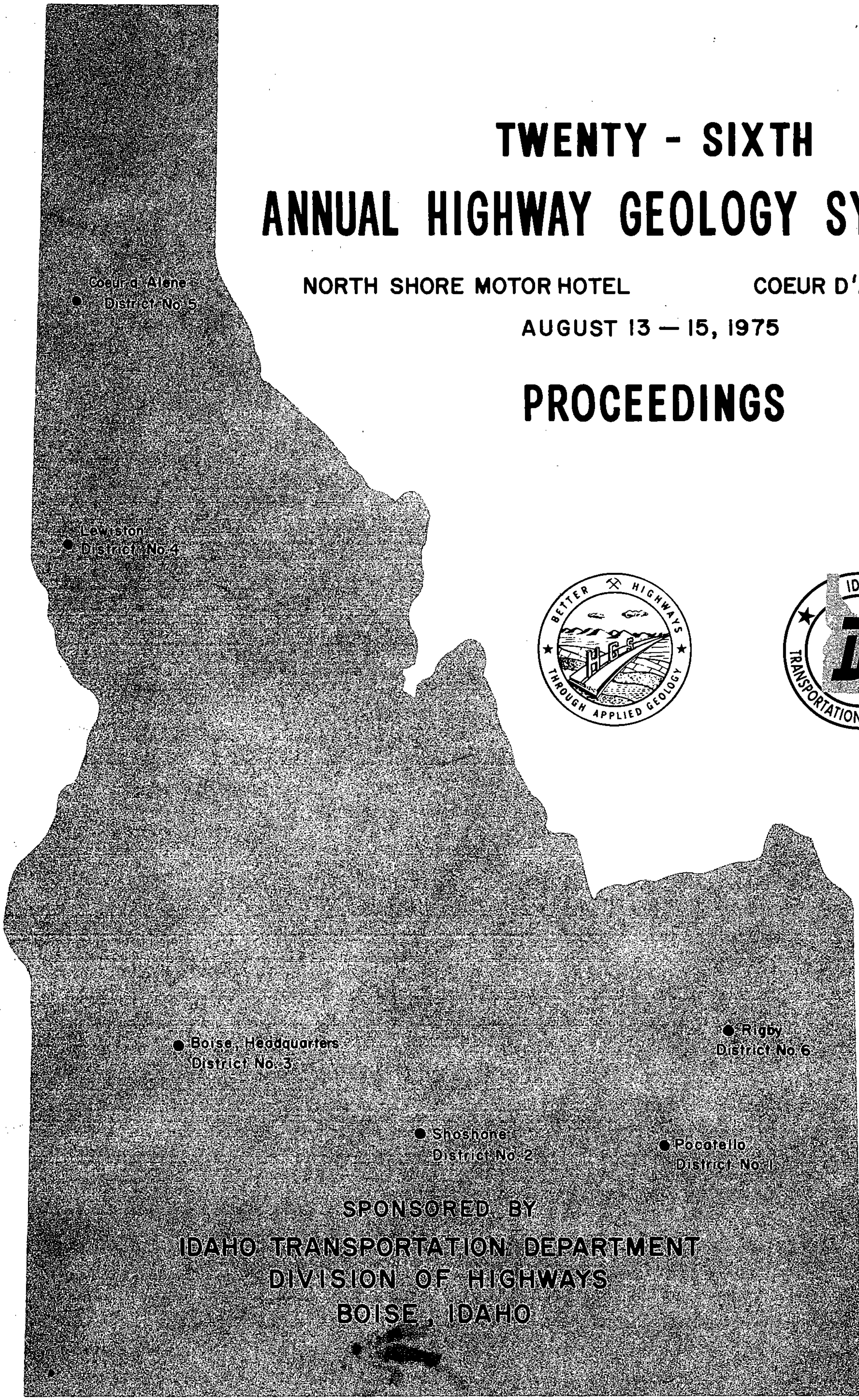
TWENTY - SIXTH

ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

NORTH SHORE MOTOR HOTEL COEUR D'ALENE, IDAHO

AUGUST 13 — 15, 1975

PROCEEDINGS



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-C-O-N-T-E-N-T-S-

Page

1 General Geology of Northern Idaho and Western Montana and Its Implication on Highway Construction. John G. Bond, University of Idaho. *

3 Application of Geotechnique to Design and Construction in North Idaho. Don H. Mathis, Idaho Transportation Department.

13 Seismic Designed Backslopes and Evaluation in a Structurally Disturbed Basalt Section. Ronald E. Larsen, Idaho Transportation Department.

43 Environmental Geology Survey, Regional Land Use and Intermodal Transportation Planning, Northeastern Pennsylvania. J. David Welch, Joseph S. Ward and Associates.

101 Use of Woven Plastic Filter Cloth as a Replacement for Graded Rock Filters. John E. Steward, U.S. Forest Service, Region 6.

115 Aggregate Resource Conservation in Urban Areas. L. E. Garner, The University of Texas at Austin.

127 Petrography as Related to Potential Skid Resistance of Paving Aggregates Used on Texas Highway Projects. Tom S. Patty, State Department of Highways and Public Transportation, Austin, Texas.

135 Rock Types and Seismic Velocities Versus Rippability. Allen D. Bailey, Nimbus Instruments.

143 Traffic Safety Instrumentation, Fountain Landslide, Oregon. Edgar L. Johnsen, Oregon Highway Division of Transportation.

159 In-Situ Measurement of Shear-Wave Velocities for Engineering Applications. Paul R. Farnham, Bison Instruments, Inc.

169 The Use of Organic Topsoils as Construction Materials. Charles L. Bartholomew, Charles Bartholomew Engineering, Inc., and Herbert O. Ireland, University of Illinois.

187 A New Analytical Approach to Split Tensile Strength of Pavement Materials. S. Kumar, University of Arkansas at Little Rock, M. Annamalai and J. G. Laguros, University of Oklahoma.

205 Collapsible Soils, State-of-the-Art. Sam I. Thornton, University of Arkansas and Kandiah Arulanandan, University of California, Davis.

---

*The above paper was not delivered to the Symposium Committee for publication.

-C-O-N-T-E-N-T-S-

Page

221 Design and Installation of an Earth Tieback Support System. David E. Weatherby and K. Ronald Chapman, Schnabel Foundation Company.

241 A Landslide in Pleistocene Deposits: Colombia (S. Am.). Bruce Denness, Institute of Geological Sciences, London (UK). Presented by Robert M. Smith, Idaho Transportation Department.

259 Skylab Explores the Earth. Verl R. Wilmarth, Johnson Space Center, John Kaltenbach, NASA, William B. Lenoir, Scientist Astronaut, and Michael C. McEwen, Johnson Space Center.

A U T H O R S

TWENTY-SIXTH ANNUAL

HIGHWAY GEOLOGY SYMPOSIUM

-A-U-T-H-O-R-S-

	Page
Annamalai, M., University of Oklahoma. A New Analytical Approach to Split Tensile Strength of Pavement Materials.	187
Arulanandan, Kandiah, University of California, Davis. Collapsible Soils, State-of-the-Art.	205
Bailey, Allen D., Nimbus Instruments. Rock Types and Seismic Velocities Versus Rippability.	135
Bartholomew, Charles L., Charles Bartholomew Engineering, Inc. The Use of Organic Topsoils as Construction Materials.	169
Bond, John G., University of Idaho. General Geology of Northern Idaho and Western Montana and Its Implication on Highway Construction.	1
Chapman, K. Ronald, Schnable Foundation Company. Design and Installation of an Earth Tieback Support System.	221
Denness, Bruce, Institute of Geological Sciences, London (UK). A Landslide in Pleistocene Deposits: Colombia (S. Am.).	241
Farnham, Paul R., Bison Instruments, Inc. In-Situ Measurement of Shear-Wave Velocities for Engineering Applications.	159
Garner, L. E., The University of Texas at Austin. Aggregate Resource Conservation in Urban Areas.	115
Ireland, Herbert O., University of Illinois. The Use of Organic Topsoils as Construction Materials.	169
Johnsen, Edgar L., Oregon Highway Division of Transportation. Traffic Safety Instrumentation, Fountain Landslide, Oregon.	143
Kalbenbach, John, NASA. Skylab Explores the Earth.	259
Kumar, S., University of Arkansas at Little Rock. A New Analytical Approach to Split Tensile Strength of Pavement Materials.	187
Laguros, J. G., University of Oklahoma. A New Analytical Approach to Split Tensile Strength of Pavement Materials.	187
Larsen, Ronald E., Idaho Transportation Department. Seismic Designed Backslopes and Evaluation in a Structurally Disturbed Basalt Section.	13
Lenoir, William B., Scientist Astronaut. Skylab Explores the Earth.	259

-A-U-T-H-O-R-S-

	Page
Mathis, Don H., Idaho Transportation Department. Application of Geotechnique to Design and Construction in North Idaho.	3
McEwen, Michael C., Johnson Space Center. Skylab Explores the Earth.	259
Patty, Tom S., State Department of Highways and Public Transportation, Austin, Texas. Petrography as Related to Potential Skid Resistance of Paving Aggregates Used on Texas Highway Projects.	127
Steward, John E., U.S. Forest Service, Region 6. Use of Woven Plastic Filter Cloth as a Replacement for Graded Rock Filters.	101
Thornton, Sam I., University of Arkansas. Collapsible Soils, State-of-the-Art.	205
Weatherby, David E., Schnabel Foundation Company. Design and Installation of an Earth Tieback Support System.	221
Welch, J. David, Joseph S. Ward and Associates. Environmental Geology Survey, Regional Land Use and Intermodal Transportation Planning, Northeastern Pennsylvania.	43
Wilmarth, Verl R., Johnson Space Center. Skylab Explores the Earth.	259

C O M M I T T E E

TWENTY-SIXTH ANNUAL

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GENERAL GEOLOGY OF NORTHERN IDAHO AND WESTERN MONTANA AND ITS IMPLICATION ON HIGHWAY CONSTRUCTION

By

John G. Bond
University of Idaho

ABSTRACT

Rocks in the region are divisible into four major types, each of which has its own engineering design and construction characteristics. The oldest and most abundant rocks are Precambrian marine sediments of the Belt Supergroup; these contain minor intrusive sheets and very locally retain inlier remnants of early Paleozoic marine detrital units. Late Mesozoic intrusives and crystalline rocks associated with the emplacement of the Idaho batholith and the Laramide Orogeny form the next younger rock type. Tertiary basalt and other volcanic rocks along the margins of the region make up the third rock type. Unconsolidated clastic deposits resulting from Pleistocene glaciation and Recent alluvial processes mantle much of the area and form the fourth type.

Rocks of the Belt Supergroup have been subjected to numerous metamorphic conditions and events. They generally respond to engineering manipulation as highly indurated, tectonically disturbed rocks exposing inclined bedding and fracture planes. The northern Idaho-western Montana region exhibits intense fault patterns and, Belt rocks, where brecciated and highly fractured, are prone to sluff or ravel. Under weathered conditions they become unstable in steep terrain or cuts.

Intrusive rocks of the Mesozoic Era respond like most massive, crystalline lithologies and offer little in the way of stability problems except where deeply weathered at the surface or along fractured planes. However, they can require the use of severe techniques to complete major excavation.

Columbia River Basalt flows make up the Tertiary volcanic rocks in northern Idaho; these respond as "bedded", highly but regularly jointed units where encountered in undisturbed, thick sequences. The flow rock can have wide engineering properties where thinly accumulated. Irregular jointing is more common in these dispursed outcrops and the basalt may not stand well vertically, particularly if weathering has altered glassy portions to clay. The more acidic volcanic rocks of western Montana generally retain enough of their initial welded characteristics to retain engineering integrity; however, they have restricted areal extent and generally one of little engineering concern.

Unconsolidated sediments in the area offer all of the common engineering benefits and problems associated with loose gravel, sand and clay. Perhaps the unique features which lead to special engineering considerations for the region are the wide distribution of till on upland slopes, the thick, fluvial debris-cored terraces and benches of the major valleys and the fine-grained lacustrine deposits of the low lands--all of which reflect glacial activity over the past few hundred-thousand years.

To footnote the region--Man's activity has also added to highway design and construction problems through his attempts to control water resources, develop mineral deposits and utilize forest products.

Editorial Note: The above paper was not delivered to the Symposium Committee for publication.

APPLICATION OF GEOTECHNIQUE TO DESIGN AND CONSTRUCTION IN NORTH IDAHO

By

Don H. Mathis
Idaho Transportation Department

ABSTRACT

The application of geotechnical data to problems encountered in design and construction of highways and related facilities can provide solutions which are both effective and economic. We have been able, over the past fifteen years, to provide workable solutions, some based on quantifiable data, many on "structured guesses."

Some examples may serve to illustrate the evolution of the present involvement of geology in design and construction:

1961. Wallace-Mullan, Interstate

The project was designed (contrary to geologic recommendation) on the "rock is rock" and will stand steep and tall forever. After a cut by cut redesign and an additional 1,400,000 c.y. of excavation we had achieved some semblance of marginally stable slopes.

Joint orientation rather than bedding generally controlled. All cuts were benched at 50' vertical intervals, a non-rational design feature arrived at by observation.

1961. Rose Lake Cut

After construction was completed on this 160' cut, a routine inspection (a procedure instituted after our Wallace-Mullan experience) disclosed some movement in a utility line above the cut slope. There was no indication of shear failure and only minor cracking in the surface duff. Rock dip seemed favorable - 70-80 degrees into the hill and the joint system was not well developed. A surface survey control net was set up to monitor direction and rates of movement in a program to determine the failure mode. As the failure progressed, the phenomonen of stress release became more evident. Corrective measures based on selective removal and buttressing effectively controlled movement.

1963. Mullan-Montana Line, Interstate

With a better appreciation for the whimsical habits of mother nature and her handmaidens, the Belt group of metasediments, we embarked on the design of our most massive project to date - a three-mile project involving 6,000,000 c.y. of excavation with cuts and fills up to 400' in height.

An extensive mapping and subsurface investigation was undertaken to provide a design data base. Although we recognized the existence of high latent stress, we did not attempt to quantify stress system.

Design features incorporated included massive drainage systems for embankment foundations, zoned fills and benched cuts.

Grading was completed in 1971 with only one surprise. The summit cut began to exhibit the symptoms of stress release. Based on our previous experience we anticipated no catastrophic failure and would have been content to wait and watch if it were not for the fact that the Burlington Northern Railroad had been relocated across the cut slope.

An extensive system of surface survey control and instrumentation was installed to provide comforting (?) data. Several possible corrective measures were studied but attendant costs were not justifiable unless it could be established that massive failure was probable.

Rates of movement varied from a high of 1.8"/day in July of 1972 to a currently stable condition. No further action is planned.

TEXT OF PRESENTATION

The three problem areas discussed in this paper lie east of Coeur d' Alene and represent those problems and solutions we have come to expect in the Pre Cambrian Belt Metasediments. Rock types range from siltstone to quartzite but for simplicity have been lumped into a catch-all argillite. Generally the engineering properties are more dependent on the diastrophic history than on the specific rock type. Tectonically the area is dominated by the Osburn Fault, a major rift traceable from Coeur d'Alene to Missoula, a distance of 170 miles. A maximum of about 16 miles of right lateral strike slip and 10,000' of throw has produced a shear zone which in places, is more than 500' wide. Numerous accessory faults and intense folding further complicate the tectonic pattern.

<u>WALLACE-MULLAN</u>

Preliminary geologic investigation of this three-mile segment of I-90 recognized the potential cut slope problems in a steep sided narrow canyon which had to accommodate a railroad, a major drainage and an interstate highway. Unfortunately, the recommendation offered could not compete with the "on-paper" economy of the then (?) prevalent concept that rock is rock and will stand tall and steep forever. Design proceeded on this premise, resulting in a beautifully balanced mass using ½:1 backslopes.

Construction proceeded in three phases, detour system and structures, Interstate grading and paving. The detour phase offered a preview of coming attractions when the only major cut began to fail shortly after it was brought to grade. With a 100' high, 250' wide drill hole to look at, it was not difficult to isolate the cause of failure - a master joint system and to effect stabilization by resloping and benching. In retrospect, this failure probably would not have been predicted from a normal preliminary geological investigation.

As anticipated from our initial experience the designed cuts on the major grading proved to be unstable. With the contractor's equipment blowing exhaust up our legs, the unstable slopes falling on our hard hats, detailed evaluation for redesign was not attempted. Investigation boiled down to visual evaluation of joint patterns, fault systems and bedding. In many places slope design was dictated by an inflexible right-of-way and only marginally stable slopes were achieved. Cuts were restarted at the right-of-way line and brought down as flat as possible until we reached more competent material and then benched out. Where we enjoyed the luxury of adequate right-of-way redesign was based on geologic features. Where possible the maximum vertical interval between major slope benches was maintained at 50'. Although there was no rational basis for this interval, it had worked for us in similar situations.

Obviously, in a situation such as this very direct communication had to be maintained with no room for detailed drawings, recommendations in geologiceese or paper shuffling.

One million dollars and 1,300,000 c.y. of material later we had achieved generally stable cut slopes. As importantly, we had sold geology as a usable problem-solving tool sufficiently responsive to provide on-the-spot answers.

ROSE LAKE INTERCHANGE CUT

At the completion of construction this cut looked good. The bedding strike was roughly parallel to the roadway with a dip of 70-80' into the cut. Although a dike sill system was intersected, no apparent planes of weakness were evident. Routine post-construction surveys of completed cuts (routine since the Wallace-Mullan job) disclosed some minor mantle cracking at the base of a 180 KV transmission line located 80' behind the top of the cut. A more detailed examination indicated extensive cracking in the zone between the power line and the top of the cut, but no expression of failure in the cut face. Three telephone poles in the zone of cracking had been moved laterally as much as three feet without losing plumb. Because of the threat to the utilities, as well as the Interstate, a tight surface survey grid was established to determine both the rate and direction of movement. Over the six-month period the top of the cut moved in excess of seven feet laterally and a graben with a back scarp approximately six feet in height developed in an area behind the cut catch. As the adjustment developed, individual beds exposed in the cut face folded out with a general decrease in apparent dip of 10°.

Survey data indicated surface movement both into the cut and laterally along the individual bedding planes. The lateral component was verified by observation of developed slickensides.

Corrective treatment consisted of effectively flattening the upper portion of the slope with a two-step bench system and using the excavated material to provide a buttress to inhibit lateral movement. In this instance, we enjoyed the luxury of time for the development of rational data on which to base corrective treatment, but lacked the sophistication and budget for a more complete investigation.

MULLAN-MONTANA LINE

This project represents the most massive job in terms of volume of cut and fill accomplished in this District - three miles of construction involving over 6,000,000 c.y. of material in cuts and fills up to 400' in height.

We were able to strongly involve geology in all phases of project design from location corridor studies to specific design recommendations.

With relatively few rock exposures we relied on aerial photos, seismic data, exploratory drilling and review of mining property data for the geologic mapping which formed a basis for recommendations relative to location. This information was used to lay out our drilling program on the selected location. One hundred and seventy-two drill sites were selected to provide specific data on groundwater, structural features and materials quality. Although we recognized the existence of significant latent stress systems, we did not feel the cost-benefit of quantification techniques justified their use.

The following specific design features were generated from mapping and drill data.

Cut slopes were variable from 1½:1 to ½:1 as were benching intervals. Unfortunately, the location of a railroad at the top of many cut sections tended to force an occasional compromise.

Embankment foundations were designed to provide for bench keys from top to bottom, and drainage for each bench. The overall subsurface drainage system incorporated over ten miles of underdrain into the three mile project.

Soils strength data generated from mechanically degraded rock was used to design zoned fills.

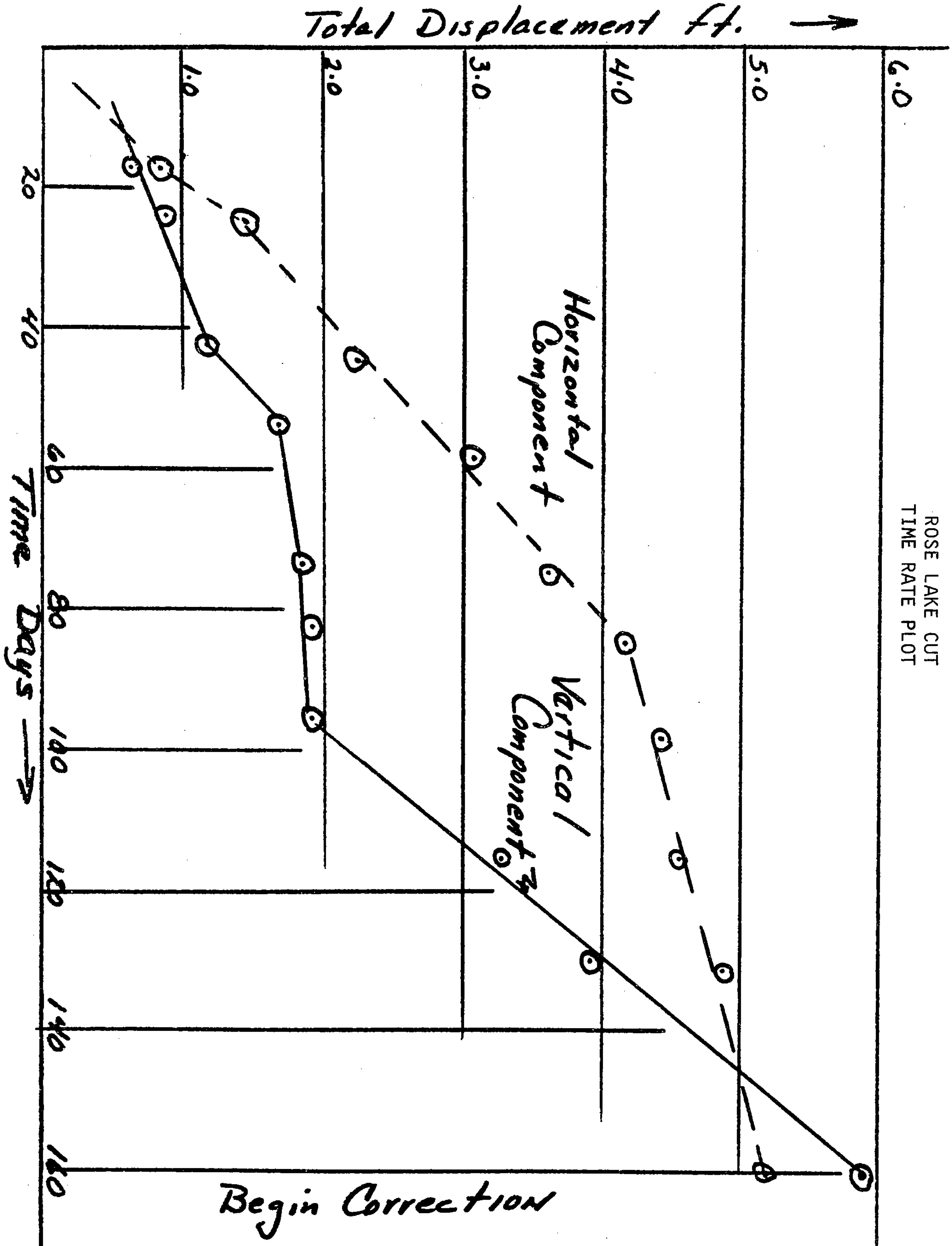
The project was completed with only one cut slope structural adjustment due to stress release and one embankment problem traceable to construction techniques - a constructed fill slope at 1.3:1 vs the 1.5:1 designed.

As you might expect, the one cut with which we experienced a problem was the one where the railroad had been relocated to a bench across the cut face. The railroad did experience significant movement both vertical and horizontal in their trackage. Although the apparent adjustment in the cut slope was considered consistent with our previous experience with stress release, there was ample incentive (and funding) to test our premise and gather data for corrective measures in case a catastrophic failure became probable.

Instrumentation included surface survey control nets, three inclinometers and two 4-position extensometers. Data provided by the extensometers indicated a somewhat alarming rate of movement reaching nearly a 0.03"/hour rate for a four-day period. The inclinometers did not, however, indicate the development of a shear plane. In addition to continued monitoring, two experimental rock bolt tie backs were placed to evaluate the feasibility of such a full scale retaining system. Although one bolt failed due to over-stressing by rock pressure, it was established that at least 90 K anchorage could be attained with grouted 120' bolts.

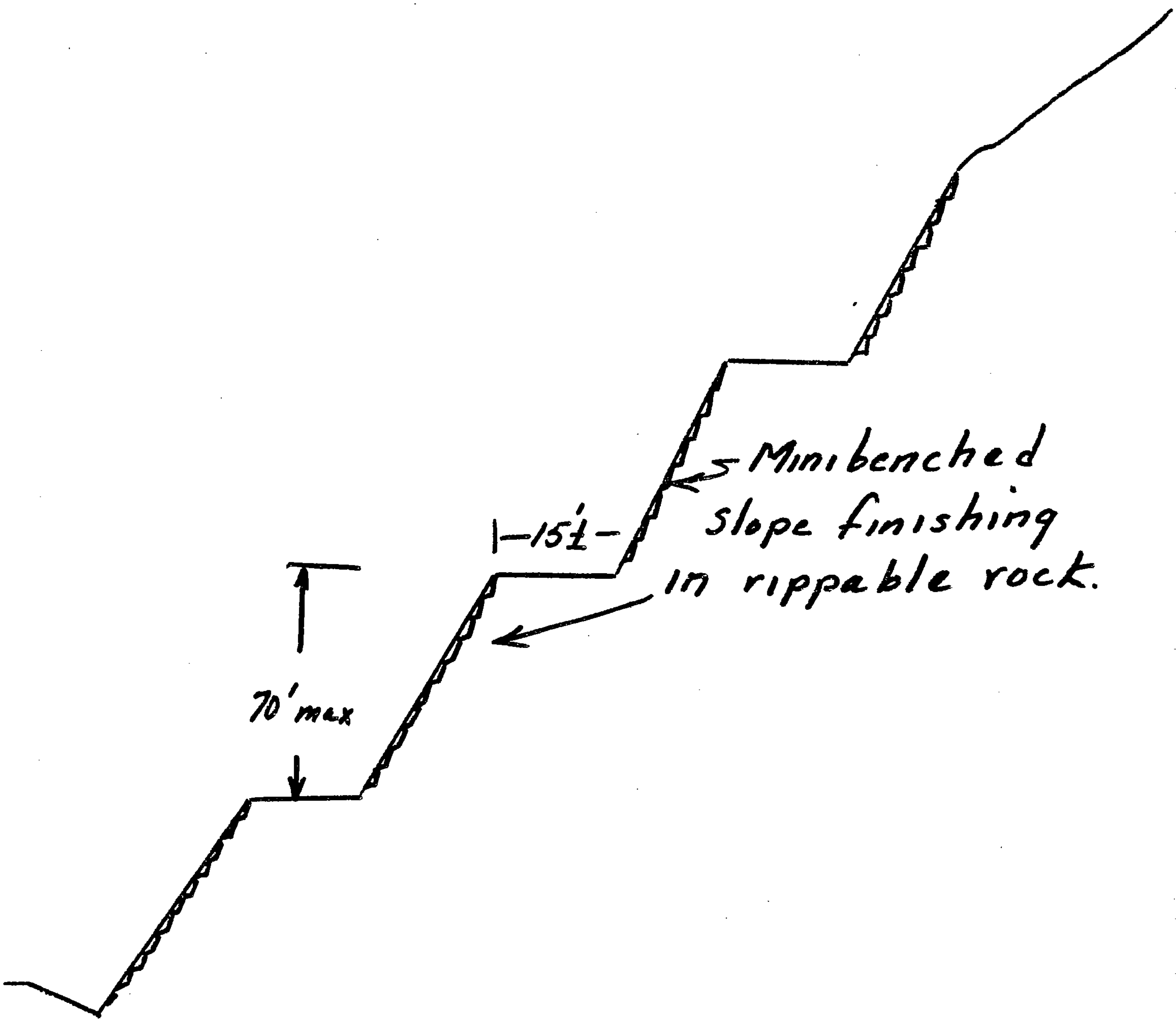
As anticipated, movement rates steadily diminished over a 14-month period with a stable condition existing for the past year.

Although we incurred approximately $6,000 in railroad maintenance costs during the period of active movement, the total cost of the wait-and-watch alternate has proven to be nearly $200,000 less expensive than proposed corrective measures.

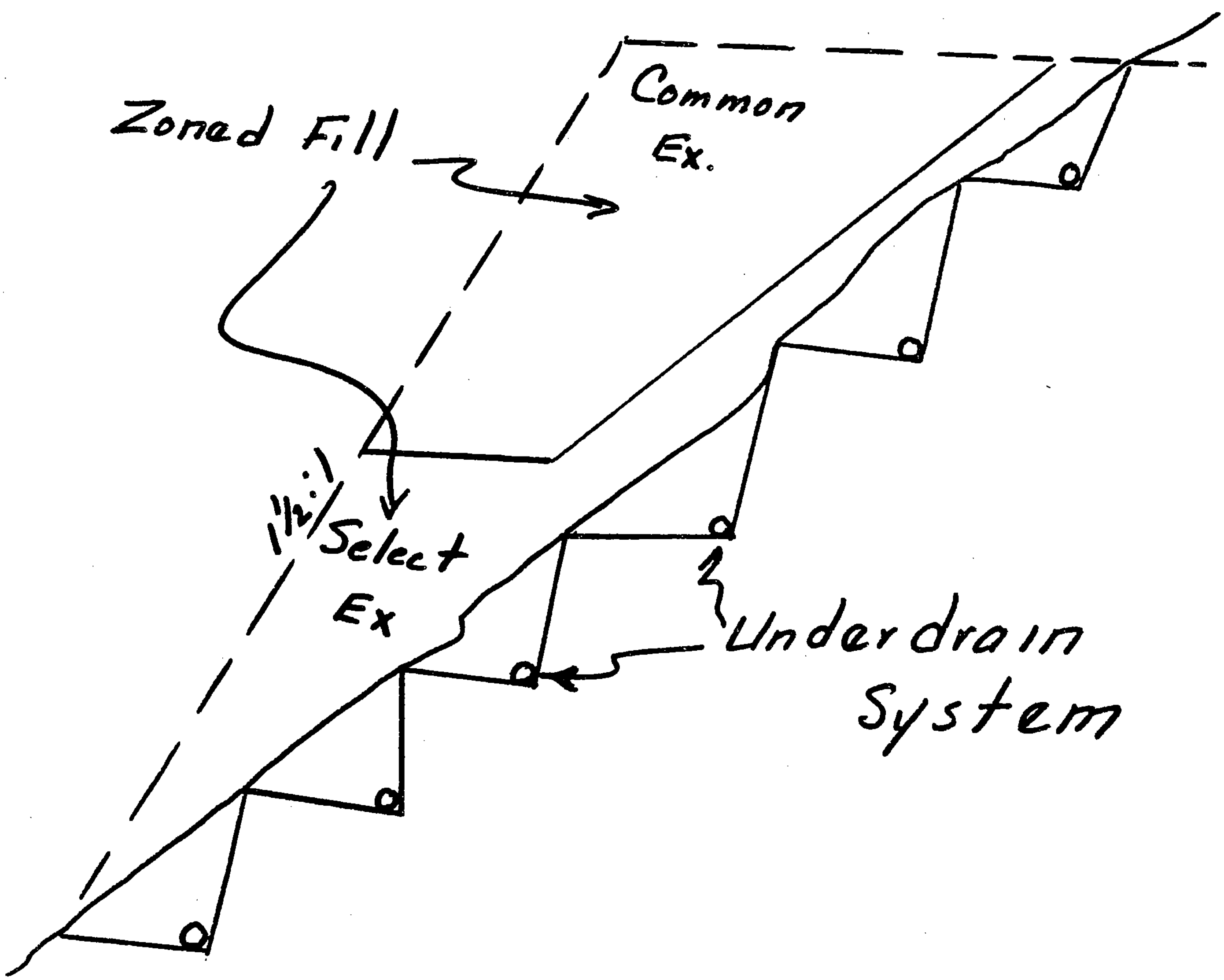


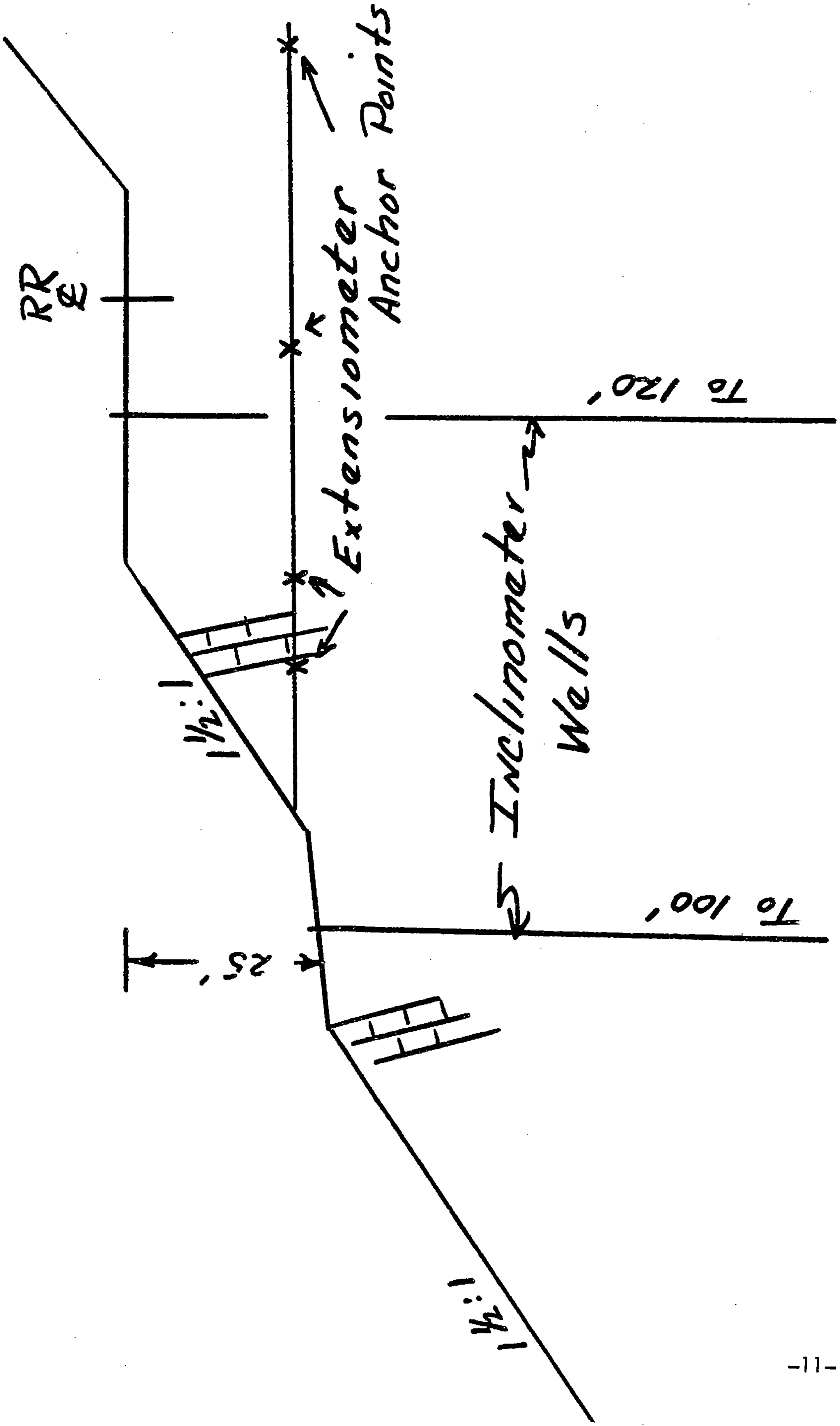
ROSE LAKE CUT
TIME RATE PLOT

LOOKOUT
TYPICAL CUT BENCHING

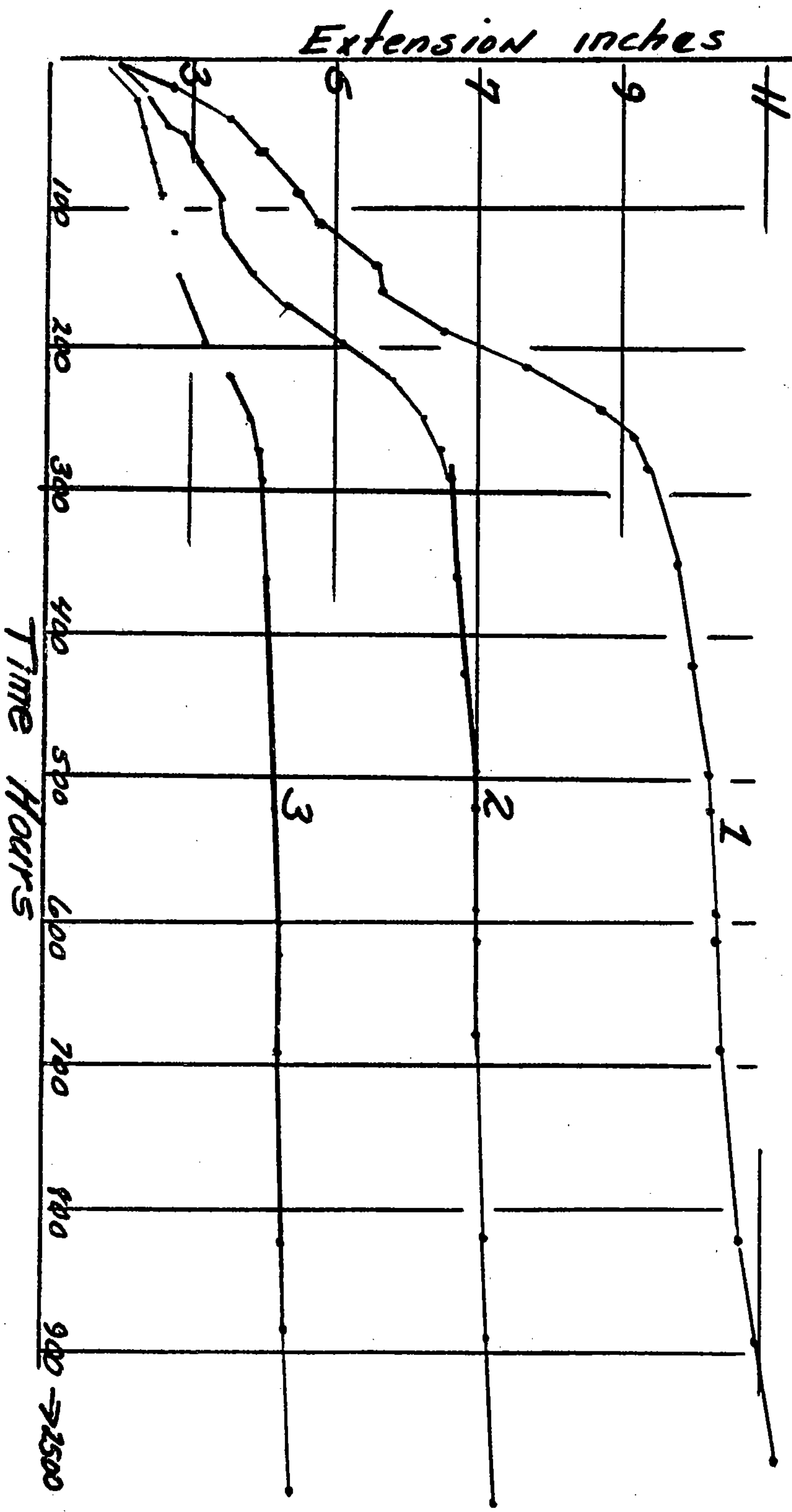


TYPICAL FOUNDATION BENCHING





IDAHO POINT INSTRUMENTATION



TIME RATE PLOTS

EXTENSOMETER
FACE WITH RESPECT TO ANCHOR POINT
AT 100' - 1 FROM FACE
30' - 2
20' - 3

SEISMIC DESIGNED BACKSLOPES AND EVALUATION IN A STRUCTURALLY DISTURBED BASALT SECTION

By

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ABSTRACT

Highway construction in mountainous terrain has suffered from limited or inadequate investigations concerning the engineering properties of rocks and geologic discontinuities.

Projection of surface mapped structural features and rock quality in relation to highway grade or backslope design is often proven to be in error, especially in geologically complex areas.

During the 1960's the Idaho Division of Highways formulated plans to locate, design and build, through stage construction, a modern, multi-lane highway through mountainous terrain of West-Central Idaho.

Although some diamond drill exploration was attempted, the bulk of the final design was based solely on geologic mapping and seismic refraction methods, in which overburden, depth of weathering, bedrock configuration, and structural elements were determined.

INTRODUCTION

The relocation of U.S. 95 in the White Bird area will replace a highway that was initially graded in 1919-1920, surfaced in 1922-1923 and finally paved in 1938. Since US 95 serves as Idaho's only North-South route and link to the "Panhandle", it is of utmost importance to connect the State with an updated mode of transportation.

The new highway along the East face of White Bird Hill will eliminate 14 switchbacks (60° curves), and the "ladder back" design.

The final design phase on the first of five projects began in the summer of 1966. The final design on the fourth project was completed in June of 1971.

Also included in this report is the background and history of both the corridor selection and various alignment considerations within the chosen corridor, some of which was based on geologic criteria, while others had political overtones.

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*Deceased, 10-20-75. Paper edited by Robert R. Elvin, Idaho Transportation Department, Division of Highways.

This paper is not concerned with the age of various units, the source of the basalt, or stratigraphic correlation on a regional basis, but only with the engineering properties of the basalt and associated sediments that will govern the success or failure of the final design on the White Bird Hill projects.

MAGNITUDE, DESIGN STANDARDS AND COST

The selected corridor was divided up into five projects to be completed by stage construction.

Contract Let	Project		Miles	Bid Price
1969	F-4113(31)	White Bird Summit-South New Construction	3.5	$1.8 Million
1970	F-4113(32)	White Bird-North New Construction	2.9	$2.2 Million
1972	F-4113(44)	White Bird Bridge		$1.7 Million
1973	F-4113(41)	Paving Contract		$1.2 Million
1973	F-4113(38)	White Bird-Salmon River New Construction	2.1	$1.1 Million
		TOTAL	8.5	$8.0 Million (Plus change orders and claims)

The design standards for these projects were as follows: 60 MPH Design Speed; 34' Minimum Width; 12'-wide Truck Lane; 10' Flat Bottom Ditch; 2.0' Rock Cap; Maximum 8° Curves; and 6.8 to 7.0% Grade on (31) and (32) projects. The highway rises from an elevation of 1486' on the river to 4245' at White Bird Summit, a distance of 8.5 miles. The completion of these projects will reduce both driving time and distance by 50%.

One of the basic reasons for this price was the amount of information supplied to the contractor in the form of a soils profile. Shown on the profile is a wealth of information, both geological and geophysical, along with the recommended slope design, which is related to the subsurface, on a typical cross section every time the slope design changes. If used, the contractor may govern his pre-splitting operation from this data, as was the case on White Bird.

STRATIGRAPHY

Stratigraphically two types of basalt will be recognized on the project, Upper and Lower Columbia River Basalt (Bond, 1963). Individual flows and flow units will be described, but no attempt will be made to equate either age, or petrographic similarities to the type Yakima or Picture Gorge sections of Washington or Oregon. The contact between Upper and Lower Basalt is recognized by the drastic break in slope and color differential of the weathered exposures. Above the contact, vertical stair-stepped or tiered outcrops, and darker colors (blue and black) predominate, whereas below the contact the slopes are more gentle and smoother and the outcrops are brown to reddish-brown in color.

If treated properly both types of basalt will provide stable backslope conditions and good compaction within the embankments.

A detailed lithologic examination will show the following observations to hold true for the Upper Basalt:

1. The basal colonade is absent to thin, but the total flow is relatively thick.

2. Irregular, thin (average 10" diameter) sliver columns are tightly jointed and resistant. The jointed surfaces are fresh, sharp-edged and relatively unweathered.

3. A fine uniformly grained texture persists throughout. Occasionally a few small phenocrysts are found.

4. Upon weathering the end product is clay.

The Lower Basalt usually possesses the following:

1. The basal colonades may be up to one-third the total thickness of the flow.

2. The columns which are highly variable in size are usually well developed. They may be vertical wavy or rosetted.

3. A porphyitic texture predominates with the cleavage surface having a resinous luster.

4. The prominent joints will be rounded and may easily disintegrate to a sandy end product.

BASALTS AS ENGINEERING UNITS

Basalts are unique from many standpoints. They all look alike, especially from a distance. For engineering purposes the flow concept is much too broad to be of any use except in correlation and structural analysis.

Each flow should be broken down into its individual components (units) and then examined for both physical and chemical anomalies. A close examination will point out significant changes that may occur in the same unit along strike. The most noticeable difference is related to the jointing habit or proximity to structural displacements. Venting, along with susceptibility to weathering and groundwater will also drastically affect the integrity of the unit.

The various engineering types or flow components which had to be contended with on the White Bird projects are as follows:

1. Massive - poorly defined columns, weakly jointed, dense, internally tight fractures, high seismic velocities (Platy-Conchoidal).
2. Columnar - well developed joints, medium to high seismic velocities.
3. Entablature - well developed, tight joints (Hackly Fracture) 6"-10" average diameter, high velocities.
4. Flow Top - poorly developed joints, highly indurated, low velocities.
5. Pillow - Plagonite - no joints, low velocities.
6. Flow Breccia - no joints, medium velocities.
7. Fault Breccia - highly fractured, occasionally semi-cemented, usually small fragments, low velocities.
8. Interbed - laminated bedding, fine grained, low velocities.
9. Basalt Talus - variable units of soil and cement. Medium to low velocities, dry to damp.
10. Colluvium - low velocities, dry to damp.
11. Lake Sediments - highly variable in both attitude and facies.
12. Ash Deposits - low velocities, no bedding.

Stress release in respect to backslope design was considered within the faulted area and with the use of seismic analysis was found to be nominal. The rock was simply more fractured, some of which indicated drag.

METHODS OF SUBSURFACE EXPLORATION AND ROCK EVALUATION

Although a Mobile B-40, 5" auger and Sprague and Henwood diamond drill was used to retrieve both soil and continuous rock core, the bulk of design information was derived from geophysics, primarily seismic refraction.

A portable single channel, engineering seismograph manufactured by Geophysical Specialties, Model MD-1, was the main source of information. Correlatable rock velocities were substituted for drill holes in inaccessible areas.

A hammer struck on a metal plate supplied the energy for most traverses of 100' in length or less, except in fault zones. Explosives (Primacord, Primadent, Dynamite--40 and 60% gel-7/8" diameter) were detonated by a battery operated blaster and seismic blasting caps.

Any explosive charge was buried at least one' below the natural ground surface and then backfilled, preferably with soil, in order to promote vibration into the rock rather than air waves.

The state-of-the-art has progressed so rapidly within the past seven years, the new digital readout devices, signal enhancement, memory recall, increased reception and sensitivity, and with portability of the multi-channel units, one would hardly recognize the prototype.

Any future work of these magnitudes should utilize a multi-channel seismograph for deep penetration, as only one explosive charge is necessary to determine a contact and substantiate slope velocity.

A signal enhancement unit is faster for shallow work, will not exhaust the manpower, and allows the operator to visually build a first arrival sine wave.

EXISTING LAB TESTS USED BY THE DIVISION

Presently, potential quarry rock is subjected to four lab tests which will determine the quality of a particular basalt. Three are mechanical in nature while the fourth is chemical.

The Los Angeles Abrasion Test (AASHTO T-96) (L.A. Wear), is a mechanical test in which a known amount of basalt is retained on a No. 12 sieve, placed in a revolving cylinder with 6 to 12 steel bearings and rotated for 500 turns. The material retained on the No. 12 sieve is then weighed and compared to the original value. The higher the number the less resistant the rock. Generally a value under 20 is quite acceptable and a value over 35 is considered a failure.

The Sand Equivalent Test (AASHTO T 186-70) (T-2-68) serves to show the relative proportions of fines or clay-silt particles (passing the No. 4 sieve) in graded aggregates that have already been crushed. The SE is computed by dividing the sand reading by the clay reading in a graduated cylinder and then multiplying by 100. The higher the SE, the more durable the material. A value below 30 is generally unacceptable.

The Idaho Degradation Test (T-15-72) is another mechanical abrasion test. The sample is run in a saturated surface dry condition (16 hours soaking), that measures the increased percentage of fines passing the No. 200 sieve. This serves as a quantitative measure of the resistance of a coarse and fine aggregate to the production of plastic fines.

The use of Ethylene Glycol to promote accelerated expansion of reactive aggregates is a chemical test used to determine the percentage of unaltered rock retained, after immersing a specific amount of basalt, in a Glycol solution for 15 days. The sample is then oven dried, and the amount retained on a 3/8" sieve is measured.

The Glycol acts as a chemical catalyst speeding up any potential reaction or expansion of any unstable minerals and swelling clays. Newton H. Olson, Army Corps of Engineers, Walla Walla, stated (personal communication) that rock samples, which normally take 10-12 months of atmospheric exposure to promote failure or degradation can by the Glycol test, 15-day immersion, be accelerated to rapid failure. The presence therefore of montrorillonite or swelling clays in a sample can be ascertained. Experience states that any less than 93% retained is questionable.

AREAS OF SPECIAL CONSIDERATION

Station 352 - Engineering vs. Ecologic Impact

The alignment throughout the Salmon River section of this project was in limbo for many years. The original proposal was to stay on natural ground, which consisted of a thick, truncated, colluvial terrace deposit that was interlaced with lenses of fluviatile sands. An existing cut slope along the old highway was standing at a ½:1 showing large blocks of sub-angular basalt within a silt matrix, that at least on the outer weathered surface was partially cemented.

Due to the excessive height and quantities involved for the proposed cuts (1¼:1), it was recommended to shift the alignment towards the river, which in places would mean a partial encroachment into the Salmon River.

This presented no real problem as there was a solid bedrock foundation to build on. River current velocities were recorded so that adequate sized riprap could be placed. The velocities were 12'/sec., which according to the Corp of Engineers should mean a stone weight of over 400 lbs.

Engineering-wise the problem could be solved, but environmentally there was a snag. The problem was that any river encroachment must have the sanctioned approval of the various environmental agencies, of which the Idaho Fish and Game Commission is one. They recommended that since willows were in abundance along the existing bank that two 20-foot-long jetties (10 feet wide) be constructed at right angles to the riprap facing. This would, hopefully, encourage sand deposition and a possible spawning area.

With the maximum daily discharge of 100,000 C.F.S. and stream velocities of 12 foot per second in a partially restricted narrow channel having a 90 degree bend in the river immediately upstream, the jetties would be washed out and would have to be replaced on a yearly basis. For this reason, the Materials Section was asked to review the originally proposed line. A seismic survey was initiated which consisted of ten normal-reverse contoured traverses which would give an overlap of information (stair-step coverage).

The contact depths were computed in the usual fashion and then plotted to the appropriate cross sections. To be exact, in the analysis one must be aware that sound waves will pick up a discontinuity that may not be in a horizontal or uniformly dipping plane. This is especially true in basalts which are typically tiered, weathered and covered by either talus or colluvium. For this reason, the depth was shown as an arc, rather than a point, and somewhere along that arc was a more resistant layer, which possibly represented bedrock. See Photo No. 1.

PHOTO NO. 1

M.P. 221.50
3/4:1 Colluvial Backslopes with 10' Benches every 50'

After the data was accumulated it appeared that most feasible slope designs would remain in colluvial debris. A 3/4:1 slope with 10-foot benches for every 50 feet of vertical height was selected even though the natural slope was 1½:1. This decision was based on excess quantity with a flatter slope and the fact that the existing cut slope was standing adequately on ½:1 slope. Another factor is the annual rainfall which amounts to 6"/year. No undisturbed soil samples were taken as the material was much too rocky to test, so the factor of safety is still unknown.

So far the slope has performed very well with only one small bench failure in the alluvial sand. See Photo No. 1.

Station 437 - Banner Ridge Section

A 120-foot cut on centerline was proposed through this area which would amount to considerable excavation, depending on the slope design.

There were a number of faults within the vicinity but what effect this would have on the rock quality was unknown. Three diamond drill holes were placed, but very little core was recovered. The bulk of the material was drilled by tricone.

Seismic traverses were attempted but the data appeared scrambled and there was no continuity between traverses. The only exposure was a thin Breccia unit and this was not continuous as it was dissected by at least two faults. Three more diamond drill holes were placed with similar results.

At this point it seemed foolish to attempt more work. The slopes were designed for the worst possible conditions; 1½:1 overall, serrated slopes (mini benched), with a 30-foot bench every 50 feet vertically. See Photo No. 2.

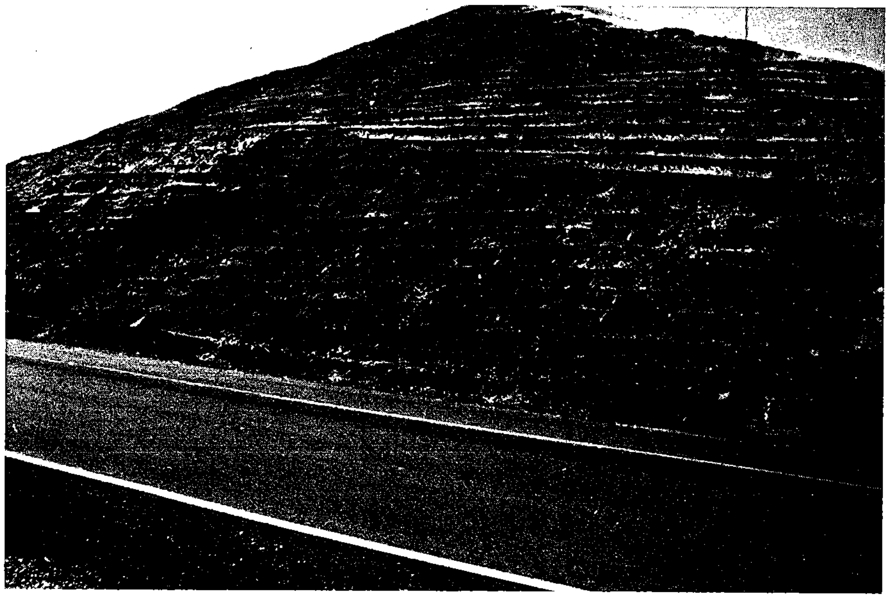
The recommendation was correct, the bulk (95 percent) of the cut was steeply dipping, hydrothermally altered, and structurally displaced. See Photo No. 2. The material throughout the cut is weathering and differentially sluffing back to the 1½:1 overall slope. Hopefully the grasses will establish their roots and eliminate raveling and look asthetically appealing.

The seismic data was useless under these circumstances, which was the reason it looked like a puzzle. The little silver box was correct, just the operator was confused.

Sta. 492.00-494+50 - Benched Embankment in Fault Gouge

A 200-foot high sliver fill was proposed in this area using conventional embankment design. But a field examination indicated that a special foundation design was necessary. The alignment was to cross a merging fault system that not only had a thick soil-talus accumulation on the footwall but severely fractured bedrock and associated gouge that indicated drag along

PHOTO NO. 2



M.P. 223.20
Banner Ridge - 1½:1 Mini-bench Slopes in an Altered Basalt

the sheared surface. A standard embankment design would be in jeopardy and would endanger the White Bird Elementary School immediately below.

For these reasons, a series of benches 20 to 25 feet wide, parallel to contour lines were excavated (See Figure No. 1) into bedrock in hopes of removing any potential slip surface, thereby keying the embankment into solid rock. At the embankment catch point, a larger bench was excavated into bedrock which would eliminate any potential base or toe failure.

To date this embankment has not experienced any movement or settlement.

Station 502 - Centerline Fault vs. Slope Design

The influence of the "centerline fault" is shown clearly throughout the vicinity of Station 502. (See Photo No. 3). A number of seismic traverses, both parallel and perpendicular to contour lines were set up in hopes of defining the limits of gouge zone that would affect rock competency within the proposed cut slope.

Although more resistant rock was encountered on the hanging wall, the proposed alignment would not allow one to take advantage of it as it was a vertical outcrop. To do so would necessitate alteration of the cur-

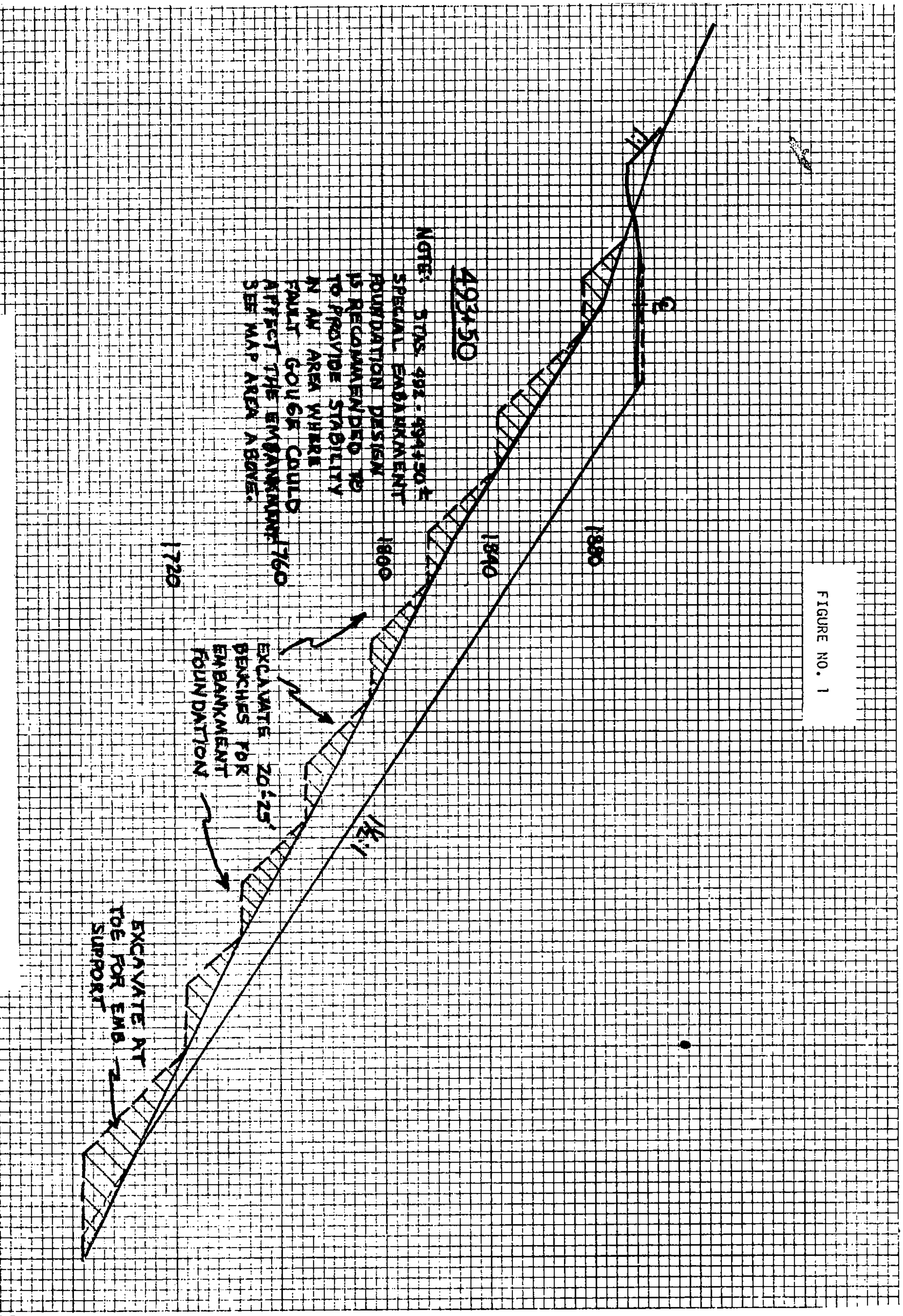
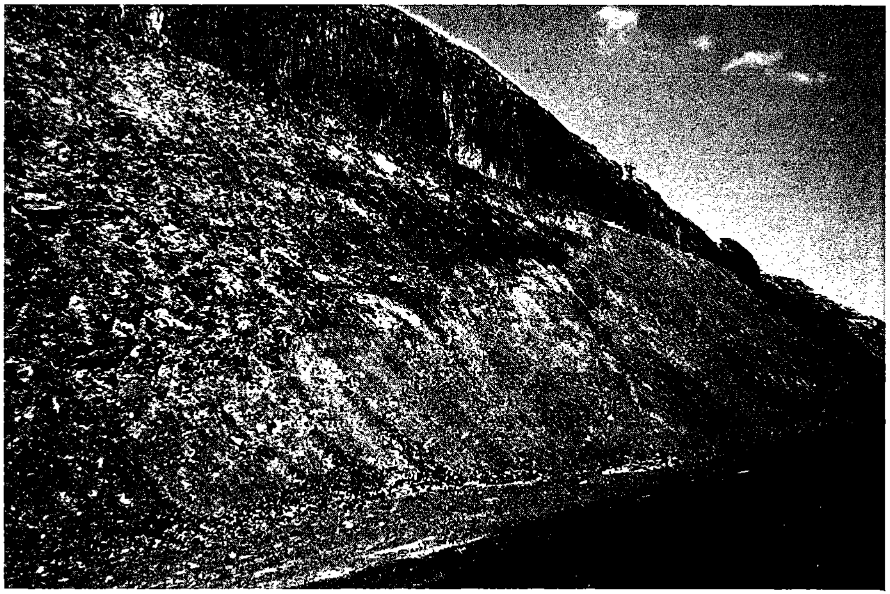


FIGURE NO. 1

Toe Tranching and Serrations for Embankment Stabilization at Sta. 493+50

PHOTO NO. 3



M.P. 224.5
Constructed Backslope and Associated
Fault Gouge at Sta. 501

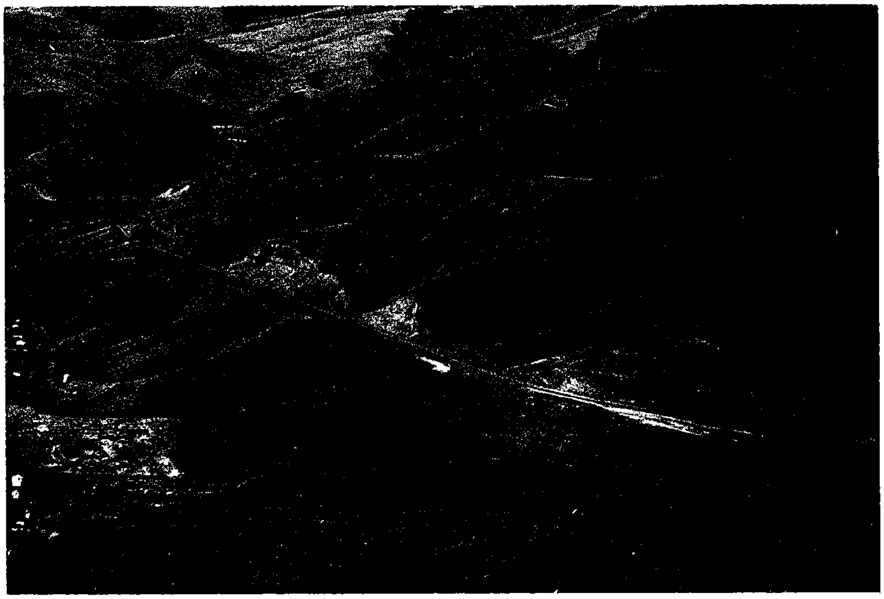
vature which would defeat the purpose, increasing both embankment and excavation. Therefore, it was decided to create the following design which was based entirely on geophysical data.

Even though it looks somewhat unusual, Photo No. 3 will indicate that the analysis was correct. The highly fractured loose gouge material encountered at grade was designed on a 1:1 slope. It has remained in place with only minimal sluff. The high angle, down the valley fault plane, is exposed throughout much of the upper section. Slopes of 1/8:1 were proposed with 15' wide bench system to catch and retain falling rock. The slope has remained stable as the wall rock behind the shear plane is quite resistant to movement. (See Photo Nos. 3 and 4).

Station 590 - Multiple Faults vs. Slope Design

A deep through cut was proposed in this area, (Photo No. 5), which again raised the question as to what would be a stable, maintenance free slope, yet not be prohibitive cost-wise.

PHOTO NO. 4



Aerial View Looking South at New Alignment at Town of Whitebird

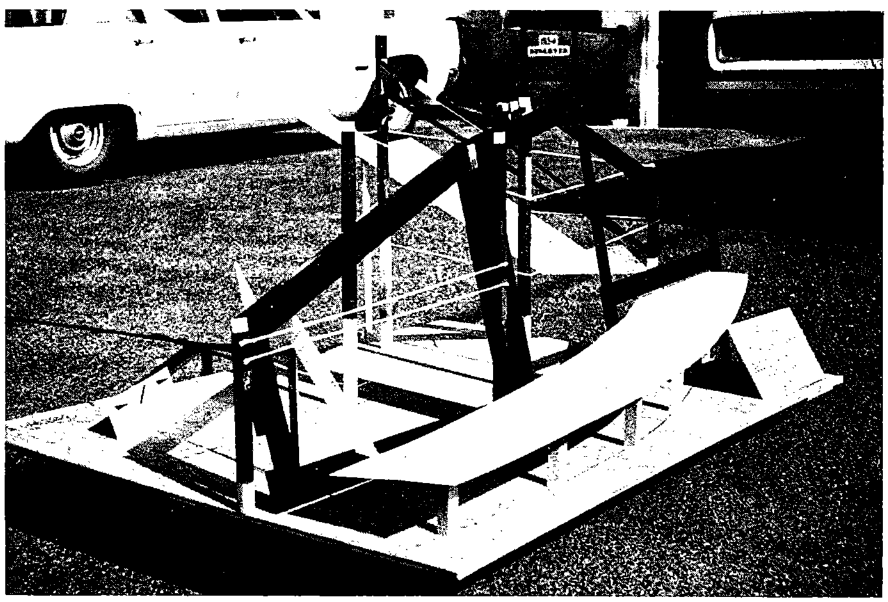
PHOTO NO. 5

M.P. 226.2
Sta. 590. Drill Investigation and Fault Limits

After attempting some detailed mapping, a high angle normal centerline fault was delineated along with some other structural complications that were not readily decipherable. Seismic was attempted but without much success. Significant amounts of dynamite were used as an energy source (60 percent Gel. 1-1/8" Dia.), but most of the shock was being absorbed in the shear material. Second cycle arrivals were common.

Six diamond drill holes were placed throughout the cut in an attempt to find uniformity or similarity of strata. After the holes were logged a three-dimensional scale model was built depicting the drill holes in relation to the proposed alignment and grade. Then, an effort was made to figure out the structure and amount of throw by correlating the test holes. See Photo No. 6 and Figure No. 2.

PHOTO NO. 6



M.P. 226.2
Scale Model of Subsurface Conditions and Associated Fault Planes

It appeared that a low angle fault intersected the high fault left of centerline. The cuttings and core indicated highly fractured and altered basalt. A variable slope (1:1 to 1½:1) and bench was designed to eliminate the loading above a three-foot clay horizon in the low angle fault. See Photo No. 7

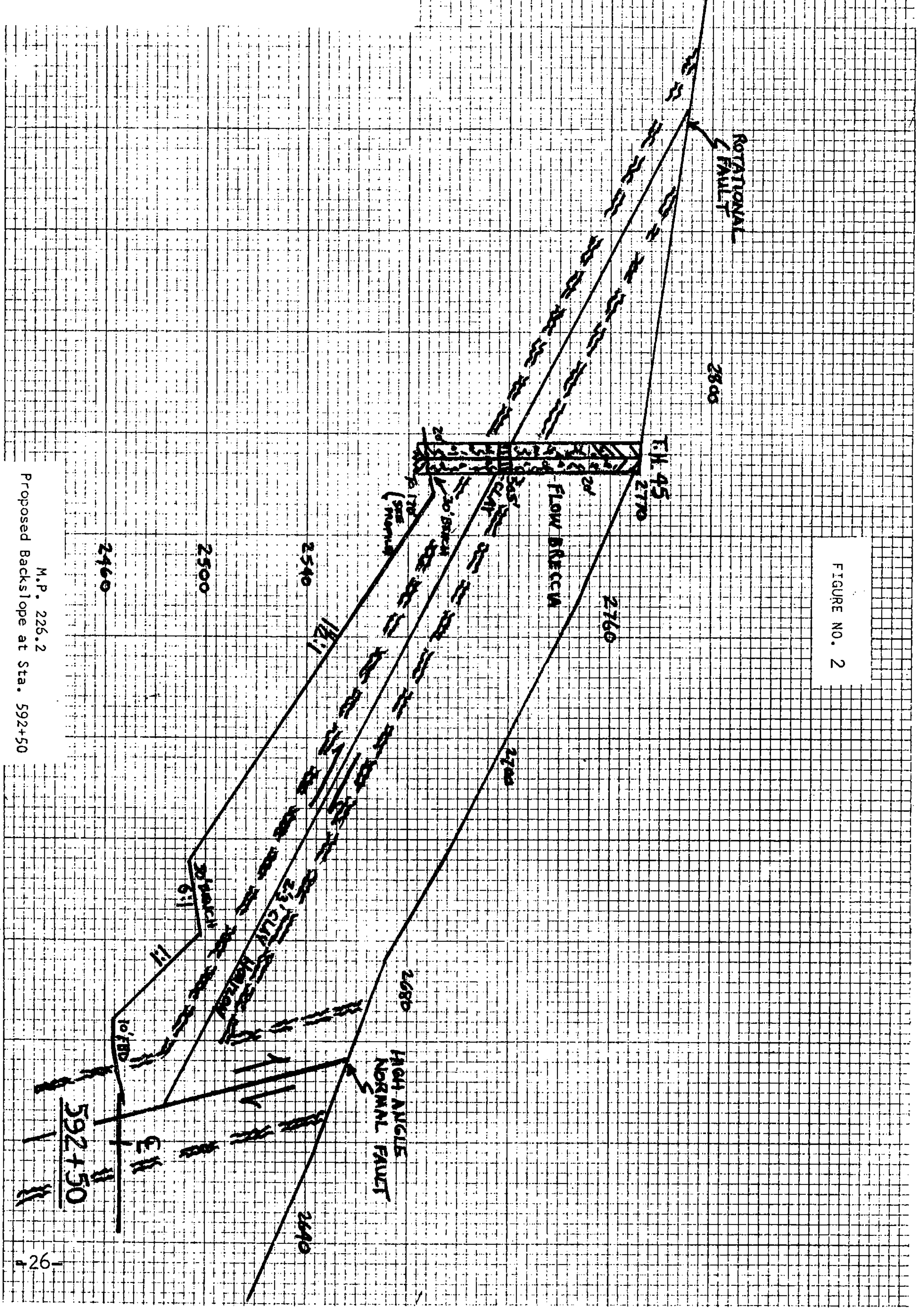


FIGURE NO. 2

M.P. 226.2
Proposed Backslope at Sta. 592+50

PHOTO NO. 7

Constructed Backslope at Sta. 590

Approximately one million yards of material was removed from this cut with this design, which has not experienced any movement or sluff. There of course, has been some criticism, saying the cut was over-designed, primarily because it has not failed.

Station 615 - Settlement Situation

This is an area which is in process of settling. The question is, why? No geologic hazards were present and no problems were expected.

In July 1972 tension cracks (1 inch to 2 inches across) appeared in the base material somewhat paralleling centerline. They more or less followed the projected natural ground line for 200 feet and then disappeared into the embankment. This was thought to be a minor adjustment and ignored.

The project was paved and along in July 1974 the tension cracks reappeared in the same location and of the same magnitude. More concern was generated, especially after an article appeared in the local newspaper.

An examination of the mass diagram and talks with the Resident Engineer showed that the bulk of the embankment material was derived from a degrading "Lower" basalt unit that upon exposure to the atmosphere began to crumble, eventually turning to sand.

Station 670

This area, (Figure No. 3) was defined as potentially unstable as existing tension cracks were present on the natural hillside. Deep talus was resting unconformably on a soil terrace. If failure occurred the two outer lanes of the proposed highway would be displaced. Seismic traverses were used to delineate the depth of the soil and talus and rock line (Figure No. 3). A dozer trench was placed to determine the amount of variation within the talus deposit, and if any thin, but detrimental clay or ash horizon was present. None was found.

Rather than shift the alignment into the hill, thereby increasing quantities which were already in excess, management decided to take the risk. The plans indicated that no wasting or side casting would be allowed in this area. Unfortunately, the Contractor was not aware of this and began to end dump. Although he was stopped the damage had been done.

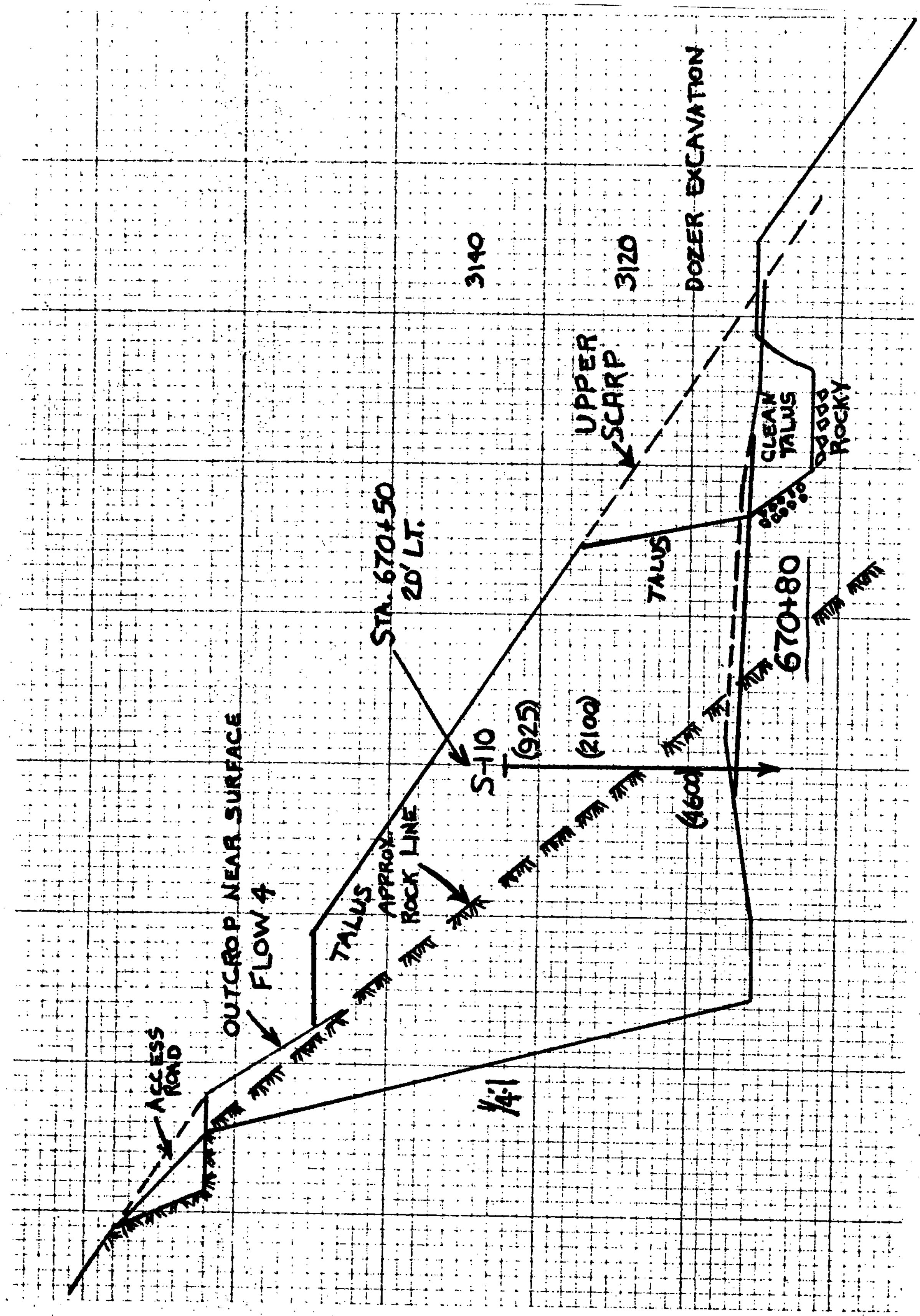
The first indication occurred during November 1970 (Photo No. 8) in which an arcuate scarp had developed along the talus-bedrock interface. By January 1971 the vertical drop was between one and two feet. Fourteen months later the vertical drop was over eight feet in places and had extended into the diced bedrock, (Photo No. 9).

At this time it was decided some corrective action was necessary as the slip plane was continuing to grow and new tension cracks were forming. The most feasible solution was to move into the hillside unless solid rock was situated at a reasonably shallow depth which would allow for a retaining structure.

Due to the nature of the material, coring and tricone drilling were out of the question. It was therefore decided to hire an air track and record the color of the cuttings and drill rate through the talus and bedrock. Twenty-eight holes (952 linear feet) and $1,008.00 later, Materials determined that the amount of variation in respect to rock line was due to a highly undulating pillow-plagonite unit. Also, a significant difference in the potential of the equipment was shown. A more shallow, competent rock line was indicated with the State drill (250 compressor) vs. a rented MR track (500 compressor).

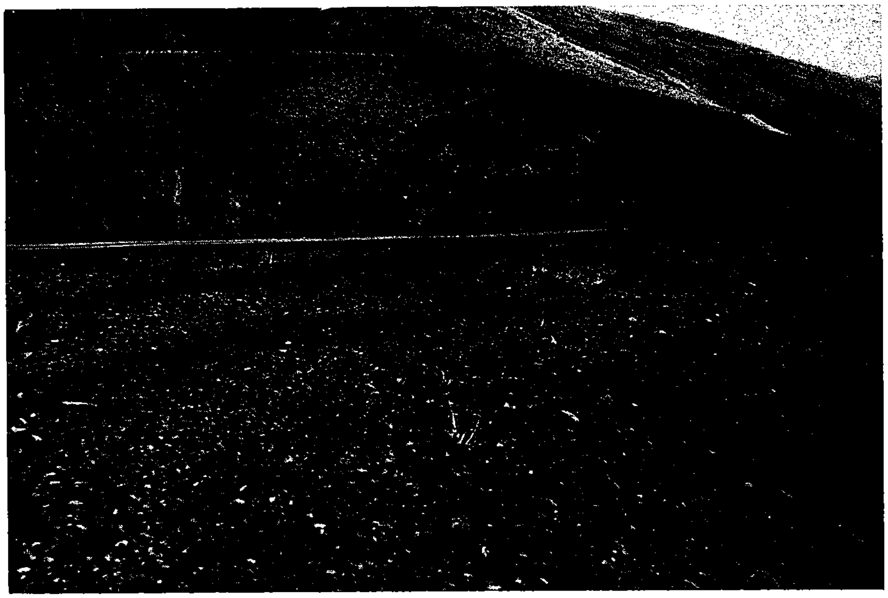
Too much variation existed in the rock line for either the seismic to prove valuable or for the operator to interpret accurately. Due to the variable rock line it was decided to shift the alignment into the hillside and reslope. During July 1975 it was noted the same tension crack has reappeared with approximately 2.0' of vertical drop. It is fortunate the alignment was shifted into the hill.

FIGURE NO. 3



M.P. 227.7
Sta. 670. Slope Design and Embankment Section

PHOTO NO. 8



M.P. 227.7
Sta. 670. Failure Plan "E" Developing After
Wasting on Hanging Terrace

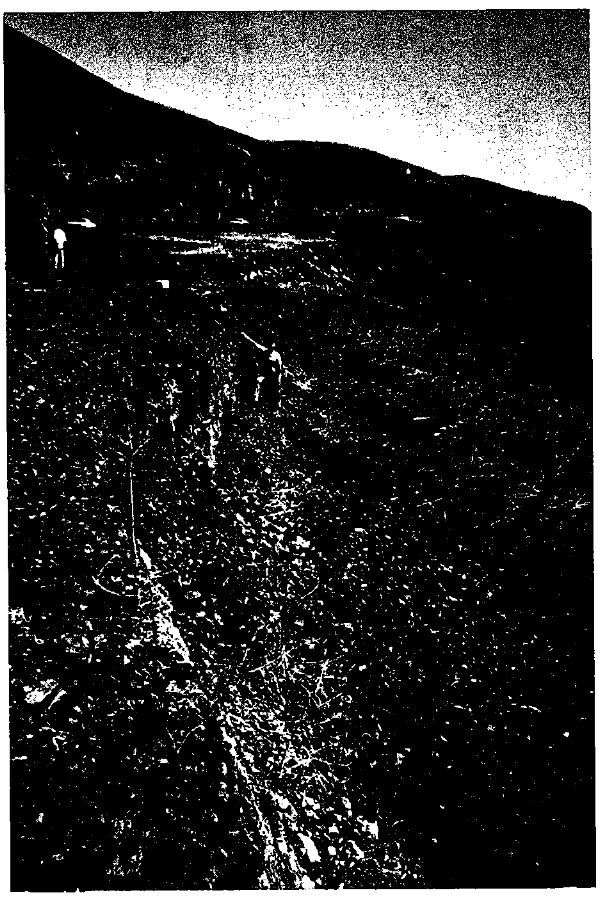


PHOTO NO. 9

M.P. 227.7

Sta. 670. 14 Months After End
Dumping Occurred

Station 696+10 to 704+45 - Retaining Structure vs. Sliver Fill

During the final design stages of this project (November 1968) there were two drainages which were still in question. These were sharp, steep, confined gulleys with minimal soil cover. The only water present was during spring runoff and this was negligible.

The question was whether to extend a sliver embankment down over the hillside some 280 feet vertically, that could be displaced by a major valley fault, or use some type of retaining structure that would stand 39 feet high and some 160 feet long at grade.

After a cost analysis and review of various types of retaining walls, sales promotions, and a lot of inter-department hassling, Gabion-type bin walls were the answer. The cost estimate for Gabions at that time was approximately half that of the conventional metal bin wall, which the Department was later to find out why. The rock baskets could be easily adapted and keyed to the steep walls of the drainage and it would be ideal to use native basalt. No haul or crushing costs were involved as the quarry rock for rock cap and plant mix was only 200 feet away. The following sequence of Figure Nos. 4 and 5 and Photo No. 10 show the construction procedure along with the end product - failure.

After the failure a backhoe was used to paw around to see where the collapse had occurred. Field examination showed that Course No. 7 of the open rock crib began to bulge taking the front row of Course No. 8 with it. The counter-forts of Course No. 8 were still in place. This was an internal failure due not only to an inadequate design but also to construction procedures. Don't use an open crib design on a relatively high wall. Colorado Highway Department has experienced similar results (Personal Communication).

After the failure the design criteria from the Manufacturer changed radically, which increased the cost roughly 100 percent. This included turning the baskets 90° to centerline which increased the rock quantity by nearly double. The twist ties to hold the baskets together were now obsolete. The Manufacturer recommended a spiral lacing procedure. Also the front row of rock facing in the cribs which is the outer wall surface should be hand-placed to insure total stability. A metal bin wall was put in as the replacement and has performed very well to date.

Station 763 - Interbed Treatment vs. Slope and Bench Design

A five-foot to ten-foot thick silty interbed sandwiched between a tightly jointed, vesicular unit below and a more loosely jointed columnar unit above was the subject of a special investigation. Fortunately the interbed was dry, which meant only spalling and differential weathering would have to be considered.

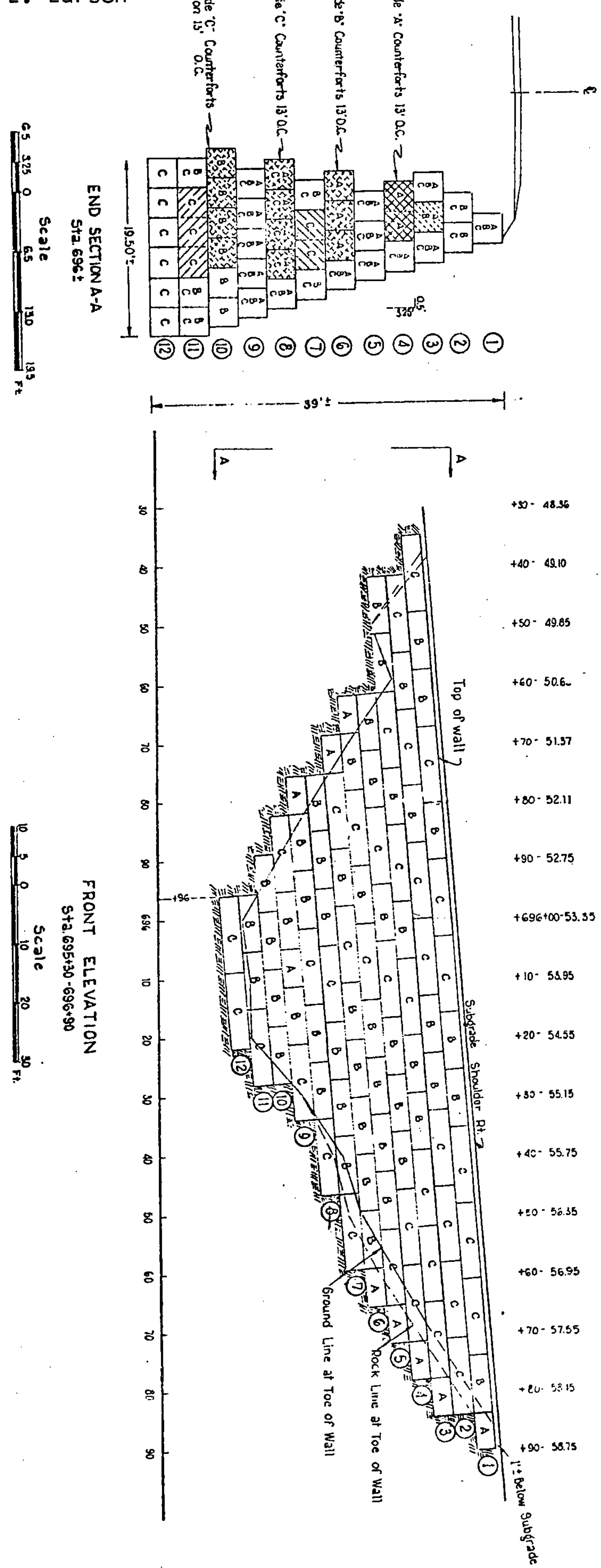


FIGURE NO. 4

M.P. 228.2
Plan Sheet of Proposed Gabion Retaining Wall

FIGURE NO. 5

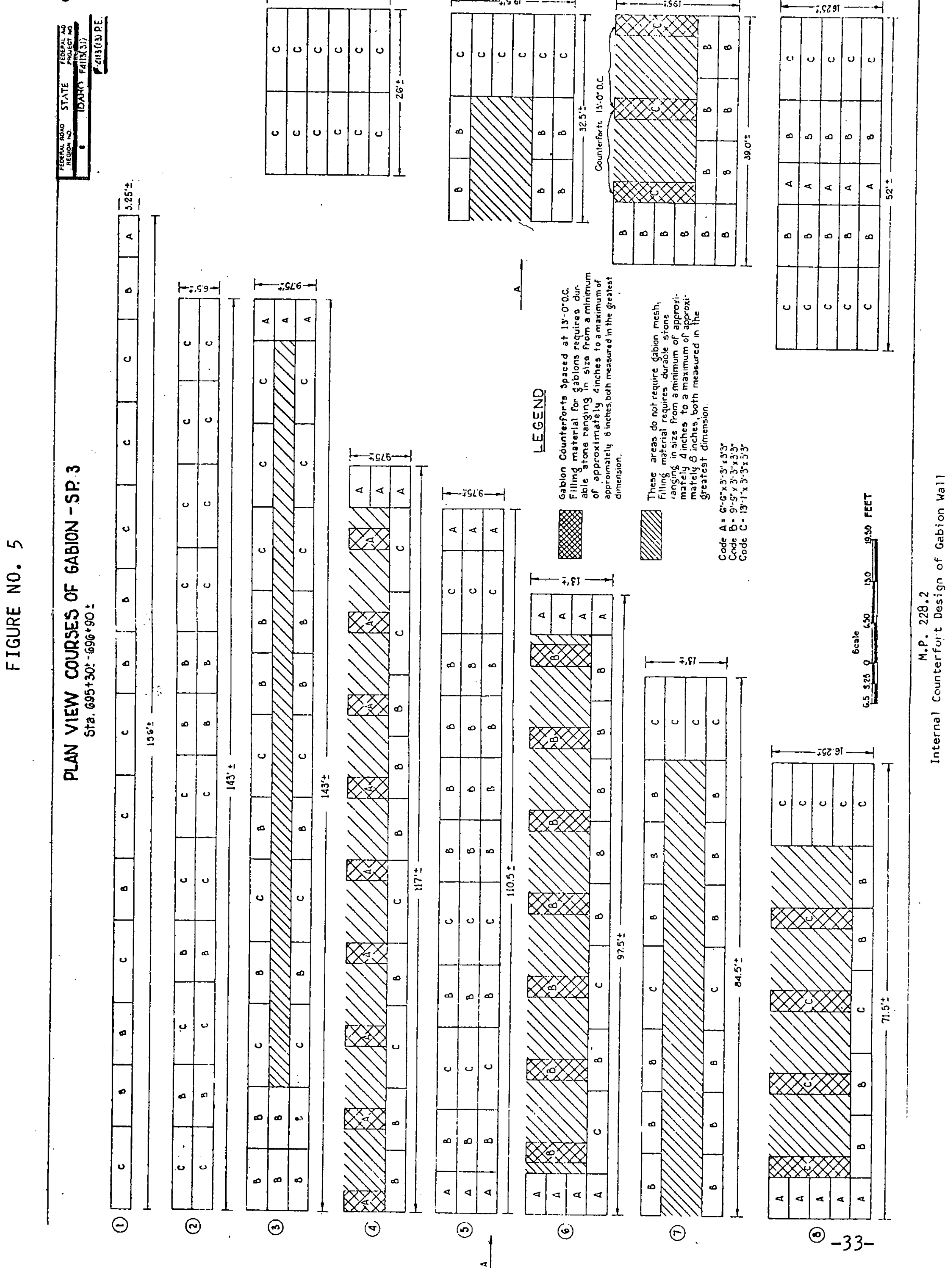

PLAN VIEW COURSES OF GABION - SP. 3
Sta. 695+30± - 696+90±
LEGEND
Gabion Counterforts Spaced at 13'-0" O.C. Filling material for gabions requires durable stone ranging in size from a minimum of approximately 4 inches to a maximum of approximately 8 inches, both measured in the greatest dimension.
These areas do not require gabion mesh. Filling material requires durable stone ranging in size from a minimum of approximately 4 inches to a maximum of approximately 8 inches, both measured in the greatest dimension.
Code A = 6'-6"x 3'-3"x 3'-3"
Code B = 9'-9"x 3'-3"x 3'-3"
Code C = 13'-1"x 3'-3"x 3'-3"
Scale
FEET
Counterforts 13'-0" O.C.
M.P. 228.2
Internal Counterfort Design of Gabion Wall

PHOTO NO. 10

M.P. 228.2
Gabion Failure

The interbed was initially discovered in the bottom of a diamond drill hole. Once it was known to exist it could be extrapolated along the regional dip to adjacent cut sections. In this instance the only way seismic could delineate the interbed was to attempt up and down slope traverses, which would be plotted to the ground line of a cross section. This was done successfully on later projects where there was minimal overburden to contend with.

Once the thickness and characteristics of the interbed had been determined, benching was the next consideration as the height of the cut was considerable. It was finally determined that benching below the interbed, within the tightly jointed, diced basalt unit would be the most satisfactory. This was determined by seismic analysis as the velocity in this unit was nearly twice as much as the overlying unit, even though the rock quality was the same, Photo No. 11.

PHOTO NO. 11

M.P. 229.20
Sta. 757 + 50. Constructed Backslope

During construction, it was discovered that a small fault (30' through) intersected the backslope which meant that the bench was composed of interbed material in part rather than in the diced unit as proposed. A post construction examination shows that the outer 3.0' of the bench is in danger as tension cracks are present. The interbed has also undercut the overlying columnar unit which has caused minor rock fall onto the bench.

GEOLOGIC MAPPING AND SUBSURFACE INVESTIGATIONS

Sound geologic reconnaissance mapping is essential to insure the successful and timely completion of any project. The answers needed to solve alignment problems may not necessarily be confined to the project but scattered throughout the geologic province. Since this may be miles off, the project management may take a dim view of such wanderings.

To understand the mechanics of the province will add greater insight into the detailed investigation in respect to drill hole placement and geophysical traverses.

The $20,000 spent in punching a dozer road not only gave easy access to the project but also created new exposures that helped define the type and quality of rock. Even with this method such units as the pillow-plagonite member and minor fault displacements, and vent areas were completely missed.

Also related is the noticeable change in the method of investigation. Prior to the White Bird projects discussed in this paper, drilling was the only method of subsurface investigation employed by the Division of Highways. This was not only expensive but also very time consuming. Such an example is the White Bird summit cut in which 31 diamond borings (4481 linear feet) were made for 2300 feet of new road construction, in order to determine the slope design. The F-4113(31) project (18,800 L.F. in length) utilized 28 diamond drill holes (1705 L.F. of drilling) and supplemental geophysical survey techniques. Project F-4113(32) (15,800 L.F. in length) relied predominately on seismic traverses with only 15 diamond drill holes (1056 L.F. of drilling). Since seismic surveys were initiated the number of backslope failures has decreased, along with both cost of investigation and amount of time spent in determining the final design.

DESIGN STANDARDS IN RELATION TO COST AND GEOLOGIC CONDITIONS

One factor which always contributes to approving a project is the total price tag. The Design Section, along with Management, are usually looking for ways to decrease total cost. The normal way, on projects of tnis magnitude, is to balance the excavation-embankment quantities. This may be accomplished in several ways: changing grade, changing alignment, or change of backslope design.

Unless these proposed considerations take into account the geologic environment and a slope of adequate safety, problems may develop as occurred at Station 670. The alignment was shifted to reduce quantities and excavation costs, but placed the line in a precarious position. The remedial measures after failure far exceeded the original savings shown by the line shift.

The problem is that geologic risk is difficult to assess in engineering terminology. There are no figures or values a geologist can rely upon to feed a computer that would indicate the total magnitude of risk involved. It is a relative, educated guess indicating a potentially unstable area which may fail tomorrow, in ten years, or never.

Management should be more attentive or in tune with geologic recommendations and not ignore or discard simply because no equation exists for a set of circumstances. The geologists must be able to convey the potential risk in terms that Management can understand.

The White Bird projects proved to be a real challenge from the quantity standpoint, especially figuring the amount of shrink-swell of the various stratigraphic units, as some of these had not been encountered previously. The ability to predict shrink-swell ratios is based more on the experience of the personnel making the prediction rather than anything else. The F-4113(31) and (32) projects had significant overruns in excavation and embankment quantities due to erroneous estimates based on this lack of experience. In the past seven years however, designing and building highways has increased at a much faster pace than before which has allowed the personnel to gain the necessary expertise. The F-4113(38) project along with later projects have been within 5% or less of the actual shrink and swell of the excavation quantities.

CULVERT PLACEMENT

All cross culverts were originally set up for placement in the bottom of the draws. These foundations had been examined in detail for settlement and compaction problems. Corrugated metal pipes were set up which varied from 18" diameter to 1 gauge multiplate types.

Unfortunately, during construction both the skew angle, location, and gradient of these culverts were changed without the consent or knowledge of Materials or Design Sections. Only the multi-plates remained in the bottom of the draws.

The smaller cross drains were raised, so the outlet end was only a few feet below sub-grade. This meant the runoff would spill directly onto the newly construction embankment which was part soil and part rock. Deep erosion channels were carved after the first cloudburst. The down-cutting was especially severe throughout the waste areas.

Not only was erosion a problem, but also separation of the pipe joint between the compacted versus the non-compacted section.

Since there was an abundance of waste on the (31) and (32) projects, the uphill side of many embankments were filled. The culverts situated in the embankments, did not flow water, but remained high and dry, as they were not extended to the "V" of the draw which is where the water is concentrated. This water would instead seep into the fill, seeking its own level and hopefully will pass through the compacted embankment without causing any damage.

The solution is easy--end the culverts into the draws. If the gradient is too severe, add dissipaters and splash pads on the outlet end to check erosion. In future projects at least keep the inlet end in the drainage at natural ground.

BENCH DESIGN

Bench width designed to alleviate rock fall conditions from the finished driving surface should in no instance be narrower than 15'. Anything less gives no margin for error, especially since both contacts and rock quantity may change drastically over a short distance. This width recommendation would be dependent on the backslope design above the bench and prevailing rock conditions. This may be an unknown in many instances until construction begins, especially if the cut was designed or extrapolated seismic data.

The question then asked is whether to design a slope near vertical with a wide ditch and let the post construction cost of rock fall become a maintenance item, or flatten the slopes, and increase quantities of excavation under the existing contract.

The Resident Engineer has the option to make any changes he feels are needed, but unfortunately the Resident is not the most qualified to analyze the situation. There should at least be a consultation with District Materials Personnel on any change either in slope or bench design, as it may be a critical item.

PRESPLIT CONCLUSIONS

A number of changes should be included within the Division of Highways specifications.

In my opinion, the use of stemming which is the backfilling of borings along the neat line with gravel or rock chips after the explosives have been placed would possibly eliminate the scalloping and over-break in high velocity, resistant flow units where the jointing and fracture habit is only nominally developed.

Even though this would be more expensive, it could alleviate the need for scaling and produce an aesthetically appealing slope with less maintenance. The increased cost of stemming would be offset by the decreased scaling cost.

Experience and accumulated knowledge of rock type and conditions seem to dictate between success and failure of a slope, rather than hard and fast rules. One way to insure this would be to pay for only the footage which shows a percentage of undisturbed hole trace left in the neat line of the backslope. The contractor would not be reimbursed for holes which wandered or drifted beyond the specified tolerances.

DEGRADING LOWER BASALT UNITS

Unfortunately these previously mentioned acceptance tests are applied only to rock which is being considered for aggregate. There are no tests for excavated rock which will be compacted in a fill section.

Basalt from these degrading flow units initially broke out in 2 to 4 foot chunks, many of which were used as barrier rocks. Within one year 66%

of these had experienced cracking, separation along joints, and decay into mounds of granular debris.

There is no reason not to expect the same phenomena to occur within the embankment, similar to the effect of an hour glass passing sand from the top through a constriction to concentrate at the bottom. With a calculated swell of 35% from adjacent degrading cuts, voids were built into the embankment. Atmospheric exposure and moisture would initiate the swelling sifting process, that eventually causes settlement.

These flows must be recognized as degrading units and be treated accordingly. The material itself is perfectly acceptable to use in embankments but rather than using the standard grid roller for rock placement, a vibratory roller should be substituted with an adequate amount of water.

This would accelerate breakdown to a sandy component while the embankment is being constructed eliminating most of the voids so the fill would not experience adverse settlement at later date.

The F.H.W.A. has recommended using benzidine dihydrochloride as a staining agent in order to detect the presence of swelling clays which may be present within basalts, especially lower basalts. This chemical, soluable in water and alcohol, may be applied to any hand specimen in the field if it is suspected to degrade. It will immediately turn blue if montmorillonite is present. This procedure should be incorporated into the investigative procedures used by the Department rather than relying on visual indications of deterioration. The geologist could and should test all stratigraphic units. During the Phase II report and set up special handling, placement and rolling for this material in the Phase V Special Provisions.

The Corp of Engineers also indicates that the application of D.S.M.O. causes a more complete breakdown than Ethylene Glycol.

SEISMIC EVALUATION

This single channel method of invetigation for deep cut analysis is slow and consumes a 3-man crew, especially if explosives are used, since charges must be set at each station in order to secure a point on the graph which automatically doubles if the traverse is to be reversed.

If the charge is not heavy enough, second cycle arrivals are received which means the station must be re-shot. If too much powder is used too close to the geophone, a lower threshold or plateau level is achieved which sometimes makes it difficult to pick contacts and blend the data into the preceding hammer line traverse. It was noted that explosives will normally duplicate the hammered slope. A differential in arrival time of 1 to 2 milliseconds is normal due to increased energy.

A charge backfilled by soil, will usually produce better first arrivals than a charge packed with loose rocks.

Traverses which extended across fault gouge zones need additional energy to develop first arrivals, usually 3 to 4 times as much as compared to a line of comparable length within unaltered rock.

SETTLEMENT OF LARGE FILLS

The greatest single factor governing the performance of a fill section appears to be time. The paving contract for the F-4113(31) and F-4113(32) projects were let in 1973 or about 2 years after completion of the embankments. During this period the fill sections had time to set and adjust. The vibration of haul traffic created by construction equipment produced additional settlement. Remeasurement, for the subsequent paving contract, determined that approximately 2.0' of settlement had occurred in the preceding period. (Station 773+50, depth of embankment 150' at centerline).

It therefore is apparent that the embankment settlement time factor becomes important for normal construction procedures. It is suggested that the use of vibratory rollers and adequate amounts of water will reduce this time factor required for settlement and produce a more tolerable limit.

RAPPORT WITH HIGHWAY CONSTRUCTION PERSONNEL

It is imperative that the Materials Section acquaint the Resident Engineer and Project Engineer with the soils-geophysical profile (Phase II Report) as there is a wealth of information that can be utilized during the construction of the project. Also, accompanying the profile is a topographic-geologic map with the various engineering units noted and described, with unstable or hazardous conditions noted. The alignment should be reviewed, with the Resident Engineer, in the field, at the time the Special Provisions are being drafted to see if he concurs with the recommendations proposed or can specify a more feasible approach.

After the project is awarded the Resident Engineer, faced with a field problem or change which will affect the previous recommendations made by the Materials Section, should consult with that Section.

Post-construction failures and problems could be alleviated by this coordination.

CONCLUSIONS - SEISMIC

1. Fault gouge may be minimal and not show a normal seismic peak. This could depend upon velocity of the adjacent flows.

2. In basalts, low velocity is a function of jointing, not necessarily of the rock quality.

3. Don't assume vertical contacts on seismic data in basalt sections that are covered with varying amounts of soil-colluvium-talus.

4. Faults-gouge and interbeds can be delineated if the stratigraphy and geologic history are understood. Lack of field time and basic mapping leads to more interpretation mistakes.

5. In areas of rock fall vs. slope design, correlation must be made from existing exposures and seismic velocities.

6. Velocity range, less crucial at higher velocities, but more room for error in flatter slope interpretations.

7. A gray area in basalts, 2500 to 4000'/sec. may indicate pillow lava, shear zones, altered or tight interlocking talus, moisture, flow top and soil. Additional investigation is warranted in these questionable areas.

8. Low velocities not always indicative of ripping vs. presplitting. Pillow lavas and breccia, due to lack of prominent jointing, can be presplit economically.

9. Contractors have found geophysical information extremely useful in bidding the job and using the information for presplitting and production round blasting vs. ripping operations, which generally means a better construction price.

10. It must be remembered that a seismograph is only a supplemental tool of your exploration program and must be coordinated and calibrated to existing norms.

ACKNOWLEDGEMENTS

Direction for this project was given by George W. Williams, the University of Idaho Geology Department Chairman, John G. Bond, P. G., Senior Geologist and Acting Director for the Idaho Bureau of Mines and Geology. J. W. Schumaker, former District Four Geologist, Department of Transportation, now with Walter Lum and Associates, Honolulu, Hawaii, prepared the prototype soils profile which the District has adopted as standard procedure and was the inspiration for the scale model approach. R. G. Charboneau, Chief Geologist, Idaho Transportation Department, Division of Highways, offered both guidance and criticism, primarily with geophysical techniques and preparation of the manuscript. William B. Hall, Professor of Geology, University of Idaho provided many of the preliminary aerial photos and M. W. Lotspeich, District Engineer, Idaho Transportation Department, Division of Highways who authorized and supported this paper.

ENVIRONMENTAL GEOLOGY SURVEY, REGIONAL LAND USE AND INTERMODAL TRANSPORTATION PLANNING, NORTHEASTERN PENNSYLVANIA

By

J. David Welch
Joseph S. Ward and Associates

ABSTRACT

The seven-county region of Northeastern Pennsylvania covers an area of 4,500 square miles and is characterized by a diversity of geologic, hydrologic, soils and land use conditions. In order to formulate realistic and effective land use and transportation strategies for this region, it is first necessary to evaluate the direct and indirect effects of environmental geologic conditions on contemplated land planning and development.

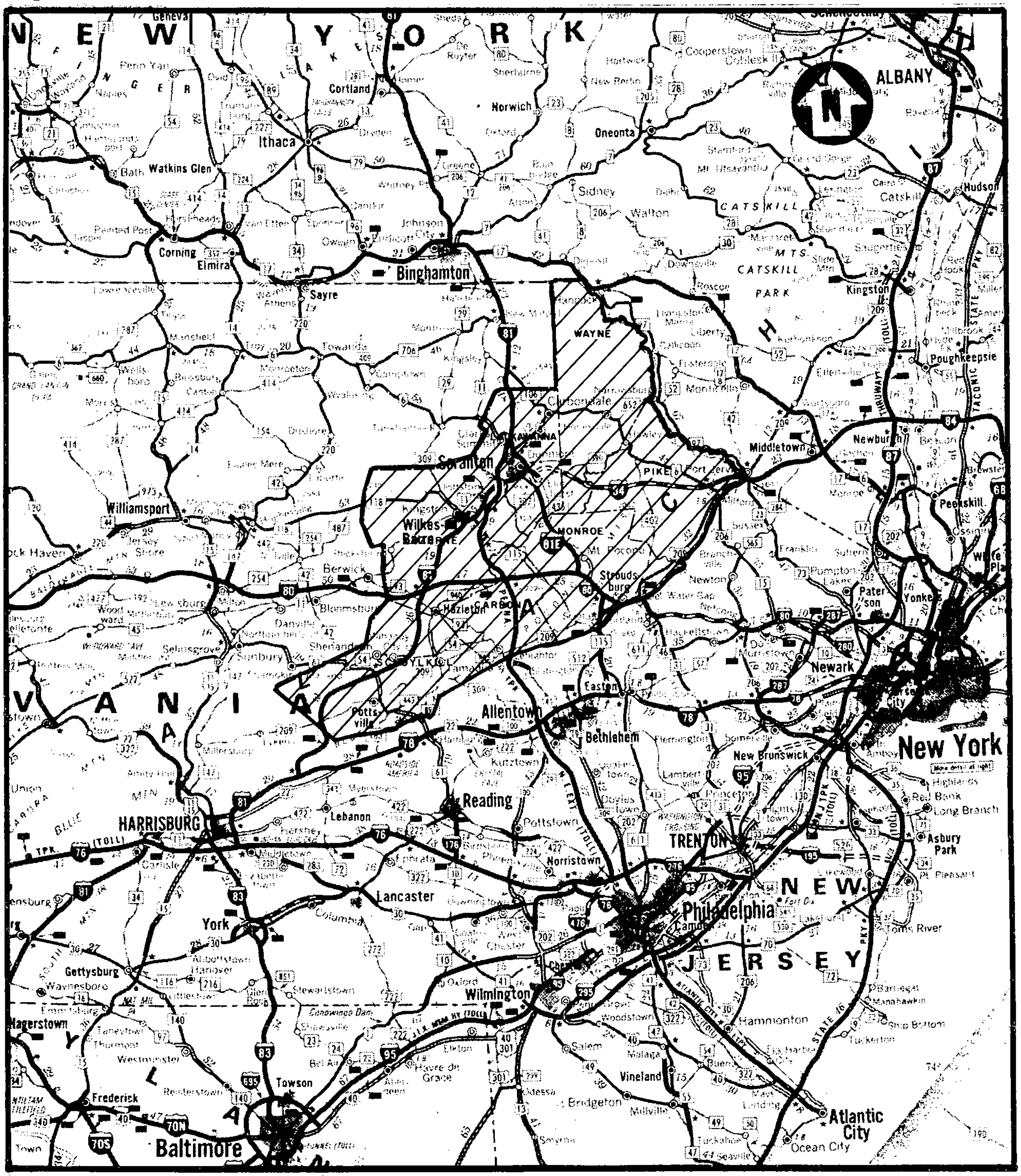
The environmental geologic investigation consisted of two phases. The first phase was to identify and map the existing geologic conditions for the entire region at a scale of 1:50,000. Specific factors investigated included regional land slope; base geology, bedrock, tectonics and engineering characteristics of bedrock; soil associations and their engineering properties; and hydrology, including flood plain delineation, high water table areas, and ground water potential. The compilation and correlation of these conditions resulted in the subsequent evaluation and mapping of potential geologic hazards and constraints which could impact regional development. Included was the identification of areas of potential subsidence due to coal mining or sinkholes in soluable limestone, landslide potential, slope instability, and seismic activity. Specific constraints delineated included areas of shallow rock, flood plains and adverse soils.

The second phase was to develop detailed geologic conditions for four representative local areas situated within the seven-county region in an effort to be more responsive to geologic constraints to planning at the local level. Base mapping for these four areas was performed at 1:24,000 scale. Specific detailed geologic conditions evaluated included local slope, base geology, soils/surficial geology and hydrology. The evaluation of this data resulted in the identification of geologic hazards and constraints to development at the local level in each of these areas.

INTRODUCTION

As Phase I of a three-phase regional land use-intermodal transportation program covering the seven-county area of the Economic Development Council of Northeastern Pennsylvania, this Environmental Geology Survey was an inventory analysis. The regional location of the study area, consisting of Carbon, Lackawanna, Luzerne, Monroe, Pike, Schuylkill and Wayne counties, is shown in Figure 1.1. The specific site location of the study area in relation to eastern Pennsylvania is shown in Figure 1.2.

The other two phases of the study relate to regional land use strategy and a multi-modal transportation program. Wilbur Smith and Associates is

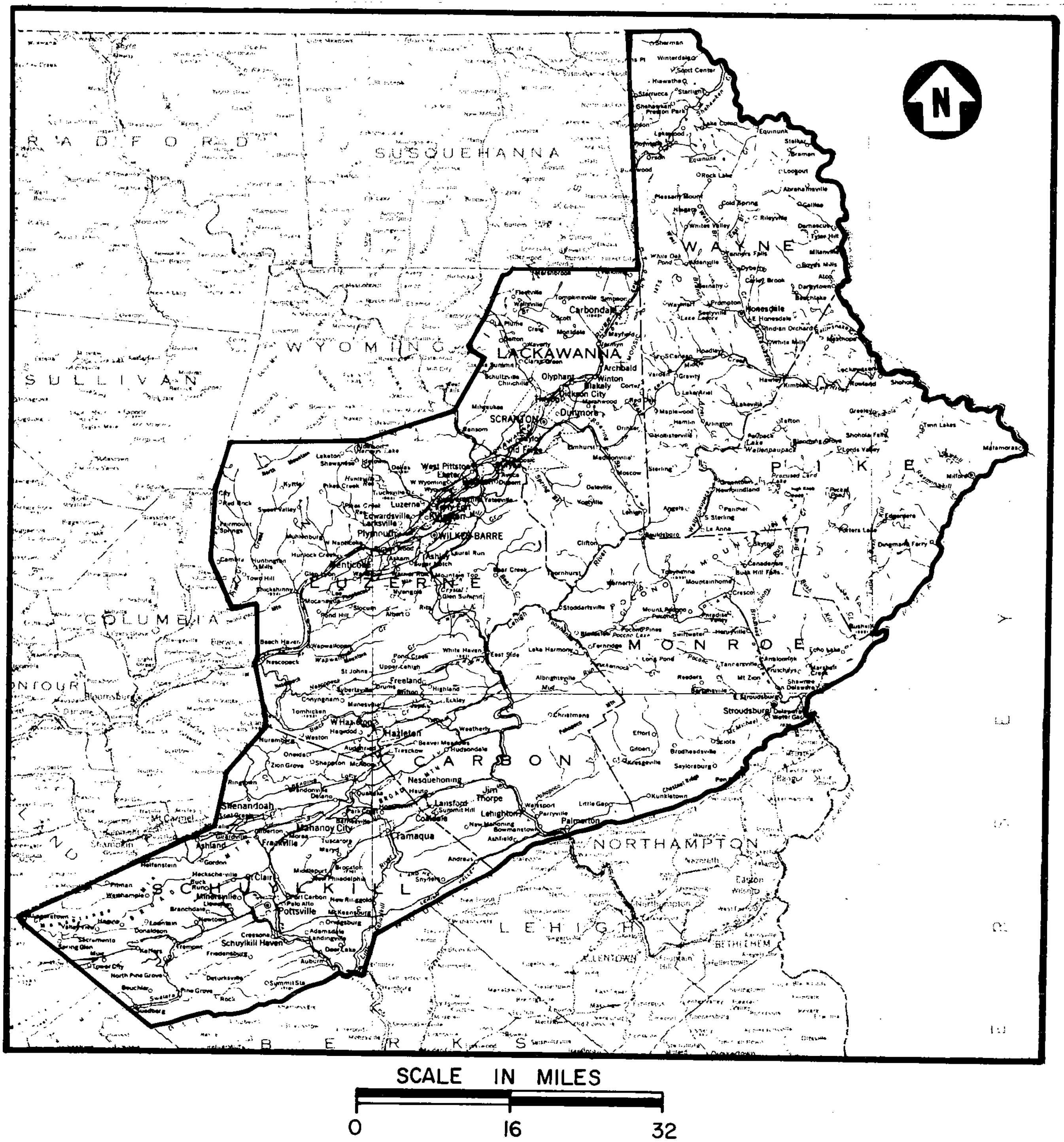


SCALE IN MILES

0 33.5 67.0

MAP SOURCE: BASE MAP WAS ADAPTED FROM AUTO MAP OF EASTERN UNITED STATES.

FIGURE 1.1 REGIONAL LOCATION OF STUDY AREA



MAP SOURCE: BASE MAP WAS OBTAINED FROM U.S. DEPT. OF INTERIOR GEOLOGICAL SURVEY MAP, STATE OF PENNSYLVANIA, 1955 EDITION.

FIGURE 1.2 SITE LOCATION MAP

responsible for the overall direction of the multi-disciplinary program with major support from Joseph S. Ward and Associates, Candeub, Fleissig and Associates, and Berger Associates.

The Environmental Geology Survey consists of the collection and evaluation of data on geologic conditions of the region and is in direct support of the other two phases. The regional land use strategy, derived from the evaluation of several viable alternatives, will result in the formulation of a policy plan for regional development. In conjunction with the land use strategy, the multi-modal transportation program is to indicate the future transportation needs of the region. These are contributed to, and supported by, this Environmental Geology Survey. The output of this multi-disciplinary program is directed towards full accomplishment of realistic and effective development plans for the Northeastern Pennsylvania region.

(As an editorial aside, I wish to express that it was extremely difficult to write this paper. I believe it was Hardy Cross who commented that he never had time to write a short paper. Not only was the time of preparation hampered by the scheduling of the Symposium, but the subject matter was voluminous. As originally conceived, this environmental geology survey was to consume six months of an overall study schedule of 14 months. The inventory graphics resulted in 55 map plates, and the supporting report had 163 pages of text, exclusive of glossary, and bibliography. A total of 10 figures, and 22 tables and matrixes supported the text.)

Statement of the Problem

The Environmental Geology Survey consists of two major tasks. The first task is to identify and map an environmental geologic inventory of the entire seven-county area. This, in turn, leads to the identification and mapping of the geologic constraints to development and the evaluation of the impact of these constraints on future land use strategies and transportation alternatives of the 4,500 square mile region. The general aspects of the geologic environment identified and evaluated include:

- Surficial Soils and Bedrock Types of the Region and their Engineering Properties
- Geologic Flood Plains
- Susceptibility of Surface Deposits to Erosion
- Natural Resources including Aggregate, Metallic Minerals, Natural Gas and Groundwater
- Landslide Potential and Construction Slope Stability
- Suitability for Septic Tanks, Leaching Fields and Sanitary Landfills

The second task develops detailed geologic conditions and constraints for four Demonstration Areas in order to be more responsive to planning at the local level. These are analyzed to identify and map the detailed geologic factors which affect future policies for land use planning, zoning, transportation and other regulatory decisions. Selections were based on those individual 7-1/2 minute quadrangle areas which represented a wide cross section in terms of physiographic, geologic, land use, and socio-economic conditions.

Methodology

Existing data pertaining to the geologic environment and the seven-county region was collected, including USGS topographic maps, geologic, hydrologic, and soils maps, and their associated publications, and various forms of intermediate altitude aircraft and spacecraft remote sensor imagery. High altitude aircraft imagery, reported to be available at the start of the program, was discovered to be unusable because of over-exposure and cloud cover.

In addition to the ERTS multispectral scanner imagery at 1:1,000,000 scale, Skylab multispectral camera imagery at 1:1,000,000 scale was also available over the region. Selected frames of Skylab high resolution color earth terrain camera stereo imagery at 1:500,000 scale were also available for portions of the region. Relative to the four Demonstration Areas, A.S.C.S. panchromatic black and white stereo photography was available at a scale of 1:40,000 and black and white photo mosaics at a scale of 1:9,600 were obtained, from Pennsylvania Power and Light Company.

THE SEVEN COUNTY REGION

All pertinent regional data was reviewed, analyzed and subsequently compiled onto a series of four intelligence overlays at a working scale of 1:50,000. The specific information derived includes:

- Regional Land Slopes
- Base Geology, Bedrock and Tectonics
- Soils and Surficial Geology
- Hydrology

The above characteristics were analyzed and evaluated. Potential hazards were identified. These include land subsidence, landslides and flooding. In addition, those characteristics which make it difficult or impossible to use a site for specific purposes, referred to as constraints to development, were set forth. These include steep slopes, shallow soils and bedrock which is difficult to move. Conversely, those areas which are suitable for selected aspects of development, such as septic tanks and solid waste disposal, were also identified.

The base geology maps delineated those areas where land subsidence and landslides could occur as well as those areas where physical properties such as hard rock or intensive fracturing impose constraints. The interpretative data provided with the soils maps showed the dominant conditions which exist within each soil area. A fifth map identified constraints imposed by steep slopes and lands with seasonal high water table as well as lands where flood hazards are apt to occur.

The regional working base maps at 1:50,000 scale were distributed across the region by relation to the USGS 7-1/2 minute quadrangle map boundaries (north-south, east-west grid) attempting to adapt to the seven county boundaries, which generally trend southwest-northeast, as dictated by topography. The regional working base maps were derived from a 0.48 reduction of the 7-1/2 minute quadrangles and combined into irregular shapes. As a result, coverage of the region is by seven base maps sheets of variable size and shape. The distribution of these regional map sheets is shown in Figure 1.3. These functioned adequately during the study phase of the program.

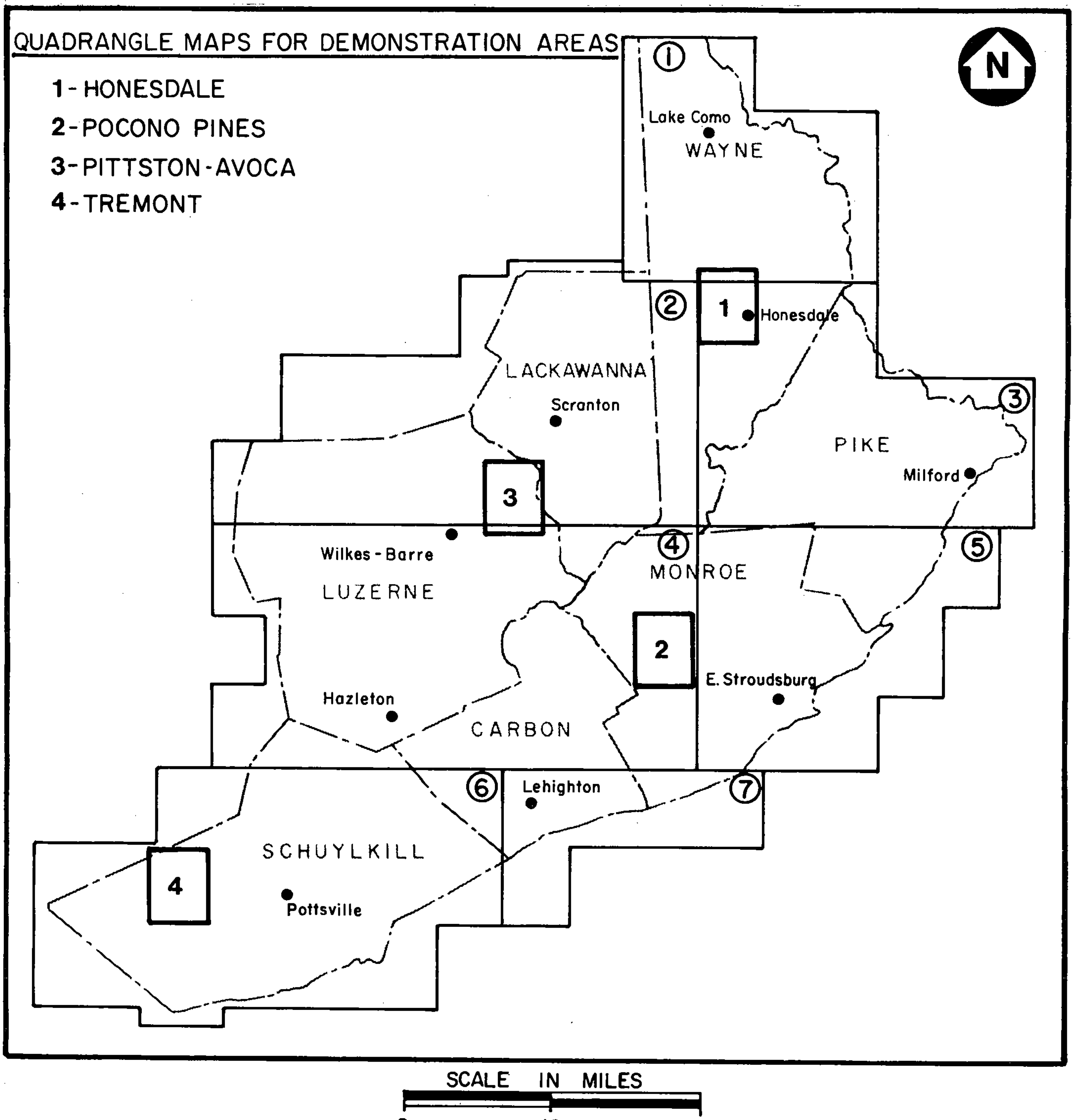
While the study progressed, seven regional base maps were prepared at 1:50,000 specifically oriented to each of the counties. These represented political boundaries, major transportation routes and principal drainage ways, as factors of orientation. For final presentation, the intelligence of the working manuscript overlays was transferred to those county base maps and they in turn were reduced to 1:125,000 scale for printing.

All this may appear as multiple drafting but it was necessary to be sure all parties were working to a common base during a greatly restricted allocation of time. To wait for preparation of the final county base maps was an unaffordable luxury, since, through contractural delays and negotiations, what had been conceived as a 14-month program had to be completed in six months.

From this point on, it must be relied on terse descriptions in order to give the full impact of the study, within time and space limitations. This paper is about the study and its presentation and not about the results of the study. If afterwards, someone's interest is further aroused, they can be referred to the actual report.

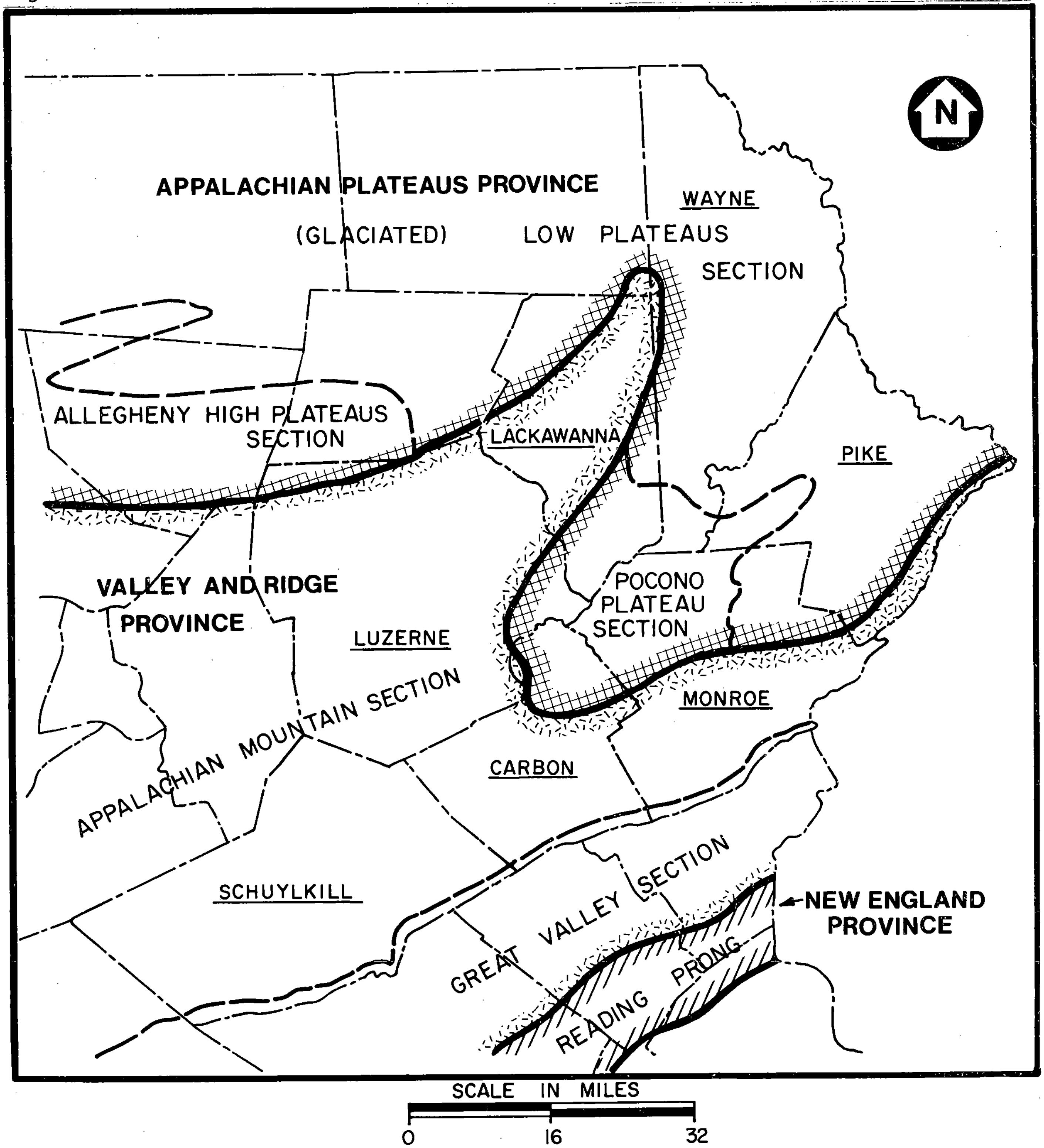
Physiographic Setting of Area

The seven county area is situated entirely within both the Valley and Ridge, and Appalachian Plateau physiographic provinces. As shown in Figure 1.4, the Glaciated Low Plateau, the Pocono Plateau and Allegheny High Plateau sections of the Appalachian Plateau province occupy approximately the northeast half of the study area. The Appalachian Mountain section of the Valley and Ridge province, including the northeast prong of the Wyoming-Lackawanna Basin, characterizes the southwest half of the study area.



MAP SOURCE: BASE MAP WAS OBTAINED FROM U.S. DEPT. OF INTERIOR GEOLOGICAL SURVEY MAP, STATE OF PENNSYLVANIA, 1955 EDITION.

FIGURE 1.3 DATA ORIENTATION MAP



MAP SOURCE: BASE MAP WAS ADAPTED FROM PHYSIOGRAPHIC PROVINCES OF PENNSYLVANIA MAP, COMMONWEALTH OF PENNSYLVANIA, DEPT. OF INTERNAL AFFAIRS, TOPOGRAPHIC AND GEOLOGIC SURVEY.

FIGURE 1.4 PHYSIOGRAPHIC MAP

The physiographic setting of the study area is particularly well defined on a portion of the ERTS-1 mosaic shown in Figure 1.5. This mosaic was compiled from cloud-free ERTS Multispectral Scanner (MSS) images from band 5 and covers the visible red (0.6-0.7 micrometer) portion of the electromagnetic spectrum. Note the clarity with which this band brings out the area's physiographic provinces and enables the correlation and delineation of major structural and stratigraphic boundaries for hundreds of miles. Particular attention is focused on the well defined mountain rims which encircle the Wyoming-Lackawanna Basin at A; the deeply dissected sandstone-capped high plateau at B; the zig-zag pattern at C caused by differential erosion of folded strata; and the overall continuity and topographic expression of Kittatinny-Blue Mountain at D.

Topography. The Appalachian Plateau is characterized by flat-topped hills formed on nearly horizontal bedded coarse-grained sandstones and conglomerates. Unconsolidated or semi-consolidated glacial and glaciofluvial materials are scattered throughout the plateau area. In Wayne, Pike and northern Monroe counties, there are a large number of natural ponds and depressions resulting from glacial deposits damming the original stream courses. The erosional and depositional effects of glaciation strongly controls the topographic expression of the region.

Topography in the Appalachian Mountain section is characterized by a series of parallel northeast-trending ridges and valleys which reflect differences in resistance to erosion by rock having a generally northeasterly strike.

Drainage. The seven county area is drained by five major southerly flowing rivers and their respective tributaries. These are from east to west: Delaware, Lackawanna, Lehigh, Schuylkill and Susquehanna Rivers. The drainage network of both major and minor tributary streams in the plateau area exhibit modified rectangular-dendritic patterns, with some indications of trellis-type drainage development in the southeast portion near the boundary with the Valley and Ridge province. This effect is the result of glacial action as well as structural control of drainage courses by the underlying rock. Drainage in the Valley and Ridge province is characteristically trellis, with individual stream segments adjusting to variations in rock resistance of the underlying folded strata. This structural control results in a general parallelism of trunk streams with marked right angle junctions of their tributaries.

Environmental Geology Inventory

The regional inventory of natural conditions pertained to the four categories previously noted; namely slopes, base geology, soils and hydrology. The report required 65 pages just to describe and discuss (not analyze) the inventory of these data. For purposes of this presentation, they will be only touched upon most briefly.

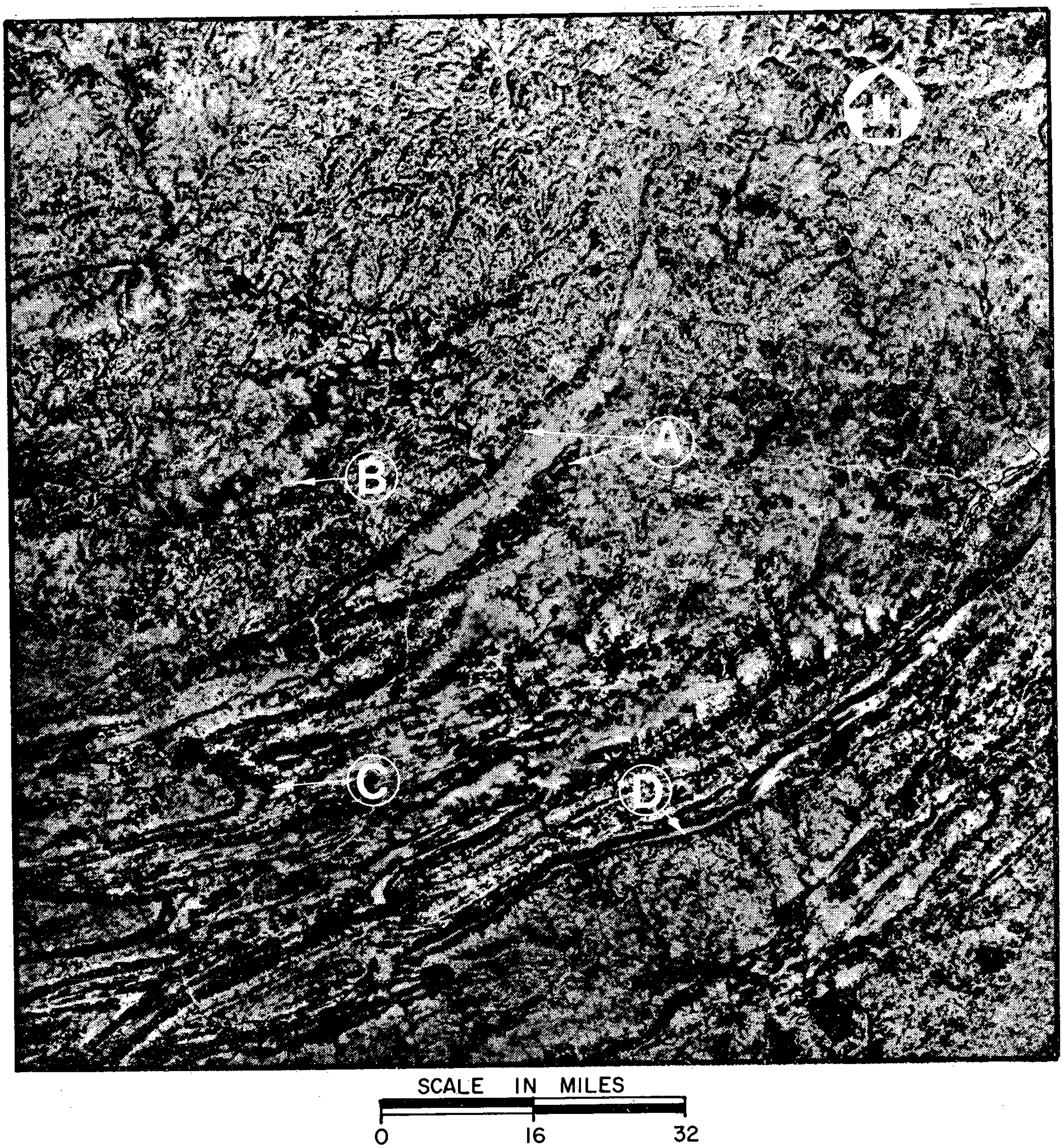


FIGURE 1.5 ERTS IMAGE - PHYSIOGRAPHIC RELATIONSHIPS

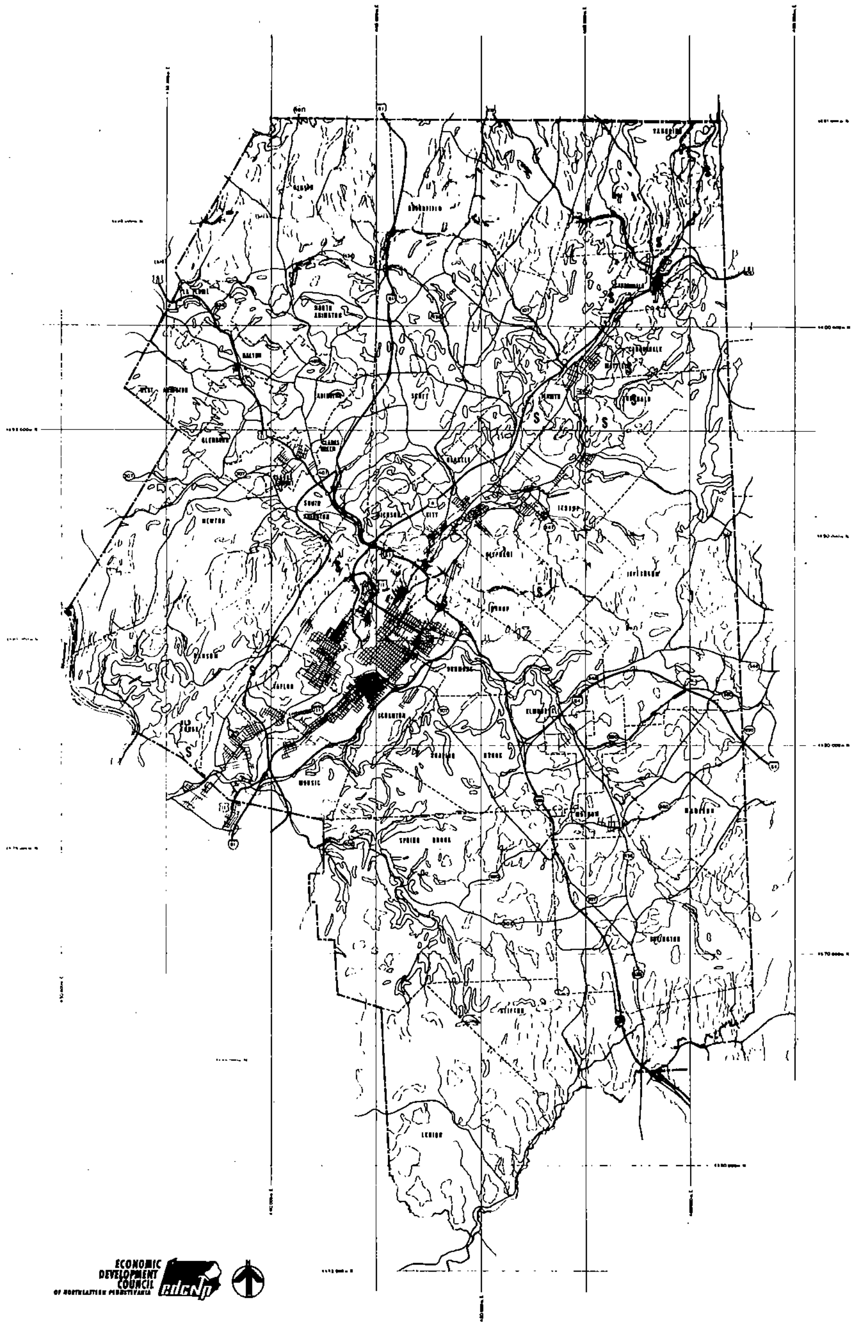
Regional Slopes. The regional slope characteristics of the seven county area, for purposes of the regional environmental geology survey, were classified and delineated in three slope classes; 0-8%, 8-25%, and over 25%. A fourth category, "variable slope", identifies those specific areas that are characterized by intricate surfaces and covered primarily by strip mining activities and related mine dumps and tailings. Variable slopes were mapped primarily in the Anthracite Basin areas of Schuylkill, Luzerne and Lackawanna counties.

As expected, a cursory examination of the Regional Slope Maps reveals that a considerable variety of slopes can be found within the seven county area. In the Plateau area of Wayne, Pike and Monroe counties, for example, the major portion of those slopes greater than 25% are confined primarily to sandstone-capped escarpments and steeply cut stream valley segments. Conversely, slopes on the upland surface generally tend to be less severe, except locally along the steeper portions of some of the higher ridges.

In the Valley and Ridge province, particularly in Schuylkill, Luzerne and Lackawanna counties, the preponderance of steep slopes can be readily correlated with the steeply dipping sandstone ridges that are underlain chiefly by the resistant Pocono and Pottsville formations. Slopes in the valley or basin portions, with the exception of the variable-slope strip-mined areas, generally range in the 8 to 25% category.

Slope is a natural environmental control and, as such, its severity could create practical limitations with regard to suitability for development. Some commonly used optimum ranges of regional slopes for various urban uses and activities are shown in Table 2.1. This table shows that natural slopes of 15% or less are generally most suited to urbanization and overall transportation development. However, this does not eliminate consideration that certain forms of recreation and scenic open space are suited to slopes greater than 25%.

Base Geology, Bedrock and Tectonics. A total of fifteen formations, or correlatable stratigraphic units, are identified for purposes of the regional base geology survey of the seven county area. The chronologic sequence of the major formations and combined map units are presented in Table 2.2. These formations are considered to be fairly representative of the bedrock occurring within the study area. The classification, extent and distribution of each of the units correspond as closely as possible to those delineated on the State Geologic Map of Pennsylvania. Additional information derived from a variety of supplemental data reference sources was also reviewed in conjunction with the State Geologic Map. This, in turn, provided the basis for the final selection of the representative stratigraphic units used in the regional geology survey.



The preparation of this map was financed in part by The Appalachian Regional Commission under section 302 of The Appalachian Regional Development Act of 1965, as amended, App. U.S.C. 302, in part by The Department of Housing and Urban Development, under provisions of section 701 of the Housing Act of 1954, as amended and as administered by The Pennsylvania Office of State Planning and Development and in part by the Economic Development Administration.

Base map preparation by Economic Development Council of Northeastern Pennsylvania.
Technical data compiled by Joseph S. Ward and Associates.
Final presentation by Wilbur Smith and Associates.

LACKAWANNA COUNTY
PENNSYLVANIA

SLOPE

I–2

Table 2-1

RANGES OF SLOPES FOR VARIOUS REGIONAL USES/ACTIVITIES

USE OR ACTIVITY	REGIONAL PERCENT SLOPE		
	0-8	8-25	>25
General Recreation Areas	x	x	x
Engineered Structures*	x	x	x
General Urban Uses	x	To 15%	
All-Weather Urban Roads	x		
Septic Field Systems	x	To 15%	
Conventional Housing	x	To 15%	
Commercial Centers	x		
Interstate Highways	x	x	
Airports	x		
Railroads	x		
Tracked Vehicle Operations	x	x	x

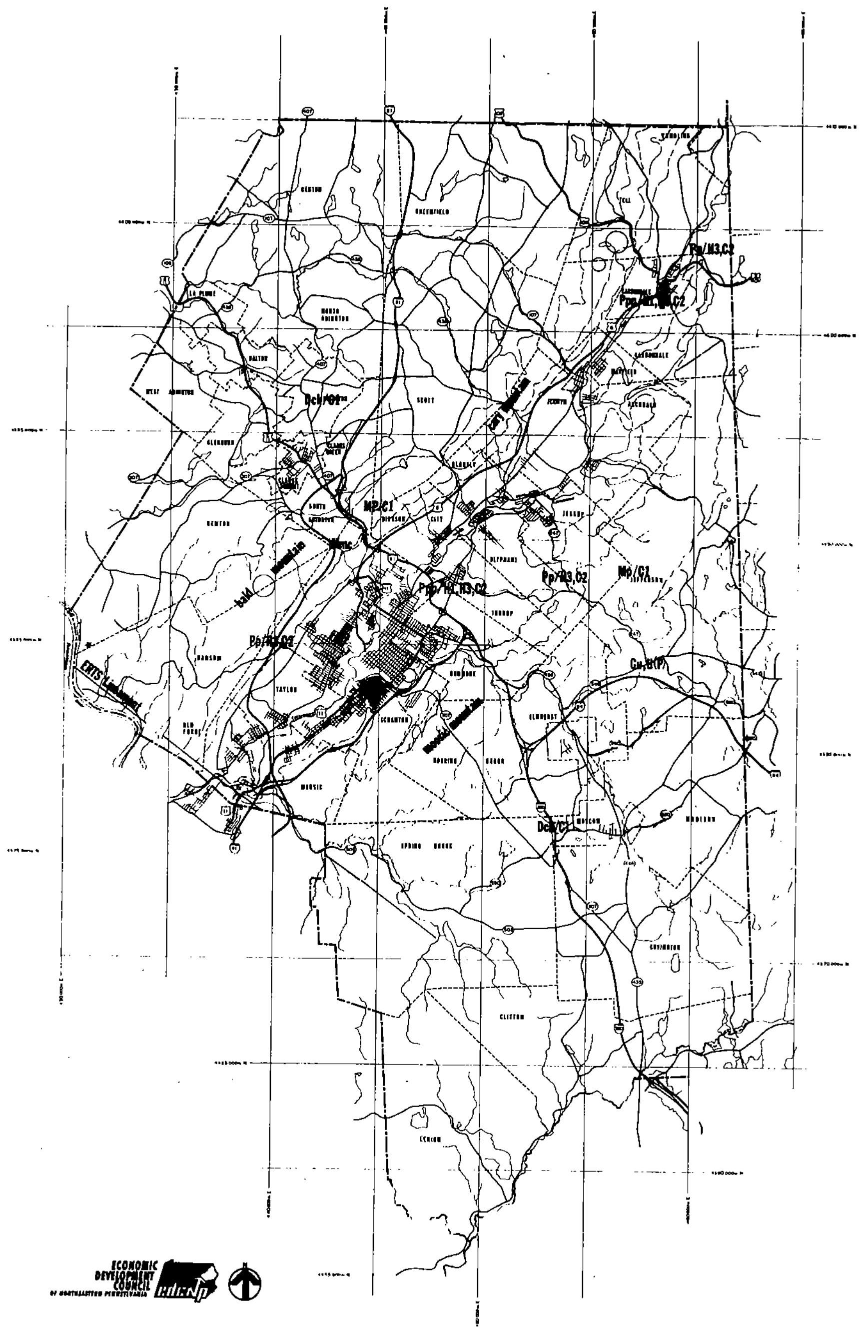
(Adapted from Mayberry, 1972)

*Dams, bridges, tunnels, pipelines, transmission lines, etc.

Table 2-2

N.E. PENNSYLVANIA - STRATIGRAPHIC SECTION, FOR REGIONAL BASE GEOLOGY

PENNSYLVANIAN	Ppp	Post - Pottsville Formations (including Llewellyn)
	Pp	Pottsville Group
MISSISSIPPIAN	Mmc	Mauch Chunk Formation
	Mp	Pocono Group
DEVONIAN	Ds	Susquehanna Group (Undiff.)
	Dck	Catskill Formation
	Dm	Marine Beds (Incl. Chemung)
	Dho	Hamilton Group & Onondaga Formation (Undifferentiated)
	Dh	Hamilton Group
	Don	Onondaga Formation
	Doh	Oriskany & Helderberg Formations (Undiff.)
SILURIAN	Skt	Keyser & Tonoloway Formations (Undiff.)
	Sbm	Bloomsburg & McKenzie Formations (Undiff.)
	Sc	Clinton Group
	Ss/St	Shawangunk & Tuscarora Formations



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Base map preparation by Economic Development Council of Northeastern Pennsylvania.
Technical data compiled by Joseph S. Ward and Associates.
Final presentation by Wilbur Smith and Associates.

LACKAWANNA COUNTY
PENNSYLVANIA

BASE GEOLOGY

II-2

With the exception of Pleistocene glacial and glacio-fluvial deposits and recent alluvium, the study area is underlain entirely by sedimentary rocks ranging in age from early Silurian through Pennsylvanian. In general, these rocks vary extensively in lithology, ranging from highly resistant sandstones and quartzites to greatly weathered shales, with interbedded coal sequences. Although limestones occur in scattered local portions of the seven county area, their distribution on the regional level is considered minor when compared to the dominant sandstone-shale lithologies.

Bedrock Occurrence and Distribution. In general, Silurian and Devonian rocks characterize the southeast border area while Mississippian and Pennsylvanian rocks occur in major parts of Luzerne, Lackawanna, Carbon and Schuylkill counties where they occupy the axial and peripheral portions of the northern, eastern middle, western middle, and southern anthracite basins respectively. A complex series of late Devonian rocks, on the other hand, underlies greater than 50 percent of the area, being particularly prominent in Wayne, Pike and Monroe counties.

Engineering Characteristics of Bedrock. Each of the fifteen regional formations, or stratigraphic units occurring within the seven county area have their own individual characteristics of lithologic (composition), physical (topography, bedding, fracturing, weathering, thickness of overburden), and hydrologic (surface drainage, porosity/permeability, groundwater) factors which makes them a unique mappable entity. These characteristics provide valuable insight with regard to determining a particular rock unit's engineering behavior and, in turn, how this impacts planned development.

The particular characteristics of rocks which are considered to have a major impact in this study include those pertinent engineering conditions such as those related to ease of excavation, cut-slope stability and foundation stability. Table 2.3 summarizes the lithologic and physical characteristics of each of the individual rock formations occurring within the seven county area and their pertinent engineering characteristics.

The information presented in Table 2.3 for each rock unit is necessarily broad in scope and reconnaissance in overview. It should be used primarily to assist in the initial phases of engineering geology investigations and to provide the basis for evaluating the regional land planning and design capabilities of contemplated developmental zones within the seven county area. This information can also provide regional baseline data for selecting specific high priority areas which require more detailed study at the local level. Detailed investigations at the local level would require that on-site sampling, borings, and laboratory analyses be performed prior to formulating design recommendations for specific structures, as well as for finalizing comprehensive land use planning decisions.

The engineering geology data presented in Table 2.3 provide a primary means for evaluating the regional geologic hazards and constraints to devel-

Table 2-3

LITHOLOGIC AND ENGINEERING CHARACTERISTICS OF MAJOR FORMATIONS
ENVIRONMENTAL GEOLOGY SURVEY, ECONOMIC DEVELOPMENT COUNCIL OF NORTHEASTERN PENNSYLVANIA

	FORMATION	MAP UNIT	DESCRIPTION	TOPOGRAPHY	EASE OF EXCAVATION	CUT-SLOPE STABILITY	FOUNDATION STABILITY	CONSTRUCTION MATERIALS
PENNSYLVANIAN	POST-POTTSVILLE	Ppp	INTERSECTED SANDSTONE AND CONGLOMERATE, MEDIUM TO COARSE-GRAINED, WITH COAL AND DARK GRAY TO BLACK SHALES	LOW RIDGES AND VALLEYS IN ROLLING AND ROUGH TERRAIN; NATURAL SLOPES ARE STABLE AT MODERATE ANGLES.	BLASTING REQUIRED, DRILLABILITY VARIES FROM MODERATE TO SLOW DEPENDING ON LITHOLOGY. DEPTH TO ROCK - 0 TO 30 FEET.	FAIR TO GOOD QUALITY IN MODERATE SLOPES. UNDERCUTTING OF SHALE AND SILTSTONE UNDER MORE RESISTANT SANDSTONE AND CONGLOMERATE UNITS CAN BE SEVERE PROBLEM, SUPPORT DRAINAGE MAINTENANCE MAY BE REQUIRED TO REDUCE WEATHERING FROM EXCESSIVE MOISTURE.	GOOD QUALITY FOUNDATION FOR HEAVY STRUCTURES, SHOULD BE EXCAVATED TO SOUND MATERIAL, EXTREME CAUTION SHOULD BE EXERCISED WHERE COALS HAVE BEEN REMOVED UNDERGROUND.	FAIR TO GOOD RESERVES OF ROAD MATERIAL AND FILL. MORE RESISTANT UNITS SUITABLE FOR BUILDING STONE, FLAGSTONE, EMBANKMENT FACING, AND RIPRAP
PENNSYLVANIAN	POTTSVILLE GROUP	Pp	FINE-GRAINED TO COARSELY CONGLOMERATIC SANDSTONE, SUBORDINATE AMOUNTS OF SHALES, SILTSTONES, LIMESTONE, COAL AND UNDERCLAY OCCUR. JOINTING AND FAULTING IMPORTANT. SUBSIDENCE FRACTURES MAY BE ENCOUNTERED IN CONNECTION WITH UNDERGROUND CLAY AND COAL MINING.	FORMS CRESTS, FLANKS OF RIDGES AND OTHER BREAKS IN TOPOGRAPHY; DISTINCTIVE RIDGE FORMER ANTHRACITE AREA; STEEP NATURAL SLOPES ARE STABLE.	BLASTING REQUIRED FOR SANDSTONES; OTHER ROCK TYPES MODERATELY EASY TO REMOVE USING HEAVY POWER EQUIPMENT; MODERATE TO SLOW DRILLING RATE. DEPTH TO ROCK - 0 TO 5 FEET.	MODERATELY CUT SLOPES REQUIRED; UNDERCUTTING IN SHALES AND UNDERCLAYS UNDER THICK SANDSTONE UNITS CAUSES ROCK FALLS AND SLUMPING. DRAINAGE MAINTENANCE MAY BE NEEDED.	GOOD TO EXCELLENT FOR HEAVY STRUCTURES EXCEPT ON/OR ABOVE UNDERCLAYS WHICH WILL DEFORM UNDER LOAD WHEN WET. UNDERGROUND COAL AND CLAY MINES WITH POSSIBLE ROOF SUBSIDENCE WILL NEED SPECIAL STUDY.	GOOD SOURCE OF ROAD MATERIAL AND FILL, BUILDING STONE, RIPRAP, EMBANKMENT FACING AND REFRACTORY CLAYS.
MISSISSIPPIAN	MAUCH CHUNK	Mmc	DOMINANT ROCK TYPES ARE SHALE, CLAYSTONE, SANDSTONE AND SILTSTONE. SANDSTONE IS FINE TO MEDIUM GRAINED AND CROSS-BEDDED. JOINTS ARE ABUNDANT.	ROLLING VALLEYS OF MEDIUM RELIEF; STABLE WITH FAIRLY STEEP SLOPES.	MODERATELY EASY TO MODERATELY DIFFICULT WITH HEAVY POWER EQUIPMENT, DEPENDS ON ROCK TYPES AND DEGREE OF FRACTURING. DEPTH TO ROCK UP TO 75 FEET	MAINTAINS A GOOD QUALITY SLOPE AT FAIRLY STEEP ANGLES IF SEVERE UNDERCUTTING OF RESISTANT BEDS DOES NOT OCCUR.	GOOD QUALITY FOR HEAVY STRUCTURES; SHOULD BE EXCAVATED TO SOUND BEDROCK.	GOOD FOR ROAD MATERIAL AND FILL, SHALE IS GOOD RAW MATERIAL FOR BRICK
MISSISSIPPIAN	POCONO GROUP	Mp	MAINLY FINE TO COARSE-GRAINED SANDSTONE; CONGLOMERATIC BEDS; OCCASIONAL THIN BEDS OF SHALES AND SILTSTONES; A FEW COAL BEDS. JOINTS ARE WELL DEVELOPED.	HIGH, ROUGH RIDGES: CRESTS OF MOUNTAINS, AND HOG BACKS: NATURAL SLOPES ARE STEEP AND STABLE.	BLASTING REQUIRED. DEPTH TO ROCK - 0 TO 5 FEET.	GOOD QUALITY SLOPES; CAN USUALLY BE CUT AT VERY STEEP ANGLES. ROCK FALLS OCCUR WHEN BEDDING AND JOINT SURFACES INTERSECT AND ROCK IS STEEPLY DIPPING TOWARD ROAD; CONDITION WOULD BE AGGRAVATED BY VERY STEEP VERTICAL CUT.	EXCELLENT QUALITY FOR HEAVY STRUCTURES.	GOOD SOURCE OF BUILDING STONE, EMBANKMENT FACING, ROAD MATERIAL AND RIPRAP.
DEVONIAN	SUSQUEHANNA GROUP	Ds	COMPLEX UNIT OF SHALES, SILTSTONES, SANDSTONES AND CONGLOMERATES. FROM THE BASE OF THE UNIT TO TOP: - GRAIN SIZE OF SEDIMENTS INCREASES; AVERAGE THICKNESS OF BEDS INCREASES; PERCENT SILICA IN ROCKS INCREASES.	PLATEAU OF MEDIUM RELIEF; NATURAL SLOPES ARE STABLE AT FAIRLY STEEP ANGLES.	BLASTING REQUIRED; RELATIVE DRILLABILITY MODERATE. DEPTH TO ROCK - VARIABLE, BUT GENERALLY GREATER THAN 30 FEET IN GLACIATED AREAS.	GOOD QUALITY, STEEP CUTS CAN BE MAINTAINED IN SANDSTONES AND CONGLOMERATES; SILTSTONE AND SHALES DISINTEGRATE RAPIDLY; UNDERCUTTING OF RESISTANT, COARSER BEDS CAN CAUSE POOR QUALITY SLOPES AND RESULT IN ROCK FALLS; MODERATE SLOPE CUTS ARE NECESSARY; DRAINAGE MAINTENANCE REQUIRED.	GOOD QUALITY FOR HEAVY STRUCTURES; SHALES AND SILTSTONES SHOULD BE KEPT WATER FREE; MUST BE EXCAVATED TO SOUND BEDROCK.	GOOD SUPPLY OF FLAGSTONE AND BRICK, RAW MATERIALS, AGGREGATE ROOF GRANULES, AND FILL.
DEVONIAN	CATSKILL	Dck	COMPLEX UNIT CONSISTING OF SHALES, SILTSTONES, SANDSTONES AND CONGLOMERATES; THICKNESS OF EXPOSED ROCKS DECREASES TO WEST.	SAME AS Ds ABOVE.	SAME AS Ds ABOVE.	SAME AS Ds ABOVE.	SAME AS Ds ABOVE.	SAME AS Ds ABOVE.
DEVONIAN	MARINE BEDS, INCL. CHEMUNG	Dm	FINE-GRAINED SANDSTONE AND SILTSTONE WITH SHALE INTERBEDS. JOINTS ARE WELL DEVELOPED.	ROLLING RIDGES OF MEDIUM RELIEF: LATERAL SLOPES ARE STEEP AND STABLE.	MODERATELY DIFFICULT TO MODERATELY EASY WITH HEAVY POWER EQUIPMENT, MODERATE DRILLING RATE EXPECTED. DEPTH TO ROCK - 5 TO >30 FEET.	FAIR CUT-SLOPE STABILITY; MODERATE SLOPE CUTS NECESSARY; SANDSTONES STABLE IN FAIRLY STEEP CUTS.	GOOD QUALITY FOUNDATION FOR HEAVY STRUCTURES; SHOULD BE EXCAVATED TO SOUND MATERIAL.	GOOD FOR ROAD MATERIAL AND FILL.
DEVONIAN	HAMILTON AND ONONDAGA UNDIFFERENTIATED	Dho	SEE Dh AND Don BELOW.	SEE Dh AND Don BELOW.	SEE Dh AND Don BELOW.	SEE Dh & Don BELOW.	SEE Dh AND Don BELOW.	SEE Dh AND Don BELOW.
DEVONIAN	HAMILTON GROUP	Dh	FINE-GRAINED SANDSTONE WITH NUMEROUS SHALE INTERBEDS. JOINTS ARE CLOSELY SPACED.	ROLLING HILLS OF MEDIUM HEIGHT WITH STABLE SLOPES; STEEPNESS VARIES FROM GENTLE TO STEEP.	MODERATELY EASY TO MODERATELY DIFFICULT TO EXCAVATE WITH HEAVY POWER EQUIPMENT; LOCALLY, BLASTING REQUIRED; FAST TO MODERATE DRILLING RATE EXPECTED. DEPTH TO ROCK - 0 TO 30 FEET.	FAIR CUT-SLOPE STABILITY; MODERATE ANGLES OF CUT SHOULD BE MADE AND BENCHES PROVIDED IN DEEPER CUTS.	GOOD FOR HEAVY STRUCTURES; NEED FOR EXCAVATION TO SOUND MATERIAL; NEED FOR UNDER DRAINAGE.	GOOD FOR ROAD MATERIAL, FILL AND POTENTIAL LIGHT WEIGHT AGGREGATE.
DEVONIAN	ONONDAGA	Don	PREDOMINANTLY SHALE WITH LIMESTONE INTERBEDS.	ROLLING HILLS OF MODERATE RELIEF; NATURAL SLOPES ARE FAIRLY STEEP AND STABLE.	MINOR BLASTING REQUIRED; MODERATE DRILLING RATE. DEPTH TO ROCK - 0 TO 5 FEET.	GOOD CUT-SLOPE STABILITY; STABLE IN FAIRLY STEEP CUTS; SHALE BEDS MAY REQUIRE CUT BENCHES.	GOOD QUALITY FOUNDATION FOR HEAVY STRUCTURES; SHOULD BE EXCAVATED TO SOUND MATERIAL. LIMESTONE SHOULD BE INVESTIGATED FOR SINKHOLES.	GOOD FOR ROAD MATERIAL AND FILL.
DEVONIAN	ORISKANY AND HELDERBERG UNDIFFERENTIATED	Doh	ORISKANY IS FINE TO COARSE-GRAINED SANDSTONE; HELDERBERG IS LIMESTONE WITH SHALE AND LOCAL INTERBEDS OF SANDSTONE. ORISKANY-JOINTS HIGHLY ABUNDANT IN INTENSELY FOLDED AND FAULTED AREAS.	ORISKANY-HOG BACKS AND MODERATELY HIGH RIDGES; HELDERBERG-ROLLING HILLS OF MEDIUM RELIEF.	ORISKANY-VARIABLE; TIGHTLY CEMENTED ROCKS REQUIRE BLASTING; FRIABLE ROCKS EASILY EXCAVATED WITH HEAVY POWER EQUIPMENT. DEPTH TO ROCK - 0 TO 5 FEET. HELDERBERG-BLASTING REQUIRED; BEDROCK PINNACLES A SPECIAL PROBLEM. DEPTH TO ROCK - 0 TO 5 FEET.	ORISKANY-STABLE IN FAIRLY STEEP TO STEEP CUTS FOR RESISTANT ROCK; WEAKER ROCK UNITS WITH STEEP DIP INTO ROADWAY HAVE POOR STABILITY IN GENTLE CUTS. HELDERBERG-GOOD CUT-SLOPE STABILITY IN FAIRLY STEEP CUTS.	ORISKANY-WELL CEMENTED, RESISTANT ROCKS GOOD QUALITY FOUNDATION FOR HEAVY STRUCTURES; SITE SHOULD BE THOROUGHLY INVESTIGATED BEFORE CONSTRUCTION. HELDERBERG-GOOD QUALITY FOUNDATION FOR HEAVY STRUCTURES, INVESTIGATE FOR SINKHOLES; SHOULD BE EXCAVATED TO SOUND BEDROCK.	ORISKANY-REFRACTORY SAND, GLASS-SAND, FILL AND ROAD MATERIAL. HELDERBERG-GOOD FOR ROAD MATERIAL AND FILL.
SILURIAN	KEYSER AND TONOLOWAY UNDIFFERENTIATED	Skt	THIN TO THICK BEDDED LIMESTONE. JOINTS ARE MODERATELY TO HIGHLY ABUNDANT.	ROLLING HILLS OF MEDIUM RELIEF; NATURAL SLOPES ARE MODERATE AND STABLE.	REQUIRES BLASTING; BEDROCK PINNACLES A SPECIAL PROBLEM; MODERATE DRILLING RATE. DEPTH TO ROCK - 0 TO 5 FEET.	GOOD CUT-SLOPE STABILITY; STABLE IN FAIRLY STEEP CUTS, LOCAL INTENSE FRACTURING AND DEPTH OF CUT MAY REQUIRE MODERATE SLOPE CUTS.	GOOD QUALITY FOUNDATION FOR HEAVY STRUCTURES; THOROUGH INVESTIGATION FOR CONCEALED SINKHOLES SHOULD BE UNDERTAKEN; SHOULD BE EXCAVATED TO SOUND MATERIAL.	GOOD FOR ROAD MATERIALS, RIPRAP, EMBANKMENT FACING AND FILL.
SILURIAN	BLOOMSBURG AND McKENZIE UNDIFFERENTIATED	Sbm	BLOOMSBURG IS SHALE AND SILTSTONE WITH LOCAL UNITS OF SANDSTONE AND LIMESTONE; MC KENZIE IS SHALE INTERBEDDED WITH LIMESTONE.	BLOOMSBURG-ROLLING HILLS OF MEDIUM RELIEF; NATURAL SLOPES FAIRLY STEEP AND STABLE. MC KENZIE-ROLLING TO ROUGH HILLS OF MEDIUM TO HIGH RELIEF: NATURAL SLOPES FAIRLY STEEP AND STABLE.	BLOOMSBURG- MODERATELY EASY WITH HEAVY POWER EQUIPMENT; RELATIVELY FAST DRILLING RATES; DEPTH TO ROCK - 0 TO 5 FEET. MC KENZIE-WEATHERED ROCK MODERATELY DIFFICULT TO MODERATELY EASY WITH HEAVY POWER EQUIPMENT; BLASTING REQUIRED OF UNWEATHERED PART, FAST DRILLING RATES MAY BE EXPECTED. DEPTH TO ROCK - 0 TO 5 FEET.	BLOOMSBURG-POOR TO FAIR CUT-SLOPE STABILITY DUE TO RAPID DISINTEGRATION WHEN EXPOSED TO MOISTURE. GENTLE SLOPE CUTS NECESSARY. MC KENZIE-FAIR CUT-SLOPE STABILITY; MODERATE CUT-SLOPES NECESSARY; SHALE PORTION SUSCEPTIBLE TO MOISTURE WITH SHORT-TERM STABILITY.	GOOD QUALITY FOUNDATION FOR HEAVY STRUCTURES; SHOULD BE EXCAVATED TO SOUND MATERIAL; ON MC KENZIE - INVESTIGATE FOR COLLAPSE POSSIBILITIES WHEN LIMESTONE IS ENCOUNTERED.	GOOD FOR ROAD MATERIAL AND FILL, POSSIBLE SOURCE OF BRICK.
SILURIAN	CLINTON GROUP	Sc	THIN BEDDED SHALE WITH INTERTONGUING "IRON SANDSTONES" AND LOCAL LIMESTONE. JOINTS ARE WELL DEVELOPED AND HIGHLY ABUNDANT.	ROLLING HILLS AND RIDGES OF MEDIUM RELIEF. NATURAL SLOPES ARE MODERATE TO FAIRLY STEEP AND STABLE.	MODERATELY DIFFICULT WITH HEAVY POWER EQUIPMENT, MINOR BLASTING MAY BE REQUIRED; MODERATE TO SLOW DRILLING RATES. DEPTH TO ROCK - 0 TO 5 FEET.	GOOD CUT-SLOPE STABILITY, STABLE IN FAIRLY STEEP CUTS.	GOOD QUALITY FOUNDATION FOR HEAVY STRUCTURES; SHOULD BE EXCAVATED TO SOUND MATERIAL	GOOD FOR ROAD MATERIAL AND FILL.
SILURIAN	SHAWANGUNK/ TUSCARORA	Ss/St	FINE TO COARSE-GRAINED SANDSTONE AND QUARTZITE; CONGLOMERATIC IN PART. WELL DEVELOPED JOINTS MODERATELY TO HIGHLY ABUNDANT	HIGH MOUNTAINS AND RIDGES; VERY HIGH RELIEF IN ROUGH TERRAIN, NATURAL SLOPES ARE STABLE AND STEEP.	BLASTING REQUIRED; BOULDER FIELDS ON LOWER SLOPES BENEATH OUTCROP AREAS A SPECIAL PROBLEM; DRILLING RATES VERY SLOW. DEPTH TO ROCK - 0 TO 3 FEET.	GOOD IN FAIRLY STEEP CUTS, CAN STAND IN VERTICAL CUTS IF BEDDING IS NOT STEEPLY DIPPING TOWARD ROAD	EXCELLENT QUALITY FOR HEAVY STRUCTURES; EXCAVATE TO SOUND BEDROCK.	GOOD TO EXCELLENT SOURCE OF ROAD MATERIAL, RIPRAP, CONCRETE AGGREGATE, EMBANKMENT FACING, BUILDING STONE AND SILICA REFRACTORY BRICK.

(ADAPTED FROM MC GLADE, ETAL, 1972)

opment. The review of engineering characteristics of potentially troublesome rock formations, provided the basis for initially defining and identifying those potential areas that are most prone to have problems associated with their development.

The seven county area contains textbook examples of Appalachian type fold structures. In general, the area is characterized by three major structural environments: the folded belt; the plateau; and the intermediate or border zone between the folded belt and the more gently deformed plateau. Figure 2.1 is a generalized map showing the major structural zones occurring with the area.

Glacial Deposits. It was not the purpose of this investigation to enter into any prolonged dissertation on the glacial history of Northeastern Pennsylvania. Nevertheless, it must be mentioned that glacial deposits comprise one of the primary modes of soil accumulation. As shown in Figure 2.2, major portions of the study area have been subjected to two, and possibly three stages of Pleistocene glaciation. These are from youngest to oldest: Wisconsin, Illinoian, and possibly a pre-Illinoian stage. Since the nature of evidence for a pre-Illinoian glacial stage is scant, this study concentrates primarily on the major unconsolidated surficial deposits resulting from the Wisconsin and Illinoian glacial stages.

Economic Resources. The major economic resource of the seven county area is anthracite coal. This study was primarily concerned with identifying those potential mineral resources other than coal. However, the area contains extensive coal deposits that occur within four major anthracite-producing basins. The location of each of these major anthracite basins with respect to the seven county area is shown in Figure 2.3. They are:

- Northern Anthracite Field (Wyoming-Lackawanna Basin) centered around Scranton and Wilkes-Barre
- Eastern Middle Field around Hazelton
- Western Middle Field around Mahanoy
- Southern Anthracite Field centered around Pottsville

Of these four fields, the southern field appears to have the best potential for future reserves of mineable anthracite.

Coal is mined primarily from key beds within strata of the Post-Pottsville group. The real occurrence of this major stratigraphic unit is delineated on the accompanying regional base geology maps. The pattern, as outlined by this broad map unit, clearly defines the extent and distribution of each of the major coal producing basins within the area. Within each area, coal is extracted by either strip and/or underground mining techniques. The location of these mine and related disturbed areas are identified on the accompanying Regional Slope Maps by the designation "variable slope."

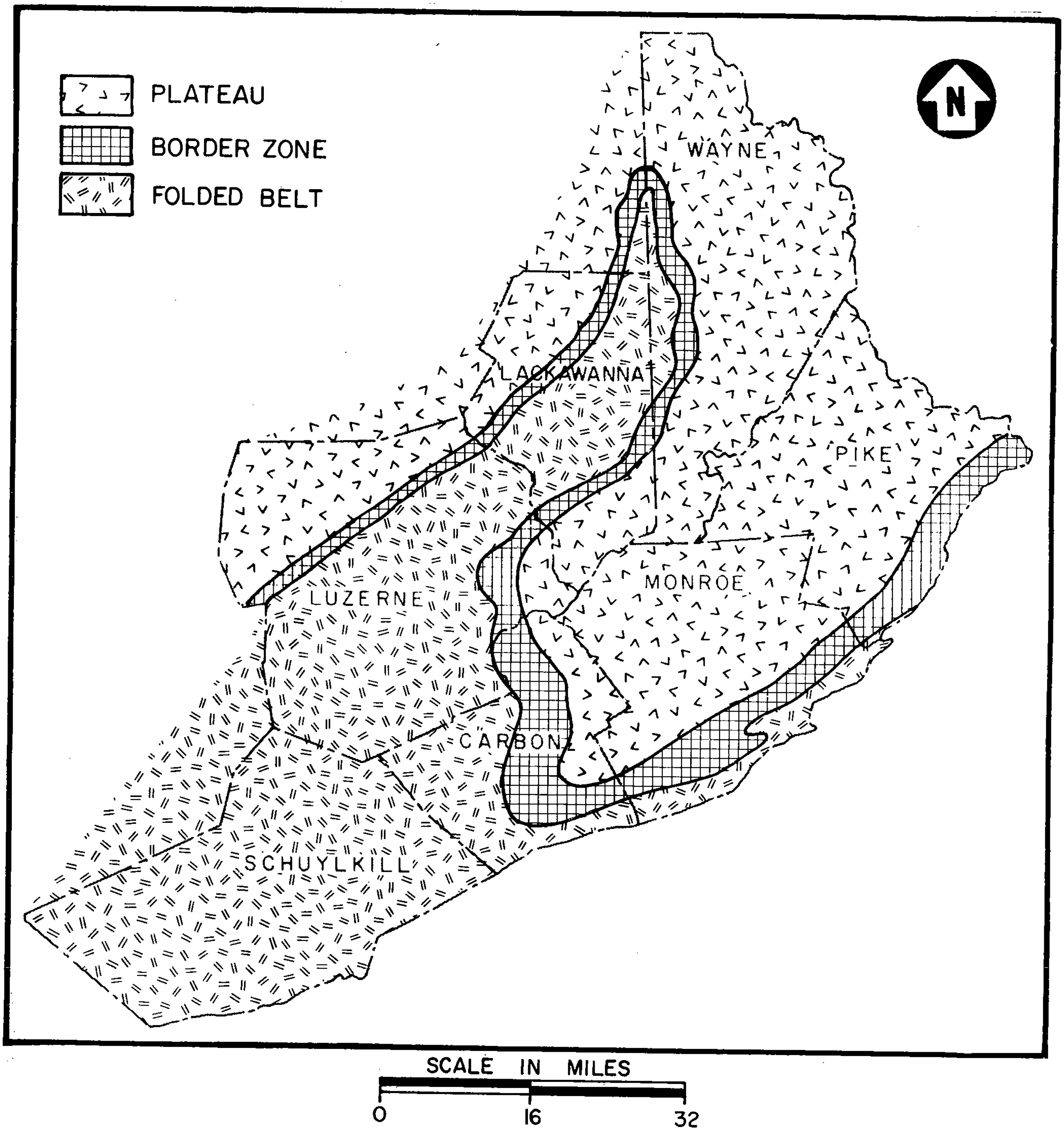


FIGURE 2.1 MAJOR TECTONIC ZONES OF STUDY AREA

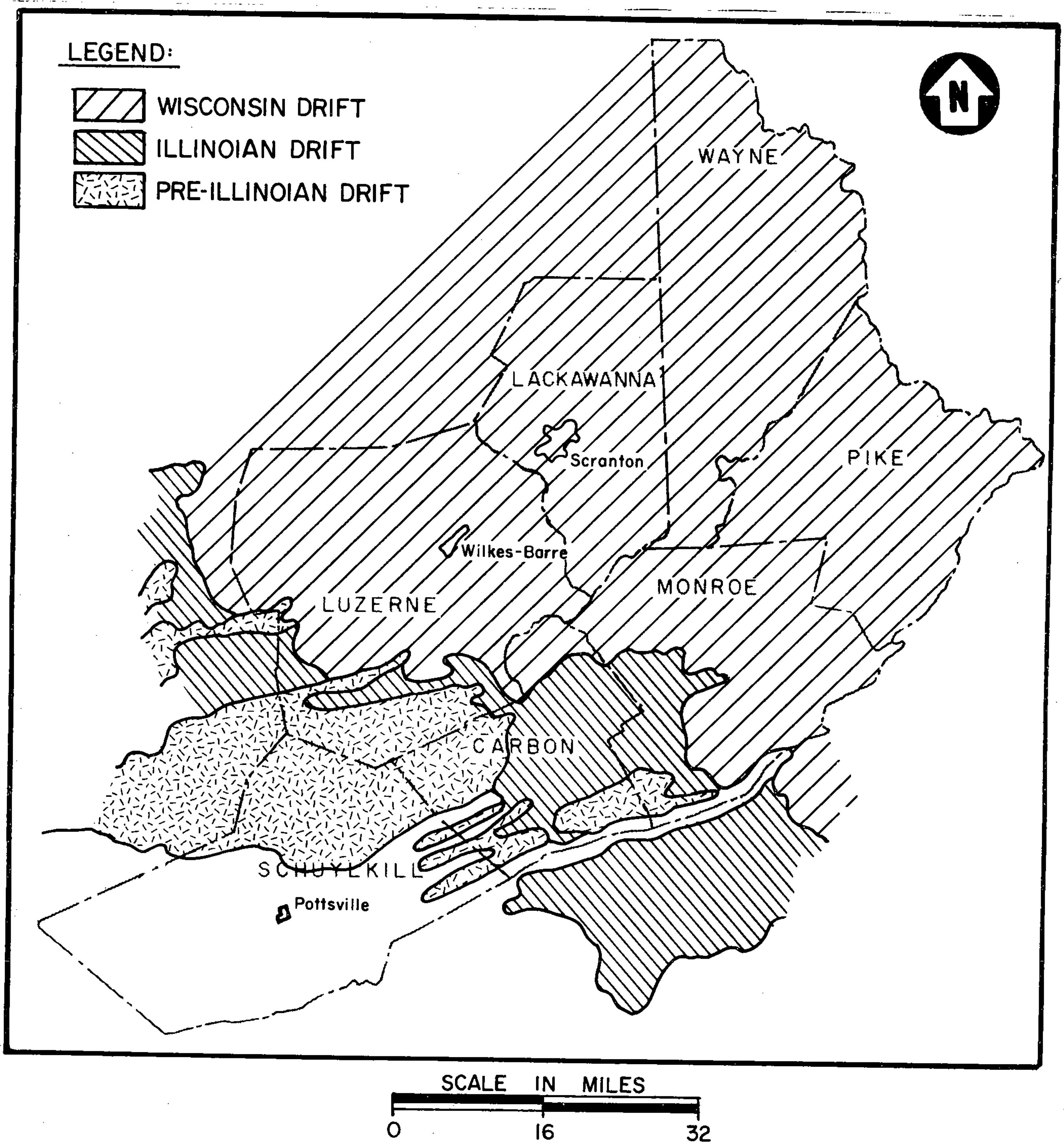
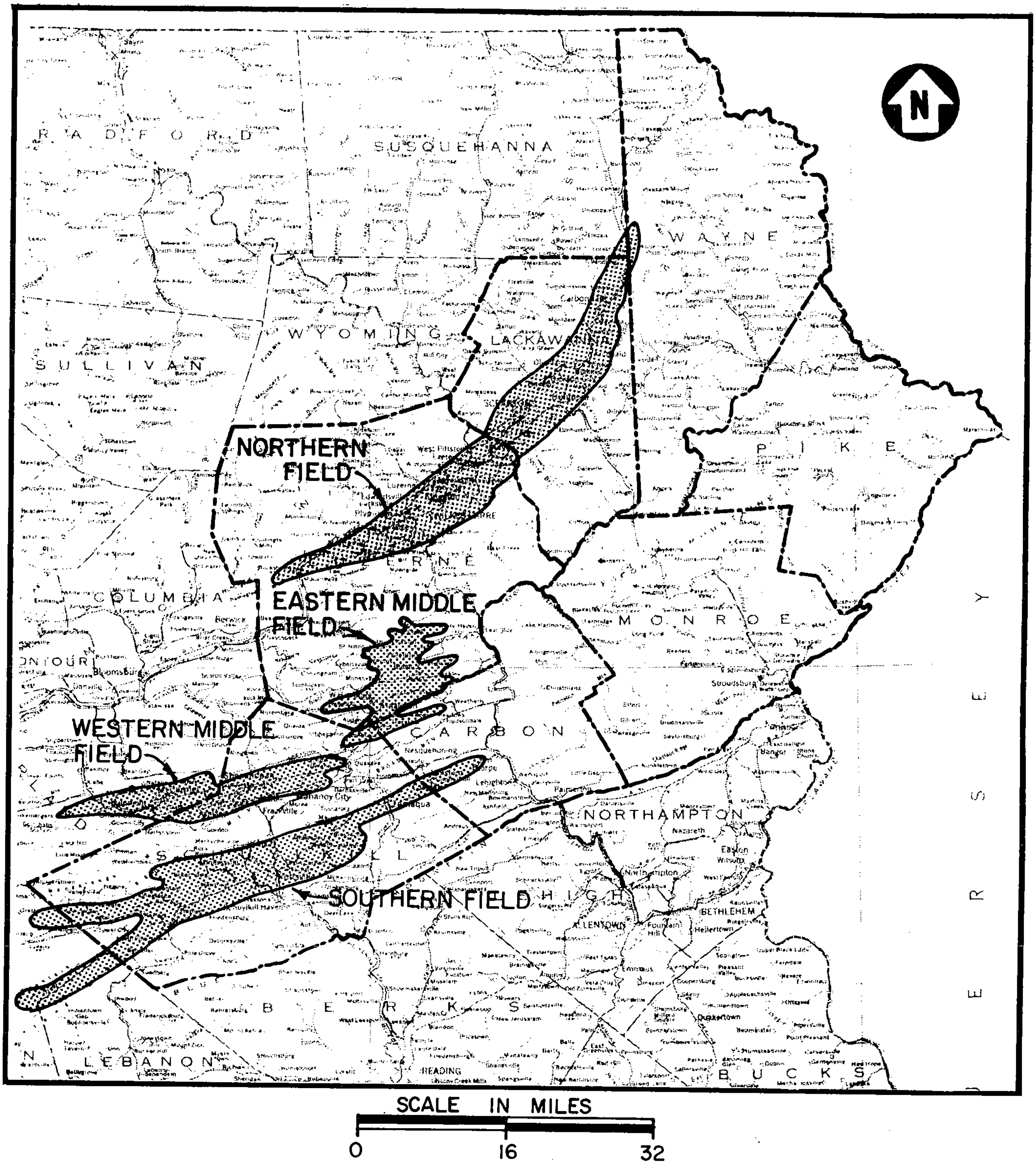


FIGURE 2.2 GENERALIZED GLACIAL MAP OF NORTHEASTERN PENNSYLVANIA



MAP SOURCE: BASE MAP WAS OBTAINED FROM U.S. DEPT. OF INTERIOR GEOLOGICAL SURVEY MAP, STATE OF PENNSYLVANIA, 1955 EDITION.

FIGURE 2.3 MAJOR ANTHRACITE BASINS OF STUDY AREA

The major economic resources of the seven county area, other than coal, appear to be building stones and aggregates; sand and gravel; metals, gas; and peat.

Table 2.4 summarizes the mineral resources potential of the study area by county. It must be observed that the majority of the mineral resources listed, with possible exception of sandstone, do not occur as extensively as coal does, but rather, tend to occur in scattered local portions of the seven county area. This tabulation also identifies the status and/or potential of a specific mineral resource (1) has been, or is being, extracted; (2) is a potential prospect; or (3) occurs in small insignificant concentrations at local sites. Appropriate symbols of each numeral reserve was employed for identification on the geology mapping.

Soil Associations and Groupings

The mapping shows the general pattern of soil association groupings occurring in the seven county area. Pertinent constraints and/or limitations that characterize the major soil groupings are also identified by appropriate alpha-numeric code on these maps.

Two methods of approach were considered in mapping and evaluating the soils of the seven county area. One method is to identify major soils associations strictly on the basis of their prominent modes of geologic occurrence (such as glacial, alluvial, aeolian, etc.) and landform pattern. This method enables one to qualitatively classify major un-consolidated surficial units on the basis of key textural characteristics such as granular, arenaceous, etc., or other classification systems such as the Unified.

A second method was selected. This method involves the use of information contained in soil survey maps and reports prepared by the U.S. Soil Conservation Service. The detailed soil maps often show areas less than 10 acres in size and are not suited for use in small maps of large areas. SCS soil scientists also prepare "general soils maps" of each county. These maps are based on the concept that each soil is normally found in the same general area in which certain other soils are found. Thus, it is possible to show areas where these associated soils (or soil associations) will usually be found. The percentage of each of the major soils in the association is estimated. SCS reports provide information with regard to the major soils within each association. Some of the more recent general soils maps prepared by SCS were available only in draft form. The general soil maps of the seven counties included 66 soil associations. In order to produce a map which could be more readily understood, these were rearranged into 15 soil associations.

The individual soil associations for the regional survey was developed primarily on the basis of prominent characteristics such as geographic position, slope, natural drainage, parent material, and to some extent, textural

Table 2-4

MINERAL RESOURCES OF THE SEVEN COUNTY AREA

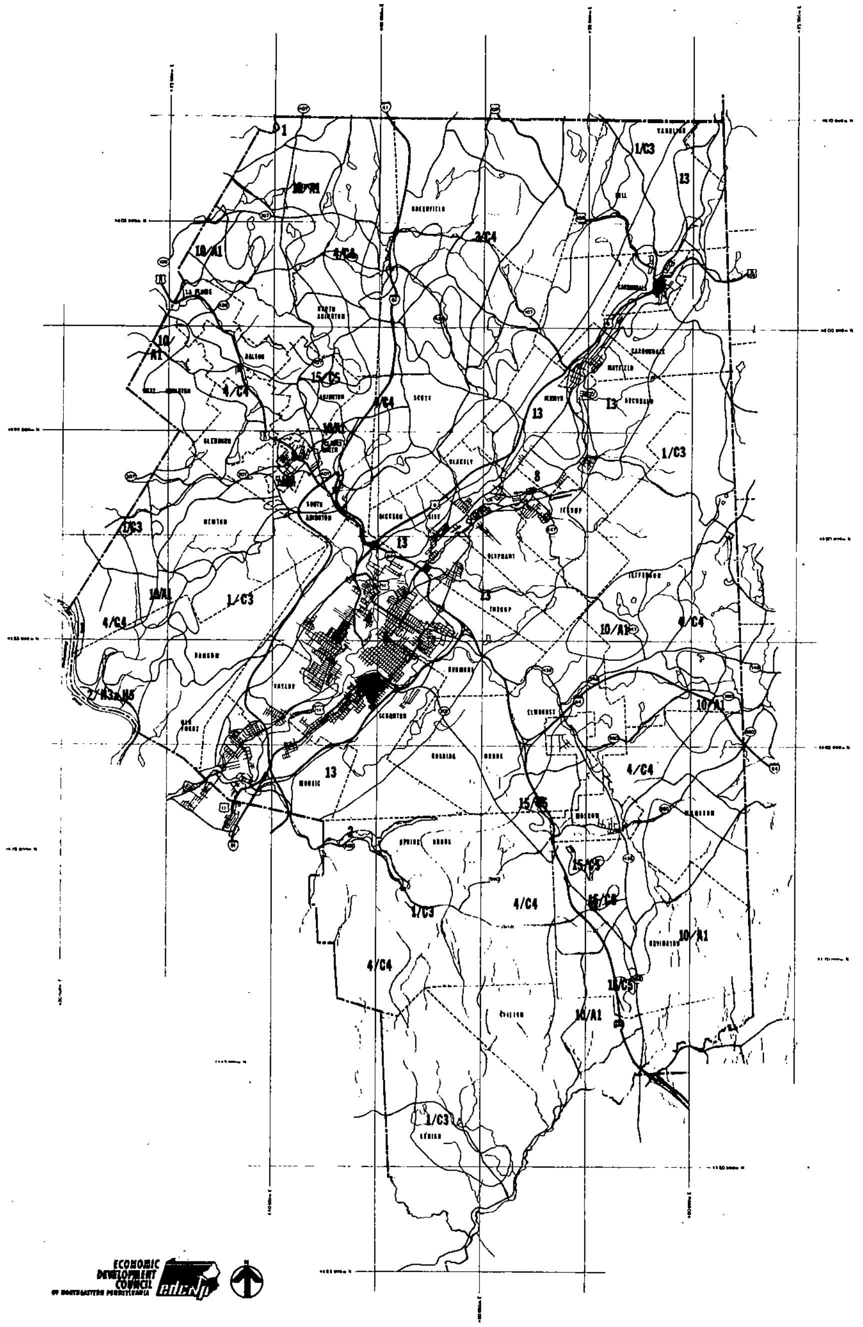
	BUILDING STONES					METALS						
	SANDSTONE	SHALE	LIMESTONE-DOLOMITE	QUARTZITE	SLATE	COPPER	LEAD	ZINC	MANGANESE	NICKEL	URANIUM	GAS
Carbon	C			C	C	P,M	M	M			C,P,M	
Lackawanna	C					P					M	C
Luzerne	C	C		C				M		M	M	
Monroe	C		C			P		P	M			
Pike	C	C				M	M	M				
Schuylkill		C	C				M				M	
Wayne	C	C				P					P	M

C = Current and/or Past Production

P = Prospect (primarily metallic minerals)

M = Occurrence (primarily metallic minerals and gas)

Blank denotes county has poor or non-existant potential for specified mineral resources.



The preparation of this map was financed in part by The Appalachian Regional Commission under section 302 of The Appalachian Regional Development Act of 1965, as amended. App. U.S.C. 302, in part by The Department of Housing and Urban Development, under provisions of section 701 of the Housing Act of 1954, as amended and as administered by The Pennsylvania Office of State Planning and Development and in part by the Economic Development Administration.

Base map preparation by Economic Development Council of Northeastern Pennsylvania.
Technical data compiled by Joseph B. Ward and Associates.
Final presentation by Wilbur Smith and Associates.

LACKAWANNA COUNTY
PENNSYLVANIA

SOILS

III–2

class. The task of reviewing the numerous soil associations that occur within the seven county area, and their subsequent classification and synthesis into 14 compatible map units was the responsibility of EDC/NP technical personnel, with valuable support from Soil Conservation Service and Penn State University soil scientists. These personnel greatly assisted in both the tabulation of soil association groupings and pertinent physical characteristics, as well as in the compilation of the regional soils maps.

It should be noted that the regional soils maps accompanying this report are not intended to show the specific type of soil occurring at any one particular locality. They do show, however, a general pattern that could include several soil associations that are characteristic of large portions of a given area. On this basis, it is possible to compare the soil associations in different parts of the seven county area, and from their representative physical characteristics, obtain a preliminary indication with regard to suitability and/or limitations to regional development.

The 14 soil associations, or map units, used in the regional survey are listed in Table 2.5. The map symbols shown on this table for each of the major groupings correspond directly to those units identified on the accompanying maps. This table is all-inclusive and summarizes the individual soil associations occurring within each major grouping and the percentage of a particular soil in that association. Also shown are pertinent soil association characteristics such as depth to bedrock; erodibility; depth to seasonal water table; subsoil permeability and internal drainage; hydrologic grouping; unified soil classification; and DER (Pennsylvania Department of Environmental Resources) soil group rating.

Two of the 14 regional soil groups that have been mapped relate to patterns of existing land use and not to prominent soil associations. These include the Group 8 map category which comprises primarily urban and associated built-up land areas, and Group 13 extractive land encompassing both surface and subsurface anthracite mining operations.

As shown in Table 2.6, the primary DER criteria for evaluating soil capabilities and limitations for sanitary disposal systems is based chiefly on probable percolation rates, thickness of soil profile, probable flooding and the existence of high-water table conditions. Note that DER Group 15 soils have the poorest rating of any of the soils occurring in the area. For this reason, the Volusia-Morris-Norwich association has been designated Map Group 15 on the prior Table 2.5, and not Map Group 14, which would normally be the case in a sequential numbering system.

Hydrology

The regional hydrologic attributes of the seven county area consider major surface drainage, including lakes and mappable bodies of standing

Table 2-5

REGIONAL SOIL ASSOCIATIONS AND THEIR PERTINENT CHARACTERISTICS

MAP GROUP	SOIL ASSOCIATIONS	% OF ASSOC.	DEPTH TO BEDROCK	DEPTH TO SEASONAL WATER	SUBSOIL PERMEABILITY	INTERNAL DRAINAGE	ERODIBILITY	HYDROLOGIC GROUPING	UNIFIED SOIL CLASSIFICATION "C" HORIZON	D E R SOIL GROUP
1	Arnot	40	1-1.5 Feet	—	Moderate	Well Drained	Low	C/D	GM	12
	Lordstown	2[illegible]	1.5-3.5	—	Moderate	Well Drained	Low	C	GM	5
	Oquaga	25	1.5-3.5	—	Moderately Rapid	Well Drained	Low	C	SM,GM,ML	5
2	Barbour	30	6+	—	Moderately Rapid	Well Drained	High	B	ML,SM,GS	13
	Basher	30	6+	1.5-3 Feet	Moderate to Rapid	Mod.Well-Somewhat poorly	High	B	ML,SM,CM	13
	Chenango	20	6+	—	Rapid	Well Drained	Low	A	GW,GP,GM,SP	1
	Wyoming	10	6+	—	Moderately Rapid	Well Drained	Low	B	GW,GP,GP-GM,SP	4
3	Wellsboro	35	5+	1.5-3	Slow	Mod. Well Drained	Low	C	GM,SM,ML,CL	14
	Morris	30	5+	0.5-1.5	Slow	Somewhat Poorly Drained	Medium	C	GM,SM,ML,CL	15
	Lackawanna	10	5+	—	Slow	Well Drained	Low	C	GM,GC	7
4	Oquaga	35	1.5-3.5	—	Moderately Rapid	Well Drained	Low	C	SM,GM,ML	5
	Wellsboro	25	5+	1.5-3	Slow	Moderately Well Drained	Low	C	GM,SM,ML,CL	14
	Morris	15	5+	0.5-1.5	Slow	Somewhat Poorly Drained	Medium	C	GM,SM,ML,CL	15
	Lordstown	15	1.5-3.5	—	Moderately Rapid	Well Drained	Low	C	SM,GM,ML	5
5	DeKalb	55	1.5-3.5	—	Moderately Rapid	Well Drained	Low	C	ML,SM,GM	5
	Buchanan	20	4+	1.5-3	Slow	Moderately Well Drained	Medium	C	ML,CL,SM,GM	14
	Hazleton	10	3.5-7	—	Moderate	Well Drained	Medium	B	SM,GM,ML	4
	Edgemont	10	3.5-7	—	Moderately Rapid	Well Drained	Medium	B	SM,GM	6(Some 14)
6	Hazelton	50	3.5-7	—	Moderate	Well Drained	Medium	B	SM,GM,ML	4
	Edgemont	30	3.5-7	—	Moderately Rapid	Well Drained	Medium	B	SM,GM	6(Some 14)
	Buchanan	15	4+	1.5-3	Slow	Moderately Well Drained	Medium	C	ML,CL,SM,GM	14
7	Hazleton	40	3.5-7	—	Moderate	Well Drained	Medium	B	SM,GM,ML	4
	Allenwood	30	6+	—	Moderate	Well Drained	Medium	B	SM,GC,GM	6
	Watson	25	5+	1.5-3	Slow	Moderately Well Drained	Medium	C	GC,ML,CL	14
8	Urban Land	95								
9	Leck Kill	50	3.5-6	—	Moderately Rapid	Well Drained	Low	B	GM,GC	2
	Klinesville	30	1-1.5	—	Moderately Rapid	Well Drained	Low	C/D	GM,GP	12
	Calvin	15	2-3.5	—	Moderately Rapid	Well Drained	Medium	C	GM,GC	7
10	Mardin	40	5+	1.5-3	Slow	Moderately Well Drained	Low	C	ML,CL,GM,GC,SM	14
	Swartswood	25	5+	—	Moderate	Well Drained	Low	C	SM,GM	6
	Lordstown	25	1.5-3.5	—	Moderately Rapid	Well Drained	Low	C	GM	5
11	Meckesville	65	5+	—	Moderately Slow	Well Drained	Medium	C	ML,CL,GM,SM	10
	Albrights	20	6+	1-3	Moderately Slow	Mod.Well-Somewhat Poorly	High	C	GM,SM,SC,ML,CL	14
12	Weikert	40	1-1.5	—	Moderately Rapid	Well Drained	Medium	C/D	GM	12
	Hartleton	35	3.5+	—	Moderate	Well Drained	Medium	B	SM,GC	2
13	Strip Mines and Mine Related Lands	80								
15	Volusia	30	6+	0.5-1.5	Slow	Somewhat Poorly Drained	Medium	C	GM,GC,ML,CL	15
	Morris	30	5+	0.5-1.5	Slow	Somewhat Poorly Drained	Medium	C	GM,SM,ML,CL	15
	Norwich	10	5+	0	Slow	Poorly Drained	Medium	D	ML,CL,SM,SC	15

Table 2-6

DEPARTMENT OF ENVIRONMENTAL RESOURCES (DER) SOIL GROUPS

DER SOIL GROUP	SOIL CONDITION
1	Soils with very rapid percolation with hazard from insufficient filtration and renovation of effluent.
2	Deep, well drained soils with probable percolation rates of 1 inch of water in 6-15 minutes.
4	Deep, well drained soils with probable percolation rates of 1 inch of water in 15-30 minutes.
5	Moderately deep, well drained soils with probable percolation rates of 1 inch of water in 15-30 minutes.
6	Deep, well drained soils with probable percolation rates of 1 inch of water in 30-45 minutes.
7	Moderately deep, well drained soils with probable percolation rates of 1 inch of water in 30-45 minutes.
8	Deep, well drained soils with probable percolation rates of 1 inch of water in 45 to 60 minutes.
10	Well drained soils with probable percolation rates slower than 1 inch of water in 60 minutes.
12	Well drained soils that are shallow or very shallow to bedrock.
13	Soil series that occur on floodplains and have a high flooding hazard. Not suitable for subsurface disposal systems.
14	Moderately well drained soils on upland sites. These soils have seasonal high water tables which are the major limitation on use for subsurface disposal systems.
15	Somewhat poorly, poorly, and very poorly drained soils on upland sites. These soils have high water tables and are unsuitable for subsurface disposal systems.

NOTE: Rates are quite variable in short distances due to variations of material immediately under the soil.

water; geologic flood plains and/or poorly drained areas subject to possible flooding; annual wetlands; and regional ground water potential based on well yield estimates from representative aquifers. The seven county area is drained by two major river basins...the Delaware and Susquehanna Rivers. Shown in Figure 2.4, is the regional divide which separates these major watershed areas.

Figure 2.5 is an ERTS black and white image from Ultispectral Scanner (MSS) band 7, recorded in the near infrared wavelength region (0.8-1.1 micrometers). This scene covers the approximate northern half of the study area. Note the intricate surface drainage detail that can be identified from this image, particularly the well defined modified rectangular-dendritic drainage patterns at A and B; and the orderly alignment of the numerous lakes and ponds in the glaciated upland plateau area at C and C'. Compare these drainage characteristics with the parallel, linear patterns at D, expressed by the channel of the Lehigh River. Structural control of this drainage segment is evident. The macro drainage divide separating the Susquehanna and Delaware Drainage Basins is outlined in E-E'. During the course of this investigation, imagery from ERTS band 7 was particularly useful for mapping surface drainage patterns and for accurately defining landwater boundaries.

Geologic Flood Plains. Planning and development at both the regional and local levels require complete baseline information regarding the distribution and extent of flood plains because the susceptibility of such areas could present serious engineering construction and sewage problems.

The term "geologic flood plain", as used in this report refers to the maximum topographic extent of the valley floor which contains the stream channel. As geologic flood plains, these limits are indicative of the maximum extent to which flooding has occurred in past geologic history as a natural feature. Therefore, they do not equate to any statistical frequency of flooding, such as a 100-year storm, that may be defined by the affected manipulation of computations of data related to precipitation, run off and backwater characteristics.

The actual mapping procedure for the geologic flood plains consisted of two methods of flood plain determination. The first method was a topographic evaluation of the stream valley configuration taking into account both surficial landform features and present stream capability. This method entails a determination as to whether a stream is under fit, over fit or normally fit for its valley. A topographically identifiable stream valley may indeed be a much less serious hazard, or no hazard at all, if it contains an under fit stream. Stream valleys of this kind can be found at the higher elevations throughout the study area and are not included in the flood plain classification. Normally fit and over fit streams are indeed a constraint in that they are quite capable of flooding during periods of high stream flow. For this reason, the stream valley floors of both normally fit and over fit streams are classified as geologic flood plains.

Environmental Geology Survey, Regional
Land Use and Intermodal Transportation
Planning, Northeastern Pennsylvania
By J. David Welch
Page 29

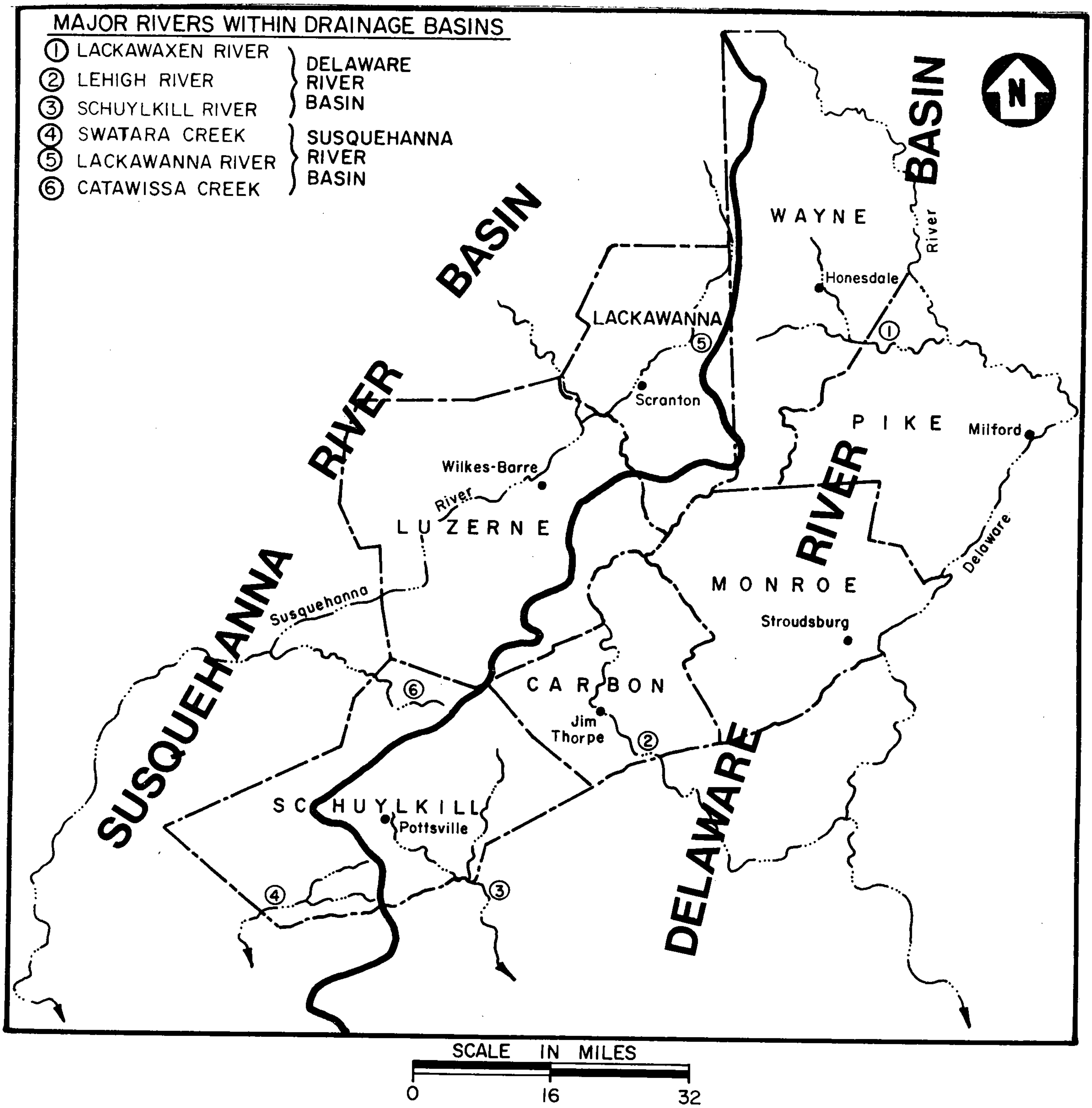
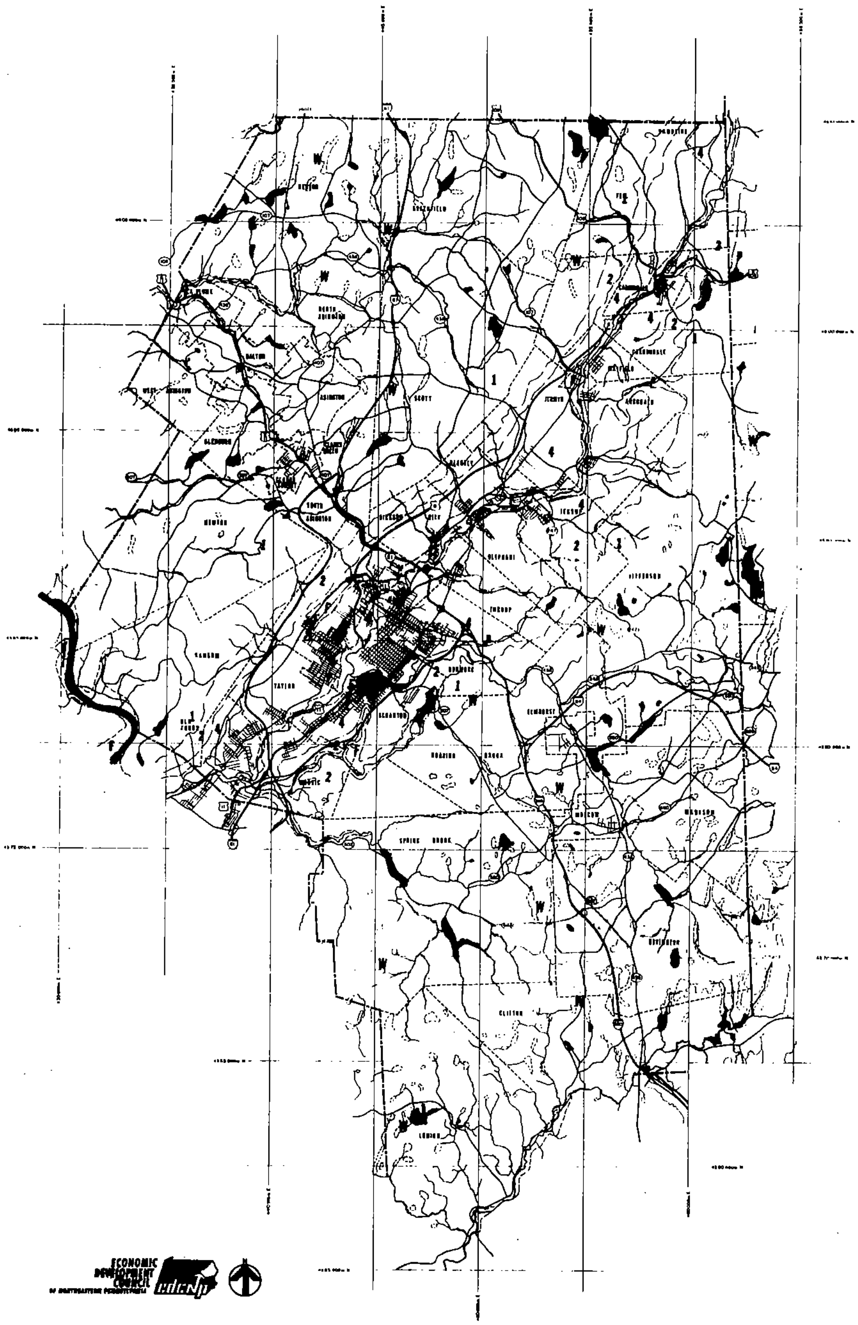


FIGURE 2.4 SURFACE DRAINAGE CHARACTERISTICS OF STUDY AREA



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Base map preparation by Economic Development Council of Northeastern Pennsylvania.
Technical data compiled by Joseph S. Ward and Associates.
Final presentation by Wilbur Smith and Associates.

LACKAWANNA COUNTY
PENNSYLVANIA

HYDROLOGY

IV–2

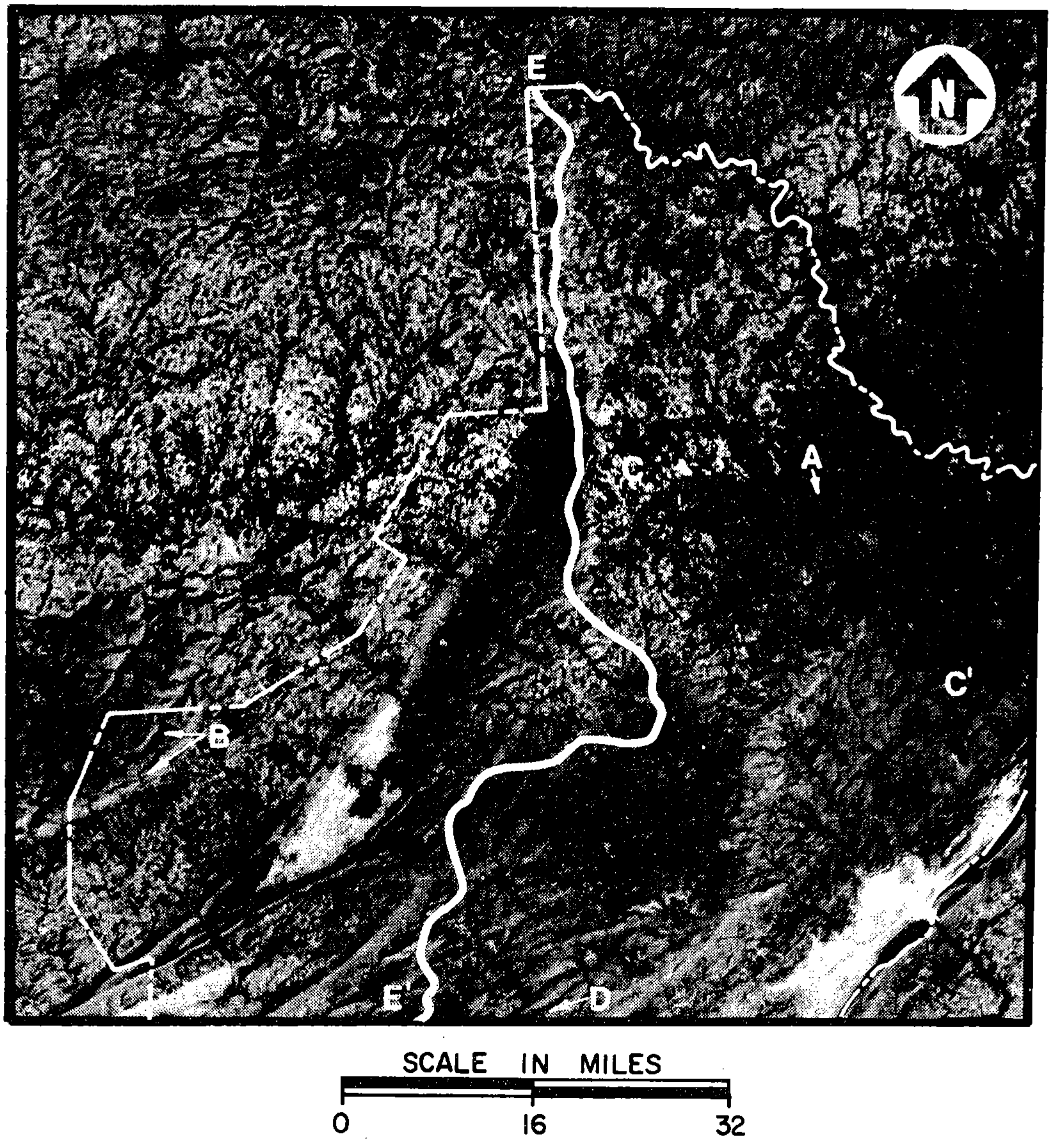


FIGURE 2.5 ERTS IMAGE OF REGIONAL HYDROLOGIC CHARACTERISTICS

The second method of flood plain determination consisted of examining published data on soil origins and relating this information to the surface hydrology conditions. The soil information used in this phase of the study was obtained from the "Land Resource Map of Pennsylvania" by W. H. Higbee (1967).

Annual Wetlands were those areas which are permanently subjected to total or partial inundation throughout the majority of the year. As such, they are areas of obvious constraint to any non-recreational development plan.

The term "annual wetland" as used in this report refers to bogs and swamps. Bogs are small pockets of organic soils and accompanying shallow water table found in the glaciated regions of the study area and formed due to the derangement of the natural drainage patterns by the deposition of glacial materials. Swamps are larger organic areas formed in natural depressions caused by ponding or blocking of surface water by natural or man-made barriers.

Groundwater in the study area is obtained primarily from aquifers; stratigraphic units that yield collectable quantities of water. Although glacio fluvial deposits, such as outwash and kame moraines, are scattered locally over the upland surfaces of the study area, their yields are generally small (5 gpm or less) and inconsistent.

Aquifers. The hydrologic characteristics of each of the 15 formations within the seven county area are summarized in Table 2.7. Note that the Pottsville, Mauch Chunk and Pocono formation are considered the most consistent aquifers in terms of groundwater yield and potential. The Catskill-Susquehanna, Marine Beds, Onondaga, Oriskany-Helderberg, and Keyser-Tonoloway formations appear to be aquifer sources with good potential. These aquifers are classified and combined into four broad mapping categories. Each category represents a qualitative rating with regard to a particular aquifer's yield and potential, i.e., primary, secondary, tertiary, low yield, and is based on the evaluation of the pertinent hydrologic characteristics of each of the individual stratigraphic units in the study area. The thickness of a particular aquifer will impact its water-bearing potential at any given locality. Beds that are too thin may be unimportant aquifer sources. Correspondingly, individual aquifer yields will also vary extensively throughout the area.

Points of Geologic Interest

From the regional standpoint, the seven county area contains some truly classic examples of Appalachian-type geology. Some of these areas could serve as potential points of interest in terms of scenic as well as scientific value. Representative points within the seven county area which were considered to have some degree of geologic significance were tabulated by county and township. Among the significant is the Delaware Water Gap.

Table 2-7

HYDROLOGIC CHARACTERISTICS OF MAJOR FORMATIONS OF STUDY AREA

FORMATION	MAP UNIT	DRAINAGE	POROSITY	GROUNDWATER
POST-POTTSVILLE	Ppp	GOOD SURFACE	TOTAL EFFECTIVE POROSITY MODERATE.	GENERALLY UNIMPORTANT SOURCE; HIGH CONTENT OF IRON SULFIDE; SMALL TO LARGE SUPPLIES RECOVERED IN AREAS REMOVED FROM MINING OPERATIONS.
POTTSVILLE	Pp	GOOD SURFACE	VARIABLE: DEPENDING ON LITHOLOGY; SANDSTONES HAVE HIGH TO MEDIUM EFFECTIVE POROSITY COMMON IN OTHER ROCK TYPES.	SANDSTONE VERY GOOD SOURCE; YIELDS UP TO 180 GPM REPORTED; LOCALLY SANDSTONES CONTAIN CONSIDERABLE IRON; IF ROCKS ARE CONSIDERABLY BELOW DRAINAGE LEVEL, THEY MAY CONTAIN BRACKISH WATER.
MAUCH CHUNK	Mmc	GOOD SURFACE	LOW TO MEDIUM POROSITY IN SANDSTONES & SILTSTONES; JOINTING PROVIDES ABUNDANT SECONDARY POROSITY IN SHALES, SANDSTONES & SILTSTONES, HIGH TOTAL EFFECTIVE POROSITY.	IMPORTANT SOURCE OF DOMESTIC & INDUSTRIAL SUPPLIES; QUALITY USUALLY GOOD TO EXCELLENT; SANDSTONE UNITS PROVIDE GREATEST YIELDS; AVERAGE YIELD OF 100 GPM AT AVERAGE DEPTH OF 590 FEET.
POCONO	Mp	GOOD SURFACE	INTERSTITIAL POROSITY & SECONDARY POROSITY GIVES UNIT A HIGH EFFECTIVE POROSITY.	VERY PRODUCTIVE AQUIFER; WELLS YIELDING 300 TO 600 GPM HAVE BEEN REPORTED; EXCEPT FOR OCCASIONAL HIGH IRON CONTENT, GOOD QUALITY AND SOFT.
SUSQUEHANNA	Ds	MODERATE TO GOOD SURFACE POOR IN GLACIATED REGIONS.	INTERSTITIAL POROSITY LOW IN COARSER ROCKS; JOINT DEVELOPMENT CREATES A MEDIUM QUALITY OF EFFECTIVE POROSITY.	FAIR TO GOOD AQUIFER; YIELDS IN EXCESS OF 300 GPM HAVE BEEN REPORTED; WATER QUALITY GOOD TO EXCELLENT.
CATSKILL	Dck	SAME AS Ds	SAME AS Ds	SAME AS Ds
MARINE BEDS	Dm	GOOD SURFACE	JOINT & BEDDING PLANES PROVIDE A SECONDARY POROSITY OF MEDIUM MAGNITUDE.	75% OF SUCCESSFUL WELLS DRILLED EXCEEDED SPECIFIC CAPACITY OF .74 OR 130 GPM; GOOD QUALITY; MAY BE HIGH IN DISSOLVED SOLIDS, MODERATE IN IRON & HARDNESS, MAY CONTAIN HYDROGEN SULFIDE & NATURAL GAS; WATER IN VALLEY AND RIDGE PROVINCE MUCH BETTER QUALITY.
HAMILTON & ONONDAGA UNDIFFERENTIATED	Dho	SEE Dh & Don	SEE Dh & Don	SEE Dh & Don
HAMILTON	Dh	GOOD SURFACE	JOINT & BEDDING PLANES PROVIDE SECONDARY POROSITY OF MEDIUM TO LOW MAGNITUDE.	SANDSTONES YIELD GOOD SUPPLIES AT MODERATE DEPTH; SHALES PROVIDE DOMESTIC SUPPLIES OF 10 GPM; WATER QUALITY MODERATELY GOOD WITH IRON REPORTED.
ONANDAGA	Don	GOOD SURFACE	JOINT & BEDDING PLANES PROVIDE A SECONDARY POROSITY OF MEDIUM MAGNITUDE	GOOD POTENTIAL; ABOUT 25% OF SUCCESSFUL WELLS DRILLED TO DEPTH OF 200 FEET IN DEPRESSIONS EXPECTED TO YIELD 36 GPM OR MORE.
ORISKANY & HELDERBERG (UNDIFFERENTIATED)	Doh	ORISKANY-GOOD SURFACE HELDERBERG-GOOD SURFACE WITH MINOR SUBSURFACE DRAINAGE.	ORISKANY-VARIABLE INTERSTITIAL POROSITY; JOINT & FAULT OPENINGS ADD SECONDARY POROSITY; MEDIUM AMOUNT OF TOTAL EFF. POROSITY. HELDERBERG-JOINTS FAULTS & SOLUTION CHANNELS PROVIDE HIGH MAGNITUDE SECONDARY POROSITY.	ORISKANY-YIELDS UP TO 150 GPM REPORTED FROM POORLY CEMENTED ROCK; WATER QUALITY USUALLY GOOD. HELDERBERG ESTIMATES OF POTENTIAL YIELD INDICATE ABOUT 75% OF SUCCESSFUL WELLS DRILLED TO DEPTH OF 200 FEET EXPECTED TO YIELD UP TO 80 GPM.
KEYSER & TONOLOWAY (UNDIFFERENTIATED)	Skt	GOOD SURFACE; MINOR SUBSURFACE DRAINAGE	SOLUTION CHANNELS, JOINT & BEDDING PLANES PROVIDE SECONDARY POROSITY OF HIGH MAGNITUDE.	GOOD POTENTIAL; ABOUT 25% OF THE WELLS DRILLED TO DEPTH OF 200 FT IN VALLEYS OR HILLSIDES MAY BE EXPECTED TO YIELD 86 GPM OR MORE.
BLOOMSBURG & McKENZIE (UNDIFFERENTIATED)	Sbm	GOOD SURFACE	BLOOMSBURG-JOINT, FAULT & BEDDING PLANES PROVIDE A SECONDARY POROSITY OF LOW TO MEDIUM MAGNITUDE. MC KENZIE-JOINTS, BEDDING & CLEAVAGE PLANES PROVIDE SECONDARY POROSITY OF LOW MAGNITUDE.	BLOOMSBURG-75% OF SUCCESSFUL WELLS PRODUCED 30 GPM OR MORE; 25% PRODUCED 200 GPM OR MORE; FAIRLY GOOD QUALITY; CONTAINS MODERATE AMOUNTS OF DISSOLVED SOLIDS, LOW TO MODERATE AMOUNTS OF HARDNESS, LOW IRON CONTENT. MC KENZIE-SMALL SUPPLIES FOR DOMESTIC PURPOSES; YIELDS 3 to 5 GPM OBTAINED FROM WELLS BETWEEN 50 & 150 FT DEPTH.
CLINTON	Sc	GOOD SURFACE	JOINTS & BEDDING PLANES PROVIDE A SECONDARY POROSITY OF LOW MAGNITUDE.	POOR POTENTIAL; YIELDS ADEQUATE FOR DOMESTIC NEEDS (3 GPM OR MORE) ARE OBTAINED, GENERALLY FROM WELLS BETWEEN 50 & 150 FT DEPTH.
SHAWANGUNK/ TUSCARORA	Ss/St	GOOD SURFACE	INTERGRANULAR POROSITY IN CONGLOMERATE, JOINT OPENINGS PROVIDE SMALL TO MODERATE SECONDARY POROSITY.	POOR POTENTIAL; MEDIUM YIELD APPROXIMATELY 4 GPM; WATER USUALLY SOFT AND GOOD QUALITY.

ANALYSIS OF REGIONAL GEOLOGIC CONSTRAINTS

The primary purpose of this phase of the investigation was to evaluate existing geologic conditions, and to define from these conditions, the major environmental hazards and constraints which could impact regional planning opportunities and transportation decisions. Table 3.1 shows the effect of certain geologic conditions on land use.

The environmental criteria considered most appropriate in terms of regional planning include potential hazards such as land subsidence from coal mining activities or soluble limestone; land slide potential and slope instability; and seismic activity. Major constraints examined include shallow soil; flood plain width and extent; poorly drained areas subject to seasonal or perennial high water table conditions; adverse soil deposits and severe slope. In addition, the acceptability of certain areas as waste disposal sites was also determined as part of this regional land capability evaluation.

It was originally planned to show all pertinent environmental geologic constraints on one summary overlay. However, during the course of the data evaluation phase, it became increasingly apparent that the extensive amount of detail derived as a result of the regional resources inventory would not allow the compilation of an all-encompassing land capability summary overlay without causing a great deal of graphic complexity and map clutter. In the best interests of graphic presentation and user communication, therefore, it was decided to portray and incorporate the results of the individual constraints evaluation phase with bedrock hazards and constraints on the base geology mapping; soils hazards, constraints, and suitabilities on the soils and surficial geology mapping; and topographic and surface drainage constraints on a separate mapping.

These individual map overlays show the potential environmental geology problem areas that could impact regional development. Specifically, these maps flag those particular areas that could present serious engineering problems, based on the evaluation of existing topographic, hydrologic, soils and bedrock conditions.

Bedrock Hazards and Constraints

Regional bedrock hazards and constraints shown on the Base Geology maps, are identified by the letters "H" and "C" respectively, followed by an appropriate number designating the specific environmental - engineering condition that could be encountered.

Table 3.2 lists the typical bedrock problems identified during the course of the investigation. As an example, a potential problem area is designated by the symbol Ppp/H1, H3, C2. This designation indicates that the area occurring within that particular map unit boundary is underlain

Environmental Geology Survey, Regional
Land Use and Intermodal Transportation
Planning, Northeastern Pennsylvania
By J. David Welch
Page 35

Table 3-1

REPRESENTATIVE GEOLOGIC CONDITIONS THAT COULD AFFECT LAND USE

GEOLOGIC CONDITIONS	TYPICAL LAND USES						
	LIGHT CONSTRUCTION	HEAVY CONSTRUCTION	WASTE DISPOSAL	BUILDING MATERIAL RESOURCES	EASE OF EXCAVATION	ROAD PERFORMANCE	AGRICULTURE
PHYSICAL PROPERTIES OF SOILS AND ROCKS	1	1	1	1	1	1	1
SLOPE STABILITY	1	1	2	2	2	1	2
THICKNESS OF SURFICIAL MATERIAL	1	1	1	1	2	2	2
DEPTH TO GROUNDWATER	1	1	1	1	2	2	1
SURFACE WATER SUPPLY	2	2	2	1	2	2	1
DANGER OF FLOODING	1	1	1	1	1	1	1
STEEP SLOPE	1	1	1	2	1	1	1

NOTE: 1 = Primary Importance; 2 = Secondary Importance

Table 3-2

REGIONAL BEDROCK HAZARDS AND CONSTRAINTS

	MAP SYMBOL	ENVIRONMENTAL-ENGINEERING CONDITION	MAJOR IMPACT
HAZARDS	H1	Potential Subsidence From Coal Mining	Safety; Foundation Stability
	H2	Potential Subsidence From Sinkholes	
	H3	Landslide Potential/Slope Instability	Safety; Slope Stability; Transportation Route Selection and Performance; Overall Land Planning and Developments
	H4	Recent Seismic Activity	Safety; Foundation and Slope Stability
CONSTRAINTS	C1	Difficult Excavation, Slow Drilling	Workability
	C2	Extensive Faulting and Fracturing	Foundation and Slope Stability

by the Post-Pottsville formation. This formation, in turn, could present serious engineering problems, regarding safety, foundation stability and slope instability due to its susceptibility to potential subsidence from mined-out coal areas, undercutting in shales and extensive faulting and/or fracturing. Correspondingly, it is evident that any contemplated development in such areas should be preceded by detailed site investigations.

An evaluation of the geologic information enabled the compilation of Table 3.3. This table identifies potential engineering problem areas that could be associated with specific formations and is based on a qualitative and relative ranking system ranging from 1 to 5. The designation "1" denotes a potential problem area, whereas a ranking of "5" indicates a minimal problem with regard to a particular formation.

Hazards. Bedrock hazards that could impact development plans within the seven county area include potential land subsidence from coal mining and/or soluble limestone; landslide and/or slope instability, and possibly, seismic activity.

TABLE 3-3

ENVIRONMENTAL-ENGINEERING PROBLEMS ASSOCIATED WITH FORMATIONS IN SEVEN COUNTY AREA

	PHYSICAL		WORKABILITY		SLOPE PROBLEMS		FOUNDATION STABILITY	
FORMATION	Depth to Rock	Fracturing	Ease of Excavation	Ease of Drilling	Local Undercutting	Stability	Mined-Out Coal Areas	Concealed Sinkholes
Post-Pottsville	4	1	2	2-3	1	2-3	1	5
Pottsville	1	1	2-3	2-3	1-2	2	2-4	5
Mauch Chunk	2-3	2	4	4	3	4	5	5
Pocono	1	2	1	1	5	5	5	5
Catskill-Susquehanna	1-5	2	1-2	3	2-3	2-4	5	5
Marine Beds	4-5	2	3	3	5	3	5	5
Hamilton & Onondaga Undiff.	3-5	1-3	2-4	2-4	4	2-5	5	1-5
Hamilton	4-5	2	3	4	4	2	5	5
Onondaga	2-3	3	2	3	4	5	5	1-2
Oriskany & Helderberg Undiff.	2	2-3	1-3	1-3	5	3-5	5	1-2
Keyser & Tonoloway Undiff.	2	1-2	1	2-3	5	2-4	5	1
Bloomsburg & McKenzie Undiff.	2	3	3-5	5	5	1-2	5	1-3
Clinton	2	2	2	1-3	5	5	5	5
Shawangunk	1	2	1	1	5	5	5	5

Note: Numerical values are qualitative and relative rankings only. Excellent Characteristics (No Problem) = 5; Very Poor, Unsatisfactory Characteristics = 1; Average Condition = 3.

Constraints. Major constraints associated with bedrock include difficult excavation/slow rates of drilling, and intensive faulting and fracturing. Shallow soil, i.e., rock occurring within four feet of the surface, is also a definite geologic constraint. Since this particular condition is directly dependent upon thickness of overburden, it was discussed in the section on Soils Hazards, Constraints and Suitabilities.

Soils Hazards, Constraints and Suitabilities

Regional soil hazards, constraints and suitabilities were delineated with potential soil hazards being identified by the letter "H"; constraints by the letter "C", and suitabilities by the letter "A". Each letter is followed by an appropriate number showing the specific environmental condition or problem that could be encountered within a particular mapped area. In general, the problem soils identified in this section were derived primarily from the evaluation of pertinent regional soil characteristics.

Table 3.4 lists the typical soil conditions identified during the course of the study. The map symbols shown on this table correspond directly to those appearing on the maps. Potential soil problem areas are identified by their appropriate numerical soil association grouping as well as an alpha-numeric symbol indicating the specific problem associated with that particular soil. For example, an area identified by the designation "11/H3a, C4" indicates that the dominant soils consist of the Meckesville-Albrights grouping and that specific areas within this soils grouping are susceptible to a high degree of surface erodibility, poor subsoil permeability and seasonal high water table. The impact on planned development in areas underlain by this particular soil group is readily apparent. Conversely, areas of favorable soil conditions are identified by the alpha-numeric designation "A1". This indicates, for example, that an area classified as 7/A1 consists of the Hazelton-Allenwood-Watson soil grouping. This particular soil group has a moderately deep to deep well-drained soil profile with generally favorable percolation rates for waste disposal activities.

The primary soil hazards occurring within the seven county area are those associated with high soil erodibility and with flood plain soils.

Major constraints associated with the soils of the seven county area include those pertaining to shallow soil, poor hydrologic characteristics and poor drainage characteristics.

Suitabilities. One of the aspects of the geologic environment that was examined during the course of the study involved the evaluation of areas which are generally suitable for liquid and solid waste disposal systems, including septic tanks, leaching fields and sanitary landfills. Based on the nature of existing terrain, bedrock, soil and hydrologic conditions, however, there appears to be a dearth of areas within the seven county region that are suited for development of waste disposal systems.

Table 3-4

REGIONAL SOIL HAZARDS, CONSTRAINTS AND SUITABILITIES

	MAP SYMBOL	ENVIRONMENTAL-ENGINEERING CONDITION	MAJOR IMPACT
HAZARDS	H3a	High Soil Erodibility	Safety; Slope Stability; Transportation Route Selection and Performance; Overall Land Planning and Development
	H5	Alluvial Soil-much of which is Flood Plain-Flood Plain areas could increase	Safety; Overall Land Planning and Development; Waste Disposal Activities
CONSTRAINTS	C3	Shallow Rock (Generally 4.0 feet or less)	Workability; Waste Disposal Activities
	C4	Poor Subsoil Permeability; High Seasonal Water Table;	Overall Land Planning and Development; Waste Disposal Activities
	C5	Very Poorly Drained Upland Soils With High Water Table	Overall Land Planning and Development; Waste Disposal Activities
SUITABILITIES	A1	Areas Characterized By Favorable Soil Percolation Rates: Moderately Deep to Deep Well-Drained Soil Profile; Slopes Ranging Generally Between 0-25 Percent	Favorable Areas for Waste Disposal Activities

Of the 14 regional soil groups occurring in the seven county area, only three were considered to be suitable in terms of having favorable soil percolation rates, including moderately deep to deep, well-drained soil profiles. However, individual soils of these groups could locally possess poor characteristics in terms of shallow rock, severe slope constraints, and/or low subsoil permeability with inherent seasonal high water table conditions. Obviously, detailed site investigations in these local areas are required prior to formulating final development plans.

Table 3.5 summarizes the total acreage and percentage of favorable DER Soil Groups (refer to Table 2.6) occurring within five representative counties of the study area. Note that Carbon and Monroe counties have the highest percentage of favorable DER Soil Groups whereas Lackawanna county has the lowest percentage of suitable soils for subsurface absorption fields.

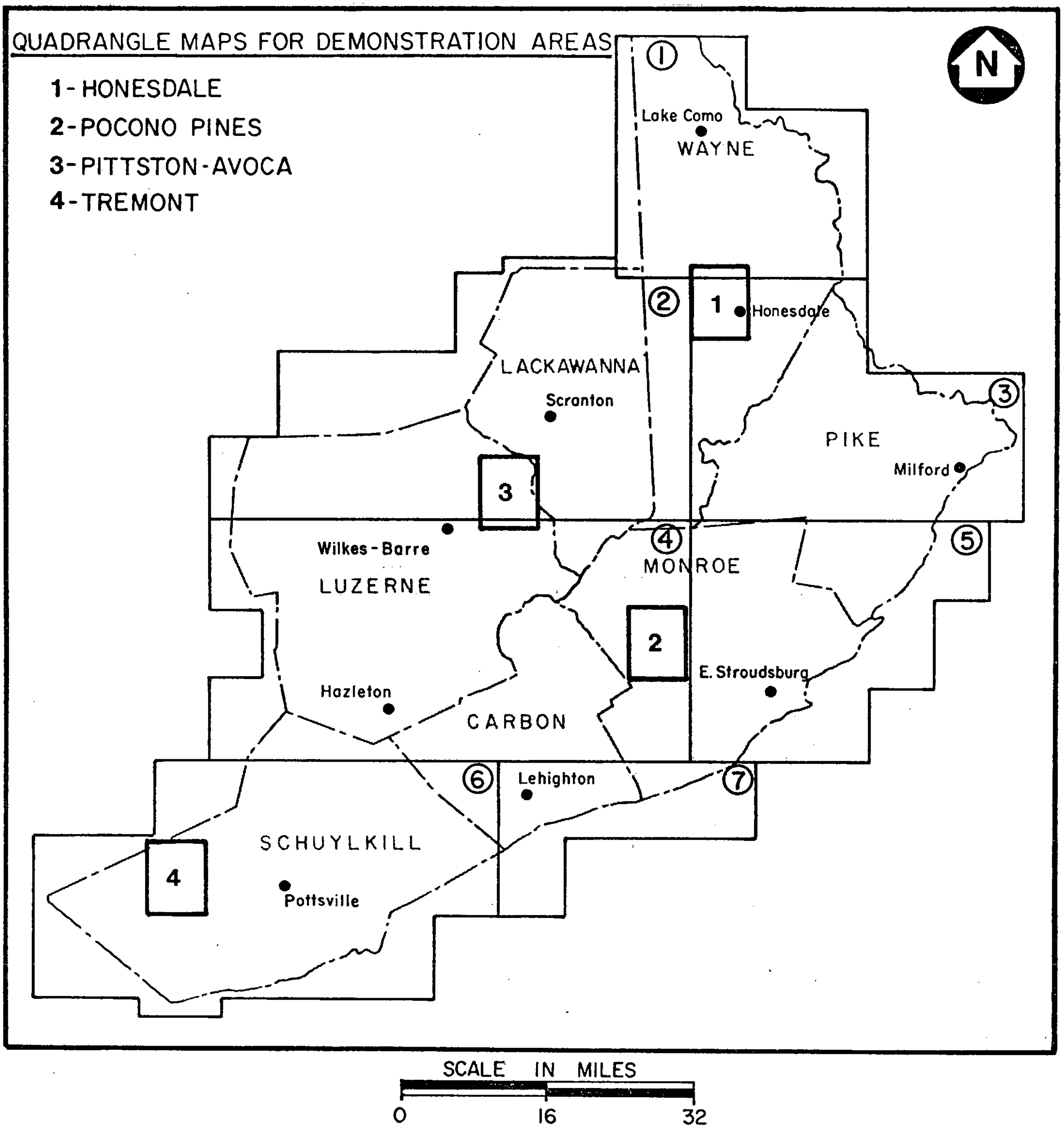
Table 3-5
ACRES OF FAVORABLE DER SOILS BY COUNTY

	DER GROUP				
COUNTY	2	4	6	8	2,4,6, & 8 as % of TOTAL
Carbon	9,870	25,347	24,671		23.1
Lackawanna	9,114			10,427	6.7
Luzerne	4,250	5,125	21,685	46,240	13.6
Monroe	22,988	3,605	4,960	16,895	23.3
Pike	29,353			27,152	15.8

Table 3.6 is a matrix summarizing the adaptation of alternate subsurface absorption areas for various types of contemplated building situations, including residential, commercial, industrial, etc.

Topographic and Surface Drainage Constraints

Topographic and surface drainage constraints occurring within the seven county area include those pertaining to severe slope; flood plains; and, wetlands, swamps, and high water table areas. These problem areas are delineated and are listed in Table 3.7.



MAP SOURCE: BASE MAP WAS OBTAINED FROM U.S. DEPT. OF INTERIOR GEOLOGICAL SURVEY MAP, STATE OF PENNSYLVANIA, 1955 EDITION.

FIGURE 1.3 DATA ORIENTATION MAP

Table 3-6

MATRIX FOR ADAPTATION OF ALTERNATE SUBSURFACE ABSORPTION AREAS

X adaptable 0 not permissible

SITUATIONS / ALTERNATIVES	subdivisions	commercial industrial institutional	existing malfunctions remedial Action	isolated building new residences	isolated hunting & fishing camps	where permissable (DER soil groups) (2)	maximum natural slope in % (5)	(6)	(7)
elevated sand mound with septic tank	X	X	X	X (1)	X	1,3,5,7,9,10 11,12,14	12	8	0
elevated sand mound with aerobic tank (3)	X	X	X	X (1)	0	1,3,5,7,9,10 11,12,14	12	8	0
sand lined trenches and beds in excessively permeable sites (3)	X	X	X	X (1)	X (4)	1,3,5,7,9	25	8	25
over-sized absorption area in well drained soil with percolation time of 60-90 minutes per inch. Aerobic tanks must be used	X	0	X	X (1)	0	10	25	8	0
shallow placement	X	X	0	X	X (4)	1,3,5,7,9,11	15	5	20

(1) There shall be sufficient suitable area for replacement of this alternate subsurface absorption area with another of equivalent size.

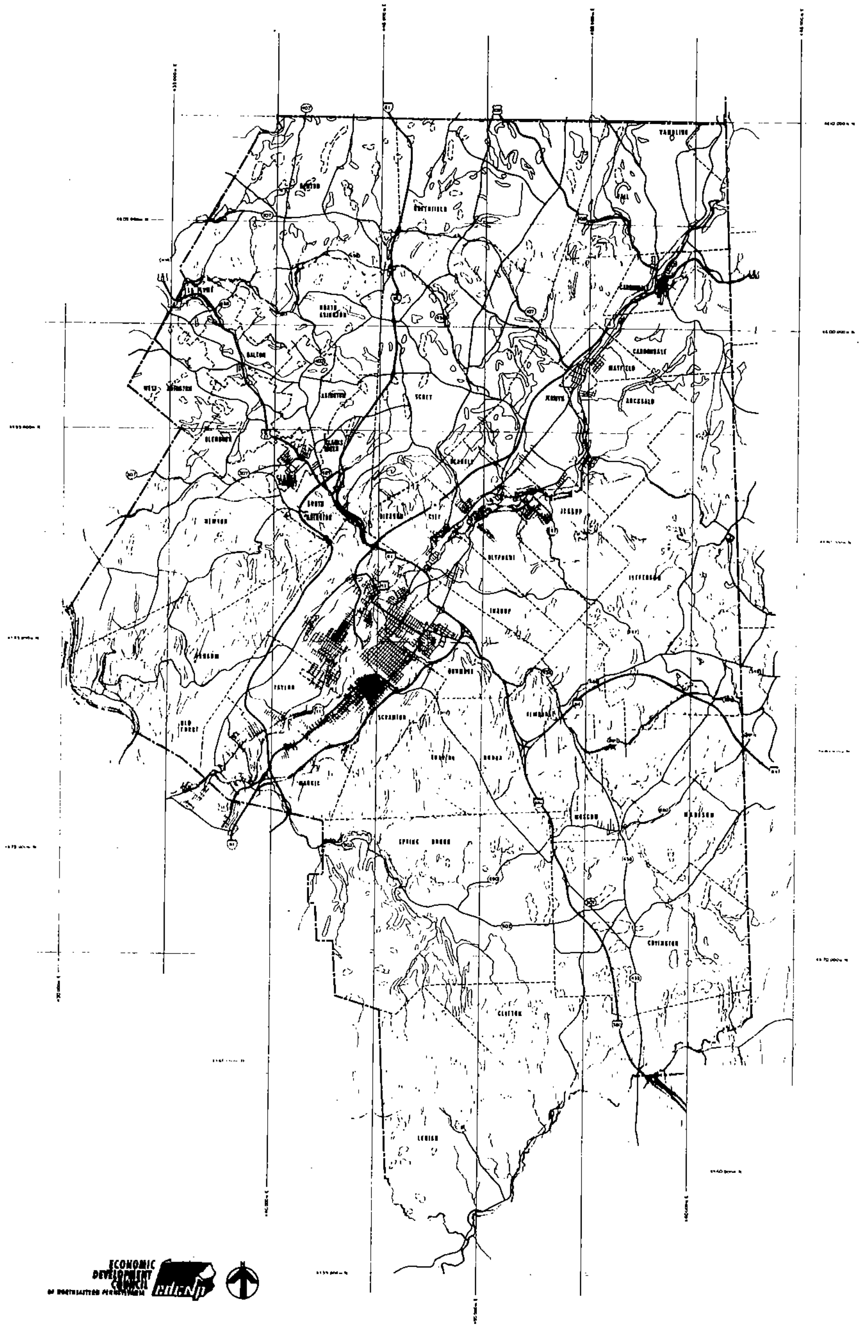
(2) Soil groups 2,4,6, & 8 are generally suited for conventional subsurface absorption areas; individual on-site investigation is required.

(3) Allowance of one-third reduction in size of absorption area may be used with an aerobic treatment tank.

(4) Aerobic Treatment Tanks not permitted.

(5) standard trench. (6) seepage bed (7) serial distribution

(8) Retention tanks, privies, chemical toilets and related on-lot sewage disposal systems are individual sewage systems and require permits. Such systems do not provide for final on-lot treatment and disposal of the sewage and require regular service and maintenance to prevent their malfunction and overflow. Holding tanks can only be used where the Department of Environmental Resources finds and gives written notice to the approving body that the proper requirements have been met.



ECONOMIC DEVELOPMENT COUNCIL OF NORTHEASTERN PENNSYLVANIA

The preparation of this map was financed in part by The Appalachian Regional Commission under section 302 of The Appalachian Regional Development Act of 1965, as amended, App. U.S.C. 302; in part by The Department of Housing and Urban Development, under provisions of section 701 of the Housing Act of 1954, as amended and as administered by The Pennsylvania Office of State Planning and Development and in part by the Economic Development Administration.

Base map preparation by Economic Development Council of Northeastern Pennsylvania.
Technical data compiled by Joseph S. Ward and Associates.
Final presentation by Wilbur Smith and Associates.

LACKAWANNA COUNTY
PENNSYLVANIA

TOPOGRAPHIC AND SURFACE DRAINAGE CONSTRAINTS

V–2

Table 3-7

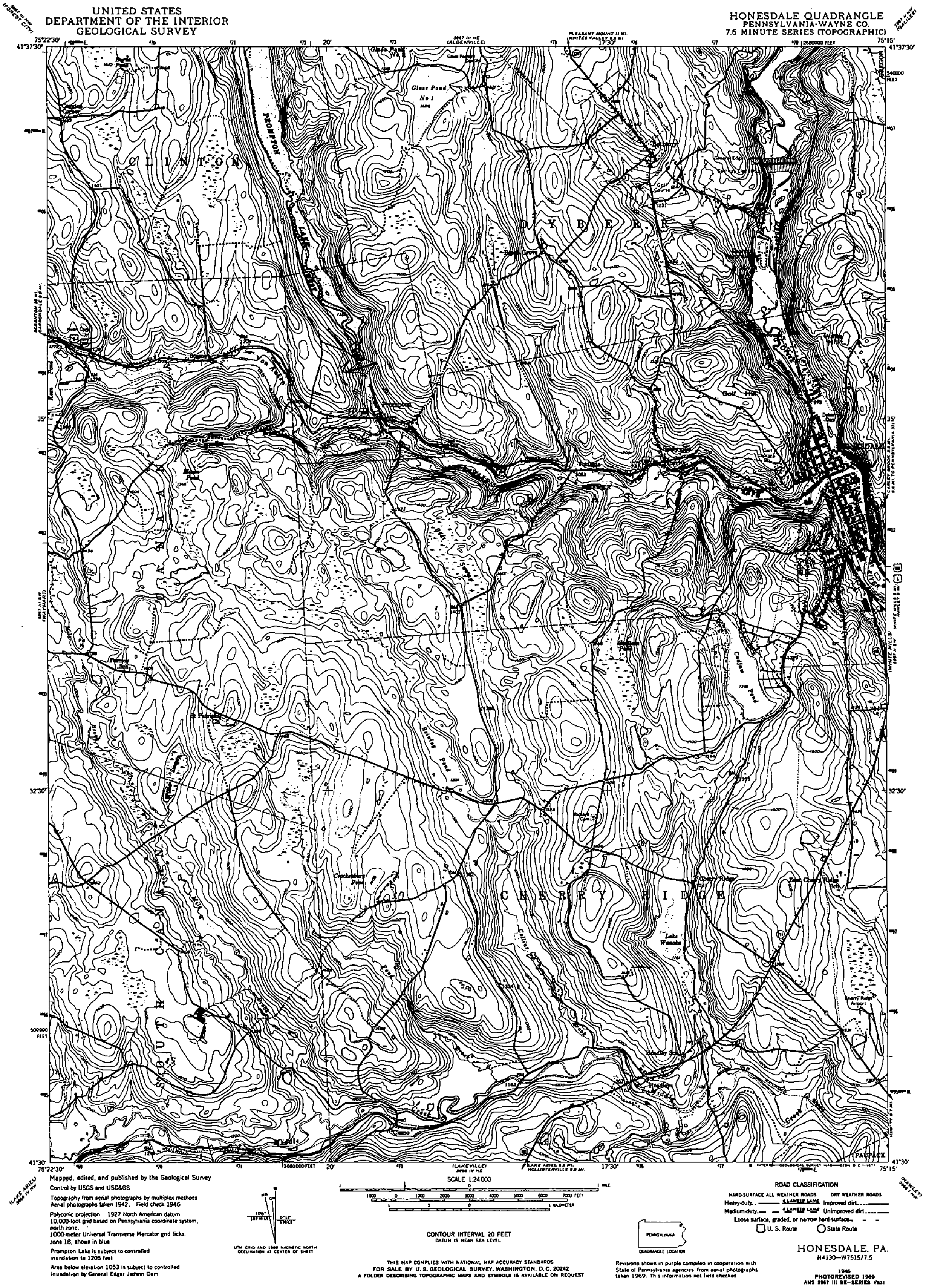
REGIONAL TOPOGRAPHIC AND SURFACE-DRAINAGE CONSTRAINTS

MAP SYMBOL	ENVIRONMENTAL-ENGINEERING CONSTRAINT	MAJOR IMPACT
S	Slopes 25% or Greater	Overall Land Planning and Development; Transportation Routing; Waste Disposal Activities
F	Flood Plain Area	Overall Land Planning and Development; Waste Disposal Activities
W	Wetlands, Swamps and Related High Water Table Areas	

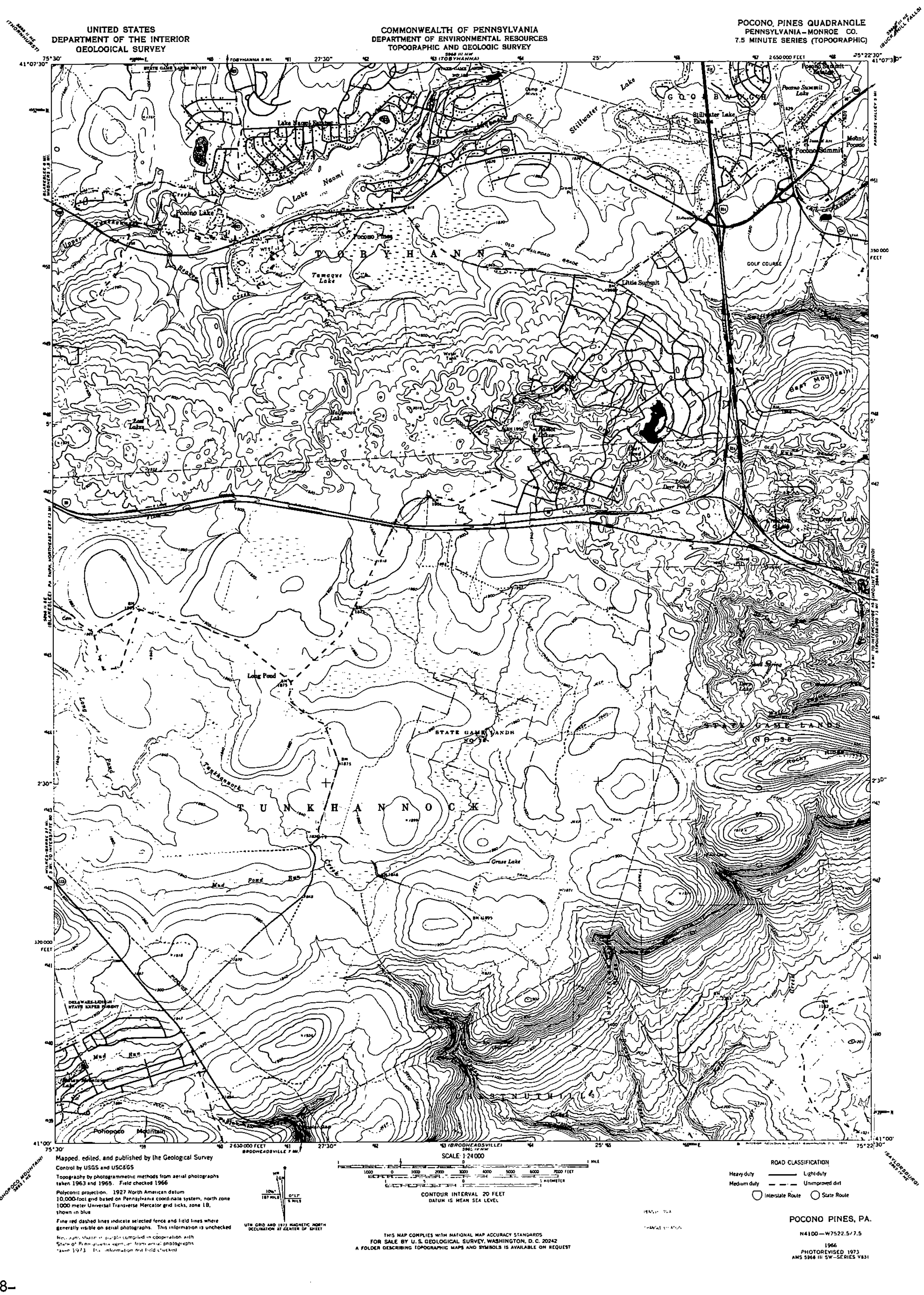
All land slopes 25-percent or greater were classified as potential constraint areas on the map sheets. More than one quarter of the seven county area is characterized by slopes that are greater than 25-percent. The primary factor contributing to this condition is physiography; namely, the undulating, high relief ridges and valleys of the Folded Appalachians, and the moderately dissected upland surface of the Appalachian Plateau.

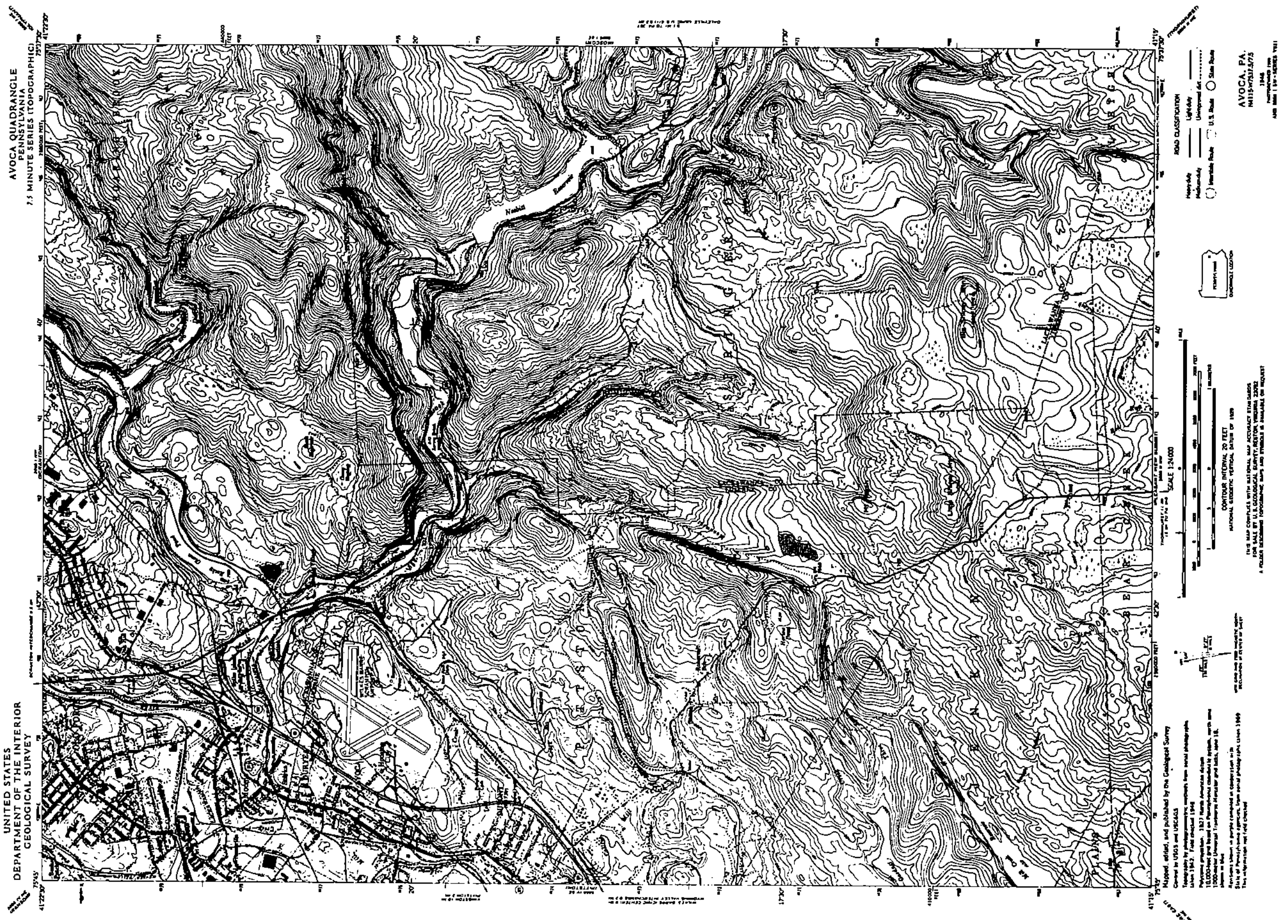
DETAILED GEOLOGIC ANALYSIS OF FOUR DEMONSTRATION AREAS

The second task of the environmental geology survey was to perform an evaluation of four local areas for purposes of defining key geologic conditions which could impact future policies for detailed land planning, zoning and decision-making. These local areas, termed "Demonstration Areas", were selected because they represent a variety of physiographic, geologic and soils conditions. Each of these demonstration areas are directly keyed to standard 1:24,000-scale USGS quadrangle maps. Their selection was also guided to cover a broad range of major land use categories which are representative of the socio-economic conditions occurring within the overall seven county region, including urban and built-up land, rural-agricultural, mixed forest land, mining-extractive land and resort-recreational areas. Specifically, the four areas that were evaluated during this study include:


UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
HONESDALE QUADRANGLE
PENNSYLVANIA-WAYNE CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
CLINTON
DYBERRY
CHERRY RIDGE
PROMPTON
PAUPACK
Mapped, edited, and published by the Geological Survey
Control by USGS and USC&GS
Topography from aerial photographs by multiplex methods
Aerial photographs taken 1942. Field check 1946
Polyconic projection. 1927 North American datum
10,000-foot grid based on Pennsylvania coordinate system, north zone.
1000-meter Universal Transverse Mercator grid ticks, zone 18, shown in blue
Prompton Lake is subject to controlled inundation to 1205 feet
Area below elevation 1053 is subject to controlled inundation by General Edgar Jadwin Dam
UTM GRID AND 1969 MAGNETIC NORTH DECLINATION AT CENTER OF SHEET
SCALE 1:24 000
CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL
THIS MAP COMPLIES WITH NATIONAL MAP ACCURACY STANDARDS
FOR SALE BY U.S. GEOLOGICAL SURVEY, WASHINGTON, D.C. 20242
A FOLDER DESCRIBING TOPOGRAPHIC MAPS AND SYMBOLS IS AVAILABLE ON REQUEST
QUADRANGLE LOCATION
Revisions shown in purple compiled in cooperation with State of Pennsylvania agencies from aerial photographs taken 1969. This information not field checked
ROAD CLASSIFICATION
HARD-SURFACE ALL WEATHER ROADS
DRY WEATHER ROADS
Heavy-duty
Improved dirt
Medium-duty
Unimproved dirt
Loose-surface, graded, or narrow hard-surface
U. S. Route
State Route
HONESDALE, PA.
N4130—W7515/7.5
1946
PHOTOREVISED 1969
AMS 5967 III SE—SERIES V831

Environmental Geology Survey, Regional Land Use and Intermodal Transportation Planning, Northeastern Pennsylvania
By J. David Welch
Page 46




AVOCA QUADRANGLE
PENNSYLVANIA
7.5 MINUTE SERIES (TOPOGRAPHIC)
UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
AVOCA, PA.


PITTSTON QUADRANGLE
PENNSYLVANIA
7.5 MINUTE SERIES (TOPOGRAPHIC)
UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
SUSQUEHANNA
RIVER
PITTSTON, PA.


UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
STATE OF PENNSYLVANIA
DEPARTMENT OF INTERNAL AFFAIRS
TOPOGRAPHIC AND GEOLOGIC SURVEY
TREMONT QUADRANGLE
PENNSYLVANIA
7.5 MINUTE SERIES (TOPOGRAPHIC)
MAHANOY
Mapped, edited, and published by the Geological Survey
SCALE 1:24000
CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL
THIS MAP COMPLIES WITH NATIONAL MAP ACCURACY STANDARDS
FOR SALE BY U. S. GEOLOGICAL SURVEY, WASHINGTON, D.C. 20242
A FOLDER DESCRIBING TOPOGRAPHIC MAPS AND SYMBOLS IS AVAILABLE ON REQUEST
ROAD CLASSIFICATION
TREMONT, PA.
1954
PHOTOREVISED 1969

1. Honesdale, in Wayne County, in the northeastern part of the study area, is characterized by a representative rural-small community land use, in glaciated low plateau physiography.

2. Pocono Pines, in Monroe County, to the southeast of the study area, is characterized by representative resort-recreational land use, in partially glaciated high plateau physiography.

3. Pittston/Avoca, (approximately one-half each) in Lackawanna and Luzerne County, to the northeast of the study area, is characterized by a variety of urban-suburban-industrial land use categories, situated in the Wyoming-Lackawanna Basin, the northern most extension of the Valley and Ridge Province in Pennsylvania.

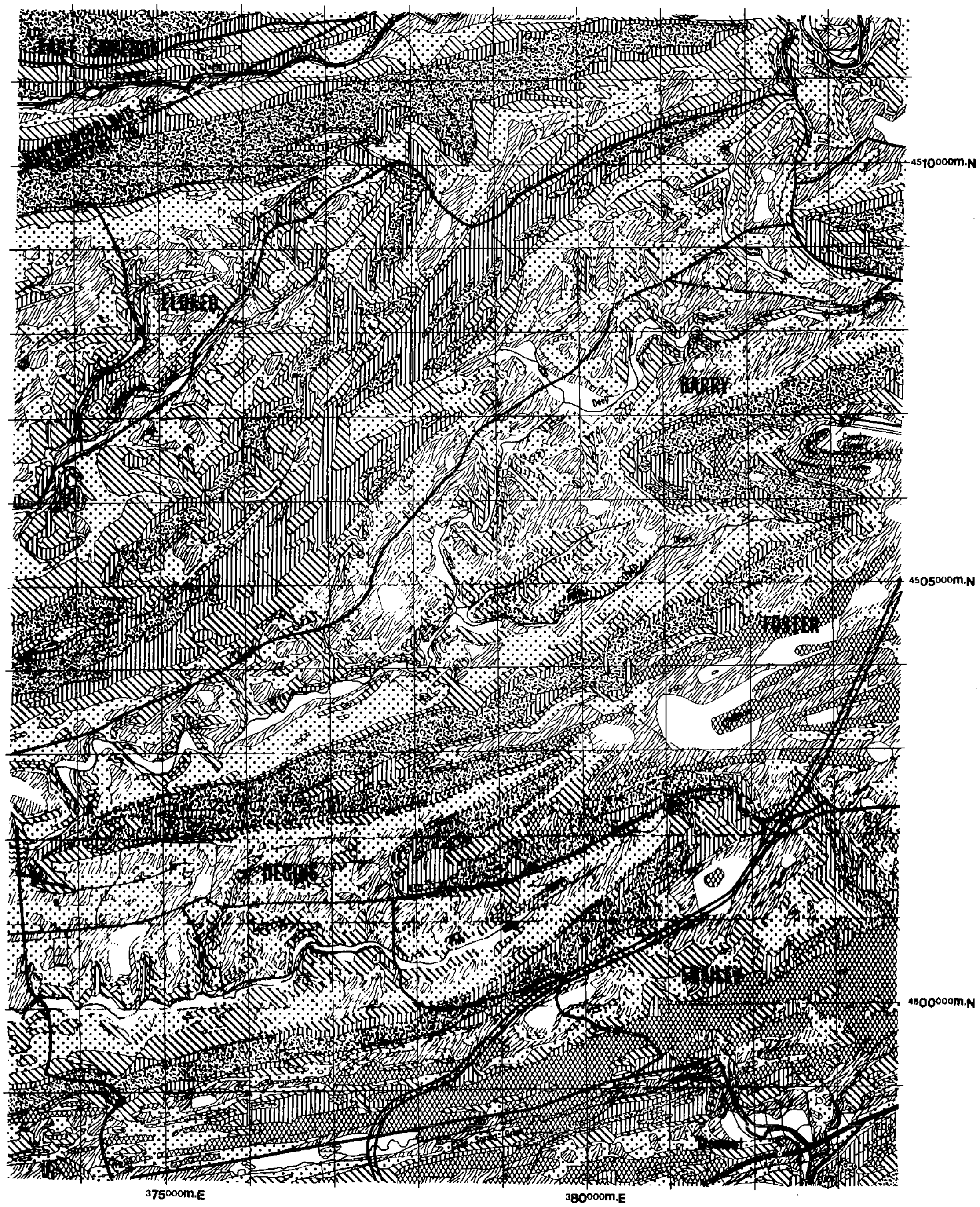
4. Tremont, in Schuylkill County, in the southwestern part of the study area, is characterized by mining and extractive land use, situated within the highly folded and faulted Valley and Ridge Province.

As was the case for the regional seven county survey, a series of four intelligence overlays were compiled for each. In addition, a fifth overlay is included which identifies certain constraints. It is readily apparent that the level of environmental geology detail on each of these overlays (i.e., level of stratification) is much greater than that shown on the corresponding regional map sheets for these same areas. This, of course, is due to the larger map scale factor which enables the inventory, compilation and presentation of more detailed conditions at the local level.

The specific environmental geology intelligence that is shown on the individual 1:24,000-scale overlays for each of the four representative areas includes the following:

Topography showing detailed land slope classification in seven categories (0-3, 3-8, 8-15, 15-25, 25-35, 35%), including areas that have slopes that are generally 3% and seldom saturated.

Base geology showing bedrock type, key stratigraphic members, unconsolidated alluvial deposits, structural characteristics, outcrop/shallow soil areas; and potential bedrock constraints and hazards to development.



TREMONT, PENNSYLVANIA

SLOPE

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Base map preparation by Wilbur Smith and Associates.
Technical data compiled by Joseph S. Ward and Associates.
Final presentation by Wilbur Smith and Associates.

ECONOMIC DEVELOPMENT COUNCIL OF NORTHEASTERN PENNSYLVANIA

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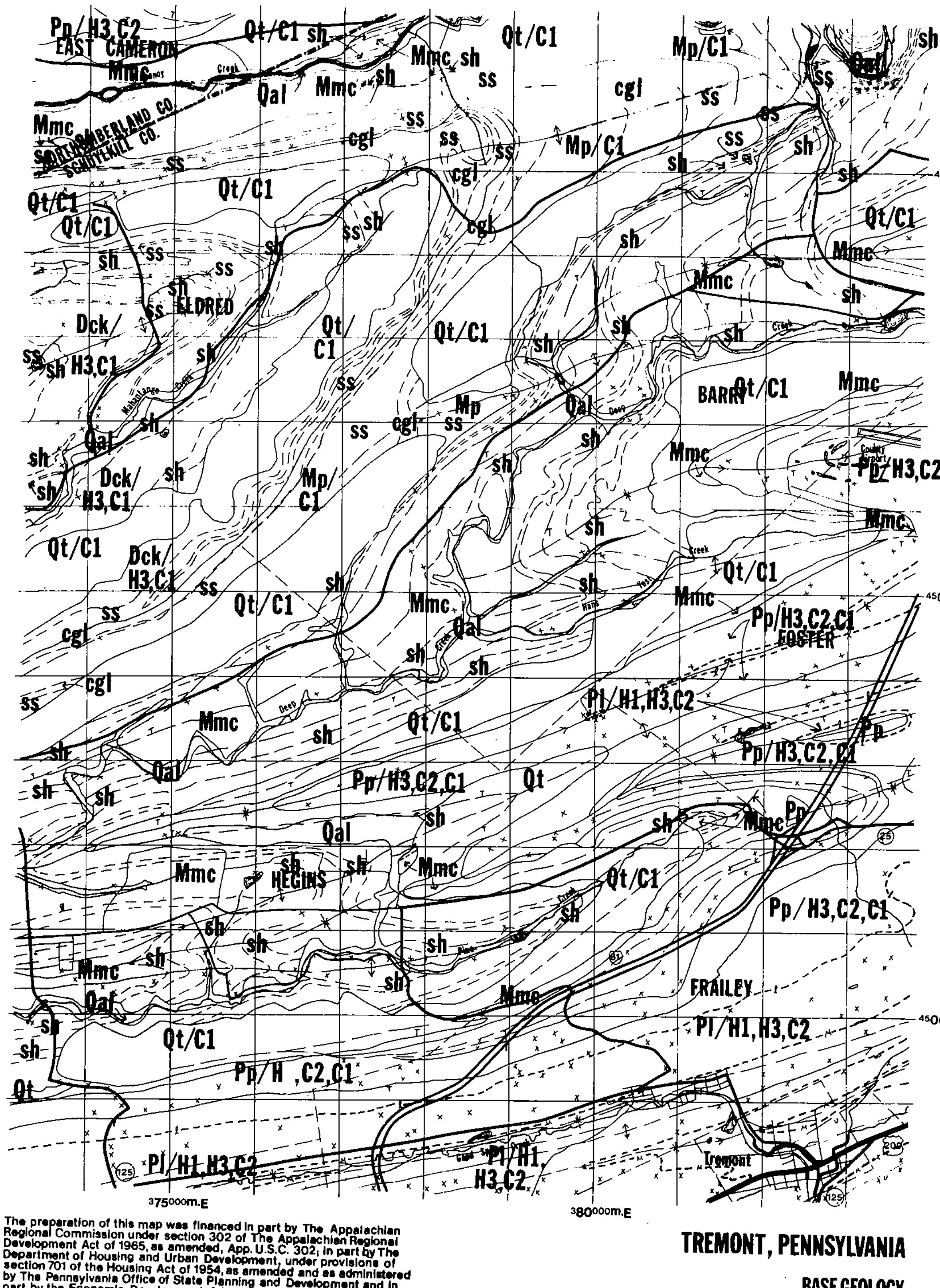

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TREMONT, PENNSYLVANIA

BASE GEOLOGY

ECONOMIC DEVELOPMENT COUNCIL OF NORTHEASTERN PENNSYLVANIA edcnp

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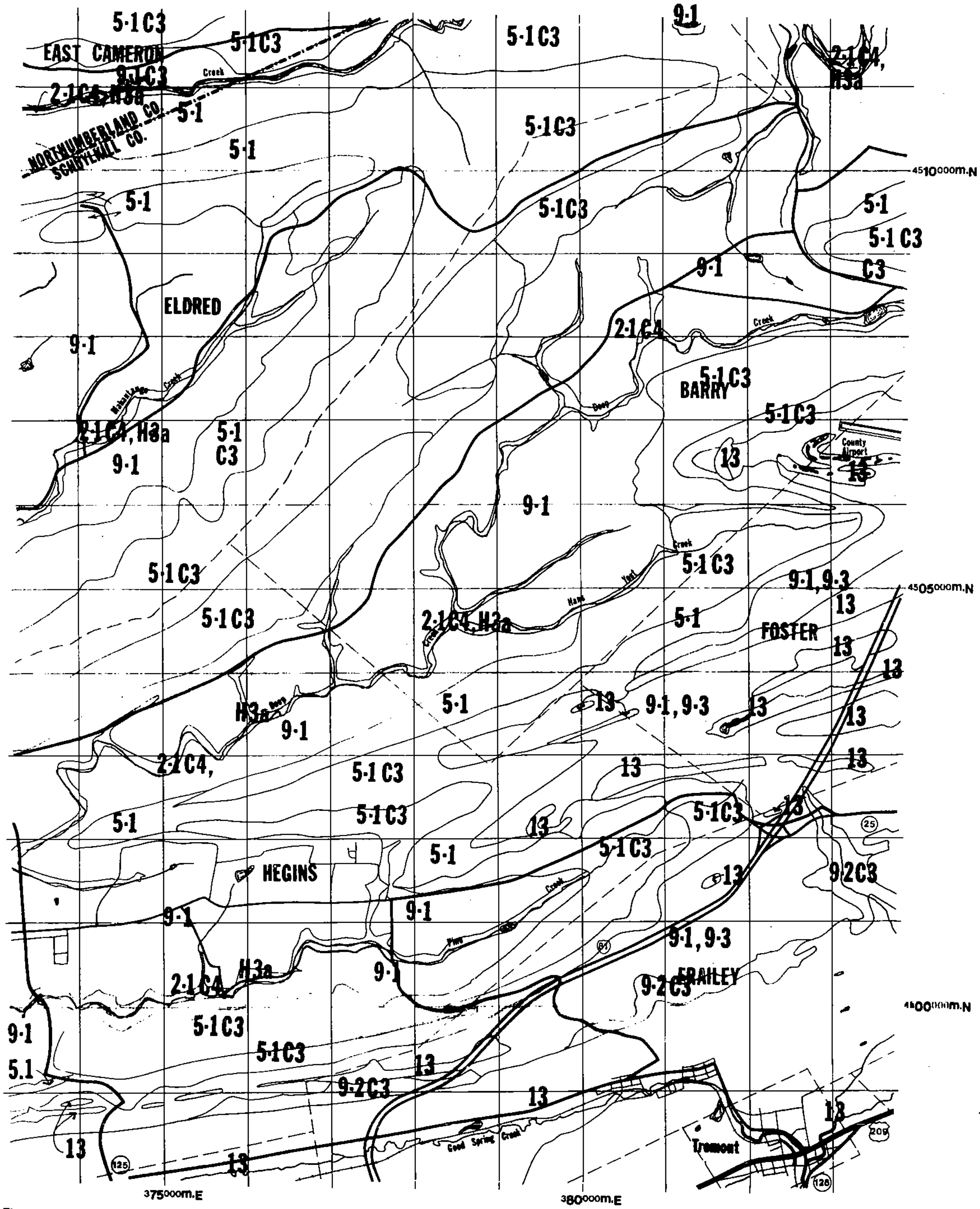

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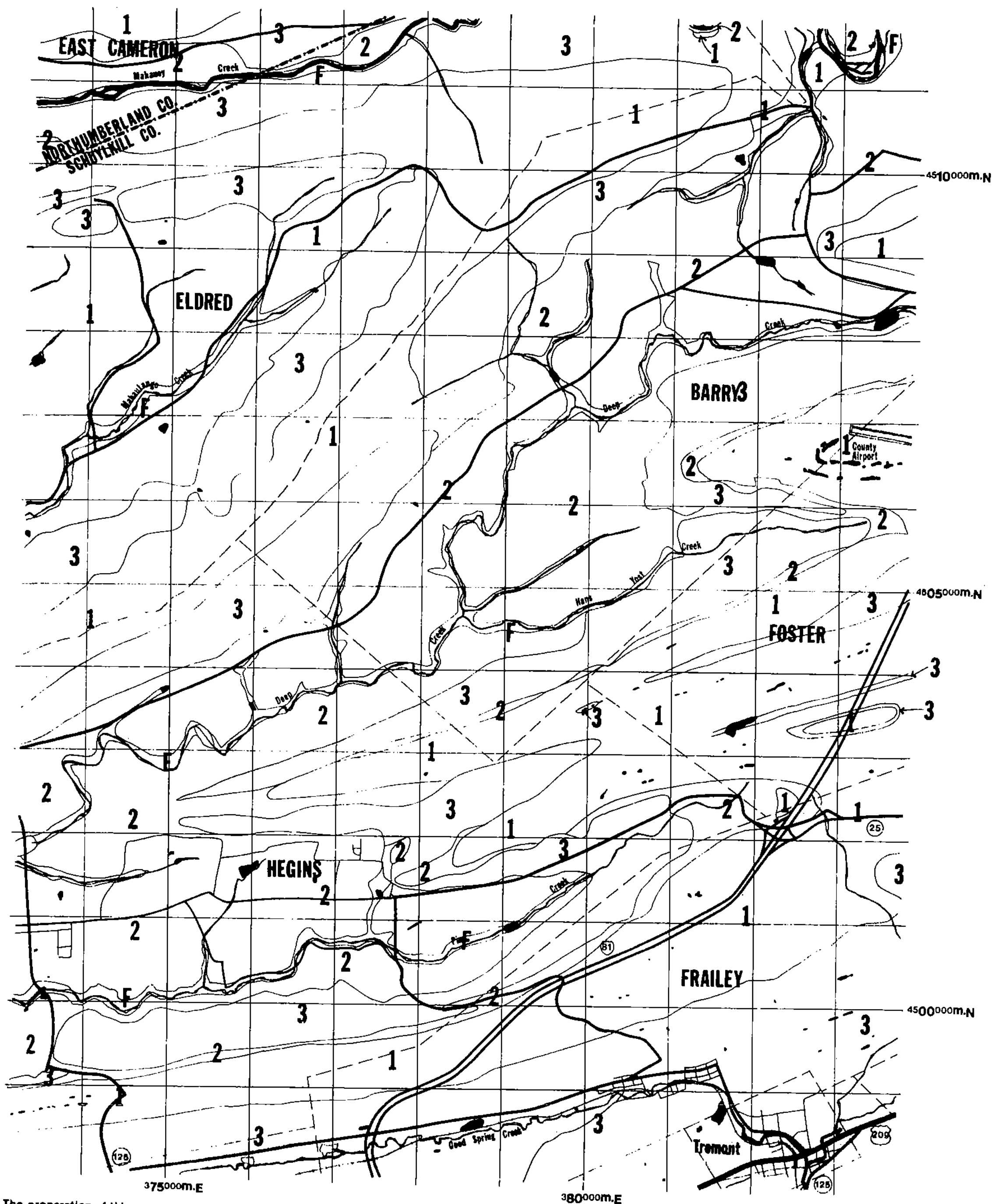

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Base map preparation by Wilbur Smith and Associates.
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Final presentation by Wilbur Smith and Associates.

TREMONT, PENNSYLVANIA

HYDROLOGY

ECONOMIC DEVELOPMENT COUNCIL OF NORTHEASTERN PENNSYLVANIA

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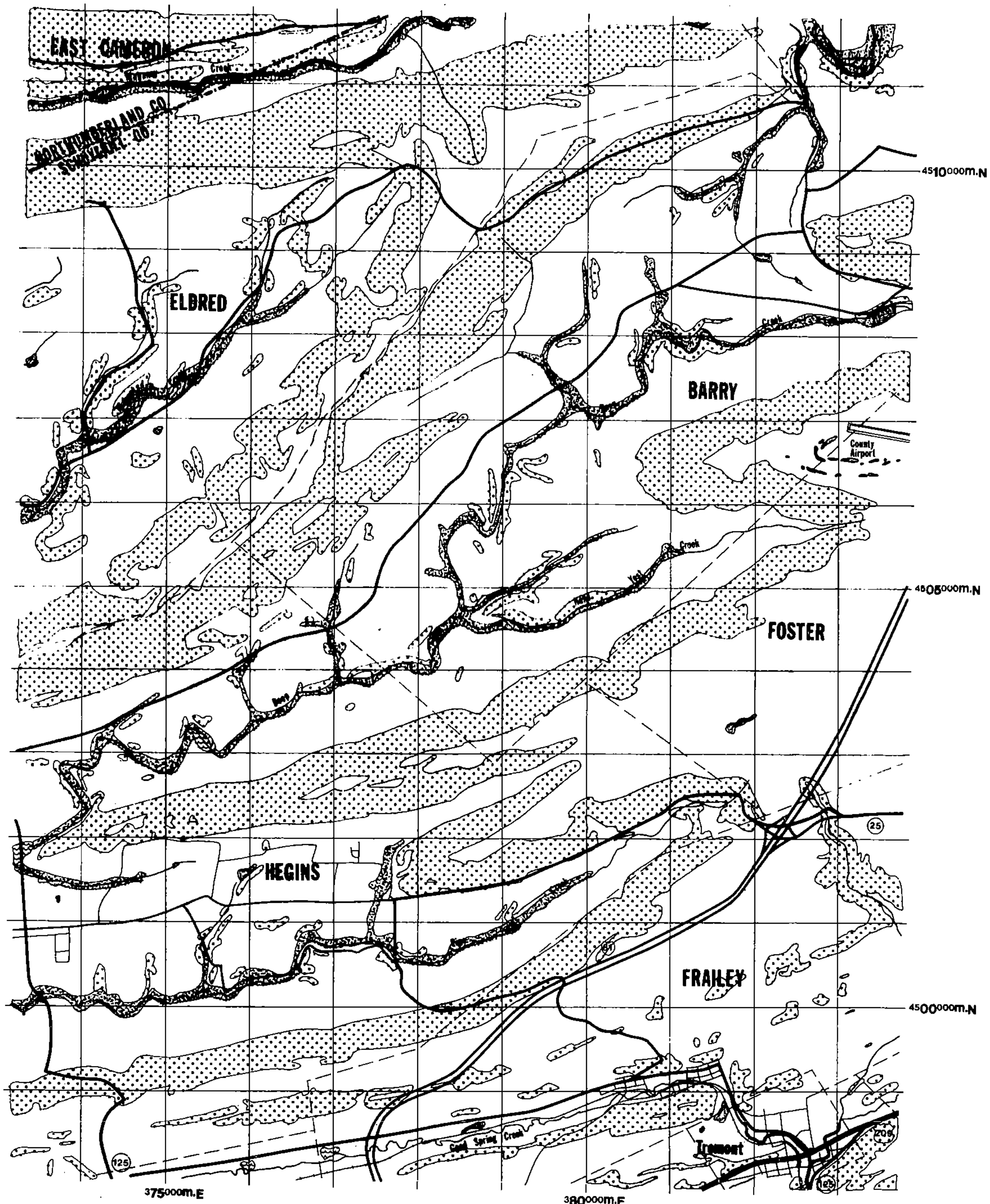

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TREMONT, PENNSYLVANIA

TOPOGRAPHIC AND SURFACE DRAINAGE CONSTRAINTS

ECONOMIC DEVELOPMENT COUNCIL OF NORTHEASTERN PENNSYLVANIA edcnp

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Local soil series, including depth to bedrock, unified soil classification, soils saturated permanently or seasonally within three feet of the surface, percolation rates of key soils strata, potential hazards, and constraints to planned development.

Hydrology showing primary, secondary and some lower order drainageways, flood plains, low flow of streams, and groundwater potential.

Local topographic and surface drainage constraints overlay showing slopes in excess of 25%, floodplains and high water table areas.

The intelligence compiled on these individual overlays for each demonstration area was derived from a variety of sources, including the review, evaluation and synthesis of existing reports and maps; the interpretation of photo maps and aircraft and spacecraft stereoscopic photography. Selected 1:9,600-scale photo maps, obtained through Pennsylvania Power and Light Company, provided a valuable data source for defining geologic conditions in portions of the Honesdale and Pocono Pines quadrangles. Conventional stereo air photos of 1:40,000-scale, acquired through ASCS, greatly assisted in the geologic and soils interpretation of Honesdale, Pocono Pines and Pittston-Avoca. In addition, selected stereo frames of Skylab high resolution earth terrain camera color infrared photography also assisted in the synoptic interpretation of sub-regional physiographic and geologic conditions in portions of Honesdale and Pocono Pines, as well as in the Tremont quadrangle.

Local Slope Characteristics. Prior to discussing the slope characteristics of a quadrangle, it was necessary to identify some practical limitations of local slope as they relate to planned uses and/or activities. Table 4.1 shows some optimum ranges of slope for representative urban and related local uses and activities.

Local Soils. The characteristics of each of the representative soil series shown on the mapping, including depth to bedrock, depth to seasonal water table, internal drainage, percolation rate, erodibility, unified soil classification, suitability as source for sand and gravel, and representative DER Soil Group, are summarized in Table 4.2, for the Honesdale quadrangle.

SUMMARY

As every paper should have a closing statement, if not earth shaking conclusion, one may be related to this presentation that is rather short. However, in actuality that briefness represents significance.

Table 4-1

OPTIMUM RANGES OF SLOPES FOR VARIOUS LOCAL LAND USE ACTIVITIES

	LOCAL MAPPING CATEGORIES (Percent Slope)					
USE OR ACTIVITY	0-3	3-8	8-15	15-25	25-35	Over 35
Recreation Areas	X	X	X	X	X	X
Engineered Structures*	X	X	X	X	X	X
General Urban Uses	X	X	X			
All-Weather Roads	X	X				
Septic Systems	X	X	X			
Sanitary Landfills	X	X				
Conventional Housing	X	X	X			
Commercial Centers	X	X				
Interstate Highways	X	X	X	X		
Airports	X					
Railroads	X					
Tracked Vehicle Operations	X	X	X	X	X	X

(adapted from Mayberry, 1972)

*Dams, bridges, tunnels, pipelines, transmission lines, etc.

Table 4-2

PERTINENT CHARACTERISTICS OF SOILS - HONESDALE QUADRANGLE

Map Symbol	Soils	Depth to Bedrock	Depth to Seasonal Water Table	Internal Drainage	Percolation Rate (in/hr)	Erodibility	Unified Soil Class.	Source for Sand and Gravel	DER Soil Group
1-2 1-3	Lordstown Oquaga	1.5-3.5 ft	3+	Well-Drained	0.60-2.0 2.0-6.0	Low	GM SM,GM,ML	Unsuitable	5
2-3	Chenango	8+	4+	Well-Drained	>6.0	Low	GW,GP,GM,SM	Good	1
3-2	Morris	5+	0.5-2.0 ft	Poor	<0.20	Medium	GM,SM,ML,CL	Unsuitable	15
3-3	Lackawanna	5+	3+	Well-Drained	<0.20	Low	GM,GC,ML	Unsuitable	7,10
4-1 4-4	Oquaga Lordstown	1.5-3.5 ft*	3+	Well-Drained	2.0-6.0 0.60-2.0	Low	SM,GM,ML GM	Unsuitable	5

*Note: The 4-1 and 4-4 soils may tend to be less shallow than the 1-2 and 1-3 soils.

No new knowledge was gained, other than some small localized refinements in formation and structure. No new procedural concepts were developed, for this methodology has been around for some time, although Skylab imagery has been handy only recently.

But it was all put together into one viable, concise, understandable and applicable package for a 4,500 square mile region. It not only contributes to the immediate support of the concurrent regional land use strategy and intermodal transportation studies, but remains available as a continued reference to the planner, engineer, administrator, political representative and decision maker. It spells out, with neither technical oppression or childish simplicity, the why's and wherefore's of the geologic environment, how they interact and so relate to inhibiting or enhancing the use of the land for man's benefit.

In a way it is a pioneering effort. The Hartford quadrangle study by USGS has been around for several years, but it only presents graphic data evaluation. It is understood that a similar study is being conducted by the USGS for a larger area over the front range in Colorado. It is known that the Maryland Geologic Survey did a mapping of the Bowie quadrangle similar to Hartford, but again little analyses, evaluation and application beyond the data presentation. Undoubtedly, several other state geology surveys have done similar studies, but one could wonder to what extent. Then there is the CARAT study about the Washington, D. C. area from ERTS imagery; however, this is again just data collection and definition.

We, as well as others, have executed numerous environmental geology studies for large land areas, but they were usually oriented to a specific purpose such as new town creation, recreational adaptation, environmental sensitive limitations to a contemplated use, etc. Also, such studies were limited in area coverage up to thousands of acres or tens, and occasionally hundreds, of square miles. To our knowledge, this is the first such study, conducted by private enterprise for such a regional area of thousands of square miles, considering a broad usage interaction to the inventoried data and defining hazards, constraints and suitabilities. In other words, more than just defining what is there but what should, or should not be done with it.

Those interested in obtaining a copy of the completed report may write Economic Development Council of Northeastern Pennsylvania, P. O. Box 777, Avoca, Pennsylvania 18641.

USE OF WOVEN PLASTIC FILTER CLOTH AS A REPLACEMENT FOR GRADED ROCK FILTERS

By

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ABSTRACT

Graded rock filters must be designed with a gradation and permeability compatible with the protected soil. In areas of varied geologic conditions, the task of designing an effective and suitable filter for each geologic zone or soil change becomes expensive and nearly impossible to achieve. Filter rock with the "unique gradations" required are often expensive to obtain and difficult to place without segregation and contamination.

From recent Forest Service experience, cost effective, reliable ground-water drainage systems can be built using woven plastic filter cloth and highly permeable open graded drainage layers. Compared to conventional graded rock systems, this combination can be used to establish adequate drainage using less engineering and soil testing, less perforated pipe, less construction quality control testing, less rigid inspection, and by utilizing commercially available aggregate gradations. Recent extensive testing at the Corps of Engineers, Waterways Experiment Station, has provided the necessary information to design and specify reliable drainage installations using woven plastic filter cloth.

This paper discusses the use of woven plastic filter cloths as a replacement for graded rock filters. The cloths are available in a range of equivalent sieve opening sizes for filtering cobble to silt sizes.

INTRODUCTION

The necessity of using properly designed and constructed, graded filters for subsurface drainage has long been recognized by soils engineers and engineering geologists. Although reliable filter design criteria have been available since 1948 (12), most low volume road drainage installations have not used drainage rock meeting these criteria. Filters meeting the established criteria are not being used for several reasons, two primary reasons being the many gradations required to satisfy the frequent soil changes and the expense and difficulty of commercially obtaining the designed gradations in small quantities.

The woven plastic filter cloths discussed in this paper are available from several manufacturers in a range of equivalent sieve opening sizes to match most soil conditions. One or two cloth opening sizes will protect most soils encountered on a project. The open drainage rock used with the cloth is more permeable than graded filters and is readily available from commercial sources. The decision to use graded aggregate filter or plastic filter cloth is usually determined by construction feasibility and economics.

Physical Properties

The plastic filter cloth discussed in this paper is a "uniform sheet of plastic mono-filament yarn woven into a uniform pattern with distinct and measurable openings---shall be calendered so that the yarns will retain their relative position with respect to each other (9)." Typical specifications (9,10) require that the plastic yarn consist of any long chain synthetic polymer composed of 85% by weight of propylene, ethylene, or vinylidene-chloride, and contain stabilizers and/or inhibitors added to the base plastic to make the filaments resistant to deterioration due to ultraviolet and/or heat exposure.

Successful filters allow a small portion of the finest grains of the protected soil to move into the filter, causing a highly permeable filter to develop adjacent to the graded filter. If the filter allows too many fines to pass, the filter will plug and become impermeable. The other condition of the filter being too fine and stopping all movement of soil fines may trap the very fine soil at the filter boundary causing surface clogging.

The Army Engineers Waterways Experiment Station (2,3) tested 7 plastic filter cloths. The cloth specifications, physical properties, and design criteria contained in this paper are a result of those tests. Of the cloths tested, two of the cloths were non-woven or woven with yarns so closely that neither cloth had any distinct visible openings. These two cloths clogged with fine soil during the research, leading to the conclusion that "only woven filter cloths with distinct openings are recommended, based on the clogging of cloths E and F during the clogging tests." Based on this testing and the filter concept stated previously, only woven filter cloths will be discussed in this paper.

Cloths tested by the Corps of Engineers which meet the Forest Service Specification, are listed in Table 1. The physical and strength requirements of the specification are shown in Tables 2 and 3, respectively (10).

Requirements of a Filter and Drainage System

A drainage system must meet two conflicting requirements:

1. Piping Requirement - The pore spaces in drains and filters that are in contact with erodible soils and rocks must be small enough to prevent most particles from being washed in or through them.

2. Permeability Requirement - The pore spaces in drains and filters must be large enough to impart sufficient permeability to permit seepage to escape freely and thus provide a high degree of control over seepage forces and hydrostatic pressures. (4)

These conflicting requirements, when combined with the rapidly changing soil conditions encountered in mountainous road construction, are difficult and expensive to satisfy using graded aggregate filters.

TABLE 1. Cloths Tested by Corps of Engineers that Comply with Forest Service Specification 6-47 (10)

Manufacturer or Fabricator	Trade Name	E.O.S. Sieve No.	Percent Open Area	Abrasion Resistance
Carthage Mills, Inc.	Filter X	100	4.6	Low
Erosion Control Div.	Poly-Filter X	70	5.2	High
Cincinnati, OH 45216	Poly-Filter GB	40	24.4	High
Advance Construction Specialties Co. 1050 Texas Street Memphis, TN 38106	Erosion Control Fabric (Type I)	100	4.3	High
Erco Systems, Inc. P.O. Box 4133 New Orleans, LA 70118	Nicolon 66411	30	36.0	Low

Notes: E.O.S. is "equivalent opening size", and is defined as the number of the U.S. Standard Sieve having openings closest in size to the filter cloth opening.

"Percent Open Area" is defined as the summation of the open areas divided by the total area of the filter cloth.

For "High" Abrasion Resistance, the strength loss after testing shall not exceed 70 percent and the abraded strength must be no less than 100 lbs. in the stronger principal direction and 55 lbs. in the weaker principal direction.

TABLE 2. Minimum Physical Requirements for Plastic Filter Cloth (10)

Test	Minimum Strength % of Unaged Tensile Strength
Alkali Treatment	90
Acid Treatment	90
Low Temperature Treatment	85
High Temperature Treatment	80
Oxygen Pressure Treatment	90
Freeze Thaw	90
Weatherometer	65
	Test Result
Brittleness	No failures at -60°F
Weight Change in Water	Less than 1.0%

TABLE 3. Minimum Unaged Strength Requirements for Plastic Filter Cloth (10)

Cloth Type	Pretested Cloths	Stronger Principal Direction (Tensile, lb)	Weaker Principal Direction (Tensile, lb)	Burst (PSI)	Puncture (lb)	Seam Breaking (lb)
AB	Poly-Filter X Poly-Filter GB Erosion Control Fabric	200	200	510	125	195
C	Nicolon 66411	180	100	250	65	90

Aggregate filters can be either a graded filter meeting the criteria in Table 4 or a zoned filter having several layers graded from fine adjacent to the soil to coarse at the perforated pipe. Each layer of the zoned filter is designed to be compatible with adjacent filter zones and/or soil. Filter criteria for filter cloths are based on the "equivalent opening size" (EOS) and "percent open area." The design criteria in Table 5 permits the designer to select a filter cloth that retains the soil being protected, yet permits drainage and prevents clogging. Open graded aggregates are used with the cloth to rapidly remove the water. Reliable gradation charts similar to Figure 1 are available for estimating aggregate permeabilities (4,5,6,8).

TABLE 4. Requirements for Filter Materials (12)

Character of Filter Materials	Ratio R_{50}	Ratio R_{15}
Uniform grain-size distribution (U = 3 to 4)	5 to 10	-
Well graded to poorly graded (non-uniform); subrounded grains	12 to 58	12 to 40
Well graded to poorly graded (non-uniform); angular particles	9 to 30	6 to 18

$$R_{50} = \frac{D_{50} \text{ of filter material}}{D_{50} \text{ of material to be protected}} \qquad R_{15} = \frac{D_{15} \text{ of filter material}}{D_{15} \text{ of material to be protected}}$$

Notes: If the material to be protected ranges from gravel (over 10% larger than No. 4 sieve) to silt (over 10% passing No. 200), limits should be based on fraction passing No. 4. Maximum size of filter material should not exceed 3 in. Filters should contain not over 5% passing No. 200. Grain-size curves (semi-logarithmic plot) of filter and of material to be protected should be approximately parallel in finer range of sizes.

TABLE 5. Requirements for Filter Cloth (9)

Equivalent sieve opening and percent open area for filter cloth should be based on the following criteria:

a. Filter cloth adjacent to granular materials containing 50 percent or less by weight fines (minus No. 200 material):

(1) $\frac{\text{85 percent size of soil (mm)}}{\text{opening size of EOS (mm)}} \geq 1$

(2) Open area not to exceed 36 percent

b. Filter cloths adjacent to all other type soils:

(1) EOS no larger than the openings in the U.S. Standard Sieve No. 70

(2) Open area not to exceed 10 percent

Notes: To reduce the chance of the cloth clogging, no cloth should be specified with an open area less than 4 percent or an EOS with openings smaller than the openings of a U.S. Standard Sieve Size No. 100. When possible, it is preferable to specify a cloth with openings as large as allowable by the criteria. However, because of strength requirements and the limited number of cloths available, it may not be possible to obtain a suitable cloth with the maximum allowable openings.

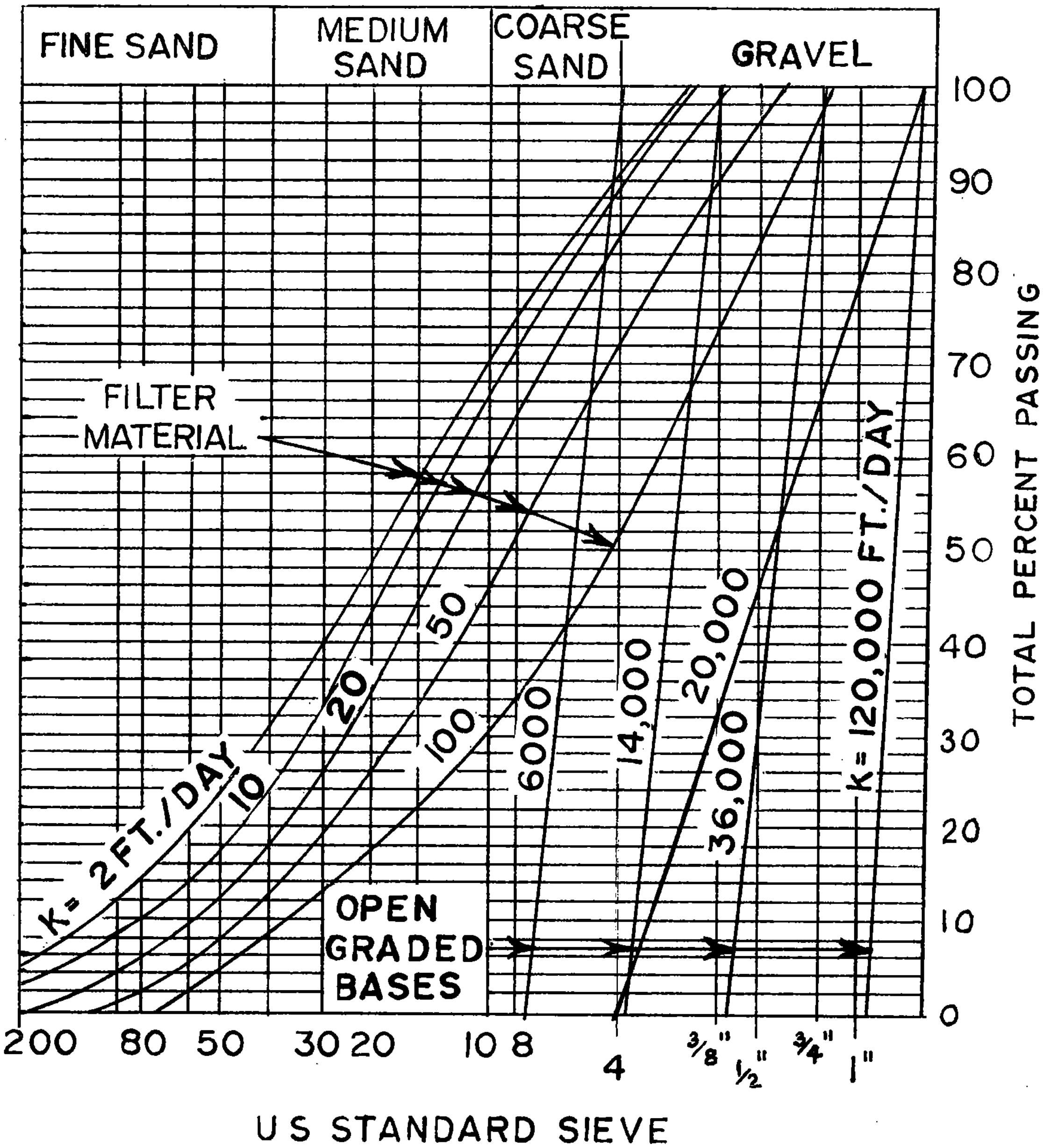

FIGURE 1. Typical Gradations and Permeabilities of Several Open-graded Aggregates and Several Filter Materials. (5,6)

For graded filters, it is well established that "close control is required in the production, handling, and placement of the materials, since even a single improperly constructed portion of a filter can lead to failure." (4) The filtration properties of filter fabrics are "built-in" in the factory and calendered to prevent enlarging of the openings in the field. Field experiments and accelerated weathering tests (1,2,3,7) have shown that the plastic filter cloths retain their strength after long periods of exposure to fresh and salt water. Stones weighing 150 to 200 pounds can often be dropped on loose fabric from heights of 2.5 to 4.5 feet without damaging the fabric (2), indicating little danger of damaging the fabric during rock placement. Unlike graded aggregate filters, the condition of plastic filters can be determined visually after installation and before placement of drain rock.

The high permeability requirement of the filter in a drainage system is often ignored. Most references containing filter criteria discuss briefly that the filter will not clog and will have a higher permeability than the protected soil. Standard plans for road drainage installations often show a narrow trench filled with concrete sand and a perforated pipe, leaving only the depth to be established by the designer. Harry Cedergren states: "Even the most minimal system designed with the help of the rational methods described will have drainage capabilities hundreds of times greater than those of most pavements that have been built in the last 30 or 40 years. Consequently, when these methods are used, even if rough estimates have to be made of inflows, the resulting systems will usually be at least two orders of magnitude better than those designed without an analysis of discharge needs." (5, pg. 75). The drainage criteria referred to by Mr. Cedergren involves: (1) Darcy's law of flow, (2) Inflow-outflow concept; outflow capabilities of a drainage system must be at least equal to the inflow from all sources, (3) Condition of Continuity; outflow capabilities of the system should increase in the direction of flow, (4) time required for water to flow through pavement-drainage systems and, (5) time required to drain after stop of inflow. Items 4 and 5 apply primarily to pavement drainage.

Graded filters for fine soils and concrete sands shown on "standard plans" usually have low permeabilities. "Standard Plan" drainage aggregates which do have high permeabilities are usually rendered ineffective by clogging. The permeability of graded filters are often too low which limits the area of influence of the drainage system and the rate of water removal. The writer has seen several projects where subdrains installed to remove ground water have actually decreased slope stability by acting as reservoirs due to inadequate outflow capacity.

Figure 1 shows the relationship between gradation and permeability for drainage materials. "Filter Materials" shown in Figure 1 are representative of the gradations required to protect the clayey and silty soils encountered in the Northwest. Like concrete sand, they have permeabilities of 20 to 100 feet/day. The "Open Graded Bases" shown are compatible with woven filter cloths. Flume test to determine head losses for the cloth alone yielded head losses of 0.5 foot at water velocities of 0.1 foot per second for the finest fabrics tested. The gradation selected for use with filter cloth will depend on commercial availability and permeability requirements of the design.

Installations with graded filters and concrete sand usually require a perforated pipe to rapidly drain the collected water. High permeability open graded aggregates used with plastic filter cloths often have adequate flow capacity to eliminate perforated pipes for the first 30 to 100 feet of normal subdrains. Additionally, the open graded aggregate and plastic filter cloth combination allows the use of thinner and more hydraulically efficient drainage layers.

CONSTRUCTION PRACTICES

For maximum benefit, the plastic filter cloth is placed between the soil and drain rock. The commercially available fabric wrapped perforated pipes are not suitable for most installations because (1) the fabric does not meet the specification discussed in this paper and (2) fabric placed around the pipe does not protect the granular drain rock.

Drain rock used with the plastic filter cloth can be from pea gravel to rip rap size, as long as it is protected from damage during construction. During construction the fabric is placed loosely so it will not be stretched during loading. Placement of rip rap sized material may need a granular cushion to prevent damage to the cloth.

Plastic filter cloth is suitable for use in subdrainage trenches, blankets, behind rock buttresses, under rip rap, around piezometers, around vertical gravel drains and many other applications where graded filters are normally used. The high abrasion resistant cloths (Table 1) should be specified for abrasive applications such as under river or lake shore rip rap.

Figure 2 shows some typical details for installation of plastic filter cloths. Vertical rock drains with horizontal drain outlets have been used successfully to remove groundwater in the layered glacial sands and silts in Seattle, Washington. Vertical rock drains protected by plastic filter cloth (Figure 2f) can be installed placing gravel in cloth tubes inserted in hollow stem augers or cased drill holes. Photographs of actual installations are shown in Figures 3, 4, and 5.

COSTS AND RELIABILITY

Woven plastic filter material costs are \$0.17 to \$0.25 per square foot delivered, depending on the cloth specified, and the quantity. Installation costs are \$0.07 to \$0.25 per square foot, depending on difficulty

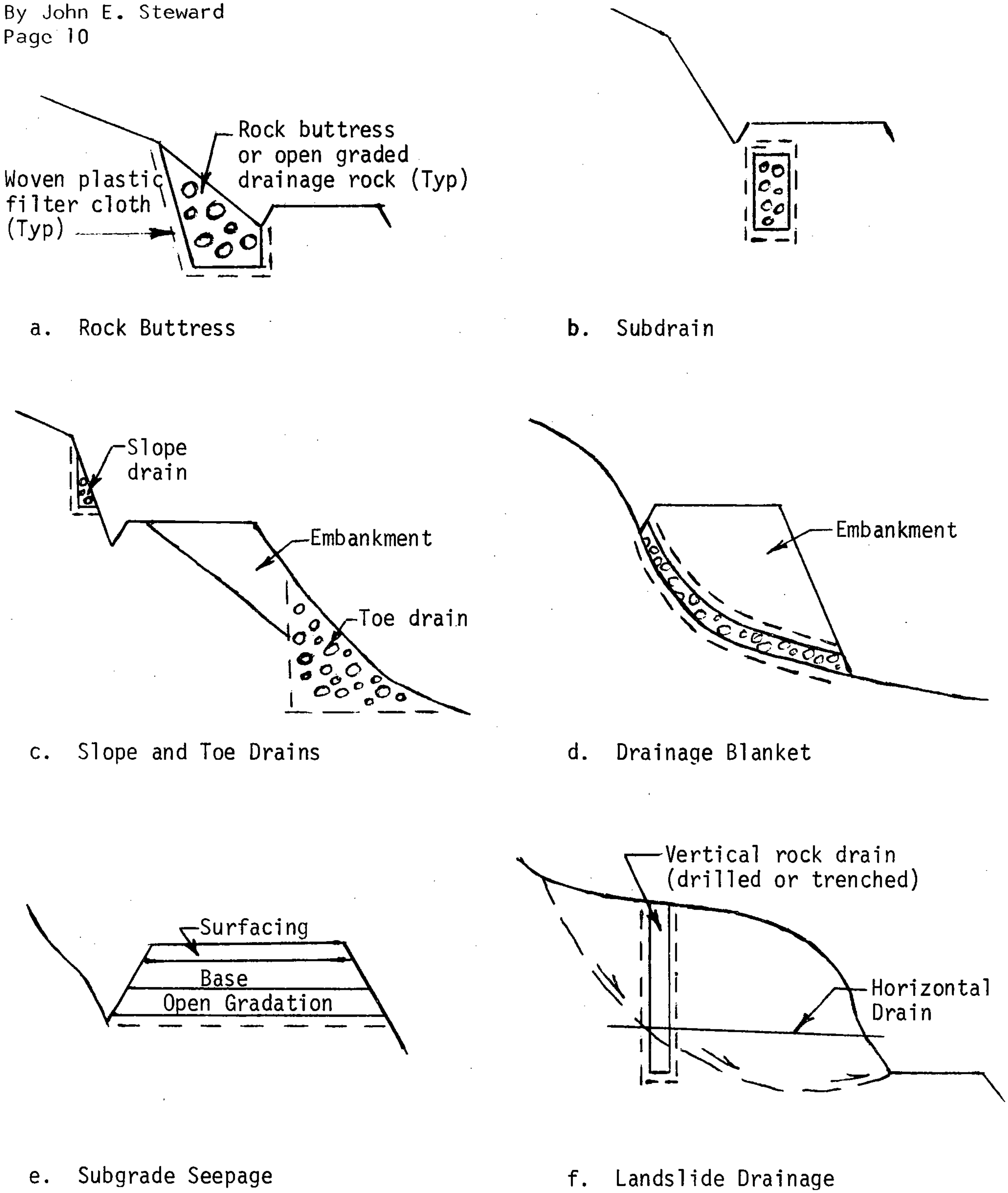

FIGURE 2. Typical Plastic Filter Cloth Installations

FIGURE 3. Woven Plastic Filter Cloth Installed in an Excavated Trench

FIGURE 4. Subdrainage Trench with Woven Plastic Filter Cloth, Open Graded Aggregate and Perforated Pipe Nearing Completion

a. Excavation

b. Installation

FIGURE 5. Drainage Blanket to Control Seepage
Under a High Earth Fill

of the project and the experience of the contractor. Drain rock manufactured during the drushing or screening process (i.e. material retained on the 3/8 or 1/2 inch screen) will cost little more than the base or surface rock being produced.

Two or three separate filter gradations per mile are normal for Forest Service roads built in western Oregon and Washington. Gradations designed to protect the soils encountered are often "bastard" gradations, not satisfied by any of our rock gradations or gradations available from local aggregate or concrete plants. When available, the aggregate costs range from $6.00 to more than $30.00 per cubic yard (11). In addition to aggregate costs, soil sampling, testing, and filter design costs will exceed $100.00 per soil change.

First cost of subdrains using woven plastic filter cloth, open graded aggregate, and perforated pipe often appear higher than installations without the filter cloth. Subdrainage costs must also include the feasibility of matching the filter to the soil and obtaining and installing the designed filter without contamination or segregation. Graded aggregate filter installations on Forest Service projects probably have less than a 50 percent chance of functioning properly. The changes of success have been necessarily low because of frequently changing soil conditions, nonavailability of specified gradations, and limited sampling, testing, and inspection capabilities. Installations using woven plastic filter cloth should have a success rate near 100 percent, even with limited testing and inspection.

SUMMARY

Soil engineers have long known that effective permanent groundwater drainage requires a highly permeable filter and drainage system that rapidly collects and removes water without clogging. Although adequate design criteria have been available for over 25 years, most drainage installations have been a compromise with the ideal due primarily to the cost of sampling, testing and design of filters to match rapidly changing soil gradations and to the difficulty of procuring and installing the designed filters. The resulting installations often fail due to plugging, clogging, or have inadequate permeabilities to rapidly remove the collected water.

Plastic filter cloths discussed in this paper are available in a range of sizes to filter a wide range of soils without clogging. Commercially available open graded aggregates are used with the cloth to provide an economical and hydraulically efficient drainage system. Criteria are presented to guide the user in designing and constructing highly reliable drainage systems.

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AGGREGATE RESOURCE CONSERVATION IN URBAN AREAS*

By

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ABSTRACT

Future utilization of aggregate resources in urban areas must be considered during early planning stages of an urban complex. This is of particular importance because urban land values are commonly greater than the mineral value of these low unit value materials and the use of the land for construction pre-empts exploitation of the aggregate materials. In addition, zoning restrictions may also prohibit resource exploitation of urbanized land. Sequential land use practices which allow the development of resource areas prior to urban encroachment can result in more efficient mineral production and lower consumer costs.

Environmental planning studies in the Austin, Texas area, serve as an example of how urban expansion can affect aggregate resource reserves. Sand and gravel and limestone deposits in this area which were considered as reserves a few years ago are now covered by developments. Furthermore, evaluation of projected aggregate consumption and population trends indicate that substantial quantities of reserves will be lost to future development if current practices prevail. Even though similar materials are present in nearby areas the unit price will be significantly increased by added transportation costs. A comparison of the possible alternatives indicates that transportation costs would be significantly higher than the cost of rehabilitating land after mineral extraction in local areas.

Many important aggregate resource deposits have been engulfed in the urban development of large cities during the past several years. Zoning regulations which promote resource preservation and sequential land use can extend the productive lives of many such deposits.

INTRODUCTION

The preservation of mineral resources within and adjacent to growing metropolitan areas is a necessary part of urban planning and a vital aspect of our mineral industry. Materials suitable for development as mineral resources occur only in certain areas; if these areas are covered by an urban complex, potential economic benefits are lost. Therefore, a knowledge of the distribution of suitable deposits and a plan for their systematic extraction prior to development is essential to obtain the maximum benefit from local resources.

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Aggregate materials are key resources for urban development because they provide the construction materials that are basic components of our roads, commercial buildings, and homes. The five-fold increase in aggregate production in Texas since 1950 (Figure 1) is closely related to the increase in construction (Figure 2) and the population increase (Figure 3). The relative importance of aggregates located in or near urban areas is also emphasized by the fact that suitable deposits in remote areas are commercially less attractive than deposits near developed areas. Transportation cost is of course the basic reason that production areas of these low unit value resources are located as near as possible to high demand areas. Even in cases where market areas are near production areas, transportation cost is commonly equal to or greater than the value of the aggregate material at the source.

Although sand and gravel and crushed stone are important resources, the relatively high real estate values, stimulated by rapidly expanding urban areas, have often caused potential mineral lands to be used for other purposes.

The Austin area of Texas is presented in this paper as an example of how urban growth can affect the availability and cost of local resources and what can be done in metropolitan centers to incorporate exploited areas into urban development. Conditions described herein are not unique to Austin or Texas.

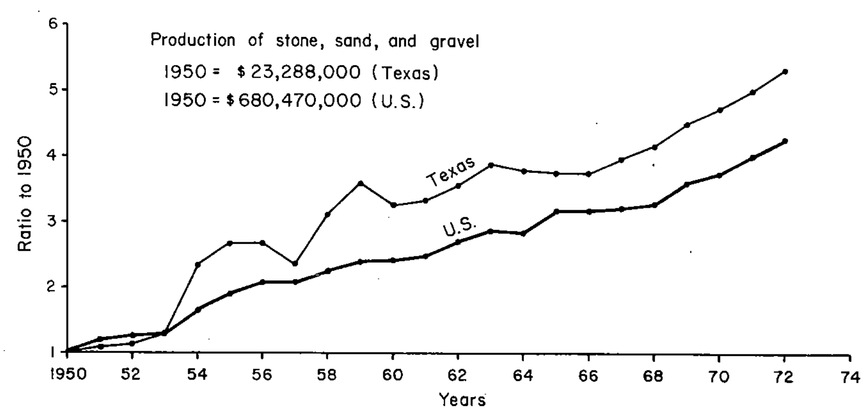
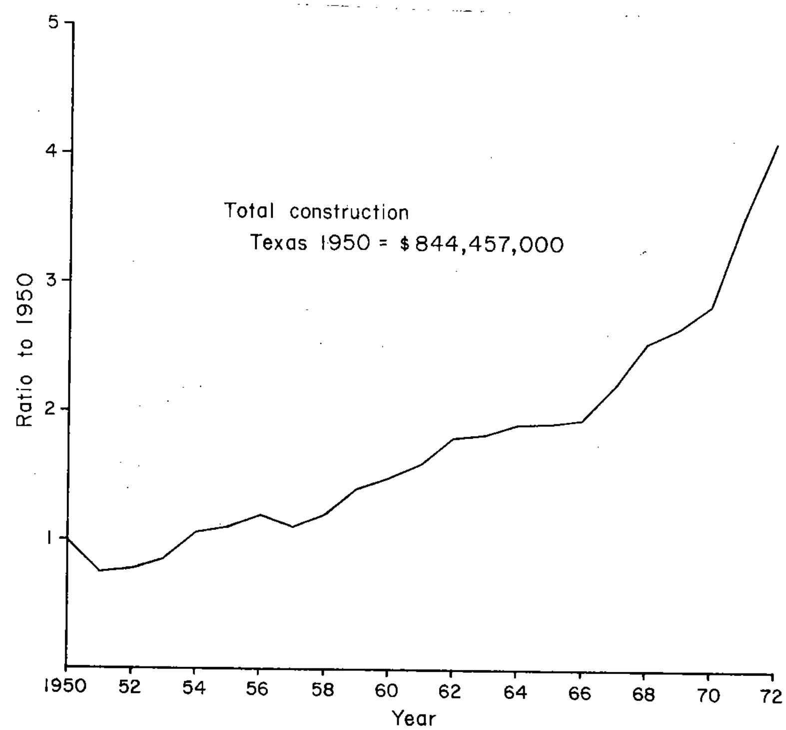


FIGURE 1. Aggregate Production in Texas and United State, 1950 to 1972 (based on U.S. Bureau of Mines, Minerals Yearbook statistics).



FUGURE 2. Total Construction in Texas, 1950 to 1972 (based on statistics from Bureau of Business Research, The University of Texas at Austin).

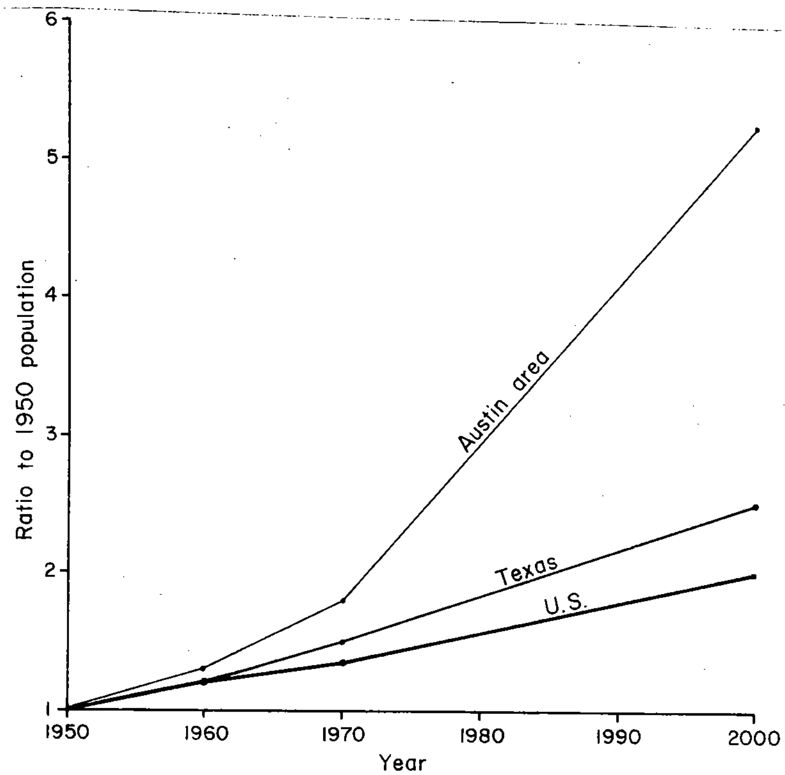


FIGURE 3. Population Growth, 1950 to 1970, and Projected Population Growth, 1970 to 2000, for Austin area, Texas, and the United States (based on statistics from Population Research Center, The University of Texas as Austin).

GROWTH

Approximately half of Austin's areal growth (Figure 4) has taken place in the past 20 years (Figure 5). This correlates with population growth which has also almost doubled in the same period (Figure 3). Even if growth is slowed somewhat, Austin's population is expected to reach or exceed one-half million during the 1980's. The growth rate for Austin (Figure 3) is several times larger than the rates for Texas and the United States. If these projected rates prove to be accurate, Austin's population in the year 2000 will be more than five times greater than the 1950 population.

The rate of development for the 1960-1970 period was approximately 2 square miles per year (Figure 5). At present population densities, this rate must increase to about 5 square miles per year to accommodate Austin's projected population in 2000.

AGGREGATE RESOURCES

Aggregate resources in the Austin area are available from limestones and alluvial materials which have been described in detail by Rodda and Fisher (1966) and Garner (in preparation).

Limestone reserves, suitable for crushed stone, occurring in the Austin area prior to urban expansion (Figure 6) totaled about 6 billion tons. Approximately 80 percent of this total is within five miles of existing railroads. As of 1970, about 350 million tons of original reserves had been eliminated from the reserve list by aggregate production and urban encroachment.

Sand and gravel deposits containing materials suitable for aggregates cover an area of about 120 square miles in the Austin vicinity (Figure 6). Estimated reserves for this area were originally about 1 billion tons. Production and urban development has eliminated about 125 million tons from the reserve list as of 1970.

The quantity of potential aggregate materials covered by urban development for the period ending in 1970 totaled about 475 million tons. A survey of abandoned pits and quarries within the urban area indicates that only about 10 percent of these aggregate materials had been extracted for use. Therefore the loss of potential resources due only to urban encroachment is about 425 million tons. At current prices, the total lost mineral value amounts to more than $600 million.

CONSUMPTION PATTERN

The current statewide average consumption for construction materials is about 7 tons per capita (3 tons per capita for sand and gravel and 4 tons per capita for crushed stone). The national consumption rate for aggregate materials is about 9 tons per capita. Projected consumption rates for the

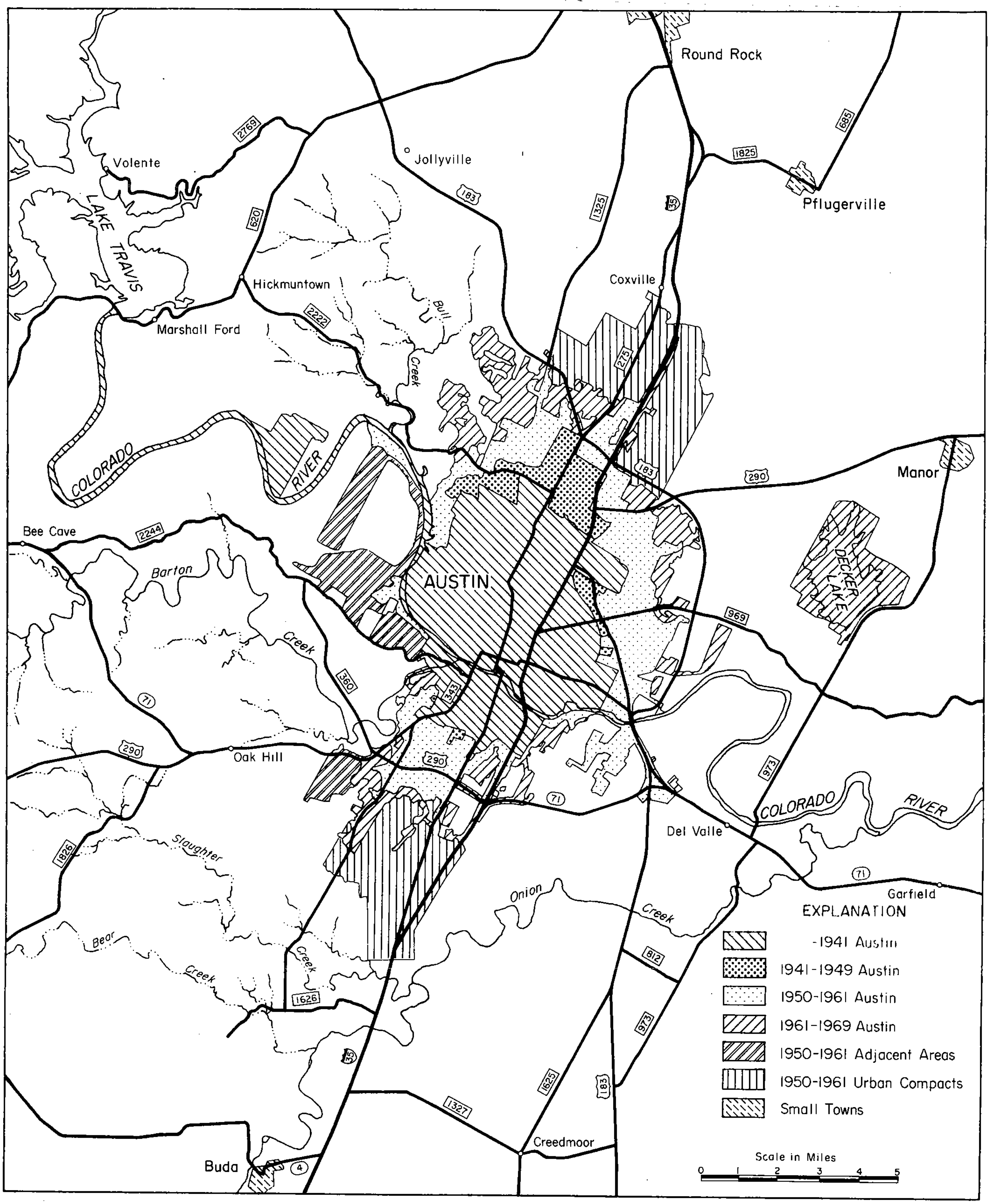


FIGURE 4. Urban Growth of the Austin Area, Texas, 1940 to 1970

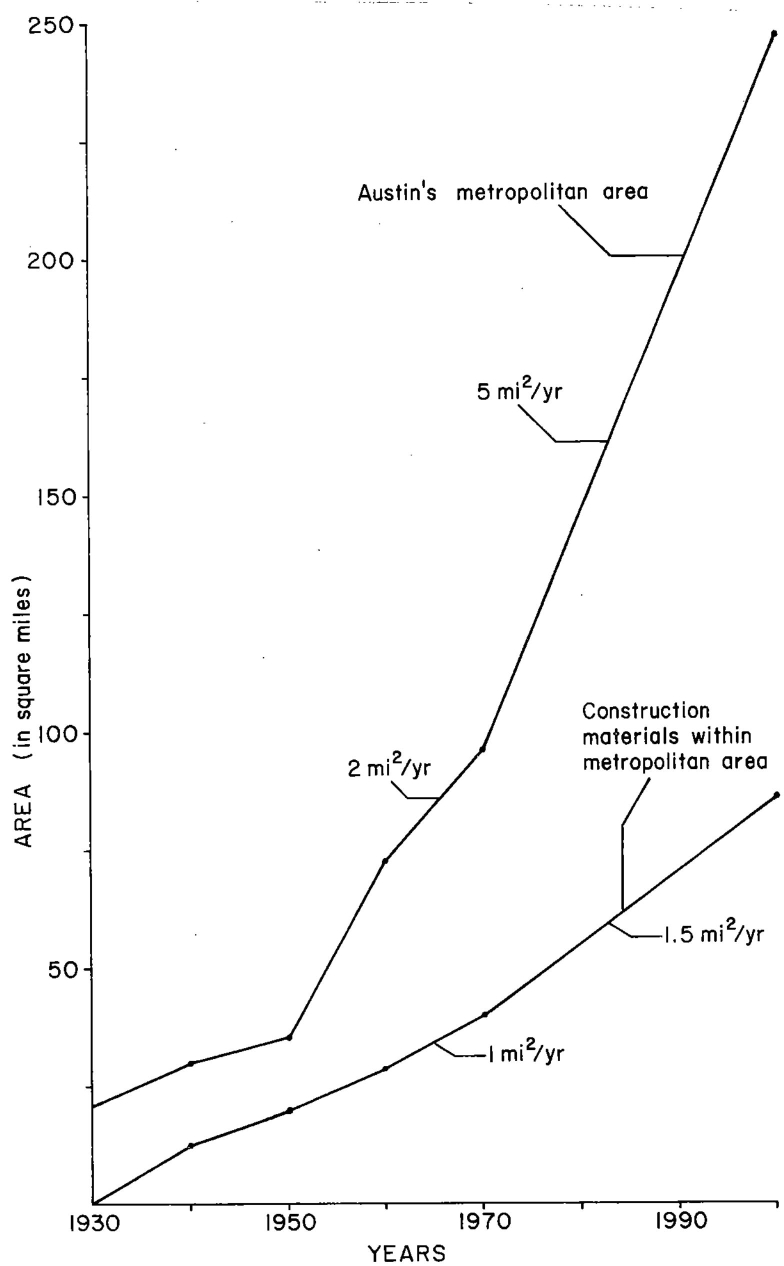


FIGURE 5. Rate of Urban Areal Growth and Rate or Urbanization of Potential Construction Materials, Austin Area, Texas

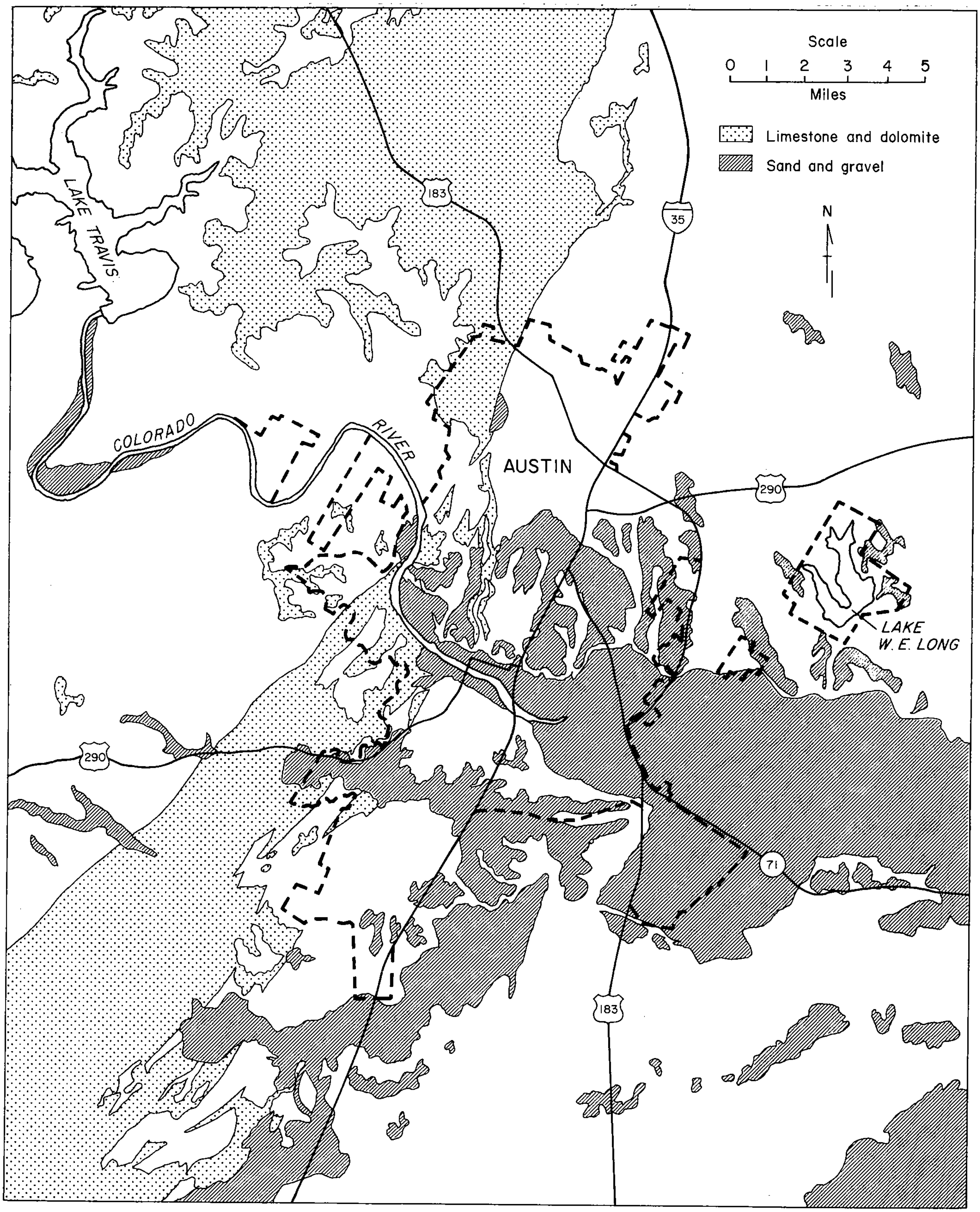


FIGURE 6. Aggregate Materials in the Austin Area, Texas

year 2000, based on forecast figures from the U.S. Bureau of Mines Mineral Facts and Problems (1970) and the U.S. Census Bureau, are about 18 tons per capita. If a constant ratio is maintained between Texas and United States total aggregate production, as indicated in Figure 1, the rate of aggregate consumption in Texas will be about 12 tons per capita in 2000.

RESERVES AND URBAN GROWTH

One can observe from the above array of numbers that Austin is not about to run out of aggregate material resources. By applying the current consumption rate to Austin's population of about 300,000 it is obvious that literally hundreds of years of reserves are present. This is true even at the projected rate of 12 tons per capita and the projected population of 850,000.

The problem with calculating reserves strictly on a numerical basis is that the area consumed by urban growth is not considered. Unless conservation measures are taken, urban development will probably obscure many times the amount of aggregate material actually produced. Based on current development patterns, urban growth (5 square miles per year) would consume approximately 1.5 square miles per year of construction materials (Figure 5), compared to a 1 square mile per year for the 1960 to 1970 period.

At the average rate of 1.5 square miles per year and assuming a production of about 50,000 tons per acre, urban growth would engulf almost 1.5 billion tons of construction materials by the year 2000 or about 25 percent of current reserves. At today's prices, this would be a mineral value loss of about $2 billion.

The problems resulting from increased urban growth are not limited to loss of aggregate resources. The total impact must also consider the interrelation of urban growth, resource conservation, and consumer cost. Evaluation of these relationships requires the consideration of present and projected haul distance, transportation cost, and consumption rate.

The average haul distance at present is about 10 miles. It is estimated that the increase in urban area will cause at least a 50-percent increase in the average haul distance by year 2000, bringing the total distance to 15 miles.

Transportation costs are currently approximately $0.04 per ton mile. Even before the energy crisis, some authorities were predicting a 100-percent increase in haul costs by year 2000. This would raise transportation rates to $0.08 per ton mile.

In Table 1, the current and projected values which contribute to the average annual per capita haul cost are listed. Currently this total value is $2.80. Projected values for the year 2000 result in an average annual per capita haul cost of $14.40 which is an increase of more than 400 percent. In the event that transportation cost is increased by only 50 percent, the average annual haul cost would still be increased by about 280 percent to $10.80.

TABLE 1. Current and Projected Transportation Cost for Aggregate Materials

Year	Average Haul Cost (per ton mile)	Average Haul Distance	Annual Per Capita Consumption (tons)	Average Annual Per Capita Haul Cost
1970	$0.04	10	7	$ 2.80
2000	$0.08 $0.06	15 15	12 12	$14.40 $10.80

As mentioned previously, these problems are not peculiar to the Austin area. A similar evaluation by the California Division of Mines and Geology (Bulletin 198, 1973) indicates that the average annual per capita haul cost in that State will increase from $10 in 1970 to $54 in year 2000. They predicted a statewide loss of construction materials for the 30-year interval of almost $17 billion, based on current development practices.

RESOURCE CONSERVATION

There are alternatives to the dim picture painted in the foregoing paragraphs. The most likely of these alternatives if the implementation of sequential land use practices to exploit mineral land prior to urban development. Although conservationists have suggested the utilization of sequential land use for many years, the economic stimulus may ultimately be the factor which causes its acceptance as a normal conservation practice. For example, in California it was estimated that about 90 percent of the predicted $17 billion construction material loss could be saved if conservation practices known and available in 1972 were rigorously applied. The estimated cost of this loss-prevention for the period from 1970 to 2000 would be about $90 million, giving a benefit to cost ratio of 170 to 1.

Fairfax County, Virginia pioneered this approach to resource conservation in 1961 when it enacted zoning ordinances to preserve land for resource development. Much of the land originally set aside has now been exploited. Completion of the project is scheduled for 1976. In 1973, Colorado passed a statute which requires a study of available mineral deposits and preservation of commercial deposits in counties having a population greater than 65,000. Sand and gravel and quarry aggregates were specifically designated. This action was prompted in part by rapid urban development in the Denver area several years ago, which radically reduced available sand and gravel resources. Vermont has also enacted a law which limits urban development of resource areas, and several other states are currently working on legislation of similar statutes.

Sequential land use practices can easily be incorporated into urban master plans. The initial reaction is that reclamation of mining sites is expensive and costs may be prohibitive. However, when reclamation of pit areas is compared to increased haul distance and higher transportation cost, it is apparent that conservation can be profitable.

An investigation of surface mining in Texas (Groat, in preparation) shows that the cost of fully reclaiming sand and gravel mining areas (including grading, revegetation, and fertilization) ranges from \$335 to \$635 per acre. Recovery of dredging operations and simultaneous reclamation during extraction using company-owned equipment is less expensive (about \$335 per acre), while secondary recovery of land after extraction is most expensive (about \$635). The cost for reclaiming crushed stone quarries is assumed to range somewhat higher than for sand and gravel pits because of the indurated nature of the materials. Reclamation cost may in some difficult situations range as high as \$1000 per acre.

For the purpose of calculations in this article, the highest reclamation cost is assumed (\$1000 per acre). Therefore, an aggregate yield of \$50,000 tons per acre would require an increase in price to the consumer of only \$0.02 per ton to offset producer overhead.

In order to see the economic benefit of sequential land use, compare the projected values for haul costs and haul distance in year 2000 (Table 1).

Haul distance x Haul cost = Total cost

15 miles x \$0.08/ton mile = \$1.20/ton

If the average haul distance could be reduced by 3 miles through the implementation of sequential land use,

Haul distance x Haul cost = Total cost

12 miles x \$0.08/ton mile = \$0.96/ton

savings in haul cost would be \$0.24 per ton. Thus, the consumer cost would be decreased by a net total of \$0.22 per ton. This would result in decreasing the annual per capita cost by \$2.64, a significant saving. Increased land values associated with reclaimed mine sties will result in additional savings.

ENVIRONMENTAL ASPECTS OF SEQUENTIAL LAND USE

The exploitation of mineral resources located within or adjacent to an urban area has many unsavory aspects. Environmental problems commonly associated with sand and gravel or crushed stone industries are stream pollution by excess sediment, air pollution by dust, land disturbance, increased traffic, noise, and general unsightliness. Objections stem primarily from the economic necessity of locating extraction and plant sites as close as possible to consumers.

Little can be done about some problems such as noise and traffic in the immediate area of mining sites. Advance planning can delay the development of nearby residential areas until extraction is complete. Vegetation and landscaping can also be used to screen pit and quarry operations, and stream and air pollution can be reduced or eliminated by trapping sediment and dust.

The most appealing aspect of sequential land use is that once mineral extraction is complete and the land is reclaimed the site is available for other uses. Therefore, it not only provides for the most effective mineral conservation but also enables property owners to obtain maximum benefit from their land.

CONCLUSION

The rapid growth of urban centers has caused a re-evaluation of many concepts about lifestyles and economic development. Increased consumer demand, increased transportation cost, and the loss of mineral reserves are causing a keener awareness of current and future resource-related problems. The finite nature of resources is realized today more than ever. Alternatives illustrated in previous sections show how resource availability and cost can be affected when development is not adequately planned.

It is possible that economic factors related to rapid urban expansion will serve to retard the development rate and the attendant price increases. In this case, the foregoing projections may be much higher than actual conditions. These circumstances would not, however, negate the premise that planned resource development can result in many benefits to both producers and consumers.

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PETROGRAPHY AS RELATED TO POTENTIAL SKID RESISTANCE OF PAVING AGGREGATES USED ON TEXAS HIGHWAY PROJECTS

By

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ABSTRACT

In the past few years petrographic analyses have become an integral part of a number of studies concerning the understanding of polishing and skid-resistant properties of paving aggregates used on Texas highway projects. Laboratory and field studies show that mineral composition and micro-texture, and to lesser degrees, grain size and particle shape have the greatest influence on the polishing behavior of both natural and synthetic aggregates on pavements with known traffic densities. Extensive laboratory testing of rock samples from commercial and non-commercial aggregate sources throughout Texas indicates that lightweight and vesicular synthetic aggregates, sandstones, sandy carbonates, rhyolites and vesicular igneous rocks have the highest skid resistance potential. Highly textured, fossiliferous and impure limestones, basalt, granite and some crushed river gravels are intermediate in skid resistance potential. Well-rounded siliceous river gravels and high purity, dense limestones and dolomites have the least potential for providing long-lasting skid-resistant pavement surfaces.

INTRODUCTION

Skidding accidents and slippery pavements are still of major concern to safety minded highway designers. For years it has been recognized that some pavements tend to polish and be less skid-resistant than others in the same length of time. Even under similar traffic volumes it has been observed that certain paving aggregates will polish more rapidly than others. In recent years this observation has led to special efforts in developing methods to effectively measure the polishing behavior of aggregates.

One measuring technique involves rating the surface of a section of pavement with a skid trailer. It has been shown that the values obtained by this method are greatly influenced by a number of factors including total traffic applications, construction practices and aggregate characteristics. The other method of evaluating an aggregate's polishing behavior is a laboratory test which provides a relative quantitative measure of frictional properties exhibited by the aggregate (Patty, 1973).

The use of the skid trailer in evaluating pavement surfaces has been in effect since the early 1960's while the accelerated polish test on specific paving aggregates had limited application from about 1970–73 and wide use since early 1974. Petrographic analysis has been conducted on a routine basis on essentially all aggregates proposed for use on paving projects since 1970. Rock classification has provided a basis for understanding the polish behavior and skid-resistance potential of paving aggregates.

LABORATORY AND FIELD STUDIES

Approximately 300 sources for paving aggregate, including commercial and non-commercial pits, have been examined petrographically and tested for relative polishing characteristics in the past five years. A few sources are located in neighboring states.

The laboratory evaluation of aggregates for polish values conducted by the Materials and Tests Division consists of testing production samples in accordance to Test Method Tex-438-A, "Accelerated Polish Test for Coarse Aggregate." The accelerated polishing utilizes a British Accelerated Polishing Machine (Figure 1) and the frictional measurements are made with a British Portable Tester (Figure 2). Representative samples of the aggregate to be tested are cast into small coupons using a polyester bonding agent (Figure 3).

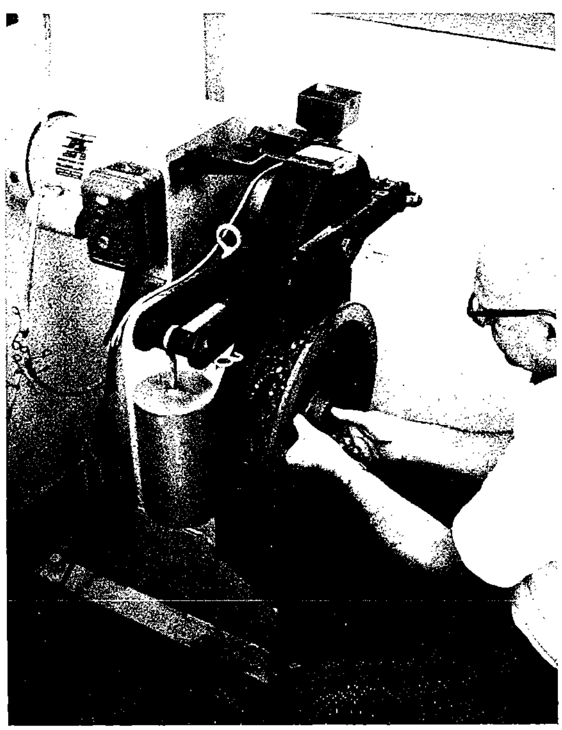

FIGURE 1. British Accelerated Polishing Machine with aggregate coupons clamped on wheel. Water and silicon carbide abrasive are applied for the 9-hour test.

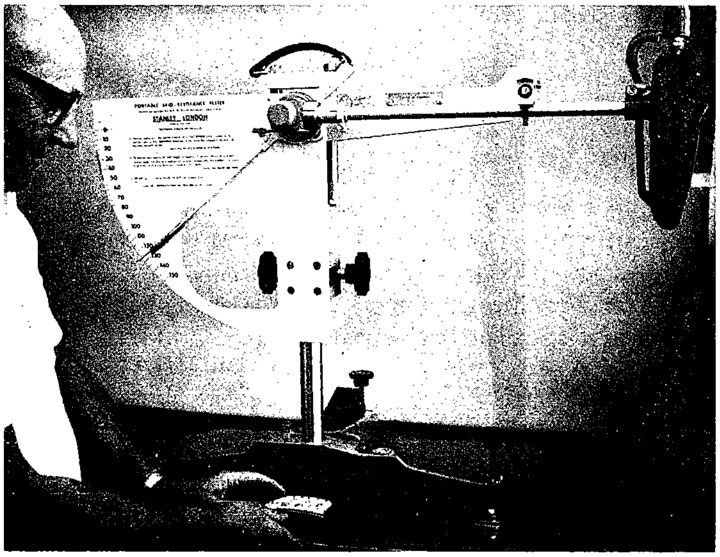

FIGURE 2. Frictional measurements are taken after the polishing test by means of the British Portable Tester. The average measured reading for an aggregate is referred to as the polish value.

FIGURE 3. Fabrication of aggregate test coupons requires close placement of the particles in a steel mold and casting with polyester. Seven coupons representing production material are tested.

Synthetic aggregates have also been examined and many types have been used on highway projects over the past 15 years. Although about 20 different sources of synthetic materials have been tested for polishing properties, the major commercial sources include expanded shale, burned clay and blast furnace slags.

Several skid trailers capable of measuring the relative frictional properties of pavements have been operated by the Department for several years. A fairly comprehensive survey of pavement surfaces is currently being evaluated and constantly updated. When a reliable sample of surfaces is examined which contains a specific aggregate type, their polishing or skid-resistance behavior is found to be a function of composition and the number of traffic applications.

RESULTS

For convenience, the sources which include both natural and synthetic materials have been grouped into 5 major aggregate categories; namely, (A) Synthetics, (B) Sandstones, (C) Igneous, (D) Carbonates and (E) Gravels (See Table I).

Petrographic studies have shown that considerable variation exists in mineral composition, hardness, porosity and grain size within any given aggregate category. These properties in turn affect microtexture and susceptibility to abrasion. All of these properties and, to some extent, particle shape have influence on the polishing behavior and skid resistance as measured by laboratory and field techniques. The range of polish values for the various types of aggregates is also shown in Table I.

DISCUSSION

The implementation of the accelerated polish test to paving aggregates is recent years has resulted in the accumulation of extensive data concerning the polishing characteristics of aggregate materials from hundreds of sources in Texas and nearby states. Petrographic analysis shows that several properties including mineral composition, grain size, porosity and hardness have the greatest influence on microtexture. The microtexture of an aggregate and to some extent particle shape is what determines its relative polish value and its ultimate potential for providing skid-resistant pavement surfaces. Data collected over several years of field testing with the skid trailer show correlations between the ability of an aggregate to reflect desirable skid resistance and the results of the accelerated polish test. For example, aggregates with relatively high polish values tend to provide a relatively high skid resistance within a given range of traffic applications. It has long been recognized that certain aggregates tend to polish and lose their relative skid resistance much sooner than other aggregates under the same traffic conditions.

TABLE I

POLISH VALUE RANGE

20 25 30 35 40 45 50 55

AGGREGATE GROUPS

----SYNTHETICS----

---SANDSTONES--

----IGNEOUS-----------

-------CARBONATES--------

-----GRAVELS------

The accelerated polish test shows that the lightweight synthetic aggregates (expanded shales, clays and fly-ash products) measure consistently high frictional properties with polish values generally in the 40's and low 50's. Skid trailer testing indicates that these materials have a very high potential for providing skid-resistant pavement surfaces. The vesicular microstructure of the particles upon exposure provides the apparent frictional property. Similar findings indicate that sandstones also show consistently high polish values in the laboratory tests and offer desirable skid-resistant pavements. (Figure 4).

FIGURE 4. Close-up view of vesicular texture of synthetic "expanded shale" aggregate. The relatively-high polish value range is indicative of a high skid-resistance potential. (Magnification 7X)

Field observations and skid tests on pavements containing crushed rhyolite reflect good skid resistance. Likewise, the polish test on rhyolites is acceptable. Trap rock basalts measure only marginal polish values; however, because of hardness and particle shape, field behavior generally is acceptable.

The carbonates (limestones, dolomites and caliche) show the greatest range of variation both as measured by the polish test as well as by the pavement skid tests. The high-purity, dense limestones and dolomites have very low polish values. Skid-trailer tests also indicate that these types tend to polish quickly and lose skid resistance in just a few months under high traffic. Under the same traffic density, the impure, sandy limestones and sandy caliches tend to maintain a higher level of skid resistance. (Figure 5).

FIGURE 5. Close-up of a moderately textured carbonate aggregate with polish values of low to mid-30's. The skid-resistance potential is intermediate. (Magnification 7X)

River gravels, as a group, generally measure very low polish values; however, certain types have been found to exhibit acceptable polish values especially when crushed. Again, the mineral composition is primarily responsible for the properties which affect polishing behavior. (Figure 6).

Current requirements for minimum polish value on federally funded projects are based on traffic density. Roadways having less than 750 vehicles per day are exempt, others may require a minimum of 30 or 33 and those with greater than 5000 vehicles per day require a polish value of 35 or more.

Present aggregate research includes the evaluation of blends, polish rates and the polishing characteristics of bituminous paving mixes.

FIGURE 6. View of uncrushed polished river gravel that exhibits polish values in the low to mid-20 range. Well-rounded gravel show least skid-resistance potential and are generally unacceptable for use on high-traffic pavements where minimum polish values are required.

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ROCK TYPES AND SEISMIC VELOCITIES VERSUS RIPPABILITY

By

Allen D. Bailey
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ABSTRACT

Laboratory and field measurements indicate the maximum rippable seismic velocity is related to the rock's tendency to "relax" upon overburden removal. This tendency appears to be related to the rock's granularity as expressed micro and macroscopically.

TEXT OF PRESENTATION

Crucial to any excavation project is the determination of whether the job can be done by conventional excavation procedures, i.e., (ripping), without blasting, and if blasting is required, at what elevation will it commence. Refraction seismic is a technique which is used to determine the general "soundness" of rock by using the relationship between rock competence and the ability of the earth materials to transmit a seismic wave.

Generally speaking, the seismic velocity of rock is related to the degree of weathering and fracturing and directly related, although not necessarily proportionately, to density and cementation. The velocity is influenced by the total character of the material, with the more prominent features, such as fracturing and weathering the grossly controlling factors. Thus, the earth scientist analyzes the time-distance graphs by relating the characteristics of the rock mass to the seismic data.

A significant factor in rippability analysis is the degree of inelastic deformation (or stress relief) of the rock resulting from overburden removal. It has been the experience of the author that massive rock with strong crystal or particle bonding and a discreet (non-gradational) high velocity interface with the overlying lower velocity materials, will usually not experience significant deformation upon unloading.

The rock that <u>has</u> been found to be susceptable to deformation is one that has one or more of the following characteristics: moderately to well fractured (with a large percentage parallel or sub-parallel to the proposed cut face), low crystal or particle bonding strength, and a gradational decrease or weathering with depth. Laboratory measurement of the compressional and shear waves by Dr. Pradeep Talwani (Talwani, et al, Journal of Geophysical Research, October 1973) of silica sand, volcanic ash and powered basalt, subjected to hydrostatic confining pressures from one atmosphere to 2.5 kilobars (about 11 km of over-burden) resulted in seismic velocities being repeatedly reversible upon pressure recycling even though the porosity was irreversible. This testing indicates a direct relationship between pressure and seismic velocity with the velocity independent of the pressure history of the materials.

A controlled laboratory environment can, and often is, quite different from field conditions, however, the laboratory results confirm the field experience of velocity decrease with pressure reversals (overburden removal) in rock which behaves as a "granular" mass much like the granular materials Dr. Talwani tested. This granular effect is felt to be produced by one or more of the following: weathering, extensive fracturing parallel to the cut face, lack of strong crystal bonding and/or cementation or a combination of these factors. Figure 1 shows the reversal of P and S wave velocities with pressure and the permanent reduction (non-reversal) of porosity which resulted from Dr. Talwani's testing.

The following are three separate rock types encountered in the field, which were analyzed for their response to overburden removal. The first is a metamorphic rock--a composite meta-sediment and metavolcanic of the Motherlode Series in northern California. This rock has been recrystallized due to the metamorphism and has numerous vertical fractures (only a few horizontal). The pre-excavation seismic survey revealed a discrete 10,000 ft./sec. contact at 16-18 ft. depth. Seismic traverses were run before and after a 30 ft. road through-cut. Blasting was required at a 20 ft. depth. Little change was measured in the seismic velocities after excavation, thus, indicating this rock does not readily inelastically deform. Figures 2 and 3 show only a small reduction of seismic velocities upon overburden removal. Additionally, this reduction was probably influenced by blasting during excavation.

The second rock is a weathered granite located in the San Gabriel Mountains of Southern California. This rock is deeply weathered (gradational with depth) and moderately to well fractured by randomly oriented fracture planes. Pre-construction rock velocities as high as 15,000 ft./sec. at depths of 200 ft. was excavated by D-9 Cats with double rippers. It appears the rock, acting as a granular mass, underwent inelastic deformation upon unloading (overburden removal) thus producing lower seismic velocities and allowing economical ripping. Figure 4 shows the pre-construction velocities.

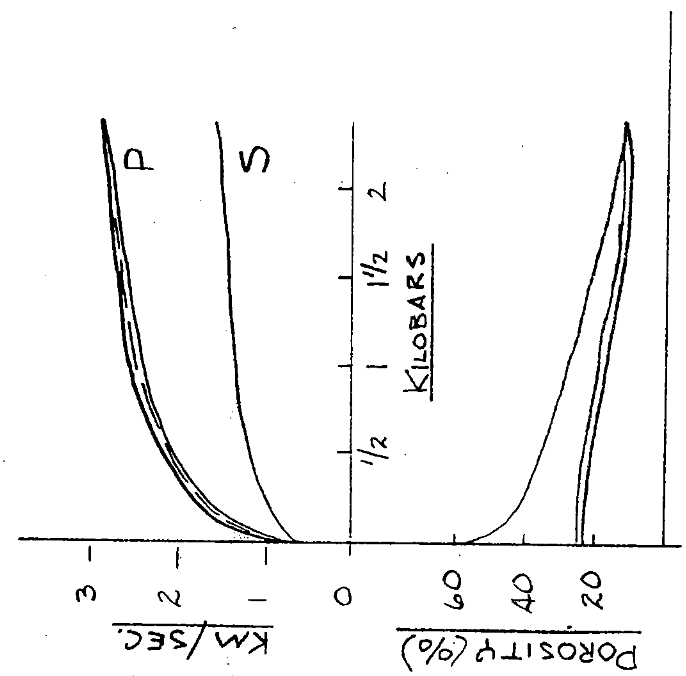
The third rock is a dense Plio-Pleistocene conglomerate encountered during excavation for a canal in Sacramento, California. The rock was seismically surveyed in an existing 20 ft. cut and again at the same stationing a week later after excavating another 20 feet. The second survey measured velocity reductions as shown in Figure 5. This conglomerate was so dense that some cobbles were sheared in two in-place by the excavating equipment, although cementation was slight. The rock, apparently acting as a granular mass, inelastically deformed or rebounded significantly and produced an appreciable seismic velocity reduction in only a weeks time. The contractor at first experienced refusal and very difficult excavating by D-9 and trimmer equipment. By working separate areas and allowing sections of the canal to inelastically deform for a few days he was able to economically excavate by ripping.

POWDERED BASALT

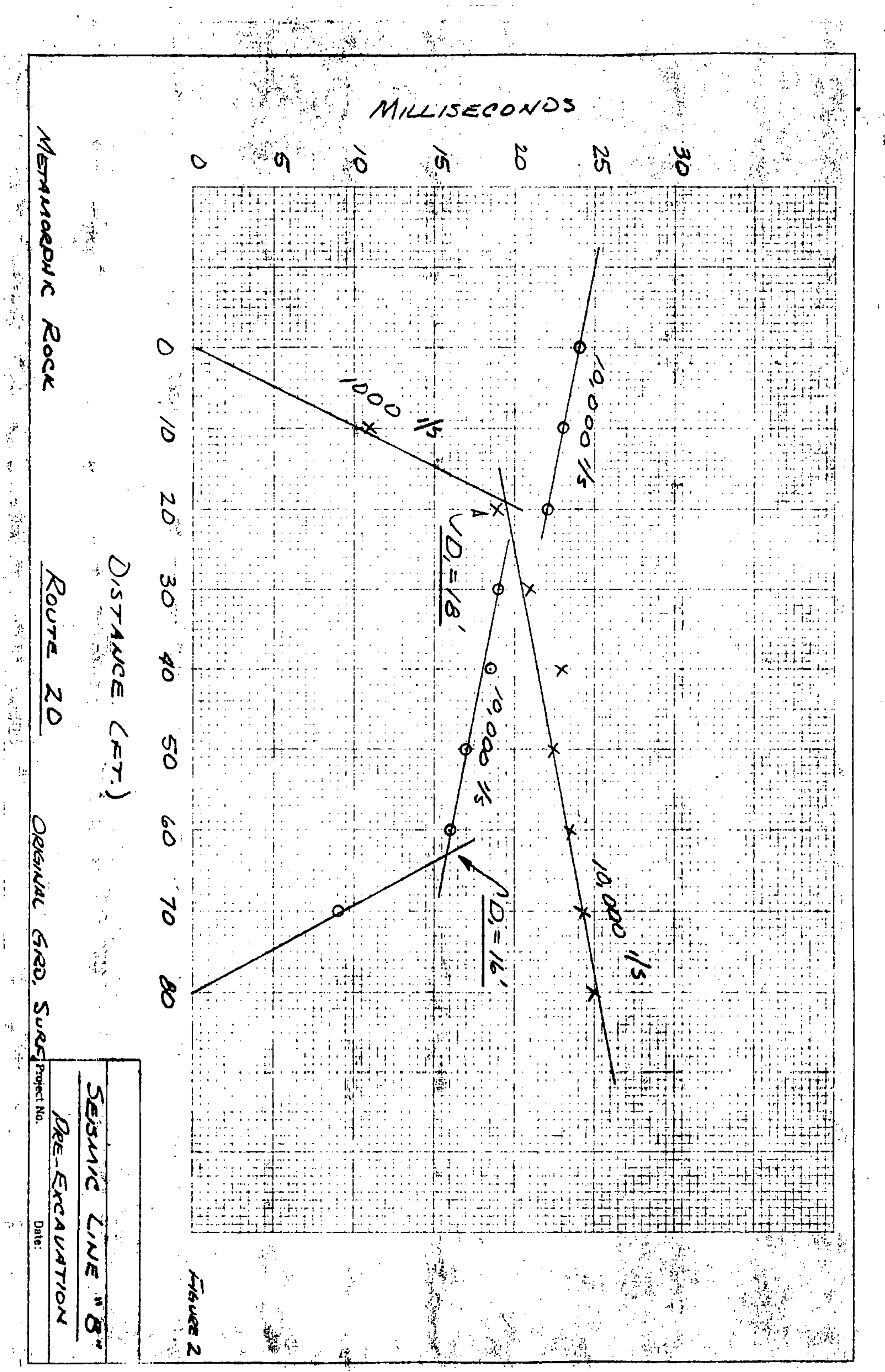
Project No.

Date:

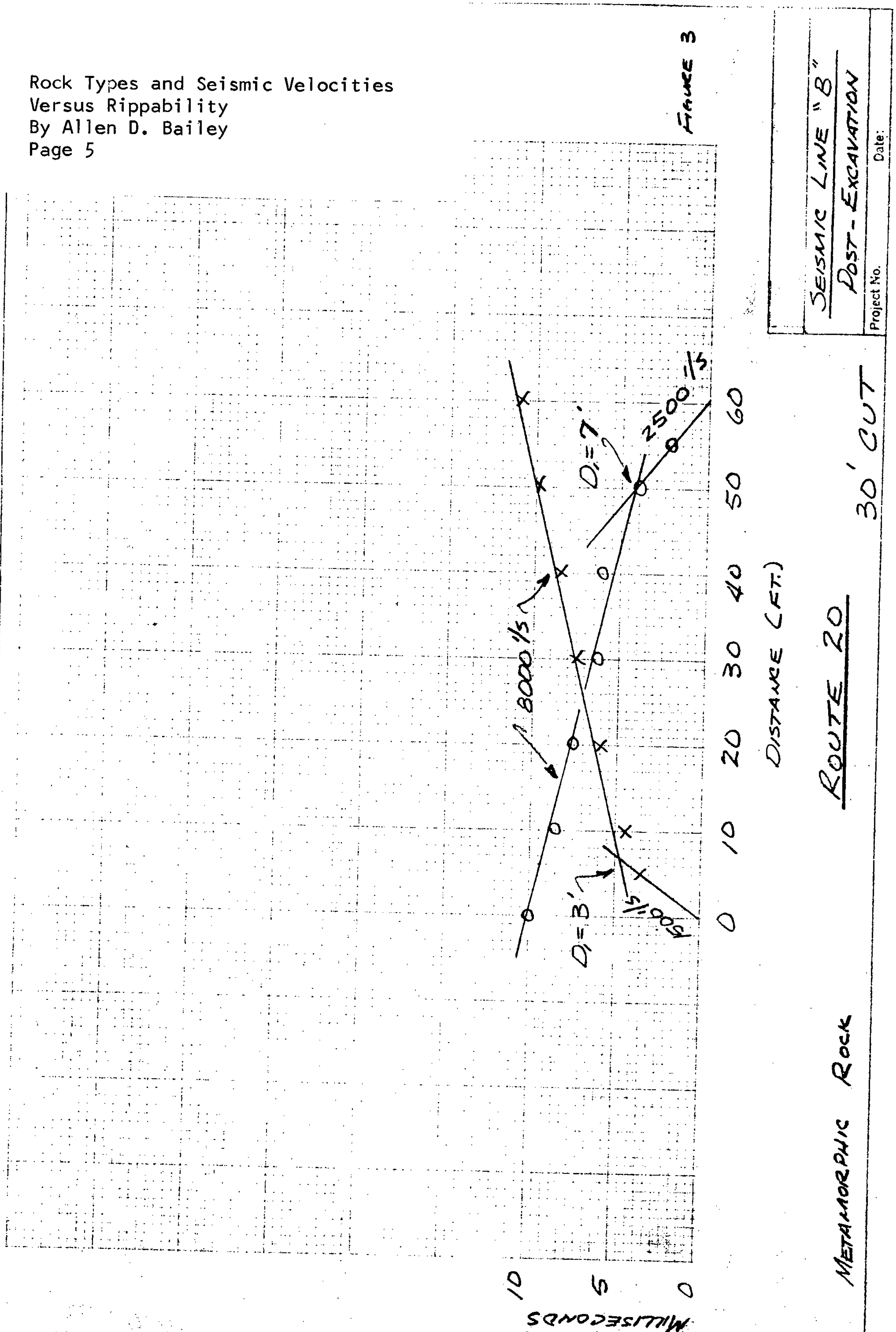
FIGURE 1



REF. – TALWANI


MILLISECONDS
0 5 10 15 20 25 30
DISTANCE (FT.)
0 10 20 30 40 50 60 70 80
1000 '/s
10,000 '/s
10,000 '/s
10,000 '/s
D=18'
D=16'
METAMORPHIC ROCK
ROUTE 20
ORIGINAL GRD. SURF.
SEISMIC LINE "B"
PRE-EXCAVATION
Project No.
Date:
FIGURE 2

Rock Types and Seismic Velocities
Versus Rippability
By Allen D. Bailey
Page 5



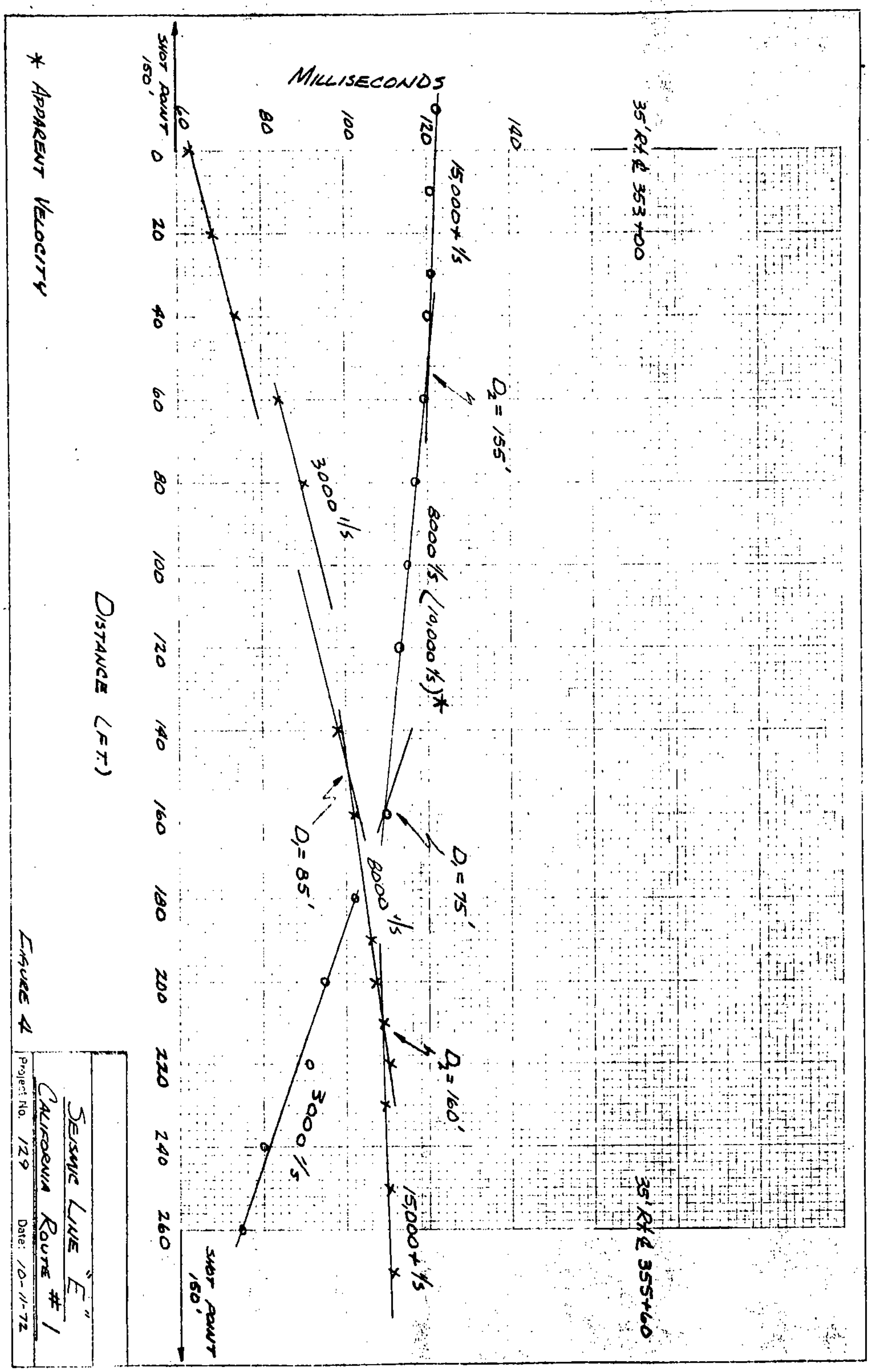
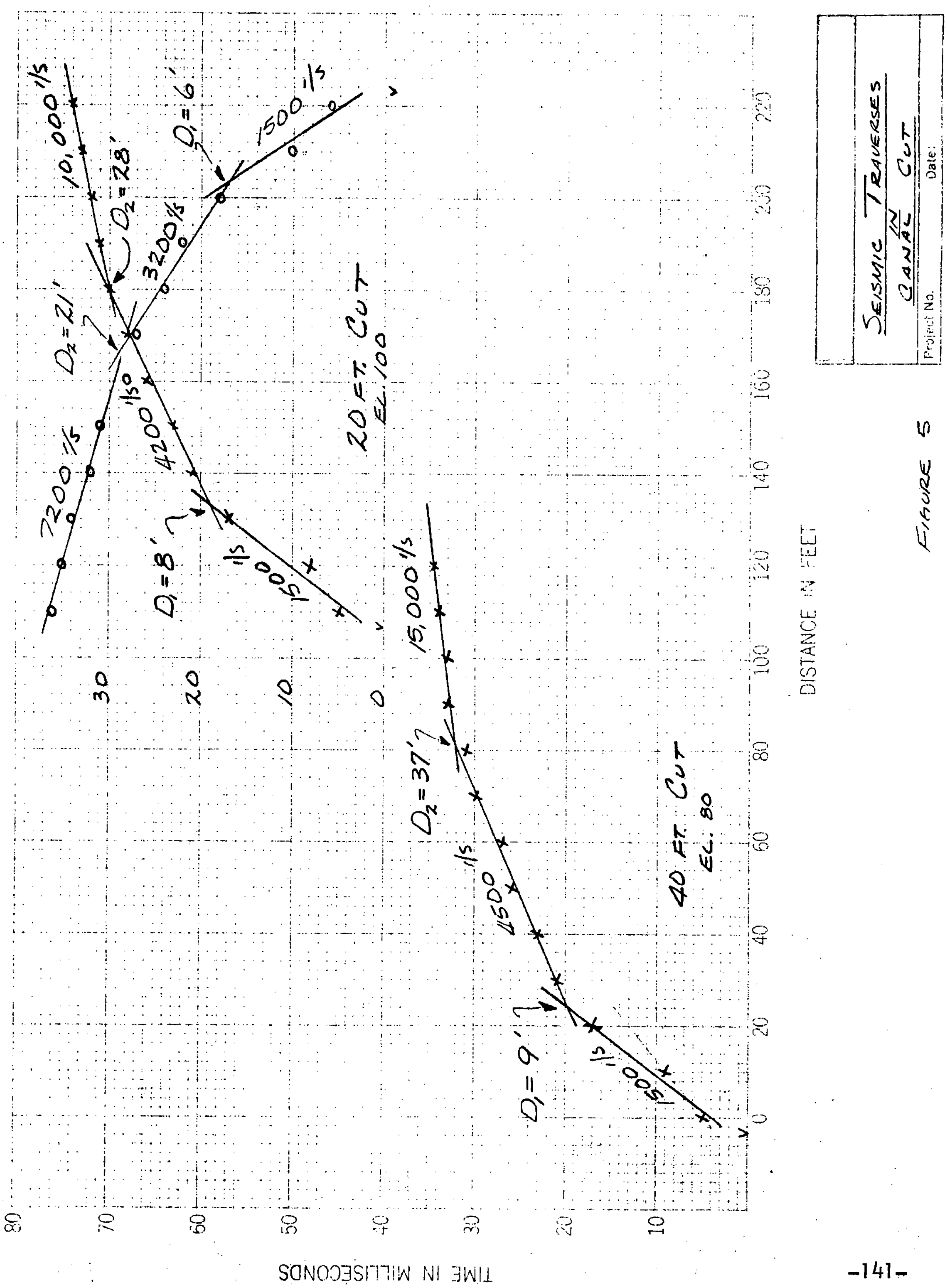

MILLISECONDS
140
120
100
80
60
SHOT POINT 150'
35' Rt. ℄ 353+00
15,000+ '/s
D2 = 155'
3000 '/s
8000 '/s (10,000 '/s)*
D1 = 75'
D1 = 85'
8000 '/s
D2 = 160'
3000 '/s
15,000+ '/s
35' Rt. ℄ 355+60
SHOT POINT 160'
0
20
40
60
80
100
120
140
160
180
200
220
240
260
DISTANCE (F.T.)
* APPARENT VELOCITY
SEISMIC LINE "E"
CALIFORNIA ROUTE # 1
Project No. 129
Date: 10-11-72

FIGURE 4


SEISMIC TRAVERSES IN CANAL CUT
Project No.
Date:
FIGURE 5
DISTANCE IN FEET
TIME IN MILLISECONDS
20 FT. CUT
EL. 100
40 FT. CUT
EL. 80
7200'/s
4200'/s
1500'/s
10,000'/s
3200'/s
1500'/s
15,000'/s
4500'/s
1500'/s
D_1=8'
D_2=21'
D_2=28'
D_1=6'
D_1=9'
D_2=37'

In conclusion, from these three examples and laboratory tests by Dr. Talwani there appears to be a significant potential for elastic deformation by rocks that tend to act as granular masses, however, this granularity occurs; whether by intense fracturing, weathering, true granular particles or low cementation. Thus, upon excavation, a previously high velocity rock can become a lower velocity rock which is capable of being excavated with less effort than presumed. Also, this deformation becomes an influencing factor in projects where slope stability (short and long-term) and foundation design are considered.

TRAFFIC SAFETY INSTRUMENTATION, FOUNTAIN LANDSLIDE, OREGON

By

Edgar L. Johnsen
Oregon Highway Division of Transportation

ABSTRACT

Settlement sensor installation in the highway subgrade triggers traffic warning signs when settlement distorts the pavement. The differential pavement elevation changes taking place are hazardous to traffic. This movement can be detected by fourteen electrically sensing mercury manometers1

Continual settlement at the Fountain Landslide area on the Columbia River Highway 40 miles east of Portland causes warping and and buckling of pavement. An early warning system will provide an alternate to other solutions not feasible because of cost.

Sensor installation as described in text was installed in 1975 during the reconstruction of I-80N across the landslide.

GEOLOGY

A generalized section of the slide is shown in Figure 1. In the vicinity of the landslide, four rock units are:

Recent landslide debris,
Columbia River lava, Miocene,
Eagle Creek pyroclastics, Oligocene,
Andesite Intrusive, Eocene.

The landslide debris is composed of dice basalt talus and cinders, with lenses of volcanic ash. Massive landslides over two square miles in area have been identified in the Columbia gorge. Many creeks and streams disappear into the talus at the base of the basalt cliffs to become groundwater. The volcanic ash beds from volcanic eruptions of the Cascade Mountains become natural barriers to the seepage of this groundwater. Both earthquakes and the subterranean water effect the sliding which mixes the ash deposits with talus. This unit locally is up to three hundred feet thick.

The Miocene Columbia River lavas are widespread in the Northwest. Locally the lava is a felsitic basalt, occasionally amygdaloidal. The bluff south of the slide rises over 4,000 feet. The bluff includes a 3,000 foot section of the brickbat jointed lava which is columnar jointed at the base of some flows. The gorge wall is the source of the basalt talus which dominates the slide debris.

Eagle Creek pyroclastics underlie the Columbia River basalt as siltstone and tuffaceous conglomerates. Wherever the explorations penetrated into this formation for sampling, the mudstone beds were badly sheared with more than ten feet of slickensided gouge near the contact with the slide debris.

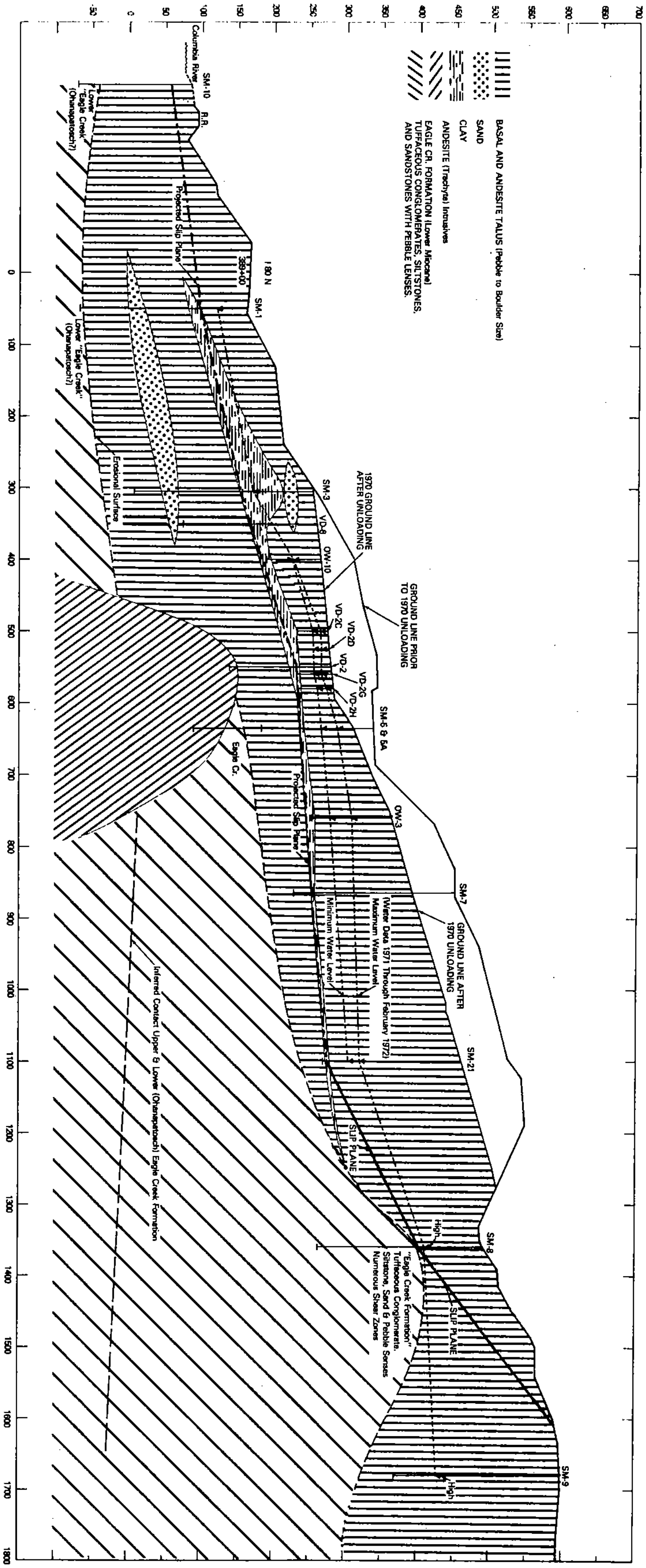


FIGURE 1. Generalized Cross-section (1)

One prominent Eocene Andesite intrusive forms part of the base to the slide which is highly irregular. Along the freeway two hundred feet west of the instrument house, a boring over two hundred feet deep failed to reach the intrusive bedrock. The instrument house and one reservoir box are founded on the intrusive.

HISTORY

Prior to 1950, both the highway and the adjacent Union Pacific Railroad were stressed little by earth creep. Routine maintenance was adequate to retain a safe alignment.

In 1950, the highway was straightened and widened with a cut through the slide. As the excavation approached subgrade, the pavement humped 12 feet overnight. And so began a long history of geologic investigation. Early corrective efforts accented drainage improvement which included test pits to locate the groundwater table.

Drainage efforts were aimed at correcting a small slide. A drainage tunnel was started in conjunction with shallow horizontal drain borings. The 50 to 100 foot deep horizontal drains were shallow compared to the tremendous mass of the slide. The drainage tunnels hit water zones and shear zones that hindered their progress and effectiveness.

In 1965, U.S. Route 30 was widened to 4-lane Interstate standards as I-80N. After the widening cut was made, the main break was located uphill far beyond its previous known extent. The pavement began humping at the rate of four to six feet per week for several weeks. The State and the contractor entered into a price agreement to unload 350,000 c.y. of slide debris from the apparent head of the slide, see Figure 2. The following year, the final excavation and paving were completed. The distortion of the roadbed continued. Test borings were added to cover the enlarged slide area.

1969 Contract

In October 1969, bids were taken on a slide correction project, see Figure 3. The contract was awarded to S. S. Mullins for removing 1,600,000 cubic yards of slide debris and incidental freeway reconstruction. The work was complete by the fall of 1970.

During the winter of '70-'71, more than a three-foot roll in the pavement occurred initially which continued to heighten throughout the season. Maintenance patching costs accumulated to more than $300,000 during the first year after the slide correction attempt.

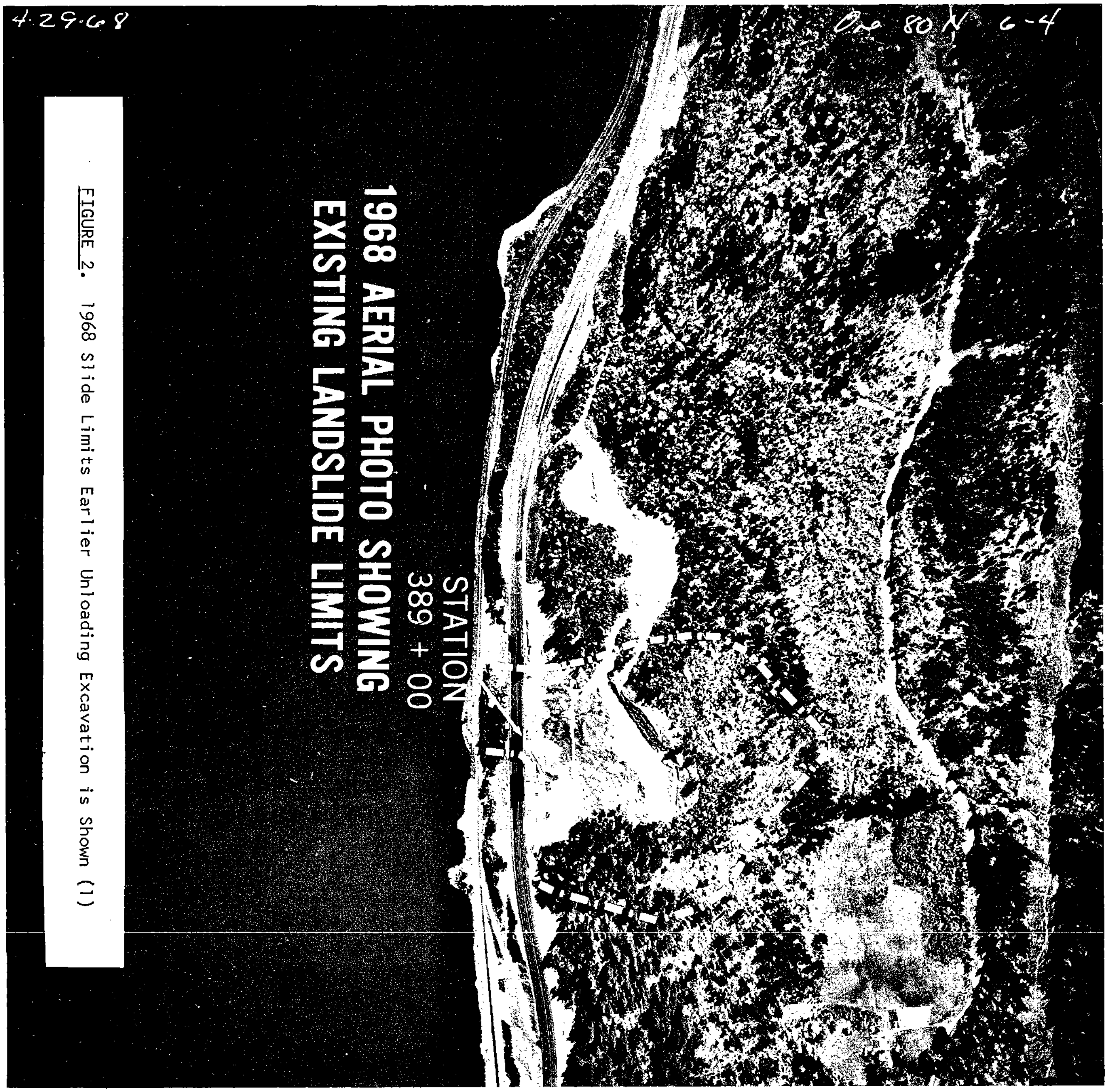


FIGURE 2. 1968 Slide Limits Earlier Unloading Excavation is Shown (1)

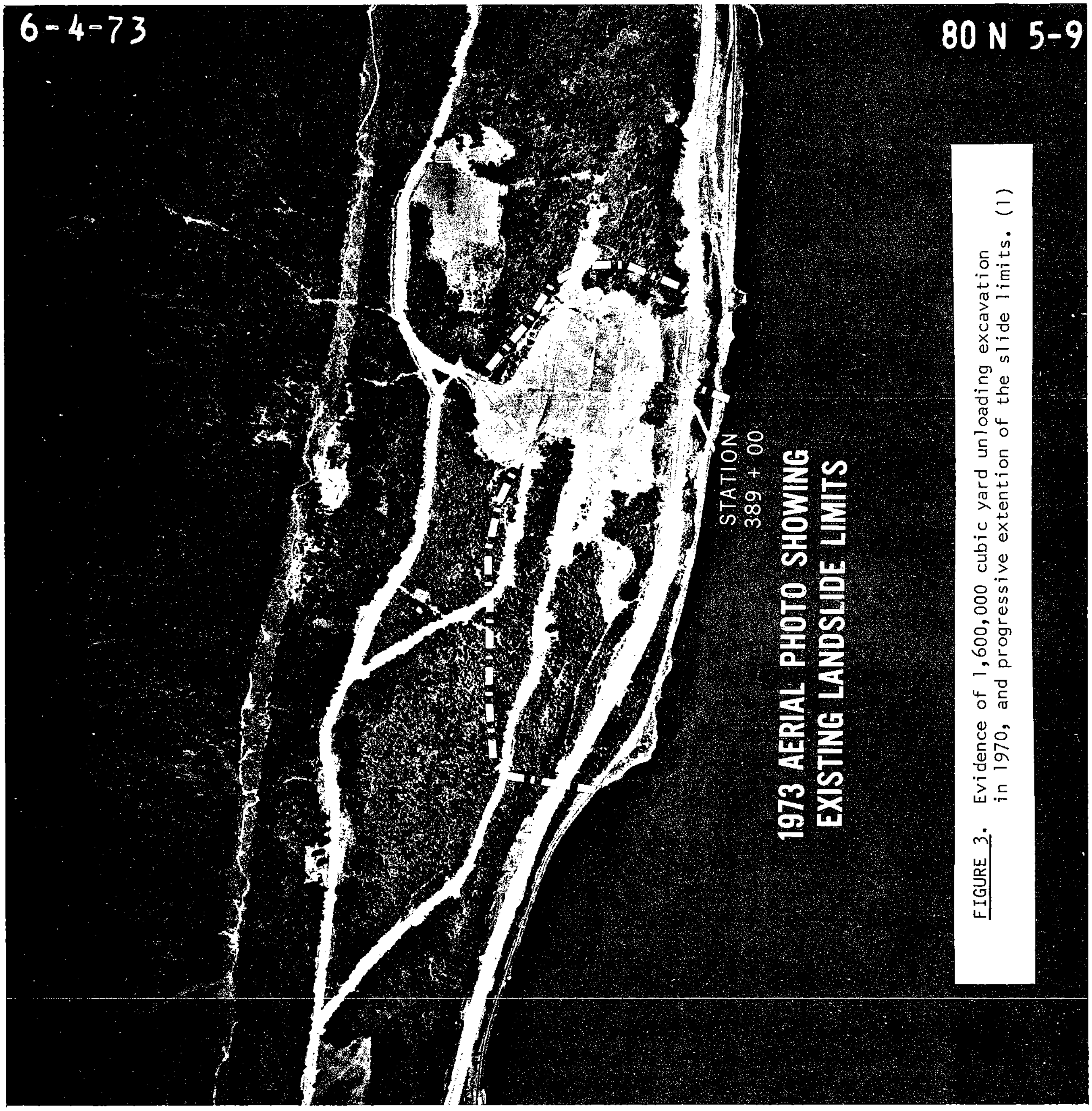


FIGURE 3. Evidence of 1,600,000 cubic yard unloading excavation in 1970, and progressive extention of the slide limits. (1)

However, the unloading reduced the rate of the total slide movement. Inclinometer readings taken subsequent to the unloading indicated a slowing from more than two feet horizontally per year to less than one foot per year.

Vertical Drainage Wells

Early in 1971, the Federal Highway Administration proposed a testing program to verify the value of vertical drainage wells. Eleven wells were installed with 48 observation wells.

Installation of earlier inclinometer tubes had verified that open dice rock occurred many places beneath the clay slip seam. If the water that collected above the sliding surface could be disposed of in the talus below, a lowering of the water table and stability could be achieved at low cost.

Two of the vertical drain wells functioned during the test, discharging about 50 GPM each into the underlying talus. Other wells either lacked a supply of water or impermeable materials were encountered in the potential discharge zone. The proposal was discarded.

RECENT PLANNING

In January 1972, the known slide area doubled by enlarging easterly, as shown in Figure 3. The original slide extended from Sta. 383 to Sta. 402. The eastern extension affected the roadbed ahead on station to Sta. 422. The new southeasterly limits of the slide was easy to recognize by one to two foot displacements in the haul roads placed by the previous contractor. Explorations were organized to identify the parameters of the sliding mass. In 1973, the limits of the slide were conclusively established.

Alternatives

The success with the prior unloading was marginal. With the controls of the shipping channel in the Columbia River to the north and the Union Pacific Railroad on the toe of the slide, unloading was considered, but dropped as too costly. The cost estimate of unloading including correction of the roadway grade exceeded $8,000,000 in 1973 dollars.

Realignment of the Freeway either at the toe of the slide in the Columbia River or south and above the slide along old Military Road was rejected. The former because of the cost $6,000,000 and the latter because the vertical grade exceeded Interstate standards.

The alternative chosen was to live with the slide and provide for the safety of the traffic in the event of hazardous pavement distortions.

Instrumentation Concept

Of the instrument alternatives including pneumatic and ethylene glycol manometers, the electrically sensing mercury manometers were selected for their automatic output. Changes in pavement elevation, either humping or dropping, present the greatest hazard to traffic. Small increments of change can be detected and recorded automatically at the time of the event. A master control console in a central location could record the data.

A threshold could be set in the console memory which would activate a traffic signal. A lower range of movement was selected that would switch on the signs. The initial threshold settings will vary from two to three inches change. This will warn traffic of driving hazards. The higher setting triggers an alarm when major distortions ranging from four inches and seven inches affect the pavement.

Whenever the warning signs turn on, a radio transmission signals the Oregon State Police office in Milwaukie. The police operator is notified by sound and sight the moment the signs are energized. The operator will then notify the patrolman nearest to the slide to inspect the pavement and report his observations, as shown in Figure 4.

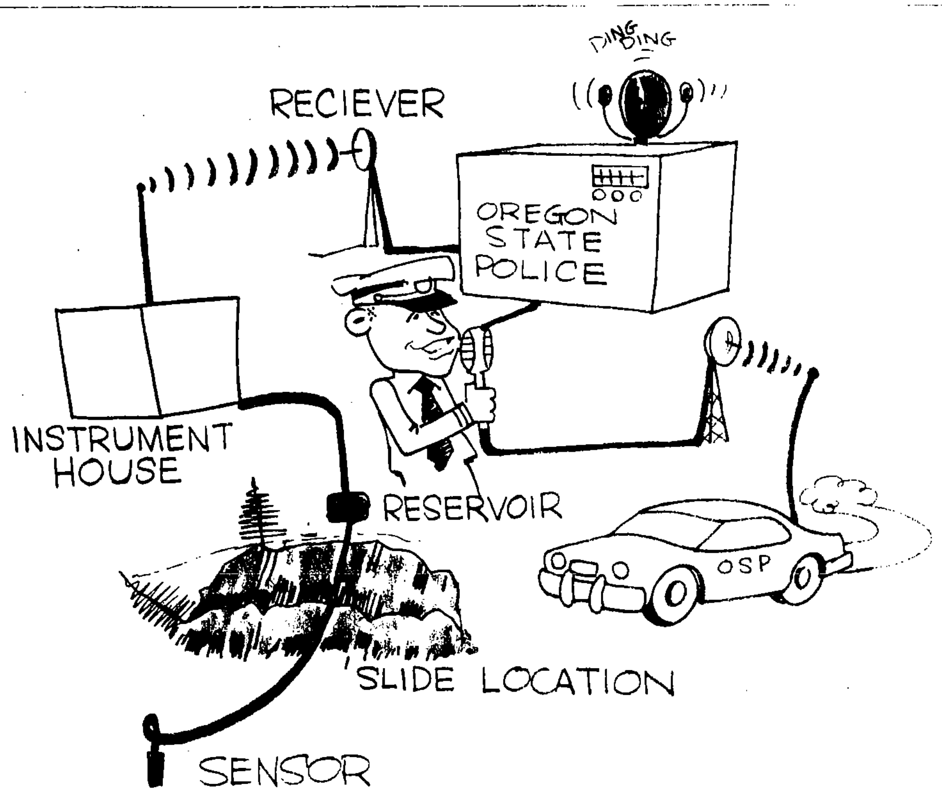


FIGURE 4. In the alarm condition the sensors initiate a radio signal to the State Police office in Portland. An officer is dispatched to the slide to examine and report road conditions.

The soils and geology unit in the Milwaukie highway office may make routine checks of the system and manually check the amount of movement since the last reading. The operator has the option of leaving the base comparative level of the sensor without change or he may select a new base elevation for threshold comparison. For instance, the instrument indicated one and one-half inches movement. With the observed field condition that the pavement surface indicated no hazard, the operator would set a new base level and adjust correspondingly the first and second thresholds. The added movement required to actuate the alarm would be the complete range of the new pre-set threshold.

Vandalism must be considered anytime the public has unsupervised access to a building. To this end, all of the instrumentation is buried or locked. Two reservoir boxes remote from the central instrument house have an elaborate bar locking them, see Figure 5. The central instrument house is concrete block construction with a locked steel door, see Figure 6.

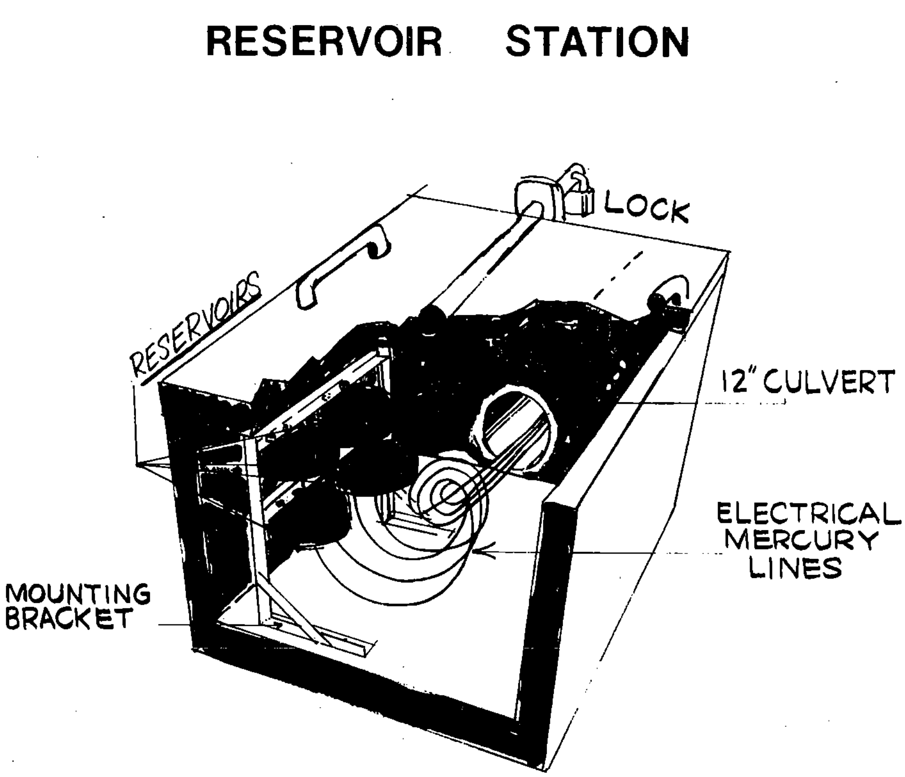


FIGURE 5. Example of the two reservoir station boxes remote from the instrument house.

12'X 10' INSTRUMENT HOUSE

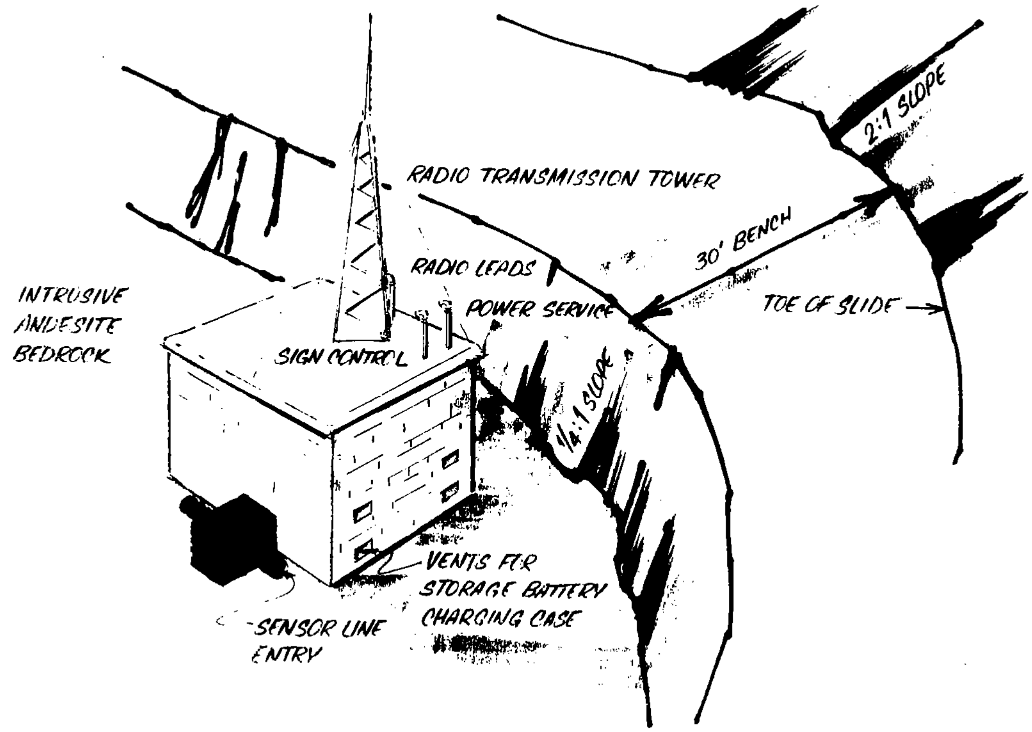


FIGURE 6. The Instrument House was constructed on Andesite bedrock. A 30-foot bench in the rock above the house will provide for creep of the slide debris in the 2:1 backslope.

THE SYSTEM

The purpose of the sensing system is to detect vertical movements in the pavement section which could be unsafe for the public traveling at normal freeway speeds.

The radio relay to the State Police is a unique safety feature. The station operator is alerted by a buzzer and a panel light each time the warning signs are triggered. He notifies the patrolman nearest the slide to check and report road conditions. The Highway Region Engineer is to be notified of the alert and given the report of field conditions. (See Figures 7 and 8).

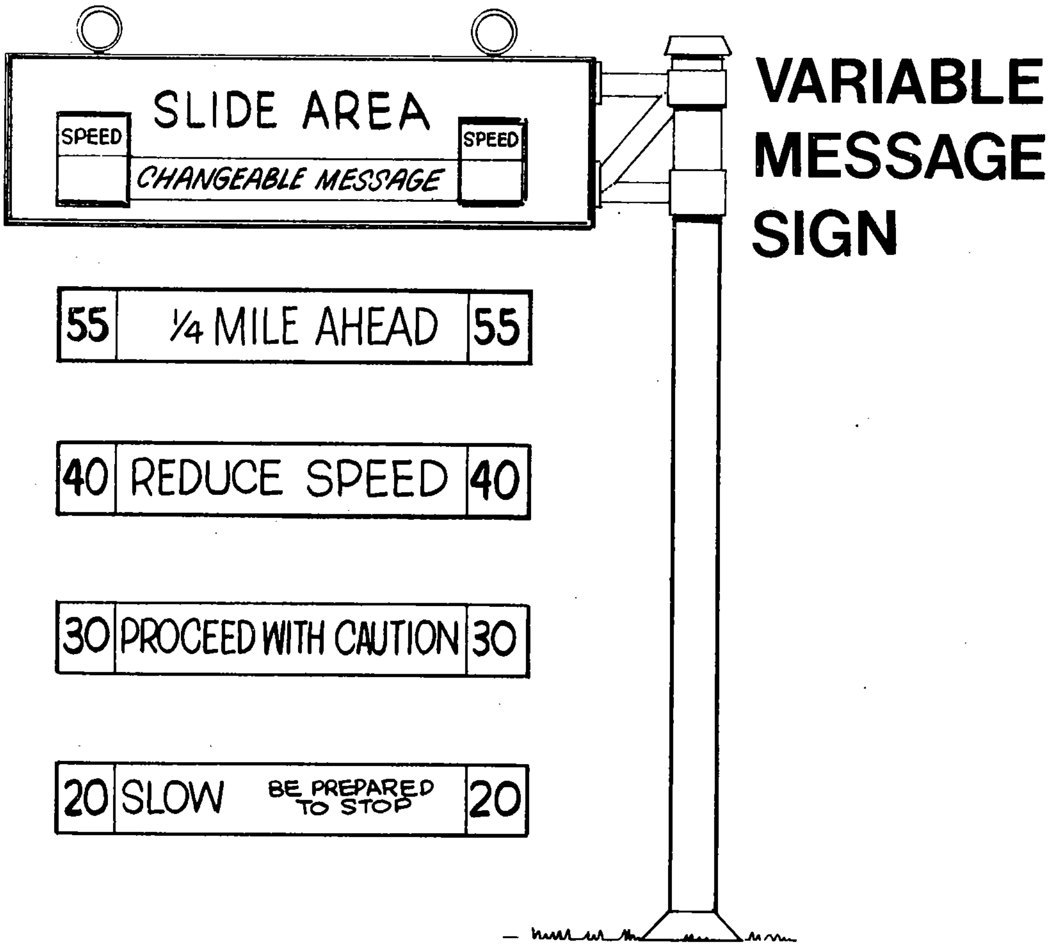


FIGURE 7. Variable Sign Messages

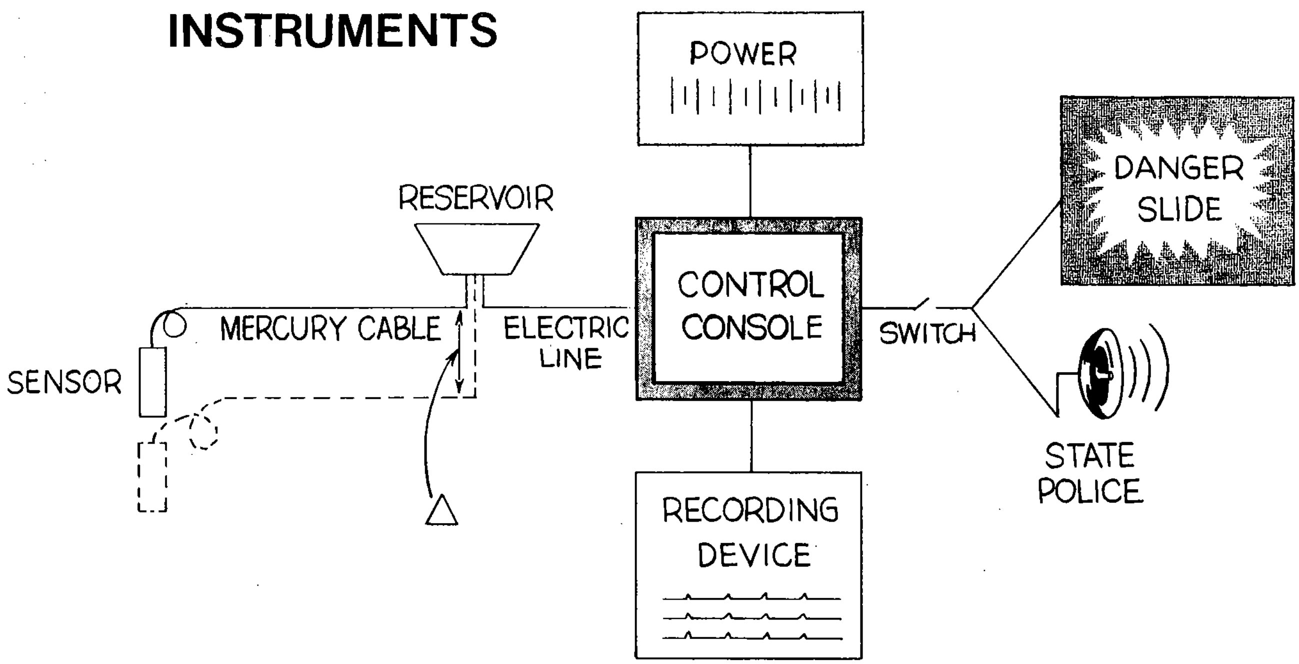


FIGURE 8. A sensor alarm triggers the warning sign, alerts the State Police and records the event.

Control Console

The control console prints a record of movement both on a routine time interval and each time a pre-set amount of change occurs, see Figure 9.

Each sensor print-out may include:

The time of day,
The day of year,
Sensor number,
Deviation.

The threshold settings may be adjusted as experience would dictate. The operator resets the base level and the thresholds of a sensor when new controlling elevations are judged safe. As a control feature, the console records are monitored and compared regularly against surveyed control.

If a rate of change (defined in unit elevation per unit time) is a valid measure for warning, then periodic inspections could reduce the requirement to manually reset the base and alarm threshold elevations as the pavement surface from the slide activity stayed within safe limits of smoothness. This rate of change control feature may be added at modest cost to this system.

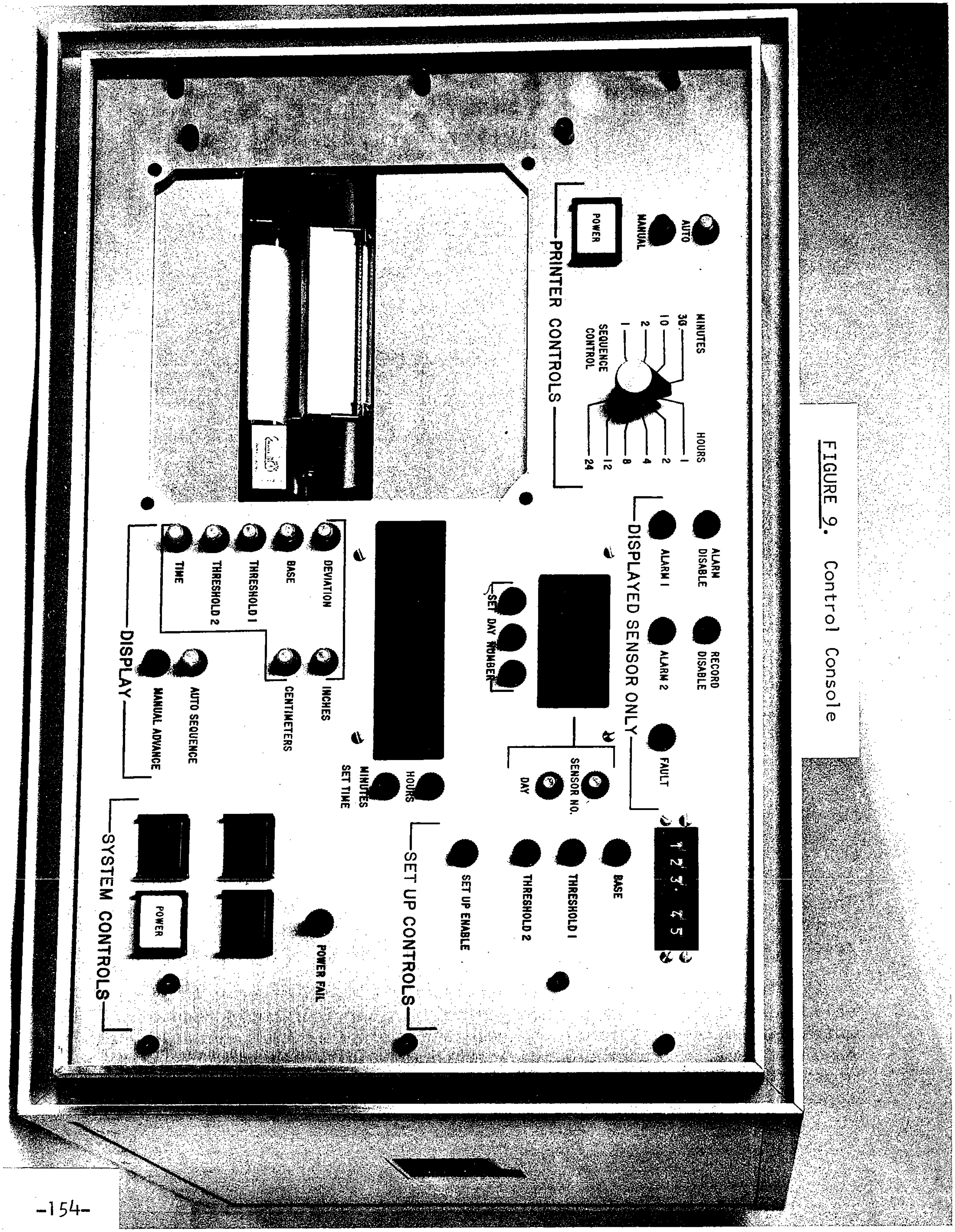


FIGURE 9. Control Console

Several safety features are built into the console:

- Power loss initiates data print-out,
- Twenty-four hour storage battery temporary power,
- Sensor failure won't activate an alarm,
- More than one sensor must exceed the threshold to set off the alarm,
- Lightning protection of electronics.

PROTECTION TO THE INSTRUMENTATION

All burial cable was jacketed with heavy vinyl and "water blocked." The equipment in the ground is equipped with lightning arresters. Each sensor reservoir is a closed system which is surrounded by at least six inches of sand. The instrument house maintains a temperature range from 60° to 110°F.

The instrument house, see Figure 6, and the two remote reservoir boxes, see Figure 5, are founded on stable ground or bedrock. Thirty-inch nestable culvert with a six-inch lift of sand in the invert protects the conduit at shear zone crossings. Manhole openings provide points of cable inspection. Distorted culverts may be replaced without severing the sensor mercury lines. Ten percent of the required cable length is added as an allowance for modest lengthening across the shear zones.

The instrumentation will be difficult to vandalize. Both reservoir enclosures are provided with locked concrete lids. The instrument house is windowless cement block construction with a concrete slab roof and a one-quarter inch steel plate door. These special precautions were recommended from the experiences with housing radio equipment in other remote unattended locations.

INSTALLATION

A contract was awarded in early April, 1975 to Terra Technology, of Seattle, Washington for the instruments and furnishing supervision for the equipment on the Fountain Landslide safety project which was awarded in May, 1975 to Roy L. Houck Construction Company. Terra Technology was given 90 calendar days to have the system available for installation by the contractor.

Unusually long lengths of mercury tube, up to 844 feet, connect the sensors to the reservoirs.

The mercury tubing and electrical lines were placed in 30" nestable culvert where the lines would cross known shear zones. Inspection manhole openings into the nestable culvert were provided at regular intervals. Damaged culvert may be maintained or replaced without injury to the sensor lines. The sensor and the lines were bedded in uniform sand to protect the cables from damage by sharp objects, see Figure 10.

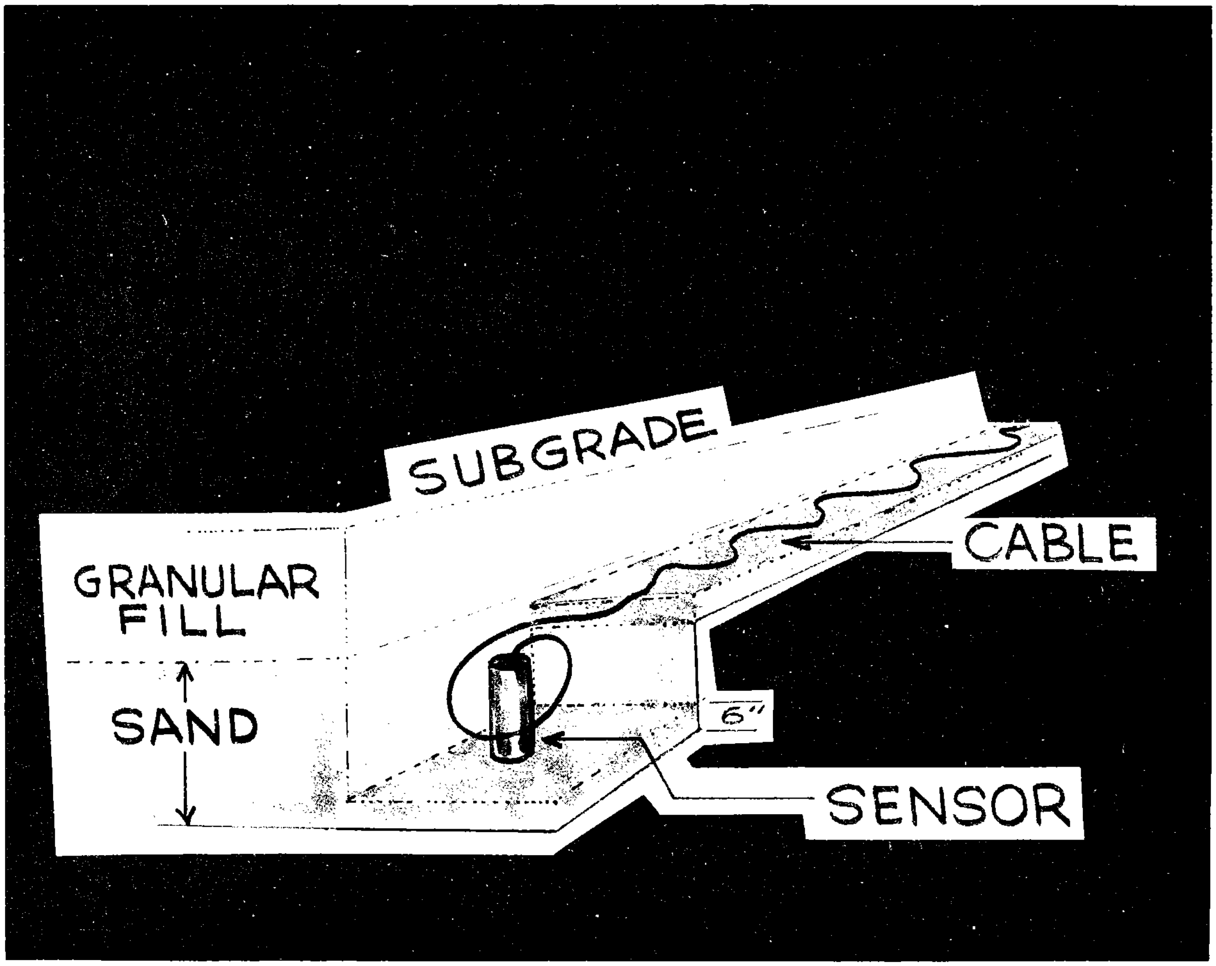


FIGURE 10. The sensors and cable are protected with a six-inch cover of sand. Where the cables cross shear zones, they are placed in open 30-inch nestable culverts.

Terra Technology provided reels to handle the lengths of cable between the sensor and the reservoir. The reels protected part of the cable and the reservoir unit during the staging of the installation.

After the contractor had built the first stage eastbound fills to subgrade, the ten sensors were placed in the subgrade. The six reservoirs located at the instrument house were buried in the reels for protection until the instrument house was completed. The remaining four sensors in the eastbound and the four westbound sensors were placed directly in their respective reservoir station box, see Figure 11.

Terra Technology retained responsible possession of the equipment until its installation. The control console was maintained in an operating mode at their Seattle plant until the instrument house was completed. The usual bugs of a new system were corrected before the equipment was supplied to the project.

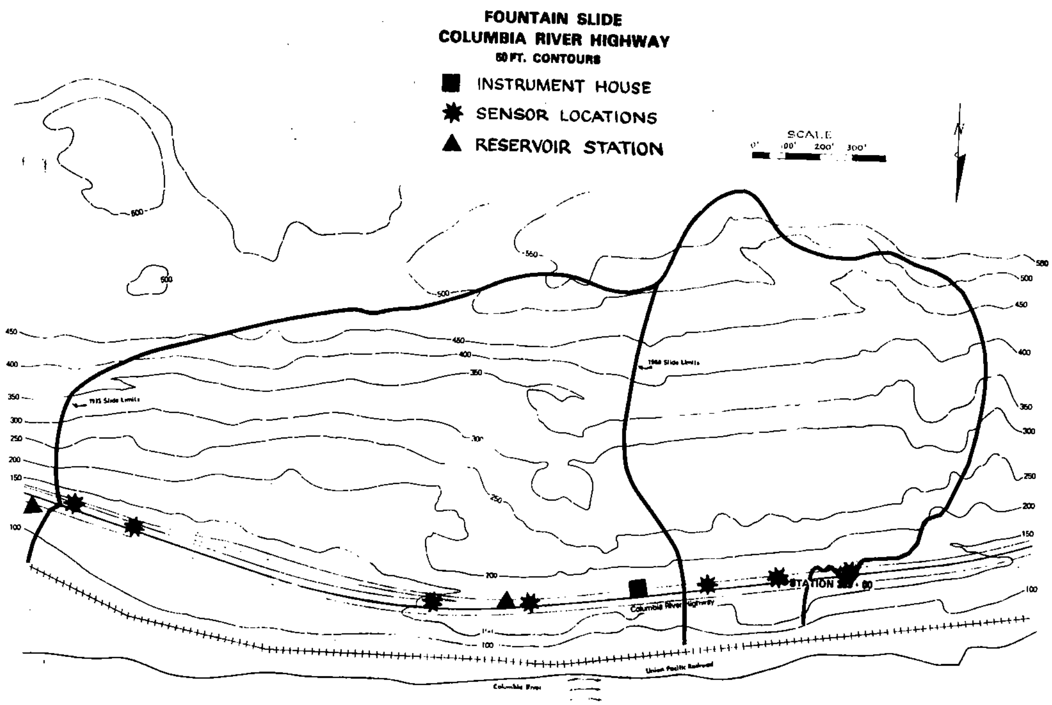


FIGURE 11. Plan view of the slide with the instrument locations.

The sign and the radio installation will be completed by October, 1975. The electrical mercury manometer sensors and the tape print-out unit are functioning properly. The completed tie-up will assure a safe trip for all past the Fountain Landslide.

CONCLUSIONS

The instrumentation system in service on the Fountain Landslide will serve as a model traffic safety device where potential slide hazards exist. Settlement sensors in the roadbed provide a low cost alternative for slide prone ground. This installation will test the long term reliability of electrically sensing mercury manometers. Automatic recordings are printed of any vertical adjustments of the pavement which will lead to a more complete understanding of this landslide.

ACKNOWLEDGEMENTS

The author recognizes many giving assistance on this project. Charles Meyers, Terra Technology, Inc. coordinated the equipment conception of the instrumentation system to its installation. Andy Munoz, FHWA Soils Engineer, made available his material and knowledge of the subsurface conditions at the slide. Dan Gano, Oregon Highway Geologist, handled effectively the field explorations and reporting of the slide over many years. E. S. Hunter, Deputy State Highway Engineer, encouraged an alternate solution to this slide problem.

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IN-SITU MEASUREMENT OF SHEAR-WAVE VELOCITIES FOR ENGINEERING APPLICATIONS

By

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ABSTRACT

Recently developed methods for making in-situ measurement of shear-wave velocities of earth materials provide some unique applications to engineering problems. New techniques for both signal generation and signal recording are discussed. These techniques permit both surface and down-hole measurements to be made.

When the density and compressional-wave velocity are known, a determination of the shear-wave velocity makes it possible to calculate all of the dynamic elastic moduli for the material. Consequently, in-situ measurement of shear-wave velocities has a wide range of engineering applications. Examples from foundation studies, particularly for protection of structures against earthquake damage, stability analysis of earth dams and other earth-fill structures, and assessment of safety of soils over small areas are presented.

INTRODUCTION

Recently developed techniques for both signal generation and signal recording have been used to make in-situ measurements of shear-wave velocities of earth materials. These techniques can be applied to both surface measurements and cross-hole measurements.

When the bulk density and the seismic compressional-wave velocity of a material are known a determination of the seismic shear-wave velocity provides all the parameters needed to compute the dynamic elastic moduli of the material. For this reason the in-situ measurement of shear-wave velocities has a wide range of application to engineering problems. Such applications would include: foundation studies to provide design parameters for protection of structures against earthquake damage, stability analysis of earth-fill structures including earth dams, and assessment of safety of soils over small areas.

This paper presents: a formulation of the relationships between seismic velocities and the dynamic elastic moduli, a review of some basic principles of seismic wave propagation, an analysis of some of the techniques for shear-wave generation and shear-wave identification which have been developed for engineering studies, and an analysis of some engineering applications of these techniques.

DYNAMIC ELASTIC MODULI

Within the temperature and pressure conditions which occur at or near the surface of the earth most soils and rocks respond as elastic materials when subjected to short-term stresses. As such their elastic deformation can be defined by the dynamic elastic moduli: Poisson's ratio (u), Young's modulus (E), the shear modulus (G), and the bulk modulus (K). The relationship between these constants for a given material is such that if any two are known the other two can be computed.

Numerous laboratory methods have been developed to measure the elastic moduli from samples of a given material. From the standpoint of engineering applications, however, the laboratory determination of the elastic moduli of earth materials has always presented some problems. Even when great care is taken to obtain "undisturbed" samples they still have to be removed from their natural setting to make the laboratory tests. As a result, their properties may be changed due to changes in pressure or alteration of their water content. A more serious difficulty in this regard is that the laboratory sample, because of its relatively small size, may not be truly representative of the much larger volume of earth materials from which it was taken. Because of these problems with laboratory tests, a considerable effort has been made to find ways of making in-situ measurements of the elastic moduli of earth materials.

A very helpful aid in making in-situ measurements of the elastic constants of earth materials is the relationship between these parameters and the properties of bulk density (d), seismic compressional-wave velocity (V_p), and seismic shear-wave velocity (V_s). Some of the equations which express these relationships are given in Table 1.

SEISMIC WAVE PROPAGATION

In general, any seismic source will generate energy which propagates in several different wave forms. Both body waves and surface waves will be produced. The body waves can travel to considerable depth while the amplitude of the surface waves dies out with depth. For many engineering applications, the determination of the elastic moduli in particular, the body waves provide the most useful information. For these reasons we will not consider the surface waves here.

There are two types of body waves generated by seismic sources: the primary or P wave and the second or S wave. In terms of their mode of propagation these two waves are distinctly different. The P wave, which always has the higher velocity, is propagated by particle motion which is parallel to the ray-path direction of the wave form. It is in reference to this particle motion that the name compressional-wave or longitudinal-wave is used.

The S wave is propagated by particle motion which is in a plane perpendicular to the ray-path direction of the wave form. As a result it is called the shear-wave or transverse-wave. An important aspect of the S-wave is that for measurements made at or near the surface of the earth it may be

Bulk density:	d
Compressional-wave velocity:	V_p
Shear-wave velocity:	V_s

$$V_p = \left(\frac{K + 4/3\ G}{d} \right)^{\frac{1}{2}} \quad \text{and:} \quad V_s = \left(\frac{G}{d} \right)^{\frac{1}{2}}$$

Poisson's ratio:	$u = \dfrac{1 - 2\ (V_s/V_p)^2}{2 - 2\ (V_s/V_p)^2}$
Young's modulus: **(Modulus of elasticity)**	$E = 2\ G\ (1 + u)$
Shear modulus: **(Modulus of rigidity)**	$G = d\ V_s^2$
Bulk modulus: **(Modulus of incompressibility)**	$K = \dfrac{E}{3\ (1 - 2\ u)}$

TABLE 1.

Equations showing relationships between elastic moduli and seismic velocities.

resolved into two mutually perpendicular components. One of these components lies in a vertical plane and it is designated the SV wave. The other component lies in a plane parallel to the surface or horizontal and it is called the SH wave. It is this resolution into the SV and SH components which has proved most helpful in engineering studies in identifying the S wave arrival even though it is preceeded by a P wave arrival.

SHEAR-WAVE GENERATION

As Mooney (1974) points out, nearly all practical seismic sources will generate shear-waves to some extent. These include explosives, impacts from hammers and weight drops, and vibratory sources. Both Ballard and McLean (1975) and Stokoe and Abdel-razzak (1975) have presented excellent discussions of the advantages and disadvantages of each of these types of sources relative to their generation of shear-waves. Consequently, we will consider here only those seismic sources which have been demonstrated to produce the best results as shear-wave generators, namely, those impact sources which are repeatable and directional.

For measurements made at the surface, the most useful technique for shear-wave generation appears to be one described by Mooney (1974) in which a plank, either wood or aluminum, held firmly on the ground by parking the wheel of a vehicle on it is struck on one end by a hammer or horizontal pendulum. Along a line perpendicular to the long axis of the plank the early seismic arrivals should consist primarily of horizontally polarized shear-waves or SH waves. Depending on the source symmetry the P wave and SV wave components in this direction can be surpressed. One very helpful advantage of this source is that it is directional, that is, it can be struck on one end and then on the opposite end. As a result, the recorded SH waveform will show a phase reversal corresponding to the reversal of the impact direction. Another advantage of this source is that it can be repeated several times. The utility of these two features will be discussed further in the following section.

For cross-hole measurements, that is, measurements made in two or more drill holes at the same depth beneath the surface, a signal generation technique described by Mirafuente, et al. (1974) has had demonstrated success. In this technique a hammer device is used which consists of a stationary mass and a sliding weight. The fixed mass can be clamped in a borehole at various depth by means of a pair of plates on opposite sides which are expanded against the wall of the hole by pistons. The unique feature of this device is that the sliding weight, which moves along guides in the stationary mass, can be operated to initiate a downward or an upward directed impulse by striking against the fixed mass. In this way the impact source is directional but with a vertical rather than a horizontal orientation. As a result of the vertical orientation, instead of generating SH waves in a direction perpendidular to the axis of the borehole this device generates both P waves and predominantly vertically polarizes or SV waves. When the direction of the impulse is reversed the SV waves recorded in adjacent boreholes show corresponding reversed polarity. In contrast, the P waves which are also recorded retain the same polarity for both a downward and an upward directed impulse. Some examples of this feature are discussed in the following section.

SHEAR-WAVE IDENTIFICATION

The two techniques for shear-wave generation previously described are both directional and repeatable. Many of the difficulties associated with identifying the shear-wave arrival on the seismic waveform can be overcome by taking advantage of these two characteristics. For example: when making surface measurements, the horizontally polarized SH waves generated by striking one end of the plank are best recorded by using a horizontal geophone oriented in a direction parallel to the length of the plank and along a line perpendicular to it; when the down-hole source is used, the SV waves are recorded on a vertically oriented geophone in an adjacent drill hole. An additional benefit of the directional character of these sources will be discussed at the end of this section.

There are two aspects of the problem of shear-wave identification, however, which bear further consideration. First, both of these signal generating techniques are comparatively low-energy sources. Second, the possibility always exists that the seismic arrivals which are used to determine the shear-wave velocity may have followed some fairly complex refracted travel-path rather than a straight-line path from source to receiver.

The problem associated with using a low-energy seismic source such as a hammer or weight-drop is that seismic energy attenuates fairly rapidly with distance such that the sought-for signals are often "buried" in the background noise. In many instances, engineering applications in particular, this difficulty can be overcome by using a signal-enhancement seismograph to record the arrivals. These units, which are designed specifically for use with low-energy seismic sources, have the capability of storing the recorded waveform in a memory and sequentially adding the waveforms from successive imputs. This, by using a repeatable source and recording several resulting waveforms with a signal-enhancement unit the coherent seismic arrivals will "build up" above the level of the random noise pulses which tend to be surpressed. An additional feature of the signal-enhancement unit is that it displays a visual record of the wave form stored in the memory. With a little practice the operator can readily identify which pulses are seismic signal and which are noise as he observes how the waveform changes after each successive imput. Some signal-enhancement units have the added feature that the waveform stored in the memory can be fed into a strip-chart recorder to provide a permanent record.

The problem of determining if the shear-wave travel-path has followed a straight line from source to receiver or some refracted path is best solved by using an array of detectors as described by Mooney (1974). Using this technique, at least two (usually several) geophones are placed along a straight line at different distances from the source. Figure 1 shows an example of a two geophone array for both the surface measurement technique (1 A) and the down-hole method (1 B). For the surface array, horizontal geophones are used which are oriented parallel to the direction of the source impulse while for the down-hole array, vertical geophones are used.

When using such an array, seismic velocities are normally determined by first making a time-distance plot of the arrival times of the seismic events recorded at each geophone. If the recorded arrival times of the onset of the shear-wave, when plotted on such a time-distance graph, fall on a straight-line segment which passes through the origin then the travel-path

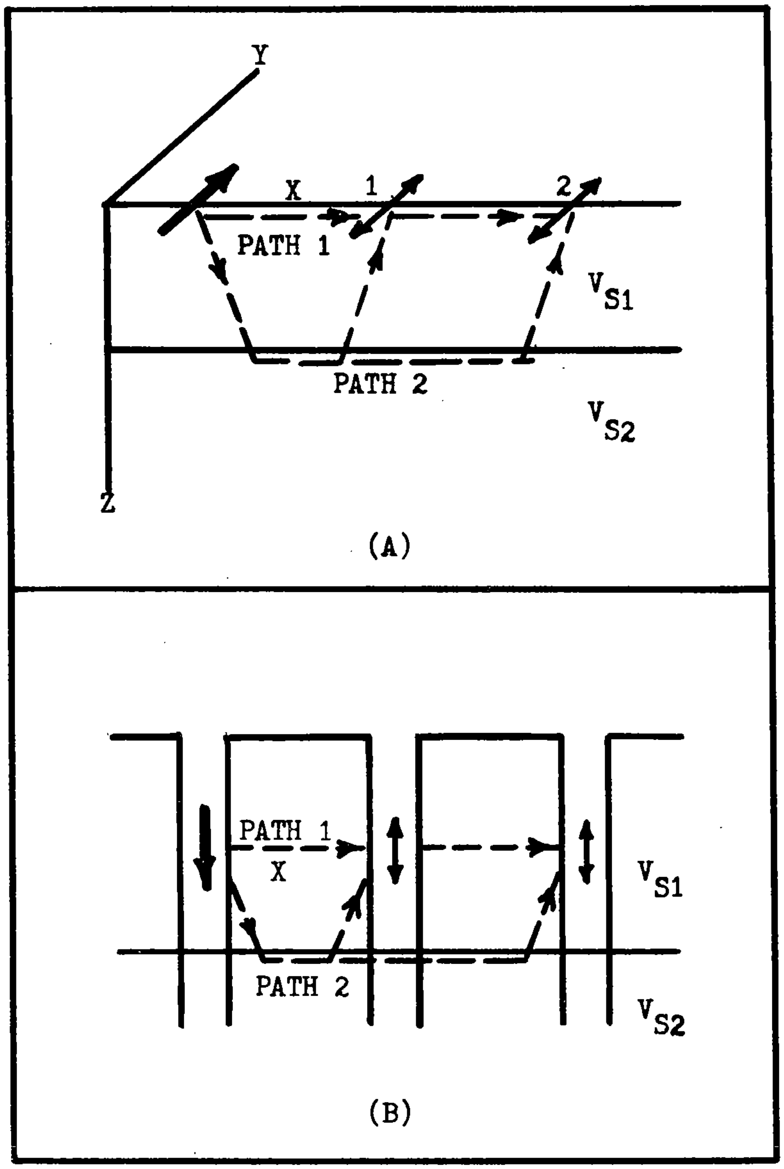

FIGURE 1.

Illustrations of the arrangement of source (heavy arrow) and reciever (double-headed arrow) arrays for surface measurements (A) and cross-hole measurements (B) of seismic shear-wave velocities.

was along a straight line from source to receiver and the inverse slope of the line segment gives the velocity of the shear-wave. It should be emphasized that the foregoing statement is true only for the time-distance plot of the first-arrival times for the shear-wave at each geophone. If some later phase in the waveform, such as a peak or trough, is used then the line segment drawn through these points will be correspondingly offset from the origin. If, however, the first-arrival time plots fall on different line segments which do not pass through the origin then the shear-wave has traveled along some refracted path. By applying the standard seismic refraction methods of interpretation to such a time-distance plot the refracted path can be identified and the solution will yield the corresponding values for the shear-wave velocities.

Another benefit of the directional character of the shear-wave generation techniques considered here is that they can be reversed in direction. As a result, the shear-wave arrivals will show a reversed polarity when the source impact direction is reversed. This polarity reversal occurs when either of the signal generating techniques is used. Excellent examples of records which show this reversed polarity are included in the literature; see: Anderson and Woods (1975), Mirafuente, et al. (1974), Mooney (1974), and Stokoe and Abdel-razzak (1975). Because of this polarity reversal the shear-wave arrival can often be accurately identified even when, as generally is the case, preceding wave trains tend to obscure the shear-wave arrival as recorded on any one record.

ENGINEERING APPLICATIONS

The use of in-situ measurements of shear-wave velocities to determine the dynamic elastic moduli of earth materials has found ever-increasing application in engineering studies. Such fields as earthquake engineering, foundation studies, soils stability analysis, and rock mechanics have already compiled an extensive list of applications. Consequently, only some representative examples will be presented here.

The increasing demand for the construction of nuclear power stations in earthquake prone areas has led the foundation engineers to collect and analyze a considerable amount of information about the distribution of shear-wave velocities and shear moduli at many proposed sites. At some sites these measurements have been made to depths of 500 feet. As a result of these studies they have gained a great deal of information about the response of structures to dynamic foundation excitation.

Shear-wave velocity studies have been used to provide information about the distribution of the shear modulus in existing earth dams. The input from these measurements is critical in making the stability analysis of such structures.

The assessment of the safety of soils for construction and structural purposes is called microzonation. Shear-wave velocity measurements and the dynamic elastic moduli derived from them play an important part in this process of assessing soil safety over small areas.

CONCLUSIONS

All of the dynamic elastic moduli of an earth material can be computed when its bulk density and seismic compressional-wave and shear-wave velocities are known. The in-situ measurement of the shear-wave velocities of earth materials thus has a wide range of engineering applications because the samples of earth materials on which the elastic moduli are measured in the laboratory often are not truly representative of the larger volume of material from which they were taken.

A review of some basic principles of seismic wave propagation shows that the shear-wave or S wave can be, for convenience, resolved into two components, a horizontally polarized SH wave and a vertically polarized SV wave.

Two different techniques for shear-wave generation have been developed which have proved particularly useful for engineering applications. One, which is used for measurements made on the surface, generates predominantly SH waves in a direction normal to the impulse direction and these horizontally polarized waves can be recorded by horizontal geophones which are placed along a line perpendicular to the source direction. The other technique, used for cross-hole measurements, generates predominantly P waves and SV saves. The vertically polarized shear-waves can be recorded in adjacent boreholes by vertically oriented geophones. Both of these techniques for generating shear-waves have the unique advantage of being directional, reversible, and repeatable.

Many of the problems associated with identifying shear-wave arrivals on the recorded waveform can be greatly reduced by using the generating techniques described above. The limitations imposed by using these low-energy sources are overcome by recording the waveforms with a signal-enhancement seismograph. The problem of determining the actual travel-path of the recorded shear-waves is solved by using an array of geophones to record the signals.

While many different kinds of engineering applications for the in-situ measurement of shear-wave velocities have already been developed it seems reasonable to conclude that the use of these newly developed techniques for signal generation and signal recording will permit even greater application to engineering problems in the near future.

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THE USE OF ORGANIC TOPSOILS AS CONSTRUCTION MATERIALS

By

Charles L. Bartholomew,[1] and Herbert O. Ireland[2]

ABSTRACT

The purpose of this program of research was to investigate the use of organic topsoils for construction purposes. The necessity of removal and wastage of topsoils was investigated with regard to the need for removal and if removed, what depth would be appropriate. Further, if topsoils were removed, the study was intended to show whether or not they could be used as compacted fill material.

A total of eleven sites were selected in central, north central, and northern Illinois for soil sampling. The site selection was accomplished largely through the use of agriculture soil maps; pedological classifications ranged from relatively inorganic topsoils to the more highly organic topsoils present in north central Illinois. The sites included cultivated fields and areas with natural vegetation. A prime criterion was that the topsoil should have a definite black or very dark brown coloration such that removal and replacement would normally be required. Actual organic contents ranged from 1.5 to 9.6 percent.

Field sampling was accomplished in test pits at four different levels. The four levels tested consisted of:

1. The upper 0 to 4 inches which is normally bladed away when clearing grass and weeds or other recent organic deposits from a site.

2. The layer of organic soil between 4 inches and 12 inches depth (provided the organic soil extended to at least 12 inches).

3. All remaining organic soil (if any) at a depth greater than 12 inches.

4. The non-organic soil which immediately underlies the organic soils that are commonly removed.

Basic index properties of each soil specimen including grain size analyses, color, organic contents, Atterberg limits, and pH were determined for each specimen. In addition, the moisture-density relationships of each material were determined. Specimens were molded and tested in accordance with standard procedures to determine their bearing ratio. Laboratory compacted specimens were tested for their consolidation characteristics at densities equal to their in situ density and at 95 percent of their laboratory maximum dry density.

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These properties were then compared to determine relative need for topsoil removal and later potential usage as fill material. Comparisons were made with previously developed empirical relationships for maximum dry density and optimum water content based on liquid limit and for compression index based on liquid limit. Good correlation was obtained from the moisture-density comparisons, but the compression index comparisons were generally unsatisfactory. It is believed that the unsaturated conditions of the specimens were responsible for the lack of correlation.

It was determined that for most construction sites, there was really no need to remove more than the upper soil layer of about 4 inches. This layer contains the majority of the roots and matted recently deposited organic material and it is usually easily removed when blading crop or weed debris from a site before beginning major construction operations.

In many cases, in order to establish a desired grade, topsoils must be removed. In those cases, all but the upper layer may be used elsewhere on the project as compacted fill, particularly if some mixing of soil layers is effected during placement.

TEXT OF PRESENTATION

Organic topsoils are commonly removed from the sites of structures and from the alignments of roadway or runway embankments prior to placing fill material. This practice has developed as an attempt to reduce the amount of structure or embankment settlement due to consolidation or undesirable volume changes in the in situ organic soils. The low shearing strength of organic topsoils is also frequently cited as a reason for the removal of these soils.

Some of the organic topsoil material is usually stockpiled for reuse as embankment topping or as topsoil around a structure. However, in many cases the topsoil is merely wasted while more suitable fill material is secured and used. An attempt to market the surplus topsoil in the local area is sometimes made. Unfortunately, in areas where considerable topsoil must be removed for the proposed construction, the topsoil is also present throughout the area in considerable quantity and the market is not substantial.

The cost of this removal and replacement can become quite high, as, for example, the cost of six inches of stripping and six inches of replacement backfill under a typical embankment on the Interstate Highway System may exceed $35,000 per mile.

A research program was conducted to investigate the behavior of organic topsoils with reference to their consolidation and strength characteristics and to develop criteria for evaluation of topsoils as to their suitability for use. An indication was desired as to whether the topsoil should be removed, the depths of optimum removal; and, if removed, could the topsoil be used as compacted embankment.

GENERAL

Organic colloids exist in soils as a direct result of decay of vegetation and occur generally as residual deposits. Erosion and subsequent deposition does, of course, account for a significant amount of displaced organic material. Decomposition of organic matter is a function of environment. In tropical climates, organic matter decomposes quickly, while in arctic and sub-arctic climates, decomposition occurs slowly. Deposition of organic colloids in soils in depressions can easily reach thicknesses of 3 to 4 feet.

Many years ago, structures and roadways were not constructed over peat deposits. Further, traffic speeds, vehicle loadings and vehicle operating characteristics were such that horizontal alignments and vertical grades were not as important as today. Roadways were built on topsoil and were located to avoid obvious peat deposits. At that time, heavily loaded structures were usually founded below the topsoil materials while light structures were occasionally founded on the topsoil.

With increasing need for better alignment of roadways and heavier traffic loads, as well as an increasing shortage of desirable building locations, experience with building over peaty areas was gained. A large portion of this experience bordered on the disastrous with severe settlements, distortions and failures in many instances. Highway engineers began observing failures in roadways built upon topsoils, and reasoning that topsoils were merely a milder or less severe form of peat, began specifying removal of organic materials. Apparently, little thought was given to the possibility that heavier and more frequent traffic loadings on an inadequate pavement structure or any of several pavement deteriorating factors may have been, at least partly, responsible for the failures rather than the presence of topsoil below the pavement.

It was noted that large structures founded below the topsoil performed satisfactorily while light structures founded on the topsoil occasionally experienced difficulty. Little credence was given factors such as frost heave which may have been partially responsible for the structural distress rather than the effects of the topsoil. The practice of removal and replacement of topsoil continues today.

The principal concentration of engineering research in this field has been concerned with behavior of peat and muskeg because of the rather severe problems associated with construction on these materials. Very little research has been conducted on the effects of organic matter in concentrations of less than 20 percent on the behavior of soils. Notable exceptions are the studies by Schmidt (1965) and Franklin, et al (1973).

It is generally recognized that the presence of colloidal organic matter imparts detrimental engineering properties to soils. For example, Lambe and Martin (1956) have stated that many engineering failures have been associated with clays containing montmorillonite clay minerals and organic matter. They fail, however, to state whether the failures were due to the montmorillonite or the organic matter or both.

DEFINITION OF ORGANIC SOILS

A. Casagrande (1947) gave three basic criteria for the identification and potential behavior of organic soils:

1. An organic soil has either an organic odor or a dark earth color, or both.

2. An organic soil shows a substantial drop in liquid limit when the soil is oven-dried.

3. The Atterberg Limit values of organic soils are commonly plotted below the "A" line on the plasticity chart.

Casagrande used the symbols "OL" and "OH" for organic soils of low and high compressibility, respectively, with a liquid limit of 50 forming the boundary between low and high compressibility. Civil engineers commonly describe organic soils as those possessing sufficient organic material to affect their engineering properties, but not to the degree of highly organic soils such as peat or organic muck. Topsoils possess some organic matter and are usually included in the category of "organic soils."

Much research has been accomplished by persons involved in agriculture/soil science specialties. However, an agronomist reserves the term "organic soils" for peat, muck, or muskeg materials because in agriculture "soil" or "topsoil" is a material capable of supporting plant growth and an organic content is implicit in the terms. In this paper, the civil engineering definition stated in the previous paragraph is used.

RESEARCH APPROACH

This study included a review of previous research on organic topsoils and a comprehensive field and laboratory testing program.

A total of eleven sites were selected in central, north central, and northern Illinois. The site selection was accomplished largely through the use of agriculture soil maps; pedological classifications ranged from relatively inorganic topsoils to the more highly organic topsoils present in north central Illinois. The sites included nine cultivated fields and two areas with natural vegetation. A prime criterion was that the topsoil should have a definite black or very dark brown coloration such that removal and replacement would be required. The site locations are shown in Figure 1. All

SITE
LOCATIONS

FIGURE 1

sites were within an area covered by Wisconsinan glaciation. The surface soils are predominantly of loessial orgin.

No site was located in an area with slopes greater than 2 percent within 100 yards of the sampling location. The minimum thickness of topsoil encountered was about 12 inches and the maximum thickness was approximately 3 feet.

It was recognized that field and laboratory tests on undisturbed samples would be affected by the climatic conditions prevailing just prior to the time of sampling. Climatic data and existing moisture conditions were therefore an important consideration in the research program.

The field investigation consisted of an in situ strength and dry density determination on each level of soil sampled. The sampling was then accomplished in test pits at four different levels. The four levels tested were:

1. The upper 0 to 4 inches which is normally bladed away when clearing grass and weeds or other recent organic deposits from a site. (Level A)

2. The layer of organic soil between 4 inches and 12 inches depth (provided the organic soil extends to at least 12 inches). (Level B)

3. All remaining organic soil (if any) at a depth greater than 12 inches. (Level C)

4. The non-organic soil which immediately underlies the organic soils that are commonly removed. (Level D)

Basic index properties of each soil specimen including grain size analyses, color, organic contents, Atterberg Limits, and pH were determined for each specimen. In addition, the moisture-density relationships of each material was determined. Specimens were molded and tested in accordance with standard procedures to determine their bearing ratio. Laboratory compacted specimens were tested for their consolidation characteristics at densities equal to their in situ density and at 95 percent of their laboratory maximum dry density. Table No. 1 lists the tests conducted.

TABLE NO. 1

Field Tests	Lab Tests
In Situ Density	Grain Size Analyses
Natural Water Content	Atterberg Limits
Cone Penetrometer	pH
Pocket Penetrometer	Color
	Organic Content
	Moisture-Density Relationships
	Bearing Ratio
	Consolidation
	A. Field Moisture-Density
	B. 95% Standard (wet side)

The various test results were then correlated illustrating the different levels of tests with the properties which may be anticipated. Of particular importance was the correlation of the lower topsoil layers and their behavior with that of the subsoil which immediately underlies these materials. The consolidation characteristics of the compacted organic topsoil specimens were also compared with the characteristics of the relatively non-organic specimens.

The lower levels were reached by excavation of a 3-foot by 6-foot pit with a hand shovel to the depth desired. Care was taken to stand only along one side of the pit and perform tests on the opposite side. Table No. 2 gives typical ranges of field test values.

TABLE NO. 2

RANGE OF FIELD TEST VALUES

Test	Range
Shear Strength	0.6 TSF to 7.2+ TSF
Color (Visual)	Black to Brown
In Situ Dry Density	51 PCF to 99 PCF
Natural Water Content	13% to 26%

A minimum of 25 shear strength readings by pocket penetrometer and cone penetrometer were made on each level of soil tested at each site. Dry density and water content determinations were averaged from a minimum of 5 tests each on each level tested. A detailed description of the field testing procedure is given by Bartholomew (1974).

As the pits were excavated approximately 100 pounds of soil from each level at each site were placed in polyethylene bags. Extreme care was taken not to allow any mixing of soil from different layers. Air-tight jar samples were also secured from each layer.

The polyethylene sample bags were tied securely and stored in a concrete curing moist room. The temperature in the room varied from 68 degrees F. to 75 degrees F. and the relative humidity from 60% to 100%.

Except for the water content specimens, all tests were preceded by air-drying a sufficient quantity of soil for the test. Air-drying was accomplished in metal trays or on small plastic sheets. Occasionally, a small fan was used to speed drying. The laboratory air-conditioning system assured that the samples were never exposed to temperatures higher than about 75 degrees F. before testing.

As air-drying progressed, the samples were broken into small pieces by hand. When the soil was air-dry, large samples were pulverized in a mechanical mixing device, and small samples were pulverized in a hand grinder. The material retained on a No. 10 sieve was reworked with mortar and pestle until essentially all soil passed the sieve. For certain tests, the pulverizing continued until the soil particles passed a No. 40 sieve.

Mixing of air-dry soil with water was accomplished in a mechanical mixer if large batches were required. Small batches were thoroughly mixed by hand in suitably sized containers or on a mixing board with a Formica surface. Because of a potential shortage of soil, portions of the moisture-density and bearing ratio samples were saved for later re-use if needed. The portions saved had not been exposed to oven drying.

LABORATORY TESTING PROGRAM

The entire laboratory testing program conformed as closely as possible to applicable American Society for Testing Materials (ASTM) and/or American Association of State Highway and Transportation Officials (AASHTO) specifications for testing. Exceptions were the determination of bearing ratio and consolidation characteristics. The bearing ratio tests were performed in accordance with Illinois Department of Transportation standards and the consolidation tests were conducted on statically remolded soils in accordance with generally accepted soil testing standards. Organic contents were determined by the wet oxidation method known as the "Walkley-Black" method.

Nearly all tests were repeated at least once or tests were run concurrently on duplicate samples. The lone exception was the consolidation testing which requires rather arduous, time-consuming procedures. In the initial phase of the consolidation testing program, duplicate tests were run. After gaining confidence that the method of molding and the test procedures were yielding proper results, single consolidation tests were subsequently run for each sample level.

TABLE NO. 3

RANGES OF LABORATORY TEST RESULTS

Test	Range
Color (Munsell)	10 YR 2/1 to 10 YR 3/4
Organic Content	1.5% to 9.6%
pH	6.5 to 8.1
Liquid Limit	non-plastic to 56%
Plasticity Index	non-plastic to 30%
AASHTO Classification	A-2-4 (0) to A-7-6 (18)
Unified Classification	SM to CL
Maximum Density (AASHTO T-99)	86 PCF to 114 PCF
Optimum Moisture	11% to 26%
Bearing Ratio	1.3 to 7.9
Percent Swell	0.05 to 3.00
Compression Index (Field Conditions)	0.13 to 0.81

Detailed descriptions of the test methods used and the results obtained are given by Bartholomew (1974). Figure 2 illustrates how the soils were classified by the Unified System. It may be seen from this figure and from Table No. 3 that the soils do represent a rather wide range of conditions. Generally the organic content decreased with depth and the plasticity, bearing ratio, and density increased with depth.

All consolidation tests were on soils statically remolded to either field condition density and water content or to 95% of the laboratory density. The water content used for molding the 95% density specimens brings up the problem of selecting either the wet or dry side of the curve. Figure 3 illustrates comparative tests. After establishing this relationship, the wet condition was used throughout the remainder of the research.

DISCUSSION OF TEST RESULTS

It should be noted that the in situ properties discussed herein represent somewhat theoretical conditions when viewed from a practical standpoint. For example, the strength and density information obtained, while representative of the actual conditions existing before construction, would certainly be altered as construction progressed. The low densities existing in the upper soils would be increased as compaction of a fill above them progressed. As the density increases, the strength would also be expected to increase. More importantly, a portion of the anticipated consolidation would be occurring before completion of an overlying embankment or structural fill. The consolidation of these soils that occurs during construction would have no influence whatsoever on a completed embankment or structure built upon them.

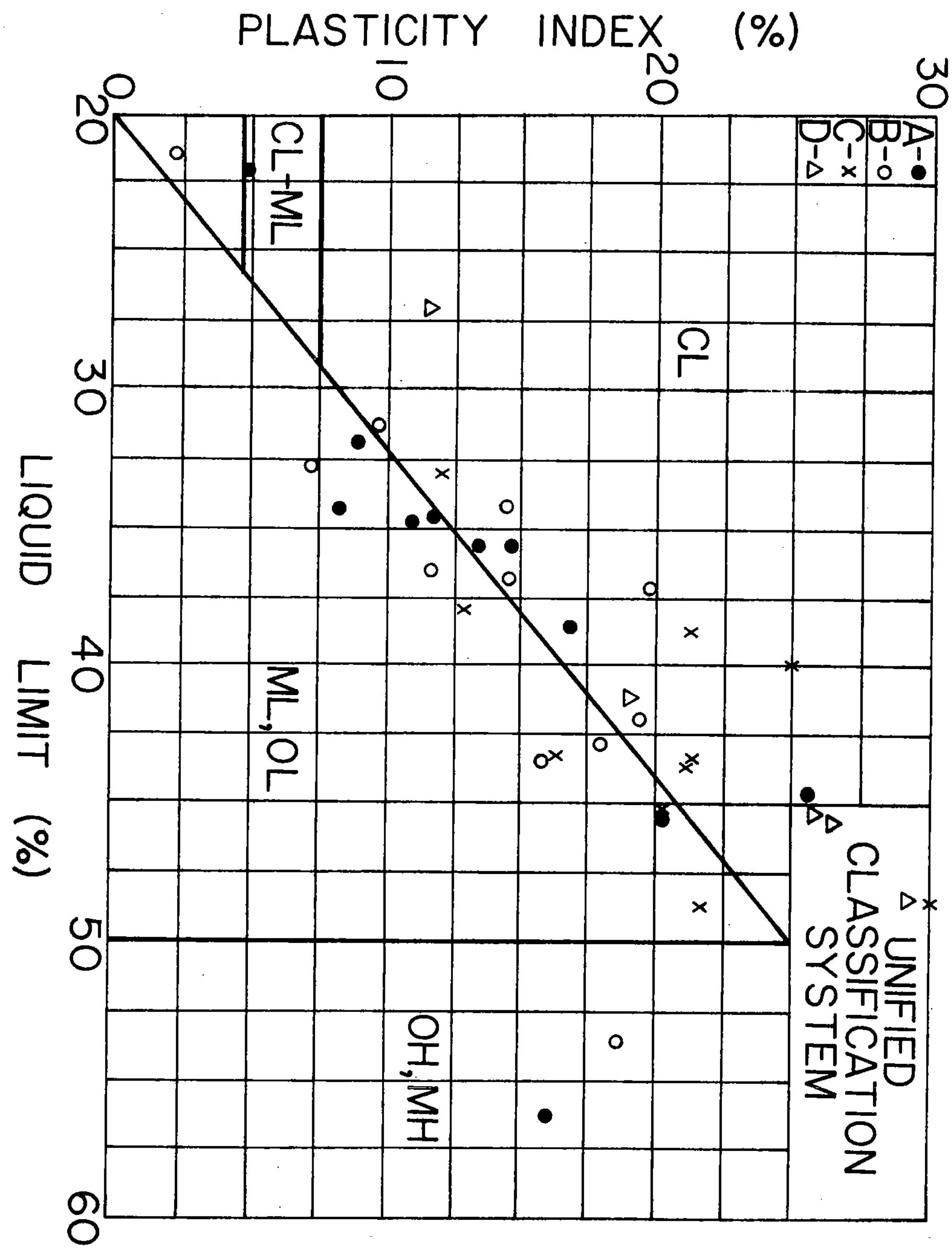
PLASTICITY INDEX (%)
LIQUID LIMIT (%)
0
10
20
30
20
30
40
50
60
A-●
B-○
C-x
D-△
CL-ML
CL
ML,OL
OH,MH
UNIFIED CLASSIFICATION SYSTEM

FIGURE 2

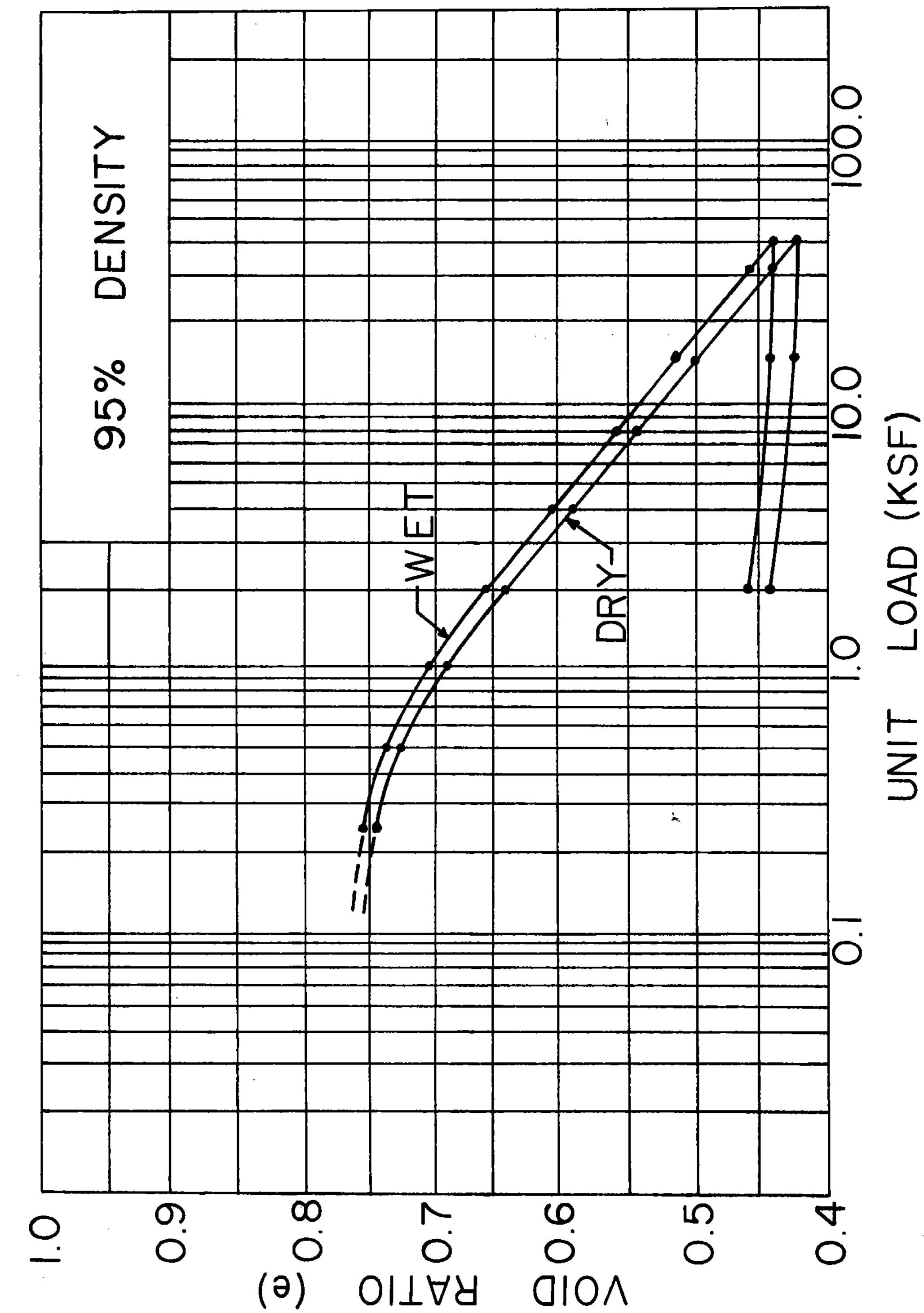
95% DENSITY
WET
DRY
1.0
0.9
0.8
0.7
0.6
0.5
0.4
VOID RATIO (e)
0.1
1.0
10.0
100.0
UNIT LOAD (KSF)

FIGURE 3

The strength test results were very scattered; the plotted data indicated only that strength generally increases with increasing density. At several sites, levels B and C strengths were higher than those obtained for subsoil level D.

Consolidation test results on field condition specimens exhibited considerable variation. It was believed that most of this variation was due to the initial low density and unsaturated condition of the specimens. Compression index results did not compare favorably with the relationship: $C_c' = .007(LL-10)$ presented in Terzaghi and Peck (1948). Compression indices appeared unrelated to molding water content, but were definitely related to density. As shown in Figure 4, only above organic contents of about 5% did it appear that the compression index was related to organic content.

On an overall basis, level A soils had compression indices 43.7 percent higher than the subsoil; level B had compression indices 16.6 percent higher than the subsoil; and level C had compression indices 11.5 percent higher than the subsoil (at the six sites where a subsoil level D existed).

When viewed from the standpoint of the densification of soils due to compaction of overlying soils, it was concluded that level B and C would likely exhibit less than 10 percent greater consolidation than the subsoil strata if left in place. Therefore, it appeared that only soils in level A need be removed.

It is commonly accepted that the Atterberg Limits are an indication of a soil's workability, compactibility, and potential for volume change. For most engineering purposes, lower plasticity soils are more desirable than high plasticity soils. In this study, it was found that the lower level soils had higher plasticity indices than did the upper soils, and on this basis, removal of the upper soils would not be necessary. If removed, however, the topsoil layers would possess plasticity characteristics suitable for compaction in embankments.

The laboratory moisture-density relationships yielded a density range of 85 PCF to 115 PCF over an organic content range of 9.6% to 1.5%. However, at about 2% organic content, the density range was 95 to 115 PCF. It can be concluded that below about 5% organic content, the maximum dry density is controlled by factors other than organic content such as textural or plasticity characteristics.

The bearing ratio tests yielded typical increases of about 0.5 for each 1% decrease in organic content. The bearing ratio increased markedly with increasing density. Level A soils consistently yielded the lowest results.

Consolidation tests on specimens molded to 95% density conditions achieved a much improved fit with the empirical relationship, $C_c' = .007(LL-10)$, probably becuase of the greater molding density and correspondingly higher saturation.

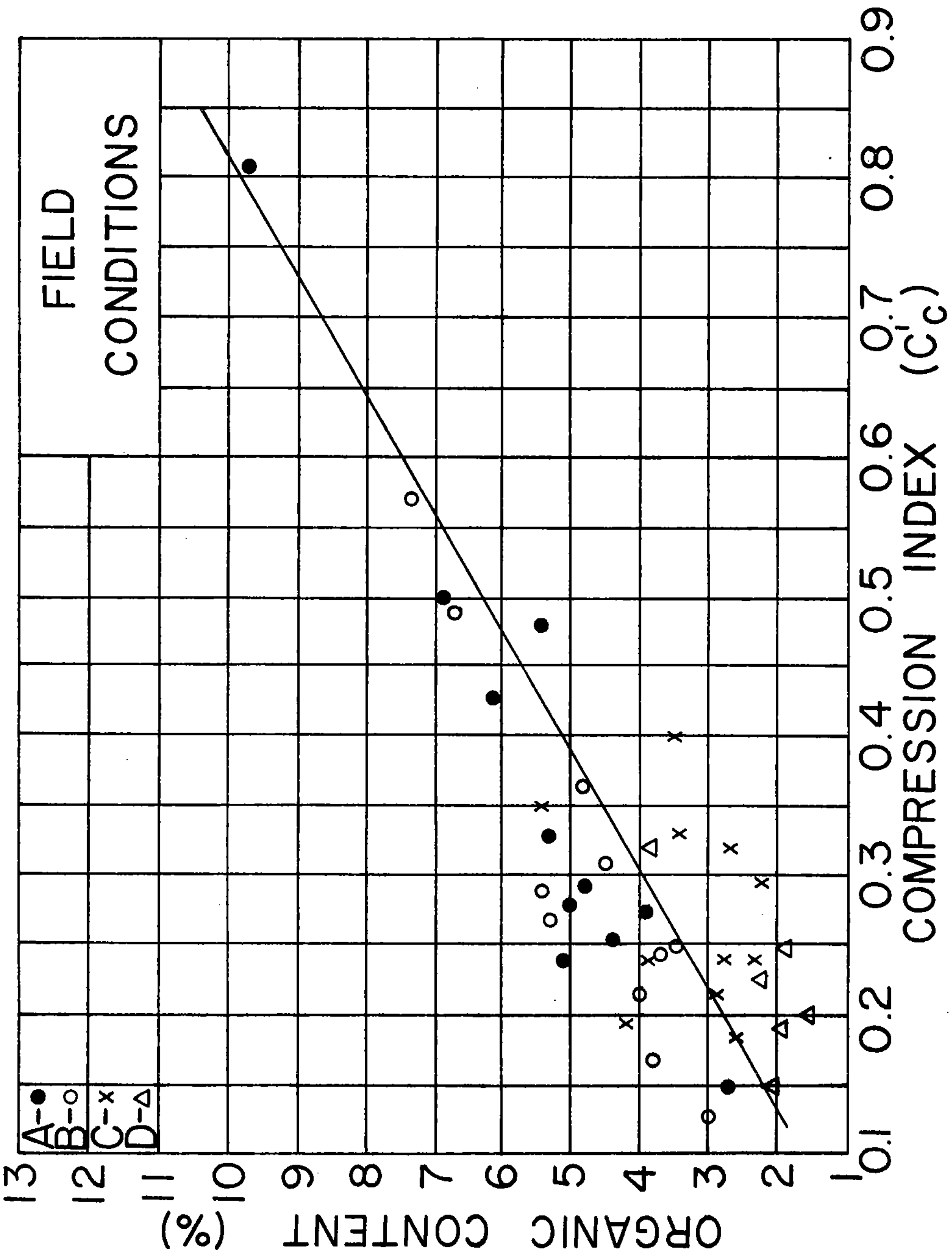
FIELD CONDITIONS
A-●
B-○
C-×
D-△
ORGANIC CONTENT (%)
1 2 3 4 5 6 7 8 9 10 11 12 13
COMPRESSION INDEX (C'c)
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9

FIGURE 4

Compression index is definitely related to dry density of specimens as shown in Figure 5. The equations of these parallel lines are:

<u>Field Conditions:</u> Dry Density = 43 + 67LOG $\frac{1}{C'_c}$

<u>95% Density Conditions:</u> Dry Density = 51 + 67LOG $\frac{1}{C'_c}$

SUMMARY AND CONCLUSIONS

At least six important factors need to be considered in arriving at design level or on-site decisions relative to the need for topsoil removal and the possibility of later use of topsoil as compacted fill.

1. Level A in this research program consisted of the upper 4 inches of soil at each site. Some or all of this layer is normally removed from a construction site or a borrow area in order to clear away vegetation prior to actual construction operations. Only in very rare cases is construction begun on a site which does not require some amount of clearing. The depth of cut in the clearing operation depends on the type and amount of vegetation present and the skill of the equipment operators.

2. This research program did not investigate the effects of mixing of soils from different layers. It is nearly impossible in normal construction operations to remove soil at one location, transport it to another, place it and fully compact it without achieving some intermixing. Indeed, many borrow areas are planned for excavation to progress at an angle to the ground surface in order to achieve desired mixing. Mixing of soils from this program would tend to improve the properties of poorer soils and diminish the desirable properties of better soils. In other words, it would be expected that all mixed soils would tend to approach an overall average condition.

3. Little is known of the effects on a soil layer when the layer immediately overlying it is undergoing densification by compaction. It is assumed that, under normal circumstances, the lower layer is also undergoing densification, but to a lesser degree. Such an effect becomes very important to the question of whether or not to remove a layer of soil. If, for example, a soil layer is undesirable solely because of its low density, it can be improved by compaction of the subgrade surface or the first layer of fill.

4. In determining whether to remove topsoil at a site, one must be careful to view the potential problems in their proper perspective. For example, a 20-foot high embankment constructed over 16.5 feet

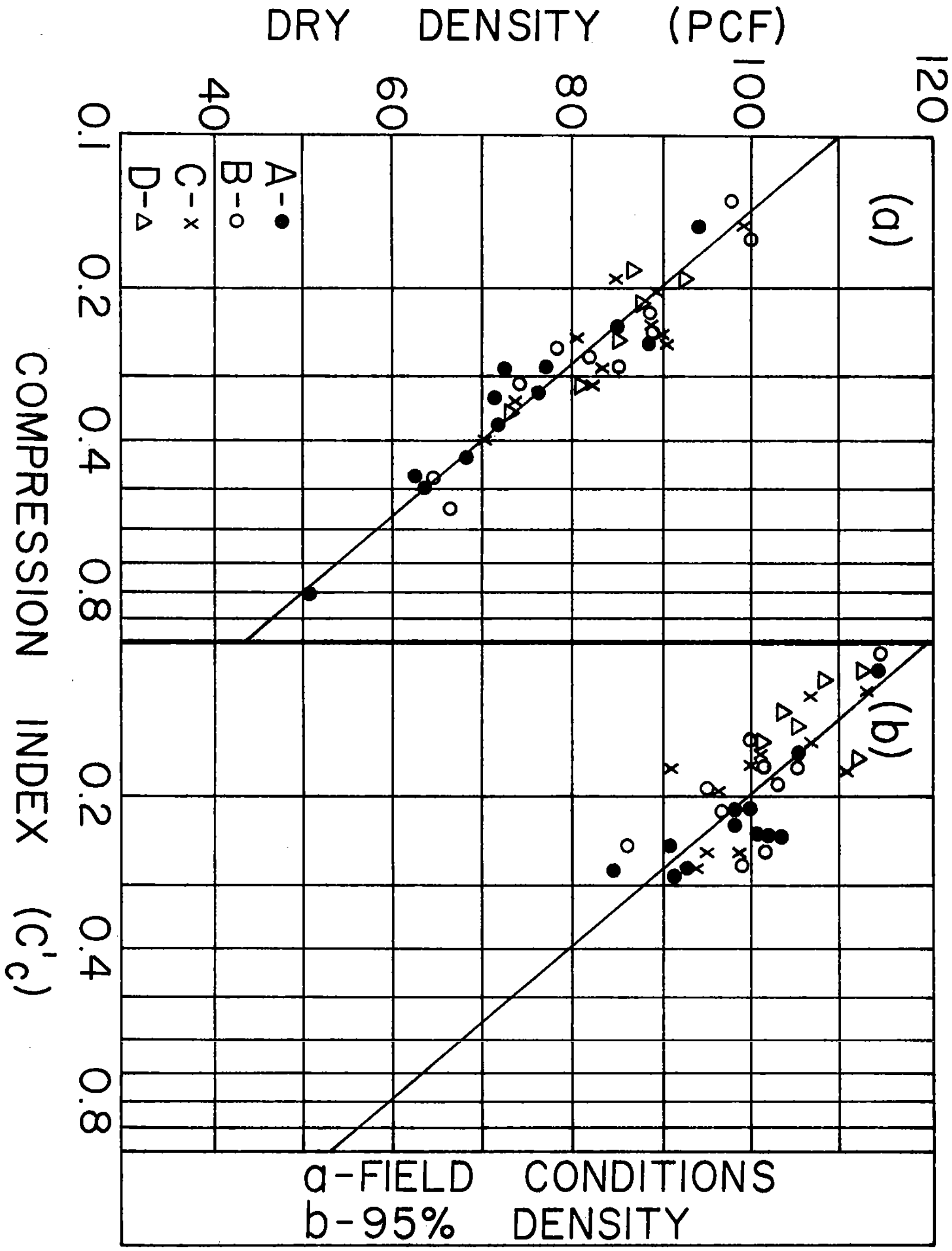
DRY DENSITY (PCF)
40
60
80
100
120
0.1
0.2
0.4
0.8
0.2
0.4
0.8
COMPRESSION INDEX (C'c)
(a)
(b)
A-●
B-○
C-x
D-△
a-FIELD CONDITIONS
b-95% DENSITY

FIGURE 5

of relatively compressible soils may settle appreciably. However, if only the upper 18 inches of compressible soil is "topsoil", the contribution to settlement of the topsoil stratum would not be significant. In fact, the contribution to settlement of the topsoil stratum may not be significant even if the subsoils are only of medium to relatively low compressibility.

5. Research presently being conducted in the Civil Engineering Department at the University of Illinois appears to indicate that soils with significant organic contents tend to yield lower strengths than comparative inorganic soils when subjected to repeated loading. This would be of no great importance to the behavior of most building foundations, but could be quite important to the behavior of pavements. Pending final results of the repeated loading research, it is suggested that organic soils should not be present in the upper two feet below a pavement.

6. Several additional factors were not investigated as a part of this research. Such environmental factors as potential frost susceptibility and in situ permeability could represent important limiting conditions on some projects.

When all factors are considered, it is clear that for most situations there is really no need to remove more than the upper soil layer of about 4 inches. This layer contains the majority of the roots and matted, recently deposited organic material, and it is usually easily removed when blading crop or weed debris from a site before beginning major construction operations.

In many cases, in order to establish a desired grade, topsoils must be removed. In those cases, all but the upper layer may be used elsewhere on the project as compacted fill, particularly if some mixing of soil layers is effected during placement. It is suggested that for areas subject to repeated loadings, the use of organic soils be limited so that none are allowed in the upper two feet below a pavement.

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A NEW ANALYTICAL APPROACH TO SPLIT TENSILE STRENGTH OF PAVEMENT MATERIALS

By

S. Kumar,[1] M. Annamalai,[2] and J. G. Laguros[2]

ABSTRACT

In a pavement system, it is well known that the base, subbase and subgrade layers undergo both compressive and tensile strains under vehicular traffic. The materials used in pavement usually have low tensile strength. Consequently they tolerate low tensile strains. In view of this, the long-term durability of the pavement appears to be controlled by the tensile strength of the pavement materials.

Split tensile strength tests are often used to estimate tensile strength of the pavement materials. In conventional analysis of split tensile tests, tensile strength is calculated from the failure load, completely disregarding any deformation that may occur during testing. However, in soil materials and asphaltic composites, deformations do occur and it is necessary to take deformation into account for a more precise estimation of tensile strength of these materials.

In this study, a new analytical method is developed in which the elastic deformation of the sample in the direction of loading is considered. Formulas for calculation of tensile and compressive stresses at the center of the sample are presented. Stress coefficients for unit load are presented for the most practical range of values of strain.

INTRODUCTION

When a layer of subgrade material is subjected to a moving load on the pavement, the area immediately below the load is compressed while the areas in front and behind the load undergo tension. On the underside of the layer the strain pattern is just the opposite. A relatively large magnitude of tensile strain compared to a small compressive strain develops at the underside of the layer and it is from this side, most probably, the crack formation initiates (3). The tension parameters at the bottom of the layer appear to be the critical factors. These parameters can be measured in the laboratory using either the flexural test or the tension tests. Generally, the tension tests require much less material and samples can be molded using the compaction equipment commonly available in most soil lab-

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oratories. Since the failure in tension test samples starts inside the soil mass, the wall effects, and the effects of surface conditions are not the determining factors; thus, using tensile tests in some ways has distinct advantages.

The most commonly used tension tests are the direct tensile strength and split tensile strength tests. The results from direct tensile strength tests are greatly influenced by the end constraints; also in such tests failure of specimen can occur along any surface. In split tensile strength tests, the surface of failure for all soil and concrete like materials is pretty well defined and is located in the neighborhood of the vertical diametral plane of the specimen. Thus, split tensile strength test appears to be the preferred test to measure the tensile strength parameters of pavement materials.

ANALYSIS OF SPLIT TENSILE STRENGTH TEST

For the split tensile strength test it is assumed that the stresses and strains at all times and at all points remain within the elastic range and in particular Hooke's law holds up to the failure (4).

Conventional Approach

From the point of view of the theory of elasticity, for a load P acting vertically along a diametral plane of a cylinder of radius R and length L, the stresses at a point (x, y) in the plane of the disk (Figure 1) as given by Frocht (2) are:

$$s_x = \frac{-2P}{\pi L}\left[\frac{(R-y)\,x^2}{r_1^4} + \frac{(R+y)\,x^2}{r_2^4} - \frac{1}{2R}\right] \tag{1}$$

$$s_y = \frac{-2P}{\pi L}\left[\frac{(R-y)^3}{r_1^4} + \frac{(R+y)^3}{r_2^4} - \frac{1}{2R}\right] \tag{2}$$

$$t_{xy} = \frac{2P}{\pi L}\left[\frac{(R-y)^2\,x}{r_1^4} + \frac{(R+y)^2\,x}{r_2^4}\right] \tag{3}$$

where, s_x = normal stress in x-direction

s_y = normal stress in y-direction

t_{xy} = shear stress in x-y plane

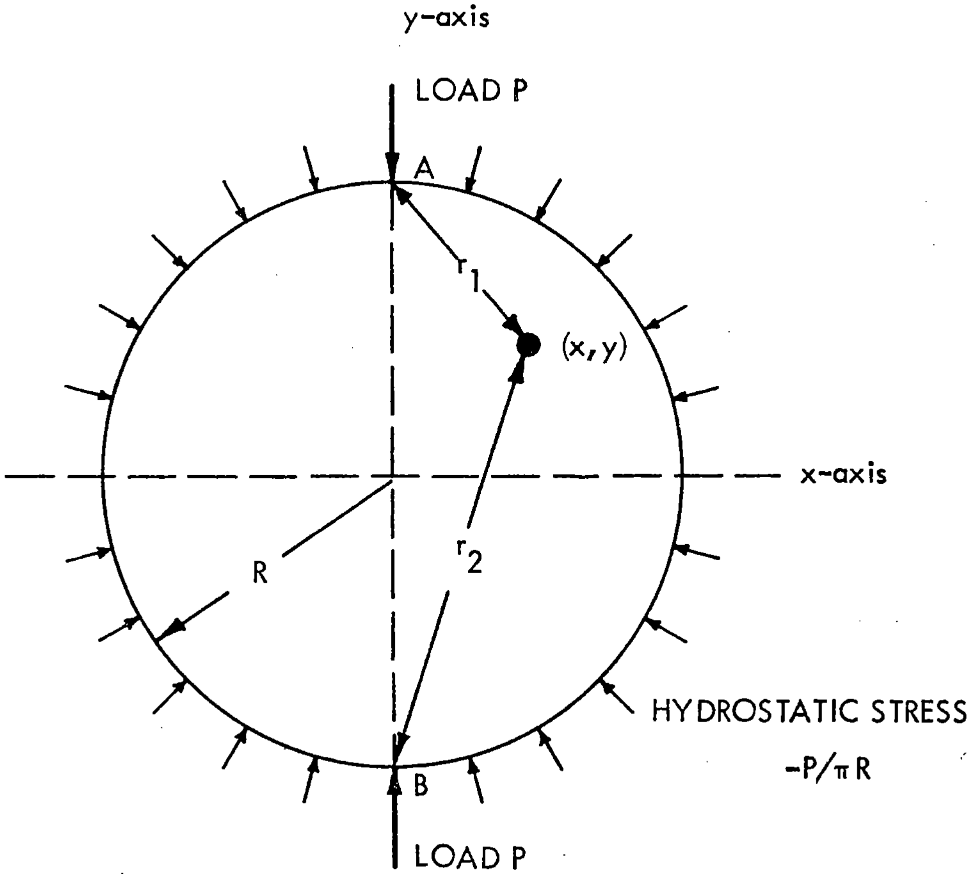

FIGURE 1. Forces acting on a disk and location of point (x, y) in split tensile strength test.

$$r_1^2 = x^2 + (R - y)^2$$

$$r_2^2 = x^2 + (R + y)^2$$

From these equations it is obvious that in the neighborhood of points A and B the stresses are close to those applied by the platens and, thus, the rupture will start from these points.

Along the y-axis, $x = 0$, $r_1 = R - y$ and $r_2 = R + y$ and hence,

$$s_{xy} = \frac{P}{\pi LR} \tag{4}$$

$$s_{yy} = \frac{-P}{\pi L}\left[\frac{4R}{(R^2 - y^2)} - \frac{1}{R}\right] \tag{5}$$

$$t_{xyy} = 0 \tag{6}$$

The distributions of s_{xy} and s_{yy} are shown on Figure 2(a).

Along the x-axis, $y = 0$, $r_1^2 = r_2^2 = x^2 + R^2$ and hence

$$s_{xx} = \frac{-P}{\pi LR}\left[\frac{R^2 - x^2}{R^2 + x^2}\right]^2 \tag{7}$$

$$s_{yx} = \frac{P}{\pi LR}\left[\frac{4R^4}{(R^2 + x^2)^2} - 1\right] \tag{8}$$

$$t_{xyx} = 0 \tag{9}$$

The distributions of s_{xx} and s_{yx} are shown in Figure 2(b).

An examination of these expressions indicates that in all cases, irrespective of the values of load P and radius R, the rupture will always start at points A and B. Along the y-axis the minimum value of s_{yy} is $-3s_{xy}$ and occurs at the center of the disk. At the contact points A and B the magnitude of s_{yy} becomes infinitely large. In general, however, the load is applied through a strip of width "a" and the limiting value of s_{yy} is P/(aL).

Except in the immediate vicinity of the platen contacts, the stresses produced during the test are mainly compressive and they are small in comparison to the compressive strength of the material; thus, they remain well within the elastic range of the material. The principal compressive stress may reach very high magnitudes without having any influence on the rupture (1, 4). The tensile stress distribution along the vertical axis is uniform and these tensile stresses are the ones which tend to exceed the elastic limit. However, even when the specimen is deformed the tension remains con-

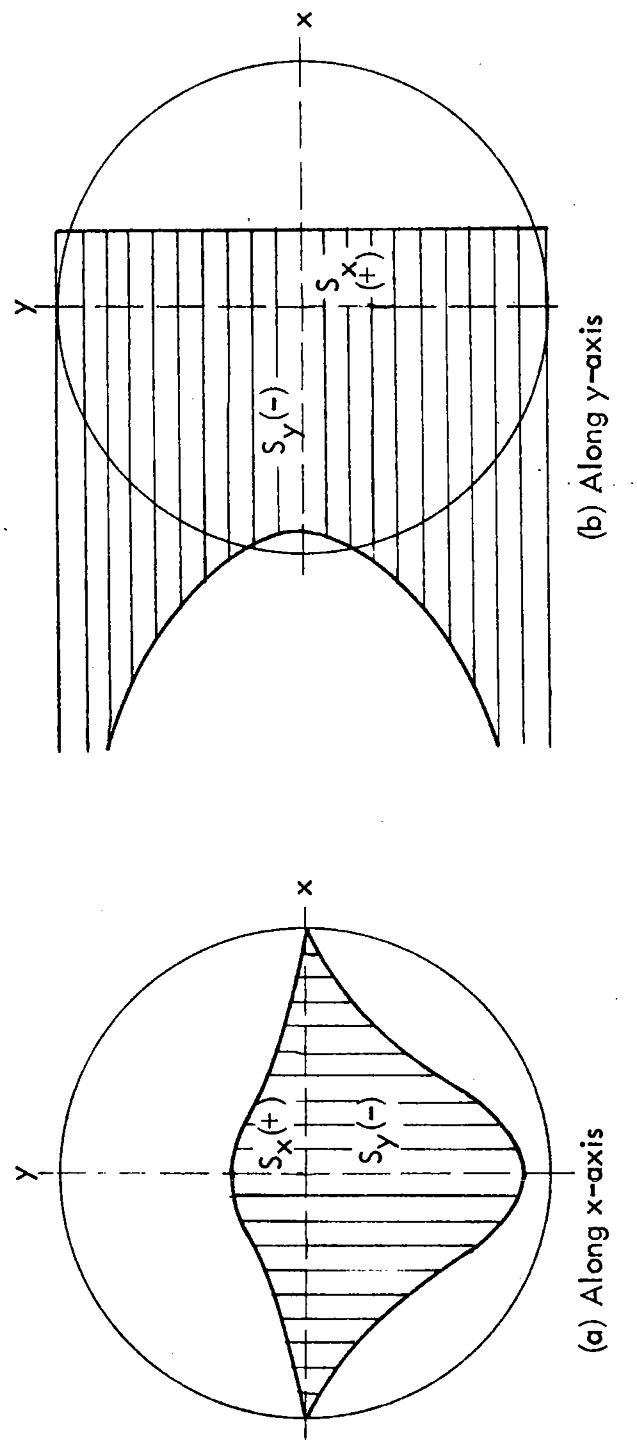

FIGURE 2. Distribution of stresses S_x and S_y along Co-ordinate axes x and y in a disk under vertical diametral compression.

stant, uniform and proportional to the load in the central part of the specimen. The test thus remains a tensile test until failure ensues. The rupture is always sharp and separation is along the vertical plane containing the generators of contact.

Unfortunately, this approach takes no account of the deformation occurring in the specimen. Thus, the analysis provides no information necessary for the actual split tension test conditions where variations in stresses occur due to the deformation taking place in the vertical diametral dimension. Also, in the case of soils the point of failure is generally not well defined and is commonly chosen on the basis of engineering judgement.

New Approach

In this, an attempt has been made to derive equations to take into account such conditions for elastic deformations assuming that the changes in other dimensions of the sample remain negligible.

Referring to Figure 3, let P be the total load applied vertically on a cylinder of length, L and diameter D. Considering that the load applied, P, produces a deformation d and that load is applied symmetrically about y axis along a distance $2x_1$, the load distribution may be assumed as a uniform load of p. Then,

$$P = p\ 2x_1\ L \tag{10}$$

The distance of load along y-axis from the center of the specimen is y_1 and

$$y_1 = (D-d)/2 \tag{11}$$

and, $$x_1 = \tfrac{1}{2}\ (2Dd - d^2)^{\frac{1}{2}} \tag{12}$$

The stresses on an element at (x_e, y_e) due to uniform load p acting on an elemental area dx_1 distance x from y axis and acting vertically from the both ends due to two platens (Figure 3) may be written as:

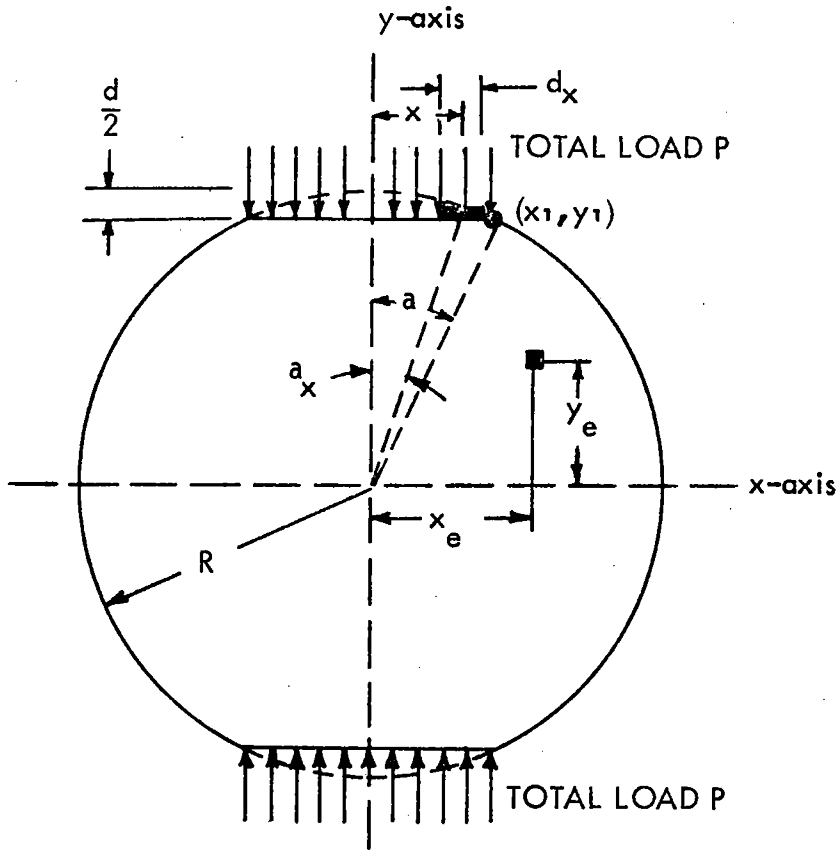

FIGURE 3. Geometrical configuration and locations of points (x_e, y_e) and (x_1, y_1) on a disk deformed by distance d along vertical diameter in split tensile strength test.

$$ds_x = \frac{-2pdx}{\pi}\left[\frac{(y_1 - y_e)(x_e - x)^2}{\left[(y_1 - y_e)^2 + (x_e - x)^2\right]^2} + \frac{(y_1 + y_e)(x_e - x)^2}{\left[(y_1 + y_e)^2 + (x_e - x)^2\right]^2} - \sin(\pi/2 + a_x)/2R\right] \tag{13}$$

$$ds_y = \frac{2pdx}{\pi}\left[\frac{(y_1 - y_e)^3}{\left[(y_1 - y_e)^2 + (x_e - x)^2\right]^2} + \frac{(y_1 + y_e)^3}{\left[(y_1 + y_e)^2 + (x_e - x)^2\right]^2} - \sin(\pi/2 + a_x)/2R\right] \tag{14}$$

$$dt_{xy} = \frac{2pdx}{\pi}\left[\frac{(y_1 - y_e)^2(x_e - x)}{\left[(y_1 - y_e)^2 + (x_e - x)^2\right]^2} - \frac{(y_1 + y_e)^2(x_e - x)}{\left[(y_1 + y_e)^2 + (x_e - x)^2\right]^2}\right] \tag{15}$$

where $\tan a_x = x/y_1$.

The stresses on the element due to the total load P obtained by integrating the expressions (13), (14) and (15) over the limits $-x_1$ and x_1 are presented below:

$$s_x = \frac{P}{2\pi x_1 L}\left[\frac{(y_1 - y_e)(x_e + x_1)}{(y_1 - y_e)^2 + (x_e + x_1)^2} - \frac{(y_1 - y_e)(x_e - x_1)}{(y_1 - y_e)^2 + (x_e - x_1)^2} + \frac{(y_1 + y_e)(x_e + x_1)}{(y_1 + y_e)^2 + (x_e + x_1)^2} - \frac{(y_1 + y_e)(x_e - x_1)}{(y_1 + y_e)^2 + (x_e - x_1)^2} + \arctan\frac{(x_e - x_1)}{(y_1 - y_e)} - \arctan\frac{(x_e + x_1)}{(y_1 - y_e)} + \arctan\frac{(x_e - x_1)}{(y_1 + y_e)} - \arctan\frac{(x_e + x_1)}{(y_e + y_1)} + \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \tag{16}$$

$$s_y = \frac{-P}{2\pi x_1 L}\left[\frac{(y_1 - y_e)(x_e - x_1)}{(y_1-y_e)^2 + (x_e-x_1)^2} - \frac{(y_1 - y_e)(x_e + x_1)}{(y_1-y_e)^2 + (x_e+x_1)^2}\right.$$

$$+ \frac{(y_1 + y_e)(x_e - x_1)}{(y_1+y_e)^2 + (x_e-x_1)^2} - \frac{(y_1 + y_e)(x_e + x_1)}{(y_1+y_e)^2 + (x_e+x_1)^2}$$

$$+ \arctan\frac{(x_e - x_1)}{(y_1 - y_e)} - \arctan\frac{(x_e + x_1)}{(y_1 - y_e)}$$

$$+ \arctan\frac{(x_e - x_1)}{(y_1 + y_e)} - \arctan\frac{(x_e + x_1)}{(y_1 + y_e)}$$

$$\left. + \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \tag{17}$$

and

$$t_{xy} = \frac{P}{2\pi x_1 L}\left[\frac{(y_1 - Y_e)^2}{(y_1-y_e)^2 + (x_e+x_1)^2} - \frac{(y_1 - y_e)^2}{(y_1-y_e)^2 + (x_e-x_1)^2}\right.$$

$$\left. - \frac{(y_1 + y_e)^2}{(y_1+y_e)^2 + (x_e+x_1)^2} + \frac{(y_1 + y_e)^2}{(y_1 + y_e)^2 + (x_e - x_1)^2}\right] \tag{18}$$

Along x axis, $y_e = 0$, therefore,

$$s_{xx} = \frac{P}{2\pi x_1 L}\left[\frac{2y_1(x_e + x_1)}{y_1^2 + (x_e + x_1)^2} - \frac{2y_1(x_e - x_1)}{y_1^2 + (x_e - x_1)^2} - 2 \text{ arc tan}\left(\frac{2x_1 y_1}{y_1^2 + x_e^2 - x_1^2}\right) + \frac{y_1}{R} \ln \frac{R + x_1}{R - x_1}\right] \quad (19)$$

$$s_{yx} = \frac{P}{2\pi x_1 L}\left[-\frac{2y_1(x_e + x_1)}{y_1^2 + (x_e + x_1)^2} + \frac{2y_1(x_e - x_1)}{y_1^2 + (x_e - x_1)^2} - 2 \text{ arc tan} \frac{2x_1 y_1}{y_1^2 + x_e^2 - x_1^2} + \frac{y_1}{R} \ln \frac{R + x_1}{R - x_1}\right] \quad (20)$$

$$t_{xyx} = 0 \quad (21)$$

at the center $x_e = 0$, therefore,

$$s_{xxc} = \frac{P}{2\pi x_1 L}\left[\frac{4x_1 y_1}{R^2} - 2 \text{ arc tan}\left(\frac{2x_1 y_1}{y_1^2 - x_1^2}\right) + \frac{y_1}{R} \ln \frac{R + x_1}{R - x_1}\right] \quad (22)$$

$$s_{yxc} = \frac{-P}{2\pi x_1 L}\left[\frac{4x_1 y_1}{R} + 2 \text{ arc tan} \left(\frac{2x_1 y_1}{y_1^2 - x_1^2}\right) \frac{y_1}{R} \ln \frac{R + x_1}{R - x_1}\right] \quad (23)$$

at the ends $x_e = R$, therefore,

$$s_{xxe} = \frac{-P}{2\pi x_1 L}\left[2 \text{ arc tan}\left(\frac{x_1}{y_1}\right) - \frac{y_1}{R} \ln \frac{R + x_1}{R - x_1}\right] \quad (24)$$

$$s_{yxe} = s_{xxe} \quad (25)$$

Along y axis, $x_e = 0$, therefore,

$$s_{xy} = \frac{P}{2\pi x_1 L}\left[\frac{2x_1(y_1 - y_e)}{(y_1 - y_e)^2 + x_1^2} + \frac{2x_1(y_1 + y_e)}{(y_1 + y_e)^2 + x_1^2} - 2 \arctan\left(\frac{2x_1y_1}{y_1^2 - x_1^2 - y_e^2}\right) + \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \quad (26)$$

$$s_{yy} = \frac{-P}{2\pi x_1 L}\left[\frac{2x_1(y_1 - y_e)}{(y_1 - y_e)^2 + x_1^2} + \frac{2x_1(y_1 + y_e)}{(y_1 + y_e)^2 + x_1^2} + 2 \arctan\left(\frac{2x_1y_1}{y_1^2 - x_1^2 - y_e^2}\right) - \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \quad (27)$$

$$t_{xyy} = 0 \quad (28)$$

at the center $y_e = 0$, therefore,

$$s_{xyc} = s_{xxc} \quad (29)$$

$$s_{yyc} = s_{yxc} \quad (30)$$

at the ends $y_e = y_1$, therefore,

$$s_{xye} = \frac{P}{2\pi x_1 L}\left[\frac{4x_1y_1}{4y_1^2 + x_1^2} + 2 \arctan\left(\frac{2y_1}{x_1}\right) + \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \quad (31)$$

$$s_{yye} = \frac{-P}{2\pi x_1 L}\left[\frac{4x_1y_1}{4y_1^2 + x_1^2} + 2 \arctan\left(\frac{2y_1}{x_1}\right) - \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \quad (32)$$

For an assumed deformation of 2 percent of the diameter and a unit load the distribution of stresses s_{xx}, s_{yx}, s_{xy}, and s_{yy} are shown in Figures 4(a) and 4(b).

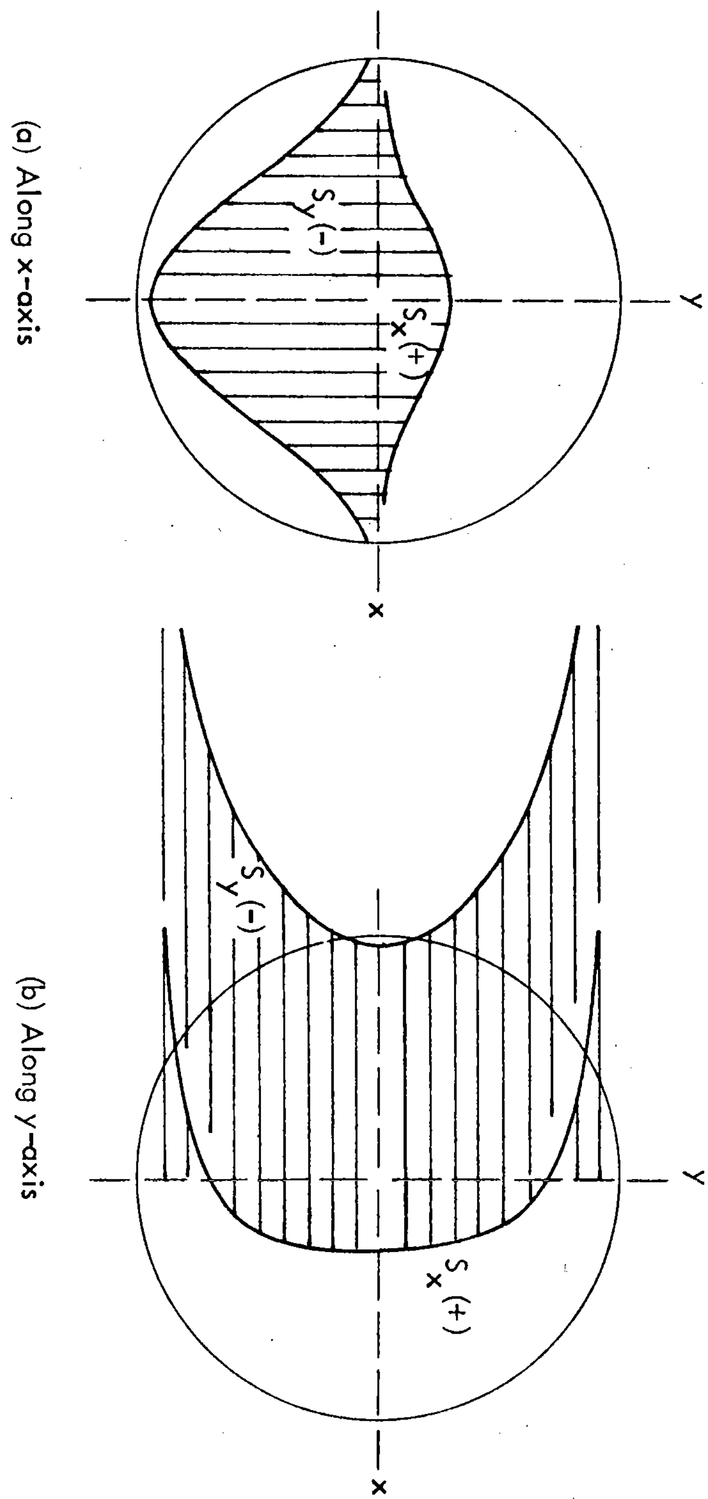

FIGURE 4. Distribution of stresses S_c and S_y along coordinate axes for sample under deformation in split tensile strength test (deformation = 0.02 x diameter).

As will be observed from these figures, the extreme values of stresses still occur either at the center or the ends of the specimen. It should be pointed out that these equations are not applicable to the case of zero deformation since in the case x_1 becomes equal to radius R and the natural logarithm term becomes undefined. Conventional approach is used in the case of zero deformation. However, great importance of these expressions is that they permit plotting of stress strain diagrams for the split tension test in much the same way these diagrams are plotted for unconfined compressive strength and triaxial compression tests.

In summary,

$$s_{xxc} = \frac{P}{2\pi x_1 L}\left[\frac{4x_1y_1}{R^2} - 2 \text{ arc tan}\left(\frac{2x_1y_1}{y_1^2 - x_1^2}\right) + \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \qquad (22)$$

$$s_{yxc} = \frac{-P}{2\pi x_1 L}\left[\frac{4x_1y_1}{R^2} + 2 \text{ arc tan}\left(\frac{2x_1y_1}{y_1^2 - x_1^2}\right) - \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \qquad (23)$$

$$s_{xxe} = \frac{-P}{2\pi x_1 L}\left[2 \text{ arc tan}\frac{x_1}{y_1} - \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \qquad (24)$$

$$s_{yxe} = s_{xxe} \qquad (25)$$

$$s_{xyc} = s_{xxc} \qquad (29)$$

$$s_{yyc} = s_{yxc} \qquad (30)$$

$$s_{xye} = \frac{P}{2\pi x_1 L}\left[\frac{4x_1y_1}{4y_1^2 + x_1^2} + 2 \text{ arc tan}\left(\frac{2y_1}{x_1}\right) + \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \qquad (31)$$

$$s_{yye} = \frac{P}{2\pi x_1 L}\left[\frac{4x_1y_1}{4y_1^2 + x_1^2} + 2 \text{ arc tan}\left(\frac{2y_1}{x_1}\right) - \frac{y_1}{R}\ln\frac{R + x_1}{R - x_1}\right] \qquad (32)$$

$t_{xy} = 0$, along both axes.

Further, by substituting D/2 for R and

$$P = \frac{P}{2\pi x_1 L}$$

$$A = \frac{16x_1y_1}{D2}$$

$$B = \frac{4x_1y_1}{4y_1^2 + x_1^2}$$

$$C = 2 \text{ arc } \tan\left(\frac{2x_1y_1}{y_1^2 - x_1^2}\right)$$

$$D = 2 \text{ arc } \tan\left(\frac{x_1}{y_1}\right)$$

$$E = 2 \text{ arc } \tan\left(\frac{2x_1}{y_1}\right), \text{ and}$$

$$F = \frac{2y_1}{D} \ln\frac{D + 2x_1}{D - 2x_1}$$

Equations suitable for computer programming can be written in the form:

$$s_{xxc} = P[A - C + F] = s_{xyc} \tag{33}$$

$$s_{yxc} = -P[A + C + F] = s_{yyc} \tag{34}$$

$$s_{xxe} = -P[D - F] = s_{yxe} \tag{35}$$

$$s_{xye} = P[B + E + F] \tag{36}$$

$$s_{yye} = -P[B + E - F] \tag{37}$$

$$t_{xy} = 0, \text{ along both axes.}$$

Equations (22) through (25) and (29) through (32) can be simplified further by using the strain values rather than the deformation values.

For a given value of deformation, d, the strain, ε, is equal to d/D

$$\text{or} \quad d = \varepsilon D \tag{38}$$

$$\text{then,} \quad y_1 = \frac{D}{2}(1 - \varepsilon) \tag{39}$$

$$\text{and,} \quad x_1 = \frac{D}{2}(2\varepsilon - \varepsilon^2)^{\frac{1}{2}} \tag{40}$$

and from this a given stress Sijk can be expressed as

$$Sijk = \frac{P}{DL} f_{ijk}(\varepsilon) \tag{41}$$

For unit values of P, D and L, S_{ijk} becomes equal to $f_{ijk}(\varepsilon)$. For a given value of ε $f_{ijk}(\varepsilon)$ has a definite single numerical value which will be called here as "stress coefficient." Stress coefficients for the values of strain ranging from 0.01 to 0.20 are given in Table 1, and shown graphically in Figure 5.

Thus, from the knowledge of the values of d, P, D, L and ε, various stress values can be easily obtained by means of equation (38), Table 1 (or Figure 5) and equation (41).

SUMMARY AND CONCLUSIONS

In conventional analysis the deformation of sample is not taken into account while calculating its split tensile strength. However, in these tests the deformations occuring in soil and asphalt concrete specimens are usually large. Thus, in this study, a new analytical approach has been developed in which the elastic deformation of the sample in the direction of loading is considered. Various stress values at the ends and the center of the sample can be calculated using the formulas developed here. The stress values can be obtained also through a simplified method that utilizes a parameter termed here as "stress coefficient."

TABLE 1. Stress Coefficients for Diametral Strain Values Ranging from 0.01 to 0.20

Strain	S_{xxc}	S_{yxe}	S_{yyc}	S_{xye}
0.01	0.6175	0.0043	1.9035	7.0846
0.02	0.5983	0.0086	1.8973	5.0167
0.03	0.5791	0.0129	1.8910	4.1006
0.04	0.5597	0.0173	1.8849	3.5541
0.05	0.5404	0.0218	1.8788	3.1808
0.06	0.5209	0.0263	1.8728	2.9048
0.07	0.5014	0.0308	1.8668	2.6899
0.08	0.4819	0.0354	1.8609	2.5162
0.09	0.4622	0.0400	1.8551	2.3719
0.10	0.4425	0.0447	1.8493	2.2495
0.11	0.4227	0.0494	1.8437	2.1438
0.12	0.4029	0.0542	1.8380	2.0512
0.13	0.3830	0.0590	1.8325	1.9692
0.14	0.3630	0.0639	1.8270	1.8958
0.15	0.3429	0.0689	1.8216	1.8295
0.16	0.3228	0.0739	1.8163	1.7692
0.17	0.3025	0.0789	1.8111	1.7140
0.18	0.2822	0.0840	1,8059	1.6631
0.19	0.2619	0.0892	1.8008	1.6161
0.20	0.2414	0.0944	1.7958	1.5723

Note: For this range of strain values $S_{xxe} = -S_{yxe}$ and $S_{yye} = -S_{xye}$.

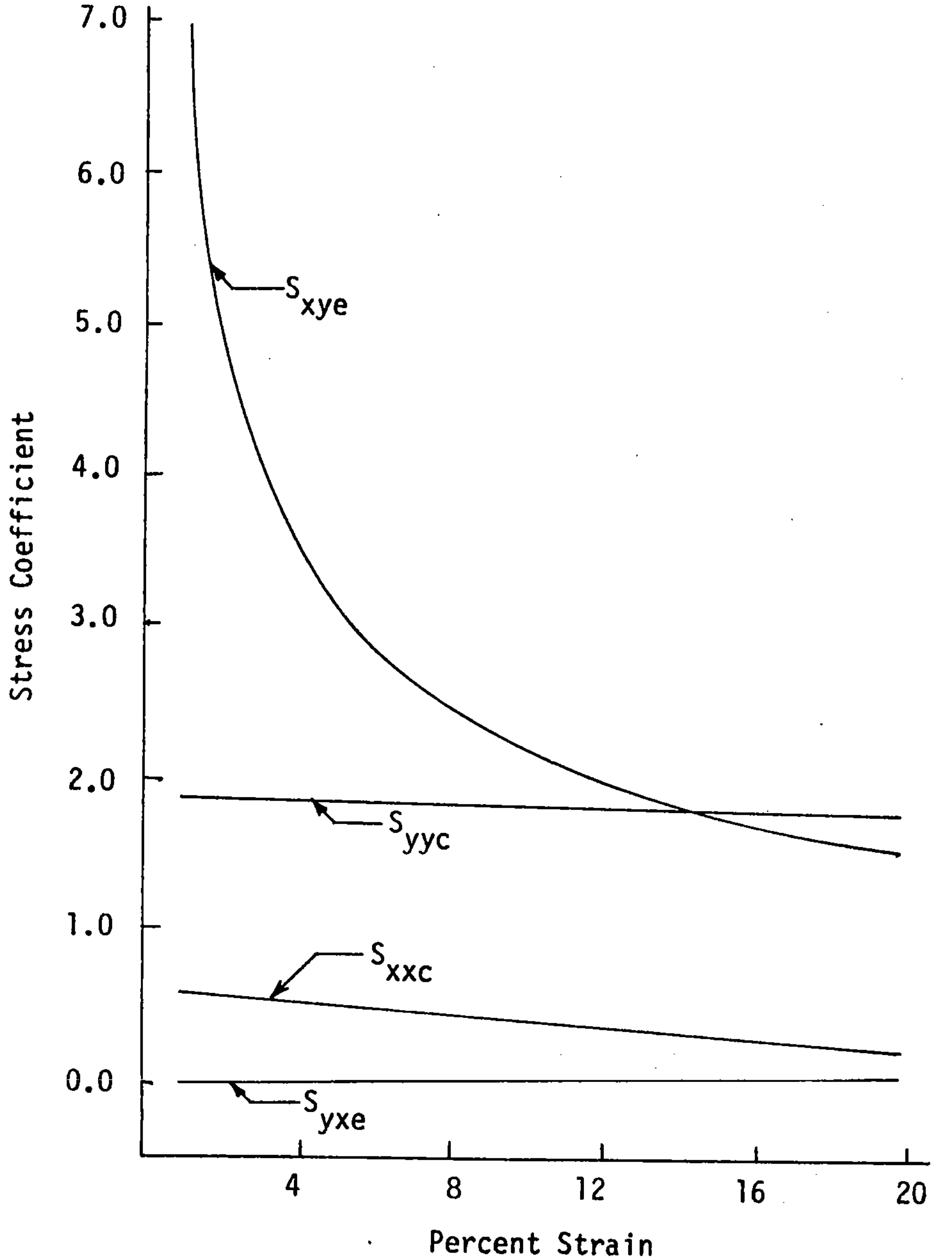

FIGURE 5. Stress Coefficients for Various Values of Strain

ACKNOWLEDGEMENTS

This paper is the result of one phase of research conducted in connection with the study 68-03-2 (OURI 1677) under the joint sponsorship of the Oklahoma Department of Highways and the Federal Highway Administration, U.S. Department of Transportation. The opinions, findings and conclusions presented are those of the authors and not necessarily those of the sponsoring agencies.

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COLLAPSIBLE SOILS
STATE-OF-THE-ART

By

Sam I. Thornton[1] and Kandiah Arulanandan[2]

ABSTRACT

A collapsible soil undergoes subsidence due to wetting, load application or both. Collapse may take several minutes or hours.

Collapsible soils have a loose structure and are geologically young. A major group of collapsible soils are windblown silt size deposits of loess. Collapsible soils are found worldwide, usually in arid areas with large deposits of loess in Europe, Asia and the United States.

Soils may collapse when wetted as a result of destruction of capillary forces, softening of the clay binder, or dispersion of clay and ions at grain to grain contacts.

Identification of collapsible soils is difficult but they may have a low unit weight (less than 80 pounds per cubic foot), low water content (less than 10%), or high void ratio (greater than the void ratio at the liquid limit). Magnitude of collapse may be estimated from a modified consolidation test.

Damage from collapse may be prevented by inducing collapse by wetting before construction, preventing collapse by reducing stress or waterproofing, or solidifying the soil.

INTRODUCTION

A collapsible soil is one that undergoes an appreciable loss of volume upon wetting, load application, or both (Reginatto and Ferrero, 1972). Collapsible soils have a discontinuous stress-strain relationship reflecting a radical change in soil structure either as the result of changes in the stress level or from an increase in the water content (Figure 1). Collapsible soils include quick clays and cemented sands as well as loess and other clayey silts or sands. Collapses are reported to be between 4 and 20 percent of the original soil height (Kassiff and Henkin, 1967). The magnitude of collapse depends on the moisture content, load, and nature of the collapsible soil. Soil collapse is not instantaneous, but may take several minutes or hours.

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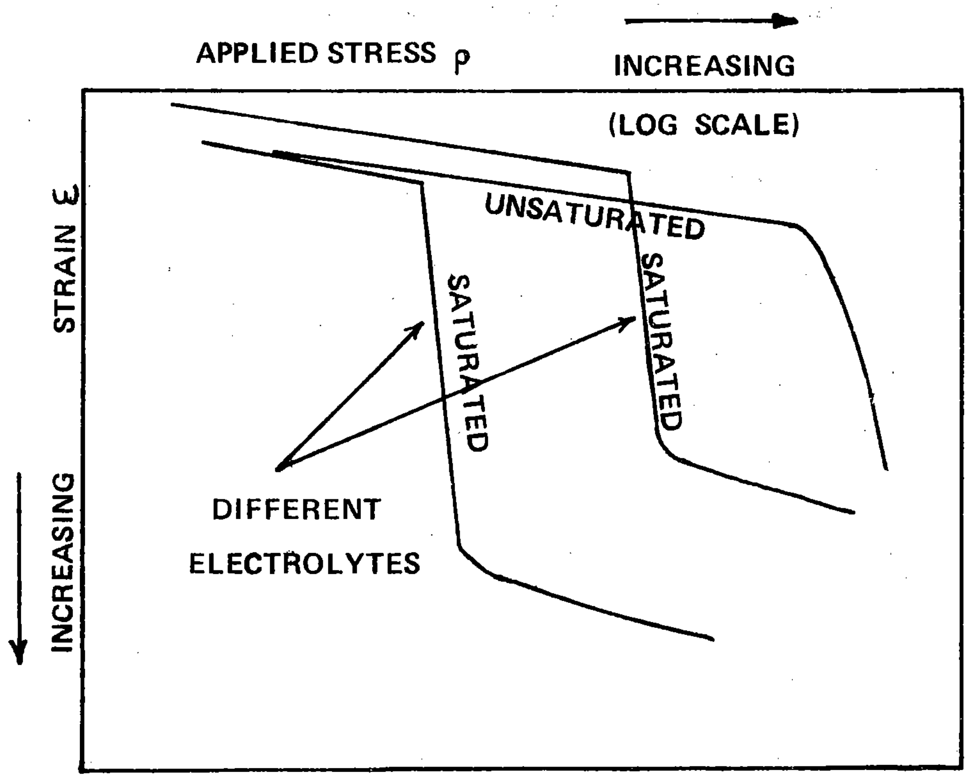


FIGURE 1. STRUCTURAL INSTABILITY SHOWN BY STRESS--STRAIN RELATIONSHIPS

This paper is addressed to collapsible loess and similar soils, i.e. collapse of a soil having a loose structure of silt or fine sand size particles held together by a clay binder.

OCCURRENCE

Collapsing soils are found throughout the world in soil deposits that are loessial, aeolian, subaerial, colluvial, mud flow, alluvial, residual and man made fills. Typically these soils are found in arid or semiarid regions and have a loose soil structure, i.e. a large void ratio, and a moisture content far less than saturation. Regardless of the formation process, most collapsible soils are geologically young.

Aeolian soils may be loose sand, like dunes, but more often they are loessial silt. The term loess represents a megascopically massive silt deposit of wind-blown origin. The soil particles in loess are loosely bonded by clay or calcareous material resulting in a low density. Saturation or flooding of loess weakens the bonds and reduces its strength. Increases in applied stress may also break the cementing bonds.

Loess covers vast areas of eastern Europe and south central Asia as well as a significant portion of the central plains area of the United States (Figure 2). Loess is a good foundation material in a dry state, sustaining vertical slopes and bearing pressure to 3.5 TSF without significant settlement. With the addition of water, a significant portion of loessial soils will collapse often without any increase in load (Lehr, 1969). In Romanis over 11 percent of the country's total surface area is reported to be covered with a collapsible loessial soil.

Collapsible soils of alluvial origin include flood plain deposits, alluvial fans, and mudflows. A major part of the flood plain deposits are the "loess-like" deposits, which are sometimes mistaken for loess. The major difference between loess and loess-like deposits is the sand content, which rarely exceeds 3% in loess, but which is usually over 5% in alluvial silt. A high sand content is not found in loess because the wind energy needed to transport sand is much greater than that for silt.

PROPERTIES

In-situ dry densities of collapsible soils range from 65 to 105 psf (1.1 to 1.7 gm/cc). These densities are reflective of porosities of from 40 to 60 percent with the porosity of 40 percent apparently a threshold value below which collapse seldom occurs (Drannikov, 1965).

Clay contents of the collapsible clayey soils in general range from 10 to 30 percent with the balance of the soil composed of silt or fine sand sized particles. Maximum subsidence in the San Joaquin Valley was found to occur when the clay was about 12 percent of the solids; below 5 percent there was little subsidence and above 30 percent the material swelled. Atterberg limits are reflective of the clay content with liquid limits below 45 and plasticity indices below 25 for most collapsing soils.

Initial water contents for the collapsing clayey soils are generally quite low, 5 to 30 percent, reflective of the arid conditions where they occur. Water contents for collapse are also less than 100 percent saturation with optimum content for maximum collapse varying with soil properties but in general between 13 and 39 percent (Dudley, 1970).

The soil structure is generally taken to be a loose honeycomb of the larger silt or sand sized particles coated and held together by a clay layer. This clay layer could be present during deposition or could result from weathering in place on the surface of the silt or sand particles. Regardless of the source of the clay and previous geological events, as movement in a mudflow, once the clay is interspersed in a loose structure of silt or sand particles and both set in position, any drying of the soil concentrates

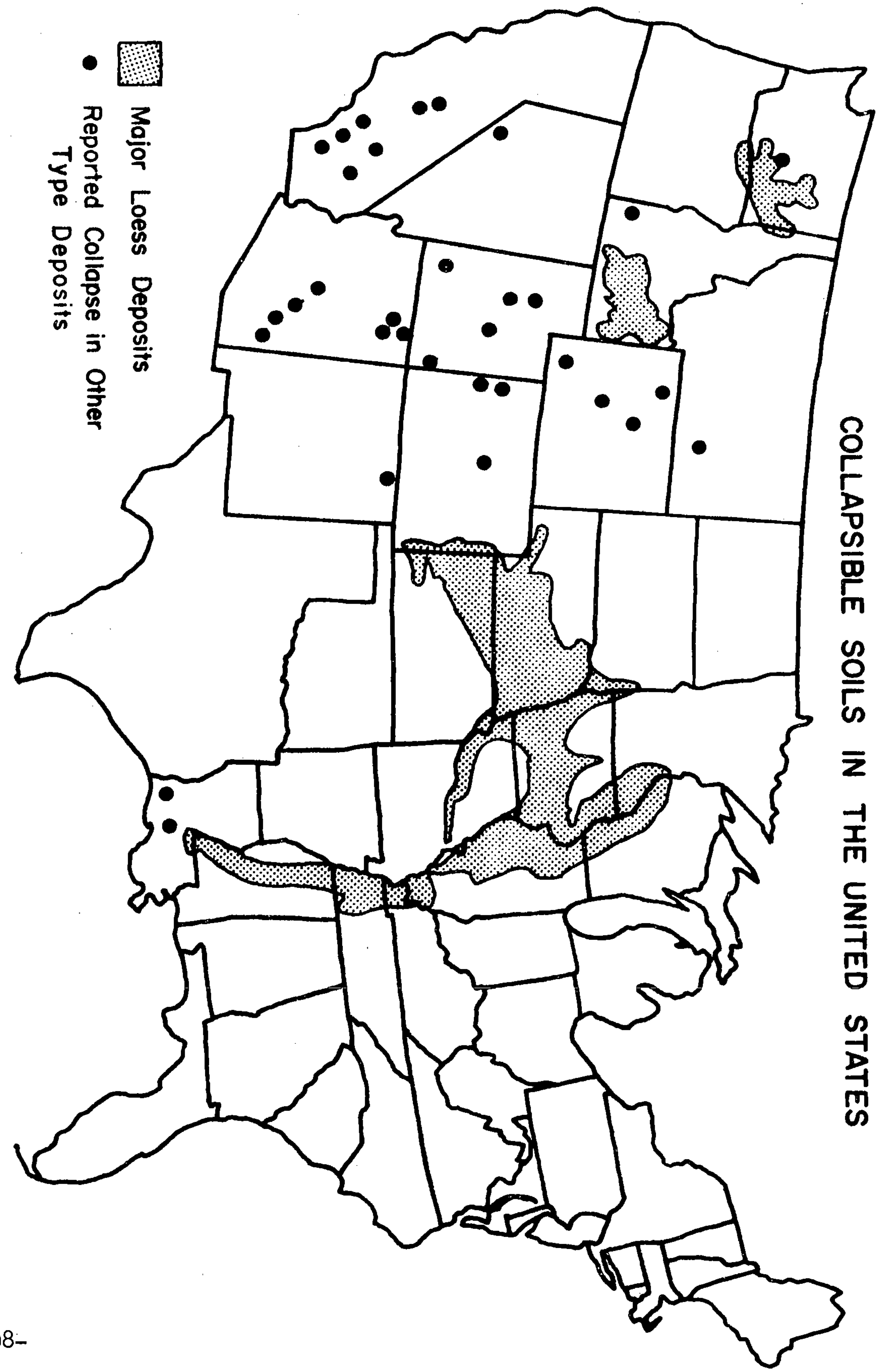


FIGURE 2

COLLAPSIBLE SOILS IN THE UNITED STATES

From Arman and Thornton P. 18

the clay and remaining water at the junctions between the particles. Interparticle forces, negative pore pressures, are created by surface tension in the water at air-water interfaces, increasing effective stress and the apparent strength of the soil, enabling it to remain in a loose structure even with increased loads and stresses. The tendency toward a loose structure remains, in some cases, even under the stresses of the standard compaction test (Arman and Thornton, 1973).

The fabric of the clay material depends on the type of clay minerals present, type and concentration of electrolyte, and stresses exerted on clay. Formation of flocculated structure is favored by the concentrating of dissolved ions by the drying process. Cementing agents would also act to weld particles together and add strength. Hence the future load carrying capacity depends on the strength gain by the clay binder as well as on changes in the negative pore water pressure and effective stress.

THE COLLAPSE MECHANISM

Addition of water as the triggering action is a widely-used explanation of collapse. Collapse may occur, however, as the result of load application wetting, or both. Thus collapse can occur either by increasing the stress above the strength or by lowering the strength below the stress.

Partially-saturated soils have shear strength as the result of capillary forces, which are destroyed at full saturation (Holtz and Hilf, 1961). In the Coulomb equation, where strength is the sum of the apparent cohesion and a function of the effective grain-to-grain pressure:

$$s = c + \bar{\sigma} \tan \phi$$

where s = shear strength

c = apparent cohesion

$\bar{\sigma}$ = effective stress

ϕ = angle of internal friction

The grain-to-grain stress is determined in part by the capillary forces and is directly related to the effective stress. Effective stress σ is the total stress minus the pore water pressure.

$$\bar{\sigma} = \sigma_\tau - \mu$$

where σ_τ = total average stress due to vertical loads

μ = pore water pressure

Because the meniscus is developed by surface tension, the pore water pressure is negative in partially-saturated soils. Negative pore pressures increase the intergranular pressure, thereby increasing the shear strength. Saturation destroys the capillary forces, thus reducing the intergranular, or effective stress, which causes a loss of shear strength (Figure 3). If the shear strength reduction is lower than the applied stress, collapse will occur.

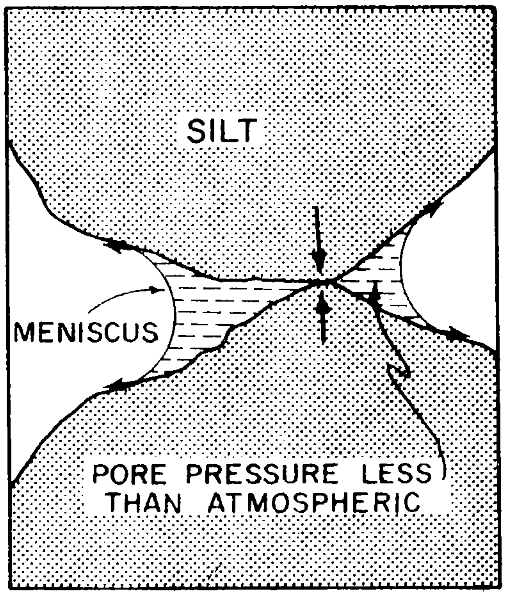


FIGURE 3

Showing how capillarity increases the grain to grain stresses in a partially saturated soil.

A hypothetical failure mechanism involving clay-covered silt particles is shown in Figure 4. The strength of this soil depends on the amount of clay binder (Kane, 1972). At a given binder content, the strength of the clay depends on its water content. When water is added to the dry soil, the clay becomes more plastic and loses strength.

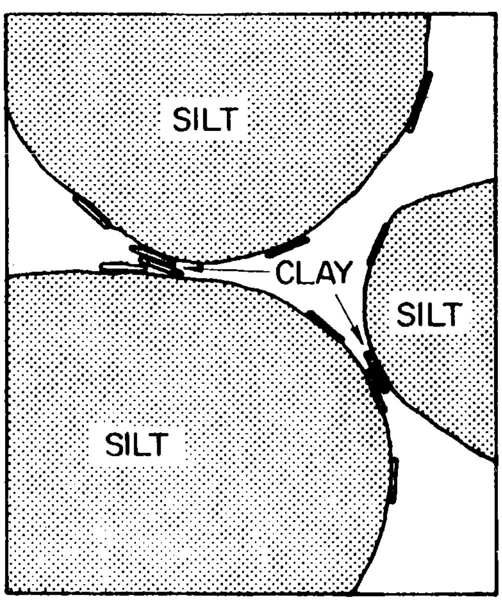


FIGURE 4

Showing silt particles whose strength depends on the moisture content of the clay binder.

After Kane, 1969, p. 34.

Soil particles supported by clay minerals and associated ions (Figure 5) may undergo collapse when saturated. Buttresses of clay-size material flocculated by ions may form around silt particles as the soil dries (Dudley, 1970), Capillary forces may add to the strength. The addition of water, besides destroying the capillary forces, will reduce the ion concentrations, which will, in turn increase the repulsive forces between particles and cause a dispersion of the supporting buttresses.

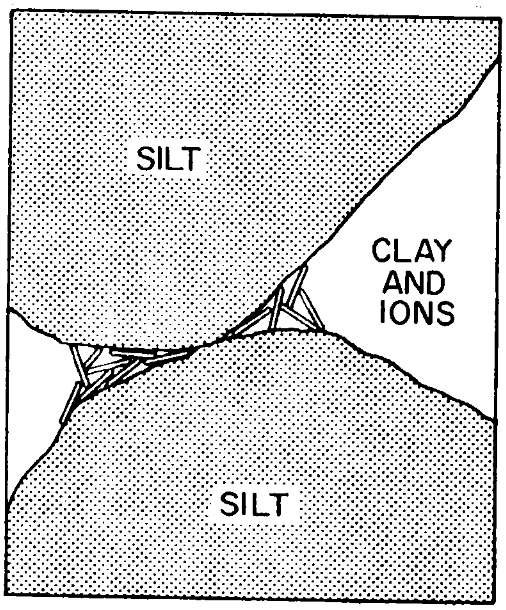


FIGURE 5

Showing soil particles supported by ring buttresses.

After Dudley, 1970, p. 937.

Soil collapse at a low level of stress can be predicted by determining the tendency of the clay to disperse as indicated by the sodium absorption ratio (Kassiff and Henkin, 1967). Changes in the solute and solute concentration also affects the collapse of soils (Reginatto, and Ferrero, 1972). Together these results indicate that the dispersive nature of the clay binder can have a significant affect on the collapse of clayey soils.

Flocculated clay particles may form buttresses which separate the silt particles (Figure 6). Lang, (1969) while working with a mixture of 94% foundry sand and 6% montmorillonite, found "a pronounced tendency for individual platelets to stand on edge between sand grains and combine in ribbon-like strands which fill the space between adjacent sand grains." Saturation of the clay buttresses in this case would cause a "mud flow" type of failure.

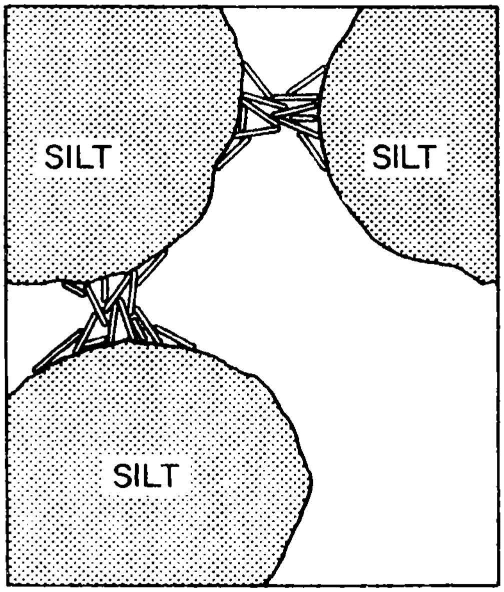


FIGURE 6

Showing silt particles separated by floculated clay buttresses.

After Lang, 1969, p. 87.

IDENTIFICATION OF COLLAPSE PRONE SOILS

The identification and prediction of collapse have proved elusive because no single criterion can be applied to all collapsible soils (Thornton, 1972). Both routine and sophisticated testing have failed to indicate the presence of collapsing soils. To date, most criteria for determining the susceptibility to collapse are based on relationships between the porosity, void ratio, water content, or in place dry density.

Because low unit weights indicate a loose structure, the in-place dry unit weight is a good criterion for collapse prediction. For loess or loessial material, an in-place dry unit weight of 80 pounds per cubic foot (pcf) or less indicates a collapsible soil (Clevenger, 1956). Unit weights between 80 and 90 pcf are transitional, while soils heavier than 90 pcf will settle very little. Holtz and Hilf (1961) added the moisture content as a criterion. Soils with less than 10 percent water content have a high resistance to settlement. Water contents over 20 percent, however, permit full settlement to occur under load.

Denisov introduced a coefficient of subsidence and a set of ranges for this coefficient which correlate with the degree of collapse (Sultan, 1969).

$$\text{Coefficient of Subsidence } K = \frac{e_L}{e_o}$$

where e_L = void ratio at the liquid limit

e_o = natural void ratio

K Value	Degree of Collapse
0.5 to 0.75	Highly collapsing soil
1.0	Non-collapsing loam
1.5 to 2.0	Non-collapsing soil

If the natural void ratio is higher than the void ratio at the liquid limit, the soil would collapse spontaneously upon saturation. Denisov's theory, however, did not account for the applied stress level or structure of the soil.

An interesting criterion incorporating a variation of the dry unit weight and water content, proposed by Gibbs and Bara (1962), was successfully applied to the San Luis Canal in the San Joaquin Valley. The soil voids must be sufficient to contain the moisture of the soil at its liquid limit. This criterion applies only if the soil is uncemented, the liquid limit is applicable, and if the liquid limit is greater than twenty.

The 1962 Soviet Building Code predicts collapse for soils that are less than 60% saturated. According to this code, a value of λ greater than minus 0.1 indicates collapse, where

$$\lambda = \frac{e_o - e_L}{1 + e_o}$$

Several workers have attempted to identify collapsing soils from the consolidation test (Knight and Dehlen, 1963). Milovic (1969), using a variation of the normal test, suggests the concept of a specific coefficient of settlement, which he defines by the equation:

$$i_m = \frac{e_n - e'_n}{1 + e_n} = \frac{\Delta e_n}{1 + e_n}$$

where e_n = void ratio corresponding to the natural water content at vertical stress σ_n

e'_n = void ratio corresponding to the increased water content at the same vertical stress σ_n

Δe = change in void ratio after wetting at the same vertical stress σ_n

Subsidence due to the addition of water can be calculated by multiplying the coefficient of settlement by the thickness of the soil layer.

Use of the double consolidometer was suggested by Knight and Dehlen (1963). One sample is tested at its natural moisture content while the other is run submerged in the confining ring. The test has the advantage of indicating the magnitude of collapse over a wide range of soil stresses.

Feda (1966) proposed a subsidence index (K_L) for predicting collapse in partially saturated plastic soils:

$$K_L = \frac{(w_n/S_n) - w_p}{I_p}$$

where w_n = natural moisture content (%)

w_p = Plastic Limit (%)

s_n = natural degree of saturation (%)

I_p = Plasticity Index (%)

Soils with a subsidence index greater than 0.85 are subsident. Those which are subsident and also have a natural saturation less than 60 percent would be expected to collapse. Since many loessial and granular soils are non-plastic, Feda suggested that they are subject to collapse if their natural porosity exceeds 40 percent:

In 1961, Gibbs developed a new criterion, the collapse ratio (R), that relates the water content at saturation to the liquid limit (Sultan, 1969).

$$R = \frac{w_{sat}}{w_L} \quad \text{or} \quad R = \frac{(\gamma_w / \gamma_d) - (1/G_s)}{w_L}$$

where w_{sat} = water content at 100% saturation

w_L = liquid limit

γw = unit weight of water

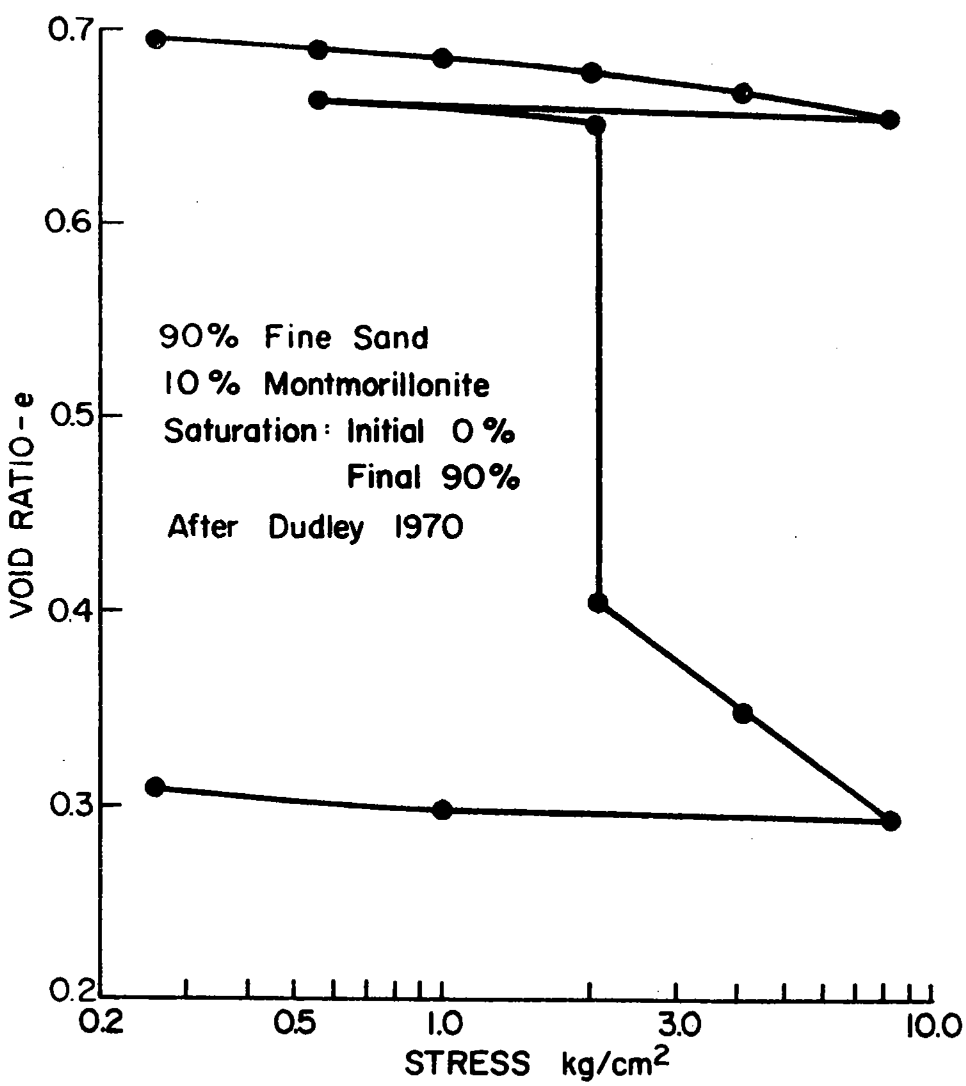
γd = dry unit weight of soil

G_s = specific gravity of solids

A soil with a collapse ratio greater than unity will be near a liquid state when saturated and therefore subject to collapse. The R values do not account for the type or nature of bond between the soil grains.

Kassiff and Henkin (1967) proposed the product of dry density and moisture content (γ_d x w) as a predictor of collapse for loess. When this product exceeds 15, large settlement may occur.

Dudley (1970) presented a description of collapse with a pseudo consolidation test (Figure 7). In this test a sample was fabricated from 90% fine sand and 10% montmorillonite. After the sample is loaded and rebounded in a dry state, it is flooded and the amount of collapse recorded.



COLLAPSE IN A CONSOLIDATION TEST

FIGURE 7

Measurements of the tendency to disperse either by calgon test (Aithchison and Tokar, 1973) or by determination of sodium absorption ratio (Kassiff and Henkin, 1967) would also indicate that a clayey soil of a dispersive nature might collapse. These tests would provide information about the initial structure of the soil and its tendency to disperse when wetted. Thus the possibility of collapse would be indicated. Dielectric dispersion tests (Arulanandan and Smith, 1973) may quantify the initial structure and provide information on collapse potential.

Quantitative data about the collapse potential of soil is generally obtained by testing the material using a double consolidometer technique. Here the soil is consolidated at its natural water content and the results compared with those run at different water contents or those obtained by saturating the sample at various load levels.

STABILIZATION METHODS AND PROCEDURES

Preventing damage from the collapse of soils falls into three categories:

1. Inducing collapse before building of structures or loading the foundation.

2. Preventing collapse by decreasing soil pressures and waterproofing measures.

3. Solidifying the soil so that the addition of water does not lower the strength.

Collapse of the soil before construction is generally induced by ponding the water on the surface. Ponding is commonly used for hydraulic conveyence facilities where wetting after construction cannot be avoided. As collapse of surface layers greatly decrease their permeability, injection wells may have to be used to insure all potentially collapsible layers are wetted. Also, the ponding time must be sufficient for the water to uniformly seep through and wet the material. In some areas of the San Joaquin Valley water was ponded for over one year prior to construction of the California Aqueduct. In Romania (Bally, et al, 1972) placing of canal linings was delayed for 2 to 4 years after their construction to insure that most of the subsidence (collapse) has occurred.

Surcharge fills are often placed where structures will be located in order to subject the soil to loads and stress equal to or greater than final values while they are wetted. Surcharge insures that collapse occurs prior to construction of the structure. In Russia (Aithchison and Tokar, 1973) surface and subsurface blasts were used to place additional loads on the wetted material. Ramming or driving of foundations into the collapsible soil to overload it and causing the soil to collapse (Aitchison and Tokar,

1973) and removing all soil to the base floor and then tamping the earth below with heavy equipment (Krutov and Viasov, 1969) was also used in Russia.

Decreasing soil loads by removal of material, use of floating foundations, and prevention of wetting are means of preventing collapse of soils with clay. For non-hydraulic structures located on collapsible soils, improvement of the surface drainage features could be the cheapest way to prevent collapse. Drainage improvement may be just insuring adequate drainage facilities to prevent ponding, or creation of surface or near-surface layers of impermeable material either by compaction or by short duration flooding which would just wet the surface layer and cause it to collapse. Buildings could be designed so that over-hanging roofs and drainage facilities conveyed water away from the immediate area of the building. In Romania (Bally, et al, 1972) hydraulic structures were located at a distance of 2 to 3 times the thickness of any collapsible layers away from the conveyence facilities (canals, etc.). Waterproof pipes or other water tight means are used to bridge the gap between the structure and the canal in order to prevent wetting the foundation of the structure.

Many means have been used to solidify the clay binder and prevent loss in strength in the soil. In Russia, soils were consolidated thermally by injecting burning hot (up to 900°C) gases through the soil to fire and stabilize the clay (Litvinov, 1960). Other more conventional means used include the addition of ammonia gas, sodium silicates, lime or portland cement which would either stabilize the clay binder or cause cementation increasing the strength of the binder.

Other means of avoiding the problems of collapsible clayey soils include removal of the collapsible material or bridging across the piles or other deep foundations.

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DESIGN AND INSTALLATION OF AN EARTH TIEBACK SUPPORT SYSTEM

By

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ABSTRACT

This paper describes an earth tieback, H-beam and wood lagging sheeting system that was used to support the sides of an excavation for Interstate 170 in Baltimore, Maryland. Section 170 - 8(7) was approximately one-half mile long, and the excavated depth varied from 20 to 60 feet below adjacent streets and utilities.

This paper discusses factors considered during the design of the tieback sheeting; earth pressure reductions due to the installation of the sheeting on a batter; coordination with other work; earth tieback capacity; tieback testing procedures; installation procedures for earth tiebacks and equipment utilized; and performance of the completed wall. Recommendations are given concerning soil deposits most suited for the types of earth tiebacks used on this project.

INTRODUCTION

General Description

Figure 1 is a plan of Section 170 - 8(7). Along the north side 1205 linear feet of sheeting and 222 tiebacks were installed. The south side had 1900 linear feet of sheeting and 301 tiebacks. An additional 166 linear feet of sheeting and 20 tiebacks were required to support temporary detour roads around the site.

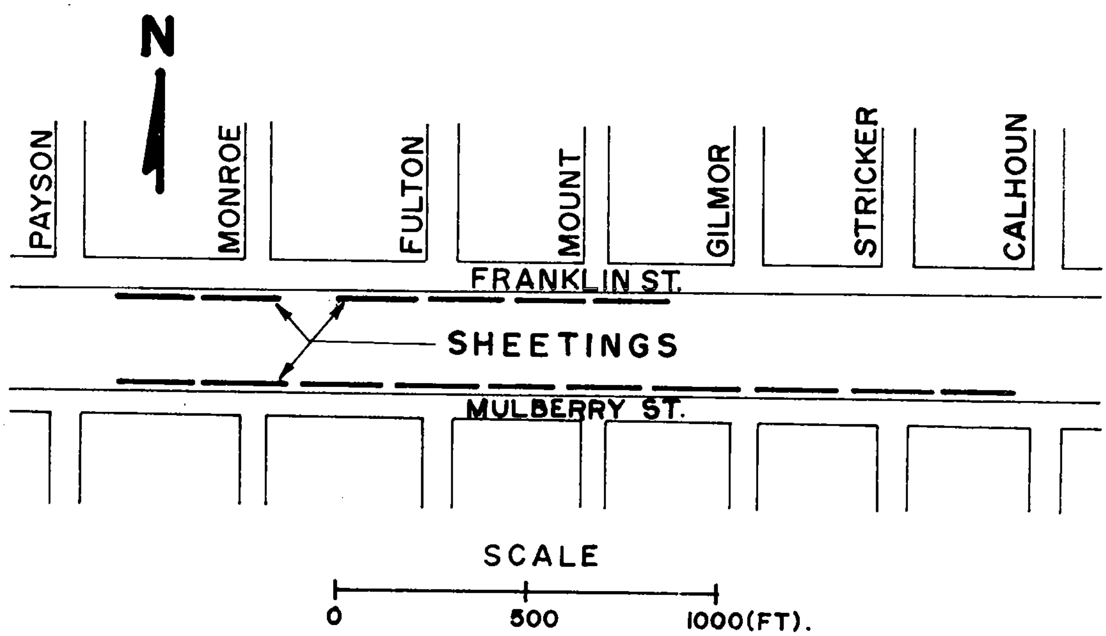


FIG. 1 PLAN

Figure 2 is a typical section of the sheeting and retaining wall. The streets behind the sheeting were laced with utilities that included a 30-inch water line, 20-inch gas line, and numerous other smaller lines and ducts.

SOILS

The project is located in the Coastal Plain physiographic province. Soils at the site were a part of the Patuxent Formation which is the "basal member" of the Potomac Group of Cretaceous Age. Primary soils in this formation were buff and light colored sands that often contained considerable amounts of kaolinized feldspar. In many cases the sands were cross-bedded and occasionally merged into gravels with pebbles of considerable size. The gravels and pebbles were sub-angular due to their short distance of transport from the Piedmont Province, which lies west of the Coastal Plain. Boring B-38 in Figure 3 is typical of the sandy material.

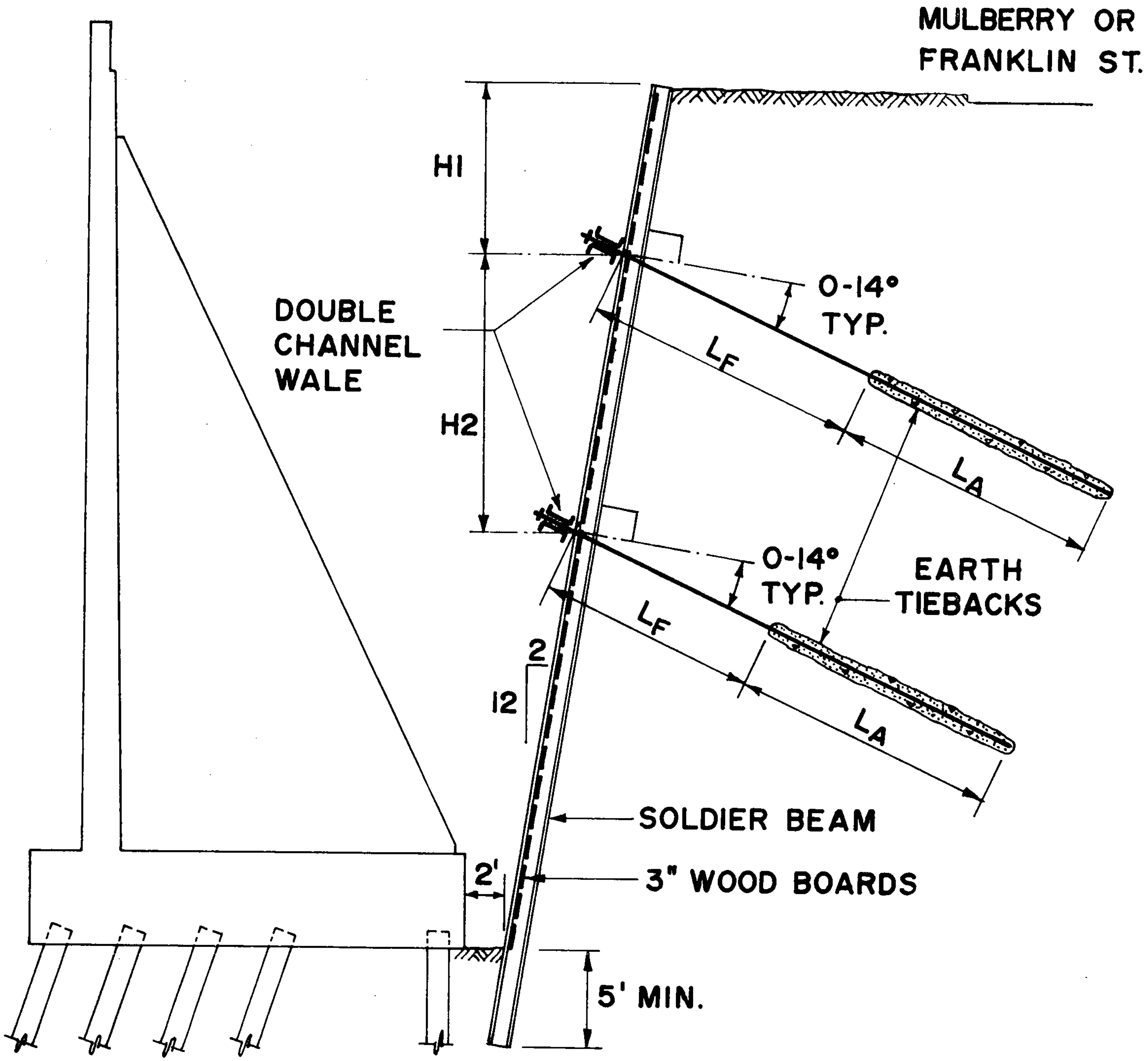
There were two secondary soil profiles in the Patuxent Formation that were less extensive than the sandy material. The first consisted of lenses of silty clay interbedded with the sands and gravels. In the second, the lenses of clay became very thick and completely replaced the sands and gravels. In both of these profiles, the silty clay was very stiff and plastic.

Across the site there was a fairly uniform water table located near or just above subgrade. No construction problems were caused by the ground water. Borings for the project had been taken every 150 linear feet along both the north and south retaining walls.

DESIGN

H-beams and wood lagging was determined to be the most appropriate and economical sheeting system for this project, since steel sheetpiling most likely would not have penetrated the dense sands and gravels. This assumption was proven true when several of the H-beams had to be preaugered after reaching refusal above subgrade. Also by using H-beams and lagging, a two inch space could be maintained between the lagging boards to allow for drainage and prevent the build-up of hydrostatic pressure behind the sheeting.

The sheeting had to be installed far enough behind the retaining wall footing to allow clearance for the pile hammer and leads during installation of battered foundation piles. It was also desirable to place the sheeting close to the footing to eliminate as much excavation and backfill as possible. By utilizing the patented method of "sloped sheeting", Schnabel (3), the toe of the sheeting could be held to within two feet of the footing, and with a 2 on 12 batter the top would be back far enough to allow clearance for the hammer and leads. Figure 4 is a schematic section showing the excavation and backfill saved as a result of using sloped sheeting.



TYPICAL SECTION

FIG. 2

DEPTH IN FT.	BLOWS PER 6" ON DRIVEN SPLIT SPOON	RECOVERY IN INCHES.	BORING NO. B-38
2.5	2-6-6	8	ORGANIG (FILL)
	3-4-5	12	GRAY & BROWN DRY MED DENSE SILTY FINE SAND.
	3-7-12	12	
	4-10-12	12	
	5-8-12	12	
	6-13-16	12	
			W. L.
	3-3-5	12	
33.0			
	6-14-17	12	BROWN & GRAY MOIST MED DENSE SAND
39.0			
	8-15-23	12	BROWN & GRAY WET DENSE CLAYEY MED. SAND & GRAVEL
	4-12-23	12	
50.0			

FIG. 3

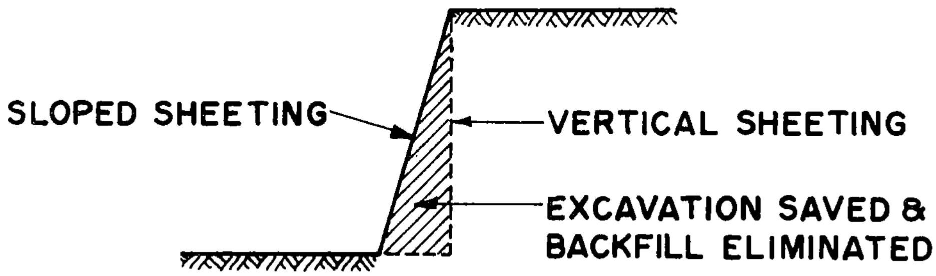
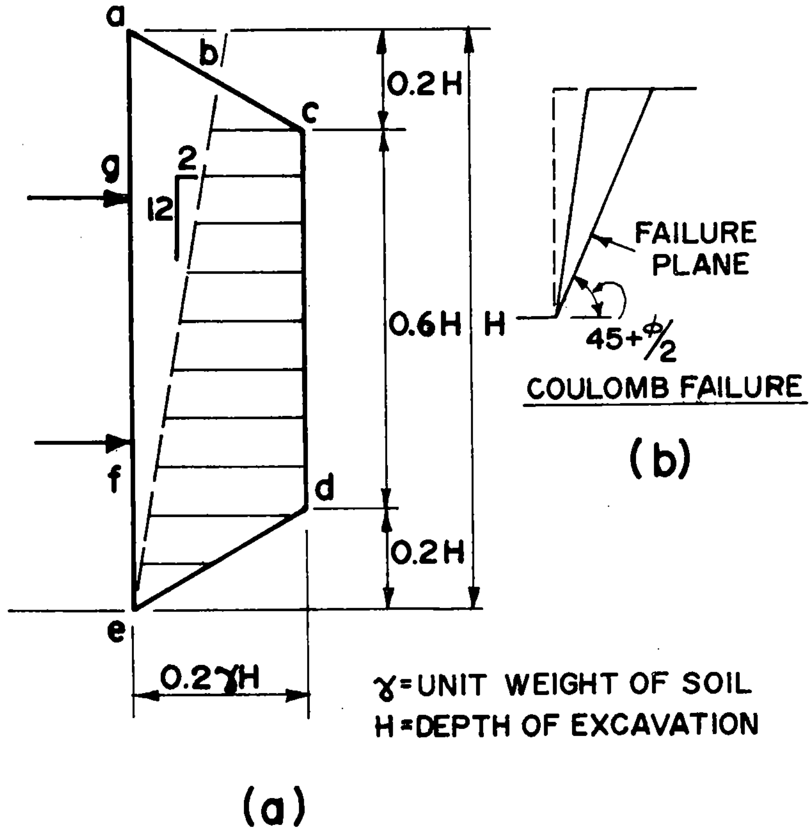


FIG. 4

In addition to reducing the amount of excavation and backfill required, sloped sheeting allowed significant reductions in the earth pressures normally used for the design of vertical sheeting. In Figure 5a the trapezoid acde is the apparent earth pressure diagram used for the design of vertical sheeting. This diagram has been derived empirically by measuring brace loads on many projects, and then converting these loads to equivalent pressures. The apparent earth pressure diagram is an envelope which encloses the pressure diagrams derived from observation. This empirical method of determining earth pressure diagrams is presented by Terzaghi and Peck (5) and Peck, et al, (2).

For sloped sheeting the earth pressures are assumed to be reduced by the ratio of the sloping wedge to that of the large right triangle, Figure 5b. These assumed reductions have been empirically verified. For this project, wedge abe was eliminated in Figure 5a, and a reduction of 30% was applied to the apparent earth pressure envelope used when designing vertical sheeting.

The possible method of supporting the sheeting consists of tiebacks or interior bracing. Interior raker bracing was a possibility, but it would have interfered with other work and substantially increased the overall construction costs. For instance, excavation of berm earth under the braces would have been slow; and foundation piles, which were spaced on five to six foot centers and in rows of three or five piles, would have been difficult to drive through a maze of braces. Also, construction of the walls with large movable gang forms could not have been accomplished with braces in place, and the rakers would have to have been positioned accurately in the areas of the deep counter-fort walls. Patching of the walls and piecing the extensive membrane water-proofing would also have been required where each brace penetrated the walls. Therefore, it was decided to use tiebacks, though somewhat more expensive than raker bracing, because the cost of the work affected by the braces would be reduced by a margin of larger than the extra cost of the tiebacks.

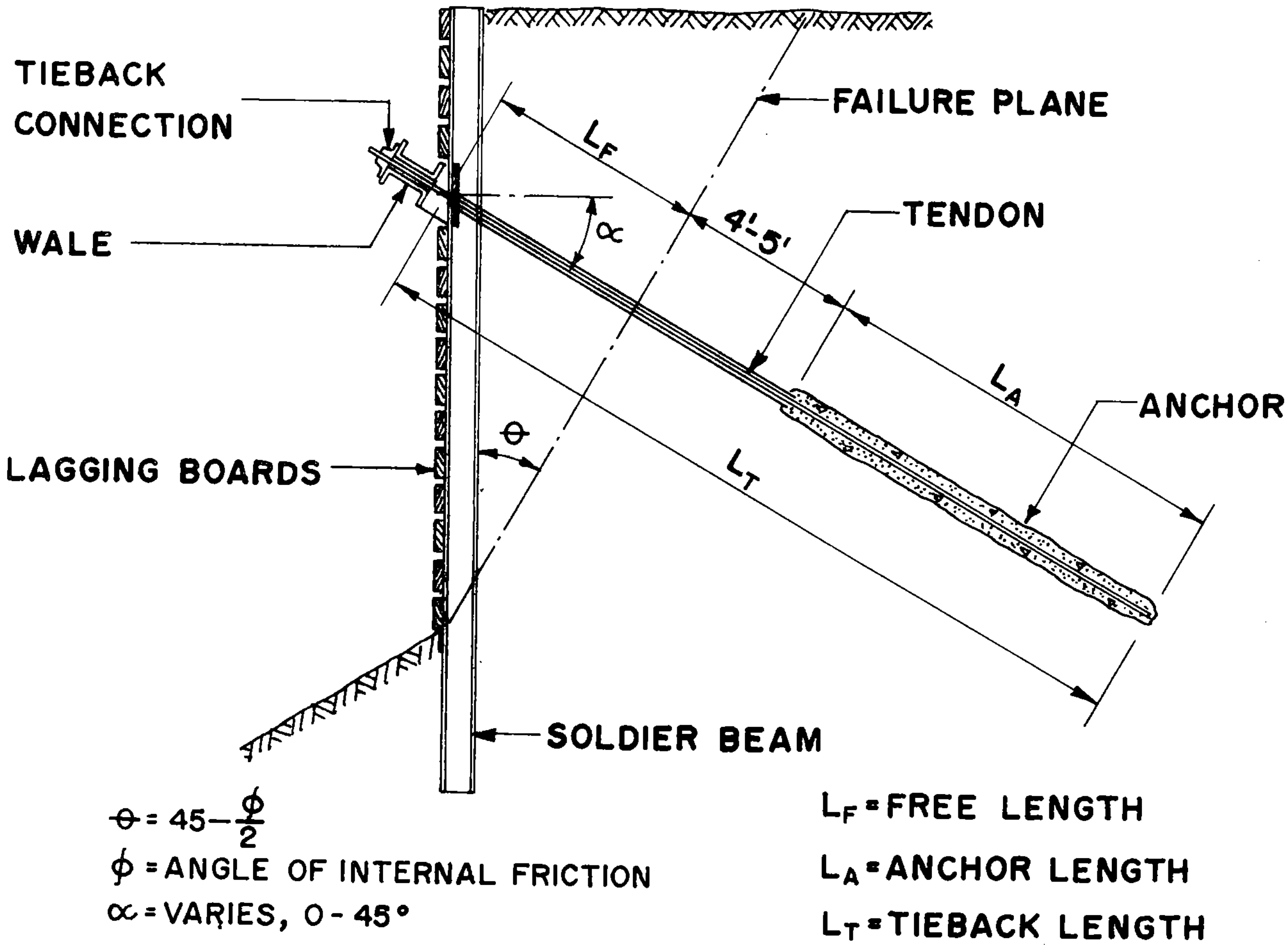


APPARENT EARTH PRESSURE DIAGRAM.

FIG. 5

On this project the specifications limited the angle of sloping of the earth to one and a half horizontal to one vertical, and set the minimum distance from the top of a slope to the nearest street. The specifications then allowed the contractor to select the type of earth support system, and to design this system. For tieback sheeting this is very important since there are many types of tiebacks and an equal variety of installation procedures. Soil conditions at this site, as on many sites, varied from boring to boring. This meant different types of tiebacks would be required for various soil conditions encountered. For this reason, it is reasonable to allow the contractor to select the types of tiebacks and methods of installation he deems appropriate, and then hold him responsible for their performance.

Figure 6 details the elements of a typical tieback sheeting system. The tendon is a standard high strength post tensioning rod. The tieback length (L_T) is made up of two parts, L_A and L_F. The anchor length (L_A) is the portion that provides the reaction to the earth pressure loadings. It



ELEMENTS OF A TIEBACK SHEETING SYSTEM.

FIG. 6

must be placed far enough behind the failure plane to prevent "pullout" and to prevent massive failure of the earth bank. The free length (L_F) transfers the load from the face of the sheeting to the anchor. No load is transferred to the soil in the free length.

Both rock and earth tiebacks were considered for the anchorage system. The rock tiebacks would have to have been very long and installed on a steep angle. The steep angled tiebacks would have tended to pull the sheeting downward, therefore, the soldier beams would have to have been driven to rock in order to provide an adequate reaction against settlement. On the other hand, earth tiebacks would be much shorter and could be installed on a nearly flat angle. This would eliminate the requirement for soldier beam resistance, and thus reduce the length of piles to be driven. In addition, the soil above the rock was of the type that could develop tie capacities of 100 to 150 kips relatively easy. Because of these advantages, earth tiebacks were selected for this job.

During the bid and design stage of the project, two types of earth tiebacks were selected for use. A pressure injected tie was considered best suited for the sandy material. Because the pressure injected tieback does not develop large capacity in cohesive soil, an auger belled tie was selected for use in the stiff silty clay. Where the clay and sand was found to be lenticular, a pressure injected tie would be used with a long anchor length.

The anchor capacity of pressure injected tiebacks is dependent upon the relative density of the soil, amount of fines in the sand, depth of anchor below existing grade, distance between the face of sheeting and the anchor, grout pressure, amount of grout take, size of the grouted bodies, and length of the anchor. Tests have been reported by Ostermayer (1) where certain of these factors were held as constants and others were then varied to determine their affect on capacity. The results provide general information, but produce no method of computing realistic anchor capacity other than field testing. Therefore, the selection of anchor capacity and length continues to be made on the basis of an engineer's knowledge of the soil conditions at a site, and his past experience with tiebacks installed in similar material. For this project, design loads up to 113 kips were selected with anchor lengths of 16 feet.

The auger belled tiebacks have a more rational design approach, Woodward, et al, (6). Basically, they perform like a belled caisson in reverse, and require a much shorter anchor length than pressure injected ties. Maximum design loads for this type of tieback were also selected at 113 kips. They were to be installed by drilling an 18 inch shaft, and then belling the end of the shaft to four feet in diameter.

After installation of the tiebacks, it is necessary to verify that an anchor actually has been installed behind the failure plane, and that it will carry the expected load. This was done by testing each tieback in accordance with procedures developed by Schnabel (4). According to these procedures, every tieback installed must have a proof test. This test is performed by incrementally loading each tie to 120% of the design load and measuring the movement at each increment. If a tie is successful, the load is reduced to between 65 and 75% of the design load and "locked-off." The entire test requires about 5 minutes. A few of the tiebacks should have a more detailed performance test. This test is performed by incrementally cycling the load in the tiebacks up to 133% of the design load, and measuring the movement at each increment. When performed accurately, the test will tell if the anchor has sufficient capacity and if it has been made behind the failure plane. This test can then be used to compare with the results of the simpler proof test to determine whether or not the production ties are performing as required. There were to be 15 performance tests on this job.

Spacing of the tiebacks, size of wales, and size and spacing of soldier beams were determined by assuming pin connections at points e and f on Figure 5a, and analyzing the beam using the reduced earth pressures. Additional reductions were allowed for the soldier beams and wales in accordance with Peck, et al, (2). The wales selected were 15 C 50, and 12 C 30 double channels, and the soldier beams ranged in size from HP 8 X 36 to HP 14 x 89 H-beams. The three and four inch thick lagging boards were not designed but selected on the basis of experience in this area.

INSTALLATION

General

The soldier piles on this project were installed by a diesel pile hammer mounted in fixed leads, and the 3 or 4 inch thick lagging boards were placed behind the flanges of the piles as the excavation progressed. In areas where water was present, or when fine sand or silt might run between the boards, straw was installed between the soil and the back of the lagging sheeting systems, only those operations associated with the installation of the earth tiebacks will be described.

When the excavation reached the level of the tieback wale, a 40 foot wide soil bench was maintained adjacent to the sheeting. The bench provided access for the tie installation equipment and storage space for materials. Because of the width of this site, the area between the benches was excavated 10 to 15 feet below the wale grade prior to installing the tiebacks. After examining the soil deposits exposed in these cuts at the depth of the future anchors, the final determination was made concerning the type of tieback to be used. Variables such as anchor length, tie angle, and grout take were then adjusted to suit the field conditions.

Pressure Injected Tiebacks

Where pressure injected tiebacks were to be installed, the wales were set in place prior to installing the ties. Then specially designed high-strength three-inch diameter steel casing was driven into the soil behind the sheeting using a standard crawler drill similar to those used for rock drilling. The end of the casing was sealed with a cast point in order to prevent soil from filling it during installation. Additional sections of casing were coupled to the first section and driven until the end of the casing reached the desired distance behind the facing of the cut. After the casing was driven for one tieback, the crew moved to the next location and repeated the process until 10 to 20 strings of casing were installed.

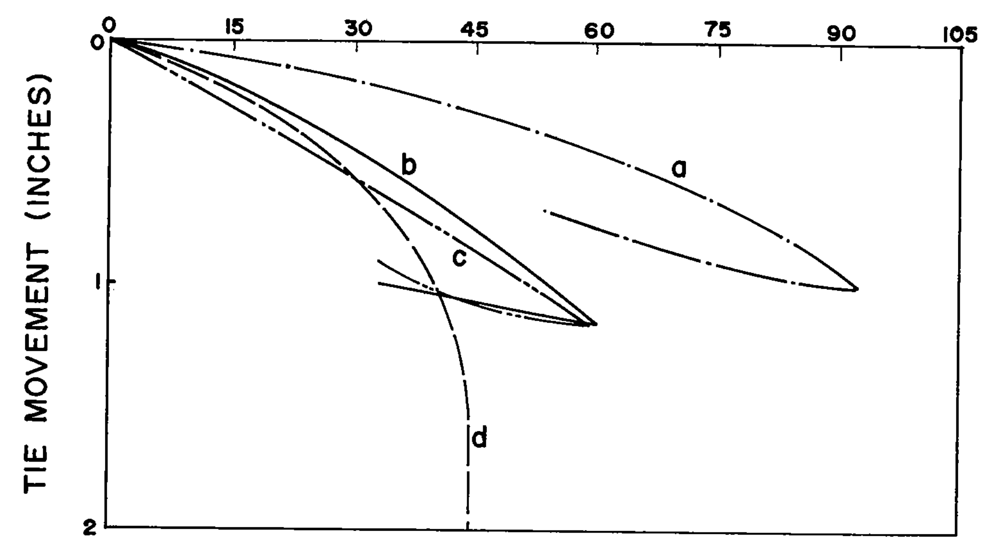
The tieback tendon, a 1¼ inch diameter Dywidag rod with an ultimate strength of 150 ksi, was then inserted into each casing. The Dywidag rod is a continuously threaded rod conforming to ASTM A-322 and ASTM A-29 specifications, and the 1¼ inch rod has an ultimate capacity of 187.5 kips. The portion of the tendon passing through the soil being supported (free length), was covered with a plastic sheath which acted as a bond breaker between the grout and the tendon. After the rod was inserted in the casing, it was necessary to drive the point off so the casing could be extracted while the Dywidag rod remained in place.

Upon driving the point off the end of the casing, a hydraulic center hole jack and gripping device were used to extract the casing while cement grout was pumped down the hole. The hydraulic jack was powered by a diesel hydraulic pump. The grout used on this project was a Hi-Early cement and water mixture with a water to cement ratio of between 4.5 and 5.5 gallons per sack. The grout was mixed by hand in an air motor operated seven cubic foot mixing tank and then dumped into another similar agitated tank where cement lumps were removed. An air operated positive displacement piston pump was used to pump the grout down the casing and around the tendon while the grout pressure and grout take were observed. When the grout pressure reached a value determined during the installation of the "test" tiebacks, usually between 150 and 450 psi, the casing was extracted. The speed at which the casing was extracted was varied in order to maintain these grout pressures. Depending on the grout take, the casing was pulled at rates up to three feet per minute.

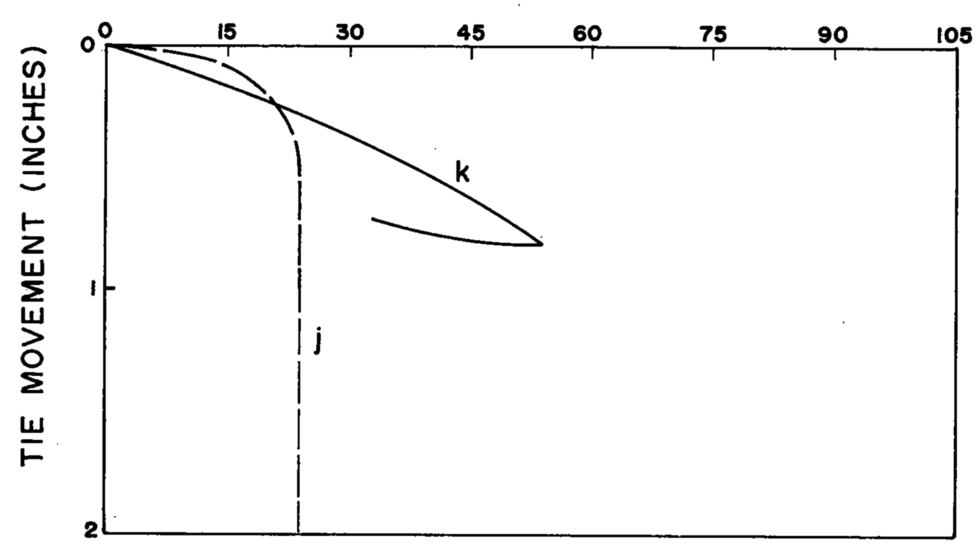
The anchors were tested as described above (a minimum of 72 hours after grouting). If a tieback failed to carry the full test load, the load was reduced to one-half the load it successfully carried, and that tie was incorporated in the system at the reduced load. Where a tieback failed, an additional tie was installed in order to pickup the full design capacity.

The first tiebacks on this project and the first ones installed in different soil profiles were used to develop grouting criterion to help evaluate the production anchors made in similar soil conditions. Usually these anchors were tested in accordance with the performance test requirements described earlier. Various combinations of anchor lengths, tie angles, grout takes, and grout pressures were tried on these "test" tiebacks in order to obtain the most economical tie. The most significant variable on this project was the anchor length and it was varied from 15 to 35 feet during the installation of the "test" tiebacks. After reviewing the "test" tieback results, it was decided that a 25-foot anchor length would be necessary to develop pullout resistances equal to 75 tons in the silty sands, and that grout takes of 10 to 15 sacks of cement per tie and grout pressures above 200 psi would produce anchors that would carry the maximum test load.

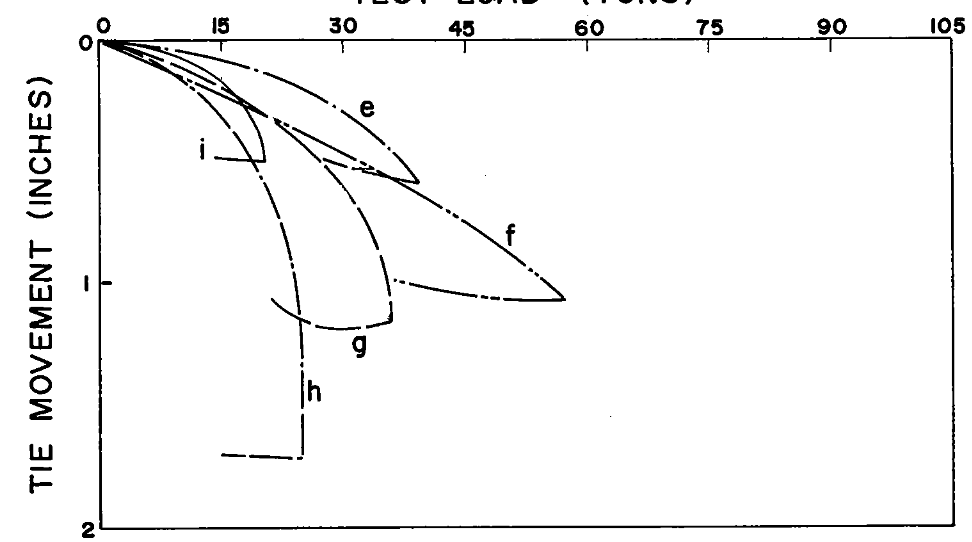
The grouting behavior and load carrying capacity of the pressure injected tiebacks were significantly affected by cohesive soils. Figures 7a, 7b, and 7c, show how these ties behaved in the various soil types encountered on this project. Following are observations and conclusions concerning the pressure injected tiebacks shown in Figure 7.



(a) TEST RESULTS FOR PRESSURE INJECTED TIES INSTALLED IN SAND.



(c) TEST RESULTS FOR PRESSURE INJECTED AND BELLED TIES INSTALLED IN CLAY.



(b) TEST RESULTS FOR PRESSURE INJECTED TIES INSTALLED IN SILTY SAND & CLAY.

PRESSURE INJECTED TIE GROUTING DATA					
TIE	ANCHOR LENGTH (FT)	FREE ANCHOR LENGTH (FT)	GROUT TAKE (SACKS)	GROUT PRESSURE (PSI)	REMARKS
a	20'	15'	9	700-800	
b	25'	10'	6	300-450	200 II
c	15'	20'	4	200-300	
d	15'	9'	6	200-250	200 I
e	25'	10'	20	100 MAX	
f	16'	18'	30	200-250	BACK PRESSURE
g	35'	10'	20	100	
h	35'	10'	20	100	289 II
i	25'	10'	20	100	289 II
j	25'	9'	57	0-75	BACK PRESSURE

BELLED ANCHOR DATA				
TIE	ANCHOR LENGTH (FT)	FREE ANCHOR LENGTH (FT)	CONC. TAKE (CU. FT.)	REMARKS
K	12'	15'	40	18" SHAFT 4'-0" BELL

FIG. 7 TIE GROUTING OR CEMENTING BEHAVIOR & TEST RESULTS.

1. Tie "c" carried the desired test load of 58.5 tons and was installed in sands and clean silts. Figure 8 shows gradation curves for two samples taken in these soils. Tiebacks installed in similar soils typically required only four to ten sacks of cement and developed grouting pressures between 300 and 800 psi.

2. Tie "d" failed to carry the desired test load and was typical of several of the initial "test" tiebacks. This tie was installed adjacent to Tie "c" and had the same anchor length and grouting characteristics as Tie "c", but its free anchor length was 11 feet less. The failure of Tie "d" was probably due to the fact that its anchor was formed closer to the face of the cut and existing ground surface than the anchor of Tie "c", thus reducing the overburden soil pressure at the level of the anchor. This reduction in confining pressure reduced the soils ability to develop pullout resistances equal to the potential capacity of the anchor.

3. Tie "b" was typical of the successful production tiebacks installed in sands and clean silts.

4. Tie "a" was made using a 1 3/8-inch diameter Dywidag rod. This tendon was capable of carrying a 93.5 ton test load, and Tie "a" successfully carried this load. This anchor had a 20-foot grouted length, and developed a "pullout" ressistance of 9.35 kips per linear foot of anchor.

5. Ties "e" and "f" were both installed in silty sands interbedded with clay and silt. Both of these tiebacks tested, but they required larger amounts of grout and they did not develop as high grout pressures as those installed in sand. Tie "f" also exhibited a behavior called "back pressure" which is the tendency of the grout to squeeze out of the hole when the grout hose was uncoupled and the pressure released.

6. Ties "g", "h", and "i" were also installed in silty sands and clays and these ties failed to carry the desired test load. The capacity of these tie-backs were not significantly affected by increasing the anchor length from 25 to 35 feet. In areas similar to those where these ties were installed, it was decided that two tiebacks would be required instead of the single tie as originally planned. About 15 extra ties were installed in these areas. Anchors in these soils do not apparently develop their pullout resistances linearly along the anchor as do ties in sands, and increasing the quantity of grout does not substantially increase their capacity.

7. Tie "j" was installed completely within a stiff clay lense. This tieback failed at 24 tons, and took 57 sacks of cement and only developed grout pressures of 75 psi. In clay, the grout formed

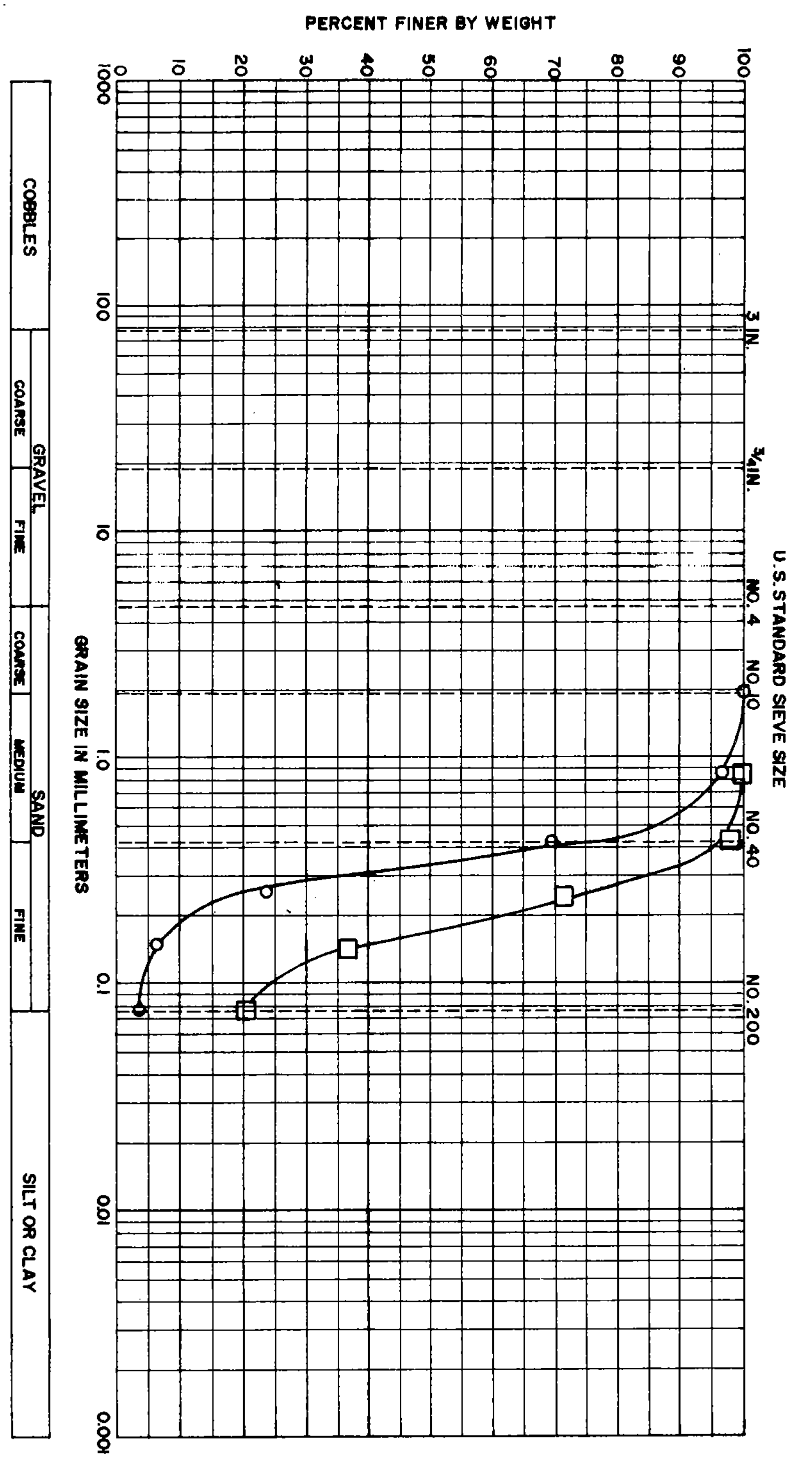


FIG. 8 GRADATION CURVES

lenses and once this occurred, a large amount of grout was pumped into the soil. The amount of grout did not increase the capacity of these ties, but it did hydraulically lift the soil above the anchor. Except for four tiebacks on this project, the grout takes were limited to 20 sacks of cement per tie in order to control grout heave.

Auger Belled Tiebacks

In one area where a thick lense of clay was encountered, auger belled ties were installed. It was decided to install this type of tie after observing grouting behavior similar to that of Tie "j", Figure 7c, and after establishing the extent of the clay lense visually in the field. The auger belled ties were installed using a truck mounted caisson drill. The tie shaft was drilled to the desired depth with an 18 inch diameter earth auger. After completing the shaft, a standard underreaming bucket was used to bell the shaft to approximately four feet in diameter. Then a Dywidag rod was inserted into the hole and supported in its center.

Next 2500 psi concrete was poured down the shaft filling the bell and 12 feet of the shaft. After the concrete was placed, the wale was installed, and 120 hours later the tieback was tested. The auger belled ties were installed in uncased holes; therefore, they could only be installed in soils which had enough cohesion so the hole would stand open. Even though there was only a small area where auger belled tiebacks were used, this alternate method saved both time and money when the cohesive soils were encountered.

Tie "k", Figure 7c, shows the test results for a typical auger belled tie installed in stiff clay with an average standard penetration resistance of 32 blows per foot. Note that Tie "j", Figure 7c, was only capable of carrying 24 tons while Tie "k" was capable of carrying 55 tons. It would have taken three pressure injected ties to develop the capacity of one auger belled tie in clay.

PERFORMANCE

Figure 9 shows the movement record of the tops of the soldier piles in an area where the depth of cut was 30 feet and the sheeting was supported by one tier of tiebacks. The soil in this area was similar to that shown in Figure 3. The zero movement datum was established pripr to excavating in the area. The August 20th movement line represents the position of the soldier piles after the ties had been installed and prior to excavating below the wale. The outward movement or movement into the soil occurred when the piles were pulled back upon testing. When the ties at piles 177 and 193 were tested, large movements into the soil occurred because voids existed behind the lagging. The large amount of movement that occurred between piles 179 and 191 was caused during the construction of a sewer and water line behind the sheeting. This construction in one area exposed a portion of the tie tendons, and braces were eventually installed as a precautionary measure. The

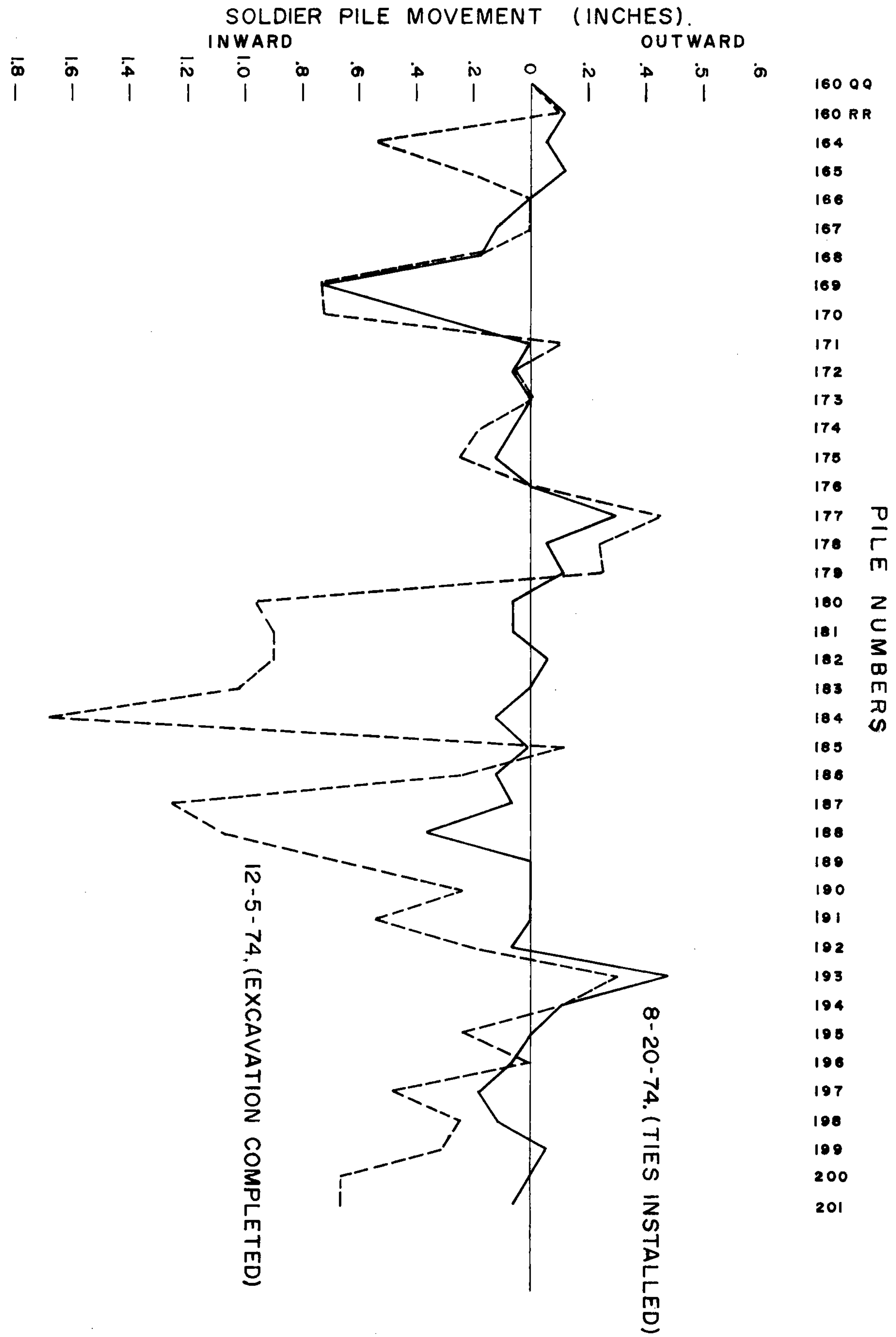


FIG. 9 SOLDIER PILE MOVEMENT

total movement of the other piles averaged about three-tenths of an inch, and only about one-tenth of an inch occurred after the ties were installed. This amount of movement was expected since movement is necessary for the earth pressures on the sheeting system to be reduced to the active earth pressure for which the system is designed. The amount of movement required to reduce the pressure to active is 0.001 times the height for sand and 0.004 times the height for clay, Wu(7). For a 30-foot cut, in silty sands, about .36 to .72 inches of movement would be expected to develop the active earth pressures.

The ties at this project can be classified into four categories; pressure injected tiebacks in sands, pressure injected tiebacks in silty sand with clay lenses, pressure injected tiebacks in clay, and auger belled tiebacks in clay. Table 1 shows the number of ties installed in each type of soil as of this writing and the percentage of ties failing to carry the test load.

TABLE 1. Tie Failure Record

Soil Type	Tieback Type	Number of Ties	Failure Rate
Sand	Pressure Injected	239	4.1% *
Silty Sand & Clay	Pressure Injected	87	21 %
Clay	Pressure Injected	5	80 %
Clay	Auger Belled	8	0 %

* - The failure rate for the pressure injected tiebacks was actually .4% for the production ties. All but one of the ties which failed in the sands occurred during the installation of the first 14 ties, which were "test" tiebacks.

The overall performance of the sheeting system was satisfactory. A portion of the system has been open for one year. No excessive movement has occurred in the sheeting, but one area above two ties was raised when large quantities of grout were pumped into a clay stratum.

CONCLUSIONS

1. Pressure injected tiebacks. This type of tieback performed successfully in soils that consisted primarily of sand and gravel, and is recommended for use in this type of material. When clay became interbedded with the sand and gravel, longer anchors and larger grout takes were required, and success rates were reduced. When installed completely in clay, the pressure injected tieback performed unsatisfactorily, and an alternate method had to be used.

2. Auger belled tiebacks. This type of tieback was used when clay was encountered without any water or sand present. Although only eight of

these were installed, they all performed successfully and are recommended for use in cohesive materials.

3. Anchor capacity and length. For the pressure injected tieback, no method is available for reliably computing the anchor capacity and length. Field testing showed that a 25-foot anchor length would develop a capacity of 113 kips. For an auger belled tie, the capacity is determined by analyzing a caisson for pullout. The length is determined by the requirement that the anchor be behind the failure plane.

4. Specification. The specifications permitted the contractor to select and design the sheeting system, and choose the methods of installation. This proved to be very desirable because of the varying soil conditions and the different methods of installation required.

5. Tieback testing procedure. Every tieback on the job was proof tested to 120% of the design load. Fifteen ties had a more detailed performance test to 133% of the design load. The tests insured that the anchors had sufficient capacity, and that they had been made behind the failure plane.

6. Movement performance. The sheeting movement was generally around 0.3 inches which was acceptable.

7. Sloped Sheeting. The sheeting performed satisfactorily with a 30% reduction of the normal vertical design loads, and allowed adequate pile driving clearance while minimizing excavation and backfill.

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A LANDSLIDE IN PLEISTOCENE DEPOSITS: COLOMBIA (S. AM.)

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ABSTRACT

Landslides in slopes on soft ground subject to a high level of water supply present problems to the Highway Engineer throughout the world. Objective methods of predicting the presence of fossil slides are required so that action can be taken to avoid their reactivation during construction and to enable the logical design of optimum remedial measures for such slides in existing road sections.

Here the solid geological structure is assessed in relation to the location of ancient and modern landslides and a fairly precise association is demonstrated. The particular example chosen to elaborate the prediction and design criteria is the reactivation of a fossil landslide above a truncated, infilled Pleistocene valley in Colombia, South America.

INTRODUCTION

The landslide site on the main road between Ibague and Armero, located in Figure 1, was examined in a collaborative study between British Government agencies and the Ministry of Public Works of Colombia. It was but one among many such landslides investigated in relation to the overall road construction and maintenance program in Colombia. The particular interest in this landslide is that it conforms to a predictable type of mass movement controlled by the so-called reservoir principle (of mass movement) described below.

At the time of the investigation it was essential that the movement should be stopped prior to upgrading the road so that it could be surfaced to a higher standard. A previous site investigation report was available at the time of the visit. The report described the findings of 5 boreholes, both vertically and horizontally inclined.

Here the main geological, geotechnical and hydrogeological characteristics of the site are related to the mechanism of failure commonly associated with the reservoir principle. It is shown that the mass movement could have been predicted prior to the construction of the road. Alternatively, now that the movement has taken place, it is shown that remedial measures appropriate to the site can be determined according to an almost standard design, but taking account of the local surface and subsurface topography of the landslide and the surrounding area.

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*Presented By: Robert M. Smith, Soils and Foundation Engineer, Idaho Transportation Department, Division of Highways.

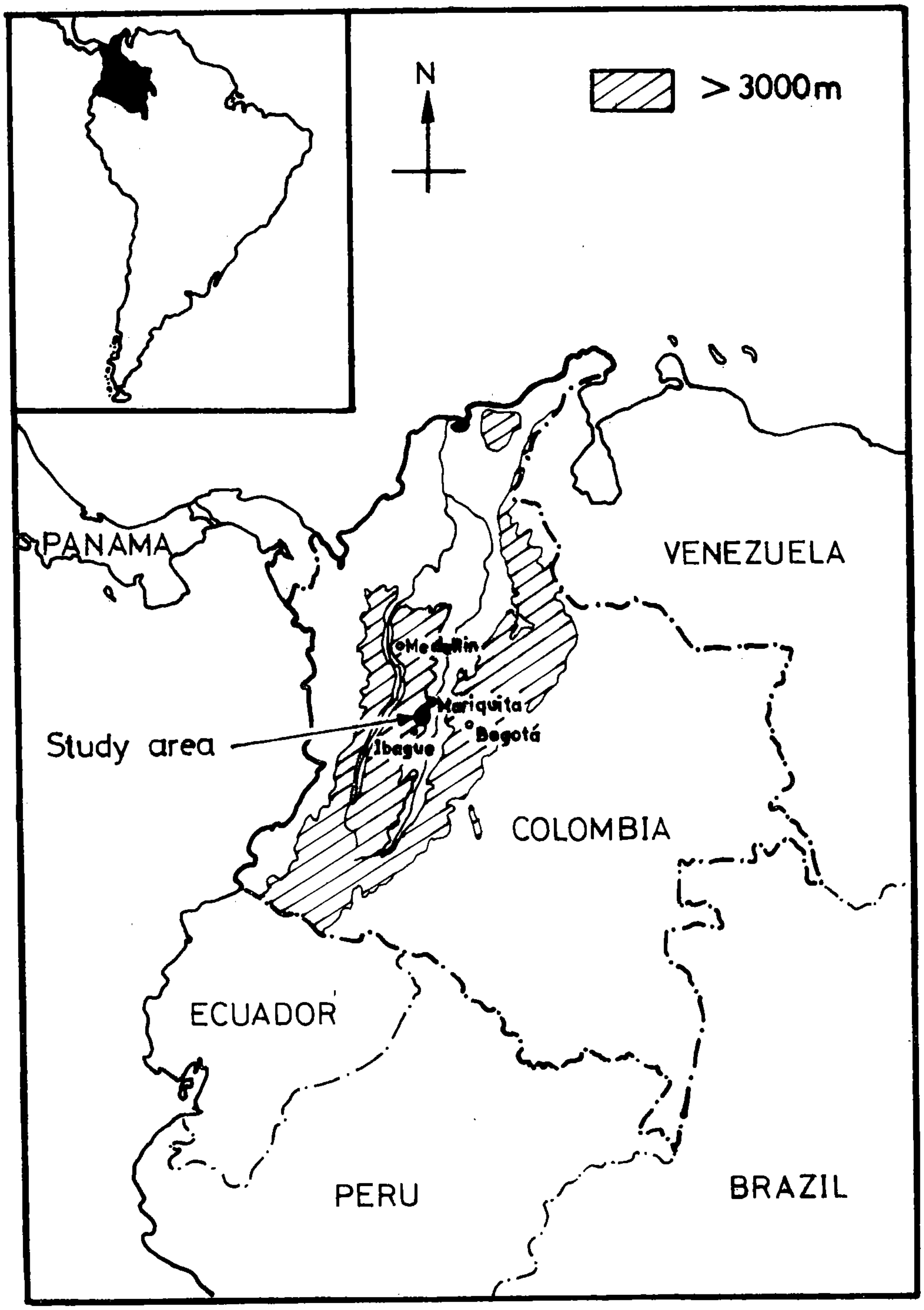

FIGURE 1. Location Map

BACKGROUND

It is appropriate to describe the reservoir principle so that the local site conditions may be related to this general concept.

The Reservoir Principle (of mass movement)

The general principle was described by Denness (1972) as that overall failure mechanism governing all types of landslide complex that degenerate more rapidly from their initial stages of failure in a relatively solid state to a more viscous state than would be the case if they were supplied only with run-off water. The necessary conditions are that a permeable stratum, able to take in and discharge its water store rapidly, overlies a relatively soft and impermeable stratum normally of higher plasticity. The effect of the available water is to permit the removal of soil from the slope as landslide or mudflow, thereby steepening it and so generating further slides to renew the slope wastage. The occurrence of geological conditions favouring the application of the principle is widespread and may usually be predicted from informed interpretation of the local geology on site or, in a general context, from a geological/topographical map.

Denness (1974 a) examined the influence of geological structure on the reservoir principle. From studies of several representative sites in the Departamento de Santander and the Departamento de Norte de Santander in Colombia it was shown that areas of particular instability, within generally unstable areas of colluvium subject to the principle, are often associated with irregularities in the structure of the underlying solid strata. Many structural irregularities are responsible for locally increased groundwater flow from aquifers onto underlying plastic aquicludes. The examples showed that such structural irregularities include general dip, folding (particularly synclinal), faulting and valleyhead erosion; special emphasis was given to the influence of faulting by Denness (1974 b) and of regional dip by Denness and Cratchley (1972). It was also shown that this type of study could not only emphasize areas of certain instability but conversely also demonstrate that neighboring areas on the same strata should be and are stable. The study also underlined the pronounced influence of seepage in the context of this type of landslide activity, emphasizing the need for strategically placed drainage as the main measures required to achieve stability in these conditions. An example of a site investigation based on these considerations and leading to the installation of remedial measures was reported by Denness et al (1975).

Local History

The landslide site, which occupies a section about 40 meters in length along the road, has been active at least since the construction of the modern road. The level of the road has been maintained by regular filling of the subsiding zone. This replacement of subsiding material followed the earlier placing of excavated material adjacent to the present roadside. The section of road across and around the landslide is currently unpaved so that the present method of dealing with the subsidence has proved adequate though expensive. However, plans have been made to upgrade the road and lay a bitumen surface so that it is necessary to prevent any further movement which would lead to considerably greater expense should further remedial measures be required in future.

SITE INVESTIGATION

The site investigation included the accurate surveying of the landslide and its surrounds by the Ministry of Public Works of Colombia and the resulting elevation contours are given on the site map in Figure 2. The sites of five site investigation boreholes are also indicated on Figure 2, two of which are inclined vertically and three horizontally. The vertical boreholes extended to a depth of 18.3 meters and the horizontal boreholes extended to penetrations from 15.25 to 18.30 meters. A core barrel drilling system was used although undisturbed soil samples were not tested. Samples were described and many were tested for relevant geotechnical index parameters in the laboratories of the Ministry of Public Works. The geological logs of all the boreholes are given in Figure 3 (vertical boreholes) and Figure 4 (horizontal boreholes). The geotechnical data is presented in Table 1.

SITE DESCRIPTION

The landslide site is shown in Photo 1. In order to facilitate interpretation of Photo 1, Figure 5 has been prepared to emphasize the superficial geological structure shown on the photograph. Figure 6 is a block diagram based on the borehole and outcrop data, to further aid the understanding of the structure. Photo 1 and Figure 5 are located on the block diagram. Various features of the site are described below.

Topography

The landslide is situated on a slope of about 20 degrees or so to the west. The road crosses the top of the slide largely in cut. However, the residue from the cut has been placed on the downhill side of the road to form a bench adjacent to it. This cut and fill construction technique is very common in sidelong ground but at this particular site, especially at the location of the landslide, the road lies almost entirely in the cut section and the fill forms a wide flat area by its side. At the landslide site the fill now rests, still in the form of a level bench, at about 4 meters below its original level beside the road. As previously mentioned the road has been maintained at its original level following each successive subsidence by continuous replenishment of its foundations. The features described in this section are indicated on Figures 2, 5 and 6. The boundary of the landslide is marked on site by an abrupt step in the level of the bench, demonstrating the movement to be bounded at least in that zone by a discrete shear

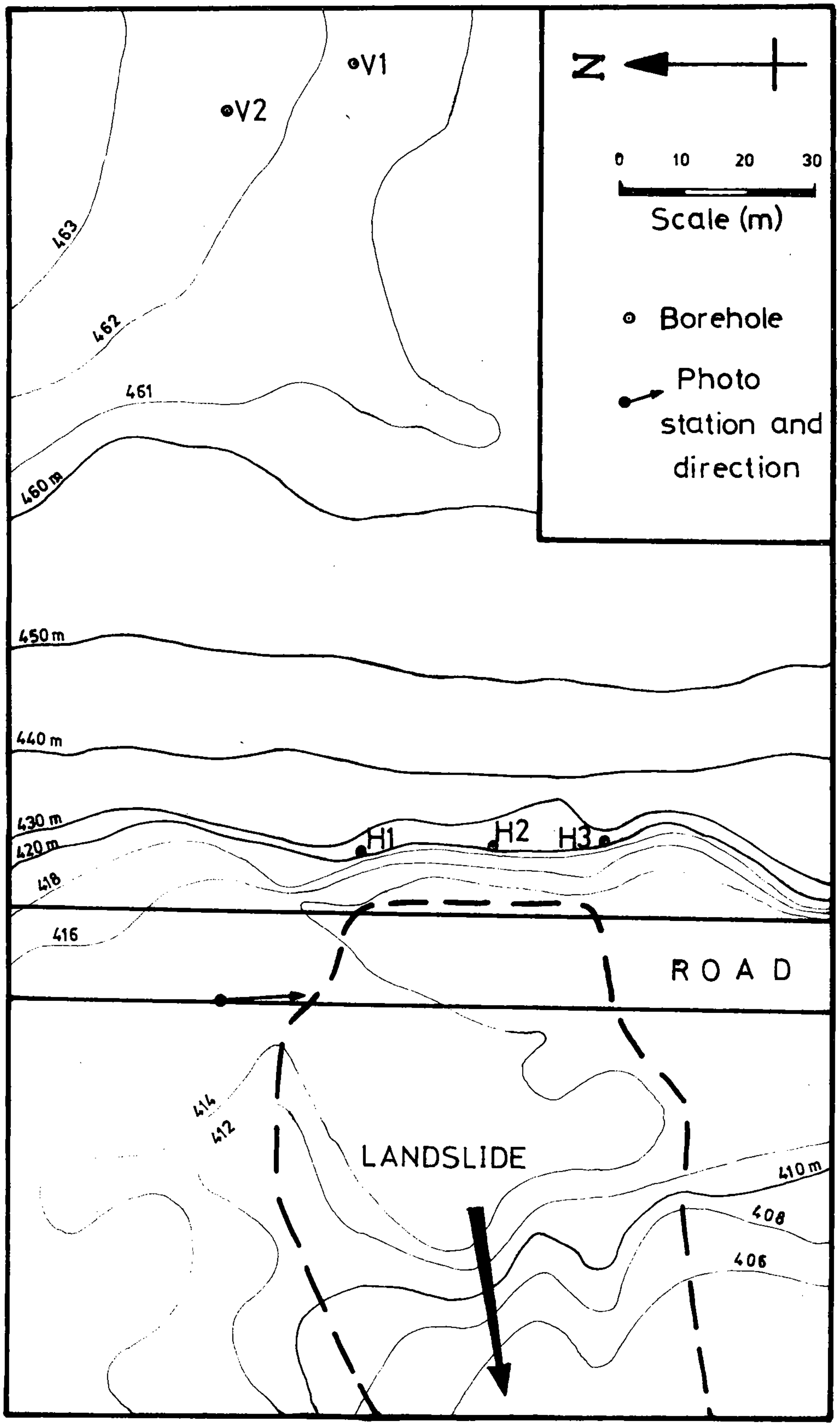

FIGURE 2. Site Plan

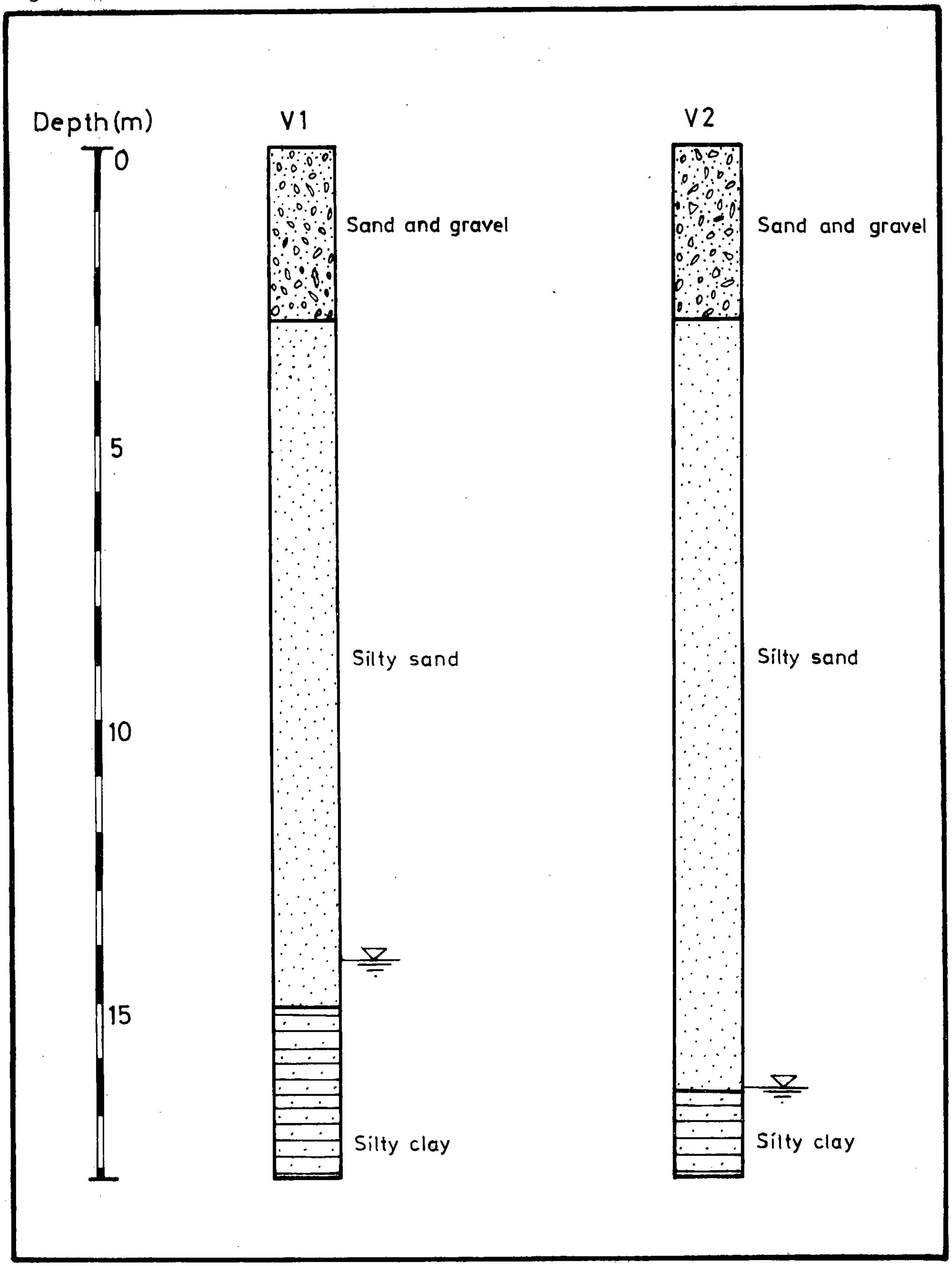

FIGURE 3. Vertical Borehold Logs

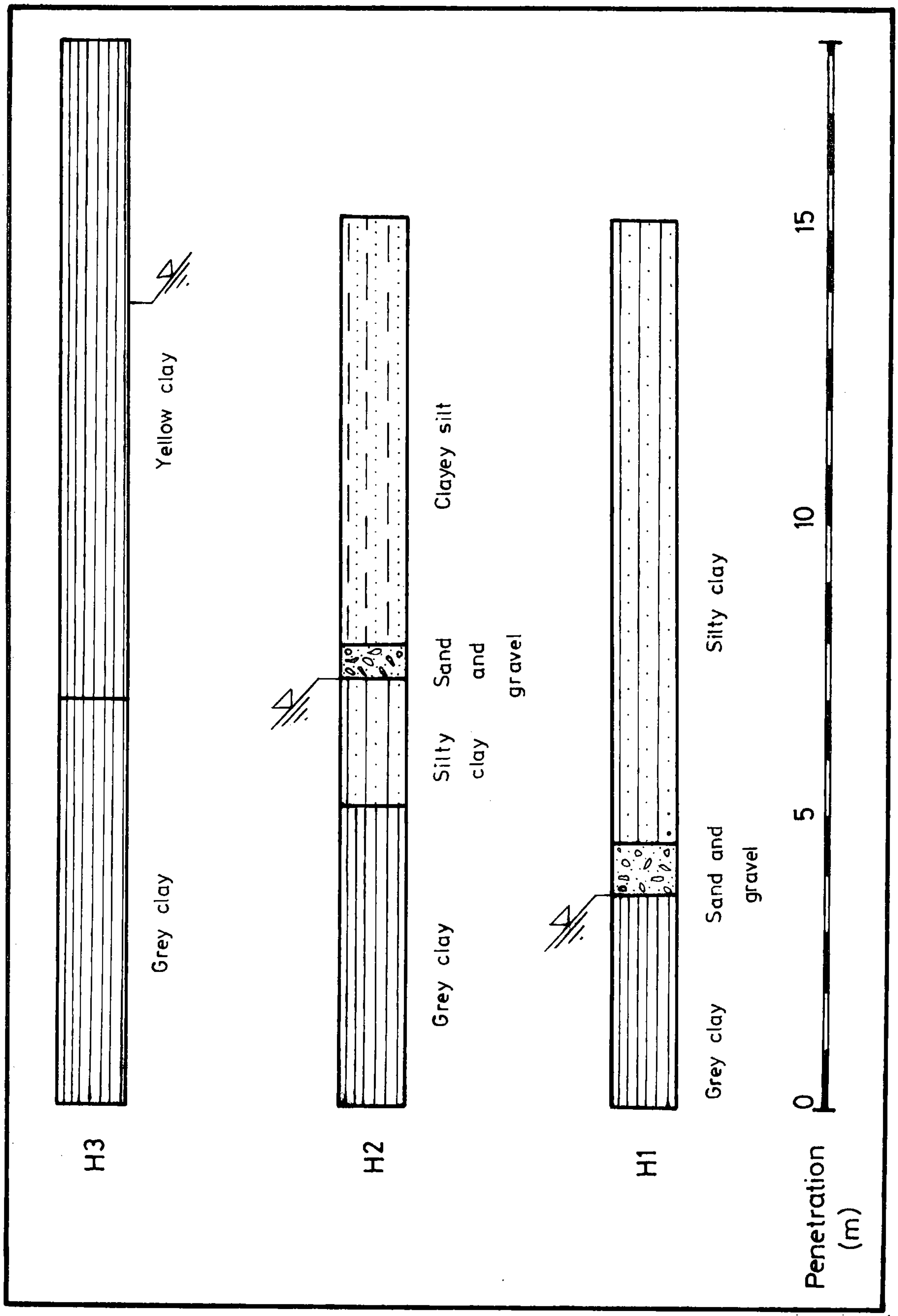

FIGURE 4. Horizontal Borehole Logs

TABLE 1. Sample Descriptions

Borehole No.	Sample depth (m)	Particle % finer				LL	PL	Observation
		Seive 4	Seive 10	Seive 40	Seive 200			
V1	2	96	89	50	22	NON-PLASTIC		Sand and gravel
	10	94	93	64	6	17	5	Silty sand
	14	99	99	79	8	NON-PLASTIC		Silty sand
	17	91	88	85	71	54	27	Silty clay
V2	2	93	87	48	19	NON-PLASTIC		Sand and gravel
	10	88	87	64	6	18	6	Silty sand
	17	100	99	97	90	54	26	Silty clay
H1	2	100	100	98	91	52	27	Grey clay
	4	100	100	98	44	NON-PLASTIC		Sand and gravel
	10	96	95	65	51	51	25	Silty clay
H2	2	100	99	97	87	52	28	Grey clay
	6	99	97	94	82	46	19	Silty clay
	8	100	99	96	34	NON-PLASTIC		Sand and gravel
	10	100	100	97	83	52	21	Silty clay
H3	5	100	99	97	87	51	27	Grey clay
	10	99	98	96	83	45	27	Yellow clay

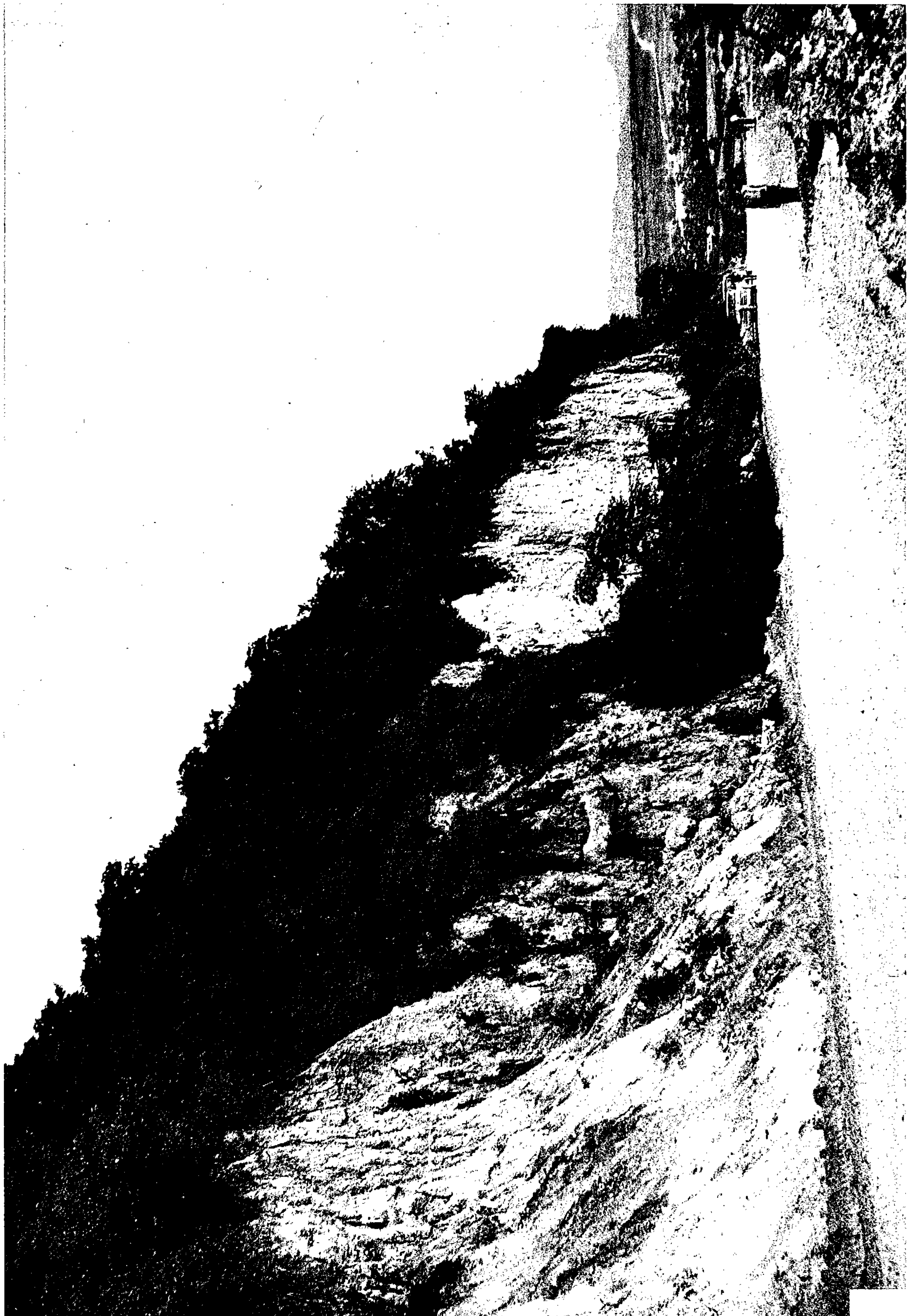

PHOTO 1. The Landslide Site

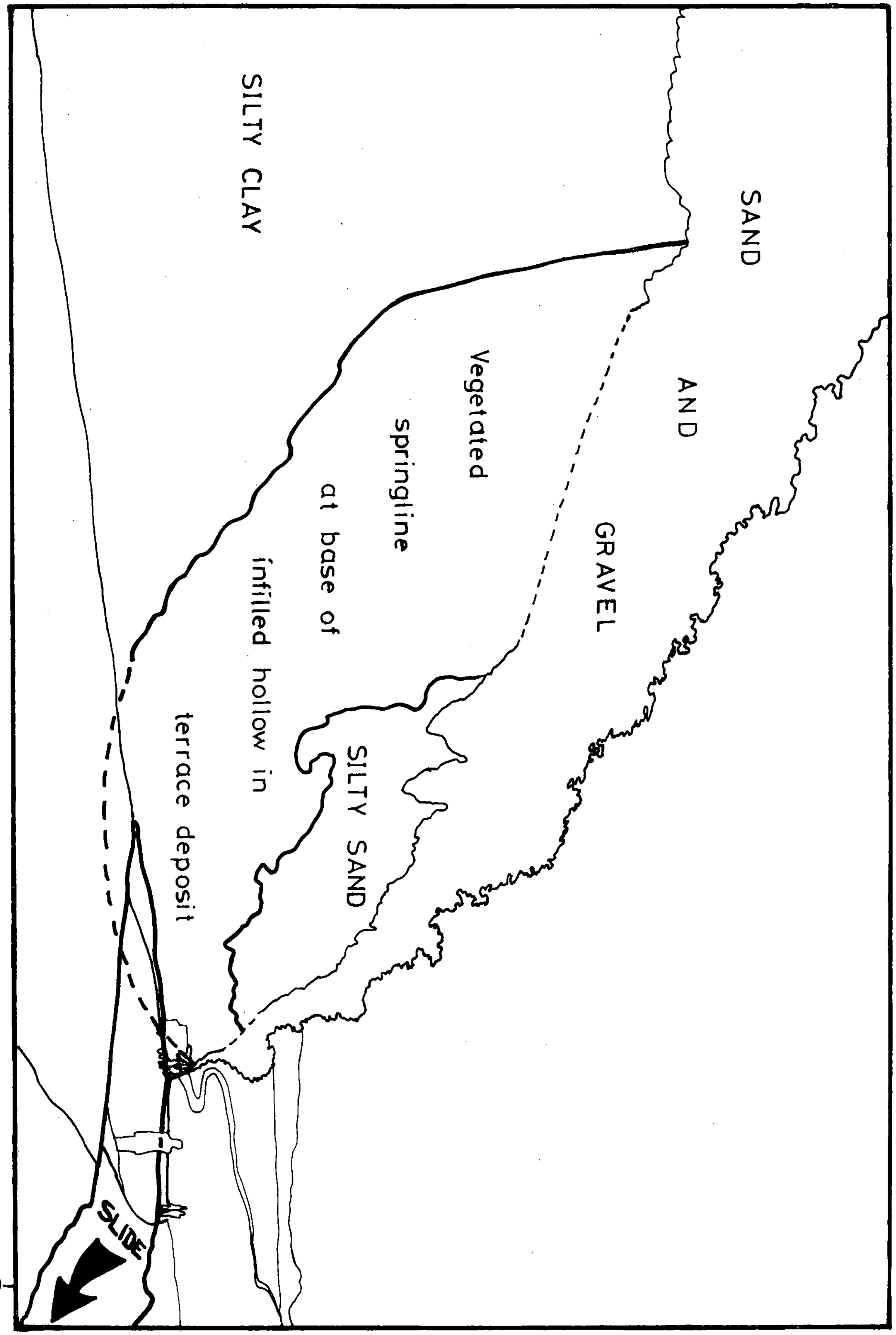

FIGURE 5. Structure and Lithology of Landslide Site

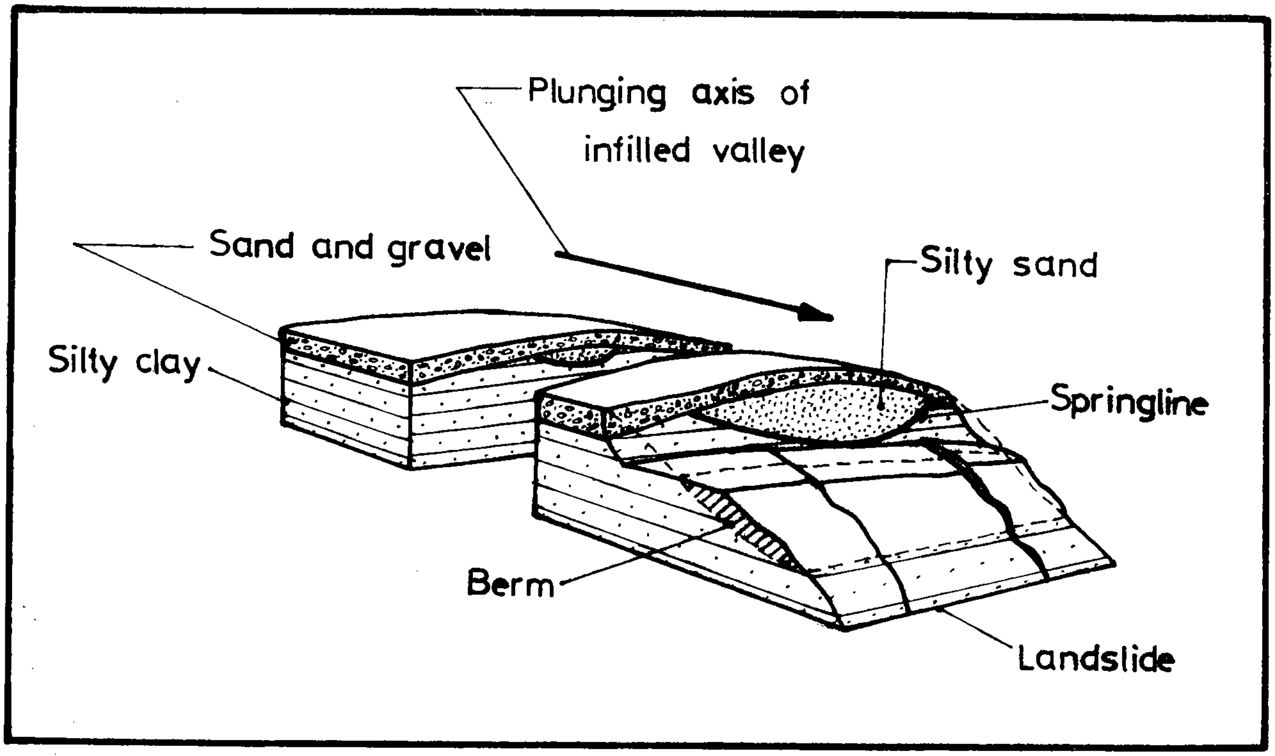

FIGURE 6. Block Diagram of Landslide Site

plane, as shown on Photo 1 and Figures 5 and 6. This demonstrates that the subsidence of the road is due to slipping and not merely to excessive local compaction or consolidation which would not induce such an abrupt boundary. The landslide topography downhill of the backscarp exhibits the form of a typical viscous mudflow with hollows and occasional backscarps and cracks near the top of the moving mass and convex lobes towards the bottom of the hillside. These features extend for several hundred meters downslope from the road.

The terrace deposit above the road stands in a vertical cut. Above the cut the terrace exhibits a typically flat form indicative of its depositional environment and minimal subsequent erosion or physical alternation. It is a very young and sharp landform.

Geology

The lithological characteristics of both the landslide and the surrounding undisturbed material along with the geological structure of the locality contribute to the understanding of the type of instability.

1. Lithology

The geological succession at the site of the landslide is a clastic Quaternary terrace deposit overlying a weakly cemented or slightly cohesive silty clay, as shown in the borehole logs described in Figures 3 and 4. The terrace deposit is made up of large rounded and subrounded cobbles and small boulders in a matrix of silty sand with abundant rounded and subrounded gravel. The terrace is effectively cohesive (probably cemented) in its undisturbed state and stands vertically in cuts to at least 20 meters at neighboring sites. The effective cohesion may be attributed to the wide range of particle sizes, assisted by a small negative pore pressure resulting from capillarity in the phreatic zone above the water table level and minor cementation. The water table is indicated by a springline at the base of the deposit.

The underlying bed is evidently less permeable than the terrace, as shown on site by the exact coincidence of the springline with the contact between the two deposits and also by its smaller constituent particles and its more plastic nature (described in Section 5). Photo 1 shows both deposits clearly and also shows a line of luxurious vegetation growth along most of the contact between them. The vegetation marks the springline at which groundwater, percolating through the terrace deposit, is expressed.

2. Structure

Much of the area surrounding the site of the landslip exhibits a similar geological succession. The road cuts on either side of the site provide good sections through the terrace deposits and occasionally cut through the underlying silty clay. The three dimensional geological structure of the area is interpreted from the borehole evidence and surface exposures in Figure 6. The linear section created by the road cut demonstrates that the contact between the terrace deposit and the underlying silty clay varies in elevation over fairly short distances across the site. At the site of the landslide the contact dips into an apparently synclinal structure, the axis of which coincides exactly with the center-line along the direction of movement of the landslide. This structure is illustrated in the partially exploded block diagram in Figure 6 and is in fact a depositional feature caused by the infilling of a small Pleistocene river channel by the later terrace deposit. The infilled valley has since been truncated by erosion to expose the local hollow in the contact.

3. The Landslide

The disturbed material downhill of the road is comprised of an admixture of rock and soil derived from both the terrace deposit and the underlying silty clay. This debris, which now forms part of a colluvial blanket to the hillside below the terrace, is variable. Some parts are largely composed of terrace deposit derivatives while others are largely silty clay. Consequently it is likely that the groundwater regime and natural drainage within the landslide is very complicated.

Hydrology

The hydrological regime of the site is concerned with both surface water and groundwater. The surface water regime, which was difficult to evaluate from the short duration site examination in a dry season, contributes to the groundwater regime. In turn the groundwater regime is controlled by the geological structure.

1. Surface Water

There is little or no natural expression of surface water on the terrace deposit. However, owing to its deployment as agricultural land, it is irrigated through irrigation channels. The water rapidly percolates into the soil and subsoil but maintains a moist topsoil prior to infiltration.

The colluvium which blankets the hillside below is similarly devoid of major natural water courses. As it is not cultivated there is no man-made water supply to supplement the natural runoff either. The main exception to this is at and adjacent to the landslide site where small surface streams are apparently fed by groundwater locally expressed at the springline along the contact between the terrace deposit and the underlying silty clay as indicated on Figures 2 and 6.

2. Groundwater

The groundwater regime within the very permeable terrace deposits is controlled by two main factors, input from irrigation and discharge along the contact with the underlying silty clay. At the road cut the springline is at that contact. The boreholes described in the following section show the ambient groundwater level at about 100 meters to the east of the road cut to be still within one meter of that contact. Just above the contact the permeability of the overlying sandy silt and gravel of the terrace deposit increases as indicated by the sudden loss of drilling fluid during the drilling operation. This band of higher permeability is less than one meter thick and rests upon a clay deposit below.

GEOTECHNICAL DESCRIPTION

The results of the drilling and testing carried out by the Ministry of Public Works are consistent with site observations. The main geological components contributing to the slide have been described as the terrace deposit and underlying silty clay. Particle size analyses in the laboratories of the Ministry of Public Works confirmed the earlier description of the terrace deposit except that the larger fraction was not recovered because of the restricted size of the core barrel. The terrace deposit is primarily a sand, which implies a high permeability and a low plasticity. The results of the testing carried out by the Ministry of Public Works, and given in Table 1, show that the terrace deposit is indeed much coarser grained than the underlying silty clay. Much of it is non-plastic while the more silty parts exhibit a very low plasticity. These properties imply that the deposit would very rapidly break down if it were disturbed. It would then flow as it remolded in the presence of water. It is possible that its natural water content might be sufficient to maintain the flowing mechanism once the deposit had been disturbed.

Both the vertical and the horizontal boreholes drilled by the Ministry of Public Works proved the underlying silty clay to be moderately plastic. The tests carried out on the samples also showed that the material is substantially finer grained than the overlying terrace deposit, implying a lower permeability and thereby providing a quantitative explanation of the presence of the observed springline along the contact between the two geological units. Unfortunately there is no data available to relate the variation of the natural moisture content of the silty clay within the hillside to its plasticity characteristics. However, it is sufficient to note that in general materials of this type can remold very easily under stresses caused by the cutting or loading of slopes in the presence of excess groundwater. When such materials are mixed with coarser grained soils their overall plasticity reduces so that they require less water to maintain their flow. In this case excess water is readily available from the overlying terrace deposit, which can also provide a coarse-grained component to the solifluction deposit resulting from an initial disturbance.

DISCUSSION

It is appropriate to describe the application of the reservoir principle to the landslide including an overall assessment of the topographical, hydrological, geological and geotechnical characteristics of the site. The suggested remedial measures that follow are a consequence of this application.

The Local Application of the Reservoir Principle

The lithological and geotechnical characteristics have been described for the main rock units involved in the landslide. These are the terrace deposit and the underlying silty clay which correspond respectively to the groundwater reservoir and the underlying plastic aquiclude referred to in the general reservoir principle. The geological structure at the site has been described as a truncated infilled valley with its axis along the centerline of the landslide in the direction of movement. The block diagram shown in Figure 6, compiled largely from borehole evidence, shows that the axis of the infilled valley plunges downslope. This type of structure is similar in effect to a syncline which has previously been shown by Denness (1974 a) to exert an exaggerating influence on the reservoir principle, thereby increasing the likelihood of instability at the site due to the concentration of groundwater discharge there while reducing it at neighboring sites. The evidence of this concentration of groundwater flow at and above the landslide site is the observed increase in permeability near the base of the vertical boreholes and at the springline, resulting in luxurious vegetation growth.

In summary the site is seen to be an excellent example of a slope instability caused by the localizing effect of a buried valley on the reservoir principle.

Possible Remedial Measures

Appropriate remedial measures fall into two categories. The main part of the system should be an adequate surface and groundwater drainage facility. This should be located primarily at the top of the landslide, i.e., at the contact between the terrace deposit (groundwater reservoir) and the underlying silty clay (plastic aquiclude), throughout the expression of the springline marked by increased vegetation cover across the truncated buried valley. The effect of such drainage would be to collect and canalize the expressed groundwater so that the groundwater reservoir would no longer discharge onto the landslide. After a short period of consolidation caused by evaporation, the landslide should stabilize provided the original slope morphology has not been substantially altered.

The optimum drainage system would be a series of short bored drains lined with perforated plastic tubes about 4 meters long at 3 meter centers inclined upwards at about 10 degrees from a line about 30 centimeters below the abrupt contact between the terrace deposit and the silty clay. The tubes should discharge into a lined collection trench about 1.5 meters deep which should drain downhill from the center of the system to a central lined collection chamber (which would also serve as a sand trap) at the axis of the buried valley. The overflow from the collection chamber should be piped in a flexible drain to a culvert which should be installed beneath the road about 7-8 meters to the north of the collection chamber. This culvert should be joined to a flexible drainage pipe leading to a natural drainage point downhill of the subsiding zone and perhaps 100 meters north of the site. The collection chamber should be low enough to drain the standing water from uphill of the road.

The second stabilization procedure involves restoring the pre-excavation topography. The overall situation was naturally precarious as a result of the historical influence of the reservoir principle prior to the road construction. The problem was further aggravated by the placing of fill as a berm immediately west of the cut, thereby increasing the load on that part of the slope downhill of the groundwater discharge from the springline. The failure resulted from increasing the stress level at the wettest (weakest) part of the hillside. Either increasing the strength by reducing the moisture content or reducing the stress level by removing the berm may succeed in stabilizing the slope. However, the optimum procedure would be both to install the drainage measures and to remove the berm. The spoil should be distributed evenly on the slope to either side of the landslide zone downhill from its present location, never exceeding one meter in thickness and avoiding the outflow from the drainage pipe discharging about 100 meters to the south of the slide. The comprehensive system is illustrated in Figure 7. It should be economical to install and become effective in the fairly short term so that the road could be permanently surfaced as part of the upgrading program. This stabilization procedure has been used in principle on many other sites controlled by the reservoir principle.

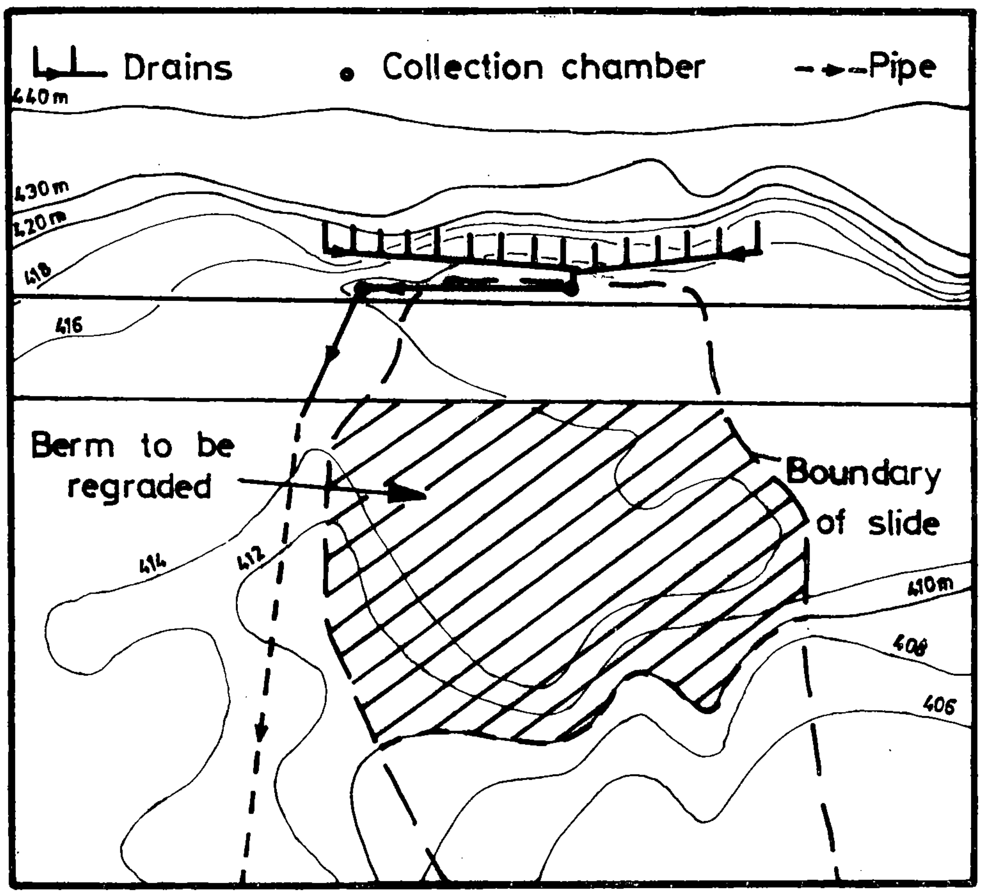

FIGURE 7. Proposed Remedial Measures

CONCLUSIONS

1. The instability is due to the influence of the reservoir principle on a truncated buried valley in silty clay infilled with terrace deposits. This has caused the local concentration of groundwater at that point with consequent softening of the near-surface materials below the spring-line.

2. The placing of fill on these softened soils triggered the ensuing landslide which has been maintained by a continuing supply of groundwater discharge and the frequent placing of more fill to achieve a level carriageway at the head of the landslide.

3. The landslide should be stabilized by the installation of drainage measures to collect and canalize the groundwater discharge and by the redistribution of the fill away from its present position at the head of the unstable slope.

4. This is but one among countless examples to which similar site investigation and remedial procedures might be applied, each tailored to the local geological structure.

ACKNOWLEDGEMENTS

The author is indebted to many members of the Colombian Ministry of Public Works for their assistance in both the field and the laboratory. He is particularly grateful to Dr. Enrique Pinilla who accompanied him to the site and helped with the site description. The author is also indebted to Mr. Donald Newill and Mr. John Dowling of the Overseas Unit of the Transport and Road Research Laboratories (U.K.) who organized most of the earlier parts of the Colombian study. He is also grateful to the Ministry of Overseas Development (U.K.) and the Institute of Geological Sciences (U.K.) for financial support in this work.

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SKYLAB EXPLORES THE EARTH

By

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ABSTRACT

Investigation of Earth's dynamic processes is of direct interest to many scientists including geologists, oceanographers, and meteorologists. Studies of features and phenomena that are indicative of these processes have been conducted with aircraft and satellite data. However, such data are limited to small coverage (aircraft) and constant viewing conditions (ERTS). During the third manned mission on Skylab, an experiment was conducted by the crewmen to determine what type of earth survey information man could obtain through visual observations and by handheld cameras.

More than 850 observations and 2,000 photographs were taken for 16 different scientific disciplines. Observations and photographs were taken over the entire range of possible Sun angles (twilight to local noon) and viewing angles (high oblique to vertical). Results of the experiment confirm that man's ability to recognize objects and patterns, to integrate his observations over a range of aspects and lighting angles, to reason, and to make selective observations, can bring another dimension to the study of the earth.

PRESENTATION OF TEXT

A new dimension in the study of Earth has evolved as the result of the Visual Observation Experiment conducted during the last Skylab manned mission. Observing the Earth was an important part of the astronauts activities during the Mercury (1962-63), Gemini (1965-66), and Apollo (1968-71), earth orbital missions, and many spectacular photographs of the land, oceans and atmosphere were obtained with handheld cameras. Prior to each Gemini ad and Apollo mission, the astronauts were briefed on the types of terrain features which were of direct interest to geologists and geomorphologists and what sites should be photographed with color and color infrared films. The practical application of this photography in geology has been demonstrated by Lowman (1969), and Short and Lowman (1973). The early manned missions were of short duration (< 14 days) and because of other mission requirements, limited time was available for earth terrain photography. The successful launch of Skylab in 1973 provided an opportunity for man to observe and study the Earth for periods of 28, 59, and 84 days (Table 1). Repair of Skylab during the first manned mission reduced the crewmen's chances

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[4]Johnson Space Center

TABLE I. Skylab Program

Mission No.	Crewmen	Lift-off Date	Splashdown Date	Days in Space
[a]1	Unmanned	May 14, 1973	--	
2	Conrad Kerwin Weitz	May 25, 1973	June 22, 1973	28
3	Bean Garriott Lousma	July 28, 1973	Sept. 25, 1973	59
4	Carr Gibson Pogue	Nov. 16, 1973	Feb. 8, 1974	84

[a]Skylab workshop launched. The workshop operated unmanned between manned missions.

to photograph the Earth. However, on the second manned mission, the crewmen requested additional work and an experiment was begun to explore man's overall capability to discern, observe, describe, and photograph a wide variety of terrain, ocean and atmospheric features. For the third manned mission (termed Skylab 4), a Visual Observation Experiment was initiated to further determine what photographic and observational data could be acquired by the crewmen when supported by multidisciplinary scientific training and an onboard comprehensive set of procedures, maps, and photographs. The objectives were to determine:

1. The types of surface, air, and water phenomena the crewmen could visually identify from the Skylab orbit.

2. Which visual observations, supplemented by photography, could be accomplished to support scientific investigations.

3. The use of observational and sensor data in the study of multiple discipline areas.

4. The types of crew training necessary to perform the visual observations desired by the scientific investigators.

Skylab was launched into a nearly circular orbit 433 kilometers (235 nautical miles) above the Earth. Moving at approximately 4 miles a second, the spacecraft orbited the Earth every 93 minutes along a flight path that cross-crossed the world between latitudes 50°N and 50°S (Figure 1). These

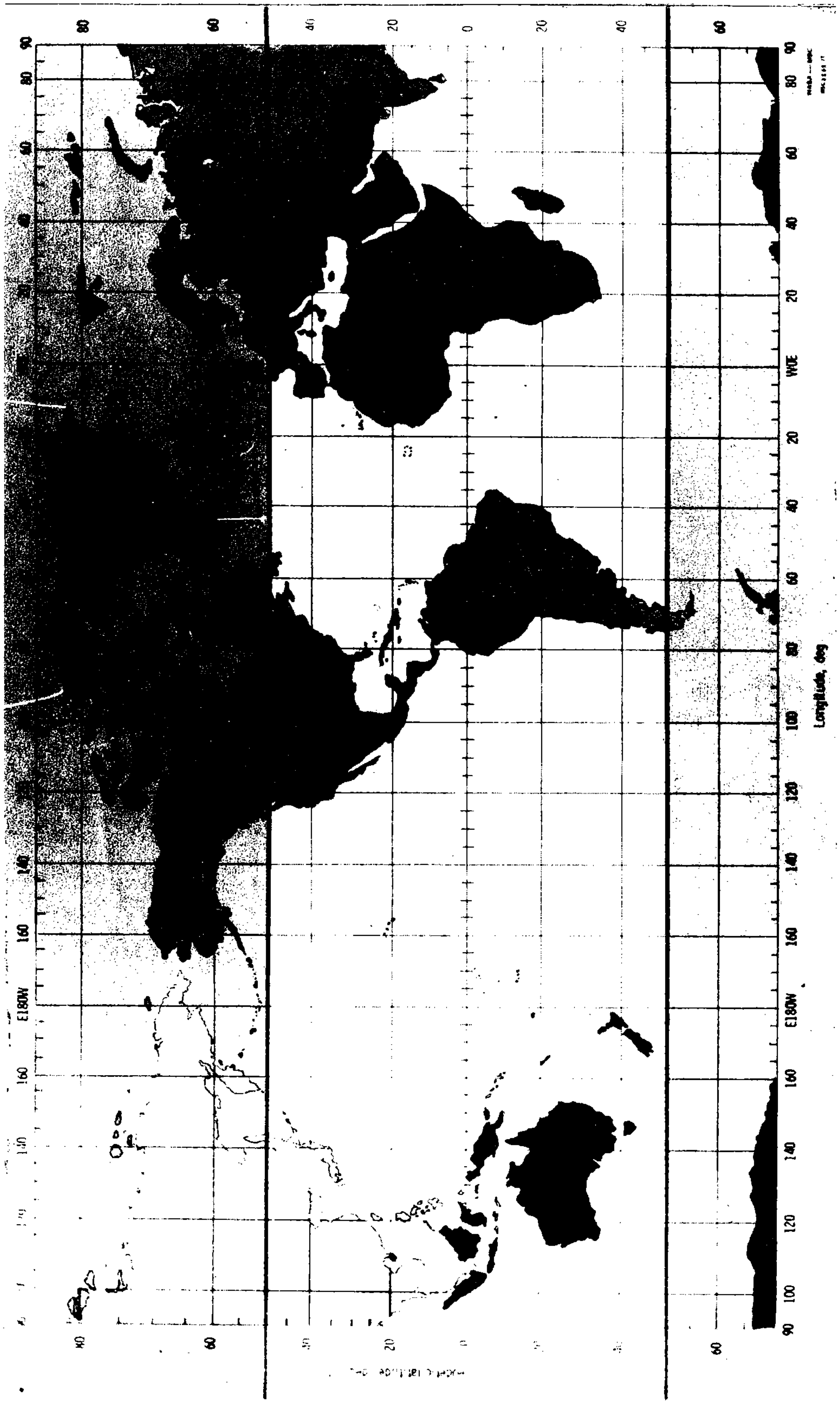

Figure 1.- Skylab ground coverage.

orbital characteristics allowed the spacecraft to overfly the same ground track every 5 days and greatly increased opportunities for the crewmen to observe and photograph the same area, features, or phenomena. During the three-month mission, several changes and variations in sun angle provided the crewmen a new perspective of the changing surface of the Earth.

Training the Skylab 4 crewmen to capitalize on the many opportunities to observe and photograph the Earth during the 84-day mission was initiated in August 1973, some 3 months before the mission. The Skylab 3 crewmembers reported difficulty both in locating observation sites with aid of the limited onboard data package and maps, and in performing the requested visual observations. To overcome these problems, a multidisciplinary team of 19 scientists lectured the Skylab 4 crewmen and assisted in the preparation of the onboard data book and maps. The crewmen received 19 hours of training on such diverse subjects as global geologic features, hurricanes, snow mapping, deserts, ocean phenomena, sea and lake ice dynamics, air and water pollution, and cultural and vegetation patterns. From these lectures and discussions, the crewmen gained insight into the significance of the observational data and an awareness of the type of photographs and visual descriptions that needed to be obtained for specific studies. The instruments onboard Skylab for the visual observations experiment consisted of Hasselblad (70mm) and Nikon (35mm) cameras equipped with color, color infrared, and black and white film. The Hasselblad camera was equipped with a 100mm focal length lens; the Nikon with a 55 and 300mm lens. For visual observations, the astronauts used 10x40 handheld binoculars and the Mark stabilized binocular with 10x50 magnifications.

The Skylab 4 mission resulted in 850 recorded, formal observations by the crewmen and approximately 2,500 photographs that documented these observations. A Preliminary Skylab 4 Visual Observations Project Report published in June 1974 (Kaltenbach, et al) described the experiment activities during the mission and gave the preliminary results of analysis of selected photographs and observations by the science team members. A final report is being prepared and will be published in the near future.

Earth-orbiting platforms such as the operational LANDSAT I and II and Nimbus, and a planned satellite designated Seasat, and the Heat Capacity Mapping Mission (HCMM) sense the Earth on command from ground stations. These systems collect information across many segments of the electromagnetic spectrum from sensors that always point at nadir or virtually toward the Earth's surface. The data obtained is in an electronic-type format and can be converted to images for detailed photo-analyses. Several tens of thousand images have been obtained by LANDSAT I and II of major portions of the land and adjacent coastal areas of the Earth. An example is the color composite of the Snake River Plains of southeastern Idaho (Figure 2). A similar view (Figure 3) taken by one of the cameras in the Earth Resources Experiment Package (EREP) aboard Skylab illustrates the high resolution capability of remote sensing cameras from a space platform. The advantages of imagery from LANDSAT and EREP sensors are synoptic coverage (~ 35,000 sq. Km,

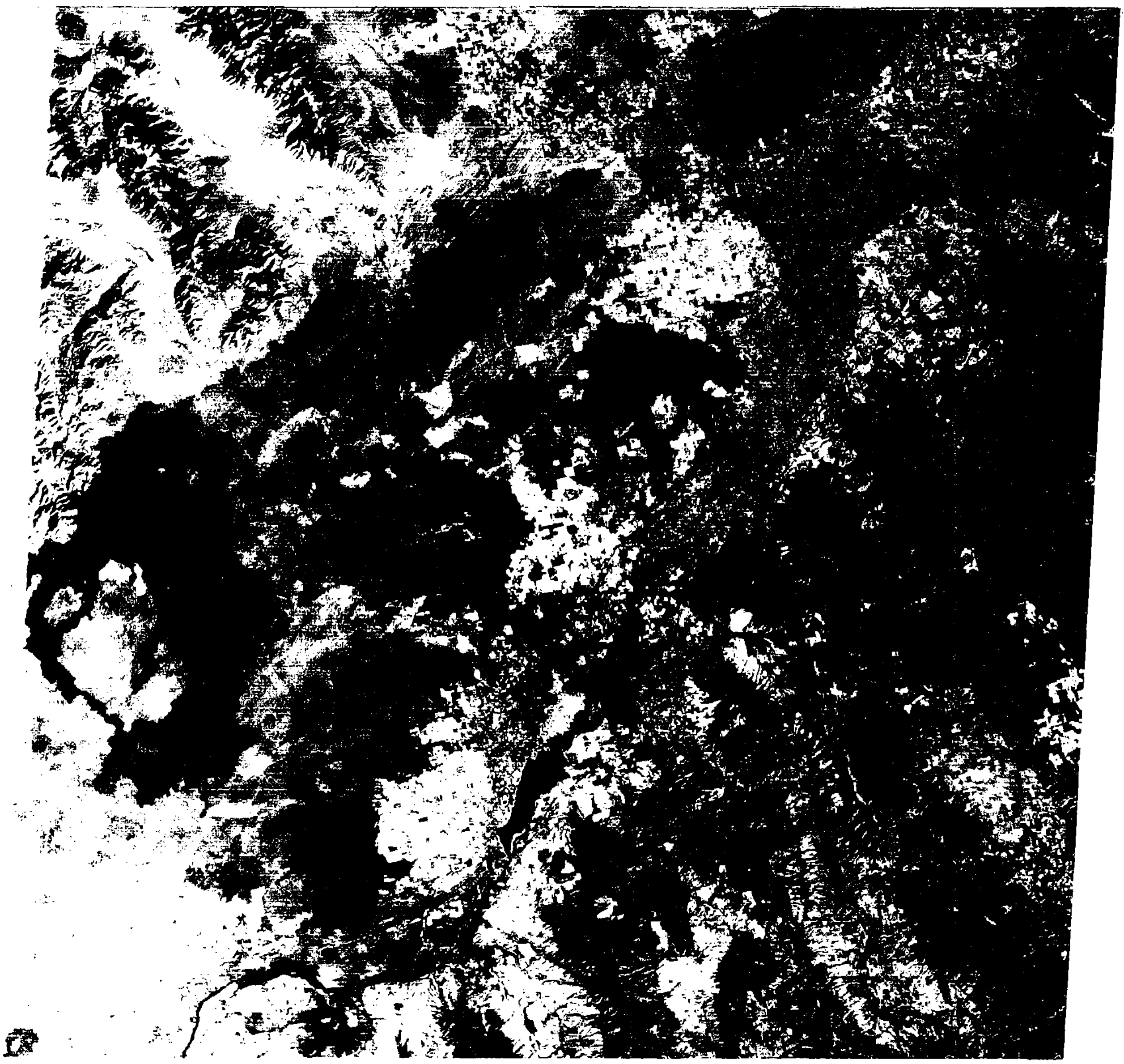

Figure 2.- LANDSAT color composite image of the Snake River Plains, central Idaho.

Figure 3.- Skylab terrain camera photograph of Craters of the Moon Region, central Idaho (SL3-86-333).

10,000 sq. miles), high resolution (~ 100m), stereo views and data recorded in the visible to near infrared spectrum. In contrast, most handheld photographs require rectification for quantitative comparison to existing maps. The spectral response of the film is low, thereby rendering data enhancing techniques such as density slicing and color separation impractical. There are inherent disadvantages of handheld photography in terms of image analysis, but the results of Skylab 4 observations, complemented by analysis of photographs obtained to enhance selected features or phenomena, have shown that new scientific data can be acquired on dynamic processes or systems on and near the surface of the Earth. This preliminary report describes these results and summarizes the utility of man to visually observe (using cameras and binoculars) and thereby perform as a data collection, integrating, and analytical system.

In the preflight training of the Skylab 4 crewmen, discipline discussions were divided equally between the terrain, oceans, and atmosphere. For convenience, this order will be followed in discussion of specific examples. Significant accomplishments in terrain analysis include discovery of new structure in impact craters, new data on volcanic eruption sequence, development of desert dune classifications, and new evidence on generation of drought conditions such as in the Sahel zone of Africa.

A striking geologic feature observed on the Canadian Precambrian shield is the circular ring outlined by waters of the Manicougan Reservoir (Figure 4). The ring, with a diameter of 65km (40 miles), was considered originally to be of volcanic tectonic origin (Hammond 1946). In 1972, however, field studies revealed presence of breccias and shock metamorphic effects and a meteorite impact was postulated (Dence, 1964). This wide-angle oblique Skylab photograph obtained by the crewmen indicates the size and relation of the crater to the terrain and provides the first clear indication of an outer circumferential depression which is interpreted as the fracture-zone boundary between disturbed and undisturbed rocks. This discovery can be applied in determining the origin of similar land forms on the Earth, Moon and other planets of the solar system.

There are approximately 600 volcanoes in the world that are classified as active and most of these form a ring around the Pacific Ocean. With the exception of a very few such as Manua Loa, Hawaii, the nature of volcanic eruptions is little understood. The Skylab 4 crewmen (for the first time from space) observed and photographed the eruptions of the volcanoes Sakura-Zima, Japan, and Fernandina in the Galapagoes Islands, Ecuador. New and important information on the distribution of volcanic dust in the atmosphere has been obtained from quantitative stereo analyses of the handheld photographs (Figure 5) combined with upper atmospheric data from the Kyushu meteorological stations. These results show that the eruption cloud from a volcanic eruption whose daily kinetic release was 10^{18} ergs, created a stratosphere dust veil of long duration, but did not penetrate the troposphere. Direct evidence such as this obtained from Skylab is important in research on contamination of the stratosphere.

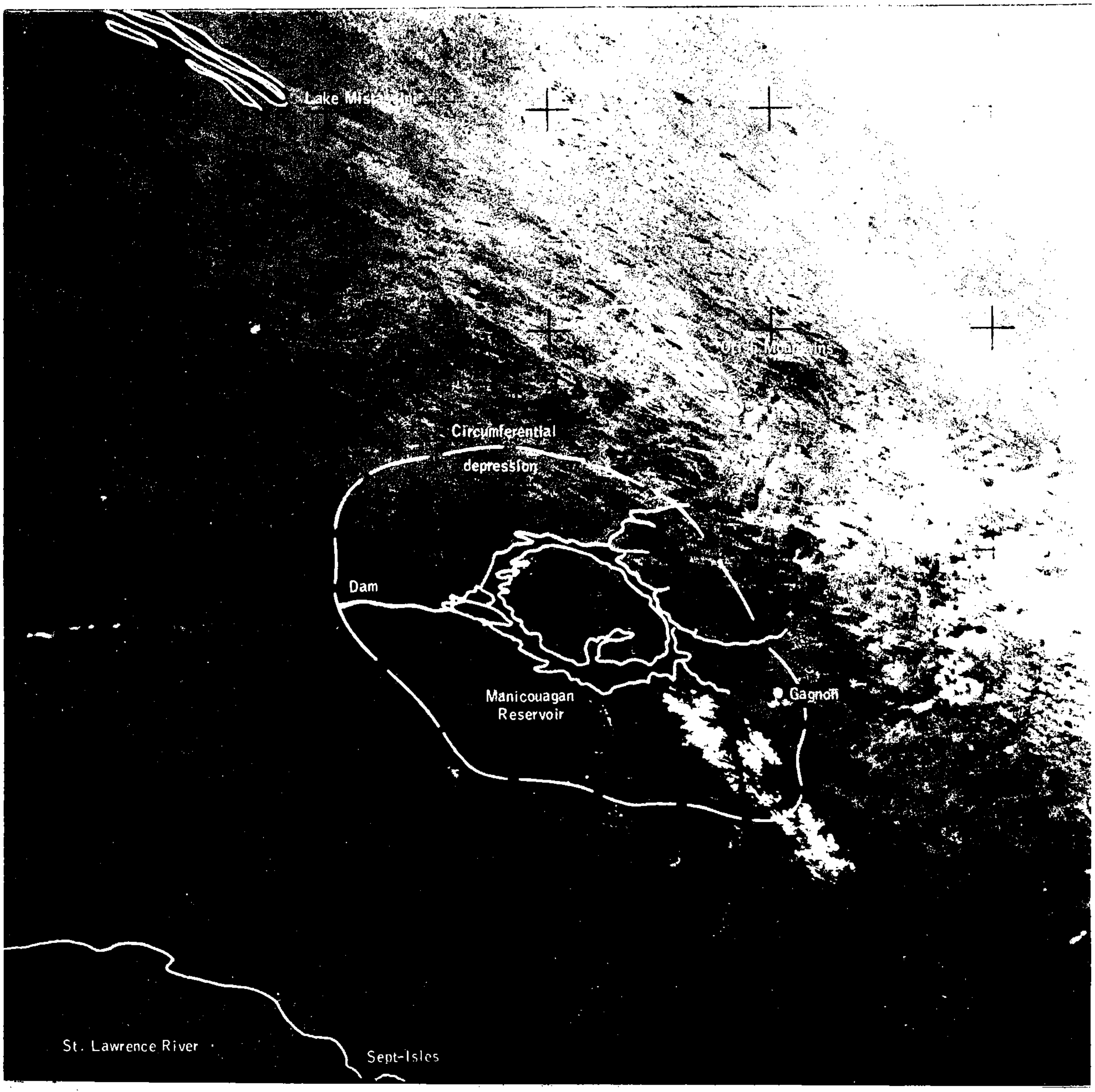

Figure 4.- Handheld photograph of Manicouagan Crater, Quebec, Canada showing distinct circumferential depression (SL3-122-2628).

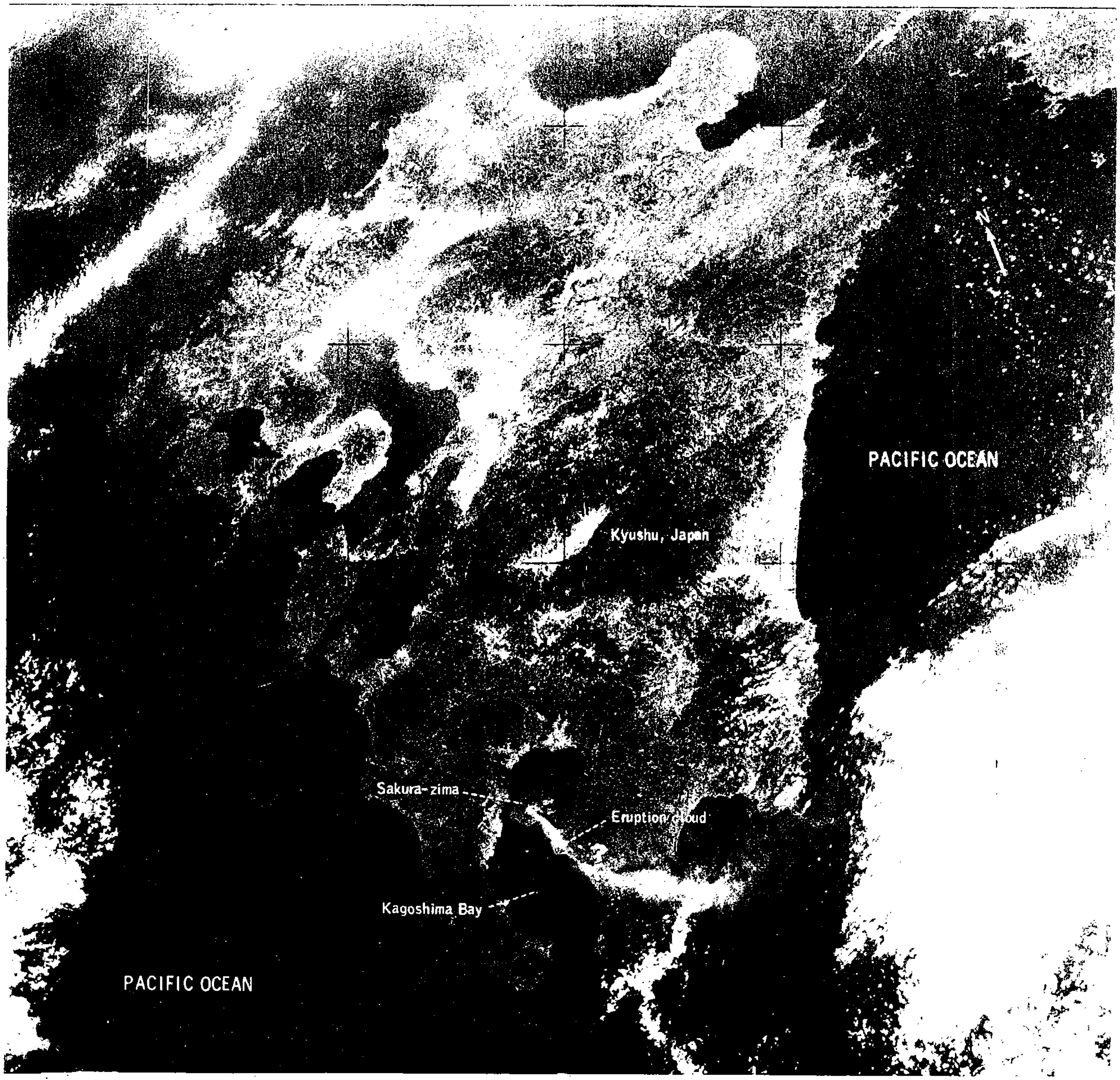

Figure 5.- Eruption cloud from Sakura-zima, Japan (SL4-139-3942).

On December 11, 1973, the crewmen were the first humans to photograph the Fernandina volcanic eruption and document the scale and extent of the plume. The view from Skylab is shown in Figure 6. As the crewmen viewed the Earth, under changing sun angles and seasonal conditions, striking photographs of surface features were obtained such as this high oblique of the Kamchatka peninsula, USSR (Figure 7). In studying these features on space photos, volcanologists see for the first time relationships of many linear features possibly tectonic in origin to volcanic centers and thus complement the data obtained from the sensors on LANDSAT.

In premission training of the Skylab 4 crewmen, a major emphasis was placed on the development of skills and interests of each crewman in conducting, by comparative analysis, studies of features and phenomena not described in the lectures. As a result, spectacular views of the vast volcanic province in the Altiplano region of Argentina, Bolivia and Chile were recorded (Figure 8). From analyses of the photographs, new data on the types of volcanoes and their extrusive features were determined; adding to the knowledge of the geologic history of this relatively inaccessible part of South America.

The analytical capability of man to apply knowledge from one region of the world to another can be applied to the study of remote regions such as deserts of Africa, China and Russia. The vastness and inaccessibility of deserts has increased the use of space data for regional analyses. Early work by McKee, et al (1974) resulted in a tentative global classification of dune sand deposits derived from limited ground surveys and LANDSAT images. The Skylab 4 crewmen, using premission instructions, confirmed the tentative classification scheme by describing similarities of features, structures and colors of the major deserts. As seen from Skylab, the dunal patterns, some as much as 1000 km long, are similar in the Simpson desert of Australia and the Empty Quarter of Saudia Arabia (Figures 9 and 10).

From the study of desert features and their relation to winds, topography, and rainfall, some insight has been obtained on the process of desertification and its companion, aridification. Within the Sahelian Drought area of West Africa the SL4 astronauts observed slash burning for land clearing along fire lines 640 km long, dust storms, and a bland appearance of the region -- later determined to be due to atmospheric dust. Over this area, they obtained photographs that indicate the land use (cultivation) patterns were similar from the Sahelian Drought Zone to rain forest that border the Sahel to the south. From interpretation of Skylab data, LANDSAT image analysis, and complementary field studies, the concept that in West Africa man's cultivation practices of forest removal (Figure 11), eventually leads to erosion, an increase in dust accumulations, aridification, and desert formation. Further analyses of this concept as it applies to other desert regions is underway by arid land experts at the American University.

Oceans cover nearly three-fourths of the Earth's surface and for centuries oceanographers have been restricted by the limited mobility of their

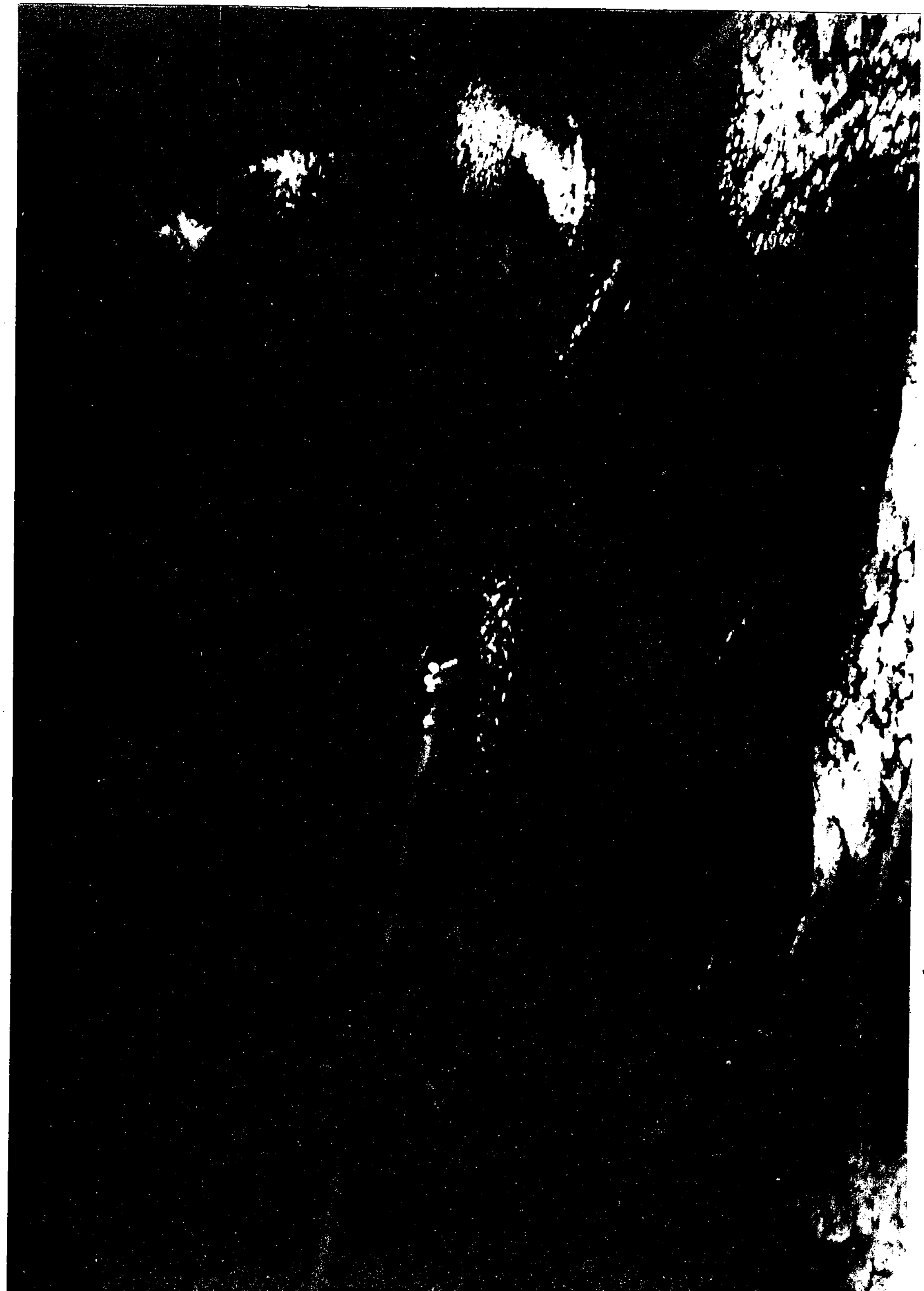

Figure 6.- Fernandina eruption cloud, Galápagos Islands, Ecuador (SL4-195-7299).

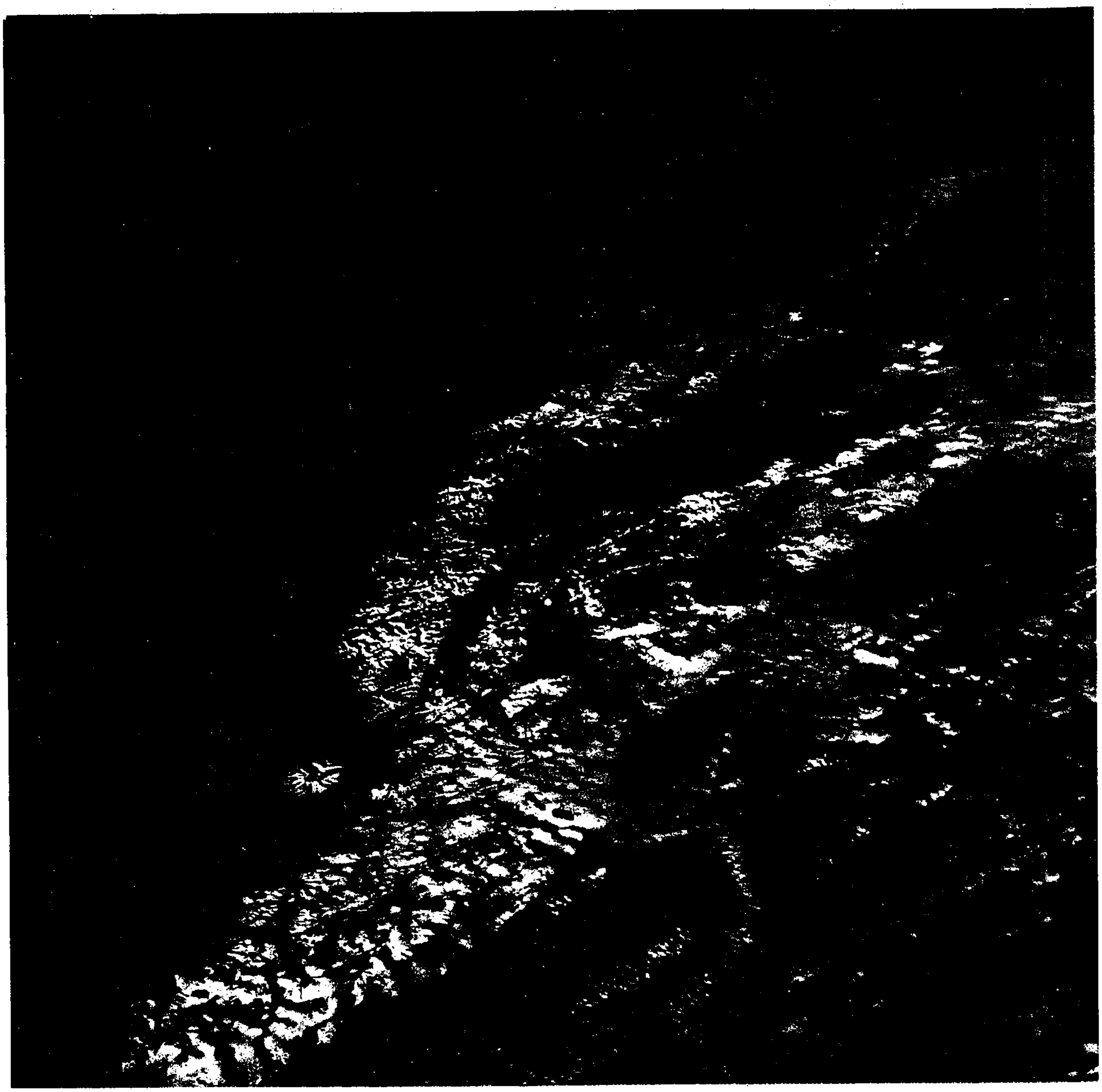

Figure 7.- Kamchatka Peninsula under snow cover showing volcanic landforms (SL4-141-4272).

Figure 8.- Altiplano region, Argentina, Bolivia, and Chile showing extensive volcanic landforms (SL4-137-3676).

Figure 9.- Simple linear dunes in Simpson Desert, Australia (SL4-143-4637).

Figure 10.- Compound dunes, Empty Quarter, Saudi Arabia (SL-143-4643).

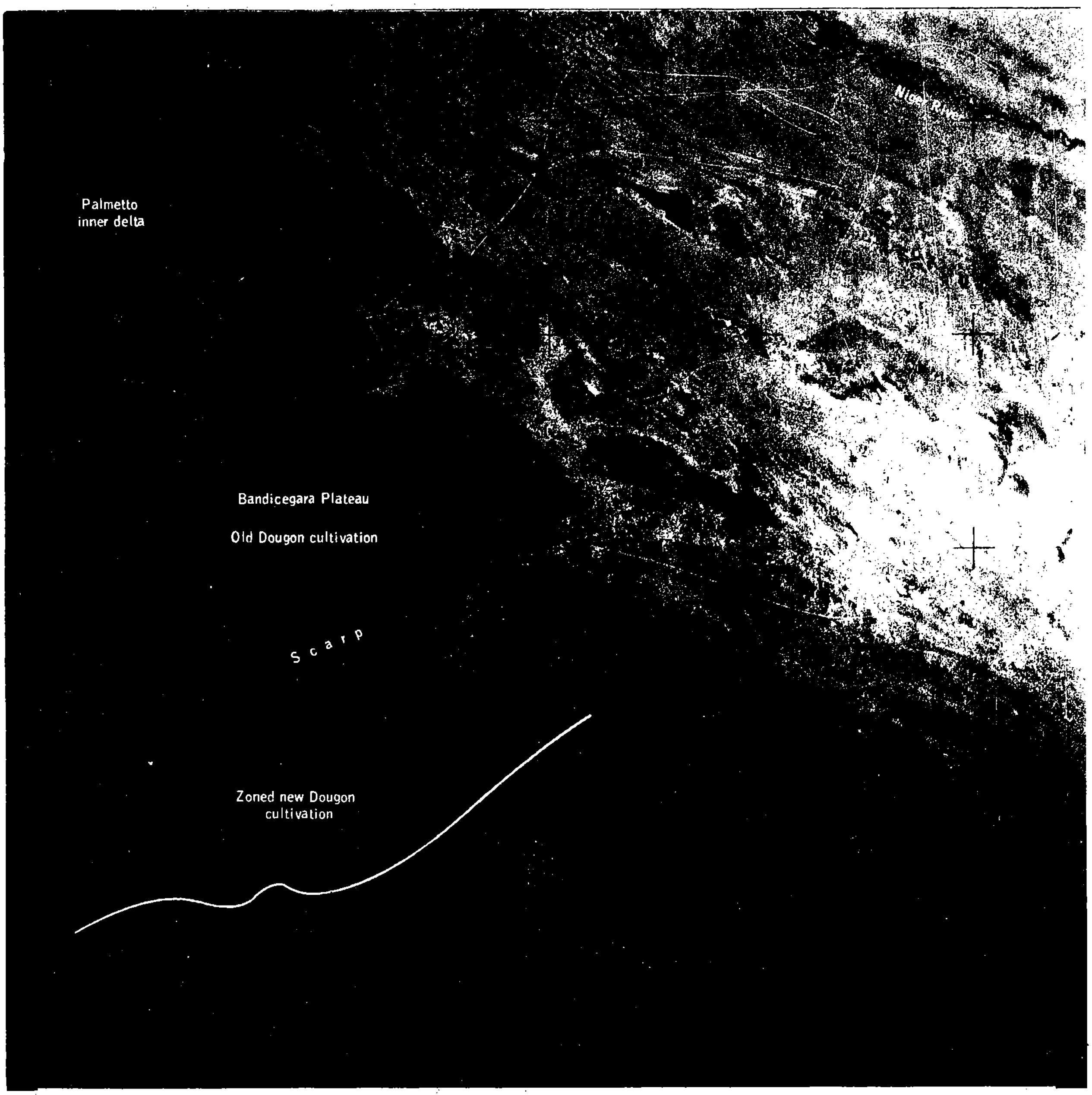

Figure 11.- Expansion of Dougon cultivation as seen in this Skylab photograph of the inland delta of the Niger River, Mali (SL4-142-4497).

research vessels. In essence, only gross features were possible to study, and the dynamic aspects were only conjectural. However, since the 1960's, greater emphasis has been directed to investigation of ocean's dynamics such as current movements, color variations, coastal effects, and relation between atmospheric and ocean energy transfer processes. From the Skylab missions, investigators learned that crewmen with only a cursory knowledge of the ocean, but supported by a scientific team listening and advising from the ground, can see and describe major dynamic conditions of the sea. Furthermore, they are able to see colors, subtle texture differences, and ocean/atmosphere relationships on the sea surface that cannot be recorded on film. However, some conditions cannot be defined by initial surface or near-surface observations because they are either too small, or near shorelines hidden among the multitude of coastal features. These features are more easily interpreted from space imagery. The key factor in the Skylab visual observations of the ocean was the ability of the crewmen to observe the condition of the ocean surface and then to evaluate, interpret, and report on the relationship of the surface features to the movements of the sea.

An example of man's capability to relate scientific knowledge from limited information to dynamic ocean conditions is shown in the study of cold water eddies in warm water currents. Cold water eddies were identified in the Yucatan Current from analysis of Skylab EREP imagery (Figure 12). The occurrence and appearance of the eddies were known to the crewmen. On subsequent passes over this region, eddies were identified by characteristic cumulus cloud formations and a variation in the ocean surface. Applying these observations, eddies were photographed near Hawaii, along the Chilean coast, in the western Mediterranean Sea, and near the Falkland Islands (Figure 13). Knowledge of the size, occurrence and distribution of eddies is of interest not only to the oceanographer in their scientific analyses of oceanic processes but also to fisherman as the cold water eddies contain more nutrients and therefore attract fish.

Differences in water color are useful as current indicators and are usually caused by concentrations of plankton of various colors. In the Falkland Current off the coast of Argentina, prolific plankton growth fingerprinted a spectacular view of the circulation systems in the ocean. The Falkland Current moves northward from Antarctica and meets the south-flowing Brazil Current near Montevideo, Uruguay. Photographs taken by the astronauts show colorful red to blue-green patterns from plankton concentrations that reflect the intricate circulation within the Falkland Current (Figures 14 and 15). As important to fish harvest as eddies, is the process of upwelling which characteristically occurs near the coast and appears as a dark blue color. Many areas of upwelling were reported by the astronauts and this photograph taken off the Portugal coast shows upwelling which supports the anchovy population and is the mainstay of the fishing industry in these waters (Figure 16). The entrainment and transportation of sediments by ocean currents were observed by the crewmen at the mouth of many large river systems. A photograph of the Atchafalaya Basin, Louisiana illustrates the ease of monitoring sediment concentrations and transport (Figure 17).

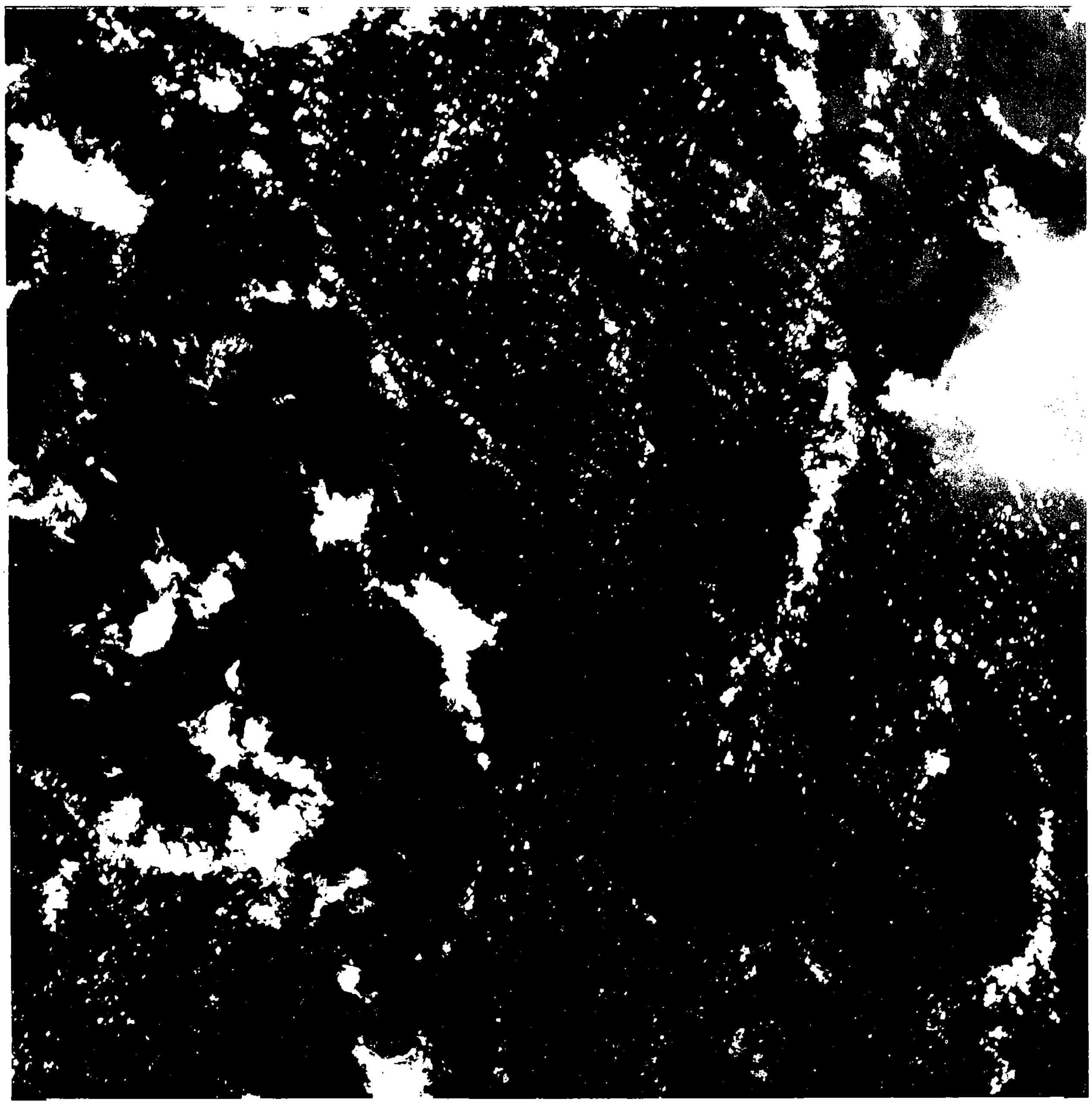

Figure 12.- Skylab photograph of northeast Caribbean Sea, showing linear cloud streets interrupted by oval-shaped clear patches. Circular eddies are defined by cloudless areas (SL2-10-072).

Figure 13.- Crescent-shaped cumuli characteristic of cold water eddies are apparent in this photograph of the Falkland Current (SL4-137-3608).

Figure 14.- Hasselblad photograph of the plankton concentrations, Falkland Current (SL4-137-3721).

Skylab Explores The Earth
By Verl R. Wilmarth, John Kaltenbach,
William B. Lenoir and Michael C. McEwen
Page 21

Figure 15.- Nikon photograph of part of the Falkland Current (NASA-S-74-4389).

Figure 16.- Upwelling off Lisbon, Portugal, as seen in this Hasselblad photograph (SL3-34-317).

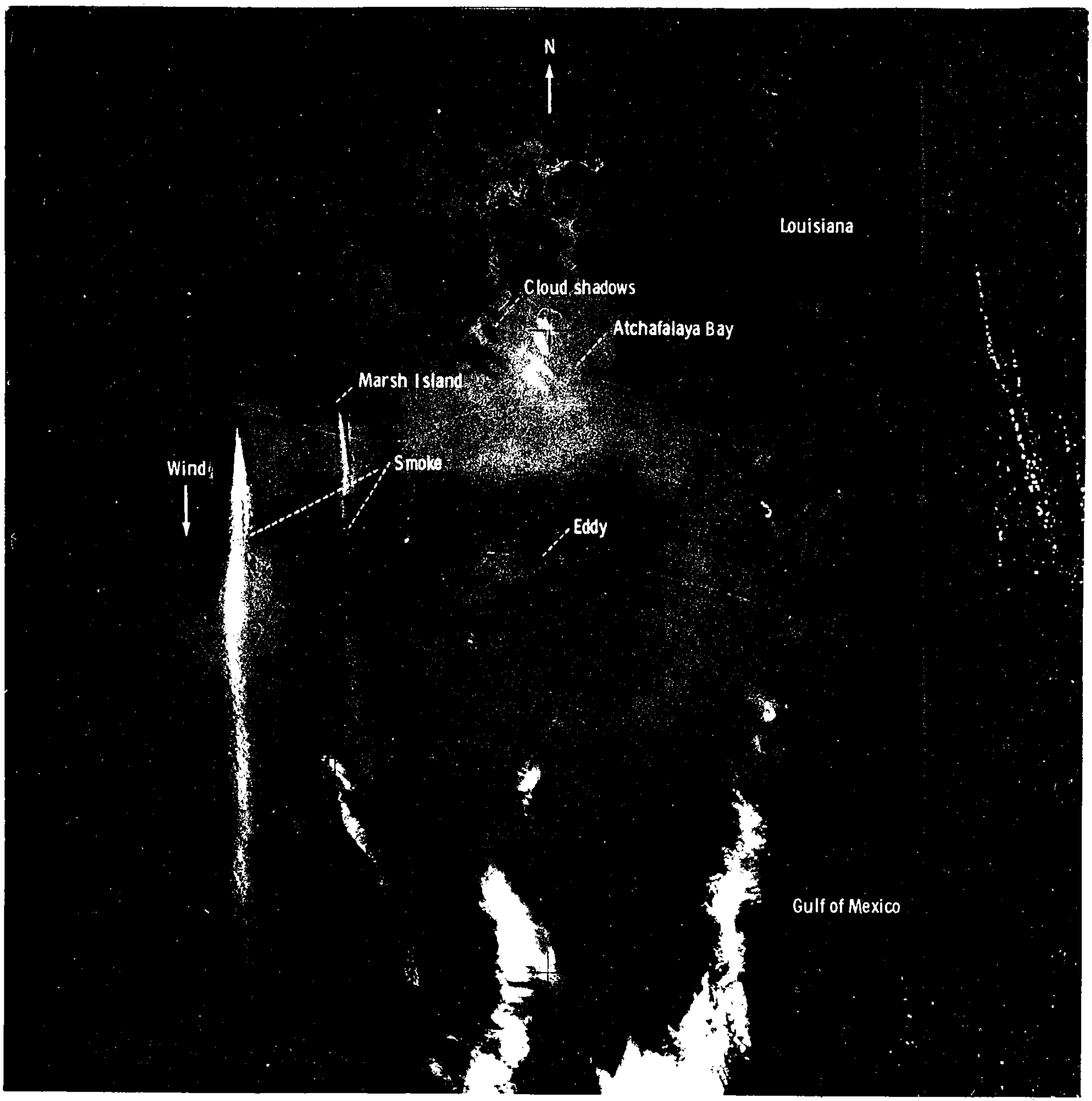

Figure 17.- Atchafalaya Basin, Louisiana and coastal sediment plumes (SL4-136-3475).

The photographic and observational data on atmospheric features returned by the Skylab crewmen have proven useful in the study of small scale phenomena which are not discernible in the synoptic views from weather satellites. Of particular importance to meteorologists are the high resolution photographs of the unusually rare, small scale phenomena that can be related to specific types of meteorological conditions.

Analysis of photographs showing cloud patterns within the wake of obstacles has provided clues to air temperatures and wind velocities. As an example, the wake cloud photographed over the Aleutian Islands shows many Karman Vortices formed downwind of the islands. The occurrence of the vortices suggest the change in temperature above the surface was small with a slow to medium wind flow (Figure 18). As the flow speed increases against a small obstacle, wake waves develop as seen in the photograph of Campbell Island in the South Pacific (Figure 19). By analogy, prediction of atmospheric conditions may be inferred when similar cloud formations occur. Descriptions and photographs of other meteorological features such as the bow shock wave developed over Snares Islands (Figure 20) can aid scientists in weather predictions.

Hurricanes are always a threat to the Gulf Coast states and research on this origin, movement and decay has led to predictive models which postulate a vertical eyewall. On Skylab 3, the crewmen photographed the eye of hurricane Ellen on September 21, 1973, and discovered the slant eyewall (Figure 21). This important result requires re-evaluation of hurricane models.

Sun glint is a transient phenomena that occurs when viewing the Earth's surface along the path of the reflected sun rays and in aerial photography, should be avoided. However, the Skylab 4 crewmen used this phenomena to describe and photograph many large scale features in the oceans such as Port Phillip Bay, Australia (Figure 24), turbulent flow patterns off the coast of Chile (Figure 22), and current patterns developed between north and south island New Zealand (Figure 23). Because approximately 5 seconds were required for the sun glint phenomena to be recognized by the crewmen, and the feature photographed, its use by satellite commanded from the ground is impractical. The detailed information that is portrayed in sun glint is shown by contrasting Figures 24 and 25. Internal structures in the water are seen only in sun glint whereas the water color (sediment content) are visible when sun glint has passed. The utility of sun glint phenomena in ocean studies has been demonstrated by the Skylab 4 crewmen; however, to exploit its application requires man to select and photograph the feature within the appropriate time frame.

In summary, the Skylab 4 Visual Observations Experiment has provided a broad background of information on the capabilities of man to serve as an integral part of a system to routinely obtain scientifically useful information for study of the Earth's dynamic systems. The Shuttle offers many potential uses of man in earth survey programs.

Skylab Explores The Earth
By Verl R. Wilmarth, John Kaltenbach,
William B. Lenoir and Michael C. McEwen
Page 25

Figure 18.- Kármán vortex streets and island wakes from Aleutian Islands (composite of SL4-140-4111 and 4112).

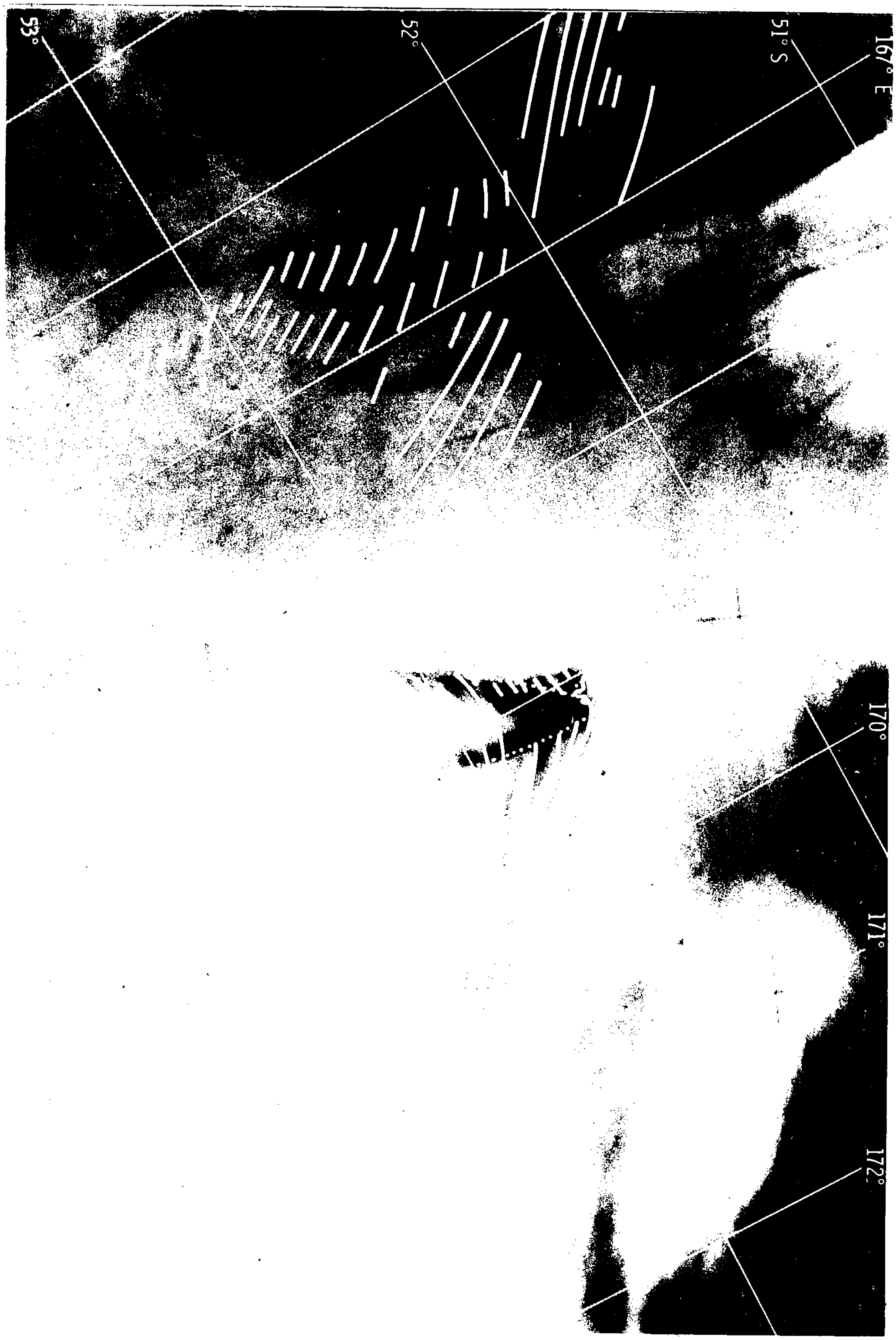

Figure 19.- Wake waves from Campbell and Auckland Islands, South of New Zealand (SL4-137-3703).

Skylab Explores The Earth
By Verl R. Wilmarth, John Kaltenbach,
William B. Lenoir and Michael C. McEwen
Page 27

Figure 20.- Bow and shock waves from Snares Islands near New Zealand (SL4-203-7764).

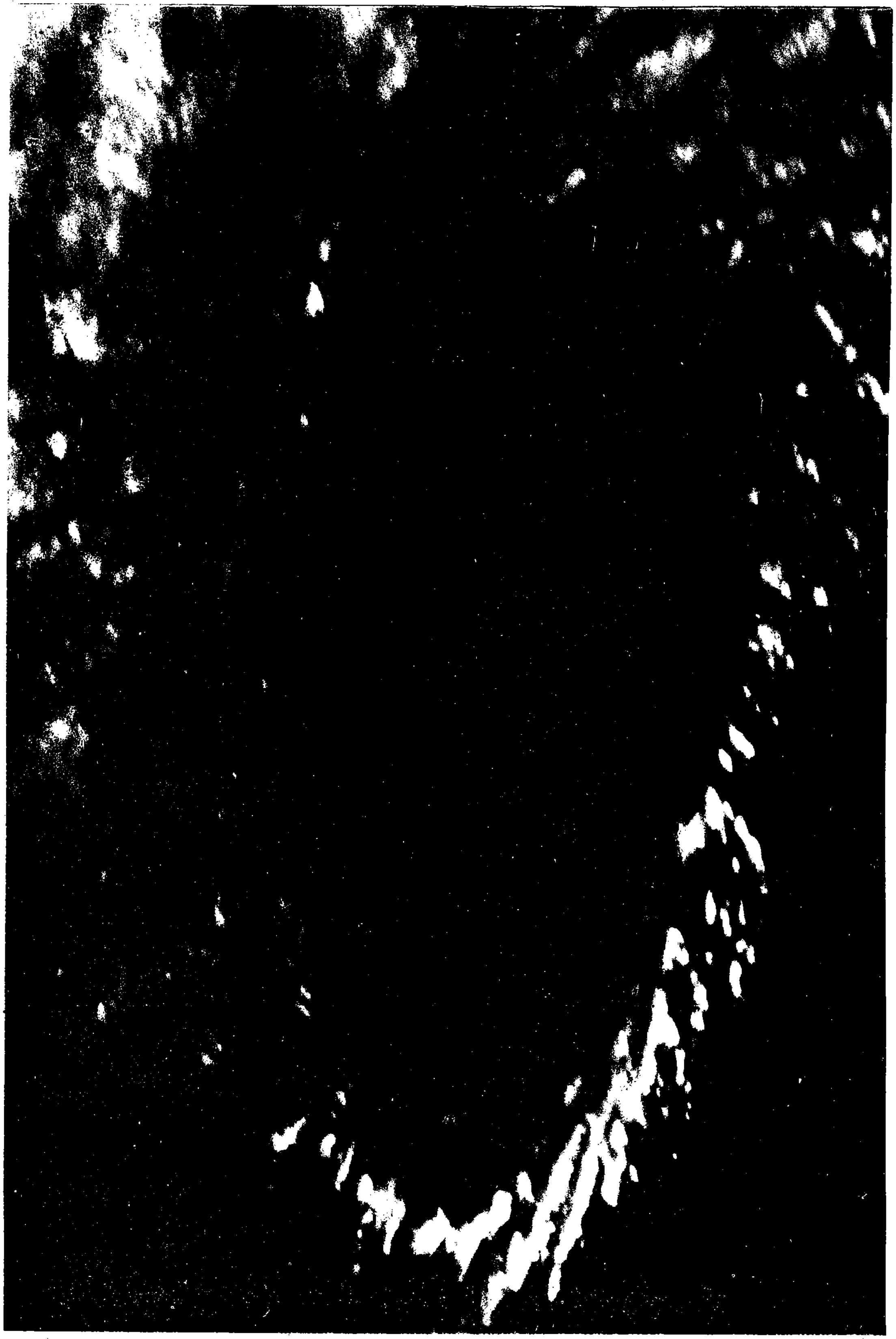

Figure 21.- Eye of Hurricane Ellen, September 21, 1973 (SL3-118-2189).

Skylab Explores The Earth
By Verl R. Wilmarth, John Kaltenbach,
William B. Lenoir and Michael C. McEwen
Page 29

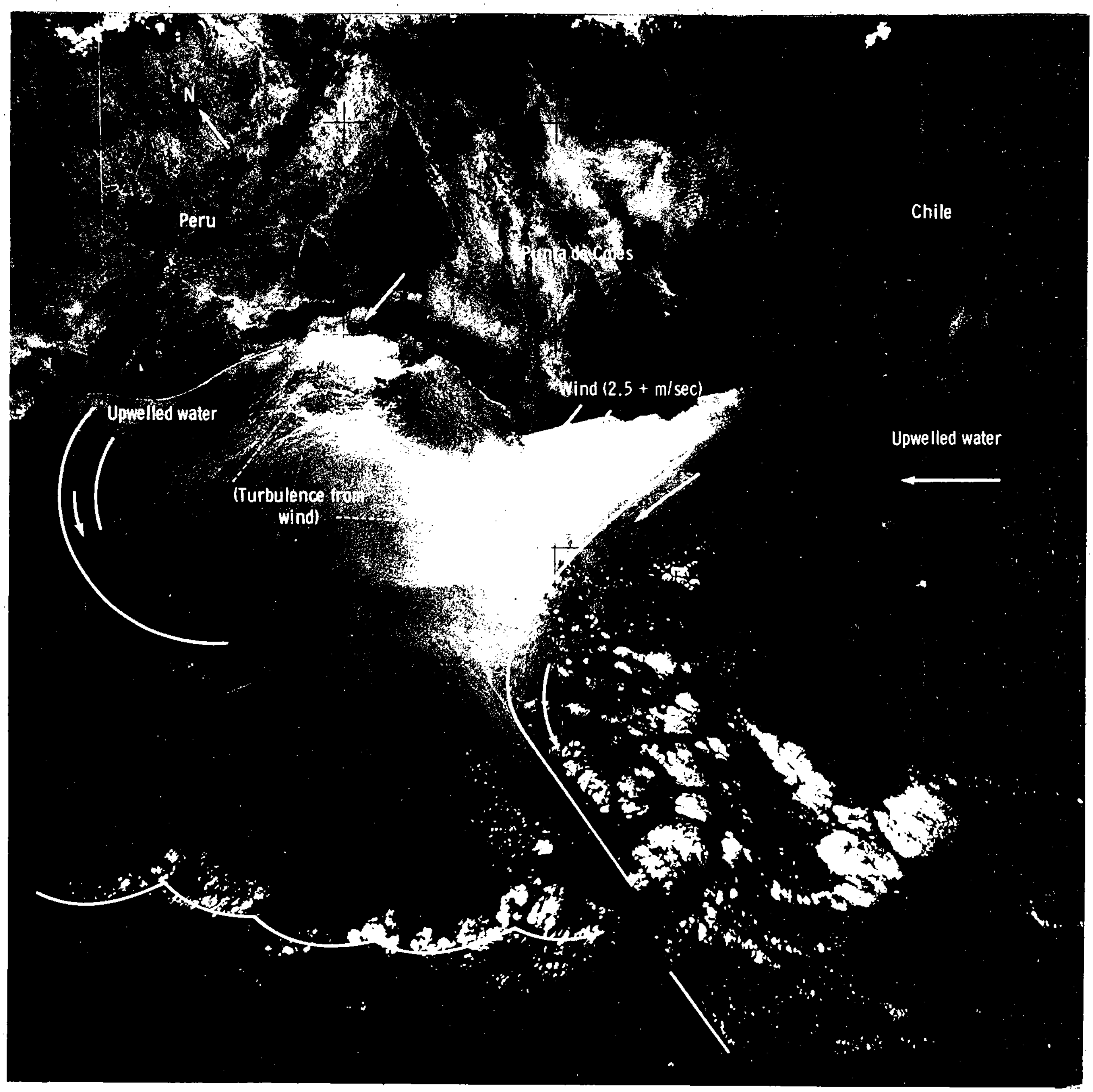

Figure 22.- Solar reflection shows illuminated turbulence and large eddies (100 to 200 km) along coast of Chile and Peru (SL4-138-3858).