISS
III FELDSPATHIC BIOTITE GRANITE GNEISS
IV MASSIVE HORNBLENDE GNEISS
V BIOTITE GRANITE GNEISS
SCARP AS OF 10/72 EXCEPT AS NOTED
ROCK CLIFF
0 300 600
SCALE IN FEET
LAW ENGINEERING TESTING CO.
CHARLOTTE, NORTH CAROLINA
SITE GEOLOGIC MAP
AND SITE TOPOGRAPHY
I-26 LANDSLIDE
POLK CO. NORTH CAROLINA
DWN. BY J H M
CKD. BY CHG
APPR'D. CHG
SCALE:
DRAWING NO.
FIGURE 3(a)
MILLER MOUNTAIN
APPROXIMATE UPPER LIMIT OF SLIDE 6-22-72
UNIT V
UNIT IV
UNIT III
UNIT II
UNIT I
WOODS
WASTE FILL
AREA 2
AREA 2A
AREA 3


LAW ENGINEERING TESTING CO.
CHARLOTTE, NORTH CAROLINA
I-26 LANDSLIDE
POLK CO. NORTH CAROLINA
SITE GEOLOGIC MAP
AND SITE TOPOGRAPHY
DWN. BY JHM
CKD. BY HG
SCALE:
DRAWING NO.
FIGURE 3(b)
LEGEND
LITHOLOGIC UNITS
I HORNBLENDE-BIOTITE GRANITE GNEISS
II BIOTITE GRANITE GNEISS W/ INTERLAYERED HORNBLENDE GNEISS
III FELDSPATHIC BIOTITE-GRANITE GNEISS
IV MASSIVE HORNBLENDE GNEISS
V BIOTITE-GRANITE GNEISS
SCARP AS OF 10/72 EXCEPT AS NOTED
ROCK CLIFFS
0
300
600
SCALE IN FEET
N
UNIT V
UNIT IV
UNIT II
UNIT I
FIGURE 3(b)

The climatic and topographic conditions in the southern Piedmont are well-suited to rapid and deep weathering of the igneous and metamorphic rocks of the region. This is because of humid conditions, with rainfall distributed throughout the year, and mild temperatures, which tend to promote chemical weathering (Sowers, 1954).

In their significant paper on slope stability in residual soils, Deere and Patton (1971) conclude that "landslides are a common and perhaps the predominant method of slope development in areas of deep residual soils, (2) they are associated with characteristics of the weathering profile, (3) they are especially common during periods of heavy rainfall and/or earthquakes - ". This process, then, naturally yields an often present mantle of colluvial and slope wash material in an area of geologically active slope formation where the rate of weathering exceeds the rate of removal by erosion.

It is beyond the scope of this paper to attempt to discuss the slope morphology of Miller Mountain in detail. However, Figure 4 has been prepared to briefly describe some observed characteristic slope angles that appear on the mountainside, and are believed to have relevance to the design and construction of I-26. Additional discussion of the topographic terraces or "benches" is included in the section on Site Geology, along with description of the Geologic Units I, II, III, IV and V shown on Figures 3 (a), 3 (b) and 4.



Region	Generalization of Slope-Forming Material and Characteristics at 14 Cross Sections Examined, Stations 420 to 480±
A	More Than About 60 ft of Colluvium and Highly Weathered/Closely Jointed Saprolite Over Geologic Units II-III-IV.
B	Less Than About 60 ft of Colluvium or Less Weathered But Generally Moderately to Closely Jointed Saprolite Over Geologic Units II-III-IV.
C	Similar to Region B, But Sometimes Thinner Overburden. Indications of State of Natural Instability in Some Locations From Inclinometer Instrumentation and Visual Evidences of Natural (Pre-construction) Landslides.
D	Higher Elevations on Mountainside, Generally Composed of Geologic Unit V.

LAW ENGINEERING TESTING CO. CHARLOTTE, NORTH CAROLINA	Interpretative Observations of Natural Slope Angles-Miller Mountain		
I-26 Landslide Ares Polk County, North Carolina	DWN. BY		SCALE:
	CKD. BY	CES	DRAWING NO. FIGURE 4
	APPR'D.	CES	

IV. INVESTIGATIVE TECHNIQUES

During construction of I-26 on the Miller Mountain area near Howard Gap in late 1968, cut slope slides were released in the saprolite and colluvium of the slope. These slides were investigated, on a preliminary basis, by the geologists of the current North Carolina Department of Transportation, Division of Highways. However, additional slides developed as the excavation was shifted to stations further down the grade, involving a total length of approximately 1 1/4 miles of roadway. By July of 1969, all of the movements subsequently reviewed in engineering studies had been initiated. The preliminary studies conducted by the Highway Geologists, and the spread of the movements, led the Department of Transportation to decide that a delay in construction was required to study the problems. A consultant was selected, and a second study was performed and reported in September, 1970. The second study required about 3 months field work, primarily test boring and probing. After considering the findings of the second study, it was concluded that a more detailed and in-depth study was required to obtain additional information and thus increase the level of confidence concerning the understanding of the causes of the landslides.

The investigation of the problem by Law Engineering Testing Company began in August, 1971 and continued over a span of approximately one year. For the investigation, use was made of all the previous data, along with surface and aerial reconnaissance, geologic mapping, airphoto studies (using both black and white and false color infrared photographs), and monitoring movements of stakes driven on the ground surface, in addition to open test pits and test drilling and sampling of the soil and rock materials. Approximately 67 borings were made for the Law Engineering study, including borings for piezometers. Split barrel drive samples, penetration tests, undisturbed samples, and rock coring were performed in the drill holes. A special orienting core barrel was used in four of the borings to determine the strike and dip of rock structure within the boreholes. In 32 of the test borings, inclinometer casing was installed and monitored for deflection with the lower end anchored in the relatively sound rock. Water level monitoring was done throughout the course of the investigation. An automatic continuous water level recorder was used to monitor water levels at one piezometer boring. In addition to the borings, a considerable number of open test pits and trenches were made to a maximum depth of about 13 feet using a tractor-mounted backhoe and in one case, a bulldozer. These excavations allowed inspection of the toes of some sliding masses, and were made at locations where slip surfaces would be exposed. Eight to 15 inch size block samples containing preconstruction slip surfaces were hand-cut from test pits at 7 locations for laboratory direct shear testing. To supplement the laboratory shear strength tests, an in-place direct shear strength test was made on a micaceous foliation zone using an 18 inch square split shear box. (Another kind of in-situ shear strength test, the Iowa Bore Hole Shear Test, was performed in borings drilled into the Waste Fill Area for the project.

Discussion of the Waste Fill problems is outside the scope of this paper).

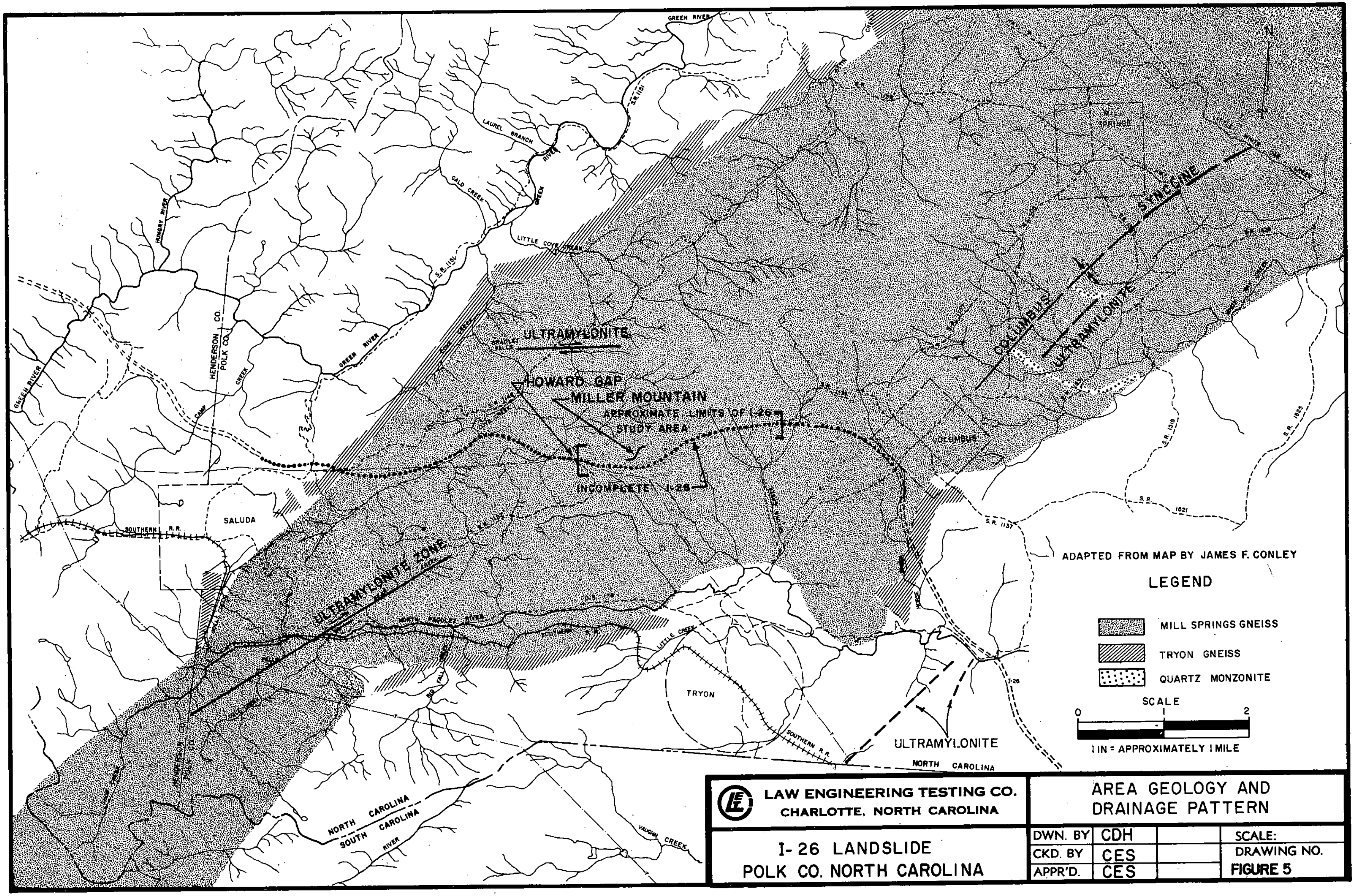
For the laboratory testing program, the ordinary classification tests, including grain size distribution, Atterberg limits, moisture content and, for the undisturbed and block samples, void ratio, specific gravity and wet density, were made. Direct shear strength tests were made on the block samples containing preconstruction and construction slip surfaces, and on some undisturbed samples. Triaxial shear strength tests were also made, but emphasis was placed on the results of the direct shear tests for the studies of cut slope slides. These data are summarized in Table 4, and were used in the geotechnical engineering analyses of stability and the effects of remedial measures.

V. AREA GEOLOGY

Figure 5 shows the general geology of the site area, adapted from work by Mr. James F. Conley, as reported by Drummond and Conley (undated). The geologic information has been superimposed on a drainage map, constructed by Law Engineering from airphotos. The Howard Gap-Miller Mountain area is underlain by the Mill Springs Gneiss, flanked by Tryon Gneiss and containing masses of quartz monzonite. The Mill Springs Gneiss consists of a group of various types of hornblende gneisses.

Structurally, the Howard Gap-Miller Mountain site area lies on the west flank of a large northeast trending syncline which Conley calls the Columbus Syncline. This structural position indicates that the rocks in Miller Mountain have a general northeast trend in outcrop pattern and a southeast trend in dip, toward the axis of the syncline.

Conley mapped several faults in Polk County, exhibited as mylonite and ultramylonite zones and interpreted as having primarily strike-slip movement. Conley believes that these faults are Triassic in age (approximately 180 million years old). Conley has traced the longest of these faults, the "Bear Creek Ultramylonite Zone", for nearly 8 miles with 2.5 miles being in Polk County. This fault is evidenced by the long linear section of Bear Creek. The direction of this



LAW ENGINEERING TESTING CO. CHARLOTTE, NORTH CAROLINA	AREA GEOLOGY AND DRAINAGE PATTERN			
I-26 LANDSLIDE POLK CO. NORTH CAROLINA	DWN. BY	CDH		SCALE:
	CKD. BY	CES		DRAWING NO.
	APPR'D.	CES		FIGURE 5

lineation is generally toward and parrallel to the Miller Mountain ridge line. However, in our opinion, based on our detailed reconnaissance and the data from this investigation, there is no direct relationship between tectonics of these faults and I-26 Miller Mountain slope stability. The faults are healed by recrystallization of the crushed material. The healed (recrystallized) fault rock is usually highly jointed, with joint spacings of 2 inches to 6 inches. Conley has also mapped soil displacements (one on Howard Gap Road between Saluda and I-26 and the other on US 176 south of Saluda) which he interprets as faults. However, in the light of present knowledge and observations during the investigation of the I-26 slides, it is the opinion of the writers that the phenomena designated as "recent faults" are very localized gravity sliding of the soils (landslides), similar to the preconstruction landslides documented at the I-26 site.

VI. SITE GEOLOGY

A. Lithology

In addition to geologic reconnaissance of the area, the second author performed detailed geologic mapping of the Mill Springs Gneiss at the I-26 site during the course of the investigation. On the basis of this mapping, and lithologies encountered in the drill holes, five fundamental rock groups were selected for mapping as an aid to understanding the landslide problems. The interpreted outcrop of the five rock groups beneath the cover of colluvium is shown on previously presented Figures 3 (a) and 3 (b). A description of each of these units, in topographically descending order, is given in Table 1.

B. Soils

Two basic soil types were found by the investigation: (1) colluvium, or material that has moved down the slope naturally by landsliding or creep; (2) saprolite, or in-place weathering products of the parent rock (Intermediate and Partially Weathered zones) still retaining the relict rock structures and banding. The Upper Zone could not be identified as undisturbed material at the site. The colluvium can be further subdivided into three types, often gradational into each other and not always having clear visual boundaries between them and between colluvium and in-place materials: (a) red boulder colluvium, (b) disaggregated saprolite, and (c) sliding (dislocated) blocks of jointed saprolite, including materials from the Partially Weathered zone.

TABLE 1

SITE LITHOLOGY

LITHOLOGIC UNIT	LOCATION	DESCRIPTION	GEOMORPHIC AND ENGINEERING RELATIONSHIPS
V	Above Slide Area.	Biotite-Granite Gneiss Light gray, medium grained Massive Joints spaced 5' to 20'	Rock cliffs above roadway No landslides. Minor natural rock falls from cliffs. Upper limit of landslide area.
IV	UNITS IN SLIDE AREAS	Hornblende Gneiss Dark Gray to Black, Massive (not thinly layered) Joints spaced 2' to 5'	Forms large boulders at head of highest slides. Hgn boulders abundant in red colluvium in and below outcrop area.
III		Feldspathic Gneiss White to Light grey, medium to coarse grained Occasional Biotite seams Joints spaced 1' to 3'	Discontinuous, forms upper part of two major slide areas. Slippage on weathered Biotite seams.
II		Thinly interlayed: Biotite Gneiss and Hornblende gneiss. Abundant Biotite seams Joints spaced 4" to 1'	Primary unstable unit, numerous slides. Slippage on weathered Biotite seams Relatively deeply weathered, underlies large benched area on mountainside.
I	Mostly Below Slide Area	Hornblende-Granite Gneiss (characterized by disseminated hornblende crystals) Light to medium grey, coarse grained, poorly foliated.	Usually stable, minor sliding at isolated mica seam locations. Sometimes forms rock cliffs below the I-26 roadway.

5/74
CHG

The red boulder colluvium typically consists of structureless clayey sand and silt usually containing jumbled soft to hard gravel to large boulder-sized rock fragments, hence the designation "red boulder colluvium". These reddish colluvial soils are practically universal in occurrence at the site, except for the exposed resistant Unit V and Unit I rock bluffs. The thickness of the red boulder colluvium, the soil portion of which classifies as A-4, A-6 or A-7-5 by the AASHO system, varies from a few inches to about 80 feet. The thicker red boulder colluvial deposits occur in troughs or pockets, probably formed by pre-colluvial landslides and subsequently filled. Some of the materials classified as red boulder colluvium during the investigation may actually be slope wash. A 1 inch to 2 foot thick zone of very clayey silt often exists at the base of the red boulder colluvium.

The disturbed saprolites vary from large blocks with dimensions of tens of feet (dislocated saprolite blocks) to strongly distorted soils with visible planes of dislocation only a few centimeters apart.

The thickness of the weathering profile is quite variable and erratic, due to lithology and jointing. Texturally, the saprolites of the Intermediate and Partially

Weathered Zones range from bands or zones consisting almost entirely of silt- to sand - sized mica flakes to zones of sandy silts and silty sands, and gravel - to boulder - sized pieces of incompletely weathered rock.

C. Site Geologic Structure

The rock layers beneath the site have a northeast trend in strike. Rock dip directions are complexed by a system of northeast trending gentle folds, causing the rocks to dip to the southeast (toward the road) in some areas and to the northwest (into the mountain) in other areas. In general, southeastward dips are predominant, toward the axis of the Columbus Syncline southeast of the site. (Figure 5).

The axial planes of these folds are nearly vertical and are spaced on the order of 75 feet to 150 feet apart. These anticlines and synclines plunge gently to the northeast (about 10 to 20 degrees), so that the rocks dip toward the northeast around the noses of the folds. An additional cross fold system with fold axes bearing northwest or west-northwest is inferred from field observations and structural measurements. This gentle cross fold system, though not commonly seen, does occasionally produce gentle dips towards the southwest. The cross folds are relatively small ripples on the side of a large southeast dipping structure.

The degree of dip of the rock layers varies from 0 degrees (along fold axes) to vertical, and even an overturned structure is visible at one location on the Miller Mountain ridge crest above the slide areas. However, predominantly the dip of the rock layers in and around the slide areas is about 20 degrees.

Rock joints at the site have predominantly steep dips, in the range of 70 to 90 degrees. Several joint sets are present, with statistical analyses yielding the following primary joint strike directions: a) N20-30W, interpreted as dip tension joints, b) N60-70E, interpreted as strike tension joints, c) N60-75W, interpreted as oblique or shear joints. A more gently dipping joint set, striking about N70E and dipping about 50 degrees SE is often present, but joints in this set are not as abundant as the other sets.

Joint spacing varies widely on the site but appears to follow a certain pattern. In the biotite-granite gneiss (V) rock bluffs above the I-26 cuts, joints are relatively widely spaced, on the order of 5 to 20 feet apart. The massive hornblende gneiss (IV) beneath the biotite granite gneiss, where seen, appears to have joint spacing on the order of 2 to 5 feet. In the feldspathic biotite-granite gneiss, (III) joints appear to be spaced from 1 to 3 feet, and in some exposures the west northwest trending joints are spaced only a few centimeters to a few inches apart. In the interlayered amphibolite biotite-granite gneiss (II) joints are closely spaced, on the order of 4 inches to 1 foot, and the amphibolite layers typically have a blocky appearance. Jointing in the lowest unit, the hornblende-biotite grantie gneiss (I) appears quite variable from limited surface exposures but tends to be in the 3 to 10 foot range.

It appears that joint spacing and alignment have exercised major topographic controls at the site and in surrounding areas. First, the ridge line, the rock bluffs, and the hollows and secondary drainage divides are lineated parallel to the main jointing directions. Second, the higher rock members forming the rock bluffs and steepest mountain slopes appear to owe their mountain-supporting ability to wide joint spacing which makes them less susceptible to chemical weathering. Third, Units I and IV are more resistant to weathering than Units II, III and IV; thus, topographic benches have been formed between Units I and V.

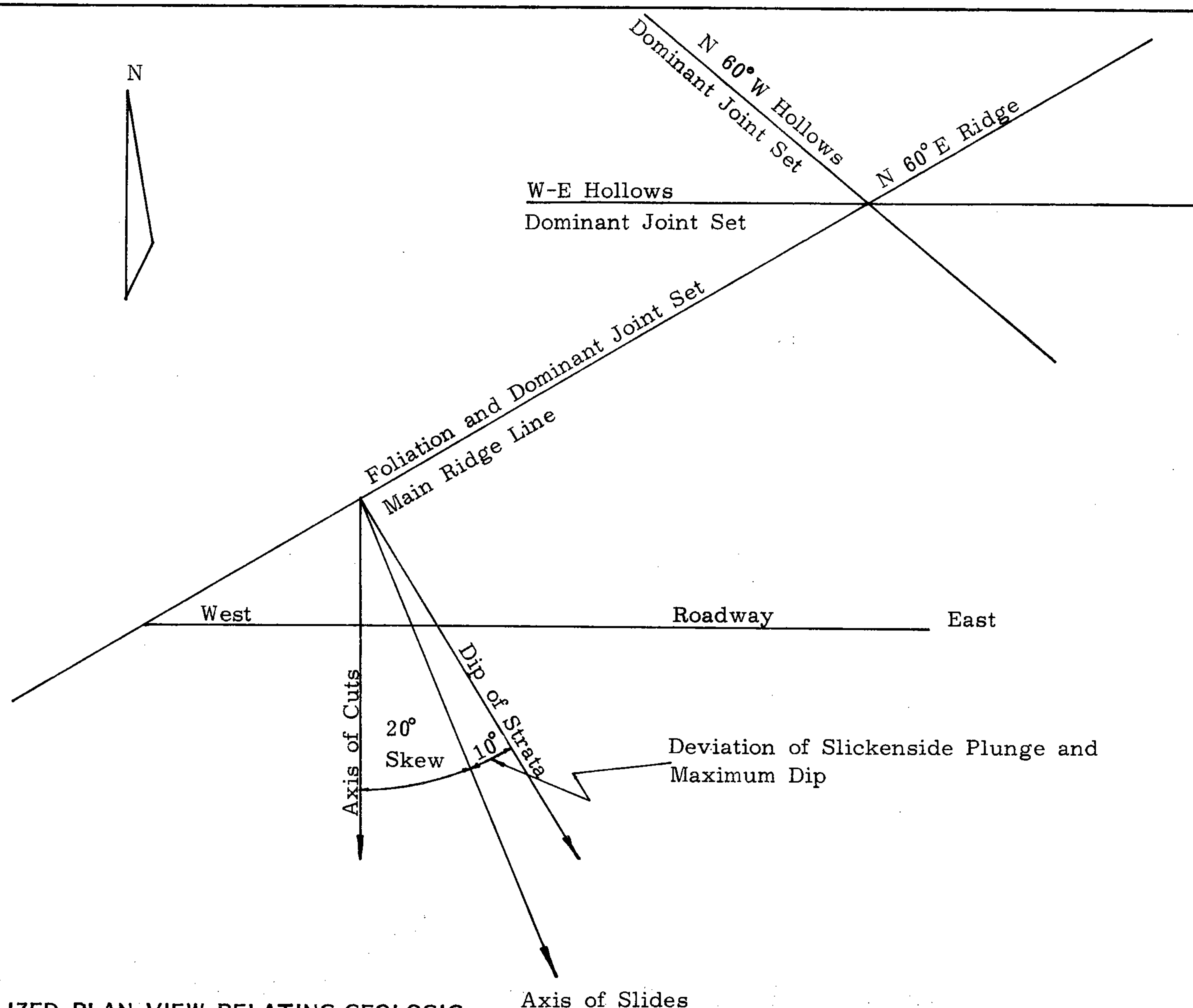
The only tectonic faulting evident at the site consists of displacements which occurred during or shortly after metamorphism, when the rocks were deeply buried, so that the faults are completely healed and are of no direct engineering significance. Faults of very small displacement were seen in a few rock exposures. An extension of a large fault mapped by Conley is indicated in Howard Gap proper by fracture-filling euhedral quartz crystals. Additionally, mylonite is found in the I-26 roadway at the foot of the mountain near the old Skyuka Road Bridge. Though fractures associated with faults such as these may affect the rate of weathering, depth of soil development, and topography, there is no evidence of movement along these zones for the past 150 to 200 million years.

There could be speculations that the steep mountain side of Miller Mountain, the rock bluffs, and the benched topography on the mountainside are the result of faulting. However, evidences of such faulting, such as breccia, gouge, slickensides, or highly fractured and deeply weathered zones, are lacking. Even though the primary origin of Miller Mountain and the associated mountains in the area has not been adequately explained, in our opinion, the topography of the mountain-side can be explained by a combination of joint spacing, differential rock weathering, and natural landslides and colluvial deposition.

There is a profound direct relationship between the site geology, weathering, and the landslides. Weathered mica seams (predominantly in Unit II, but occuring in Units III and IV) which dip at a low angle toward the roadway yield a condition conducive to sliding failures. Ancient pre-construction gravity mass movements (landslides) in residual soils created benches which accumulated deposits of inherently unstable colluvium. Weathering and ancient sliding of Units II-III-IV produced topographic benches that influenced the selection of the I-26 route up the side of Miller Mountain. Thus, though there is nothing geologically unique in either the structure or lithology of the site, a complex combination of geologic conditions materially contributes to the landslide problems.

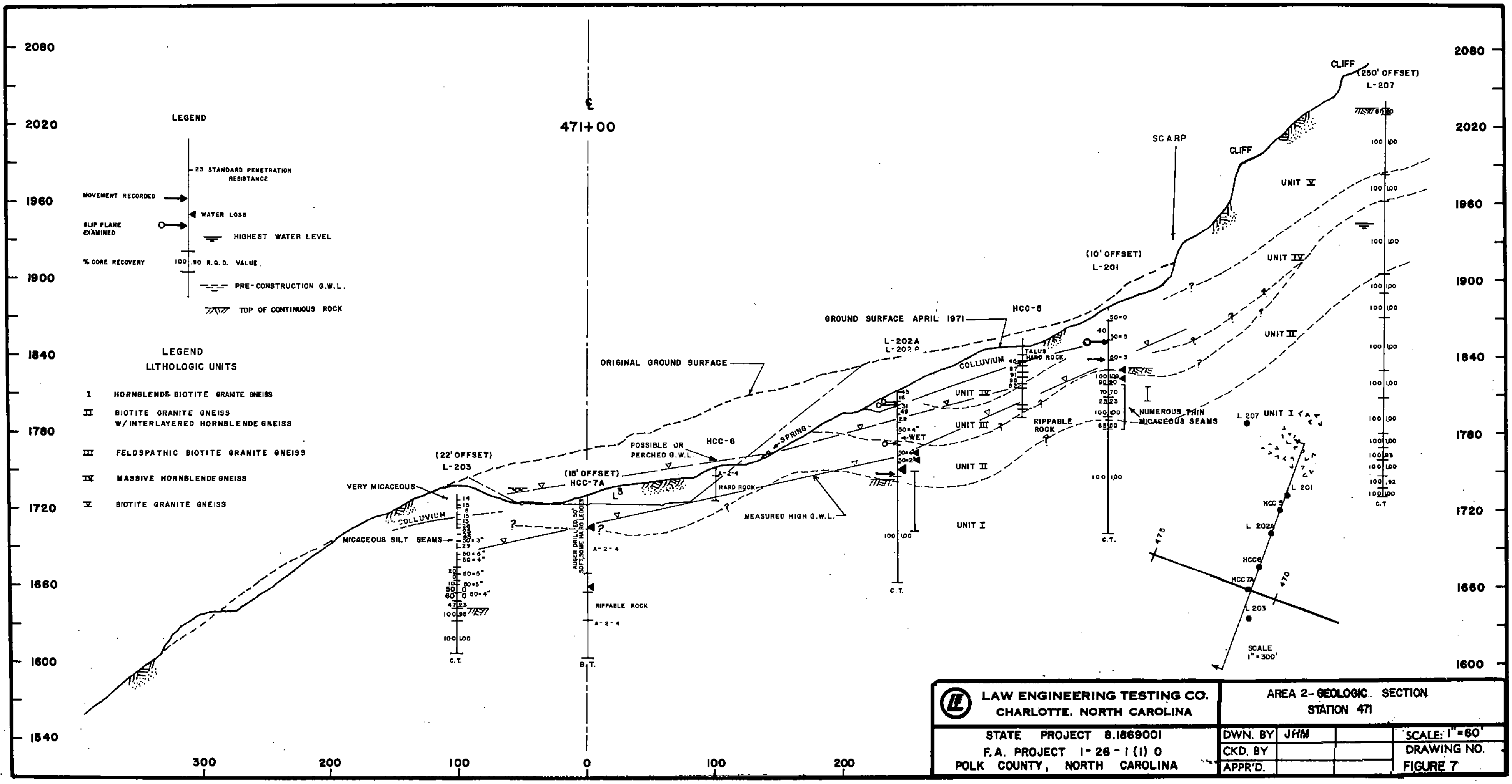
Figure 6 is a generalized plan view relating the geologic structure to topography and to the orientation of the slides.

Figure 7 is a geologic cross-section of one of the slide areas. It illustrates the subsurface lithology and structure. Specific sliding surfaces are indicated.



GENERALIZED PLAN VIEW RELATING GEOLOGIC STRUCTURE TO TOPOGRAPHIC LINEATIONS AND SLIDE ORIENTATIONS - I-26 LANDSLIDES

LAW ENGINEERING TESTING CO. CHARLOTTE, NORTH CAROLINA					
STATE PROJECT 8.1869001 F.A. PROJECT I-26-1(1) 0 POLK COUNTY, NORTH CAROLINA	DWN. BY			SCALE:
	CKD. BY			
	APPR'D.	CHG		FIGURE 6



LAW ENGINEERING TESTING CO.
CHARLOTTE, NORTH CAROLINA

STATE PROJECT 8.1869001
F.A. PROJECT I-26-1(1) 0
POLK COUNTY, NORTH CAROLINA

AREA 2-GEOLOGIC SECTION
STATION 471

DWN. BY	JHM		SCALE: 1"=60'
CKD. BY			DRAWING NO.
APPR'D.			FIGURE 7

Slickensided movement planes were found during geologic mapping at about 25 locations in the Miller Mountain cut slopes, both within and outside of active slide areas. Many of them showed evidences of pre-construction movement. Table 2 presents a classification of the directly observed slip planes with respect to types of materials and age of movement. It also shows that movement is primarily parallel to foliation, even where the sliding material consists of colluvium in contact with saprolite.

TABLE 2

CLASSIFICATION OF EXPOSED SLIP PLANES

I-26 LANDSLIDES

TYPE	TOTAL OBSERVED	AGE OF MOVEMENT				SLIP PARALLELS FOLIATION	AVERAGE SLIP PLANE DIP	AVERAGE STRIAE PLUNGE
		PRE-CONSTR. (1)	CONSTR. (2)	BOTH	UNKNOWN			
Red Colluvium/ Red Colluvium	4	3			1	1 (a)	28^0	28^0
Red Colluvium/ Saprolite	5	3	3	2	1	4	36^0	31^0
Saprolite/ Saprolite	16	11	8	3		14	27^0	18^0
Significant Totals	25	17	11	5		19		

(1) Evidences for pre-construction movement: a) Striated Mn-Fe secondary mineral coatings; b) slip-plane found outside of slide area.

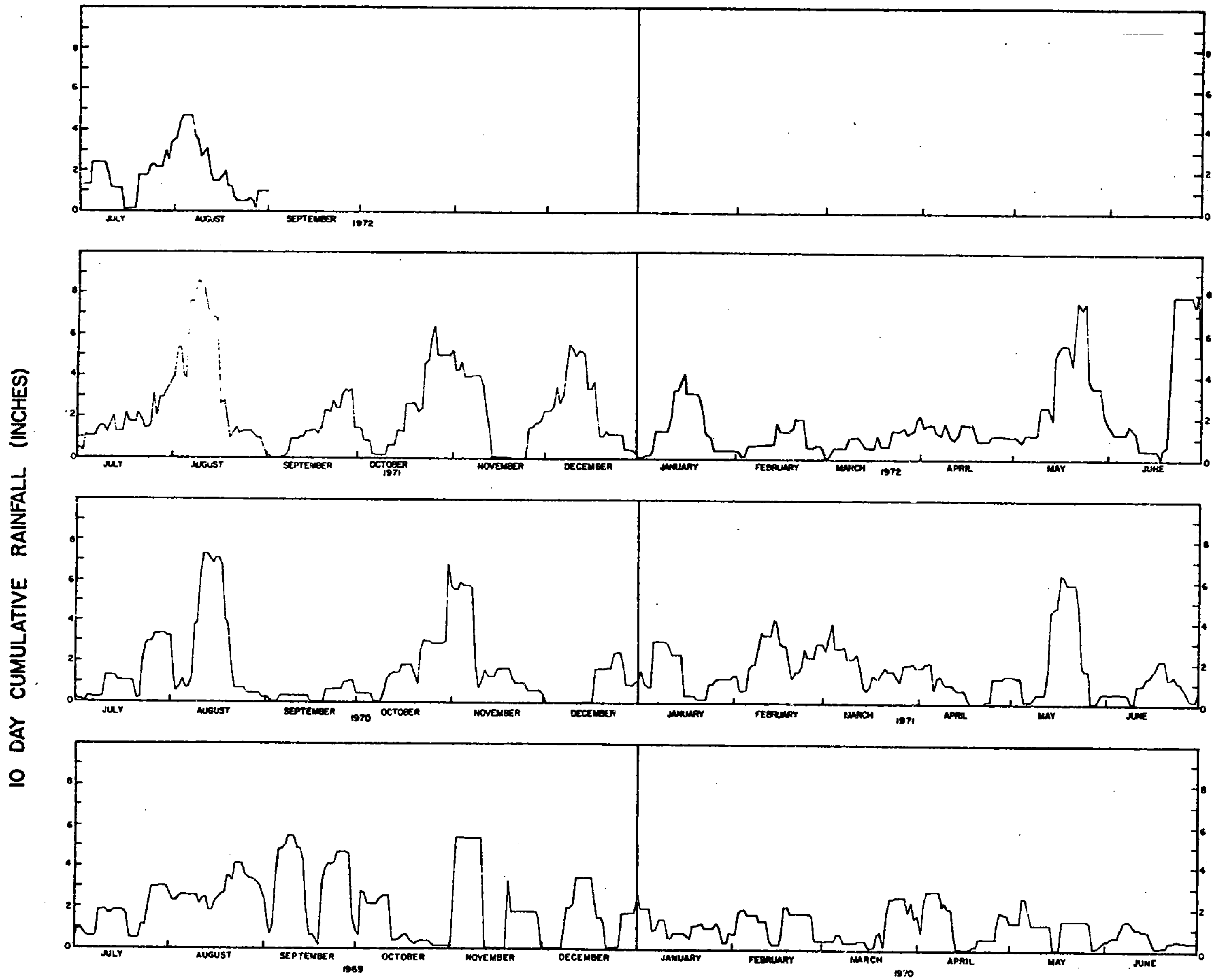
(2) Evidence for construction movement: obvious disruption of ground surface after grading was terminated.

(a) Parallels foliation in saprolite few inches below elevation of slip plane in colluvium.

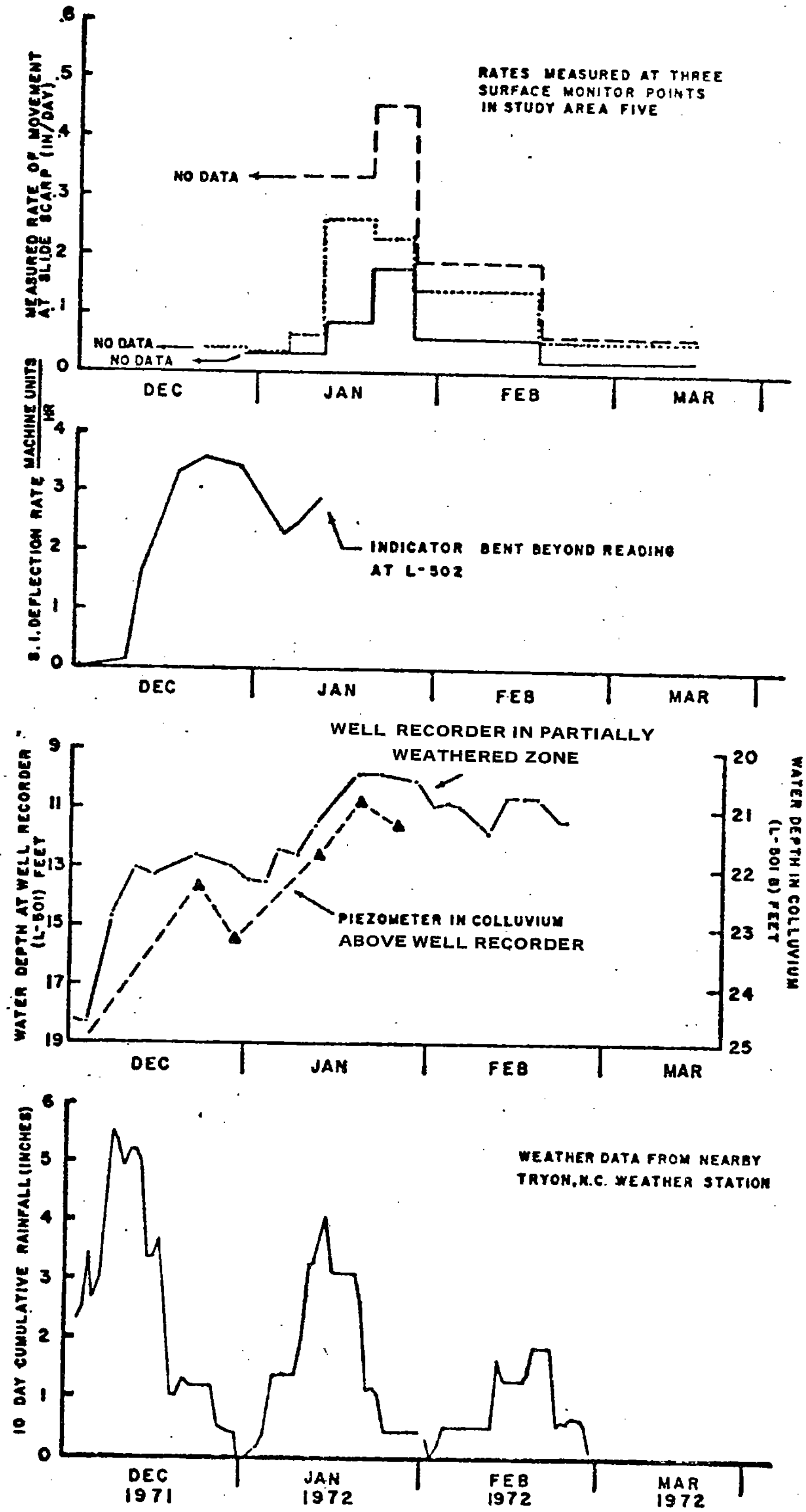
5/74
CHG

D. Rainfall, Drainage, and Groundwater

The mean annual rainfall for the mountain region of Polk County is about 63 inches (from Polk County Agricultural Soil Survey) as compared with about 53 inches for the Piedmont area. Precipitation is fairly well spread over the four seasons but usually is slightly greater and more intense in summer. Figure 8 represents the rainfall data from the nearby Tryon, North Carolina, weather station for the 3 year period beginning July 1969, through August, 1972. Comparison of data from a rain gage on the site during parts of the investigation with more complete data from the weather station confirmed that the weather station data represents closely the conditions at the site. The data on Figure 8 are plotted as the sum of the rainfall for the 10 days (an arbitrary choice) preceding the given date. Studies of slide movement rate versus rainfall have indicated it is the cumulative rainfall which has the most significant effects on movements of the slide masses. Figure 9 illustrates for a particular slide locality a distinct relationship between cumulative rainfall, groundwater levels (including an artesian condition), and movement rates measured by an inclinometer installation at depth and on the surface by measurement of the distance between pairs of stakes driven into the ground (one off the slide mass) at 3 different locations. It is interesting to note that the data on Figure 9 indicates a few day lag between the development of peak movement rates-groundwater levels and the peak cumulative 10 day rainfall. This implies that



LAW ENGINEERING TESTING CO. CHARLOTTE, NORTH CAROLINA	TEN DAY CUMULATIVE RAINFALL TRYON, POLK COUNTY, N.C.		
STATE PROJECT 8.1869001 F.A. PROJECT I-26-1(1) 0 POLK COUNTY, NORTH CAROLINA	DWN. BY		SCALE:
	CKD. BY	DGM	FIGURE 8
	APPR'D.	CES	



AREA 5-MOVEMENT RATE VS, CUMULATIVE RAINFALL & G.W.L.

MOVEMENT RATES AND EFFECT OF WINTER RAINFALL AT SELECTED LOCATIONS INVOLVING CONSTRUCTION SLIDES for this location, the cumulative rainfall over a period slightly longer than 10 days would correlate slightly better with the movements.

Because of the steepness of the upper mountain slopes, runoff is probably high. Surface drainage paths on the mountainside are restricted to relatively few deep hollows or draws, which suggests that much of the drainage occurs by sheetflow, and/or downward percolation.

Infiltration into the ground is aided by the forest and humus development, by the widespread and sometimes permeable colluvium, the terraced topography and on the higher slopes, by joints in the rock bluffs.

The basal part of the upper red boulder colluvium in some places contains concentrations of clay and silt which form an aquiclude and produce a perched water table in the colluvium and an artesian condition in the dislocated saprolites and rock beneath the colluvium, as shown by the data in the middle of Figure 9. Downhill drainage of groundwater in the colluvium is controlled by erratic variations in colluvium permeability. In the saprolite and rocks beneath the colluvium, drainage is controlled by jointing. The resulting groundwater configuration is somewhat complex. Springs are common on the mountainside during wet weather, but many of these abate during the dry seasons.

Groundwater was measured at depths of 0 to 25 feet below original grade near the centerline in a number of the preconstruction exploratory auger borings. Preconstruction auger borings and reconnaissance uphill of centerline in the eastern part of the site measured groundwater 0 to 15 feet below the original surface. Preconstruction auger borings uphill of centerline in other areas typically penetrated 10 to 40 feet, some encountering groundwater. The cut excavations along I-26 lowered the original groundwater table in some areas.

E. Specific Landslide Characteristics

The sliding problems involved a total length of approximately 1 1/4 miles of roadway, and thus it is impossible to describe and discuss the details of all the individual slides in this paper. Instead, we have summarized some of the pertinent characteristics of the slides on Table 3, including depth of excavation, depth of slide movement below original ground, sliding materials, groundwater, and computed safety factors. Depths of slide movement in Table 3 were determined by inclinometer measurements in all cases except study area 2A, where an estimate was made from observations in test pits dug into the slope. The strength parameters used for analysis were derived from the data presented on Table 4.

The pseudo-dynamic safety factors shown for several cases in Table 3 were calculated assuming a horizontal earthquake acceleration of 0.10 g. These calculations show that the factors of safety of the slopes are considerably reduced by the addition of earthquake loading. The studies of remedial measures showed that, at best, remedial construction would provide only marginal factors of safety, even without considering seismic loadings. It was concluded that, should the site be subjected to earthquake shocks of any duration, many of the slides will be displaced, requiring, as a minimum, repair of damage to remedial installations. However, many other highways in mountainous parts of the region would be similarly affected to varying degrees, and this problem would not be unique to I-26.

The slides were observed to be progressive with time, beginning near the excavation and preceeding upslope, as can be observed on Figure 3 (a) for the upslope limit of the slides at stations 450 and 455. These two slide masses seemed to advance up the mountainside with new cracks forming in increments of 150 to 200 feet horizontally.

TABLE 3

GENERALIZED DATA ON CUT SLOPE LANDSLIDES (1¼ Miles of Roadway)
I-26
POLK COUNTY, NORTH CAROLINA

SLIDE STUDY AREA NO.	TYPICAL COMPLETED DEPTH OF EXCAVATION BELOW ORIGINAL GROUND FT.	GREATEST MEASURED DEPTH OF MOVEMENT BELOW ORIGINAL GROUND FT.	PRINCIPAL SLIDING MATERIAL AT DEPTH	GROUNDWATER MEASURED ABOVE SHEAR ZONE	STABILITY ANALYSES			
						CALCULATED FACTOR OF SAFETY		
					SHEAR STRENGTH PSF/DEGREES	FAILURE TYPE	FS (Static)	FS' (Pseudo-dynamic)
1	75 ft. 25 ft.	75 ft. 55 ft.	MICACEOUS SAPROLITE UNIT II - PARTIALLY WEATHERED ZONE UNIT III/IV - INTERMEDIATE ZONE	YES YES	$C' = 150$ $\phi' = 14^{o}$	ROTATIONAL	.84/.90 †	
2	45 ft ±	92 ft.	UNIT II - MICACEOUS SAPROLITE PARTIALLY WEATHERED ZONE	YES	$C' = 400$ $\phi' = 16^{o}$	PLANAR	.91/1.06 †	≈ .80 †
	45 ft. ±	100 ft.	UNIT II/III - MICACEOUS SAPROLITE PARTIALLY WEATHERED ZONE	YES	ACROSS FOLIATION $C' = 900$, $\phi' = 19^{o}$ ALONG FOLIATION $C' = 400$ $\phi' = 16^{o}$	ROTATIONAL	.75/.99 †	≈ .80 †
2A	20 ft.±	15 ft - 25 ft.	COLLUVIUM - "DISAGGREGATED UNIT II - SAPROLITE" AND RED BOULDER	YES	------	--------	------	------
3	30 ft.	45 ft.	COLLUVIUM - "DISPLACED UNIT II - SAPROLITE" BLOCKS AND RED BOULDER	INTERMITTENT SPRINGS OBSERVED	$C' = 400$ $\phi' = 16^{o}$	ROTATIONAL	.91/1.01 †	< .80 †
	25 ft.	25 ft. (PRECONSTRUCTION SHEAR PLANES TO 50 ft)	COLLUVIUM - "DISPLACED UNIT II SAPROLITE AND RED BOULDER	INTERMITTENT SPRINGS	$C' = 400$ $\phi' = 16^{o}$	ROTATIONAL PLANAR	.97/1.03 † .87/.96 †	.80 ± †
4	15 ft.	30 ft. (FROM INSPECTION OF SAMPLES)	COLLUVIUM - RED BOULDER AND "DISPLACED UNIT II/IV SAPROLITE"	INTERMITTENT	$C' - 400$ $\phi' = 16^{o}$	ROTATIONAL PLANAR	.89/.97 † .92/1.08 †	

† COMPUTED FS ASSUMING NO WATER PRESENT OR REMOVED BY REMEDIAL DRAINAGE, GROUND SURFACE AS OF APRIL, 1971 UNLESS OTHERWISE NOTED.

TABLE 3, CONTINUED

GENERALIZED DATA ON CUT SLOPE LANDSLIDES (1¼ Miles of Roadway)

I-26

POLK COUNTY, NORTH CAROLINA

SLIDE STUDY AREA NO.	TYPICAL COMPLETED DEPTH OF EXCAVATION BELOW ORIGINAL GROUND FT.	GREATEST MEASURED DEPTH OF MOVEMENT BELOW ORIGINAL GROUND FT.	PRINCIPAL SLIDING MATERIAL AT DEPTH	GROUNDWATER MEASURED ABOVE SHEAR ZONE	STABILITY ANALYSES: SHEAR STRENGTH PSF/DEGREES	CALCULATED FACTOR OF SAFETY: FAILURE TYPE	FS (static)	FS' (Pseudo-dynamic)
4	30 ft.	40 ft.	DISPLACED SAPROLITE BLOCK	NOT DETECTED OR OBSERVED - EVIDENCE OF PAST GROUND-WATER FLOW IN OPEN JOINTS	C' = 400 $\phi' = 16^{o}$	PLANAR 25^{o} SLOPE AND PARALLEL TO ORIGINAL GROUND	.97 †	
5	45 ft.	60 ft.	COLLUVIUM - RED BOULDER AND "DISAGGREGATED/ DISPLACED SAPROLITE" UNITS II/IV	YES (ARTESIAN BELOW BASE OF COLLUVIUM)	C' = 450 $\phi' = 24^{o}$	ROTATIONAL	.78 *	
					C' = 400 $\phi' = 16^{o}$	ROTATIONAL	.78 ** /1.08 †	.89 †
					C' = 450 $\phi' = 24^{o}$	PLANAR 17^{o} SLOPE	1.13	
6	60 ft.	55 ft. (PRECONSTRUCTION SHEAR PLANES TO 95 ft IN PARTIALLY WEATH-ERED ZONE OF UNIT I?)	COLLUVIUM - RED BOULDER AND DISAGGREGATED SAPROLITE	YES (ARTESIAN LAYERS IN COLLUVIUM)	C' - 400 $\phi' = 16^{o}$	ROTATIONAL	.71/<1 †	
					C' = 450 $\phi' - 24^{o}$	PLANAR 24^{o}	.80 **	

† COMPUTED FS ASSUMING NO WATER PRESENT OR REMOVED BY REMEDIAL DRAINAGE, GROUND SURFACE AS OF APRIL, 1971 UNLESS OTHERWISE NOTED.

* ASSUMING PRECONSTRUCTION GROUNDWATER CONDITIONS AND INITIAL CUTTING WITH ORIGINAL TOPOGRAPHY.

** ASSUMING CONSTRUCTION GROUNDWATER CONDITIONS AND APRIL, 1971 TOPOGRAPHY (AFTER SOME SLUMPING).

TABLE 4
SHEAR STRENGTH PARAMETERS
I-26
POLK COUNTY, NORTH CAROLINA

IDENTIFICATION NUMBER	CONTACT MATERIALS IN SHEAR ZONE	STRENGTH PARAMETERS		REMARKS	SLOPE REGION ON FIGURE 4
		C' PSF	ϕ' DEGREES		
1B	Silty And Sandy Sand - Sized Mica Flakes	500	16.5°	Block Sample - Slide Surface Direct Shear Test Parallel To Foliation	C
1D	Thin Red-Brown Sandy Clayey Silt Sliding On Mica	0	21.4°	Block Sample - Direct Shear - Slide Surface	A/B
2A	Red Clayey Sandy Silt	510	15°	Block Sample - Direct Shear - Slide Surface	B
3A	Sandy And Silty Sand - And Silt - Sized Flakes Of Mica	690	18.3°	Block Sample - Slide Surface - Direct Shear Test Parallel To Foliation	B
3A	Sandy And Silty Sand - And Silt - Sized Flakes Of Mica	1780 $C'_r = 0$	12.5° $\phi'_r = 22°$	Block Sample - Slide Surface Direct Shear Test Across Mica Plate Orientation	B
L-304	Clayey Sand	1000 $C'_r = 600$	20° $\phi'_r = 20°$	Undisturbed Sample Of Deep Colluvial Deposit Direct Shear Test	A
L-304	Clayey Sand	580 $C'_r = 80$	27.6° $\phi'_r = 27.2°$	Undisturbed Sampled Of Deep Colluvial Deposit Direct Shear Test	A
3C	Red Clayey Silty Sand	500	14.5°	Block Sample - Direct Shear Test Of Slickensided Surface	C
4A	Silty And Sandy Sand - To Silt - Sized Mica Flakes	150	14.5°	Block Sample - Direct Shear Test Of Slide Surface, Parallel To Foliation	C

TABLE 4, CONTINUED
SHEAR STRENGTH PARAMETERS
I-26
POLK COUNTY, NORTH CAROLINA

IDENTIFICATION NUMBER	CONTACT MATERIALS IN SHEAR ZONE	STRENGTH PARAMETERS		REMARKS	SLOPE REGION ON FIGURE 4
		C' PSF	ϕ' DEGREES		
4A	Silty And Sandy Sand To Silt - Sized Mica Flakes	1029 C'_r = 180	16° ϕ'_r = 18°	Block Sample - Direct Shear Test Across Mica Plate Orientation	C
7A	Colluvial Clayey Silty Sand Sliding On Sandy And Silty Sand - To Silt - Sized Flakes Of Mica	260	16.5°	Block Sample - Direct Shear Test Of Slide Surface	--
7A	Colluvial Clayey Silty Sand Sliding On Sandy And Silty Sand - To Silt Sized Flakes Of Mica	220	14°	Block Sample - Direct Shear Test Of Slide Surface - From Another Place In Block	--
1B	Micaceous Foliation Zone In "Saprolite"	900	19°	18" Square Direct Shear Tests In Field	C
JOINT COATING FOUND IN A RESIDUAL SOIL, REPORTED IN LITERATURE					
	"Black Seams"	--- ---	ϕ_r = 10.5° ϕ_r = 14.5°	Joint Filling - Seam With Slickensides - Seam Without Slickensides C U Triaxial Tests	St.John, Sowers And Weaver (1969)
COMMON MINERALS FOUND IN RESIDUAL SOILS, REPORTED IN LITERATURE					
	Mica (Hydrous)	---	ϕ_r = 16° to 26°		Kenney (1967)
	Muscovite	---	ϕ_r = 17° to 24°		Kenney (1967)

C' = effective cohesion, ϕ' = effective angle of shear resistance ϕ'_r = residual angle of shear resistance

C'_r = residual cohesion

Note: Except as noted, all direct shear tests were on preconstruction or construction slip surfaces, and all C' and ϕ' values are believed to be representative of residual strengths.

VII. GENERAL FEATURES OF REMEDIAL DESIGN

When construction was halted, the excavation for the roadway was still considerably above design grade at a number of locations - up to 55 feet at the right edge of the roadway. The findings and analyses showed that further excavation into the mountainside to achieve the original design grade would aggravate existing construction slides and start new ones. It became apparent that a redesign of the grade and alignment in the affected areas was necessary to allow completion of the routes in the same corridor without either triggering much additional sliding or expending very large amounts of money to retain these masses.

The general guidelines for grade and alignment shifts for the entire area affected by slides or potential slides were as follows:

(1) Shift the roadway away from the mountainside slopes a sufficient distance to allow establishment of a design grade without significant additional excavation, avoiding the construction slides as much as practicable by leaving accessable space between the edge of the roadway and the toe of construction slide movements,and/or;

(2) Raise the design grade to allow construction of the roadway without significant additional excavation.

It was concluded that a redesign of the grade and alignment could best be accomplished by a team effort involving members of the geotechnical consulting firm, Law Engineering Testing Company, and designers and engineers of the State Highway and FHWA Organizations. This was accomplished by an iterative process, and produced a design meeting Interstate standards, yet accomplishing, to the greatest extent possible, the two objective redesign guidelines noted above.

Other major elements of the remedial design were (1) the excavation of slide materials from the head of the major slide mass immediately below the rock cliffs at stations 465-475 to reduce the driving forces; (2) complete excavation of the slide masses at stations 477-480 and at stations 450-455; (3) partial excavation of the colluvial slide mass at station 420; (4) provide surface drainage ditches well above the slopes to divert run-off; (5) utilize horizontal drilled drains to remove as much groundwater and perched subsurface water as possible from the saprolite and colluvial materials, thus attempting to reduce the rate of slide or potential slide movement and provide lower mainatenance costs.

It was not possible to avoid requirement of some additional excavation to achieve design grades in the area of stations 450-455. The use of tied-back cylinder pile walls socketed into rock upslope of the active slide masses to retain the materials further uphill was considered. Such walls would be quite expensive and subsurface investigation revealed a non-uniform depth to rock along the proposed wall alignment. However, a major factor entering into the final decision not to build the walls was the fact that fill materials would be needed to raise the design grade elsewhere. Therefore, the decision was made to excavate the sliding materials at station 450-455 and re-use the soils as embankment fill to raise the design grade elsewhere. It was recognized that portions of the excavated soil would not be desirable as embankment fill, either because of large boulders or excessive water content. The excavation bottom line will average 30 to 40 feet depth and will parallel the original natural slope of the mountainside for 700 to 900 feet uphill of the highway, or until the Unit V rock exposures are encountered, whichever comes first. This will protect the highway from slides that will inevitably occur at the steeper excavation slope required to intersect original ground where the rock exposures are not present at the upper edge of the excavation.

VIII. SUMMARY AND CONCLUSIONS

The following summary of the geotechnical aspects and conclusions about the I-26 landslide site is offered:

(1) Miller Mountain's south slope can be classified as a colluvial slope, or one which has had a geologic history of landslides.

(2) The topography of the mountainside has some characteristic slope angles that appear to correlate, on a gross examination, with the subsurface geologic data.

(3) There is nothing geologically unique in either the structure or lithology of the mountainside, but a subtle, complex combination of geologic conditions contributes to the landslide problems. A predominant factor is the presence of weathered mica seams in both the intermediate and partially weathered zones of the weathering profile. These mica seams have an overall average dip at a low angle toward a nearby synclinal axis, which thus produces a condition conducive to sliding toward the roadway excavations on micaceous foliation planes on a slight skew to the centerline.

(4) Effects of rainfall and groundwater play a critical part in the slope stability of the mountainside, both present and past.

(5) There is no evidence of geologically recent tectonic influence of the site. Several old faults exist in the area, but there is no evidence of movement on these faults since at least early Mesozoic time.

(6) Geotechnical study of the landslides was greatly facilitated by the ability to obtain and test the shear strength of samples of slip surfaces, from both the preconstruction and construction slide masses. These data are presented herein and may prove to be helpful guidelines in the investigation of shear failures of geologically similar earth and rock masses.

(7) The uses of instrumentation, (primarily inclinometers) and special investigative techniques such as the orienting core barrel were invaluable in developing an understanding of the landslides.

(8) Since the mountainside slopes were metastable to unstable by their very nature, even before the construction of I-26, it would not be economically feasible to provide the necessary retaining structures throughout the length of the project to create slope conditions better than metastable. Instead, the decision was for a remedial redesign to (1) avoid the construction slides, insofar as practicable, to (2) remove some of the slide-prone materials for several hundred feet from the roadway (and thus isolate the roadway from the new slides that will result from the excavation), and to (3) reduce slide movements by removing as much water as possible by surface ditches and by drilled horizontal drains.

(The discussion of design conditions for the required embankments for the remedial design and construction is beyond the scope of this paper.)

IX. ACKNOWLEDGEMENTS

The writers express their appreciation to Mr. A. Carter Dodson, State Highway Geologist, for permission to present data generated during the study of the landslide problems. Also, a number of engineers and geologists from Law Engineering participated in the exploration, testing and analyses of the slides, and their significant contributions to the site studies are acknowledged.

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